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Contents

Track Strength Characterization Program: An Overview

By D. P. McConnell,* A. Zarembski,** W. S. Lovelace,***

Paper summarizes the background, accomplishments, and scheduled test activities of the Track Strength Characterization Program. Recent data from laboratory tests on a track section at the AAR's Track Structures Dynamic Test Facility and pilot studies of field measurement facilities are reviewed. Details of scheduled field characterization tests are outlined and the design of a field test vehicle reviewed.

This paper represents the first formal report to the members-at-large of the AREA Engineering Division of the progress on this effort.

1. Overview

The growing awareness of the increasing demands placed on conventional track structure, by large capacity cars and six axle locomotives has led to the initiation of a major research effort to define rational performance requirements for current track. The joint industry-government Track Strength Characterization Program is intended to develop the technical data and track assessment techniques required to define rational requirements for the track. Working in close cooperation with the joint FRA-AAR effort to investigate performance specifications for track, these activities mark a dedicated attempt to improve the performance and safety of track.

The working group of the Track Strength Characterization Program is a committee comprising working researchers from each of the participating organizations. The Committee was formed in December of 1977 to investigate issues raised in the Phase II work of the Track Train Dynamics Program. The Committee has 16 members made up of railway engineers, AAR Research personnel, and representatives of the FRA and TSC.

If a railroad engineer was to review the significant research work made in the field of track structures in this country and Europe over the past 100 years or so, he would probably recall: (1) Talbert's work on Vertical Track Modulus in the United States beginning in 1918; (2) French National Railway's (SNCF) work in the late 1940's on their derailer wagon and; (3) the Track Train Dynamics effort that began in 1972 which brought together and expanded recent track related research. To a large extent, the present Track Strength work is a continuation of these efforts to define and improve the performance of track. The Track Strength Committee is working in close support with the AAR-FRA Ad Hoc committee on Track Standards to define functional requirements for track.

The committee receives support from the organizations shown in Figure 1 which include the AAR Research and Test Department, the Transportation Systems Center, the AREA, the Engineering Division of the AAR, the Track Train Dynamics Program, Individual Railroads, and the Federal Railroad Administration (FRA).

The primary concern of railroad engineering departments is to provide a track structure which is safe and reliable at a minimum cost. To reach this goal in maintaining track, it is critical that the load bearing capacity of the track to support vertical, lateral, and longitudinal loads imposed by train operations and environmental conditions be known.

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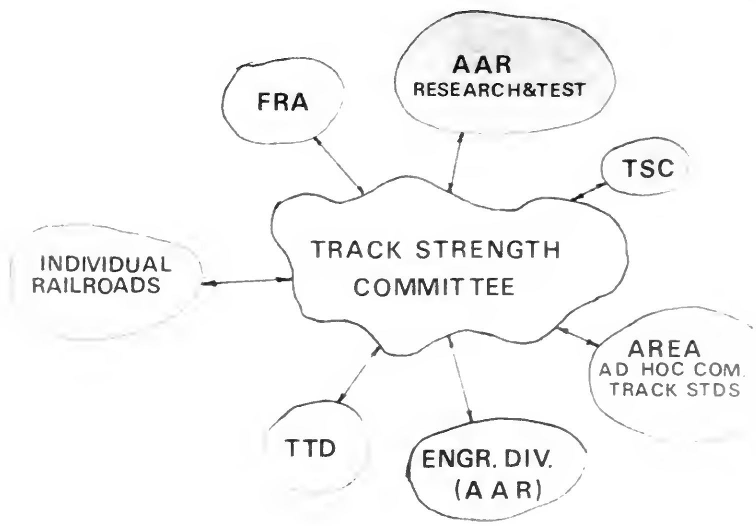

AAR
RESEARCH&TEST
FRA
TSC
TRACK STRENGTH
COMMITTEE
INDIVIDUAL
RAILROADS
AREA
AD HOC COM.
TRACK STDS
TTD
ENGR. DIV.
(AAR)

Figure 1

In general, both the load bearing capacity of our track and, in fact, the actual service loads imposed on the track are not quantitatively known. Consequently, both the actual strength of the track and the required strength is undefined. This problem is reflected in uncertainty as to the required level of maintenance.

Four overall objectives of the Track Strength Characterization Program have been defined (1). These are summarized in the following required activities:

1. *Determine What Influences Track Strength*—Define the parameters that influence the strength of the track, i.e., the load carrying capacity of the track.

2. *Measurement of the Parameters that Affect Track Strength*—Once these parameters are established, their effect on the track strength must be measured and understood.

3. *Develop Practical Methods of Determining Strength of Track*—Techniques and equipment must be developed to accurately determine the strength characteristics of track *in the field.* It does little good to measure track in a laboratory if we are unable to measure the same parameter in the real world.

4. *Demonstrate the Benefits of Quantifying Track Strength*—This will include investigation and demonstration of the benefits of track strength measurements, such as:

   (a) an aid to maintenance planning.

   (b) input to traffic planning.

   (c) input to rolling stock design.

   (d) an aid to derailment prevention.

The Track Strength Program is proceeding to attain these objectives by a combination of *laboratory* and *in-track* tests. Tests in the AAR's new track laboratory in Chicago have been underway since April 1978. These tests are being conducted to evaluate and determine the vertical track modulus under various static loadings and track conditions. A full range of vertical, lateral, and longitudinal loads have been placed on a 39-ft. section of mainline track, and the resulting displacements measured. These tests are proving to be of considerable value in establishing a base line for track strength data. In addition to these laboratory tests, static load-deflection tests have been carried out in-track on two railroads to further examine and confirm the track reaction to these static loads.

In December 1978, a series of perturbed track tests were conducted at the Transportation Tests Center in Pueblo for the purpose of evaluating locomotive response to track irregularities. The Track Strength Program was able to piggy-back on these tests with measurement of track responses under train loads. Extensive in-track tests were made at locations where horizontal misalignments were introduced into the track structure. By monitoring the rolling stock forces *and* track displacements at these locations, significant insight into the level of forces imposed on mainline track structures was gained. The evaluation of this data is in progress at this time.

In November of 1978, a prototype gage widening device designed by Mr. Jerry Magee, retired Assistant Vice President of Research for the AAR, was used to evaluate the concept of rolling a wheel pair along the track with a constant lateral force to spread the wheels apart. The result of this single test was most interesting, and showed a potential for measuring gage widening *and* lateral strength of track.

Based on these initial laboratory and in-track test results, plans to pursue this work by testing mainline track on several railroads during the third and fourth quarters of 1979 have been developed. Both static load tests and dynamic gage widening tests will be made.

Finally, using the FRA derailment information data for the years 1976, 1977, and 1978, a review of the derailments caused by *buckled tracks* is being undertaken. A detailed summary is being made of the track operating, and environmental conditions that prevailed at the time of the derailment. This data will be analyzed to develop common characteristics that led to buckled track.

It is very difficult to improve our track structure and gain a more stable, reliable, and safer track without a quantitative definition of *what* factors affect the strength of track *or* how to measure these factors. As a better understanding of factors involved in track strength is gained, the ability to provide safe and reliable track *at* a minimum cost will increase.

2. Preliminary Track Testing

For the initial phase of track strength testing, the basic measurements shown in Figure 2 have been selected as initial candidate test configurations for track strength testing. These measurements are directly related to both vertical and lateral track strength characteristics. The strength characteristics of interest include:

- resistance to gage widening (including both rail roll and translation)
- resistance to track alignment deterioration
- resistance to track surface deterioration

These aspects of track behavior under load were selected as most directly relatable to the capacity of the track to resist undesirable deformation under load. These factors will form the basis for the definition of the initial indices of track strength.

The activities performed under this preliminary track testing phase consisted of two basic series.

(1) Track Laboratory tests

(2) Preliminary Rail Spreader Tests

Both test series were completed in 1978, and are currently being reviewed and analyzed to help prepare for the next series of field tests.

Track Laboratory Tests

The preliminary tests of track response to loads required to define track strength indices and develop test procedures were conducted at the Association of American Railroads Track Structures Dynamic Test Facility, Chicago, Illinois. This facility shown in Figure 3 provides a closely controlled environment in which the load-deflection behavior of the track can be documented under controlled vertical, lateral, and longitudinal loads.

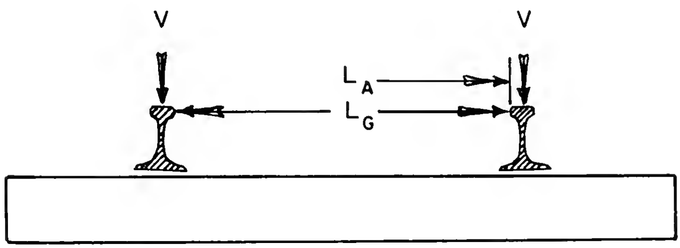
The test track consisted of 136 RE rail on wood crossties at $19\frac{1}{2}$ inch spacing resting on 12 inches of Limestone Ballast (AREA Number 4), and 6 inches of Limestone Subballast (Illinois Specification CA-10). The parent subgrade material was a sandy fill (USCS classification SP).

The Track Laboratory Test consisted of three test series:

(a) Vertical Modulus Tests

(b) Gage Widening Tests

(c) Track Misalignment Tests



FAILURE MODE	TEST LOAD	STRENGTH PARAMETER
GAGE WIDENING	L_G, V	RAIL ROLL AND TRANSLATION RESTRAINT
LOSS OF ALINEMENT	L_A, V	LATERAL TRACK RESTRAINT
LOSS OF SURFACE	V	TRACK VERTICAL SUPPORT

TRACK STRENGTH PARAMETER

Figure 2

Figure 3

In the Vertical Modulus tests, vertical loads were applied at one point on each rail, through a loading bolster, Figure 4. Deflection measurements were taken using both linear displacement transducers and a Surveyors level at the point of loading and at several points on either side of the loading point. The deflections were taken for different load levels, ranging from 25 to 50 kip wheel loads, and then converted to equivalent track modulus values using three different techniques (2). Figure 5 shows a plot of vertical modulus, calculated using beam on elastic foundation theory, vs applied wheel load. Note that the track modulus varies with the applied load. For wheel loads of less than 10,000 lb, this variation is quite non-linear. In view of this load dependent behavior, it appears desirable that modulus values be obtained at defined load levels, representative of actual service loads, so that uniform modulus values can be obtained under differing test conditions. For a more detailed description of this test and the corresponding results, the reader is referred to Reference (2).

The gage widening tests were designed to begin to answer the question "Can non-destructive gage widening tests be used to detect damaged or weakened track and to predict ultimate strength and failure modes of track" (3). In order to answer this question, a series of 86 gage widening tests were conducted for various combinations of vertical and lateral loads and at different levels of track condition. These tests utilized a hydraulic jack, located between the rails, to exert a lateral load on each rail, and two vertical jacks to apply the vertical loads (Figure 6).

Lateral deflections of the rail head and rail base, and vertical uplift of the rail base were measured. Figure 7 shows a typical set of gage widening deflection results. Note, the dominant mode of deformation appears to be rigid body rotation of the rail section. A set of adjacent load tests were included within the gage widening series in order to evaluate the effect of a second adjacent vertical load on gage widening. This was accomplished through the use of a second lateral jack and two additional vertical jacks. Figures 8 and 9 show the results of the adjacent axle test. V1 represents the vertical load applied at the same location where the lateral load is applied, V2 represents a second vertical load applied 72"" (a truck spacing) away from V1. Note that although the deflection of the rail (at V1) ia decreased by the application of a second adjacent load, significant deflections are obtained even when the truck L/V (L/(V1 + V2)) is less than .5. Finally, a set of gage widening tests were conducted where a hydraulic rail puller was used to introduce longitudinal force in the rail under investigation. (Figure 10). The results of these tests (Figure 11) indicate that for a track structure in good condition, the presence of longitudinal loads does not result in a significant increase in gage widening under lateral and vertical loading. This result differs from the results of tests by Heron and Flassig (4) where on a track in poor condition (worn 100 lb. rail, poor ties) they found that the presence of longitudinal loads does result in a significant increase in gage widening. A full report on the results of the gage widening test series is currently being prepared and will soon be available.

The third series of tests performed in the track laboratory were those directed at the investigation of track shift. In these tests, a net lateral load was applied to the track structure through a loading bolster (Figure 12) and the net deflection of the track was measured. Data from these tests are currently being reduced and it is expected that this data will be available within a short period of time.

Preliminary Rail Spreader Tests

Though some research effort had been directed in the past towards the understanding of the mechanism of gage widening and rail overturning (5), only very limited information was available to ascertain whether over-the-road measurement of widening resistance was possible. Furthermore, in order for any measurement technique to be effectively utilized by the

Figure 4

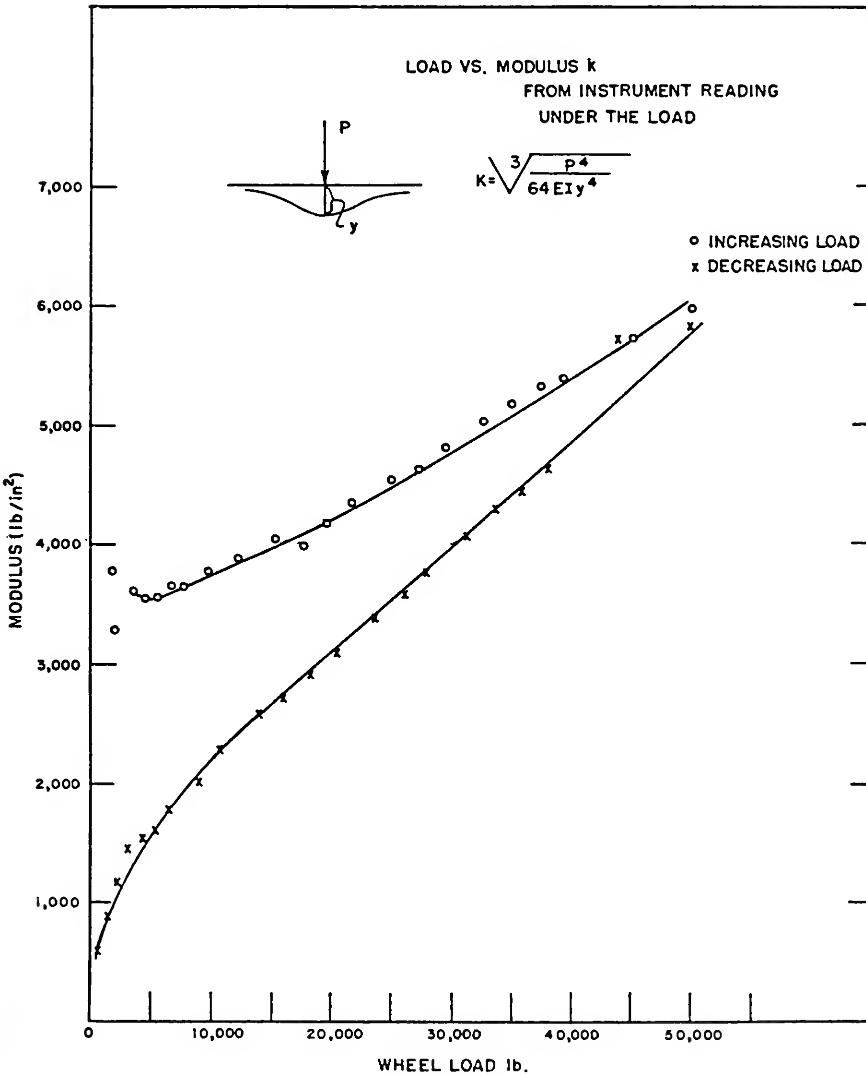

LOAD VS. MODULUS k
FROM INSTRUMENT READING
UNDER THE LOAD
P
y
$K = \sqrt[3]{\frac{P^4}{64EIy^4}}$
o INCREASING LOAD
x DECREASING LOAD
7,000
6,000
5,000
4,000
3,000
2,000
1,000
MODULUS (lb/in²)
0
10,000
20,000
30,000
40,000
50,000
WHEEL LOAD lb.

Figure 5

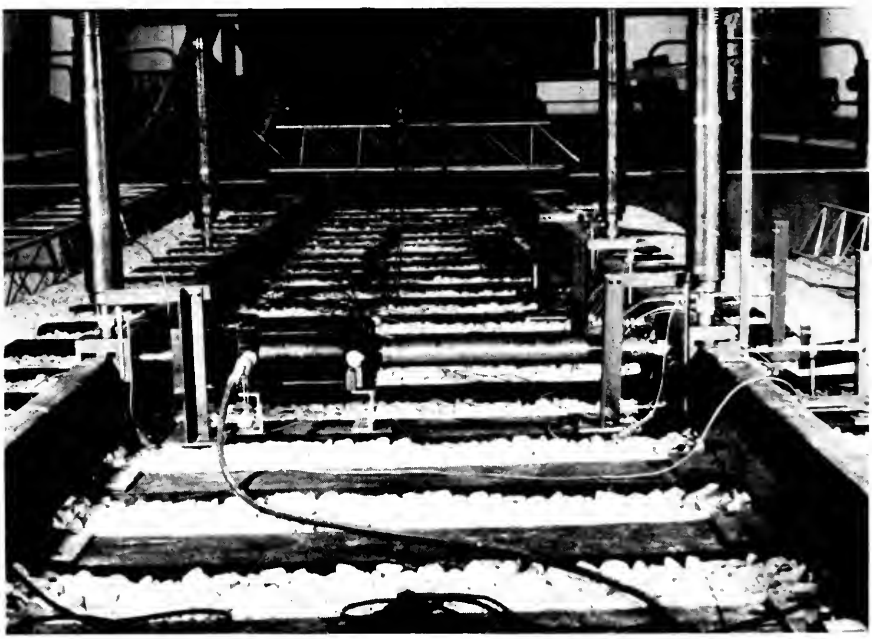


Figure 6

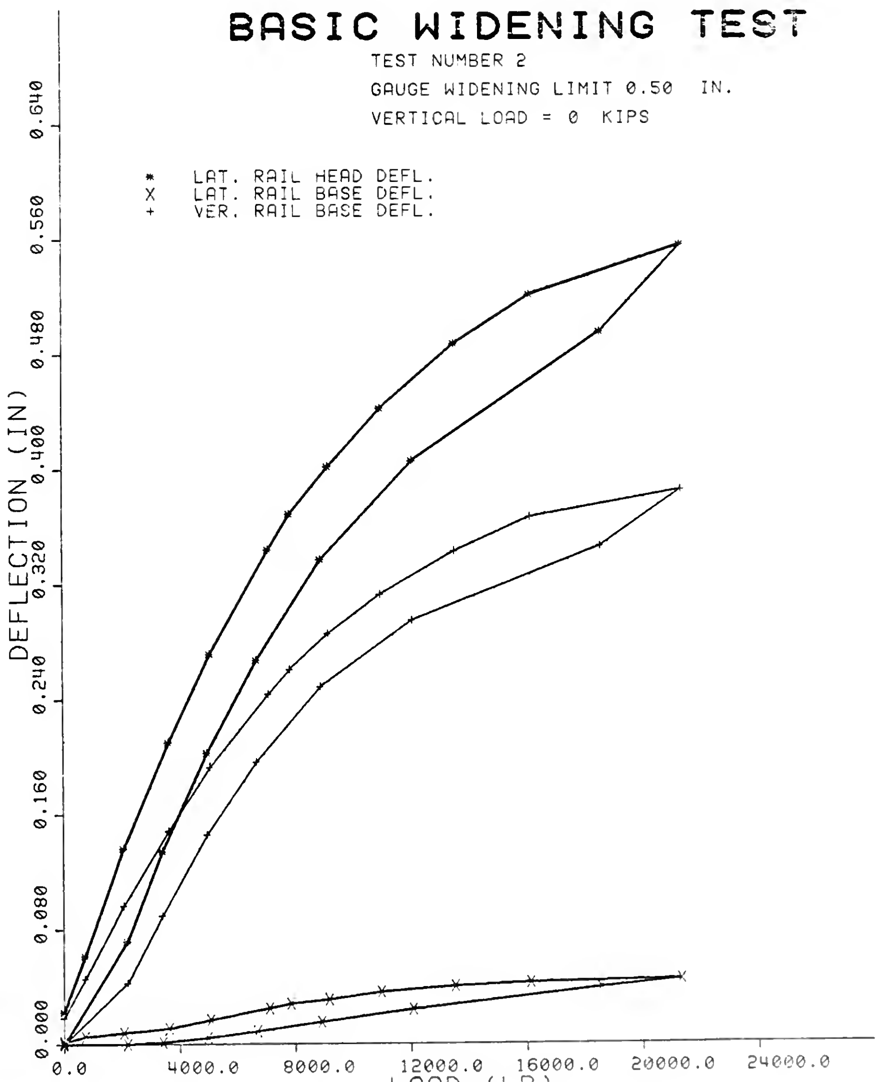

BASIC WIDENING TEST
TEST NUMBER 2
GAUGE WIDENING LIMIT 0.50 IN.
VERTICAL LOAD = 0 KIPS
* LAT. RAIL HEAD DEFL.
X LAT. RAIL BASE DEFL.
+ VER. RAIL BASE DEFL.
DEFLECTION (IN)
0.000
0.080
0.160
0.240
0.320
0.400
0.480
0.560
0.640
0.0
4000.0
8000.0
12000.0
16000.0
20000.0
24000.0
LOAD (LB)

Figure 7

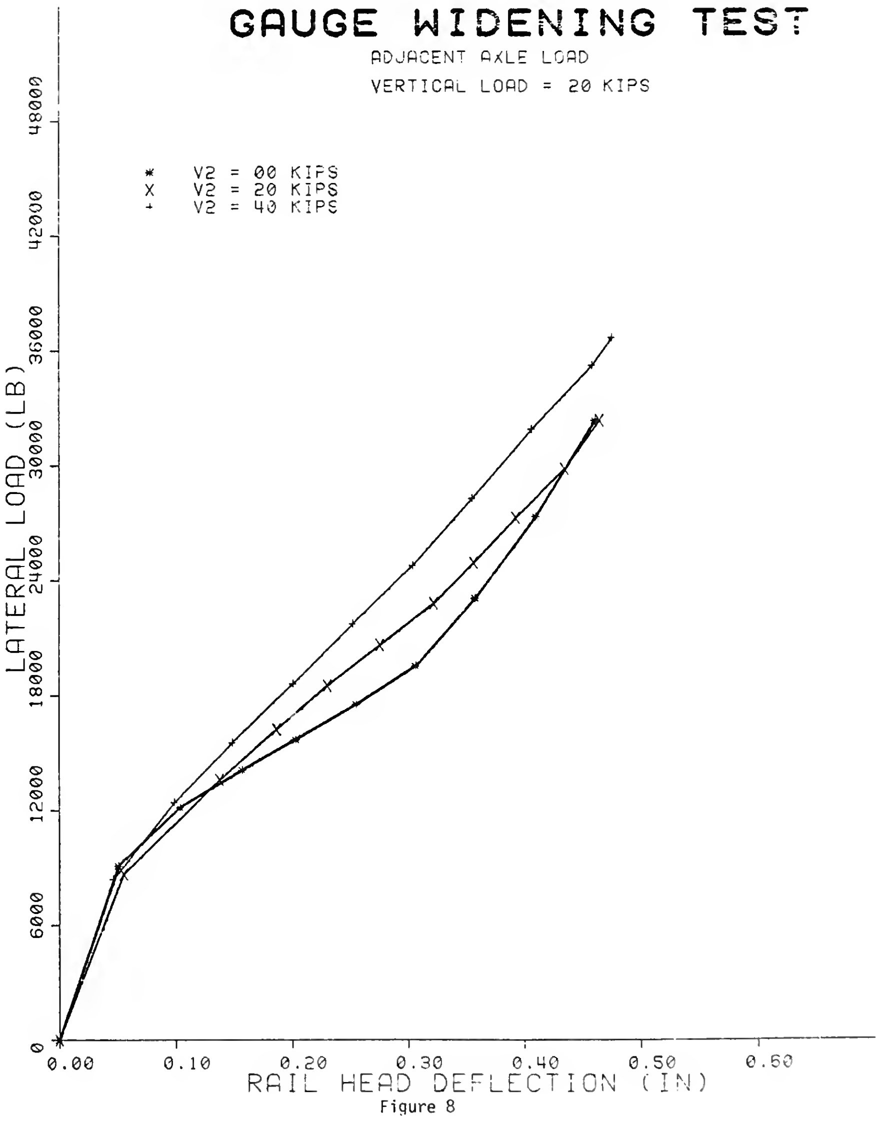

GAUGE WIDENING TEST
ADJACENT AXLE LOAD
VERTICAL LOAD = 20 KIPS
* V2 = 00 KIPS
X V2 = 20 KIPS
+ V2 = 40 KIPS
LATERAL LOAD (LB)
0
6000
12000
18000
24000
30000
36000
42000
48000
0.00
0.10
0.20
0.30
0.40
0.50
0.60
RAIL HEAD DEFLECTION (IN)

Figure 8

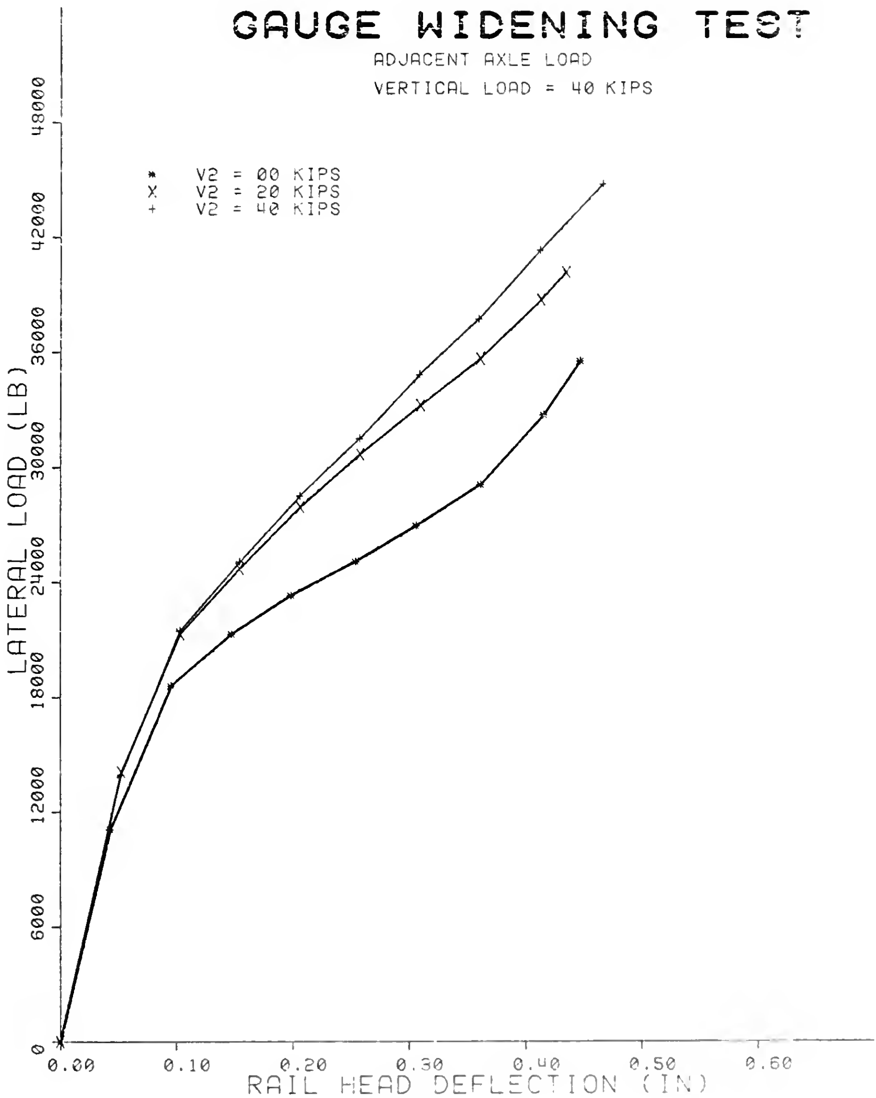

GAUGE WIDENING TEST
ADJACENT AXLE LOAD
VERTICAL LOAD = 40 KIPS
* V2 = 00 KIPS
X V2 = 20 KIPS
+ V2 = 40 KIPS
LATERAL LOAD (LB)
0
6000
12000
18000
24000
30000
36000
42000
48000
0.00
0.10
0.20
0.30
0.40
0.50
0.60
RAIL HEAD DEFLECTION (IN)

Figure 9

Figure 10

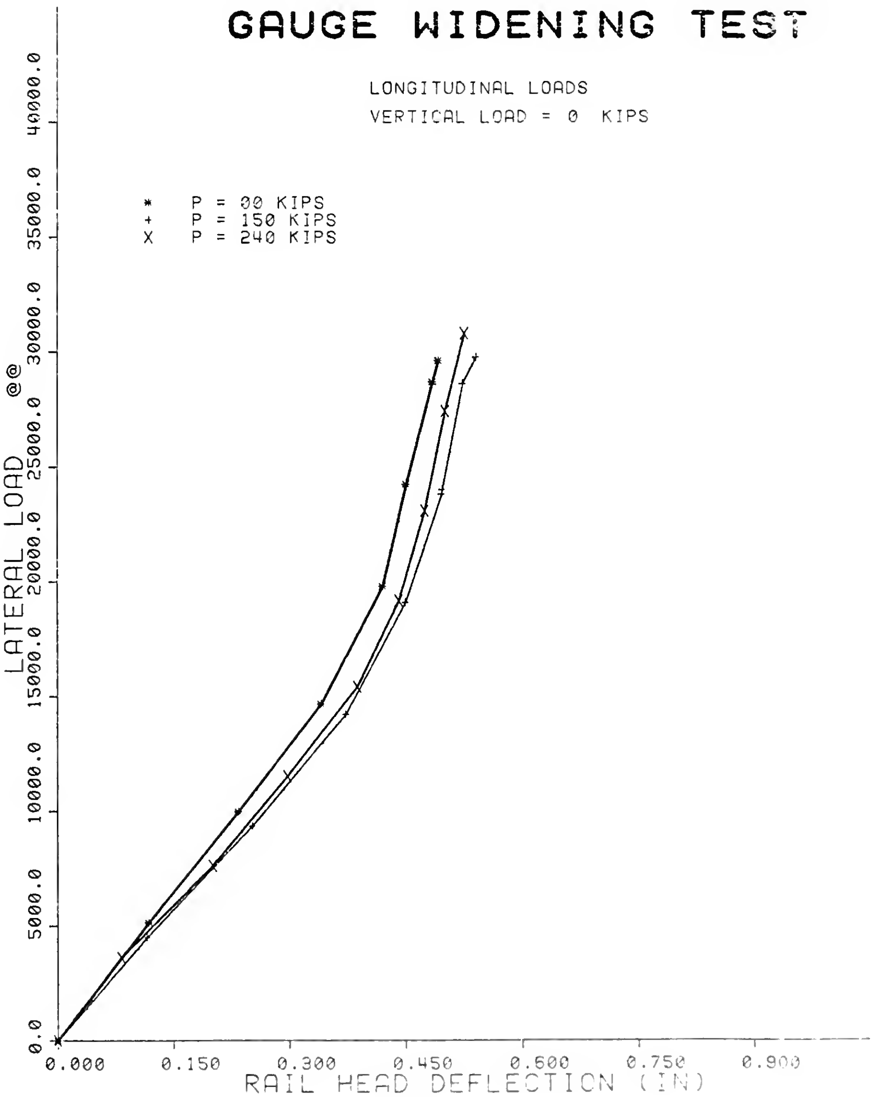

GAUGE WIDENING TEST
LONGITUDINAL LOADS
VERTICAL LOAD = 0 KIPS
* P = 00 KIPS
+ P = 150 KIPS
X P = 240 KIPS
LATERAL LOAD @@
0.0
5000.0
10000.0
15000.0
20000.0
25000.0
30000.0
35000.0
40000.0
0.000
0.150
0.300
0.450
0.600
0.750
0.900
RAIL HEAD DEFLECTION (IN)

Figure 11

Figure 12

railroad community in a field environment, it would be necessary to have a test apparatus capable of evaluating long stretches of track quickly, without interfering with traffic, and non-destructively, without causing permanent damage to the track structure.

In order to investigate the feasibility of such a measurement, it was decided to conduct a restraint measurement test utilizing the rail spreader apparatus developed by the AAR Technical Center in 1968 (6). Since this equipment was currently available and was in generally good working condition, it was determined that the utilization of this apparatus to measure widening resistance continuously over a short stretch of track would provide a quick and inexpensive test to evaluate the feasibility of the basic concept. (6)

Using the Rail Spreader apparatus to exert known lateral loads onto the track structure, and fixing the vertical load, the test vehicle was run down a test track at the AAR Technical Center. (Figure 13)

The initial test results indicated that by monitoring the rail deflections and load variations it appears possible to measure track strength and specifically locate and identify ties or fasteners in poor condition. This is supported by the test observations that for several different test runs, a drop in recorded pressure in the hydraulic system coincided with passage over a tie in poor or fair conditions. (Figures 14). Furthermore, by monitoring individual rail head displacements, it appears that a distinction could be made between the support conditions in the two rail seat areas. These observations, together with the observed spring back of the rail upon removal of the load, indicated that the development of an over-the-road test vehicle does indeed appear to be feasible.

3. Field Tests of Rail Restraint

A rapid, efficient and accurate means of testing the gage restraining characteristics of the track is essential to rationally assess the adequacy of track for train service. The need for such a technique has been highlighted by the concern over the increasing demands which large capacity cars and six axle locomotives placed on the track.

Figure 15 illustrates the essential aspects of the gage restraining characteristics of the track. The operation of trains over the track produces a spectrum of vertical and lateral loads on the rail. Ultimately, with the passage of wheelsets, the track is subjected to the peak lateral load, L^*, and the associated simultaneous vertical load V^*. This action causes the railhead to displace laterally by the increment S shown in Figure 15. The gage restraining performance required of the track can be quantified in that the total dynamic gage, taken as the sum of the static gage plus the rail displacement under load must be less than a limiting value.

This functional requirement forms the essential element of a performance based standard for track.† This requirement also provides a basis for defining an index of the track. This index can be defined as the deflection of the rail, δ^*, under applied loads L^* and V^*. The influence of axial load is assumed to be negligible in this definition.

Field tests are currently in planning to evaluate the practicality of gage restraint testing. The primary objectives of this activity, as shown in Figure 16, are two-fold. The first goal is the evaluation of the gage restraining characteristics of track which is in-service and thereby evaluate the typicality of the laboratory test results and the validity of the test procedures developed in the lab. Equally important is the investigation of the variation of rail restraining behavior with varying track conditions. These conditions include the type of fastening and the

†Standards based on such functional requirements are currently under evaluation by the FRA and the AAR Ad-Hoc Committee on Standards.

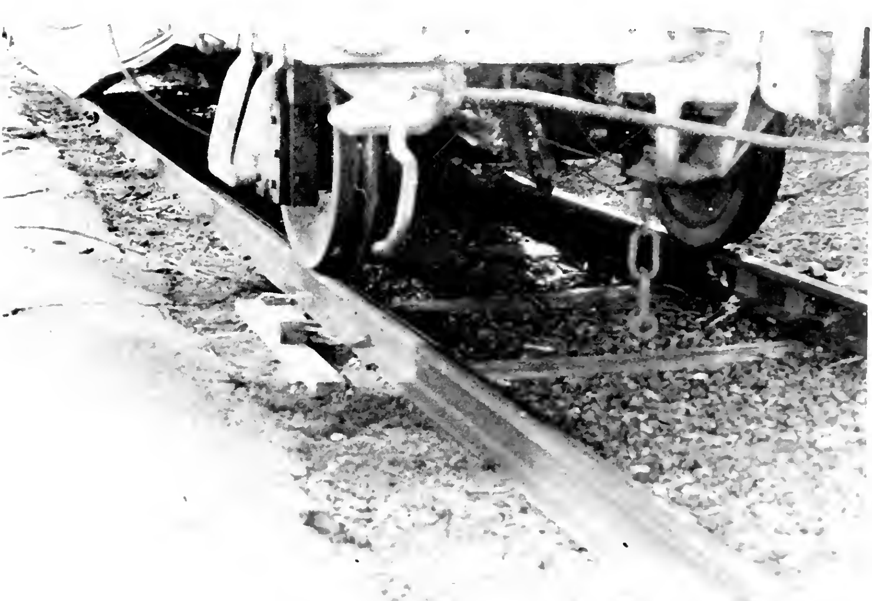


Figure 13

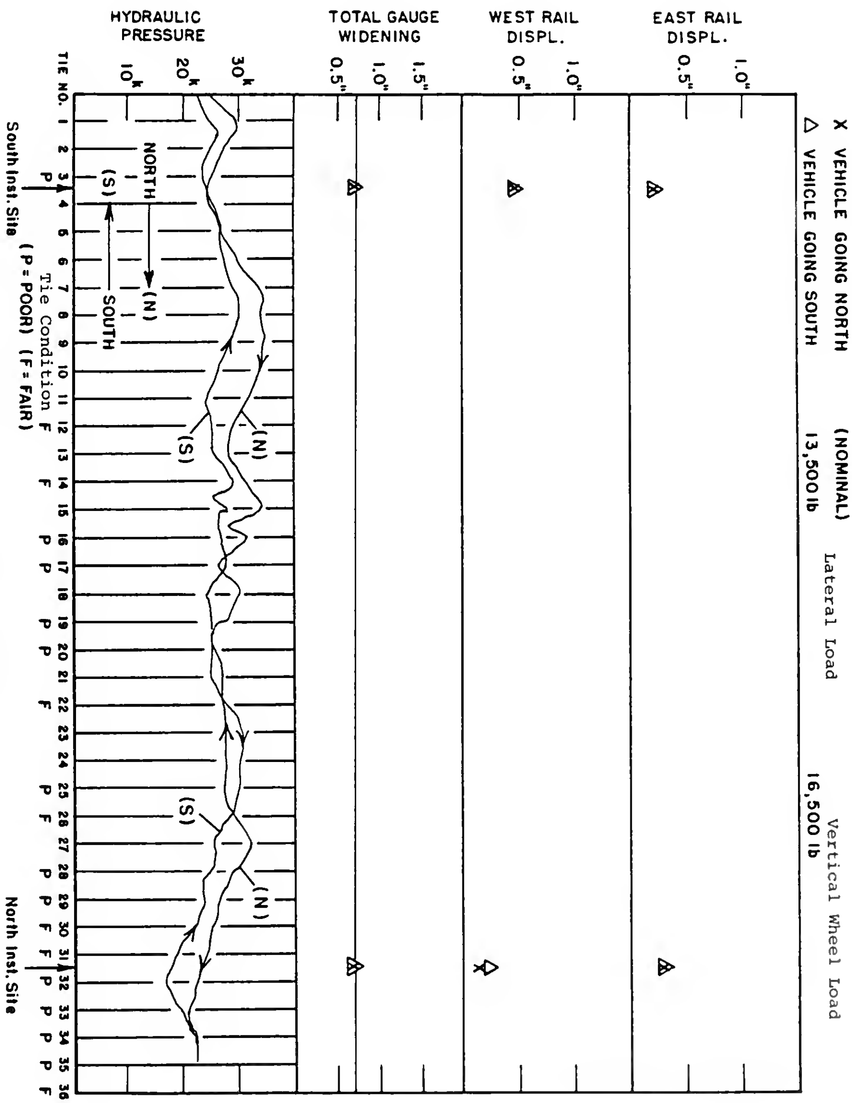
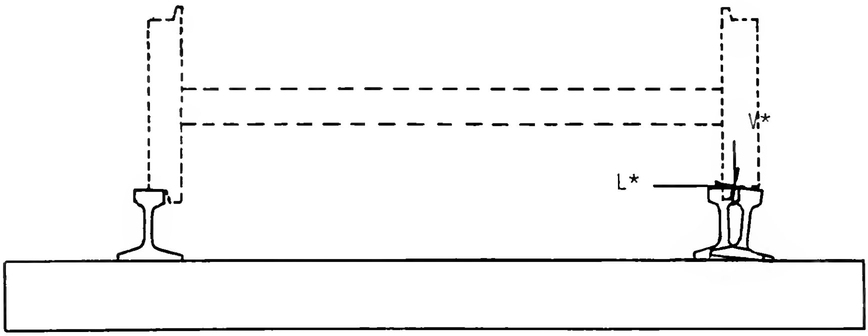
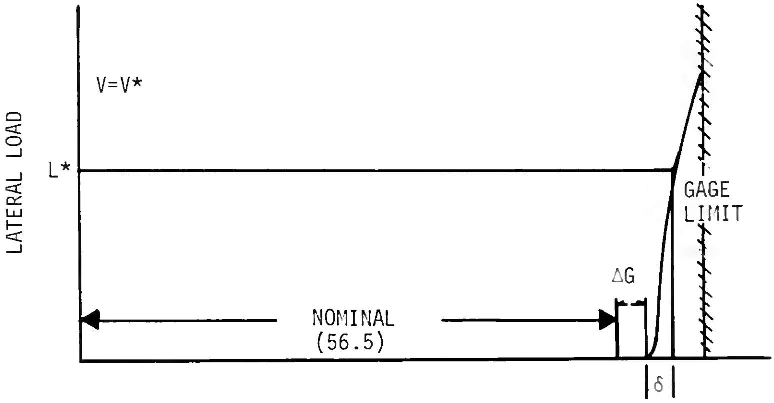


FIGURE 14 TEST DATA. RUN NUMBER 7





RAIL RESTRAINT INDEX

δ* AT V* AND L*

Figure 15

TRACK STRENGTH FIELD TEST OBJECTIVES

- TO EVALUATE RAIL RESTRAINT TESTING PRACTICES IN THE FIELD FOR:
  - PRACTICALITY
  - UTILITY
  - VALIDITY
- TO INVESTIGATE THE RANGE OF RAIL RESTRAINT BEHAVIOR FOR:
  - MAINLINE VS. SECONDARY LINE
  - NEWLY TIMBERED VS. WORN TRACK
  - VARIOUS LOCATIONS ALONG TRACK

Figure 16

weight of rail and, more significantly, the condition of the ties and spikes in worn or weathered track. This latter aspect is crucial to confirming the preliminary indications of the original rail spreader tests regarding the ability to detect inadequate tie condition.

The test approach for field investigations builds directly on the gage spreader and laboratory testing techniques. The key elements of the approach, as summarized in Figure 17, are the use of a spreader type test to survey long track segments and identify locations of anomalous rail load-deflection response. These locations will then be tested using a stationary loading fixture to obtain a full load displacement curve such as that shown earlier in Figure 7. The comparison of such data from both anomolous sites and sites showing no anomoly in the survey will permit an assessment of the utility of the survey technique to evaluate the adequacy of the gage restraint behavior of the track.

Correlating this data with the prior assessments of the track condition by engineering and maintenance of way officers will provide an indication of the comparability of the test results with experience with the track in service.

This test approach defines the basic functional requirements for a field test device for gage restraint testing. These include:

- The ability to non-destructively evaluate large segments of track and detect restraint variations or anomalies.
- The ability to test track objectively in both the moving "survey" mode and in the stationary "proof" test mode without introducing a bias or substantive errors in the data due to test car design.

The laboratory test data developed as part of the track strength characterization program has been analyzed to assess the ability of a non-destructive test to evaluate gage restraint ability. Figure 18 illustrates the gage widening measured under a vertical load of 15,000 pounds and lateral loads of 10,500 pounds and 7,500 pounds.

This corresponds to single wheel lateral to vertical force ratios of 0.7 and 0.5 respectively. The maximum lateral rail head displacement, or gage widening, is plotted as a function of the "pre-damage" to which the track had been subjected. Pre-damage was produced by deflecting the newly installed rail to 0.25, 0.50, and 1.0 inches gage widening prior to applying the test loads. The desired condition is the lowest test load values producing a monotonically increasing deflection as a function of prior gage widening.

The laboratory data summarized in Figure 18 indicates that a load condition of 15,000 pounds vertical and 10,500 pounds lateral is the lowest load values at which interpretable data is generated. These values of load have been selected as candidate test loadings for the initial field tests. This condition will serve as the baseline load state for continued evaluation of the load conditions required for survey testing.

The development of unbiased test data with a vehicle borne field test system is also complicated by the presence of adjacent axles. The critical issue is the interaction of adjacent wheel loads with the load-deflection response of the rail. Field data generated by J. Lundgren of the Canadian National Railroad (7) had been evaluated for the action of adjacent loads. The test data shown in Figure 19 clearly show that the presence of substantial adjacent vertical loads of 27,000 pounds influence not only the shape of the deflection curve but may also influence the measured response at the plane of combined vertical and lateral loading. The later laboratory tests have indicated that an error in estimating gage restraint capability from test data of 10-15% may result from such interactions . The source of error would be expected to increase with stiffer rail sections and would act to limit the sensitivity of the test vehicle to detect restraint anomalies.

TRACK STRENGTH FIELD TEST

TEST APPROACH:

- SET UP TEST CAR CONSIST ON SIDING
- SECURE TRACK FROM REVENUE TRAFFIC
- SURVEY TRACK WITH GAGE SPREADER
- SPOT TEST LOCATIONS AT LOW LOAD LEVELS
    - ANOMALIES
    - "AVERAGE" NOMINAL SITES
- LIMITED "PROOF" TEST
- STATISTICALLY SUMMARIZE TRACK SECTIONS

Figure 17

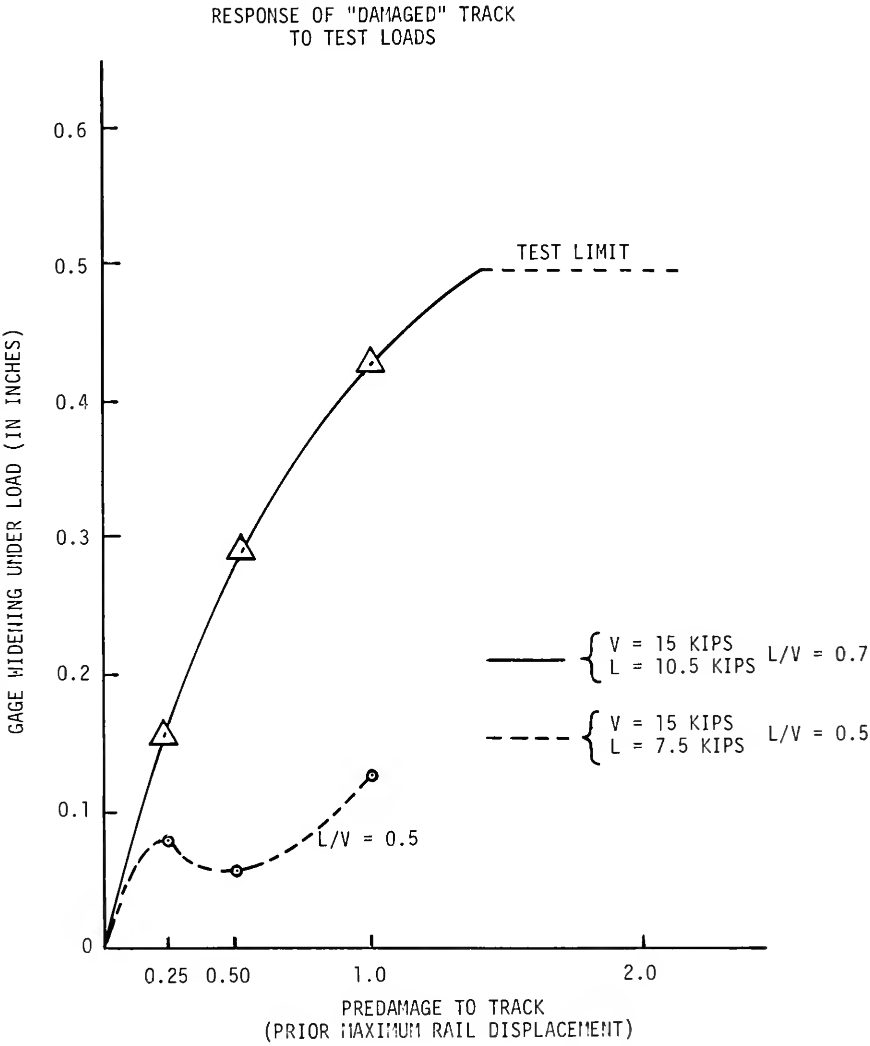

RESPONSE OF "DAMAGED" TRACK
TO TEST LOADS
GAGE WIDENING UNDER LOAD (IN INCHES)
0.6
0.5
0.4
0.3
0.2
0.1
0
TEST LIMIT
V = 15 KIPS
L = 10.5 KIPS
L/V = 0.7
V = 15 KIPS
L = 7.5 KIPS
L/V = 0.5
L/V = 0.5
0.25
0.50
1.0
2.0
PREDAMAGE TO TRACK
(PRIOR MAXIMUM RAIL DISPLACEMENT)

Figure 18

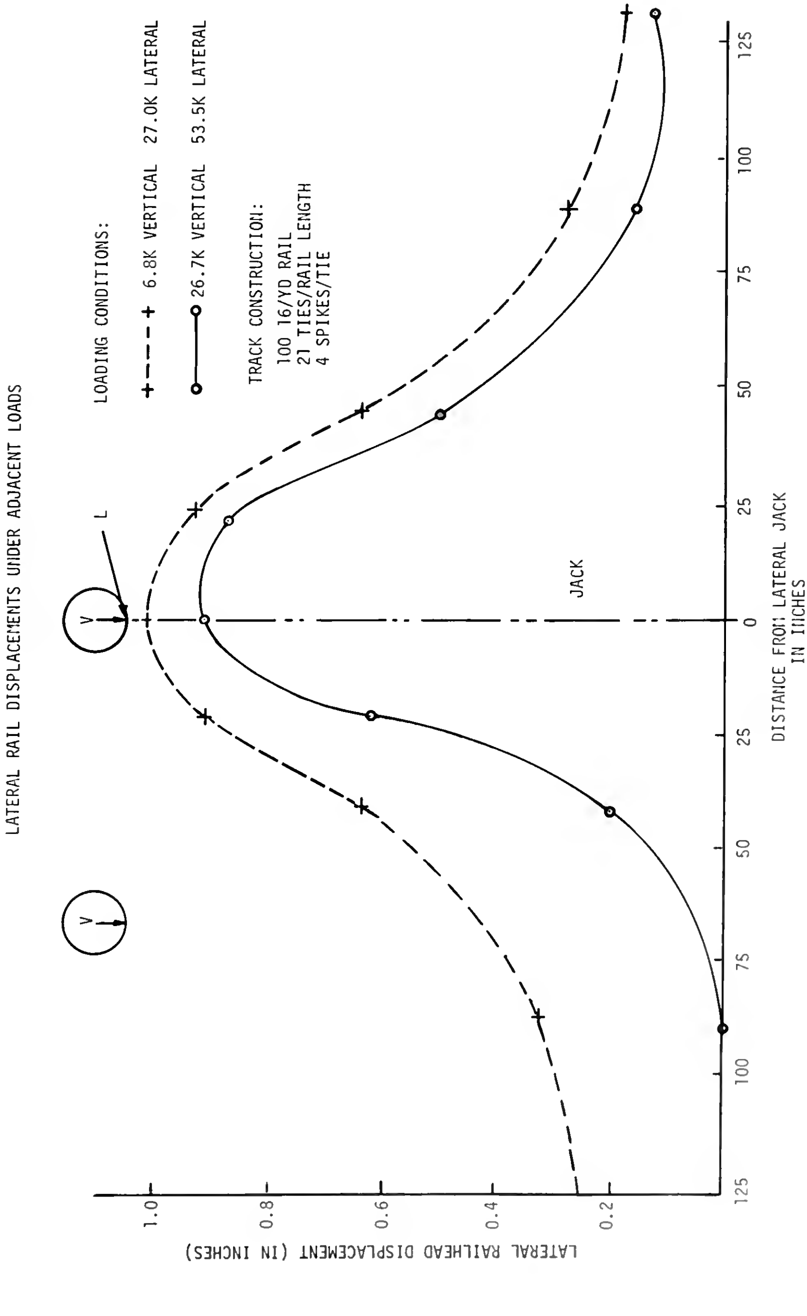

LATERAL RAIL DISPLACEMENTS UNDER ADJACENT LOADS
LOADING CONDITIONS:
6.8K VERTICAL 27.0K LATERAL
26.7K VERTICAL 53.5K LATERAL
TRACK CONSTRUCTION:
100 16/YD RAIL
21 TIES/RAIL LENGTH
4 SPIKES/TIE
V
V
L
JACK
LATERAL RAILHEAD DISPLACEMENT (IN INCHES)
1.0
0.8
0.6
0.4
0.2
125
100
75
50
25
0
25
50
75
100
125
DISTANCE FROM LATERAL JACK
IN INCHES

Figure 19

The analysis of deflection curves from gage widening tests indicates that a separation of 12 to 15 feet from the plane of load application to the nearest axle will virtually eliminate the bias introduced by adjacent loads for the range of track conditions likely to be encountered.

Based on these considerations, a candidate test vehicle configuration has been defined. A schematic of such a system for mounting on a standard box car is shown in Figure 20. The basic design incorporated the ability to run both survey and stationary controlled load-deflection tests. Current designs required only weldments to the car to mount the system which cables data back to an adjacent instrumentation car. The test car, now in construction, will be available for test work in late summer of 1979.

The candidate requirements for initial test sites are listed, in Figure 21. The segments of track listed are intended to cover a range of gage restraint characteristics while concentrating on the issue of variation of gage restraint with tie/fastener deterioration. Most critical is the pre and post timbering tests. This test sequence is essential in evaluating the ability of gage restraint testing to identify functionally required maintenance. Additional tests will permit correlation of restraint test data with track conditions such as rail weight, wear of the rails, the type and weathering of the track.

It should be noted that the interest in the rational testing of track performance is not limited to North America. Currently both British Railway Research and a major maintenance equipment manufacturer are involved in the development of such track tests. The vehicle shown in Figure 22 is the British Rail "Decapod" test car. It is intended primarily to evaluate lateral track restraint behavior in conjunction with the introduction of the Advanced Passenger Train. Testing with this car has been in progress for over a year.

4. Conclusions

Although it is very early in the program, the research into the characterization of track strength has indicated a number of preliminary conclusions which are shaping the course of continued research. The combination of these efforts in quantified performance requirements for track and the demonstration of their utility will require three to five year. Preliminary findings have been reached in the following areas:

- *Feasibility of Non-Destructive Testing*

  The laboratory data has shown that an indication of the restraint characteristics of the track may be obtained at test load levels substantially below the level of peak train loadings. This response does not appear to exhibit a substantial sensitivity to axial load. This finding is contrary to earlier reported results by Heron and Flassig (4). The extension of this data beyond laboratory test conditions is required to confirm these findings.

- *Applicability of Gage Spreading Testing*

  The preliminary test employing the fixed displacement gage spreader device suggests that an indication of deteriorated rail restraint capacity and the associated degraded track conditions may be obtained from a survey testing technique. Trends in rail restraint parameters comparable to the manually assessed track deterioration have been identified. The accuracy and fidelity of the test technique has yet to be determined against a range of track conditions. However a modification of the test procedure to constant load control will be necessary in future applications. Such evaluations are a prime concern of the field test activity planned for this year.

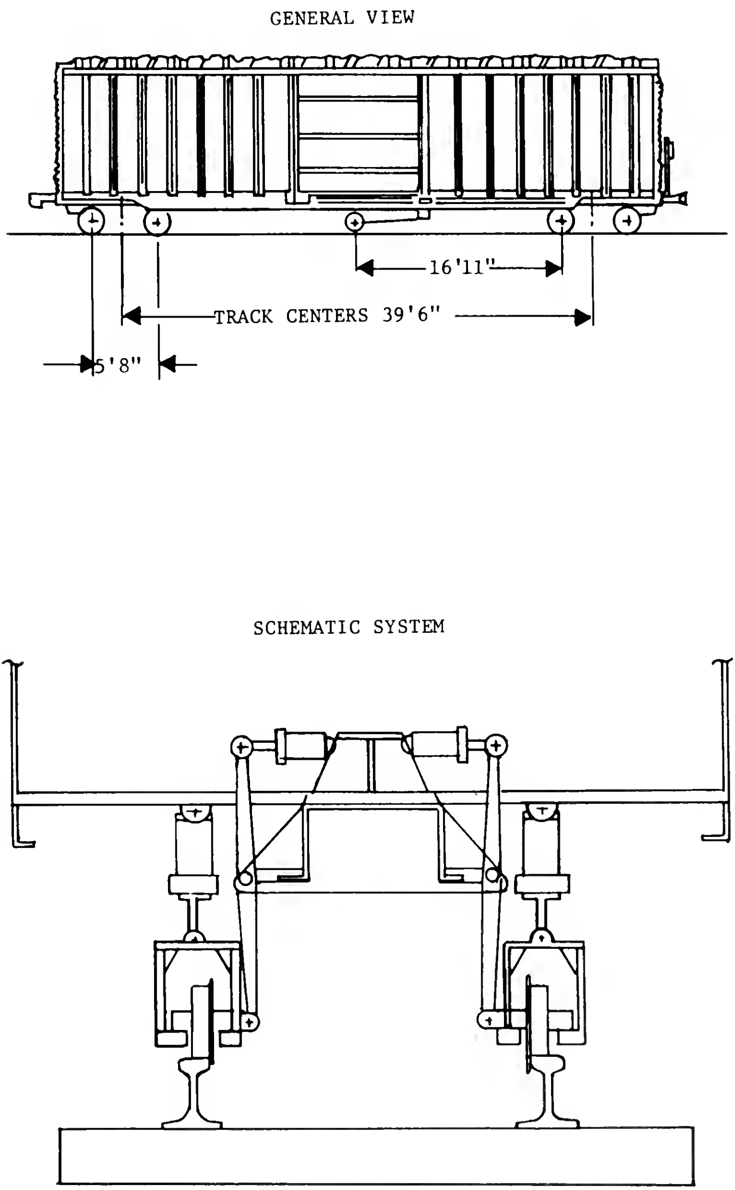


Figure 20

TRACK STRENGTH FIELD TEST

CANDIDATE SITE TYPES:

- MAINLINE TRACK SEGMENT (RAIL _ 130# / YD)
  - PRE AND POST TIMBERING
  - 3, 5, 7, YRS POST TIMBERING
- SECONDARY TRACK (_ 115# / YD)
- YARD TRACK

Figure 21

Figure 22

- *Test Vehicle Requirements*

  Laboratory and field data on rail deflection under combined vertical and lateral loads has suggested that a test system based on a controlled load test is feasible. Consideration of the prime issues of the interpretability of deflection data at non-destructive load levels and the interference of adjacent wheel loads with deflection data has been initiated.

Study of available deflection data indicates that the load levels of 15,000 pounds vertical and 10,500 lateral can serve as a base line for field rail restraint. Laboratory data showed a monotonic increase in deflection under these test loads prior to restraint damage. Similarly, lateral rail deflection curve data indicates that offset of the loading axle from adjacent axles by 12 to 15 feet is adequate to limit the influence of adjacent wheel loads.

Measurements of unloaded and loaded gage can then be used to identify rail deflection under load. Deflection under test loads would then be used as an index of rail restraint capacity.

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Dynamic Response of Concrete Railway Bridges

by William J. Venuti* and Frank J. Huebsch**

Concrete railway bridge designers are handicapped by lack of a rational impact formula which provides a reasonable estimate of impact effects of moving trains. The impact formula for reinforced concrete bridges recommended by the American Railway Engineering Association (AREA), which is based on dead and live loads only, is quite conservative. The impact formula for prestressed concrete bridges is based on field studies conducted by the Association of American Railroads (AAR).

The research presented herein is concerned with the dynamic load factor of concrete railway bridges when subjected to trains of various speeds. The theoretical study includes the effects of train speed, wheel loads, axle spacing, span lengths, bearing pads, bridge weight, and track modulus. Track irregularities, rail joints, wheel flats and transverse variations are not considered.

Data from 23 existing single span bridges of lengths from 7.6 m (25 ft.) to 43.3 m (142 ft.) was used to develop dynamic characteristics. Cooper's E-80 loading with common engine and car axle spacings and track moduli of 34.5 MPa (5000 lb/in^2), 68.9 MPa (10,000 lb/in^2), and ∞ (open deck bridges) was considered.

The results indicate that, for single span bridges, the dynamic load factor is not simply a function of live load, dead load, train speed, or span length but a function of the dynamic characteristics of both bridge and loading. The calculated dynamic load factors compare favorably with field data of prestressed concrete bridges obtained by the AAR. However, there is little correlation between the results of this study and the recommended AREA impact factor for reinforced concrete bridges.

Introduction

The dynamics of bridges under railway loading is a complex phenomenon which involves the effects of a wide range of parameters such as the dynamic characteristics of the span, loading configurations, dynamic properties of the track, vehicle dynamics, and the statistical aspects of the loading. Recent research findings (*1,2*) and the inadequacy of present design codes reflect the need to develop a more rational approach to impact loading consistent with modern design practice. Availability of highly versatile digital computer programs utilizing the finite element approach to the normal mode method of structural dynamics makes possible the investigation of the effects on response of a number of selected parameters with minimal computational effort. These parameters include: dynamic properties of the span, dynamic properties of the trackbed, velocity of the moving load, and track irregularities.

This investigation is aimed at obtaining simplified mathematical models which would allow the dynamic effects in concrete railroad bridges to be predicted with sufficient accuracy for design purposes, and serve as a basic guide for subsequent field testing.

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Method of Analysis

Many of the previous studies, (see for example, Fryba (*3*) for a comprehensive development and excellent bibliography) of the dynamic nature of structures subjected to moving load and variables affecting the problem have followed the classic assumptions of Inglis (*4*), or later refinements, to attempt closed form or numerical solutions to partial differential equations of motion for the distributed parameter system. Although treatment is broad in scope, the resulting differential equations are frequently nonlinear leading to solutions by numerical integration of the differential equations or by means of transformation. Findings available by this approach provide insight into the important parameters of the problem.

The alternative formulation using a finite element idealization of the structure approximates the motion in terms of a select number of nodal generalized displacement coordinates. If simplifications of the loading conditions are justified, time histories of response can be obtained by mode superposition utilizing commercially available structural dynamic analysis programs such as STRUDL-DYNAL (*5*).

Figure 1 (a) shows a Cooper's E-80 loading on a typical span. Figure 1 (b) shows the idealization used for a typical structure analyzed. Each span is modeled as a simply supported beam with discrete spring-mass elements representing the dynamic effect of a rail system supported by ties and ballast. The structure is thus divided into standard beam and axial bar elements by definition of nodal points whose displacement components are sufficient in number to provide a good definition of the deflected shape.

For a structure of this simplicity, using a consistent approach (distributed structural mass), with the number of dynamic degrees of freedom equal to the static degrees of freedom, is more efficient than use of a static condensation procedure (lumped-mass matrix). Additionally, non-structural mass contributions due to ballast, rail, and vehicle, which are lumped at appropriate nodal freedoms, are added to the respective diagonal positions of the mass matrix. Consistent with the usual assumptions made for analysis of dynamic response of long span bridges to the passage of vehicles, the inertial effect of the moving mass is considered to be separable from the gravitational effect. In this manner the motion is approximated by the response of a beam with fixed vehicle mass position and subjected to a moving force system. For a moving lumped mass the variation in response frequency is smallest when its position is at midspan (*3*).

In this analysis, an evaluation of dynamic properties of mode shapes and frequencies is made with the assembled equations of motion for free vibration. A transient response for each case of externally applied nodal loading then proceeds by the mode superposition method, assuming the motion to be composed of its modal contributions and the uncoupled equations of motion solved by the Duhamel integral, using a modal damping ratio at 2 percent of critical damping. Modal results are then combined to obtain midspan dynamic deflection time histories from which impact is found as the maximum dynamic deflection expressed as a percentage of maximum static deflection due to live load.

In the study of the basic causes of dynamic effects in railway bridges and the determination of impact effect for design purposes, the use of simplified models with results obtained for systematic variation of the parameters studied by means of available digital computer programs are found to be highly practical. For a complete analysis of response, these programs are ordinarily limited to linear elastic structures subject to dynamic loads that can be defined in terms of time histories.

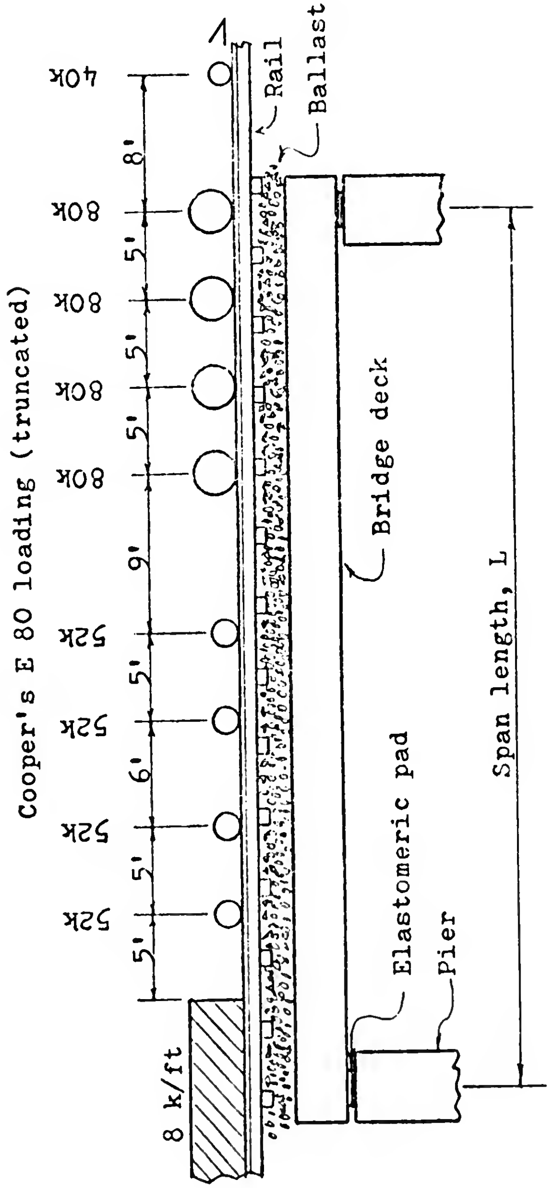


FIG. 1(a) - LOADING AND MODEL

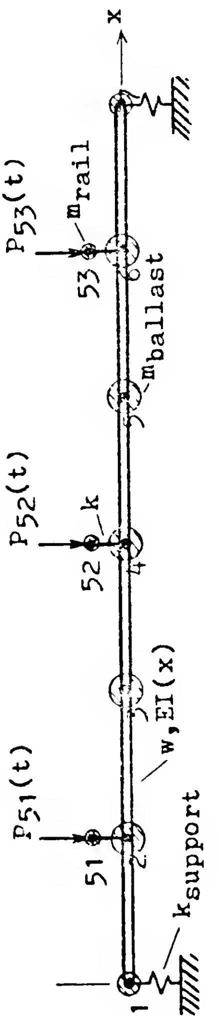


FIG. 1(b) - IDEALIZATION AND PROPERTIES

BRIDGE PARAMETERS

The paramenters considered are as follows:

Span Length, L: 7.6, 15.2, 22.9, 30.5, 34.8, & 43.3 m
(25, 50, 75, 100, 114, & 142 ft.)

Velocity, c: 32, 64, 97, 129, & 161 km/h
(20, 40, 60, 80, & 100 mph)

Ballast Foundation Modulus, k: 34.5 MPa, 68.9 MPa, & ∞
(5000 lb/in^2, 10,000 lb/in^2, & ∞)

Frequency Parameter, $\sqrt{\frac{wL^4}{EI}}$:
2.9, 4.8, 9.6, & 14.5 for span lengths of
7.6, 15.2, 22.9, & 30.5 m, respectively
or: 1.8, 3.0, 6.0, & 9.0 for span lengths of
25, 50, 75, & 100 ft., respectively.

Member properties for each model were obtained by analysis of gross section properties from actual designs of 23 existing prestressed concrete or reinforced concrete railway bridges of spans ranging from 6.7 m (22 ft.) to 43.3 m (142 ft.) (Table 1). Although more data was available for short and medium spans, a relation between frequency parameter (which is proportional to the theoretical fundamental period) and span length was found to be possible with relatively low scatter for prestressed concrete bridges. A conservative fit which was used to obtain section moments of inertia, I, span lengths, L, modulus of elasticity, E, and section unit weights, w, is shown in Figure 2.

The replacement of a continuous rail on a uniform elastic foundation with an equivalent single degree of freedom spring-mass system is based on fundamental mode shapes and frequencies. The assumed fundamental mode shape of an elastically supported rail is taken to be that portion of the resulting deflected curve between points of zero slope when a statically applied concentrated force is applied at the midpoint. This wave length is $2\pi/\beta$, where $\beta = 4\sqrt{k/4EI}$ (*6*) (k and EI corresponding to the foundation modulus and rail flexural stiffness, respectively) which neglects the velocity effects of moving loads as discussed in Volterra and Zachmanaglou (*7*). The tributary length for lumped track mass is thus π/β, which for a 136 CF&I rail is approximately equal to 2.5 m (100 in), and the discretized spring constant is taken to be $k\pi/\beta$. The fundamental frequency of the equivalent spring-mass system is roughly equal to that of a beam on an elastic foundation with fixed boundary conditions.

Foundation moduli taken at 34.5 MPa (5000 lb/in^2) and 68.9 MPa (10,000 lb/in^2) were found to be in reasonable agreement with resilience formulas for the modulus of an elastic layer on a rigid plane with a depth of 8 in. The unit weight of the ballast material was assumed at 1920 kg/m^3 (120 lb/ft^3) distributed over a 380 mm (15 in) depth for the full width of the bridge deck.

Vehicle Parameters

A major complexity involved in obtaining transient response to various conditions of moving loads by this method is the complete description of the external load as it passes over each equivalent spring-mass system. The region of influence for each equivalent nodal point is assumed equal to the discrete spring spacing and the intensity of concentrated force is

TABLE 1 SUMMARY OF GROSS SECTION PROPERTIES

Bridge	Type	f'_c (MPa)	E_c (MPa)	Span, L (m)	Width, b (m)	w (kg/mm)	I (m^4)	$\sqrt{\frac{wL^4}{EI}}$
1	R/S	24.1	24,500	6.7	4.0	8.8	0.081	2.99
2	P/S	34.5	29,600	7.3	4.0	6.9	0.099	2.59
3	P/S	48.3	36,500	7.6	4.0	7.5	0.100	2.62
4	P/S	34.5	29,600	8.2	4.0	7.8	0.115	3.24
5	P/S	34.5	29,600	9.1	6.1	16.7	0.294	3.78
6	P/S	34.5	29,600	9.1	5.2	8.5	0.127	3.97
7	P/S	48.3	36,500	11.4	4.0	9.2	0.220	4.42
8	P/S	34.5	29,600	12.2	17.1	30.4	1.022	4.71
9	P/S	34.5	29,600	12.8	5.5	11.1	0.458	4.70
10	P/S	34.5	29,600	13.7	4.0	7.6	0.296	5.52
11	P/S	34.5	29,600	15.2	5.0	10.9	0.719	5.26
12	P/S	34.5	29,600	16.2	4.0	9.0	0.612	5.82
13	P/S	34.5	29,600	18.3	4.0	9.1	0.624	7.42
14	P/S	34.5	29,600	18.3	4.0	9.0	0.641	7.27
15	P/S	34.5	29,600	21.3	4.0	9.4	0.782	9.14
16	P/S	34.5	29,600	21.3	4.0	9.4	0.782	9.14
17	P/S	34.5	29,600	22.9	6.1	13.3	1.307	9.69
18	P/S	34.5	29,600	26.2	4.0	10.3	1.292	11.28
19	P/S	34.5	29,600	26.8	5.2	15.8	2.123	11.39
20	P/S	34.5	29,600	30.2	5.2	17.9	2.294	14.75
21	P/S	34.5	29,600	30.5	4.6	17.3	2.256	14.93
22	R/C	27.6	26,200	34.7	12.5	43.8	16.41	12.18
23	R/C	27.6	26,200	43.3	6.1	39.0	14.69	18.84

Note: 1 MPa = 145 lb/in.^{2}
1 m = 3.28 ft.
1 kg/mm = 56 lb/in.
1 m^4 = 2.40 x 10^6 in.^{4}

Frequency Parameter, $\sqrt{\frac{wL^4}{EI}}$ = FP

1.61*FP (in U.S. Customary Units) = FP (S.I. Units)

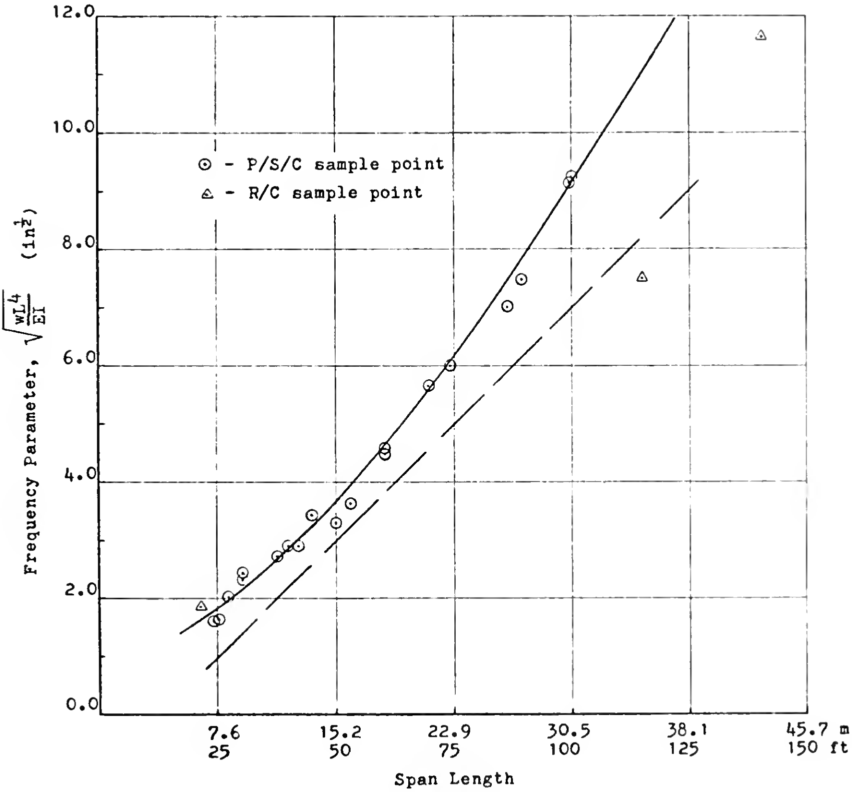


FIG. 2 - FREQUENCY PARAMETER DATA FOR 23 EXISTING CONCRETE RAILWAY BRIDGES

1.61 xFP (in U.S. Customary Units) = FP (S.I. Units) assumed to be distributed linearly over the region. The loading time history can then be specified in terms of magnitude-time pairs from which intermediate values are linearly interpolated. A more complete description of the loading conditions investigated is described within the following presentation of results obtained under design loading, unit train loading, and test train loading.

Results For Design Loading

A complete study was first made to investigate the nature of the dynamics of prestressed concrete railway bridges subject to moving loads, concentrated and uniformly distributed. Externally applied loads were assumed on the basis of current AREA design specifications for live load, i.e., Cooper's E-80 axle loading as recommended by AREA (*8*). Since maximum values of impact for short spans would be expected to be primarily due to the application of the first set of 36,000 kg (80k) axle loads specified, the locomotive axle loading was truncated at the second 23,000 kg (52k) axle load and thereafter replaced by the uniform car loading of 12,000 kg/m (8k/ft). Modal analyses and transient results were obtained for spans ranging from 7.6 m (25 ft) to 43.3 m (142 ft) with applied forces of constant magnitude moving at constant velocities of 32, 64, 97, 129, and 161 km/h (20, 40, 60, 80, and 100 mph). The effects of changes in frequency parameter, elastic properties of track support, and type of support for the span were also considered.

From the results of modal analyses of all spans the natural frequencies for the first four modes are presented in Table 2 for values of foundation moduli k = 34.5, 68.9 MPa,and ∞ (5,000, 10,000 lb/in/in, and ∞). The first three frequencies of transverse beam vibration f_1, f_2, f_3 are slightly effected by the elastic foundation and to a lesser extent for increasing span lengths, while spring-mass excitation of the track system occurs at frequencies near f_s. In subsequent transient analyses the first three modes (lowest three frequencies, ignoring longitudal modes f) were used to obtain the results. It should be noted that no account was made of the effect of vehicle mass on beam vibrations under design loading, and the above frequencies are the natural frequencies of the unloaded systems. The approximation to the loaded system is usually considered realistic for longer spans, those in excess of 30.5 m (100 ft) for which the ratio of live load to dead load is small.

Transient results for the truncated Cooper's E-80 locomotive axle loading and uniform car loading for tne 7.6 m (25 ft) span, as shown in Figure 1 (a), are displayed in Figure 3 for concentrated loading and Figure 4 for uniform loading, as a percentage of maximum static deflection. The dependence of impact on elastic layer stiffness, generally effecting both frequencies and mode shapes, tends to diminish with span length. For constant moving force inputs, a more flexible foundation modulus tends to increase impact, excluding consideration of track and wheel irregularities. The effect of elastic supports on mode shapes and frequencies is greater, but seems to influence impact in short and long spans differently.

A typical response time-history plot of midspan dynamic deflection is illustrated in Figure 5 for the design load traversing the 7.6 m (25 ft) span for k = 68.9 MPa (10,000 lb/in/in) at 97 km/h (60 mph), which shows the relative influence of forced (static component) and free (cyclic component) vibration on impact. At most velocities of moving load considered herein, dynamic response is primarily due to fundamental mode excitation.

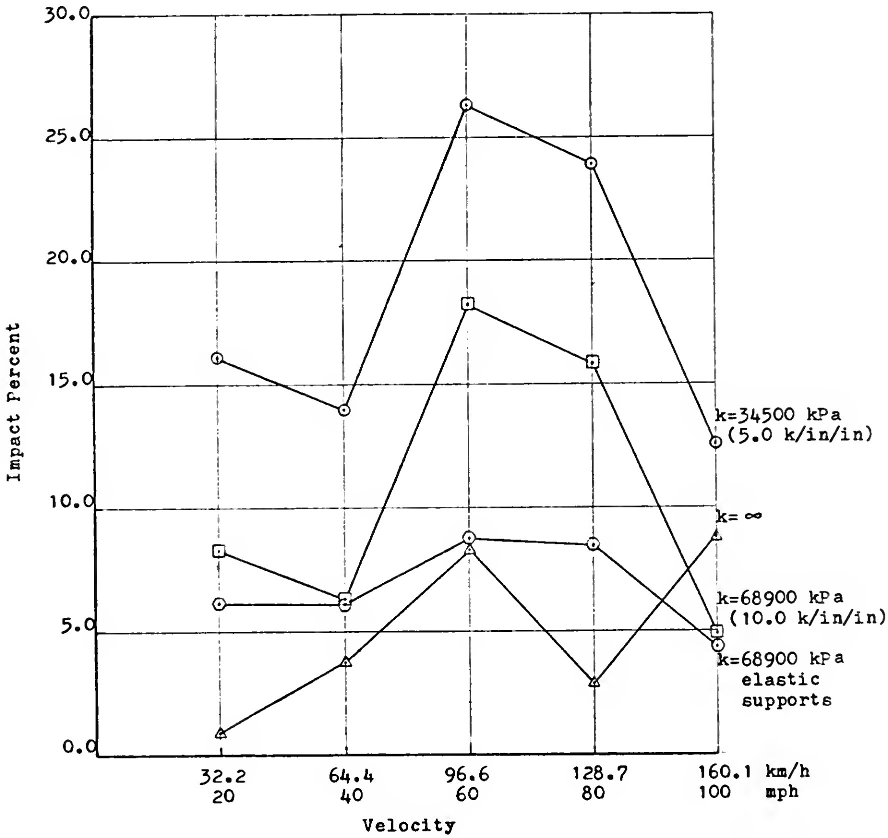
For design purposes the important parameter is span length, which can be assumed to be related to the fundamental natural period of the structure. A plot of impact results obtained for each velocity of moving load considered is shown in Figure 6. Since it is not practical to design bridges for specific operating speeds, it is desirable to establish a maximum impact vs. span length envelope for velocities less than or equal to an established design limit. This envelope is shown on Figure 6 as a broken line which decreases with span length, although it is expected that many more velocity points would be required to establish a well defined limit.

TABLE 2 MODAL RESULTS

			Natural frequencies, Hz				
Span (m)	$\sqrt{\frac{WL^4}{EI}}$	k (MPa)	f_1	f_2	f_3	f_s	f_ℓ
7.6	2.9	34.5	17.36	62.56		65.33	
		68.9	17.37	69.73		91.38	
		68.9*	16.30	54.33		85.78	
		inf.	21.33	84.16	178.85		115.53
15.2	4.8	34.5	10.24	40.30		65.12	57.77
		68.9	10.25	40.43	85.52		57.77
		inf.	13.03	51.16	111.85		57.77
22.9	8.1	34.5	6.16	24.40	53.49		38.45
	9.7	34.5	5.13	20.42	45.29		38.45
		68.9	5.13	20.43	45.49		38.45
		68.9*	5.04	18.97	38.39		38.45
		inf.	6.21	24.59	54.48		38.45
30.5	14.5	34.5	3.42	13.62	30.34		28.82
		68.9	3.42	13.62	30.36		28.82
		inf.	3.92	15.57	34.65		28.82
34.8	11.8	68.9	4.20	16.62	36.68		23.76
43.3	18.5	68.9	2.67	10.59	23.56		19.12

See Table 1 for Conversion Factors

*Elastically supported



Span Length - 7.6 m (25 ft)

Frequency Parameter - 2.90 (1.80)

Cooper's E 80 Loading (truncated) -

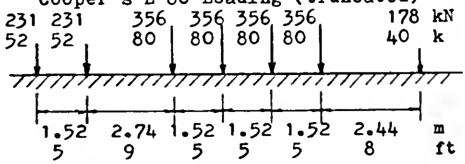
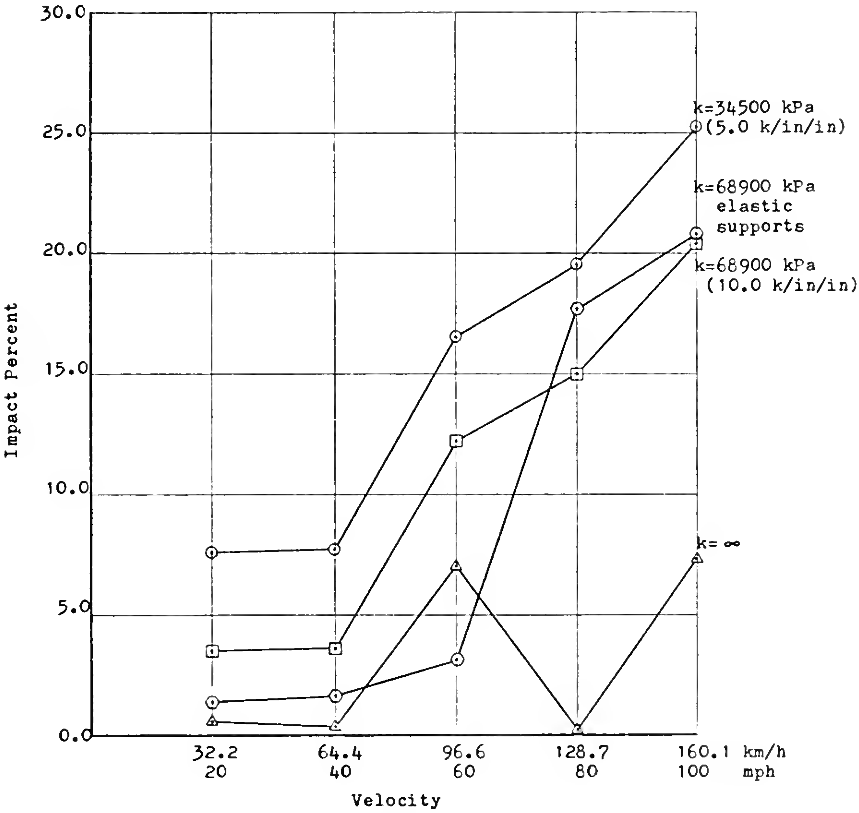


FIG. 3 - EFFECT OF BALLAST, ELASTIC SUPPORTS AND VELOCITY ON IMPACT



Span Length - 7.6 m (25 ft)

Frequency Parameter - 2.87 (1.80)

Cooper's E 80 Loading (unif. distrib.)

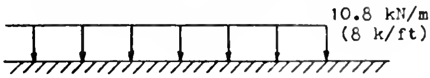


FIG. 4 - EFFECT OF BALLAST, ELASTIC SUPPORTS AND VELOCITY ON IMPACT

Comparison with the AREA impact formulas, for reinforced concrete bridges (which is based on dead and live loads only), and for prestressed concrete bridges (which decreases with increasing span lengths), shows that the componeent which is due primarily to the effects of moving design load without consideration of track and wheel irregularities may only be a small part of these conservative impact factors.

Results For Unit Train Loading

In order to comparatively assess impacts attainable on bridges subjected to actual loads, a study of steady-state response due to the passage of a unit train (158T) of hopper cars was undertaken for span lengths of 7.6 m to 43.3 m (25 ft to 142 ft). Velocities of 16, 32, 64, 97, 129, and 161 km/h (10, 20, 40, 60, 80, and 100 mph) were considered for the unit train idealized as a system of constant moving axle forces, in which axle loads and spacings, taken from Southern Pacific specifications, were converted into nodal load time histories. A short auxiliary Fortran program was used to generate the runs. Since structural models used are idealizations of existing railroad bridges, it is hoped that these results can be verified by field tests.

The approximate effect on response of the variation in fundamental period caused by the change in mass distribution as a train passes over the bridge is most pronounced for short spans where the ratio of live to dead load is greatest. When the train is traversing the span and before it completely departs, the period of response is slowed by the addition of the full train mass, which tends to increase lower velocity excitations. Hereafter, a full train mass based on average distributed vehicle mass allowances, was universally used to account for this phenomenon.

Another major influence on the complete response of bridges to a moving unit train of hopper cars are the initial conditions taken when the cars arrive on the span. Within the limitations of this type of analysis, the variation of bridge impact by a passing unit train with a locomotive was investigated at times just after the response to the locomotive (under first two cars) and at the time when steady-state conditions have developed (after passing of several cars). Since few oscillations are generally required to reach steady-state vibration, this condition was simulated by using a loading configuration of two hopper cars preceded and followed by the immediate trucks of adjacent cars. Although the pseudo steady-state loading configuration achieved somewhat lower maximum impact than obtained for the configuration with the locomotive, it was felt that the deviation was not sufficiently signficant to justify the 50% increase in computer (CPU) time for the longer integration period required to include the locomotive. Even assuming a slightly larger impact would be felt once by a structure during the passing of a train, many cycles of steady-state oscillations would be experienced for a long train.

Thus, pseudo steady-state responses were obtained for the remaining ballasted spans considered using full train mass allowance and a loading configuration representing the axle loads of tandem hopper cars in a unit train. Only the fundamental mode was generally used since higher modes for constatnt force systems moving at the velocities considered herein, are not excited, and have negligible contribution. The impact percentages are summarized in Table 3 along with the dimensionless frequency ratios $c/2f_1L$, computed at each velocity, c, for a given span of length, L, and fundamental response frequency, f_1. These data are then plotted for all spans in Figure 7 from which can be observed a general pattern in the results obtained with each span for the assumed loading configuration, characterized as follows:

(1) a local maximum impact occurring for low frequency ratios ($c/2f_1L < 0.15$) which generally prevails for short spans;

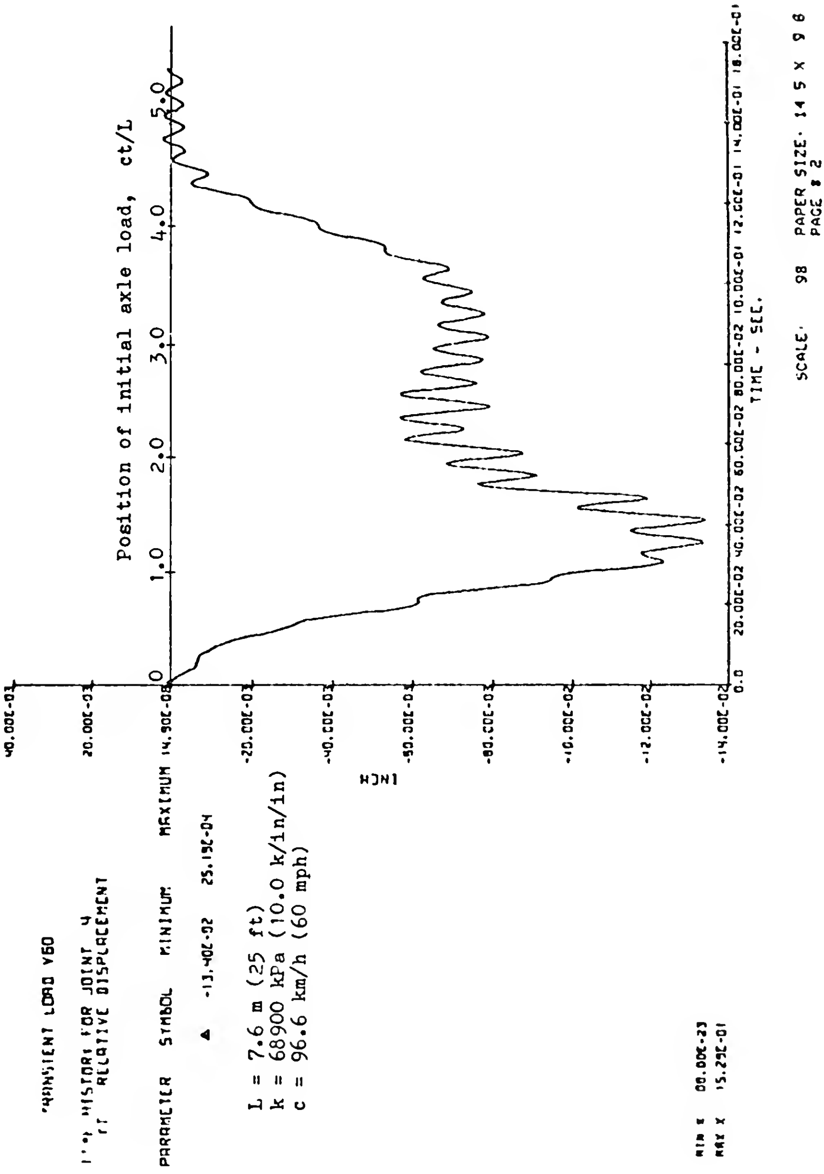
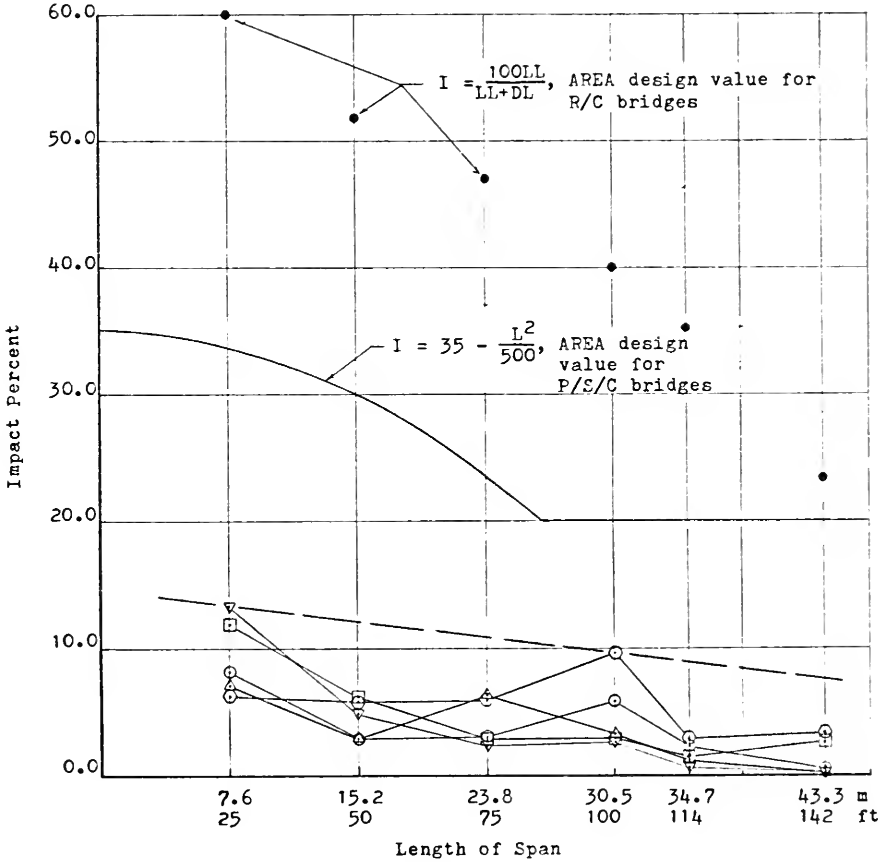


FIG. 5 - MIDSPAN DEFLECTION TIME HISTORY, COOPER'S E 80 LOADING
(1 in = 25.40 mm)



⊙ - Velocity of design load = 32.2 km/h (20 mph)
△ - Velocity of design load = 64.4 km/h (40 mph)
▽ - Velocity of design load = 96.6 km/h (60 mph)
⊡ - Velocity of design load = 128.7 km/h (80 mph)
⊙ - Velocity of design load = 160.1 km/h (100 mph)

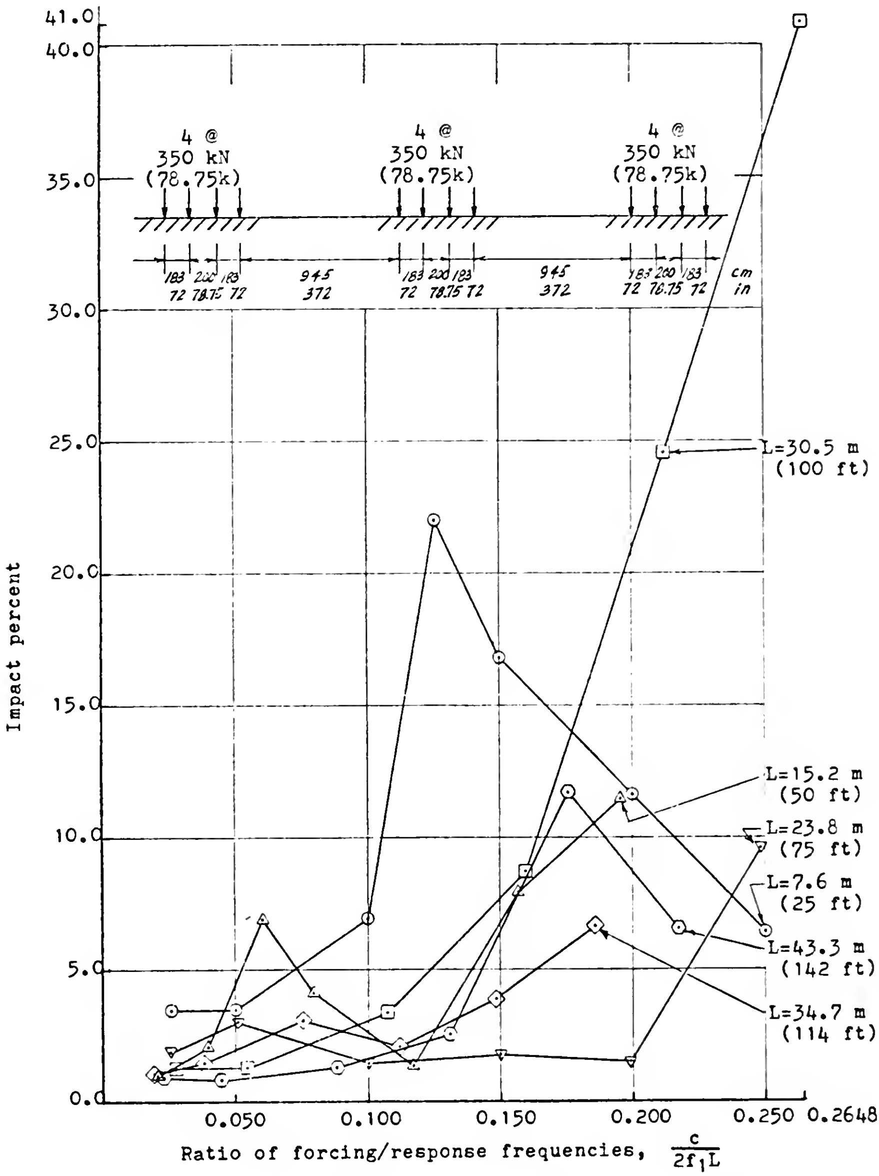
FIG. 6 - SUMMARY OF IMPACT DATA FOR DESIGN LOADING

TABLE 3

Frequency Ratio, $\frac{c}{2f_1L}$							
Span (m)	f_1	velocity, c(km/h)					
		16	32	64	97	129	161
7.6	11.734	0.0250	0.0500	0.1000	0.1500	0.2000	0.2500
15.2	7.538	0.0195	0.0389	0.0778	0.1167	0.1557	0.1946
22.9	3.936	0.0248	0.0497	0.0994	0.1490	0.1987	0.2484
30.5	2.769	0.0265	0.0530	0.1059	0.1589	0.2119	0.2648
34.8	3.480	0.0185	0.0370	0.0739	0.1109	0.1479	0.1848
43.3	2.389	0.0217	0.0433	0.0867	0.1300	0.1733	0.2167

IMPACT PERCENT								
	velocity, c(km/h)							
Span (m)	16	32	48	64	80	97	129	161
7.6	3.38	3.47	-	6.85	22.10	16.80	11.68	6.47
15.2	1.01	2.09	6.91	4.17	-	1.37	7.99	11.51
22.9	1.88	3.00	-	1.46	-	1.76	1.55	9.63
30.5	1.34	1.24	-	3.32	-	8.73	24.59	40.99
34.8	1.03	1.42	-	3.13	-	2.05	3.97	6.65
43.3	0.94	0.78	-	1.28	-	2.58	11.75	6.64

1 ft. = 0.305m 1 mph = 1.61 km/h



Span Lengths - 7.6 to 43.3 m (25 to 142 ft)

Ballast Moduli - 68900 kPa (10.0 k/in/in)

FIG. 7 - SUMMARY OF IMPACT DATA FOR UNIT TRAIN LOADING

(2) impact then falling off for increasing velocities until a local minimum is achieved at midrange frequency ratios;

(3) at higher velocities, increasing impact to a maximum which prevails for longer spans.

Even though it may be somewhat questionable whether the large impacts obtained at higher velocities would be realized in field tests, since the feedback of the vehicle mass has been neglected in this moving force analysis, these results can, perhaps, be interpreted as a reasonable upper bound on actual impacts experienced for this type of loading.

The observed behavior of maximum response occurring at lower velocities with short spans and at high velocities with longer spans can be qualitatively explained in terms of the cumulative effect of successive axle loads moving across the bridge, in which the superimposed response may be strongly influenced by the axle spacing.

The response of a uniform beam to a single concentrated moving force is known from the theory of vibration of continuous systems as is developed in Timoshenko and Weaver (9). The dynamic amplification factor depends most importantly on the frequency ratio, $c/2f_1L$, and to a lesser extent on damping, initial conditions, and the phase relation between free and forced vibration. The oscillations due to consecutive axle loads spaced at a constant distance, d, tend to build up when the loads are in phase with the fundamental frequency, such that $f_1d/c = n$, for $n = 1, 2, \ldots$, and tend to interfere when the loads are out of phase with the fundamental frequency, such that $fd/c = n/2$, for $n = 1, 3, 5, \ldots$

The above simplification suggests that local maximum impacts occur at velocities which are a function of only the axle spacing, d, and response frequency, f_1. Using the predominant spacing of approximately 1.8 m (72 in) for the four axle loads of adjacent coupled trucks, these optimal velocities (max. and min.) are computed for each span in Table 4(a). For example, a local maximum impact is expected to occur at a velocity equal to or slightly greater than 77 km/h (48 mph), for the 7.6 m (25 ft) span and a minimum impact is expected to occur at 154 km/h (60 mph), which agrees well with the results obtained. Although this analysis appears to be meaningful only for short spans, it serves to demonstrate the importance of closely controlled velocities in making field measurements.

For long spans the cumulative effect of successive moving resultants for axle groups of adjacent trucks moving at high velocities for which $f_1d/c << 1$, can produce a similar resonance phenomenon. Using instead, the spacing of such axle groups, D, in the above analysis, axle groups are found to be in phase with the fundamental frequency when $f_1D/c = n$, for $n = 1$, and so on. Table4(b) shows optimal velocities that occur for long span bridges, which are applicable for velocities greater than a critical velocity based on the overall axle group dimension.

Thus, the computed results for the unit train show a high degree of dynamic response is possible with heavier loading configurations which display regularity in distribution. Optimal velocities for such loading can be predicted knowing only the fundamental response period of the structure and the predominant spacing of the wheel loads.

Test Train Results

A further aspect of the applicability of results obtained by using detailed calculation methods based on gross idealizations toward establishing a practical design impact formula is the need for empirical verification by experimental measurement. Although the complexities of a systematic field investigation are beyond the scope of this phase of study, some demonstration of the validity of the methods used was considered desirable. It has been shown that

TABLE 4 OPTIMAL VELOCITIES - CUMULATIVE AXLE EFFECT

(a) predominant single axle spacing, d = 1.83 m					
Span (m)	f_1 (Hz)	In phase (Max) velocities $c = \frac{f_1 d}{n}$ (km/h)			
		n = 1	n = 2	n = 3	n = 4
7.6	11.734	77	39	26	19
15.2	7.538	50	25	17	
22.9	3.936	26	13		
30.5	2.769	18			
34.8	3.480	23			
43.3	2.389	16			

Span (m)	f_1 (Hz)	Out of phase (Min) velocities $c = \frac{2f_1 d}{n}$ (km/h)			
		n = 1	n = 3	n = 5	n = 7
7.6	11.734	154	51	31	22
15.2	7.538	99	33	20	
22.9	3.936	52	17		
30.5	2.769	37	12		
34.8	3.480	46	15		
43.3	2.389	31			

(b) predominant axle group spacing, = D = 15.1 m				
Span (m)	f_1 (Hz)	Critical Velocity (km/h)	Max. Veloc. $\frac{f_1 D}{n}$	Min. Veloc. $\frac{2f_1 D}{n}$
7.6	11.734	478		
15.2	7.538	307		
22.9	3.936	160	214	428
30.5	2.769	113	151	301
34.8	3.480	142	189	378
43.3	2.389	97	130	276

1 ft = 0.305 m 1 mph = 1.61 km/h

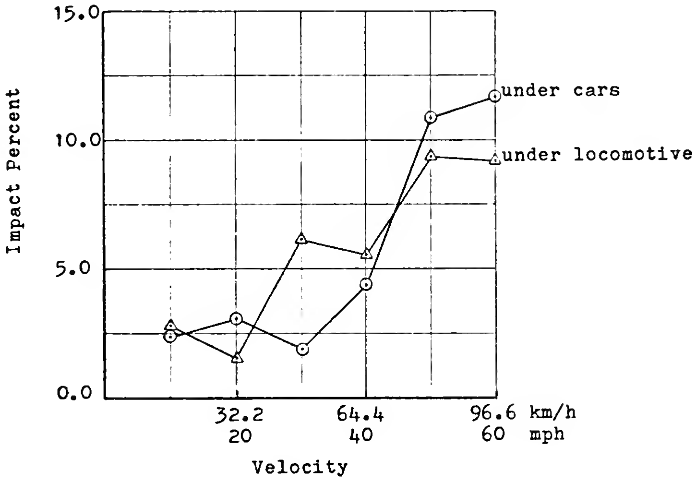
the response for a given span depends primarily on the forcing/response frequency ratio ($c/2f_1L$), and that for multiaxial load configurations the magnitude and spacing of the axle loads has an important effect on the extent to which each axle contributes to the total response. In a normal loading environment, of course, the variation in these parameters would be considered random variables, as are other sources of input including rail joints and wheel irregularities.

For the purpose of simulating test train runs within the limitations of a moving force analysis, nodal load time histories were generated based on the wheel load data for the test train used in the "Field Investigation of Prestressed Concrete Beams and Piles on the Western Pacific Railroad" (*10*) for velocities of 16, 32, 48, 64, 80, and 97 km/h (10, 20, 30, 40, 50, and 60 mph). As described in that report the test train consisted of two diesel-electric locomotives, five loaded cars (two cars of 44,000 kg (50 tons) and three cars of 62,000 kg (70 tons) nominal capacity, and a caboose), for which the scale weight of each car was assumed equally distributed over the four axles. In the interest of economy of computer time, the train loading used in the analysis herein was truncated at the fourth car, inasmuch as the adjacent trucks of the third and fourth cars were considered most severe. The test span is actually a 7.6 m (24 ft 11 in) approach span of a 3-span prestressed concrete bridge carrying a single track on a ballast deck, which was assumed to be reasonably represented by the 7.6 m (25 ft) beam model, with a ballast modulus of 68.9 MPa (10,000 lb/in/in) and full train mass allowance. The approach span is constructed of six 7.6 m (24 ft 11 in) precast concrete beams having a 0.71 m (28 in) square cross-section with 0.41 m (16 in) diameter void, and pretensioned with 38 9.5 mm (3/8 in) ASTM A-416 strands. The beams are placed on 13 mm (1/2 in) thick neoprene bearing pads and post-tensioned transversely at the midspan diaphragm and end blocks. Shear keys are grouted for load distribution. Gross-section properties based on design information furnished in the report are included in Table 1 (Bridge No. 3) and compared with the model properties (Figure 1 b) below:

Property	*W. P. Bridge*	*Model Span*
span length, L	7.6 m	7.6 m
unit weight, w	7.5 kg/m	7.5 kg/m
section area, A	1.88 m^2	1.91 m^2
moment of inertia, I	0.100 m^4	0.102 m^4
compressive strength, f'_c	48.3 MPa	34.5 MPa
modulus of elasticity, E_c	36,500 MPa	29,600 MPa
section width, b	4.0 m	4.0 m
ballast depth	380 mm	380 mm
frequency parameter, $\sqrt{\frac{wL^4}{EI}}$	2.62	2.90

(See Table 1 for conversion factors)

Impact results obtained from transient analyses for the test train loading configuration shown in Figure 8 are somewhat smaller in magnitude than results obtained for the more regular loading configuration of the unit train. Maximum dynamic midspan deflections, which generally occurred at times when the final locomotive wheel and the last wheel of car 3 were positioned over midspan, tend to increase with velocity. Impacts are somewhat greater under cars at higher velocities as shown, although static midspan deflection is slightly (3%) greater under locomotive loading. These results are summarized as follows:



Span Length - 7.6 m (25 ft)

Fundamental Frequency - 11.734 Hz

Ballast Modulus - 68900 kPa (10.0 k/in/in)

Test Train Loading -

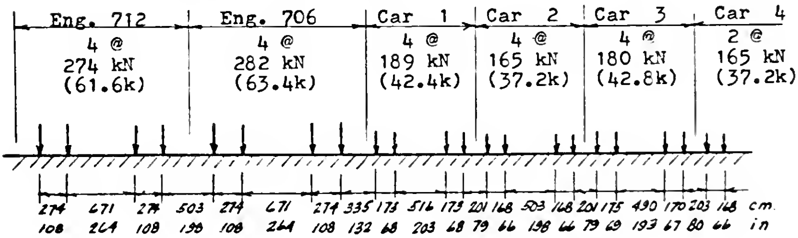


FIG. 8 - IMPACT PERCENT VS. VELOCITY FOR TEST TRAIN LOADING

VELOCITY	FREQUENCY RATIO	IMPACT PERCENTS	
c, km/h*	$c/2f_1L$	under cars	under locomotive
16	0.0250	2.38	2.77
32	0.0500	3.08	1.55
48	0.0750	1.91	6.19
64	0.1000	4.36	5.53
80	0.1250	10.82	9.32
97	0.1500	11.63	9.16

*1.61 km/h = mph

The maximum impact under cars obtained at 97 km/h (60 mph) might well be exceeded at an intermediate velocity between 80 km/h (50 mph) and 97 km/h (60 mph), and greater impact values would be expected under the locomotives for velocities in excess of 97 km/h (60 mph) due to the larger axle spacing. However, since the mass of the train of cars is distributed more uniformly and effects less change on the response frequency while traversing than the locomotives, the results under cars should be more realistic.

On Figure 9 the computed impacts (line plot) for the 7.6 m (25 ft) span produced under car loading by each test train run are compared with maximum recorded impacts (solid point) per rail of the 7.6 m (24 ft 11 in) test span. As discussed in the AAR report these impacts are based on the increase in average strain at a particular velocity over the average strain at slow velocities of less than 16 km/h (10 mph), as measured with two SR-4 strain gages applied to the bottom of each beam at a distance of 0.3 m (1 ft) from the midspan diaphragm. The maximum recorded impact per rail is the average impact for the beams under each rail. The total impacts as observed include the effects of roll, track and wheel irregularities, as well as velocity of the moving load. Since the scatter in the test points above the computed results is somewhat larger for the exterior beams, the higher observed values are presumed to reflect load distribution and unsymmetrical sources of loading input. The maximum recorded impact under cars was 24.8 percent at 72 km/h (45 mph) with corresponding tensile strain well below strain due to precompression of the bottom fibers. The moderate increase in impact with increasing velocity agrees with the characteristics of recorded values.

Conclusions

Some of the basic causes of dynamic effects in railway bridges when subjected to moving loads can be investigated by use of simplified models. The results of this study indicate that, for single span bridges, the dynamic load factor is not simply a function of live load, dead load, or span length, but depends principally on the fundamental response frequency of the structure, the velocity of moving load, and the magnitude and spacing of axle loads. For short spans, the response is also influenced by the presence of a ballast layer on the bridge and bearing pads at the supports, although it is certain that these components serve to reduce the effect of wheel and track irregularities.

Rail joints, wheel flats, and transverse effects associated with the vehicle dynamics represent random sources of load input that make up a relatively large contribution to total impact. However, there appears to be much evidence to justify further research into the development of a more rational impact formula than is currently recommended for design of concrete railway bridges.

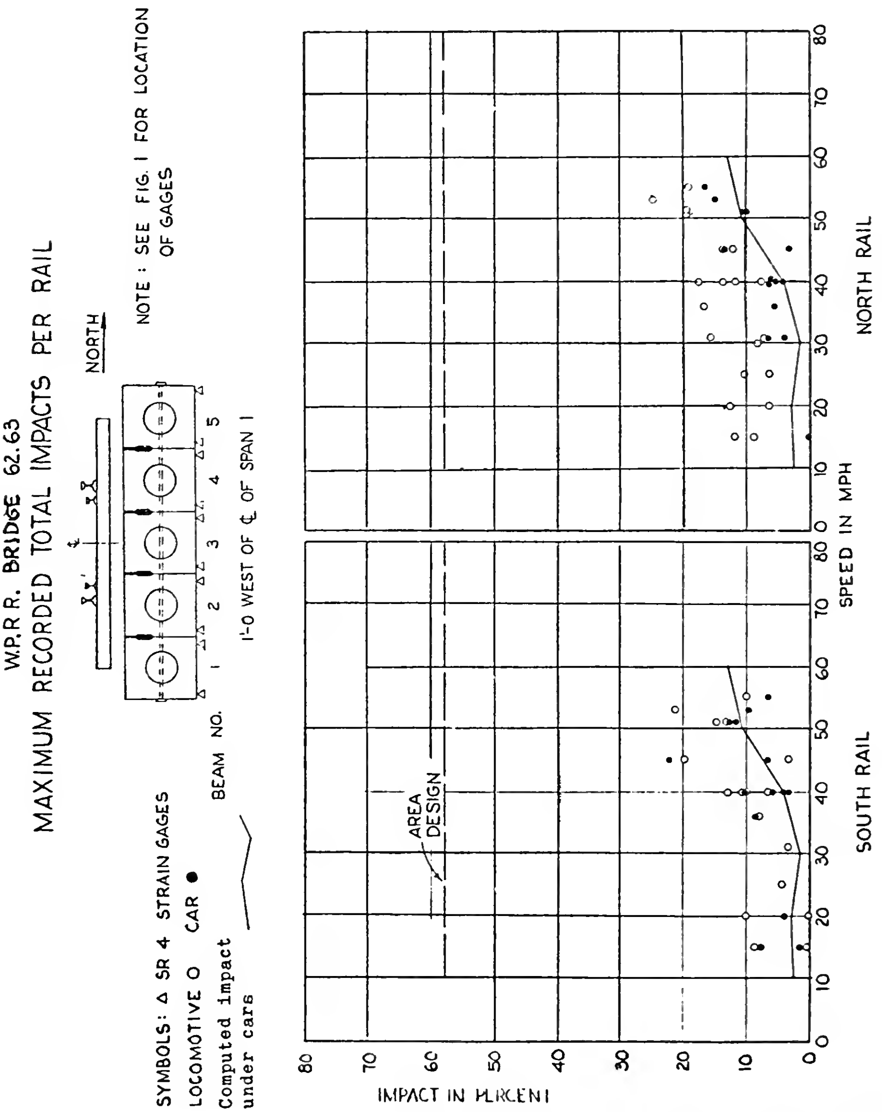


FIG. 9 - COMPARISON OF COMPUTED AND EXPERIMENTAL IMPACT RESULTS - TEST TRAIN
(1 mph = 1.609 km/h)

Acknowledgements

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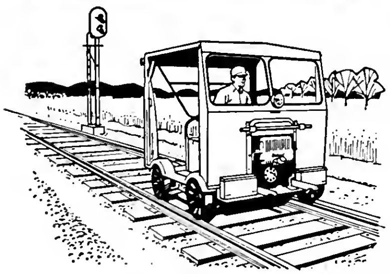

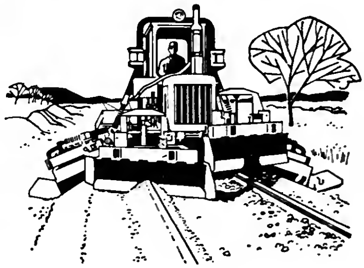

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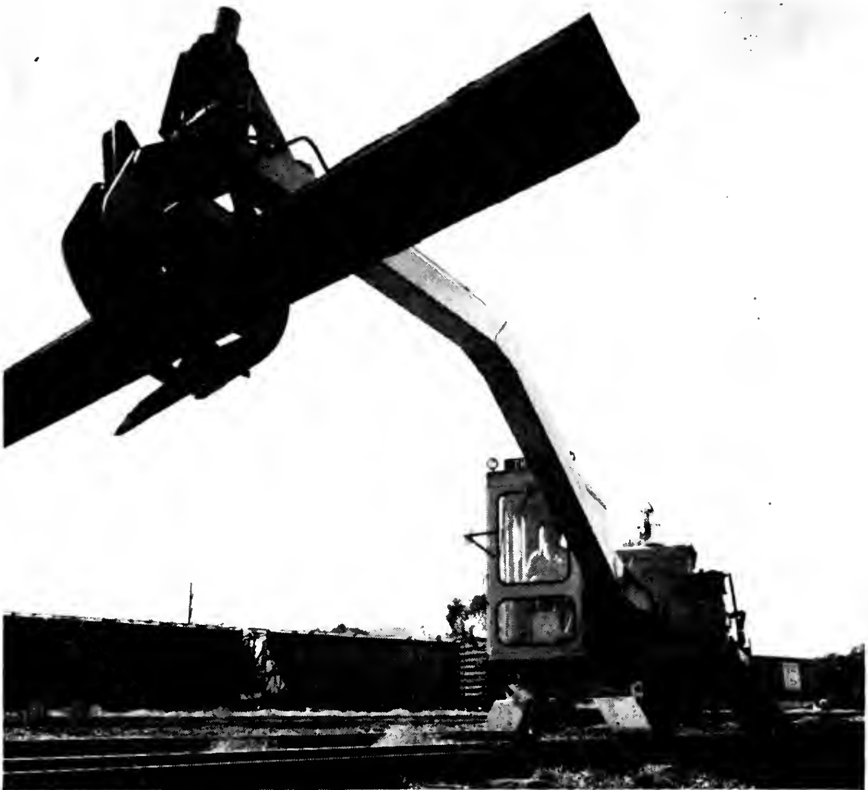

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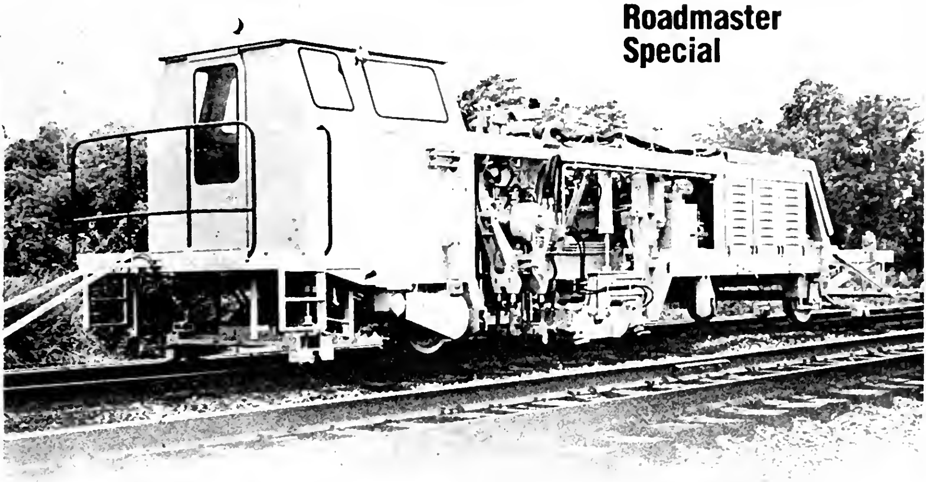

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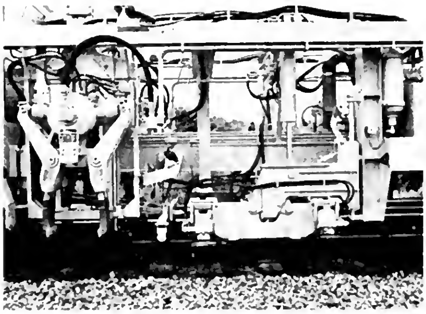

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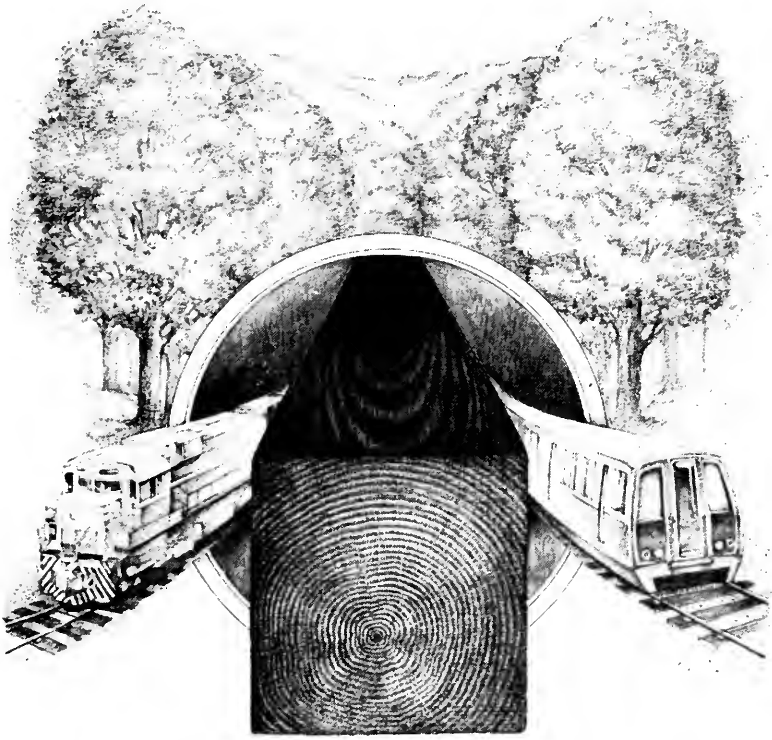


MANUAL RECOMMENDATIONS

Committee 8—Concrete Structures and Foundations

Part 23

Pier Protection Systems at Spans over Navigable Streams

T. R. Kealey (Chairman, Subcommittee)

1979

23.1 GENERAL

23.1.1 Scope

These specifications cover the design, construction, maintenance, and inspection of protective systems for railway bridge piers located in or adjacent to channels of navigable waterways.

23.1.2 Purpose

The purpose of the protective systems is to protect supporting piers of railway bridges from damage caused by accidental collision from floating vessels. Such protection should be designed to eliminate or reduce the impact energy transmitted to the pier from the vessel, either by redirection of the force or by absorption, or dissipation of the energy, to non-destructive levels.

23.2 SPECIAL CONSIDERATIONS

23.2.1 Vessel

The size and type of vessel to be chosen as a basis for design of the pier protection should reflect the maximum vessel tonnage and velocity reasonably to be expected for the specific facility involved.

23.2.2 Waterway

Consideration should be given to the exposure of the structure in the waterway, including the alignment of the channel, visibility for approaching vessels, as well as effect of wind, ice, current, or tide in the vicinity.

Depth of water may dictate the type of protection to be chosen. If the depth is so great, or the character of the waterway bottom does not lend itself to proper anchorage and support for an independent protective system, it may be necessary to design a suspended or floating protective system.

23.2.3 Types of Construction

The type of construction to be chosen for the protective system should be based on the physical site conditions and the amount of energy to be absorbed or deflected, as well as the size and ability of the pier itself to absorb or resist the impact.

Some of the more common types of construction are as follows:

23.2.3.1 Integral

Where the pier is considered to be stable enough to absorb the impact of floating vessels, it may be necessary to attach cushioning devices to the surfaces of the pier in the areas of expected impact to reduce localized damage such as spalling of concrete surfaces and exposure of reinforcing steel, or disintegration of masonry jointing. Such cushioning may include strips of material attached to the face of the pier, such as solid rubber, timber, rubber-pneumatic, hydraulic or hydrocushion strips.

23.2.3.2 Dolphins

Where depth of water and other conditions are suitable, the driving of pile clusters may be considered. Such clusters have the piles lashed together with cable to promote integral action. The clusters should be flexible to be effective in absorbing impact through deflection.

Cellular dolphins may be filled with concrete, loose material or material suitable for grouting. Cells filled with uncemented materials may lose fill material in the event of rupture due to collision.

23.2.3.3 Floating Sheer Booms

Where the depth of water or other conditions precludes the consideration of dolphins or integral pier protection, floating sheer booms may be used. These are suitably shaped and positioned to protect the pier and are anchored to allow deflection and absorption of energy. Anchorage systems should allow for fluctuations in water level due to stream flow or tidal action.

23.2.3.4 Hydraulic Devices

Suspended cylinders engaging a mass of water to absorb or deflect the impact energy may be used under certain conditions of water depth or intensity of impact. Such cylinders may be suspended from independent caissons, booms projecting from the pier, or other supports. Such devices are customarily most effective in locations subject to little fluctuations of water levels.

23.2.3.5 Fenders

Construction of fender systems, using piling with longitudinal wales, is a common means of protection where water depth is not excessive and severe impacts are not anticipated.

23.2.3.6 Other Types

Various other types of protective systems have been successfully used and may be considered by the engineer.

23.2.4 Permits

Proposed protective systems must receive approval of the U.S. Coast Guard and probably other regulating agencies prior to installation. Advance handling with these agencies to determine waterway clearance, lighting and any other special requirements, is recommended.

23.3 DESIGN

23.3.1 General

Criteria for the design of protective systems cannot be specified to be applicable to all situations. Investigation of local conditions is required in each case, the results of which may then be used to apply engineering judgment to arrive at a reasonable solution.

The location of the protective system (regardless of the type of construction) with respect to the navigation channel limits, stream current, prevailing winds, water depth, and normal water traffic approach angle is extremely important. The protective system should be located so that it will not hinder the vessel in negotiating the bridge opening, insofar as it is practical to do so.

In any type of pier protection system, general details should be designed to provide the following:

a. Replacement of damaged parts.

b. Elimination of sparking upon vessel impact.

c. Adequate mass and resilience so that the railroad facility will not be vulnerable to damage from normal collision of marine traffic.

23.3.2 Design Loads

Design loads to be used shall be determined for each individual structure, based on factors peculiar to the location. Information may be available from ship owners and operators, port facility authorities, industry representatives, the U.S. Army Corps of Engineers, and the U.S. Coast Guard.

23.3.2.1 General factors to be considered in determining the desired degree of pier protection include, but are not limited to, the following:

a. Piers at the edge of a channel having wide horizontal clearance may require only minimum protection.

b. The type of construction of the pier should be considered.

(1) A massive pier may be capable of resisting most anticipated loads so that the additional resistance offered by a protective system may not be warranted.

(2) A pier incapable of resisting anticipated loads should be provided with greater protection than a massive pier might require.

c. Piers may be especially vulnerable because of difficulty of navigation caused by high stream velocity or tidal flow, wind velocity, limited horizontal clearances, channel curvature, proximity of other obstacles, or other similar factors.

d. Foundation conditions will have a bearing on the resistance capability of the pier and on the practicality of providing the desired degree of protection.

e. The history of collisions with existing bridges or other obstacles in the vicinity should be considered.

23.3.2.2 To determine the actual collision forces which could be encountered, and their effects, the following items should be known:

(a) Maximum sizes and types of vessels.

(b) Impact velocity of vessels.

(c) Crushing resistance of hulls.

(d) Stream velocities.

(e) High and low water elevations.

(f) Impact angle.

(g) Wind velocities.

(h) Velocity and mass of floating ice.

23.3.2.3 The kinetic energy in the moving vessel may be determined as follows:

$(KE) = \frac{1}{2} MV^2$

Where (KE) = Kinetic energy

M = Total mass of the vessel

V = Velocity of the vessel

23.3.2.4 Energy may be dissipated according to the following (see fig. 1):

$E = F \times d$

Where E = energy dissipated

F = Average force applied to the moving vessel

d = Distance vessel moves (in the direction of F) during the time F is applied

The distance (d) is measured after initial contact and is composed of deflection of the protective system, crushing of the system and vessel, or a combination thereof. System flexibility determines, to a large extent, the relative amounts of deflection and crushing, and is more fully discussed in the appended commentary.

23.3.2.5 The effects of stream flow forces, wind forces and ice forces, where applicable, should be taken into consideration in the design of pier protection systems.

23.3.3 Suggested Design Procedure

As a practical matter, pier protections will not always be adequate to completely dissipate the kinetic energy of a vessel at high speed. However, in many cases, the protection will deflect a vessel, reducing damage that may otherwise occur. The outline presented here provides an approach to the problem of evaluating the effect on the kinetic energy of a vessel when a collision occurs:

23.3.3.1 Compute the kinetic energy (KE) based on the mass and impact velocity of the vessel.

23.3.3.2 Assume trial configuration of pier protective device and compute force (F) to be resisted:

a. Assuming allowable stresses equivalent to 1.5 times basic allowable unit stress of the material.

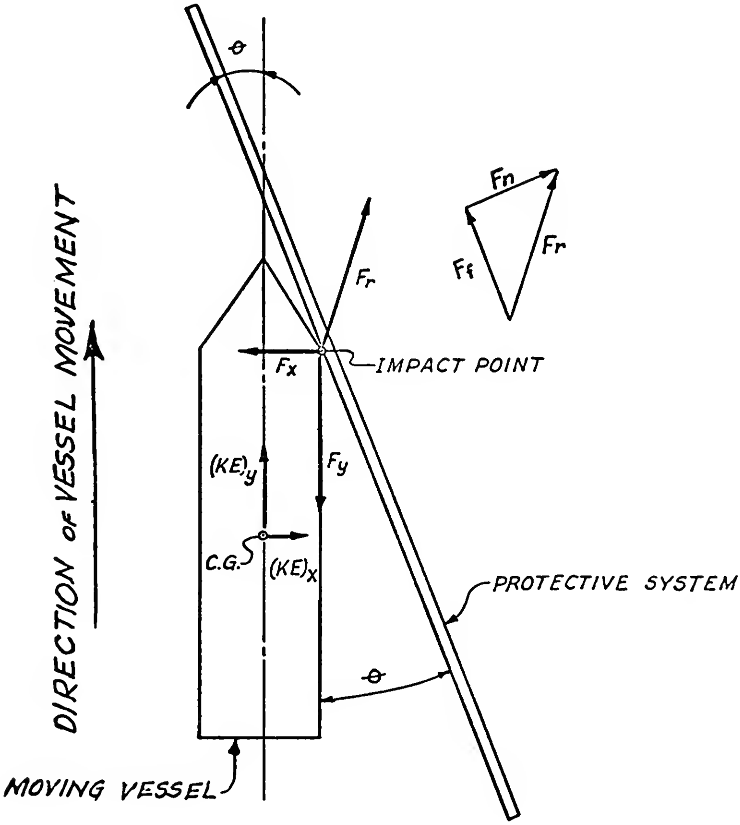
b. Assuming ultimate strength of materials.

23.3.3.3 Equate kinetic energy (KE) with energy dissipated (E):

$\frac{1}{2} MV^2 = F \times d$

solve for d to determine total movement required to dissipate energy (see commentary).

23.3.3.4 The above outline provide a basis for evaluating the amount of energy that can be dissipated without damage to the pier protection, and the total resistance capability.



F_x = FORCE ON VESSEL LATERALLY
F_y = FORCE ON VESSEL AXIALLY
$(KE)_x$ = INERTIA FORCE (KINETIC ENERGY) LATERALLY
$(KE)_y$ = INERTIA FORCE (KINETIC ENERGY) AXIALLY
F_r = RESULTANT FORCE ON PROTECTIVE SYSTEM
F_f = FRICTIONAL OR TANGENTIAL FORCE ON PROTECTIVE SYSTEM
F_n = NORMAL FORCE ON PROTECTIVE SYSTEM
θ = IMPACT ANGLE
C_g = CENTER OF GRAVITY

FIGURE 1

23.3.4 Types of Protection

The following types of protection are commonly used; however, other types may be considered.

23.3.4.1 Sheet Pile Cell Dolphins (see fig. 2).

Sheet pile cells preferably should be of circular configuration. A typical cell includes interlocking steel sheet piles filled with concrete or grouted material. If loose fill materials are used, a reinforced concrete top having a minimum thickness of 2′-0″, with an opening to allow for adding additional fill should be provided. The concrete top should be adequately anchored to the sheet piles. Desirable qualities of fill material include free draining characteristics, high unit weight, shear strength, and high coefficient of friction.

The designer should make an evaluation of the cell stability and resistance to overturning and sliding. Factors to be considered include characteristics of the underlying soil or rock and the cell fill material, interaction of the cell fill material with the cell walls, and friction of the sheet piles embedded in the underlying soil.

Additional resistance against overturning may be provided by driving and attaching additional piles around the perimeter of the cell. Increased penetration into the underlying soil may be obtained in this manner, in lieu of extension of all sheet piles.

The possibility of scour occurring near a dolphin should be investigated and protection should be provided, if required.

23.3.4.2 Pile Cluster Dolphins (see fig. 3).

Pile cluster type dolphins should be composed of groups of battered and/or vertical piles which are held together at the top. The designer should evaluate the resistance to lateral forces, considering the effects of any battered piles, and the interaction of the piles and the surrounding soils.

23.3.4.3 Gravity Pendulum Dolphin (Hydrocushion Type) (see fig. 4).

Typically, a heavy cylindrical mass of steel or concrete is suspended from a cantilevered supporting structure, which may be a part of the pier, or may be an independent support. Energy is dissipated by movement of the pendulum when a force is applied by a striking vessel.

The designer should evaluate the energy dissipated by the pendulum, taking the following items into account.

a. Movement of the pendulum. When the pendulum is suspended in water, the effective mass includes an amount of water which moves along with the pendulum; in the case of a ring, (as shown in Figure 4) the volume of water enclosed by the ring is part of the total mass to be moved.

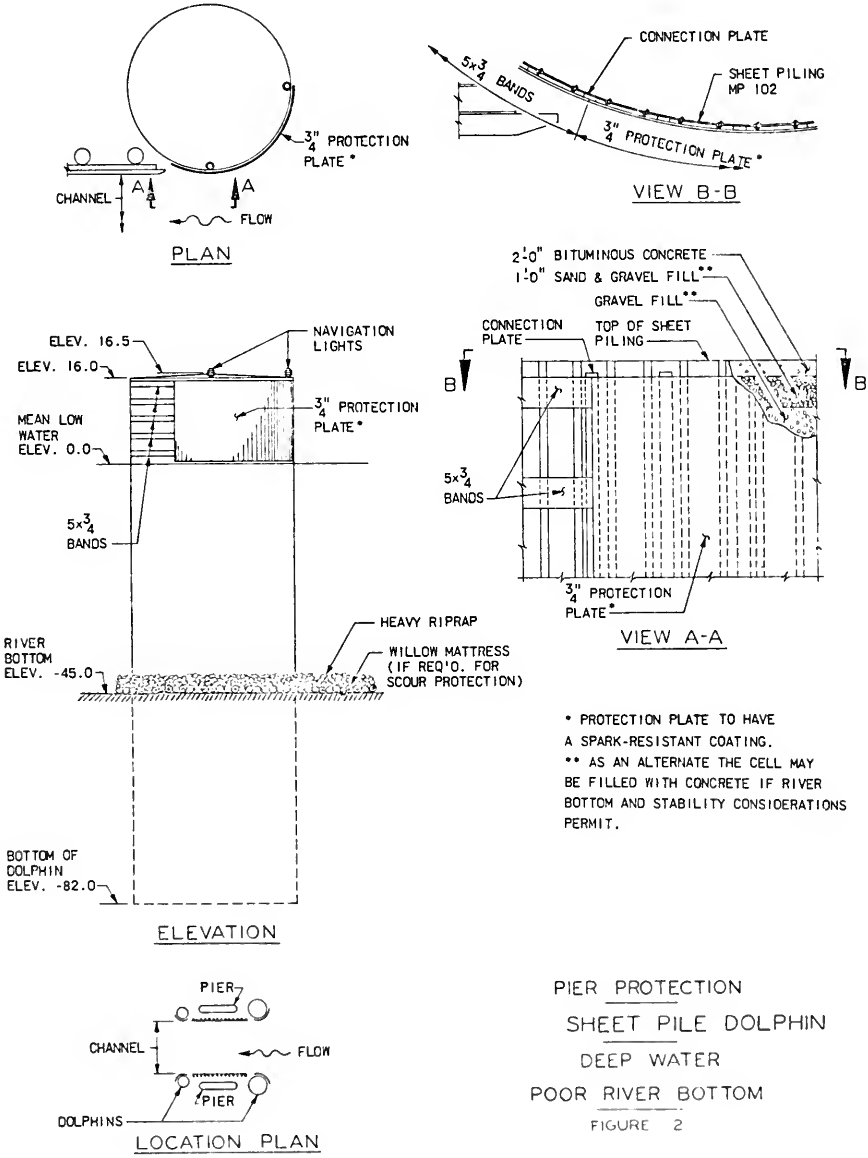
b. The resisting horizontal force component $= Wr\left(\frac{x}{l-y}\right)$

in which: Wr = Weight of the ring

x = The horizontal displacement of the ring

l = Length of hanger to the ring

y = The amount the ring is lifted



* PROTECTION PLATE TO HAVE A SPARK-RESISTANT COATING.

** AS AN ALTERNATE THE CELL MAY BE FILLED WITH CONCRETE IF RIVER BOTTOM AND STABILITY CONSIDERATIONS PERMIT.

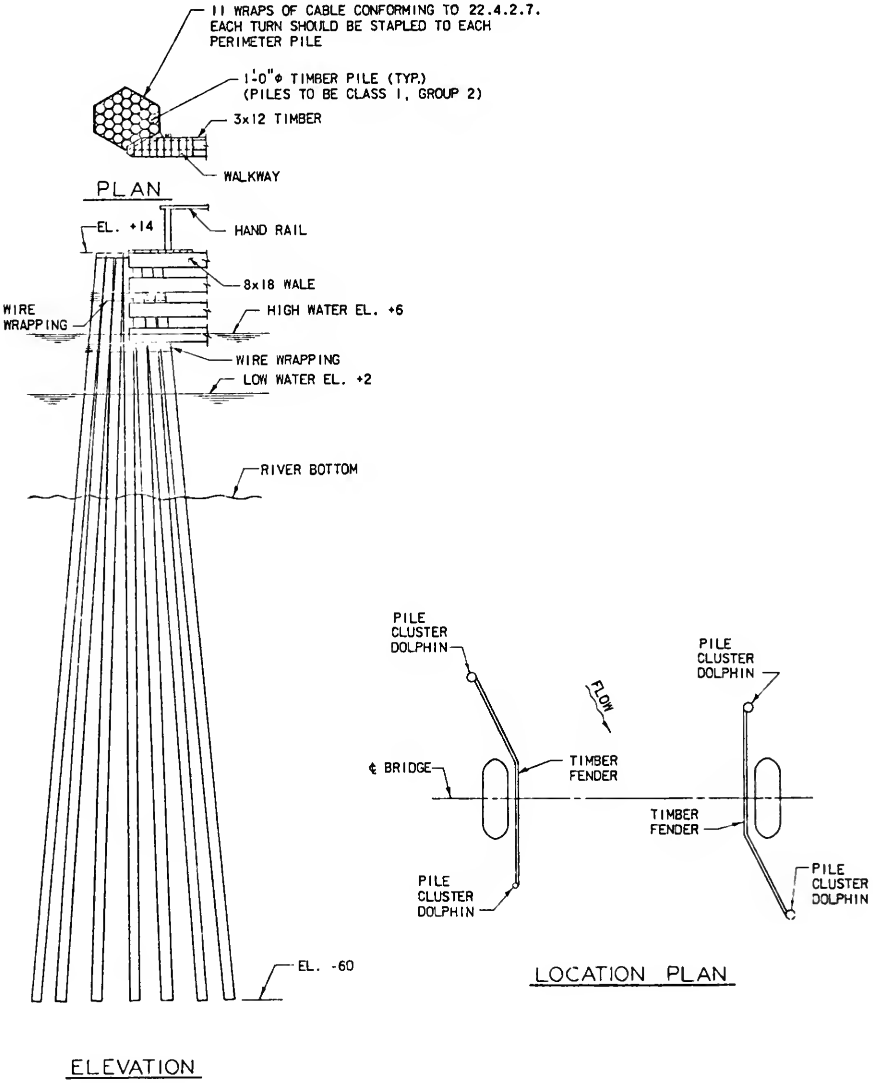
PIER PROTECTION

SHEET PILE DOLPHIN

DEEP WATER

POOR RIVER BOTTOM

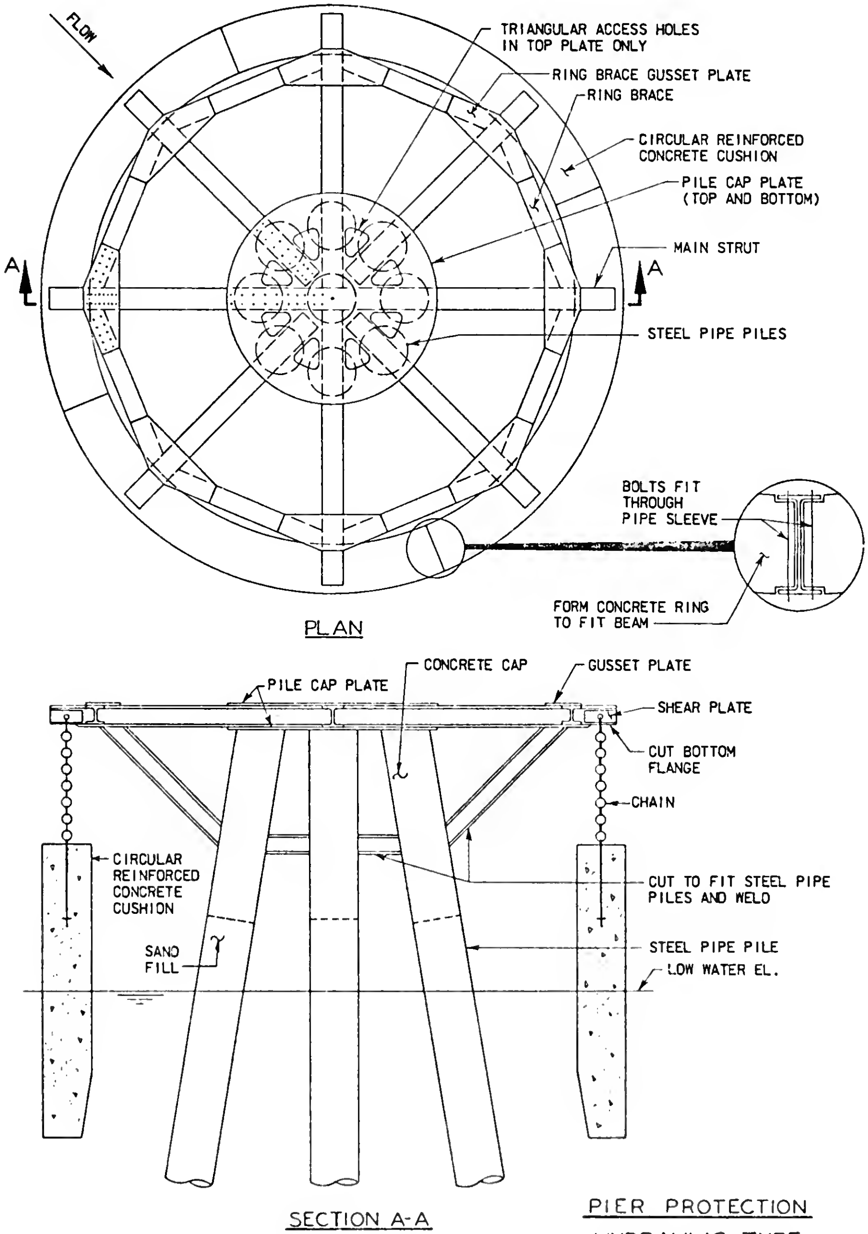
FIGURE 2



PIER PROTECTION

TREATED TIMBER PILE DOLPHIN

FIGURE 3



PIER PROTECTION
HYDRAULIC TYPE
HYDRO-CUSHION DOLPHIN

FIGURE 4

23.3.4.4 Floating Sheer Booms (see fig. 5).

The configuration of a sheer boom will depend upon the requirements of a particular location.

The designer should evaluate the capability of the device to dissipate energy, recognizing the following:

a. The mass to be considered as part of the moving element includes a volume of water which will be forced to move with the boom.

b. Deflection movements of supporting elements will account for some energy loss.

c. Frictional resistance is provided by the water adjacent to the moving elements.

23.3.4.5 Fenders (see figs. 6, 7, 8 and 9).

Pier fenders are constructed to provide for some degree of protection to the pier in the event of contact by a vessel. Fenders are usually positioned to anticipate the direction of impact from a vessel to be at a relatively small angle with respect to the fender line. A fender may be supported by the pier it is intended to protect, or it may be independently supported.

Independently supported fender systems typically consist of vertical and/or battered piles with horizontal members connecting the piles so the fender system acts as a unit. The horizontal members may be used as rubbing strips or separate rubbing strips may be attached to these members.

Pier-supported fenders vary in type from simple rubbing strips attached directly to the pier face to more elaborate installations which provide for some energy dissipation by the fender when struck by a vessel.

The designer should consider the following items pertaining to fenders:

a. Fenders should preferably be detailed so that a maximum number of piles, or other supporting elements, will participate in resisting applied loads.

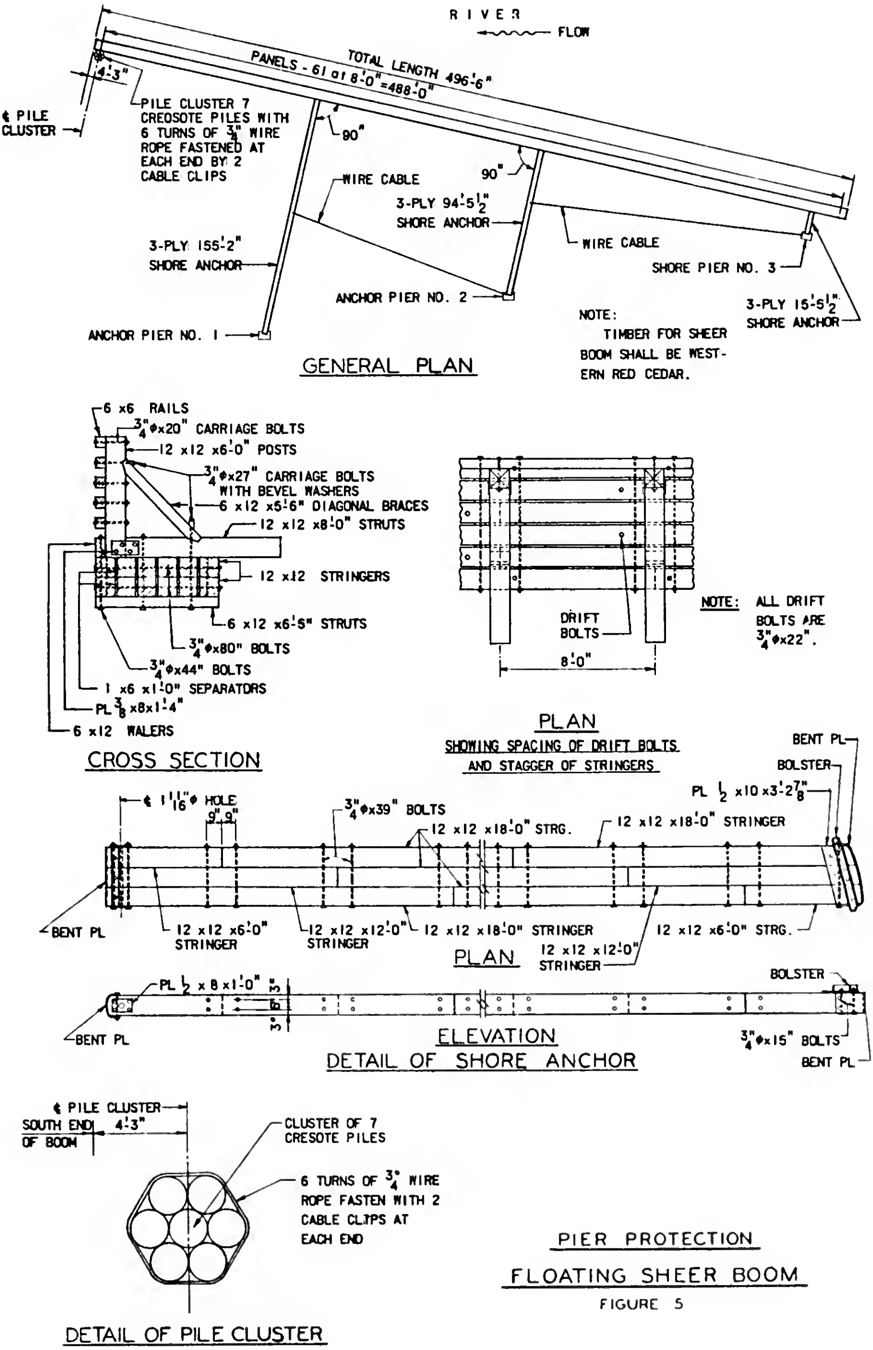
b. Generally, a somewhat flexible arrangement that provides for deflection movement of the fender is preferred to provide for energy dissipation.

c. The effects of battered piles and pile-soil interaction should be considered when evaluating the capability of the fender to resist lateral forces.

d. Consideration should be given to providing a weak point in the design, thus causing the unit to fail in a pre-planned manner when struck by a force in excess of the capacity. Details can then be arranged to facilitate the replacement of damaged elements.

23.3.4.6 Riprap Used as Pier Protection

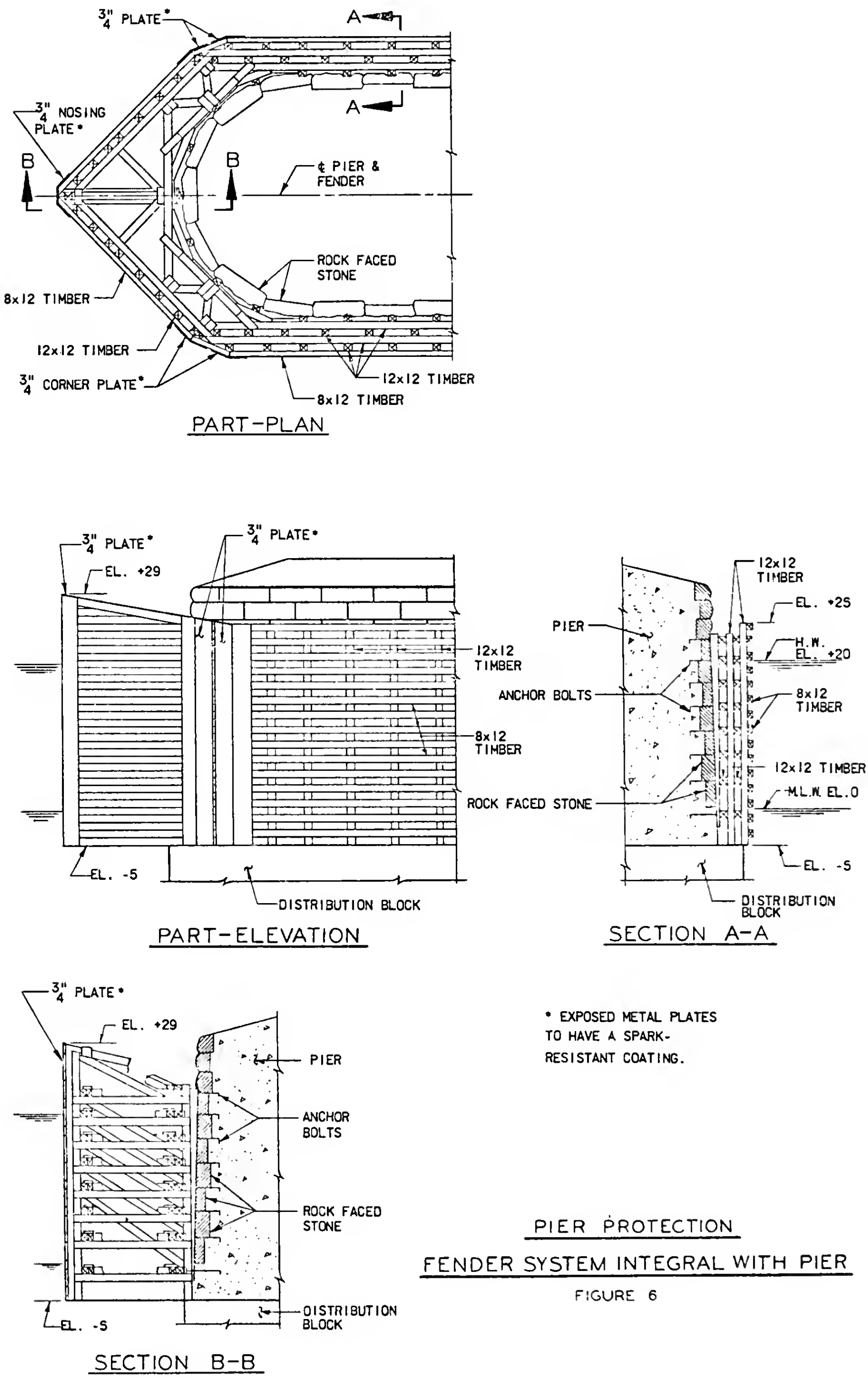
Piers which are located near the shoreline or in shallow water at the edge of a ship channel may require minimum protection. Riprap may be deposited near a pier for the purpose of preventing erosion and to reduce the water depth, thus protecting the pier from vessels by stopping them before contact is made.



PIER PROTECTION

FLOATING SHEER BOOM

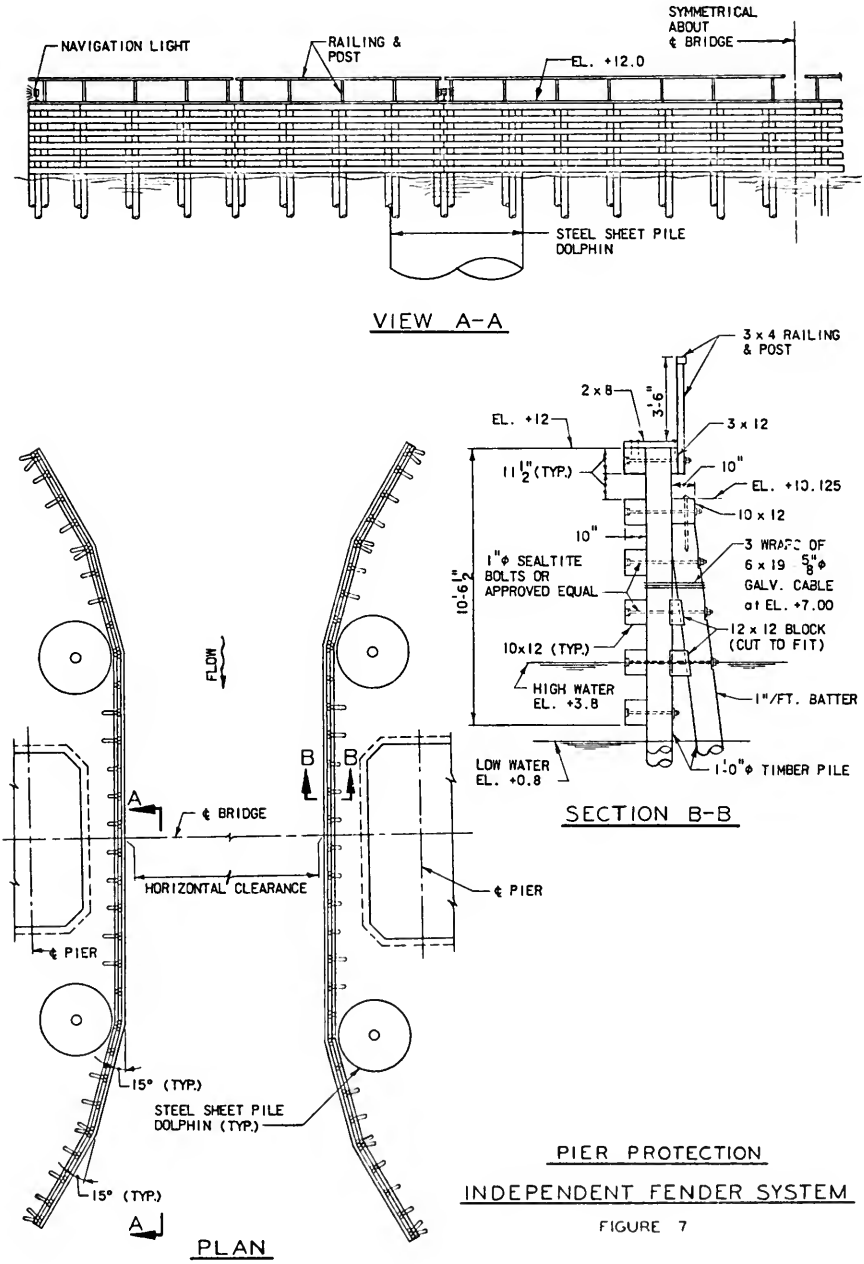
FIGURE 5



PIER PROTECTION

FENDER SYSTEM INTEGRAL WITH PIER

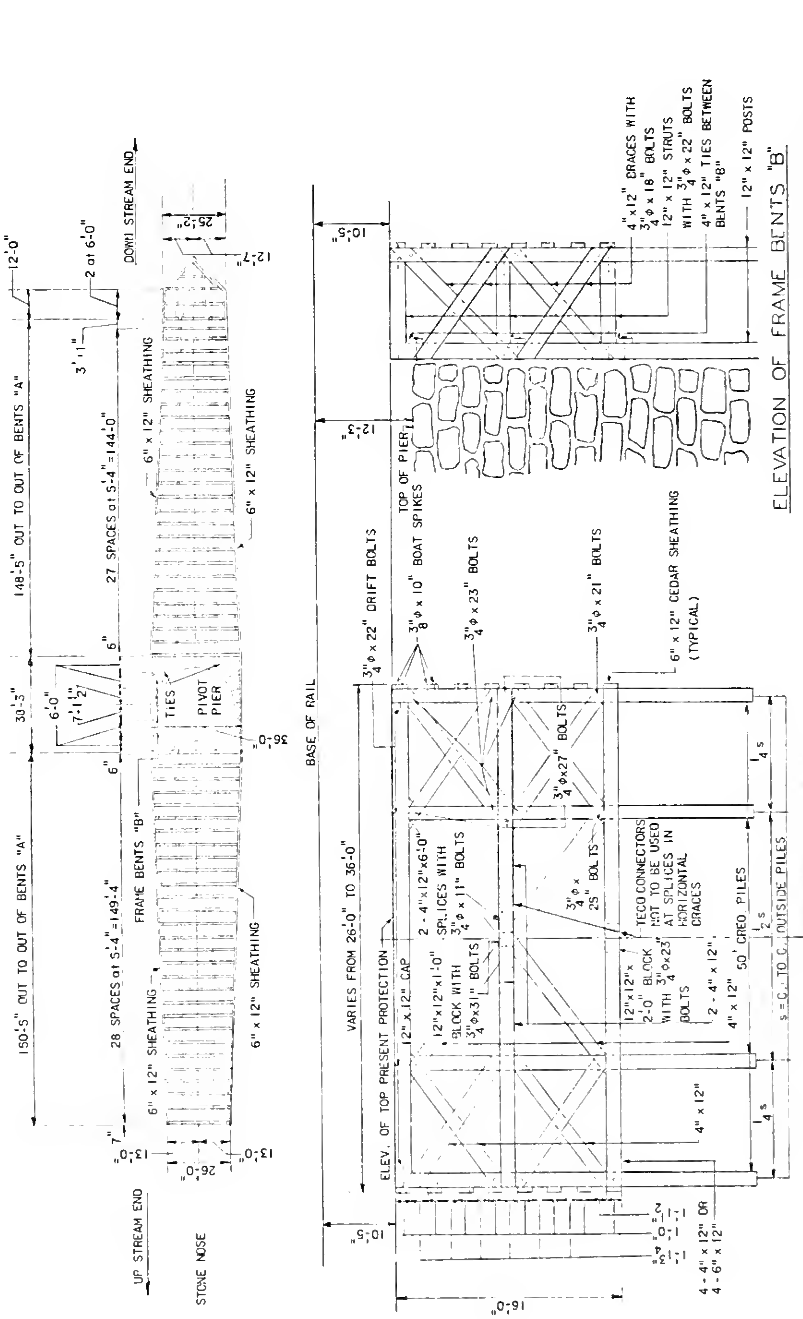
FIGURE 6



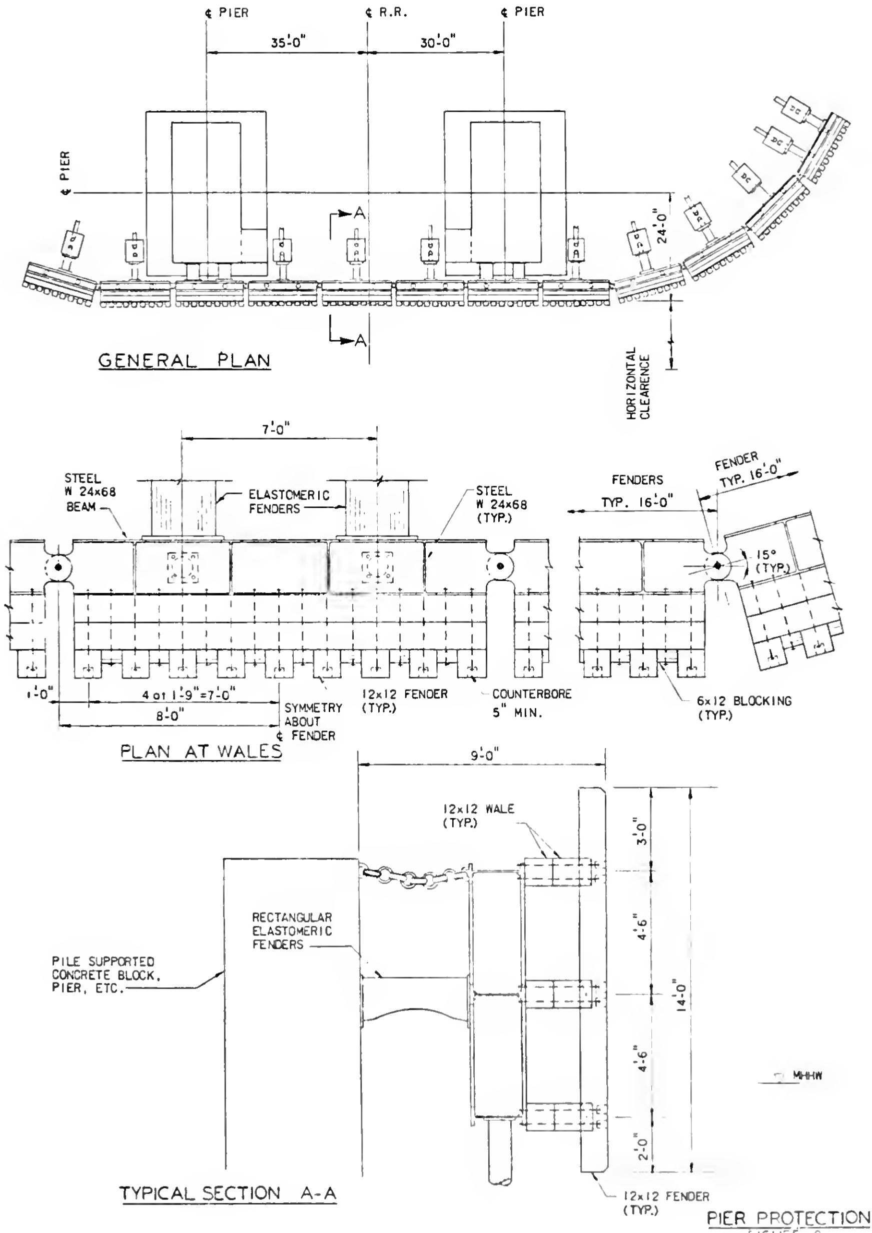
PIER PROTECTION

INDEPENDENT FENDER SYSTEM

FIGURE 7



PIER PROTECTION
PIVOT PIER
FIGURE 8



PIER PROTECTION
FIGURE 9

23.4 CONSTRUCTION

23.4.1 General

23.4.1.1 Construction permits from all federal, state and local regulatory bodies should be obtained prior to beginning construction.

23.4.1.2 Construction should be performed in accordance with all local laws and regulations including navigational clearances, maintenance of marine traffic, navigation lighting and temporary warning signs and devices.

23.4.1.3 All temporary construction facilities should be approved by the company and the concerned regulatory bodies. Temporary construction should be removed upon completion of the work and the construction site returned to a condition acceptable to the regulatory bodies and the company.

23.4.1.4 Excavated material and debris of demolition and of construction should be disposed of in accordance with all applicable laws and regulations.

23.4.1.5 Construction inspection safeguards should be provided to insure that pier protection structures are constructed in the correct location with respect to the navigation channel. Underwater inspection services should be provided if necessary to determine conditions relevant to the construction. As-built plans should be furnished to the company upon completion of the work.

23.4.2 Materials

23.4.2.1 Timber

a. All new timber should meet the requirements of the current standard Methods for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber, ASTM Designation D245. Timber should be Dense Structural 65 or Long-Leaf Structural 65, southern yellow pine, conforming to the Grading Rules of the Southern Pine Inspection Bureau; or No. 1 Douglas Fir conforming to the Standard Grading Rules for West Coast Lumber; or other species conforming to the flexural strength specified for Southern Pine and Douglas Fir, other requirements being comparable.

b. Timber for joists, planks, beams, wales and walkways should be square edge and shall be grade marked.

c. Timber, except walkway planking and handrails, should be preservative treated with coal tar creosote by the full cell process. The preservative treatment should be in accordance with AREA Chapter 3, Parts 6 through 9 inclusive. Walkway planking and handrails may be treated wither with creosote or pentachlorophenol. Pentachlorophenol should be used if the member is to painted with exterior oil paints.

23.4.2.2 Timber Piles

a. Timber piles should be First Class piles in accordance with AREA Chapter 7, Parts 1 and 3, and should conform to ASTM D25.

b. Coal tar creosote preservative treatment is required and should confomrm to AREA Chapter 3, Parts 6 through 9 inclusive.

23.4.2.3 Steel Piles

a. W and HP steel shapes should have minimum flange and web thicknesses of $^3/_8$ inch and should conform to ASTM A36, or ASTM A709, Grade 36, with a minimum 0.2 percent copper; or should conform to ASTM A588, Grade 50, or ASTM A709, Grade 50.

b. Steel pipe piles should have a minimum wall thickness of $^3/_8$ inch and shall conform to ASTM A252, Grade 2, with minimum 0.2 percent copper.

c. Steel sheet piles should have a minimum thickness of $^3/_8$ inch and shall conform to ASTM A328, with minimum 0.2 percent copper, or shall conform to ASTM A690. The designer should specify the minimum strength required in the interlock joint.

23.4.2.4 Concrete

a. Workmanship, materials and proportioning for concrete members used in pier protection structures should be in accordance with requirements for Concrete and Reinforced Concrete Railroad Bridges and Other Structures, Part 1 of this Chapter.

b. The design of concrete members used in pier protection structures should be in accordance with the requirements for Plain and Reinforced Concrete Members, Part 2, this Chapter.

c. The minimum cover on reinforcing steel in concrete faces subject to impact should be 3 inches.

23.4.2.5 Structural Steel

Structural steel shapes and plates should conform to the Standard Specification for Structural Steel, ASTM A36, or ASTM A709, Grade 36, with a minimum of 0.2 percent copper. Other steels may be used having greater strength and enhanced corrosion resistance as required by the design of the pier protection work. The recommended minimum thickness of metal is $^3/_8$ inch.

23.4.2.6 Hardware

Bolts, nuts, washers, spikes, lag bolts, staples, cable clamps and similar hardware items should be galvanized steel or stainless steel. In lieu of using galvanized or stainless steel hardware, the company may wish to provide other means of corrosion protection.

a. Galvanized standard carbon steel fasteners should conform to the standard Specification for Carbon Steel Externally and Internally Threaded Standard Fasteners, ASTM A307; or to the Standard Specification for High Strength Bolts for Structural Steel Joints, Including Suitable Nuts and Plain Hardened Washers, ASTM A325, Type 1. Galvanizing should be in accordance with the requirements of ASTM A153, Class C. If galvanizing is not required, fasteners should conform to the Standard Specifications for High Strength Bolts for Structural Steel Joints, Including Suitable Nuts and Plain Hardened Washers, ASTM A325, Type 3.

b. Stainless steel hardware should be manufactured from material conforming to the Standard Specifications for Stainless and Heat-Resisting Steel Bars and Shapes, ASTM A276, Type 304 or 316. Type 316 should be used in a salt water atmosphere and treated with a positive corrosion resistant material.

23.4.2.7 Wrapping for Dolphins

Cable for wrapping dolphins should be galvanized 6 × 7 steel rope; or galvanized 7 × 7 mild plow steel rope. Where enhanced corrosion resistance is required, 7 × 19 stainless steel wire rope shall be used. The designer should evaluate the cost, expected life, usage, susceptibility to damage and other pertinent factors when choosing the dolphin wrapping cable. All wire ropes should have steel wire cores.

23.4.2.8 Corrosion Protection

Consideration should be given to protecting submerged steel surfaces cathodically and exposed surfaces by means of suitable paint systems or by galvanizing.

23.4.3 Handling and Storage of Materials

23.4.3.1 All timber, lumber, timber piles and associated hardware should be handled and stored in accordance with Chapter 7, Part 3, Sections 2 and 3.

23.4.3.2 Concrete materials such as cement, aggregates and steel reinforcement, should be stored in accordance with Chapter 8, Part 1, Sections F-1, F-2, and F-3.

23.4.3.3 Handling and Storage of steel items should be in accordance with Chapter 15, Part 4, Section 4.8.

23.4.3.4 Miscellaneous parts and materials should be handled in a manner as to prevent loss and damage, and should be stored on blocking or on platforms above the ground. Weather and fire protection should be provided as necessary.

23.4.4 Framing of Timber

Timber should be cut and framed in accordance with Chapter 7, Part 3, Section 5.

Bolt heads and washers on the navigation side should either be recessed below the rubbing surface of the fender or be of the dome-head type flush with the rubbing surface.

23.4.5 Fabrication of Structural Steel

Fabrication of structural steel should be in accordance with the requirements of Chapter 15, except as noted herein:

a. Substitution of stronger, but less energy absorbing members will not be permitted.

b. Substitution of higher grade, but less ductile steel will not be permitted, such as A490 bolts for A325.

c. Shop assembly will not be required.

d. Field welding will not be permitted, unless specifically authorized by the engineer. All field connections should be held to a minimum and should be made by means of bolts with appropriate washers and nuts.

e. Washers should be placed under both the heads and nuts of all bolts (except dome-head bolts) bearing on timber. Suitable lock nuts should be provided where fastenings may tend to loosen.

23.4.6 Pile Driving

Pile driving should be performed in accordance with Chapter 8, Part 4, Section I (or Chapter 7, Part 3).

23.4.6.1 Pile Driving Records

An accurate record should be kept of all piles driven, on the form perscribed by the engineer. The log should show date, type of pile driven, pile number, location, type of hammer used, water depth and elevation, pile depth into soil, and ultimate driving resistance. The form should be signed by the person recording the information, including his job title. The record should be made a permanent part of the job statistics.

23.5 TYPICAL PLANS FOR PIER PROTECTION SYSTEMS

The various types of pier protection systems shown in this section are for general information only. For the most part, they have been taken from protection systems currently in use on both highway and railway bridges in the United States. Member sizes, numbers of units, types of material, and details of construction are those used for specific installations and cannot be considered standards since the design of pier protection systems depends on many parameters that may vary markedly from one installation to another. Each pier protection system must be chosen and designed to fulfill the unique requirements at the given location.

Commentary on Pier Protection Systems at Spans over Navigable Streams

Energy Dissipation

A moving vessel has a certain amount of kinetic energy, which is dependent upon the mass of the vessel and its velocity. If we are to redirect or stop this vessel in protecting the pier, a portion or all of this kinetic energy must be absorbed or dissipated. This energy is dissipated by applying a force to the vessel over a given distance. For the fender to function properly, this distance must be less than the distance from initial contact until the vessel would strike the pier. For large vessels, traveling at fair speeds, in deep water, the amount of kinetic energy provided is large and the resistance of the fender is relatively small and it is very difficult to design a fender that will completely protect a pier for such a collision if the vessel is headed directly at the pier.

The energy in any contact with the fender is dissipated by deflection of the fender itself, by lifting a portion of the fender, by lifting the vessel out of the water, by crushing of the fender, by crushing of the bow of the vessel, by displacement of the water adjacent to the vessel, by displacement of the ground or river bottom, etc.

Several general facts should be considered and are noted briefly:

1. It should be recognized that the total resisting force is not developed immediately upon impact, but requires some movement until it develops.

2. If the crushing force of the vessel is greater than the ultimate resisting force of the fender, then dissipation of the kinetic energy occurs in two phases. In the first phase, the impact creates a force between the vessel and the fender, which causes the vessel to decelerate and the fender to accelerate (F = mass x acceleration). At some point, the fender and the vessel reach the same velocity and move along together, being slowed by the resisting forces of the fender and/or the soil being acted upon. This will continue until either the vessel stops, the fender breaks or some combination of the two.

3. If the crushing force of the vessel is less than the total ultimate resisting force of the fender, then the velocity of the fender will increase from zero to a maximum and decrease to zero again without a common velocity being achieved. When the fender stops, the vessel continues to decelerate, acted upon by the crushing force.

Fender Flexibility

An ideal pier fender would be constructed so that the fender itself absorbs all of the energy of the moving vessel in stopping the vessel before it hits the pier and then returns to its normal position without damage to either the fender or the vessel. Except for relatively small vessels and low speeds, design of such a fender is impractical due to the large required resisting force and the short distance in which to stop the vessel.

A flexible fender, one that acts elastically, will absorb energy with little or no damage to the vessel; however, the horizontal force that such a fender can resist is usually relatively small and may be insufficient to protect the pier. On the other hand, a rigid fender is capable of resisting a considerably larger force, although this force may only be applied over a small deflection before the member breaks, or is damaged locally. In this case, the total amount of energy absorbed may be far less than is absorbed in a flexible fender, although a considerable amount of energy is absorbed in breaking of the fender parts. In most cases, some compromise between a truly flexible and a very rigid fender is the better solution.

In fender systems, incorporating steel pipe piles or sheet pile cells, a concrete fill will provide a much more rigid device than will one filled with sand or stone or riprap. In the latter case, the energy absorbing qualities are improved due to the rubbing of the fill particles on each other, by friction in the interlocks of the sheet piles and the like. On the other hand, one must be extremely careful that the pile wall or the sheet pile wall is protected to prevent damage resulting in the loss of fill, which would materially reduce the effectiveness of the fender and its energy absorbing capability. The sand filled pipe is much more likely to deflect and to bend than the concrete filled pipe, which will only deflect a small distance before shearing-off.

The type of fender used in any particular application must take into account the size and velocity of the vessel, flow of the stream, the depth of the water, the founding conditions, the distance between the pier protection and the pier, the strength of the pier itself and the types of cargo that are normally carried. The engineer must normally use his discretion in selecting a pier protection design that best suits all of the parameters of the individual case considered.

Sources of Information

Stream velocities for various river stages on most navigable waters can be obtained from the U.S. Corps of Engineers. Channel locations, navigation maps and scour potential, may be available from the U.S. Corps of Engineers and the U.S. Coast Guard.

Information regarding principal sizes, capacities and power of various vessels, as well as the type of cargo is usually available for navigable waters from the U.S. Corps of Engineers, the U.S. Coast Guard, the American Waterways Operators, Inc., ports authorities, pilots associations and others.

Specific site parameters such as, riverbed conditions, soil information, local wind and current effects on navigation usually must be developed by the design engineer, although local pilots associations and waterway users associations may be able to help with the latter.

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MANUAL RECOMMENDATIONS
Committee 8—Concrete Structures and Foundations

Part 5

RETAINING WALLS AND ABUTMENTS

G. Cooke, (Chairman, Subcommittee)

SPECIFICATIONS FOR DESIGN OF RETAINING WALLS AND ABUTMENTS[1]

1977

5.1 DEFINITIONS

5.1.1 Types of Retaining Walls and Abutments

A retaining wall is a structure used to provide lateral support for a mass of soil which, in turn, may provide vertical support for loads acting on or within the soil mass.

The principal types of retaining walls are as follows:

a. The gravity wall, which is so proportioned that no reinforcement other than temperature steel is required.
b. The semi-gravity wall, which is so proportioned that some steel reinforcement is required along the back and along the lower side of the toe.
c. The cantilever wall, which has a cross section resembling an L or an inverted T, and which requires extensive steel reinforcement.
d. The counterfort wall, which consists of a reinforced vertical face slab supported laterally at intervals by vertical reinforced counterforts extending into the backfill, and supported by a reinforced base slab which usually projects in front of the face slab to form a toe.
e. The buttress wall, which is similar to the counterfort wall except that the vertical members, called buttresses, are exposed on the face of the wall rather than buried in the backfill.
f. The crib wall, which consists of an earth-filled assembly of individual structural units, and which relies for its stability on the weight and strength of the earth fill. The design of such walls is treated in the AREA Specifications for Crib Walls, Part 6, this Chapter.

An abutment commonly consists of a retaining wall that incorporates a bridge seat in its face. It may also be of the spill-through type, however, in which the bridge seat rests on horizontal beams supported by piles or columns between which the fill is permitted to extent.

5.2 INFORMATION REQUIRED

5.2.1 Field Survey

Sufficient information shall be furnished, in the form of a profile and cross sections or a topographic map, to determine the structural requirements. Present grades and

[1]References, Vol. 18, 1917, pp. 85, 1564; Vol. 28, 1927, pp. 1055, 1453; Vol. 52, 1951, p. 384; Vol. 54, 1953, pp. 798, 1342; Vol. 58, 1957, pp. 633, 1182; Vol. 59, 1958, pp. 676, 1188; Vol. 62, 1961, pp. 438, 869.

[2]Latest page consist: 1 and 2, 15 and 16 (1966); 3 and 4, 7 and 8 (1961); 5 and 6, 13 and 14, 17 and 18 (1958) to 12, incl., (1953).

alinements of tracks and roads shall be indicated, together with the records of high water, low water, and depth of scour, the location of underground utilities, and information concerning any structures that may affect or be affected by the construction.

5.2.2 Subsurface Exploration

For specifications concerning this topic, see Part 22 of this Chapter.

5.2.3 Controlling Dimensions

Information shall be assembled concerning clearances, proposed grades of tracks and roads, and all other factors that may influence the limiting dimensions of the proposed structure.

5.2.4 Loads

Loads to be superimposed either on the wall or abutment, or on the backfill, shall be indicated.

5.2.5 Character of Backfill

Backfill is defined as all material behind the wall, whether undisturbed ground or fill, that contributes to the pressure against the wall.

The backfill shall be investigated and classified with reference to the following soil types:

TYPES OF BACKFILL FOR RETAINING WALLS

Type
1. Coarse-grained soil without admixture of fine soil particles, very free-draining (clean sand, gravel or broken stone).
2. Coarse-grained soil of low permeability due to admixture of particles of silt size.
3. Fine silty sand; granular materials with conspicuous clay content; or residual soil with stones.
4. Soft or very soft clay; organic silt; or soft silty clay.
5. Medium or stiff clay that may be placed in such a way that a negligible amount of water will enter the spaces between the chunks during floods or heavy rains.

Types 4 and 5 backfill shall be used only with permission of the Engineer.

5.2.6 Character of Foundation

The character of the foundation material shall be investigated as specified under Sec. B. Art. 4 of the AREA specifications for the Design of Spread Footing Foundations, Part 3 of this Chapter.

5.3 COMPUTATION OF APPLIED FORCES

5.3.1 Loads Exclusive of Earth Pressure

In the analysis of retaining walls and abutments, due account shall be taken of all superimposed loads carried directly on them, such as building walls, columns, or bridge structures; and of all loads from surcharges caused by railroad tracks, highways, building foundations, or other loads supported on the fill back of the walls.

In calculating the surcharge due to track loading, the entire load shall be taken as distributed uniformly over a width equal to the length of the tie. Impact shall not be considered unless the bridge bearings are supported by a structural beam, as in a spill-through abutment.

5.3.2 Computation of Backfill Pressure

Values of the unit weight, cohesion, and angle of internal friction of the backfill material shall be determined directly by means of soil tests or, if the expense of such tests is not justifiable, by means of the following table referring to the soil types defined in Section 5.2.5. Unless the minimum cohesive strength of backfill material can be evaluated reliably the cohesion shall be neglected and only the internal friction considered. See chart on page 8-20-9.

Soil Type	Unit Weight LB per Cu Ft	Cohesion c LB per Sq Ft.	Angle of Internal Friction
1	105	0	33°42′ (38° for broken stone)
2	110	0	30°
3	125	0	28°
4	100	0	0
5	120	240	0

The magnitude, direction and point of application of the backfill pressure shall be computed on the basis of appropriate values of the unit weight, cohesion and internal friction, by means of one of the following procedures.

When the backfill is assumed to be cohesionless; when the surface of the backfill is or can be assumed or be plane; when there is no surcharge load on the surface of the backfill; or when the surcharge can be converted into an equivalent uniform earth surcharge, Rankine's or Coulomb's formulas may be used under the conditions to which each applies. Formulas and charts given in Appendix A and the trial wedge methods given in Appendix B are both applicable.

When the backfill cannot be considered cohesionless, when the surcharge on the backfill is irregular, or when the surcharge cannot be converted to an equivalent uniform earth surcharge, the trial wedge methods illustrated in Appendix B should be used.

If the wall or abutment is not more than 20 ft. high and if the backfill has been classified according to Sec. B, Art. 4, the charts given in Appendix C may be used.

If the surcharge is of a lesser width than the height of the wall, a more satisfactory design can be obtained by the use of the trial wedge methods given in Appendix B.

If the wall or abutment is prevented from deflecting freely at its crest, as in a rigidframe bridge or some types of U-abutments, the computed backfill pressure shall be increased 25 percent.

In spill-through abutments, the increase of pressure against the columns due to the shearing strength of the backfill shall not be overlooked. If the space between columns is not greater than twice the width across the back of the columns, no reduction in backfill pressure shall be made on account of the openings. No more than the active earth pressure shall be considered as the resistance offered by the fill in front of the abutment. In computing the active earth pressure of this fill, the negative or descending slope of the surface shall be taken into consideration.

If local conditions do not permit the construction of drains and, consequently, water may accumulate behind the wall, the resulting additional pressure shall be taken into account. Consideration should also be given to the eventual plugging of the drains due to infiltration of soil.

5.4 STABILITY COMPUTATION

5.4.1 Point of Intersection of Resultant Force and Base

The resultant force on the base of a wall or abutment shall fall within the middle third if the structure is founded on soil, and within the middle half if founded on rock, masonry or piles. The resultant force on any horizontal section above the base of a solid gravity wall should intersect this section within its middle half. If these requirements are satisfied, safety against overturning need not be investigated.

5.4.2 Resistance Against Sliding

The factor of safety against sliding at the base of the structure is defined as the sum of the forces at or above base level available to resist horizontal movement of the structure divided by the sum of the forces at or above the same level tending to produce horizontal movement. The numerical value of this factor of safety shall be at least 1.5. If the factor of safety is inadequate, it shall be increased by increasing the width of the base, by the use of a key, by sloping the base upward from heel to toe, or by the use of batter piles.

In computing the resistance against sliding, the passive earth pressure of the soil in contact with the face of the wall shall be neglected. The frictional resistance between the wall and a non-cohesive subsoil may be taken as the normal pressure on the base times the coefficient of friction of masonry on soil. For coarse-grained soil without silt, f may be taken as 0.55; for coarse-grained soil with silt, 0.45; for silt 0.35.

If the wall rests upon clay, the resistance against sliding shall be based upon the cohesion of the clay, which may be taken as one-half the unconfined compressive strength. If the clay is very stiff or hard the surface of the ground shall be roughened before the concrete is placed.

If the wall rests upon rock, consideration shall be given to such features of the rock structure as may constitute surfaces of weakness. For concrete on sound rock the coefficient of friction may be taken as 0.60.

The factor of safety against sliding on other horizontal surfaces below the base shall be investigated and shall not be less than 1.5.

5.4.3 Soil Pressure

The allowable soil pressure beneath the footing shall be determined in accordance with AREA Specifications for the Design of Spread Footing Foundations, Part 3, this Chapter.

5.4.4 Settlement and Tilting

The soil pressures determined in accordance with Sec. 5.4.3 provide for adequate safety against failure of the soil beneath the structure. If the subsoil consists of soft clay or silt it is necessary to determine the compressibility of the soil and to estimate the amount of settlement.

If the compressibility of the subsoil would lead to excessive settlement or tilting, the movement can be reduced by designing the wall so that the resultant of the forces acting at the base of the wall intersects the base near its midpoint.

If the pressure on a subsoil containing fairly thick layers of soft clay or peat is increased by the weight of the backfill, the wall may tilt backward because of the compression of the clay or peat. The tilt may be estimated on the basis of a knowledge of the compressibility of the subsoil. If the tilt is likely to be excessive, it is advisable to use backfill of lightweight material, to replace the backfill by a structure, or otherwise to change the type of construction so as to avoid overloading the subsoil.

5.4.5 Progressive Creep or Movement

If the weight of the backfill is greater than one-half the ultimate bearing capacity of a clay subsoil, progressive movement of the wall or abutment is likely to occur, irrespective of the use of a key, a tilted base, or batter piles. In such cases, it is advisable to use backfill of lightweight material, to replace the backfill by a structure, or otherwise to change the type of construction so as to avoid overloading the subsoil.

5.5 DESIGN OF BACKFILL

5.5.1 Drainage

The material immediately adjacent to the wall should be non-cohesive and free draining. Cinders shall not be used. If a special back drain is installed, the grain size of the drain shall be coarse enough to permit free flow of water, but not so coarse that the fill material may ultimately move into it and clog it. Where economical, it is preferable that free-draining material be used within a wedge behind the wall bounded by a plane rising at 60 deg to the horizontal. Water from the free-draining material shall be removed, preferably by horizontal drain pipes or by weep holes. Horizontal drain pipes, if used, shall not be less than 8-in diameter and shall be installed in such a position that they will function properly. Such drains shall be accessible for cleaning. Weepholes are considered less satisfactory than horizontal drains. If used, they shall have diameters not less than 6 in. and shall be spaced not over 10 ft.

5.5.2 Compaction

The backfill shall preferably be placed in layers not to exceed 12 inch in thickness. Each layer shall be compacted before placing the next, but overcompaction shall be avoided.

No dumping of backfill material shall be permitted in such a way that the successive layers slope downward toward the wall. The layers shall be horizontal or shall slope downward away from the wall.

5.6 DETAILS OF DESIGN AND CONSTRUCTION

5.6.1 General

The principles of design and permissible unit stresses for walls and abutments shall conform to the Specifications for Design of Plain and Reinforced Concrete Members, Part 2, this Chapter, with the modifications or additions in the following paragraphs.

The width of the stem of a semi-gravity wall, at the level of the top of the footing, shall be at least one-fourth of its height. Abutments shall be of the gravity or semi-gravity type.

The base of a retaining wall or abutment supported on soil shall be located below frost line, and in no case at a depth less than 3 ft. below the surface of the ground in front of the toe. The base shall be located below the anticipated maximum depth of scour. Where this is not practicable the base shall be supported by piles or other suitable means.

To prevent temperature and shrinkage cracks in exposed surfaces, not less than 0.25 sq. inch of horizontal reinforcement per foot of height shall be provided, irrespective of the type of wall. Consideration shall be given to providing additional reinforcement above horizontal joints.

The backs of retaining walls and abutments shall be damp-proofed by an approved material.

At horizontal joints between the bases and stems of retaining walls, raised keys are considered preferable to depressed keys. The unit shearing stress at the base of such a key shall not exceed $0.25f'c$.

Vertical keyed joints shall be placed not over 60 ft. apart to take care of temperature changes. They shall be protected by membrane waterproofing or noncorrosive water stops.

The walls above the footings shall be cast as units between expansion joints, unless construction joints are formed in accordance with the provisions of these specifications.

The heels of cantilever, counterfort and buttress retaining walls shall be proportioned for maximum resultant vertical loads, but when the foundation reaction is neglected the permissible unit stresses shall not be more than 50 percent greater than the normal permissible stresses.

5.6.2 Cantilever Walls

The unsupported toe and heel of the base slabs shall each be considered as a cantilever beam fixed at the edge of the support.

The vertical section shall be considered as a cantilever beam fixed at the top of the base.

5.6.3 Counterfort and Buttress Walls

The face walls of counterfort and buttress walls and parts of base slabs supported by the counterforts or buttresses shall be designed in accordance with the requirements for a continuous slab, Specifications for Design of Plain and Reinforced Concrete Members, Part 2, this Chapter. Due allowance shall be made for the effect of the toe moment on shears and bending moments in the heel slabs of counterfort walls.

Counterforts shall be designed in accordance with the requirements of T-beams. Stirrups shall be provided to anchor the face slabs and the heel slabs to the counterforts. These shall be proportioned to carry the end shears of the slabs. Stirrups shall be U-shaped with their legs in the counterforts, and shall extend as close to the exposed face of face walls and the bottom of base slabs as the requirements for protective covering permit. It is desirable to run reinforcing bars through the loops of the U.

Buttresses shall be designed in accordance with the requirements for rectangular beams.

Appendix A

EARTH PRESSURE FORMULAS FROM RANKINE-COULOMB THEORIES

The following formulas are applicable only to materials that may be considered cohesionless.

Case 1 to 3 are for vertical walls without heels. The pressure P is the same as the pressure on a vertical plane in the backfill. Vertical walls with heels come under Cases 4 to 6.

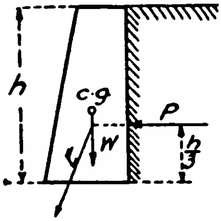
Case 4 to 6 are for walls with heels. The wall may be vertical or may lean forward, or may lean backward as long as the upper edge of the back of the wall is in front of the vertical plane through the edge of the heel.

Cases 7 to 9 are for walls without heels. Walls with heels come under Cases 4 to 6 as long as the upper edge of the back of the wall is in front of the vertical plane through the edge of the heel; if the upper edge of the back of the wall extends back of the vertical plane through the edge of the heel, the problem can be solved by combining the solutions of Cases 4 to 6 and 7 to 9.

For walls leaning forward or walls with the base extending into the backfill the pressure of the backfill on a vertical plane through the back of the heel of the wall is to be combined with the weight of backfill contained between this vertical plane and the back of the wall.

For walls leaning toward the backfill the resultant pressure P will be horizontal for a wall without surcharge, or for a wall with uniform surcharge, if the surface of the backfill is horizontal, and will make an angle λ with the horizontal for a wall with a sloping surcharge. The values of λ will vary from ζ, where the wall is vertical, to zero, where Rankine's theory shows that the resultant pressure is horizontal. Values of λ and values of K, where $P = \frac{1}{2} wh^2K$, are given in Diagram 10.

1· Vertical Wall, Horizontal Fill

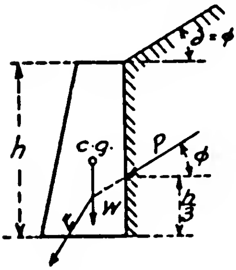


$$P = \frac{1}{2} wh^2 \frac{1-\sin\phi}{1+\sin\phi}$$
$$= \frac{1}{2} wh^2 \tan^2\left(45^\circ - \frac{\phi}{2}\right)$$

For $\phi = 1\frac{1}{2}$ to 1 ($\phi = 33^\circ 42'$)

$P = 0{\cdot}143\, wh^2$

2· Vertical Wall, Sloping Surcharge

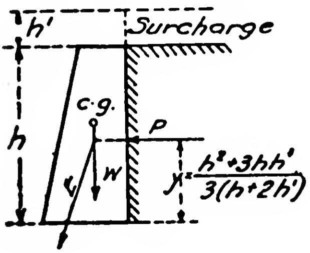


$$P = \frac{1}{2} wh^2 \cos\phi$$

For $\phi = 1\frac{1}{2}$ to 1 ($\phi = 33^\circ 42'$)

$P = 0{\cdot}416\, wh^2$

3· Vertical Wall, Loaded Surcharge

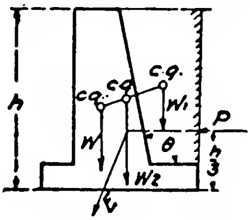


$$P = \frac{1}{2} wh(h+2h') \frac{1-\sin\phi}{1+\sin\phi}$$

For $\phi = 1\frac{1}{2}$ to 1 ($\phi = 33^\circ 42'$)

$P = 0{\cdot}143\, wh(h+2h')$

4. Wall Leaning Forward, Horizontal Fill

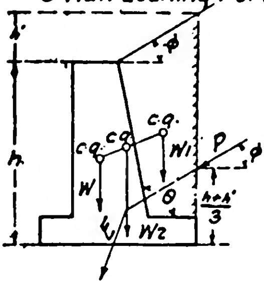


$$P = \frac{1}{2} w h^2 \frac{1-\sin\phi}{1+\sin\phi}$$
$$= \frac{1}{2} w h^2 \tan^2 (45^\circ - \frac{\phi}{2})$$

as in Case 1.

W = total weight of wall one ft long.
W1 = " " earth wedge " " "
W2 = W + W1.

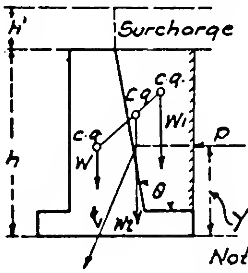
5. Wall Leaning Forward, Inclined Surcharge



$$P = \frac{1}{2} w (h+h')^2 \cos\phi$$

W = total weight of wall one ft long
W1 = " " earth wedge " " "
W2 = W + W1

6. Wall Leaning Forward, Loaded Surcharge



h' = surcharge per sq ft ÷ w

$$P = \frac{1}{2} w h (h+2h') \frac{1-\sin\phi}{1+\sin\phi}$$

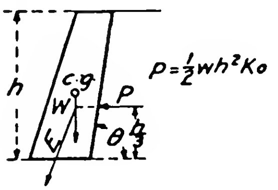
as in case 3

W = total weight of wall one ft long
W1 = " " earth wedge " " "
W2 = W + W1

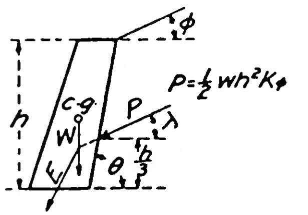
$$y = \frac{h^2 + 3h'h}{3(h+2h')}$$

Note: Wall should be investigated when W1 includes surcharge, and when surcharge over wedge is omitted.

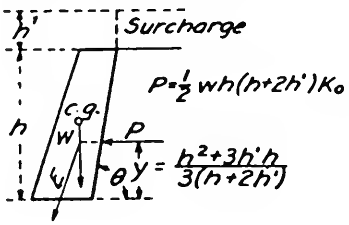
7·Wall Leaning Toward the Filling, Horizontal Fill

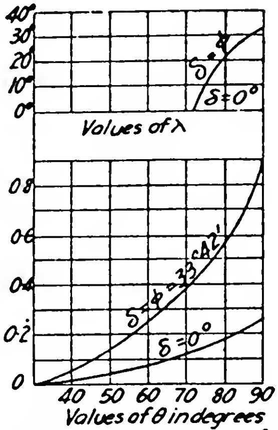


8·Wall Leaning Toward the Filling, Inclined Surcharge



9·Wall Leaning Toward the Filling·Loaded Surcharge





10·Values of Ko and Kφ for φ-33°42'

Appendix B

TRIAL WEDGE METHOD OF EARTH PRESSURE COMPUTATION

The trial wedge method is applicable for backfills of soils possessing cohesion, internal friction, or both; for backfills having any configuration of ground surface; and for surcharges located at any position on the backfill. The procedure, illustrated in Fig. 1, is outlined as follows:

Computation of Total Pressure

1. Make scale drawing of wall with backfill and any surcharge loads.
2. Locate surface AB against which earth pressure is to be computed. For walls with heels use vertical section as shown in Chart 1, Fig 1. For walls without heels use back of wall as shown in Chart 2.
3. Establish direction of earth pressure with respect to line AB, by the procedure described below.
4. Compute depth h_o of tension cracks if soil has cohesion.
5. Draw boundaries of trail wedges BC1, BD2, etc., wherein BC, BD, etc., are assumed plane surfaces of sliding.
6. Compute weights of successive wedges ABC1, ABD2, etc., including any surcharge acting on the ground surface within the limits of each wedge.
7. Lay off weight vectors for successive wedges.
8. Compute total cohesion of each surface of sliding BC, BD, etc.
9. Lay off cohesion vectors from lower ends of weight vectors, each parallel to the surface of sliding on which it acts.
10. From end of each cohesion vector draw line parallel to earth presser P.
11. From point B in force diagram lay off radial lines BC, BD, etc., each making an angle ϕ with the normal to its respective surface of sliding (as force R on surface BF).
12. Locate intersections of vectors R with corresponding lines drawn in step 10 and connect intersections with smooth curve. This is the earth pressure locus.
13. Determine maximum distance between line TT′ and the earth pressure locus, measured parallel to line of action of P. This distance represents the active earth pressure P.

Direction of Pressure P

For walls with heels, the following procedure is applicable:

1. Determine height h of wall, measured from point a.
2. Locate point b on the surface of the backfill at a distance 2h measured horizontally from a.
3. Draw line ab.
4. Take direction of resultant earth pressure P as parallel to line ab.

For walls without heels, where AB is the back of the wall, take angle ϕ equal to 2/3 ϕ.

Point of Application of Pressure

The point of application of the resultant pressure P can be obtained by determining the approximate pressure-distribution diagram, Fig. 1. The procedure is as follows:

1. Subdivide the line BB′ into about 4 equal parts h_1 below the depth h_0 of tension cracking.

2. Compute the active earth pressures P_1, P_2, P_3, etc., as if each of the points C′, D′, E′, etc., were at the base of the wall. The trial wedge method is used for each computation.
3. Determine the average pressures P_1, P_2, etc., over each distance B′C′,C′D′, etc., as indicated in Fig. 1.
4. Determine the elevation of the centroid of this approximate pressure diagram. This is the approximate elevation of the point of application of the resultant earth pressure P.

If the backfill may be considered cohesionless, the point of application of pressure may be obtained as follows:

a. Determine the center of gravity of the earth and ballast in the wedge between the plane of rupture and the vertical plane passing through the heel of the wall (Chart 1) or the back of the wall (Chart 2), Fig. 1.
b. Assume the center of gravity of the surcharge loads to be located at the surface of the backfill.
c. Determine the center of gravity of the combined loads and draw a line from this point parallel to the plane of rupture to a point of intersection with the vertical plane through the heel of the wall (Chart 1) or the back of the wall (Chart 2), Fig. 1.

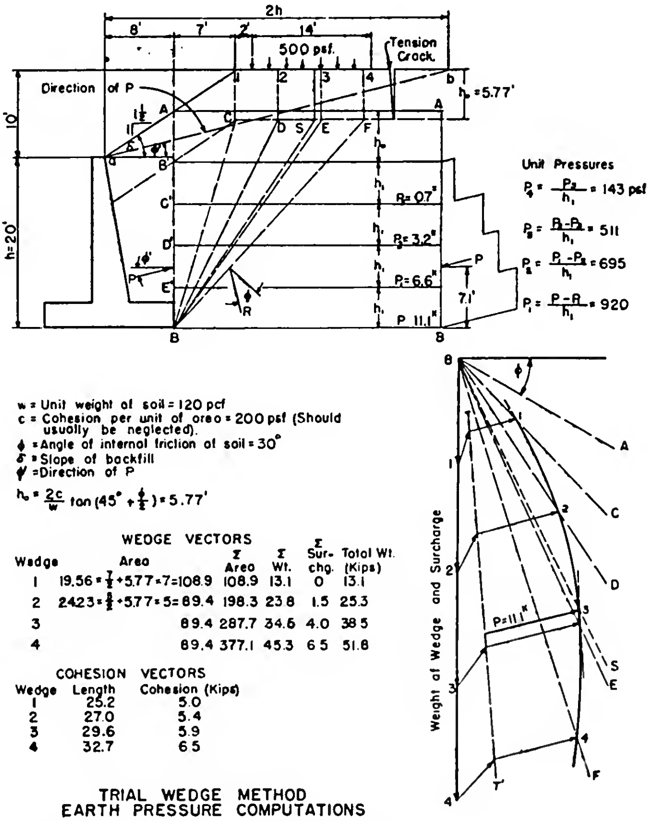
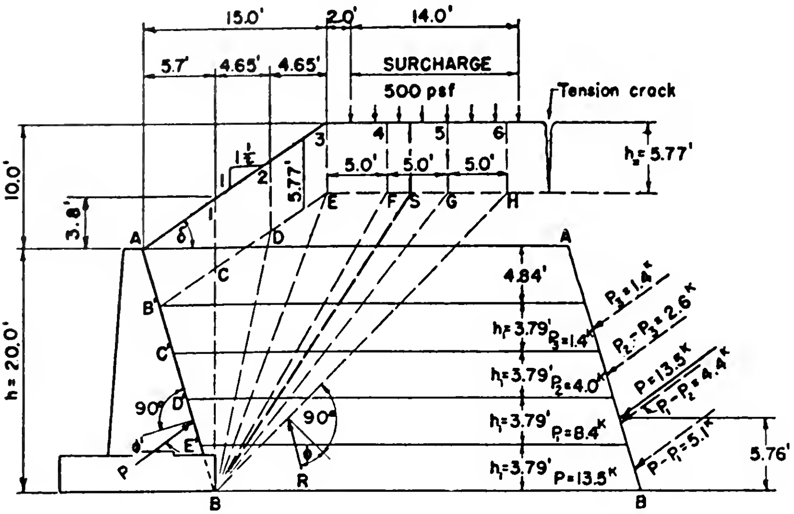
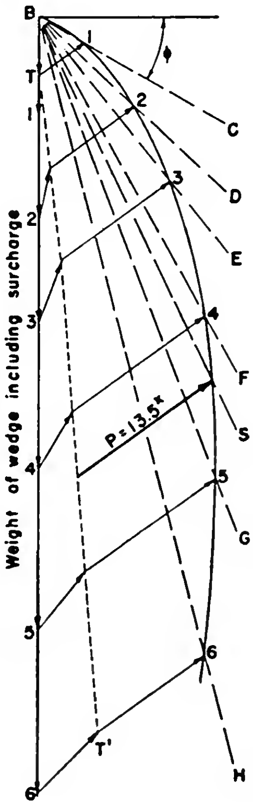


Fig. 1—Chart 1 (Walls with large heels)





w = Unit weight of soil = 120 pcf
c = Cohesion per unit of area = 200 psf
(Should usually be neglected)
ϕ = Angle of internal friction of soil = 30°
δ = Slope of backfill
ϕ' = Direction of P = $\frac{2}{3}\phi$ = 20°
$h_o = \frac{2c}{w}\tan(45° + \frac{\phi}{2}) = 5.77'$

WEDGE VECTORS

Wedge	Area		Σ Area	Σ Wt.	Σ Sur-chg.	Total Wt. (Kips)
1	$\frac{(20+3.8)\times 5.7}{2}$	= 67.8	67.8	8.1	0	8.1
2	$\frac{18.03\times 4.65}{2} + 5.77\times 4.65$	= 68.7	136.5	16.4	0	16.4
3	Same as 2	= 68.7	205.2	24.6	0	24.6
4	$\frac{24.23\times 5.0}{2} + 5.77\times 5.0$	= 89.4	294.6	35.4	1.5	36.9
5	Same as 4	= 89.4	384.0	46.1	4.0	50.1
6	" " "	= 89.4	473.4	56.8	6.5	63.3

COHESION VECTORS

Wedge	Length	Cohesion (Kips)
1	18.0	3.6
2	21.6	4.3
3	26.0	5.2
4	28.1	5.6
5	31.0	6.2
6	34.3	6.9

TRIAL WEDGE METHOD
EARTH PRESSURE COMPUTATIONS
FIG. 1 - CHART 2 (Walls with small heels)

Appendix C

EARTH PRESSURE CHARTS FOR WALLS LESS THAN 20 FT HIGH

Figs. 2 and 3 may be used for estimating the backfill pressure if the backfill material has been classified in accordance with Sec. B, Art. 4.

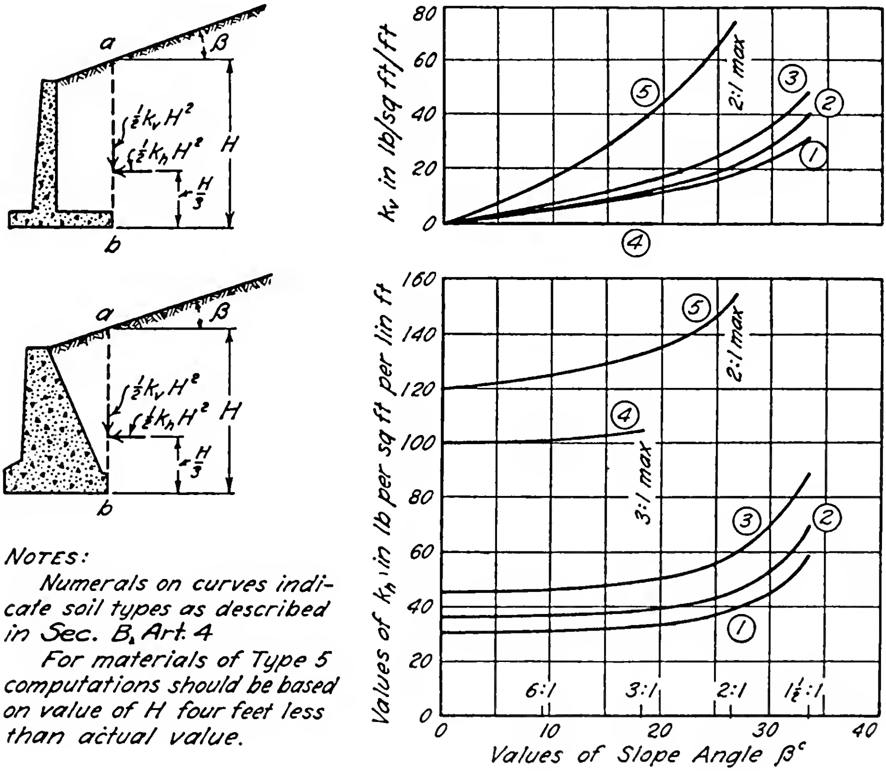


Fig. 2.

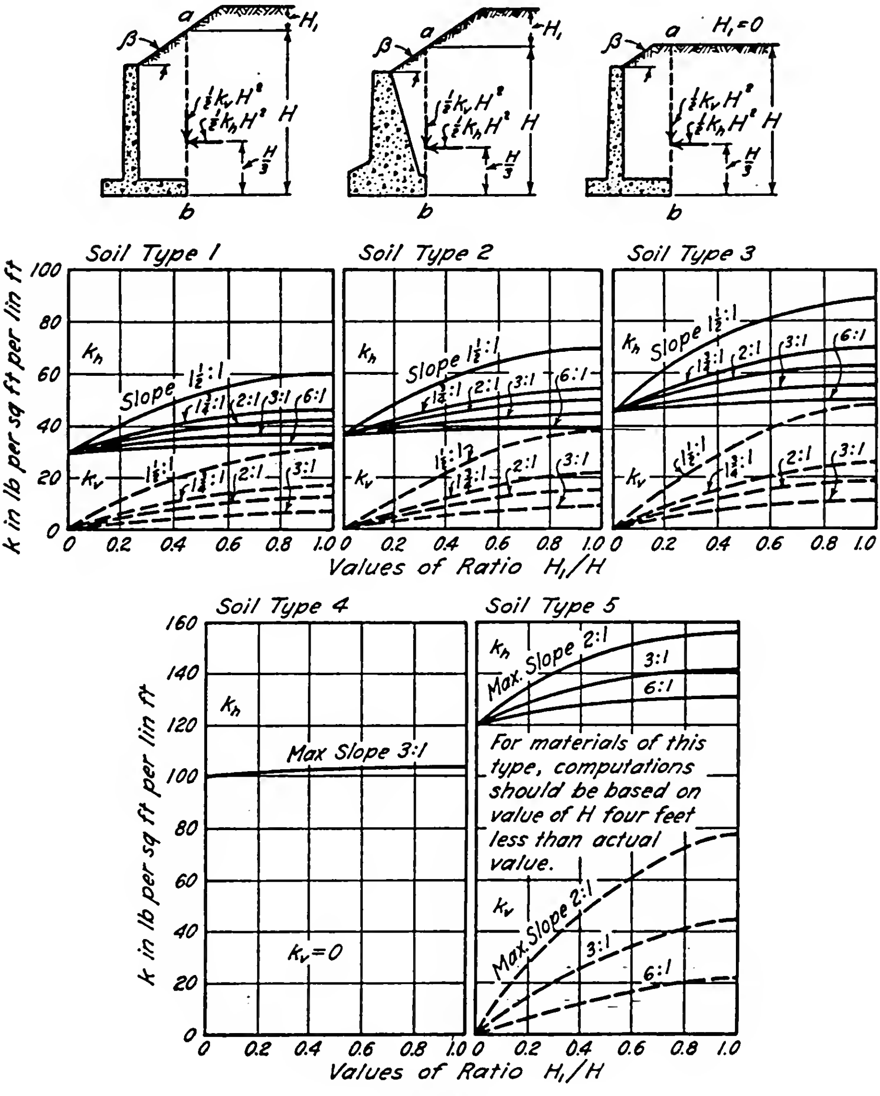

Soil Type 1
Soil Type 2
Soil Type 3
Soil Type 4
Soil Type 5
k in lb per sq ft per lin ft
Values of Ratio H_1/H
Max Slope 3:1
$k_v=0$
For materials of this type, computations should be based on value of H four feet less than actual value.
Max. Slope 2:1

Fig. 3.

PUBLISHED AS INFORMATION

COMMITTEE 6 - BUILDINGS

Part 1 DESIGN CRITERIA FOR FREIGHT FORWARDING FACILITIES

S. D. ARNDT (CHAIRMAN, SUBCOMMITTEE)

1979

0.1 FORWARD:

0.1.1 A Freight Forwarding Facility is a specialized facility designed for freight consolidation and distribution with an operating plan relating to the area it serves, and the volumes of freight to be handled.

This article outlines the planning required to assure that the facility will be adequate for the service standards it must meet.

0.1.2 There are three key areas that must be well planned and engineered in order to assure efficient shipping and receiving operations:

a. Dock Design and Yard Layout: Space for truck maneuvering, parking areas, track layout, weather protection, etc.
b. Number of inbound and outbound Dock Spots - Truck and Rail.
c. Accumulation Space: Space required behind shipping and receiving areas for accumulating shipments so truck or rail cars can be processed efficiently.

0.1.3 Consideration must be given to the number of shipments, pounds of freight and revenue dollars. This information is usually determined by the shippers and receivers operating the terminal.

The information contained in the following report is divided into two sections:

1. DESIGN INFORMATION: Information relating to specific factors, i.e., door sizes, dock heights, parking areas, etc.
2. DESIGN CHECK LIST: Check list to be used in determining requirements for a specific project.

0.2 DESIGN INFORMATION

0.2.1 Type of Operation: The four basic types of freight handling facilities are:

a. Rail to Rail
b. Rail to Truck
c. Truck to Truck
d. Combination of Rail to Rail and Truck to Truck

Figures 1, 2, 3 and 4 illustrate possible arrangements of the various types of operations indicated above.

The ultimate size of the facility should be determined by the types of traffic, the average amount of traffic to be handled initially, the variation of the peak from the average, and projections of possible growth. Thus consideration for expansion capabilities is a necessity.

Minimum operating costs will result when house tracks are placed between inbound and outbound freight houses with truck connections. Track areas may also be paved for trucks, and trucking doors provided in track houses for additional trucking operation if required.

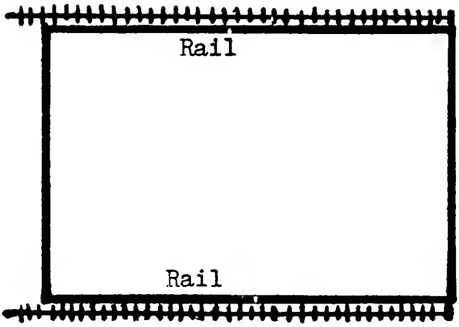


FIGURE 1

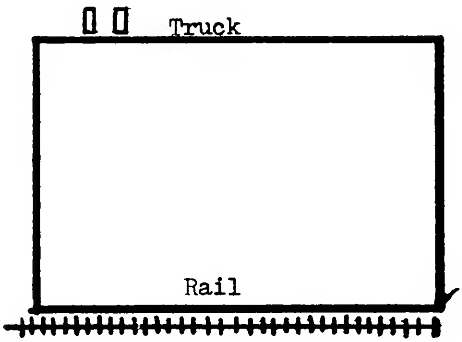


FIGURE 2

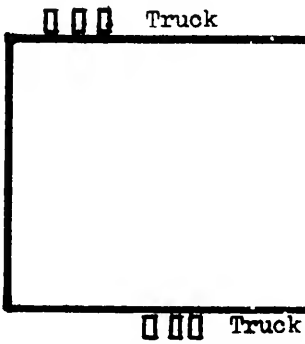


FIGURE 3

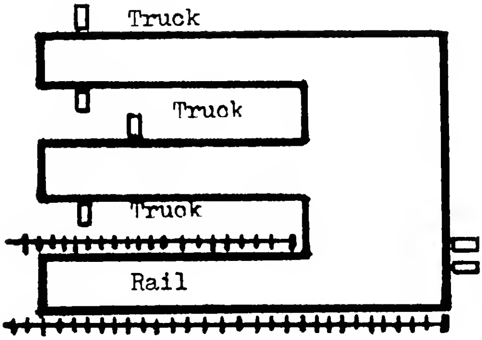


FIGURE 4

The facility should be designed to minimize freight handling labor. Where economically feasible mechanization should be considered.

0.2.2 Site:

a. Where there is a choice of sites, the following factors should be considered:

1. highway accessibility,
2. nearness to city pick-up,
3. space for future expansion,
4. proximity to existing switching service,
5. space for a new yard or proximity to existing supporting yard,
6. possible inclusion of TOFC/COFC facilities,
7. economics of location near terminal yards even though remote from city, and
8. relative land values.

b. Service Roads and Gates:

Traffic movement to and from the dock is very important and consideration should be given to the following factors:

1. Estimated amount of automobile, truck and pedestrian traffic.
2. Separation of pedestrian and automobile traffic from truck traffic is desirable.
3. Recommended Standard

A. Gates and Approach Roads: 16′ - one way traffic
28′ - two way traffic
6′ additional - if pedestrian traffic involved

B. Service Roads: 12′ - one way
24′ - two way
28′ - mixed traffic

C. Right Angle Roadway Intersections: 50′ radius desirable.
D. Traffic Circulation: Counterclockwise recommended, as it is easier for drivers to make left hand turns and back into docks from this position.
E. Traffic Control: Speed limit regulations should be posted, and wide-angle mirrors installed at blind corners.
F. Roadway Surfaces: Heavy wheel loading (40,000 lbs. on tandem axles in some states is legal) will be encountered. Refer to the local Highway Department design.

c. Waiting, Maneuvering/Loading Areas:

Since most facilities cannot be designed for peak loads, provisions should be made for a truck waiting area, or in the case of rail service for a large facility, storage tracks should be provided or the facility located near an existing yard. The waiting areas for trucks or rail cars should be designed and placed so that they will not interfere with movement of trucks or cars into the dock area. The rail car waiting area depends on switching operations, and the layout should be discussed with the railroad serving the facility.

In planning truck maneuvering areas consideration should be given to possible changes in truck sizes. An overall tractor-trailer length of 55′ is allowed in all states. Predictions are that 65′ or 70′ could be the average within the next five to ten years.

The length of the maneuvering area is determined by the direction of traffic flow.

1. Counterclockwise - Maneuvering - 40′ beyond loading area.
2. Clockwise - Maneuvering - 100′ beyond loading area.

See Figure 5

The loading area (space allowed at dock for tractor-trailer unit) should extend a minimum of 65′ from the dock face. If the loading area is asphalt, a concrete pad should be laid parallel to the dock to support dolly wheels. Pad should be as long as the dock, 20′ wide, and cover the area shown in Figure 6.

d. Width of Berth:

The most desirable berth width of 12′ will require 24′ column spacing. However, to lower construction costs, 11′ width with 22′ column spacing is frequently used. The overall width of tractor-trailers is 8′-0. The use of 11′ berths allows 3′-0″ on each side, which is acceptable.

e. Area Lighting:

Lights should be provided for the entire area, of sufficient intensity for loading, unloading, inspection and/or security.

f. Security Fencing:

The entire area should be adequately fenced to discourage entry and theft.

g. Security House:

A security house or houses should be located at tractor-trailer entrances and exits. Consideration should also be given to making provisions for a toilet in this facility. TV monitoring and other security devices should be considered.

0.2.3 Dock Building:

a. Orientation: The long dimension of the building should parallel prevailing winds if possible.
b. Column spacing: 24′ recommended, for 12′ berth width.
c. Width of building: Varies depending upon the type of operation, 45-70 ft. common widths used.
d. Height: Clear interior 15 ft. minimum
e. Canopy width: 3 ft. minimum, usual 12 ft. See Figure 7 for truck and track canopy suggested sizes.
f. Doors: 8′ wide x 10′ high for single doors and 20′ wide by 10′ high for double doors.
g. Floor: Reinforced concrete, designed for 400-500 PSF Loading, with nonslip finish.
h. Bumper: Wood with steel channel and/or rubber bumpers.
i. Steps: Provide one set of steps per four berths. Iron bar rungs set in dock front are acceptable.
j. Lighting: 30 foot-candles recommended. Skylight optional. Lights should be provided at trailer berths to illuminate trailer interiors.
k. Heat: Warming rooms should be provided in the freight house areas if heating of the dock area is impractical due to the number of doors open at various times.
l. Toilet facilities: Unless provided in office areas, toilet facilities should be located in the freight house. If warming room or rooms are provided, toilet should be adjacent.
m. Ventilation: Gravity ventilation is normally adequate, however with the use of gas forklifts, mechanical ventilation may be required.

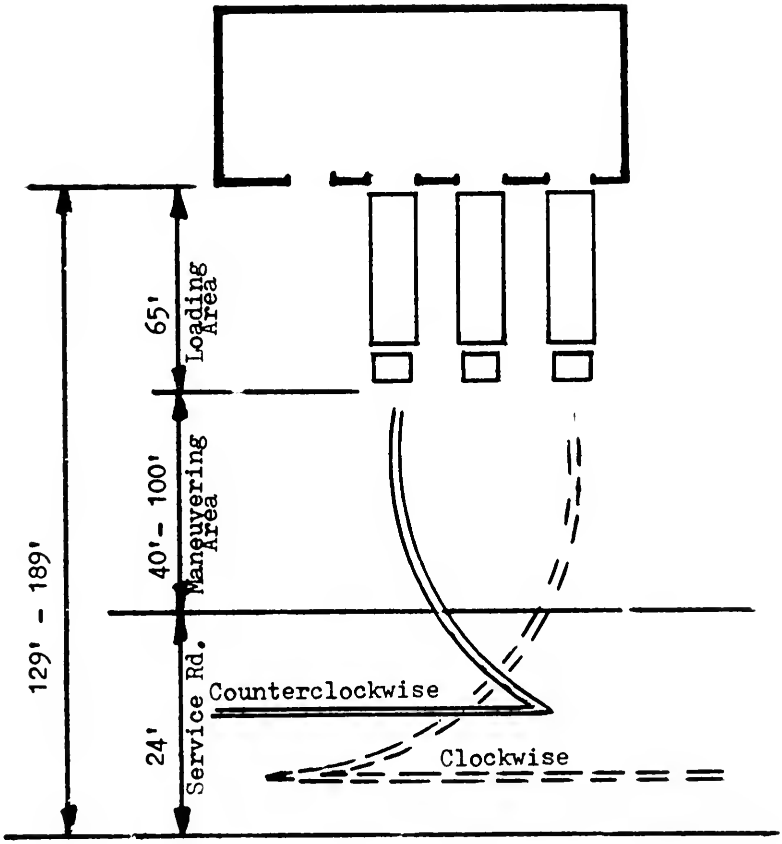

129' - 189'
65'
Loading Area
40'- 100'
Maneuvering Area
24'
Service Rd.
Counterclockwise
Clockwise

FIGURE 5

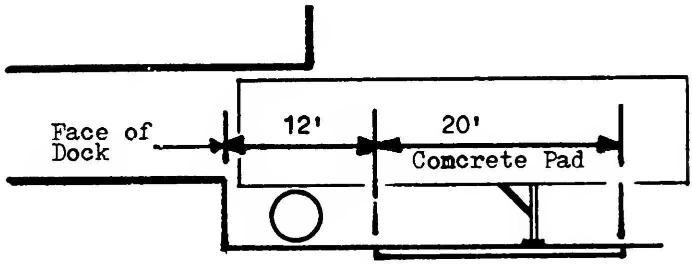


FIGURE 6

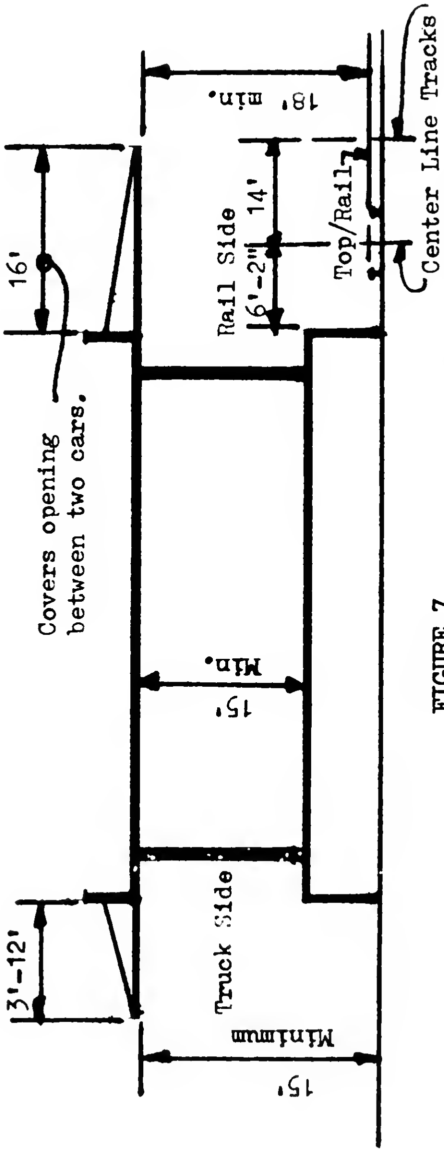


FIGURE 7

Note: Check Local Railroad Clearance Requirements at all tracks and Platforms.

n. Sprinkler system: Recommended for the entire dock area. Dry type system will be required if area is unheated.

o. Type of Docks:

1. Flush Dock - Outside wall of building is flush with face of dock. Provides common foundation for dock and wall, encloses dock area and provides support for dock seals. (Note: When using dock seals, wall will have to be offset 10-12 in. from face of dock - verify with supplier of dock seal.) Flush dock design requires a 4-10 ft. level area in front of the dock to prevent top of truck from hitting the building wall. See Figure 8.

2. Enclosed Docks - Two types; totally enclosed and straight-in-straight-out. See Figures 9 and 10.

Totally Enclosed Dock

Provides total protection from the elements, greater control of pilferage, and increased freight handling efficiency.

Straight-in-Straight Out

Requires a minimum of enclosed space, truck must back in, requires more maneuvering area outside enclosures than a flush dock.

Both types of enclosed docks require interior floor drains and mechanical ventilation for exhausting the tractor fumes.

3. Open Docks—Dock width should allow two way traffic of material handling equipment. Dock width is determined by multiplying the width of material being handled by (4), and adding the length of the dock board being used. See Figure 11.

Open dock design provides minimum security for materials, and usually requires high intensity lighting, safety switches, etc.

4. Saw Tooth Dock—Where space is critical and distance from dock edge to nearest obstruction is less than the length of a tractor-trailer unit, a saw tooth design should be employed. This type dock, however, reduces the useable space in the dock, and also dictates that trucks depart within the angle of the dock and approach from the opposite side. See Figure 12.

Direction of Truck Movement

5. Rail Docks—Modernization will make many existing rail siding facilities obsolete, and require that many more be extensively remodeled. Planners should take possible future equipment designs into consideration; for example, future cars may be 85′ in length, incorporate different door arrangements, etc.

a. Open Platforms:

Open platforms eliminate the necessity of exact spotting of cars... as well as the cost of "spotting" equipment, since either permanent rail ramps or portable plates can be brought to the car openings. Open platforms also substantially reduce building, heating and shelter costs... even if canopies are used.

Platforms should be 43″ high for servicing of new boxcars and 48″ high for servicing of refrigerator cars.

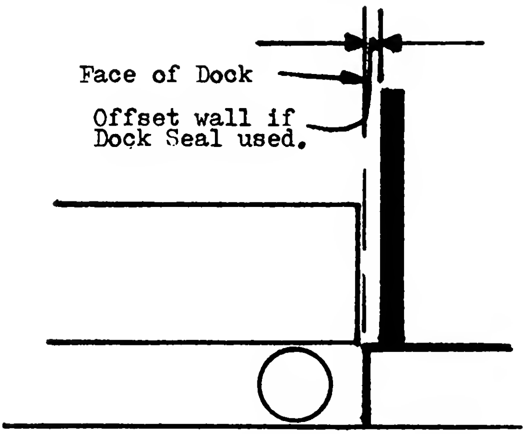


FIGURE 8

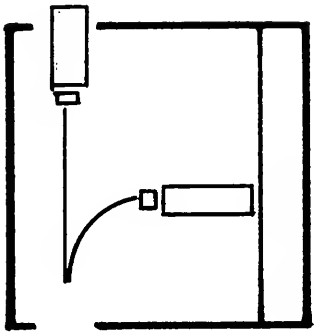


FIGURE 9

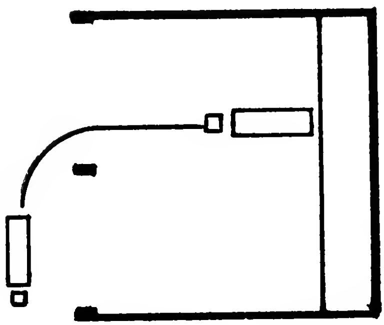


FIGURE 10

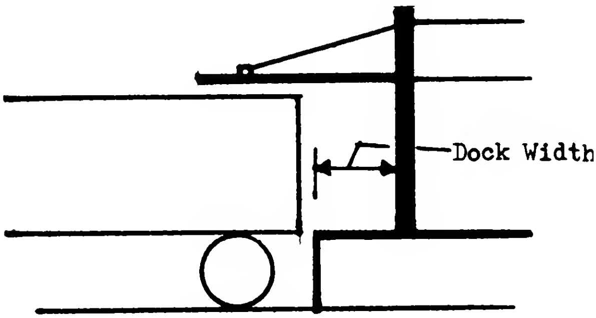


FIGURE 11

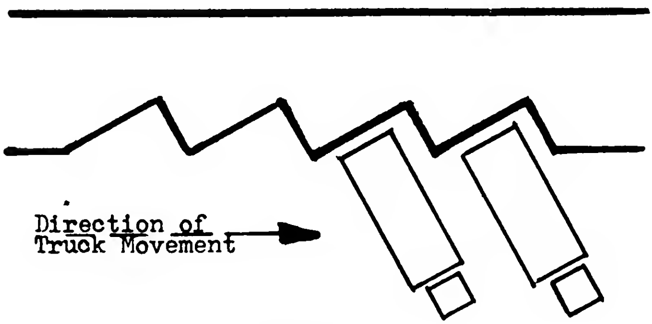


FIGURE 12

Clearance dimensions from edge of dock to centerline of track are established by state law. These generally differ for inside and outside docks. Legal minimums should be used whenever possible, to keep car to dock spans to a minimum.

For maximum material handling efficiency platforms should have a clear 14′ width. This permits 4′ wide loads to pass, with 2′ clearance between the material handling units, and 2′ between the unit and the edge of the platform.

b. Completely Enclosed Docks:

This type requires the same design principals discussed under open docks... as platform configurations are identical. The chief advantage is complete independence from the outside environment.

c. Flush Docks:

Many times a completely enclosed building is necessary for proper handling of freight. However, it is important that current design trends be given due consideration, since remodeling at a later date will be extremely costly.

1. Height. (See Figs. 13 and 14)

2. Clearance. (See Figs. 13 and 14)

3. Openings. Any permanent openings should be equipped with overhead doors and be large enough to accommodate the largest anticipated car door opening. Since car door openings of 16′ are being projected, building openings of 16′ should be considered, to permit full access to even largest unit loads, require less critical spotting, and permit greater flexibility of operation.

Depending on climate and types of freight, shelter should be considered. If not provided, it is recommended that a vestibule be constructed inside the building for each outside opening. The vestibule should be equipped with impact type traffic doors. Traffic door size should be a minimum of 8′ × 8′. Space between vestibules can be used for storage or production, since the doors afford excellent environmental control.

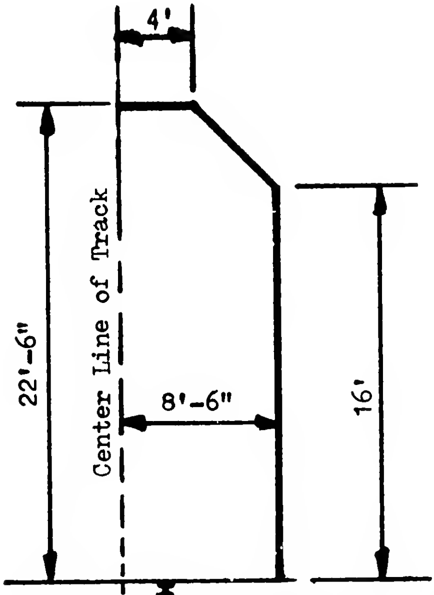
Main aisles leading from openings should be a minimum of 14′ wide, to permit two-way traffic and turning.

p. Dock Height: Truck beds normally vary from 48-52 inches; however, pickup and delivery will have heights from 44-50 inches. Note 48″ height requires guard rail and 46″ height will eliminate.

Recommended dock height is such that truck beds are above dock level, to prevent a runaway load trapping a man in a truck. A dock height of 46″ should serve most types of trucks.

q. Self-Leveling Dockboards: Dock levelers speed up turnaround of trucks, and increase dock productivity. Permanently installed boards are safer than portable boards. The one exception to the desirability of dockboards would be at a facility utilizing conveyors exclusively to move material.

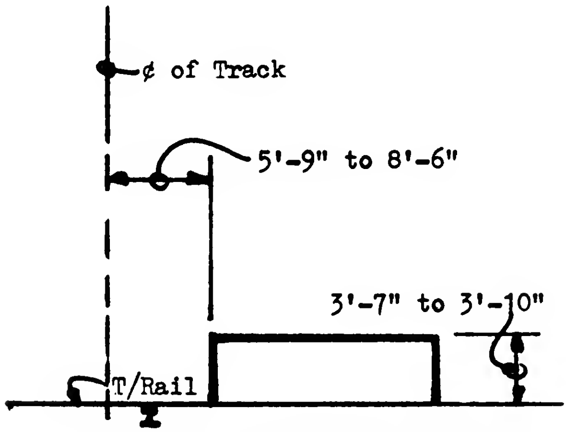
1. Greatest height difference between the dock and bed of trucks or trailers seserviced.
2. Type of materials handling equipment used.



Clearance Diagram for Structures
Adjacent to Industrial Tracks.

FIGURE 13

Note: Check Local Railroad
Requirements.



Clearance Diagram for Platforms

FIGURE 14

Note: Check Local Railroad Requirements.

3. Type of loads handled into and out of vehicles.
4. Type of vehicle road equipment picking up or delivering freight.

After determining the maximum height difference from dock level, the length of the dockboard can be calculated by using the allowable percentage grade for the type of handling equipment used. Allowable grades are shown in Table 1.

TABLE 1—Percent of Grade for Material Handling Equipment

Type of equipment	Allowable percent of grade*
Powered handtrucks	3
Powered platform trucks	7
Low-lift pallet or skid trucks	10
Electric fork trucks	10
Gas fork trucks	15

*Contact manufacturer and check manufacturer's specifications before operating beyond operating beyond allowable percent of grade.

Most standard truck dockboard lengths range from 6 to 10 ft. For most applications, dockboards should be 6 ft. wide, except that 7 ft. wide dockboards are recommended with fork trucks.

Additional information relating to design requirements is available from:

Shipper—Motor Carrier Dock Design Manual
Operation Council
American Trucking Association
1616 P Street, N.W.
Washington, D.C. 20036
Manual No. MH8-1

Reference is also given to a design manual covering Modular Freight Centres published by the Public Transport Commission of New South Wales. Contact:

The Investigation and Planning Section
Ways and Works Branch
Room 1008 10th Floor
Transport House
11-31 York Street
Sydney, New South Wales 2000
Australia

FREIGHT FORWARDING FACILITY

0.3 DESIGN CHECK LIST:

0.3.1 Location:

a) City .... State ....
b) In Yard...
c) Applicable Codes ....
d) Zoning ....
e) Fire Zone...

0.3.2 Type of Operation:

a) Rail to Rail ....
b) Truck to Truck ....
c) Rail to Truck...
d) Combination (Rail and Truck) ....

0.3.3 Facility Requirements:

a) Freight house .... Maintenance Shop ....
.... Scale .... Office Building ....
Fueling Area....................... Trailer Parking ....
Auto Parking .... Heater Outlets Outside ....
Other ....
....

0.3.4 Site:

a) Layout indicating property lines, easements, utilities, on site
b) Existing Topography
c) Utilities Available: Gas, water, sewer, electrical
d) Accessibility to site by truck and rail
e) Expansion flexibility capabilities
f) Security capabilities
g) Roadways
h) Lighting

0.3.5 FUNCTIONAL REQUIREMENTS:

A) Site:

1. Trailer storage area for .... trailers, size ....
2. Security guard house required .... , size ....
3. Security fencing required...................., type ....
   location .... Gates: Manual or Mechanical
4. Pavement requirements: Blacktop ....Concrete...............
5. Dolly Pads ...., size, location.......................
6. Employee parking.................... No. of Cars....................
   Location ....
7. Visitor parking .... No. of Cars....................
   Location ....
8. Storm drainage requirements ....
9. Outside Area Lighting (Foot candles required) ....

B) Dock Building:

Freight House (Inbound or Outbound) Information required for each area.

1. Width of house ....Height ....Bay Spacing ....
2. Width of berth to be provided ....
3. Truck spots required .... Door Size and Type ....
4. Rail spots required .... Door Size and Type ....
5. Dock height at truck side .... Rail side.................
6. Dock width at truck side........................ Rail side.................
7. Canopy width—truck side...................... Rail side.................
8. Dock levelers/boards—truck side................ Rail side ....
   (Provide info. on type and capacity required)
9. Weather seals at doors (Size and Type)—Truck Side ....
   Rail Side ....
10. Floor loading (PSF) ....
11. Lighting (Foot Candles Required)...
    a) Skylights Required ....
12. Fire protection (Type and Location)...
13. Heat (Type and Location) ....
14. Ventilation (Gravity, Mechanical, Make-up Air)...........................
    ....
15. Material Handling—Fork Lifts...................Type ....
    Infloor Conveyor .... Others ....
16. Scale (Type and Location) ....
17. Security Area (OS & D), Size & Location ....
18. Refrigerated Area, Size & Location...
19. Cranes..
20. Warming Room Areas/Toilet Facilities Required at Dock and Location ....
    ....
21. Ladders or Steps Required at Dock ....Spacing................
22. Cranes, Monorails or Towveyor (Type and Capacity Required)..............
    ....
23. Towveyor Cart Repair Area..
24. Towveyor Cart Surge Bank Area (No. of Carts) ....
25. Fork Lift Maintenance Shop..
26. Fork Lift Fueling or Battery Changing Area ....
    ....

C) Office Area (Building)

The following office areas may be required. Check those areas required and provide sketch to indicate relationship of areas to each other and location in respect to the Freight House and other terminal facilities.

1. General Office
   Size, number of people, etc. ....
2. Message Center..
3. Billing Office ....
4. Cashier ....
5. Telephone Room ....
6. Foreman's Office ....
7. Office Manager ....

8. Terminal Manager . . . .
9. Operations Manager. . . . .
10. Salesmen's Office . . . .
11. Record Storage . . . .
12. Sanitary Facilities—Office, Freight Handlers and Lunch Room
. . . .
13. Central Checking . . . .
14. Driver's Ready Room . . . .
15. Other. . . . .
. . . .

D) Miscellaneous Facilities

1. Maintenance Shop . . . .
2. Fueling Area . . . .
3. Weighing Area . . . .
4. Customs Facilities. . . . .

MEMOIRS

ROY SEVAN BELCHER
1883-1979

Roy Sevan Belcher, retired manager of tie & timber treating department of the AT&SF Railroad, died at his home in Topeka, Kansas after a short illness on January 25, 1979.

Mr. Belcher was born January 6, 1883 at Stoughton, Massachusetts and moved to Topeka, Kansas in 1920.

Mr. Belcher was employed in the tie & timber treating department of the AT&SF Railroad for 37 years, beginning in 1912 at Sommerville, Texas. Before his retirement in 1949, he was manager of the department.

He was graduated from Lombard College with a degree in industrial chemistry. He was a member of the First Presbyterian Church, Masonic Lodge, Siloam No. 225, Beulah Chapter No. 34, Order of the Eastern Star and Knights Templar all of Topeka. He was also a member of the Shawnee County Historical Society, Sons of the American Revolution, Kansas Society of Mayflower Club and Member Emeritus of Committee III - Ties and Wood Preservation.

Mr. Belcher served as president of the American Wood Preservers Association in 1933 and received that organizations Award of Merit in 1967.

Mr. Belcher is survived by a son, retired Navy Capt. Roy S. Belcher, Jr., South Walpole, Mass.; two daughters, Helen Jane of the home and Mrs. Sumner J. Logan, Wilmette, Illinois.

Mr. Belcher joined the American Railway Engineering Association on August 16, 1920 became a Life Member 1950.

Member Emeritus Committee III, Sept. 30, 1957.

Member Emeritus Committee 17, Dec. 7, 1953.

Member Committee III, 1923 to date of death.

Member Committee 17, 1921 to consolidation with Committee III in 1975.

M. BRUCE MILLER
1917-1979

Merritt Bruce Miller died on Monday, February 26, 1979 at his home, 1479 Lexington Lane, Wayne, Pennsylvania.

Mr. Miller was born at Joliet, Illinois, September 12, 1917. A graduate of Iowa State University with a B.S. in Civil Engineering, Mr. Miller was a Registered Professional Engineer in the state of Pennsylvania.

Bruce began his career in the Engineering Department of the former Pennsylvania Railroad in June of 1940. Starting as an Assistant on the Engineering Corps, Mr. Miller advanced through the levels of Track Supervisor, Division Engineer and Regional Engineer, all at various locations throughout the railroad. Upon creation of Penn Central, Mr. Miller was named Chief Regional Engineer and later served at system headquarters. Upon his death, Mr. Miller was Director of Passenger Transportation Engineering for Conrail.

He entered military service in July 1942 and was discharged from active duty in January of 1946, after having obtained the rank of Captain, serving in the field artillery in the European Theater of operation.

Joining the American Railway Engineering Association in 1948, Bruce became immediately active in association affairs. He worked his way through various sub-committee chairmanship, became Vice Chairman of Committee 16 in 1973 and served as Chairman of the Full Committee from 1975 to 1977. Mr. Miller was for many years active on the Annual Technical Conference Arrangements Committee and was nominated to serve on the Association's Nominating Committee for 1979. He was also a member of the Wayne Presbyterian Church, the Valley Forge Lions Club and the Masons.

Mr. Miller is survived by his wife, the former Patricia Samsel, two daughters, Mrs. Shannon (Deborah) Patrick of South Bend, Indiana, Jenifer, at home and two grandchildren, Kevin and Megan Patrick.

L. E. Ward
T. C. Nordquist

ALTON VINCENT JOHNSTON
1909-1979

Alton V. Johnston, for more than two decades a senior Canadian National Railways executive in Montreal, passed away at his home in St. Thomas, Ont. on Wednesday, August 15th.

From 1971 until his retirement in 1974, Mr. Johnston was President of CANAC Consultants Ltd., a CN subsidiary that carries out railway consulting work abroad. Most of its work is in developing countries. It often involves full transportation programs in which Air Canada provides the aviation expertise.

Before CANAC Consultants Ltd. was set up in 1971, Mr. Johnston was General Manager of its predecessor organization, CNAC, involved in the same type of international consultants' work.

He was CN's Assistant Chief Engineer from 1955 to 1958, and Chief Engineer from then until 1968 and his appointment to head the consulting service.

Mr. Johnston was a Director of the A.R.E.A. in 1957 and 1959. He became 1st Vice-President in 1964-65 and President in 1965-66.

He was born in St. Thomas, Ont. July 31, 1909, and joined CN as an apprentice in 1927, gained a Bachelor of Science Degree from Queen's University in 1935, and held various executive posts in Ontario before moving to Montreal.

He is survived by his widow, Grace Irene, and two sons, David of Toronto and Eric of Ottawa.

Advance Report of Committee 3
Ties and Wood Preservation

Report on Assignment 5
Service Records

K.C. Edscorn (Chairman, sub-committee), L.C. Collister, M.J. Crespo, E.M. Cummings, J.K. Gloster, H.E. Richardson, R.H. Savage, G.D. Summers

Statistics providing information on cross tie renewals and average tie costs for the 1978, as compiled by the Economics and Finance Department, Association of American Railroads, are presented on the following pages in Tables A and B.

The 1978 statistics on new tie renewals by Class I, U.S. Railroads compared with 1977 are as follows:

Year	Total New Tie Renewals	Renewals Per Mile
1977	25,363,218 *	91
1978	25,033,738 **	88

By geographical districts, the Eastern Roads inserted in replacement 105 ties per mile, the Southern Roads 103 ties per mile and the Western roads 75 ties per mile. Average for the United States was 88 ties per mile.

"Indicated" wood tie life determined by dividing the total number of ties in track ''67 figures) by the number of new ties inserted in 1978 is as follows:

Eastern Roads	29 yrs.
Southern Roads	30 yrs.
Western Roads	41 yrs.
All U.S. Class I Roads	35 yrs.

Although the total number of new cross ties renewed increased less than 2% during 1978, two very significant increases are called to your attention in the statistics. The use of second-hand ties was 28% higher than 1977 and tie costs range from 5% to 18% more on January 1, 1979 compared to 1978 reports.

As they were affected in 1974, maintenance budgets are being squeezed again by the inflationary spiral of higher timber, labor and preservative costs experienced by all producers. A Railway Tie Association survey of Class I Railroads resulted in an estimated requirement for ties of 29,627,000, an 18% increase over actual renewals in 1978. The question becomes how much increase can the budget stand before maintenance begins to be deferred. The substantial increase in the use of second-hand ties seems to be an indication that some maintenance engineers are already taking steps to reduce expenditures.

*Excludes 96,932 concrete ties and 600,696 secondhand ties

**Excludes 42,079 concrete ties and 769,656 secondhand ties

Table A

CROSS TIE STATISTICS (EXCLUDING SWITCH & BRIDGE) FOR CLASS I RAILROADS IN THE UNITED STATES
Calendar Year ended December 31, 1978

District and Road	Wooden cross ties laid in replacement (number): New Ties	Wooden cross ties laid in replacement (number): Second hand ties	Other than wooden cross ties laid in replacement (number)	Track maintained by reporting railroad: Miles occupied by cross ties a	Track maintained by reporting railroad: Total cross ties b	Cross ties per mile (1967)	Equated gross ton miles (000) c: Total	Equated gross ton miles (000) c: Per mile of track	New cross tie replacement averages: Percent renewal to all ties	New cross tie replacement averages: Number laid per mile
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
EASTERN DISTRICT										
Baltimore & Ohio	388 025	6 172	-	9 524	27 619 600	2 900	64 465 135	6 769	3 22	93
Bessemer & Lake Erie	49 328	1 279	-	435	1 305 000	3 000	3 816 619	8 774	3.78	113
Boston & Maine	88 907	-	-	1 519	4 481 050	2 950	6 448 904	4 245	1 98	59
Chesapeake & Ohio	770 561	6 559	-	7 916	23 748 000	3 000	51 619 968	6 521	3 24	97
Conrail	4 874 864	21 129	-	34 427	103 108 865	2 995	256 134 380	7 440	4 73	142
Delaware & Hudson	17 275	3 312	-	1 171	3 630 100	3 100	8 592 912	7 338	0 48	15
Detroit, Toledo & Ironton	52 811	-	-	606	1 745 280	2 880	3 373 767	5 567	3.03	87
Elgin, Joliet & Eastern	55 742	-	-	896	2 739 968	3 058	1 801 331	2 010	2 03	62
Grand Trunk Western	134 952	-	-	1 979	6 237 808	3 152	11 116 544	5 617	2.16	68
Long Island	135 554	2 392	-	695	1 966 155	2 829	4 616 074	6 642	6.89	195
Norfolk & Western	615 903	-	-	12 871	40 041 681	3 111	93 732 099	7 282	1 54	48
Pittsburgh & Lake Erie	41 731	-	-	676	2 067 884	3 059	2 448 104	3 621	2 02	62
Western Maryland	33 531	-	-	1 069	3 102 238	2 902	3 780 944	3 537	1 08	31
Total Eastern District	7 759 184	40 843	-	73 784	221 793 629	3 002	511 946 781	6 938	3 90	105
SOUTHERN DISTRICT										
Clinchfield	47 248	216	-	486	1 574 640	3 240	7 062 581	14 532	3 00	97
Florida East Coast	22 137	-	41 985	876	2 641 140	3 015	5 771 230	6 588	0 84	25
Illinois Central Gulf	1 381 810	-	-	13 677	43 397 121	3 173	72 276 925	5 285	3 18	101
Louisville & Nashville	967 908	-	-	10 196	29 405 264	2 884	91 723 936	8 996	3 29	95
Seaboard Coast Line	1 241 362	-	86	13 790	42 928 270	3 113	93 391 903	6 772	2 89	90
Southern System d	1 728 937	90 705	-	13 431	41 797 272	3 112	138 717 441	10 328	4 14	129
Total Southern District	5 389 402	90 921	42 071	52 456	161 743 707	3 083	408 944 016	7 796	3 33	103
WESTERN DISTRICT										
Atchison, Topeka & Santa Fe	1 732 002	48 915	-	19 528	62 352 904	3 193	197 086 933	10 093	2 78	89
Burlington Northern	2 756 468	18 110	-	32 126	97 630 914	3 039	271 249 148	8 443	2 82	86
Chicago & North Western	578 821	371 306	-	12 793	38 135 933	2 981	68 143 064	5 327	1 52	45
Chicago, Milwaukee, St. Paul & Pac	673 222	104 020	-	12 469	37 818 477	3 033	49 183 763	3 944	1 78	54
Chicago, Rock Island & Pacific	378 969	34 609	-	9 087	27 079 260	2 980	37 216 295	4 096	1 40	42
Colorado & Southern	55 186	1 309	-	754	2 307 240	3 060	10 140 978	13 450	2 39	73
Denver & Rio Grande Western	328 628	475	-	2 770	6 550 990	3 087	26 374 573	9 522	3 84	119
Duluth, Missabe & Iron Range	37 986	9 975	-	798	2 377 242	2 979	4 223 652	5 293	1 60	48
Fort Worth & Denver	110 603	555	-	1 407	4 222 407	3 001	9 345 141	6 642	2.62	79
Kansas City Southern (incl L&A)	287 196	-	8	2 369	7 578 431	3 199	6 084 909	2 569	3.79	121
Missouri-Kansas-Texas	257 735	-	-	2 348	7 522 992	3 204	13 953 770	5 943	3 43	110
Missouri Pacific	1 315 975	-	-	15 423	47 471 994	3 078	127 993 406	8 299	2 77	85
St Louis-San Francisco	701 460	12 800	-	6 309	19 803 951	3 139	44 953 831	7 125	3 54	111
St Louis Southwestern	212 760	-	-	1 818	5 568 534	3 063	28 399 333	15 621	3 82	117
Soo Line	341 436	2 179	-	5 532	16 640 256	3 008	23 664 898	4 278	2 05	62
Southern Pacific	1 322 661	-	-	17 188	50 704 600	2 950	190 366 760	11 076	2 61	77
Union Pacific	701 490	33 739	-	14 398	40 703 146	2 827	202 646 244	14 075	1 72	49
Western Pacific	92 554	-	-	1 887	5 632 695	2 985	15 855 067	8 402	1 64	49
Total Western District	11 885 152	637 992	8	159 004	482 101 966	3 032	1326 881 765	8 345	2 47	75
Total United States	25 033 738	769 656	42 079	285 244	865 639 302	3 033	2247 772 562	7 880	2.89	68

a Total mileage operated at end of year, excluding mileage under trackage rights
b Column 5 based on cross ties per mile of track in 1967, the last year for which reported. Conrail average estimated
c Gross ton miles of cars and contents, plus two times gross ton miles of locomotives in freight service, plus three times gross ton miles of locomotives in passenger service
d Includes all Southern System Class I railroads

TABLE 8

NUMBER OF NEW CROSS TIE RENEWALS PER MILE OF MAINTAINED TRACK AND RATIO OF NEW CROSS TIE RENEWALS TO TOTAL CROSS TIES IN MAINTAINED TRACK

Class I roads in the United States, by years, and for the average of five years 1974 to 1978 inclusive

Note: All figures are exclusive of switch and bridge ties

District and Road	Number of new cross tie renewals per mile of maintained track						Percent new cross tie renewals to all ties in track					
	1974	1975	1976	1977	1978	5 year average	1974	1975	1976	1977	1978	5 year average
EASTERN DISTRICT:												
Baltimore & Ohio	50	76	75	85	93	76	1.72	2.63	2.59	2.93	3.22	2.62
Bessemer & Lake Erie	167	148	146	143	113	143	5.58	4.94	4.87	4.77	3.78	4.79
Boston & Maine	71	75	107	103	59	83	2.39	2.56	3.63	3.50	1.98	2.81
Chesapeake & Ohio	24	48	63	71	97	61	0.80	1.60	2.11	2.36	3.24	2.02
Conrail	45	79	112	123	142	100	1.51	2.64	3.74	4.09	4.73	3.34
Delaware & Hudson	32	124	117	88	15	75	1.04	4.12	3.77	2.83	0.48	2.45
Detroit, Toledo & Ironton	42	39	55	89	87	62	1.47	1.35	1.90	3.08	3.03	2.17
Elgin, Joliet & Eastern	95	99	106	98	62	92	3.12	3.24	3.46	3.19	2.03	3.01
Grand Trunk Western	57	51	71	85	68	66	1.82	1.61	2.25	2.71	2.16	2.11
Long Island	83	107	120	135	195	128	2.92	3.78	4.24	4.77	6.89	4.52
Norfolk & Western	88	61	77	98	48	74	2.82	1.95	2.47	3.15	1.54	2.39
Pittsburgh & Lake Erie	68	65	62	80	62	67	2.22	2.13	2.01	2.62	2.02	2.20
Western Maryland	28	48	40	44	31	38	0.96	1.67	1.36	1.51	1.08	1.32
Total Eastern District	53	72	95	104	105	86	1.75	2.41	3.17	3.46	3.50	2.86
SOUTHERN DISTRICT:												
Clinchfield	92	105	109	105	97	102	2.83	3.26	3.37	3.23	3.00	3.14
Florida East Coast	193	28	78	27	25	70	6.42	0.94	2.57	0.89	0.84	2.33
Illinois Central Gulf	83	30	94	107	101	83	2.61	0.93	2.96	3.39	3.18	2.61
Louisville & Nashville	61	92	94	96	95	88	2.12	3.19	3.26	3.34	3.29	3.04
Seaboard Coast Line	62	75	82	89	90	80	2.00	2.42	2.65	2.85	2.89	2.56
Southern System a	160	94	170	103	129	131	5.15	3.03	5.47	3.32	4.14	4.22
Total Southern District	97	71	112	97	103	96	3.14	2.32	3.63	3.16	3.33	3.12
WESTERN DISTRICT:												
Atchison, Topeka & Santa Fe	64	68	81	88	89	78	2.01	2.14	2.55	2.77	2.78	2.45
Burlington Northern	63	71	88	90	86	80	2.06	2.34	2.89	2.97	2.82	2.62
Chicago & North Western	52	38	34	42	45	42	1.47	1.27	1.14	1.42	1.52	1.36
Chicago, Milwaukee, St. Paul & Pac.	45	23	27	56	54	41	1.80	0.76	0.90	1.85	1.78	1.42
Chicago, Rock Island & Pacific	54	33	58	81	42	54	1.80	1.11	1.93	2.71	1.40	1.79
Colorado & Southern	29	75	53	46	73	55	0.96	2.46	1.75	1.52	2.39	1.82
Denver & Rio Grande Western	59	55	85	75	119	79	1.90	1.78	2.75	2.43	3.84	2.54
Duluth, Missabe & Iron Range	43	55	88	20	48	51	1.45	1.84	2.96	0.68	1.60	1.71
Fort Worth & Denver	76	71	85	92	79	81	2.55	2.38	2.83	3.05	2.62	2.69
Kansas City Southern (incl. L&A)	188	180	189	178	121	171	5.87	5.64	5.92	5.55	3.79	5.35
Missouri-Kansas-Texas	48	54	60	75	110	69	1.50	1.69	1.87	2.36	3.43	2.17
Missouri Pacific	77	78	94	110	85	89	2.51	2.52	3.06	3.58	2.77	2.89
St. Louis-San Francisco	53	102	106	116	111	98	1.69	3.26	3.37	3.68	3.54	3.11
St. Louis Southwestern	147	78	85	122	117	110	4.80	2.55	2.77	3.99	3.82	3.59
Soo Line	42	41	52	61	62	52	1.38	1.35	1.74	2.01	2.05	1.71
Southern Pacific	50	50	67	80	77	65	1.70	1.69	2.28	2.70	2.61	2.20
Union Pacific	50	43	63	61	49	53	1.78	1.51	2.22	2.17	1.72	1.88
Western Pacific	51	54	53	66	49	55	1.70	1.82	1.78	2.20	1.64	1.83
Total Western District	61	59	72	81	75	70	2.00	1.94	2.39	2.68	2.47	2.30
Total United States	65	65	86	91	88	79	2.13	2.16	2.89	2.99	2.89	2.61

a Includes all Southern System Class I railroads

TABLE C

TYPICAL CROSS TIE PRICES
As of January 1

9 Selected Class I Railroads

District/description of cross tie	1973	1974	1975	1976	1977	1978	1979
EAST:							
7"x9"x8'6" oak treated	$6.67	$9.02	$12.94	$12.25	$13.18	$13.56	$15.15
Grades (4&5 Latest A.R.E.A. Spec.) 60/40 Cr/Coal	7.68	8.23	10.72	10.54	11.28	13.77	15.93
SOUTH:							
6"x7",7"x8" and 7"x9" by 8'6" treated oak & mixed hardwood	5.67	6.08	9.09	9.10	9.83	10.95	12.54
7"x9"x8'6" treated	5.71	6.48	10.77	10.99	10.41	11.10	12.36
7"x9"x8'6" oak, creosoted	6.89	8.63	11.20	10.65	11.50	11.98	12.58
WEST:							
7"x9"x8'6" red oak, Gr. 5	5.59	6.53	8.95	10.75	9.30	11.90	12.40
7"x9"x9'	6.40	8.95	13.55	12.88	12.00	13.32	14.18
7"9"x8' Doug. Fir rough - No. 1 & better	6.06	7.50	10.90	9.80	10.58	11.85	14.00
7"x8"x9' Hardwood treated	5.05	8.10	10.37	10.55	9.56	10.87	12.81

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July 26, 1979

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Contents

Cover Photo—South bound Amtrak train on Amtrak track south of Bowie, Maryland, November 1979.

MANUAL RECOMMENDATIONS

MANUAL RECOMMENDATIONS
Committee 15—Steel Structures
Report on Assignment B
Revision of Manual

D. L. Nord (Chairman, subcommittee), E . S. Birkenwald, J. G. Clark, L. F. Currier, R. E. Davis, G. F. Fox, V. K. Garg, D. V. Messman, G. E. Morris, Jr., W. W. Sanders, Jr., F. D. Sears, J. E. Stallmeyer, C. R. Wahlen, R. H. Wengenroth.

Your committee submits for adoption the following revisions to the SPECIFICATIONS FOR STEEL RAILWAY BRIDGES, Chapter 15 of the Manual:

Revise Art. 3.1.1 to read as follows:

Article 3.1.1—Quality of Workmanship

a. Structural steel fabricators shall be certified for the type of structure being fabricated under the AISC Quality Certification Program (Category I—simple rolled beam bridges or Category III—major steel bridges including rolled beam bridges) or another suitable program as determined by the Engineer. Evidence of certification shall be submitted to the Engineer for his approval before beginning any work.

b. The workmanship and finish shall be equal to the best general practice in modern bridge shops.

Revise Art. 7.3.4.2 to read as follows:

7.3.4.2 Fatigue (See also Commentary Article 9.7.3.4.2)

a. Welded or rolled members and welded and high strength bolted connections subject to repeated fluctuations of stress shall be investigated as to their capability to meet the fatigue requirements of Arts. 1.3.13 or 2.3.1.

b. Members with riveted and other mechanically fastened connections with low slip resistance that are subject to repeated fluctuations of stress shall be investigated as to their capability to meet the requirements of Category D of Arts. 1.3.13 and 2.3.1. If the engineer can verify that the rivets or fasteners are tight and have developed a normal level of clamping force, fatigue Category C may be used to determine the fatigue resistance.

c. Riveted and other mechanically fastened connections and members that do not satisfy the requirements of Art. 7.3.4.2(b), may have these requirements waived at the discretion of the engineer if the connections or members will retain their structural adequacy if one of the elements cracks. The connection, member or span must have adequate capacity to carry redistributed load and a frequency of inspection which will permit timely discovery of the local failure and corrective action.

d. Eyebars and pinplates subject to repeated fluctuations of stress shall be investigated as to their capability to meet the requirements of Category E of Arts. 1.3.13 and 2.3.1 for the nominal stresses acting on the net section of the eyebar head or pinplate.

e. When the actual stress cycles can be estimated from traffic which has used the structure, an effective stress range can be determined for the total variable stress cycles, N_v, as $S_{re} = \alpha(\Sigma\gamma_i S_{ri}^3)^{1/3}$.The coordinates and for the applicable design detail must be less than fatigue strength curves shown in Fig. 9.1.3.13A. The appropriate value of shall be taken from Table 9.1.3.13A unless an appropriate analysis provides a more accurate estimate. γ_i and S_{ri} are the ratio of the number of occurrences of S_{ri} to the number of occurences of cyclic stress, N_v and corresponding stress range of the estimated traffic, α is defined in Art. 9.1.3.13.

On page 15-3-8, revise Article 3.2.5(c) by adding the words "For riveted construction" at the beginning of the last sentence contained therein.

On page 15-1-26, in last sentence of Article 1.7.2.2(a), change "Art. 1.10.2" to read "Art. 1.10.1".

On page 15-1-35, Appendix, change maximum moment for a 9 foot span from 90.00 to 93.89.

On page 15-3-4.2, revise Article 3.1.10(a) by deleting the words "if permitted by Art. 1.10.5 or Art. 2.8.2".

On pages 15-1-3 and 15-2-2, revise the footnote at the bottom of these pages to read: *Refer to Arts. 9.1.2.1 and 9.2.2.1

On page 15-1-18, replace designation Art. 3.25 with Art. 3.2.5; where it appears in the footnote at the bottom of this page.

On page 15-3-6, replace designation Art. 3.2.3(a) with Art. 3.2.3(e); where it appears in the second line of Art. 3.2.3(b).

On page 15-9-2, replace designation Arts. 15.1.2.1 and 15.2.2.1 with Arts. 1.2.1 and 2.2.1 where it appears in the fourth paragraph.

On page 15-9-2, replace the last sentence with: (See Arts. 9.1.3.13 and 9.2.3.1).

On page 15-5-9, replace the designation Art. 1.2.4 with Art. 1.6.4.3; where it appears in the fourth line of Art. 5.3.2(a).

On page 15-5-10, replace the designation Art. 1.2.3 with Art. 1.2.4 where it appears in the first line of Art. 5.3.4(a).

Revise Article 6.6.7 to read as follows:

6.6.7 Wire—Physical Properties

a. The wire from which wire ropes are made shall be tested in the presence of an inspector designated by the engineer. Excepting that the filler wires may be made to the manufacturer's standards, the physical properties of the bright (uncoated) individual wires before fabricating into the rope shall be as follows:

b. The tensile strength shall be within the following limits:

Diameter of Wire	Tensile Strength	
Inch	Minimum, Psi	Maximum, Psi
0.038-0.060	244,000	286,000
0.061-0.100	237,000	280,000
0.101-0.140	231,000	273,000
0.141-0.190	223,000	266,000

c. The test specimens of the wire shall be subjected to a torsion test in which the distance between the jaws of the testing machine is 8 inches. The number of complete successive turns of 360 degrees in one direction through which an 8-inch-length wire can be twisted around its longitudinal axis without breaking or showing any signs of splitting or other defects shall be not less than the following:

Diameter of Wire Inch	Number of Turns
0.038-0.060	2.3 divided by diameter of wire in inches
0.061-0.100	2.2 divided by diameter of wire in inches
0.101-0.140	2.1 divided by diameter of wire in inches
0.141-0.190	2.0 divided by diameter of wire in inches

d. In this torsion test, one end of the wire is to be rotated with respect to the other end of the wire at continuous uniform speed until breakage occurs. During the test the applied tension shall be sufficient to straighten the wire. The speed of rotation shall not exceed 60 twists per minute. Such tests shall be carried out by a mechanically driven device, such as a motor or belt drive, in order to secure operation at constant uniform speed.

e. All of the tests specified above shall be made upon fair samples which may be taken from either end of any coil of wire, and such samples shall be taken from not less than 10 percent of the total number of coils.

f. The tolerance limits on diameters of like-positioned wires in the strands of the wire rope shall not exceed the following value:

Diameter of Wires Inch	Total Variation Inch
0.038-0.060	0.002
0.061-0.100	0.0025
0.101-0.140	0.003
0.141-0.190	0.0035

Report on Assignment 7
Bibliography and Technical Explanation

J. G. Clark (Chairman, subcommittee), D. S. Bechly, E. Bond, H. B. Cundiff, J. L. Durkee, E. R. Estes, J. M. Hayes, G. K. Gillan, D. V. Messman, W. W. Sanders, Jr., R. H. Wengenroth.

Add a new Section 9.7 as follows: 9.7.3.4.2 Fatigue

The intent of evaluating a structure for fatigue in Art. 7.3.4.2 is to minimize the probability of failure as a result of crack growth. This primarily affects the maximum service life that the structure is designed for. If a reduced life is acceptable, higher loads are permissible providing the serviceability is not impaired throughout the shortened useful life.

Welded structures do not have the inherent redundancy of older riveted construction. Hence the consequences of fatigue crack growth are more serious for most welded connections and members than for the riveted structure with built-up sections. The internal component redundancy of such members has generally permitted part of the member to fail and redistribute its load elsewhere.

Experience with several welded highway bridges that have experienced fatigue cracking has demonstrated that the members usually fail before the crack is discovered. (40.41). As a result it appears prudent to use the requirements of Arts. 1.3.13 and 2.3.1 when rating all welded bridge members. High strength bolted joints provide very high fatigue resistance and should not be critical.

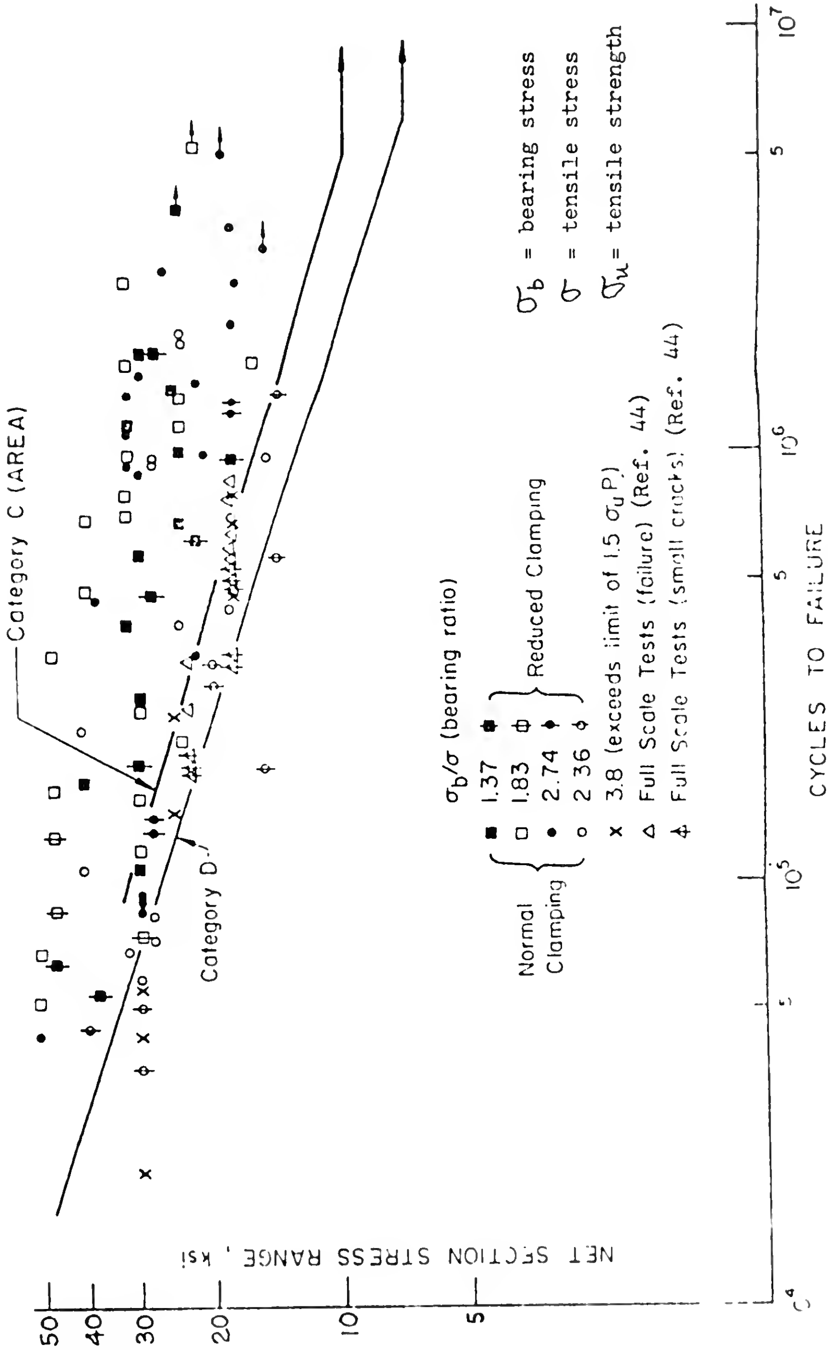
Members with riveted and other mechanically fastened connections with low slip resistance have had their fatigue resistance defined by Category D as a result of study and review of available test data. (42). This data is plotted in Fig. 9.7.3.4.2 and shows that the lower bound fatigue resistance has been defined by connections with reduced levels of clamping force. Also plotted in Fig. 9.7.3.4.2 is the fatigue design line corresponding to Category C. It is readily apparent from this comparison that nearly all test data on riveted joints with normal levels of clamping force fall to the right of the Category C fatigue line. Hence it is reasonable to permit a more liberal fatigue stress range if it can be demonstrated that the connection or member in question has tight riveted joints.

This discretion has been left to the engineer dependent on his verifying the tightness of the rivets or bolts and the adequacy of the clamping force.

For riveted construction where the members are fabricated from multiple elements, the immediate consequences of fatigue cracking may not be as serious as in welded structures. Such construction often has built-up members and connections, so that if one element fails, there is normally sufficient capacity and redundancy to permit the load to be redistributed long enough to be detected by routine inspection and this permits corrective action before more serious damage develops.

The intent of Art. 7.3.4.2(c) is to permit a waiver of the fatigue provisions when the engineer can show that the structure has an adequate level of redundancy, so that should cracking develop it can be accommodated.

For eyebars and pinplates the critical section is at the pine hole normal to the applied load. Several studies have indicated that the stress concentration at that location is in excess



FATIGUE STRENGTH OF RIVETED CONNECTIONS

Fig. 9.7.3.4.2

of 4. (42,43). It has been suggested that Category E provides a conservative estimate of fatigue resistance at such connections. Particular attention should be given to any forge seams or other unusual flaw-like conditions that may exist at the bore of the eyebar normal to the applied stress.

When the actual stress cycles can be estimated from traffic known to have used the structure, the total variable stress cycles can be estimated and the effective stress range calculated as $S_{re} = \alpha(\Sigma\gamma_i S_{ri}^3)^{1/3}$. (42,45). The resulting coordinates can be compared with Fig. 9.1.3.13A for the applicable design detail. The values of for various spans and member classification are tabulated in Table 9.1.2.13A. The factor γ_i is the ratio of the number of occurrences of S_{ri} to the total number of occurrences of cyclic stress, N_v.

Add to bibliography on page 15-9-24 as follows:

40. Fisher, J. W., Pense, A. W. and Roberts, R. EVALUATION OF FRACTURE OF LAFAYETTE STREET BRIDGE, Journal of the Structural Division, ASCE, Vol. 130, No. ST7, July 1977.

41. Fisher, J. W., Sullivan, M.D. and Pense, A. W. Appendix F "Analysis of Crack Growth in the Quinnipiac River Bridge", Final Report NCHRP Project 12-15, Improving Fatigue Strength and Repairing Fatigue Damage, 1975.

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45. Wyly, L. T. and Scott, M. B. AN INVESTIGATION OF FATIGUE FAILURES IN STRUCTURAL MEMBERS OF ORE BRIDGES UNDER SERVICE LOADINGS. AREA Bulletin, 524, Proceedings Vol. 57, P. 175, October 1955.

Subject	Article Reference
Fabrication tolerances	3.1.7.1(d)(e); 3.1.7.2.
Fatigue	1.3.13; 2.3.1.
Field welding	4.20; 9.1.5.10.
Fillet welds	1.7.4(b); 1.10.3; 3.5.5(b)(c); 5.1.3.2(a); 5.1.3.3(j); 5.2.9.2; 7.2.2.3(a)(b); 7.3.4.3(b); 9.1.10.4.
Flange to web welds	1.7.4(b); 3.3.3; 3.5.5(c); 9.1.7.4; 9.3.3.3.
Full penetration groove welds	1.5.9(b); 1.7.5(c); 1.7.6(c); 3.1.10(a); 3.5.5(b); 6.5.36.10(c); 9.1.7.4.
Groove welds	1.10.1; 3.5.5(b); 7.3.4.3(b).
Groove weld backings, extension bars and run-off plates	3.3.5; 9.3.3.5.
Inspection	3.5.5; 7.4.7(a)(b); 9.3.5.5.
Intermittent fillet welds	1.10.2(c); 7.2.2.3(b).

Subject	Article Reference
Lacing bars, fillet-welded	1.6.4.2(g).
Lateral bracing	1.11.2(c).
Longitudinal force-welded rail	1.3.12(b).
Machine welding	9.3.3.3.
Plate girder	1.2.3; 1.7.1; 1.7.2.2; 1.7.4(b). 1.7.5(c)(d); 1.7.6(c); 1.7.8(a)(b); 2.7.1; 2.7.3(a); 9.1.7.1; 9.2.7.1; 9.1.7.4.
Plug welds	1.10.2(b).
Sealing welds	1.5.5; 1.5.13(c).
Seam welding	6.7.9.35(c)
Sheaves	6.5.36.10.
Shielded metal arc process	5.1.3.3(j)(k).
Shop-painting joints	3.4.1(b).
Slot welds	1.10.2(b).
Splices	1.5.9(b); 1.7.5(c)(d); 5.3.11; 6.6.6.
Spot welding	6.7.9.35(c).
Stay plates	1.6.3(d).
Stiffener plate	1.7.7(a); 1.7.8(b); 1.10.4; 2.7.3(a)(b); 3.1.10.
Structural welding code	1.2.1; 1.10.2(a); 3.3.1(a); 3.3.6(a); 3.5.5(b); 9.1.4; 9.2.4; 9.3.1.6.
Stud: end welding	5.1.3.3(g).
Stud: shear device	5.1.3.2(a); 5.1.3.3(a); 9.5.1.
Stud: welding equipment	5.1.3.3(g); 5.3.3.3(n).
Stud: welding procedure	5.1.3.3(i)(n).
Stud: weld zone	5.1.3.3(i).
Tack welding	3.3.4; 3.3.6; 9.3.3.4.
Welded construction	1.7.2.2; 1.10; 3.3; 5.2.9.2; 6.2.11(a)9.
Welder and welding operator qualifications	3.3.6.
Welding: connection angle flexing (O.S.L.) leg	1.8.3(a).
Welding: electrodes	1.2.1; 2.2.1; 5.1.3.3(j)(k).
Welding: machinery weldments	6.2.11(a)9; 6.5.36.10.

MANUAL RECOMMENDATIONS
Committee 27—Maintenance of Way Work Equipment
2.4—HYDRAULIC AND ELECTRICAL SYSTEMS

J. P. Zollman (Chairman Subcommittee),
D. C. Johnson, M. E. Kerns, Dave Schulz

Your committee submits the following report as *information* for *guidance* of equipment manufacturers and railway maintenance personnel in the design, construction and evaluation of hydraulic systems.

2.4.1 Hydraulic systems shall conform to the recommendations of the National Fluid Power Association (NFPA), American National Standards Institute (ANSI) and International Standards Organization (ISO) except where a conflict occurs, then the following will apply:

2.4.1 1. Upon completion of manufacture and before any operation shall begin, all parts of the hydraulic system shall be clean and free from scale, rust, dirt and any other contaminant. Threads, flares, holes, cuts and machining must be deburred and cleaned.

2.4.1 2. Hydraulic reservoirs of ten (10) gallon capacity or larger shall be designed with the following considerations:

a. Place the baffle(s) in the reservoir so as to separate the pump inlet part from the settling part of the reservoir. The baffle(s) should direct the flow toward the reservoir walls for maximum cooling capacity and maximum lay-over time.

b. Provide sufficiently large access panels for complete periodic cleaning, maintenance and inspection.

c. Provide an air inlet large enough to maintain conditions of Item 2.4.2.12. The air inlet shall be equipped with a 25 micron or less filter. A cartridge type is preferred.

d. Provide a filler with at least a 100 mesh screen protected from external damage with a minimum capacity of five gallons per minute with five thousand (5000) SSU fluid viscosity and with a filler cap that can be locked with a large railroad padlock.

e. Provide thermometer to indicate reservoir operating temperature protected from damage.

f. Provide a static fluid level gage to show full-point and addpoint protected from damage.

g. When immersion heaters are provided to control fluid viscosity during cold weather start up. Place the heater(s) so removal is possible without draining reservoir.

h. A non-integral reservoir is preferred.

2.4.1 3. Fluid temperature shall not exceed 180 degrees F. maximum in the reservoir outlets while operating in a 110 degree F. ambient. The minimum fluid temperature after 45 minutes operation shall be 85 degrees F. with ambient temperature of 20 degrees F.

2.4.1 4. A full flow testing tee(s) shall be provided adjacent to the pressure side of hydraulic pump(s). A return line full flow tee shall be placed ahead of return line filter.

2.4.1 5. Where failure of power plant or pump can immobolize components in a position which could prevent moving the machine, an emergency hand pump shall be provided in the circuit. Large machines shall be equipped with battery operated emergency pump where more than five minutes are required to move all components within the clearance diagram of the track occupied by means of a hand pump.

2.4.1 6. The total return and/or pressure line flow shall pass through filters rated at 25 microns or finer equipped with a condition indicator. Filtration shall not be less than recommended by manufacturers of components.

a. In closed loop system, filtration as recommended by pump manufacturer will apply.

b. Magnetic particle attraction shall be provided in the filters or reservoir.

c. Filtration of the return flow from the pilot section of pilot-operated valves is not required.

2.4.1 7. All hydraulic hose assemblies must have reusable screw-together hose fittings, if available, at the required pressure specifications.

a. "Rubber" cover hydraulic hose should NOT be type-T (thin cover) as specified in the SAE specifications, because the heavier cover protection is required to resist abrasion on railroad maintenance equipment.

b. Hydraulic hose must meet (or exceed) SAE specifications SAE J-517c 1978 standards and all future changes to be made to these standards.

c. Hydraulic pump supply hoses shall meet the requirements SAE 100R4.

d. Field attachable reusable hose fittings must be capable of installation without the use of special machinery.

e. Hoses shall not be flexed to less than the specified minimum bend radius.

f. Hoses shall not be exposed to twisting, pulling, kinking, crushing, or abrasion.

g. Hoses shall not be exposed to operating and/or ambient temperatures above or below the manufacturer's specified temperature range.

h. All hydraulic hoses shall be replaced at the first sign of environmental damage or rubber degradation.

i. The SAE 37° flare shall be standard for all flared tubes.

j. For tube wall thicknesses that are too heavy for flaring, as per SAE J-1065, silver brazing or butt welding the connector is recommended.

k. The SAE J-518c 4-bolt split flange connection is recommended for all connections over one-inch (1").

l. Ports in hydraulic components should be either the SAE straight thread O-ring boss or the SAE 4-bolt split flange type.

m. Pipe threads are not recommended, but where they are used, they must be NPTF (dry seal type).

2.4.1 8. Tubing and piping shall be mounted to minimize vibration and tubing shall have only gentle bends to change direction or compensate for thermal expansion. Tube bend radii shall not be less than three times inside diameter.

2.4.1 9. Wherever practicable, valves shall be manifold mounted.

2.4.1 10. Complete circuit diagram shall be provided. Only NFPA, ANSI and ISO symbols shall be used in graphical diagrams. Pictorial and cutaway diagrams are also permissible where they add to the case of understanding the circuit. Diagrams shall be large enough to be easily followed for trouble shooting.

2.4.1 11. Galvanized pipe and fittings shall not be used.

2.4.1 12. The vacuum at the pump inlet(s) shall not be more than 60% of pump manufacturer's recommendations or four inches mercury whichever is less at 500 feet altitude fluid at 100°F or standard conditions. Test opening shall be provided.

REPORT ON ASSIGNMENT 2
Machine Design—Hydraulic & Electrical Systems

J. P. Zollman (Chairman, Subcommittee),
D. C. Johnson, M. E. Kerns, Dave Schulz

Your committee submits the following report as *information* for *guidance* of equipment manufacturers and railway maintenance personnel in the design, construction and evaluation of electrical systems.

Electrical systems shall conform to American National Standards Institute (ANSI) and International Standards Organization (ISO) except where a conflict occurs, then the following will apply:

2.5.1 1. Upon completion of manufacture and before any operation shall begin, all parts of the electrical system shall be clean and free from scale, rust, dirt and any other contaminant. All material and workmanship must be of satisfactory quality for the intended use.

a. A sequence of operation, along with a correct electrical physical and schematic drawing large enough to be easily followed for trouble shooting shall be provided. Subsequent changes shall be described in new drawings provided to all customers.

b. Whenever practical, various components shall be interchangeable.

c. Cable shall be routed to prevent exposure to physical damage. Thin wall conduit should not be used except in a protected area.

d. All machines must have negative ground.

e. Battery charging generators and alternators must have rated capacity to handle all operating equipment and accessories with a 50% reserve.

f. If the battery system is 24V, using two standard 12V batteries, the manufacturer shall not connect any 12V load to one battery.

g. Battery box shall contain a battery disconnect switch and shall have a cover that can be locked with a large railroad padlock.

h. Standard, readily available components shall be used in construction of electrical system.

2.5.1.2. 2. ELECTRICAL APPARATUS CABINETS

a. Cabinets for electrical apparatus shall be of steel interior. Clearances between walls of cabinet and bare live parts of all apparatus or apparatus or switchboard shall be not less than 1-1/2 inches, where a potential in excess of 50 volts exists.

b. All cabinets must be arranged so that panels can be readily removed and all parts requiring adjustment are easily accessible.

c. Adequate lighting shall be provided in electrical cabinets.

d. Pipe lines, other than electrical conduit, shall not enter electrical cabinets.

e. Metal surfaced interior walls of electrical cabinets must be painted with approved electrical insulating paint, or coated with other approved insulating material. The floors of lockers must be covered with an approved electrical insulating material.

f. Nominal voltages used must be plainly indicated outside the electrical apparatus locker.

g. All components and/or integral units, such as circuit boards, must be plainly identified showing capacity, use and exact reference to electrical drawings and parts list. Complete parts information shall be shown, where practical, to minimize errors and time referring to drawings and lists.

h. Wires shall be equipped with good quality terminals and identified with markers. Terminal posts must be plainly marked. All wires in cabinets shall be neatly dressed and clamped.

i. Electrical and electronic equipment cabinets must be weatherproof and equipped to be locked by a large railroad padlock.

j. Electrical apparatus lockers, boxes, and housings which contain heat producing elements, must be properly ventilated to maintain a temperature of less than 140° F.

2.5.1.3. MOTOR CONTROL APPARATUS OVERLOAD RELAYS

a. All fractional horsepower motors shall have fuse protection.

b. All motors 1/4 horsepower to one horsepower shall have automatic reset thermal protection within the motor itself rather than in the starter and fuse protection in the line.

c. Motors in excess of one horsepower shall have starter relays with built in thermal protection.

d. Overload relays shall be in each line of a three phase starter in ungrounded systems.

MANUAL RECOMMENDATIONS

Committee 28—Clearances
Report on Assignment B
Revision of Manual

R. R. Snyder, (Chairman, Subcommittee)

Delete pages 28-3-8 and 28-3-9 and revise 28-3-7 as follows:

Clearances 28-3-7

3.2 SUGGESTED METHOD OF PRESENTING PUBLISHED CLEARANCES

The following data is desirable in presenting published clearances in the publication "RAILWAY LINE CLEARANCES".

1. Date of last change.
2. A map showing the lines and the interchange points, junctions or other major points involved in the routes and weight limits and the column numbers.
3. Identification of the individual or department to notify for advance authority, dimensions exceeding published clearances and other specific questions.
4. A statement providing description of car on which carrier's clearances and weight limitations are based.
5. A notice section to provide both specific and general information concerning clearance and other important information not shown elsewhere.
6. A list of routes to show maximum gross weight between specific cities or areas and including cross reference to clearance columns.
7. Notes. To include additional data, exceptions, further information related to weight or clearances, allowable combined center of gravity, etc.

PUBLISHED AS INFORMATION

COMMITTEE 14—Yards and Terminals
Report on Assignment 7
Yard System Design for Two-Stage Switching

H. B. Christianson (Chairman, Subcommittee), M. J. Anderson, G. H. Chabot, M. R. Gruber, Jr., H. L. Haanes, J. N. Hagan, W. P. Robbins, W. A. Schoelwer, P. E. Van Cleve, P. C. White.

Your committee submits the following report as information with the recommendation that the subject be discontinued.

The purpose of a classification yard is to create destination order out of disordered arriving cars. It is a car sorting system. Ideally, each car has a known destination, a distant or sometimes a local storage, repair, industry or interchange track.

We try not to switch unil trains at all—coal, ore, grain, auto and trailer trains. We encourage multiple car shipments, mostly because they use less switching. Nevertheless, most North American cars arrive at unloading sites in one- to six-car blocks. To process these cars the United States uses over 4,000 yards, about 125 of which are hump yards. Hump yard design technology has advanced rapidly, but, with few exceptions, sorting techniques are unchanged. This report is intended to encourage transportation and engineering personnel to design and install yard subsystems using two-stage techniques for specific needs.

The demand for sorting trains near origins and destinations increases because we have:

— more long, through and interline trains.

— fewer intermediate yards.

— greater range of heavy/light and long/short cars which affect train dynamics.

— more loads, empties and trailer/containers of hazardous materials (hazmat), with in-train positions mandated.

— tighter schedules for placing cars with customers some of whose docks are open less than an average of 5 shifts in 21.

SORTING, TODAY

The usual procedure is to assign a track for each car destination, called a block. If a track overflows, the yardmaster assigns another track to that block. If he has no more cars for a block and the track is not full, he sometimes begins another block in that track. If blocks exceed the number of tracks, he will assign a track for several small blocks, then reswitch these cars as tracks or part-tracks become empty. Most yards do this part of the time. Most switching is done from one end of a yard. Switching the overflow reswitch cars from the other or trim end sometimes enables the yardmaster to pack more blocks simultaneously into a set of tracks. In some situations where switching is done independently at both ends of a yard, some tracks may hold two blocks. Ultimately the tracks are emptied in prescribed sequence, assembling a blocked train for dispatch.

Traffic patterns and habits in larger yards are such that usually an empty track is assigned the same block continuously. But the yardmaster may "swing" a track to another block. In this way some yards produce perhaps one and one-half blocks per track during three shifts. Short local trains may makeup on borrowed tracks or part-tracks. But often large yards produce fewer daily blocks than the number of class tracks. This one-block-per-track procedure is accepted in typical automatic hump yards and variations such as herringbone, train-maker and tandem hump yards. Bowl occupancy of an active hump yard seldom exceeds 20 percent capacity.

Sorting is less than half the effort in a one-block-per-track yard. We use more effort to make solid, withdraw and assemble blocks into trains. Two trim engines per hump engine is a typical ratio in modern hump yards.

TWO-STAGE SWITCHING

Various methods of multi-stage sorting were described by K. J. Pentinga of the Netherlands Railways. [1] The simplest of these methods—two-stage switching—was described by H. B. Christianson of the Chessie System.[2] For specific workloads the two-stage sorting technique uses fewer human and physical resources and out-produces conventional one-stage sorting.

Briefly, two-stage switching is a technique for sorting each car twice so as to yield more blocks than tracks used. Unlike conventional one-stage switching, each track in each stage holds more than one block. When two-stage switching is complete, train assembly is partly complete.

The following illustrates how
(1) to make four blocks on a main and a stub track:

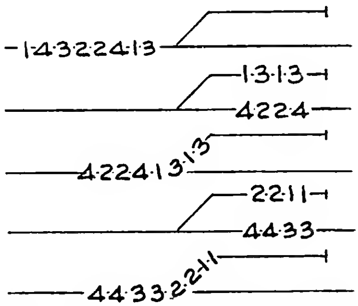
(a) get disordered cars

(b) sort to 2 tracks

(c) then double back

(d) and sort again

(e) then assemble train



In this example each block has two cars, but any number of cars will do until the stub track overflows.
(2) to make nine blocks on three tracks:

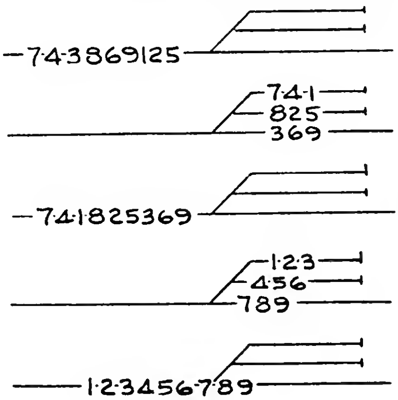
(a) get cars

(b) sort 1st stage

(c) withdraw

(d) sort 2nd stage

(e) and assemble



[1]Railway Gazette, May 22, 1959
[2]RSMA Railway Management Review, Vol. 72, No. 2 (1972)

To reverse the assembled train, change the sequence of withdrawal after first stage.

To find the maximum number of blocks which two-stage switching can produce, multiply first stage tracks by second stage tracks:
$2 \times 2 = 4, \quad 3 \times 3 = 9, \quad 4 \times 3 = 12, \quad 4 \times 4 = 16.$

Aside from yard investment, which always favors two-stage plans, operating costs favor two-staging as the numbers of blocks increase. The breakeven point probably is somewhat above nine blocks. To sort nine blocks two-stage on three tracks a yard engine makes seven moves. To sort them one-stage on nine tracks it makes nine moves.

Moves	3 tracks two-stage	9 tracks one-stage
sort	2	1
pull	1	
double	4	8
	7	9

If typical times are 30 min to sort, 5 min to pull, and 10 min per track plus 10 min fixed time to double, the conventional one-stage system is a little faster. Improve the hump rate and the two-stage system is faster.

TWO-STAGE PLANNING

To plan two-stage switching you must select the blocking pattern for the outbound train or trains. More than one train can be produced. Consider inbound car availability; generally, the second stage sorting cannot begin until all cars are on hand. Consider also the block sizes, which vary, and track lengths. Two planning aids for sorting are illustrated. The RSMA aid shown in Fig. 1 is experimental.

A number-letter tag method is used by SP. In this example the four 4-block trains in Fig. 2 are produced simultaneously on four tracks. They switch first stage all the l's into track A, 2's into track B, etc., as in Fig. 3. The second stage sort produces the blocked trains as in Fig. 4.

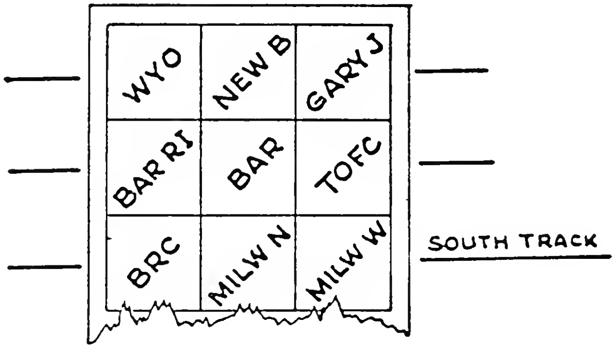
A computer plans two-stage switching for SP at City of Industry Yard, CA, and for AT&SF at Barstow Yard, CA. Figs. 5, 6 and 7 illustrate portions of switch lists for a Barstow train.

Productivity and output quality rest partly on the skill of the yardmaster-planner or how the computer programmer interprets system blocking policy. For example, a block may be any of these types of cars for a block destination:

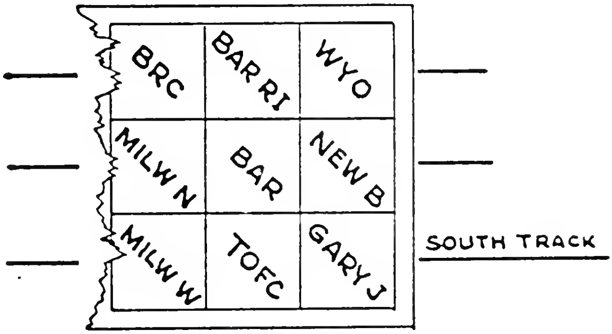
- random cars
- caboose
- excess dimension loads to move at head end
- long cars
- short cars
- medium length transition car
- overflow cars—if a yard track is too short for a long block, plan to fill the track and switch it first stage only; switch the overflow as a separate block.
- hazmat car or cars
- spacer cars for hazmat cars

FOR YARD FOREMEN ONLY:

1. decide how you want to block the train. If you want 9 blocks like this:
   WYO NEW B GARY J BAR RI BAR TOFC BRC MILW N MILW W
2. then write the blocks in a 3 x 3 table like this:



   which is how you want the cars to line up in the 2nd stage.
3. next, turn the table 90°



   and SORT the cars to tracks assigned in this 1st stage.
4. then DOUBLE the tracks keeping the west-east relationships correct
   WYO and others center track others and MILW W
5. now SORT cars to tracks assigned in the 2nd stage as in your original table.
6. finally, DOUBLE the tracks in correct sequence. Now your train is blocked as you specified in step 1.

—To reverse the blocks, change the doubling sequence.

—You can exchange tracks. If, for example, the south track is long and the TOFC block is long, mate them in both stages, and change the doubling sequence.

—Use the same procedure for 3 x 4, 4 x 4 or any 2-stage processing. Try it!

Fig. 1

TRAIN A		TRAIN B		TRAIN C		TRAIN D	
West Oakland	1A	(S.F. to) Redwood Jct.	1B	Watsonville Jct.	1C	Ozol	1D
Emeryville	2A	Sunnyvale	2B	Newark	2C	Pittsburg	2D
Richmond	3A	Niles	3B	Mulford	3C	Tracy	3D
Suisun	4A	Hayward	4B	Davis	4C	Lodi	4D

Fig. 2

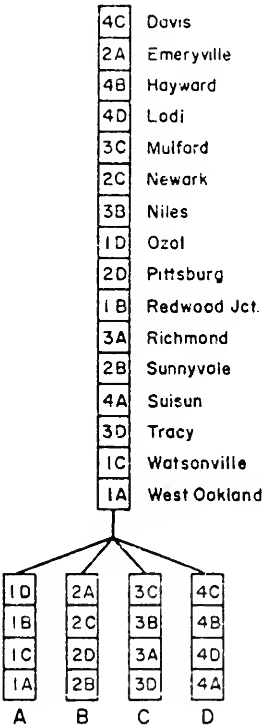


Fig. 3

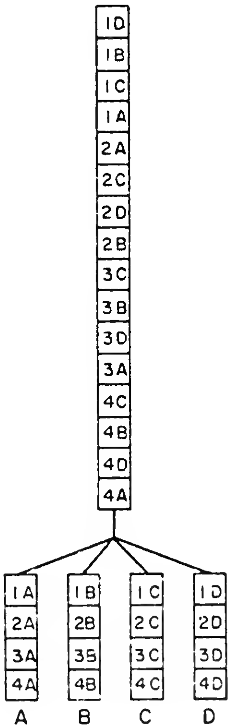


Fig. 4

MINI SWITCH LIST 406
PULL BB02 COME OUT WITH SUNX060005

PASS 1
LIST IS FROM WEST TO EAST

LINE NO	INITIAL	NUMBER	SET OUT ORDER BLOCK NUMBER	TRACK
001	SUNX	060005	2	46
002	RAIX	003332	2	46
003	RAIX	003343	2	45
004	PTLX	126021	2	45
[illegible]	ATSF	045997	4	46
042	[illegible]	052200	5	47
043	ACFX	[illegible]35	5	47
044	RAIX	0026[illegible]	2	45
045	NATX	029177	[illegible]	45

TRACK	CARS
45	20
46	3
47	14
48	8
TOTAL	45

1ST PULL BB45 COME OUT WITH RAIX003343 20 CARS
2ND PULL BB46 COME OUT WITH SUNX060005 3 CARS
3RD PULL BB47 COME OUT WITH GATX052200 14 CARS
4TH PULL BB48 COME OUT WITH ATSF014677 8 CARS

Fig. 5

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              PASS 2
     LIST IS FROM WEST TO EAST

LINE                                SET OUT ORDER
NO       INITIAL     NUMBER          BLOCK NUMBER          TRACK

ØØ1        ATSF      Ø14677               4                  46
ØØ2        SOU       527526               4                  46
ØØ3        SOU       525159               4                  46
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Fig. 6

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          FINAL  LIST  B1Ø2
     LIST IS FROM EAST TO WEST

LINE                          ONLINE
NO       INITIAL    NUMBER    DESTINATION

ØØ1        NATX     229177        8Ø11
ØØ2        RAIX     ØØ261Ø        8Ø11
ØØ3        ACFX     Ø17577        8Ø11
ØØ4        ATSF     [illegible]   9354
           RTMX     ØØ2135        9354
Ø45        GATX     Ø522ØØ        9354

TOTAL      45 CARS

END MINI-SWITCH LIST  4Ø6
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Fig. 7

— heavy loads placed to improve train dynamics
— empties for the same reason
— priority "shutdown" cars
— ice-breaker car

GROWTH

Switching two-stage is not new: train crews have used it for decades, but sparingly. SP pioneered large scale two-stage switching in the U.S. at its Roseville, CA, yard in 1974. Roseville now two-stages 500 cars daily, about 20 percent of its output. SP's City of Industry, CA, yard two-stages 400 cars daily, and several other SP yards regularly two-stage smaller amounts.

AT&SF in 1976 was first to construct a hump yard designed in part for two-stage switching. Barstow Yard sorts first stage from the primary hump into a four-track bowl group. Second staging is from the trim end over a mini-hump with tangent point retarders.

Chessie System's Queensgate, now under construction in Cincinnati, OH, will two-stage sort 23 percent of its designed capacity of 3,500 cars daily. The six-car-per-minute hump rate will enable both first and second stage sorting over the primary hump.

UP's enlarged yard at North Platte, NE, will sort first-stage over a master retarder and one of five group retarders into an outer ten-track group. A flank mini-hump will re-use the group retarder and sort second-stage into the same ten-track group.

You can two-stage switch at any yard, large or small, flat or hump, but efficiency is related to blocking policy, communications and yard design. A two-stage yard has four temporary storage functions, arrival, first stage, second stage and departure. Various combinations are feasible as illustrated in Figs. 8 through 11.

FOLDED TWO-STAGE YARD

The folded two-stage yard, Fig. 12, does not exist. A thesis in 1967 by L. C. Davis for the University of Pennsylvania suggests the plan. Later analysis shows that a seven-track folded two-stage yard should outperform a conventional 48-track hump yard. Comparative physical features: 1/4 the land area, 1/5 the widest land width, 1/3 the track length, 1/4 the turnouts, 1/5 the crest height, 1/3 the crest-to-clearance distance. If so, it presents an opportunity for more innovative yard system design. Potential devices to enhance folded two-stage yard systems are:

— car accelerator, not gravity or brakes, to separate cars
— automatic pin-puller or personnel carrier
— short, multiroute switch
— trimmer to prevent critical stalls
— propulsion system for second stage sorting

DISADVANTAGES

A two-stage system has some disadvantages. Each car is sorted twice, exposing it to more overspeed impacts, stalls and uncoupling/coupling defects, but in a specifically designed yard, these risks are less. It is difficult to add cars after second stage sorting has begun, but cutoff time is also critical for a one-stage system if train assembly has begun. Finally, a two-stage system may be more sensitive to late trains, faulty communications and poor planning, but the conventional system's ability to alleviate these flaws often only conceals them.

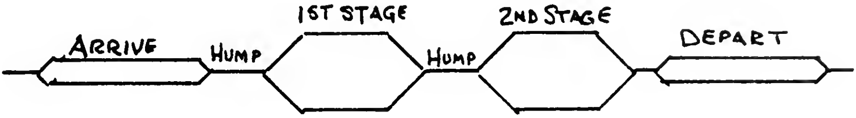

Fig. 8 – Tandum Hump

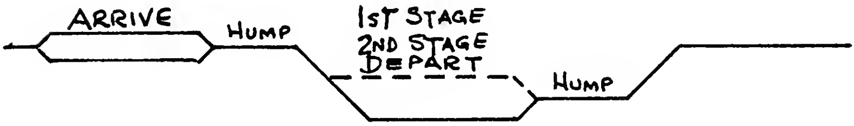

Fig. 9 – Barstow Yard

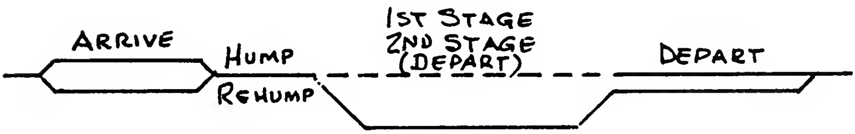

Fig. 10 – Roseville and Queensgate Yards

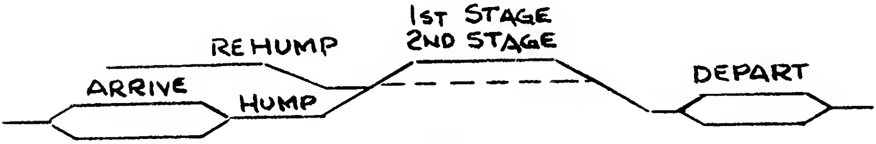

Fig. 11 - North Platte Yard

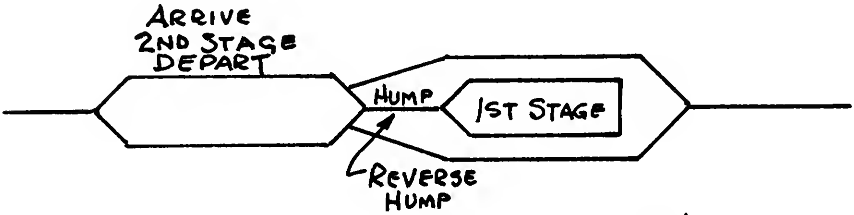

Fig. 12- Folded Two-Stage Yard

While a two-stage system can work effectively alongside a one-stage system, it is useless for unit trains and for single trains with few or large blocks. But examine these large blocks. Perhaps, for example, connecting lines with better information and policies would exchange multiblocks more efficiently.

One alleged disadvantage is that personnel cannot learn and effectively use two-stage switching, but a seven-day test at a large flat yard and a two-day experiment at a medium-sized hump yard refuted this. Two-staging will work in a normal environment with normal delays and problems.

SUMMARY

Where demand and policy produce small blocks, consider a two-stage yard system. The technique is proven feasible. Each car is sorted twice, and the second sort also partly assembles a blocked train. Maximum number of blocks equals first stage times second stage tracks used. In each stage, each track holds more than one block's cars. A block may be a type of car or lading, not just a destination. The two-stage system is a process, not a physical plan, and several layouts are feasible.

ON THE MEASUREMENT AND CALCULATION OF VERTICAL TRACK MODULUS +

Allan M. Zarembski*
John Choros**

ABSTRACT

This paper presents the results of a series of tests and analyses directed towards the characterization of the track structure under vertical loads. It also presents and evaluates different analytical techniques for the calculation of the vertical track modulus.

In a series of tests at the Association of American Railroads's Track Structures Dynamic Test Facility, the response of the track was obtained by monitoring track deflection under increasing vertical loads. This load and deflection data was then used to calculate vertical track modulus, track stiffness and track compliance. Three widely used techniques were utilized to calculate the vertical modulus.

The results of the tests indicate that the modulus of the track is related to the level of loading; thus identical track can give different modulus values for different load levels. Of the three different techniques used to calculate track modulus, the beam-on-elastic-foundation technique was found to be the most applicable to field measurements since it requires a minimum number of track deflection values.

INTRODUCTION

Since the early days of the railroad industry, when track constructed with longitudinal steel rails and transverse wooden crossties was introduced, track engineers have desired a reliable method to quantify the response of the track structure to given loads. The ability to specify the load-carrying capacity of track, to determine the resulting rail stresses and accompanying track deformation, is considered to be essential to proper track design and maintenance.

Winkler (1) first proposed the use of an elastic beam theory to analyze rail stresses. His method assumed the rail to behave like a beam that was continuously supported on a uniform elastic foundation. He proposed the calculation of a fundamental parameter, called the track modulus, which was related to both the applied load and the resulting track deflection, measured at one location relative to the loading point. As more modern track structures evolved, using decreased tie spacings and heavier wheel loads, Winkler's original theory was shown to be justified.

Other investigators, including Gough (2), Czitary (3) and Wasiutynski (4), independently analyzed a track structure by two different methods, assuming: (1) a beam on discrete supports and (2) a beam on a distributed elastic (Winkler) foundation. Both methods produce similar results, although the Winkler method involves simpler calculations, and has gradually become accepted by the railroad industry for use in track design. More recent investigators using the method include Timoshenko (5) and the ASCE-AREA Special Committee on

+ Research sponsored by the Federal Railroad Administration Under Contract DOT-FR-30038, and the Track Train Dynamics's Track Strength Charaterization Program

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Stresses in Railroad Track under the leadership of Prof. A. N. Talbot(6). In the First Progress report of this committee, Talbot, employing the theory first proposed by Winkler, developed the notation which has been in common use by American railway engineers ever since.

The cornerstone of the beam on elastic foundation theory was the identification of a parameter which attempted to quantify in single term (u) the combined effects of cross ties, ballast and subgrade. This parameter, was referred to by Talbot as the modulus of elasticity of rail support. It is important to note that the effect of the rail itself enters into the beam on elastic foundation theory directly, and is consequently not represented by the modulus term.

After the validity of the Winkler method had been established, track moduli calculations became very important. In the original Winkler model (1), the foundation was assumed to behave like a continuous linear spring, and the calculated modulus was a measure of the spring's stiffness. This method, however, failed to account for interactions among soil particles in the foundation. In an attempt to correct this deficiency, many early investigators either modified the Winkler model, or tried to develop new models that could more accurately describe an actual track foundation's behavior under various applied loads. Reference (7) describes some of these alternate foundation models attributed to Filonenko-Borodich, Hetenyi, Pasternak, Vlasov and Reissner.

Although many mathematical track foundation models have been developed, little was done to determine track moduli from experimental data. The first attempt to do so was undertaken by the ASCE-AREA Special Committee on Stresses in Railroad Track (6). This second phase of the Talbot committee's investigation consisted of field measurement conducted on Illinois Central Railroad trackage near Champaign, Illinois. From the test results, Talbot determined the track modulus* for various combinations of rail size, tie size and spacing, and ballast depth and consolidation. Talbot's method assumed that the modulus was proportional to the applied load divided by the area under the track section's deflection curve. Since deflections were measured over the entire length of the depressed section caused by the load, both soil particle interactions and load distribution by beam action of the rails were taken into account. A major advantage of this method is the averaging effect acting over the entire length of the depressed area, which compensates for any track discontinuities that may be present.

This method, however, has three distinct disadvantages, namely (1) a large number of deflection measurements are needed on both sides of the applied loading point in order to accurately determine the shape of the deflection curve (2) since the foundation experiences compression only, any slack in the track is not taken into account, and (3) as described above, the effects of differing rail size are not taken into account.

To correct for the slack in the track, this method was modified, such that the modulus became equal to the difference between a light and heavy load, divided by the net area between the load deflection curves. While this eliminates the effects of free play, twice the number of deflections must be measured.

A third method for determining track modulus from experimental data is to use a modified version of the beam-on-an-elastic-foundation theory. This method, which accounts for differences in rail size, uses Winkler's equation to calculate the track modulus. The advantages of this method are: (1) measurements are required at only one deflection point, and (2) by taking rail stiffness into account, there is an averaging effect over the entire length of the depressed track section. This method for the determination of track modulus from

*The committee described the parameter measured as the modulus of elasticity of rail support. However since the effect of rail stiffness was not considered, the terminology does not precisely correspond to the notation and terminology (u) previously defined by the committee.

measured data appears to be the easiest to use and has been used by a number of railroad investigators including Schoeneberg (8), Code, (9), and Way (10). However, it has not been directly compared with other techniques on the same or closely similiar tracks.

In order to compare these three methods under identical track configuration and loading conditions, tests were conducted at the Association of American Railroads's Track Laboratory in Chicago, Illinois. This paper presents the test objective, instrumentation procedures and results. Theoretical track modulus values were calculated, for track loadings ranging from zero to 50 Kips. Other related variables, such as track stiffness and compliance, track deflections and rail bending stress were also obtained. This paper also discusses the three different methods, and compares the results with each other and with previously-published data.

VERTICAL MODULUS TESTS

A series of vertical modulus tests were conducted at the Association of American Railroads's Track Laboratory in the fall of 1979. The test area (Figure 1) contained a 45 foot section of track constructed with 136 RE rail, hardwood cross-ties at 19-½ inch spacing, 12 inches of AREA #4 limestone ballast, and 6 inches of limestone sub-ballast, all resting on the parent foundation of poorly graded sand.

Vertical load was applied to the track structure through a specially designed loading bolster (Figure 2). A set of 50 Kip hydraulic jacks was used to apply the vertical load. Two jacks were used for the major portion of the test to represent single axle loading, and four jacks were used to simulate truck loading. The loading bolster was designed to approximate a conventional freight car track. Four 36 inch wheel segments were used to duplicate wheel—rail contact geometry.

The test series consisted of three loading sequences in which simulated axle loads were applied through the loading bolster and measurements taken of track deflection and rail bending strain. In the first sequence, the load was applied in increasing increments from 0 to 50 Kips and data was recorded after each load increment. The second sequence was the unloading sequence and the data was taken after each decreasing increment of loading. At no time was the load returned to zero during the increasing or decreasing sequence. In the third sequence, the load was applied directly and then released to zero for each of the defined load levels.

Track deflections were measured at three locations using linear variable displacement transducers (LVDT) and at twenty-one locations using a surveyor's level. The deflections measured with LVDTs were read after each loading increment whereas the deflections measured with the level were read at only a limited set of load levels. All deflections measured were absolute, i.e. relative to a fixed zero point constant for all tests. To achieve this the LVDTs were mounted on a reference frame supported at the concrete walls of the test pit. (Figure 2) A triangular aluminum truss section was used for the reference frame. A cantilever beam extending from reference frame to rail provided the transducers support at each measurement station. The level readings were taken with the level outside the test pit and using a one-hundredth of an inch graduated scale held at the measurement point.

In addition to displacements, strains were monitored in the rails at five points. At two locations, strain gauge arrays were used to measure the applied load on each rail. This was done to provide a check on the load applied to the track. The other three arrays measured bending stresses in the rail at the load point, at 28.5 inches and 66.5 inches away from the load.

For all the loading sequences, data were recorded on both magnetic and paper tape, and reduced according to the techniques defined in Appendix A of Reference 11.

Figure 1. Test Track Used for Determining Vertical Track Modulus

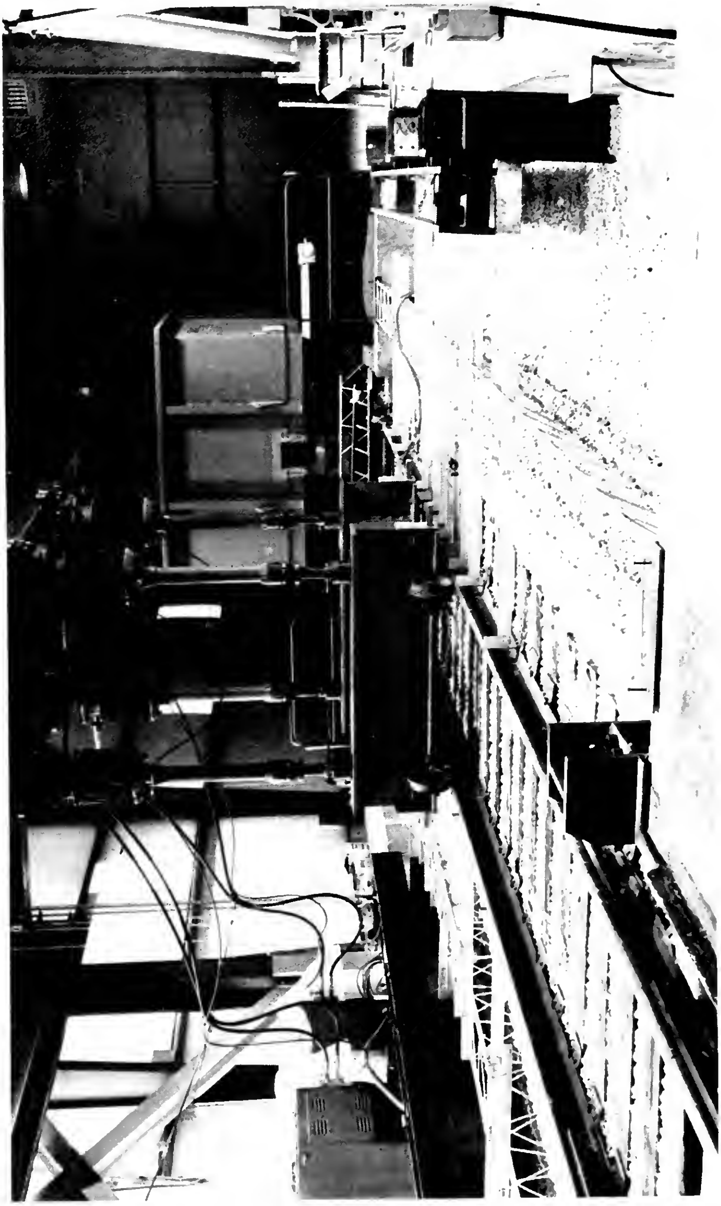

Figure 2. Test Set Up Showing Hydraulic Loading Jacks, Loading Bolster, Vertical Instrumentation, and Reference Fram.

TEST RESULTS

Three different analytical techniques were used to reduce the data with each method assuming a different definition of track modulus and also utilizing a separate procedure for calculating the modulus. The three methods are:

1. Deflection curve
2. Heavy-light wheel load deflection curve
3. Beam on elastic foundation

1. *Deflection Curve*

This method was used by the ASCE-AREA Special Committee (6) under the leadership of Talbot. The basic assumption of this method is that the applied wheel load divided by the area under the deflection curve is the track modulus, i.e.

$$u = \frac{P}{s \sum_{i}^{n} y_i} \qquad \text{-------------------- (1)}$$

Where u is the track modulus* (lb/in^2)

P is the applied wheel load (lb)

y_i is the deflection of the i^{th} tie (inches)

s is the tie spacing (inches)

n is the number of depressed ties.

Using this method, the modulus of the test track was found to be 4,712 lb/in^2 for a load of 39,566 lb and 4,796 lb/in^2 for a load of 50,327 lb. Figure 3 shows the deflection curve under the two loads.

2. *Heavy-Light Wheel Load Deflection Curve*

This method differs with the previous one in the way the applied load is taken into account. This method assumes that the track modulus is the difference of a heavy and a light wheel load divided by the net difference in area under these loads, i.e.

$$u = \frac{P-p}{s \sum_{i}^{n} (y - y_1)} \qquad \text{-------------------- (2)}$$

Where u is the track modulus (lb/in^2)

P is heavy wheel load (lb)

p is light wheel load (lb)

*u was defined to be the pressure per unit length of each rail necessary to depress the track one unit (modulus of elasticity of rail support) s is tie spacing (inches)

y is the individual tie depression (inches) under P

y_i is the individual tie depression (inches) under p

n is the number of depressed ties

Using this method, the modulus of the track was found to be 5,016.5 lb/in^2 for a heavy load of 37,536 lb and a light load of 5,695 lb. Figure 4 shows the deflection curves under the heavy and light loads. It is evident from the data that it requires approximately 2,000 lb before the slack in the system is removed.

In methods one and two, the level measurements were used in determining the area under the load-deflection curve. The number of deflection points measured with the LVDT's was insufficient to establish a valid deflection curve.

3. *Beam On Elastic Foundation*

This method is based on the beam-on-an-elastic-foundation-theory, as defined by Winkler (1), and discussed by Talbot (6), which relates the deflection of the track, the applied load, and the track modulus. Solution of the beam on elastic foundation equation for the track modulus yields an equation for the modulus

$$u = \sqrt[3]{\frac{P^4}{64EIy^4}} \qquad \text{------------ (3)}$$

where u is the track modulus (lb/in^2)

EI is the stiffness of the rail (lb-in^2)

P is the applied wheel load (lb)

y is the deflection under the load (inches)

This method differs from the previous two techniques in that the stiffness or bending rigidity of the rail is directly taken into account in the calculation of track modulus. Furthermore, only one deflection measurement, at the point of loading is required, rather than the entire deflection curve of the track.

Evaluation of the test data showed that the track modulus varied with the applied load. Figure 5 shows the track modulus *vs* the applied load for the loading and unloading sequence. It can be seen that for loads above 5,000 lb and up to 50,000 lb the modulus varies linearly with respect to the load for increasing loads. For decreasing loads it varies linearly from 50,000 lb to 10,000 lb. At loads less than 10,000 lb on the decreasing sequence, and less than 5,000 lb on the increasing sequence, the modulus variation was quite non-linear.

The difference in values for the track modulus shown for the loading and unloading sequence can be attributed to the permanent deformation in the track. The variability of the track modulus with load suggests that the track modulus should be measured as close as possible to the expected load environment of the given track. The difference between loading and unloading curves shown in Figure 5 would suggest that the values of the track modulus are dependent on the time duration of the load.

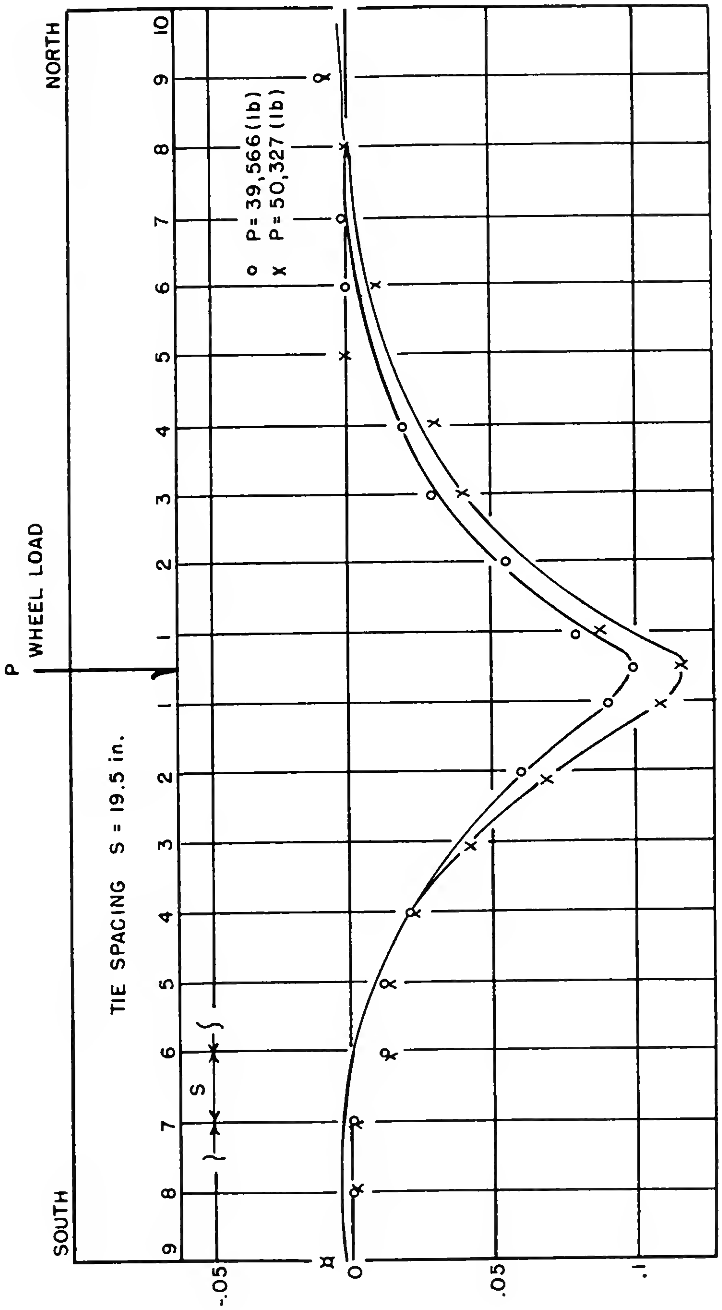

FIGURE 3. LOAD-DEFLECTION CURVE FOR HEAVY WHEEL LOADING.

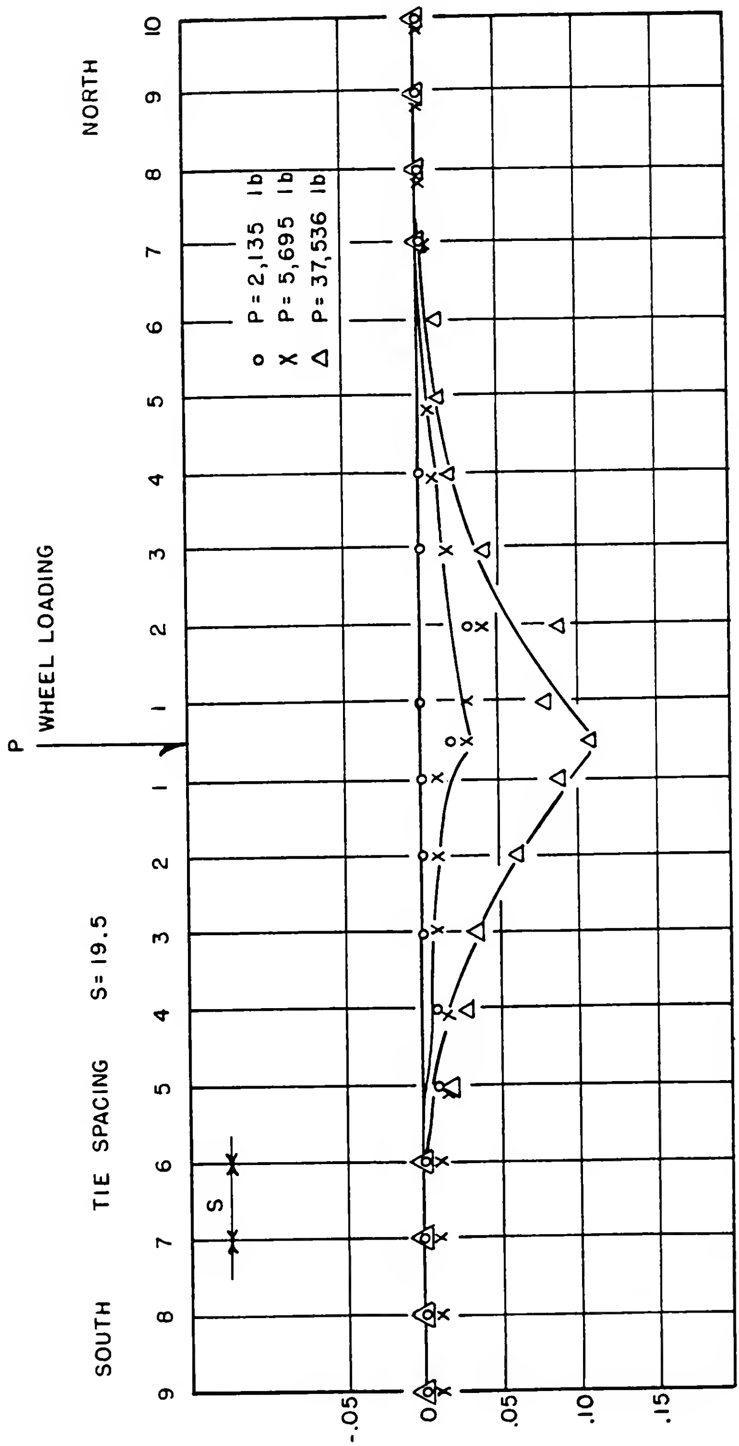

FIGURE 4. LOAD-DEFLECTION CURVE FOR HEAVY AND LIGHT WHEEL LOADS.

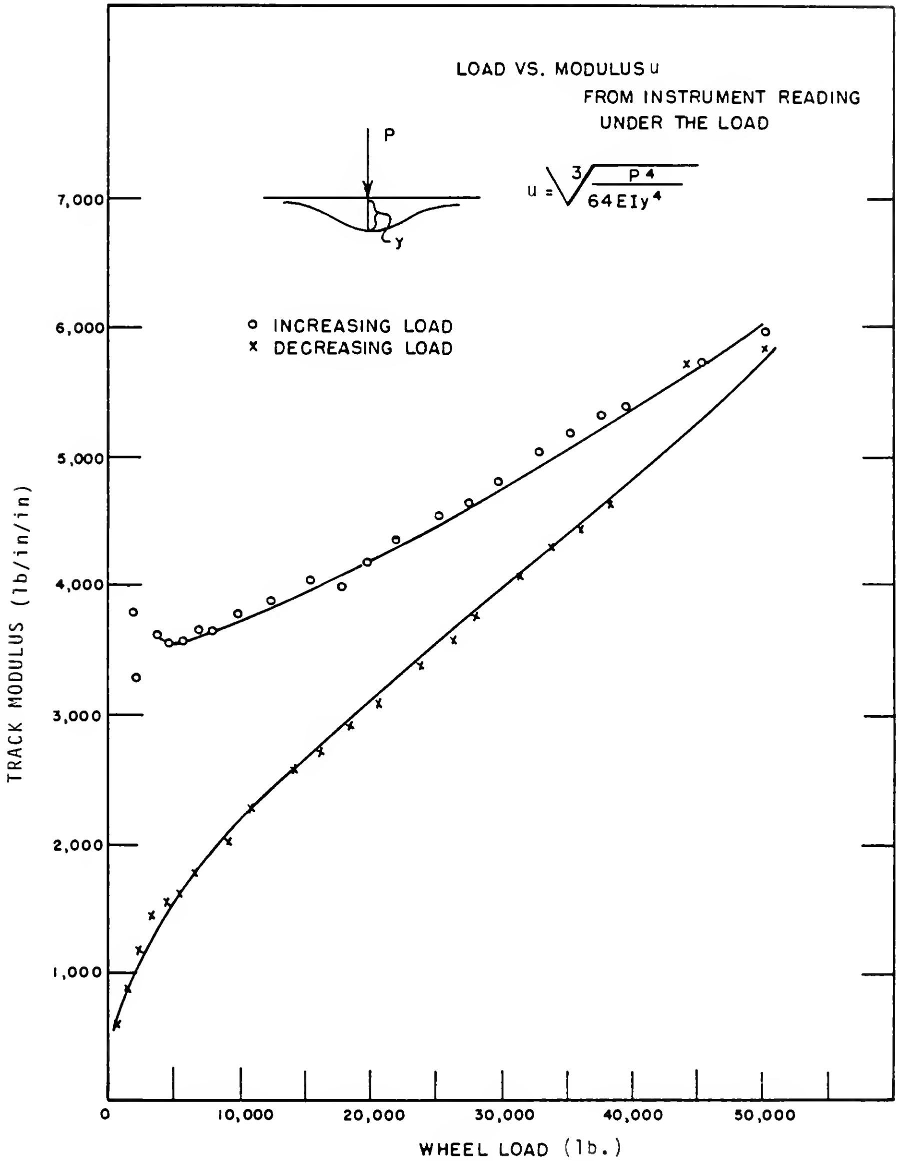

FIGURE 5. TRACK MODULUS VS. WHEEL LOAD FOR THE INCREASING AND DECREASING LOAD SEQUENCE.

Table 1 gives the calculated modulus values for the increasing and decreasing load sequences. Table 2 gives the modulus value calculated for the third loading sequence.

Table 3 presents a summary camparison of the track modulus values calculated for each of these three methods for three sets of applied wheel loads. Note that in all cases shown, the three methods all yielded modulus values within a 20% band.

In order to calculate track modulus under multiple wheel loading using the deflection curve methods, it is necessary only to sum the applied loads and divide this applied load value by the area of the deflection curve, as appropriate. However in order to utilize beam on elastic foundation theory, an iterative calculation is required to determine the modulus.

Noting the equation for deflection of the track

$$y(x) = \frac{P\beta}{2} \eta(x) \quad \ldots\ldots (4)$$

where y(x) is the track deflection at x (inches)

x is the distance from the load (inches)

P is the applied load (lb)

u is the track modulus (lb/in^2)

$$\eta(x) = e^{Bx} (\cos \beta x - \sin\beta x) \quad \ldots\ldots (5)$$

and

$$\beta = \left(\frac{u}{4EI}\right)^{1/4} \quad \ldots\ldots (6)$$

where EI is the stiffness of the rail and using superposition theory for multiple loads, the track modulus equation becomes:

$$u = \frac{\beta}{2y} \sum_{i=1}^{n} P_i \eta_i \quad \ldots\ldots (7)$$

where n is the number of loads

i is the i^{th} load.

Rewriting equation (7) as

$$\left| u - \frac{\beta}{2y} \sum_{i=1}^{n} P_i \eta_i \right| \leq \epsilon \quad \ldots\ldots (8)$$

a solution can be obtained using an iterative approach by choosing a value of u and systematically changing it until equation (8) is satisfied, to a preassigned accuracy, ϵ.

Track stiffness and track compliance values calculated from the test data are also given in Tables 1 and 2. The track stiffness was determined by dividing the applied load by the deflection of the track under the load, i.e.

$$K = \frac{P}{y} \quad \ldots\ldots (9)$$

TABLE 1: SUMMARY OF RESULTS FOR THE LOADING AND UNLOADING SEQUENCE

LOAD (LB)	DEFLECTION (IN)	TRACK MODULUS (LB/IN x IN)	TRACK STIFFNESS (LB/IN)	TRACK COMPLIANCE (IN/LB)
	INCREASING LOADS			
641.9	.0013	6967.50	0.49379E+06	0.20252E-05
1867.4	.0060	3765.39	0.31124E+06	0.32130E-05
2135.9	.0076	3286.26	0.28104E+06	0.35583E-05
3583.1	.0119	3602.80	0.30110E+06	0.33211E-05
4586.9	.0154	3550.97	0.29785E+06	0.33574E-05
5695.6	.0191	3556.59	0.29820E+06	0.33534E-05
6804.4	.0224	3645.40	0.30377E+06	0.32920E-05
7808.2	.0257	3646.21	0.30382E+06	0.32914E-05
9990.7	.0321	3765.39	0.31124E+06	0.32130E-05
12185.0	.0384	3863.76	0.31732E+06	0.31514E-05
15476.3	.0472	4036.33	0.32789E+06	0.30498E-05
17588.8	.0541	3990.92	0.32512E+06	0.30758E-05
19771.4	.0588	4174.13	0.33625E+06	0.29740E-05
21930.6	.0636	4316.62	0.34482E+06	0.29001E-05
25151.9	.0704	4525.68	0.35727E+06	0.27990E-05
27486.2	.0755	4640.63	0.36406E+06	0.27468E-05
29738.8	.0797	4795.56	0.37313E+06	0.26800E-05
32936.7	.0854	5011.68	0.38568E+06	0.25929E-05
35247.7	.0894	5161.12	0.39427E+06	0.25363E-05
37535.3	.0933	5301.89	0.40231E+06	0.24857E-05
39566.1	.0975	5363.46	0.40581E+06	0.24642E-05
45285.1	.1063	5722.46	0.42601E+06	0.23474E-05
50268.8	.1144	5963.71	0.43941E+06	0.22758E-05
	DECREASING LOADS			
50222.1	.1166	5806.95	0.43072E+06	0.23217E-05
44246.3	.1111	5230.83	0.39826E+06	0.25109E-05
38188.9	.1048	4646.44	0.36440E+06	0.27443E-05
35983.0	.1019	4455.71	0.35312E+06	0.28319E-05
33835.4	.0987	4283.11	0.34281E+06	0.29171E-05
31524.5	.0954	4078.37	0.33045E+06	0.30262E-05
28128.1	.0899	3791.94	0.31288E+06	0.31961E-05
26097.3	.0868	3595.73	0.30066E+06	0.33260E-05
23751.3	.0825	3393.64	0.28789E+06	0.34735E-05
20553.4	.0759	3127.58	0.27080E+06	0.36928E-05
18382.5	.0714	2923.89	0.25746E+06	0.38841E-05
16153.2	.0659	2738.54	0.24512E+06	0.40797E-05
13982.3	.0597	2577.28	0.23421E+06	0.42697E-05
10807.7	.0507	2273.29	0.21317E+06	0.46911E-05
8496.8	.0435	2023.21	0.19533E+06	0.51196E-05
6501.0	.0366	1782.45	0.17762E+06	0.56299E-05
5333.8	.0320	1637.59	0.16668E+06	0.59994E-05
4423.5	.0283	1503.10	0.15631E+06	0.63977E-05
3443.1	.0224	1469.88	0.15371E+06	0.65058E-05
2205.9	.0170	1172.74	0.12976E+06	0.77066E-05
1283.9	.0117	937.84	0.10973E+06	0.91132E-05
543.6	.0072	576.60	0.76188E+05	0.13125E-04
175.1	.0047	222.08	0.37249E+05	0.26846E-04

TABLE 2: SUMMARY OF RESULTS FOR THE THIRD LOADING SEQUENCE

LOAD (LB)	DEFLECTION (IN)	TRACK MODULUS (LB/IN x IN)	TRACK STIFFNESS (LB/IN)	TRACK COMPLIANCE (IN/LB)
	INCREMENTED LOADS			
5940.7	.0230	2936.55	0.25829E+06	0.38716E-05
10037.4	.0359	3263.80	0.27959E+06	0.35766E-05
19794.7	.0596	4106.05	0.33213E+06	0.30109E-05
29937.2	.0795	4854.50	0.37657E+06	0.26556E-05
39659.5	.0963	5469.92	0.41183E+06	0.24282E-05
50420.5	.1142	6001.71	0.44151E+06	0.22650E-05

TABLE 3. SUMMARY OF MODULUS VALUES

Load (lb)	Modulus (lb/in^2)		
	Method 1	Method 2	Method 3
37,536	4,465	5,167	4,257
39,566	4,712	5,658	4,667
50,327	4,769	5,529	5,604

Where K is the track stiffness (lb/in)

P is the applied wheel load (lb)

y is the deflection under the load (inches)

The track compliance is defined as the inverse of the stiffness:

$$C = \frac{1}{K} \qquad \text{.........................(10)}$$

Where K is the track stiffness (lb/in)

C is the track compliance (in/lb)

The relationship between track modulus and track stiffness is shown in Figure 6.

Finally, utilizing data from the bending strain gauge arrays, the bending stresses in the rail were obtained and compared with the moment influence line predicted by beam on elastic foundation theory (12). Note the excellent agreement between the test data and the theory. (Figure 7)

For a more detailed discussion of the test data and test results, the reader is referred to reference (13).

CONCLUSIONS

In comparing the methods of determining track modulus, one should concentrate not only on the results that best represent the track response but also on the technique that is easiest to use.

Thus, before any conclusion can be made as to the "best" method for calculating track modulus, consideration should be given to the practical problem of collecting data. As noted previously, method 1, the deflection curve technique, appears to be the most accurate because it takes into account a large portion of the track, thus eliminating local effects. However, the use of this method in the field could be cumbersome, since along with the load information, at least six absolute deflection measurements have to be taken on each side of the load. This can be done either manually with a level, thus creating a time delay in the readings, or mechanically with displacement transducers, which requires significant instrumentation. Time delay especially, in soft track, could give erroneous readings due to creeping under load. These disadvantages tend to outweigh the accuracy obtained by using this method. Method 2, the heavy-light load deflection curve, has the same disadvantages as method 1. In fact, they are even more severe since two deflection curves have to be obtained, thus doubling the number of measurements and consequently the test time. Furthermore, it is expected that this method would always give a higher track modulus than that "seen" by a vehicle in service since the initial slack in the track structure has been eliminated.

Thus, of the three methods, method 3, the beam on elastic foundation, appears to be the most suitable for general use. This method is substantially easier to collect data for, since it requires only one deflection value together with the applied load. Its accuracy, as compared to method 1, which is considered by many to be the "correct" method, is quite good. Therefore, it is recommended by the authors that the beam on elastic foundation theory be used where track modulus is required. Furthermore, it should be determined using a load level corresponding to the level of traffic experienced by the track. Thus for track that sees 100 ton car traffic, a wheel load of approximately 33,000 lb should be used to calculate the track modulus. For track that sees lighter traffic, an appropriate lower load should be used. If a

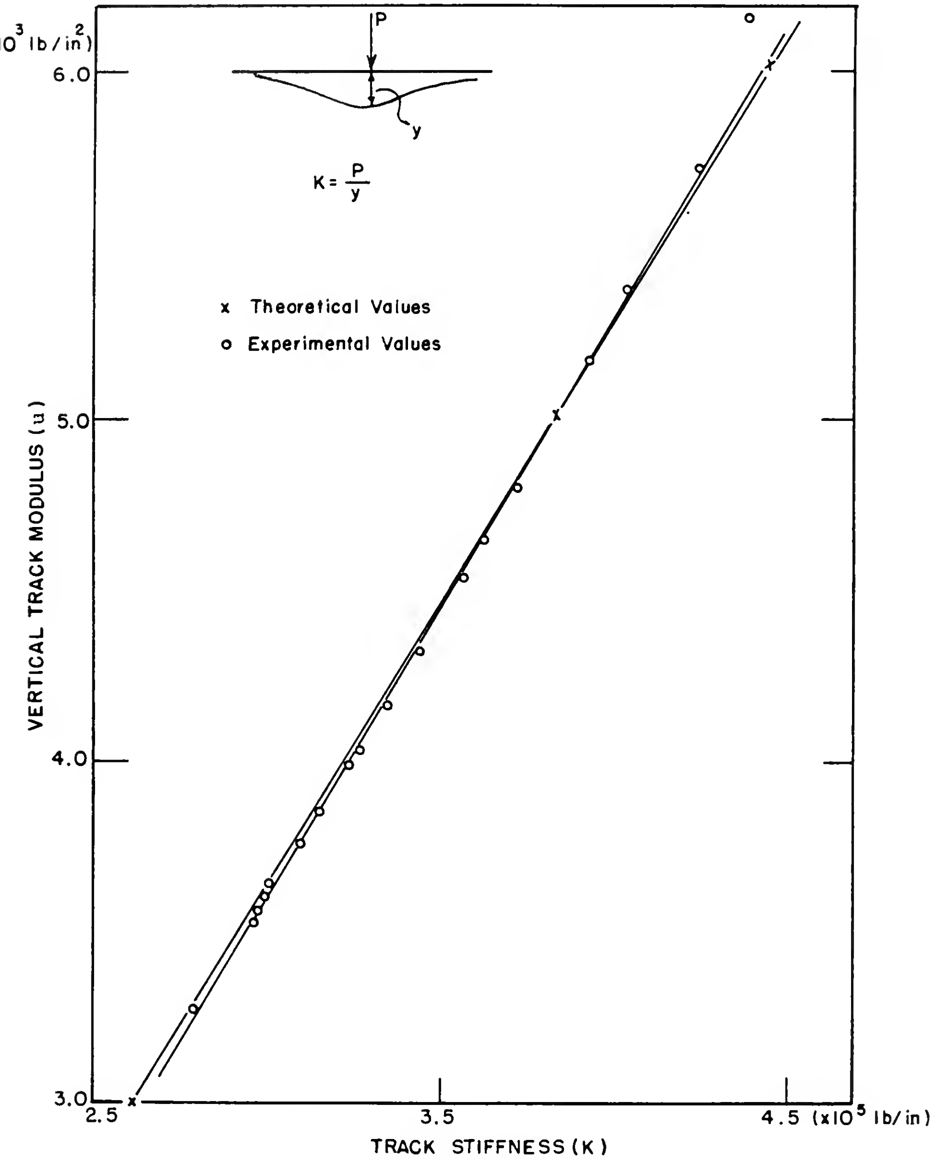

FIGURE 6. TRACK MODULUS VS. TRACK STIFFNESS FROM RAIL DEFLECTION UNDER THE LOAD.

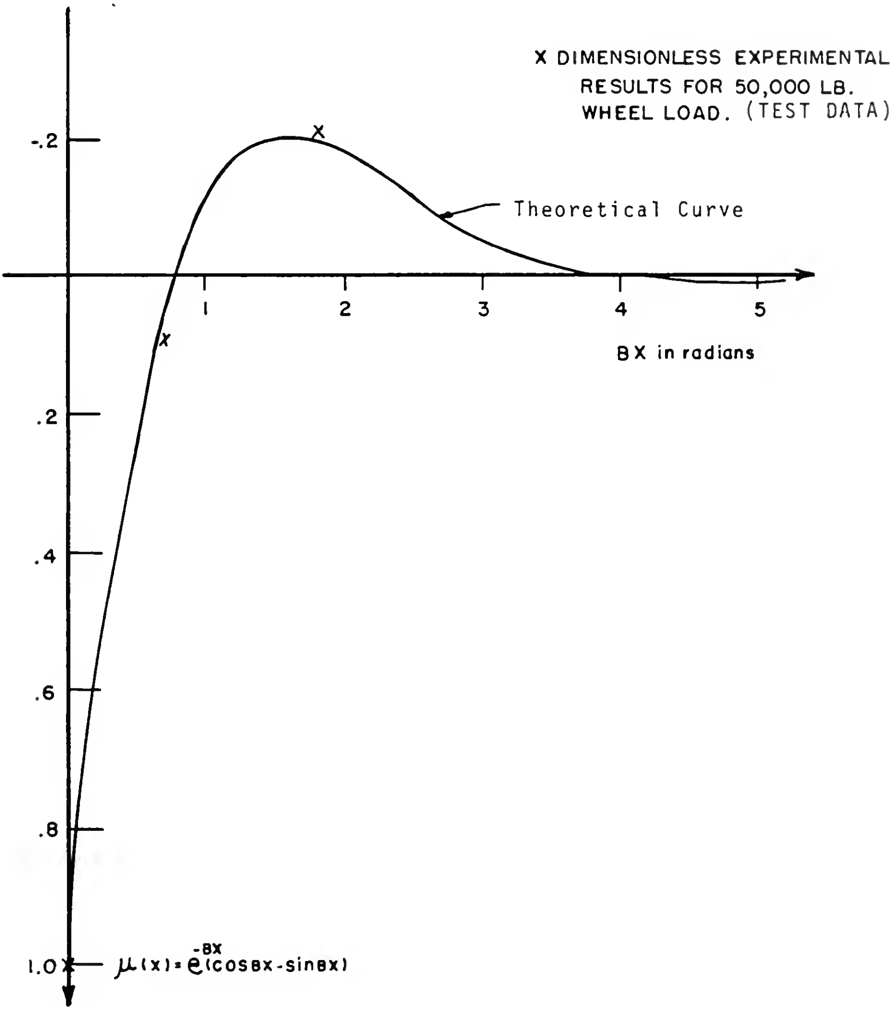

FIGURE 7

DIMENSIONLESS INFLUENCE LINE FOR THEORETICAL MOMENT comparison of different track with varying support conditions is desired, a lighter wheel load, possibly 27,5000 lb (70 ton car), should be used.

Once the track modulus is known, the stiffness and the track compliance can be readily determined from Figure 8, which gives the relation between track modulus and track stiffness for a range of rail sizes. With this information, the track engineer is then in a better position to evaluate the condition of his track, and it ability to support service traffic.

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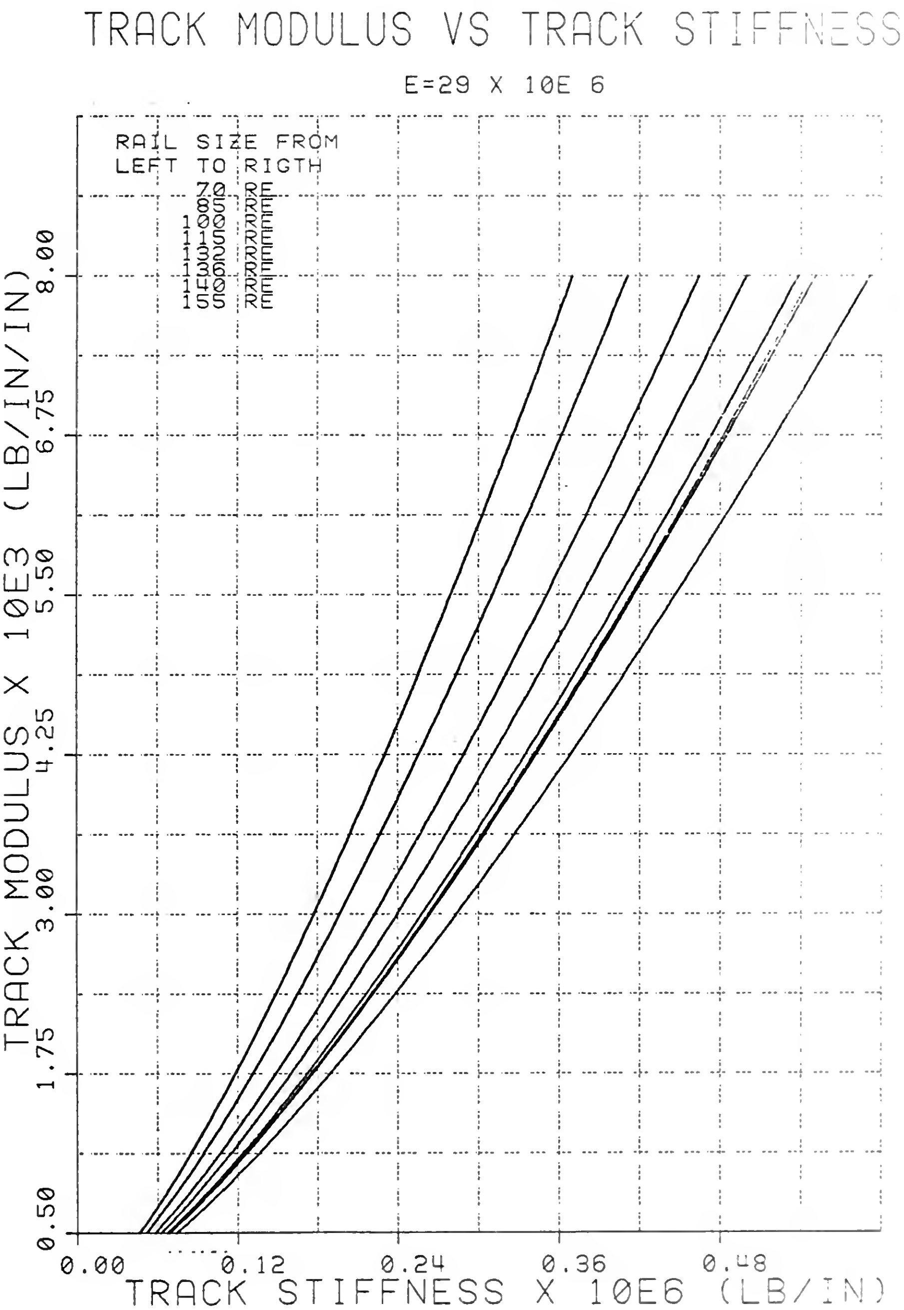

FIGURE 8. TRACK MODULUS VS TRACK STIFFNESS, THEORETICAL RESULTS FOR VARIOUS SIZE OF RAILS.

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   Brief progress report, presented as information .... page 178

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   Progress report, presented as information............................. page 179

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Report on Assignment 1

Administrative Systems—Disseminate Information Pertinent to Design and Implementation, Including Specific Applications or Techniques Within the Scope of Railroad Engineering

R. H. Knittel (Chairman, Subcommittee), R. S. Allen, D. J. Faulkner, L. F. Grabowski, R. H. Johnston, M. J. Nelson, A. W. Polich, L. T. Richards, Milan Velebit, C. F. Wiza, J. R. Wood, Jr., Andrews Youhanie.

Your committee has developed a proposed section of the manual entitled "Track and Roadway Inventory," and a final draft is being prepared. Purpose of the section is to provide a checklist for elements to be included in a track and roadway computerized inventory as well as a suggested format for the inventory. It is anticipated that the material will be submitted to the committee for letter ballot in November or December of this year (1979).

As a follow-on to the track and roadway inventory, we plan to develop systems for accessing and utilizing the inventory. Also, we anticipate developing a similar check list for maintenance-of-way equipment.

Report on Assignment 2

Make available a digest of present railroad engineering systems applications, including all present system-oriented applications of the assignments of other AREA technical committees.

F. S. Mitchell (Chairman, Subcommittee), D. E. Bartholomew, D. M. Harlan, W. L. Piotrowski, C. T. Popma, T. W. Toal

A digest of railroad engineering systems applications was last published in 1975. The value of such a digest has been challenged and the committee has not decided whether to publish a new digest, and if so, what vehicle should be used. A decision will be made during the coming year.

Report on Assignment 3

Systems Engineering Education—Collect and disseminate information to the association membership by means of special features, seminars, demonstrations, and printed material.

A. D. M. Lewis (Chairman, Subcommittee), D. R. Bergmann, F. A. Daly, R. Dirvonis, A. R. Hermann, J. W. Jenkins, C. J. Porterfield, F. E. Young

Two specific items are included in this assignment, namely,

(a) programming systems for engineering design and analysis problems, and
(b) application of computer graphics to railroad engineering.

With regard to computer graphics, on October 16 and 17, the committee sponsored "A Symposium on Computer Graphics in Railroad Engineering," Some 60 registrants participated representing some 15 railroads, 6 suppliers and consultant organizations, and 3 government or industry agencies and research organizations.

Report on Assignment 4

Provide interface for coordination of effort in railroad engineering systems.

R. F. Tuve (Chairman, Subcommittee), L. P. Diamond, R. P. Howell, C. E. Law, H. R. Williams

Assignment 4 specifically provides for coordination of effort in railroad engineering systems "between AREA and DOT (FRA)." Two FRA contracts fall in this categroy. One of these is a contract with the Mitre Corporation to supply certain data concerning long-range management use of data produced from a track geometry car. It is the intention of this committee to become fully informed on this contract—its purpose or object, status, and the overall intentions of the FRA with respect to track geometry car data. The committee will then request a meeting with the AREA board and solicit its advice and guidance.

The second FRA contract deals with the development of an industry track data base to "...provide information for impact accessments of tracks standards implementation. In addition, the data base should provide information that will be useful in economic evaluation of railroad maintenance techniques." Specifically, the data base is intended to include the following:

(1) Weight of rail in pounds per yard

(2) Rail type—continuous welded or bolted jointed track

(3) Rail age—date rail was laid

(4) Date of last update of the track chart

(5) Resurfacing history—date of last resurfacing

(6) Tie replacement year—date ties were replaced

(7) Ballast type

Again, it is the intention of this committee to determine the status of this contract, the general approach to fulfilling the contract, and the role of the AAR. This committee will then request a meeting with the AREA board to determine what role the AREA should play.

Report on Assignment A
Recommendations for Further Study and Research

Four new assignments are being explored by the committee. They include:

(1) Utilization of track geometry data for

(a) track maintenance planning

(b) measurement of track maintenance quality

(c) measurement of track degradation

(2) Economics of track maintenance—track deterioration modeling and optimum scheduling

(3) Hazardous material transport—considerations in route selection and track maintenance where hazardous commodities are involved

(4) Exploration of feasibility of a student design project sponsored by the committee.

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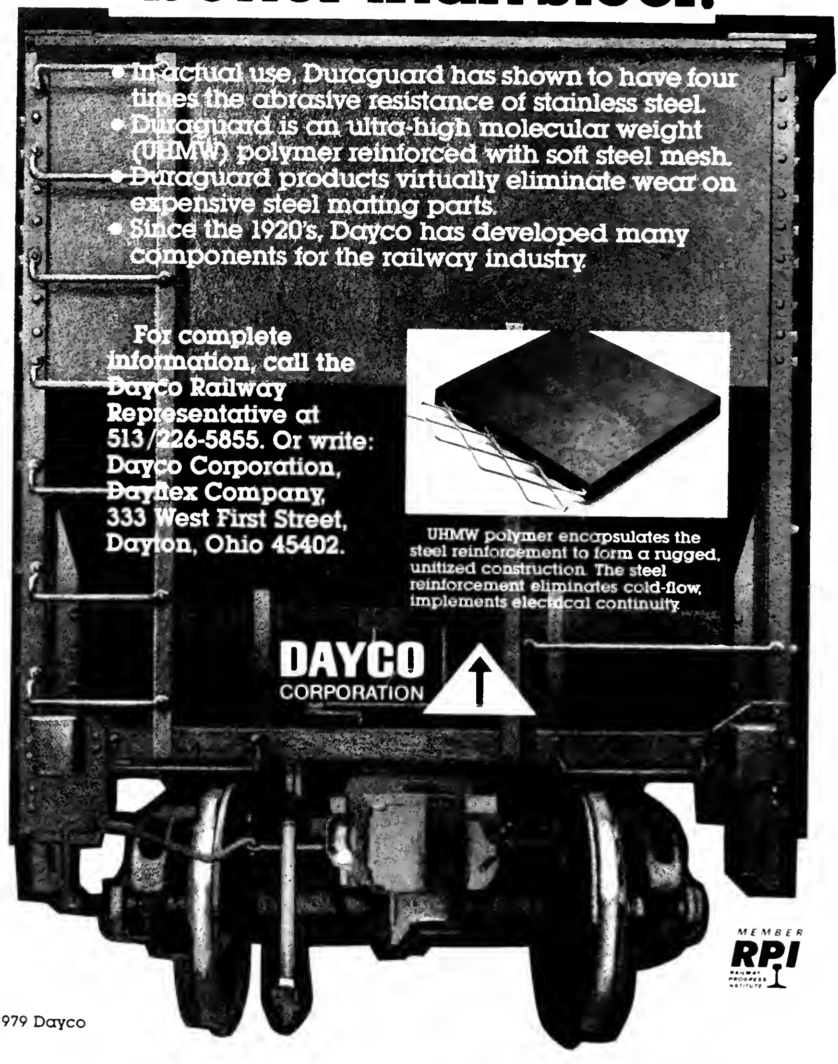

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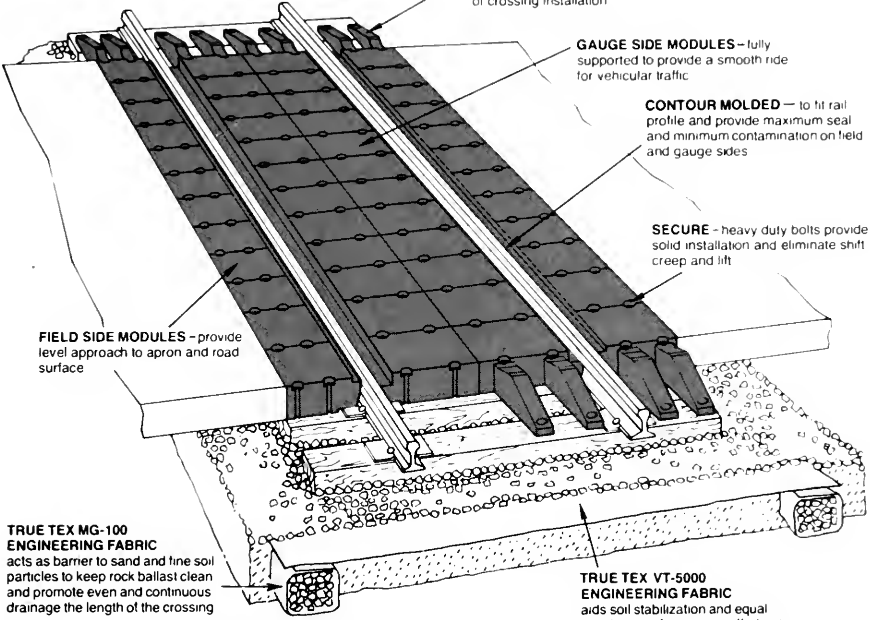

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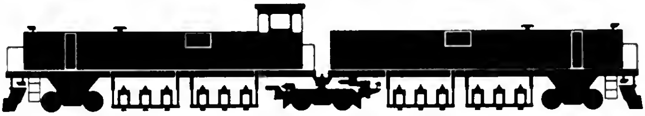
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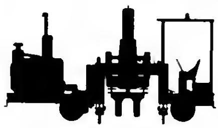
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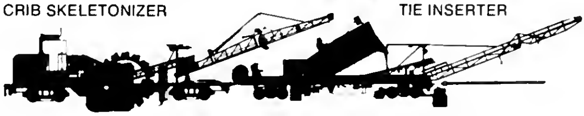
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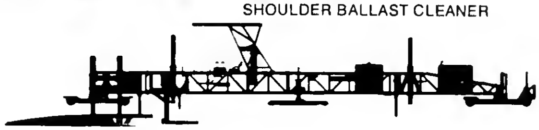
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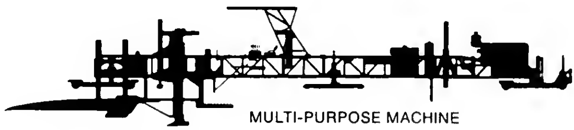
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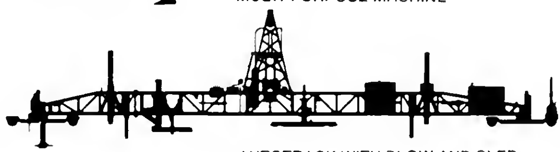
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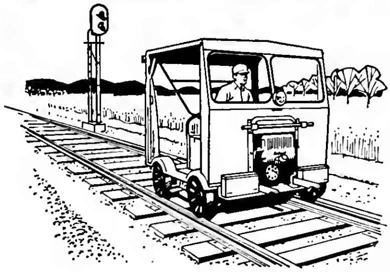

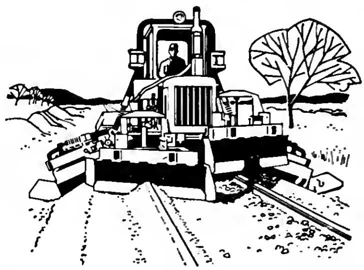

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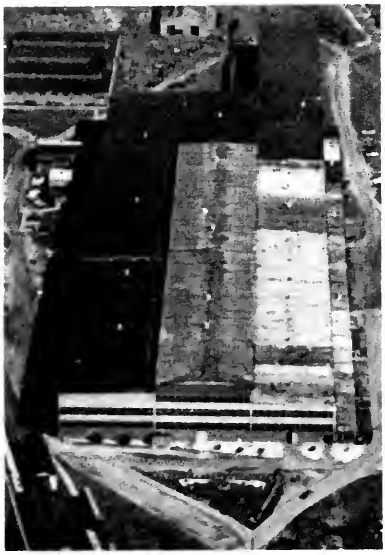

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SPECIAL REPORT

Committee 24—Special Report

RAILROAD ENGINEERING EDUCATION IN THE UNITED STATES

by E. Y. Huang*

In an effort to determine the extent to which today's civil engineering graduates are academically prepared for work in the railroad industry, a study on railroad engineering education in the U.S. was conducted by Committee 24. The study was carried out with two questionnaires. One questionnaire was prepared and sent to colleges and universities with 4-year civil engineering bachelor's degree programs in an attempt to determine the education in railroad engineering and related fields offered by educational institutions. The other questionnaire was designed to survey the opinions of major employers in the railroad field as to the qualifications of recently graduated civil engineering students hired between 1973-1977. Committee members participated in the development of the questionnaires were W. S. Autrey, B. M. Brown, C. D. Chambers, D. N. Cortright, B. M. Davidson, E. T. Franzen, W. W. Hay, E. Y. Huang, J. F. Pearce, C. T. Popma and D. V. Sartore.

Many universities and railroad companies responded to the questionnaires in detail, and to those organizations the Committee wishes to extend its fullest thanks and appreciation.

In the following the findings of this study are presented.

Undergraduate Instruction in Railroad Engineering and Related Areas

For the purpose of obtaining a general picture of the status of railroad engineering and related subject matter areas in today's American civil engineering curricula, a questionnaire containing a list of course work in various subject areas was sent to each of the heads or chairpersons of 187 civil engineering departments with ASCE Student Chapters, as listed in the ASCE Official Register 1977. Each respondent was requested to indicate the number of credit hours that was required or available as electives for each of the courses so listed. A total of 99 questionnaires or 52.9 percent of those solicited have been returned. The departments that responded graduated a total of 23,967 students between 1973 and 1977.

An examination of the returned questionnaires indicated that 73 of the responding departments operate on a semester basis, and the other 26 on a quarter basis. In order to provide a common base for comparing emphasis placed on the various subject areas under study, all the credit hours were expressed as semester hours. The average number of credit hours required for graduation by the responding departments was 132.

In Table 1 the required undergraduate coursework in railroad engineering and related areas offered by the responding civil engineering departments are summarized. The average number of semester hours devoted to each of the required undergraduate subject areas in this study was based on the number of departments which offer each individual subject. It may be noted that none of the responding departments requires a course on railroad engineering. However, 75 percent of the departments require a course in general transportation engineering, including railroad, highway, waterway, air and pipeline transportation, carrying an average of 3.2 credit hours. In addition, 20 percent of the departments have a course in highway

*Vice Chairman, AREA Committee 24.

engineering in their required curricula with an average credit of 2.8 hours. Near or over 90 percent of the departments require courses in the subject areas of surveying, graphics, properties of engineering materials, soil mechanics and foundation engineering, structural analysis and design, environmental engineering, and computer programming and numerical analysis techniques, providing a unifying background in all the major areas within civil engineering. The total amount of time devoted to the above subject areas amounts to 32 semester hours which is approximately 25 percent of the average of 132 semester hours required for graduation.

Table 2 summarizes the average number of semester hours devoted to elective courses available in the subject areas covered by this study. As before, the averages represent only those departments indicating elective hours in each area. Also included are the percentage of responding departments offering courses in each subject area.

Industry's Evaluation of Recently Graduated Civil Engineers

Civil engineers in railroad work are engaged primarily in the design, construction, and maintenance of the "fixed property" including tracks, bridges, buildings and related facilities to house many operations, and equipment and systems to reduce or prevent pollution. In soliciting opinions from railroad companies and consulting firms regarding the basic education and engineering background that were required for satisfactory performance by recently graduated civil engineers, the various civil engineering activities were presented in the form of a check list. Each respondent was requested to indicate by a yes, no, or unknown in regard to the degree of technical preparedness of these engineers in undertaking each of the listed activities that was assigned to them.

The questionnaire was mailed to the chief engineers of 57 railroad companies and 82 consulting engineering firms. As of this date, replies have been received from 23 railroad companies and 24 consulting engineering firms, which are respectively 40 percent and 29 percent of those solicited. It should be noted that although only a relatively small percentage of railroads responded to the questionnaire, those responding included almost all the major railroad companies who were also large employers of railroad engineers in the U.S. Belonging in this category are Amtrak, Burlington Northern, Chessie System, Consolidated Rail, Illinois Central Gulf, Louisville and Nashville, Milwaukee Road, Norfolk and Western, Seaboard Coast, Southern Pacific, Southern Railway, and Western Pacific. It may also be noted that 4 of the 23 railroads who responded did not employ any newly graduated civil engineers during the five-year period. Consequently, only 19 railroads involving a total of 416 recently graduated engineers have been evaluated in this study. The percentage of consulting firms responding to the questionnaire was even smaller than that of the railroads. However, a total of 352 persons was involved. In Table 3 the number of graduate civil engineers employed by the responding railroads and consulting firms is indicated.

The returns of the questionnaire from the railroads and the consulting firms were analyzed separately in terms of the percentages of recently graduated civil engineers having the basic education and engineering background for satisfactory performance for each of the activities listed. Since the responses by railroad companies with and without formal training programs were divergent for some of the listed activities, the returns from these two groups of railroads were further separated for analysis. The results of the analyses are presented in Table 4. The principal findings are summarized as follows:

A. Newly graduated civil engineers employed by railroad companies, either with or without a formal training program, were generally considered to be well qualified in the following activities:

1. Routine surveying operations.

2. Interpretation of aerial maps for routine location, network layout, and drainage design.
3. Design of simple buildings and structures.
4. Routine hydrological studies and drainage design.
5. Preparation of detail drawings and specifications.
6. Interpretation of construction plans and specifications.
7. Inspection and supervision of construction to insure workmanship.
8. Exercise of judgments involving economic alternatives based on principles of engineering economy regarding plant, equipment, and operations for construction and maintenance.
9. Preparations of routine engineering reports.
10. Handling of business correspondence.
11. Effective communication with associates, superiors and subordinates.

B. Between 50 to 75 percent of the newly graduated engineers employed by railroad companies were considered capable in the following activities:

1. Engineering evaluation of soil and rock materials.
2. Application of soil stabilization techniques to unstable soil conditions.
3. Application of high speed electronic digital computers to engineering computations, including basic programming skills.
4. Effective public speaking.
5. Application of basic knowledge of managerial principles to engineering operations.

C. Newly graduated civil engineers employed by railroad companies were not generally considered capable in the following activities:

1. Engineering evaluation of ballast materials.
2. Maintenance operations involving routine procedures and equipment.
3. Application of routine methods and equipment to reduce or prevent air, water, and noise pollution.

D. The percentage of newly graduated civil engineers capable of performing the activities listed below was higher in railroad companies with formal training programs than those without these programs. It appears that formal training programs provided by the railroad companies would improve the capabilities of newly graduated engineers in these activities.

1. Engineering evaluation of soil and rock materials.
2. Application of soil stabilization techniques to unstable soil conditions.
3. Routine track layout, including a basic understanding of track components.
4. Application of routine methods and equipment to reduce or prevent air, water, and noise pollution.
5. Effective public speaking.
6. Application of basic knowledge of managerial principles to engineering operations.

E. Newly graduated civil engineers employed by consulting engineering firms were generally considered well qualified only in the following activities:

1. Design of simple buildings and structures.
2. Routine hydrological studies and drainage design.
3. Applications of high speed electronic digital computers to engineering computations, including basic programming skills.
4. Effective communication with associates, superiors, and subordinates.

F. Sixty-seven (67) percent of the newly graduated engineers employed by consulting firms were considered capable in the following activity:

1. Interpretations of aerial maps for route location, network layout, and drainage design.

G. Newly graduated civil engineers employed by consulting firms were not generally considered capable in the following areas:

1. Routine surveying operations.
2. Engineering evaluation of soil and rock materials.
3. Application of soil stabilization techniques to unstable soil conditions.
4. Engineering evaluation of ballast materials.
5. Routine track layout, including a basic understanding of track components.
6. Preparation of construction plans and specifications.
7. Interpretation of construction plans and specifications.
8. Inspection and supervision of construction to insure quality workmanship.
9. Maintenance operations involving routine procedures and equipment.
10. Application of routine methods and equipment to reduce or prevent air, water, noise pollution.
11. Exercise of judgments involving economic alternatives based on principles of engineering economy regarding plant, equipment, and operations for construction and maintenance.
12. Preparations of routine engineering reports.
13. Handling of business correspondence.
14. Effective public speaking.
15. Application of basic knowledge of managerial principles to engineering operations.

H. There was a considerable difference of opinion between railroad companies and consulting engineering firms as to the capabilities of newly graduated civil engineers in their organizations. This seemed to be largely attributable to the different types of civil engineers they had employed. Engineers employed by consulting firms appeared to be primarily oriented toward design activities, whereas engineers employed by railroad companies were involved in general civil engineering practice, including also construction and maintenance operations.

A number of respondents from both the railroad companies and the consulting engineering firms also made comments regarding specific deficiencies that might exist in the typical civil engineering undergraduate education. These comments were generally explanative in nature, restressing opinions that were registered in the check list. Nine railroad companies remarked, for example, that it is unfortunate that most schools have dropped railroad engineering in their curricula and do not include railroad related problems in their transportation engineering courses. Consequently, the majority of civil engineering graduates have little knowledge of the diversity and complexity of railroad operations, and have had limited or no technical courses applicable to railroad engineering, dealing with track layout and design, yard design, and shop layout. Although holding a minority opinion among his peers, one railroad official also indicated that civil engineering graduates do not learn to communicate well, especially in the art of letter-writing, and they do not get enough liberal arts courses.

Among the consulting firms, eight commented that civil engineering graduates are deficient in the ability to prepare good written reports and make oral presentations. Seven indicated that there are too many computer experts and stress analysts among recent civil engineering graduates. Because practical knowledge is definitely lacking, very few are able to effectively apply the theoretical background to a real life situation. One consulting engineer felt that no new graduate is qualified immediately to undertake the jobs listed without close supervision. Three also suggested that civil engineering graduates would be much more useful if they could spend at least one summer of on-the-job experience prior to graduation, participate in co-op programs while at college, or serve a 6-month internship after graduation.

Conclusions

A comparison of the results of the railroad industry survey with those of the civil engineering curricula study indicates that today's civil engineering curricula are reasonably effective in preparing engineers to enter the railroad industry. As shown by the data in Table 4 and summarized in the previous section, the railroad companies were largely satisfied with the basic education and technical background of recently graduated civil engineers, even though no civil engineering department which responded to the questionnaire was offering a required course in railroad engineering. The consulting engineering firms, on the other hand, were more critical in their appraisal of these graduates. The difference in opinion between the two groups of employers probably arose from the different demands made on these graduates. Since the work of the consulting firms was primarily in engineering analysis and design, it was also possible that graduates with strong background in structures, computer programming and numerical analysis techniques were particularly sought by these firms. Consequently, those employed by these firms might not have as broad a background as those employed by the railroad companies, but were equally able to meet the performance requirements of their employers.

It should also be noted, however, that the civil engineering curricula represented in this study are far from "ideal" in the viewpoint of the railroad industry. This is particularly evident in the lack of technical knowledge of civil engineering graduates in the areas of engineering evaluation of ballast materials, the design and layout of track components, and railroad maintenance operations. It was suggested by several railroad companies that this deficiency was caused by the omission of railroad engineering courses in many civil engineering departments. The railroad industry cannot expect the colleges to change their already crowded curricula to fit the exact needs of the industry, however, since only a small portion of the civil engineering graduates are employed by the industry. To insure civil engineering graduates' academic preparation for railroad work, it seems therefore necessary that adequate instruction on the aforementioned subject areas be provided in the courses of transportation engineering which are being offered by 74 percent of the civil engineering departments that responded to the questionnaire. It also seems possible that railroad employers may find it necessary to provide in-service training in these skills if they cannot be introduced in the regular academic programs.

The data in Tables 2 and 4 also revealed that although there is evidence of a reasonable amount of effort being spent on soil mechanics, foundation engineering (96 percent of responding departments offering an average of 3.7 semester hours and 60 percent departments having an average of 4.2 semester hours available for electives), and environmental engineering (88 percent of responding departments offering an average of 4.0 semester hours and 67 percent departments having an average of 7.5 semester hours available for electives), it apparently is not sufficient to satisfy the needs of the railroad industry. Only 23.0 to 55.5 percent of the newly graduated civil engineers were considered to have satisfactory background in these areas. The courses are most likely not geared to the needs of the industry.

Another subject area that should be of concern to both the railroad industry and the civil engineering educators is the communication skills of the graduates. The deficiency in these skills is, however, college-wide and not only pertaining to civil engineering graduates. Many colleges have instituted tutorial courses and writing laboratories to improve the students' communication skills and apparently have achieved success.

TABLE 1

REQUIRED UNDERGRADUATE COURSEWORK IN RAILROAD ENGINEERING AND RELATED AREAS OFFERED BY RESPONDING CIVIL ENGINEERING DEPARTMENTS

Subject Area	Average No. Of Semester Hrs. Devoted to Coursework	% of Responding Departments Offering Coursework	% of Graduated Civil Engineers Having Taken Coursework
a. Surveying (including boundary surveys, construction surveys, and route surveying)	3.2	89	93
b. Photogrammetry	1.8	6	9
c. Graphics	2.9	90	87
d. Geology	3.0	55	62
e. Airphoto Interpretation	0.3	1	3
f. Properties of Engineering Materials	3.6	93	92
g. Soil Mechanics and Foundation Engineering	3.7	96	95
h. Fluid Mechanics	3.5	96	96
i. Hydrology and Hydraulic Engineering	3.4	69	72
j. Concrete Materials (including PCC and Bituminous mixtures)	2.9	49	62
k. Structural Analysis and Design (including bridge design)	7.8	96	97
l. Railroad Transportation Engineering (including roadway design, track structures, and operational characteristics of locomotives)	0	0	0
m. General Transportation Engineering (including railroad, highway, waterway, air and pipeline transportation)	3.2	74	78
n. Highway Engineering	2.8	20	23

TABLE 1 (continued)

REQUIRED UNDERGRADUATE COURSEWORK IN RAILROAD ENGINEERING AND RELATED AREAS OFFERED BY RESPONDING CIVIL ENGINEERING DEPARTMENTS

Subject Area	Average No. Of Semester Hrs. Devoted to Coursework	% of Responding Departments Offering Coursework	% of Graduated Civil Engineers Having Taken Coursework
o. Environmental Eng.	4.0	88	86
p. Computer Programming and Numerical Analysis Techniques	3.3	97	99
q. Engineering Economy	2.7	70	69
r. Statistics	2.4	37	46
s. Contracts and Specifications	2.0	28	32
t. Construction Equipment and Methods	2.3	12	12
u. Construction Management	2.6	18	13
v. Report Writing and Business Correspondence	3.1	44	47
w. Public Speaking	2.3	25	31

TABLE 2

ELECTIVE UNDERGRADUATE COURSEWORK IN RAILROAD ENGINEERING AND RELATED AREAS OFFERED BY RESPONDING CIVIL ENGINEERING DEPARTMENTS

Subject Area	Average No. of Semester Hrs. Devoted by Coursework	% of Responding Departments Offering Coursework
a. Surveying (including boundary surveys, construction surveys, and route surveying	5.2	44
b. Photogrammetry	2.9	39
c. Graphics	3.5	14
d. Geology	4.1	44
e. Airphoto Interpretation	3.3	18
f. Properties of Engineering Materials	4.2	25
g. Soil Mechanics and Foundation Engineering	4.2	60
h. Fluid Mechanics	4.3	20
i. Hydrology and Hydraulic Engineering	8.4	63
j. Concrete Materials (including PCC and Bituminous mixtures)	3.4	37
k. Structural Analysis and Design (including bridge design)	8.4	71
l. Railway Transportation Engineering (including roadway design, track structures, and operational characteristics of locomotives)	4.4	7
m. General Transportation Engineering (including railroad, highway, waterway, air and pipeline transportation)	5.6	43
n. Highway Engineering	4.3	49
o. Environmental Engineering	7.5	67
p. Computer Programming and Numerical Analysis Techniques	5.0	35
q. Engineering Economy	3.5	27
r. Statistics	3.6	52
s. Contracts and Specifications	2.6	29
t. Construction Equipment and Methods	3.4	36

TABLE 2 (continued)

ELECTIVE UNDERGRADUATE COURSEWORK IN RAILROAD ENGINEERING AND RELATED AREAS OFFERED BY RESPONDING CIVIL ENGINEERING DEPARTMENTS

Subject Area	Average No. of Semester Hrs. Devoted by Coursework	% of Responding Departments Offering Coursework
u. Construction Management	3.7	41
v. Report Writing and Business Correspondence	2.9	26
w. Public Speaking	2.9	28

TABLE 3

NUMBER OF GRADUATE CIVIL ENGINEERS EMPLOYED BY RESPONDING RAILROAD COMPANIES AND CONSULTING FIRMS

	Railroad Companies	Consulting Firms
Number of organizations to which questionnaires were distributed	57	82
Number of organizations returned questionnaires	23	24
Number of organizations for which questionnaires were analyzed	19	24
Number of graduate civil engineers presently in organizations:		
Total	981	707
Size range	5-263	2-150
Number of newly graduated civil engineers employed (1973-1977):		
Total	416	352
Size range	1-125	4-34
Number of newly graduated civil engineers employed and retained (1973-1977):		
Total	288	235
Retained percentage	69.2%	66.8%
Number of organizations with formal training programs	8 (42.1%)	0 (0%)

TABLE 4

PERCENTAGES OF RECENTLY GRADUATEDCIVIL ENGINEERS HAVING BASIC EDUCATION AND ENGINEERING BACKGROUND FOR SATISFACTORY PERFORMANCE OF VARIOUS ACTIVITIES

Activity	Evaluated by			
	Railroad Companies			Consulting Firms
	With Training Program	Without Training Program	Average	
a. Routine surveying operations (including boundary surveys, construction surveys, and route surveying).	91.4	87.2	90.1	25.6
b. Engineering evaluation of soil and rock materials.	75.9	8.0	55.5	33.8
c. Application of soil stabilization techniques to unstable soil conditions	69.8	12.8	52.6	13.9
d. Engineering evaluation of ballast materials	26.2	8.0	20.3	10.2
e. Interpretations of aerial maps for route location, network layout, and drainage design.	82.8	80.8	82.2	67.1
f. Design of simple buildings and structures.	100.0	92.0	97.6	88.9
g. Routine hydrological studies and drainage design.	90.4	89.6	90.1	77.9
h. Routine track layout, including a basic understanding of track components.	70.4	19.2	55.0	16.5
i. Applications of high speed electronic digital computers to engineering computations, including basic programming skills.	64.3	73.6	67.1	94.9
j. Preparation of detail drawings and specifications.	90.4	93.6	85.3	21.3
k. Interpretation of construction plans and specifications.	90.4	90.4	90.4	46.3
l. Inspection and supervision of construction to insure quality workmanship.	75.3	79.2	76.5	17.6

TABLE 4 (continued)

PERCENTAGES OF RECENTLY GRADUATED CIVIL ENGINEERS HAVING BASIC EDUCATION AND ENGINEERING BACKGROUND FOR SATISFACTORY PERFORMANCE OF VARIOUS ACTIVITIES

Activity	Evaluated by			
	Railroad Companies			Consulting Firms
	With Training Program	Without Training Program	Average	
m. Maintenance operations involving routine procedures and equipment	22.3	43.4	31.7	4.5
n. Application of routine methods and equipment to reduce or prevent air, water, and noise pollution.	57.4	12.8	44.0	23.6
o. Exercise of judgments involving economic alternatives based on principles of engineering economy regarding plant, equipment, and operations for construction and maintenance.	80.8	60.0	74.5	37.1
p. Preparations of routine engineering reports.	100.0	90.4	97.1	48.9
q. Handling of business correspondence.	82.8	82.4	82.7	48.0
r. Effective communication with associates, superiors, and subordinates.	100.0	88.8	96.6	79.3
s. Effective public speaking.	77.3	24.0	61.3	37.5
t. Application of basic knowledge of managerial principles to engineering operations.	71.8	41.6	62.7	18.5

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Cover Photo—Eastbound Chessie System Coal Train,
Point of Rocks, Maryland, December 1979

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Those whose names are shown in boldface, in addition to the chairman, vice chairman and secretary, are the subcommittee chairmen.

To the American Railway Engineering Association:

Your committee reports on the following subjects:

1. Roadbed.
   No report.

2. Ballast.
   No report.

3. Natural waterways.
A report is being prepared on use of geotextiles and rip-rap for slope protection.

4. Culverts and drainage pipe.
Working on expansion of tables to include larger diameter pipes, cooperating with Committee 15. Also working on development of specifications for aluminum culvert pipe. Plan to upgrade part 4 to decimal format. Material intended to be submitted to the Committee in the Spring of 1980.

5. Pipelines.
A table is being prepared covering casing pipe larger than 42" and also revision of the table for E-80 loading in lieu of the present E-72 loading. Material should be ready for Spring 1980 meeting. Subcommittee also has to study the use of non-metallic materials for pipelines.

6. Fences.
No report.

7. Signs.
No report.

8. Tunnels.
Work on revision of this section is being held up pending determination by Committee 28, Subcommittee 4, as to need for any clearance data revision.

9. Vegetation control.
No report.

The Committee on Roadway & Ballast,
W.J. Sponseller, *Chairman.*

Report of Committee 3—Ties and Wood Preservation

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To the American Railway Engineering Association:

Your committee reports on the following subjects:

B. Revision of Manual.
No report.

1. Metric Conversion.
Our Committee recommends that no action be taken towards metric conversion at this time.

2. Cross and Switch Ties.
No report.

3. Wood Preservatives.
Although there is no report at this time, we are taking an active interest in any changes.

4. Preservative Treatment of Forest Products.
This Subcommittee has been discussing vapor drying and Boultonizing.

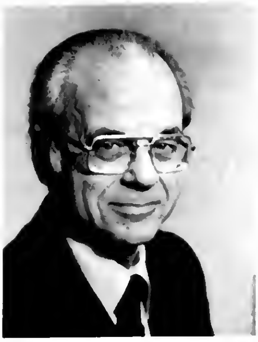
5. Service Records of Forest Products.
The annual tie renewal statistics as compiled by the Economics and Finance Depart-

ment, AAR, were published as an advance report in Bulletin 674, September-October 1979.

6. Collaboration with AAR and Other Organizations.
   Our Committee has been actively involved in the discussion of substitutes for wood ties, especially in laminated wood ties and reconstituted wood ties.

The Committee on Ties & Wood Preservation,
E. M. Cummings, *Chairman.*

Report of Committee 4—Rail



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To the American Railway Engineering Association:

Your committee reports on the following subjects:

3. Rail Statistics.
Progress report, submitted as information in this bulletin.

The Committee on Rail,
H.F. Longhelt, *Chairman.*

Report on Assignment 3

Rail Statistics

D.L. Banghart (*Chairman, Subcommittee*), B.G. Anderson, R.M. Brown, R.F. Bush, L.S. Crane, P.K. Cruckshank, R.C. Faulkner, M.A. Ferguson, R.G. Garland, W.J. Gilbert, T.B. Hutcheson, R.R. Lawton, H.F. Longhelt, W.S. Lovelace, J.F. Lyle, A.B. Merritt, Jr., F.W. Michael, B.J. Murphy, B.F. Overbey, R.C. Postels, J.M. Rankin, I.A. Reiner, W.A. Smith, R.K. Steele, G.S. Triebel, E.H. Waring, G.H. Way, M.J. Wisnowski

Rail Shipped

During the past year, Subcommittee 3 has secured from the American Iron and Steel Institute Technical Committee on Railroad Materials a summary of the tonnage of rail shipped from Canadian and United States steel mills to North American Railroads during 1978. A tabulation of this information is included herewith.

It is noted that 924,593 tons or 90.30% of the total rail shipped during 1978 was in sections which have been previously recommended that purchases of new rail be confined.

Also, included is a tabulation indicating the net tons of new rail shipped to North American Railroads from foreign rail mills in 1978.

Total shipments of new rail to North American Railroads from North American and foreign rail mills declined by 22% in 1978 compared to 1977 as indicated below:

Year	North American Mills	Foreign Mills	Total
1973	920,747	N/A	920,747
1974	1,018,446	N/A	1,018,446
1975	1,276,953	N/A	1,276,953
1976	1,442,243	N/A	1,442,243
1977	1,312,776	22,168	1,334,944
1978	1,023,901	18,523	1,042,424

Continuous Welded Rail

Your committee also presents as information the accompanying statistics pertaining to track miles of continuous welded rail (CWR) laid by years from 1933-1978; the track mile breakdown of CWR laid in 1978 between oxyacetylene and electric flash method; miles of CWR laid new and second-hand and further separated by that laid in main tracks, sidings and yard tracks.

The total miles of CWR reported laid in track by individual roads at the end of 1978 is also tabulated.

Inquiries were sent to 134 North American Railroads with replies received from 109 roads, reflecting 98% of the total track miles reported.

During 1978, more miles (6,823) of CWR was reported laid in track than in any other year previously reported. As an indication of the continuing increase in the use of CWR, 65% of the CWR reported since 1933 has been laid during the past nine-year period, 1970-78. At the end of 1978, over 65,600 track miles of CWR was reported in track, representing approximately 29% of the total miles of main track in the United States. An additional 8,600 track miles of CWR was reported in North American tracks outside the continental USA.

CONSOLIDATED REPORT OF RAIL SHIPPED TO NORTH AMERICAN RAILROADS FROM NORTH AMERICAN RAIL PRODUCING MILLS IN 1978 BY WEIGHT AND SECTION

WEIGHT	SECTION	TONS SHIPPED	% TOTAL
140*	AREA	33,049	3.23
136*	AREA	199,143	19.45
133	AREA	79,936	7.81
132*	AREA	512,696	50.07
122	CB	7,577	0.74
119*	AREA	37,921	3.70
115*	AREA	132,224	12.91
100*	AREA	7,648	0.75
100	ARA-A	8,161	0.80
90*	ARA-A	2,411	0.23
85	CP	1,912	0.19
75	BS	1,223	0.12
TOTAL		1,023,901	100.00

REPORT OF NEW RAIL SHIPPED TO NORTH AMERICAN RAILROADS FROM FOREIGN RAIL MILLS IN 1978 BY WEIGHT AND SECTION

		Net Tons Received From				
Weight	Section	Britain	Germany	Japan	Other	Total
136*	AREA	—	985	6,500	—	7,485
132*	AREA	4,038	—	—	—	4,038
122	CB	—	7,000	—	—	7,000
TOTAL		4,038	7,985	6,500	—	18,523

*Recommended Section.

TRACK MILES OF CONTINUOUS WELDED RAIL LAID BY YEARS, 1933-1978

	Total		Oxyacetylene	Electric Flash	Total
1933	0.16	1955	194.50	72.00	266.50
1934	0.95	1956	372.33	89.10	461.43
1935	4.06	1957	390.47	159.65	550.12
1936	1.52	1958	148.11	312.13	460.24
1937	31.23	1959	378.65	691.92	1,070.57
1939	6.04	1960	299.42	961.20	1,260.62
1942	5.48	1961	94.13	926.50	1,020.63
1943	6.29	1962	310.59	1,183.34	1,493.93
1944	12.88	1963	497.52	1,360.48	1,858.00
1945	4.81	1964	586.76	1,796.74	2,383.50
1946	3.91	1965	700.59	1,655.74	2,356.33
1947	18.70	1966	746.61	1,984.71	2,731.32
1948	29.93	1967	784.28	1,800.27	2,584.55
1949	33.05	1968	643.10	2,543.61	3,186.71
1950	50.25	1969	674.35	2,930.01	3,604.36
1951	37.25	1970	800.30	5,378.32	6,178.62
1952	40.00	1971	504.28	3,604.72	4,109.00
1953	80.00	1972	422.91	4,011.29	4,434.20
1954	87.00	1973	465.68	4,084.27	4,767.37*
		1974	273.79	4,183.48	4,457.27
		1975	139.58	4,151.83	4,291.41
		1976	294.04	5,658.89	5,952.93
		1977	300.35	6,158.14	6,458.49
		1978	192.85	6,630.16	6,823.01
			Total		73,214.62

*Total includes 217.42 miles reported in 1973, but breakdown not available.

BREAKDOWN OF CONTINUOUS WELDED RAIL LAID IN 1978—TRACK MILES

	Oxyacetylene		Electric Flash		
	New	Secondhand	New	Secondhand	Totals
Main Tracks	60.28	122.42	4,137.24	2,044.31	6,364.25
Sidings & Tracks	—	10.15	25.44	423.17	458.76
Total	60.28	132.57	4,162.68	2,467.48	6,823.01

TRACK MILES OF CWR REPORTED BY INDIVIDUAL ROAD THROUGH 1978

RAILROAD	TOTAL TRACK MILES	OXYACETYLENE NEW MAIN TRK.	OXYACETYLENE NEW SIDING & YD.	OXYACETYLENE SECONDHAND MAIN TRK.	OXYACETYLENE SECONDHAND SIDING & YD.	ELECTRIC FLASH NEW MAIN TRK.	ELECTRIC FLASH NEW SIDING & YD.	ELECTRIC FLASH SECONDHAND MAIN TRK.	ELECTRIC FLASH SECONDHAND SIDING & YD.	TOTAL CWR ON PROPERTY
Akron, Canton & Youngstown RR Co.	171.00	-	-	-	-	-	-	8.40	-	8.40
Alaska RR (The)	536.00	-	-	-	-	-	-	-	-	-
Aliquippa & Southern RR Co.	48.44	-	-	-	-	-	-	-	-	-
Alton & Southern Ry. Co. (The)	32.00	-	-	-	-	2.59	-	-	-	2.59
Apalochiola, Northern	96.00			(No Reply)						
Amtrak	1,709.00	-	-	-	-	809.00	-	7.30	0.50	816.80
Ashley, Drew & Northern Ry. Co.	41.00	-	-	-	-	-	-	-	-	-
Atchison, Topeka & Santa Fe Ry. Co.	12,251.00	448.29	-	5.40	-	3,901.09	21.26	1,795.83	290.17	6,462.04
Atlanta & St. Andrews Bay Ry. Co.	89.00	-	-	-	-	23.07	-	2.40	-	25.47
Atlanta & West Point RR Co.	222.65	-	-	-	-	35.93	-	-	-	35.93
Auto-Train Corp.	1,685.00	-	-	-	-	-	-	-	-	-
Bangor & Aroostook RR Co.	504.00	-	-	2.00	-	-	-	-	-	2.00
Belfast & Moosehead Lake RR	33.00	-	-	-	-	-	-	-	-	-
Belt Ry. Co. of Chicago (The)	455.44	3.83	-	-	-	26.79	.92	11.33	16.72	59.59
Bessemer & Lake Erie RR	204.87	-	-	-	-	73.87	2.38	117.04	9.63	202.92
Boston & Maine Corp.	1,574.00	52.30	-	20.10	-	-	-	9.50	-	81.90
British Columbia Hydro & Power Auth.	104.00	-	-	-	-	-	-	-	-	-
British Columbia Ry. Co.	1,260.80	-	-	-	-	-	-	-	-	-
Burlington Northern	22,988.00	352.00	-	-	-	4,342.32	110.33	793.91	52.33	5,650.89
Butte, Anaconda & Pacific Ry.	120.00	-	-	-	-	.07	-	-	-	.07
California Western RR	40.00	-	-	-	-	-	-	-	-	-
Cambria & Indiana RR	57.20	-	-	-	-	-	-	-	-	-
Cartier Ry. Co.	276.00	-	-	-	-	156.00	-	-	-	156.00
Central Romana RR	235.00			(No Reply)						
Central Vermont Ry., Inc.	364.00	4.00	-	-	-	-	-	-	-	4.00
Chesapeake & Western Ry.	52.00	-	-	-	-	-	-	-	-	-
Chessie System	11,217.00	-	-	-	-	3,084.30	-	3,433.50	570.40	7,088.20
Chicago & Illinois Midland Ry. Co.	121.02	5.40	-	-	-	17.90	-	-	-	23.30*
Chicago, Milw., St. Paul & Pacific RR	9,903.00	-	-	-	-	54.62	-	26.44	-	81.06
Chicago & Northwestern Trans. Co.	9,139.00	16.90	-	-	-	456.10	-	604.85	16.98	1,094.83
Chicago, Rock Island & Pacific RR Co.	7,103.42	-	-	-	-	626.26	-	394.38	7.01	1,027.65*
Chicago So. Shore & So. Bend RR	76.00	39.55	-	-	-	.25	.25	4.25	-	44.30
Chicago Union Station	13.52	-	.88	-	-	-	-	-	-	.88
Chicago & Western Indiana RR	27.00	-	-	-	-	-	-	-	-	- *
Chicago, W. Pullman & Southern RR Co.	30.61	-	.19	-	-	-	-	-	-	.19
Clinchfield RR Co.	275.00	71.80	-	-	-	56.00	-	-	-	127.80
Colorado & Southern Ry. Co. (The)	692.00	-	-	-	-	113.62	-	-	-	113.62
Colorado & Wyoming Ry. Co.	110.05	-	-	-	-	-	-	-	-	-
Columbia, Newberry & Laurens RR	75.00	-	-	20.40	-	-	-	45.00	-	65.40*
Columbus & Greenville Ry. Co.	168.00	-	-	-	-	-	-	-	-	-
Conemaugh & Black Lick RR Co.	39.67	-	-	-	-	-	-	-	-	-
Consolidated Rail Corp.	35,079.00	710.00	-	157.00	164.00	3,512.00	-	1,904.00	554.00	7,001.00

TRACK MILES OF CWR REPORTED BY INDIVIDUAL ROAD THROUGH 1978 (CONT.)

RAILROAD	TOTAL TRACK MILES	OXYACETYLENE NEW MAIN TRK.	OXYACETYLENE NEW SIDING & YD.	OXYACETYLENE SECONDHAND MAIN TRK.	OXYACETYLENE SECONDHAND SIDING & YD.	ELECTRIC FLASH NEW MAIN TRK.	ELECTRIC FLASH NEW SIDING & YD.	ELECTRIC FLASH SECONDHAND MAIN TRK.	ELECTRIC FLASH SECONDHAND SIDING & YD.	TOTAL CWR ON PROPERTY
Delaware & Hudson Ry. Co.	1,600.00	-	-	-	-	107.99	-	110.62	-	218.61
Denver & Rio Grande Western RR	1,848.00	52.00	-	-	-	340.31	-	51.31	4.86	448.48
Detroit & Mackinac Ry. Co.	405.00	-	-	-	-	-	-	-	-	-
Detroit, Toledo & Ironton RR Co.	478.00	3.00	-	2.60	-	-	-	-	-	5.60*
Detroit & Toledo Shore Line RR Co.	59.00	1.00	1.00	-	1.00	11.50	-	6.50	-	21.00
Duluth, Missabe & Iron Range Ry. Co.	461.00	1.51	-	-	-	132.00	.31	11.62	-	145.44
Durham Southern Ry. Co.	59.00	-	-	-	-	-	-	-	-	- *
East Erie Commercial RR	12.00	-	-	-	-	-	-	-	-	-
Elgin, Joliet & Eastern Ry. Co.	199.80	88.00	-	.90	-	22.90	-	-	-	111.80
Esquimalt & Nanaimo Ry.	197.00			(No Reply)						
Florida East Coast Ry. Co.	554.00	-	-	-	-	245.20	-	170.30	19.30	434.80
Fort Worth & Denver Ry. Co.	1,343.00	-	-	-	-	111.91	-	-	-	111.91
Gainesville Midland RR Co.	40.00	-	-	2.70	-	34.60	-	-	-	37.30*
Genesee & Wyoming RR Co.	12.20	-	-	-	-	-	-	-	-	-
Georgia RR Co.	301.84	-	-	-	-	80.68	-	.39	-	81.07
Grand Trunk Western RR Co.	1,219.00	97.89	-	72.93	15.27	100.00	-	10.00	5.00	301.09
Great Western Ry. Co.	58.00	-	-	-	-	-	-	-	-	-
Green Bay & Western RR Co.	254.00	-	-	-	-	-	-	-	-	-
Harbor Belt Line RR	108.67	-	-	-	-	-	-	-	-	-
Houston Belt & Term. Ry. Co.	53.17	-	-	-	-	32.63	.21	.05	-	32.89
Illinois Central Gulf RR Co.	9,659.49	840.45	3.95	903.82	46.81	306.79	3.65	61.48	4.41	2,171.36
Illinois Term. RR Co.	448.18	-	-	-	-	19.66	-	-	-	19.66
Indiana Harbor Belt RR Co.	114.00			(No Reply)						
Kansas City Southern Ry. Co.	881.77	58.14	4.39	10.49	.25	-	-	-	-	73.27
Kansas City Term. Ry. Co.	26.98	-	-	-	-	2.48	-	11.58	-	14.06
Kentucky & Indiana Term. RR	131.00	-	-	-	-	-	-	-	-	-
Lake Superior & Ishpeming RR Co.	98.00	-	-	-	-	-	-	-	-	-
Long Island RR Co.	322.00	-	-	-	-	166.69	1.60	81.75	2.39	252.43
Los Angeles Jct. Ry. Co.	66.00	-	-	-	-	-	-	-	-	-
Louisiana & Arkansas Ry. Co.	745.51	42.12	-	32.48	2.58	23.65	-	-	-	100.83
Louisville & Nashville RR Co.	6,617.00	762.81	2.48	438.41	11.23	664.53	17.54	406.63	56.45	2,360.08
Maine Central RR Co.	850.00	-	-	-	-	-	-	-	-	-
Manufacturers Ry. Co.	42.00	-	-	-	-	-	-	-	-	-
Maryland & PA RR Co.	90.59			(No Reply)						
McCloud River RR Co.	96.00	-	-	-	-	-	-	-	-	-
Meridian & Bigbee RR Co.	51.00	-	-	-	-	-	-	-	-	-
Michigan Northern Ry. Co.	248.00	-	-	-	-	-	-	-	-	-
Minneapolis, Northfield & Southern Ry.	77.00	-	-	-	-	-	-	-	-	-
Minnesota Transfer Ry. Co. (The)	13.00	-	-	-	-	-	-	-	-	-
Mississippi Export RR Co.	42.00			(No Reply)						
Missouri-Kansas-Texas RR Co.	2,168.00	-	-	-	-	161.47	-	-	-	161.47
Missouri Pacific RR Co.	11,500.87	-	-	-	-	3,023.40	-	1,427.00	17.00	4,467.40
Monongahela Conn. RR. Co.	40.58	-	-	-	-	-	-	-	-	- *

TRACK MILES OF CWR REPORTED BY INDIVIDUAL ROAD THROUGH 1978 (CONT.)

RAILROAD	TOTAL TRACK MILES	OXYACETYLENE NEW MAIN TRK.	OXYACETYLENE NEW SIDING & YD.	OXYACETYLENE SECONDHAND MAIN TRK.	OXYACETYLENE SECONDHAND SIDING & YD.	ELECTRIC FLASH NEW MAIN TRK.	ELECTRIC FLASH NEW SIDING & YD.	ELECTRIC FLASH SECONDHAND MAIN TRK.	ELECTRIC FLASH SECONDHAND SIDING & YD.	TOTAL CWR ON PROPERTY
Monongahela Ry. Co.	184.09	-	-	-	-	48.20	-	-	-	48.20*
Montour RR Co.	51.00	-	-	-	-	-	-	-	-	- *
Nevada-Northern Ry. Co.	148.00	-	-	-	-	-	-	-	-	-
New York Dock Ry.	58.00			(No Reply)						
Norfolk, Franklin & Danville Ry. Co.	205.00	-	-	13.50	-	-	-	6.50	-	20.00
Norfolk & Western Ry. Co.	7,481.00	351.03	-	260.86	28.52	1,482.96	-	805.21	131.58	3,060.16
Northwestern Pacific RR	313.00			(No Reply)						
Patapsco & Back Rivers RR Co.	99.30	-	-	-	-	-	-	-	-	-
Peoria & Pekin Union Ry.	16.00	-	-	-	-	5.50	-	-	-	5.50
Philadelphia, Bethlehem & New England	56.90	-	-	-	-	-	-	-	-	-
Pittsburgh & Lake Erie RR Co.	273.00	6.90	-	6.50	-	114.95	-	89.30	15.40	233.05*
Pittsburgh & Ohio Valley RR Co.	7.21	-	-	-	-	-	-	-	-	-
Pittsburgh & Shamut RR Co. (The)	96.00	-	-	-	-	-	-	-	-	-
Portland Term. RR Co.	76.00	-	-	-	-	-	-	-	-	-
Providence & Worcester Co.	176.00			(No Reply)						
Richmond, Fredericksburgh & Potomac RR Co.	118.00	58.34	-	-	-	64.11	3.39	57.97	7.84	191.65
Roscoe, Snyder & Pacific Ry. Co.	32.00	-	-	-	-	-	-	-	-	-
Sacramento Northern Ry.	451.00			(No Reply)						
St. Louis-San Francisco Ry. Co.	4,538.00	135.30	-	-	-	1,013.16	-	432.23	8.00	1,588.69
St. Louis Southwestern Ry. Co.	1,441.00	-	-	-	-	341.63	1.33	212.29	8.09	563.34
St. Paul Union Depot Co.	1.00	-	-	-	-	-	-	-	-	-
San Diego & Arizona Eastern Ry. Co.	171.00			(No Reply)						
Sand Springs Ry. Co.	43.00	-	-	-	-	-	-	-	-	-
Seaboard Coast Line RR Co.	9,028.00	1,370.30	-	995.30	-	364.70	-	19.40	-	2,749.70
Soo Line RR Co.	4,589.00	242.17	-	36.25	-	520.89	-	195.93	13.13	1,008.37
So. Buffalo Ry. Co.	70.49	-	-	-	-	-	-	-	-	-
So. Pacific Trans. Co.	11,294.00	-	-	-	-	2,791.17	10.77	2,236.35	216.75	5,255.04
Southern Ry. System	10,200.00	-	-	-	-	3,705.34	42.56	2,216.81	309.89	6,274.60
Steelton & Highspire RR Co.	26.45	-	-	-	-	-	-	-	-	-
Terminal RR Assn. of St. Louis	300.00	-	-	-	-	-	-	-	-	-
Texas-Mexican Ry.	157.00	-	-	-	-	-	-	-	-	-
Toledo, Peoria & Western RR Co.	301.00	35.00	-	14.20	2.00	9.30	-	34.44	-	94.94
Toledo Term. RR Co.	28.59	1.50	-	-	-	9.90	-	2.35	-	13.75
Union Pacific RR Co.	9,460.00	-	-	-	-	2,429.47	2.21	59.75	40.12	2,531.55
Union Railroad Co.	31.31	-	-	-	-	5.03	-	-	-	5.03
Utah Ry. Co.	95.00			(No Reply)						
Vermont Ry., Inc.	131.50	-	-	-	-	-	-	-	-	-
Virginia & Maryland RR Co.	96.00			(No Reply)						
Western Pacific RR Co.	1,486.00	-	-	-	-	226.24	-	16.27	-	242.51
Western Ry. of Alabama	222.65	-	-	-	-	44.12	-	-	-	44.12
White Pass & Yukon Route	110.00			(No Reply)						
Totals		5,851.53	12.89	2,995.84	271.66	36,056.84	218.71	17,892.16	2,377.95	65,677.58

*Year 1977 report used as no reply received for year 1978.

TRACK MILES OF CWR REPORTED OUTSIDE CONTINENTAL U.S.A. THROUGH 1978

		OXYACETYLENE				ELECTRIC FLASH				
		NEW		SECONDHAND		NEW		SECONDHAND		
RAILROAD	TOTAL TRACK MILES	MAIN TRK.	SIDING & YD.	MAIN TRK.	SIDING & YD.	MAIN TRK.	SIDING & YD.	MAIN TRK.	SIDING & YD.	TOTAL CWR ON PROPERTY
Algoma Central Ry.	322.00	-	-	-	-	-	-	-	-	-
Aroostook Valley RR Co.	25.00	-	-	-	-	-	-	-	-	-
Canadian National Rys.	22,247.00	10.00	-	-	-	5,510.00	-	143.00	125.00	5,788.00
Canadian Pacific Lmtd.	15,806.77	-	-	-	-	2,408.60	-	80.10	-	2,488.70
Ontario Northland Ry.	753.61	-	-	-	-	21.20	-	-	-	21.20
Quebec, North Shore & Lab	357.00	-	-	-	-	245.36	.25	15.21	19.60	280.42
Toronto, Hamilton & Buffalo Ry.	110.00	.58	-	-	-	13.96	8.77	-	-	23.31
Northern Alberta Rys. Co.	923.00	-	-	-	-	-	-	-	-	-
Totals		10.58	-	-	-	8,199.12	9.02	238.31	144.60	8,601.63

Report of Committee 5—Track

J. R. Masters, *Chairman*
R. J. Jones, *Vice Chairman*
C. W. Morrison, *Secretary*

A. J. Schavet
E. F. Pittman
C. R. Fulghum
E. H. Taylor
G. E. Fisher
R. G. Huston
W. B. Dwinnel
C. W. Alexander
R. W. Anderson
R. C. Arnlund
J. O. Born
S. W. Brunner
J. E. Campbell
N. H. Clark
R. E. Clemons
C. M. Cowles
E. D. Cowlin (E)
W. G. Crone
D. E. Crouser
D. R. Davis
E. R. English
E. E. Frank
R. G. Garlund
C. H. Gaut
S. W. George
C. A. Gerstner
P. E. Gordenier
W. R. Hamilton, Jr.
E. P. Hatten
E. E. Howard
J. D. Jardine
D. L. Jerman
R. A. Kendall
A. D. Kerr
G. G. Knupp
M. P. Kornspan
A. J. Kozak
R. E. Kuston
E. S. Laws
J. M. Letro
G. H. Maxwell
C. J. McConaughy (E)
M. E. McGinley
L. P. McKenna
H. V. Meek
B. Mohl
R. R. Morrish
T. C. Netherton
H. W. Newell
W. B. O'Sullivan
B. E. Pearson
L. A. Pelton
R. V. Perrone
C. E. Peterson (E)
S. H. Poore (E)
L. E. Porter
B. Post
L. C. Randolph
L. L. Rekuch
P. R. Richards
J. M. Salmon, Jr. (E)
R. N. Schmidt
V. M. Schwing
N. J. Skoutelas
H. E. Storey
S. K. Talukder
L. L. Tameling
R. L. Teeter
A. C. Trimble
A. F. Ubaldi
K. H. von Kampen (E)
W. J. Wanamaker
M. E. Wilson
A. M. Zarembski
A. J. Zierow

Committee

(E) Member Emeritus.

Those whose names are in boldface, in addition to the chairman, vice chairman and secretary, are the subcommittee chairmen.

To The American Railway Engineering Association:

Your committee reports on the following subjects:

B. Revision of Manual
a. Develop specification for rail anchors.
b. Review anchor patterns for bolted rail and CWR.
Progress Report, submitted as information.

2. Track Tools
a. Review inclusion of new and additional track tools.

No report. See Manual Recommendations published in the January-February 1980 Bulletin 676.

4. Track design, collaborating as necessary or desirable with Committees 1, 3 and 4.
b. Tie plates, including pads under plates.
(1) Evaluate canted tie plate design relative to 1:20 cant vs. 1:40 cant vs. 1:14 cant vs. 1:30 cant.
2. Evaluate tunnel and/or transit type tie plates.
c. Hold-down fastenings:
(1) Wood ties
No report.

5. Turnout and crossing design, collaborating as necessary or desirable with Committees 1, 3 and 4.
a. Review of guard rails protecting turnout frogs.
b. Review tolerances on plans 1010 and 1011.
c. Investigate use of epoxy on bolt threads for trackwork.
d. Investigate use of gage plates on turnouts to maintain gage.
e. Investigate use of cast iron or steel in lieu of manganese steel for heel extension on AREA 62 design railbound manganese frogs.
No report.

6. Track construction, collaborating as necessary or desirable with Committees 1, 3, 4 and 22.
a. Review specifications for track construction.
b. Review recommendations on joint spacing—centers vs. quarters.
No report. See Manual Recommendations published in the January-February 1980 Bulletin 676.

7. Track maintenance, collaborating as necessary or desirable with Committees 1, 3, 4 and 22.
a. Review requirements for rail lubrication.
b. Review use of gage rods.
Progress Report, submitted as information.

8. Criteria for track geometry design as related to modern equipment, collaborating as necessary or desirable with other AREA technical committees, and with the Engineering, Mechanical and Operating-Transportion Divisions, AAR.
a. Study minimum tangent lengths required between reverse curves with spiral and sup relevation.
b. Investigate design criteria for vertical curves.
No report.

The Committee on Track,
J. R. Masters, *Chairman*

Report On Assignment B

Develop Specifications For Rail Anchors

A. J. Schavet *(Chairman, Subcommittee)*, J. O. Born, E. R. English, A. B. Hillman, Jr., D. L. Jerman, W. B. O'Sullivan, L. A. Pelton, V. M. Schwing, S. K. Talukder, W. J. Wanamaker

This committee is working with various rail anchor manufacturers in an effort to develop a specification covering the manufacture and quality control of conventional drive on and spring type rail anchors. It is expected that a specification will be prepared and submitted for manual material next year.

MANUAL RECOMMENDATIONS

Committee 5 — Track

Report on Assignment 2

Track Tools

E. F. Pittman (*Chairman Subcommittee*), G. E. Fischer, R. G. Garland, R. G. Huston, J. M. Letro, T. C. Netherton, H. W. Newell, B. Post, L. L. Rekuch, R. L. Teeter

Your committee has been working with specifications and plans for track tools for several years as a result of reports received indicating the breaking of claw bars, spike mauls and the spalling of material from struck tools.

The existing specifications covering the manufacture of track tools have been found inadequate as there is no current specification covering the manufacture of percussion hand tools. Also, many of the percussion hand tools are made from carbon steel and the findings of this committee indicate that all percussion hand tools should be made from alloy steel. Carbon steel cannot be depth hardened sufficiently to provide a safe and durable steel. Therefore, it is recommended that all percussion tools be made from alloy steel.

Two grades of steel are noted on the recommended tool plans; Grade A which is basically the same alloy steel that some track tools are now being made from; Grade B which includes molybdenum for additional hardness and durability. These recommended specifications require a cleaner steel along with a new head contour design for tools made from Grade A alloy steel which will result in a stronger and safer tool than those now produced.

Changes in the design and specification for the spike maul were made due to the failure of some mauls in the neck area and due to the spalling of material from the striking face of the maul. The ring at the reduced neck area has been eliminated. A new head contour has been designed with recommendation that all spike mauls be made from alloy steel, preferably Grade B.

Changes in the design and specifications for the claw bar resulted from reported failures of the bar in the toe section and general dislike with the design. Plan No. 11-62 has only a 1-$\frac{1}{16}$" claw depth opening and the constant use enlarges this opening to an unacceptable width within a short period of time. Plan No. 11-78 has a 2-$\frac{1}{8}$" claw depth opening and a deeper heel section for better spike extraction.

Changes in the design and specifications for the double-faced sledge, track chisel, round track punch and the track spike lifter were made to provide a safer and more durable tool. It is recommended that these tools be made from alloy steel, preferably Grade B, with the new head contour design.

The nut cutter is a new tool to be included in the manual for use in removing "frozen" nuts from the track structure. In many instances, the track chisel is now used for nut removal which is not the intended purpose of this tool. It is recommended that the nut cutter be made from alloy steel, preferably Garde B, with the new head contour design.

Your committee recommends for adoption and publication in Chapter 5 of the manual, the following material to replace existing material on pages 5-6-1 through 5-6-5 and revised plans on pages 5-6-9 through 5-6-26.

This material covers Assignment 2, specifications for track tools.

Specifications and Plans for Track Tools

General

1.0 WORKMANSHIP

1.1 The steel used in the manufacture of all tools shall be free from pipe, porous centers, gross non-metallic inclusions or any other defects.

1.2 All tools shall be made in a workmanlike manner and shall be free from cracks, seams, laps and other injurious discontinuities.

1.3 Eyes of tools with handle holes must be on center and in true alignment.

2.0 FINISH

2.1 Percussion Tools

The body of the tools shall be unpainted. Ground working surfaces can be coated with a transparent lacquer type rust preventative.

2.2 Non-Percussion Tools

The body of the tool shall be coated with paint, oil or varnish to prevent corrosion, and each polished cutting edge shall be oiled or coated with a clear lacquer.

3.0 MARKING

Each tool shall be legibly marked with the manufacturer's name and trademark, coded as to production lot and the purchaser's initials. This identification shall be located in a position which will not interfere with the quality or performance of the tool and will not be removed by subsequent redressing.

4.0 INSPECTION

4.1 The inspector representing the purchaser shall have free entry, at all times while the work on the contract of the purchases is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector free of charge, all reasonable facilities and necessary assistance to satisfy him that the material is being furnished in accordance with these specifications. Tests and inspections shall be made prior to shipment at the place of manufacture unless otherwise specified.

4.2 The purchaser may make tests to govern the acceptance or rejection of the material in his own laboratory or elsewhere. Such tests shall be made at the expense of the purchaser.

4.3 Rejection—Material represented by samples which fail to conform to the requirements of these specifications will be rejected.

4.4 Material which, subsequent to test and inspection at the manufacturer's plant or elsewhere, and the acceptance shows injurious defects will be rejected and the manufacturer shall be notified.

5.0 SHIPMENT OR DELIVERY

Tools shall be properly packed for shipment to avoid damage. All bundles and boxes shall be plainly marked with the name of the purchaser, purchaser's order number, the name of the manufacturer, and the point of shipment.

6.0 WARRANTY

The manufacturer shall warrant that all tools are free from defects in material workmanship and heat treatment, that the tools meet all requirements of this specification, and that any defective tools will be replaced free of cost to the purchaser. Certified test reports may be requested at the purchaser's option.

Percussion Tools

7.0 SCOPE

7.1 This section of specification covers the contouring and metallurgical requirements for the manufacturing, ordering, inspection and acceptance of the following percussion tools.

7.1.1 Metal to Metal Contact Striking Tools

Double-faced sledge hammers (Plan No. 13-78). Spike Maul (Plan No. 3-78).

7.1.2 Metal to Metal Contact Struck Tools

Track chisels (Plan No. 17-78), nut cutter (Plan No. 35-77), round track punch Plan No. 19-78, Track Spike Lifter (Plan No. 32-78).

8.0 MANUFACTURE

8.1 Process—The shock resisting steel shall be made from electric furnace, vacuum degassed carbon deoxidized special quality fine grain size alloy bar.

8.2 Heat Treatment

8.2.1 Each tool classified in 7.1.1 and 7.1.2 shall be hardened by liquid quenching and subsequent tempering in such a manner that the hardness range specified in 8.3.2 will be maintained to a sufficient depth to absorb the normal working stresses. This heat treatment shall be such that a fracture test of the tool will exhibit a silky, fine grained appearance according to Shephard Standard No. 6 or finer.

8.2.2 All tools made with Grade B Steel to be redressed without subsequent heat treatment shall be initially heat treated so that the hardness specified in 8.3.2 is maintained to a depth from the end not less than the average cross sectional thickness. Tools made from Grade A Steel shall not be redressed without subsequent heat treatment.

8.3 Chemical and Hardness Requirements

8.3.1 All striking and struck tools (7.1.1 and 7.1.2) shall be made of shock resisting steel of Grade A or Grade B chemical composition with standard AISI residuals.

	Carbon		Manganese		Phos	Sul-fur	Silicon		Vanadium		Molyb-denum	
Gra	Min	Max	Min	Max	Max	Max	Min	Max	Min	Max	Min	Max
A	.56	.64	.75	1.00	0.25	0.25	1.80	2.20				
B	.51	.60	.75	1.00	0.25	0.25	1.80	2.20		.45	.35	.50

8.3.2 Hardness—All hardness tests shall be performed according to ASTM Spec. E-18. Frequency of testing should be performed according to the requirements in MIL-STD-105D, "Military Standard-Sampling Procedure and Tables for Inspection by Attributes."

8.3.2.1 All struck surfaces shall be 44/48 Rockwell "C" hardness.

8.3.2.2 All striking surfaces shall be 51/55 Rockwell "C" hardness.

8.3.2.3 All cutting surfaces shall be 56/60 Rockwell "C" hardness.

8.3.2.4 All punch ends shall be 52/56 Rockwell "C" hardness.

8.4 Hardenability

8.4.1 Grade A Steel—Composition of the steel shall be such that in the standard Jominy test the hardness is greater than 40 Rockwell C at $\frac{9}{16}$ inch from the quenched end of the specimen.

8.4.2 Grade B Steel—Composition of the steel shall be such that in the standard Jominy test the hardness is greater than 50 Rockwell C at $\frac{8}{16}$ inch from the quenched end of the specimen.

8.4.3 Frequency of Testing—The steel manufacturer shall have conducted a Jominy test from the first, middle, and last ingot of each heat of steel purchased.

8.5 Microscopic Inclusion Evaluation

8.5.1 Grade A and B steel shall meet the following requirements for inclusions.

8.5.2 Test Specimen—Speciments shall be prepared from approximately 4 inch (100-mm) forged, square section taken from the top and bottom of the first, middle and last ingot. The specimen shall be $\frac{3}{8}$ by $\frac{3}{4}$ inch (9.5 by 19 mm) and shall be taken from an area midway between the center and outside of the test section. Procedures outlined in Methods E 45 shall be followed.

8.5.3 Examination and Limits-Specimens shall be examined in accordance with Method E 45, Method D, using the modified JK Chart, Fig. 12 of Plate III. The worst field in any specimen shall not exceed the following limits:

	A	B	C	D
Thin	3.0	3.0	2.5	2.0
Thick	2.0	2.5	1.5	1.5

8.6 Nondestructive Test Requirements

8.6.1 To insure that all tools are free from the defects listed in 1.0, each tool shall be inspected *after finish grinding* by the supplier according to one of the following procedures.

8.6.1.1 Magnetic Particle Inspection in accordance with ASTM Method A-275.

8.6.1.2 Liquid Penetrant Inspection in accordance with ASTM Recommended Practice E-165.

9.0 DESIGN

9.1 All tools shall conform substantially when applicable to the dimensions here set forth. Dimensions for head contours as shown on Page 5-6-28, sheets A, B, C will conform to the following:

9.1.1 Head Contour

9.1.1.1 Heads of tools with a round cross section shall be ground to the corner contours prescribed in sheets A, B, C.

9.1.1.2 Heads of tools with a hexagonal or octagonal cross section shall also be ground to the corner contours prescribed in sheets A, B, or C. In addition, the arcs not tangent to the hexagonal or octagonal corners shall be "blended" into a smooth contour similar to that shown on sheet C.

9.1.1.3 Punch ends shall have corner radii according to 9.1.1.1, but with no crown radius.

9.1.1.4 All ground surfaces shall be free of decarburization.

Non-Percussion Tools (Inspection and Physical Tests)

10.0 Clay Pick—Plan No. 1

No special tests required.

11.0 Tamping Pick—Plan No. 2

No special tests required.

12.0 Track Wrenches—Plan No. 4

One wrench to be tested from each lot of 10 dozen or less by applying for 1 minute a load of 400 lb. at a point distant from the jaw end equal to 95 percent of the total length of the wrench without any spreading of the jaw or any permanent set in the handle.

13.0 Lining Bars—Plan No. 5

One bar to be tested from each lot of 10 dozen or less by applying a load of 350 lb. 9 in. from the end of the handle, with the point suitably secured 6 in. from the end, without leaving a permanent set in excess of 1/4 in.

14.0 Rail Tongs—Plan No. 6

No special tests required.

15.0 Tie Tongs—Plan No. 7

No special tests required.

16.0 Timber Tongs—Plan No. 8

Three pairs of tongs to be tested from each lot of 10 dozen or less by suspending a load of 300 lb. to 400 lb. workwise in the tongs with the handles in a horizontal position and supported at a point 2 in. from the end. Deflection with 300 lb. weight shall not exceed 1 in. with no permanent set, and with 400 lb. weight deflection shall not exceed 1-$\frac{1}{4}$ in. with a permanent set of not to exceed $\frac{1}{8}$ in.

17.0 Spike Puller—Plan No. 9

Sample of each lot manufactured to be tested in actual use by pulling a spike with a standard claw bar.

18.0 Rail Fork—Plan No. 10

No special tests required.

19.0 Claw Bar—Plan No. 11

One bar from each lot of 10 dozen or less to be tested by placing the claws of the bar $\frac{1}{2}$ in. under the head of a standard track spike, rigidly placed and so located as to hold the bar in a horizontal position while a shock load equivalent to that of a 200 lb. weight falling a distance of 1 ft. is applied to the handle at a point 5 in. from its end, without the toes showing any cracks or the handle taking any permanent set.

20.0 Track Adz—Plan No. 12

Test one adz in each lot of 10 dozen or less by subjecting cutting edge to 5 normal blows on metal of the same composition as a railroad spike without breakage or serious nicking.

21.0 Carpenter's Adz —Plan No. 12A

No special tests required.

22.0 Tamping Bar—Plan Nos. 14-15

No special tests required.

23.0 Track Gage—Plan No. 20

No special tests required.

24.0 Track Gage with Wood Rod—Plan No. 20-A

No special tests required.

25.0 Track Shovel—Plan 21

25.1 Scope and Design

This specification covers the welded or riveted type and the solid shank type with either wood, malleable iron or combination wood metal handle tops. Dimension shall conform to plans, which are made a part of this specification. A variation of $\frac{1}{2}$ in. more or less from the dimension shown on the plan for the length of the strap or shank and handle will be allowed. A variation of $\frac{1}{4}$ in. more or less from the dimensions shown on the plan for the width or length of the blade will be allowed, but the total variation in the overall length of shovels shall not exceed $\frac{1}{2}$ in. more or less of the dimensions shown on plan.

25.2 Materials

Blades shall be of carbon or alloy steel, with a Rockwell (Rc) hardness for carbon steel of 45 to 50.

Carbon-steel blades shall have a thickness of not less than No. 13 gage and alloy blades shall be of not less than No. 14 gage U.S. standard, the gage to be measured at the point where the hardness is taken. For welded or riveted types, the straps shall be welded or riveted to the blade.

25.3 Handles and Tops

Handles shall be made of ask and shall conform to Grade AA as set forth in the general Specifications for Handles for Track Tools. The tops of handles shall be of the design specified and shall conform to plans that are made a part of this specification.

25.4 Tests

One shovel from each lot of 10 dozen or less shall be selected and metal straps (curved to fit the contour of the handle) shall be clamped to the upper and lower parts of the handle, after which the shovel shall be placed in a prying position, supported at the end of the blade by clamps and shall be capable of sustaining a load of 200 lb. suspended from the end for a period of 2 minutes without showing any permanent set, fracture or distortion.

Alloy steel shovels which have been given a heat treatment to insure uniformity in hardness shall be subjected to a shock test to insure against brittleness. The test shall be made by forcibly striking the blade of the shovel with a hand hammer at several places when placed on an anvil.

26.0 Ballast Fork - Plan No. 22

26.1 Scope and Design

The dimensions shall conform to the plans, which are made a part of this specification. The total variation in the overall length of the forks shall not exceed 1/2 inch more or less of the dimensions shown on plan.

26.2 Material

Forks shall be made of high-grade carbon steel. Tines of forks shall show Rockwell (Rc) hardness of 35-45. Straps shall be 0.04 U. S. Standard gage steel.

26.3 Handles

This specification covers either wood, malleable iron or combination wood-metal handle tops. Handles shall be made of ash and shall conform to Grade AA and be in accordance with the general Specification for Handles for Track Tools.

27.0 Track Tool Handles - Plan No. 25

No special tests required.

28.0 Rail Tongs for use with crane - Plan No. 31. In the manufacture of the Rail Tongs, Section 8.6, Nondestructive Test Requirements will be adhered to.

29.0 Drive Spike Extractor Socket Wrench - Plan No. 33

No special test required.

PLANS FOR TRACK TOOLS
1978

Plan Number	Description	Grade of Steel	Hardness
1-62	Clay Pick	Carbon or Alloy	425-500 BHN
2-62	Tamping Pick	Carbon or Alloy	425-500 BHN
3-78	*Spike Maul	Alloy, Grade A or B	51-55 Rc
4-62	Track Wrenches	Carbon	375-450 BHN
5-62	Lining Bars	Carbon	300-375 BHN
6-62	Rail Tongs	Carbon	
7-62	Tie Tongs	Carbon or Alloy	
8-62	Timber Tongs	Carbon or Alloy	
9-62	Spike Puller	Carbon	375-450 BHN
10-62	Rail Fork	Carbon	275-350 BHN
11-78	Claw Bar	Carbon	300-375 BHN
12-62	*Track Adz	Carbon or Alloy	375-450 BHN
12-A-62	*Carpenter's Adz	Carbon or Alloy	
13-78	*Double-Faced Sledge	Alloy, Grade A or B	51-55 Rc
14-62	Chisel End Tamping Bar	Carbon	425-500 BHN
15-62	Spear End Tamping Bar	Carbon	425-500 BHN
16-62	Tie Plug Driver	Carbon	
17-78	*Track Chisel	Alloy, Grade A or B	44-48 RC (head) 56-60 Rc (Point)
19-78	*Round Track Punch	Alloy, Grade A or B	44-48 Rc (head) 52-56 Rc (punch end)
20-62	Track Gage-Pipe Center	See Plan	
20-A-62	Track Gage-Wood Center	See Plan	
21-62	Track Shovels	Carbon or Alloy	45-50 Rc
22-62	Ballast Forks	Carbon	35-45 Rc
25-78	Track Tool Handles		
26-62	Scoop	Carbon or Alloy	45-50 Rc
27-68	Aluminum Combination Track Level & Gage (Insulated)		
28-62	Scythe	See Plan	54-58 Rc
29-62	Snatch	See Plan	
30-62	Spot Board	See Plan	
31-78	Rail Tongs for Use with Crane	See Plan	
32-78	Track Spike Lifter Grade B	Alloy, Grade A 44-48 Rc (Claw)	44-48 Rc (Head)
33-62	Drive Spike Extractor Socket Wrench	Carbon	300-350 BHN
34-71	Rail Thermometer		
35-77	Nut cutter	Alloy, Grade A or Grade B	44-48 Rc (Head) 56-60 Rc (Point)

* When specified, the small-eyed track tools will be furnished with AREA handles. The handles are to be properly fitted and wedged; 36-inch handles are to be furnished for tools

weighing 6-lb and over, and 16-inch handles for tools weight under 6-lb., except that 22-inch handles are to be furnished for track chisels, tie plug punch, and round track punch.

Chemical Specification for Carbon Steel Track Tools

+Carbon	0.55 to 0.70	Phosphorous	0.05 max.
Manganese	0.60 to 0.90	Sulfur	0.05 max.

+Applies to all carbon steel tools

Chemical Specifications for Alloy Steel Track Tools, Grade A & B

	Carbon		Manganese		Phos	Sulfur	Silicon		Vanadium		Molybdenum	
Gra	Min	Max	Min	Max	Max	Max	Min	Max	Min	Max	Min	Max
A	.56	.64	.75	1.00	0.25	0.25	1.80	2.20				
B	.51	.60	.75	1.00	0.25	0.25	1.80	2.20		.45	.35	.50

Report on Assignment 6-a
Track Construction

G. E. Fischer (*Chairman, Subcommittee*), D. R. Davis, C. R. Fulghum, C. A. Gerstner, M. P. Kornspan, E. S. Laws, R. R. Morrish, R. V. Perrone, E. F. Pittman, L. E. Porter, P. R. Richards, E. C. Rudolph, R. L. Teeter, A. M. Zarembski

Your committee recommends for adoption and publication in Chapter 5, the following material to replace existing material on Pages 5-4-1 through 5-4-6.

2. Track construction may be performed in two distinct manners:

(a) employment of contractor.

(b) employment of railroad track forces. If the latter manner is used, replace subsequent references to contractor with railroad track forces.

2.1 The railway company's authorized representative shall arrive at a clear understanding with the contractor as to the force to be employed and the speed with which the work shall proceed. Prior to starting the work, the contractor shall notify the railroad company's representative at least five working days in advance so that adequate arrangements can be made for the prosecution of the work.

3. (a) The railway company will furnish track materials on cars or on the ground in the material yard and or

(b) The contractor will supply and transport track materials to the job site and all material shall be subject to the approval of the railway company.

6. The contractor shall supply the necessary supervision and labor to prosecute the work properly and in such numbers as may be required by the railroad's chief engineer or his authorized representative, and at the request of the chief engineer or his representative will remove any supervisor or man not satisfactory to the railway company.

10. Railway company will determine size of tie to be used and type of timber acceptable.

10.1 Ties shall be placed in the track with the wide surface nearest the heart down and square to the line of the rail.

11. When necessary the ties must be adzed to get a full and even bearing for the tie plate. Excessive adzing must be avoided. All newly adzed surfaces shall be coated with an approved preservative.

12.1 Tie plates will be used under running rails on all tracks.

12.2 Tie plates should be free of dirt and foreign material when installed.

12.3 Care must be exercised to see that canted tie plates are applied so as to cant the rail inward.

12.4 Tie plates must be placed square with the rail and centered on the tie. Particular care must be given to see that the tie plate shoulders are never under the base of the rail and that the tie plates are well seated on the ties and the rail properly seated on the tie plate.

13. Ties shall be spiked with two rail-holding spikes on each rail and with additional rail-holding and plate-holding spikes as specified by the railway. Other railway approved fastening devices must be used.

14. All cut spikes shall be started and driven vertically and square with the rail and so driven as to allow 1/8 in to 3/16 in space between the under side of the head of the spike and the top of the base of the rail. In no case shall the spikes be overdriven, or straightened while being driven.

16.1 Spikes on gage side of running rail to be placed across from each other and spikes on the field side of the running rail are to be placed across from each other. The pattern to be held consistent.

16.2 Switch ties will be placed in turnouts and crossovers as shown on AREA Track Work Plans unless otherwise specified by Railway.

20. All rail fastening systems for concrete ties shall be installed per manufacturer's specifications. Any exposed metal components shall be protected against corrosion.

20.1 Use of concrete ties under joints should be avoided.

22. When laying jointed rail, approved expansion shims shall be used to provide the proper opening between rails, and a rail thermometer shall be used to determine the thickness of shims in accordance with the recommendations in Temperature Expansion for Laying Rails, Part 5, This Chapter.

26.1 Prescribed corrosion resistant lubricant shall be applied to bolts by supplier or prior to installation.

26.2 Track bolts will be retightened within an appropriate period after track has been put into service, as determined by the railway.

29. Rail joints will not be placed in road crossings or within the limits of switch points or guard rails.

29.1 A lubricant shall be applied on the rail within the area of the joint bar at time of installation.

29.2 Rail joints shall be applied so that bars are not cocked between base and head of rail. Bars to be properly seated in rail.

31. Rail anchors shall be applied in accordance with Rail Creepage—Number and Position of Rail Anchors, Part 5, this chapter, unless otherwise specified by the railway. Rail anchors pattern shall be spaced approximately uniformly along the rail length. To avoid tie skewing, the anchors must be applied against the same tie on opposite rails. Rail anchors when applied must have full bearing against a sound tie.

33. All switches, frogs and guard rails shall be placed in accordance with the proper plan of the AREA Trackwork Plans, unless otherwise specified by the railway.

33.1 Switches shall be left in proper adjustment, special care being given to the bending of the stock rail.

34. Drainage of the roadbed is necessary before good track can be secured or maintained. It is of the first importance that drainage be given careful detailed consideration at all points.

34.1 Cross drains shall be installed wherever necessary to obtain proper drainage.

34.2 To prevent water from coming over the top of cuts, interception ditches must be constructed to carry the water along the top of the cut and drain into a water course at the ends of the cut.

34.3 Side ditches along the track shall be constructed to a grade that will permit water to flow freely and not form pools and seep into the roadbed. Ditches must be examined frequently and cleared of obstacles interfering with the free flow of water.

41. Traffic shall not be permitted upon the newly constructed track section until the track has been accepted by the railroad or upon receipt of a written order from an appropriate representative of the railroad.

44. The contractor is to understand that any work not specifically mentioned in the specification, but which is necessary, either directly or indirectly, for the proper carrying out of the intent thereof, shall be required and applied, and he shall perform all such work just as if it were particularly defined or described. Unless specifically mentioned above, all work shall conform to the standards of the railway company.

All of the other paragraphs of this section will remain as printed in the current manual.

Your committee recommends for adoption and publication in Chapter 5 of the manual, the following material to replace Appendix 1—Where Track Is Constructed With Continuous Welded Rail (CWR) on Page 5-4-5.

3. Prior to laying of CWR, the maximum and minimum rail temperatures in the areas shall be determined and recorded by railway.

4. It is the railway company's responsibility to establish the desired laying temperature. The Contractor shall record the temperature of each rail laid. When it is not possible to lay rail at the desired laying temperature, the contractor shall make the necessary adjustment at a later date. All adjustments shall be made as per instructions of a railway company representative. Buffer rails, in any length between 30 and 37 feet, may be used if permitted by the railway. (For recommended practices, refer to Page 5-5-4.4 in AREA Manual on Temperature Laying of CWR).

7. A string of CWR should not end on the deck of an open-deck bridge nor may it be less than a minimum distance specified by the railroad from the face of the backwall on the at-grade side.

8. The contractor should apply all rail anchors immediately behind the laying of CWR.

The rail should be anchored in accordance with the AREA recommendations contained in Rail Creepage—Number and Position of Rail Anchors (Continuous Welded Rail), Part 5, this Chapter. Ballast must be unloaded and all cribs filled as soon as rail anchors have been applied. The track will be surfaced and tamped as soon as possible after the laying of the CWR.

All of the other paragraphs of this Appendix will remain as printed in the current manual.

Report on Assignment 7

Track Maintenance

R. G. Huston (*Chairman, Subcommittee*), J. O. Born, J. E. Campbell, N. H. Clark, E. E. Frank, R. G. Garland, C. H. Gaut, S. W. George, E. E. Howard, J. D. Jardine, G. G. Knupp, G. H. Maxwell, B. Mohl, C. W. Morrison, T. C. Netherton, B. E. Pearson, A. C. Trimble, A. J. Zierow

(a) Lubrication of Rail on Curves

(b) Gage

Your committee has reviewed and rewritten these subjects and after a letter ballot, has submitted same for publication by bulletin and subsequent adoption as Manual material next year.

(h) Tamping

This is a new project which your committee started work on during the meeting in October and will try to finalize during the meeting in March, 1980.

(i) Ballast Compaction

This is also a new assignment agreed upon by your committee and preliminary format will be started at the March, 1980 meeting.

Report on Assignment 7-a

Track Maintenance

R. G. Huston (*Chairman, Subcommittee*), J. O. Born, J. E. Campbell, N. H. Clark, E. E. Frank, R. G. Garland, C. H. Gaut, S. W. George, E. E. Howard, J. D. Jardine, G. G. Knupp, G. H. Maxwell, B. Mohl, C. W. Morrison, T. C. Netherton, B. E. Pearson, A. C. Trimble, A. J. Zierow

Your committee recommends for adoption and publication in Chapter 5, Part 5 of the manual, the following material to replace existing material on Page 5-5-10.

This material covers Assignment 7-a, Lubrication of Rail on Curves.

Suggested Material to Replace Manual Material on Page 5-5-10:

Lubrication of Rail on Curves

5-5-10.1 The life of the high or outside rail in curves can be extended through lubrication of the contact area between wheel flanges and side of rail head.

5-5-10.2 Lubrication can be accomplished through the use of wheel actuated mechanical devices which dispense lubricant to passing wheel flanges for rolling distribution, or, in special cases, manual application of lubricant may be necessary.

5-5-10.2 Specially compounded lubricants are available for use in mechanical lubricators.

Note: Care must be taken to assure that lubricant is deposited only to the gage side of the rail head. Excessive lubricant, especially in the wheel-tread-rail-head contact area, may have a detrimental effect on train handling.

5-5-10.4 In addition to reducing the wheel flange cutting on the side of the rail head, lubrication accomplishes the following:

10.4.1 Wheel (flange) wear is reduced on rolling stock.

10.4.2 The tendency for wheel climbing is reduced.

10.4.3 Train resistance is reduced, permitting increased train tonnage, better time, or less fuel consumption.

10.4.4 Longer service life for ties with less regaging of rails.

5-5-10.5 Economic justification for lubrication depends upon the estimated annual saving resulting, as compared with the total estimated annual cost of lubrication.

Report on Assignment 7-b

Track Maintenance

R. G. Huston (*Chairman, Subcommittee*), J. O. Born, J. E. Campbell, N. H. Clark, E. E. Frank, R. G. Garland, C. H. Gaut, S. W. George, E. E. Howard, J. D. Jardine, G. G. Knupp, G. H. Maxwell, B. Mohl, C. W. Morrison, T. C. Netherton, B. E. Pearson, A. C. Trimble, A. J. Zierow

Your committee recommends for adoption and publication in Chapter 5, Part 5 of the manual, the following material to replace existing material on Page 5-5-7.

This material covers Assignment 7-b, Gage.

Suggested Material to Replace Manual Material on Page 5-5-7:

Gage

5-5-7.1 The gage tool shall indicate standard track gage.

5-5-7.2 The rail shall be held to gage while line spikes are being driven.

7.2.1 The rail shall be properly seated in the tie plates with the edge of the rail base and the field shoulder of the tie plates aligned and in contact.

7.2.2 A minimum of two rail holding spikes is required. These spikes shall be so staggered that all outside spikes are on the same side of the tie and inside spikes on the opposite side of the tie.

7.2.3 The rail and tie plates shall be spiked to each tie in accordance with the standard of the railway.

5-5-7.3 Within proper limits, a slight variation of gage from the standard is not seriously objectionable, provided that the variation is uniform and constant over long distances. For new track construction, see Manual Chapter 5, Part 4.

5-5-7.4 Wide gage, due to rail worn within permissible limits, shall be corrected by regaging or by interchanging the low and high rails, or by replacing the rail.

5-5-7.5 Under ordinary conditions, where speed does not dictate otherwise, it is not necessary to regage track if the increase in gage is not more than $\frac{1}{2}$ inch (12 millimeters) provided such increase is uniform.

7.5.1 Old spike holes should be plugged when regaging.

5-5-7.6 Gage rods, gage plates, rail braces, or inner guard rails may be used on curves where it is difficult to maintain gage.

Report of Committee 6—Buildings

E.P. Bohn
Chairman
T.H. Seep
Vice Chairman
O.C. Denz
Secretary
R.E. Phillips

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W.F. Armstrong
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J.J. Brandimarte
G.J. Chamraz
J.A. Comeau
F.D. Day
C.M. Diehl
R. Evans
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C.S. Graves (E)
A.R. Gualtieri
R. Hale
W.G. Harding (E)
H.R. Helker
K.E. Hornung
W.C. Humphreys
K.N. Kerns
C.R. Madeley
R.J. Martens
J.N. Michel
L.A. Palagi
T.F. Peel
D.C. Pharant
R.D. Powrie
J.G. Robertson
H.A. Shannon, Jr.
J.E. Smith
J.H. Smith
J.S. Smith
R.E. Smith
W.C. Strum
S.G. Urban (E)
W.M. Wehner
T.S. Williams (E)

Committee

(E) Member Emeritus.

Those whose names are shown in boldface, in addition to the chairman, vice chairman and secretary, are the subcommittee chairmen.

To the American Railway Engineering Association:

Your committee reports on the following subjects:

B. Revision of Manual.

Current assignment is the revision of Chapter 6, Part I of the manual to conform with the Construction Specifications Institute (CSI) format. Rough draft of this assignment was reviewed and discussed at the June, 1979 meeting. Chairman noted recommended changes and will correct and expand the assignment for further review at the November, 1979 meeting. I anticipate that this report will be submitted as manual information in 1980.

1. Design criteria for freight forwarding facilities.

This assignment has been approved and was published as information in Bulletin 674, September-October 1979. It is proposed to continue this assignment for revision as manual material. Hopefully we will be able to get a majority vote at our June 1980 meeting and submit to letter ballot in time to meet our November 1, 1980 filing date.

2. Inspection of railway buildings.

The chairmanship of this committee changed in early 1979 and at the June meeting the new Chairman recommended that the report be handled in two separate sections and the title of the first section be changed as written above. The proposal was put in the form of a motion, seconded and approved by majority of members present. A preliminary report was prepared, and the first section should be ready for submission as information in 1981.

3. Design criteria for locomotive load test compartments.

This assignment has proven to be quite technical from a mechanical engineering standpoint and controversial because of proposed regulations governing noise emission standards for transportation equipment interstate proposed by EPA and the many state and Local Codes, Ordinances, agencies such as OSHA, etc. having jurisdiction. The chairman has gathered much information on this subject and prepared a revised and expanded report for the November 1979 meeting. It is the concensus of the Committee that this assignment be progressed with caution. It has become a learning experience for all involved. It is anticipated that this report will be submitted for publication as information in 1981.

4. Roofing Systems

This new subject was proposed at the June meeting. The Committee feels a need for study and research in this area in order to bring the railroad industry up to date on the many new and innovative roofing systems now available for use in new construction and maintenance. Rough draft should be prepared by June, 1980, estimate 1981 publication.

5. Architectural Education-Student Design Contest 1981.

The current assignment of this subcommittee is to conduct the architectural design competition, judging of competition and presentation of awards at the 1981 Annual Technical Conference.

6. Building Design for Energy Conservation.

The Chairman submitted a fifty-page report on "Design for Physically Handicapped" at our June, 1979 meeting. The controversial nature and need for this report was discussed at length and finally a motion was made that the subject be dropped, the motion was seconded and approved by majority of the members present, subject to board approval.

The new subject this committee would like to undertake is "Building Design for Energy Conservation".

The Committee on Buildings,
E. P. Bohn, *Chairman*

Report of Committee 7—Timber Structures

J.A. Gustafson
Chairman
J. Budzileni
Vice Chairman
J.W. Chambers
Secretary

T.E. Markvaldas
D.C. Meisner

J.H. Huzy
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W.L. Anderson
T.E. Brassell
D.W. Brookings
J.C. Calhoun
A.J.S. Carr
G.K. Clem
M.J. Crespo
A.R. Dahlberg
H.E. Dearing
K.L. De Blois (E)
D.J. Engle
R.W. Ezelle, Sr.
S.L. Goldberg
D.C. Gould
W.J. Gunkle
J.A. Hawley

J.M. Helm
L.H. Kelley
W.C. Kirkland
D.I. Kjellman
H.G. Kriegel
L.R. Kubacki
R.E. Kuehner
C.V. Lund (E)
R.F. Mc Guire
R.C. Mc Master
C.H. Newlin (E)
W.A. Oliver (E)
R.P. Rasho
D.V. Sartore
F.E. Schneider (E)
G.N. Sells
J.W. Storer
W.A. Thompson
N.E. Whitney
A.P. Yannotti

Committee

(E) Member Emeritus.

Those whose names are shown in boldface, in addition to the chairman, vice chairman and secretary are the subcommittee chairmen.

To the American Railway Engineering Association:

Your committee reports on the following subjects:

2. Grading rules and classification of lumber for railway uses; structural timber collaborating with other organizations interested.

   Mr. D. C. Meisner is the new chairman of this subcommittee. A great deal of work has taken place in previous years to assemble the data required to revise the stress tables but each time, prior to approval by the committee as a whole, the Industry, through its grading bureaus, has changed some basis stresses. We have been told that agreement has now been reached and new tables are being prepared for the subcommittee's approval. We expect approval by the whole committee prior to the complete revision of the Manual.

3. Specifications for design of wood bridges and trestles.

   The chairman of this subcommittee, Mr. J. H. Huzy, has been very active in reviewing each item of our specification that deals with design. In doing this an attempt has been made to break the portions into manageable parts. To date, about 130 items have been sent to the subcommittee for review and comment. The majority of these items are being resubmitted to the subcommittee as a composite of comments received. This will then become part of the material submitted by Subcommittee B in their reorganization of the Manual.

5. Design of glued bridges and trestles.

The chairman of this subcommittee, Mr. R. C. Moody, has submitted to the committee as a whole a complete revision of the material for the design of glued laminated members including tables that follow current industry standards. It is expected that this will be approved and ready for the Manual by next year.

7. Repeated loading of timber structures.

Mr. W. S. Stokely, subcommittee chairman, is currently reviewing the data from testing that was done some years ago at the AREA laboratory and together with his subcommittee will determine how this material can be reflected in our specifications. There has been controversy as to the validity of the test which should be resolved in the near future so that a report may be written.

8. Protection of pile cut-offs; protection against marine organisms by means other than by preservatives.

Subcommittee 8 is now preparing questions and a mailing list of persons and institutions active in Marine Pile Protection and Maintenance for a questionnaire regarding current methods being used to protect Marine piling. The amount of borer attack and effectiveness of protection will also be covered in the questionnaire. The Subcommittee chairman, Mr. B. J. King, is an active member of the "West Coast Committee on Marine Barriers", an informal group interested in protection of Marine Piling, that meets annually to exchange information relative to the use of barriers to protect Marine Piling and related subjects. This committee provides an important source of information regarding current uses and innovations of Marine Pile Protection, such as the recent trend in the use of heavy polyethylene (150 to 190 mil) barriers which will protect against both borer attack and abrasion.

The Subcommittee's work on protection of pile cut-offs has been limited by lack of information regarding the different methods of protection being utilized. However, they have recently learned of a research project on Pile Cut-off Protection sponsored in part by Oregon State University Sea Grant College Program, supported by the Department of Commerce. A test group of 198 piling with various cut-off protection was installed in 1976, and is being monitored. Correspondence is being initiated with the University for information regarding the research work, which should supply excellent information for a Subcommittee report.

Completion of "State of the Art" report should be forthcoming in approximately one year.

9. Study of in-place preservative treatment of timber trestles.

There has been a recent change in the chairmanship of this Subcommittee with the appointment of R. W. Thompson, Jr. With the large number of timber structures in use the committee continues to review new products that are developed and attempts to evaluate the effectiveness of products currently in use. If treating schedules can be arranged we will visit a bridge that is being treated in the fall of 1980 with the Board's permission.These products now have longer service life and could be reevaluated by laboratory analysis to determine their effectiveness. We would request that monies be made available for this type of evaluation.

The Committee on Timber Structures,
J. A. Gustafson, *Chairman*

Report of Committee 8—
Concrete Structures and Foundations

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J.W. Weber
J.O. Whitlock
J.M. Williams
J.R. Williams
W.R. Wilson (E)
J.L. Youngblood

Committee

(E) Member Emeritus.

Those whose names are shown in boldface, in addition to the Chairman, vice chairman and secretary are the subcommittee chairmen.

To the American Railway Engineering Association:

Your Committee reports on the following subjects:

B. Revision of Manual.

"Part 23-Pier Protection Systems at Spans over Navigable Streams" and "Part 5-Retaining Walls and Abutments," were presented as Manual material in AREA Bulletin

674, September-October 1979. "Part 10-Reinforced Concrete Culvert Pipe" is presented for manual material on page 235.

1. Design of Masonry Structures

   (a) Part 2 - Reinforced Concrete Design was published in Bulletin 670 as a Manual Recommendation. This assignment is complete.

   (b) Investigate and report on precast and/or prestressed components for use in construction, maintenance, or repair and restoration of railway bridges, trestles and culverts. Subcommittee 1 was given this new assignment in February 1979 and anticipates having a report to be published as information ready by fall of 1980. The report will be on the "State of the Art" and the information should be of special interest and usefulness to bridge engineers and others seeking information on precast and/or prestressed concrete elements for construction and maintenance of railway bridges and structures.

   (c) Part 1 - Concrete and Reinforced Concrete Railroad Bridges and Other Structures. Subcommittee 1 has started work on rewriting Part 1 to update the material in Part 1, much of which is out of date. No date has been set for completion of this assignment.

2. Foundations and Earth Pressures

   (a) Part 4 - Pile Foundations was published in Bulletin 670 as a Manual Recommendation. This assignment is complete

   (b) Part 10 - Reinforced Concrete Culvert Pipe. Part 10 is being submitted as a Manual Recommendation.

   (c) Subcommittee has started work on updating Part 6 - Crib Walls. No date for completion has been set.

   (d) Subcommittee had started work on earthquake loadings. The assignment was dropped when a warning to designers in new Part 2 was published. The topic is too complex for the average engineer not familiar with earthquake loadings and the seismic response of soils.

3. Waterproofing for Railroad Structures

   (a) Subcommittee Chairman, L. A. Kuston accepted the appointment at the June 1979 meeting of Committee 8. Several new members and a new associate member with experience and expertise in the field have joined the Subcommittee. The Subcommittee has started work on rewriting Chapter 29 - Waterproofing. New Chapter 29 is to be based on type or degree of waterproofing required for protection and a performance specification as a guide line. No completion date has been set.

5. Strengthening existing mansory structures, and repair and restoration of existing masonry structures to restore original structural capacity and durability

   (a) The Subcommittee will handle this material as a new Part of Chapter 8. There will be need to update or incorporate existing Parts of Chapter 8 into this new Part. Tentative outline adopted by the Subcommittee breaks the project into five sections. The Subcommittee has been divided into four groups with a leader who will develop a scope for each section. The fifth section will consist of typical details and will be developed after the first four sections are drafted. No target date for completion has been set as the Subcommittee just got started at the June 1979 Committee meeting.

The Committee on Concrete Structures and Foundations,
W. E. Brankensiek, *Chairman*

Manual Recommendations

Committee 8—Concrete Structures & Foundations

Part 10—Reinforced Concrete Culvert Pipe

Specifications for the Placement of Reinforced Concrete Culvert Pipe

G.W. Cooke (Chairman, subcommittee)

10.1 SCOPE

These specifications cover the placement of reinforced concrete culvert pipe for railway culverts.

10.2 PREPARATION OF SUBGRADE

10.2.1 Excavation

Trenches shall be excavated in accordance with bank stability requirements to a width sufficient to allow for proper jointing of the pipe and thorough compaction of the bedding and backfill material under and around the pipe. Where feasible, trench walls shall be vertical. A maximum trench width in conformance with the design assumptions, should be specified on the construction plans. Wide trenches generally require the use of stronger pipe. The completed trench bottom shall be firm for its full length and width. Where required, the trench shall have a longitudinal camber of the magnitude specified on the plans.

When so specified on the plans, excavation for pipe to be placed in embankment fill shall be made after the embankment has been completed to the specified or directed height above the pipe.

10.2.2 Foundation

10.2.2.1 If the foundation is soft or spongy, an adequate support shall be supplied by excavating the unstable soil and backfilling with firm material, or by such other means as may be specified or approved by the engineer.

10.2.2.2 If the foundation is muck, or similarly yielding material, the pipe shall be supported on piling, or by such other means as may be specified or approved by the engineer.

10.2.2.3 For Class B and C Beddings, subgrades should be excavated or over excavated, if necessary, so a uniform foundation free of protruding rocks may be provided. Special care may be necessary with Class A or other unyielding foundations to cushion pipe from shock when blasting can be anticipated in the area.

10.3 PIPE INSTALLATION

10.3.1 Laying Pipe

Pipe laying shall begin at the downstream end of the culvert. The bell or groove end of pipe shall be placed facing upstream. No culvert shall be placed in service until a suitable outlet is provided. Elliptical and elliptically reinforced pipe shall be placed with the vertical axis within 5 degrees of a vertical plane through the longitudinal axis of the culvert.

10.3.2 Bedding

Pipe bedding and placement shall conform to one of the Bedding Classes illustrated in Figures 1 through 5 as specified or approved by the engineer. When the pipe cannot be placed on Class A, B or C bedding but, instead, must be placed on an unprepared surface, it shall be considered as having Class D Bedding. Class D bedding should normally be used only for emergency work, and is not permitted for permanent installations. For typical Class D bedding see Figure 1 and 3.

10.3.3 Joining Pipe

Pipe may be bell and spigot or tongue and groove design unless otherwise specified. When bell pipe is used, a shallow excavation shall be made underneath the bell of sufficient depth so the bell does not rest on the bedding material.

Pipe sections shall be jointed so that the ends are fully entered and the inner surfaces are reasonably flush and even.

Joints shall be made with either mortar, grout, rubber gaskets, plastic mastic compounds, or by a combination of these types, as approved and specified by the engineer. Joints in pipe that is jacked in place shall not be sealed with mortar, if so specified, until the jacking of the culvert is completed.

In areas where a tendency exists for pipe sections to separate, suitable ties shall be installed.

If for environmental or other reasons, water tightness is a problem, rubber gasketed pipe is recommended. When such joints are specified the pipe should be tested for either infiltration or exfiltration as stipulated or approved by the engineer. The maximum recommended leakage requirement shall conform to:

Infiltration—0.6 gallon per inch of internal diameter per 100 feet of pipe per hour.

Exfiltration—same criteria for a basic 2 foot minimum internal head, and increased by 10% for each additional 2 feet of head.

Culverts shall be inspected before any backfill is placed. Any pipe found to be substantially out of alignment, unduly settled, or damaged shall be taken up and relaid or replaced.

10.3.4 Backfill and Embankment

10.3.4.1 General

The filling around and over the culvert shall be placed in accordance with the Bedding requirements illustrated in Figures 1 through 4, the Induced trench requirements as illustrated in Figure 5, and the requirements of this section, as specified or approved by the engineer.

10.3.4.2 Embankment Bedding—See Figures 1 and 2

Where rock or noncompressible foundation material is encountered the hard unyielding material should be excavated below the elevation of the concrete cradle (Class A) or the bottom of the pipe or pipe bell (Class B & C Beddings) for a depth of at least 6 inches or $\frac{1}{2}$ inch for each foot of fill over the top of the pipe whichever is greater, but not more than $\frac{3}{4}$ the nominal diameter of the pipe. For Class D Bedding, the depth should be in inches. The width of the excavation should be one foot greater than the outside diameter of the pipe. The excavation should be refilled with selected fine compressible material, such as silty clay or loam, lightly compacted and shaped as required for the specified class of bedding.

10.3.4.3 Trench Bedding—See Figures 3 and 4

a. Materials for backfill on each side of the pipe for the full trench width and to an elevation of 1 foot above the top of the pipe shall be fine, readily compactible soil or granular material, and shall not contain frozen lumps, stones that would be retained on a 2-inch sieve, chunks or highly plastic clay, or other objectionalbe material. Granular backfill material shall have 100% passing a $^3/_4$" sieve not less than 95 percent passing a $^1/_2$-inch sieve and not less than 95 percent retained on a No. 16 sieve. Oversize material if present, shall be removed at the source of the material, except as directed by the engineer.

b. When the top of the pipe is even with or below the top of the trench, backfill material shall be placed at or near optimum moisture content and compacted in layers not exceeding 6 inches (compacted) on both sides and to an elevation 1 foot above the top of the pipe. Care shall be exercised to thoroughly compact the backfill under the haunches of the pipe. The backfill shall be brought up evenly on both sides of the pipe for the full required length. Except where negative projecting embankment-type installation is specified, the backfill material shall be placed and compacted for the full depth of the trench, see Figure 3 and 4.

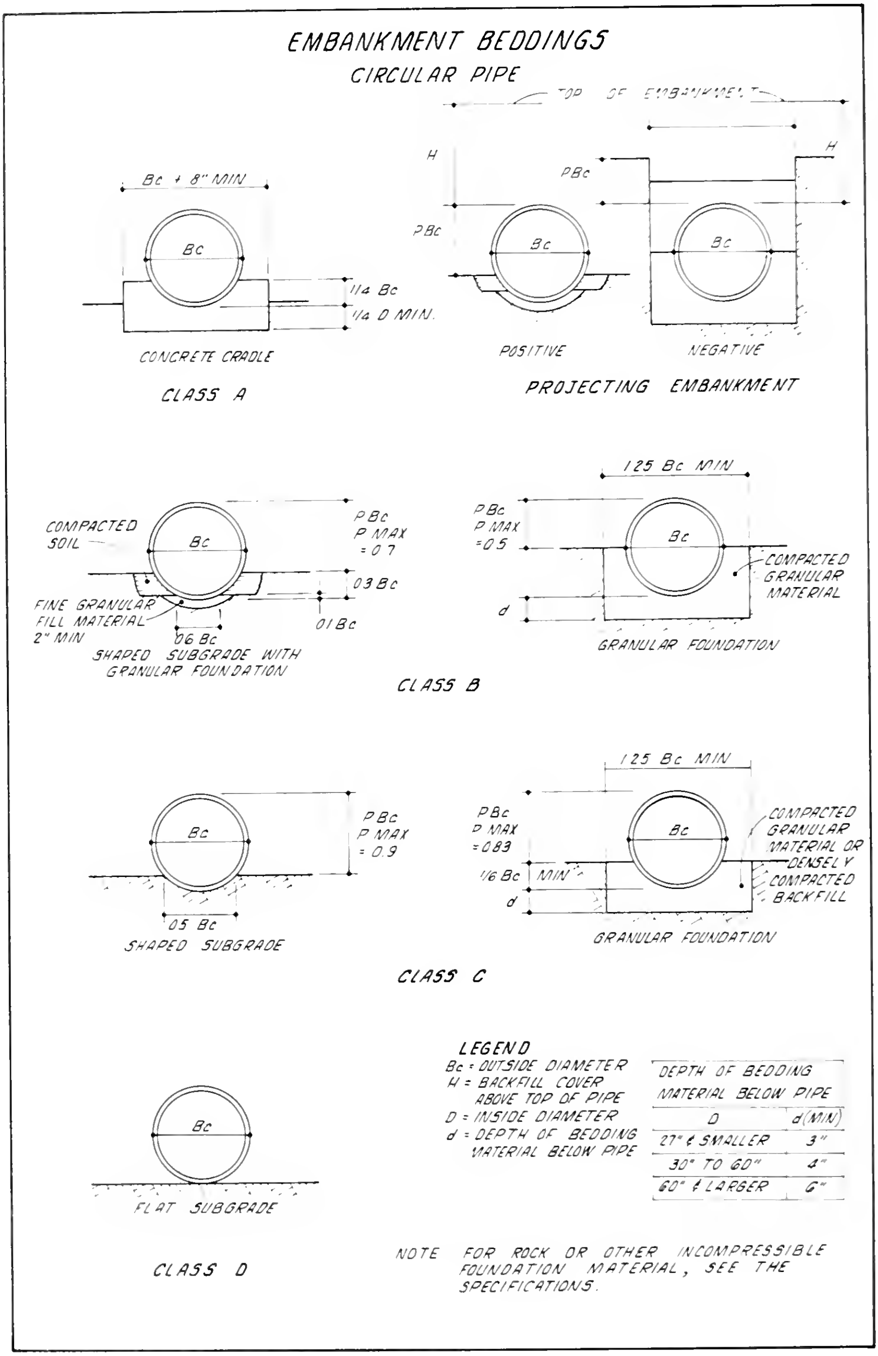
c. Backfill material shall be placed and compacted for the full depth of the trench, unless induced trench installation is used.

d. When the top of the pipe is above the top of the trench, backfill shall be placed at or near optimum moisture content and compacted in layers not exceeding 6 inches (compacted) and shall be brought up evenly on both sides of the pipe for its full length to an elevation 1 foot above the top of the pipe. The width of backfill on each side of the pipe for the portion above the top of the trench shall be equal to twice the diameter of the pipe or 12 feet whichever is less. The backfill material used in the trench section and the portion above the top of the trench for a distance on each side of the pipe equal to the horizontal inside diameter and to 1 foot above the top of the pipe shall conform to the requirements for backfill material in the first paragraph of this subsection. The remainder of the backfill shall consist of material from excavation and borrow that is suitable for embankment construction.

10.3.4.4 Induced Trench Bedding (Figure 5)

a. When the Induced Trench method is used, the embankment shall be completed as required in Section 10.3.4.3 preceding and as illustrated in Figure 5, to a height above the pipe equal to the vertical outside diameter of the pipe plus one foot. A trench equal in width to the outside horizontal diameter of the pipe and to the length shown on the plans or as directed by the engineer shall then be excavated to within one foot of the top of the pipe, trench walls being as nearly vertical as possible. This trench shall be loosely filled with highly compressible material. Construction of embankment above shall then proceed in a normal manner, using regular fill material.

b. The length of the Induced Trench method will be determined by the designer in keeping with the pipe strength being used. Induced trench installation shall not be used when the pipe is subject to track live load.



DEPTH OF BEDDING MATERIAL BELOW PIPE	
D	d(MIN)
27" & SMALLER	3"
30" TO 60"	4"
60" & LARGER	6"

FIGURE 1

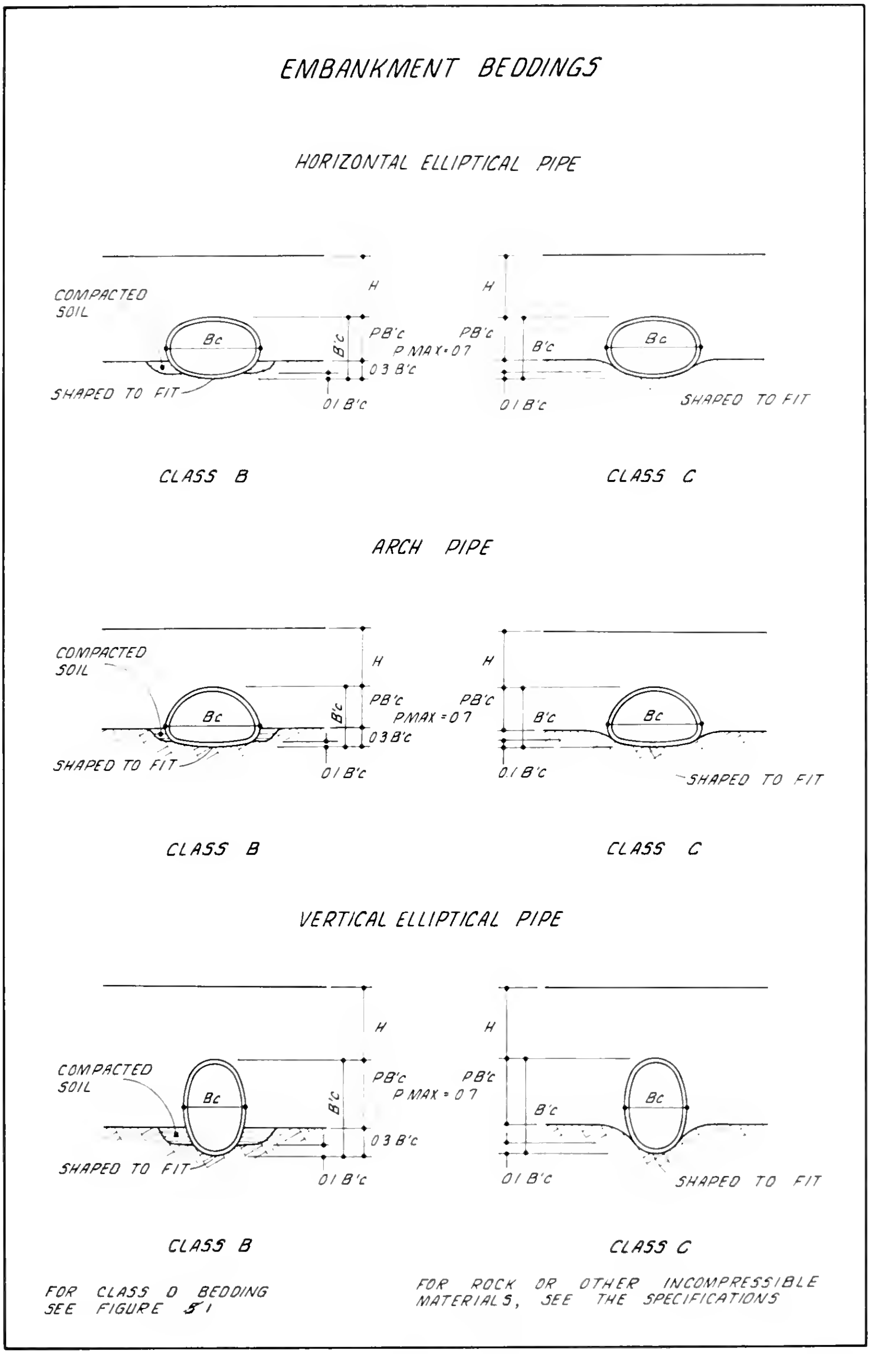

EMBANKMENT BEDDINGS
HORIZONTAL ELLIPTICAL PIPE
COMPACTED SOIL
H
Bc
B'c
PB'c
P MAX = 0.7
0.3 B'c
0.1 B'c
SHAPED TO FIT
CLASS B
CLASS C
ARCH PIPE
VERTICAL ELLIPTICAL PIPE
FOR CLASS D BEDDING SEE FIGURE 5.1
FOR ROCK OR OTHER INCOMPRESSIBLE MATERIALS, SEE THE SPECIFICATIONS

FIGURE 2

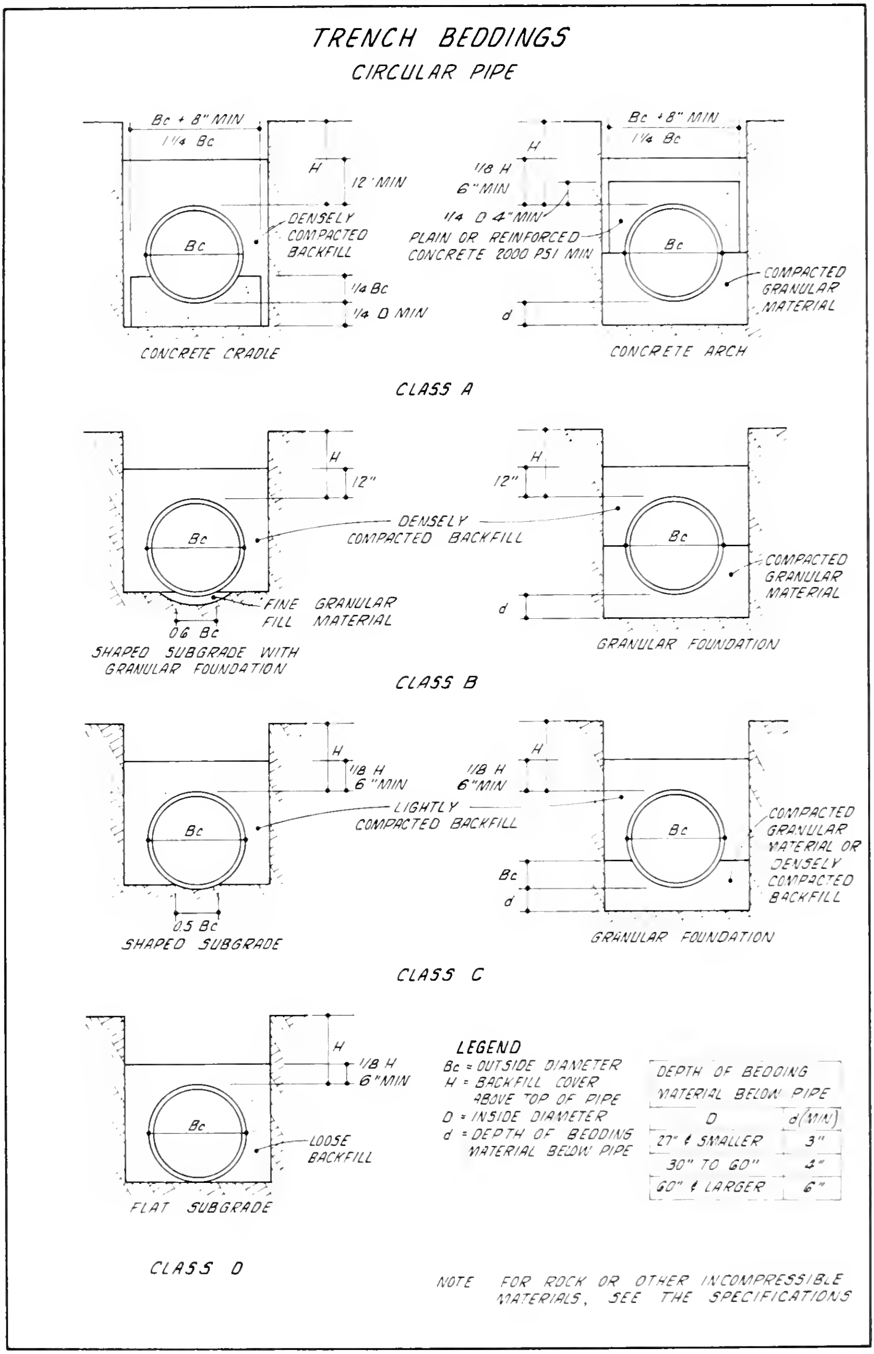

TRENCH BEDDINGS
CIRCULAR PIPE
Bc + 8" MIN
1¼ Bc
H
12" MIN
DENSELY COMPACTED BACKFILL
Bc
¼ Bc
¼ D MIN
CONCRETE CRADLE
1/8 H
6" MIN
¼ D 4" MIN
PLAIN OR REINFORCED CONCRETE 2000 PSI MIN
COMPACTED GRANULAR MATERIAL
d
CONCRETE ARCH
CLASS A
12"
FINE GRANULAR FILL MATERIAL
0.6 Bc
SHAPED SUBGRADE WITH GRANULAR FOUNDATION
GRANULAR FOUNDATION
CLASS B
LIGHTLY COMPACTED BACKFILL
COMPACTED GRANULAR MATERIAL OR DENSELY COMPACTED BACKFILL
0.5 Bc
SHAPED SUBGRADE
CLASS C
LOOSE BACKFILL
FLAT SUBGRADE
CLASS D
LEGEND
Bc = OUTSIDE DIAMETER
H = BACKFILL COVER ABOVE TOP OF PIPE
D = INSIDE DIAMETER
d = DEPTH OF BEDDING MATERIAL BELOW PIPE
DEPTH OF BEDDING MATERIAL BELOW PIPE
D
d (MIN)
27" & SMALLER
3"
30" TO 60"
4"
60" & LARGER
6"
NOTE FOR ROCK OR OTHER INCOMPRESSIBLE MATERIALS, SEE THE SPECIFICATIONS

FIGURE 3

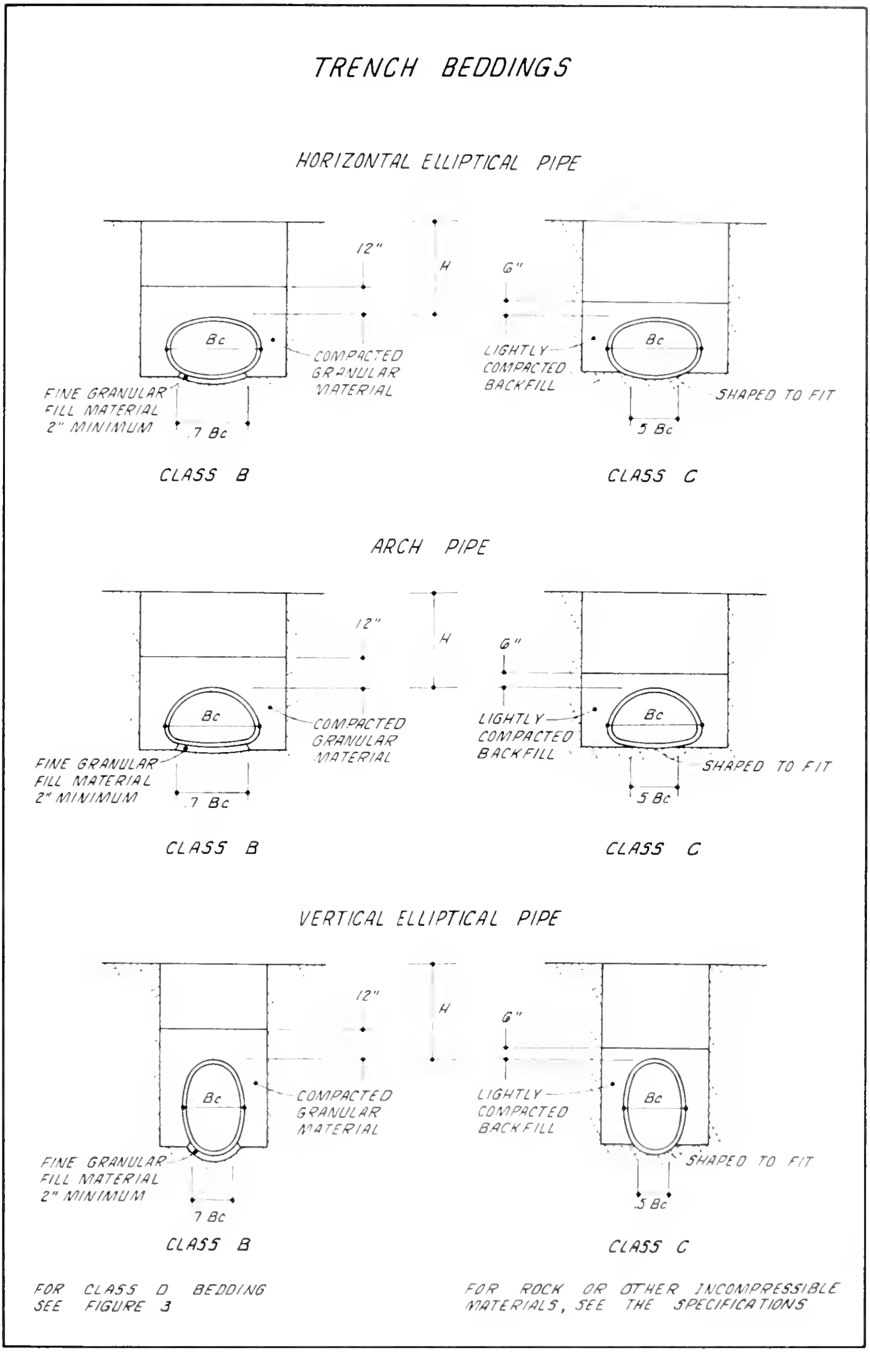

TRENCH BEDDINGS
HORIZONTAL ELLIPTICAL PIPE
12"
H
6"
Bc
COMPACTED GRANULAR MATERIAL
LIGHTLY COMPACTED BACKFILL
FINE GRANULAR FILL MATERIAL 2" MINIMUM
.7 Bc
SHAPED TO FIT
.5 Bc
CLASS B
CLASS C
ARCH PIPE
12"
H
6"
Bc
COMPACTED GRANULAR MATERIAL
LIGHTLY COMPACTED BACKFILL
FINE GRANULAR FILL MATERIAL 2" MINIMUM
.7 Bc
SHAPED TO FIT
.5 Bc
CLASS B
CLASS C
VERTICAL ELLIPTICAL PIPE
12"
H
6"
Bc
COMPACTED GRANULAR MATERIAL
LIGHTLY COMPACTED BACKFILL
FINE GRANULAR FILL MATERIAL 2" MINIMUM
.7 Bc
SHAPED TO FIT
.5 Bc
CLASS B
CLASS C
FOR CLASS D BEDDING SEE FIGURE 3
FOR ROCK OR OTHER INCOMPRESSIBLE MATERIALS, SEE THE SPECIFICATIONS

FIGURE 4

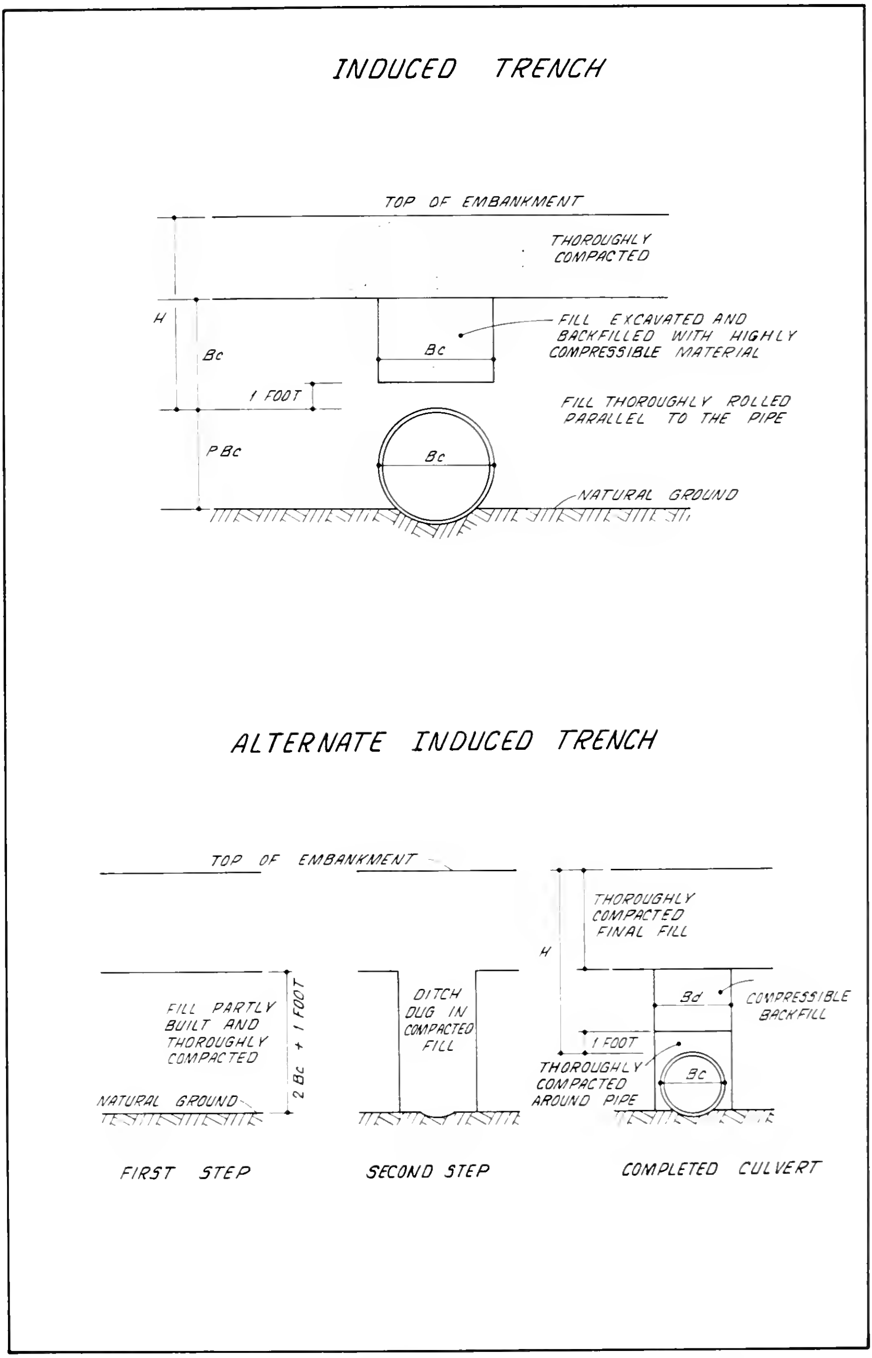

INDUCED TRENCH
TOP OF EMBANKMENT
THOROUGHLY COMPACTED
H
Bc
Bc
FILL EXCAVATED AND BACKFILLED WITH HIGHLY COMPRESSIBLE MATERIAL
1 FOOT
FILL THOROUGHLY ROLLED PARALLEL TO THE PIPE
PBc
Bc
NATURAL GROUND
ALTERNATE INDUCED TRENCH
TOP OF EMBANKMENT
FILL PARTLY BUILT AND THOROUGHLY COMPACTED
2 Bc + 1 FOOT
NATURAL GROUND
DITCH DUG IN COMPACTED FILL
THOROUGHLY COMPACTED FINAL FILL
H
Bd
COMPRESSIBLE BACKFILL
1 FOOT
THOROUGHLY COMPACTED AROUND PIPE
Bc
FIRST STEP
SECOND STEP
COMPLETED CULVERT

FIGURE 5

10.3.4.5 Alternate Induced Trench Bedding (Figure 5)

When the Alternate Induced Trench method as illustrated in Figure 5 is used, the embankment shall be constructed in a normal manner to a height above the required culvert bedding elevation equal to twice the outside diameter of the pipe plus one foot. A trench as required in Figure 5 shall be excavated with walls as nearly vertical as possible and the pipe bedded and backfilled as required by Section 10.3.4.3.b. The remaining trench shall be loosely filled with highly compressible material. Construction of embankment above shall then proceed in a normal manner.

10.3.4.6 In no case, shall the length of the compressible material for the induced trench methods extend the full length of the culvert.

10.3.4.7 Rock fill shall not be dumped over the culvert without a sufficient cushion of earth to prevent breakage of the pipe.

10.3.5 Special Installation Conditions

10.3.5.1 Jacking Pipe

Pipe used for jacking through fills shall be tongue and groove design. The tongue shall preferably be at the downstream end. Jacking frames shall be so constructed as to avoid breaking the pipe or forcing it out of alignment. The pipe shall preferably be jacked upgrade to provide drainage at the heading during excavation. Satisfactory means shall be provided for maintaining the lead pipe in the correct grade and direction.

The pipe shall be installed according to especially prepared plans and specifications. The contractor shall set forth the construction procedure, extra pipe reinforcement and jacking shield if required, jacking pit location and shoring, and other special items necessary for the safe and satisfactory completion of the work. Plans prepared by the contractor giving the construction details shall be submitted for review by the engineer.

10.3.5.2 Constructing Culverts in Tunnels

When it is necessary to place culvert pipe by tunneling, plans and specifications for the completed structure shall be prepared by the engineer. The Contractor shall set forth construction procedures and other necessary details and submit them for review by the engineer.

10.4 PIPE MATERIALS

Pipe shall conform to the following ASTM Standards for type, size, shape and strength requirements as specified by the engineer:

10.4.1 ASTM C76, Specification for Reinforced Concrete Culvert, Storm Drain and Sewer Pipe.

10.4.2 ASTM C506, Specification for Reinforced Concrete Arch Culvert, Storm Drain and Sewer Pipe.

10.4.3 ASTM C507, Specification for Reinforced Concrete Elliptical Culvert, Storm Drain and Sewer Pipe.

10.4.4 ASTM C655, Specification for Reinforced Concrete D-load Culvert, Storm Drain and Sewer Pipe.

10.4.5 ASTM C789, Specification for Precast Reinforced Concrete Box Sections for Culvert, Storm Drains and Sewers.

10.4.6 ASTM C850, Specification for Precast Reinforced Concrete Box Sections for Culvert, Storm Drains and Sewers With Less Than 2 Ft. of Cover Subjected to Highway Loadings.

10.5 MANHOLES

Concrete manholes, if required, shall conform to ASTM C478, Specification for Precast Reinforced Concrete Manhole Sections.

10.6 RUBBER GASKETS

Rubber gaskets, if required, shall conform to ASTM C443, Specification for Joints for Circular Concrete Sewer and Culvert Pipe.

10.7 WATERWAY

For determination of the required size and shape of the culvert see Chapter 1, Part 3.

10.8 MINIMUM COVER

In order to permit unrestricted maintenance, a minimum cover of 2 feet below the bottom of tie is recommended.

10.9 STRUCTURAL DESIGN

10.9.1 Design Criteria

10.9.1.1 General

The design of reinforced concrete pipe culverts must take into account the type of installation and bedding, the soil constants of the natural ground and the backfill, the settlement of the subgrade, the physical measurements such as depth and width of cut and height of fill, and any surcharge loads.

10.9.2 Loads

10.9.2.1 Embankment Loads

The earth, live, and impact loads carried by the pipe is to be determined by a complete set of criteria developed for use by theoretical and experimental analysis carried out at the Engineering Experiment Station, Iowa State University.[1] Design literature gives the procedures for this determination. See Section 10.9.3.1.

10.9.2.2 Live Loads Plus Impact

Design live loads shall conform to the following chart.

Depth Below Bottom of Tie	Live Load Plus Impact psf
2 Feet	3800
5 Feet	2000
8 Feet	1200
10 Feet	900
12 Feet	700
15 Feet	600
20 Feet	400

Impact taken as
40% at 0 feet
0% at 10 feet

10.9.3 Design

10.9.3.1 Design of the reinforced concrete pipe culverts shall be carried out according to the procedures of the referenced investigations at Iowa State University. Design methods are set forth by several organizations two of which are referenced here:

a. United States Department of Agriculture Soil Conservation Service Engineering Division Technical Release No. 5

b. American Concrete Pipe Association Design Manual Concrete Pipe

Conformance with the procedures set forth therein shall be considered to be a satisfactory analysis for the design of the culvert in question.

10.9.3.2 Minimum Pipe Class. Pipe under a main line railroad shall have a minimum class of IV even if analysis indicates that a lighter section will be satisfactory.

A live load factor of 1.5 shall be used for all types of bedding conditions for a pipe subjected to track load.

10.9.3.3 Factor of Safety. The minimum factory of safety against the formation of a 0.01 inch crack shall be 1.0.

10.9.3.4 Alternate Design. In lieu of carrying out the extensive design analysis required by the procedure set forth herein, the designer may use Class V pipe for all sizes up to a fill depth of fourteen (14) feet. For greater fill heights, the designer must make an analysis. For Elliptical or Arch Pipe where Class V is not available a design analysis must be made.

Report of Committee 11—Engineering Records and Property Accounting

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G. R. Gallagher, *Vice Chairman*
R. F. Nelson, *Secretary*

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C. E. Bynane
P. G. McDermott
T. H. Stevenson
J. W. Kelly
W. A. Baker
F. B. Baldwin (E)
P. J. Beyer
R. M. Davis
C. R. Dolan
W. V. Eller
L. D. Farrar
G. L. Fisher
C. C. Haire (E)
A. P. Hammond, Jr.
S. P. Hoffman
P. R. Holmes
A. E. Hussey, Jr.
R. E. Igou
G. F. Ingraham
A. J. Ivaskevicius
R. W. Janssen
W. C. Kanan
C. E. Lee
J. G. Maher
J. L. Manthey
C. J. McDonald
S. Miller, Jr.
F. S. Mitchell
H. L. Restall (E)
G. S. Rogers
R. S. Shaw
V. E. Smith
J. B. Styles
T. A. Valacak
H. R. Williams
L. D. Wilson

Committee

(E) Member Emeritus.

Those whose names are shown in boldface, in addition to the chairman, vice chairman and secretary, are the subcommittee chairmen.

To the American Railway Engineering Association:

Your committee reports on the following subjects:

B. Revision of Manual.
No Report.

1. Accounting.
Our emphasis on this committee for the immediate short term will be to continue to study and report on the ICC's proposed change in accounting rules regarding depreciation of track structures. Members of our committee have submitted to the AAR a paper outlining our understanding of the proposed ICC Rule and our view of the impact that will be created by its implementation. The study included numerous questions and statements that must be resolved if such an order is to be implemented by the railroad industry. This committee will also continue to keep the industry abreast of depreciation and life study problems including the use of the ICC's Depreciation Programs. The standard practice of reporting on all accounting changes will continue.

2. Bibliography.
The committee will continue as in the past to furnish pertinent articles of interest as they apply to engineering and property accounting and records.

3. Office and Drafting Practices.
   The committee will continue to study techniques for performing the property accounting function.

4. Taxes.
   The thrust of this committee is to report to the industry the changes in the federal income tax laws, state and local tax laws and other tax laws affecting railroad property.

The Committee on Engineering Records and Property Accounting,
L.F. Grabowski, *Chairman.*

Report of Committee 13—Environmental Engineering

PHOTO
NOT AVAILABLE
AT PRESENT
TIME

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Committee

(E) Member Emeritus.

Those whose names are shown in boldface, in addition to the chairman, vice chairman and secretary, are the subcommittee chairmen.

To the American Railway Engineering Association:

Your committee reports on the following subjects:

B. Revision of Manual

1. Water pollution control.
   This subject is discontinued due to the lack of EPA guidelines for the railroad industry on NPDES discharges.

2. Air pollution control.
   New subject is to be an update of previous subject in line with current and projected energy costs of primary fuels.

3. Land pollution control.
   Will continue to work with sub-committee 2 and continue its study of railroad sludge disposal.

4. Industrial hygiene.
   It was voted on and passed by the full committee at the June meeting to discontinue this committee assignment since the subject is primarily one concerning OSHA rather than EPA regulations. New title—Environmental Economics.

5. Plant utilities.
   Design, construction and operation. This sub-committee will be inactive and will assist sub-committee 8.

7. Noise pollution control.
   Continue study of realistic limits for boundary noises for various railroad equipment and facilities.

8. Student design contest.
   To be awarded at 1980 Annual Technical Conference.

The Committee on Environmental Engineering,
R. S. Bryan, Jr., *Chairman*

Report of Committee 14—Yards and Terminals

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To the American Railway Engineering Association:

Your committee reports on the following subjects:

1. Fire Prevention in yards.
   No action has been taken on this subject to date. The matter was discussed at some length at our last meeting. There was concern expressed by certain of the members that if the rail industry did not itself specify some minimum fire prevention standards for its yard facilities, some government agency would. Defining the scope of the subject has been somewhat difficult. It is felt that this problem is behind us and that a preliminary report can now be progressed.

2. Bulk material handling systems, collaborating as necessary or desirable with Committees 6, 8 and 15.
   Chairman has changed jobs recently and has been unable to progress this subject. Early phases were completed by previous chairman who has retired. May have to seek a new chairman to complete the work.

5. Trends in intermodal facilities.
Information assembled and a report expected at next meeting. Suggest changing title to "Design of" rather than "Trends in."

6. Guidelines for intertrack drainage and surface construction in yards.
Preliminary report has been submitted and it has been reviewed and discussed by the whole committee. Final report in progress.

7. Yard system design for two-stage switching.
A preliminary draft report was reviewed by entire committee at its most recent meeting. The report was accepted with modifications and appeared in Bulletin 675, November-December 1979.

8. Grades in yards with tangent point retarders.
A preliminary report was submitted and minor changes suggested by the full committee. It is now in the process of being completed.

9. Bibliography of reports by other organizations and bodies concerning yards and terminals.
A verbal report was presented at the last meeting by Mr. Anderson. The format of the proposed bibliography was discussed and it was decided that the sequential order as presented in the Contents of Chapter 14, Manual would be followed. It has been determined that computerized listings and digests of publications are available to search for a nominal fee. This course is being pursued as the most economical method to put together the proposed bibliography.

10. Cooperate with DOT-TSC on study of methodical design of classification yards, collaborating as necessary or desirable with other AREA committees or AAR units.
The subcommittee has met with the contractor and others engaged by the TSC on this project a number of times. On one occasion the entire committee 14 was given a briefing on Phase I of the effort. Coordination has been good as things progress. The FRA scheduled a "Classification Yard Technology Workshop" in Chicago on October 30 and 31, 1979. Many committee members and Sub-committee members attended.

The Committee on Yards and Terminals,
P. C. White, *Chairman.*

Report of Committee 15—Steel Structures

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Committee

(E) Member Emeritus.

Those whose names are shown in boldface, in addition to the chairman and vice chairman, are the subcommittee chairmen.

To the American Railway Engineering Association:

Your committee reports on the following subjects:

B. Revision of Manual

Revisions to specifications for Steel Railway Bridges submitted for adoption published in this Bulletin.

1. Develop criteria for the Design of Unloading Pits, collaborating with Committees 7 and 8.
   No Report.

2. Obtain data from which the frequency of occurrence of maximum stress in steel railway bridges may be determined under service loading.
   No Report.

4. Develop specifications for the earthquake design of steel railway bridges.
   Subcommittee has been organized and is collecting data.

7. Bibliography and technical explanation of various requirements in AREA specifications relating to steel structures.
   Additional items were published in Bulletin 675, November-December 1979.

10. Continuous Welded Rail on Bridges.
    Your committee considers this assignment complete.

The Committee on Steel Structures,
C. A. Hughes, *Chairman*

Manual Recommendations

Committee 15—Steel Structures

8.5 Walkways and Handrails on Bridges

D.L. NORD (Chairman, Subcommittee)

1979

8.5 WALKWAYS AND HANDRAILS ON BRIDGES

8.5.1 Locations

8.5.1.1 Bridges to be equipped with walks and handrails will be designated by the Engineer.

8.5.2 Clearances and minimum dimensions

8.5.2.1 Clearances shall not be less than specified in Art. 1.2.6. A guide to legal requirements in the various states may be found in Chapter 28, Sec. 3.6.

8.5.2.2 Handrails

(a) In through structures, handrail need not provide more clearance than the structural members.
(b) Top of handrail shall be not less than 3′-6″ above surface of walkways. An intermediate rail, or rails, shall be provided, with clear space between rails, or between rail and top of walkway, not to exceed 1″-9′.
(c) The ends of the rails shall not overhang the terminal posts except where such overhang does not constitute a projection hazard.

8.5.2.3 Walkways

(a) In general, walkways shall not be less than 2″-0′ wide and shall extend to the inner face of the handrail. On ballasted deck bridges the ballast may be considered the walkway, or a separate surface may be provided. On open deck bridges, not more than 2″ gap shall be allowed between the line of the ends of ties and edge of walkways.

(b) On bridges with two or more tracks, walkway may be located between the tracks, without handrails.

(c) Structural members (such as floorbeam brackets) shall not be considered an obstruction to the walkway.

(d) Walkways on bridges over highways or other locations where spillage of ballast or lading is a consideration shall be solid material (i.e., not grating) and shall be provided with a curb or toe board.

8.5.3 Loads

8.5.3.1 Handrails

(a) Each railing and its fastening shall be designed for a single load of 200 lbs., applied either laterally or vertically, and at any point in the span.

(b) Where steel cable is used for rails, sag under load at midspan shall not exceed 2″.

(c) Posts shall be designed for a single load of 200 lbs., acting either laterally or vertically, applied at the point of attachment of the top railing.

8.5.3.2 Walkways

(a) Walkways shall be designed to support a uniformly distributed load of not less than 85 pounds per square foot without exceeding the allowable stresses of the material used.

(b) In addition to the strength requirements, the walkway material shall be so proportioned that the deflection under a single concentrated load of 250 pounds per foot of width, applied at midspan, does not exceed 1/160 times the span length.

(c) Where off-track work equipment may be expected to be driven across the bridge, walks should be designed for the appropriate wheel loads, without regard to deflection.

8.5.4 Materials

8.5.4.1 Stresses

(a) Walkways and handrails may be designed for higher stresses than allowed for members subject to railroad live loading.

8.5.4.2 Handrails

(a) Where rails or posts are constructed of timber, minimum thickness shall be 2″ nominal. Rails shall be surfaced material.

(b) Where rails or posts are constructed of structural steel, minimum thickness shall be 1/4″.

(c) Where cable rails are used, they shall be minimum 3/8″ diameter, 7-wire galvanized steel. Cut ends shall be suitably protected to prevent injury from the sharp strand.

(d) Where posts are connected to a structural member, the post or its connection shall be designed to fail under overload without damaging the member.

8.5.4.3 Walkways

(a) In addition to providing sufficient structural strength, materials used for walkways shall also present a suitable walking surface.

(b) Where timber is used as walkway material, minimum nominal thickness shall be 2″, with walking surface rough. It shall be fastened to each support with the equivalent of 2 - 20d spikes.

(c) When structural steel plate is used for walkway material it shall have a roughened tread surface (checker plate), with a minimum thickness of 1/4″.

(d) When metal grating is used as walkway material, it shall be of galvanized steel or other corrosion resistant material. Fastenings shall be designed to prevent longitudinal movement which may result in loss of bearing.

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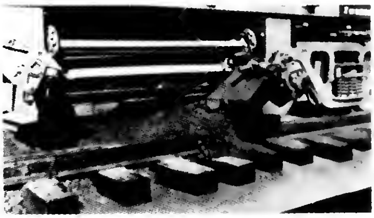

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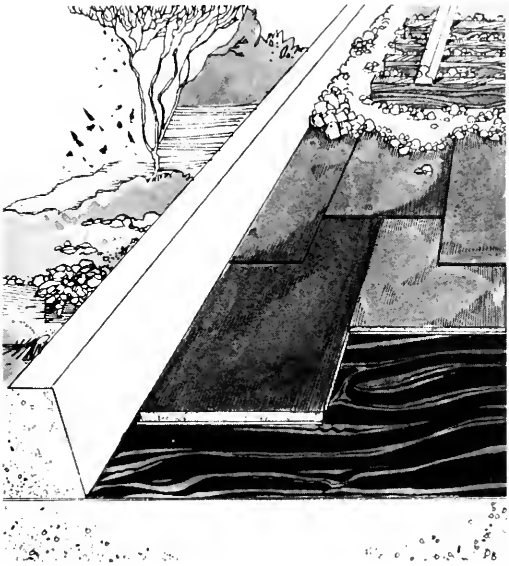

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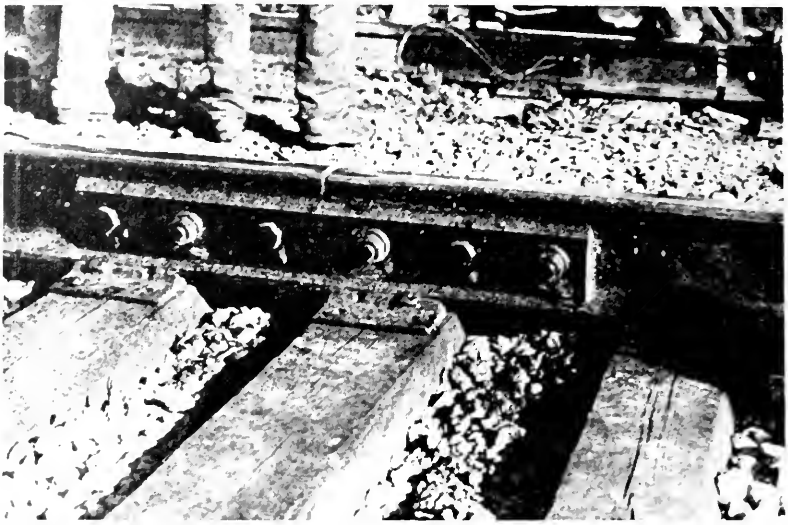

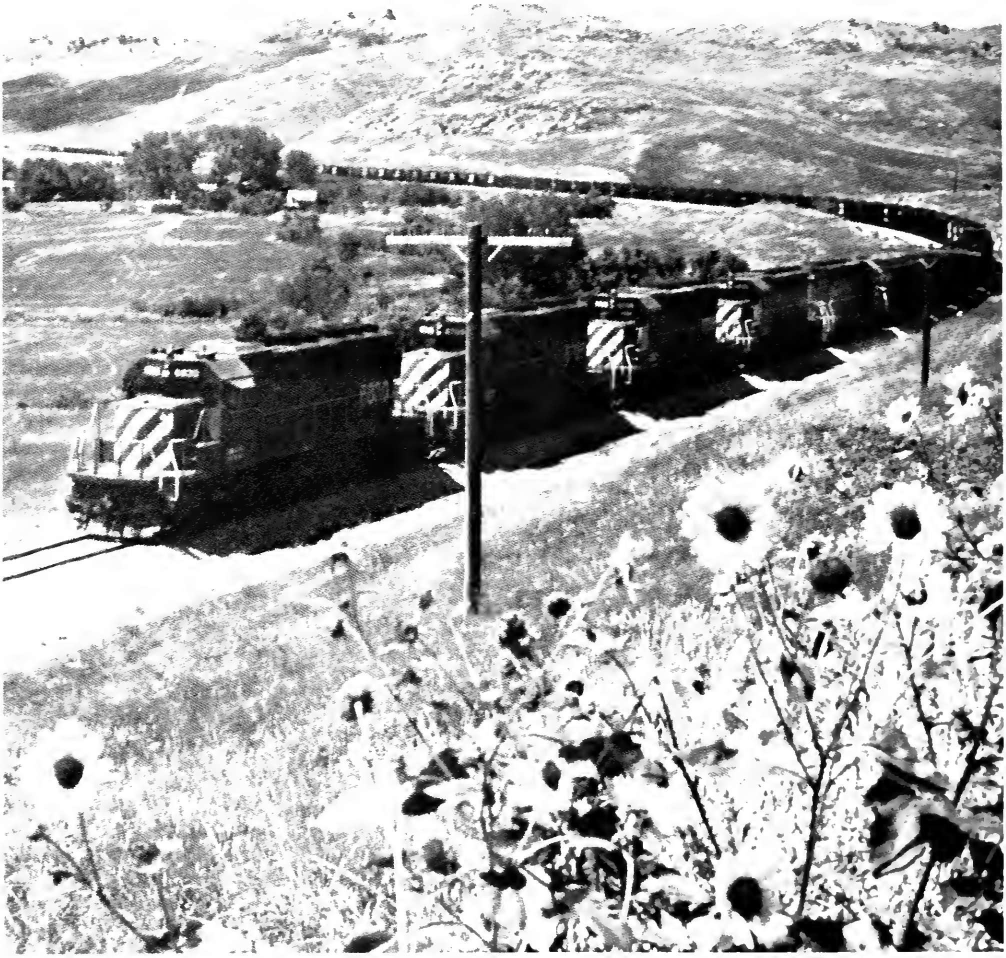

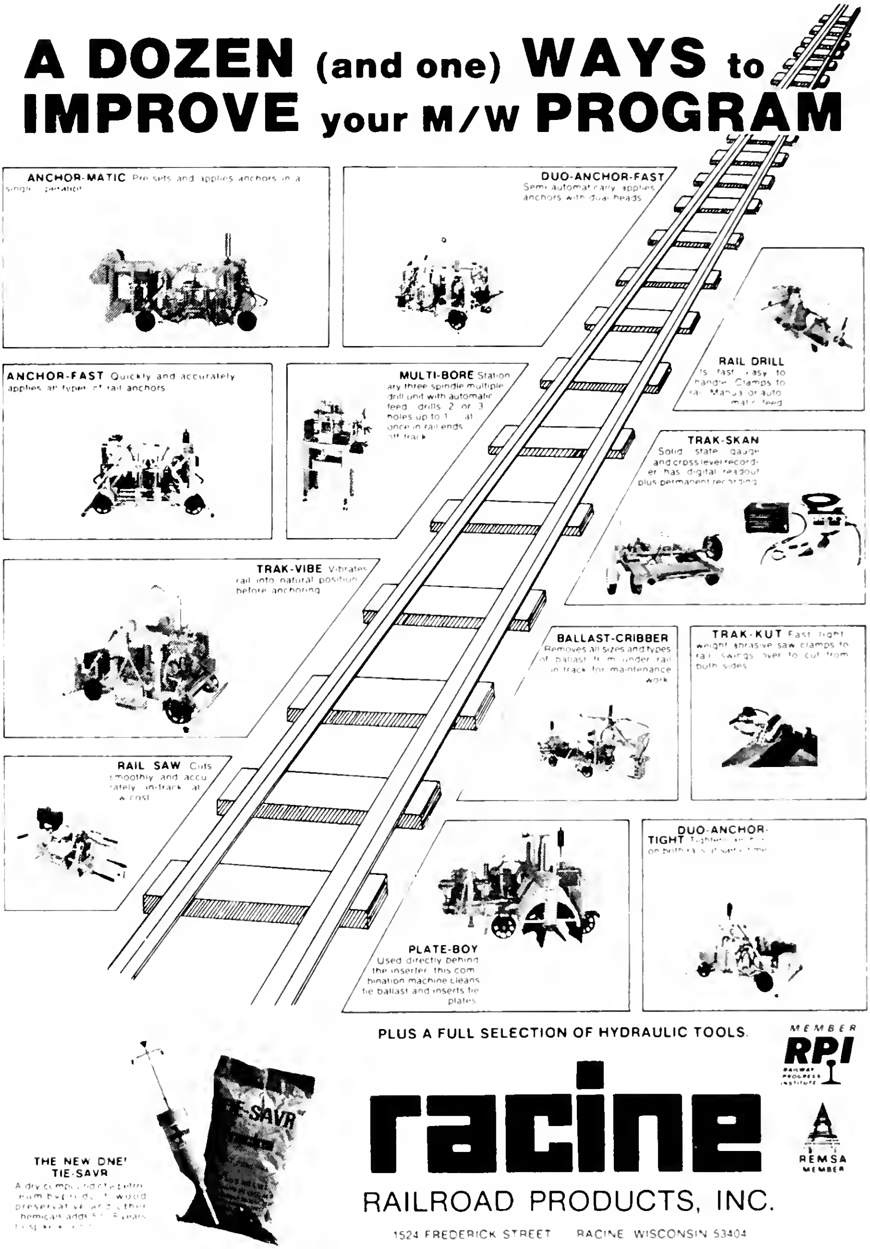
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Committee

(E) Member Emeritus.
*Died Feb. 26, 1979.
**Died Sept. 16, 1979.

Those whose names are in boldface, in addition to the chairman, vice chairman and secretary, are the subcommittee chairmen.

To the American Railway Engineering Association:

Your committee reports on the following subjects:

B. Revision of Manual.
This subcommittee has been very diligently working on a new section for Chapter 16, Part 4. This section covers Railway Operations and the text is now in final form and was submitted to the Board for approval and manual insertion after receiving a 72% approval by Committee 16. This section would greatly expand Chapter 16 and be of great value and assistance to the railroad industry.

3. Determination of Factors, including various traffic volumes affecting maintenance of way expense and effect of using such factors in terms of equated mileage or other derived factors, for allocation of available funds to maintenance of way, collaborating as necessary or desirable with Committees 11 and 22.

   a. Additional maintenance cost due to operating 100 ton unit trains.
   This committee has been wrestling with subsubject (a) for a number of years. They have now completed a report which appears in this bulletin, 676. This report deals directly with unit train operation and its impact on existing track. It also offers some good guidelines to follow when necessary to construct new track and refurbishment of existing track. This committee should be congratulated for its continued effort in progressing a very complicated and highly concerned subject.

4. Economic evaluation of methods for reducing the probability of derailments.
This committee has been in the process of collecting data and information on procedures followed by each railroad in the investigation and reporting of derailments. It is expected that the information collecting will be completed shortly and that a summary report will follow.

5. Economics of freight cars with characteristics approaching the limits of accepted designs.
This committee has attempted to resolve the subject matter but as you know from a paper presented by Mr. George Way at the last Technical Conference in Chicago even the AAR has not determined "the bottom of the bath tub." On this basis Committee 16 will give consideration to the discontinuance of this subject.

6. Factors involving the rationalization of railway systems.
This committee had a change in chairman and a review of the past work is presently being conducted.

7. Applications of industrial engineering functions to the railroad industry.
This committee is giving inputs on the scope of service the industrial engineers provide for the railroad industry. They are finding that industrial engineers are used extensively and have a very important place in the operating of a railroad plant. Even though there is a full section in Chapter 16, this committee feels that additional material can be added dealing with the real functions of the industrial engineer.

The Committee on Economics of Plant, Equipment and Operations,
T.C. Nordquist, *Chairman.*

Committee 16—Economics of Plant, Equipment and Operations

Report on Assignment 3(a)

Additional Maintenance of Way Cost Due To Operating 100-Ton Car Unit Trains

G. M. Tabor (Chairman, Subcommittee)

The purpose of this report is to identify areas where additional maintenance of way costs are incurred with respect to operating 100-ton car (263,000 lbs. gross weight) unit trains and to provide desirable track standards.

As covered in various AREA Committee 16 reports over the past several years, there has been a continuing effort by railroads and the federal government to develop a consistent approach to the allocation of maintenance of way costs. (1,2)* At present, any accurate description of maintenance of way costs is elusive and railroad engineers and budget officials have to rely on judgment and empirical estimates in determining maintenance of way costs.

The report "Procedures for Analyzing the Economic Costs of Railroad Roadway for Pricing Purposes—January 1976" prepared for the U.S. Department of Transportation, Federal Railroad Administration currently represents the most notable endeavor in recent years to develop procedures for determining maintenance of way costs. (3) The report takes into account significant factors affecting the physical and economic behavior of a roadway. Considered along with track-related and traffic-related factors (including the effects of heavy wheel loads) are such variables as managerial policies and budgetary constraints which govern roadway maintenance expenditures. Although this report provides a basis for the determination and allocation of variable roadway costs, it concludes that even with the expansion of well-documented track research data there is still insufficient data to provide railroad management with adequate decision-making capability for establishing rail service prices and effective operations and maintenance planning and control

A principal recommendation of the aforementioned reports is the development and use of improved procedures to extend rail life. To this end AREA Committee 4-Rail has introduced a methodology for the calculation of the flexural fatigue life of rails in service in connection with the Committee's study of the effect of heavy wheel loads on rail. The results presented show the detrimental effect of increased wheel loads on rail life and further show that it is necessary to properly match rail size to anticipated wheel loads to reduce occurrence of rail defects.(4,5) Authors of the Committee paper believe that the technique they have developed can be expanded to encompass other track components.

The increased frequency of heavy wheel loads and 100-ton car unit train operations directly contributes to the deterioration of rail and other track components and as reported herein procedures have been developed for determining maintenance costs. However, before these procedures can be adopted by the railroad industry, considerable refinements are needed.**

*These numbers indicate references at end of paper.

**References 6, 7, 8 and 9.

It is anticipated that the results of the cooperative government-industry research program, "Facility for Accelerated Service Testing" (FAST), will greatly increase present-day knowledge regarding the fatigue life of track components. In addition, these results should provide a basis for comparing track design and maintenance practices, thus making it possible to refine costing procedures. The "FAST" program makes use of a loop track constructed through cooperative efforts of industry and government at the United States Department of Transportation's test center at Pueblo, Colorado. Data now has been accumulated by DOT personnel from the first experiment using 4-axle locomotives and a consist of loaded, nearly all 100-ton cars, which got under way September 22, 1976. This data is being analyzed by the Association of American Railroads, but the full results of this analysis may not be known for several years. Some railroads, however, have benefited from the results produced by the "FAST" program.

For the present, there is a problem basic to all maintenance of way costing efforts. There are no available, reliable data on the cumulative, long-term response of track to repetitive wheel load application. Instantaneous effects can be measured or computed, but the long-term effects, especially regarding life of line and surface for various types of ballast and subgrade materials and ballast section design are unknown. Therefore, railroad engineers cannot design a track for a known life in terms of a given traffic density and wheel load. For example, it cannot be determined in advance the difference in length of maintenance cycle for 8-inches of top ballast versus 10-inches of top ballast depth. There is no direct relation between life of rail and strength of subgrade or, in other words, no good index of track quality and life.

A well constructed and maintained track structure requires lighter and less frequent maintenance. However, all other things considered equal, the cost of maintenance for the same revenue tonnage is greater for 100-ton car trains than for trains of 50-ton or 70-ton cars. For tracks adequately constructed for the usual high density freight traffic mix and maintained in first class condition, the additional costs of maintenance due to superimposing 100-ton car unit train operations will rise in excess of the proportionate increase in gross tonnage. For lightly constructed or poorly maintained tracks, the rise in maintenance costs would be dramatic.

Whether it is proposed to operate 100-ton car unit trains over existing tracks or to construct new tracks, it will be necessary to commit funds for higher quality construction and maintenance standards than normally required to operate trains of mixed consist. This may mean upgrading or constructing with more expensive track components, depending on the traffic and the existing condition of the tracks being upgraded. If a railroad is not adequate for 100-ton car unit trains, deterioration of the track structure will magnify at an ever increasing rate and train operations will become more expensive.

Characteristics of 100-ton car unit trains that cause deterioration of the track structure are:

1. Heavy, repetitive loads.

2. Movement of heavy loads in one direction.

3. Dynamic loading unique to unit trains.

4. High center of gravity of cars used in some unit trains.

Repetitive heavy axle loads contribute to higher contact stresses and internal defect development in rail, resulting in greatly accelerated metal flow, corrugation, shelling, and sharply reduced rail life. Heavy, repetitive loads in long trains reduce the elastic response of ballast and subgrade, tending to "beat down" the track, hastening the abrasive wear, fouling, and eventual cementation of ballast materials. This contributes to poor track line and surface, and to centerbound track. Cemented ballast does not drain internally, loses its elasticity, and further accelerates ballast and tie abrasion. Ultimately, it requires undercutting or extensive raising on new ballast.

The movement of heavy loads in one direction can cause uneven wear on rails, other track components and wheels. Uneven wheel wear causes additional wear on rail and other track components.

Dynamic loading is more severe for 100-ton car unit trains because of the equipment uniformity and stiffness (lack of lateral freedom) in roller bearing trucks usually found on unit train equipment. Uniformity causes each car in a train to respond in the same manner to each irregularity in track geometry. This concentrates the wear and makes such irregularities more severe. Stiffness reduces the absorption by the car of dynamic loads and increases hunting at speeds above 45 MPH. Hunting contributes to gauge widening and increased truck and car component wear.

High center of gravity cars have a greater tendency to develop harmonic "rock-and-roll". "Rock-and-roll" increases dynamic loading and the tendency of the cars to derail. It is most severe in a critical speed range that is dependent on truck and rail joint spacing.

Train speed is directly related to impact loading and other dynamic loads due to train operation. Reduced train speed will reduce the deterioration of the track structure due to 100-ton car unit trains; however, reduced speed increases other operating expenses.

When designing new tracks or planning the use of existing lines for 100-ton car unit train operation, the adverse characteristics of this operation must be considered and a determination made regarding the adequacy of each component of the track structure and its supporting structure. As with any other load-supporting structure, a railroad must be designed to handle the heaviest loads contemplated and there is a profound change in relationship between various components of the track structure when heavier loads must be supported.

In order to hold maintenance of way costs to a minimum, good line and surface must be maintained. Some items to be considered in accomplishing this are as follows:

Alignment—Accelerated wear on the rails in curves can be expected from the wheel flanges when operating heavy car unit trains. Accordingly, curves should be as light as possible and have adequate elevation and spirals for the actual operating speed of the loaded unit trains. It is recommended that curvature be no sharper than 1 degree 30 minutes for unit trains operating at the relatively high speed of 50 MPH and limited to a maximum of 6 degrees for speeds less than 25 MPH. Superelevation of curves above 4½ inches should be avoided by reducing the maximum allowable speed. Where unit train speeds of 50 MPH are contemplated, there should be at least 500 ft. of tangent track between spirals or superelevation runoffs, where practical, whether the curves are reversed or in the same direction, but no less than 200 ft. of tangent track between spirals or superelevation runoffs. In the event these alignment requirements cannot be met in the construction of new roadbed or the upgrading of existing roadbed, authorized unit train speeds should be reduced below that allowed for normal operation of freight trains with mixed consist. In some instances, the combination of curvature, elevation and speed selected for existing tracks might be dictated by the track-train dynamics of 6-axle locomotives. Curvature for loadout loop tracks should be no more than 7 degrees 30 minutes.

Gradients—Except for minor changes, revisions in gradients of existing roadbeds generally are not practical. Therefore, it is imperative in constructing new rail lines to design for the forseeable traffic. In order to provide for safe operation and for efficient allocation of motive power, gradients should not exceed those of the main line. If feasible, ascending gradients for loads should not exceed 1.0 percent compensated for curvature and descending gradients should not exceed 2.0 percent but be preferably less in both instances.

Roadbed—Stability is absolutely essential for maintenance of good line and surface. Consequently, particular attention should be paid to drainage and compaction. It has been said that almost any material will make a stable subgrade if it can be made dry and kept dry. However, if this is not possible and the subgrade materials are fine-grained and weakened by the presence of excessive moisture, pumping action from repetitive wheel loads will turn the subgrade materials into a soft plastic or slurry-like substance. As a consequence, unless stabilization is provided there will be a continual problem in maintaining good surface and line adding to maintenance and operating costs. To eliminate this condition it will be necessary to install subdrains and/or undertake special measures, as replacing unstable soil, treating the soil with an additive such as lime or installing waterproof membrane on the roadbed surface. It costs roughly one-half as much to provide good drainage, ballast and ties as to relay rail. Therefore, good drainage is a first priority item for maximum cost-effectiveness in upgrading track to handle heavier cars or to reduce maintenance of way costs. (6)

Bridges—Ratings should be reviewed and modification or reconstruction undertaken as necessary. Some problems experienced are:

1. The heavier loads of unit trains will reduce the durability of timber structures and can precipitate early failure in older bridges.

2. There is a potential for fatigue damage to steel bridge components and new timbers as discussed in "Address by R.E. Ahlf" before the AREA's Seventy-Fourth Technical Conference and reported on Page 628 of AREA Bulletin 653. (6)

3. Additional stringers and/or piles are required to increase carrying capacity of some timber trestles.

4. Extra maintenance is necessary on bridge approaches to prevent mechanical damage to ties, stringers, caps and piling at the ends of bridges and trestles.

5. Depending on traffic and operating speeds conversion from open decks to ballast decks is warranted to provide support consistent with the roadbed and to better control forces transmitted by welded rail. Also, conversion to ballast decks reduces the potential of fire from hot brake shoes or fusees.

Ballast—High quality ballast will be required to:

1. Minimize settlement due to fracturing of sharp corners of the ballast under heavy loads and to maintain drainage.

2. Uniformly distribute the load and to better anchor the track against heavy lateral and longitudinal loads.

3. Provide for more efficient maintenance.

AREA specifications for ballast have recently been revised to establish new material standards and test requirements in order to provide adequate support for the track under heavier wheel loads. (10, 11) It is recommended that ballast meeting these specifications be used. The ballast section should be placed and maintained with full cribs and high shoulders and with no less than 8 inches of ballast under the ties. Placing and maintenance procedures should result in a well packed and tamped ballast section. The AREA specifications for sub-ballast were also revised and it is further recommended for newly constructed tracks that at least 6 inches of sub-ballast meeting these specifications be placed on the finished subgrade prior to laying tracks. Depth of sub-ballast should be based on adequate study of soil bearing and shear strength by a qualified geotechnical expert. Although the subgrade should be crowned for drainage, the sub-ballast should not be crowned under the ties.

Ties—Closer tie spacing with larger and longer ties may be necessary to minimize roadbed disturbance and to hold surface and line. Treated 8 ft. 6 in. minimum timber ties meeting current AREA specifications should be used. The ties should have a minimum cross-section of 7 in. x 9 in. and 3,113 (equivalent to 23 per 39 foot panel) should be installed per mile of track. At locations where unit trains are operated in heavy curve territory and the volume of traffic restricts time for maintenance, consideration should be given to the use of concrete ties of approved design.

Tie Plates—Larger and thicker plates may be needed to provide increased bearing area and better load distribution, thus holding the physical damage to ties to a minimum and affording better line and surface. To accomplish this, heavy-duty double shoulder 14 in. or larger plates with no less than 1 to 40 cant should be used. Also, consideration should be given to use of 18 in. plates on curves over 4 degrees to provide improved load distribution and a greater frictional area to resist lateral thrust and the tendency of rail to overturn.

Spikes—Additional spikes may be required to maintain gauge and to prevent overturning of rails. In tangent track and in curved track under 4 degrees, a minimum of four spikes per tie plate can be used. On curves over four degrees, at least one additional holddown spike should be driven on the gauge side of each rail to provide a minimum of five spikes per tie plate. In certain territories where a combination of factors such as degree of curves, tonnage, grades, speeds, superelevations, train handling, train consist, type of locomotives and weather conditions work together to increase the probability of rail turnover, special rail fasteners should be considered. These fasteners would also provide toe load on the base of rail to resist longitudinal movement. There are several types of special rail fasteners ranging from rigidly bolted to spring clip type. In addition to tests undertaken at Pueblo, some railroads are conducting extensive tests to determine the effectiveness of these fasteners in preventing rail turnover. Concrete ties are less prone to develop irregularities in alignment because of their greater resistance to gauge spreading and rail overturning.

Rail Anchors—Sufficient anchors must be provided to resist rail movement from the heavy directional traffic pattern of 100-ton car unit trains. For tracks of jointed rail carrying traffic in both directions, every third tie should be box anchored (an anchor on each side of the ties). Within continuous welded rail territory, every other tie should be box anchored. In some situations additional anchors will be necessary to restrain the rail from running due to the effects of heavy carloads. Where insulated or field butt welded or conventional joints occur in continuous welded rail territory, each tie should be box anchored for 195 ft. in both directions from each joint. Turnouts should be anchored throughout.

Rail Joints—Rail joints are designed to connect rails so that when joined together the rails act as a continuous rail with uniform surface and alignment. Bolt holes of the joint bars are slotted to allow for expansion and contraction of the rails without overstressing the bolts. As a consequence, when the rails contract in cold weather an opening occurs between abutting rails and the rail ends become battered and worn down under traffic. Heavy repetitive axle loads produce damage to the rail ends and to joint ties which receive less wear under normal traffic. Subsequently, low joints occur and as the joints are staggered heavy cars develop "rock-and-roll" action which increases the probability of derailments. While this condition would be eliminated by the use of welded rail, this is not always economically feasible. It is standard practice to stagger rail joints as near as possible to the center of the opposite rail to minimize dynamic impact at the joints, but certain types of high capacity, high center of gravity equipment, not equipped with suitable truck snubbing devices such as used in captive service on unit trains, develop a harmonic rocking motion which is aggravated by rhythmic center staggered rail joints. The speed range at which this "rock-and-roll" occurs is between 10 and 25 MPH with the exact speed being determined by such factors as the wheel base, height of the center of gravity, spring dampening, length of rail and relative difference in the elevation of successive joints in jointed rail territory. In extreme cases, wheel lift occurs which can result in derailment. Laying rail having a nominal length of 36-39 feet so that the joints will be on one-quarter rather than one-half stagger in branch line main tracks and other tracks where 10 to 25 MPH speeds prevail may reduce harmonic "rock-and-roll". When joint bars are used, it is recommended that these be made from standard rolled steel sections and the bars meet current AREA specifications for 6-hole angle bars 36 inches in length. It is important to support the bars at each rail joint by 3 ties to minimize the effects of 100-ton car unit trains.

Rail—Although the girder strength of lighter weight jointed rail with good support may be adequate for short-term unit train operations at reduced speeds, relay with at least 132-lb. rail is recommended by most railroad engineers for general unit train traffic. The repetitive passage of heavy axle loads, especially on jointed track, severely punishes the track structure and accelerates deterioration. Welded rail of heavy section is recommended to minimize this deterioration but is not a complete answer. Heat treated or special alloy rail of lighter section will yield longer life and more economy than standard rail in track having sharp curvature and handling unit trains. However, selection of a rail section significantly lighter than 132-lb. rail should take into account the effects of fatigue on rail life and the need for more frequent surfacing cycles. Also, experience has shown that even heavy rail will incur damage due to high contact pressure between wheels and rails. New rail will deteriorate more rapidly than rail that has been work-hardened under lighter traffic. Grinding of the rail heads will restore the rail head profile and slow the deterioration as long as the grinding is not overdone. Also, the use of higher quality and, therefore, more expensive rail, will retard the deterioration. However, until less costly and improved metallurgy is provided, greater expenditures for relays should be anticipated. On the other hand, the use of hardened and heat treated, and thus more expensive, rails in curve territory is warranted where there is heavy traffic. Additional measures which will prolong rail life are the transposition of curve rails, the use of rail lubricators and laying special alloy rails in high wear areas.

There are other items to be considered in 100-ton car unit train operations such as dynamic braking, train operations and to expedite the movement of trains, the extension of sidings and the installation of longer turnouts and speed signalling. The installation of dragging equipment detectors and hotbox detectors will reduce train delays by decreasing the number and severity of derailments. Also, where traffic volumes are high or visibility problems occur train delays can be cut by the use of cab signalling. All of these items will add to maintenance of way cost. Some of these items would be necessary due to movement of frequent long trains regardless of weight, but where long trains are not being operated the cost of these items must be considered when unit train operation is introduced. The railroad engineer must identify those areas where improvements have to be made to handle anticipated traffic and justify to management the need for additional expenditures. In the final analysis, however, the availability of funds will dictate what can and what will be done to hold maintenance costs to a minimum.

This report is submitted as information with the recommendation that the assignment be continued for further study regarding the feasibility of developing specific cost figures.

REFERENCES

1. AREA Bulletin 635, Proceedings Volume 73, Page 239, "Determination of Maintenance of Way Expense Variation with Vaiious Traffic Volumes and Effect of Using Such Variation, in Terms of Equated Mileage or Other Derived Factors, for Allocation of Available Funds to Maintenance of Way".
2. AREA Bulletin 646, Proceedings Volume 75, Page 566, "Determination of Maintenance of Way Expense Variation with Various Traffic Volumes and Effect of Using Such Variation, in Terms of Equated Mileage or Other Derived Factors, for Allocation of Available Funds to Maintenance of Way".
3. U.S. Department of Transportation, Federal Railroad Administration, "Procedures for Analyzing the Economic Costs of Railroad Roadway for Pricing Purposes," January 1976 Final Report, Report No. RPD-11-CM-R, Volume 1, DOT-FR-30028.
4. AREA Bulletin 666, Proceedings Volume 79, Page 191, "On the Prediction of the Fatigue Life of Rails".
5. AREA Bulletin 673, Proceedings Volume 80, Page 514, Address by Allan M. Zarembski "Effect of Rail Section and Traffic on Rail Fatigue Life".
6. AREA Bulletin 653, Proceedings Volume 76, Page 616, Address by G. H. Way, "Heavy Cars—What Are the Issues?" and Page 622, Address by R. E. Ahlf, "Heavy Four-Axle Cars and Their Maintenance of Way Costs".
7. AREA Bulletin 661, Proceedings Volume 78, Page 427, "Additional Maintenance Cost Due to Operating 100-Ton-Car Unit Trains".
8. AREA Bulletin 663, Proceedings Volume 78, Page 611, Address by J. R. Sunnygard, "Effect of Heavy Cars on Rail".
9. Interstate Commerce Commission Docket No. 36612, Incentive Rate On Coal—Gallup, New Mexico to Cochise, Arizona—Verified Statement of E. C. Honath".
10. AREA Bulletin 660, Proceedings Volume 78, Page 110, "Manual Recommendations—Ballast".
11. AREA Bulletin 673, Proceedings Volume 80, Page 428, Address by Gerald P. Raymond, "Ballast Properties That Affect Ballast Performance".

Report of Committee 22—Economics of Railway Construction and Maintenance

W. Glavin, *Chairman*
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Committee

(E) Member Emeritus.

Those whose names are shown in boldface, in addition to the chairman, vice chairman and secretary, are the subcommittee chairmen.

To the American Railway Engineering Association:

Your committee reports on the following subjects:

2. Study to establish new equated mileage parameters.

   This subcommittee has developed a report on this subject for inclusion in the 1980 Manual revision. A copy of it can be found at the end of this report.

4. Economics of tie renewals by cutting vs. one piece renewal.

   The Subcommittee's report on this subject has been edited and should be finalized in early 1980.

The Committee on Economics of Railway Construction and Maintenance,
W. Glavin, *Chairman*

Manual Recommendation

Committee 22—Economics of Railway Construction and Maintenance

Report on Assignment 2

H.R. Davis (Chairman, Subcommittee)

Study to Establish New Equated Mileage Parameters

1979

INSTRUCTIONS FOR USE OF EQUATED MILEAGE PARAMETERS

TABLE 1: This table will produce a direct cost ratio between various track maintenance functions and FRA track classifications, using Class 4 first main track as unity.

EXAMPLES: (a) Compare branch line Class 3 track maintenance cost with that of Class 4 first main track:

Answer $\frac{0.72}{1.00} = 0.72$

(b) Compare cost to maintenance Class 3 branch line track with Class 4 branch line track:

Answer $\frac{0.72}{0.90} = 0.80$

TABLE 2: Additional factors have been established that will apply within a particular class of track to adjust Table 1 for conditions that either contribute to or reduce track maintenance cost.

EXAMPLE: (a) Three unit trains per day are operated over Class 3 first main track with axle loads of over 66,000 lbs. The track is 132# continuous welded rail. What would be the expected ratio increase in maintenance costs if track was changed to Class 4:

Answer $\frac{1.06 \times 1.70 \times 0.80 \times 1.02 \times 1.00}{1.02 \times 1.50 \times 0.76 \times 1.01 \times 0.87} = 1.44$

TABLE 3: This tabulation permits a comparison between various FRA track classes in relationship to the annual tonnages the track carries. It uses Class 4 track with 20 to 25 million gross tons per year as the unity figure. This table is to be used independently of Tables 1 and 2.

EXAMPLE: (a) How does track maintenance cost of Class 3 track with 13 MGTZ per year compare with Class 4 track with 28 MGT per year:

Answer $\frac{0.73}{1.07} = 0.68$

APPROVED

Sht. 1 of 2

A.R.E.A. COMMITTEE 22 SUB COMMITTEE 2

STUDY TO ESTABLISH NEW EQUATED MILEAGE PARAMETERS

		A.R.E.A. PUBLISHED PARAMETERS	CLASSES of TRACK-OPERATING SPEED LIMITS: CLASS 1, SPEED FRT/PSGR 10/15	2, 25/30	3, 40/60	4, 60/80	5, 80/90	6, 110/110
MAIN TRACKS 1ST	MILE	1.00	.55	.69	.87	1.00	1.13	
2ND	"	0.83	.45	.58	.78	.89	1.01	
3RD & 4TH	"	0.75	.37	.52	.67	.77	.95	
BRANCHLINE TRACKS	"	0.49	.50	.52	.72	.90		
OTHER TRACKS								
PASSING & THOROUGHFARE	"	0.43	.32	.43	.50	.80		
CTC PASSING TRACKS	"		.40	.63	.83	.95		
YARD & SIDE TRACKS	"	0.32	.39	.50				
SWITCHES								
MAIN TRACK	EACH	0.07	.04	.05	.12	.12	.15	
SIDE TRACK	"	0.05	.03	.08	.09			
POWER OR SPRING	"		.06	.07	.17	.19		
RAILWAY CROSSINGS	"	0.10	.10	.15	.18	.20	.24	
ROAD CROSSINGS								
PAVED ST OR HIGHWAY	EA/TR	0.07	.09	.09	.09	.10	.10	
UNPAVED ST OR HWY	" "	0.03	.04	.05	.05	.06	.06	
UNIMPROVED ROAD	" "	0.01	.02	.02	.03	.03	.03	
FARM OR PRIVATE	" "		.02	.02	.02	.02	.02	
FACTORS TO BE APPLIED TO ABOVE FOR:								
CWR			.59	.70	.76	.80	.82	
CURVES 0°-2°			1.03	1.03	1.04	1.05	1.08	
2°-4°			1.20	1.22	1.25	1.30		
4°-6°			1.40	1.42	1.50			
OVER 6°			2.00	2.02	2.23			
AXLE LOADS 45000 Lbs			.95	.95	.95	.96	.97	
55000 "			1.02	1.02	1.02	1.06	1.09	
66000 "			1.24	1.30	1.30	1.40	1.45	
OVER 66000 "			1.50	1.50	1.50	1.70	2.07	
UNIT TRAINS-EACH DIRECTION 1-5 PER DAY			1.02	1.02	1.02	1.06	1.09	
OVER 5 " "			1.09	1.09	1.13	1.18	1.29	
RAIL WEIGHT-UNDER 100 Lb/Yd			1.08	1.09	1.16	1.43		
100-116 "			1.05	1.05	1.05	1.20		
116-132 "			1.00	1.00	1.01	1.02	1.02	
OVER 132 "			.83	.90	.92	.95	.97	
BALLAST-CRUSHED ROCK			.90	.93	.96	.98	1.00	
-CR WASHED & SCREENED			.95	.96	1.00	1.20		
-CR PIT RUN GRAVEL			1.04	1.04	1.12			
-PIT RUN GRAVEL			1.06	1.11	1.22			

3

CLASS of TRACK-OPERATING SPEED LIMITS

MILLION GROSS TONS PER YEAR	CLASS 1 SPEED MPH FRT/PASS 10/15	2 25/30	3 40/60	4 60/80	5 80/90	6 110/110
0-5	.39	.50	.56	.70	.75	
5-10	.44	.56	.64	.74	.83	
10-15	.51	.62	.73	.84	.93	
15-20	.56	.67	.81	.90	1.03	
20-25	.63	.75	.89	**1.00**	1.14	
25-30	.73	.81	.95	1.07	1.23	
30-35	.74	.89	1.00	1.12	1.32	
OVER 35	.82	.93	1.05	1.22	1.41	

June 12, 1979

Report of Committee 24—Engineering Education

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Vice Chairman
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E.N. Wilson
B.J. Worley
A.M. Zarembski
R.W. Zimmer

(E) Member Emeritus

Those whose names are shown in boldface, in addition to the chairman, vice chairman and secretary, are the subcommittee chairmen.

To the American Railway Engineering Association:

Your committee reports on the following subjects:

A. Recommendations for further study and research

E. Y. Huang has submitted a report published in Bulletin 675, September-October 1979, on railroad engineering education as viewed by editors in the industry. This report was very well received by members of the committee and has been of great assistance to the consultants retained by the FRA in connection with railraod engineering needs.

1. Recruiting

The committee on recruiting submitted a report last year indicating job progression of top railroad officials.

4. Student affiliates

The student affiliate group under the very active leadership of Chuck Chambers has worked closely with other AREA committees to re-establish the student design contest. As you are well aware, Committee 13, Environmental Engineering, will hold the contest for 1980. Committee 6, Buidlings, will sponsor the contest for 1981. Committee 8, Concrete Structures and Foundations, will sponsor the contest in 1982. With the assistance of the AREA headquarters group, we plan to expand on our student affiliate membership.

5. Continuing education

Proposals have been submitted to the committee membership in connection with seminars to be held concurrent with the AREA regional meetings. After discussion these proposals will be implemented if desired.

6. Speakers

This committee has been rather dormant and will be reviewed at the next meeting.

The Committee on Engineering Education,
C.T. Popma, *Chairman*

Report of Committee 28—Clearances

D. W. LaPorte, *Chairman*
E. E. Kessler, *Vice Chairman*
J. E. Beran, *Secretary*

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R. R. Snyder
J. E. Teall
D. J. Moody
M. L. Power
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J. M. Quesada
E. W. Jantz
R. E. Barrows
E. W. Berendt
E. S. Birkenwald (E)
A. V. Bodnar
E. C. Castellanos
J. W. Coles
J. A. Crawford
D. D. Cunningham
S. M. Dahl (E)
G. D. Graff
G. E. Henry
C. F. Intlekofer
R. G. Klouda
A. J. Kozak
A. E. Mooney
W. E. Morgus
H. D. Niebur
J. T. Ormsby
C. E. Peterson (E)
L. Schmitz
P. A. Schoville
W. P. Silcox
E. C. Smith
C. C. Smoot
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L. H. Wantanabe
A. P. Wivagg
G. Wilwerding

Committee

(E) Member Emeritus.

Those whose names appear in bold face, in addition to the chairman, vice chairman and secretary, are the subcommittee chairmen.

To the American Railway Engineering Association:

Your committee reports on the following subjects:

B. Revision of Manual.
Former Sub-Committee 5's assignment, revise suggested method of presenting published clearance now in the manual, is completed and has received the necessary votes by the full committee. The revised methods were included in Bulletin 675.

1. Investigate the Practicability of Using Disposable Placards or Other Appropriate markings for Identifying Shipments of Excessive Dimensions and/or Weight.
Committee to further investigate and explain all advantages to AREA Board.

2. Compilation of Railroad Clearance Requirements of the Various States.
The chart showing these requirements is now in the manual as a matter of information and will be updated as significant changes are reported.

3. Investigate the criteria for handling heavy shipments, collaborating as necessary or desirable with Committees 5, 7 & 15. A questionnaire was sent to all members of Committee 28 asking their practice for handling overweight shipments in relation to their rail and bridges. These questionnaires are being studied and will serve as a basis for further recommendations on this subject. This assignment is scheduled for completion in Oct. 1980.

4. Restudy and Possibly Revise "Clearance Diagrams-Fixed Obstructions" now in the Manual.
   The continuing increase in the number of large size shipments has dictated the need for larger clearances on new structures for larger clearance routes in the future. The larger clearance diagrams have been completed except for a few minor changes. Completion and Committee approval is scheduled for February 1980.

6. Study the Effects of Shipment Center of Gravity in Relation to Train Speed and Track Curvature.
   We are, at this time, waiting the possibility of computer test runs at the AAR Research Center and test to be conducted at the Fast Loop II Project at Pueblo, Co. This is a very technical subject and the computer runs and testing at the Fast Loop Track seem the only way to get final and definite results.

7. Liaison Committee to Work with AAR Management Systems Department In Implementation of Umler Phase II to include needed Car Characteristics Data for Use in the Official Railway Equipment Register.
   This Committee continues to monitor any activities with the AAR Management Systems Department in connection with Umler Phase II, adding of data to the Equipment Register.

8. Restudy of Clearance Allowances for Horizontal Movement of Passenger Cars due to Lateral Play, Wear and Spring Deflection. Response was recently received from Amtrak on Amfleet Tilt Test by the Budd Co. and by Pullman Standard for Bi-level Superliners. The data when submitted by Amtrak should be sufficient for this Committee assignment to be advanced to the final stages.

9. Methods of Modifying and/or By-passing Obstructions for Increased Clearances.
   This assignment discontinued.

10. Review the Form for Reporting Loads Which Exceed Line Clearances now in Chapter 28 of the Manual.
    A Composite form was developed from forms collected from about 20 railroads. This form was sent to each Committee Member for their study and use. A final Committee vote should take place in February, 1980.

The Committee on Clearances,
D. W. LaPorte, *Chairman.*

Report of Committee 33—Electrical Energy Utilization

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Chairman
R.U. Cogswell
Vice Chairman

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H.S. March
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D.M. Twine
K.B. Ullman
E.F. Weitz
H.W. Wittmann
T.P. Woll

Committee

Those whose names are shown in boldface, in addition to the chairman, and vice chairman are the subcommittee chairmen.

To the American Railway Engineering Association:

Your committee reports on the following subjects:

A. Recommendations fur further study and research.

1. Electrification Economics.

   A section entitled "Factors to Consider in Making Electrification Economics Studies" was adopted in 1976 and is now included in the AREA Manual. Future assignments include a review of the costs used in North American Electrification studies in the January-February 1974 Bulletin.

2. Clearances.

   A narrative on clearances was adopted in 1978 and is now included in Part 2 of the manual. The narrative supplemented the Clearance Drawing adopted in December 1975. Work continues on a clearance profile for electrified railroads at 25kVac and 50 kVac. DC clearances are being investigated.

3. Voltage standards.

   Part 3 of the Manual includes "Recommended Voltages" adopted in 1978. At present this Committee is on stand-by having completed its assignments.

4. Catenary/Pantograph Systems.

   A uniform method for computing catenary ampacity was adopted in 1978 and is now included in the Manual-Part 4. Future work includes preparation of an errata sheet for contact wire ampacities, glossary of terms, envelope for electrical clearances and short-term overload ampacity ratings.

5. Signals and Communications.

   Ballots have recently been received on the section entitled "Signal Compatibility with Railway Electrification" for inclusion in Part 5 of the Manual. Further work will include the effects of the power distribution system on wayside signal circuits and the use of three-phase traction motors.

6. Power Supply and Distribution.

   A section for the Manual has been prepared entitled "Power Supply Requirements, Railroad Electric Traction System", voted on by the entire Committee. A new electric power assignment has been suggesteded: Moveable bridge control and electric power.

7. Contact Rails.

   This committee was abandoned since expertise did not exist on committee 33 to develop technical recommendations

8. Wire and Cables.

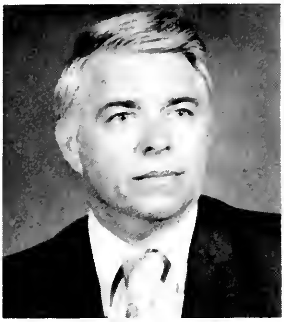
   This committee has been retired since cable and open wire standards can generally be obtained from anaufacturers or suppliers catalogues on railway equipment.

9. Illumination.

   A review of the various standards that are presently available is now being carried out. It is intended that a table with various light efficiencies be included in the new Part 10 section on Illumination in the Manual. It has also been suggested that a safety level for proper illumination should be investigated.

The Committee on Electrical Energy Utilization,
L. D. Tufts, *Chairman*

Report of Committee 34—Scales



H. E. Buchanan, *Chairman*
N. A. Wilson, *Vice Chairman*
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E. L. Robinson
J. J. Robinson
E. Szaks
K. D. Tidwell
L. J. Walker
P. C. White
R. S. Yarnell
J. Zaenger

Committee

Those whose names are shown in boldface, in addition to the chairman, vice chairman and secretary, are the subcommittee chairmen.

To the American Railway Engineering Association:

Your committee reports on the following subjects:

B. Revision of Manual.
A new chairman was appointed to complete revision of the manual.

2. Statistical Data for Coupled-in-motion Weighing and Testing.
Submitted a report indicating that test results of coupled in motion track scales, while meeting established tolerances, will continue until year's end.

3. Innovations in track scales.
Asked the scale manufacturers to furnish design specifications for scales not requiring pits.

4. Metric Planning for Track Scales.
Study indicates that there is no great compelling force for the U.S. track scale industry to go metric at this time.

The Committee on Scales,
H.E. Buchanan, *Chairman.*

SPECIAL REPORT

Laboratory Investigation Of Track Gauge Widening +

Allan M. Zarembski*

John Choros**

ABSTRACT

This paper presents the results of a series of track gauge widening tests conducted at the Association of American Railroad's Track Structures Dynamic Test Facility. The tests investigated the gauge widening behaviour of conventional track structure under various combinations of vertical, lateral and longitudinal loads. The effect of single axle *vs* dual-axle loading and static *vs* dynamic lateral loading were also examined.

The tests indicated that under loading representative of that imposed by traffic, significant widening of the track gauge can occur. It was further observed that the level of damage to the tie—fastener interface can be measured and evaluated by means of gauge widening type testing and that the potential exists for conducting "nondestructive" gauge widening tests in service track.

INTRODUCTION

With the introduction of the modern T-rail section into the railroad track structure, the potential for track gauge widening and rail overturning soon became evident. These phenomena, which are caused by excessive lateral forces applied to the head of a rail, can result in train derailments, usually occuring when the gauge widens sufficiently under load to allow the wheels to drop down. In order to prevent derailments from this condition and maintain a suitable track integrity, the track engineer must understand the behavior of his track structure under traffic loading and be able to determine to what extent his track has been "weakened" by traffic. The purpose of this study was to examine, more fully, the gauge widening behavior of conventional track structure and to determine if track damage resulting from excessively-high lateral forces can be detected without further damage to the track.

The historical background of track gauge widening is well documented (1). Many well-known investigators have studied gauge widening of one type or another. A. N. Talbot studied bending stresses in rails due to lateral forces (2), and railhead deflections and rail rotation in tangent track (3). S. Timoshenko and B. F. Langer looked at the torsional resistance of rail, and various methods for measuring lateral forces applied to rail (4). Numerous other investigators (1) have studied gauge widening under static and dynamic loading conditions in order to determine safe wheel loads and train speed.

In order to answer some of the outstanding questions on the phenomenon of gauge widening, to determine if nondestructive gauge widening testing was feasible, and to generate a data base for field data correlation, a series of tests were conducted at the Association of American Railroads' Track Laboratory located in Chicago, Illinois.

+ Research sponsored by the Federal Railroad Administration under Contract DOT-FR-30038, and the Track Train Dynamics's Track Strength Characterization Program

*Manager-Track Research, Association of American Railroads, Chicago, Illinois

**Track Research Engineer, Association of American Railroads, Chicago, Illinois

This report presents the results of this gauge widening test series and discusses the significance of these results.

OBJECTIVE

The purpose of this series of tests was to quantify the lateral resistance characteristics of track, to investigate the ultimate strength and failure modes of track due to high lateral loads on the rail, and to determine if nondestructive gauge widening tests can be used to detect damage or weakened track. To accomplish this goal, the gauge widening test series was designed and conducted using the following guidelines (5).

1. Conduct a sequence of single-point lateral rail loadings, whereby the track gauge is progressively damaged, in order to quantify the basic gage widening mechanism and determine if "nondestructive" loading and measurements can be used to identify deteriorated track conditions.

2. Continue these tests until the track's resistance to gage widening is "seriously weakened," and determine what nondestructive loading levels are required to detect this weakened condition.

3. Further reduce the resistance to gage widening by sequentially removing first the gage spikes, and then the field spikes, to determine what effects the missing spikes have on gage widening resistance.

4. Determine the effects of vertical and lateral loads from the adjacent axle on gage widening.

5. Determine the effects of dynamic lateral loads on gage widening

6. Determine the effects of longitudinal rail loadings of gage widening.

BASIC GAGE WIDENING TEST

This series of single point gage widening tests was designed to progressively weaken the test track by the application of combined lateral and vertical loads, using predetermined gage widening limits, e.g. maximum allowable railhead lateral deflections of 0.25, 0.05. 1.0 and 2.0 inches. With the track in "weakened" condition, two additional series of tests were conducted in which nine gage spikes and then nine field spikes were sequentially removed (6).

Test Procedure and Instrumentation

The test track used for the entire test series was of conventional North American construction consisting of 136 RE Rail, AREA #5 cross ties, (7 inches by 8 inches by 9 feet), spaced on 19.5 inch center, AREA #12 tie plates, with two cut spikes per plate (spikes were fully driven), AREA #4 limestone ballast, (12 inches deep) with 12 inch shoulders, and Illinois Specification CA-8 limestone subballast, (6 inches deep).

Track loading was accomplished using two 50-ton capacity hydraulic jacks for vertical loads, and one 25-ton capacity hydraulic jack for lateral loads, as shown in Figure 1.

Vertical and lateral wheel loads were applied to both railheads by means of a specially-designed loading fixture (6). Use of this fixture resulted in vertical load application 0.50 inch from the 136 RE railhead center line, and lateral load application 0.69 inch below the top of the rail. During the tests, vertical and lateral loadings were applied equally to both rails. All applied loads were measured by strain gage load cells inserted between the loading fixture and the jack stilts.

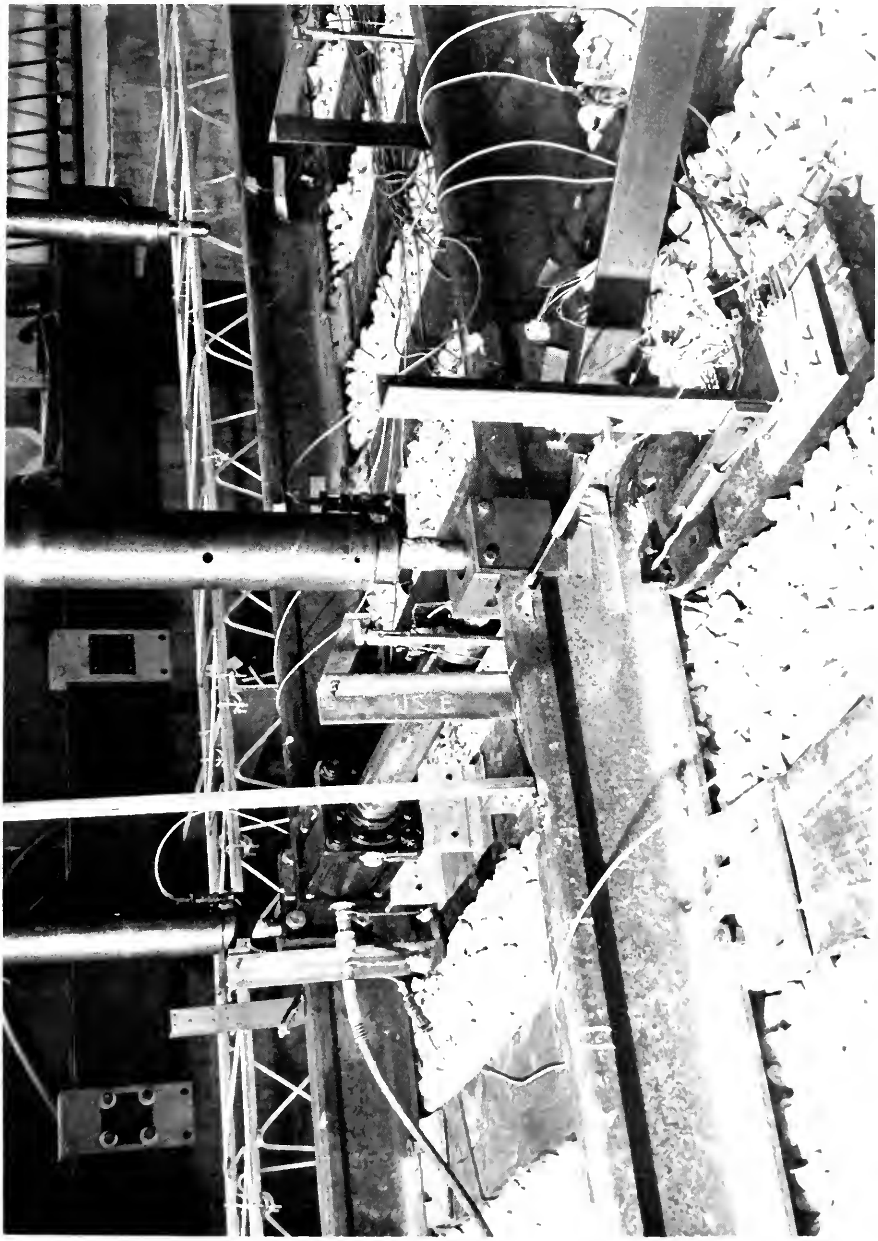


Figure 1. Basic Gage Widening Tests - Hydraulic Equipment for Vertical and Lateral Load Applications During Simulated Single Axle Loadings.

Rail deflections were measured at specific locations on both rail, e.g. 0, 39, and 78 inches from the east rail loading point. Three sets of deflections were measured at each location: railhead and base lateral, and rail base vertical, i.e. gage side rail base vertical displacement (Figure 2). All rail deflections (railhead, rail base-lateral, rail base-vertical) were measured relative to the ties. Prior to load application, zero reference readings (for all data channels) were recorded. During the actual loading sequences, the vertical load was applied initially followed by the lateral load which was increased until a predetermined deflection of the railheads was achieved. At this time a set of instrumented reading was taken. This procedure was continued until the defined gage widening limit was reached. The lateral load was then decreased to zero, the vertical load decreased to zero, and the procedure repeated.

It is important to note that, during the entire test series no attempt was made to rearrange or repair the track. The position of each rail after each test was taken as the reference zero for the following test.

Results*

As can be seen in a typical set of load deflection curves for *new* track in good condition, (Figure 3), most of the measured lateral railhead deflection results from rotation of the rail section rather than lateral translation of the rail base. Furthermore, noting the strong correlation between the railhead lateral deflection curve and rail base vertical deflection curves, it can be additionally surmised that little or no bending of the rail section takes place. This behavior is observed even under the largest combination of vertical and lateral loads (7).

Figures 4 and 5 present the combined vertical and lateral load levels necessary to produce ½ inch and one inch deflection of each *individual* rail head. As can be clearly seen in these figures, as the vertical load decreases, the amount of lateral load required to achieve an equivalent amount of rails head displacement, i.e. gauge widening, decreases. This indicates that under conditions where vertical unloading occurs, such as in spiral of curves, the potential for dynamic gauge widening increases.

Further examination of Figures 4 and 5 indicate that as the track is weakened by prior application of significant loading, and in the case of "weakened" track, where spikes are actually missing, the strength of the track, its ability to resist deformation under load, is significantly decreased. Although these "weakened" conditions obtained in the laboratory tests have not, as yet, been correlated with the track deterioration experienced in the field, one can extrapolate this tendency towards loss of trackstrength to field service conditions. Tests performed by the Canadian National Railroad (8) do in fact confirm this tendency.

Finally, examination of railhead deflection as a function of predamage history, i.e. track subjected to previous deflections of a given magnitude, as shown in Figure 6, indicates that load levels exist, at which it is possible to achieve sufficient sensitivity to level of predamage without requiring "unsafe" load levels. The curve corresponding to $L/V = 0.7$ in Figure 6, clearly indicates this sensitivity with the railhead experiencing displacements of less than 0.5 inches.

ADJACENT LOAD TEST

The objective of adjacent load test series was to study the effects of a second (adjacent) set of vertical and lateral loads on the gage widening characteristics of the track structure.

*For a complete set of test results the reader is referred to References (6) and (7).

Figure 2. Typical Measurement Location for Rail Deflections: Railhead Lateral and Rail Base Lateral/Vertical.

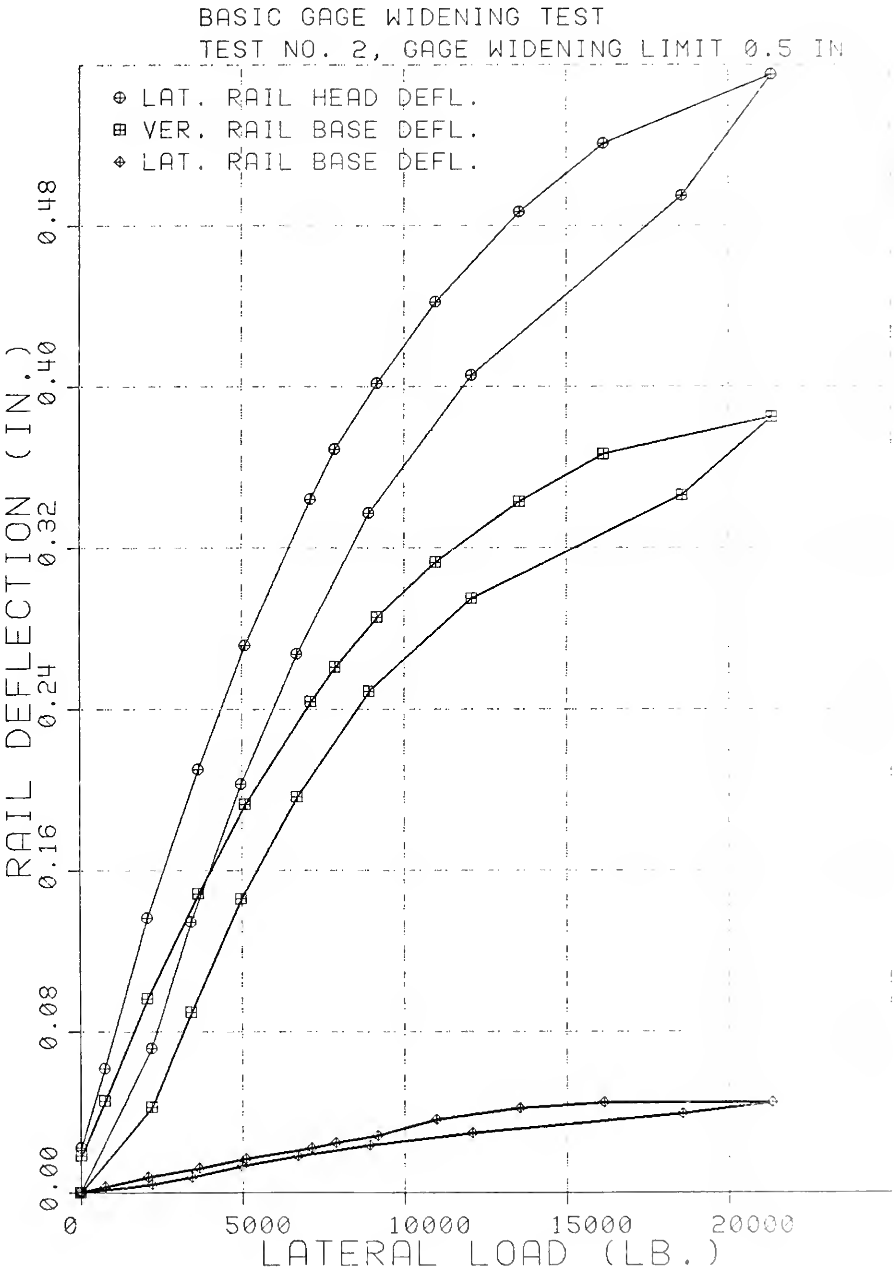


FIGURE 3. GRAPH SHOWING VARIOUS RAIL DEFLECTIONS VS LATERAL LOADS, FOR ZERO VERTICAL LOAD.

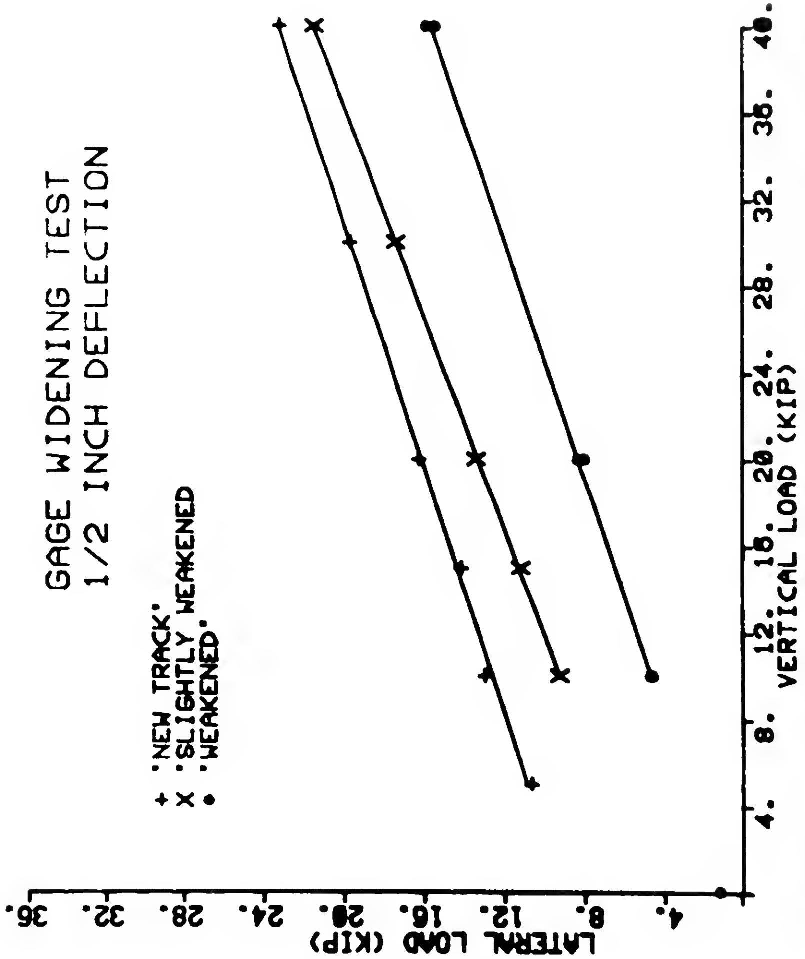


FIGURE 4 VERTICAL LOAD VS LATERAL LOAD, FOR 1/2 INCH RAIL HEAD DEFLECTION AND DIFFERENT LEVELS OF DAMAGED TRACK.

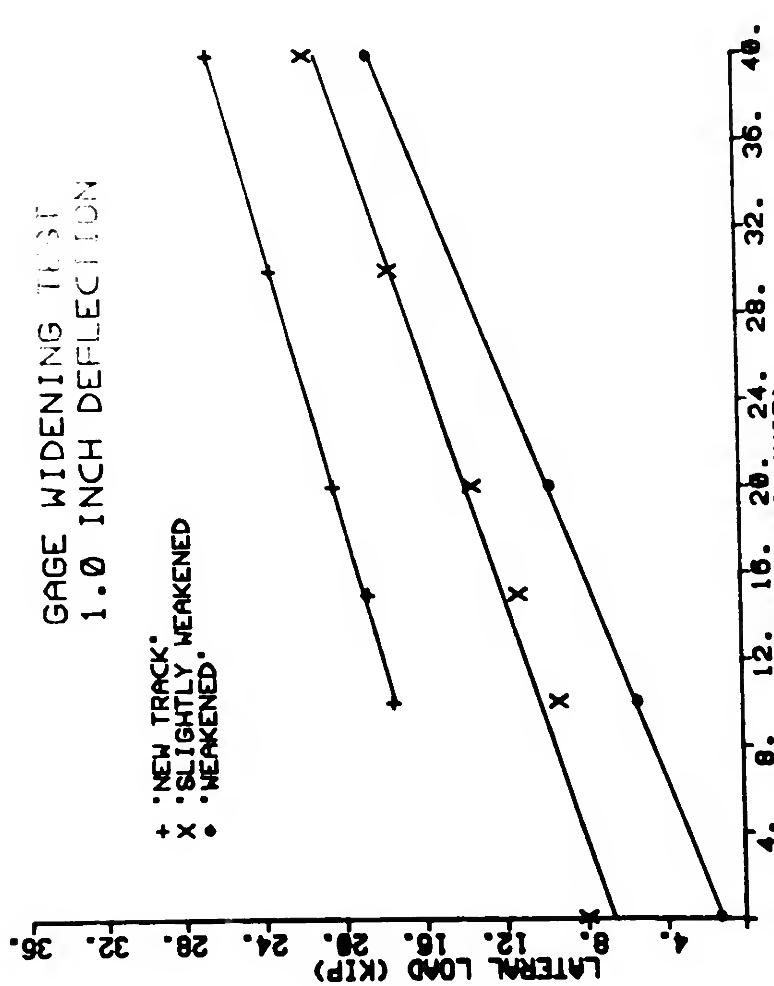


FIGURE 5 VERTICAL LOAD VS LATERAL LOAD. FOR 1.00 INCH RAIL HEAD DEFLECTION AND DIFFERENT LEVELS OF DAMAGED TRACK.

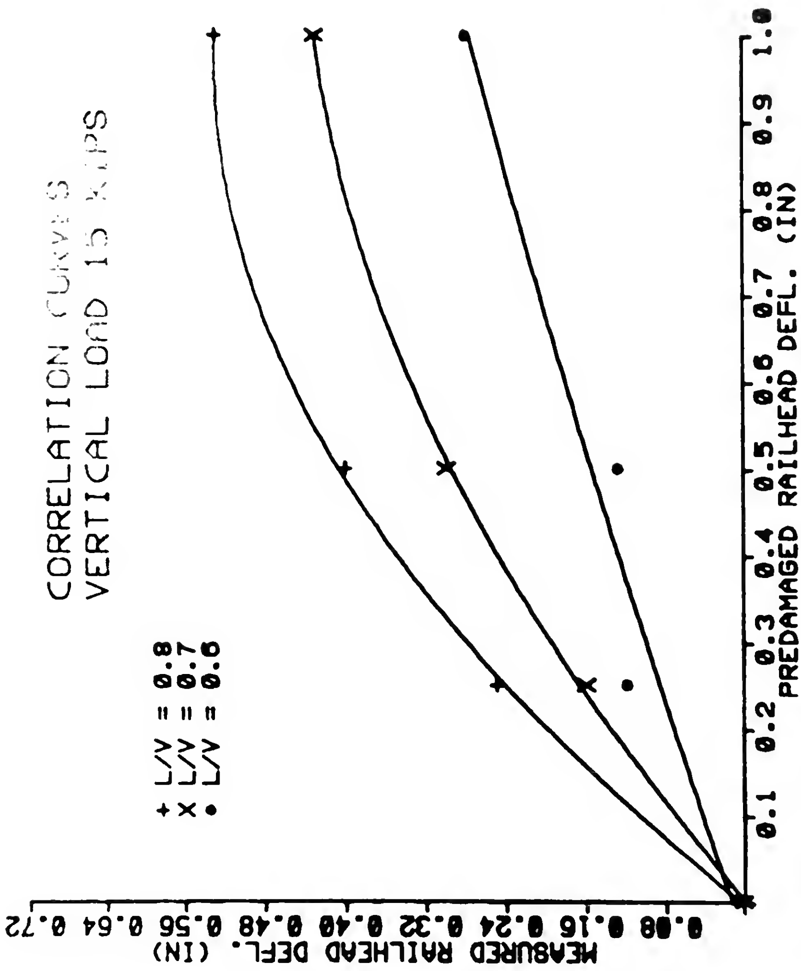


FIGURE 6 MEASURED VS PREDAMAGED RAIL HEAD DEFLECTIONS. FOR VARIOUS L/V LOADING RATIOS, AT CONSTANT VERTICAL LOAD.

Test Procedure and Instrumentation

The test track used in the adjacent load test series was the same one used in the basic tests. The spike holes resulting from the pulled spike tests were filled with Racine Tie Saver and new spikes driven. The remaining spikes were checked, and either redriven or replaced after using Tie Saver to plug the spike holes.

The track was loaded in a manner similar to the previous test test series, except that a second set of vertical and lateral jacks were applied 70 inches from the primary set, thus approximating the wheel spacing of a 100-ton capacity freight car truck. Both sets of vertical and lateral applied loads were independently controlled during the tests.

Deflections were measured at the four rail loading points. As in the previous test series, three sets of deflection data were taken at each location: lateral railhead, and vertical and lateral railbase deflection.

Results*

The effect of the adjacent vertical load, located 70 inches from the primary load is illustrated in the lateral load *vs* railhead deflection curves given in Figure 7.

It can be observed from this figure that for railhead displacement greater than .1 inches, the presence of the adjacent vertical load has an apparent "stiffening" effect on the track structure. This stiffening effect can also be seen in Figure 8, which presents the primary lateral and vertical loads necessary to deflect the railhead 0.4 inches for different adjacent vertical loads. For increasing adjacent vertical loads, the lateral load required to displace the railhead 0.4 inches increases.

Figure 9, presents a camparison of single axle L/V ratio with simulated truck L/V ratio under various vertical and lateral load combinations. It is significant to note that for single axle L/V ratio below 0.5, no significant railhead deflection is observed. However, for truck** L/V ratio below 0.5, railhead deflections of 0.4 inches were measured. It is therefore concluded that the use of truck L/V ratios for evaluating the potential for gauge widening be used with caution, and whenever possible, single axle L/V ratio should be used as the gage widening and rail overturning criterion.

DYNAMIC LOADING TEST

In order to examine the effects of dynamically-applied lateral impulse loads on gage widening, a series of dynamic load tests were conducted.

Test Procedure and Instrumentation

The set-up for the dynamic gage widening test series differed from the previous tests in that lateral loading was applied only to one rail. The same test track was used, however the west rail was braced (at a point opposite the load application point on the east rail) to prevent movement.

The Amsler hydraulic system was used to apply static vertical loads to both test track rails, as in the earlier test series. The lateral load, applied to the east rail only, was obtained from

*For a complete set of test results the reader is referred to References (6) and (7).

**Truck L/V ratio (L1 + L2)/(V1 + V2)

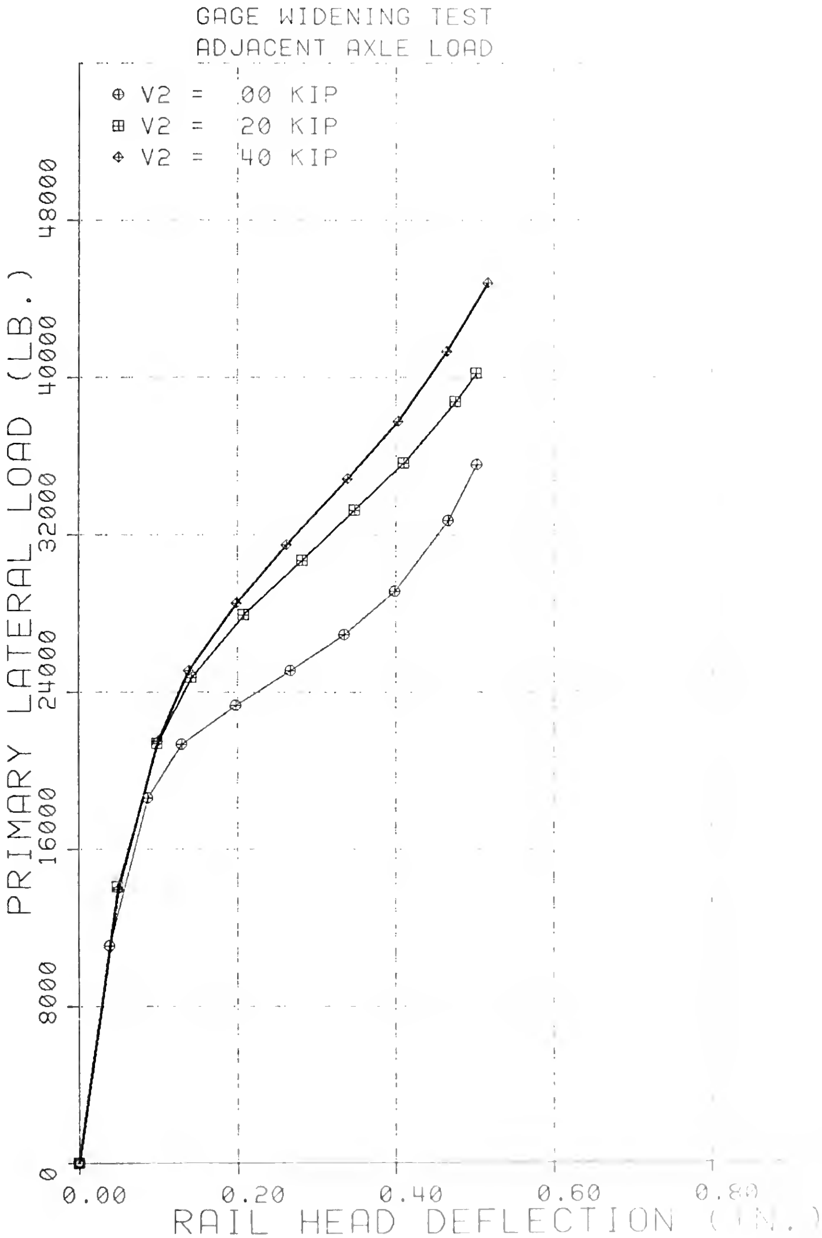


FIGURE 7. RAIL HEAD DEFLECTION VS PRIMARY LATERAL LOADS, FOR VARIOUS ADJACENT VERTICAL LOADS, AND CONSTANT 40 KIP PRIMARY VERTICAL LOAD.

GAGE WIDENING TESTS - SIMULATED
ADJACENT AXLE LOADINGS

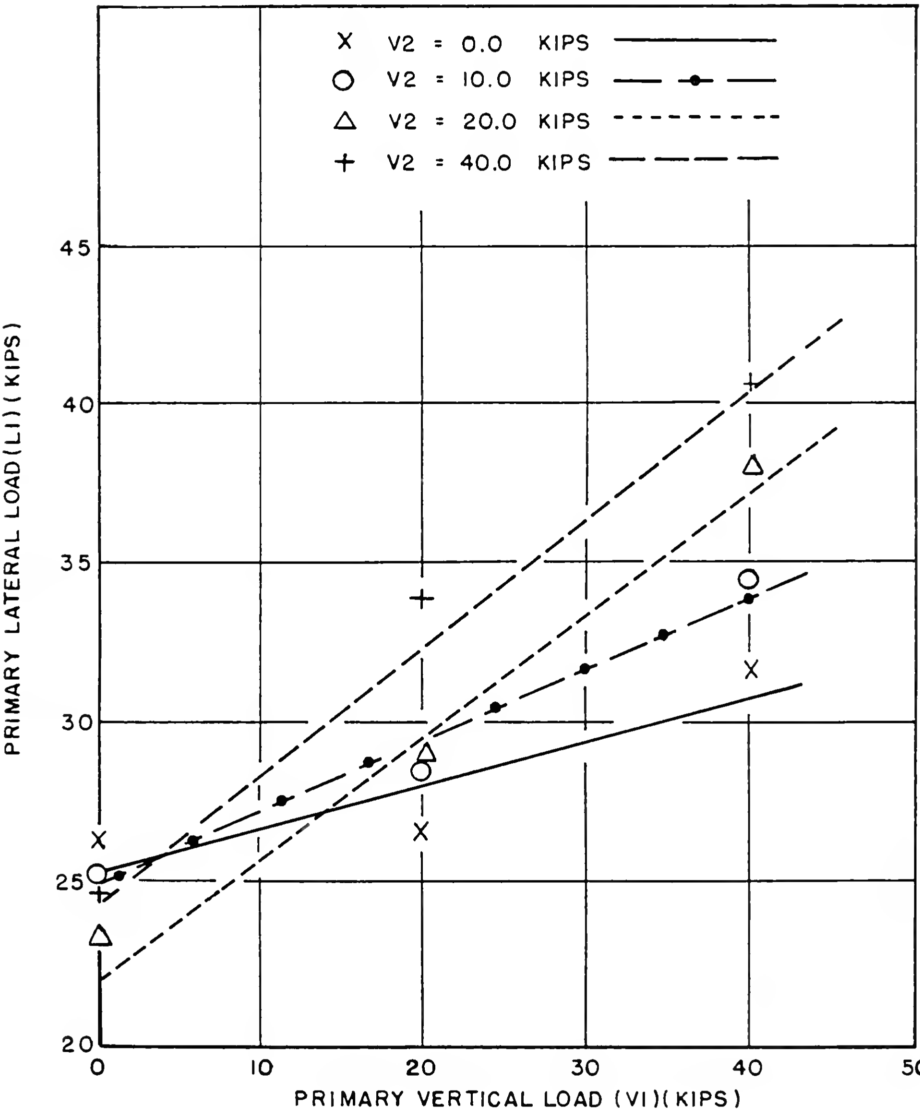


FIGURE 8. PRIMARY VERTICAL VS. PRIMARY LATERAL LOADS REQUIRED TO PRODUCE 0.4 INCH OF RAIL HEAD DEFLECTION, FOR VARIOUS ADJACENT VERTICAL LOADS.

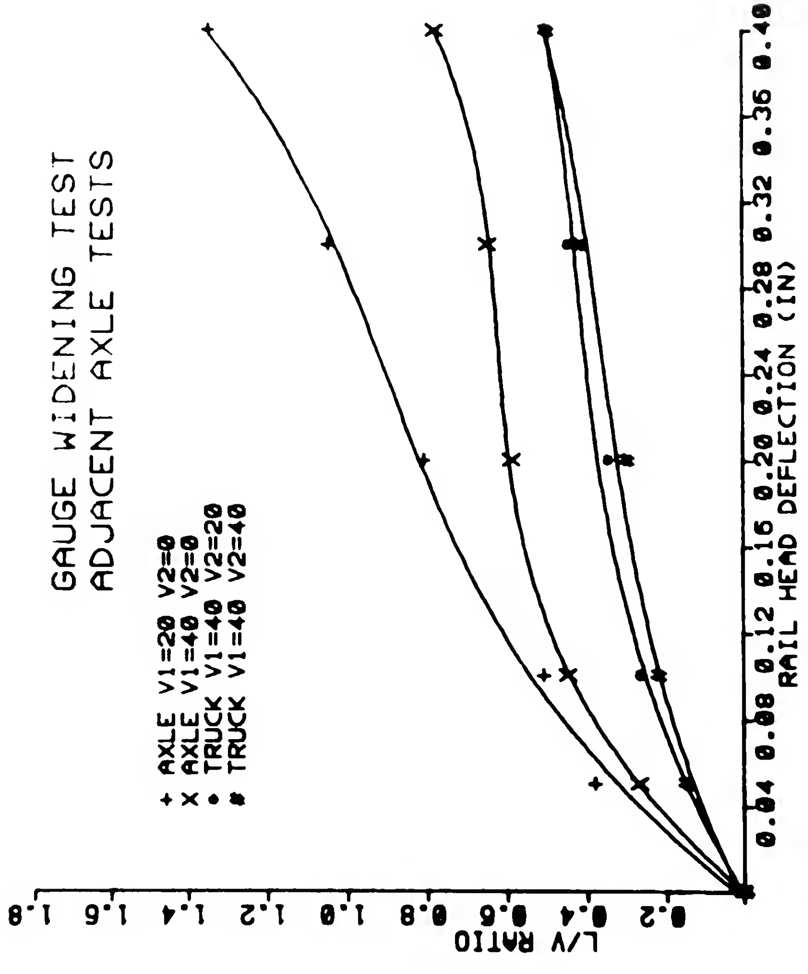


FIGURE 9 COMPARISON OF AXLE AND TRUCK L/V RATIOS FROM ADJACENT AXLE TESTS.

the same hydraulic system used in the earlier test runs, except for system modification* to enable the generation and application of dynamic impulse loads of variable time duration.

The lateral impulse load time durations were controlled by means of directional control valves. Load cells, of the type discussed earlier, were used to measure lateral load amplitudes and time durations. The east railhead lateral and rail base lateral and vertical deflections were measured at the lateral loading point.

Results**

Due to limitations in the hydraulic loading system, only a limited set of dynamic loads, with durations greater than 200 milliseconds were obtained. Thus only limited conclusions could be drawn from the test results.

Examination of Figure 10, which compares the impulse load and and railhead response wave forms shows that:

1. The in-phase nature of the two wave forms indicates no physical separation occuring between the impulse loading jack and the rail.
2. The dynamically-loaded railhead exhibits a slightly oscillatory displacement wave form, which correlates with the results from published mathematical models (9) for this type of system.
3. The railhead returns to its original (unloaded) position within the first 100 milliseconds after the lateral impulse load is released.

LONGITUDINAL LOAD TEST

In order to determine the effects of longitudinal compressive rail loads on gage widening and to create a data base of load deflection curve for various combinations of static vertical, lateral and longitudinal forces, a series of combined load tests were conducted.

Test Procedure and Instrumentation

The test track set-up for this test series was the same as for the previous dynamic load tests. The west rail bracing arrangement was retained, and all of the spikes were redriven.

During these tests, the track was subjected to combined vertical, lateral and longitudinal loads. The vertical loads were applied by the Amsler hydraulic system and measured using a strain-gauged load cell. Longitudinal load to the east rail were applied by means of hydraulic rail puller (Figure 11). Additional lengths of rod allowed the anvils to be placed 14 feet from the vertical and lateral loading points, which permitted a total test track length of 28 feet. The rail pullers were capable of applying a 240 Kip (maximum) conpressive load to the rail. Lateral loads, which were also measured with strain-gauged load cells, were applied using actuators and hand pumps, as in the previous test series. The various vertical, lateral and longitudinal static loads were applied to the east rail only.

Rail deflections were measured at the east rail loading point, and at distances of one, two, four and six tie centers from the loading point, in one direction only.

During each test, the vertical load was applied first, followed by the longitudinal load. The lateral load was then applied and incrementally increased until the desired maximum

*For a more complete description of the dynamic loading apparatus the reader is referred to Reference (6).

**For a complete set of results the reader is referred to References (6) and (7).

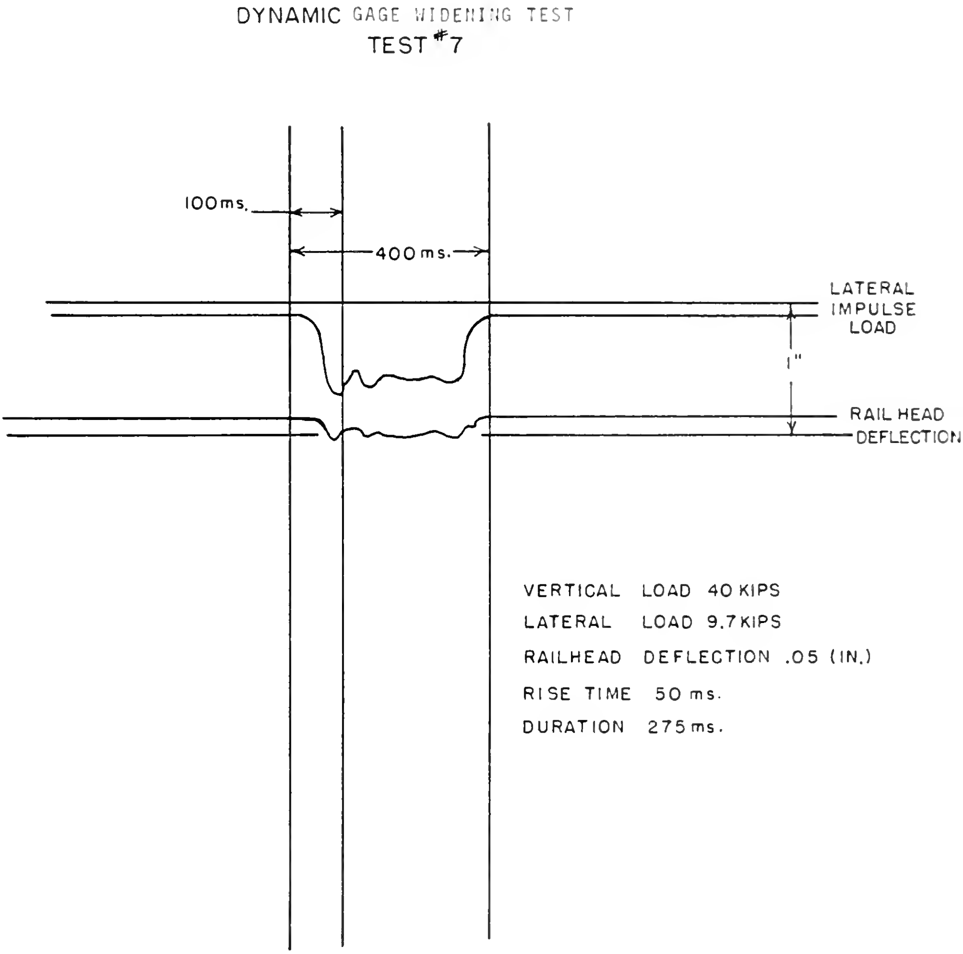


FIGURE 10. RECORDING OSCILLOGRAPH TRACE SHOWING WAVEFORMS OF 9.7 Kip LATERAL IMPULSE LOAD, AND RESULTING 0.05 INCH EAST RAILHEAD DEFLECTION, FOR A CONSTANT 40 Kip VERTICAL LOAD.

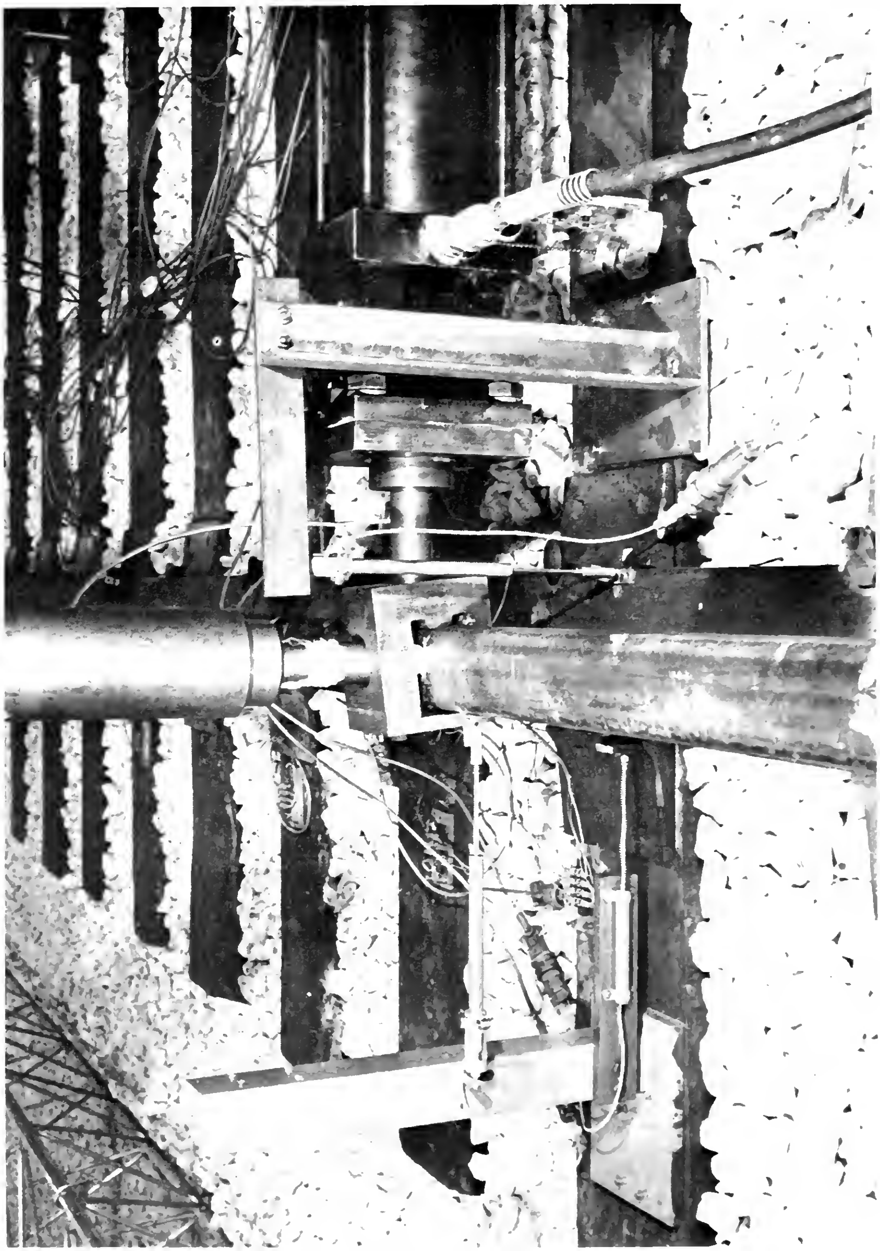


Figure 11. Test Set-Up for Longitudinal Load Tests, Showing Hydraulic Rail Puller in Position.

deflections was reached. Deflection data, corresponding to each increment of applied lateral load, was measured and recorded. The lateral load was then reduced incrementally to zero, the vertical and longitudinal loads released, and the procedure repeated for the next test in the loading sequence. The spikes were not redriven between tests.

Results*

The effect of longitudinal rail force applied in conjunction with simultaneous vertical and lateral loads is typified* by the results shown in Figure 12. Note that for the test track, which can be considered to be in "good" condition, the presence of up to 240,000 lbs of longitudinal load results in only a very limited increase in the railhead deflection. This general behavior is also shown in Figure 13, where the maximum lateral railhead deflection is plotted against longitudinal load for a constant 30,000 lb lateral load and several different vertical loads. Though there is some effect of longitudinal load, particularly for the 40,000 lb vertical load case, the order of magnitude of this effect is significantly less than that noted in an earlier test series (10).

Figure 14 presents rail deflections relative to the test section center line for various longitudinal loads at constant 20 Kip lateral and zero vertical loads. It also presents a comparison between the present laboratory test data and previously published field test data (9). The graph shows excellent agreement between the two data sets at zero applied longitudinal load, but poor agreement when longitudinal loads were present. Although the field test used 42 Kips, and the laboratory tests used 75 Kips of applied longitudinal load, the close proximity of the latter curve to the zero longitudinal force curve suggests that the two data curves should be extremely close. The field test lateral deflection were, however, about 174% higher than the corresponding ones obtained in the laboratory tests for "comparable" load levels. The authors believe that the observed discrepancies can be explained by the differences between the two track sections and the difference in test procedures. The laboratory test track had new 136 RE rail and new cross ties. The field test track was a section of branch line with worn 100 lb. rail and 25 year old hardwood ties. The sequence of load application for the two tests was also different. Though both tests applied the vertical load first, in the laboratory tests the longitudinal load was applied next, followed by the lateral load. The field tests reversed this order of lateral and longitudinal load application, which would suggest the presence of a lateral eccentricity in the rail at the time the longitudinal load was applied. As a result, the longitudinal forces produced significant changes in the lateral railhead deflections.

Based upon the results of these laboratory loading tests, it appears that longitudinal rail forces have negligible effects upon the gage widening of track in good condition. Additional work, however, is needed to more fully understand the effect of longitudinal forces on other track structures, such as aged or deteriorated track.

CONCLUSIONS

This laboratory test program was conducted to obtain load and deflection data for the gage widening mode of track failure. The basic objectives of this test program were to quantify the lateral resistance characteristics of track, to investigate the ultimate strength and failure modes of track subject to large lateral loads, and to determine if non-destrtructive gage widening can be used to detect damaged or weakened track.

Examination of the basic railhead deflection data for lateral load combinations representative of those encountered in the field (Figure 4 and 5) indicate that *significant* gauge widening

*For a complete set of test results the reader is referred to References (6) and (7).

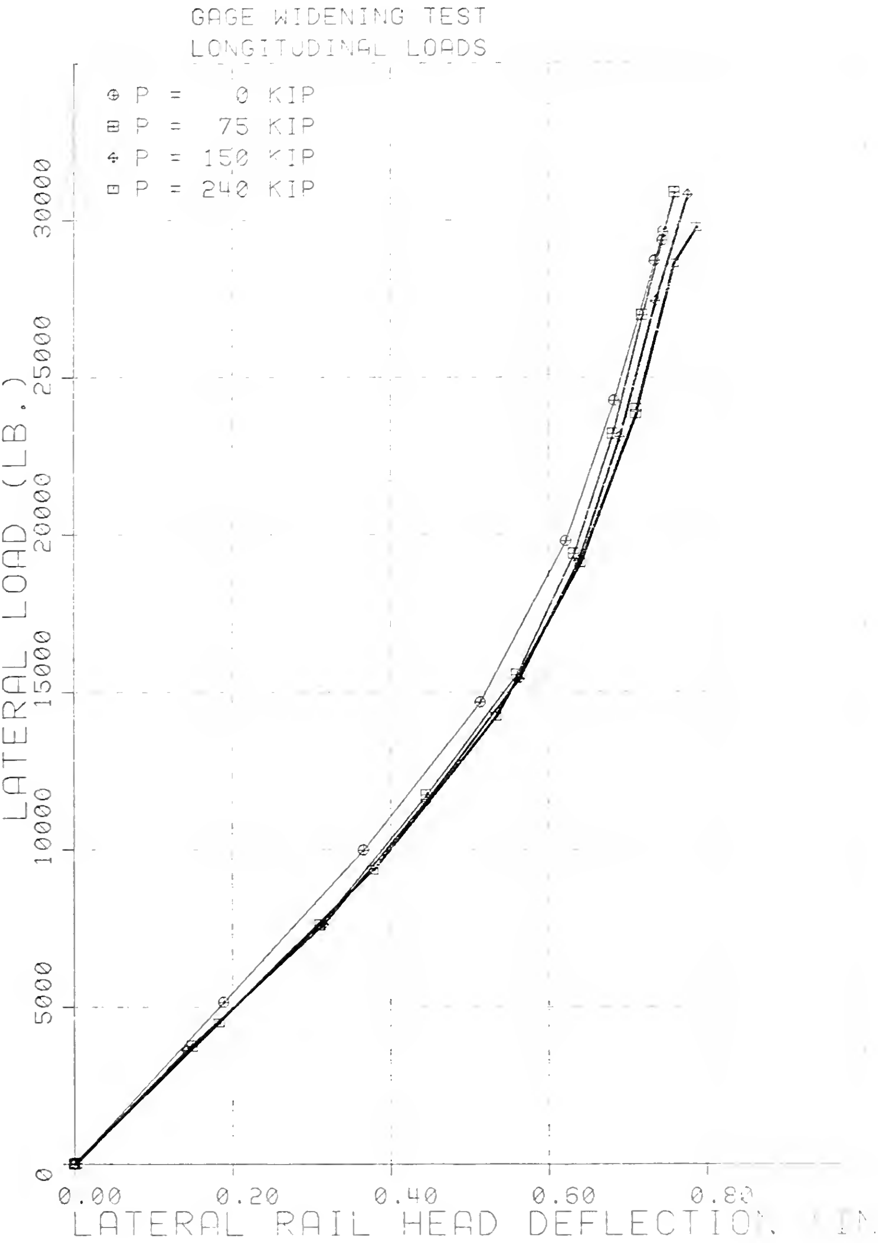


FIGURE 12. LATERAL RAIL HEAD DEFLECTION VS LATERAL LOAD, FOR VARIOUS LONGITUDINAL LOADS AND ZERO VERTICAL LOAD.

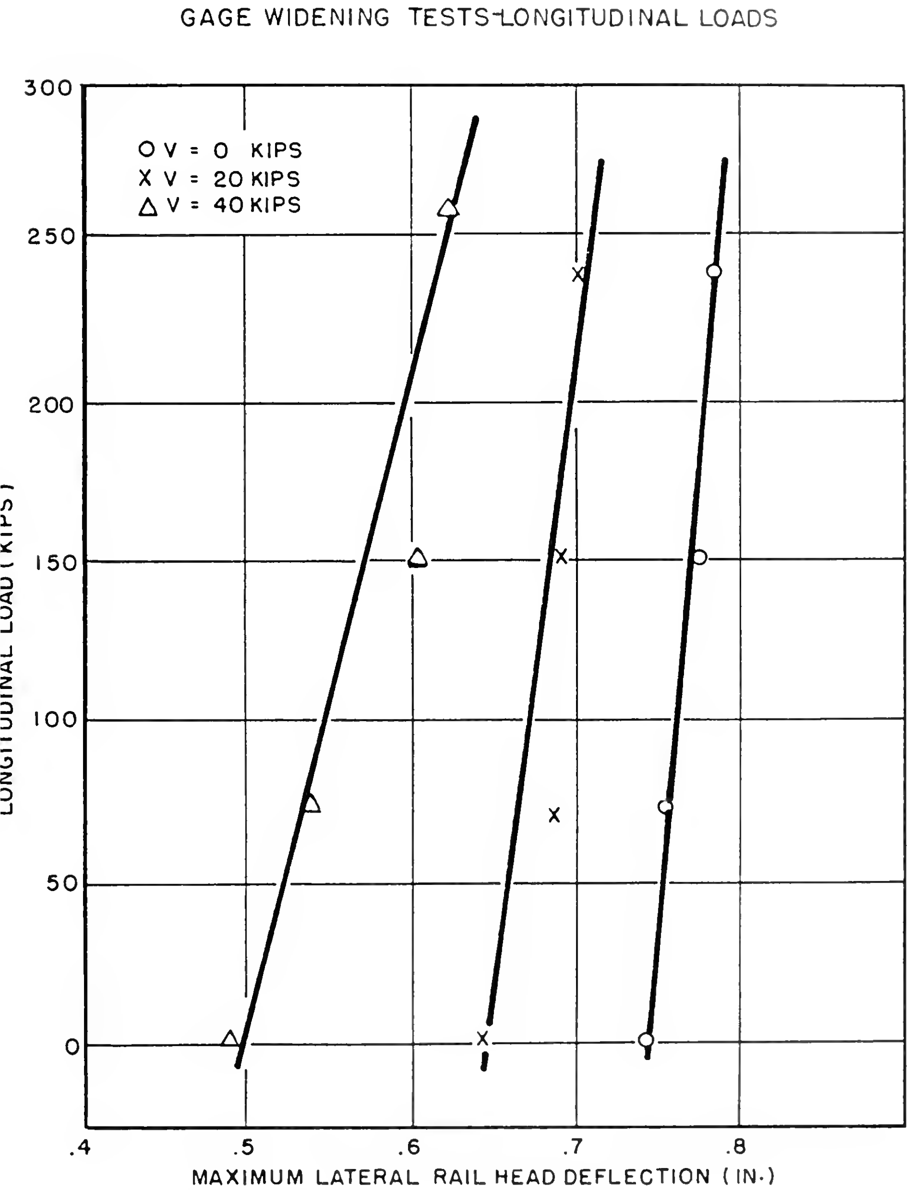
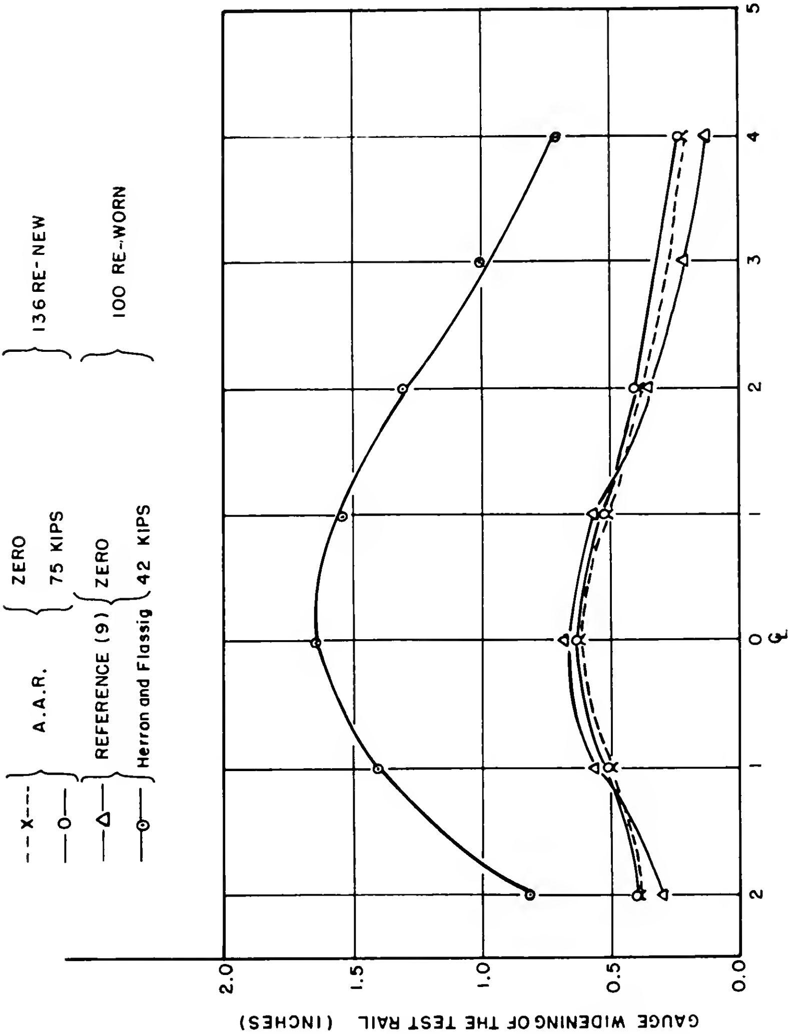


FIGURE 13. MAXIMUM LATERAL RAIL HEAD DEFLECTION VS. LONGITUDINAL LOAD, FOR VARIOUS VERTICAL LOADS AND CONSTANT 30 Kip LATERAL LOAD.



NUMBER OF TIES (IN EITHER DIRECTION) FROM CENTER LINE, (TIE CENTERS ARE 19.5 INCHES)

FIGURE 14. GAUGE WIDENING OF TEST RAIL VS. TIE LOCATION. (RELATIVE TO TEST SECTION (CENTERLINE) FOR VARIOUS LONGITUDINAL LOADS, AT CONSTANT 20 Kip LATERAL AND ZERO VERTICAL LOADS. COMPARISON OF A.A.R. LABORATORY AND FIELD TEST (9) DATA.

can occur. Noting that lateral load of over 30,000 lbs per axle for locomotives, (Figure 15, Reference 11) and up to 12,000 lb per axle for freight cars, (Figure 16 and Reference 12), have been measured in the field, it can be seen from the test data that dynamic gage widening of over one inch can be experienced by track subjected to "normal" train operations. Furthermore, it must be noted that Federal Railroad Administration Track Safety Standards permits nominal gage for track classes 4 and 5 to be up to one inch wide on curves (13). "Combining these observations with the *worst case* wheel and rail wear situation depicted in Figure 17, it can be seen that a potential exists for dynamic gage widening and consequently wheel drop in to occur".

Additional evaluation of the basic gage widening test data indicates that gage widening "predamage" of up to 1.0 inch can be detected by applying a "safe" loading level, i.e. a suitable lateral and vertical load combination that would not cause the railhead to deflect (laterally) more than 0.5 inches, even for track in a "relatively-weakened" condition. The correlation curves, shown in Figure 6 illustrate this concept. However additional field testing must be conducted to determine if these relationships are valid for actual field conditions, including both main and branch line track with various rail sections, tie sizes and spacings, fastener configurations etc. It is recommended that these field tests be conducted using a vertical load of 15,000 lbs and a lateral load of 10,000 lbs., resulting in an L/V ratio of 0.7. This ability to seriously weaken track by a field measurement technique would provide the track engineer with a useful tool to supplement existing visual inspection methods.

In examining the effects of simulated truck (two axle) loading on the gage widening behavior of the track, the data indicates that the presence of a second, adjacent vertical load has a "beneficial" effect, i.e. the resistance of the track gage widening is increased. "However, in examining the effect of simulated single and dual-axle (truck) loading L/V ratios, as shown in Figure 9, it was found that the use of truck L/V ratios as an indication of gage widening potential, could be misleading. Thus, while single axle L/V ratios less than 0.5 were found to correlate well with small rail head deflections, truck L/V ratios did not. It is therefore concluded that whenever feasible, the single axle L/V ratio should be used for evaluation of gage widening and rail overturning potential."

Finally, in examining the effect of longitudinal rail loads the results showed that the longitudinal loads have only a minimal effect on gage widening behavior of track in "good" condition, i.e. heavy rail and relatively new ties. As an example, the presence of longitudinal rail loads at levels normally encountered in track (e.g. up to 100,000 lbs) increased the lateral railhead deflection (constant lateral and vertical load) by less than ten percent. These results did not agree with those obtained by Herron and Flassig (10), who conducted tests on actual track in "poor" condition, i.e. worn 100 RE rail and old ties, and a different load sequence.

The test results, summarized in this report and reported in References (6) and (7), represent a comprehensive study of track gage widening behavior under various static and quasi-static loading conditions. It is hoped that they will provide a useful data base for future field and laboratory studies and provide a more complete understanding of the gage widening restraint characteristics of conventional track structure.

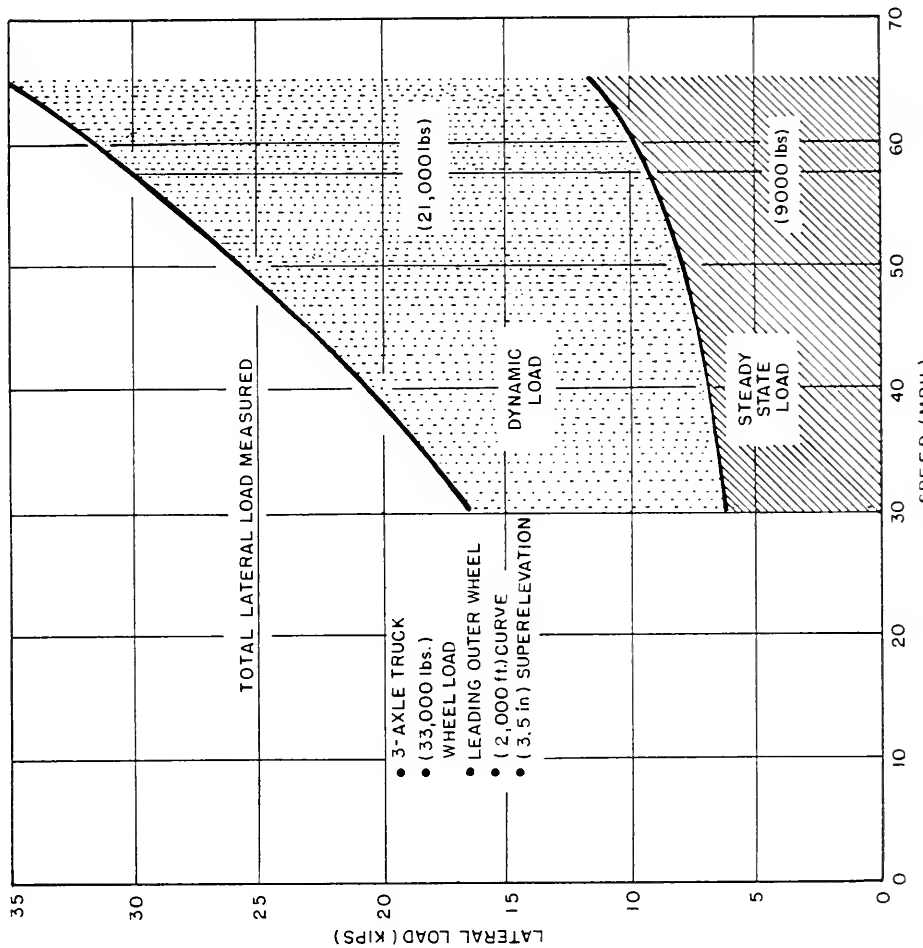


FIGURE 15. STEADY STATE AND DYNAMIC LATERAL LOADS
(6-AXLE LOCOMOTIVE)

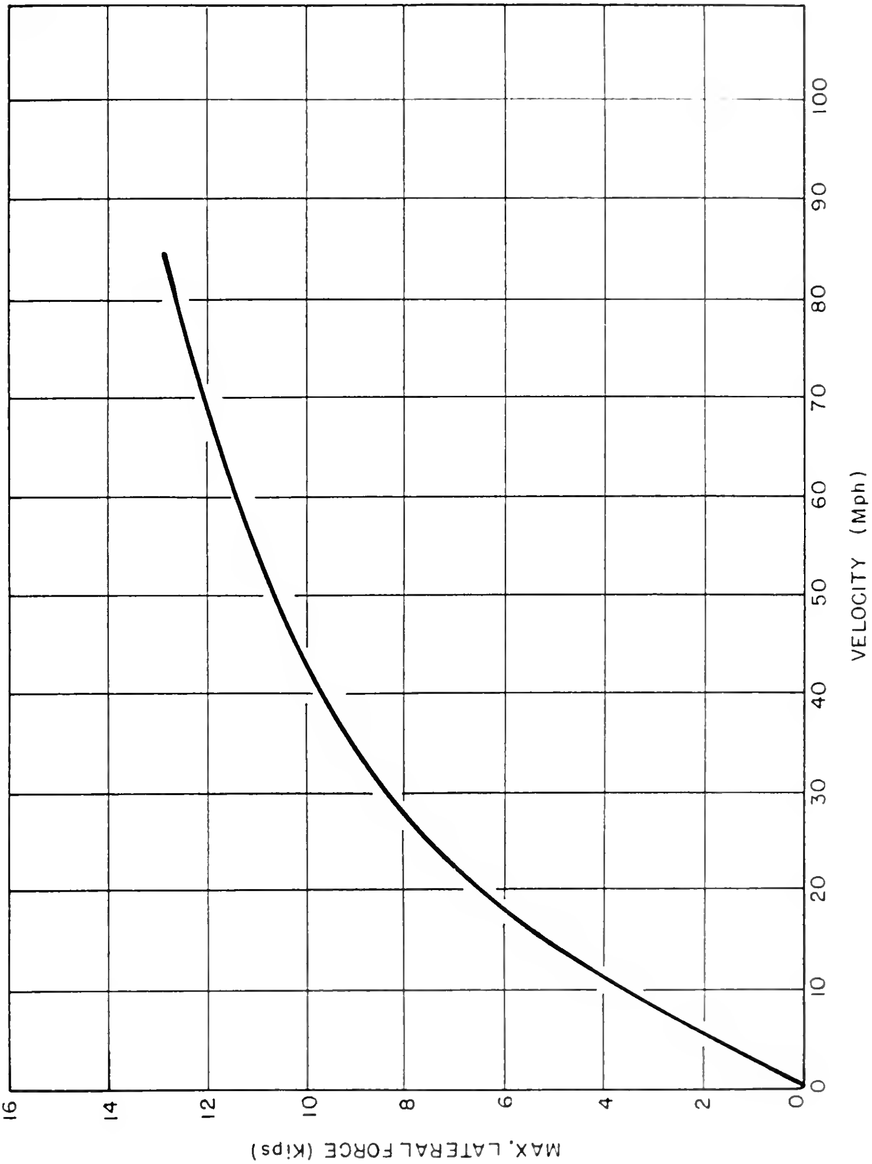
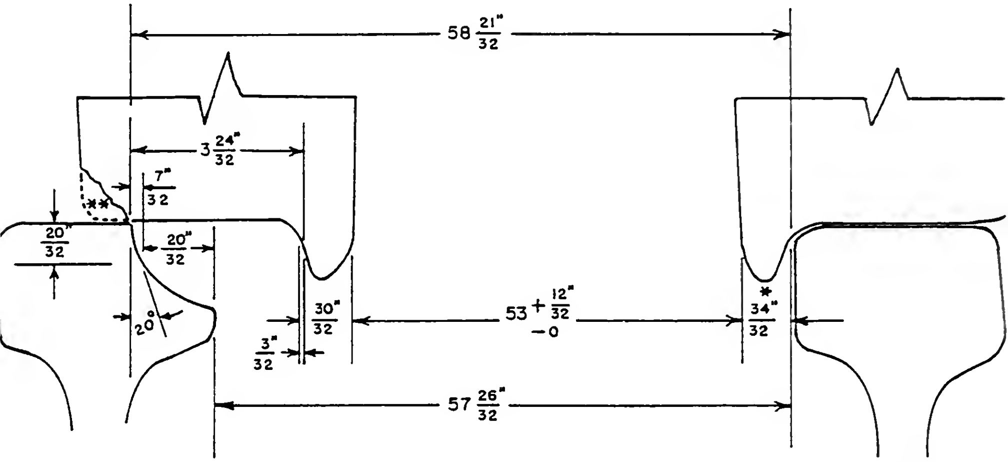


FIGURE 16. MAXIMUM LATERAL FORCE VS. VELOCITY FOR 5° CURVED MAINLINE TRACK (Type II Trucks.)



THICKNESS SUM OF 2 WORN FLANGES (AAR RULE 41)	$\frac{64}{32}$
BACK TO BACK OF WHEEL FLANGES (MINIMUM)	+ 53 —
AVAILABLE TREAD CHIPPED WHEEL	$3\frac{24}{32} - \frac{3}{32} = +3\frac{21}{32}$
	$58\frac{21}{32}$
EXTREME HEAD WEAR AT GAGE POINT	$\frac{20}{32}$
EXTREME HEAD WEAR TOP OF RAIL	$\frac{27}{32}$
MAXIMUM TRACK GAGE SHORT OF DERAILMENT	$58\frac{14}{32}$

$1\frac{30}{32}$ (1.94) INCH OVER $56\frac{1}{2}$ INCH TRACK GAUGE

* Assume That Opposite Wheel Flange Has Half of The Allowable Wear Surface.

**Chipped Rim is Unusual Under Normal Conditions But it Could Exist Under AAR Rule 41.

FIGURE 17. MAXIMUM TRACK GAGE - WORN WHEEL FLANGES AND CHIPPED RIM WHEEL ON CURVE WORN RAIL (CAR NOT RIDING HIGH RAIL)

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March 24–26, 1980

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VOLUME 81

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1980 ANNUAL MEETING

MARCH 24–26, 1980

PALMER HOUSE, CHICAGO

PROGRAM

MONDAY, MARCH 24

OPEN SESSION—Red Lacquer Room—9:30 am

Invocation—Rev. Donald Wheat, Minister, Third Unitarian Universalist Church of Chicago.

Recognition of speakers table guests

Presidential Address—L.A. Durham, Jr., Chief Engineer, Norfolk & Western Railway

Headquarters Report—L. T. Cerny

Greetings from Railway Engineering—Maintenance Suppliers Association—R. C. Crosby, President

Keynote Address—Richard E. Briggs, Executive Vice President, AAR

Burlington Northern's New Gillette–Orin Line (Illustrated)—B. G. Anderson, Assistant Vice President—Engineering, Burlington Northern, Inc.

Engineering Division Session—Red Lacquer Room—1:30 pm

Recognition of speakers table guests

Remarks by Chairman L. A. Durham, Jr.

Remarks by A. W. Johnston, Vice President, Operations & Maintenance, ARR (given by L. T. Cerny)

Remarks by G. H. Way, Jr., Assistant Vice President—Research & Tests, AAR

Update on Results from F.A.S.T. Track—J. R. Lundgren, F.A.S.T. Technical Manager, AAR; G. P. McIntosh, F.A.S.T. Program Manager; and T. P. Larkin, F.A.S.T. operations Manager

Rail Behavior Under 125 Ton Cars on Monongahela Railway (Illustrated)—D. H. Stone, Director—Metallurgy, AAR and L. T. Cerny, Excutive Director, AREA.

Track-Related Performance Guidelines & Economic Analysis of High-Capacity Covered Hopper Car Designs—Keith Hawthorne, Director, Track-Train Dynamics, AAR; M. B. Hargrove, Manager—Applied Technology Division and Deputy Director, Track-Train Dynamics, AAR.

REMSA RECEPTION—Grand Ballroom—6:00 pm–7:30 pm

TUESDAY, MARCH 25

Technical Session—Red Lacquer Room—8:30 am

Use of Reinforced Earth Techniques on Clinchfield Railroad (Illustrated)—J. A. Goforth, Chief Engineer—Maintenance of Way, Clinchfield Railroad; Victor Elias, Vice President, Engineering, Reinforced Earth Co.

Results from Caldwell, Texas Geotextile Tests on Southern Pacific (Illustrated)—Tom Barnett, Sales Manager—Western Region, Bidim Engineering Fabrics/Monsanto Textiles Co.; Jack Newby, Geotechnical Engineer, Southern Pacific Transportation Co.

60-Mile Track Rehabilitation Using Geotextiles on Southern Pacific near Flatonia, Texas (Illustrated)—H. B. Berkshire, Assistant Vice President—Maintenance of Way and Engineering, Southern Pacific Transportation Co.

New C. F. & I. Rail Mill (Illustrated)—John J. Burke, Manager, Railroad Products, C. F. & I. Steel Corp.

New Facilities for Increased Traffic at Alliance, Nebraska (Illustrated)—M. O. Woxland, Director Construction Projects, Burlington Northern

ANNUAL LUNCHEON—GRAND BALLROOM—12:00 NOON

Presentation of guests at speakers table

Introduction of Committee Chairmen

Announcement of newly elected officers

Presentation of 1980 Student Design Competition Award—Walt Studabaker, Environmental Engineer, AAR.

Address—J. R. Neikirk, Vice President—Administration, Norfolk & Western Railway Co.

Technical Session—Red Lacquer Room—2:00 pm

New Car for Measurement and Evaluation of Gage—Widening Resistance of Track (Illustrated)—Allan Zarembski, Manager—Track Research Division, AAR; Scott Lovelace, Manager—Research & Tests, Southern Railway System.

Field Evaluation of New Ballast/Subgrade Radar System—Tom Hutcheson, Consultant; Tony So, Research Engineer, AAR.

Construction of New Urban Rail Systems (Illustrated)—D. A. Shoff, Chief Trackwork Engineer, Daniel Mann, Johnson & Mendenhall Engineers.

Northeast Corridor Track Laying System (Illustrated)—R. D. Johnson, Project Manager—NECIP, Amtrack

WEDNESDAY, MARCH 26

Technical Session—Red Lacquer Room—8:30 am

Improved Method of Determining Size of Transverse Defects—W. J. Rogovsky, Nondestructive Inspection Engineer, Failure Analysis Associates.

Polymer Concrete Material for Structural Concrete Restoration on Railroads (Illustrated)—Wilfried H. Reisterer, Manager—Application Engineering of Silikal North America, Inc.

Ballast Research (why tamping and compaction do not fully restore density and stability of the ballast section)—Dr. Ernest T. Selig, Professor of Civil Engineering, University of Massachusetts.

Installation of Officers

Illustrated Case Studies of Bridge Problems Caused by Unit Trains (Illustrated)—Tim Christenson; Art Fish.

Adjournment

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G. H. Maxwell (West)
Division Engineer, Union Pacific Railroad

R. W. Holt (Canada)
Director, Road Maintenance Management Systems, Canadian Pacific Ltd.

T. C. Nordquist (At Large)
Assistant Director Construction Projects, Burlington Northern, INc.

Committee of Tellers, 1980 Election

The following committee was appointed to canvass the ballots for Officers, Directors, and Members of the Nominating Committee, the count being made on February 22, 1980.

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For President

Mike Rougas, Chief Engineer, Bessemer and Lake Erie, Greenville, Pa.

For Senior Vice President*

J. W. Brent, Director Operations Planning, Chessie System, Baltimore, Md.

For Junior Vice President

William Glavin, Vice President—Administration, Grand Trunk Western Railroad, Detroit, Mi.

For Directors

EAST

B. J. Gordon, Chief Engineering Officer, Conrail, Philadelphia, Pa.
J. C. Hobbs, Chief Engineer, Richmond, Fredericksburg & Potomac Railroad, Richmond, Va.

WEST

J. R. Masters, Director—Engineering, Burlington Northern, Inc., Minneapolis, Mn.

SOUTH

H. L. Rose, Assistant Vice President, Maintenance of Way & Structures, Southern Railway, Atlanta, Ga.

For Members of 1980 Nominating Committee

EAST

A. L. Maynard, Director—Engineering Administration, Chessie System, Huntington, W. Va.

SOUTH

R. E. Frame, Assistant Chief Engineer—Maintenance, Louisville & Nashville Railroad, Louisville, Ky.

WEST

H. G. Webb, Assistant Chief Engineer, Atchison, Topeka & Santa Fe Railway, Chicago, Il.

CANADA

C. W. Wagner, Engineer—Standards, Canadian National Railways, Montreal, Quebec

AT LARGE

R. S. Bryan, Jr., Director—Environmental Protection, Norfolk & Western Railway, Roanoke, Va.

*Under the provisions of the AREA Constitution, J. W. Brent advances automatically from Junior Vice President to Senior Vice President.

OPENING SESSION FEATURES

Address by President L. A. Durham, Jr.*

Members of the American Railway Engineering Association, ladies and guests: welcome to the 79th Annual Technical Conference. This welcome is directed to each of you, but especially to the ladies present. It is a pleasure to have you attend the opening session, and you are invited to attend any of the sessions which may be of interest to you. I hope you will join us Wednesday morning when our new officers will be installed.

The past year has been a productive one for your Association. I am pleased to advise you our membership as of February 28, 1979, was 3,567; as of February 15, 1980, our membership numbers 3,725—an increase of 158 with a substantial number of new applications being processed. It is vital to our future that we promote the professional growth of our membership, both railroad employees and those not directly employed by railroad companies. Consulting engineers, college professors, officers or engineers in other engineering or scientific societies or associations, railroad related employees of the government, editors of trade and technical journals, and our good friends in the railway supply business, all have the potential of making valuable contributions to the work of AREA.

On July 1, 1979, AREA headquarters moved from Chicago to Washington, D.C. Some of our members feared that such a move would result in a deterioration of AREA's influence in railroad engineering and maintenance matters. This situation has not developed, and the move has resulted in even greater cooperation with AAR and FRA.

The executive director and the engineering division of AAR have become more directly involved in the principles of good railroad engineering practices and problems of our railroads than ever before. They are in a position to monitor any proposed governmental regulations concerning railroads and work to insure that these proposals are realistic and will accomplish the goals intended.

Governmental agencies accept AREA's unique organization of professional railroad engineers bolstered by the expertise available from AREA members from the academic community, railway supply industry, governmental agencies involved with railroad activities and consulting engineers. AREA technical committees are still the greatest source of expertise in the railroad engineering field. It is imperative that we maintain this leadership.

The Association is in sound financial condition, and you will hear more about finances later in Lou Cerny's headquarters report.

On October 11, 1979, our Association held a very successful Regional Meeting in Philadelphia under the able chairmanship of J. T. Sullivan, Chief Engineer–Design and Construction, ConRail. The 1980 Regional Meeting is scheduled for October 23, 1980, in Los Angeles, California, under the chairmanship of W. S. Autrey, Chief Engineer, Atchison, Topeka & Santa Fe Railway.

A committee chairmen's meeting was held in Philadelphia on October 12, 1979, to review committee structure and procedures. A concerted effort will be made to strengthen committees by the assignment of appropriate personnel to the committees and elimination of nonproductive members. I have been concerned greatly for sometime with the lengthy response time required on railroad problems assigned for study and reporting. We must disseminate this valuable information faster. Efforts will be made to publish manual material on a more timely basis. Committee chairmen will submit, with their annual report, a list of research needs in

*Chief Engineer, Norfolk & Western Railway their committee's area, listed in priority order, for AREA headquarters' ready reference should they have an opportunity for the research work to be performed.

The dedicated work of the standing committees continues to be the backbone of AREA efforts to develop, maintain and disseminate timely railway engineering and maintenance specifications and procedures. Our committees are led by competent chairmen and vice chairmen who, too often, must achieve committee goals through the work of a relatively small number of hard-working committee members.

The Technical Activity Committee is charged with monitoring and reviewing activities of AREA study and research committees, and to review the activities of the personnel assigned thereto. This committee carefully reviews and passes upon the recommendations for subjects to be investigated, considered and reported on by committees during the ensuing Associaiton year, and reports to the Board of Direction for its approval. This committee also reviews and passes upon applications of members for appointment to study and research committees, and appoints the chairman and vice chairman of each committee, subject to Board of Direction approval.

The Technical Activity Committee and Board of Direction will continue to make critical appraisals of our committee assignments to eliminate obsolete assignments and assign studies on the many problem areas we have today.

Our AREA publications, namely the Manual for Railway Engineering, Portfolio of Trackwork Plans and the Proceedings, are accepted standards for railway enginering construction and maintenance in North America. For AREA to remain in the forefront of railroad engineering, construction and maintenance these publications must be updated as required to cope with current loads, operating and economic conditions.

An ad hoc committee of the AREA is in the process of being formed by W. S. Autrey to improve the communication between AREA and FAST. The FAST Project will benefit from the vast experience and expertise available in AREA and AREA committees, and AREA members will gain valuable information from the tests being conducted by FAST. Many committees have informal contacts with FAST and the ad hoc committee will strengthen these relations. The result will be the development of more meaningful experiments with more practical applications.

Public hearings were held by FRA in Washington, D.C., in December 1979, on miscellaneous proposed revisions to Federal Track Safety Standards. The AREA Committee on Track Safety Standards, AREA officers, AAR staff officers and attorneys, the Executive Director of AAR Engineering Division and Executive Director of AREA, and other railroaders worked long hours preparing AAR's testimony for the hearings and final written statements in reply to the proposed revisions. The chief engineering officers and senior engineering staff members of approximately thirty railroads devoted a substantial amount of time—which can be measured in "man-years"—evaluating standards and the proposed revisions.

AREA Ad Hoc Committee on Concrete Ties has been disbanded and the Board of Direction of AREA has approved establishment of a new AREA Committee 10, Concrete Ties.

I wish to express my appreciation for the cooperation and hard work performed by the AREA officers and Board of Direction. Executive Director Lou Cerny has done an outstanding job for our Association under very trying conditions. He has recruited an able staff and the transition of AREA offices from Chicago to Washington was handled in a most efficient and professional manner. I also commend our committee chairmen, vice chairmen and committee members who work so diligently to perpetuate the reputation and leadership of AREA in the railway engineering and maintenance field.

Our conference operating committee is performing in an excellent fashion under the able leadership of Santa Fe's Vic Hall. I extend to them the appreciation of our Association for their fine work which is so essential for a successful technical conference.

Our good friends in the railway supply industry continue to render valuable assistance to our Association. We are grateful for their efforts to supply us with more efficient machines and reliable material to improve our properties.

Thank you.

Headquarters Report

by L. T. Cerny*

As you probably know, the year since our last Technical Conference has been a time of great change as far as the AREA is concerned. Following a favorable vote by the membership on a constitutional change to allow the headquarters to be moved to Washington and the vote of the Board of Direction to do so, the headquarters was moved to Washington from Chicago on July 2nd of last year.

The AREA staff has had a 100% turnover since last February 12th, when I took the position of Executive Director following Mr. Hodgkins leaving active duty due to illness in December of 1978. Because of the move to Washington, a complete turnover in staff besides myself occurred between May 6th and July 3rd.

During this time, it was necessary to keep pushing forward, and it was decided to try to put all of the publications back on schedule regardless of what time and effort it would require. Even though the combined 1977 and 1978 revisions to the Manual came out early '79, the 1979 Manual revision was issued on schedule in August. Regular, on-schedule publication will increase the regard in which the Manual is held, as it will allow material to go from committee approval to publication in the Manual in less than one year. In addition, the Trackwork Portfolio, which had not been revised since 1973, was revised, and the 1979 revision was published last fall. The 1980 revision should be out in a few months, and revisions should continue on a yearly basis thereafter. Sales of AREA publications reached all-time highs in 1979, partially due to an international advertising campaign last fall.

Another publication problem was the AREA News, which had been decreasing in frequency of publication and only came out twice in 1978. A survey of members indicated that it was very seldom read because of the repetitive nature of its articles from year to year. Negotiations were begun with Railway Track and Structures magazine on placing a monthly AREA News section in that magazine, and a special subscription price was worked out so that the AREA was able to purchase for all of its members a subscription to Railway Track and Structures. Each AREA member should be sure to read the AREA News in Railway Track and Structures each month, as one of the reasons we were able to afford this is that it can save many general mailings to the membership, which can run to over $1000 apiece.

Financially, 1979 was a very good year for the Association, despite the extra expenses involved due to the move to Washington, with a record positive cash flow. Details of our finances in 1979 will be contained in Bulletin 678. In order to keep the AREA in a strong position, it will probably be necessary to ask the membership for authority to adjust dues and other fees for inflation. For instance, when Past President George Brooke, who is now 101 years old and living in Chesapeake Beach, Virginia, joined the AREA in 1907, his initiation fee was $10, and it is still $10 today, 73 years later. The number of members is at a record level, and the growth in 1979 was the highest in recent years, with a *net* growth of almost 200 members.

The new headquarters in Washington is fully-functioning, and our most important goal is good service to the committees and the membership as a whole. The location in Washington has enabled us to play a much more active part in the track standards situation and maintain a closer relationship with other disciplines in railways, such as mechanical and operations, so as to make our work as useful as possible to the profession and the industry. We have worked to develop better communications with other organizations, such as the Transportation

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Research Board and agencies of the federal government, so that the positions of the railway civil engineering profession are made known.

The infusion of federal funds has created a much more favorable situation for railway engineering research, and over 300 railway research projects are underway at the present time. Additional input from practicing railway engineers is needed in many cases to make the results of those studies relevant to actual field conditions, and AREA is in an excellent position to provide this input. Federal funding has also led to an increase in the employment of engineers in the engineers in the railway field, and has been the inspiration for more railway engineering curriculums than have existed in the recent past.

The energy situation has created great opportunities for North American railroads, and it has already influenced new line construction and doubling and tripling of lines, something envisioned only by the most optimistic railroaders of 15 years ago. 1979 was the all-time record year for freight ton-miles in the United States. The resurgence in the passenger train and the rail transit industry has also been something that was not anticipated 10 or 20 years ago.

With the new vitality in the AREA and the new vitality in the railroad industry, it is important that we remember that the future of railroading depends on the work of railway engineers and their work on AREA committees.

Many fears were expressed by AREA members that the move to Washington would result in a reduction in the AREA's independence from the AAR. This has certainly not happened, and, in fact, a study which was commissioned by the AAR's Research and Test Department in 1979 has reaffirmed the differences between AREA and the AAR Engineering Division, and has indicated the importance of an independent AREA. You can be assured that I will carry out the mandate of the AREA Board of Direction to keep the AREA fully indpendent while providing better service than ever to the railroad industry in our functions as the Engineering Division.

With the complete changeover in personnel since February 12th, obviously we at the AREA headquarters staff have gone through much of the last 12 months with no previous experience to guide us, and it is possible that some things that have been done in the past may be overlooked. Please feel free to call the AREA headquarters if you sense that something is amiss or something you feel that you should have received has not been received, because there is no intention of cutting services to members, and, if this has happened, it is simply an oversight on our part.

At this time, I would like to give proper credit to the new staff, to Susan Rich, my very capable secretary, who has done an excellent job of filtering out my mistakes before anybody else but her knows of them, and whose sense of humor brightens the whole headquarters staff; and to Gary Wade, who handles publications and keeps that function in a neat, efficient manner.

A special mention is due to Susan Chambers, the Manager-Headquarters of AREA. Sue deserves much of the credit for the successful move to Washington and is in fact responsible for the upkeep of the organization and the preparation and printing of all of its publications. She is editor of the AREA News, and in general takes care of most of the things that the AREA headquarters did in the past. The massive detail involved in running an organization like the AREA becomes fully apparent only when it is necessary to transfer all of that knowledge to new individuals. Susan Chambers, with only a three week overlap with her predecessors, has done a truly outstanding job in keeping the processes of the AREA moving along and getting the publications back on schedule.

The heavy burden of organizing and running this conference fell mainly on the shoulders of Cynde Kraft, who also handles all of the membership matters at headquarters. Cynde has done an enormous amount of work in this connection, and is doing an excellent job, especially considering that she has never been to an AREA conference before.

In closing, let me repeat that the AREA headquarters exists to serve the committees and the members of the AREA, so please let us know of any suggestions that you have for improvement. We also invite you to drop in at our new headquarters at 2000 L Street in Washington.

Keynote Address by Richard E. Briggs*

It is indeed a pleasure for me to be here with you today. I would like to preface my remarks by extending to you Bill Dempsey's apologies for being unable to attend today's session. I suspect he would find your company more congenial than that which he will be keeping today. But when Congress calls on you to testify, you have but two choices: testify or find yourself stuck with legislation that is impractical.

So he and Bill Johnston are spending the day before the Senate Commerce Committee's Surface Transportation Subcommittee. I imagine most of you would feel at home there today if you had to join them—for the Subcommittee is holding hearings on the Federal Railroad Safety Act specifically, and railroad safety in general.

When it comes to safety, I think we have a very positive story to tell. Particularly in comparison with other modes. For example, big trucks carry only 30 percent of the country's hazardous materials, excluding oil. But they're involved in more than 90 percent of the accidents, 75 percent of the injuries and 80 percent of the fatalities that occur during the movement of hazardous materials.

Railroads, by way of stark contrast, carry about 70 percent of the hazardous materials and are involved in fewer than 10 percent of the accidents.

Indeed, we have made tremendous safety strides in recent years. And the people in this room have contributed substantially to those gains . . . gains that are being reflected in several ways. Overall, our accidents are now beginning to decline. And especially significant is the fact that neither injuries nor deaths from railroad accidents have ever before been as low as they have been over the last few years.

I don't mean to suggest our record is perfect. Obviously, it is not. We still have accidents —far more than any of us would like. We still do not have answers to every question that could be raised. We are aware that the continuing trend toward larger freight cars and longer, heavier trains does intensify the need for preventative maintenance along the railroad right-of-way.

But importantly, we are matching our expressed commitment to safety with more than words. We are taking the concrete actions that we as an industry must if we are to further improve safety and prepare for the future.

For example, from 1976 to 1978 railroads laid more than 2.6 million tons of new rail—an average of more than 800,000 tons a year. Not since the mid-1950s have railroads been able to lay that much rail annually.

It's also been two decades since railroads were able to put in as many new crossties as they have over the same three years—27 million a year, 81 million in all.

Indeed, for the first time in years, railroads in every part of the country have been able to make significant inroads into deferred maintenance. Railroads have been putting in both crossties and rail at a rate faster than would be required just to maintain the rail plant at existing levels.

We are also putting more money than ever before into capital improvements—more than $3.3 billion last year. Another mile-stone of sorts was reached last year when railroads for the first time in history put more than $1 billion into capital improvements for roadway and plant. Inflation accounts for some of that increase—but nowhere near all of it. Last year's roadway improvement program of one-billion-forty-million dollars was 21 percent above the 1978 level.

*Executive Vice President, Association of American Railroads

Railroads have invested this money despite continuation of the industry's long-term financial problems. Last year, for example, railroads moved more freight than ever before. And earnings were up. But they still did not approach even a minimally acceptable level.

Preliminary figures indicate the industry earned about 2.7 percent on investment—the best since 1974, but hardly adequate. Virtually every economist who has studied the railroad industry has concluded we need to earn something in the range of 10-to-13 percent on investment in order to provide adequate service, make needed improvements and provide a fair return to stockholders.

I might add that the problem of low earnings is not restricted to just a few railroads. Nor is it limited to lines in the Middle West and Northeast. The fact is that not even one of the country's ten largest railroads earns the 11 percent on investment that the ICC recently concluded railroads would need in order to provide both adequate service and a reasonable return to investors.

The immediate prospects are dim for railroads to significantly improve earnings. It is true our traffic is holding up well. Indeed, thus far in the first quarter, traffic is up about 11 percent over last year. However, last year's traffic was unusually low because of the harsh eastern and Midwestern winter. But President Carter's recent actions regarding the economy are likely to slow business later this year so it is unlikely that we can continue to show traffic gains of that magnitude. A combination of a business slowdown and continued inflation will certainly not be a spur to railroad earnings.

But looking beyond the immediate future, I do see some grounds for optimism—optimism for both the continued growth of railroads and for a solution to our long-standing earnings problem.

It is relatively easy to find reasons why railroads should be able to reverse their three-decade-long decline in traffic share.

It starts with energy. Depending upon the traffic, railroads are up to four times as efficient as trucks. Of course, on some traffic the advantage is not that great: piggyback, for example, where the most recent studies put the rail advantage about 2-to-1. But even there, technology is moving to increase the rail advantage with developments like the Road Railer and the Fuel Foiler. As fuel becomes more and more expensive, motor carrier rates likely will have to increase more rapidly than rail, giving railroads a significiant competitive advantage for many types of freight now going by truck.

Railroads also stand to benefit as the country increasingly turns to coal as an energy alternative to petroleum products. We now carry about 65 percent of the country's coal and have experienced a 20 percent traffic gain over the last two years. I might add that the ease with which railroads have handled that increase speaks well for our ability to handle significant traffic gains over relatively short time periods.

Railroads—again depending upon the specific movement—also have, on the average, a marginal energy advantage over water carriers. And railroads also have environmental advantages over both motor and water carriers—we pollute less and can handle major traffic gains without construction of new lines.

My reasons for optimism about industry profits are perhaps less obvious than my reasons for optimism about traffic growth. Deservedly less obvious because solution to our earnings problem depends largely on the actions of others.

Here I am talking about two things we need: deregulation and competitive equity.

You have probably heard quite a bit about deregulation over the last year, so I won't dwell too long on that subject.

There is some good news on the deregulayion front. Within the past year, the ICC has taken several steps to ease regulation—it has reversed its historic opposition to contract rates; it is moving to simplify its market dominance standards, an action which could free many individual rates from regulation; it is considering exempting all piggyback from regulation; and it has already freed most fresh fruits and vegetables from regulation.

This latter action has proven particularly significant. Because it shows the potential of deregulation for railroads. As some of you may be aware, the railroad market share in this commodity group has been declining for about a quarter century.

That trend continued last year until the traffic was deregulated in May. For the first five months, volume was down almost 6 percent. But over the next seven months, the rail volume soared almost 32 percent, bringing a full year increase of 14 percent over 1978.

Congress is also moving to ease regulation. Tomorrow the Senate is supposed to begin consideration of S. 1946—the Railroad Transportation Policy Act of 1979. That bill is not really a deregulation measure. Rather it reforms instead of deregulates. But the reforms included would be a step in the right direction.

The bill would remove all regulation from any rate that falls below the average required to cover variable and fixed costs plus a reasonable return on investment. That average lies somewhere between 150-and-165 percent of variable costs. Since the average rate for all rail traffic lies about 130 percent of variable cost, this would free a considerable amount of traffic from regulation.

Railroads would also be permitted to increase individual rates by an amount equal to inflation, plus four percent, annually. However, the ability to use general rate increases to cover inflationary cost increases would be restricted.

The bill also gives railroads specific contract rate authority; it speeds up the abandonment process slightly; and it restricts the ICC's authority to issue car service orders.

Unfortunately, special interest groups have succeeded in attaching amendments to the bill to protect grain shippers and recyclable dealers from the realities of the market place. During tomorrow's consideration of the bill, special interest groups representing the electric utilities are expected to try to amend the bill to restrict rail rates for moving coal. If they are successful, railroads will have little choice but to oppose the bill. Because instead of easing regulation so that railroads could improve earnings, the net effect of the bill would be to increase regulation and make it more difficult for railroads to overcome earnings problems.

As you can tell from this, the battle for deregulation is not an easy one. But it is an essential one. Deregulation is an essential element to any solution of railroad earnings problems.

The other essential element is competitive equity—in other words, an end to the subsidies given our competitors for their rights-of-way.

As engineers, I suspect you would find your jobs a lot easier if you didn't have to pay anything to build and maintain your rights-or-way and someone else paid the entire cost.

That is the situation with barge lines today. They have never paid anything to build and maintain the facilities necessary for our system of navigable waterways. The result is that for every revenue dollar collected by water carriers, taxpayers kick in 41 cents for subsidies to build and maintain the waterways. Obviously, if the barges had to pay the cost of that system, their rates would be considerably higher—and railroads would be in a position both to increase some rates that are artificially low today and to gain back some business that had been lost to subsidized barge rates.

The motor carriers don't have quite as good a deal as the barge lines. They do pay a user charge. They just don't pay their fair share.

A federal cost allocation study completed in 1970 found that combination trucks would need to pay user charges about one-third higher than they were paying in order to cover their full allocation of construction and maintenance costs. That same study found that user charges for the largest trucks then permitted—semi-trailers plus full trailers—would have to be increased almost 79 percent in order to cover their full share. The problem has gotten worse since that study was completed.

As engineers, you know what effect increasing weight has on right-of-way. It increases the need for preventative maintenance. It is a problem we have, too. To a degree this can be compensated for by designing the structure so it can take the heavier wieght. We're certainly doing that.

The basic interstate system, however, was designed for maximum 18,000 single-axle and 32,000 pound tandem-axle loads. What happens when these loads are exceeded?

Quite a bit. And none of it good.

One study sponsored by the American Association of State Highway and Transportation Officials found that increasing the maximum single-axle load from 18,000 to 20,000 pounds—11 percent—reduces remaining highway life by 25-to-40 percent.

A study in Mississippi found that state's annual maintenance costs would increase $18.5 million if single-axle load limits were raised to 20,000 pounds and maximum truck weights to 80,000. That same study said the state would have to raise another $76.8 million to replace bridges that would otherwise become critically overstressed under the higher weight limits.

Despite this, the federal government in 1975 increased the maximum limit for single axle loads from 18,000 pounds to 20,000 and the maximum gross vehicle weight from 73,280 to 80,000.

Were user charges increased to reflect the greater-than-proportionate increase in wear caused by the higher weights?

Of course not. And today the interstate system is afflicted with a problem we railroaders are all to familiar with—deferred maintenance. More than half of the interstate system—26,000 miles in all—needs upgrading just to meet current safety and service standards. One-fifth of the system is in need of major 3-R work—the "Rs" standing for resurfacing, restoration or rehabilitation. Just catch up on this 3-R work would cost $2.6 billion. Projections into the future put the total bill at $19 billion by 1995.

The problem is hardly limited to the interstate system. Other highways have the same problem—and to an even greater extent. One federal study says that all levels of government will have to raise $300 billion over the next 15 years just to keep the highway system in the same condition it was in during 1975.

Despite this, those states that have not gone along with the increased weight limits are now under intense pressure to conform to the federal standards. Legislation to force states to conform has already passed the Senate when it was tacked onto a highway safety bill.

Why? Allegedly to save energy, since 80,000 pound trucks are marginally more efficient that the 73,280 pound vehicles. Never mind the energy that would be wasted from traffic that might be diverted from even more energy-efficient railroads. Or the reduced energy efficiency caused by broken highways. Or the energy that would be wasted in repairing those crumbling pavements.

As you can see, the battle for competitive equity will not be easy. Nevertheless, I remain optimistic that we can win that fight.

For one thing, barges are about to lose their free ride. They will begin paying a tiny user charge this fall—4 cents a gallon on diesel fuel. Gradually that will rise to 10 cents a gallon. Even then, however, barge lines will be paying less than 10 percent of their fair share. But at least a start has been made. That battle will be fought again. I believe there is a growing realization in Congress that barges need to pay more. I can assure you we are working diligently with environmental and other groups to reinforce that realization.

There is also a growing recognition of the fact that the big trucks need to pay more. Last year the General Accounting Office studied the rapid deterioration of the highways and laid much of the blame squarely on the big trucks.

The federal government is supposed to complete a new highway cost allocation study in 1982. It is expected to show that big trucks underpay their fair share of the highway system by an even greater margin than was shown by the 1970 study. That will increase the pressures on Congress and the states to impose fully compensatory user charges on big trucks.

The costs of railroad right-of-way amount to about one-third of all operating revenues. Barges spend nothing. Trucks about 5 percent. Until there is an end to the subsidy that produces and magnifies that inequity, railroad earnings will remain depressed and traffic will be diverted to other modes of transportation.

But with an end to that inequity and with an end to overregulation, both problems can be solved. Railroad earnings can be brought to adequate levels. Railroad traffic can grow at an unprecedented rate.

As members of the railroad industry . . . as Americans concerned about energy and the environment . . . as taxpayers unable to pay subsidies . . . it is in the best interest of each of us to see that these problems are solved and that the railroad industry does indeed live up to its bright future potential.

Burlington Northern's New Gillette-Orin Line

by B. G. Anderson*

Mr. President, officers of AREA, honored guest, ladies and gentlemen. Thank you for that kind introduction. I have the privilege this morning of reporting to you the most interesting and challenging project of my railroad career—the construction of a new main line 127 miles long in northeastern Wyoming.

Burlington Northern has two main lines which cut diagonally across northeastern Wyoming. See Figure 1. Both are well maintained secondary main lines which carried an annual gross tonnage of about five million tons prior to 1970. The northerly of the two routes was laid mostly with 112 lb. rail The southerly route east of Casper is laid entirely with 112 TR rail with 36 inch joint bars and on crushed rock ballast. It is in excellent shape. These two lines are strategically located with respect to the enormous body of low sulfer bituminous commonly referred to as the Eastern Powder River Coal Body.

In 1972, Gillette, Wyoming, a subdivision point on the northerly of the two lines between Lincoln, Nebraska and Billings, Montana, was a quiet little cattle town of 7,400 people, bulging only slightly from the oil drilling activity in its surrounding agricultural area. The rail line through the town was well maintained secondary main line with an average of five daily freight trains and no passenger service. Today, Gillette and the surrounding area originates fifteen unit coal trains a day, and the population has grown to 17,000

It was that year that the Amax Coal Company first approached us concerning their plans for opening a coal mine fourteen miles south of Gilette. Burlington Northern was just emerging from the expensive process of merging its properties and was short of cash, so an agreement was reached for the Amax Company to finance a railroad spur into the mine. Because of this agreement, we more or less acquiesced to their standards of construction on this line and therefore the first fifteen miles of the new line that I will be talking about this morning was not constructed to the same first class main line standard of the rest of the line.

As the Amax mine to be known as Belle Ayr became a reality and the full impact of the potential for this large body of coal became apparent, environmentalists raised strong opposition to (1) the opening of additional open pit mines, and (2) to the construction of a railroad that would serve them. This opposition led to the requirement of an extensive environmental impact statement for the entire Eastern Powder River Coal Basin, including not only the mines, but the railroad construction. Even after this impact statement was completed by the U.S. Geological Survey, it was never completely accepted by the federal E.P.A. Nevertheless, the Interstate Commerce Commission in January 1976 authorized the Burlington Northern and the Chicago & North Western to jointly construct and operate a new line through this area and connecting with the Burlington Northern's other main line through Wyoming at a town called Orin, located sixty-seven miles east of Casper, Wyoming. This line would also connect with the Chicago & North Western's branch line in the same general area. Shortly after this order was released and before we could accomplish a great deal of work, the Sierra Club and others secured an injunction from federal court prohibiting us from further construction until the matter of the environmental impact was resolved. This did not, however, stop the coal companies from continuing with their plans, and in 1975, Sunedco started construction of their Cordero mine six miles south of the Amax mine. We negotiated an agreement with them to construct a track into their mine as an industrial spur. However, now we understood where we were going with this project, and this segment was built to our main line standards, which

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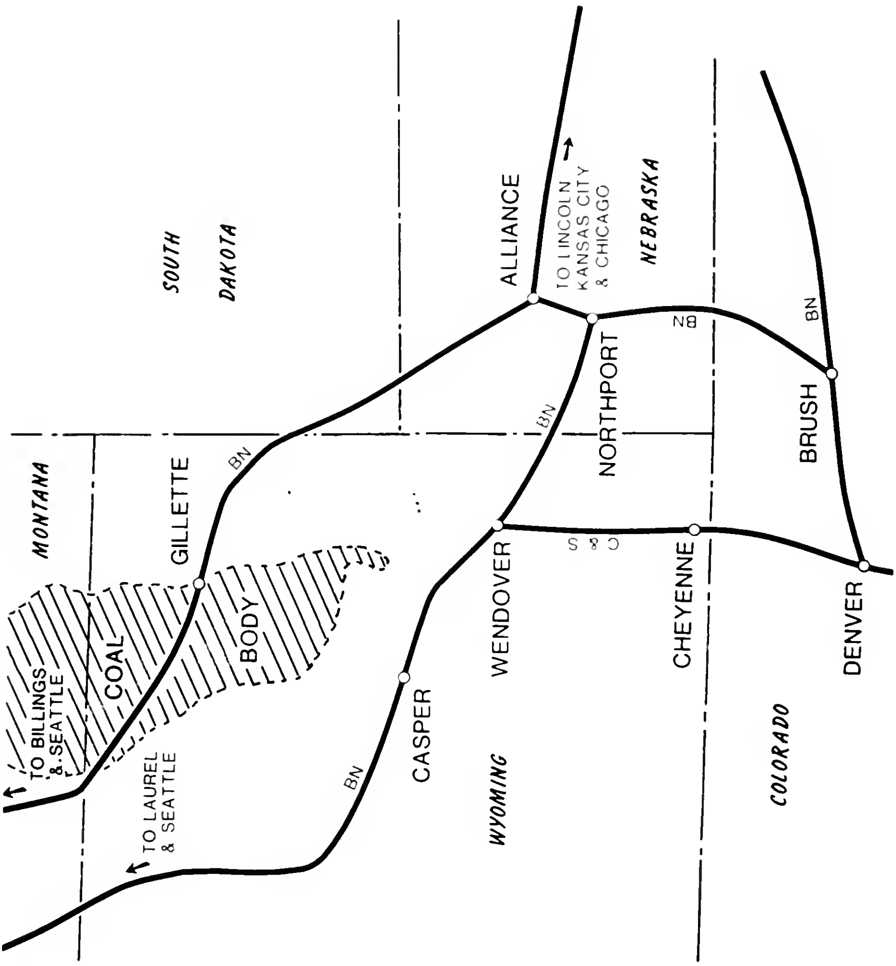


Figure 1.—General diagram of the geographical location of the Burlington Northern main lines through the State of Wyoming.

include 132 lb. CWR rail, twenty-three No. 5 wooden ties per panel, $7^3/_4 \times 14$ tie plates and eight inches of C.R. ballast on twelve inches of select sub-ballast. Also careful analysis and consideration was given the soil that we encountered in constructing the subgrade. We also established maximum gradient of one percent and maximum curvature of three degrees for the line. The extension into the Cordero mine was completed in the fall of 1976 and we immediately started construction on the same basis on a second spur continuing south to serve mines being opened by the Arco and Kerr McGee companies. The former mine was to be known as the Black Thunder mine and the Kerr McGee mine as Jacobs Ranch. Construction was continued to Reno Junction, milepost 42, in 1977 and onto the east a short distance to serve these two mines. This was as far as we could go, however, without resolving our differences with the ecologists and the Sierra Club. See Figure 2.

Irregardless of the opposition from the ecologists, coal tonnages shipped from the area were increasing. See Figure 3. We kept a careful eye on the actual and projected growth of this business, and could see that by 1979 our line across from Gillette to Alliance was going to be severly taxed if not completely saturated and that the new connection would most certainly be required by the end of that year. This secondary main line was undergoing some remarkable changes during this period of time. Originally a railroad without signals, we were rapidly constructing C.T.C. across the territory, replacing the jointed 112 lb. TR rail with 132 lb. CWR, reballasting and constructing long sidings which eventually will become segments of second main line to alleviate congestion at the more critical areas. Nevertheless, even with these improvements, the line would not be able to carry the projected eighty-two million gross tons annually beyond the year 1979 or even during the latter half of the year 1979 without serious delays in moving this traffic.

For this reason, those of us in the engineering division became increasingly nervous as the delay continued over the lifting of the injunction that would permit us to continue with the construction of the eighty-five miles of railroad southerly from Reno Junction to Bridger Junction, the name given the connection point at Orin.

At Orin, the Burlington Northern's new line had to cross the Chicago & North Western's line extending from Fremont, Nebraska to Casper. See Figure 4. As a part of our agreement resulting from the order of the Interstate Commerce Commission to construct this line jointly, we were to rebuild a three mile segment of the Chicago & North Western line which would then become jointly owned but maintained and operated by Burlington Northern. In addition, there was a three-quarter mile connection necessary between the Chicago & North Western's and Burlington Northern's line at Orin. The construction of this connection involved a long high fill, the relocation of a short section of state highway no. 87 and the construction of a bridge over this highway.

As we waited during the winter of 1977-78 to continue south from Reno Junction, we spent the time finalizing plans for twenty bridges, nineteen box culverts, and fourteen million cubic yards of grading itself. The land department also used this time to acquire the last of the right-of-way. This was not an easy task as ranch owners and county commissioners were not overly anxious to see a railroad built through this open range country. It is worth noting that all the necessary right-of-way was acquired without having to resort to time consuming condemnation proceedings albeit this was but one of the reasons we had to build so many overhead highway bridges and large cattle and vehicular underpass structures. See Figure 5.

Finally, after a great deal of effort on the part of our law department, the injunction was lifted in March of 1978. Immediately, we solicited bids for the lower forty miles of grading, bridging and culvert work. This contract included the three-quarter mile connection between Burlington Northern and Chicago & North Western tracks at Orin but excluded the recon-

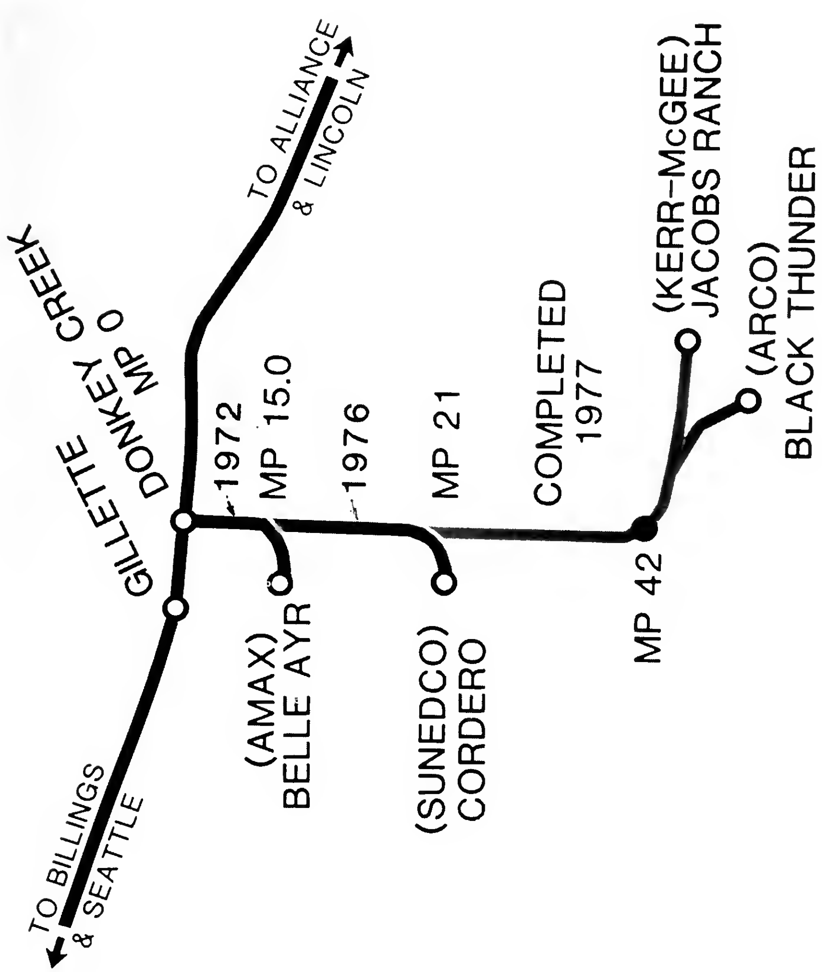


Figure 2.—Diagram of track construction to Milepost 42.

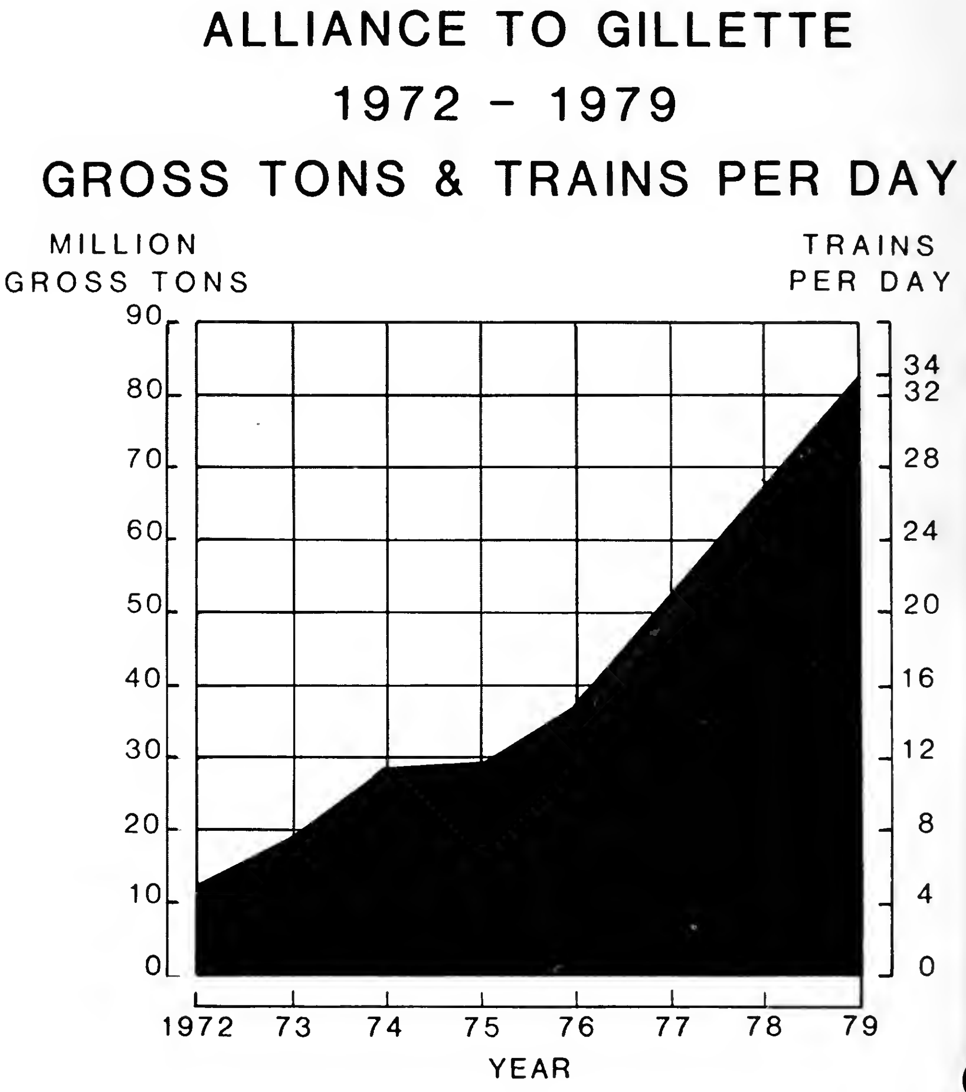


Figure 3.—Chart of gross tons and trains per day on line Alliance to Gillette 1972-1979.

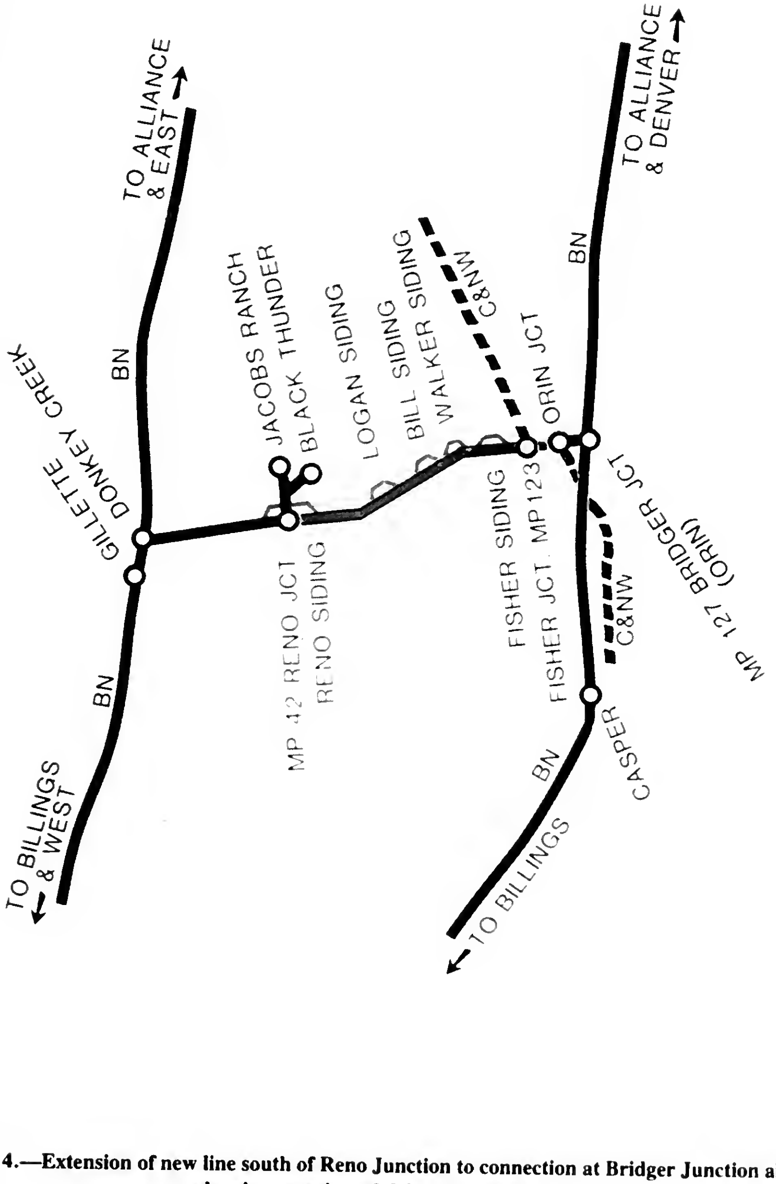


Figure 4.—Extension of new line south of Reno Junction to connection at Bridger Junction also showing crossing of C&NW railroad.

CONSTRUCTION QUANTITIES

RENO JCT. - BRIDGER JCT.

LENGTH	85 ROUTE MILES
132 CWR TRACK	107.5 MILES
GRADING	14,000,000 CY.
RAILROAD BR.	14
OVERHEAD HWY. BR.	6
BOX CULVERTS	19
SUB-BALLAST	615,000 CY.
C. R. BALLAST	310,000 CY.

Figure 5.—Construction quantities Reno Junction-Bridger Junction.

struction of the three miles of C&NW track. This work, along with the four new bridges required in this section, was awarded later in 1978 to Neosho Construction Company.

The contract for the forty mile southern section involved five million four hundred thousand yards of grading, six bridges to carry the railroad structure, and two overhead highway bridges as well as numerous large box culverts and other drainage structures. This contract was awarded to Johnson Brothers and Gustafson Construction Company, operating as a joint venture. The Johnson Brothers is a medium sized road grading contractor out of Fort Pierre, South Dakota, and Gustafson operates out of Sioux Falls. They divided up the grading with the Johnson Brothers assuming about two-thirds of the work and Gustafson, one-third. Their area of responsibility is as shown. They, in turn, sublet the construction of the bridges to a local firm in Casper, Wyoming. See Figure 6.

In July of 1978, a second grading contract was awarded to Neosho Company, involving eight million cubic yards of material along with four railroad bridges and three overhead highway bridges. They, in turn, subcontracted the territory between milepost 88 and milepost 70 to a contractor known as A-B-C Company, and the upper ten miles of the project from Reno Junction, to Harley Hall Construction Company. Neosho kept the remaining grading work in the center of this northerly area for themselves which comprised the heaviest yardage and the longest hauls. Also involved in this contract are four bridges, including the bridge over the Antelope Creek which is five hundred and eighty-five feet long and sixty feet high. There are four overhead highway bridges involved in this segment. Neosho Company did all of their own bridge work, except the bridge over state highway no. 59 which the state insisted on building themselves. Since this was a critical structure on our construction plan, we were a little nervous about the state keeping their contractor on schedule. So we had four grading operations in progress, the completion of which, along with the bridges, had to be carefully sequenced and completion dates staggered to meet the planned schedule for the track construction. Therefore, each contractor was given very specific requirements with respect to completion of each of the bridges, the placement of subballast and of the grading itself so that this work would be done, permitting track construction to proceed on a definite schedule.

I would like to stop here for a minute and tell you a little about our planning regarding the actual grading work itself. See Figure 7. We knew we were going to encounter some difficult soil conditions throughout this route, and we engaged the services of a very reputable soils engineering firm, Cooper-Clark, who operate out of Palo Alto, California but who had established a regional office in Gillette, Wyoming. These poeple arranged for soil borings along the entire route and carefully analyzed the soil conditions from these borings and along with visual inspection, gave us instruction and advice as to specifying for the excavation and the placement of the embankment. A more detailed account of this study is included in a paper that I presented at the annual National Transportation Research Board in Washington, D.C., in January, 1979, excerpts of which were repeated in an article in April, 1979 issue of "Progressive Railroads."

Briefly, however, we were very specific regarding the density requiring compaction to 90% modified proctor in all fills and the moisture content of the material be maintained two to three percent greater than optimum in order to reduce the void ratio and improve the soil's other characteristics. During those months of the year when the temperature was well below freezing, we also kept careful record of the actual soil temperature before it reached its final compaction to be sure this occurred before the soil temperature dropped below the freezing point. Special treatment was provided to those locations where we passed from cut to fill because we found from experience this was a critical area where soft spots often develop and if there was any question of stability, we undercut three feet for a distance of four hundred feet into the cut, and replaced the waste soil with selected material. For an equal distance out

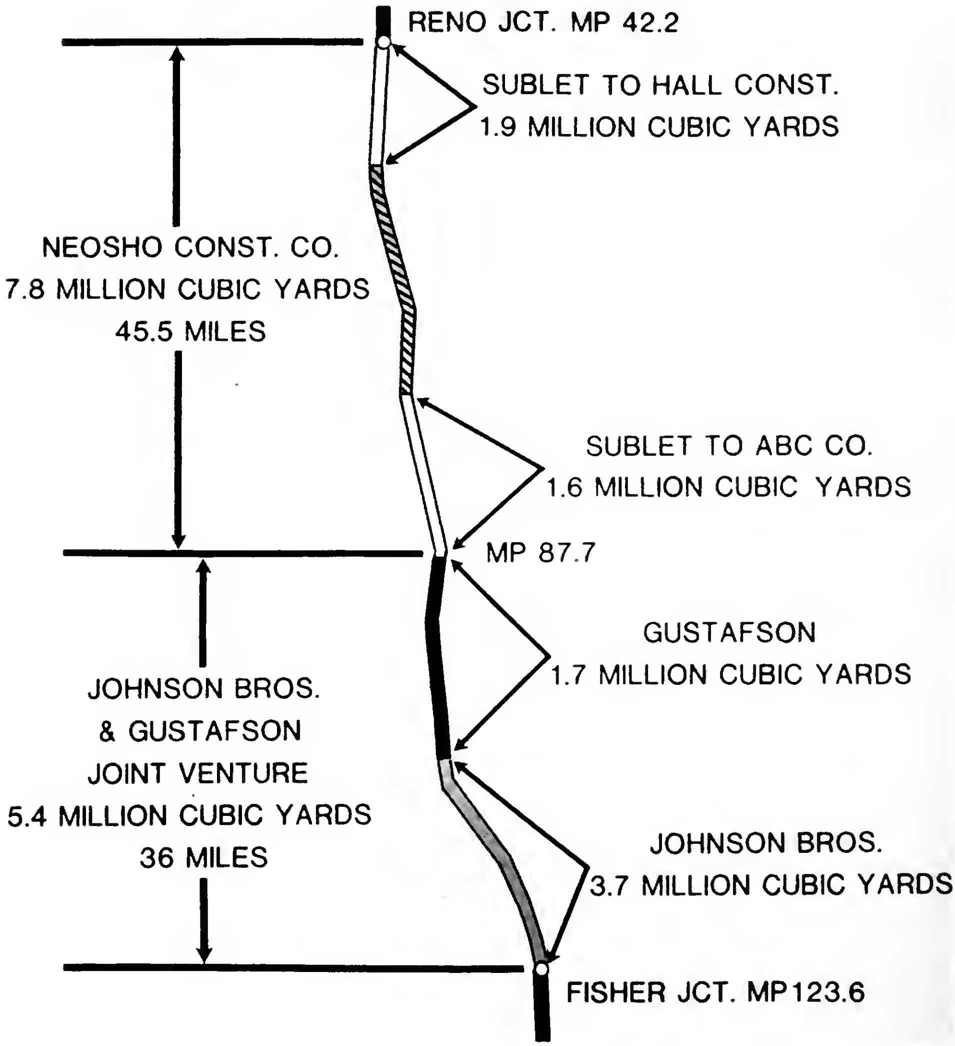


Figure 6.—Distribution of grading between prime contractors and their subcontractors.

onto the fill we placed selected material in the upper three feet. Any material that was suspected of not providing a good substantial embankment was wasted. Moisture was added as required to obtain the proper density and compaction. Each spread of the grading contractors was monitored by a soils technician from the soils engineering firm so that tests were made frequently during the day as the embankment was placed and as the material was excavated from the fills. One of the requirements of the environmental impact statement was that we make all of our cut slopes three to one where they were less than forty feet deep and the same for fills where they were under forty feet high. See Figure 8. Cuts and fills greater than this were permitted two to one slopes.

In the final analysis, although this increased the cost of the project considerably, it did make for a much better roadbed, particularly considering the fact that we are in very heavy snow territory and the flatter slopes provided much greater area for storage of snow during winter months. It also facilitated seeding these slopes which was another requirement of the impact statement but which was also desirable to stabilize the soil which is easily eroded during the summer months from rain and wind. We feel this attention to the soil problem has given us an excellent subgrade and expect that we will have a minimum amount of trouble with it during the years ahead.

Because our source of ballast was located at Guernsey, Wyoming, a short forty miles east of Orin, and because it was much easier to bring other materials including ties and rail into the area over the less congested main line through Casper, it was decided that we would start track construction from the south end. However, because of the shortened construction time, since we knew the line would have to be in service by the end of 1979, it became apparent we would also have to start a track construction gang from the north at some later date.

In September of 1978, we let a contract for the construction of the railroad track itself. Neosho Construction Company won this bid, and we were quite pleased that it turned out this way as they had been involved in the construction of all of the contracts leading down to milepost 42 at Reno Junction. As a result, we were quite satisfied with their capability to progress this project on the expedited schedule we had to meet. As I have said earlier, it was necessary that the project be completed by the end of 1979 and the contract with Neosho carried a completion date of December 31, 1979. Clare Hutchinson, president of Neosho Construction Company, recognized as well as I that to meet this date it would be necessary to have the line operational by December 1, and so our original plans were progressed on this basis. Early in our discussion, we had agreed with the contractor to a rate for laying rail which would govern all of the activities associated with the construction of the track. I felt it important that this rate be reasonably attainable on an average basis throughout the full progress of the job. For this reason, we selected the modest rate of one-half mile of new track per day and established a schedule for the construction on this basis. See Figure 9.

A prior contract had been let to Neosho for reconstructing the three miles of Chicago & North Western track to Fisher Junction. This also included construction of the track on the embankment and across the bridge connecting the Burlington Northern and the Chicago & North Western Tracks at Orin. This work progressed through late 1978 and early spring of 1979 and was substantially completed on May 1 of that year. This date was selected as the date to start construction north out of Fisher.

Grading and placement of sub-ballast and bridge work was scheduled to be completed ahead of tracklaying operations by at least two weeks; however, in one or two locations this time was actually reduced to a mere two or three days. There are four sections of double track on this segment, each of which is five miles in length. Also, it is intended to extend this double track another five miles at each location and grading and bridging were provided in the initial contracts for this second extension. The siding at Fisher is two and one-half miles long and will

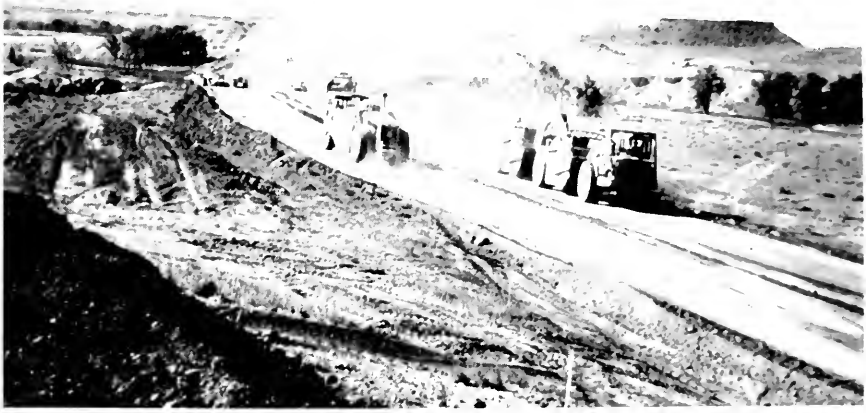


Figure 7.—Panoramic view of grading operations in progress south end of project.

Figure 8.—Typical cut section with 3:1 slopes.

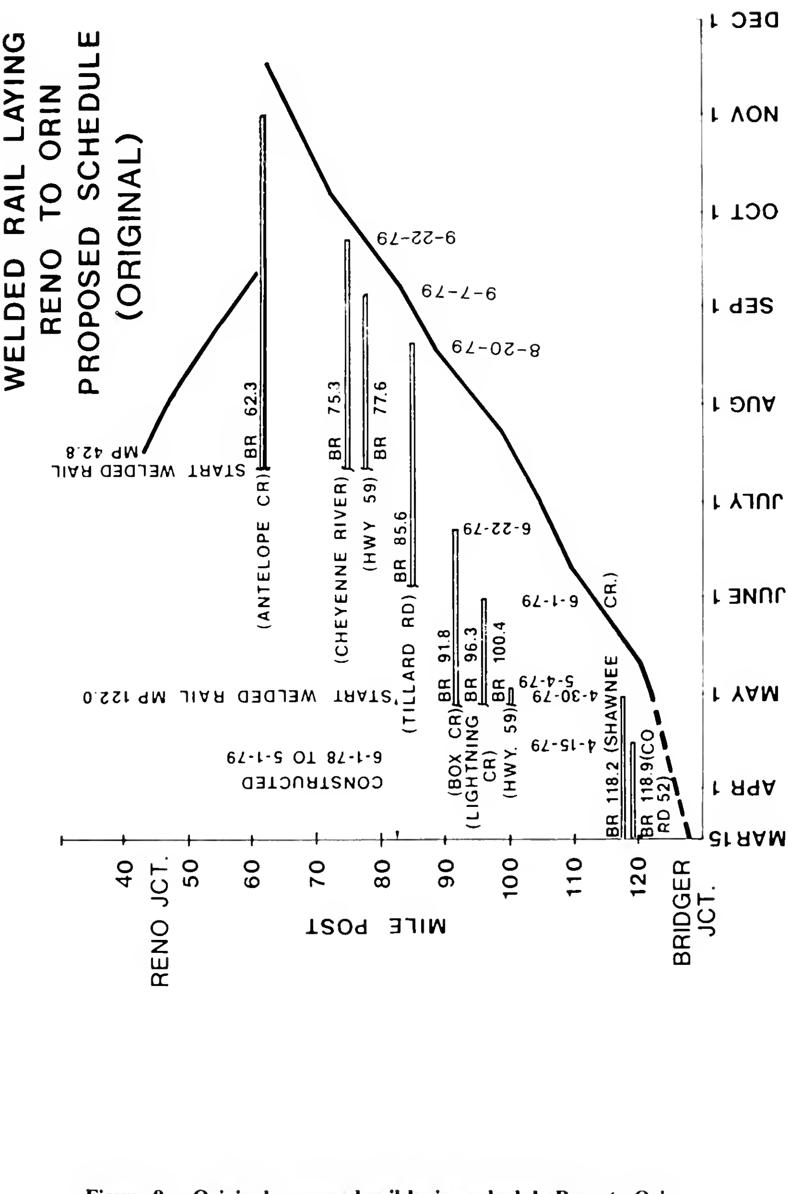


Figure 9.—Original proposed rail laying schedule Reno to Orin.

be operated as a siding and not a segment of double track. Therefore, as our progress chart indicates, where we encountered double track conditions, we slowed down the rate of laying rail to correspond to one-quarter *route* mile a day.

On the 15th of July, the second tracklaying gang was started south out of Reno Junction. Here, there were two obstacles in the way of making equal progress. In the first place, there was no handy source of ballast and ballast would have to be brought into the project through the congested coal territory between Gillette and Alliance. Secondly, the bridge over Antelope Creek, the largest on the project, was considered to be the last structure completed and therefore tracklaying across this bridge could not occur until very late in the fall of 1979. In fact, it was our original plan to connect the two track gangs at this bridge as shown on this diagram. The scheduling of laying south out of Reno was based on the same one-half mile per day with consideration taken to the double track segment that extended five miles south of Reno Junction. On this basis we would have completed the tracklaying by the middle of November and would have allowed us only two weeks to finish the surfacing and other cleanup work behind the tracklaying operation to finish by November 30. The contractor realized this would be cutting it pretty short and therefore he increased his efforts at the bridge over Antelope Creek. In fact, he worked all through the winter months of 1978-79 and was able to advance the completion date of this bridge. See Figure 10. This was not easily accomplished, however, because if you will recall, the 1978-79 winter across the prairie states was particularly severe. Nevertheless, this was accomplished and moved the completion of the Antelope Creek bridge and work south and meet up with the tracklaying gang coming from the north at a point in the vicinity of milepost 72 or ten miles south of Antelope Creek bridge. These two track gangs joined their rail at milepost 72 on October 5. They were not delayed a single day by any of the other work on the project. Bridges as well as grading and sub-ballast were completed within scheduled requirements so as to permit the rail laying operation to continue at its planned rate of one-half mile per day. See Figure 11.

The surfacing operation was started one month behind the rail gang. It was intended that this surfacing would progress so as to be within three weeks of the rail laying operation by the time the two relay gangs connected. The actual progress of the surfacing was of some concern to us, and in August the situation became critical enough for me to step in and make some basic decisions for the contractors. The surfacing subcontractor had one 6500 Jackson Tamper working on Burlington Northern property in the Chicago area, and I directed him to move that machine immediately to the work on the new line. At the same time, the prime contractor located a second 6500 and had it moved into the area. These two machines, working togehter, were able to continue and pick up the tempo on the surfacing. If we could keep ballast to them, they could make three miles a day. Our rail laying operations proceeded on the following basis. We had a requirement in the specification that rail was to be anchored and tied down with ballast between a temperature range of 60°F and 90°F. We watched this very meticulously and this resulted in the contractor adopting the practice of dragging out the four strings of rail in the late afternoon. The first pair would be pulled from the train and spiked sufficiently to the *pre-plated ties* to permit the rail train to be shoved forward on this pair of rail. The second pair of rail was then pulled out and left on the rollers. See Figure 12. The rail train was pulled back off of the newly laid pair of rail and left for the night. In the morning, before the rail temperature reached the maximum limit, the rail was spiked to the ties, anchors were applied, and initial spread of ballast was dumped in order to restrain the rail. See Figures 13 & 14. This sequence of operation was repeated each day.

Our tracklaying operations required a continual supply of forty cars of ballast each day for each tracklaying crew. This we were able to do without any problems since we were close to our source on the south end of the job. On the north end of the job, we stockpiled sixty thousand yards of material during the winter months and then reloaded it and distributed it

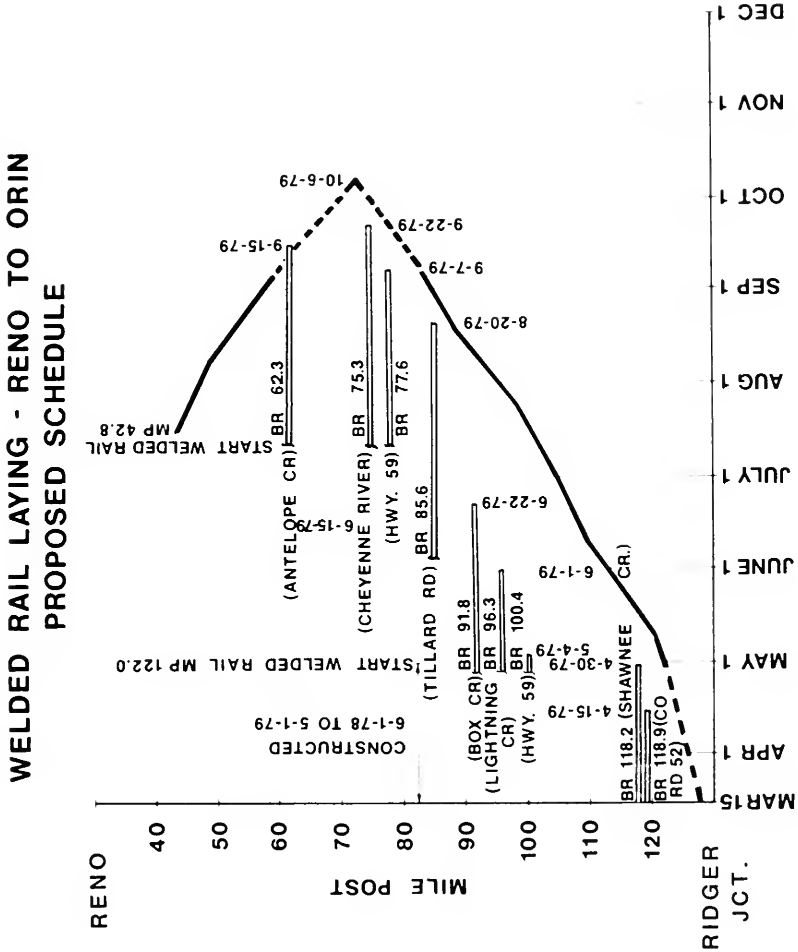


Figure 10.—Revised rail laying schedule Reno to Orin with completion of rail laying October 6.

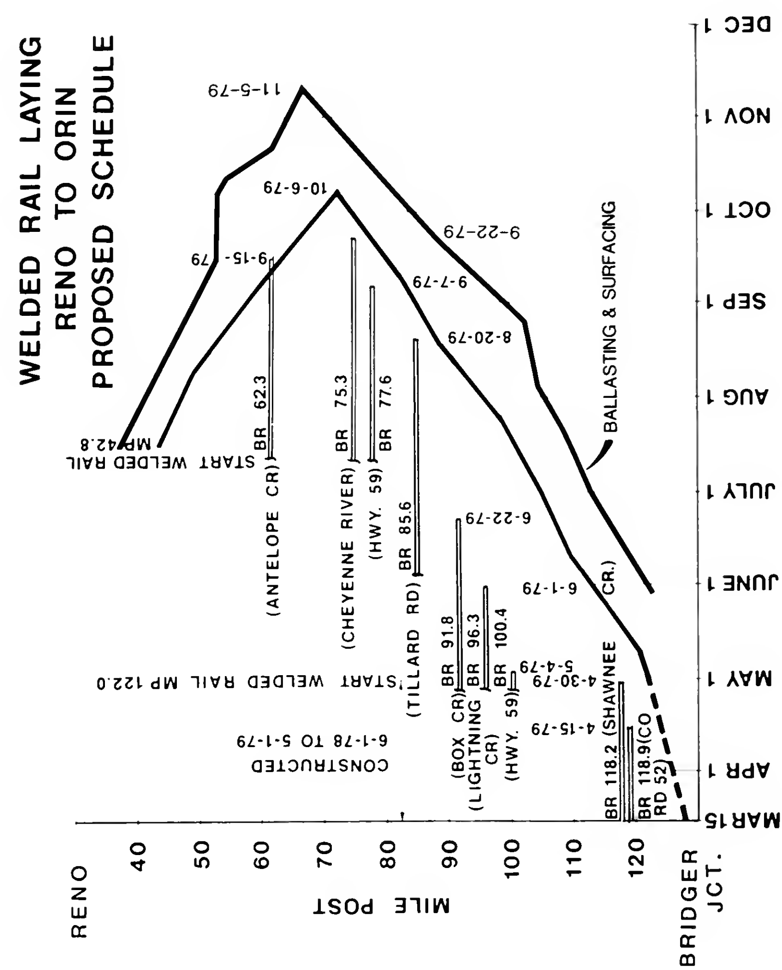


Figure 11.—Actual schedule welded rail laying and ballast surfacing Reno-Orin Junction.

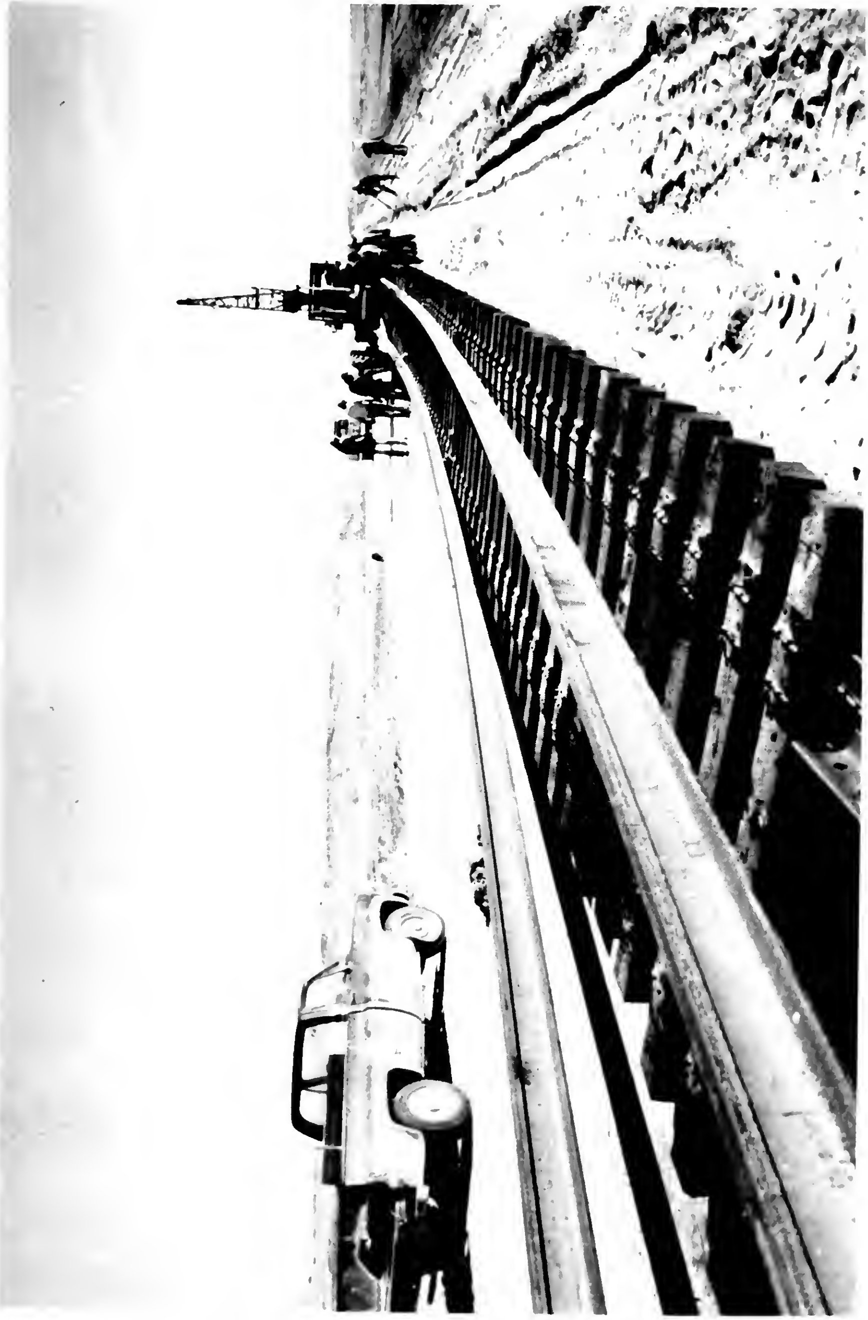


Figure 12.—Pulling pair of welded rail strings onto pre-plated ties.

Figure 13.—Welded rail seated in the tie plates preparatory to being spiked when temperature reaches proper level.

Figure 14.—Initial spread of ballast.

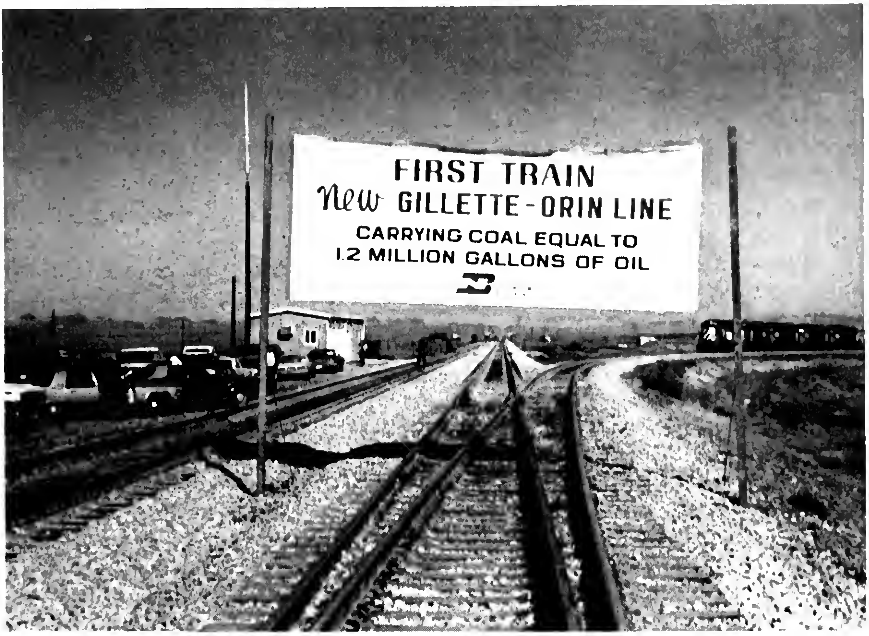


Figure 15.—First unit coal train moving on new line.

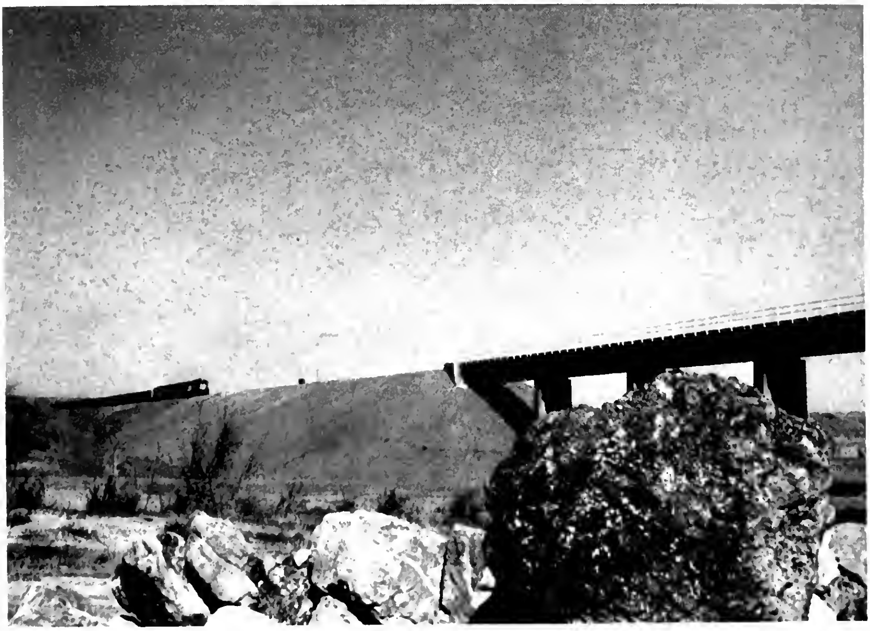


Figure 16.—Antelope Creek Bridge

as necessary, following the tracklaying operation. When the two rail laying gangs joined, we immediately supplemented the ballast on the northerly section with ballast material brought from the south end and we increased our tamping force on the north end so that we were able to make a completion of the surfacing operation by November 5 and permit the first traffic to move over this line on November 6. See Figure 15. This was seven weeks ahead of our original schedule and a full three weeks ahead of the revised schedule that we had agreed to after stepping up construction on the Antelope Creek Bridge. See Figure 16.

As I review the accomplishments of this project, there are several factors that stand out and of which I am quite proud. In the first place, I have already explained to you the care with which we prepared the subgrade. Secondly, with the exception of the consultant on soils engineering and consultant to furnish us topography and calculate our grading quantities, Burlington Northern Engineering Division performed all engineering for this project. This included design of all twenty-eight bridges except the one across U.S. Highway 87 at Orin, all of the culverts, and all of the track and other subgrade structures. Burlington Northern engineering forces also managed this project entirely. Finally, we completed the project well within the estimated cost, ninety million dollars, and as I have mentioned, not only on schedule but ahead of schedule. Still to be done is the installation of C.T.C. and construction of an additional five miles of track on grade and bridging already provided, extending the four five mile sidings to ten mile sections of double track.

I feel that we have constructed a high quality, heavy duty main line railroad for hauling unit coal trains and which, as has been pointed out, is the longest piece of new main line constructed in the U.S. since 1932. The line will benefit Burlington Northern directly by reducing the cycle time between mine and destination of unit coal trains as a result of shortening the distance and relieving the congestion on the northerly route. It also affords the opportunity of greater flexibility in the event of emergencies. These are important considering the vast volume of coal that is moving out of this area today and will not only continue but in fact increase in the years ahead. Not only is this line of great benefit to Burlington Northern itself, but it is an important contributor to the energy effort for the entire country. It will give us a greater ability to deliver coal which is in abundance in this part of the country to the power plants in the middle and south central parts of our nation on schedule and to meet their ever growing demands.

I thank you.

SPECIAL FEATURES

Reinforced Earth Techniques on the Clinchfield Railroad

by J. A. Goforth[1] and Victor Elias[2]

This report will be a presentation on the application of the Reinforced Earth technique to rebuild an unstable roadbed and slide area on the Blue Ridge mountain in North Carolina.

The Clinchfield's line descending the south side of the Blue Ridge mountain was constructed in 1906-08 and is an engineering masterpiece, even by today's standards. From an elevation of 2,628 feet at Altapass, North Carolina, the line is on a 1.2% compensated grade, unbroken for twenty miles. The maximum curve is 8 degrees. The development loops require 18 miles of track to cover a distance of 2.3 miles as a crow flies. There are 17 tunnels in a distance of 11 miles.

The many high fills were constructed by methods typical of that era—that is, without compaction. The steep sloping hillsides were not benched prior to placing the fill material and the fills were not started at the lowest point and built up in compacted, uniform layers, Instead, the fills were built by dumping the loose material from the ends of the cuts and keeping the fill surface to profile height as the dumping progressed across the fill. The fills were "humped" toward the center in an amount determined by a questionable formula to allow for settlement.

I have seen pictures of the fills under construction. The tree stumps were left standing some 18 inches and the vegetation of the surface of the ground was not removed prior to placing the fill material.

The native soil material with which the fills were built is a micaceous, sandy clay typical of that found in the lower elevations of the Piedmont which was forced into the higher elevations by upheavals in the past. The solid rock is of igneous origin, being a mica schist or trap rock, very resistant to weathering as evidenced by the many unlined tunnels in the area. The native soil becomes very unstable when wet. Each year during the period from late January through April, several of the high fills on the Blue Ridge begin to settle. It is an annual ritual during this period of patrolling the Blue Ridge mountain and correcting the line and grade on the fills.

The month of March, 1979, was a very wet period on the Blue Ridge. There were continuous rains for several days climaxed by a heavy deluge. The several troublesome fills began to act up but our maintenance forces were able to control them. The fill located at M. P. 189.3, first slid-off from the heads of the ties. Restoration work was underway with on-track ditching equipment, when the heavy deluge mentioned above hit the area and the entire fill gave way, sliding down the mountain like so much fresh concrete, leaving the track structure hanging in the air like a suspension bridge.

The first efforts to restore the fill with timber cribs supporting the track and filling the void with ballast dumped from hopper cars proved to be futile. A roadbed was cut along the hillside above the slide area, a run-around track installed and traffic was restored. The annual traffic density at this location is 25 million gross tons per mile, much of it being unit coal trains.

Having restored the line to traffic, the next consideration was rebuilding the fill and replacing the track on the original alignment. Several alternative solutions were considered.

[1]Chief Engineer, Clinchfield Railroad.

[2]Vice President, Engineering, Reinforced Earth Co.

The first and quickest would be rebuilding the fill with native soil from adjacent mountain side. Work was actually started and then abandoned in favor of some method that would give a stable, permanent, trouble-free fill. We considered building a concrete retaining wall near to toe, but poor foundation conditions ruled this out. We considered gabions of rock-filled, wire cages. I recalled having seen a presentation at the Annual Transportation Conference at the University of Tennessee of how an unstable fill on Rockwood Mountain on Highway Interstate 40 had been permanently contained with a new technique as the "Reinforced Earth Fill."

An inquiry was made to the home office of The Reinforced Earth Company as to the adaptability of the reinforced earth fill to our situation. We were told that a reinforced earth fill had not to that time been installed under a live railroad loading, but the concept was sound and would give satisfactory results in such a location. Studies were made of the several alternative solutions. The reinforced earth was selected, (1) permanence; (2) short installation period (the time was now late summer and it was imperative that we have the work finished and the track moved off the temporary grade by winter); (3) favorable costs when compared to alternative methods.

Engineering was completed, the materials were acquired and moved to the job site. The actual work began in late October consisting of the excavation, placing the filter media, sub-surface drains and footing. The first wall units were set on December 5, with the wall being completed December 23.

The erection of the wall units required no special skills and was performed by one of our Bridge and Building forces with no previous experience in this type of work, assisted by a local contractor.

Mr. Victor Elias of The Reinforced Earth Company, Washington, D.C., will now make a slide presentation of the construction procedures.

The roadway fill above the reinforced earth portion was built with selected material and compacted. Filter fabric was laid on the finished grade with no sub-ballast. The track was surfaced on an average of eight inches of ballast and opened for traffic on January 7. Up to this time, we have observed no settlement in the fill and the track supervisor reports that he has not had to do any resurfacing or lining to the track since the initial work.

We have several other troublesome fills on the Blue Ridge which we expect to begin permanent restoration work on this summer. The reinforced earth fill will be given prime consideration at each location.

II. DESIGN PRINCIPLES

The key element is the friction between the earth and the reinforcements. Through this friction, the earth transmits to the reinforcements the stresses which develop in the mass. The reinforcements are thereby placed in tension and the composite material gains a pseudocohesional strength which is directly proportional to the tensile strength of the reinforcements and acts in the direction of their placement.

The design of Reinforced Earth structures, at present, consists of considering the local equilibrium between the facing elements and the reinforcing strips under the assumption that the reinforced volume is in a state of limit equilibrium and that the principal directions of the stresses are vertical and horizontal. The reinforced backfill is treated as a composite material that has both the frictional strength of the granular soil and the pseudocohesion imparted by the reinforcement.

The design further considers that if a failure wedge develops it will occur in the unreinforced fill beyond the reinforced volume, thereby treating the reinforced fill as a single gravity unit. The applicaton of surcharges such as from seismic events or imposed dynamic tract forces requires the calculation of the additional tensile forces which the reinforcing strips must resist due to this additional load, as schematically shown in Figure 3.

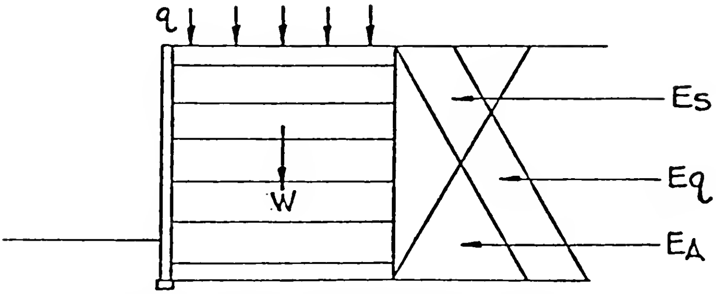


Figure 3. Schematic of Forces of a Reinforced Earth Structure

q	:	is a surcharge load
W	:	is the total weight of the Reinforced Earth mass
E_A	:	is the horizontal pressure due to earth pressure
E_q	:	is the horizontal pressure due to surcharge
E_S	:	is the pseudostatic horizontal pressure due to a seismic or dynamic event

III. DESIGN METHODS

The present design method requires that the external mass stability, as shown on Figure 3, based on initial proportioning, is checked by conventional static methods prior to internal stability design. For structures not subjected to large surcharge loads, the length of reinforcing strips is generally chosen at 70% of the wall height, except that a minmum length of 14 feet is necessary regardless of size to insure integrity.

Specifically, the checks for external mass stability consist of determining the adequacy of the mass against overturning moments generated by earth pressure and other imposed loads as well as sliding stability along the interface with the foundation soils. Consistent with the above, bearing capacity analyses for both general and local shear are made using Meyerhoff theory for bearing capacity of the foundations under eccentric loadings. It should be noted that the relevant factor of safety is computed with respect to the ultimate bearing capacity of the foundation and should not be confused with more conventional foundation analyses for rigid structures which generally compare the imposed loads to an allowable bearing capacity. This latter capacity in cohesionless soils is usually established to limit settlements, which are not a problem for flexible structures such as Reinforced Earth. Therefore, Reinforced Earth structures can be successfully constructed in areas of weaker foundation soils without the added cost of positive foundation support required by traditional rigid retaining walls.

Internal stability design consists of developing the appropriate horizontal pressure envelope and designing the reinforcing strips for sufficient cross-sectional areas to carry the horizontal loads as well as for sufficient surface area to transfer in friction the stresses which develop in the mass. Minimum bond lengths can be obtained under the assumption that the earth-reinforcement friction is fully mobilized, and that the normal stress is uniform and approximately equal to the overburden pressure.

Significant data have been published recently by the junior author on the variation of the apparent coefficient of friction f* used in bond calculations in Reinforced Earth structures. It was shown that the phenomenon is complex in which the density and dilatancy of granular fill are predominant factors as well as the nature of the strip surface. In general, the apparent coefficient becomes asymptotic to a value equal to the tan. ϕ for ribbed strips where the shear is essentially a soil to soil interface phenomenon.

IV. MATERIALS OF CONSTRUCTION

Reinforced Earth structures contain three essential materials of construction:

1). the reinforcing strips

2). the facing material

3). a granular backfill

The reinforcing strips used on most applications are rolled from steel billets to a 5mm thickness, with transverse ribs, and standard widths of 40 and 60mm. They are subsequently cut to the required length. The steel properties conform to ASTM A-36 and they are hot dip galvanized in accordance with ASTM A-123. They are bolted to the facing panels, which have protruding clips or tabs cast on their back side.

The galvanization provided in excess of 2 oz./sq.ft., is adequate to insure a longevity well in excess of 150 years in environments not characterized by excess acidity or alkalinity. In severe environments, other materials or coatings may be used, such as aluminum or more recently fusion-bonded epoxy coatings.

The facing material most commonly used is a square concrete panel having dimensions of 5 feet by 5 feet generally 7 inches thick and weighing in the neighborhood of 1 ton. These panels are produced in an industrialized mass production environment in precast yards generally in the vicinity of projects. They are lightly reinforced and cast to a concrete strength of 4000 psi.

The required backfill material is a well graded granular material conforming to the following gradation specifications:

Size	*Percentage Passing %*
6 inches	100
2 inches	75-100
#200 Sieve	25

Adherence to these specifications insures both that the required friction is developed between the backfill and the strips and that this generally free-draining fill will preclude the development of hydrostatic forces on the wall face.

During panel erection the joints are filled with a cork strip on all horizontal joints and a polyurethane foam in the vertical joints.

V. CONSTRUCTION METHODS

The erection of Reinforced Earth walls is a simple repetitive process of erecting in an alternate fashion, rows of panels which are then bolted to the reinforcing strips. The strips are covered with backfill up to the next row of strips and the process is repeated until the completion of the wall.

Specifically, the base of the wall is made up of alternate full panels and half panels. These stand directly on the previously poured levelling pad. The half panels are positioned first, and the full panels inserted between them to constitute the first lift of panels as shown in Figure 4.

Backfill is placed in the direction in which the course of panels is placed. After the first course of panels is positioned, the fill is brought to the level of the lower tie strip on the half panels. Reinforcing strips are then attached to the lowest panel tabs and backfill is brought over this layer and compacted utilizing conventional embankment compaction equipment as shown in Figures 5, 6, and 7. The process is then repeated until the top of the structure is reached. The intermediary stage and the finished structure is shown in Figures 8 and 9.

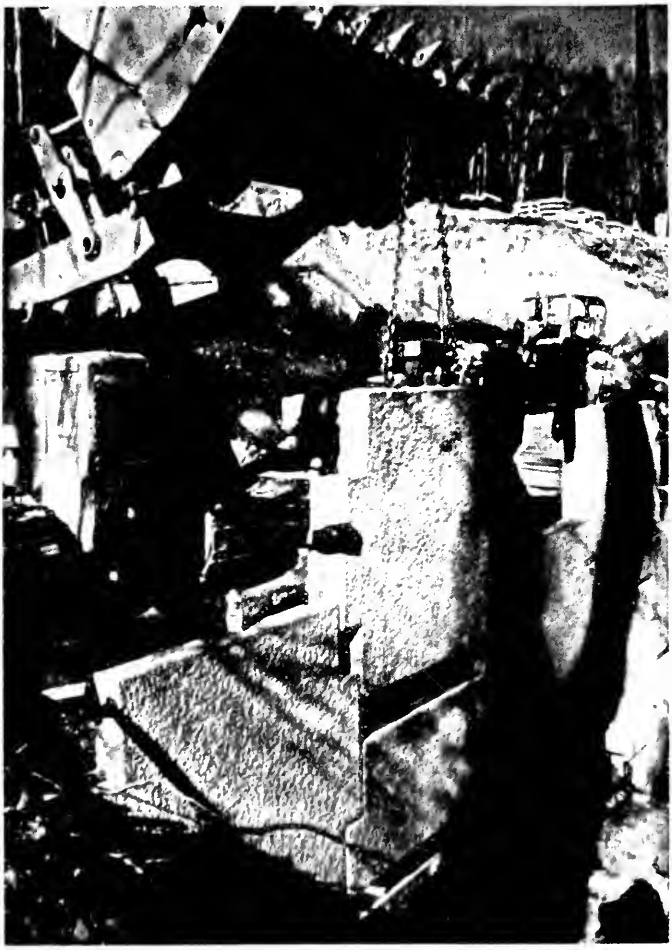


Figure 4. Erection Sequence

Figure 5. Placement of Backfill

Figure 6. Placement of Reinforcing Strips

Figure 7. Compaction of Granular Backfill

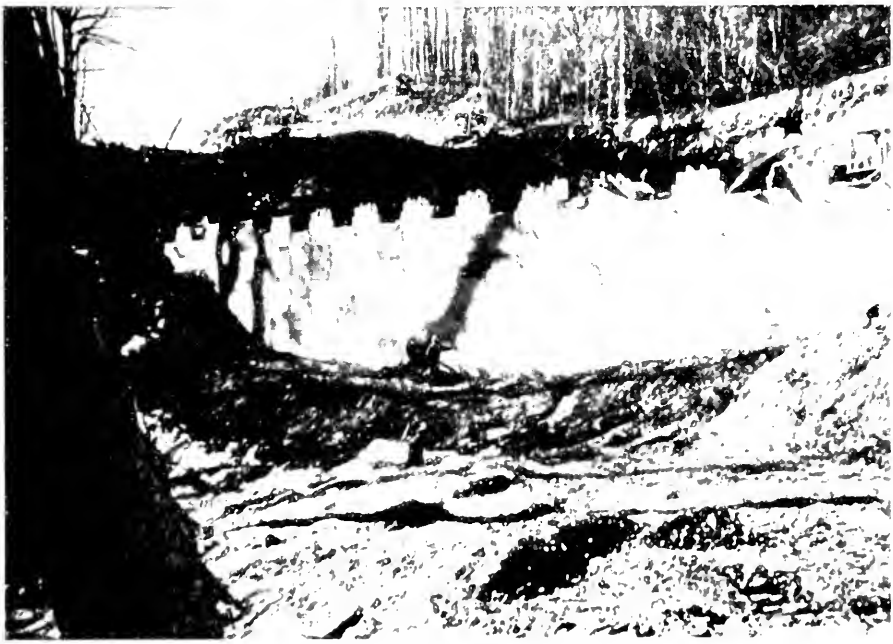


Figure 8. Reinforced Earth Under Construction

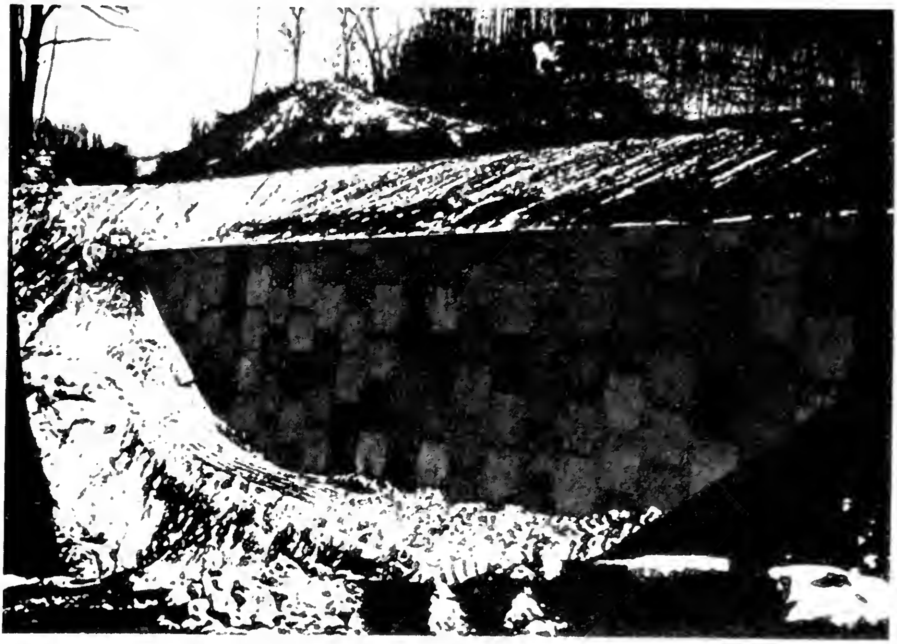


Figure 9. Completed Reinforced Earth Structure

Results from Caldwell, Texas Geotextile Tests on Southern Pacific

by Tom Barnett[1] and Jack Newby[2]

Track Structure Test Program Review

Today, we would like to give you an overall picture of the track structure test program, currently underway at our test site in Caldwell, Texas, to evaluate the functions of Geotextiles in the rail track structure. This program, a joint technical effort between Southern Pacific and Monsanto, was structured to provide new insight into the behavior of these materials which are coming into increased use, both at the southern Pacific and at other roads.

The earliest known use of fibrous, nonwoven structures in the track structure in the United States was done by Mr. Newby in 1968 when he used a glass fiber mat in a tunnel near Chatsworth, California and in roadway crossings and switches in the Pacific system. In 1974, Monsanto began marketing Bidim® engineering fabrics in the United States and began to install fabrics with many railroads.

The early results obtained were excellent. Fabrics solved my problems associated with ballast contamination, ballast pocketing, and track structure stability. Dramatic differences were observed when side-by-side tests were conducted. Many early tests were conducted in crossings as they were perennial problem areas and offered the opportunity for side-by-side comparisons with little risk, if the promises of the fabric suppliers were not fulfilled.

Because of the mounting evidence of the effectiveness of fabrics in the track structure, but faced with a lack of knowledge on the precise mechanisms of fabric behavior in railroad applications, Monsanto, in 1977, approached the Southern Pacific Transportation Company with a proposal for a full scale test program in which geotextiles would be evaluated using an instrumented track structure.

Various functions of fabrics had been proposed. One, which is the most obvious, is that of filtration, key to controlling the upward migration of fine soil particles into the ballast.

In our field test, we are defining the level of filtration performance through evaluation of ballast samples removed from above the fabric and, in a more general way, by visual means. Mr. Newby will present data which will clearly show the effectiveness of geotextiles in preventing ballast contamination, through the filtration mechanism.

Of equal importance to filtration is the function of separation whereby the materials of the ballast and/or subballast maintain their individual properties and downward movement of the granular materials into soft subgrades is eliminated. Our field experiments include analysis of ballast and subgrade field samples, visual examination, measurements of top of rail and subgrade profiles.

Prior to this field test, we had seen evidence of fabrics altering the movement of moisture into the subgrade. We also felt that there was an influence played by the role of fabric structure since both vertical permeability, that is permeability perpendicular to the plane of the fabric and planar permability, varied from fabric to fabric as a direct result of such things as fiber size, fabric weight and bonding system.

[1]Sales Manager-Western Region, Bidim Engineering Fabrics/Monsanto Textiles Co.
[2]Geotechnical Engineer, Southern Pacific Transportation Co.

Fourthly, our experience with fabrics in access roads and the results of a test program conducted by Law Engineering showed that fabrics could undergo in-situ tensioning and had greater load carrying capacity than would have been anticipated had they not been contributing a vertical upward force. Our test program was designed to measure earth pressures, displacements, accelerations, and deflections.

The program consists of two elements—the major phase has been the design construction, field testing and data analysis of the Caldwell Test, but concurrently we have been evolving a mathematical model to treat the track structure with or without a fabric. From the integration of these data, we will develop design guidelines for the proper utilization of geotextiles in the railroad industry.

The program is designed to look at total system response from the top of rail into the subgrade. The outputs showed how loads are distributed from the rails to the ties, from the ties to the ballast and from that point into the subgrade.

The load carrying capacity of weak soils and its modification by the introduction of a fabric layer will be a significant output.

Other outputs will be the effect of fabrics on moisture movements into and out of the subgrade, the role of fabrics in controlling ballast contamination, how fabrics can lengthen the maintenance cycle and lessen MOW costs through improved track stability.

And importantly, for those of us involved in supplying geotextiles, is what role is played by polymer types such as polyester (PET) compared to polypropylene since these materials have substantially different physical properties. And, also, to define the role of fabric structure, that is, the influence of fiber denier, fiber orientation, web bonding techniques on the performance of geotextiles in track stabilization.

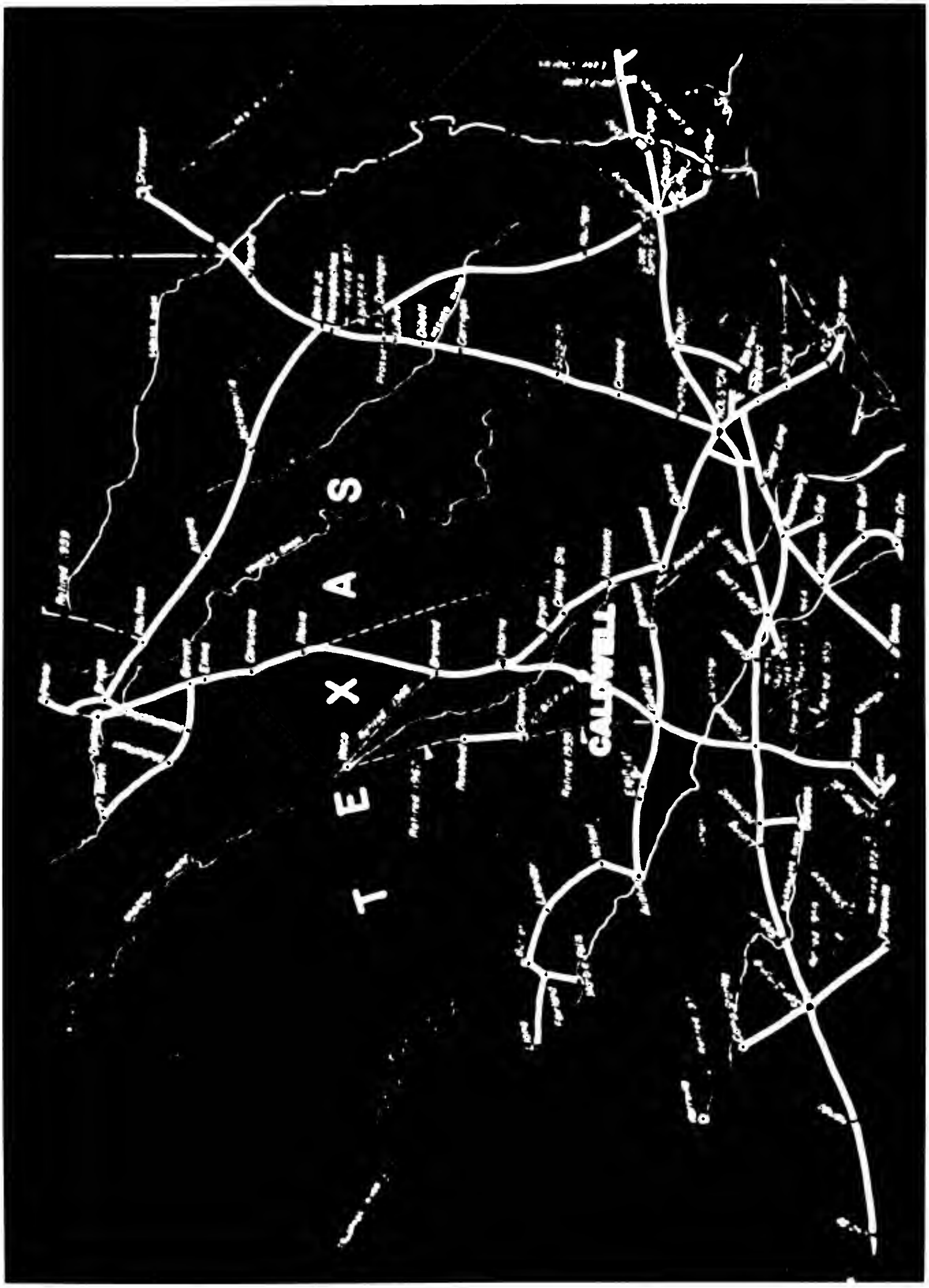
An extensive site selection program was undertaken to find a suitable location for the test site. The presence of very poor soil conditions was a prime requirement as was a high level of rainfall. The final choice was a location near Caldwell, Texas located about 25 miles west of College Station, Texas. (Figure 1.) The test site was located in a siding and was built as part of a siding enlargement project.

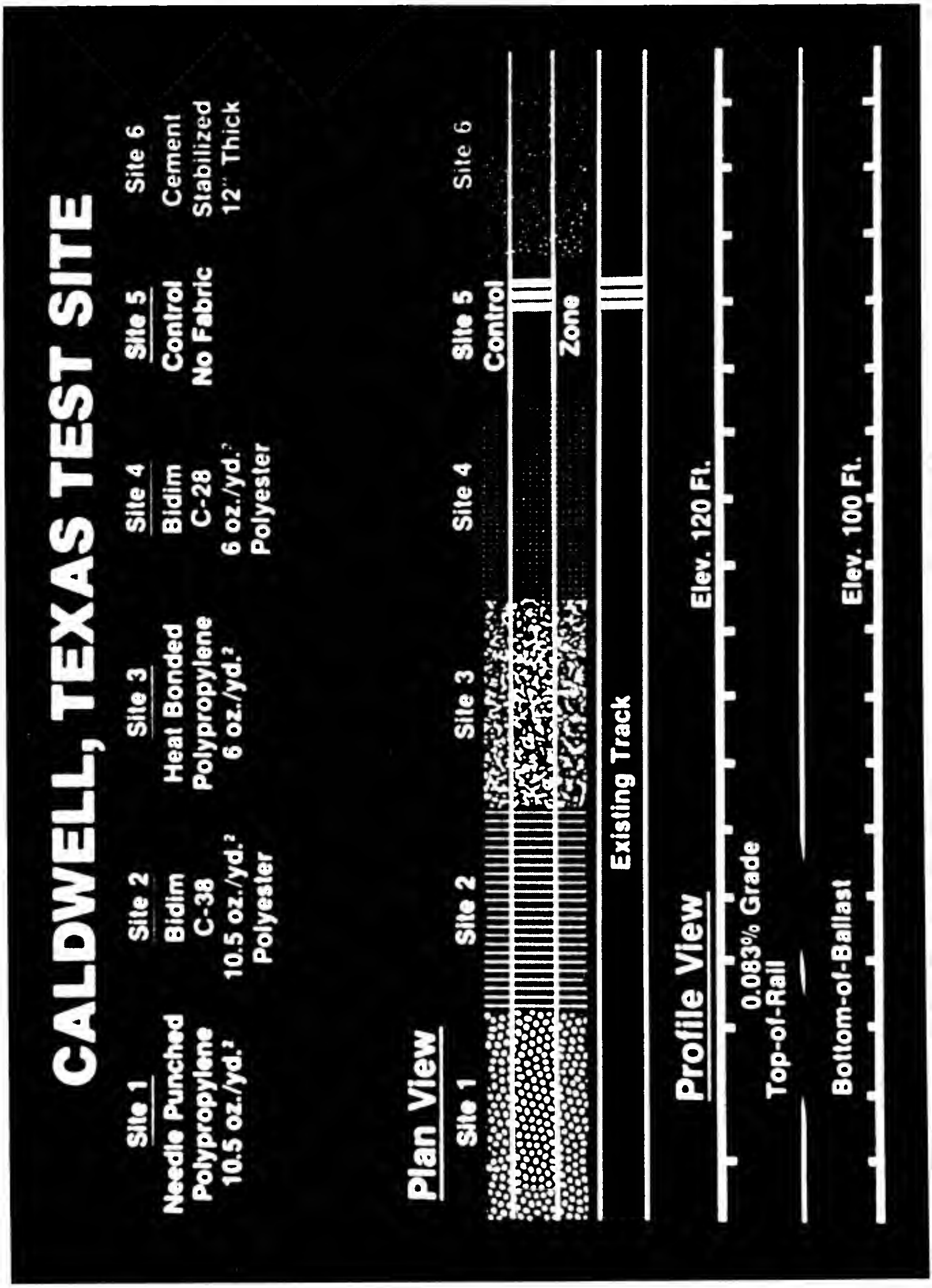
Within the site, there are six test sections. (Figure 2). Four have fabric; one is a control zone without fabric and one is a cement stabilized section. This particular selection of fabrics was done to allow us to make comparisons that could be translated in a much more general manner.

First we wanted to see if fabric weight was an important factor. Would heavier fabrics outperform lighter weight fabrics? Secondly, how do equivalent weight fabrics compare if they use different base polymers or different bonding systems? How do each compare with the control zone having no fabric, and how do they compare with conventional stabilization techniques, in this case, cement stabilization?

In Site 1 is a 10 oz./sq. yd. needlepunched polypropylene fabric. In Site 2, there is a 10 oz./sq.yd. needlepunched polyester fabric—Bidim C38. Sites 1 and 2 are used as comparisons of different polymer types with fabric of equal weight and equivalent structures, both using the needlepunch method of fabric fiber bonding. The data from these sites is used to define the performance of heavy weight fabrics and this contrasted against the control zone, cement stabilized site and the two lighter weight fabrics in Sites 3 and 4.

In Site 3 there is a 6 oz./sq.yd. heat bonded polypropylene fabric. In the heat bonding process, a fiber web is passed through heated rolls which causes bonding at the fiber crossover points. The resulting fabric is thin in cross section and has low lateral moisture permeabilities compared to the more lofty needled fabrics. This site is used to compare differences in


T E X A S
CALDWELL


CALDWELL, TEXAS TEST SITE
Site 1
Needle Punched
Polypropylene
10.5 oz./yd.²
Site 2
Bidim
C-38
10.5 oz./yd.²
Polyester
Site 3
Heat Bonded
Polypropylene
6 oz./yd.²
Site 4
Bidim
C-28
6 oz./yd.²
Polyester
Site 5
Control
No Fabric
Site 6
Cement
Stabilized
12" Thick
Plan View
Site 1
Site 2
Site 3
Site 4
Site 5
Control
Zone
Site 6
Existing Track
Profile View
Elev. 120 Ft.
0.083% Grade
Top-of-Rail
Bottom-of-Ballast
Elev. 100 Ft.

moisture transport characteristics between thin fabrics and thicker fabrics and to compare the performance of light weight versus heavy weight fabrics.

In Site 4 a 6 oz./sq.yd. needlepunched polyester fabric is located. This site compares a light weight polyester fabric to the light weight polypropylene fabric in Site 3, contrasting the effect of polymer type and fabric structure on performance and also serves as a comparison site to the heavier fabrics in Sites 1 and 2.

All fabrics test sections are 300 feet in length. Site 5 is the control site which uses no fabric.

The control zone is 150 feet in length to reduce the amount of maintenance projected since we anticipated early failure of the zone.

Site is the cement stabilized section which was incorporated to compare a conventional stabilization technique currently used by the SP in areas of Texas and Louisiana.

The cement stabilized section is 12 inches thick and used 80# of cement per sq.yd. to bind the screening aggregate.

Eight inches of ballast was used in all sites.

Following the preparation of the subgrade and the installation of the subgrade instrumentation, the fabric was put in place.

The siding was constructed with new ties and 136# CWR simulating current mainline practice, 9 foot ties used with a spacing of 22½ inches. Construction of the track took place on the fabric which was placed directly on the subgrade.

The first lift of ballast was placed and the track lifted the initial 4 inches. A second lift was placed and the entire test section tamped.

In each of the test sections, there is an extensive instrumentation package (Figure 3) designed to measure the responses of the subgrade to varied test conditions. Each of the systems will be reviewed.

Instrumentation is designed to start at the top of rail and follow the loads as they pass through the structure into the subgrade. Rail strain gages (Figure 4) which are placed at four places around the rail, two locations under the ball and two locations on the rail flange at the toe. This grouping of four is placed in five places longitudinally along the rail, two over ties and three mid-span.

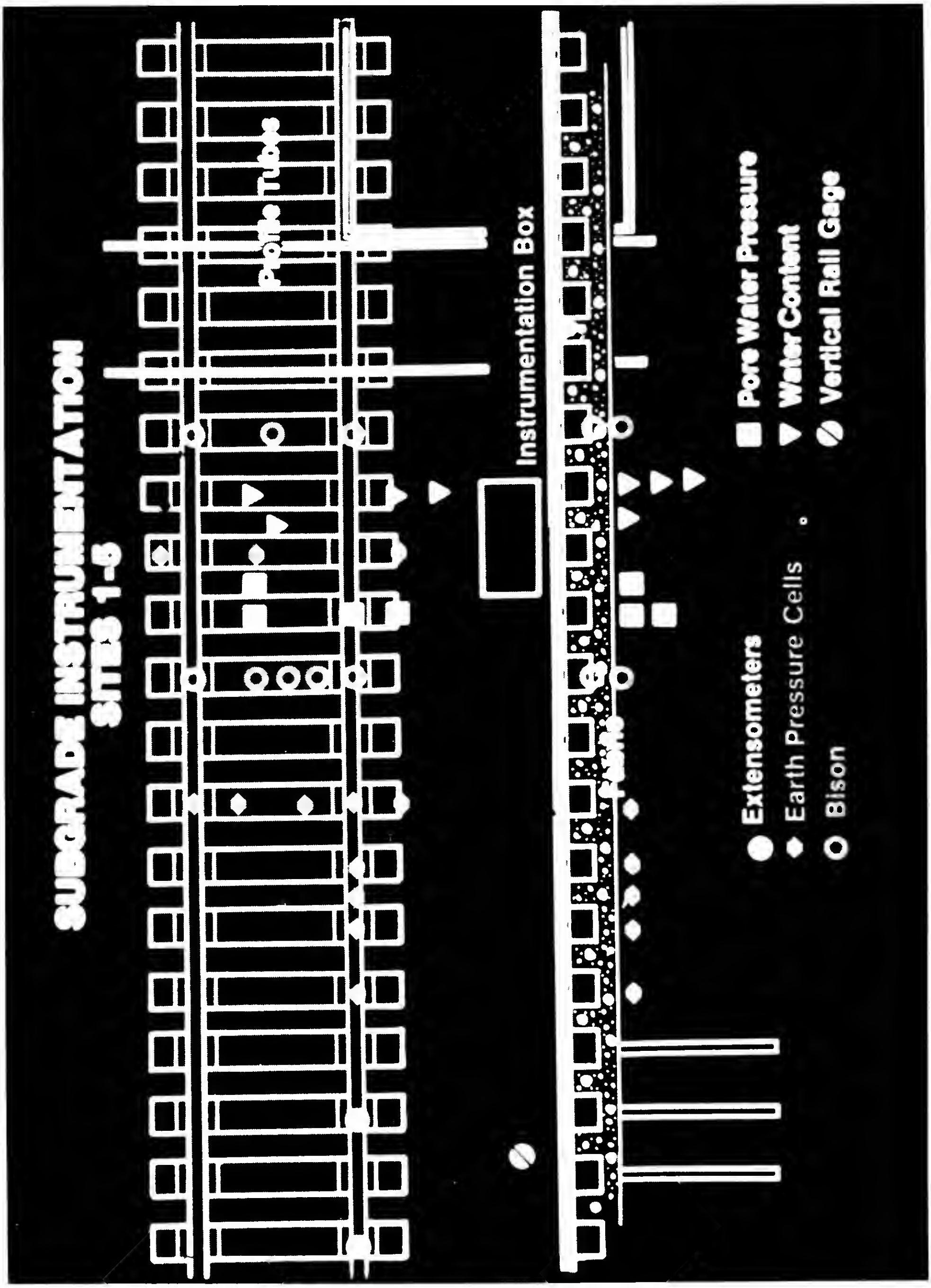
Beneath the rail are tie-plate load cells which measure the actual loads fed from the rail onto the tie. Another feature is the ability to couple output from the tie-plate load cell to extensometers to generate load/deformation curves using an X-Y plotter.

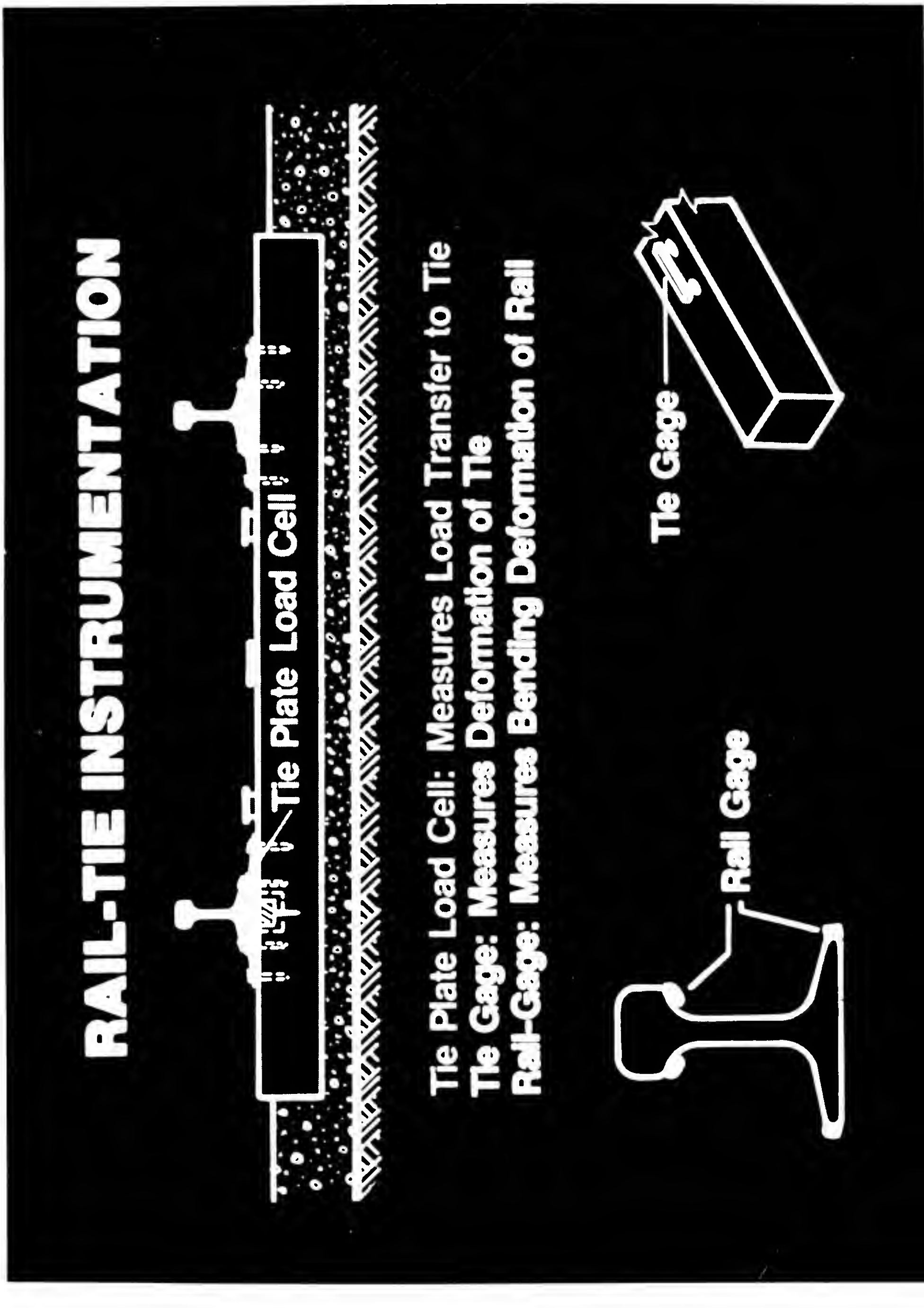
On the tie itself, we placed tie strain gauges at three points along the span.

The tie-plate load cell, which was built into the same type of tie-plate used in the construction, is easily moved from location to location and has a capacity of 50 KIPS.

For measurement of dynamic vertical displacements in the subgrade, a series of three extensometers are placed longitudinally along the rail under the tie-rail intercept. These devices have resolution of .001 inch. The measurements are taken at a point just below the fabric or at the original ballast/subgrade interface in the control zone and at the subgrade/cement stabilized section interface in Site 6.

The extensometers are placed in telescopic plastic tubes which in turn are in borning holes that are 10 feet deep. The extensometers are anchored in the soil at the 10 foot depth to isolate the subgrade movements at the top of extensometer.


SUBGRADE INSTRUMENTATION
SITES 1-5
Profile Tubes
Instrumentation Box
Fabric
Extensometers
Earth Pressure Cells
Bison
Pore Water Pressure
Water Content
Vertical Rail Gage


RAIL-TIE INSTRUMENTATION
Tie Plate Load Cell
Tie Plate Load Cell: Measures Load Transfer to Tie
Tie Gage: Measures Deformation of Tie
Rail-Gage: Measures Bending Deformation of Rail
Tie Gage
Rail Gage

To evaluate the distribution and intensity of earth pressures in the subgrade, a series of earth pressure cells are located both longitudinally and transversely along the center line of the rail and the tie. These cells have a sensitivity of .01 psi. These 2 inch diameter sensors were placed in specially prepared cavities at a level approximately 3-4 inches below the ballast/subgrade interface. The sensors have given some very interesting and unanticipated results which Mr. Newby will cover later.

A series of flexible tubes have been placed in the subgrade at two different depths transversely and at one depth coaxially. The function of these tubes is to detect any permanent deformation within the subgrade. We measure these contours using a sophisticated electronic manometer. The profile tubes were placed in carefully prepared trenches and carefully compacted in place.

Two types of soil mositure instrumentation are used. The first, is a resistance type of gauge manufactured by Soil Test. The second type, called the Ecotec moisture gauge uses a capacitance principle for operation. Calibration curves for both instruments were generated using native soils.

In addition to the soil moisture gauges, pore pressures are measured using specially modified Petur pore pressure gauges.

A variety of accelerometers are used to evaluate total system response. These are located on rail, tie, ballast, and at various locations in and on the subgrade.

Besides taking these readings, soil samples are periodically taken to evaluate soil moisture levels and the gradient of soil moisture within the subgrade.

Because of the extensive data output potential with this amount instrumentation, it was necessary to employ a high-speed data acquisition system to record and store the data. We used a Kinemetrics Data Acquisition System which can handle forty channels of data at 10,000 data points per second. Data from the various dynamic sensors such as rail strain gages, earth pressure cells, and extensometers is converted from analog to digital inputs on magnetic tape. The tapes are then processed using a computer and the desired data retrieved, subjected to statistical analysis, or plotted by Calcomp or other line plotting techniques.

Data from static sensors, such as soil moisture and pore pressure gauges are taken manually using the readout boxes associated with the particular system.

In addition to these data, other data is generated by survey, soil sampling for moisture, and by visual inspection.

A trailer houses the data acquisition systems and a communications system tied in to the SP operational lines. It serves as the base for all test operations.

As part of the overall test program, an attempt is being made to model the track structure and to simulate responses seen in the various test sections.

This has been a difficult and complex problem, but recent developments indicate that these problems are nearing solution and that a design methodology using fabrics as a component of the track structure is a distinct engineering possibility and that cost/performance value for fabrics can be validated.

Now I'd like to introduce Mr. Jack Newby, Geotrchnical Engineer of the Southern Pacific Transportation Company, and the man responsible for the SP interface in this joint technical effort.

Quite rightly, Jack could be called the father of geotextiles in the American Railroad Industry as it was his pioneering efforts in using unique materials, such as plastic films, glass batts, and the newer synthetic geotextiles that showed that it was possible to control pumping, ballast contamination, and develop a stable track structure. Jack will review some of the interesting and significant findings that have arisen from this test at Caldwell, Texas.

Introduction

The installation of instruments in railroad track structures and subgrades for test purposes was a dream that came true. Many changes in railroad operations and track structure—such as heavier loads, increased speed, heavier rails, longer cross ties, etc., and more precise instruments for testing are available—since Mr. Talbot performed his tests about 1919. There have been attempts to analogue subgrade behavior of railroads with highways, but in my opinion, major difference in the reaction of these two types of facilities prevents direct comparison. The tests at Caldwell, Texas are on only one type of subgrade, but the results should provide a major step in further development of field, laboratory tests and design of railroad subgrade.

Existing methods of railroad roadbed stabilization have been very expensive with long train delays, but stabilization by using recently developed fabrics offers relatively low cost and a minimum of train delays. The behavior of these fabrics is one of the main reasons for Caldwell Tests, but beyond this, we hope to get a much expanded understanding of the track structure.

Before we selected a particular site for this test, we established a set of criteria for the test location. First, it had to have soil conditions which were known to cause problems with pumping, intrusion, contamination and track stability. Secondly, the rainfall must be high and frequent enough to allow these conditions to exist over a long period of time. Thirdly, since our instrumentation package installation time window was an unknown, we couldn't use a main, and fourthly, we wanted to be in undisturbed soil that was as homogeneous as possible throughout the site. After six months of visits and evaluation of various potential locations in Texas and Louisiana, we selected the Caldwell location as best filling the desired conditions.

The Caldwell mainline railroad and old siding had a long history of unstable subgrade conditions. When a cut was made into the shoulder of the main track during construction, it exposed the contaminated ballast under the track. The clays had penetrated up to near the base of cross ties.

Further evidence of the type of pumping associated with these types of soils could be seen on a crossing which adjoins the test site on the mainline section of track.

The subgrade properties are average shear strength, at field water content, of less than 20.5 psi, average field water content—34%, liquid limit—75%, plastic limit—30%, and plasticity index—45%. The classification of soil is A-7-5 by AASHTO and CH by Uniform Classification System. This is a very highly expansive clay commonly found in the Gulf Coast area of Texas and Louisiana.

Having the test site in an area of high rainfall was of major importance. During the period from October, 1978, through October, 1979, the total accumulated rainfall was 65 inches. The spring months are clearly the wettest and there is a drying trend in the late summer and fall. Temperatures range from very hot in summer, to the 10's and 20's in winter.

The inputs fro SP into the test are as follows:

Test Site	TOPS Program
Construction	Maintenance Monitoring
Test Trains	Communication
Track Geometry	Technical Liaison w/SP Headquarters

Now we would like to show you examples of the type of outputs that are generated in the test. Obivously it is impossible to show all the results so single examples have been selected. Of particular interest are some of the data from the earth pressure cells in the control zone versus those in Site 4. (Figures 1 & 2).

We have used a variety of loading procedures in our test programs. In our earliest tests, we took data on entire trains. The outputs were interesting, but the problem of data reduction was enormous, so we simplified this approach by using dual locomotives; in this case two U33C locomotives weighing 410,000 lbs. over six axles. This worked very well but caused problems when there was a shortage of motive power. A third approach was to use a 270,000 lbs. (over four axles) switch engine from the Caldwell Yard to service as a more constant source of loading.

Figure 1 shows the earth pressure response across the section of the tie and rail in Site 4 under two different loading conditions. We find the average of the four most recent tests are entirely consistent with data collected in May, 1979, indicating there has been virtually no change with time.

You can also see the influence of the heavier loads applied by the U33C.

The loads have a symmetrical distribution around the center line of the rail-tie intercept.

The control zone is showing us an entirely different profile. (Figure 2).

The October, 1978, run shows a similar loading curve to the curve for Site 4, but offset slightly towards the center. The most surprising finding is that in the four recent tests the highest pressures are located directly under the center of the tie. We feel that this is an indication of the structure going into a center blind condition caused by downward movement of ballast under the rail.

From the extensive soil mositure instrumentation and from soil and ballast samples taken from the site since testing began in late 1978, we have seen the following results. The data (Figure 3) shows that following installation in June of 1978, which was during an extended drought in this part of Texas, the subgrade began to gain moisture until it reached an equilibrium moisture level. Tests conducted recently show little change occurring—with a ± drift of 2-3% being the norm.

The cement stabilized section shows a very low percentage of fines and these probably have come from crusher dust and from wind deposited soils. (Figure 4)

The needled fabrics in Site 1, 2 and 4 also show a very low level of fine soil particles.

The heat bonded fabric in Site 3 shows a greater level of fines than the other three fabrics, but clearly the control zone has the worst contamination.

In December, 1979, we excavated the ballast in all fabric test sections and in the control zone to evaluate the level of contamination, the condition of the fabric and to generate subgrade profiles which would help us understand the mechanisms of fabric deformation and ballast/fabric interaction.

The control zone showed three distinct strata. One upper level of clean ballast down to about 4 inches below bottom of tie, an intermingled zone where clay and ballast have had relative motion to each other with clay moving upward and ballast downward. This zone is approximately 8 inches thick. Then the third zone, the subgrade.

Part of the excavation work in December was to look at permanent deformations.

The section where heat bonded fabric (Site 3) was installed under the ballast in where we had, what I call, a failure by December, 1979. (Figure 5) The surface was 3.2 inches below

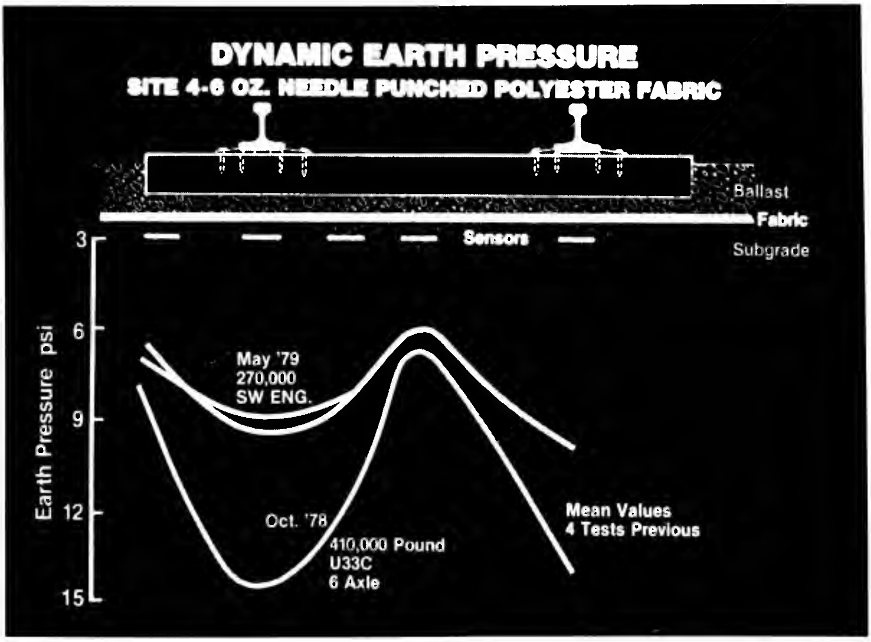


Figure 1

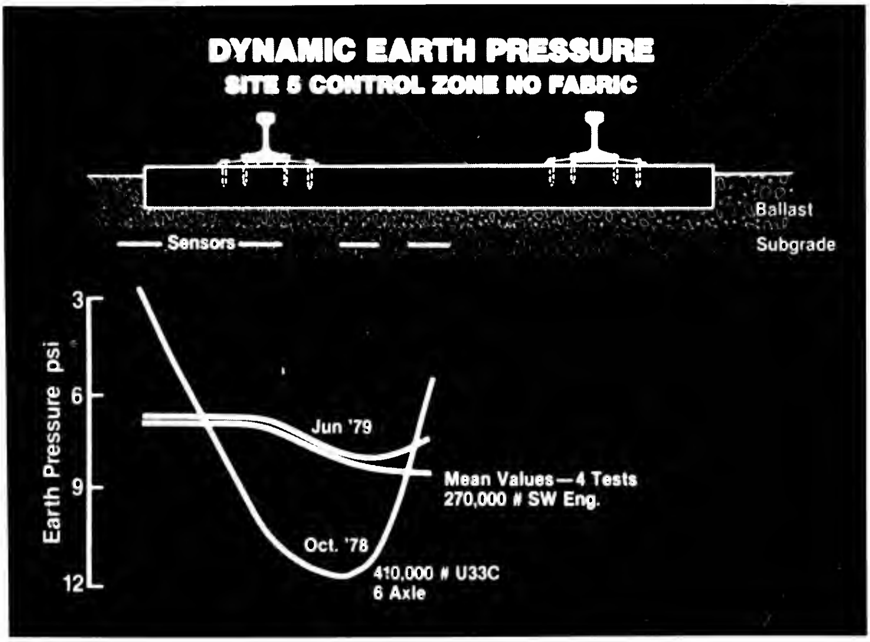


Figure 2

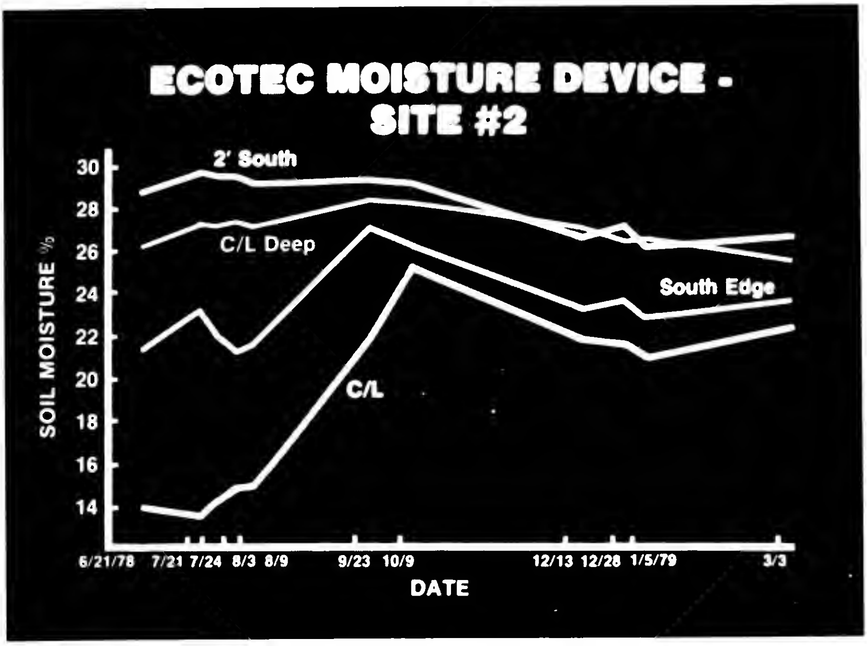


Figure 3

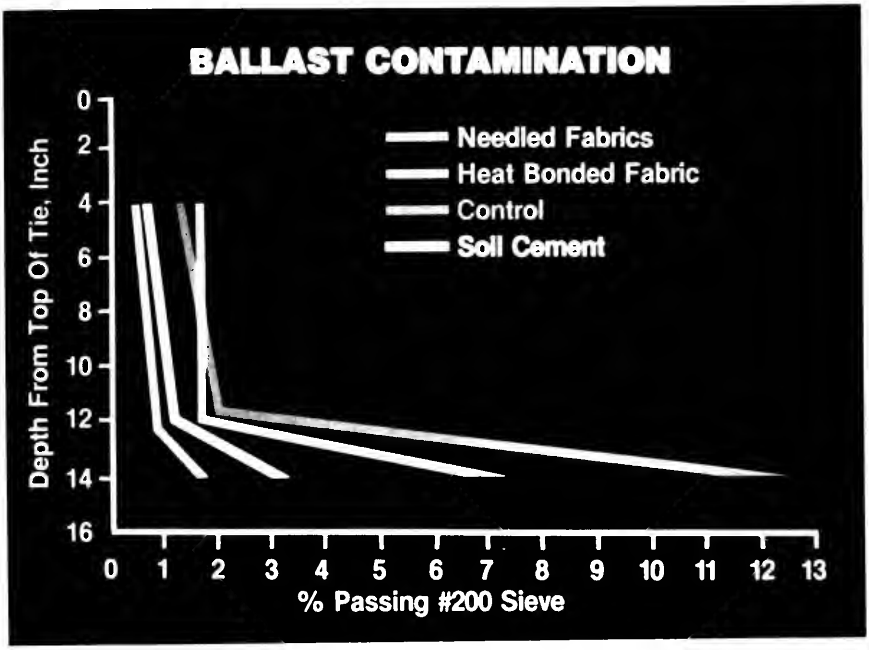


Figure 4

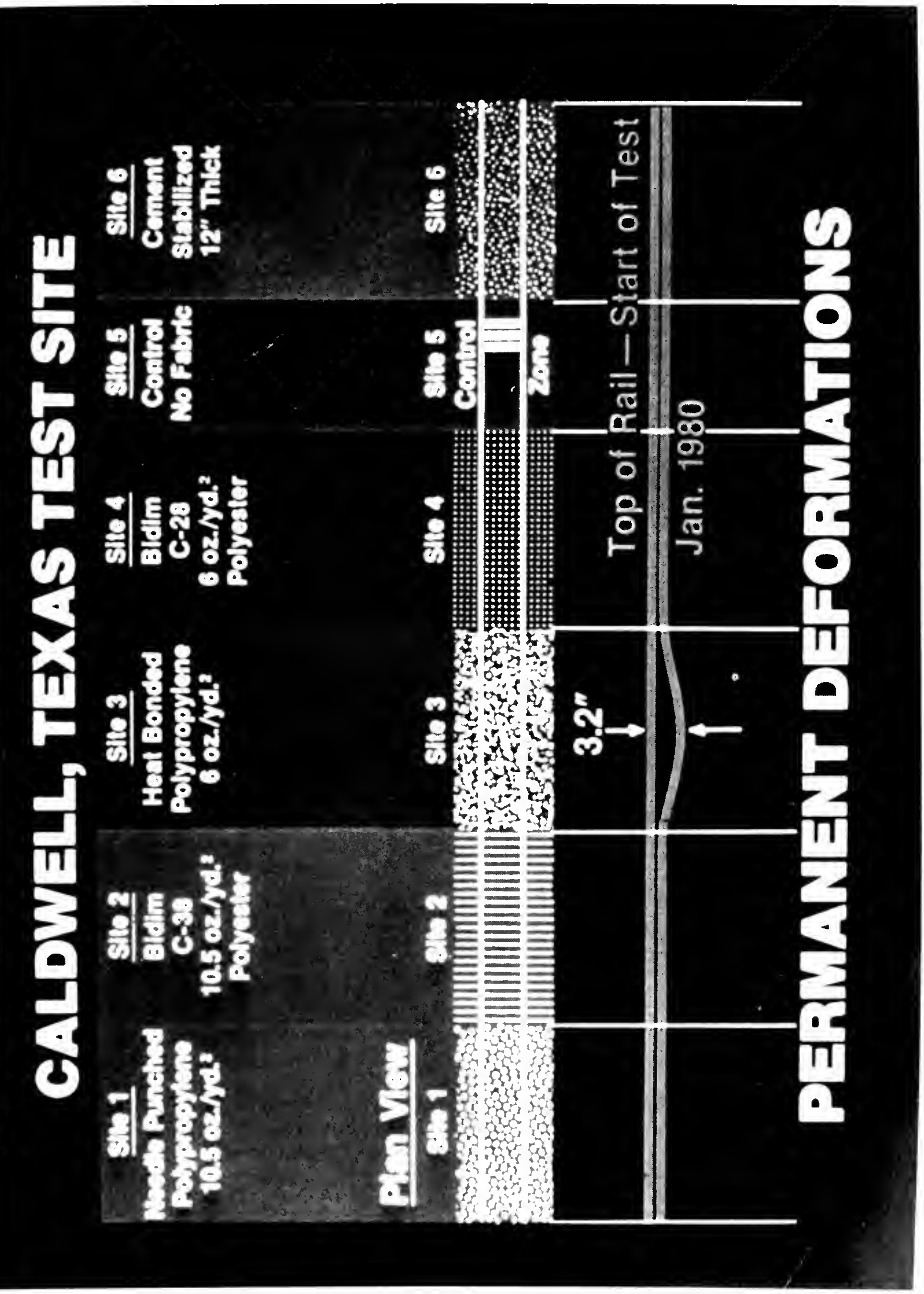

CALDWELL, TEXAS TEST SITE
Site 1
Needle Punched
Polypropylene
10.5 oz./yd.²
Site 2
Bidim
C-38
10.5 oz./yd.²
Polyester
Site 3
Heat Bonded
Polypropylene
6 oz./yd.²
Site 4
Bidim
C-28
6 oz./yd.²
Polyester
Site 5
Control
No Fabric
Site 6
Cement
Stabilized
12" Thick
Plan View
Site 1
Site 2
Site 3
Site 4
Site 5
Control
Zone
Site 6
3.2"
Top of Rail—Start of Test
Jan. 1980
PERMANENT DEFORMATIONS

Figure 5

grade, alignment was about $\frac{3}{4}$ inches outward on side away from main track, and the shoulders were near the top of rail. It was assumed that a bearing type of failure (rotation) had occurred.

After excavation, we found that the failure was not due to a bearing failure as expected but was caused by a plastic flow less than one inch of depth below the fabric; i.e., the soil was over saturated just below the fabric and was being squeezed outward similar to squeezing toothpaste out of a tube. The beat bonded fabric appears to be trapping water. The higher moisture content of the less than one inch below the fabric occurred in all fabrics but not to the same degree as in the heat bonded fabric.

In the control section, the ballast had penetrated into the subgrade about 4 inches, but the mud had replaced the ballast with only about one inch deformation at this time.

The deformations observed in each fabric were surveyed and the profiles plotted.

In addition to the survey data, we made plaster casts of each of the defomations observed in the fabric sites. These three dimensional casts indicate that some tensioning of the fabric occurs and also that the ability of the fabric to conform to the shape of the ballast generates an effective ballast restraint in certain of the fabrics. A combination of fabric elongation and surface frictional characteristics is beginning to emerge as an important fabric property in controlling ballast structural integrity.

From this test program, we are learning the fundamental mechanisms of fabric behavior in track structures. These findings are sometimes much different than we would have anticipated two years ago. As in many test programs, more new questions are raised as others are answered, but we are now reaching the point where we can demonstrate that some fabrics are very effective in stabilizing the track structure. We also know that all fabrics do not perform the same way and that fabric structure is a key element in their performance in this application. We also see that heavier fabrics do outperform light weight fabrics and we are now using heavier fabrics as our standard practice.

The comparison with the cement stabilized shows that some fabrics approach the performance levels of the conventional, but very costly cement stabilization method. We are now in the process of defining their relative cost/performance values.

Certainly these tests are vital to the development of rational design methodology so that fabrics can be effectively utilized by the railroad industry.

The end result or bottom line is that we can use fabrics to same valuable MOW dollars.

An example of geotextiles effectiveness in stabilization, other than the Caldwell tests, is shown in Figure 6. In our Eugene, Oregon Hump Yard, three very busy track leads with switches and crossovers feed traffic to the hump lead. Very little time is available for maintenance on these tracks, but a small derailment occurred on one of the tracks which gave the Roadmaster an opportunity to rehabilitate two of the tracks. Fabric was installed under one of the rehabilitated tracks and no stabilization was performed under the other. I inspected the two tracks about 13 months after the rehabilitation, and the track without fabric had about one inch space around each cross tie filled with water and ballast which was completely fouled. The ballast under the track with fabric gave the appearance of being completely clean.

I have seen some reports that expressed doubt that geotextile fabrics will be of much benefit in stabilization of railroad roadbeds based on laboratory results, but actual results such as this one proved otherwise.

Figure 6

60-MILE TRACK REHABILITATION USING GEOTEXTILES ON SOUTHERN PACIFIC NEAR FLATONIA, TEXAS

by H. B. Berkshire*

This morning I am going to talk to you about our coal route, The coal originates in Axial, Colorado, and its destination is Fannin, Texas, as shown in Figure 1.

The coal trains are delivered to us at Caldwell, Texas, and then proceed in a southerly direction through Flatonia on to Victoria and then across to the power plant at Fannin —covering a distance of 180 miles. The route from Caldwell to Flatonia is one of our major routes and no special work was necessary on this portion to handle the traffic. From Flatonia to Victoria and across to Fannin, a distance of 64 miles, is a different story.

We were looking at a typical low tonnage branch line that many of you are quite familiar with. Besides the normal tie and rail considerations, this line was plagued with many subgrade problems. Even under the light tonnage, we were experiencing heavy warp and line variations. In considering subgrade weakness, we were certain of one thing—the drainage on the branch was very poor and must be corrected. Our first task was to put together a grading gang of company equipment and operators to handle this task.

We used Company forces and equipment because we felt that the writing of specifications, and asking for contractor bids, would require more engineering effort and time than we had available. Hence, our approach was to assign an engineer to be grading supervisor. Thus, each area was surveyed and a drainage plan developed as the work progressed. I am sure you realize that to have a contractor to reestablish drainage over 64 miles of track would be a very expensive method. We now believe, based on our experience with this project, that use of Company forces and equipment is the only way to go when many miles of right-of-way are involved.

Our next consideration in solving the subgrade problem was to determine those spots that were extremely muddy but did not indicate a weak subgrade. The investigation was relatively simple, in that visual observations pinpointed 90% of the problem areas. These areas were examined by our Soils Engineer to determine the cause of water retention. In most cases, the soil in the subgrade was of a nature that water was held in pockets because of impervious clay or mud dams.

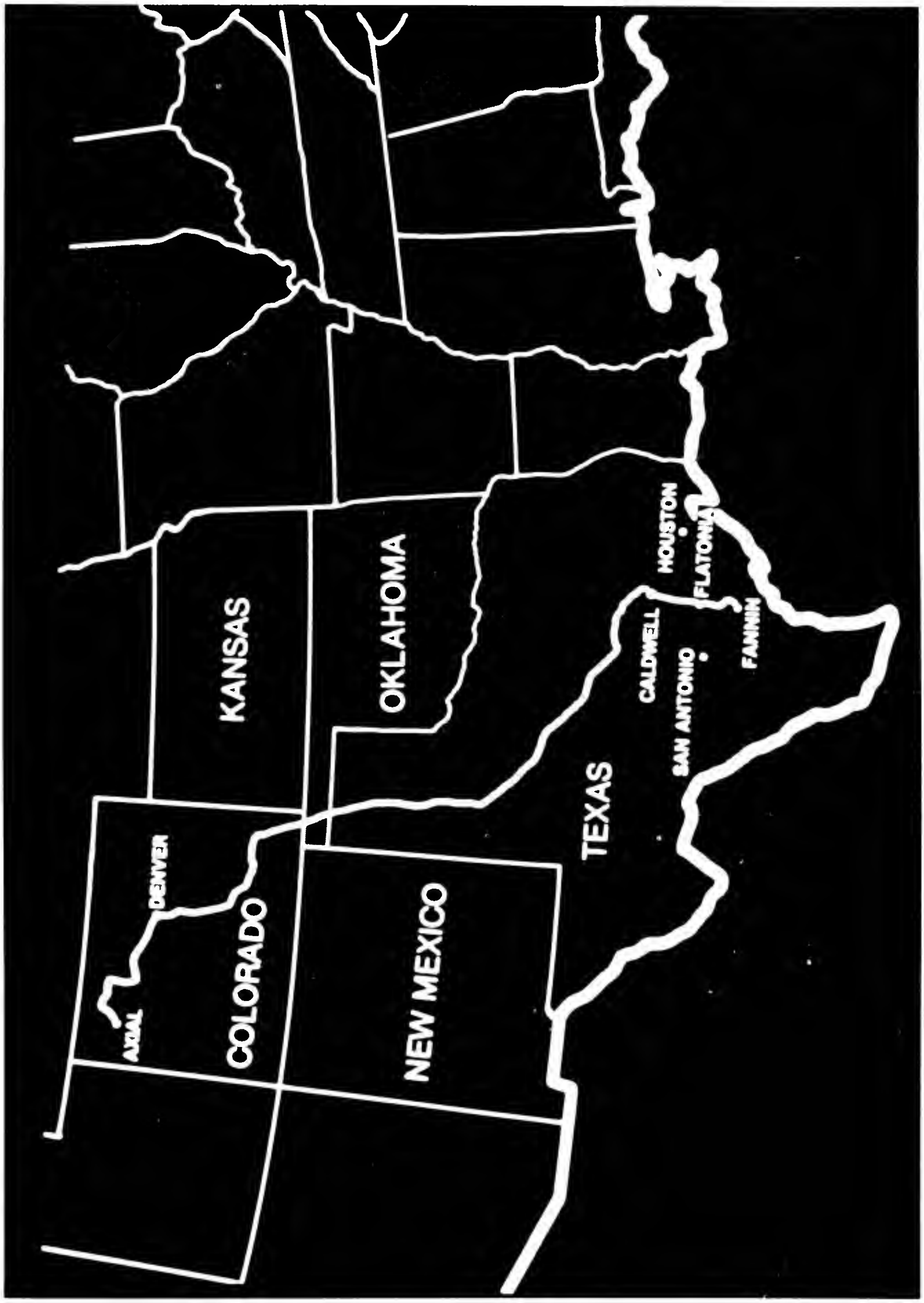
Our tests at Caldwell and over four years of experience with fabrics indicated that fabrics would solve almost all of this type of subgrade problem.

It was determined to use a track lowering plow (Figure 2) to remove the fouled ballast and to lower the subgrade. This lowering of subgrade removed water pockets, leaving a uniform smooth subgrade and, at the same time, allowing our finished grades to meet the existing grades at bridges and road corssings.

Before the plowing was to be done, it was advisable to cut the shoulders down, using combination of dozer and spreader. This made it easier to plow the excessive materials to the outside, which provided a wider shoulder to hold ballast.

We lowered the track an average of 6 inches. Ballast Regulators were used after the tie installation and before ballast dumping to slope the sub-base material away from the tie ends.

*Assistant Vice President, Maintenance of Way and Engineering, Southern Pacific Transportation Co.


KANSAS
OKLAHOMA
TEXAS
COLORADO
NEW MEXICO
DENVER
AXIAL
HOUSTON
FLATONIA
CALDWELL
SAN ANTONIO
FANNIN

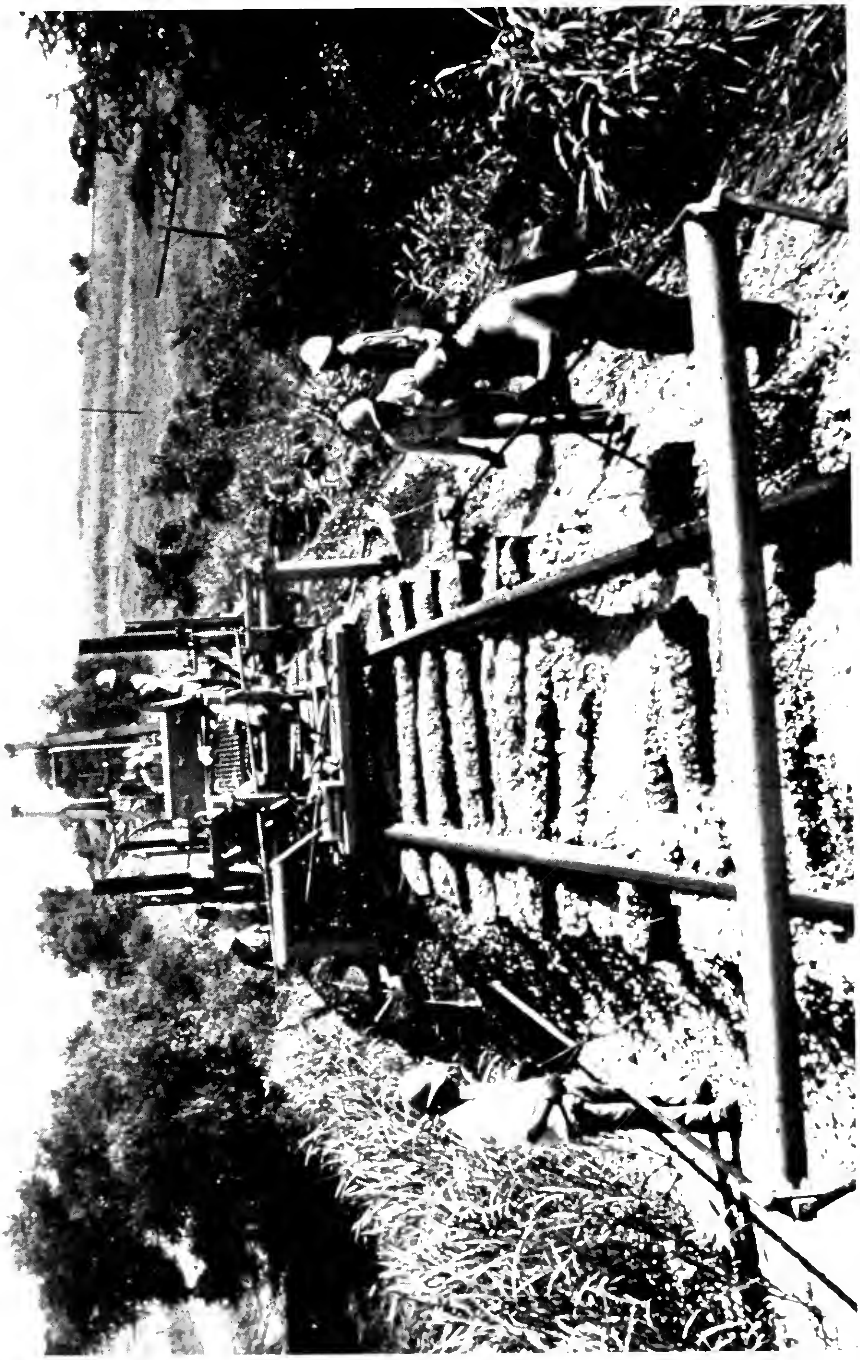


Figure 2

Figure 3

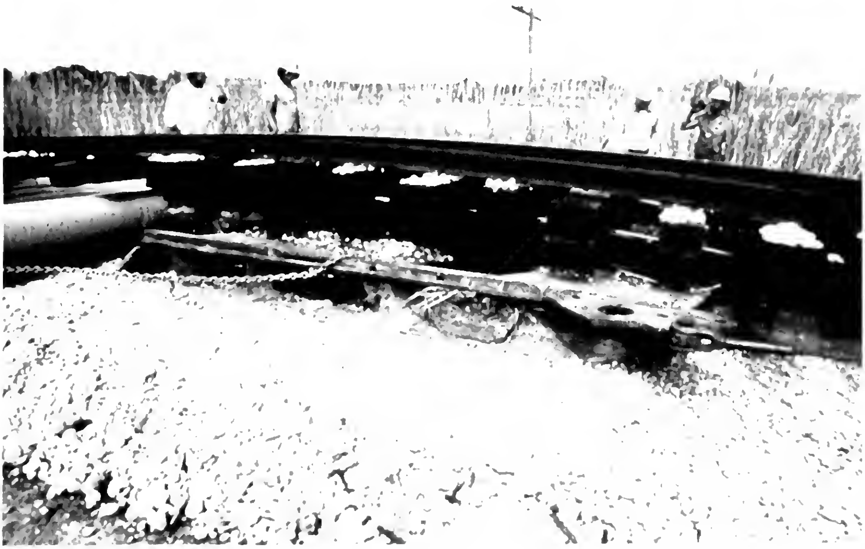


Figure 4

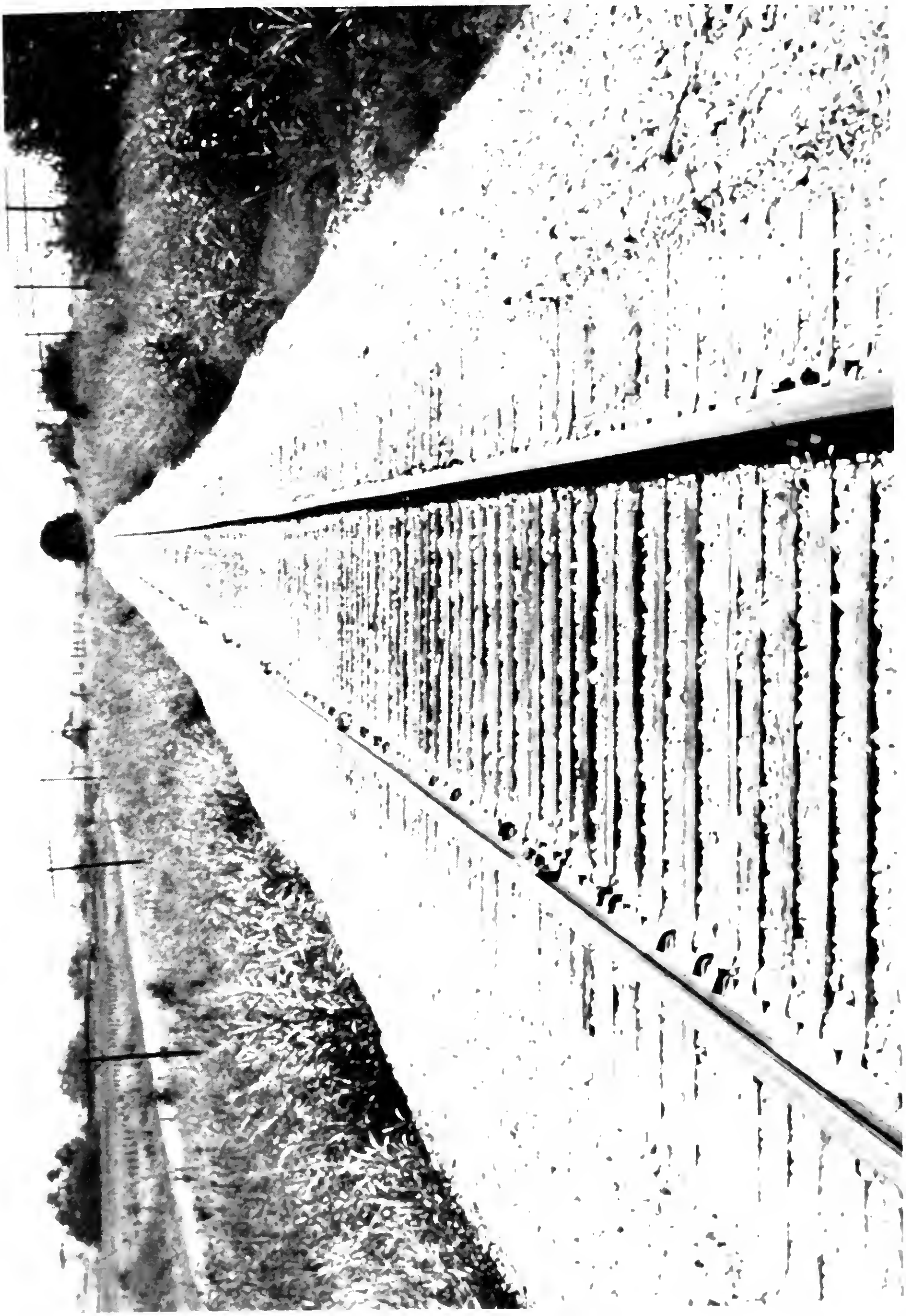


Figure 5

Ties were removed in conjunction with the track lowering plow. In some areas, it was necessary to run a tie again ahead of the plow operation. Regular tie renewal equipment was used in either operation. It was necessary to respace the ties as we did not install anchors ahead of the plow operation. Anchors were not installed because the rail is to be renewed after ballasting and surfacing.

The fabric was installed under the rail by pulling a fabric roll behind the shed. The shed restored a smooth subgrade after installation of new ties and spacing work was performed (Figures 3 and 4).

We were able to pull the sled and fabric roll with a track crane. Fabric installed by this method was at a rate of about 200 feet per minute which includes the time of putting on a new roll.

The ballast was distributed to full height of rail and then the track was raised by power jacks. Because of the light rail, it took two raises to obtain the full 6 inches of ballast under the ties. We had determined that this depth was required before tamping work started in order to prevent punching holes in the fabric. The first raise was accomplished by again pulling the led with the track crame.

The finished product is shown in Figure 5. We made the second ballast dump slightly above the top of tie and used various types of tampers to provide final line and surface.

Old wooden structures were replaced by either corrugated metal pipe or by prestressed concrete deck structures.

The laying of Continuous Welded Rail will be completed during our 1980 production season. We feel that the rehabilitation will stand the stress of unit coal trains and the use of fabric will reduce maintenance expense. It is our intention to monitor the line closely for any development of mud spots and for subgrade strength.

The historical information to be gained from projects such as this will provide much needed data for future work.

New CF & I Rail Mill

by J.J. Burke*

Mr. President, ladies and genelemen, on behalf of CF&I Steel Corporation, I wish to thank you for your invitation to speak before this 1980 A.R.E.A. Technical Conference. I certainly consider it both an honor and a distinct privilege to be able to bring you up to date on our rail mill expansion and moderinization.

Speaking to railroad engineers about rails, to use an old cliche, is like the minister preaching to the choir, and it is even more awkward when speaking to experts in the field such as represented by this group today. I do, however, welcome the opportunity since we at CF&I want you to know how we feel about the A.R.E.A., the railroads, the railway industry in general, and the role we as suppliers can fill in the resurgence of the railroads during the 1980's.

CF&I Steel Corporation, will celebrate its 100th anniversary in the production of rails on April 12, 1982. From the first production of 30-foot T-rail sections to assist the Denver & Rio Grande Railway in building a narrow gauge route, CF&I responded throughout the next 98 years to the demands of western railroads producing over 20 million tons of rail. It played a significant role in the development of the 39-foot rails; the 136 lb. and 119 lb. rail sections, and the high silicon rail. The only manufacturer of rails west of the Mississippi River, CF&I was the first to roll rails from steel produced by the new basic oxygen furnaces. Now as CF&I approaches its 100th anniversary in rail production, the pioneer spirit is still evident in the production of Rails up to 25 meters, high performance CROMORAILS and the application of new technology to meet growing energy related demands. Whether participating in tests at the Transportation Test Center in Pueblo, or laying track within the Royal George of Colorado, historically American railroads and CF&I rails have been synonymous with the vital economic development of the western United States.

The rail mill expansion program, representing the largest single capital expenditure in the history of our Pueblo Plant, was prompted by increasing demands for rail from major consumers, basically the western roads and the rapid growth of the coal industry with the attendant rise in unit coal train and rail wear.

Our confidence in the railroads coupled with the outlook for the future was the moving factor in our decision to expand and modernize our rail mill. I'm delighted to inform you that this project was completed during 1979 after two and one-half years of construction and an expenditure of over $85 million. etc.

During the construction phase, we were imposing a new facility on an old facility which hampered our productivity and our capability to respond to railroad needs. It was a very trying time for us and I'm sure for the railroads. Often times, we felt like a doctor trying to perform an arterial by-pass operation to the heart while the patient was running at 30 mph.

This now fortunately represents past history!

This massive project has increased CF&I's rail making capacity by 50% to 550,000 tons yearly—or enough to span the United States coast to coast annual with new rails.

Aside from the market demand and potential, many other factors were also carefully considered in the decision process for expansion and modernization. Principal among the

*Manager, Railroad Products, CF&I Steel Corporation additional factors were: 1) Total rail production capacities within the U.S.; 2) the high percentage, approximately one-third, of all our plant production.

Our objectives were threefold: First, to modernize our existing facilities and to incorporate the ability to produce long length rails.

The **Second** objective was to increase thruput or capacity for the manufacturer of all standard T-rails, sections from 90# RA through 140# RE to insure more than adequate domestic supply for even the most ambitious future consumption plans. **Last but by no means least, our third objective** was to continue to improve product quality and performance. This has been achieved as you will see later by the installation of various new equipment. The most important piece that will immediately help your industry is the roller straightener for improvement to weld plant performancance. During the last three years, we have developed a new high strength alloy rail steel which I will discuss later

Since rail production is extremely important to CF&I Steel, accounting as it does for over one-third of our shipments over the past five yars—two major restraints were placed on all planning for the aforementioned objectives. There were to augment and improve—the basic rail manufacturing process CF&I Steel Corporation has developed ove r the years, and which produces a high quality rail, and to design the facilities in such a manner that they could be installed with a minimum of interference with existing operations.

CF&I's fully integrated rail steelmaking actually begins with the mining of coal, iron ore, and limestone to be refined into coke in coke batteries, iron in blast furnaces, and actual steel in electric and basic oxygen furnaces.

An electric furnace melts and refines a heat (batch) of rail steel. Three carbon electrodes are removed prior to tapping (or pouring) of the heat.

After tapping, the lade is transferred to the teeming isle for teeming into ingots.

Steel is received by cast iron ingot molds. Average weight of these ingots is five tons. Normally we get 22 ingots per heat. This will yield approximately 100—39′ rails or 50—long length rails per heat. Our plant's current ingot making capacity approaces 1.9 million tons annually.

We have added eight new soaking pits to augment the orgiinal 24 already in place. These new soaking pits were installed in 1978. They operate on coke oven gas, natural gas, and #6 fuel oil with future capabilities of mixed blast furnace and coke gas.

The pits are circular style with eight burners. Combustion air is preheated to 1,000° F through a combination tile and metallic recuperator which yields an approximate 35% fuel savings (over past practices).

These eight new pits will continue to insure that proper heating cycles can be followed —even with the increased rolling rates. Proper heating and soaking of ingots is necessary to maintain the uniform and consistent section tolerance required on all rails.

The ingot is then drawn from the pit to be put on the ingot buggy.

The new ingot buggy is cable driven and is capable of running 1,200 feet per minute. It delivers ingots from individual soaking pits to the ingot transfer crane. The buggy has a scale hopper which catches the scale that falls off the ingot and carries it to the ingot transfer where the bottom hopper opens and dumps the scale.

The new ingot transfer is a stiff legged crane with "C hooks" to carry ingots from the ingot buggy over to the new 45-inch blooming mill approach tables.

Ingots are always positioned on the approach tables so they roll bottom first. This improves interior quality.

The 45-inch mill and tables were installed in 1978. This new 45-inch blooming mill is thought to be the world's fastest. Reversal time is less than one second and a maximum bloom runout speed is 1,200 feet per minute.

Driven by twin 4,000 horsepower electric drives, the mill is fully automated. It uses a stored program computer to set the opening between rolls, position the manipulator, measure the bloom length and control mill direction and speed.

Fast acting load cells in the mill housing automatically detect when the ingot or bloom leaves the mill to initiate mill reversals, roll setting, and manipulator positioning for the next pass.

Due to the automatic pass control schedule seen on this close up of the CRT tube, roll spacing for any given pass will be the same for all ingots bloomed.

The result is blooms delivered to the rest of the mill that are uniform and consistent. This consistency improves the quality of section on the finished rail because succeeding roll stands do not recieve too much or too little metal.

Construction and preliminary de-bugging of this mill was accomplished with a varying degree of interruption to mill operations. Naturally, we experienced some normal start-up delays as we put this mill on stream.

The new down up cut bloom shear is driven by twin 700 horsepower drives. It will cut the bloom into two pieces, as well as discard the top and bottom of the ingot. The shear has a cycle time of five seconds and a capacity of 225 square inches. The new crop hoist is a skip car which is under water; it collects top and bottom ingot discards and carries them outside where they are dumped into a pit of water. Crops are later loaded into railroad cars for return to steel production.

The new bloom transfer bed is to transfer rolled blooms from the 45-inch area into the 36-inch mill roll line. It is constructed of rope driven buggies which carry blooms from one roll line to the other.

To take further advantage of the precise rolling made possible by the computer, it is necessary for the rolls on all mills to be machined very carefully. To produce these rolls, we have installed the only roll turning lathe in the United States which has computer numerical control for turning and dressing rolls. It turns and dresses both blooming mill rools and rail shaping and finishing rolls. The lathe is fully operational.

A new, fully automated, computerized 36" mill (or breakdown mill) has replaced the former steam driven mill. The new 36" mill is driven by a single 4,000 horsepower motor through a pinion stand.

This mill makes five passes and starts the formation of the rail base. The last two passes were previously rolled in the roughing mill. This change in pass schedule speeds up the overall operation. Savings in rolling time minimized heat loss from the bloom, resulting in better section conformance through the balance of the mill.

The original roughing mill was steam driven and required use of an 80-ton flywheel. In the early part of the century, when the mill was electrified with an A.C. motor, the flywheel was left in as part of the drive train.

The drive on the No. 1 intermediate is very similar to the existing roughing mill. This mill also remains intact. However, rolling passes have been reduced from 4 to 2, thereby speeding up the rolling process and, again permitting better dimensional characteristics.

The new intermediate mill took two of the four intermediate passes from the existing intermediate.

The existing 26″ 2-Hi finishing mill which rolls the one finishing pass remains as is except for a new 2,000 horsepower drive motor.

The entire hot rolling sequence is accomplished with no intermediate heating of the bloom. To the energy conscious, this is an impressive contribution to conservation of fuel in that bloom reheating would require as much as 4 million BTU's per ton equivalent to 7,052 gallons of diesel fuel per track mile!!

To handle the designed number of ingots per hour capacity of the blooming mills, an additional hot saw was required. Since we roll various lengths of rail, this saw had to be moveable. The saw raides on a pair of tracks and will travel up to 85′ from the original fixed saw. The existing hot saw is basically unchanged and is also used to cut the finished product to length.

Immediately downstream from the existing saw is our new traveling gage stop. This enables us to cut rails to exact lengths. This gage stop can also move up to 85′ from the fixed saw.

The stamping machines, which were rebuilt, stamp the heat number indentification onto the rails. As you know, this identification is required by A.R.E.A. specifications.

The new rail "walking beam" cooling bed was built in 1978. It was absolutely necessary to design and install a complete new cooling bed capable of handling both conventional 39′ rails and longer lengths up to 82′ (25 meters).

This bed was completed and operational before demolition of the old rail cooling beds. This was accomplished by installing an automated walking beam type bed with adjustable stroke and lift off unloading devices which operate automatically to remove rails one at a time as they reach proper temperature on the cooling bed.

Rails enter this bed at approximately 1,800° F and are scheduled to leave the bed at 1,000°—725° F—within the prescribed A.R.E.A. parameters—for transporting to one of the stacking tables.

Long rails and their approximate 1½ ton weight per rail created a handling problem because they could no longer be man-handled. From this cooling bed through the completion of rail finishing, the rails are carefully lifted mechanically, conveyed and handled without sliding on the base to eliminate possible scratching and thus improve quality.

The rails are then transported from the new cooling bed where they are turned 90° head up, closely packed in groups of ten for magnet loading into the cooling boxes. Rails are charged into the cooling boxes between 725°—1,000° F. As previously mentioned, to begin their controlled cooling cycle.

A.R.E.A. specifications require rails to be slow cooled from 1,000° to 300°over a ten hour period in slow cooling boxes; this eliminates potential shatter cracking from entrapped hydrogen.

To slow cool long rail, the new boxes which you see, were required. These boxes can hold over 8,000 39′ rails or over 4,000 long rails.

Since CF&I started producing rails, almost 100 years ago, we and all other domestic rail producers gag straightened rails. This method was more an art than a science. It was decided to take as much of the human element as possible out of rail straightening and a two-plane roller straightener was purchased.

Our new rail straightener was designed and built by Alfred Wirth & Co. The Wirth Co. is the only company in the world to manufacturer a roller straightener specifically designed to straighten rails.

Since a rail is always straightend head up, a rail-turning device was necessary. Occasionally a rail will turn over onto its side. The turnover device will turn the rail 90° to a head-up position and push it into the horizontal and vertical rolls. The rail then passes through the straightener at 150 FPM to the inspection station downstream.

While several similar rail roller straighteners are operational in other parts of the world, CF&I has the only such equipment in the U.S.

At the outset of our planning, it was decided to purchase a straightener powerful enough to straighten the high performance alloy rails. Our straightener is the largest ever built. It can exert up to 200 tons of pressure on the rail to straighten it.

From a quality point of view, railroads that are receiving our roller straightened rail have made numerous comments about the increased productivity rates on both 39′ and long rail in their weld plants. This higher productivity is directly traceable to uniform and consistent rail end straightness as compared to conventional gag straightened rail.

No matter how skilled the gag straightener, this system left rails with a series of small kinks—while the roller straightener provides a much smoother overall configuration. Rail welding plant response thus far has been excellent when comparing roller straightened rails to those straightened by the original hand gagging process.

The rail is then inspected as it exits the straightener.

After inspection, rails are routed to the ending milling operation, cold saw operation, or the hydraulic gag from this distribution table, depending on finishing requirements.

Also built was this two plane hydraulic straightener. On occassion, the roller straightener will not completely straighten a short hook on the end of a rail. This is due to the distance between the centers of the working rolls, or a rail will exit the straightener with unusual sidesweep. This hydraulic straightener takes these "out of tolerance" rails and restraightens them to specification.

After they have been straightened, it is necessary to finish rail ends for the intended application. The majority of rails produced are for CWR (continously welded rail), requiring only that ends of the rails be milled square. At each of two finishing lines, rails are milled with carbide cutters to specification. These milling machines are also equipped with three spindle drilling machines which can drill holes in rails—simultaneously with milling—for customers who intend to bolt the rails together.

The drills are also carbide and were developed at our plant to become some of the first used in the rail making industry. Carbide tools permit cleaner cutting, as well as working materials of high hardness, such as alloy rail. Rails can now be milled and drilled in approximately one minute. This productivity rate is necessary because the straightener will straighten three 39′ rails per minute, or two long rails per minute.

Rails can then be end hardened after drilling.

A very small percentage of the overall rail production requires some kind of rework due to metallurgical or mechanical defects.

This line has two saws with 40-foot scrap beds where a rail can be cut to proper length and the defect removed. Rails with defects can be salvaged down to 20′.

Our modernized rail finishing handling equipment is designed so that, under normal operating conditions, rails can be loaded directly from the finishing beds into rail cars for shipment to our customers. One new overhead crane and one rebuilt shipping crane, both being able to handle rails from 25′ to 85′, load the railroad cars using magnets.

Before I continue on the other areas, I would like to take a few minutes to discuss long rails. Welding plants that have welded our long rail have experienced dramatically high productivity rates due to making fewer welds per 1,440′ string. Fewer welds are both economically advantageous and desirable from a track maintenance point of view since welds are a potential source of problems in track. We have now had the ability to produce long rails for two years. It should be remembered that it was *your industry cry* for many years to the rail producers of the country—"*We want longer rails*"—yet, today we have not fully utilized our capability to manufacturer long rails. We find it disappointing to think we at CF&I have made a sizable capital expenditure to produce a product that is not being fully utilized. I hope each of you after this meeting is over will look at your welding facilities to see if long rails can be utilized. I am sure with some imagination on both our parts we can find a way to utilize long rail—the "*Right track*" to economy.

The second area of CF&I contribution to track improvement has been in the area of CROMORAIL. (High performance alloy rail for critical and high wear application.)

During the past decade, major U.S. railroads indicate having experienced problems with standard carbon steel rail. This is due to increased traffic density, heavier wheel loadings and the operation of long haul, high capacity unit trains.

As previously mentioned, when CF&I's rail mill modernization and expansion program was being planned, it was decided at that time that new equipment should be designed to accommodate high strength materials.

Therefore, the roller straightener, enders, and drills were designed to handle rails with yield strengths up to 200,000 psi. These levels should prove adequate for current alloy compositions—and even higher strength levels in the future for any other high performance rails.

It was recognized from the inception of the development of alloy rail that several types of alloyed and heat treated rails were commercially available. Field experience with these rails confirmed predictions that higher strength steels could, in fact, reduce problems of gauge face wear, head flow, corrugation, shelling, and fatigue failure.

However, from information available, alloyed grades failed to achieve the strength and hardness of the best performing heat treated rails.

Available alloy rails were consequently regarded as deficient on the basis of cost versus performance analysis. Heat treated rail appeared to suffer from occasional limited availability, and costly energy consumption. Prospects for added capacity to meet rising demand were considered poor due to constraints imposed by availability of capital, energy or environmental aspects.

In view of these factors, the prime objective of CF&I's alloy research program was defined to be the development of an improved rail steel—with strength, hardness, toughness and ductility equal to or better than heat treated grades.

A second objective was to insure that the new alloy rail would be weldable by established or slightly modified welding techniques.

Chemistry changes in the CROMORAIL steel solved this problem.

Some of our customers have found that welding CROMORAILS at their centralized welding facilities can be accomplished in the same time as that required for standard carbon steel rails. It will also be of interest to you to know that Orgotherm and U.S. Thermit have developed a standard field welding kit and practice suitable for CROMORAIL.

No premature failures due to welding of alloy rails have been observed, despite the severe service conditions. Also, the weld zone of CROMORAIL is not as soft as in heat treated high performance rails.

CROMORAILS are in service around the world and throughout the United States and in all cases the alloy rail is performing very well indeed.

I will not attempt a quantitative comparison of the various rail types not in test or service. This is because field tests are continuing and the final outcome of the comparisons is not known at this time. CF&I strongly feels that the alloy approach is valid and extremely important for track improvement. Some track has accumulated hundreds of millions of gross ton miles with no complications. So, to respond to the engineer who "has a critical need for an improved quality rail," we now have hot topped CROMORAIL—in quantity—with a Brinell hardness of 321 to 388, with less than 3% martensite in the weld zone and which can be welded conventionally.

Summary

Long rails up to 25 meters, roller straightened rails, new hgih performance CROMORAILS—that's history—where do we go from here—to continue to answer the needs of the railroad engineer and his unending quest for higher quality in the 1980's. Let's briefly discuss some of these areas:

We are continuing our efforts to meet as many aspects as possible of the new A.R.E.A. specifications for rail. I will briefly review the areas where we are currently meeting the new specification.

1. While not consistent with our manufacturing process, a commitment to produce limited quantities of rail in hot top ingots. (CROMORAIL)
2. The restricting of the carbon range to one point narrower (.67-.80 and .72-.82).
3. The lowering of the sulfur maximum from .050 to .040.
4. The widening of the silicon range from .10-.25 to .10-.35.
5. The raising of the manganese range on rails 121#/yard and heavier from .70-1.00 to .75-1.05.
6. The lowering of the phosphorus maximum from .040 to .035%.
7. The specification requiring that all ladle tests individually must meet the specified chemistry.
8. The reduction of the quantity of No. 2 rails permitted from 8% to 5%.
9. The new wording concerning removal of fins and burrs at the edge of bolt holes is now included.
10. The method and requirements for hooks on ends of rails have been modified to include a .023-inch maximum hook within 9 inches of the end of a rail, using a 36-inch straightedge.
11. The quanity of short rails permitted has been reduced from 11% to 9%.

CF&I top management wants to be the first major integrated steel producer in the U.S. to continuously cast all of its major products. This would conceptually include a rail bloom caster, sometime in the future. Once we are able to concast rail steel blooms this would further improve the quality of the rail interior. To date, no specific timetable has been determined. However, during the last decade, CF&I spent over $^1/_2$ billion dollars improving and expanding our plant and equipment so the idea of adding this equipment is certainly not without precedent.

Also, our Research & Development Dept. is involved in a continuing search for better fastening devices to improve on the conventional cut spike and rail anchor.

Again on the research level, we have been invited by a major western railroad to explore the potential of applied laser technology in developing a laser hardened alloy rail steel for the future.

Perhaps a more appropriate description of our comments today would have been to subtitle our presentation, "Rails for Tommorrow" or "Rails for the 80's" or **most importantly** "The Railroad Engineer and His Critical Need for an Improved Quality Rail."—For these are critical areas that we will continue to stress.

This concludes my update on CF&I's rail modernization and expansion program—with emphasis on what **we have done** and are **continuing to do** toward product improvement and quality.

New Facilities for Increased Traffic at Alliance

by M.O. Woxland*

Construction of the Alliance Locomotive and Car Repair Facility was one of the most challenging and interesting projects with which Burlington Northern's Engineering Division has been involved. In addition to being a very large and costly project, with total expenditures of approximately $46,000,000, much of the construction and design were done on a "Fast-Track" basis, about which some more will be said later.

In pre-coal days Alliance was a small terminal able to handle the small amount of work on locomotives and cars generated by the comparatively light traffic in the area. The roundhouse located there handled the assignment of 150 locomotive units. With the advent of coal traffic Alliance became an important junction for unit coal trains originating in the Powder River Basin in Wyoming and Montana, which were destined for power plants southerly through Denver to southern Colorado and Texas and easterly via Lincoln to points in Missouri, Iowa and Illinois. A great preponderance of the Powder River Basin coal traffic goes through Alliance. Projections of the increae in traffic resulting from the coal movement were made in early 1977 and they indicated that in 1980 32 trains with 3,360 cars would be passing through Alliance and that this would increase to 42 trains and 4,410 cars per day in 1983. Forecasts of the amount of maintenance work resulting from the increase in traffic indicated that in 1983 over 100 cars per day would be requiring maintenance work at Alliance and that over 540 locomotives would be assigned to the Alliance shops for maintenance. To handle this projected work decision was made to construct a new facility for which the design criteria proposed that the locomotive maintenance shops would be able to handle the assignment of 625 units, with up to 60 of the units being shopped daily, and that the car shop would be able to handle in excess of 120 cars per day.

A great deal of study was given to the type of overall arrangement of the facility that would be constructed at Alliance. The design did not spring completely from Burlington Northern experience, although the recently built mechanical facility designed by Burlington Northern at Northtown Yard was working out well. To extend the Northtown experience Burlingtron Northern officers visited a number of the recently constructed mechanical maintenance facilities on other railroads in the country. The final concept for the Alliance Facility as developed by Burlington Northern from the overall study was then validated by an industrial engineering overview study which analyzed the flow of labor, material, material handling equipment, locomotives and cars through the shop, and determined that the size and layout of the various component facilities provided for optimum productivity.

Although the design of the Northtown Yard facility was carried out by Burlington Northern design engineers and these engineers had the capability to prepare the plans for the Alliance Facility, it was considered that preparation of such plans would completely dominate the efforts of the Burlington Northern Building Section for up to eight months to the exclusion of other work, which of course could not be tolerated, so it was decided to retain a consultant to prepare the bulk of the plans. After considering a number of large and capable consulting firms it was decided to retain a local organization from the Twin Cities area, the Ellerbe Company, which firm is a large architectural and engineering design firm with international experience. They had the force available to progress the design to completion and further, they were located close at hand and would be convenient to work with as plans developed. We considered that even though Ellerbe was retained to do the design in complete fashion, they

*Director Construction Projects, Burlington Northern, Inc.

would have to be provided a substantial amount of assistance in explaining just what the facilities were intended to do. The Ellerbe part of the design included the main shop building. Other portions of the design plans for the facility were carried out by Burlington Northern, including the plans for the support facilities outside the main shop building, the site preparation and grading, and the track layout for the repair facility.

The concept for the facility as finally developed (figure 1) proposed construction of car repair, materials and locomotive maintenance facilities under one roof with four walls dividing the structure into five sections—car repair, materials, light/heavy locomotive repair, light locomotive repair and wheel truing. Support facilities such as bulk oil storage, fusee building, oil house, wastewater treatment plant, wheel storage and unloading platforms, along with a new yard office, were proposed at locations convenient to the main building. The underground tunnel provided for access to the shop for employees, and also provided space for utility conduits and pipes.

The wheel truing shop is 177 ft. long by 32 ft. wide. It houses a Heggenscheidt Model 106 under-floor wheel lathe with an automatic chip conveyor.

The Heggenscheidt machine can carry out the complete turning of all the wheels on a six-axle locomotive in five hours and 30 minutes, which is a substantial improvement over other wheel truing machines on Burlington Northern which take approximately nine to one half hours to do the same job.

Cuttings from the wheels are automatically removed via a conveyor to a chip car located on a track outside the north wall of the wheel truing building.

It is projected that three locomotive units will require having their wheels trued each day beginning in 1983.

The light repair shop is 324 ft. long by 182 ft. wide. There are five tracks within the building, each capable of holding four locomotive units. Two five-ton bridge cranes over each track. Three of the five tracks are equipped with single wheel assembly drop tables. These drop tables were installed to enable changing out single wheel assemblies without having to move the locomotive to the full truck drop tables located in the light/heavy locomotive repair building.

Work proposed to be carried out in the light repair shop is the kind of work that will generally require no more than eight and three quarter hours total locomotive stall time. Work on locomotives requiring in excess of this amount of time will be assigned to the light/heavy area.

Each track in the light repair shop has an upper and a lower work area. The lower area is equipped for work pertinent to the locomotive running equipment below the car body, such as traction motor brush exchange, brake equipment repair, piping repairs and the like. The upper area will carry out such work as oil changes, scheduled maintenance, work on fuel pumps, water pumps, bearings, electrical work, and the like.

Work areas are provided with ample room for storage of material needed in the maintenance operations without interfering with the movements of the work force or of fork lifts.

A total of 20 locomotives can be located in the light repair shop at one time.

Locomotives will not be run inside the shop. A startup service shelter has been located east of the light repair shop, at which the locomotives will be finally tuned as they come out of the facility. The locomotives will be moved to the startup shelter using a device that will power the locomotive traction motors or, when the shop is unoccupied, a switch engine will move the locomotive. The startup shelter is covered and will have infrared heating to protect employees during wintertime operation.

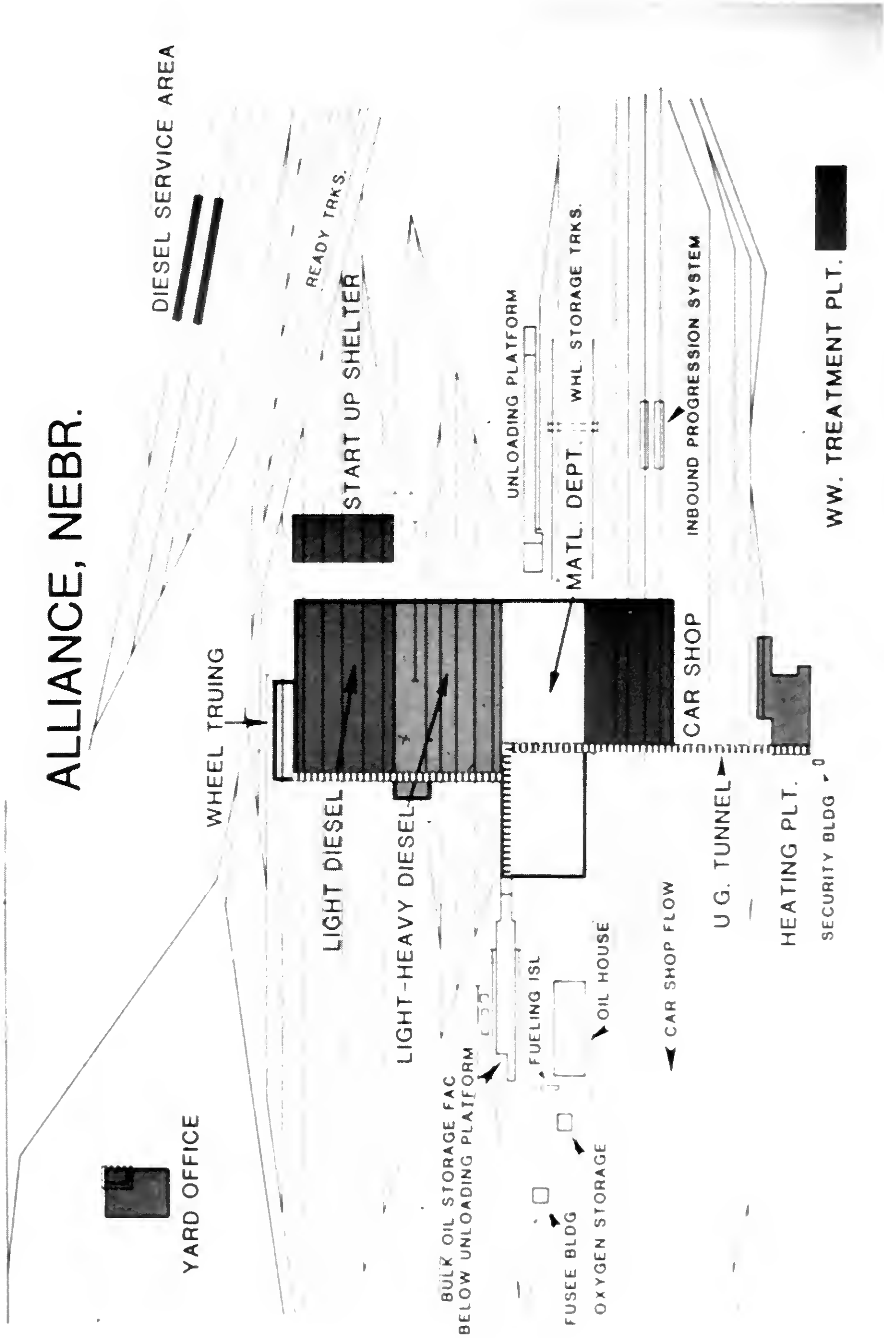

ALLIANCE, NEBR.
DIESEL SERVICE AREA
READY TRKS.
START UP SHELTER
UNLOADING PLATFORM
WHL. STORAGE TRKS.
MATL. DEPT.
INBOUND PROGRESSION SYSTEM
WW. TREATMENT PLT.
WHEEL TRUING
LIGHT DIESEL
LIGHT-HEAVY DIESEL
CAR SHOP
U.G. TUNNEL
HEATING PLT.
SECURITY BLDG
CAR SHOP FLOW
OIL HOUSE
FUELING ISL
BULK OIL STORAGE FAC
BELOW UNLOADING PLATFORM
YARD OFFICE
FUSEE BLDG
OXYGEN STORAGE

Figure 1

The light/heavy repair shop is 324 ft. long and 191 ft. wide. There are four through tracks in the shop for maintenance purposes, two of which are equipped with full truck drop tables that have auxiliary single-axle drop tables. Two other tracks are located in the shop, one a full truck release table track and the other a release table track for the single-axle assembly drop table system that complements operations in the light repair shop.

Four locomotives can be handled on each of the maintenance tracks. (Figure 2) However, because two of these tracks have full truck drop tables, only two locomotives can consistently be positioned for maintenance because two locomotive lengths are required when truck changes are accomplished. Accordingly, the light/heavy shop can handle twelve locomotive units for maintenance and two locomotives for truck changes simultaneously for a total capacity of 14 locomotive units.

The light/heavy shop also includes support areas such as a machine shop, tool room, pipe and tin shop, truck repair shop, truck painting booth, cleaning room and a traction motor electric shop.

The truck rebuild area is of sufficient size to allow rebuilding six trucks simultaneously. Truck cleaning will be accomplished in a cleaning room to the west of the truck repair shop. Two lye vats will be used to clean the frames and the gear cases. A two-year truck changeout cycle will be the policy at Alliance and accordingly every second day at least one locomotive will be directed to the shop for truck changeout.

The materials building is 500 ft. long by 150 ft. wide. There is a second floor over the easterly half of the building in which is located offices, meeting rooms and employees' lunch and locker rooms.

The car shop is 265 ft. long by 156 ft. wide. Four tracks run through the building. Two of these tracks are RMC one-spot progression repair tracks and two tracks are designated for light/heavy repairs. The shop design provides for repairing 16 cars per shift on each of the two onespot tracks on a three shift basis for a total of 96 cars per day. The light/heavy tracks provide capacity for a repair of six cars per track on a two shift basis for a total of 24 cars per day. In addition there are two tracks outside of the building designated for shifting loads and for shipping wrecks.

A 25-ton bridge crane covers the two light/heavy repair tracks and tiedowns are provided on each light/heavy track for straightening car frames. Jacking stations are provided on each one-spot repair track, which provide for quickly raising the car for retrucking and so on. The hydraulic jacks are built into the floor and are capable of lifting a car for removal of trucks in 25 to 30 seconds. The jacks are of the fixed type, two 75-ton jacks being positioned outside of the rail and one 150-ton jack between the rails.

Support facilities for the mechanical shop building include a wheel storage area located east of the materials building and a bulk oil storage area whose tanks are located below the west side unloading platform. In addition, a heating plant and a wastewater treatment plant were constructed.

Shown in Figure 3 is the new yard office to handle the train operations through Alliance which was constructed as part of the project.

Locomotives coming to the facility from operation in trains will be directed to the fueling area. This area and the ready track area will be under the control of a foreman, who will be stationed in a control tower constructed in an existing building adjacent to the fueling facility and who can view the locomotives coming to the fueling area and the ready track area. The foreman will ascertain if scheduled maintenance is due for any locomotives and as the units ae being serviced they will be inspected for mechanical condition. The tower foreman, after

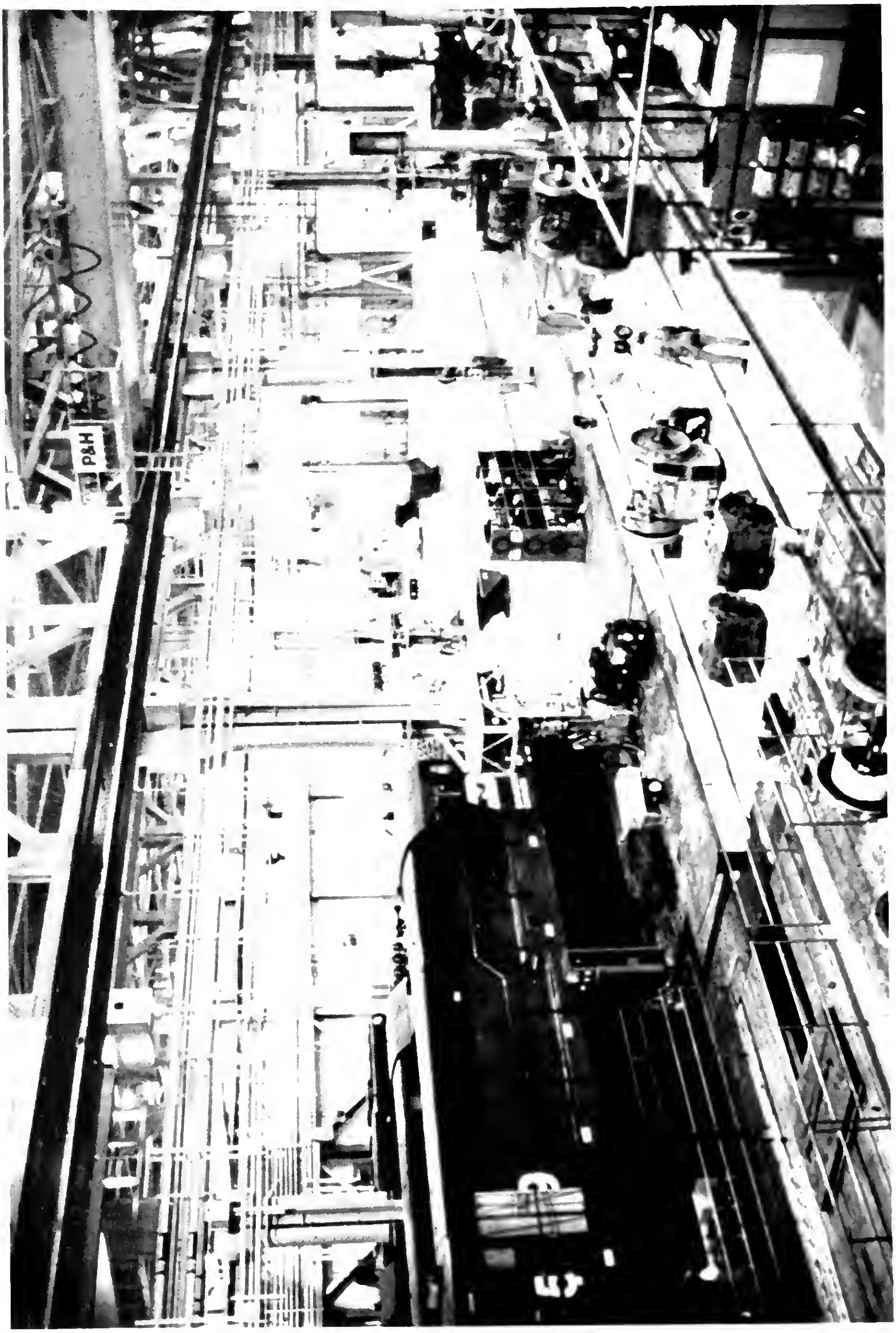


Figure 2

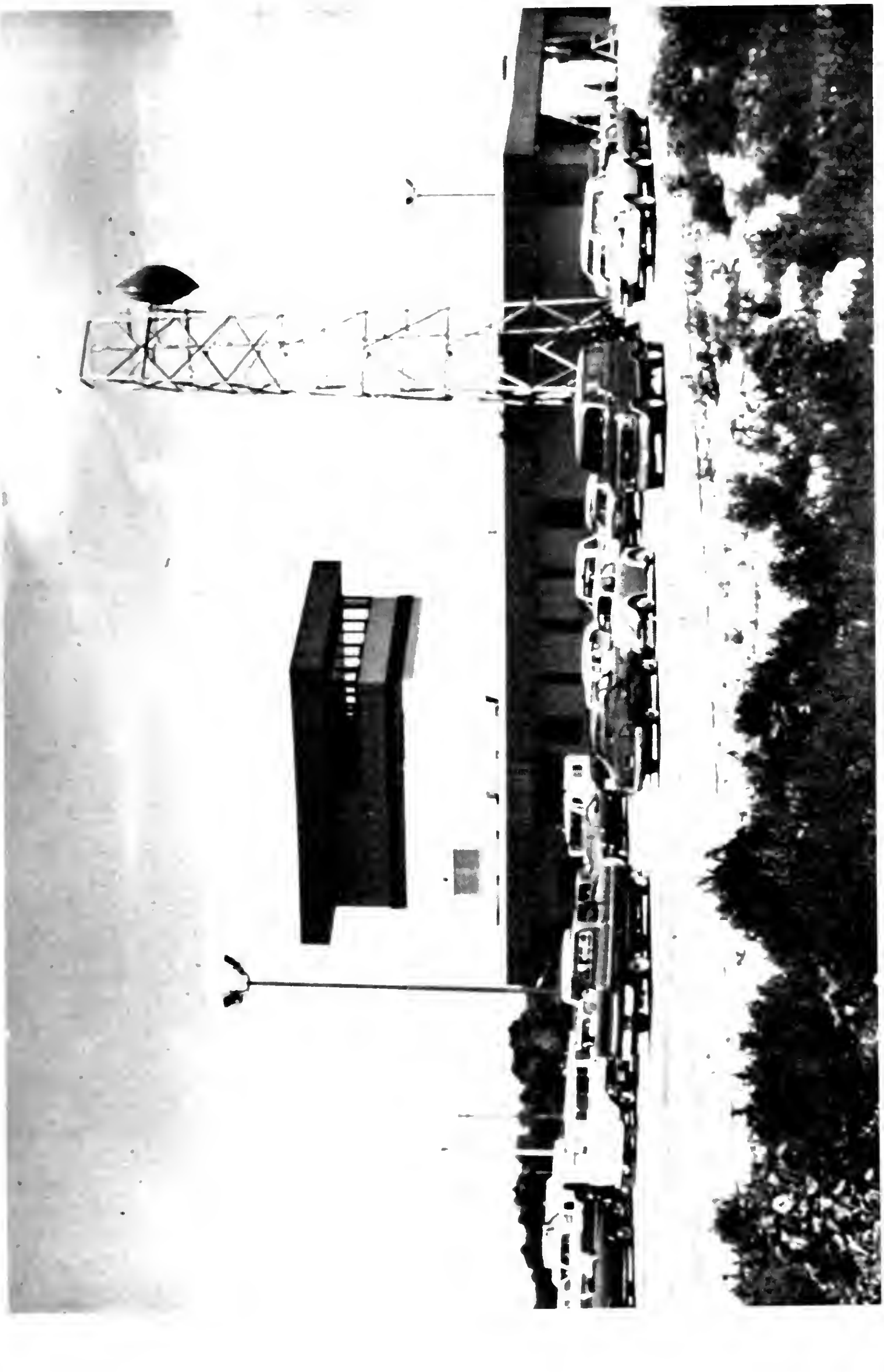


Figure 3

receiving the information concerning the locomotive needing maintenance, will advise the maintenance shop that he is directing the locomotive to the shop for repair and will advise what inspection has revealed as to the nature and the extent of the failure. This sets in motion shop procedures which will direct the inbound locomotive to the proper tracks and start the flow of materials needed for the repair work. Orders for material are placed with the Material Department via a telewriter in the foreman's office. The Material Department is endeavoring to schedule their work so that upon receipt of an order for locomotive material, no more than 30 minutes will ensue before materials are delivered at the proper location.

Bad order cars that are set out from trains will be placed on the inbound tracks at the east end of the car shop, from where they will move through the shop to the outbound tracks on the west end and then be switched to the proper locations in one of the yards at Alliance.

Design work was started in early 1977. As this work progressed Operating officers were viewing with some concern the tight situation that was developing with respect to locomotive maintenance and had determined that certain parts of the Alliance Facility, including the light/heavy locomotive repair shop, were needed no later than October, 1978 and that the entire facility was needed in July 1979. This time frame was so tight as to preclude a normal sequence of construction activity in which plans and specifications are fully developed before advertising for bids and starting construction and made necessary the "Fast-Tracking" of the project. We accordingly determined this by a step-by-step construction schedule, which allowed for early start of the work on the phases which required early completion and with other phases following in proper sequence.

Phase I work included those components needed by October, 1978 which were the light/heavy locomotive repair shop, the materials building and the wheel truing building, along with oil fired part of the heating plant.

The Phase II work included the light locomotive repair shop, the car repair shop and the coal fired part of the heating plant.

The "Fast-Track" procedure applied to the construction of the building foundation and to the Phase I and Phase II sequences. The schedule called for letting of contracts as soon as possible after completion of the plans for each of the component items of work, which in effect meant that several contractors might be working in the same general area at the same time.

Each one of the various activities, except for the tracklaying work which was done by company forces, was the subject of a separate construction contract. The first contract, that for grading of the building site, was started on time but before it was completed we had already lost two weeks of time. The building site grading called for levelling of the ground to the elevation of the building and also was intended to provide a soils layer on which spread footings for the building foundations could be established, and included excavation of between 7 ft. and 10ft. of soil below the proposed footing level as to remove a soils layer that was determined during the soils investigation as being unsatisfactory from a density standpoint along with replacement of the removed soil recompacted to a density of 95% Standard Proctor. The contractor found it extremely difficult to replace the excavated soil to the required density and finally decided to use another type of soil located in a borrow pit about a half mile distant, so as to obtain a material that could be readily compacted and which would provide the required soil bearing stratum. These circumstances resulted in the time lag of two weeks, which in turn set off a series of additional time problems involving the Phase I foundation contractor, who in addition to getting started late, ran into problems caused by the early onset of the most severe winter in the recorded weather history of the Alliance area which resulted in time lags that impacted on succeeding contractors for structural steel and the large general contract, all of which required our field engineering crew's most energetic efforts in carrying out the coordinating work between the various contractors, as well as an extreme amount of cooperation and display of resourcefulness from the contractors in order to make the Phase I completion date come out as scheduled. For several months during the Phase I work the labor force on the job numbered more than 500 men.

Work on the Phase II items and the Yard Office contract did not have the complications associated with Phase I. The foundation concrete, structural steel, and general contracts proceeded pretty much as planned and Phase II was completely slightly ahead of schedule.

Grading work which involved the preparation of foundation stratum for the maintenance shop building, as well as the construction of the subgrade for the trackage serving the facility amounted to 1,600,000 cubic yards. This was all carefully controlled as to type of material and compaction to satisfactory density and moisture content.

Phase I foundation concrete work started at the south west corner of the mechanical shop building and progressed through the utility tunnel and the building footings. Much of this work had to be done in unanticipated, severe, winter weather. The lye vat foundation, along with the section of the utility tunnel between the light/heavy building and the wheel truing building and the south wall of the wheel truing building were constructed.

As soon as room was available in the utility tunnel the Phase I General contractor started the work of installing piping and conduits.

Structural steel frame erection for the Phase I work was done during the winter season which was according to the established plan, but heavy winds along with the extremely cold weather, which were not anticipated, impeded the scheduled progress of the work and aggravated the difficult schedule problems. Figure 4 shows the full structural steel frame for the materials and the light/heavy buildings.

Bar joists were installed for support of the floor decking and the roof. Placement of the floor decking and the roof decking in the windy winter weather turned out to be another item that affected scheduled progress detrimentally. Concrete block wall work was one of the large items of work involved in the interior walls in the buildings. The exterior walls were composed of 10ft. high precast concrete panels above which was placed 2 inch preformed insulated metal panel.

The entire project involved placement of 28,000 cubic yards of concrete, 6,000,000 pounds of structural steel, along with 16 miles of trackage and 98 turnouts.

This project was our first experience with "Fast-Track" construction and it turned out to be both challenging and interesting.

Burlington Northern construction engineers had the overall management of the project. A basic field engineering crew of ten engineers was assigned to the work. This crew was augmented when required by specialists engaged on a consulting basis for such items as survey help, soils engineering and soils compaction, contract programming, structural steel shop fabrication and steel erection. These men carried out the staking of track work and the establishing of basic survey control for the grading and building work, inspection of contract construction work and the scheduling and monitoring of progress. We gave a good deal of consideration to the procedures that we thought would be best suited to the scheduling and monitoring of the work. Burlington Northern forces had developed and used a critical path arrow diagram with satisfactory results on several phases of the Northtown Yard project, but some reservations as to our capabilities for using this procedure for the Alliance work were raised. We engaged a consulting firm with broad experience in large construction projects to assist us in making the determination as to the proper form of the overall construction program and schedule. These people proposed that the various contracts could be best scheduled and monitored by using bar charts and also that the overall construction schedule should be

Figure 4

represented by a bar chart. The bar charts were based on precedence diagrams and each of the bars or parts of bars between division points represented about $75,000 or less of the project. The schedules for the Phase I and Phase II general contracts involved a large number of bars to adequately represent the various activities but they turned out to be of manageable size. The schedules for the other contracts were of a more routine nature. Each contractor was required by his contract to prepare a detailed bar chart schedule and our field engineers had the task of reviewing those schedules to determine that they were in sufficient detail to facilitate monitoring the work to keep it on schedule, and that the individual schedules could be coordinated into an overall project schedule. In the various charts the bars representing the activities were, of course, placed in their proposed time frame as established by the precedence study and further they were subdivided by suitable intervals of no greater than a half month to show the proposed status of each activity by percent as of the end of the interval. The proposed rate of progress was shown along the top of the bar, the actual progress as measured during the montioring process was shown along the bottom of the bar. Adequate information as to the physical quantities of work represented by the various bars was provided to the monitors.

Information from the detail bar charts was condensed on an overall chart showing the major divisions of work for purposes of reporting progress to the St. Paul office.

This kind of scheduling also provided the basis on which the delivery of long lead time items of equipment and material could be watched. However, this item turned out to be a difficult one to monitor. Although the contractors all had material expeditors on their staffs and we had representatives from Burlington Northern telephoning and visiting several of the contractors' major suppliers during the shop fabrication period, there were still a number of items of equipment that were delivered late and in turn required juggling of the construction schedule.

An experienced engineer was furnished by the construction programming consultant to assist in the monitoring and scheduling work. A Burlington Northern engineer was also assigned full time to such work along with another engineer who gave about one half of his time to the monitoring work. This force was able to keep up with the monitoring work and to provide data necessary to assist in keeping the contractors on schedule.

The primary way in which the monitoring effort was transmitted to the contractor was by means of a weekly meeting on Thursdays attended by all contractors at which was discussed scheduling and coordinating problems and on Friday a detailed day by day activity outline of work as proposed for the following week was developed by the monitors in consultation with each of the contractors. This system proved to have enough flexibility to ensure that the frequent problems arising in the schedule could be handled and responses made to the variances in progress so as to bring the overall project into line.

Figure 5 is a view of the east end of the facility showing the completed trackage. Figure 6 is an overall view of the trackage.

Although all of the contractors used the most up-to-date procedures in carrying out the project, we look back on the successful completion of the work as more of an example of organization and planning than of new or innovative construction methods. All of the contractors provided a good scheduling effort which contributed greatly to the success, and the part played by Burlington Northern field engineers in obtaining the very substantial coordination between contractors needed to make up for the various delays was vital. It is a source of satisfaction to us that Burlington Northern engineers played a substantial role in the on-time completion of this "Fast-Track" project.

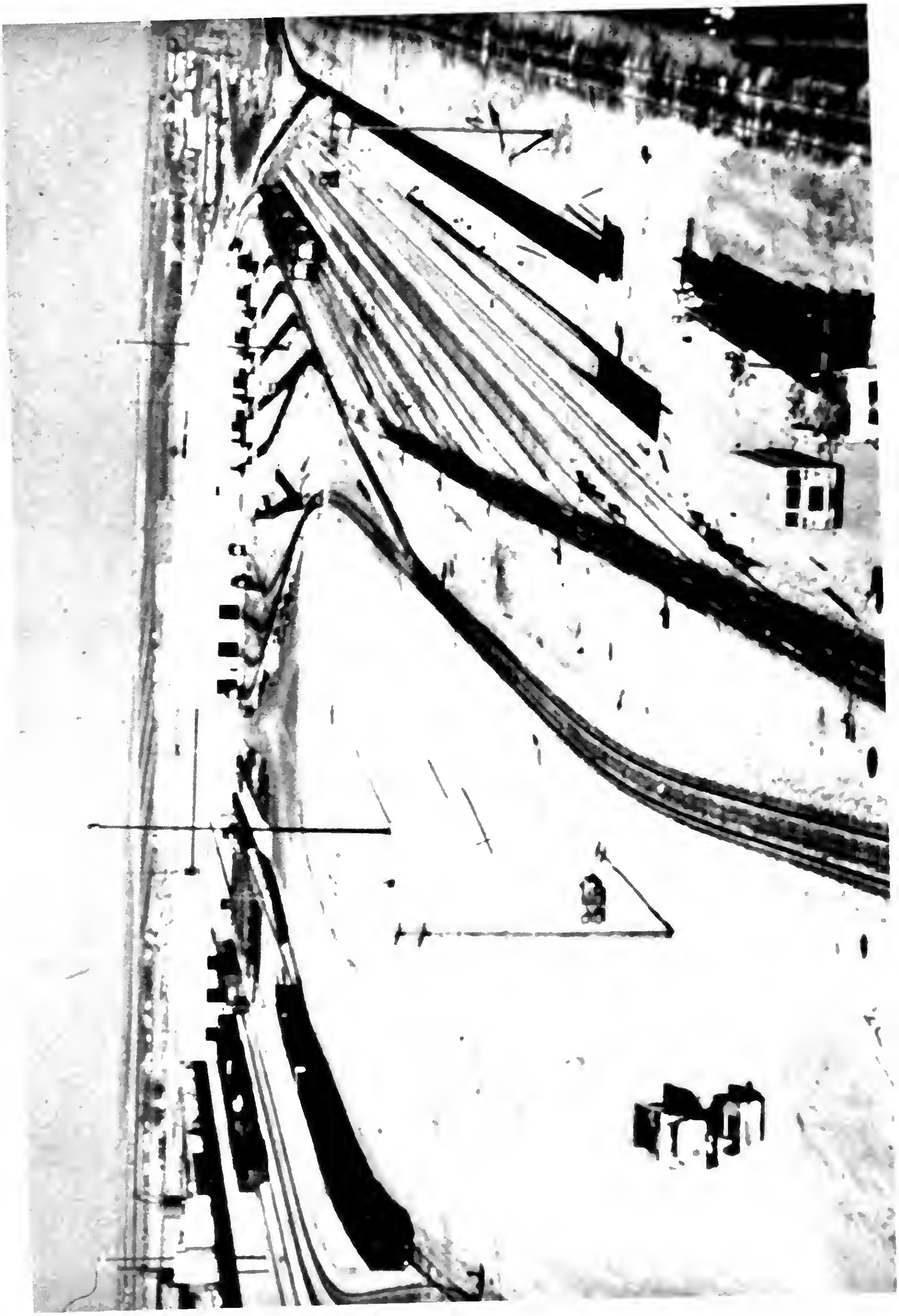


Figure 6

New Car for Measurement and Evaluation of Gage-Widening Resistance of Track

by Allan M. Zarembski,[1] Donald P. McConnell[2] and W. Scott Lovelace[3]

Acknowledgement

This report was prepared as part of the ongoing work of the Track Strength Characterization Program. This program is sponsored by the Track Research Division of the Association of American Railroads Research and Test Department and the co-operative Industry-Government Track Train Dynamics Program.

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Introduction

The primary concern of railroad engineering departments is to provide a track structure which is safe and reliable at a minimum cost. To reach this goal, it is critical to know the capacity of the track to support vetical, lateral, and longitudinal loads imposed by train operations and environmental conditions. For example; the lateral strength requirements for tangent track, a 3° curve, and a 6° curve are certainly not the same.

It is very difficult to improve our track structure and gain a more stable, reliable, and safer track without a sound definition of the factors that affect the strength of track. As a better understanding of these factors involved in track strength is gained, the ability to provide safe and reliable track at a minimum cost will increase.

The Track Strength Characterization Program was initiated in 1977 to coordinate and progress *track strength* research.

The four overall objectives of the Track Strength Characterization Program can be defined in the following activities[1, 2, 3]:

1. *Determine What Influences Track Strength*—Define the parameters that influence the strength of the track, i.e., the load carrying capacity of the track.

2. *Measure The Parameter That Affect Track Strength*—Once these parameters are established, their affect on the track strength must be measured and understood.

3. *Develop Practical Methods of Determining Strength of Track*—Techniques and equipment must be developed to accurately determine the strength characteristics of track *in the field.* It does little good to measure track in a laboratory if we are unable to measure the same parameter in the real world.

4. *Demonstrate the Benefits of Quantifying Track Strength*—This will include investigation and demonstration of the benefits of track strength measurements, such as:

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[1]Manager—Track Research, Association of American Railroads.
[2]Manager—Standards Research, Track Systems Branch, Transportation Systems Center.
[3]Manager—Research and Tests, Southern Railway.

(a) an aid to maintenance planning.
(b) input to traffic planning.
(c) input to rolling stock design.
(d) an aid to derailment prevention.

Under the guidance of the program, a series of laboratory tests, aimed at defining track strength parameters and obtaining basic data on these parameters, were conducted at the Association of American Railroad's (AAR) Track Laboratory, in Chicago, Ill.[3] These tests provided basic information about the strength of track. However, in order to investigate the feasibility of measuring track strength in the field, a gage restraint measurement test was conducted at the AAR Technical Center in October, 1978.[4] This test utilized a special rail spreading apparatus, mounted under a flat car, which measured the rail deflections, as well as the vertical and lateral loads applied to the track, as the vehicle moved along the test track. The results of those tests indicated that it appeared possible to measure the strength of track, in general, and to identify specific weak spots along the track, in particular. It was furthermore noted that this type of testing could be performed with minimal damage to the track structure.[4]

As a result of these gauge restraint rests, a new track strength measurement test vehicle was constructed under the auspices of the co-operative government-industry Track Train Dynamics Program.[3, 5] It is the purpose of this report to describe and discuss the preliminary tests of this track strength test vehicle, the "DECAROTOR," conducted at the Southern Railway's Alexandria, VA. Yard in January, 1980.

Test Objectives and Scope

In order to develop a track strength measurement technique that could be effectively utilized by a railroad to evaluate it's track conditions, it is necessary to have a test apparatus, capable of evaluating long stretches of track quickly, without interfering with traffic, and non-destructively, i.e. without causing permanent damange to the track structure. Furthermore, it should be directed towards the detection of track strength "problems," that are of real concern to the railroad maintenance departments.

In reviewing the causes of track-related accidents, it was found that a significant percentage of these accidents were attributed to a lack of sufficient rail restraint, i.e., poor ties, or fasteners, or both. These conditions are generally not detectable through geometry measurements, and consequently, their detection is left to the "eye" of the track inspector. Thus, a judgement must be made as to whether the track conditions can safely carry the expected traffic. Since a lack of sufficient rail restraint can result in a derailment, such as when a rail overturns, this area was selected for the initial investigations.

Recent laboratory and field investigations,[6, 7] directed towards the problem of gage widening and rail overturning, provided a basic set of data to guide the development of a testing methodology. The concept of a continuous measurement technique, aimed at examining the gage restraint "strength" was established through the rail spreader tests.[4] However, the actual feasibility and practicality of conducting moving gage restraint strength measurements on track was yet to be demonstrated.

The prototype test vehicle constructed to make these tests has the capability of applying variable vertical and lateral loadings independently to each rail using a continuous feedback system to maintain the load levels, and also permits the direct measurement of the corresponding rail deflections under the uniform loads. It also has the capability of testing at speeds of up to 7 miles per hour, and is able to test through curves of up to eight degrees.

The preliminary tests were designed to fulfill two specific objectives and were consequently carried out in two states.[8] The first stage of the tests was a shakedown test series, aimed at determining the operational characteristics and measurement accuracy of the DECAROTOR. The second stage was a feasibility test, aimed at examining the capability of the system to measure track strength, and identify "weak" track of good *yard* quality. Both tests were essential prerequisites for preliminary evaluations of the system's capability, and for future operations over mainline track.

Test Car Design

The Track Strength Test Car "Decarotor," shown in Figure 1, was designed to measure the ability of the track to maintain gauge under controlled lateral and vertical loads. To do so, the car must measure the unloaded track gauge and the change in gauge which occurs under closely modulated vertical and lateral loads.

The design of the car is based on a concept developed by the Transportation Systems Center which employs a mechanically stabilized load wheel and a "closed loop" feedback load control system. The 12 inch diameter loading wheels, located at the car center, rotate on an instrumented stationary axle carried in a carrier housing shown in Figure 2. The hydraulic loading cylinders are reacted to the centersill of the car through a kingpost and auxiliary structure, welded to the ballasted 50 ton box car. The design of the mechanical and structural components of the system was accomplished by the Research and Test Department of Southern Railway.

In operation, the loads applied to the rails are controlled by utilizing the load cell axle in the loading wheels to continuously monitor the vertical and lateral loads on the load wheels. An electronic controller located in the Southern R-2 instrumentation car, Figure 3, compares the instantaneous applied load with the preset command, and adjusts a servohydraulic valve to maintain the applied load. This system controls the loads, while the car is in operation, to within 2 per cent of the desired load up to the peak loads of 15,000 pounds lateral and 20,000 pounds vertical.

The track gauge is monitored at the three locations shown as G1, G2, and G3 in Figure 4. Unloaded gauge prior to and following the load wheels is measured to within ± 0.1 inches at locations outboard of the trucks, by the spring loaded system, shown in Figure 5. The loaded gauge G2, is measured by monitoring the gauge at the load wheels. The track gauge at these locations, as well as the change in gauge under load and the applied loads are recorded on FM magnetic tape as well as on the chart recorders shown in Figure 6.

In addition to the load control system, the car is equipped with two safety systems to prevent spreading of the track gauge. The controller is equipped to retract the load wheels both manually and automatically. The automatic system will retract the wheels whenever preset limits on maximum gauge or change in gauge are met. Once the safety limits are tripped, the load wheels must be manually reset on the rails.

Initial Evaluation of the Decarotor

In order to evaluate the performance of the track strength test car and to determine the accuracy of the control and measurement systems, a preliminary series of tests were conducted on the Southern Railway in January, 1980. The test series was designed to accomplish the dual objectives of:

1. Characterizing the operational capabilities of the car design to control load and measure track displacement, and

2. Conduct an initial evaluation of the ability of the car to locate weak gauge restraint.

Figure 1 TRACK STRENGTH TEST VEHICLE (DECAROTOR)

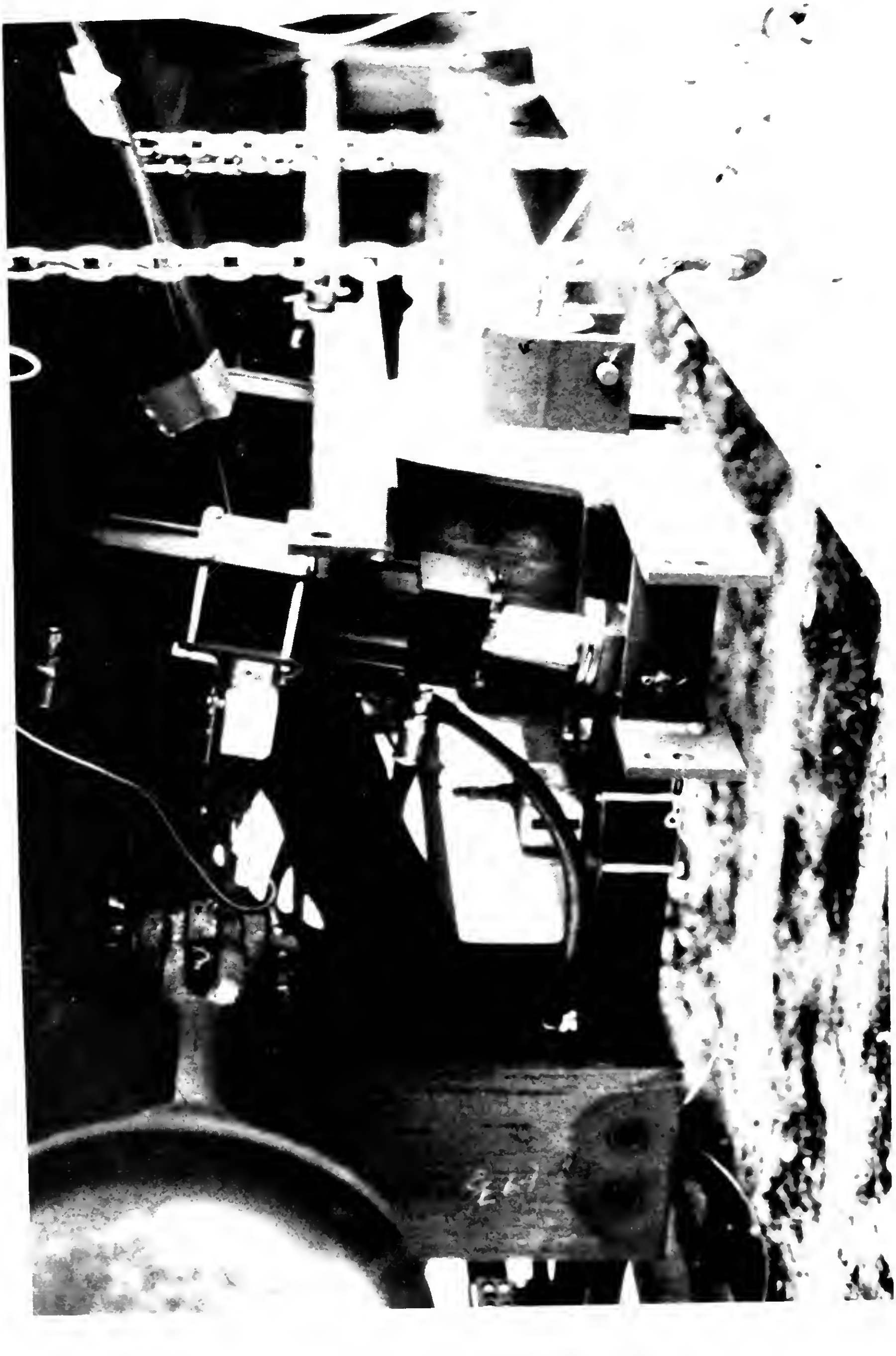


Fig. 2 DECARBOTOR LOADING WHEEL AND CARRIER HOUSING

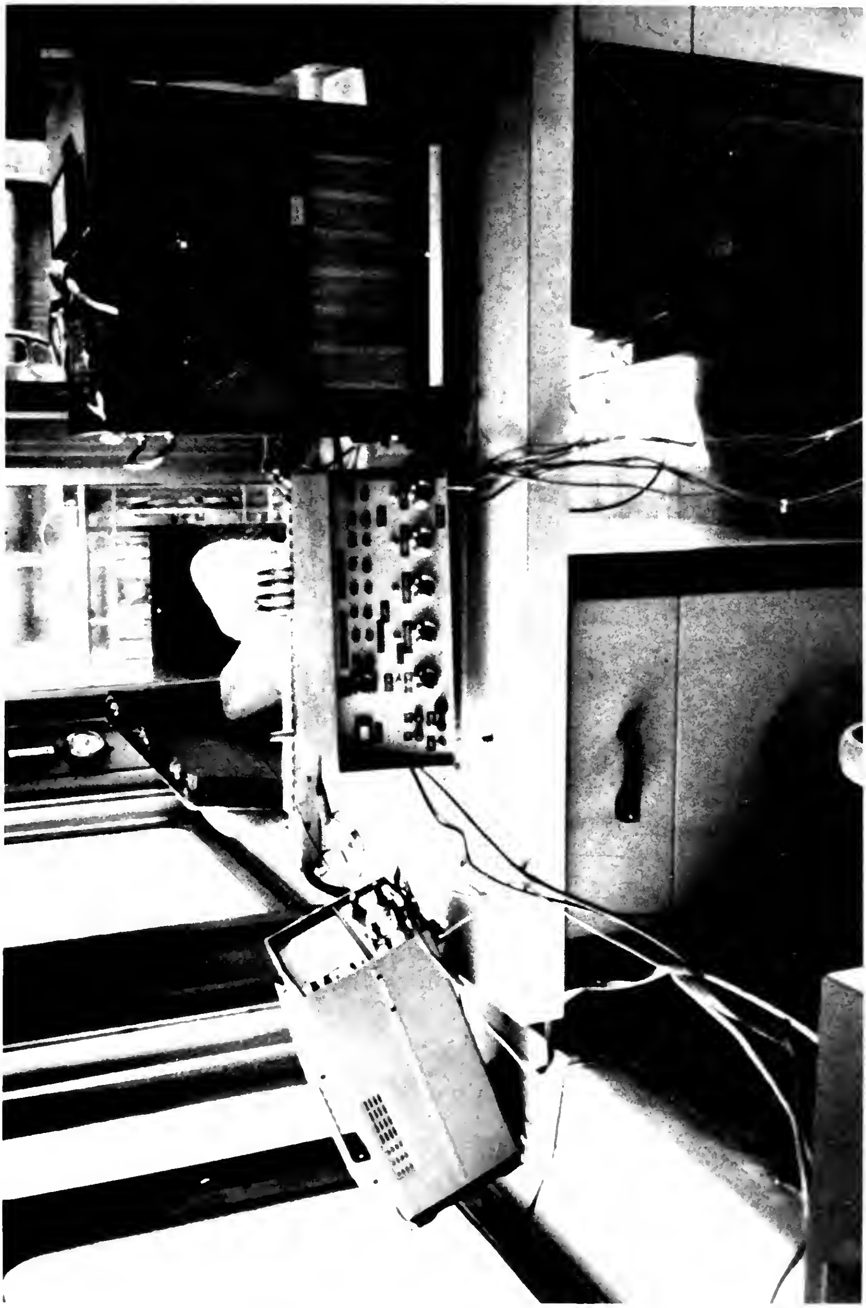


Figure 3 ELECTRONIC CONTROLLER FOR AUTOMATIC LOAD CONTROL

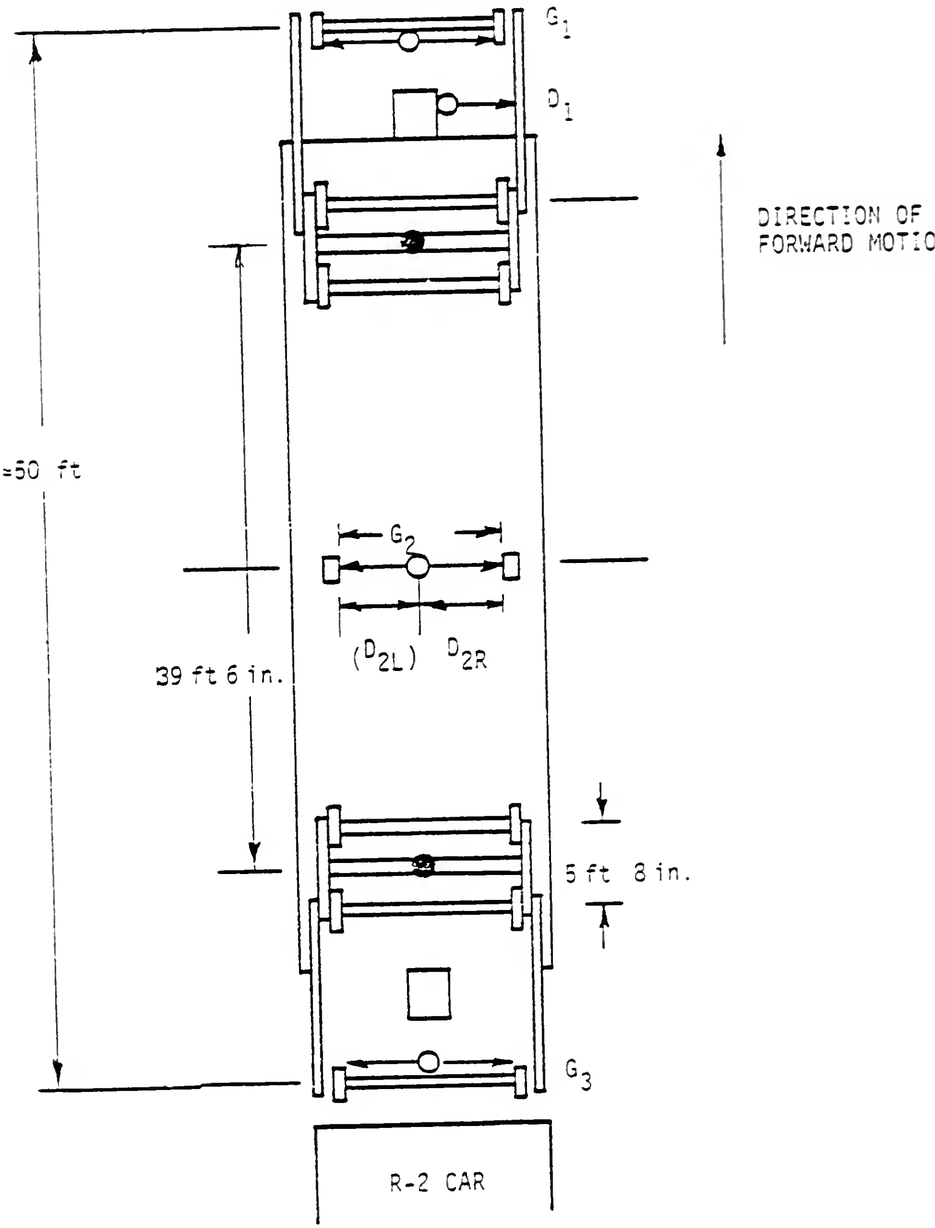


FIGURE 4 OVERALL TRANSDUCER LAYOUT OF THE RAIL RESTRAINT TEST CAR

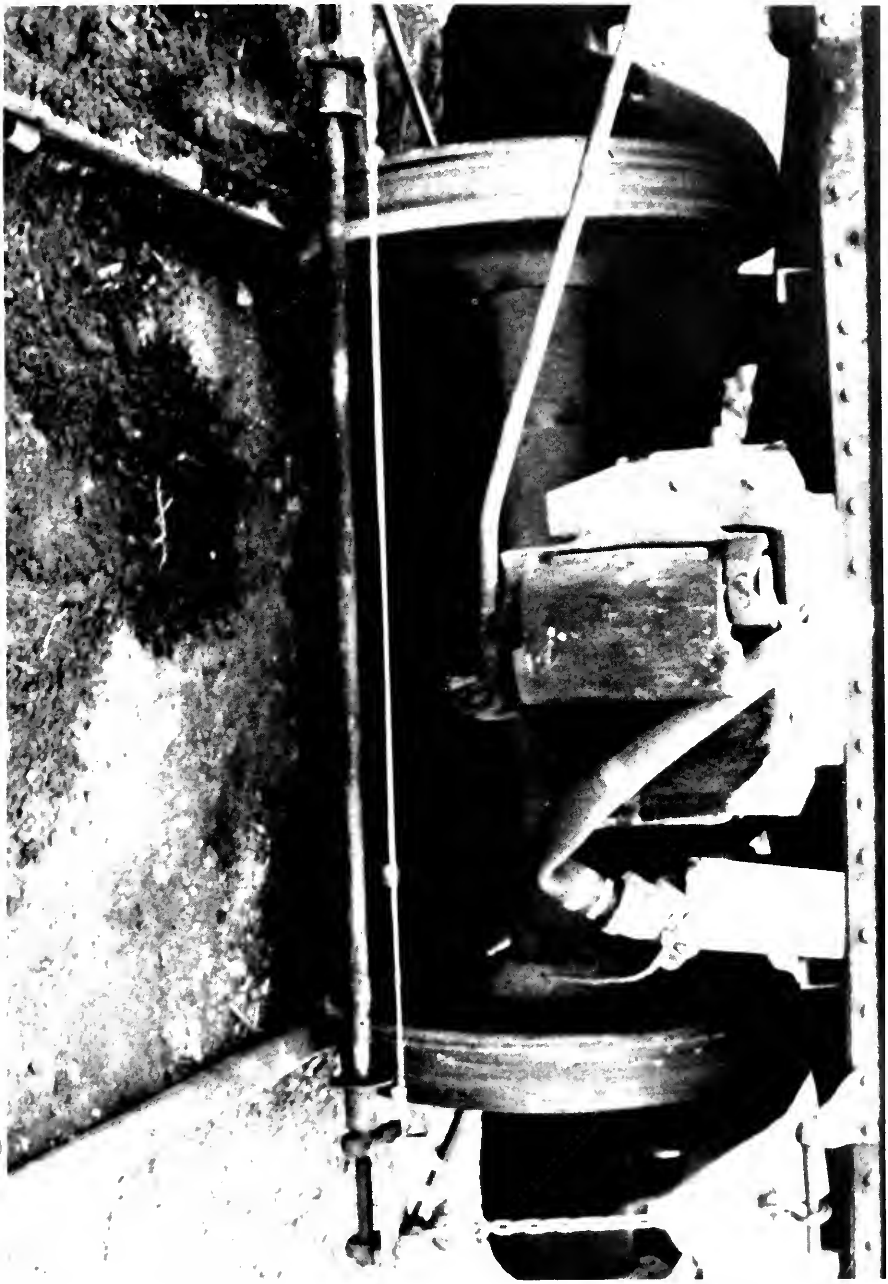


Figure 5 UNLOADED GAGE MEASUREMENT SYSTEM

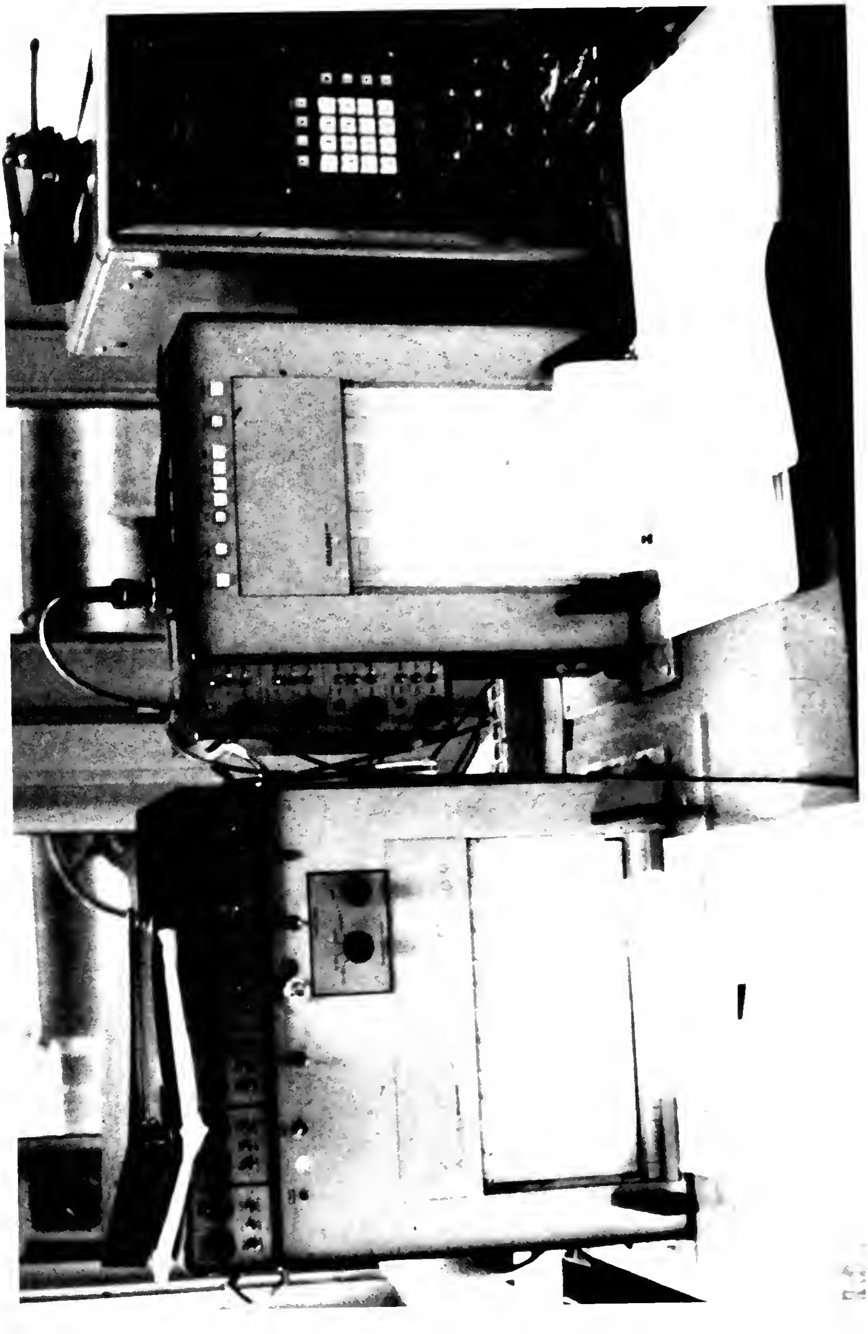


Figure 6 STRIP CHART DATA RECORDERS

The test series was organized in two parts. The first, known as the vehicle evaluation test series, was designed to test the car on track of known characteristics. The second series of tests on yard track was designed to compare the output of the test car with visual evaluation of tie conditions.

The vehicle evaluation test series was conducted on the "WHARF" track at the Southern Railways Research Laboratory, Alexandria, VA. The track was modified to include both a stiffened segment equipped with compression clips and gauge bars, as shown in Figure 7 and a weakened section. Both of the sections were instrumented by mounting strain gauge load arrays and rail displacement transducers, as shown in Figure 8. The wayside instrumentation was monitored by a mobile instrumentation van provided by Portec Inc. The wayside instrumentation provided the reference against which the test car systems were calibrated.

The comparison of the output of the car with wayside tie condition was conducted on a 410 tie segment of track in the Southern Railways Middle Yard at Alexandria, Virginia. The track, consisting 100 lb RAIL layed on wood ties at 24 inch spacing, with some sections last timbered in 1960, and exhibited a range of tie conditions from very poor to sound. Ties judged to be poor exhibited significant plate cutting, splitting, or elongation of the spike holes. This segment of track was visually inspected and tie condition tabulated as a basis for comparison with the rail restraint indication of the test car. Table 1 presents this evaluation of tie condition for the first 140 ties.

Results

The Track Strength Vehicle Test was conducted on January 14-18, 1980 at the Southern Railways' Alexandria Yard in Alexandria, Virginia. The test consisted of two distinct parts. The first part was the calibration, evaluation and check out of the test vehicle and test consist. This test was conducted in the Southern Railways Research and Test Department's test compound. The second part of the test was the actual track strength system test and was conducted on the South Wharf Track in Southern Railways' Middle Yard, Alexandria, VA. A total of ten test runs were conducted on the Yard Track, at speeds of up to 7 mph. A complete breakdown of the test series is given in Table 2.

Vehicle Evaluation Test Series

The objectives of the Vehicle Evaluation test series were to conduct a preliminary check out of the test vehicle, to calibrate the on board load and deflection measurement systems, and to evaluate the ability of the test vehicle to follow and detect various conditions of gage restraint.[8] As can be seen in Table 2, these objectives were accomplished through a series of static system checks and calibrations and a moving evaluation run. The stationary calibration of the load system was carried out the reinforced test section shown in Figure 6, which used independently calibrated lateral and vertical strain gage arrays to measure the applied lateral and vertical loads.

The resulting calibration curves for the vehicle load system are given in Figures 9 and 10. Note that for the full range of lateral (and simultaneous vertical) loads outlined in Table 3, the load calibration curves are quite linear,with an extremely tight scatter band. Furthermore, monitoring of the rotation of the wheelbox indicated that inclination of the loading system, for the full range of calibration loads was always less than 2°. Thus, in the initial calibration, the vehicle system performed extremely well.

During the slow speed (1 MPH) evaluation run, the ability of the system to automatically retract at an "extreme" weak spot was validated at a compromise section of 131 RE to 100 RE rail on very "poor" tie conditions. The system, upon undergoing a loaded gage widening of over 1 inch, automatically unloaded and retracted. This was repeated twice, and the slow

Figure 7 REINFORCED TRACK SECTION FOR VEHICLE SYSTEM CHECK OUT

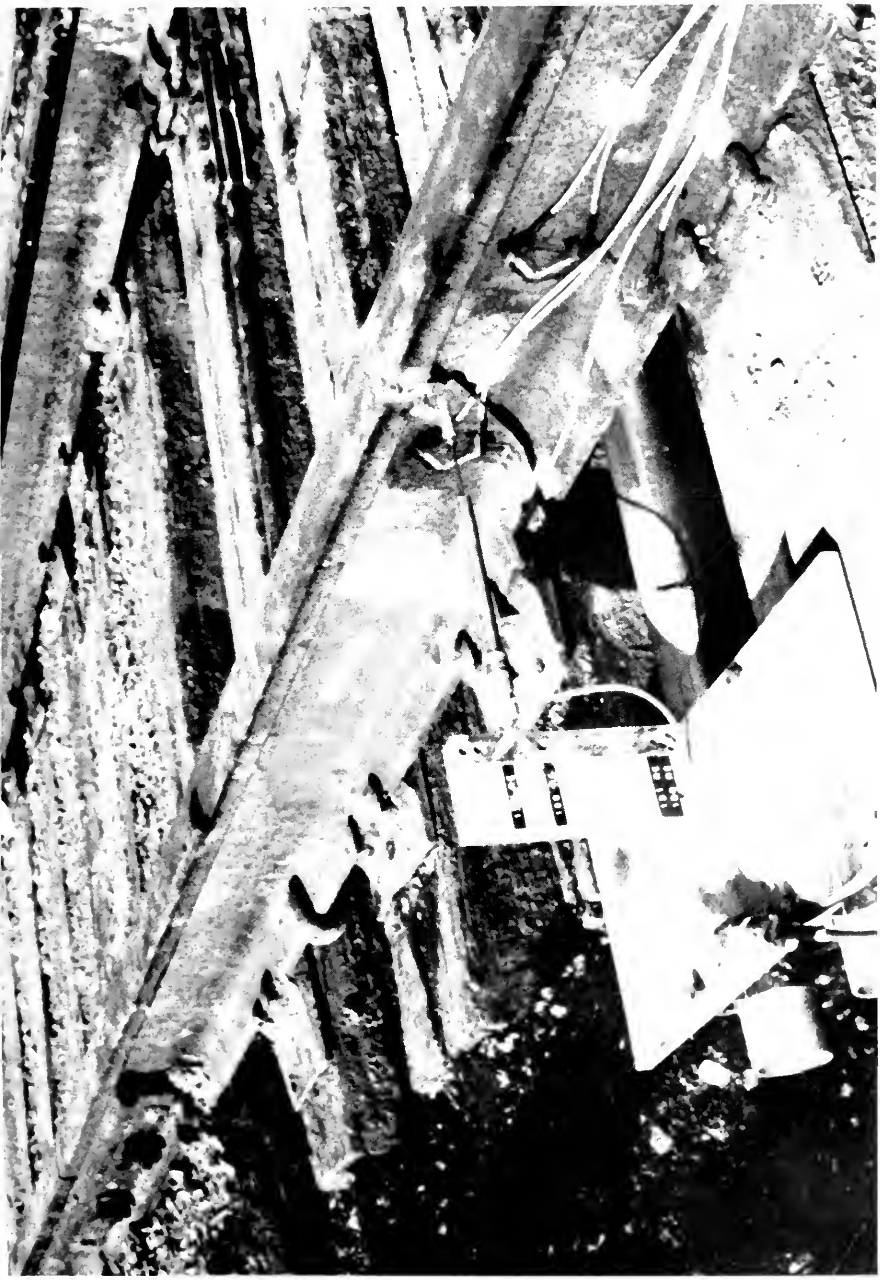


Figure 8 WAYSIDE DATA INSTRUMENTATION (DISPLACEMENT TRANSDUCERS AND STRAIN GAGE LOAD ARRAYS)

TABLE 1

TRACK CONDITION SOUTH WHARF TRACK—ALEXANDRIA YARD

(100 lb. Rail)

(Could only see right side.
Left covered with ballast.)

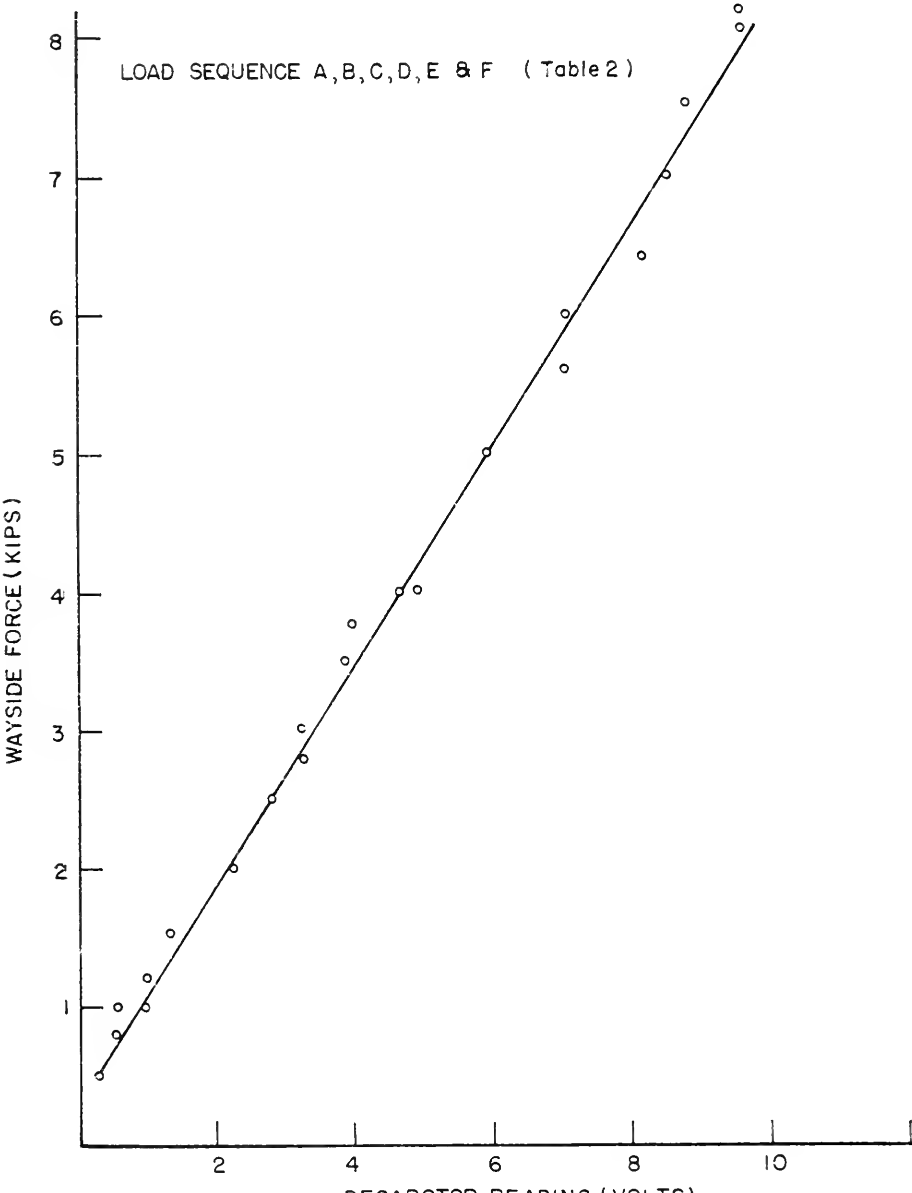
1. Fair
2. Good
3. Good
4. Good
5. Good
6. Good
7. Fair (Slight plate cut)
8. Good
9. Poor
10. Good
11. Good
12. Poor
13. Good
14. Poor
15. Good
16. Good
17. Good
18. Good
19. Poor
20. Good
21. Good
22. Poor
23. Good
24. Good
25. Poor (Tie plate
26. Poor Cutting)
27. Good
28. Poor
29. Good
30. Good
31. Good
32. Good
33. Good
34. Good
35. Good
36. Good
37. Good
38. Fair
39. Fair (Ends split)
40. Poor
41. Good
42. Fair
43. Good
44. Good
45. Poor
46. Good
47. Fair
48. Fair
49. Fair
50. Good
51. Good
52. Good
53. Fair
54. Poor
55. Good
56. Good
57. Good
58. Fair
59. Fair
60. Poor
61. Poor
62. Good
63. Good
64. Good
65. Poor
66. Good
67. Fair
68. Good
69. Fair
70. Fair
71. Good
72. Good
73. Fair
74. Poor
75. Fair
76. Fair
77. Fair
78. Good
79. Good
80. Good
81. Fair
82. Fair
83. Good
84. Fair
85. Good
86. Good
87. Good
88. Good
89. Good
90. Fair
91. Good
92. Fair
93. Fair
94. Good
95. Fair
96. Good
97. Fair
98. Good
99. Good
100. Fair
101. Fair
102. Fair
103. Poor Angled
104. Poor in Track
105. Good
106. Poor
107. Fair
108. Good
109. Good
110. Poor
111. Good
112. Good
113. Poor
114. Fair
115. Fair
116. Fair
117. Good
118. Good
119. Poor
120. Good
121. Could not
122. see tie
123. ends. Covered
124. with ballast.
125. Good
126. Fair
127. Fair
128. Good
129. Good
130. Fair
131. Good
132. Good
133. Good
134. Good
135. Good
136. Good
137. Poor
138. Poor

TABLE 2

TRACK STRENGTH VEHICLE TEST SERIES

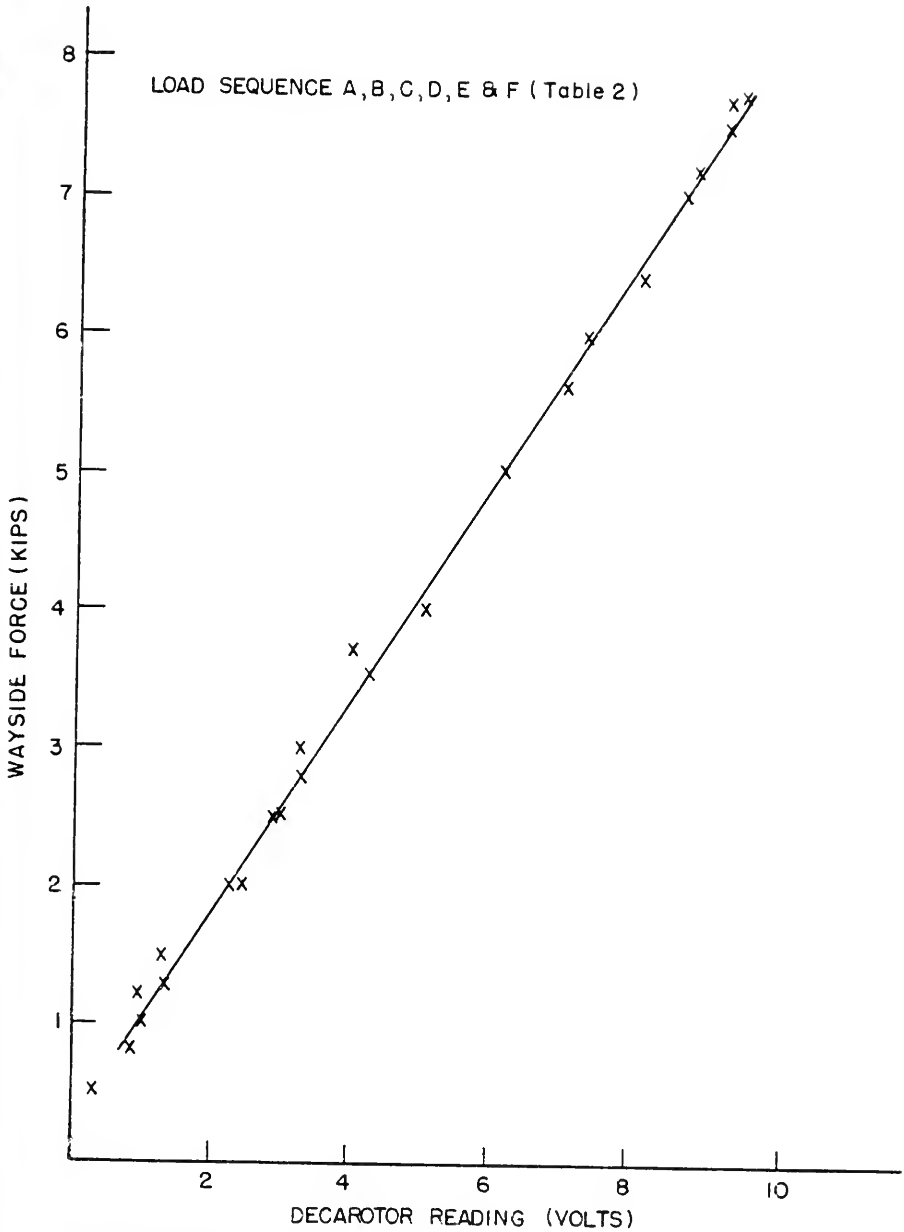
Test Series	Run No.	Applied* Loads (Kips)	Test Speed (MPH)
		V, L	
Evaluation/Check Out	1	Table 3	Stationary
" "	2	10, 7	1
Yard			
"	1 West	10, 7	1
"	1 East	10, 7	1
"	2 West	10, 7	1
"	2 East	10, 7	1
"	3 West	10, 7	3
"	3 East	10, 7	3
"	4 West	10, 7	3
"	4 East	10, 7	3
"	5 West	10, 7	7
"	5 East	10, 1	1

*L = Lateral Load (Wheel)
V = Vertical Load (Wheel)



STATIONARY LATERAL LOAD CALIBRATION: WAYSIDE FORCE VS. DECAROTOR RIGHT LATERAL FORCE (IN VOLTS)

FIGURE 9



STATIONARY LATERAL LOAD CALIBRATION: WAYSIDE FORCE VS. DECAROTOR LEFT LATERAL FORCE (IN VOLTS)

FIGURE 10

TABLE 3

STATIONARY LOAD CALIBRATION LOAD SEQUENCE

Sequence	Vertical Load Targets	Lateral Load Targets	L/V
A	15.0 kips	1.50 kips	0.10
		3.75 kips	0.25
		7.50 kips	0.50
		8.00 kips	0.55
B	12.0 kips	1.20 kips	0.10
		3.00 kips	0.25
		6.00 kips	0.50
		8.00 kips	0.66
C	10.0 kips	1.00 kips	0.10
		2.50 kips	0.25
		5.00 kips	0.50
		7.00 kips	0.70
		8.00 kips	0.80
D	8.0 kips	0.80 kips	0.10
		2.00 kips	0.23
		4.00 kips	0.50
		5.60 kips	0.70
		6.40 kips	0.80
E	5.0 kips	0.50 kips	0.10
		1.25 kips	0.25
		2.50 kips	0.50
		3.50 kips	0.70
		4.00 kips	0.80
F	4.0	1.00 kips	0.25
		2.00 kips	0.50
		2.80 kips	0.70
G	2.0 kips	0.50 kips	0.25
		1.00 kips	0.50
		1.40 kips	0.70
H	1.0 kips	0.50 kips	0.5
		0.70 kips	0.7

speed evaluation run clearly indicated that the Track Strength System was able to operate on railroad trackage.

In-Track Test Series

The objective of the In-Track Test Series, conducted on the South Wharf Track of Southern Railway's Middle Yard, was to evaluate the performance of the test vehicle on unaltered trackage of yard quality. The test vehicle measurements would be compared with a visual survey of tie and fastener conditions in order to determine repeatability of the system, examine overall operational characteristics of the test car; and evaluate the capability of the test car measurements to detect anomalies in the track structure.[8]

The objectives were accomplished in a series of ten test runs conducted over an 800 foot length of track. (Table 1). These runs were conducted at a fixed load level of 7,000 lbs lateral load (per wheel), and 10,000 lbs vertical load (per wheel) for an L/V ratio of .7. The test runs were conducted at several speeds ranging from one to seven miles per hour (mph). The test consist, shown in Figure 11, consisted of a Southern Railway's locomotive, the Southern R-2 Instrumentation Car and the Track Strength Test Vehicle (Decarotor).

During each test run, which consisted of one pass either eastbound or westbound along the 400 tie test section, the load level was fixed, and the loaded gage (G2) together with the unloaded gage 25 ft from the load pint (G1,G3) were recorded on analog type and on strip chart. Thus, since the load levels were held constant, the system recorded the net gage widening of the track as the load passed. This can be see in Figure 12 which shows the loaded gage (G2) and the unloaded gage (G3) plotted for the first 140 ties in the test section. The net gage widening or $\Delta G = G2 - G3$ is shown in Figures 13 together with a brief evaluation of the tie and joint conditions. The reader should note that every time the system encountered a section of weakened track, such as at a weak joint or at a concentration of poor ties (as determined by visual observation) the change in gage or ΔG exhibited a significant increase. For example, at the concentration of four poor to fair ties around tie number 60, a significant widening of the gage occurred. As can be seen Figure 14, which is a photograph of tie 60, the tie condition is quite poor. A similar comparison can be made at tie number 139 by noting the tie condition shown in Figure 15 and the large difference in loaded and unloaded gage shown in Figures 12 and 13.

In Figure 12, the reader can observe the additional information to be obtained from a "strength" or loaded gage measurement (G2) as opposed to a conventional unloaded gage measurement (G3). If the track section were of a perfectly uniform quality it would be expected that G2 would exhibit a constant shift or offset, at all points, from G3. However, since the test section was not of uniform condition, the shift under load was not uniform, but rather tender to increase around the weaker tie or fastener locations. The degree of sensitivity can be best seen in the ΔG measurement of Figure 13 where relatively small differences in strength, can be detected.

This ability to detect weakened sections, was found to be generally repeatable for the different test runs. Thus, while the relative magnitude of the gage widening or ΔG measurement did change somewhat from run to run, the ability to detect the *same* weakened test sections, i.e., the repeatability of the measurements, was found to be quite good. Even for runs in opposite directions, where the wave effect of the rail deflection curve would expect to cause a slight shift of the peak values, the ability to identify the same weak spots in different runs was found to be quite reasonable. This can be seen in Figure 16, where the gage widening (ΔG) for an eastbound and westbound run, at the same vehicle speed, indicate a slight shift due to the directional effect, however, the basic strength information appears to be the same.

Similarly with increases in the test speed, it was found that though there is some fluctua-

Figure 11 DECAROTOR TEST CONSIST

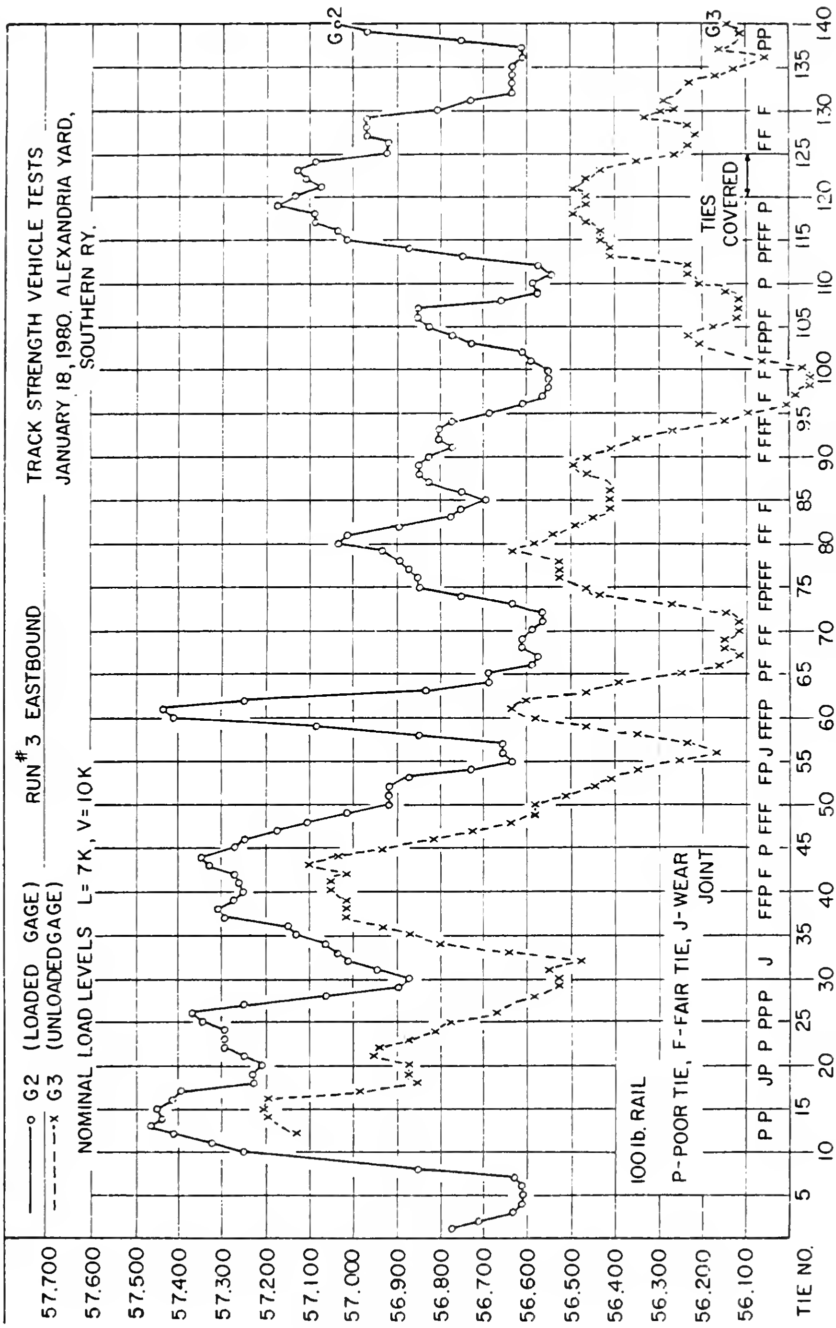


FIGURE 12 COMPARISON OF LOADED AND UNLOADED GAGE MEASUREMENTS.

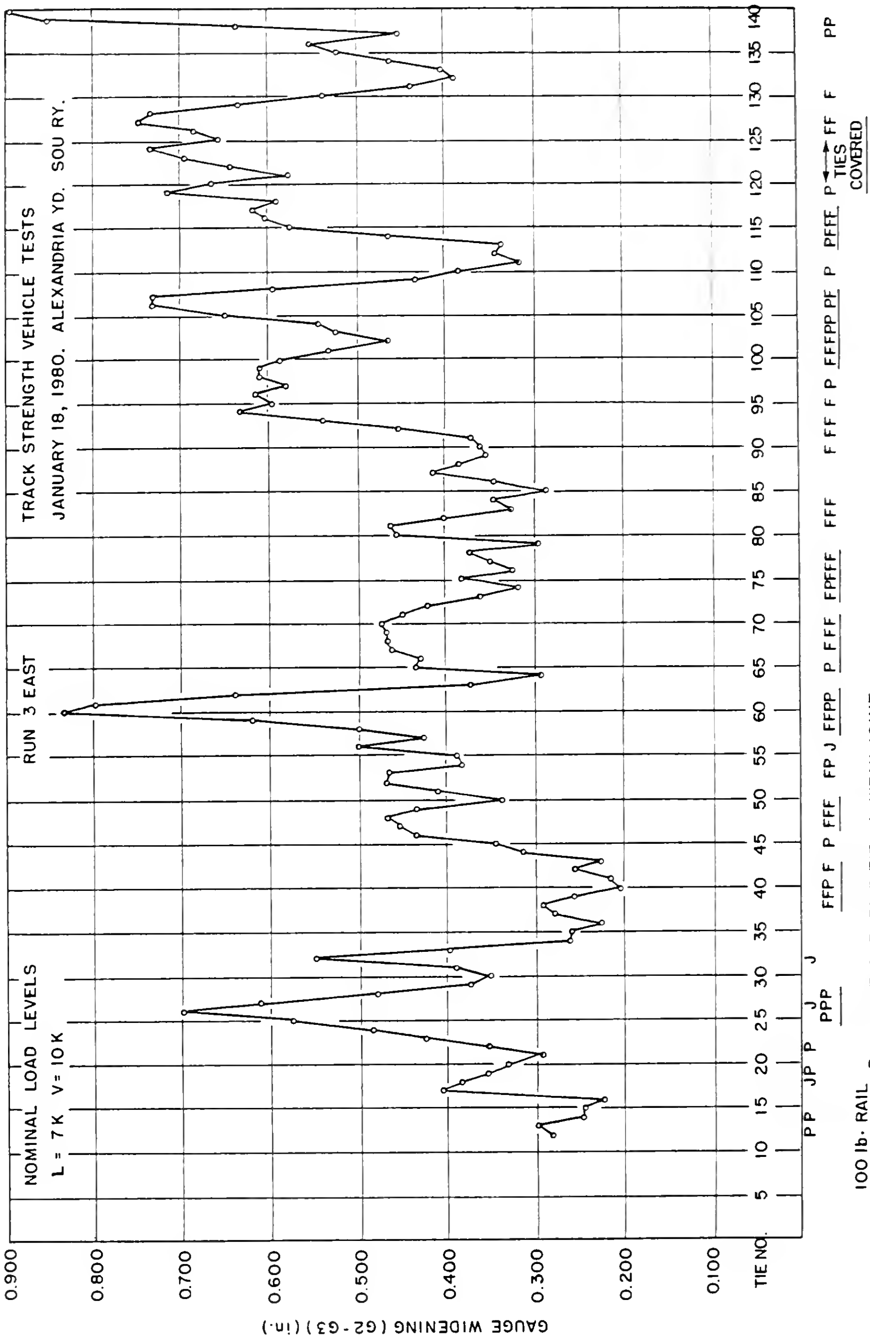

TRACK STRENGTH VEHICLE TESTS
JANUARY 18, 1980. ALEXANDRIA YD. SOU RY.
RUN 3 EAST
NOMINAL LOAD LEVELS
L = 7 K V = 10 K
GAUGE WIDENING (G2 - G3) (in.)
TIE NO.
TIES COVERED
100 lb. RAIL
P - POOR TIE, F - FAIR TIE, J - WEAK JOINT

FIGURE 13

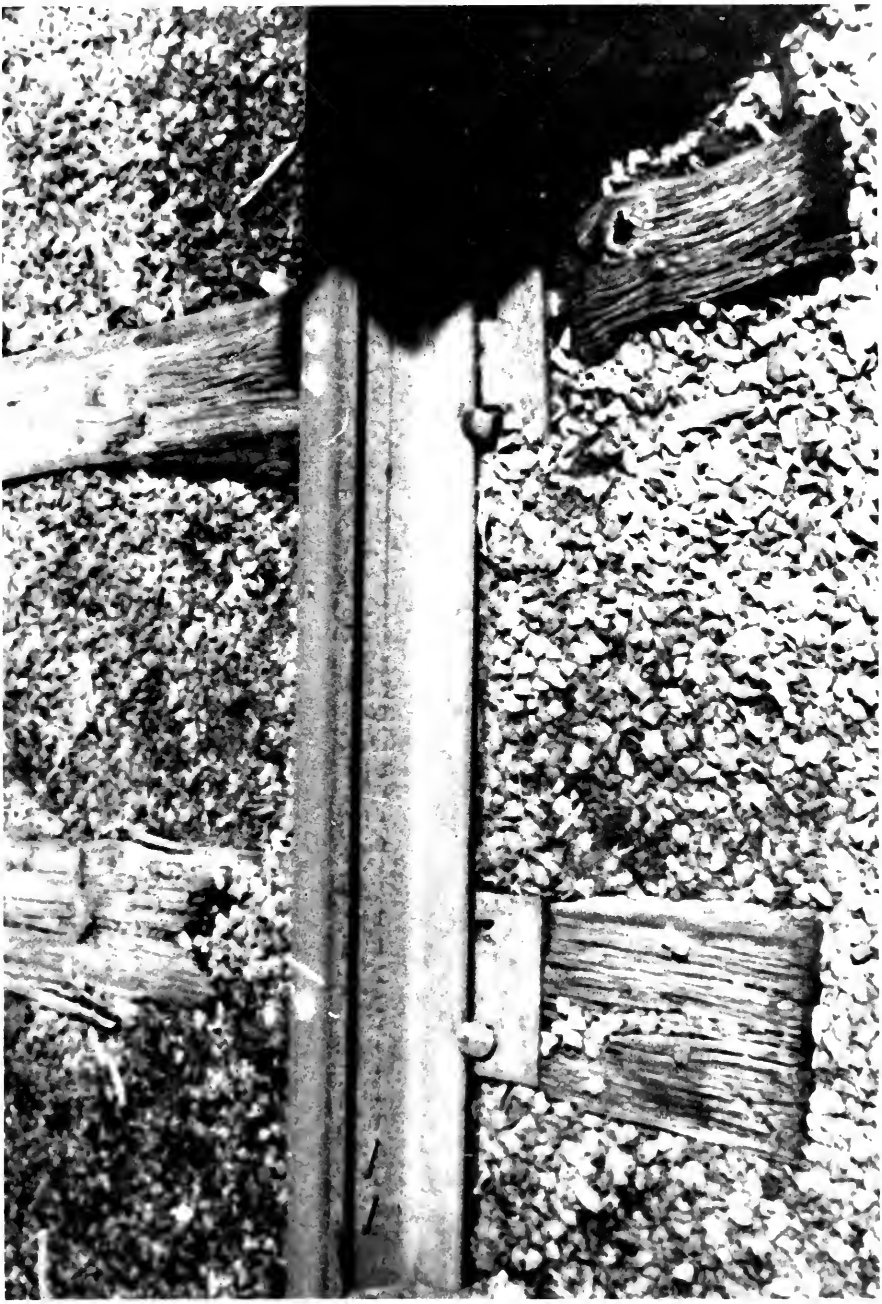


Figure 14 VIEW OF TIE NUMBER 60

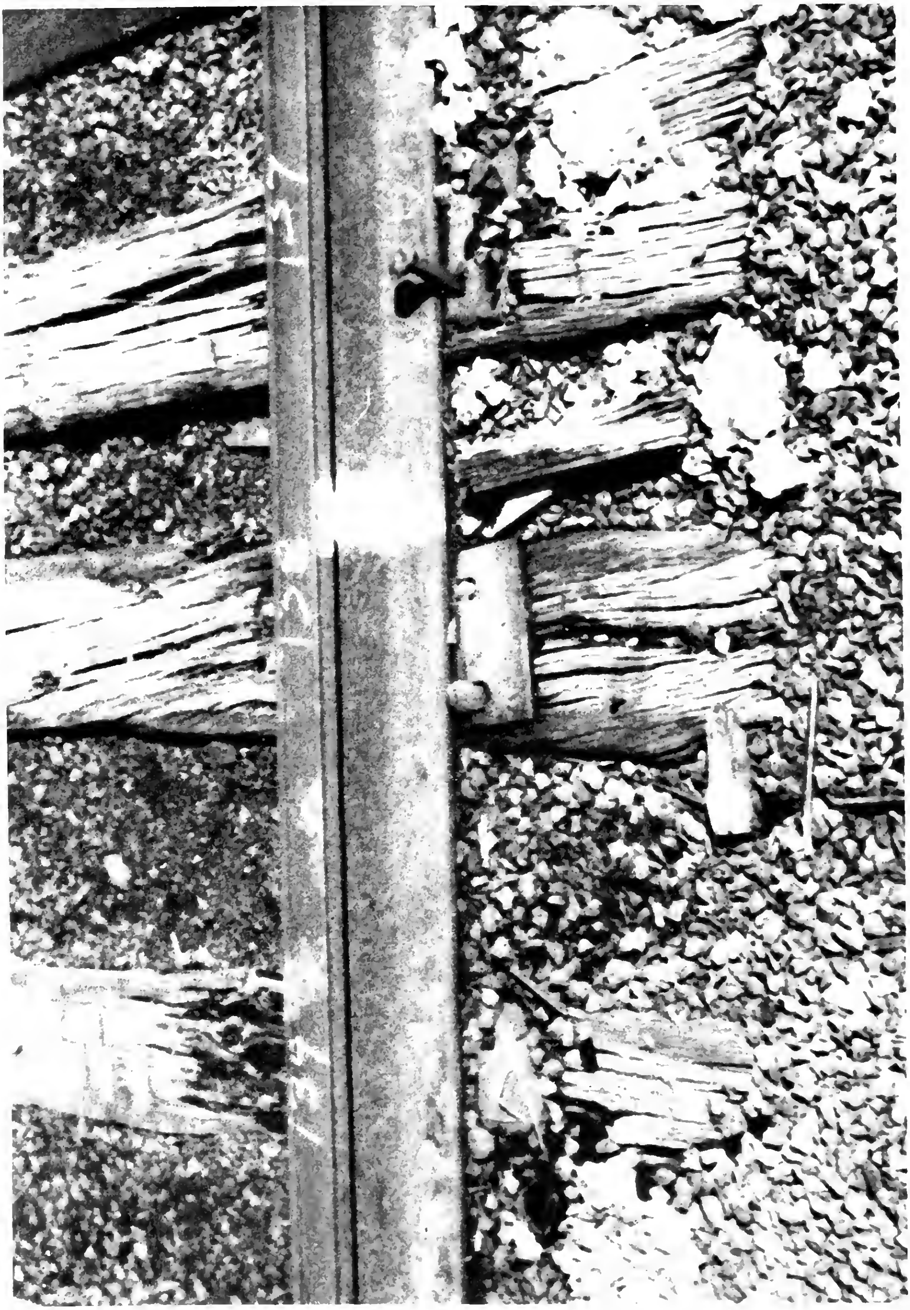


Figure 15 VIEW OF TIES NUMBER 138 and 139

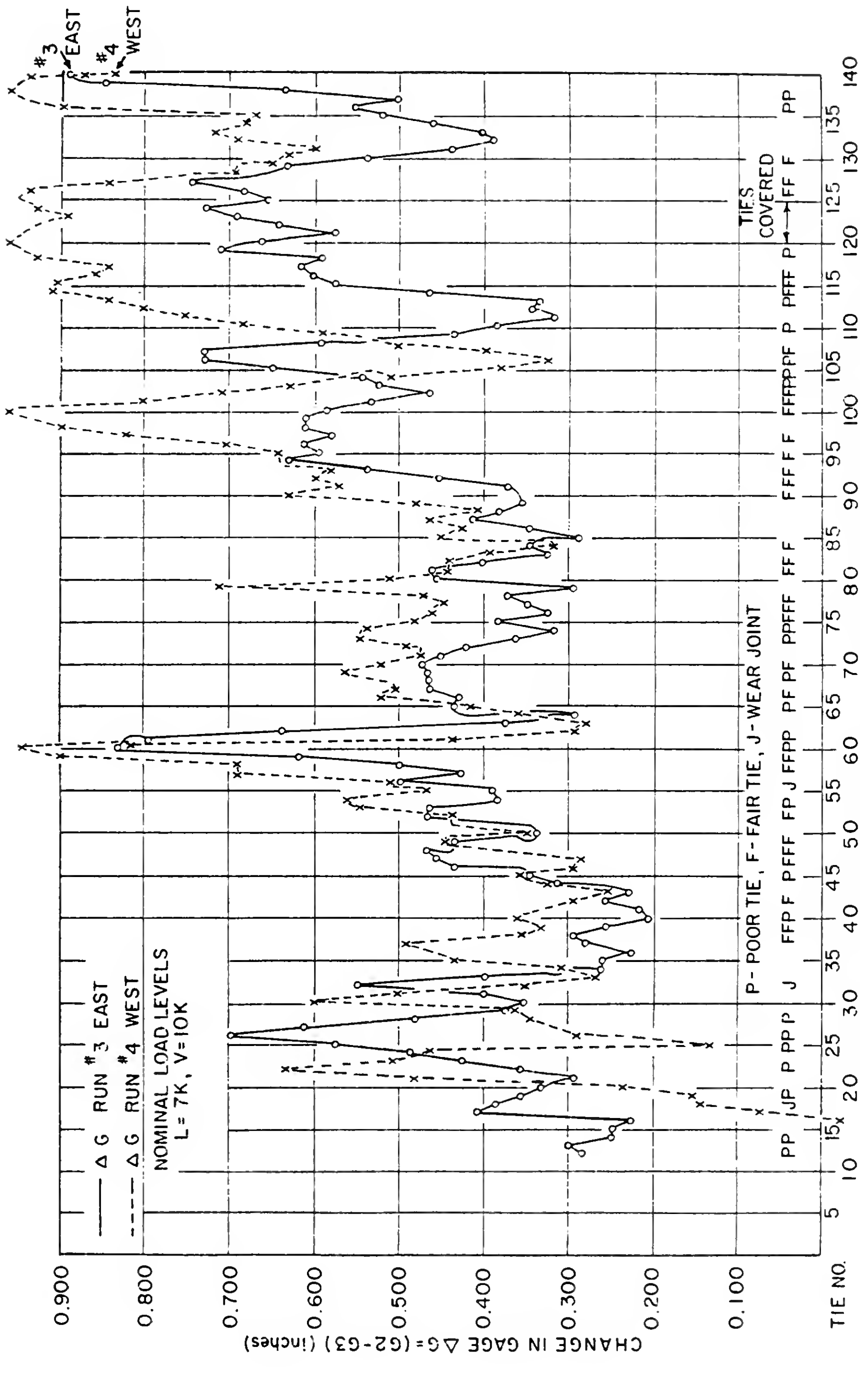


FIGURE 15 COMPARISON OF TEST RUNS IN OPPOSITE DIRECTIONS.

tion in the absolute magnitudes of the gage widening (ΔG) values and some consolidation of information, due to the increased vehicle speed, the ability to pin point weak spots in the track remains. This can be seen in Figure 17, which compares a test run made at 3 mph with a later test run made at 7 mph in the same direction. Furthermore, at the maximum speed (7 mph) the system exhibited no difficulties in following the rail and operating properly. It thus appears that the potential exists for ultimately conducting track strength field measurements at speeds compatible with other current track measurement systems.

Finally it should be noted that although dynamic gage widening of over 1 inch was observed, upon removal of the load, i.e. after the vehicle passed over the section of track, the gage returned to normal in almost all cases. In fact, as can be seen in Table 4, of a total of 40 independent gage measurements made using a steel tape at intervals of every 10 ties, the gage returned to normal, i.e. *no change* occurred, at 75% of the measurement points. This reflects the interval of time between the first set of tape measurements taken after runs number one and the last measurements taken after runs number 5, or 8 passes of the load vehicle. Of the remaining eleven measurements, ten showed changes in permanent gage of less than $^{1}/_{8}$ inch and only one out of 42 measurements showed a significant widening of the gage. Thus, while the gage did widen significantly under load (Figure 5), the gauge was found to have returned, in almost all cases, to the same or a reasonably close value even for repeated passes of the test vehicle.

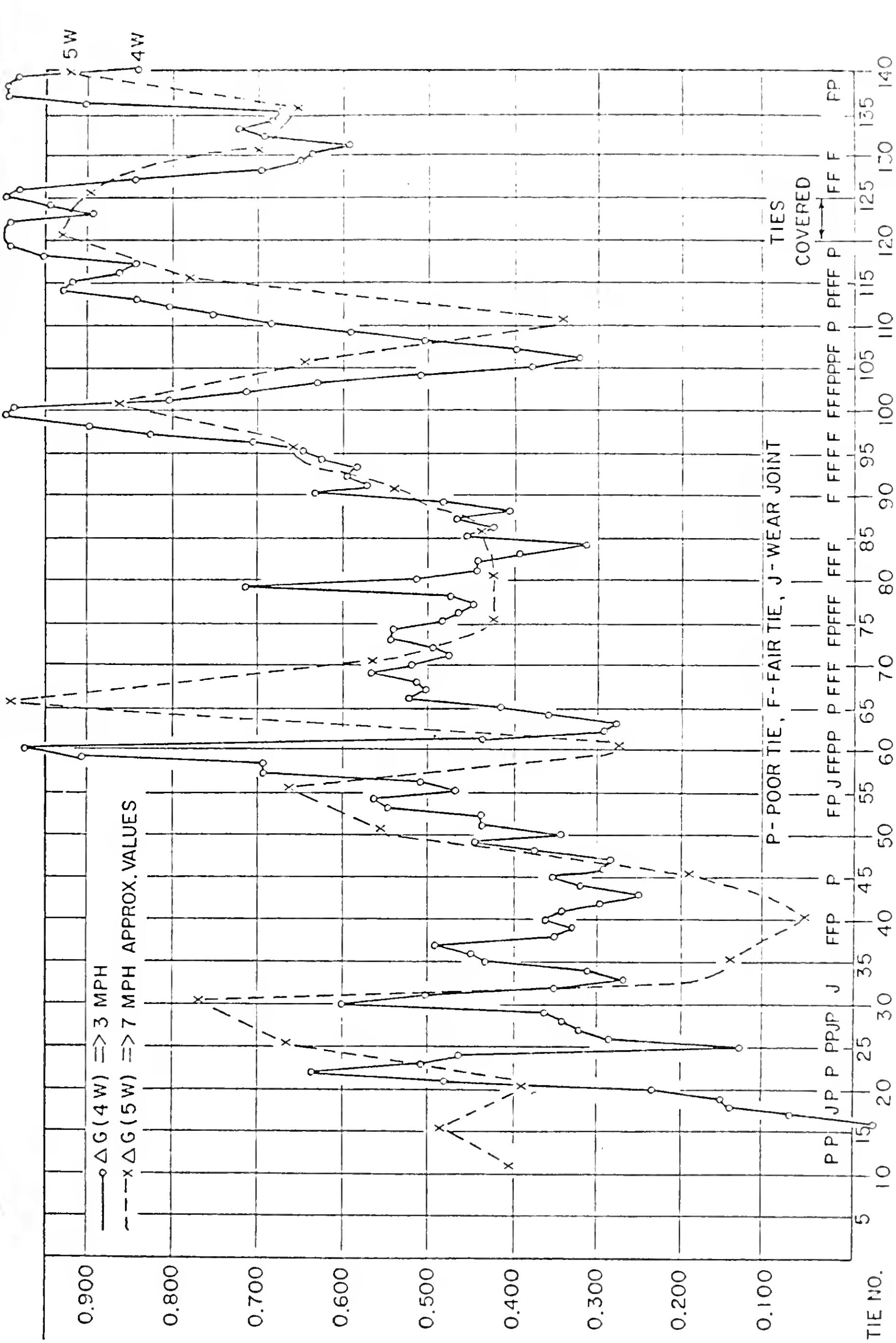
Conclusions

The results of these preliminary tests of the Track Strength Vehicle, the Decarotor, indicated that it appears to be possible to identify "weak" spots in the track structure through continuous measurement of loaded gage. The results appear to confirm and reinforce the results of the earlier Rail Spreader Tests which stated that "it appears possible to measure track strength and specifically locate and identify ties or fasteners in poor condition."[4] For track of yard quality, this quite clearly appears to be the case. It furthermore appears that these track strength measurements can be made at a reasonable test speed and without causing significant damage to the track structure.

In particular, the results of these tests show that the Track Strength Vehicle can (in yard track):

1. Detect weakened sections in the track.
2. Identify poor or weak joints.
3. Indicate locations of poor individual tie or fastener condition.
4. Repeatedly identify the same weakened sections (with a slight directional effect).
5. Provide a continuous loaded gage measurement at test speed.
6. Test without significant damage to the track structure.

It now remains to verify these test results in mainline quality track to see if the same levels of sensitivity and repeatability can be found in track where the variance between "good" and "bad" tie or fastener condition is much narrower than in the yard track tested to date. These tests, scheduled for spring 1980, will more fully indicate the versatility and potential usefulness of the continuous track strength measurement concept.



ΔG(4W) => 3 MPH
ΔG(5W) => 7 MPH APPROX. VALUES
P-POOR TIE, F-FAIR TIE, J-WEAR JOINT
TIES COVERED
5W
4W
TIE NO.
0.900
0.800
0.700
0.600
0.500
0.400
0.300
0.200
0.100

FIGURE 17

TABLE 4

NOMINAL GAGE MEASUREMENTS*

Tie No.**	Nominal Gage (Inches) (After Runs 1)	Nominal Gage (Inches) (After Runs 5)	Change In Gage (In.)
0	56 5/8	56 5/8	—
10	57	57	—
20	57	57	—
30	56 3/4	56 3/4	—
40	57 1/8	57 1/8	—
50	56 5/8	56 5/8	—
60	56 5/8	56 5/8	—
70	56 3/8	56 3/8	—
80	56 5/8	56 3/4	+ 1/8
90	56 5/8	56 5/8	—
100	56 3/4	56 3/8	—
110	56 3/8	56 1/2	+ 1/8
120	56 5/8	56 3/4	+ 1/8
130	56 1/2	56 1/2	—
140	56 1/4	56 3/8	+ 1/8
150	56 1/2	56 5/8	+ 1/8
160	56 1/4	56 3/4	+ 1/2
170	56 1/2	56 1/2	—
180	56 7/8	56 7/8	—
190	56 1/2	56 1/2	—
200	56 7/8	56 7/8	—
210	56 7/8	56 7/8	—
220	56 3/8	56 1/2	+ 1/8
230	56 1/2	56 1/2	—
240	56 7/8	56 7/8	—
250	56 1/2	56 5/8	+ 1/8
260	56 1/2	56 1/2	—
270	56 3/8	56 3/8	—
280	56 5/8	56 5/8	—
290	56 3/4	56 3/4	—
300	56	56 1/8	+ 1/8
310	56 1/2	56 1/2	—
320	56 3/8	56 3/8	—
330	56 3/4	56 3/4	—
340	56 1/2	56 5/8	+ 1/8
350	56 5/8	56 5/8	—
360	56 1/2	56 5/8	+ 1/8
370	56 1/2	56 1/2	—
380	56 1/2	56 1/2	—
390	56 3/4	56 3/4	—
400	56 5/8	56 5/8	—
410	56 3/4	56 3/4	—

*measurements taken by hand using a steel tape
**tie numbers increase in the west direction

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Field Evaluation of a Ballast-Subgrade Radar System

by W. So,[1] T. B. Hutcheson[2] and R. F. Breese[3]

Abstract

In order to develop a means of survey which will provide the necessary subsurface profiles and load response data to allow a realistic evaluation of existing railroad embankment and subgrade conditions, the Research and Test Department of the Association of American Railroads and the Cooperative Track Train Dynamics Program have undertaken a research effort to determine if ground penetrating radar will allow for extensive field survey and evaluation of existing track subgrades.

A field test was conducted at Natick, Massachusetts, on December 19 and 20, 1979, to evaluate a pulse radar system from Geophysical Survey Systems, Inc. The test results show that the system does locate and display to a usable scale the ballast-subgrade interface; it does give some indication of the moisture content in the lower ballast area and in the ballast-subgrade or subballast interface; it does locate and display areas of contrasting moisture content in the subgrade; it demonstrates that the data is repeatable over the same trackage and, therefore, is, with sufficient experience, capable of interpretation.

Successful use of the system in its present state of development will require, for railroad service, an operator having a high skill in its use, and one capable of bringing a high level of skill to an interpretation of the data. It may well be possible, through additional work with data processing techniques, to improve the objectivity of the data interpretation.

Introduction

As part of the ongoing Track Research Program, the Research and Test Department, Association of American Railroads (AAR), and the Cooperative Government Industry Track Train Dynamics Program (TTD) have been interested in characterizing and quantifying the relative strengths of track components in existing track structures. An important aspect of this research is the development of non-destructive test methodologies which will assist railroads in their inspection of existing track to detect potential failure conditions in the track structure and take remedial action before the condition reaches a critical level.

An important factor in track stability is the condition existing in the ballast, subballast and subgrade. Changing conditions of axle loads, speeds, train lengths and weights have, for many existing rail lines, increased the loadings experienced by ballast and subgrade sections, resulting in increased difficulty in maintaining an acceptable level of track alignment and surface, even on long established and well compacted embankment and cut sections.

Older existing subgrades were traditionally constructed from natural materials found along the right of way, using a minimum of haul consistent with the grading equipment then available, except at locations where these materials were of such low strength characteristics as to render them completely unusable. In such cases, those materials were wasted, and more suitable nearby materials were borrowed and placed in the subgrade.

This procedure naturally produced widely varying conditions of subgrade strengths and placed, or left within the subgrade, materials having a variety of conditions of compaction, moisture content and cohesiveness. Because of these widely varying subgrade conditions, it

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is not possible to work with a concept of "average" subgrade condition. It becomes desirable, therefore, to determine with some degree of accuracy the conditions of moisture, including the location and size of water pockets, of ballast cementation and fouling, and the location of cohesive materials existing in the subgrade and along the ballast-subgrade interface.

Soil borings, the traditional means of subgrade exploration, are expensive, time consuming and in addition, generally interfere seriously with train operations. Thus, this means of investigation is not easily incorporated into maintenance programs.

Ground penetrating radar has been used for a number of years in a wide variety of geotechnical applications. At least two research efforts are presently under way to determine if this newer technology will allow for extensive field survey and evaluation of existing track subgrades by non-destructive means in a framework of time and cost which will allow for their incorporation regularly into track maintenance programs. The goal is to develop a means of survey which will provide the necessary subsurface profiles and load response data to allow a realistic evaluation of existing embankment and subgrade conditions.

In one such effort, the Federal Railroad Administration has commissioned the Transportation Systems Center (TSC) in Cambridge, Massachusetts, to conduct a program of accumulation of data and experience on railroad foundation problems, with the ultimate goal of improving the railroad track safety record. As part of this program, TSC has engaged the Waterways Experiment Station, United States Army Corps of Engineers, Vicksburg, Mississippi, to construct and test a swept frequency radar system capable of collecting reflectivity data on railroad embankments and capable of operating from a position immediately above the embankment surface. This project is now getting well under way. Arrangements have been made between the Track Research Division of AAR and TSC to coordinate the research activities in this area.

A second effort, and the one to be discussed here, is work under way by Geophysical Survey Systems, Inc., (GSSI) of Hudson, New Hampshire, to develop a ground penetrating pulse radar system which may be moved along a railroad right of way at test speeds and which would allow the equipment to be incorporated into regular track inspection programs. Such a device would provide a means of inspection for anomalies in the ballast-subgrade interface and in the subgrade itself. Speeds of operation varying from walking speed to 40 miles per hour have been indicated by GSSI to be within the capabilities of the equipment.

In the fall of 1979, the Track Research Division of AAR contacted GSSI to determine the state of development of its ground penetrating radar system and the extent of its experience with the system in subgrade investigations. This contact resulted in arrangments to visit GSSI at its headquarters.

After initially viewing the equipment in operation by GSSI at a location near its plant in New Hampshire in November of 1979, arrangments were concluded with GSSI and with the Consolidated Rail Corporation (Conrail) to conduct a field test of the equipment under rail service conditions. The tests were conducted on a section of Conrail's Northeast Region, near Natick, Massachusetts, on December 19 and 20, 1979, under the auspices of TTD.

The test program was under the general direction of Dr. Allan M. Zarembski, Manager of AAR Track Research Division and Deputy Director of TTD. Mr. Russell A. Abbott, Assistant Manager of Track Rehabilitation Planning, represented Conrail during the test and was assisted by Mr. Robert Sutton, Assistant Division Engineer for Conrail. Mr. William S. Stokely, Roadway Engineer for Illinois Central Gulf Railroad Company, Mr. Thomas B. Hutcheson, AAR Consultant, and Dr. R.F. Breese, Senior Research Engineer (Electrical) of AAR, assisted in the field evaluations of the radar equipment. Dr. W. Tony So, Senior Research Engineer, Track Research Division of AAR, was Project Manager for these studies.

Ground-Penetrating Radar Applications

As previously stated, ground penetrating radar systems have already been used in a wide variety of geotechnical applications.

The following selected references have been summarized and listed in their approximate chronological order of development, in order to illustrate some typical radar system applications.

In 1960, Cook [3] proposed the use of a monocycle-pulse VHF radar system for air-borne snow and ice measurements. The methodology was later extended to include fast air-borne determinations of the thickness of layered, but radar transparent, materials, such as floating fresh-water ice and the polar continental ice sheets.

In 1973, Calspan Corporation [2] announced the development of a portable radar system capable of penetrating concrete and several feet of soil. Potential uses included the detection of buried objects and the monitoring of highway subsurface conditions.

In 1974, Birchak, et al. [1] reported an investigation of two designs of microwave sensors, to be used as buried soil moisture sensors in the long-term monitoring of highway subgrade moisture conditions.

In 1974, Dolphin, et al. [6] succesfully tested, in a California dolomite mine, a radar system to be used in Egyptian archaeological explorations.

In 1974, Porcello [10] discussed the Apollo Lunar Sounder Radar System that was used to detect subsurface lunar discontinuities, and to generate lunar surface profiles and images.

In 1975, Cook [4] measured the relative transparencies, to radar signals, of a variety of rock materials, encountered in mining, tunneling and various engineering works. The test results predicted the low-loss propagation of radar signals through certain granites, limestones, coals and dry concretes.

In 1976, Stewart and Unterberger [11] used a VEHF radar system in a salt mine to probe horizontally for the dome flank, and vertically to locate the top of the salt deposit.

In 1976, Moffatt and Puskar [9] used a pulsed radar system to investigate various subsurface geologies, including faults, joints, cavities, and lithographic contrasts in soft rock materials.

In 1977, Cook [5] reported the results of borehold radar tests in coal seams. This technique of being able to explore coal seams by means of boreholes from the surface, has the potential for being of great importance.

In 1977, both ground-penetrating radar and soil resistivity measurements [7] were used to support a limited exploration and excavation of Victoria Peak in New Mexico. The survey confirmed the existence of large caverns, and various tunnels and fissures.

An FM-CW microwave system was investigated by Ellerbruch and Belsher in 1978 [8] for measuring coal seam thicknesses. The results showed that, in most cases, the thicknesses could be determined.

Fowler of Ensco, Inc., presented at the 1979 SME-AIME Fall Meeting the results of field experiments using a ground-penetrating radar within coal mines.

GSSI conducted a radar survey of a section of railroad track in Nashua, New Hampshire, in September of 1979. However, no excavations were made to confirm the radar measurements.

Description of Equipment

The basic principles involved in a ground penetrating radar system are relatively simple. The system periodically generates very high intensity, high frequency, short time duration, electro-magnetic signals, called transmit pulses, which are directed downwards into the ground. The speed at which they move is determined by the dielectric constant (related to the moisture content) of the various soil materials, and the depth of penetration of the signals is determined by the overall electrical conductivity of the soil. Each time a transmit pulse encounters a change in dielectric constant, such as at a discontinuity (e.g., a buried pipe), or change in moisture content, its downward speed is changed accordingly, and an echo pulse is generated at the interface. Under favorable geometrical conditions, these echo pulses return to the surface, where they are detected by the receiving antenna and associated electronic circuitry. The greater the depth at which the echo-generating interface is located, the longer it takes for the echo pulses to return to the surface. If a very accurate time clock can be thought of as starting at time zero, at the instant each transmit pulse is generated, then the progressive series of returning echo pulses, differing from each other only in the time lag with respect to time zero, become direct indicators of depth below the surface. Since the average velocities of radar pulses in various types of soil are known, this type of system can estimate depth to about twenty percent accuracy. If boreholes are used to determine the exact distances of various strata below the ground surface, the radar system can be calibrated to one percent accuracy.

The GSSI radar system consists of a D.C. power supply, a transmit/receive antenna assembly, a radar return-signal monitoring unit and an "EPC Graphics" spark gap recording system (Figure 1). On the output chart from this recorder, the vertical axis is proportional to depth in the ground, and the horizontal axis is proportional to time, or distance along the surface, when the antenna assembly is moving. An optional magnetic data tape recorder can also be used, if desired, to record the processed return signals for subsequent laboratory playback and analysis.

This radar system is unique, because it does not use conventional microwave carrier signal generation or modulation techniques. Instead, it generates the transmitted radar output signal by applying a very sharp rise-time square wave to the transmitting antenna, at a repetition frequency of 50 KHz. The antenna responds to the leading edge of the applied square wave by resonating or "ringing," but certain circuit parameters have been carefully selected, so that the antenna's oscillatory output is highly (but not critically) damped. The resulting output is approximately three cycles of a rapidly-decaying sinusoid, at some nominal center frequency, f_o. The antenna has a relatively broad band output, ranging from $\frac{1}{4} f_o$ to $4 f_o$, with the one-half power points at approximately $\frac{1}{2} f_o$ and $2 f_o$, respectively. The peak power is about 15 watts, with an average power of only seven milliwatts, due to the low duty cycle (ratio of "on" to "off" times).

The antenna assembly consists of a pair of properly-scaled, dimensioned and geometrically-oriented "bow-tie" antennas; one for transmitting the generated radar "pulses," and the other for receiving the much-weaker ground reflected "pulse echo" signals. Each antenna is covered by a suitable metallic reflector, in order to transmit and receive signals in the downward direction only. This results in a total beam width of 16 inches, with a total angular spread (from the vertical) of about 45 degrees. The entire assembly is contained in a rectangular fiber glass housing, with the antennas and associated equipment encapsulated in a plastic foam material, which provides non-metallic structural rigidity and light weight (figure 2).

Special data-sampling circuits, located in the antenna assembly, selectively sample the return echo signal amplitudes, and develop reconstructed return signal waveforms, with greatly-stretched time scales. Although the time windows for the reconstructed waveforms can

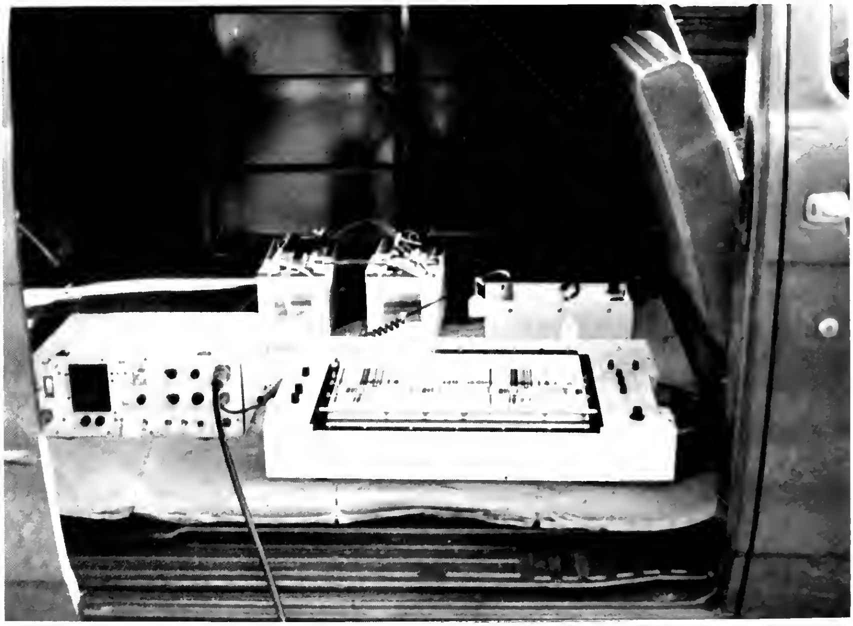

Figure 1.—Radar Test Equipment

Figure 2.—Radar Antenna Assembly

be varied from 24 (minumum) to 800 (maximum) nanoseconds, they are always composed of 2000 discrete sampling points. Since the reconstructed waveforms are in the audio frequency range, they can be graphically recorded.

The system is easily calibrated by temporarily replacing the antenna with a small, solid-state, ten nanosecond time pulse generator. Since a typical radar signal velocity (in the ground) is about 5-6 nanoseconds per ft, each ten nanosecond pulse represents a 2 ft depth.

The maximum usable ground-penetrating depth, and corresponding signal resolution, are both related to the system's center frequency, f_0. In general, the higher the center frequency, the greater the resolution, but the lower the penetration depth.

GSSI has designed and constructed antennas for use at 900, 300, 120, and 80 MHz, with corresponding usable penetration depths of 3, 30 to 50 and 200 ft, respectively. The antenna at 300 MHz is most suitable for railroad usage, because of both its penetration depth and resolution capabilities, and was consequently used in this field test. Subsequent to the field test, GSSI constructed another antenna for use at 600 MHz.

Radar Field Test

In order to evaluate the GSSI radar unit (Type SIR System 7) with 300 MHz antenna, a field test was conducted on a stretch of main line track of Conrail near Natick, Massachusetts. The general condition of the track was good to excellent. However, some stretches of the track had free water standing in ditches along the sides of the track (Figure 3). No problem spots were readily evident. The construction of the track was 127 lb welded rail with wood ties and trap rock ballast.

The radar unit was first towed eastwards behind a hy-rail vehicle (Figure 4) from mile-post 21 to mile-post 15 to obtain a general survey of the track. The speed of the vehicle was 3 to 4 mph. Portions of the track were then re-surveyed with the radar unit to ensure that the radar measurements were repeatable and the results confirmed that. From the general survey, three areas of interest were selected (Figure 5). The radar measurements of these areas showed more irregular and darkened patterns which might indicate high moisture spots. The radar unit was hand-towed eastwards over these three sites to obtain more detailed readings (Figure 6).

The radar data were marked electrically at every 10th tie and the markings are shown as vertical lines at the top of the Figures 7, 8 and 9. The radar readings of all areas showed a well defined layer at the top, which evidently indicated the ballast layer. Under the ballast, various layers of light to dark intensity were shown. A dark zone generally indicated high moisture content in the subgrade.

Site 1 started at the fifth tie east of the signal at mile 17.52 and the radar was pulled over 300 ties. From the radar data (Figure 7), a dark area and a light area were chosen to be excavated to confirm the amount of moisture in the subgrade. These were designated as Cross-Hole 1 and Cross-Hole 2, respectively.

Site 2 extended over 240 ties, starting at the 110th tie west of the overhead bridge at mile 17.16 and ending to the east of the bridge. Again, a dark area and a light area in the radar reading (Figure 8) were designated as Cross-Hole 1 and Cross-Hole 2, respectively.

Site 3 started at mile-post 16 and extended over 200 ties to the east. A dark area in the radar data (Figure 9) was chosen to be Cross-Hole 1.

The radar measurements taken across a previous track-bed adjacent to the existing track at the above sites showed several layers that formed pockets directly below the former rail locations (Figure 10). These evidently indicated the ballast pockets.

Figure 3.—Ditches Along the Side of Track Near Natick

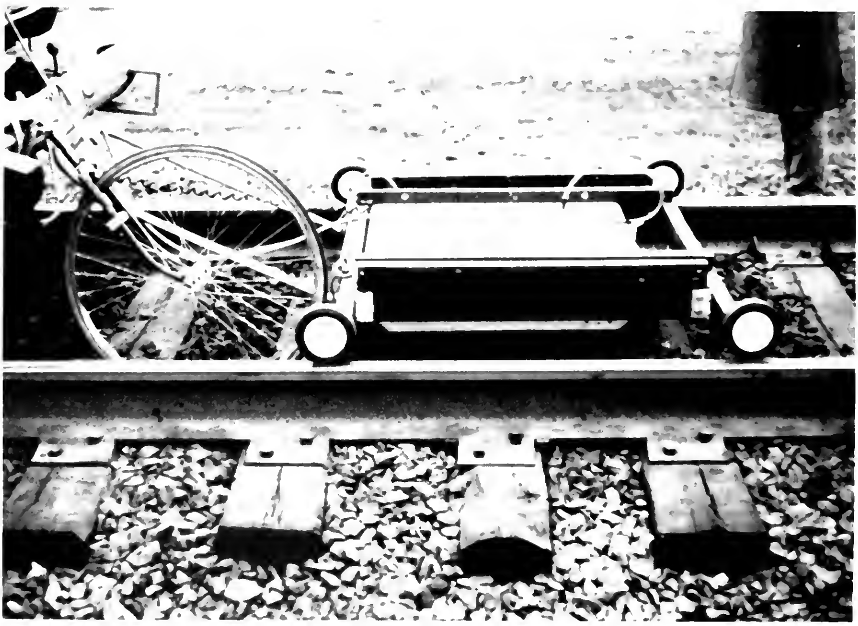

Figure 4.—Radar Antenna Assembly Towed Behind Hy-Rail Vehicle.

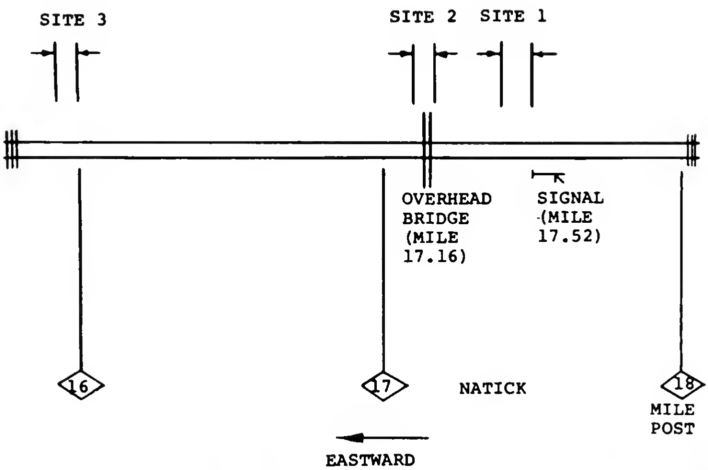

Figure 5.—Location of Test Sites for Radar Investigations

Figure 6.—Radar Antenna Assembly Towed Manually

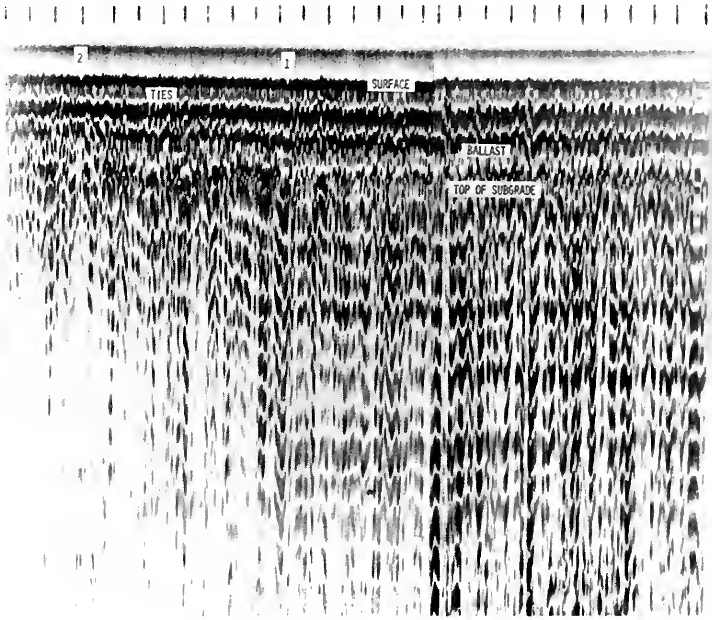

Figure 7.—Radar Ground Profile for Site 1

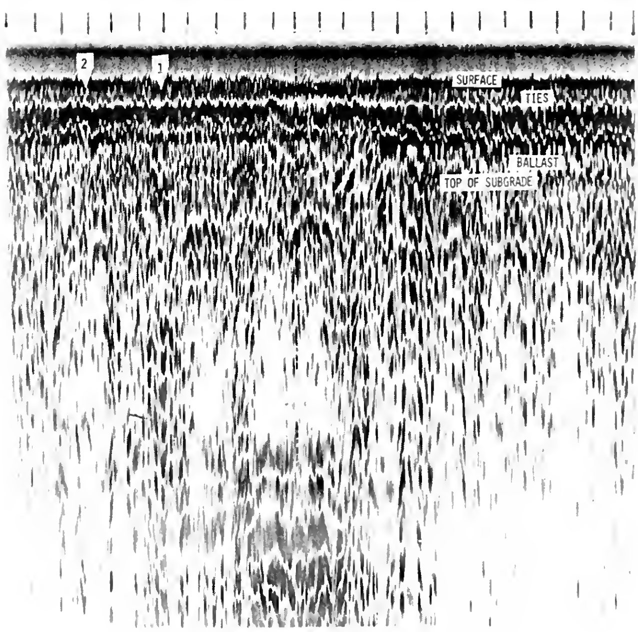

Figure 8.—Radar Ground Profile for Site 2

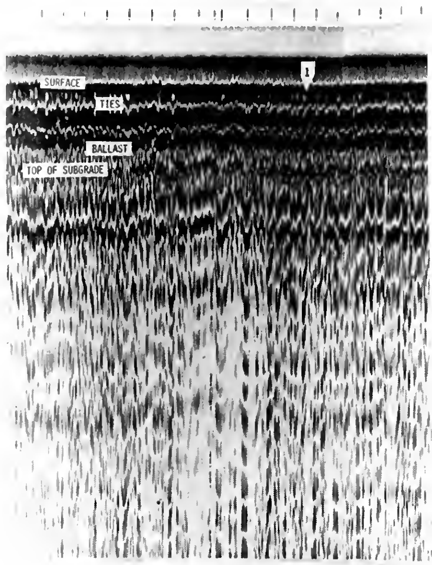

Figure 9.—Radar Ground Profile for Site 3

Figure 10.—Radar Ground Profile Across Previous Railroad Track Bed

Cross-Hole Excavations

Cross-holes were excavated with a back-hoe on the shoulders of the track on the sites to depths of about 4 ft to 5 ft and the soil structure was recorded for comparison with the radar measurements.

From the cross-holes, Site 1 was seen to have about 18″ of ballast which consisted of ½″ to 2″ trap rock in dirty conditions (Figures 11, 12, 13, and 14). Under the ballast was a sand layer about 1″ to 12″ thick with some free water. Beneath that was moist sandy silt. The radar reading (Figure 7) of Cross-Hole 1 indicated a dark zone while that of Cross-Hole 2 indicated a light zone under the ballast. Upon excavation, Cross-Hole 1 (Figures 11 and 12) did show a larger amount of free water than Cross-Hole 2 (Figures 13 and 14) in the sand layer under the ballast, but only slightly so. The sandy silt at the bottom of Cross-Hole 1 did contain higher moisture than that at Cross-Hole 2, but again only slightly so.

At Site 2, the cross-holes indicated about 12″ to 18″ of ballast consisting of ½″ to 2″ trap rock (Figures 15, 16, 17 and 18). Under that was a sand layer about 12″ thick and saturated with free water. Beneath was a rock ledge. The radar reading (Figure 8) of Cross-Hole 1 indicated a dark zone under the ballast while that of Cross-Hole 2 indicated a light zone. However, the excavations at both cross-holes showed saturation of free water.

The cross-hole at Site 3 showed about 15″ to 30″ of ballast consisting of ½″ to 2″ trap rock (Figures 19 and 20). Under the ballast was a sand layer about 6″ thick and containing some gravel. Beneath that was sand with some cobbles. The radar reading (Figure 9) at Cross-Hole 1 showed a dark zone under the ballast. Upon excavation, however, no saturation of water was found.

Conclusions

The equipment used and the test described in this report represent the first known documented use of a ground penetrating pulse radar system in the railroad environment. At best, it is an early beginning, and there remains much yet to be learned about its capabilities and use.

There also remains considerable work to be done as to the development of suitable equipment to move the radar antenna along the track in a manner which will avoid interference with its signal and thus provide a clear reflected signal. Incorporating the radar antenna and its support and recording system into standard inspection vehicles is another area requiring examination.

What is clearly seen from the results at the Natick test is that the system does locate and display to a usable scale the ballast-subgrade interface; it does give some indication of the moisture content in the lower ballast area and in the ballast-subgrade or subballast interface; it does locate and display areas of contrasting moisture content in the subgrade; it demonstrates that the data is repeatable over the same trackage and, therefore, is, with sufficient experience, capable of interpretation.

Successful use of the system in its present state of development will require, for railroad service, an operator having a high skill in its use, and one capable of bringing a high level of skill to an interpretation of the data display to produce information useful in maintenance planning.

It may well be possible, through additional work with data processing techniques, to improve the data interpretation, provided the demonstrated repeatability of the system is upheld in future tests. Additional work in this area holds the promise of successful future use of the system.

Figure 11.—Site 1, Cross-Hole 1

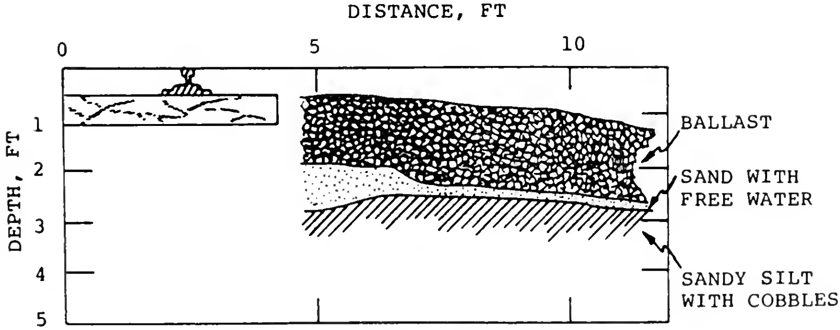

Figure 12.—Transverse Cross Section at Site 1, Cross-Hole 1

Figure 13.—Site 1, Cross-Hole 2

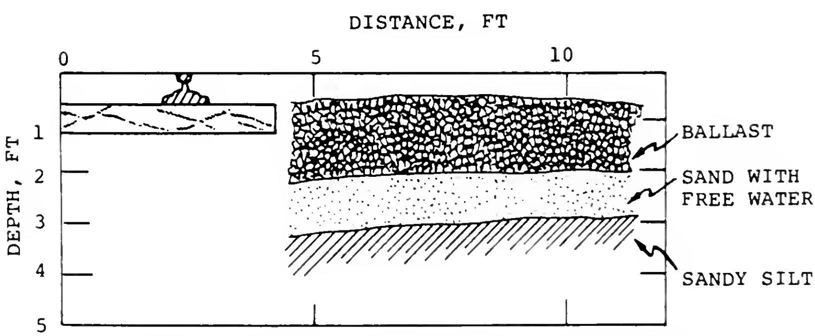

Figure 14.—Transverse Cross Section at Site 1, Cross-Hole 2

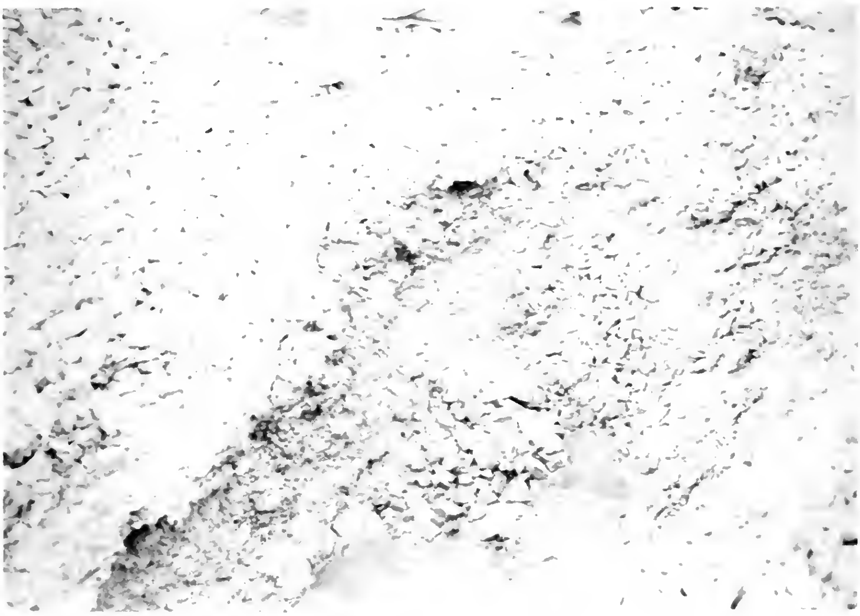

Figure 15.—Site 2, Cross-Hole 1

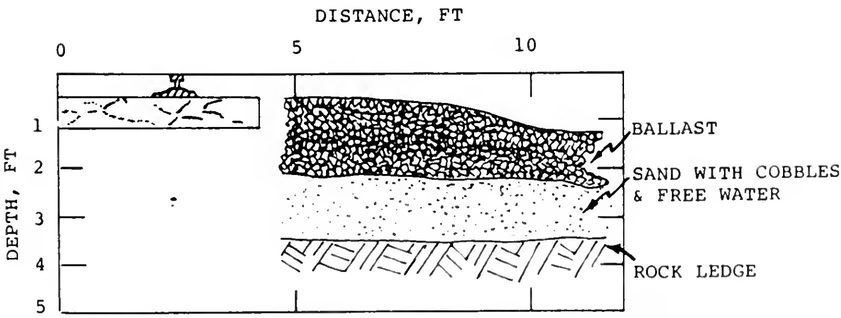

Figure 16.—Transverse Cross Section at Site 2, Cross-Hole 1

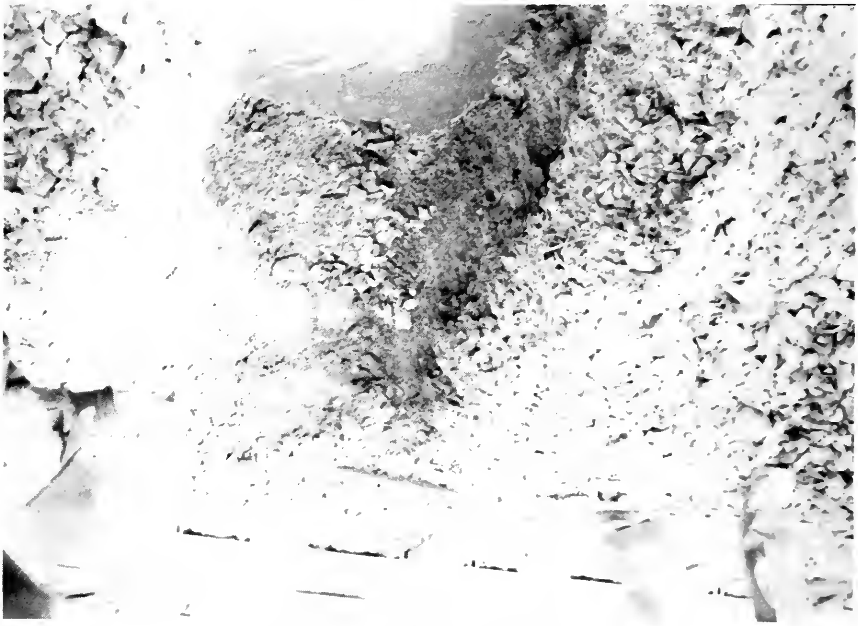

Figure 17.—Site 2, Cross-Hole 2

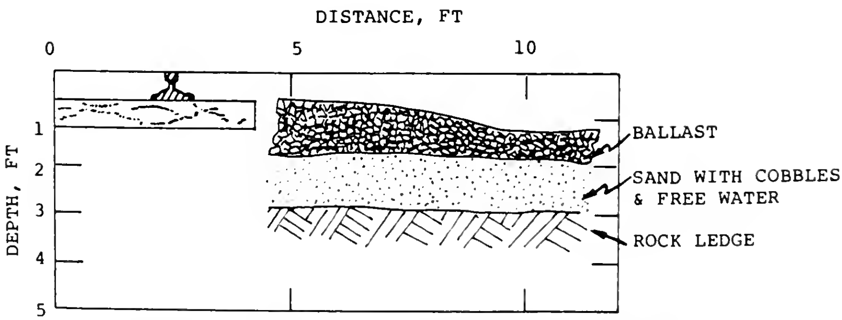

Figure 18.—Transverse Cross Section at Site 2, Cross-Hole 2

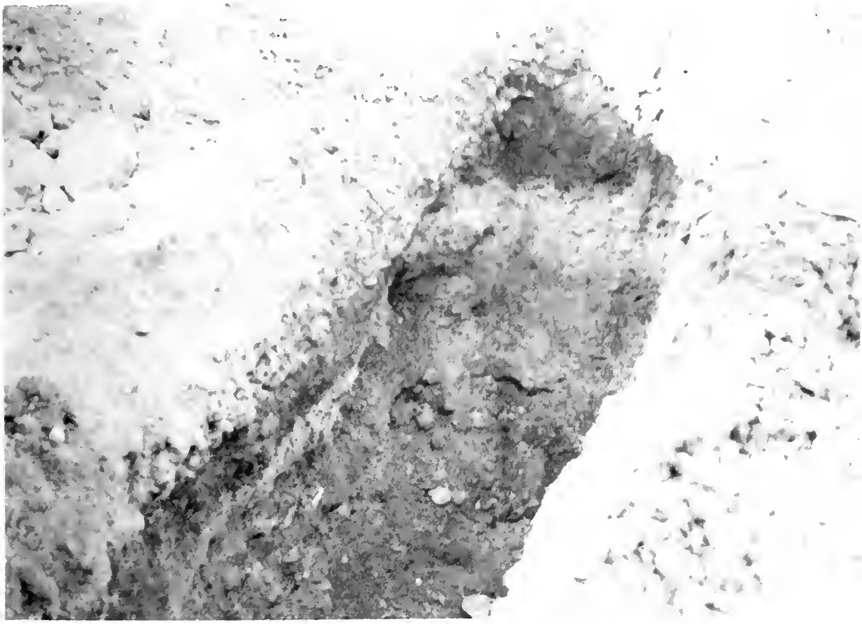

Figure 19.—Site 3, Cross-Hole 1

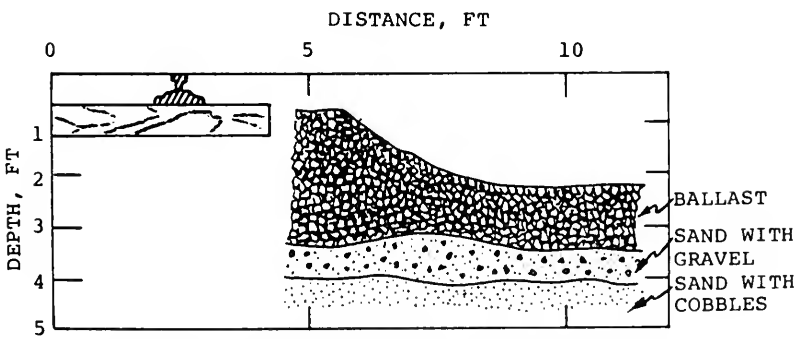

Figure 20.—Transverse Cross Section at Site 3, Cross-Hole 1

To achieve such a goal, it will be necessary to work with the system over a period of time on railroad subgrades having a variety of well defined and known anomalies. This would speed the development under railroad service conditions, and would increase the level of confidence and skill in the data interpretation.

With further development and testing, such systems do appear to have the capability of meeting the claims of various organizations developing or operating ground penetrating radar systems that they can map the ballast-subgrade interface; detect high moisture areas in the subgrade and crossties; and delineate soil layers and water pockets.

Successful systems which produce these inspection results at normal inspection speeds currently used, with vehicles such as inspection motor cars, inspection trucks, rail defect inspection vehicles and the like, would provide information badly needed in planning successful track maintenance and improvement programs.

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CONSTRUCTION OF NEW URBAN RAIL SYSTEMS

by Donald A. Shoff*

Introduction

Coincident with the evolution of engineering technology in the railroad industry has been a similar evolution, perhaps even a revolution, in a related field, the rail transit industry. From its beginnings before the turn of the century as horse-drawn transportation, the rail transit industry has developed into a number of electrified networks which have shaped the growth patterns of cities, business, industry, and individuals. It is hard to imagine how such cities as New York, Boston, Philadelphia and Chicago would have developed without the network of commuter railroads and rapid transit systems to provide for the movement of people in those areas. Some of the more recent engineering developments in the urban rail field will be discussed here this afternoon.

Types of Urban Rail Systems

Urban rail systems can be generally categorized as commuter railroad and light rail or heavy rail transit, and The American Public Transit Association applies the following definitions:

1. Commuter Railroad: That portion of "main-line railroad" transportation operations which encompasses urban passenger train service for local short-distance travel between a central city and adjacent suburbs; suburban rail passenger service, using both locomotive-hauled and self-propelled railroad passenger cars, is characterized by multi-trip tickets, specific station-to-station fares, railroad employment practices, and usually only one or two stations in the central business district.

At present, 17 railroads operate some form of commuter operation in 10 major markets.

2. Light Rail Transit: Streetcar-type transit vehicle railway constructed on city streets, semi-private right-of-way, and/or exclusive private right-of-way; formerly known as "streetcar" ("trolley car") and "subway-surface" depending upon local usage or preference.

Presently 13 cities including Toronto, Edmondton, Pittsburgh, Cleveland, Boston and San Francisco operate forms of light rail and more systems are being developed.

3. Heavy Rail Transit: Subway-type transit vehicle railway constructed on exclusive private right-of-way with high-level platform stations; formerly known as "subway" or "elevated (railway)". This concept has been familiar for years to inhabitants of Boston, New York City, Philadelphia and Chicago, and more recently to those in Washington, Atlanta, and the Bay area. Presently there are 13 such operations in the United States and Canada. Both this category and light rail, as opposed to the railroad commuter operations, are devoted primarily to the movement of passengers in a metropolitan area, and without exception are publicly owned and operated by an authority created to provide transit services.

4. Other: There are additional types of urban systems such as monorail, people movers in downtown and airport complexes, inclined planes, and the famous San Francisco cable cars. These forms of urban transportation, while important in their own right to the areas they serve, will not be discussed further here.

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Urban Transit Summary

To help appreciate the scope of activity in urban rail transit in this country, a table has been prepared (See Figure 1), which shows the single track mileage of transit main line by type of construction and by weight of rail. It also shows the quantity of turnouts and crossovers in service or in the construction phase. It becomes immediately apparent that the rail sections for main line track are generally lighter than in railroad main line service, with the newer properties tending toward the 115 pound section. Similarly, turnout sizes are generally smaller than in comparable railroad service, and the use of double crossovers in rail transit is not uncommon. These facts can be attributed to the much smaller radius curves (higher degree of curve) which can be negotiated by the transit vehicle and to the fact that in many instances, especially on existing properties, there is no room to install larger turnouts or two single crossovers. In the case of aerial or subway construction, the overall construction cost is greatly reduced where double crossovers are used.

Figure 2 shows some general engineering characteristics of transit properties, and it is obvious that there are considerable differences among the properties.

TRANSIT TRACK SUMMARY

Track Location and Structure	Miles of Single Track		
	In Operation	In Design or Construction	Planned
Subway or Tunnel			
Concrete Slab	378.9	68.7	11.2
Ballasted	257.3	0.6	—
Surface, Ballasted			
Concrete Ties	95.2	20.3	40.1
Wood Ties	255.9	41.2	54.0
Elevated			
Open Structure	270.3	0.4	—
Concrete Slab	56.2	56.5	32.8
Ballast on Concrete	27.0	—	—
TOTAL	1340.8	187.7	138.1
Rail by Weight and Joint Type			
85-lb. CWR	36.0	14.0	—
100-lb. CWR	123.6	2.1	—
112-lb. CWR	5.4	—	—
115-lb. CWR	104.4	174.4	138.1
119-lb. CWR	136.1	3.2	—
132-lb. CWR	22.3	—	—
85-lb. BJR	13.2	—	—
90-lb. BJR	71.6	—	—
100-lb. BJR	791.3	—	—
115-lb. BJR	12.8	0.3	—
119-lb. BJR	8.0	—	—
130-lb. BJR	2.0	—	—

Turnouts					Crossovers				
	In Operation		In Design or Construction			In Operation		Construction	
Size	Line	Yard	Line	Yard	Size	Line	Yard	Line	Yard
3.5	6	45	—	—	4S	23	22	—	—
4	17	329	—	—	5S	32	12	—	—
5	34	463	—	1	5D	19	8	—	—
6	126	144	—	135	6S	113	36	—	—
7	40	14	—	—	6D	63	17	—	3
8	89	195	33	66	7S	30	—	—	—
9	7	2	—	—	7D	6	—	—	—
10	166	5	5	2	8S	93	9	14	6
11	5	1	—	—	8D	41	1	5	—
12	18	2	—	—	9S	24	—	—	—
15	21	—	2	—	10S	25	—	22	—
20	10	—	2	—	10D	21	—	8	—
SPE	47	284	—	—	11S	4	—	—	—
					12S	4	—	—	—
					12D	1	—	—	—
					15S	4	—	—	—
					20D	1	—	—	—
					SPE.S	89	17	—	—
					SPE.D	63	20	—	—

Figure 1

ENGINEERING CHARACTERISTICS

ITEM	RANGE
Grade, maximum	3 to 5 percent
Curve, Horizontal, minimum	115 to 1000—foot radius
Curve, Vertical, minimum length	100 to 800 feet
Superelevation, balanced, maximum	4.5 inches
Superelevation, unbalanced, maximum	1.75 inches
Lubrication in curves	None to all curves to 1500—foot radius
Rail Cant	1 in 40 to 1 in 20
Gage on Tangent	56¼ to 66 inches ±⅛ inch
Gage Widening on curves	½ to 1 inch ±⅛ inch
Wood Tie Spacing, open deck brides	
deck bridges	15 to 18 inches
surface	24 to 30 inches
subways and tunnels	22½ to 24 inches
Concrete Tie Spacing	30 inches
Direct Fastener Spacing	26 to 36 inches
Welded Rail Joints	Tangent only to all track

OPERATING CHARACTERISTICS

ITEM	RANGE
Hours of operation	24 hours/day, 7 days/week, ultimate
Headways	2-4 minutes during peak periods
Control	Semi-automatic operation
Car size	75 × 10.5 feet
Seats per vehicle	75
Practical capacity	165
Maximum cars per train	6-8
Maximum speed	70-80 mph
Average speed (including station stops)	35-40 mph
Normal acceleration/deceleration	3.0 feet/sec./sec.

Figure 2

There is no one set of standards for the industry, such as a standard gauge. Each older property developed its own set of standards throughout the years to the point that these standards still dictate engineering policies. Although the newer properties are somewhat more comparable in their standards, there are still some major differences, an example being the 5′-6″ track gauge used on BART.

The lower portion of Figure 2, indicates a range of operating characteristics that the newer properties are adopting. They do not apply to every system, rather they represent somewhat of a composite of characteristics.

Transit Construction

It must be stated by way of introduction to this subject that each operating property has plans and funding applications to make additions and improvements ranging from line extensions and station renovations to the acquisition of new vehicles and service improvements. In addition, many cities or metropolitan areas which do not now have rail transit services are in various stages of planning and design for such systems. Time constraints do not permit discussion of all of these topics, so this discussion will be limited to those new transit systems which have recently been opened for service, are still under construction, or those existing systems which have some major improvement under way.

Newly Constructed Systems

1. The first of the new generation of rail transit lines to enter service in the United States was the Lindenwold high speed line operated by the Delaware River Port Authority (PATCO) serving Philadelphia and the New Jersey suburbs to the southeast. By utilizing an old existing subway in downtown and an existing bridge across the Delaware River into New Jersey, much of the initial heavy capital construction work was avoided. The outer portion of this line utilized railroad right-of-way when possible, and was constructed with conventional railroad type materials, 132# rail and timber ties, on surface and with direct fixation fasteners on the aerial section. The yard and shop facility is located at the present outer terminus of the line. The line is 15 miles long, and several plans for extensions are in the making.

2. The opening of the Bay Area Rapid Transit (BART) in 1972, introduced new levels of vehicle and systems technology and aesthetic quality to the transit industry. Portions of the ballasted track are constructed with concrete ties and the aerial and subway sections are constructed generally with specially designed direct fixation fasteners set on a "second pour" of concrete. The track gauge is 5′-6″ and rail is 119# CWR. Shortly a new contract to finish two Oakland stations and to place track in a tunnel which was built as part of the first construction phase will be completed. The line is approximately 75 miles in an "X" shape with the center being in downtown Oakland.

3. With the Bicentennial celebration came the first segment of the Washington D.C. Metro (WMATA). Subsequent extensions included the Red Line to Silver Spring. Md., the Blue Line between National Airport and R.F. Kennedy Stadium, and the Orange Line between New Carrolton, Md. and Ballston, Va. The Blue Line, now virtually complete, will be opened for service to Addison Road shortly, and work continues south of National Airport and north of Dupont Circle. The entire system will cover approximately 100 route miles. Construction consists of subway through downtown and aerial and ballasted in the outlying areas. 115# CWR is used in conjunction with timber ties or direct fixation fasteners which are set on the basic structure by means of grout pads.

4. The latest city to enjoy the services of heavy rail transit is Atlanta, Georgia. The first two segments have been opened to service in the past year, and under construction is a system comprising an east-west line and a north-south line, intersecting under downtown Atlanta. Each of the legs will branch outside of the central area to increase coverage. About 65% of the network parallels existing railroad rights-of-way to minimize neighborhood disruption and strengthen existing transportation corridors. Notable exceptions are in downtown Atlanta and east of the city in Decatur. In the central city, one line tunnels under Peachtree Street, providing access to almost every major educational, recreational, and employment center.

By late 1981, MARTA projects the 14-mile Phase A segment of the system to be carrying 110,000 passengers daily. With a full 53-mile system with 39 stations operating by 1990, patronage in Atlanta has been set at 500,000.

Concrete cross ties are used in the ballasted sections, about half of the total system, and an aerial and subway sections, direct fixation fasteners similar to those used in BART and Washington are used. One notable exception to this criteria is the testing of a resiliently-supported two block tie system in two 1200 foot sections. This French concept is new to this country, and its prime potential advantage is to alleviate noise and vibration. Expanded use of this concept will depend on the results of this testing.

5. Montreal, Quebec's metro began carrying its first revenue passengers in October 1966. By early 1967, a network of three lines was complete.

Since 1967, the east-west line Number 1 has been extended twice in June 1976, and in September 1978. A four-station extension of Line Number 2, west of the Bonaventure Terminal, is in service. The full extension program for Line Number 2 includes 15 new stations over nearly 10 miles of track. Excavation is under way for about half of the route with planning and design activities being conducted on portions of the remainder.

At present, construction has begun on a short segment of Line Number 5, a wholly new route, and design work is nearing completion for much of the rest. With about 23 miles and 43 stations in service now, implementation of the expansion program will add over 27 new miles and 52 stations.

To serve the program of expansion, 423 new rail cars have been purchased. Like their earlier counterparts, they will run on rubber tires. Moreover, signaling, central control, supervisory control, and trainphones have been revamped completely throughout the existing system. The entire extension project is estimated to cost $1.6 billion (Canadian dollars), with almost $700 million (Canadian dollars) worth of contracts awarded so far.

6. The infant of light rail operations in North America is Edmonton, Alberta, with its first revenue passengers carried in April 1978. In about three and one-half years since the first earth was turned, Edmonton residents were riding in and out of downtown on a 4.5 mile rail transit line.

Most of those residents will be traveling to Edmonton's core, where the line begins in subway. With one mile and two stations downtown, the cars then surface and travel 3.5 miles within the existing Canadian National Railways right-of-way, between a service track on the north side and the main line on the south side. A second service track on the south side completes the five-track layout.

A number of additional lines are being investigated for potential development at the present time, with preliminary results of these studies expected by the end of this year.

These systems discussed so far are operating at least part of their lines. The balance of the systems in this section are under construction, but have not yet started operation.

7. In Buffalo, N.Y., tunnel construction commenced in late 1979 for a 6.5 mile light rail line, of which 1.3 miles will be surface and the balance underground. Six surface stations and eight subway stations will serve the projected 40 million riders annually. The single yard is downtown in and around the former Lackawanna Railroad Station.

The Main Street line will be underground from the SUNY South Campus terminal to Tupper Street where it will emerge onto a pedestrian mall at grade.

Trackwork will be standard gauge, T-rail or girder rail. Construction will be of the CWR-direct fixation type. Special track support systems, or floating slabs, designed to reduce the effects of noise and vibration, are planned in selected locations. The projected opening of this first line is mid 1984.

8. The casual observer in Miami would likely be unaware today that a major heavy rail system is now underway. However, many construction and procurement contracts have been awarded, including a multi million contract for the revenue vehicle, and much of the long-lead trackwork material.

The 20.5 mile, 20 station first stage of the heavy rail system, to be opened by 1983, will be mostly elevated (18 miles) with some sections at-grade (2.5 miles). The rail route will begin south of Miami and proceed northeasterly along the Florida East Coast Railway right-of-way, generally parallel to one of the county's main traffic arteries. The route proceeds to the central business district in Miami and then continues northwesterly through the county's second most populous city, Hialeah.

9. In Baltimore, the first leg of an ultimate spoke-shaped system is under construction. Revenue service is to commence in late 1982. The first leg is 7.5 miles long, of which 4¼ miles is underground, 2¼ miles is aerial, and the balance at grade. The cost is estimated at $768 million and to date the actual expenditures are "under budget". There are 9 stations planned, 6 underground and 3 aerial, each station platform being between the double main tracks and 450 feet long to accommodate the maximum 6 car trains.

Before digging of the two identical parallel tubes began, several important preliminary steps were taken. Underground utilities, such as gas, water and sewer lines, were relocated closer to the street surface so as not to interfere with eventual excavation. Then access shafts were dug from the street surface down into the earth, which will allow men and machinery to enter the work site.

Wooden bracing, or lagging, was placed along the walls of the shaft to support it and wooden decking is used to cover most of the shaft opening so that traffic can continue while tunneling proceeds below the street surface.

Depending on the location, tunneling operation takes place between 25 and 60 feet underground.

The aerial portion is being constructed with pre-cast girders, which are fabricated locally, transported to the site, and erected onto the piers. Pre-cast concrete sound barrier walls are used in selected locations. Both the aerial deck and tunnel invert are pre-formed to accommodate the trackwork installation.

10. By mid-1981, the San Diego area will have a new 15.9 mile, $86 million light-rail transit system between the Santa Fe Depot in Centre City, and the International Border with Mexico. About 14 miles will be built on existing railway right-of-way and about 2 miles of new tracks will be built in the streets in Centre City San Diego.

Construction of the line involves several important activities, including:

- Purchasing 108 miles of right-of-way of the San Diego and Arizona Eastern Railway.
- Upgrading about 14 miles of this right-of-way to make it suitable for high-speed LRT service.
- Building 11 passenger shelters along the right-of-way, with parking lots at 7 of them and convenient bus transfer and handicapped facilities at all of them.
- Laying down all new track for about two miles on 'C' Street and 12th. Avenue in Centre City San Diego.
- Buying and testing a fleet of 14 modern LRT cars that will be run in 2-car trains along the line.

11. Calgary, Alberta has completed an evaluation process that led to the selection of light rail to serve one of the city's most heavily traveled corridors. Although construction has barely begun, revenue service is targeted to begin during the summer of 1981.

A 10-year, $1.12 billion plan to curtail street congestion includes this $311 million light rail system. Hopes are that the system, in conjunction with an upgrading of key raods throughout the city, will reduce street congestion, particularly since the city is expected to have one million people by the year 2005.

In July 1977, implementation had begun with the purchase of 27 light rail vehicles, and the vehicles are starting to arrive. From a southern terminus, the 10 mile line follows a Canadian Pacific rail right-of-way for the greater portion of the run.

At a point outside of downtown, the cars will travel successively along a street in a protected arrangement, through a subway section, along one side of an arterial roadway, and through another tunnel, emerging on Seventh Avenue. Seventh Avenue, a major central district street, will be closed to private automobiles with the thoroughfare turned over to buses, emergency vehicles, and the light rail service. Five stations at approximately three-block intervals will be located on the street, providing access to intersections and to Calgary's overhead pedestrian system.

Construction on Existing Systems

The previous discussion has covered those systems which are newly completed or are still under construction. But amidst the groundbreakings and ribbon-cuttings, there are cities whose residents consider taking the subway or getting on the streetcar as routine. These cities are presently carrying out comprehensive programs of rail expansions and improvements which indicate a confidence in and dependence on the rail mode as the backbone of their urban transportation grids.

1. Toronto, Ontario remains a stronghold of rail. It is served by an integrated system of commuter, light, and heavy rail lines with a number of expansions under way.

In January 1978, the opening of the Spadina subway added 6.17 miles to metropolitan Toronto's 26 mile heavy rail system. It signalled the beginning of high-speed, high-capacity subway service between the region's northwest corridor and downtown.

Toronto's approach to rail has been of both an incremental and integrated nature. The existing network grew from 4.5 miles to over 32 in short segments. Subway construction has been under way in some part of the region for 30 consecutive years. At the same time, rail growth has been accompanied by policies which are designed to ensure use of the subway to its greatest effect, and, in turn, to ensure that rail supports other investments.

Construction work continues at both ends of the Bloor-Danforth heavy rail line.

On the east end of the line, 1.6 miles of route is being added, also the beginning of the planned Scarborough light rail line. At the west end, the subway is being extended approximately one mile at a cost of about $49 million (Canadian dollars).

Most of the $91 million (Canadian dollars) eastern extension will be underground, and, when completed the multilevel station will include parking space for 510 automobiles and kiss-and-ride lanes along with the Scarborough transfer terminal. Also scheduled for a 1980 opening date, the western extension will include a 1221 car parking lot and a kiss-and-ride facility.

The Scarborough line will run from an extension of the east-west Bloor-Danforth subway, north on Ontario Hydro Electric Power Commission land and unused Canadian National Railway right-of-way, into the town center. From there it will continue a short distance to a large park-and-ride lot, a total of 4.4 miles.

The first construction contracts have been awarded for the $108.7 million (Canadian dollars) project with an opening date sometime in 1982.

2. Boston, Mass., finds itself in the unique position of operating both the nation's oldest subway and its newest rail extensions. The trains first began running under that city's streets in 1897, and over the years they have continued to run farther and farther out into the metropolitan region.

Several years ago, the Massachusetts Bay Transportation Authority's Red Line extended service into the coastal Quincy area. Construction has pushed the line 3.2 miles farther south into Braintree. Track construction included a two-block concrete cross tie in ballast which to date has been utilized only here and in Chicago.

Improvements on the Orange Line, however, provide perhaps the most dramatic illustration of rail's advantages and versatility. On its southern leg, the Orange Line is involved in a four-mile wholesale relocation. 70-year old elevated tracks over busy Washington Street will be removed as the stretch of rail is diverted to a nearby railroad right-of-way. The improvement will open the corridor to a variety of neighborhood improvement projects as well as open new transportation opportunities to local residents. The line will share right-of-way, but not tracks with Amtrak and commuter rail operations.

Other improvements include the addition of 175 new light rail cars to operate on the 100 mile system, the first of which were placed in service during the severe winter of 1978-79.

3. The nation's rail transit capital, New York, N.Y., is continuing its massive rail expansion and capital rehabilitation program. The fact that this program continues in spite of the city's well-publicized financial woes is ample proof of the priority assigned to rail transportation in this region.

The most important single rail project is the 63rd. Street-Queens Line, which is being built to connect existing subway lines in Manhattan on Sixth and Seventh Avenues with the borough of Queens to the east.

The new 63rd. Street Line is deliberately designed to parallel the existing 53rd. Street route, thereby alleviating overcrowding and strengthening the preeminence of rail transit in serving this corridor. In addition, a station will be provided at Roosevelt Island located in the middle of the East River between Manhattan and Queens. This island is the site for the development of a massive new residential community of approximately 15,000 dwelling units or 45,000 people. Actual completion of this development has been hindered by the fact that its only direct connection to Manhattan has been by aerial tramway, a delightfully picturesque mode, but one lacking the capacity or convenient distribution of the new subway line whose opening should ensure Roosevelt Island's completion.

Work is also continuing on another key piece of the Queens subway expansion, the Archer Avenue subway. This line will provide passengers with a choice of either the Jamaica Avenue BMT or the Queens Boulevard IND service, which should help to balance loadings by diverting passengers for downtown Manhattan to the less heavily used BMT line. It also will provide convenient transfer access to the nearby Jamaica station of the Long Island Rail Road, the national's busiest commuter railroad, through which all but one of its branches operate.

4. Pittsburgh is one of a few cities in the United States to retain trolley service. What remains today are 23 miles of trolley lines operating in South Hills through a mixture of working class and upper middle income communities.

In 1976, the Port Authority of Allegheny County turned to light rail—recommended by a comprehensive comparative analysis—as most suitable for South Hills, economically and pragmatically.

This work pertains to stage one of the project, which includes complete reconstruction of 10.5 miles of existing lines, renewal of electrification and signal facilities governing all 23 miles, a new distribution pattern in downtown Pittsburgh, and purchase of 80 light rail cars.

The 10.5 mile line in stage one will run southward from Penn Central Station downtown, on railroad right-of-way across the Monongahela River and into South Hills via the Mt. Washington Transit Tunnel. It then will follow PAT's Dormont, Mt. Lebanon route through the city's Beechview section, and suburban Dormont, Mt. Lebanon, Castle Shannon, Bethel Park, and Upper St. Clair. The southern terminus, where a new car maintenance-storage center is to be built, will be located at South Hills Village, one of the largest shopping malls between Chicago and New York.

5. The Greater Cleveland Regional Transit Authority operates two very different rail transit systems, the 19 mile rapid transit (high platform) line from Cleveland Hopkins Airport, opened in 1968, through downtown to East Cleveland and the light rail line from downtown to Shaker Heights. These two systems have a characteristic unique in North America, sharing the same tracks for 2.5 miles.

A fleet of 48 new articulated light rail cars is now on order, with delivery expected this year. These extra-large, double-end cars will be constructed of stainless steel, will be air conditioned and will each seat 84 passengers.

Rehabilitation of track, power and supply, and overhead distribution on light rail will cost about $18 million, with 26 one-way track miles being rebuilt with new rail, ties, and ballast in order to provide a smoother and more comfortable ride for passengers and to allow the new equipment to deliver its maximum performance.

The higher power requirements of the new cars will be met by adding to the existing power conversion system and replacing an existing substation with a larger capacity unit. The entire overhead catenary on the light rail lines, which is now 60 years old, will be replaced with a heavier-duty system designed especially to accommodate the new equipment, improving the efficiency of conveying power.

Other right-of-way improvements will cost about $5 million, including replacement or rehabilitation of the retaining walls and drainage system along the light rail lines.

Design work has also begun on a new central rail maintenance and operations facility adjacent to the tracks of both systems which will provide space for maintaining both light rail and rapid transit cars at a single location. Total cost for this major project is estimated at $34 million. A cab signal system will be installed throughout the rail system as well.

6. San Francisco—Within this city, the San Francisco Municipal Railway, the oldest publicly owned transit system in the country, operates five light rail lines connecting outer neighborhoods with the central business district, and three cable car lines. At present, the five street car lines travel from various sections of the city to a point where they merge, just northeast of the Twin Peaks hills and near the beginning of Market Street. From there, the lines run along the surface to the Transbay Terminal, near the foot of Market Street.

With the opening of Muni Metro light rail lines will feed into the subway under Market Street. Nine new stations are included in the project, of which four downtown stations will be shared with BART. As constructed, BART occupies the lower level and Muni Metro upper level.

7. Chicago remains one of the most rail-oriented cities in the nation. To augment an already effective network of rapid transit and commuter rail, a major extension is in the work to the existing 103 route miles.

An extension along the median strip of the Kennedy Expressway will link the present terminus of one line with O'Hare International Airport, the nation's busiest air terminal.

The O'Hare link will cover a distance eight miles between the airport and the Jefferson Park Transit Center. Jefferson Park is the present terminus of the Kennedy rapid transit line and transfer point for 16 suburban bus routes and commuter trains. The two-track line will continue in the median strip of the Kennedy Expressway to a point where it will break off into the median strip of the O'Hare access road. About 500 feet west of the airport runways, the route will enter a tunnel and curve into a station beneath the main parking garage. Intermediate stations are planned for Harlem, Cumberland, and East River Road. At those locations, parking for more than 2500 autos will be provided. Supporting facilities will include a storage yard for 180 rail cars, an inspection shop, and electrical substations. The project is estimated to cost $136 million.

An entire program could be devoted to any one of the subjects or cities discussed in this presentation; however, it is hoped that this brief summary of the present construction activities of the urban transit properties will give an appreciation of the extent of the committment to improve rail transit in North America.

Northeast Corridor Track Laying System

by R. D. Johnson*

I wish to thank you for inviting me to speak to you at this convention today, to describe to you hopefully in greater detail and to possibly clear up any questions you may have in your mind, concerning a very exciting development in track maintenance.

I believe the Track Laying System, which we are using on Amtrak's Northeast Corridor, as we upgrade the trackage for the Northeast Corridor Improvement Program, to be a significant change in track work methods and maintenance philosophies and is basically heralding the future method for maintaining track structure. As we look into the future and contemplate changes, it is always a good idea to look first into the past to ensure we have a correct perspective, relating to these envisioned changes.

A great many of the "old time" railroaders in this room can remember when track raising was performed with hand jacks and forks. If we were "really modern", an air compressor and air guns were used. Our Production Gang then consisted of either an 8 or 16 tool raising outfit using a 105 or 315 ft air compressor with daily production somewhere between 700 to 800 feet of track raised per day. Our rail gangs, with the latest equipment available then, a burrow crane, with approximately 125 people would lay about 200 sticks rails/day. Our tie gangs, with about 250 men, would replace and insert about 20,000 ties per season per subdivision.

When the first automatic tampers and specialized equipment arrived in our gangs to raise track, install ties and lay rail, I'm sure a lot of us wondered if they would give us the type of refinement which was being secured by the known hand methods and in fact would replace what was familiar to us at that time. We met this change with resistance.

History has proven that not only did change take place and that new maintenance philosophies were established, but that these changes far exceeded our initial conception.

The tampers we first used were, in retrospect, miniatures of the machines now used in our daily operations. I would propose to you that the Track Laying System is a further refinement in the progression of modifications in the method to maintain, repair, and rehabilitate track structure.

The Track Laying System is a total track renewal system in that after its passage over a section of track, all components of the track structure have been renewed including the road bed beneath the track. The total system is made up of approximately 30 pieces of maintenance of way equipment plus specialized rolling stock. The manpower requirements are approximately 170 people.

The main machine, which is new to most, is the P-811 Track Laying Machine. The first unit, known as the Valvatere System, was developed in Italy by a contractor working in conjunction with Matisa of Luzurne, Switzerland. Matisa is an operating Division of the Canron Rail Group and we purchased our machine from the Tamper Division of The Canron Rail Group and it was built at Columbia, South Carolina. Many of these types of machines are used extensively throughout the European Continent.

The primary goals of the Track Laying System are similar to those of the Northeast Corridor Improvement Project and are to increase speed, improve ride quality, and improve maintainability.

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*Project Manager—NECIP, Amtrak.

TLS Script

Amtrak's Track Laying System, or TLS, began its first year of operation in 1978. The '78 season served as an educational period with the usual start-up problems associated, with not only personnel and equipment, but method of operation as well. However, significant production was achieved. By the 1979 work season, between Boston and New York many of the first year's difficulties had been overcome. As a comparison, our 1979 daily TLS production improved 70% over 1978! We expect to make more changes in the TLS organization this year to further refine and improve the operation.

A great amount of materials must be handled, both in the installation process as well as the removal process, in order for the TLS to install one mile of track per day.

There are 5 basic functions in the TLS operation. These are:

- Track Preparation
- Material Exchange
- Destressing & Clipping
- Undercutting
- Surfacing & Lining

Please keep in mind that it is possible to relay existing rail that is already in track, or to replace existing rail with new CWR.

The track preparation phase begins with the removal of bolts from the jointed rail. The nutters leave two bolts in each joint to allow the jointed rail to thread through the guide rollers of the Track Laying Machine. All rail anchors must be completely removed. This allows the rail to flow freely through the Track Laying Machine's rollers and guides.

The spike pulling crew pulls all the spikes except for one gage and one anchor spike in each tie plate. These remaining two spikes hold the gage for the Track Laying Machine and keep the plates on the old ties as they are removed. Scrap clean up is accomplished by using two self-propelled clean up units. Periodically, the scrap is dumped along the track for later collection. A cribbing machine is utilized to dig out the ballast in the cribs at selected locations where the Track Laying Machine will begin or end its concrete tie installation. This also occurs at open deck undergrade bridges, and at all switches and road crossings. As required, manual labor and machinery such as a backhoe may assist the cribbing operations. The required perfect alignment of the new CWR may not occur as it is unloaded from the rail train. The excess rail in this misalignment would hamper the operation of the Track Laying Machine, and so to avoid this problem, a Pettibone crane is used to lay the new CWR onto the ends of the ties to achieve proper alignment with the running rails. The rail ends are pulled together by the rail pulling machine and joined together by the rail pulling machine and joined together for threading through the Track Laying Machine. The rail pulling machine operator must keep in radio contact with the personnel at the location where the rail is misaligned. The operator adjusts the rail to a final position within approximately one foot from the field side of the running rail.

It may be easier to understand the second phase, "material exchange" of the TLS operation by first reviewing the events happening at the TLM. (See Figures 1 through 4) The entire machine weighs 346 thousand pounds, is 222 feet long, has a width of 10 feet 4 inches, and a height of 13 feet. As you will see, the TLM has 3 main parts, the first being the accumulator car. This car receives the new ties from the concrete tie flat cars and provides momentary storage for the old ties. The old ties are later removed to the flat cars.

The second main section is the 110 foot long working beam which supports the material exchange locations. Here, the old ties are removed from the track simultaneously with the installation of the new ties as the rail is also exchanged. The third main section, the power car,

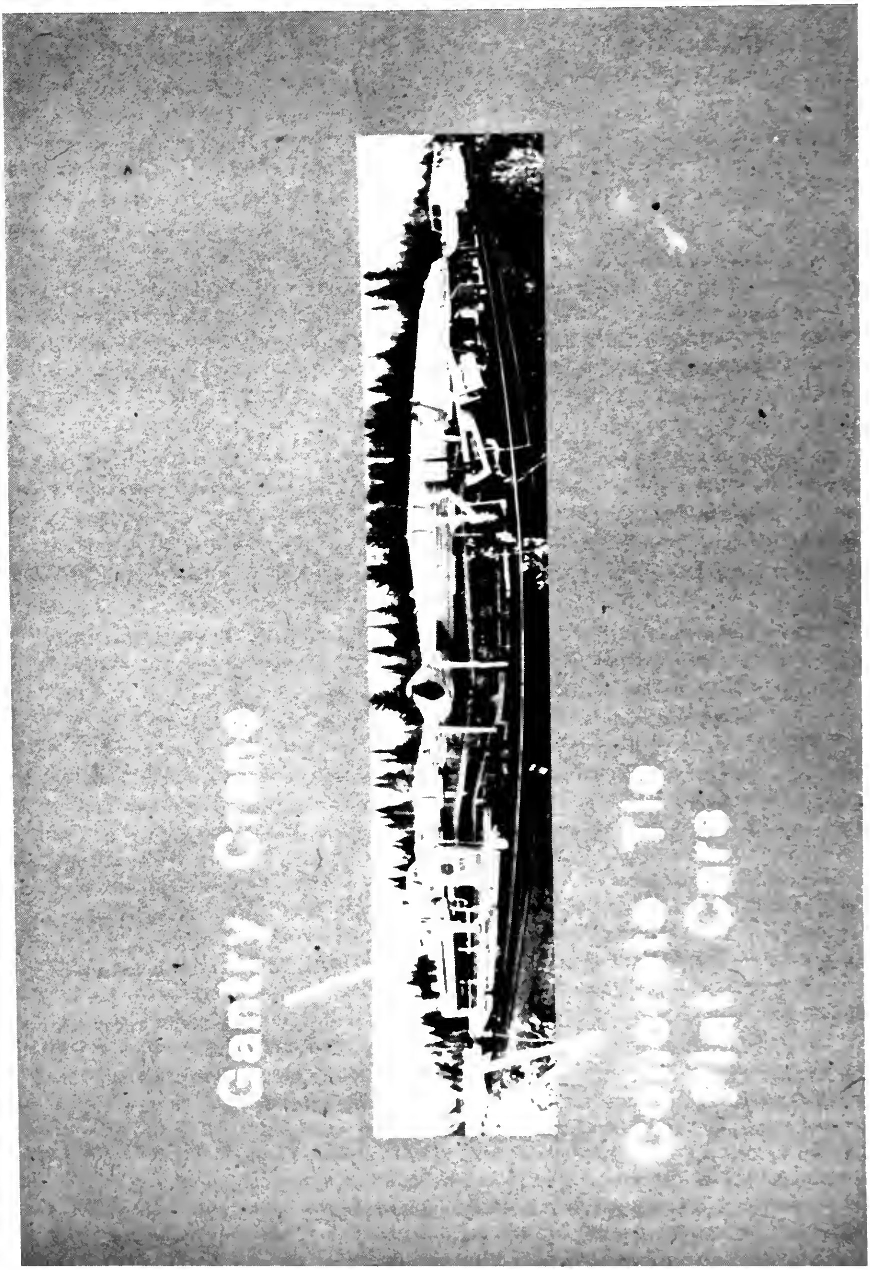


Figure 1

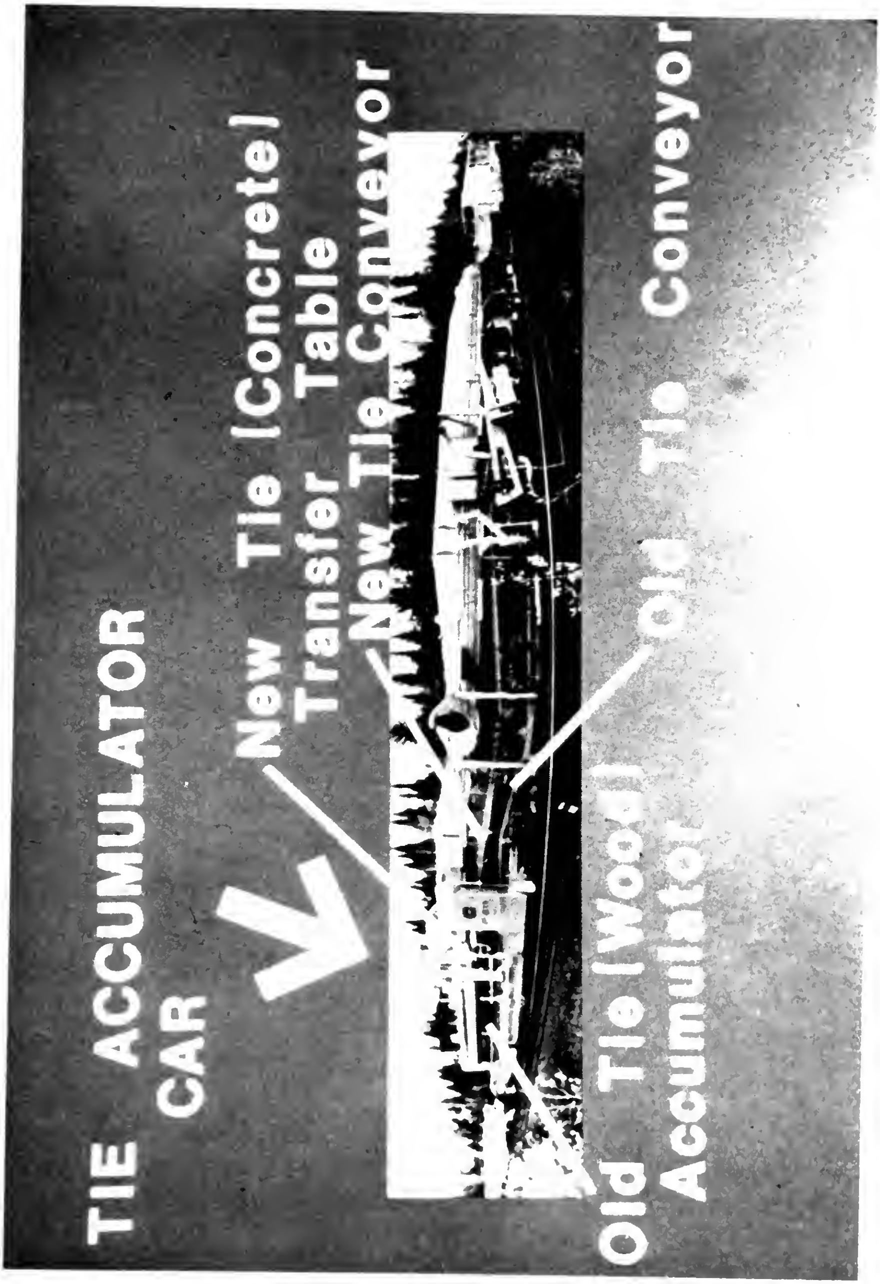

TIE ACCUMULATOR CAR
New Tie (Concrete) Transfer Table
New Tie Conveyor
Old Tie (Wood) Accumulator
Old Tie Conveyor

Figure 2

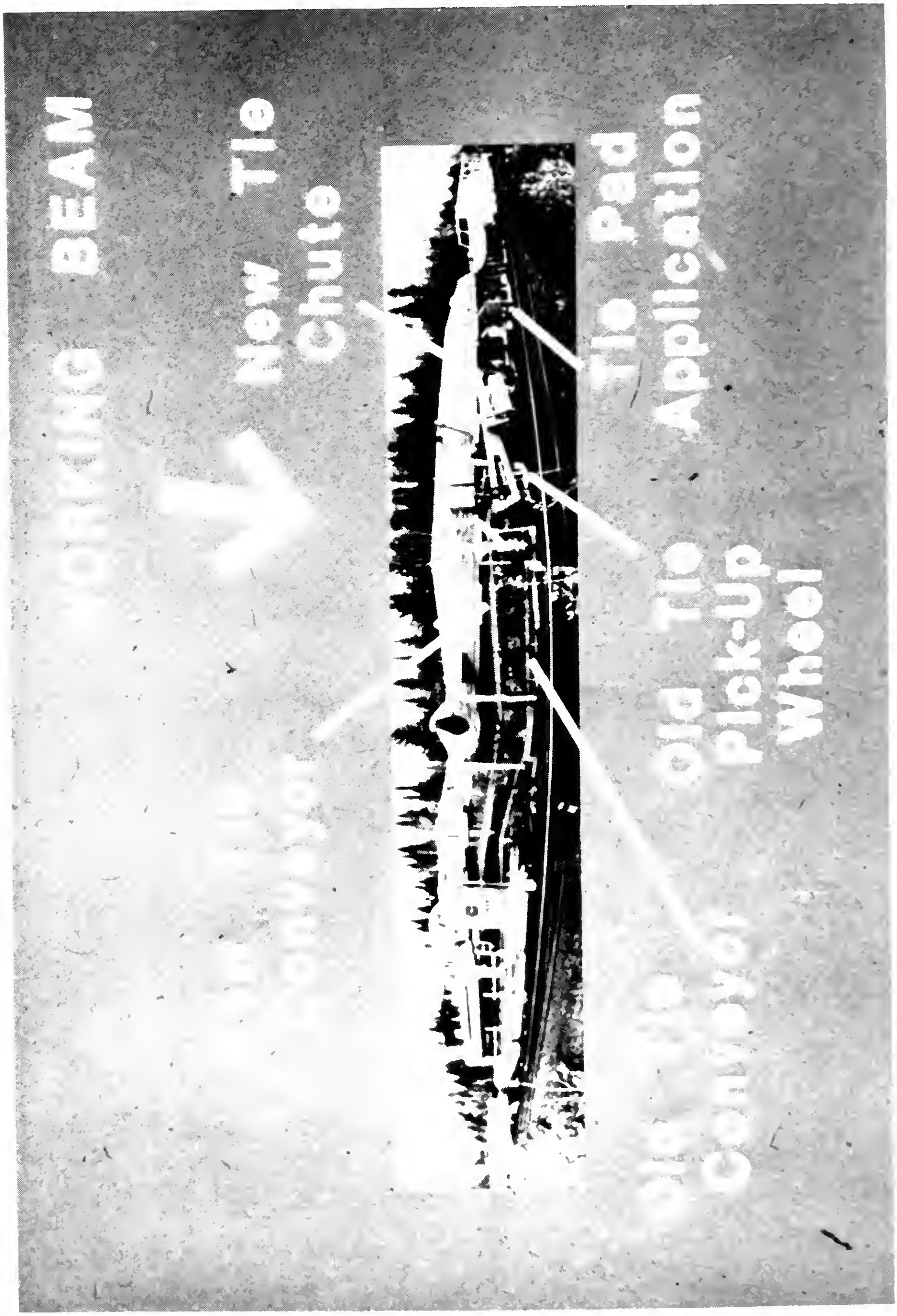


Figure 3

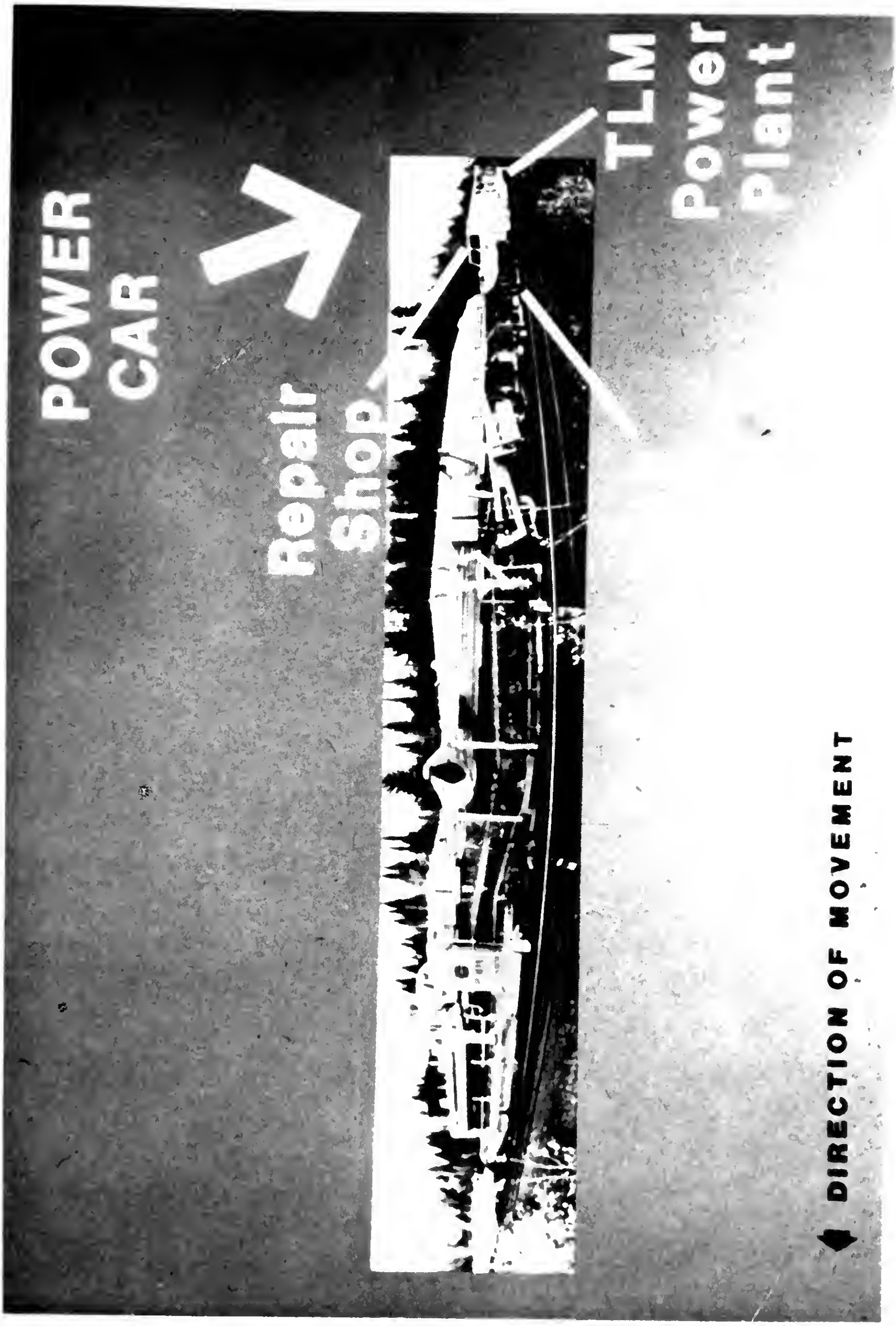

POWER CAR
Repair Shop
TLM Power Plant
DIRECTION OF MOVEMENT

Figure 4

runs the entire TLM and provides the motive power to drive all of the TLM's axles. During tie installation, the TLM moves itself, several material cars, and sixteen concrete tie cars. A repair area is also located within the power car. As you may have noticed, the normal operating direction of the TLM has the concrete tie cars on the head end with the power car pushing. There are three self-propelled gantry cranes that run on the concrete tie cars. (Figure 5) The gantry seen here is in position, picking up a load of concrete ties for the TLM. Here are several views of the TLM.

The concrete tie cars were specially built for the project. The bulkheads restrain two stacks of concrete ties; each stack having 4 layers of 21 ties each. This gives a total of 168 concrete ties per car. The top layer of concrete ties is secured by two longitudinal channel angles and chains. The gantry cranes run on special running rails fixed to the sides of the flat cars. To keep the lower three layers of ties from shifting in transit, rectangular restraining rails are placed between the layers. They lock against the bulkheads to prevent tie movement. On empty tie cars, the chains, channel angles and restraints are stored in floor compartments until needed. During the day's work the angles and restraints are stored on temporary hangers on the car sides, having been applied as needed. This keeps them available but out of the gantry's way. As the concrete ties are placed on the roadbed, the gantrys bring back the old ties for loading and securing on the flat cars. Every layer of wood ties is secured with channel angles and the entire stack is secured with the chains.

It is necessary to keep the tie plates square with the ties so a maximum quantity of old ties can be loaded. The three gantry cranes shuttle the new ties to and from the accumulator car, and use bridge rails to cross between cars. Locking pins secure the bridge rails to keep them secured to the tie cars during movement.

The gantrys have to work together, passing the new ties from one to another, ensuring that the gantry closest to the accumulator car has the shortest distance to travel and can keep the TLM constantly suppled. New ties feed into the TLM on the top transfer table while old ties gather on the lower accumulator. Before the old ties are picked up, any misaligned ties must be straightened as required. After releasing a load of new concrete ties, the gantry positions itself over the old wooden ties, lifts them, and takes them back to the other gantrys for loading on the empty tie cars. Later, at the end of the day, the fully-loaded tie cars with old ties will be set out and shipped to the old tie reclamation area.

The stage is now set for the actual material exchange under the TLM. We will start with the old tie removal sequence, and then discuss the installation of the new track. (Figures 6, 7 and 8) As the TLM advances, the old tie pick-up wheel rotates, picking up the old ties. The plow follows, cutting a smooth bed for the new ties. Plow guides, located beneath the old ties, work with the rotating old tie pick-up wheel to lift the old ties from the track bed. If a tie has a tie plate hanging upside-down, it will be flipped over to put it in loading position. The size and weight of the old ties varies and sometimes human assistance is needed to offset the tie flipper's force.

The lower conveyor takes all the old ties back to the old tie accumulator to be picked up by the gantry.

Underneath the upper conveyor leading to the tie chute, there is a vibrator plate which forms and compacts the track bed movements before the new ties are placed.

The old rail is removed and crossed over the new rail. The rails are contained and guided as they are relocated by the guide rollers. After the tie pad placement, the rail lining station operator centers the rail between the Pandrol lugs, so the power car can follow on the new trackage just laid. Four Pandrol clips are dropped between each tie by the men in the gondola. This operation has recently been changed by cutting a hole in the car's floor to allow the clips

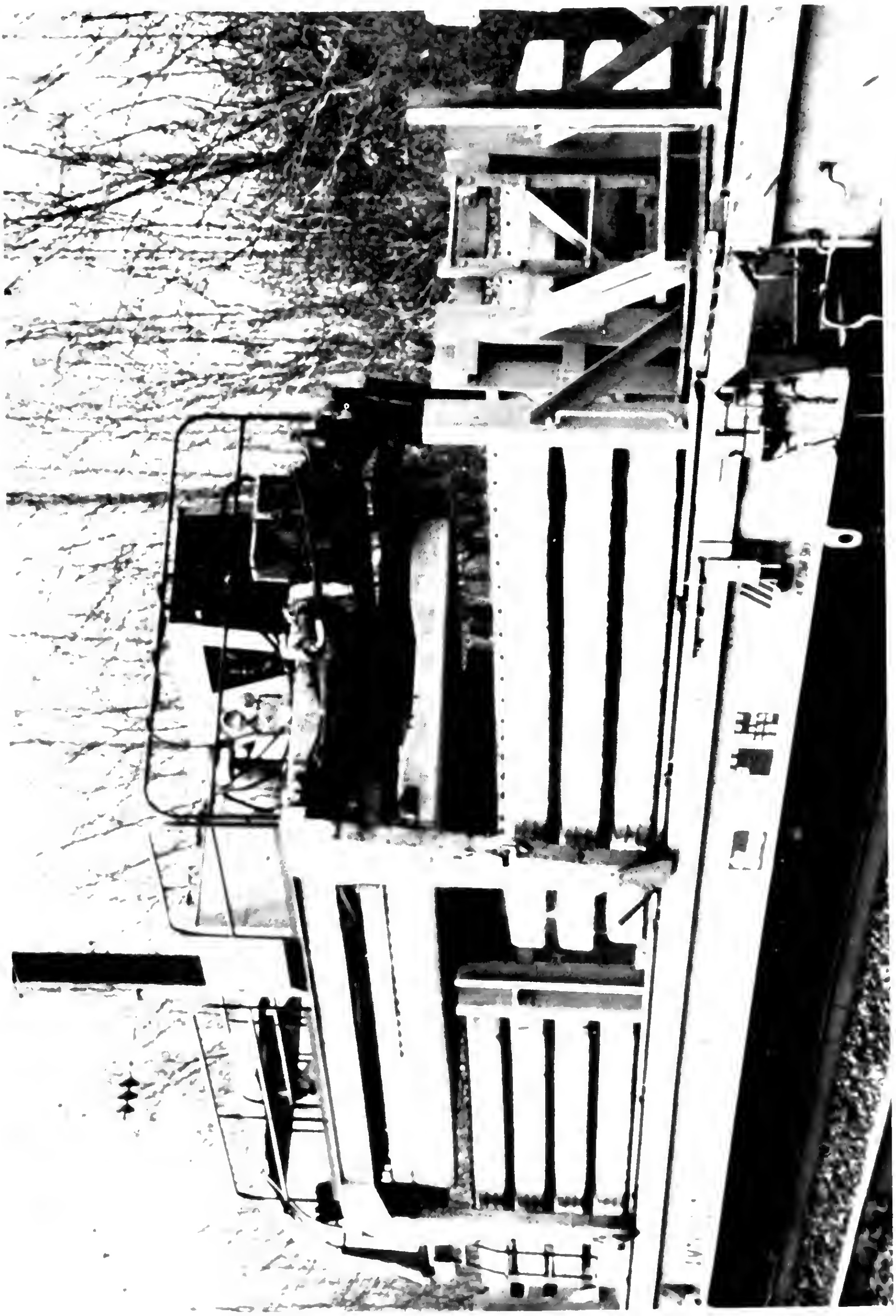


Figure 5

Figure 6

Figure 7

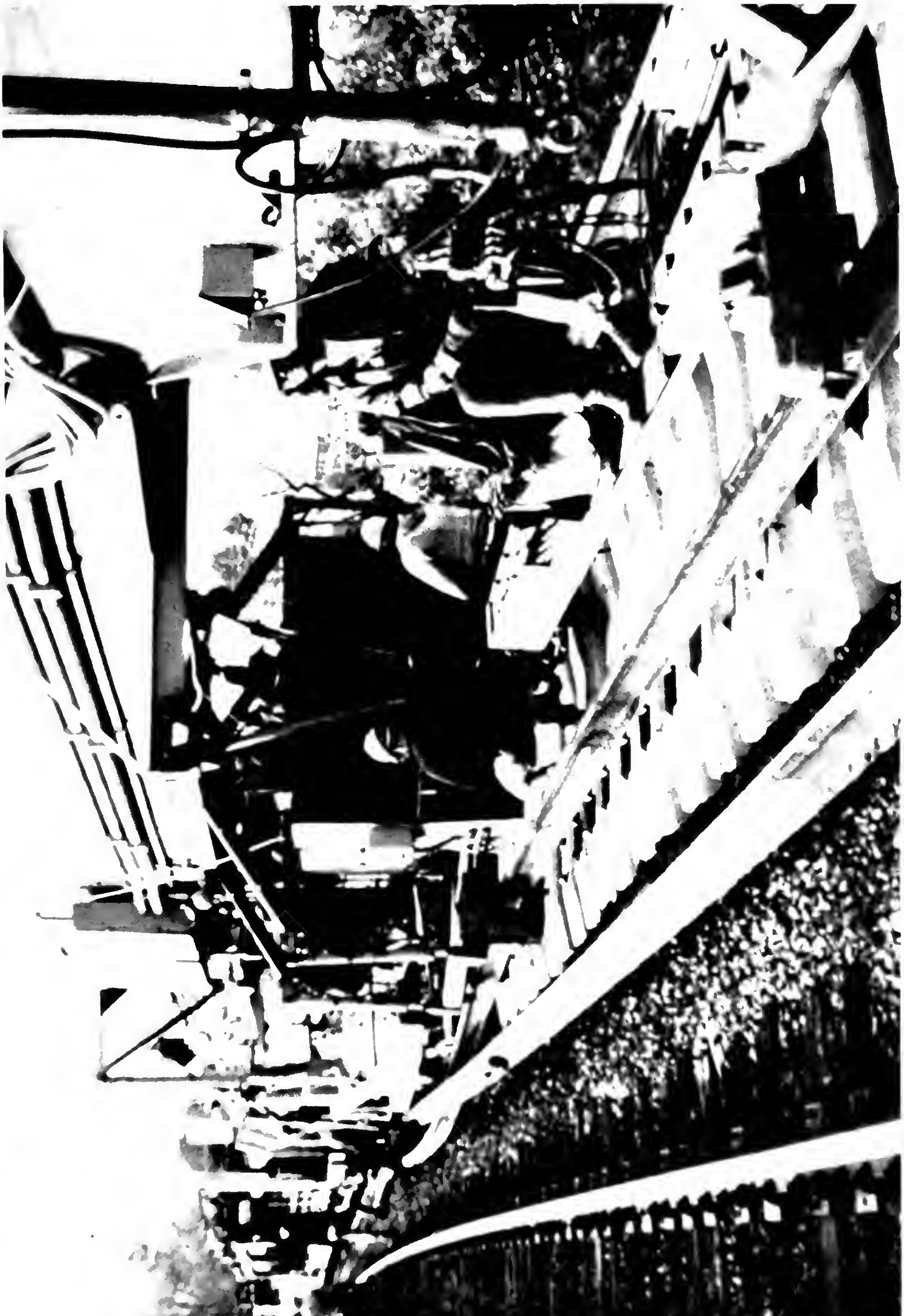


Figure 8

to be dropped directly into the crib. The TLM leaves very good line and surface behind its operation.

The third phase in the TLS operation, destressing and clipping, uses propane rail heater equipment, followed by a dual rail vibrating machine, and a chiller unit for hot days. Insultors are set, by hand, (Figure 9) and sometimes the rail is not centered perfectly and must be manually adjusted so that the insulators can be inserted. A hand tool is used to re-position the rail and allow correct insulator placement. The rail is now ready for clipping, so the pandrols are positioned (Figure 10) and the pandrol driving machine (Figure 11) secures them in place. As you know, the pandrols may be also secured manually.

The thermit welding gang follows behind the clipping operation as time permits. A hydraulic shear removes excess material from the weld. Shearing is followed by final grinding.

We utilize safety straps for extra protection on the high speed passenger tracks.

Prior to undercutting the jointed rail must be removed out of the undercutter's reach. An air wrench handles most of the bolts but a cutting torch is required for frozen bolts. In most areas the drainage needs improvement so a Jordan spreader is used ahead of the undercutter to the maximum extent possible. A Plasser undercutter is utilized to undercut and clean the ballast so that a minimum of 8 inches of clean ballast exists under the tires. About 40% of the volume of material passing over the screens is spoil, as this is the first time the corridor trackage has ever been undercut. Some of the cleaned ballast is returned directly behind the undercutting chain to provide support for the new ties. The remaining clean ballast is distributed immediately in advance of the trailing wheels. When the undercutter spoils can't be cast along the right-of-way, for embankment stabilization, they are loaded into air dump cars equipped with conveyors. A ballast regulator follows right behind the undercutter to evenly distribute the cleaned ballast prior to the arrival of the ballast train. Extra ballast is added as required for final surfacing and lining operation. There are three separate self-contained gangs in the surfacing and lining operation. These gangs place the track on a pre-set alignment located earlier by field survey crews. Each gang consists of a head-end ballast regulator followed by a Canron Mark 2 raising and lining machine with an absolute base lining system, a Canron Mark 1 switch tamper, a ballast compactor, and lastly, a rear ballast regulator. The ballast compactors are utilized in the crib areas, as well as the shoulder areas. The rear ballast regulator finishes the track with its track broom and leaves the track ready for operation. (Figure 12) The TLS, I believe, is introducing a new generation of track maintenance philosophy in our industry.

To provide a complete picture of the TLS operation, I would be misleading if I did not provide a thumb nail introduction to concrete tie manufacturing and logistics. The 1.1 million concrete ties required for the program are being supplied to Amtrak by the joint venture of Santa Fe Pomeroy, Inc. and San/Vel Concrete Corporation. Amtrak's concrete cross tie is 8 feet 6 inches in length, 11 inches wide at the bottom, 9½ inches deep at the rail seats; and weighs 780 pounds. Pandrol limited supplies Santa Fe/San Vel with the steel rail clips, tie pads, insulator clips, and malleable cast iron shoulders. (Figure 13) The production facility has the capacity to operate 24 hours a day, producing 2240 ties. The tie manufacturing process begins with cleaning the forms. These are cleaned with a high-pressure air jet and mechanical cleaning then sprayed with form release oil. The malleable cast-iron shoulders for the Pandrol clips are placed in the form and round identity buttons with the date codes are put in the center of each tie. The pre-stressing wire is stored in reels adjacent to each production line. The prestressing strands are put in templet blocks which are placed on a utility cart and drawn through the forms. Gate bars are placed periodically along the forms to separate the strands to their proper locations. At the jacking end of the forms the templet blocks are secured, and the slack is removed so that each strand is correctly positioned. The strands are next stressed

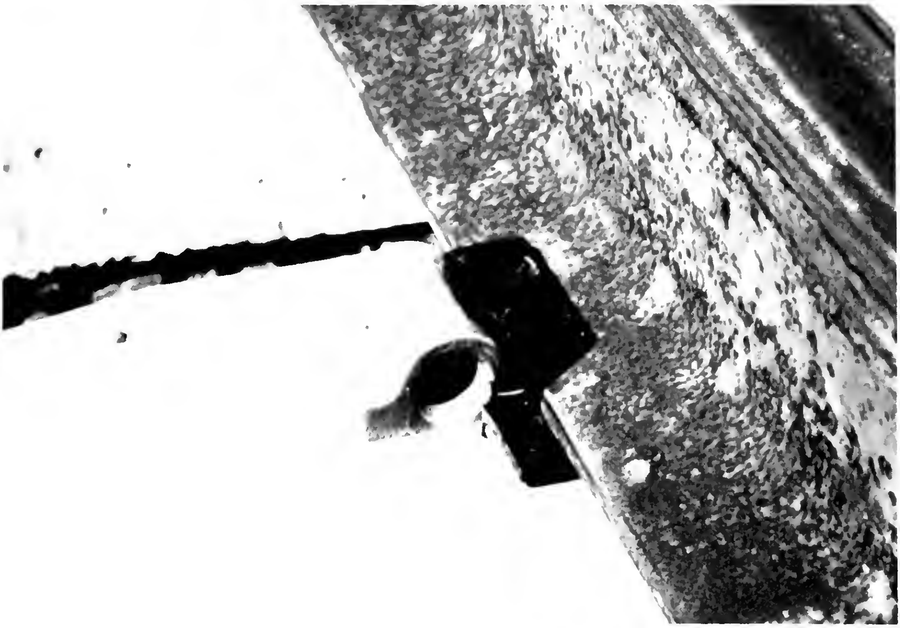


Figure 9

Figure 10

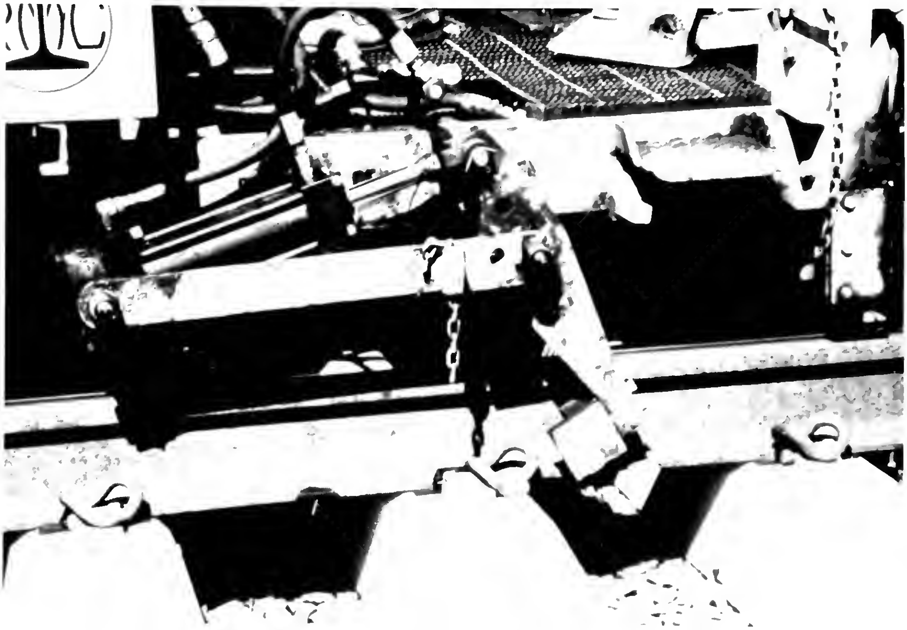


Figure 11

Figure 12

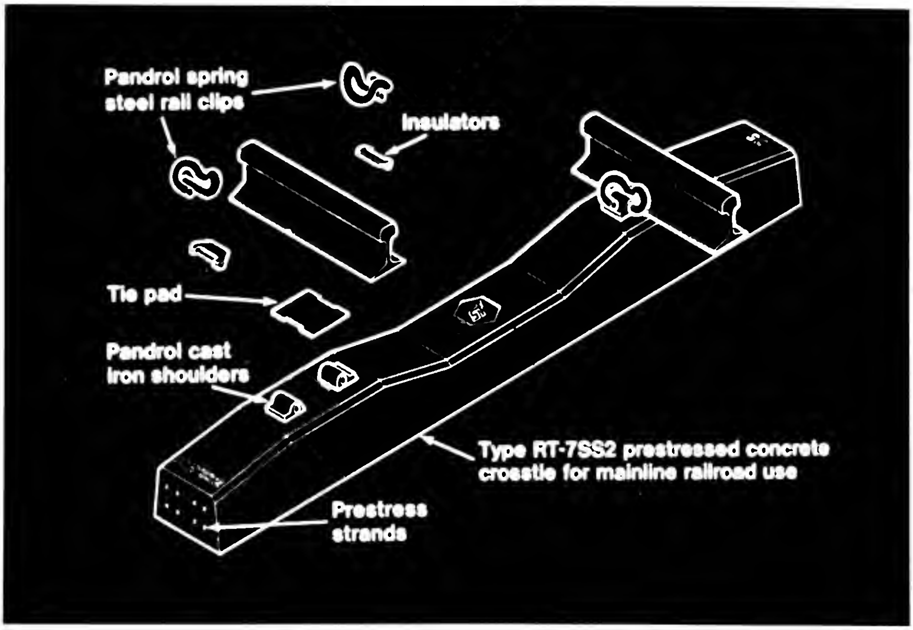


Figure 13

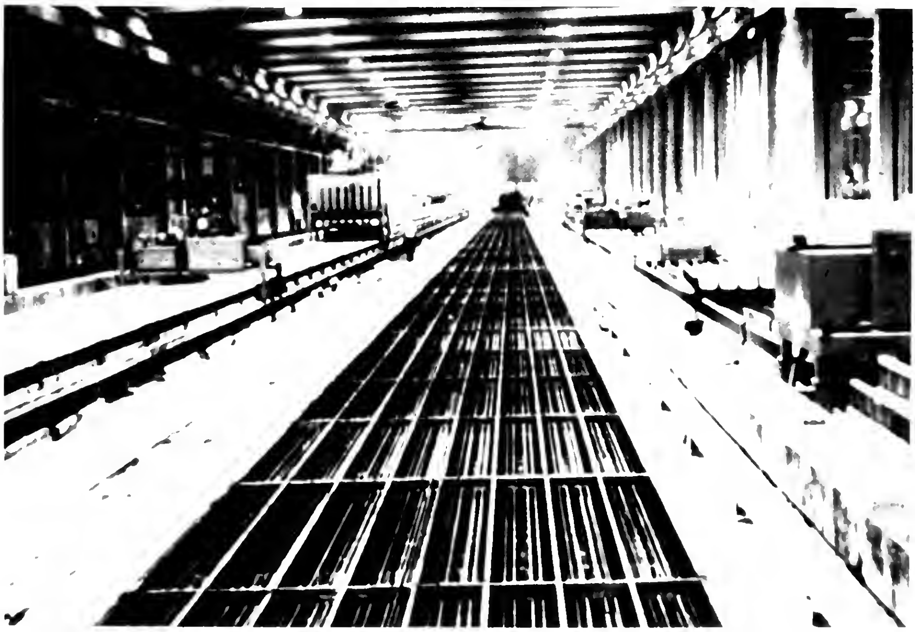


Figure 14

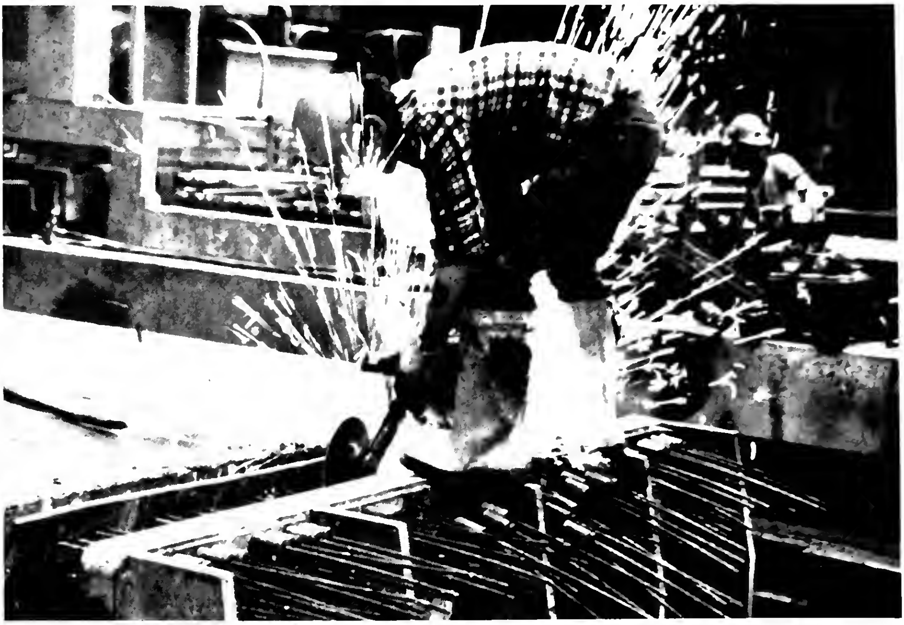


Figure 15

Figure 16

to the required tension. Elongation measurements are recorded at this time. Santa Fe/San Vel manufactures its fine and coarse aggregates from its 200 million cubic yard gravel deposit. The concrete is mixed in 4 yard batches and the quality of the mix is carefully controlled and monitored by a computer. A forklift transports concrete from the batch plant to the assembly line where the concrete is transferred to a self-powered delivery machine straddling the forms. The delivery machine dumps the fresh concrete into the 6 yard hopper of the pouring machine.

The pouring machine discharges concrete to all 8 form cavities simultaneously. It vibrates, screeds, and finishes the bottoms of the ties in one continuous operation. Approximately one hour after pouring, the end gate bars are removed from the forms to allow for expansion and contraction during the curing cycle, an overhead crane then removes all pouring equipment. An insulated neoprene tarp is placed over the forms after the concrete had its initial set. Hoops under the tarp allow sufficient steam circulation for the curing cycle. Six test cylinders are also placed under the tarp at this time. Automatic temperature recorders permit the curing crew to monitor and adjust the steam curing cycle. The test cylinders are removed from under the tarp after 14 hours; two are tested immediately, the others are tested at 7 and 28 days respectively. The curing cycle ends when the test cylinders attain a 4000 PSI compressive strength. Upon completion of the curing cycle, the tarp is removed and the tensioning jacks gradually released. The end strands are then cut loose with an abraisive saw. (Figure 15) A water-cooled diamond edge saw cuts the strands between the ends of the ties. The ties are lifted out of the forms by a special pick-out machine equipped with vacuum released. (Figure 16) The overhead crane then transfers the new ties to flat bed truck for the trip to the tie storage area approximately one mile away. A specially equipped fork lift with hydraulic forks grips both the top and bottom of the new ties as it unloads them from the flat bed and flips the inverted ties to permit their storage in an upright position. Each day's production is segregated for ease of identification as well as for quality control and acceptance purposes. Three test ties per production line are chosen at random and delivered to the quality control laboratory.

When the testing machine indicates that the 7000 PSI design strength has been attained, ties for that production run are subjected to rail seat positive bending moment, tie center negative moments, bond developments tendon anchorage, and ultimate strength tests. The ties are also examined under 5 power magnification to detect minute cracking which is cause for rejection of the entire production line at loads up to 52 KIPS. After the ties have passed all requirements they are loaded onto Amtrak's concrete tie flat cars by a travel-lift gantry. These gantries are equipped with special frames capable of handling 21 ties at a time. The plant's storage yard which has a storage capacity of over 100 thousand ties.

Concluding Remarks

Underlining any assembly line type operation are two basic requirements. These are proper machinery maintenance and preplanning of all activities. Machinery must be ready to operate, and operate efficiently and consistantly, when called upon. In addition to providing an overall general work plan for the TLS which identifies responsibilities and accountabilities, site specific work plans (SSWP) are formulated for each block of track to be worked. These SSWP's contain a statement of work, a construction schedule, configuration control data, established budget and manhours for each sub-element work item, and an identification of any additional resources required with location specified where they are located.

Improved Method of Determining Size of Transverse Defects

by Alexander J. Rogovsky*

INTRODUCTION

Ground inspection of rails following detector car inspection is carried out for verification of detector car results and flaw characterization and sizing. This ground inspection is performed by using portable hand-held ultrasonic flaw-detectors in the echo-pulse mode. The sequence of operations performed during the ground, or so-called hand-held device, inspection of rails can be explained by means of the flow chart, Figure 1. Because of the short time that is available for the hand-held inspection to characterize flaws found by the detector car, the flaw size estimation is approximate. Consequently, apparent sizes of flaws determined by the hand-held inspection are sometimes quite different from actual sizes[1]. Errors in size determination can cause inappropriate remedial actions, resulting in financial losses through possible failures or unnecessary rail replacement. A survey conducted on flaw characterization and sizing of the defects in rails has indicated considerable uncertainty in flaw size determination. Therefore, analysis of sizing procedures and their improvement is important for increasing rail inspection reliability, prevention of derailments, and unnecessary rail rejection. As a part of the program on cost-risk analysis of defective rail sponsored by the Association of American Railroads (AAR)[1], a study of the hand-held inspection has been carried out with emphasis on transverse defects. This paper summarizes the results of that study.

MODELLING OF THE GROUND RAIL INSPECTION

For a simplified modelling of the ultrasonic testing in the echo-pulse mode, rail transverse defects (transverse fissures, detailed fractures) can be approximated by flat reflectors. Sizing techniques employed in ultrasonic testing essentially depend on the ratio S_f/S_u between the flaw area S_f and the cross section of the interrogative ultrasonic beam S_u, Fig. 2. For small defects with the ratio $S_f/S_u<1$, techniques are used based on the comparison of the ultrasonic response from real flaws with responses from well-characterized artifical reference flaws by using standards as well as standardless methods. The most common type of reference flaw is a flat bottom hole (FBH), or a disk-shaped defect. The ultrasonic response A_f from such a flaw located on the axis of the ultrasonic beam in the far field of the transducer[2] can be expressed by the formula:

$$A_f = \frac{S_s S_f}{Z^2\lambda^2} \tag{1}$$

where S_s is the area of the ultrasonic transducer,
S_f is the area of the disk-shaped defect,
Z is the distance between the transducer and defect,
λ is the wavelength.

If sizes of flaws are greater than the diameter of the ultrasonic beam, i.e., $S_f/S_u>1$, the response from such flaws can be compared to the response from an infinite plane. The amplitude of the signal A_∞ from the infinite plane is expressed by the formula:

$$A_\infty = \frac{S_s}{2Z\lambda} \tag{2}$$

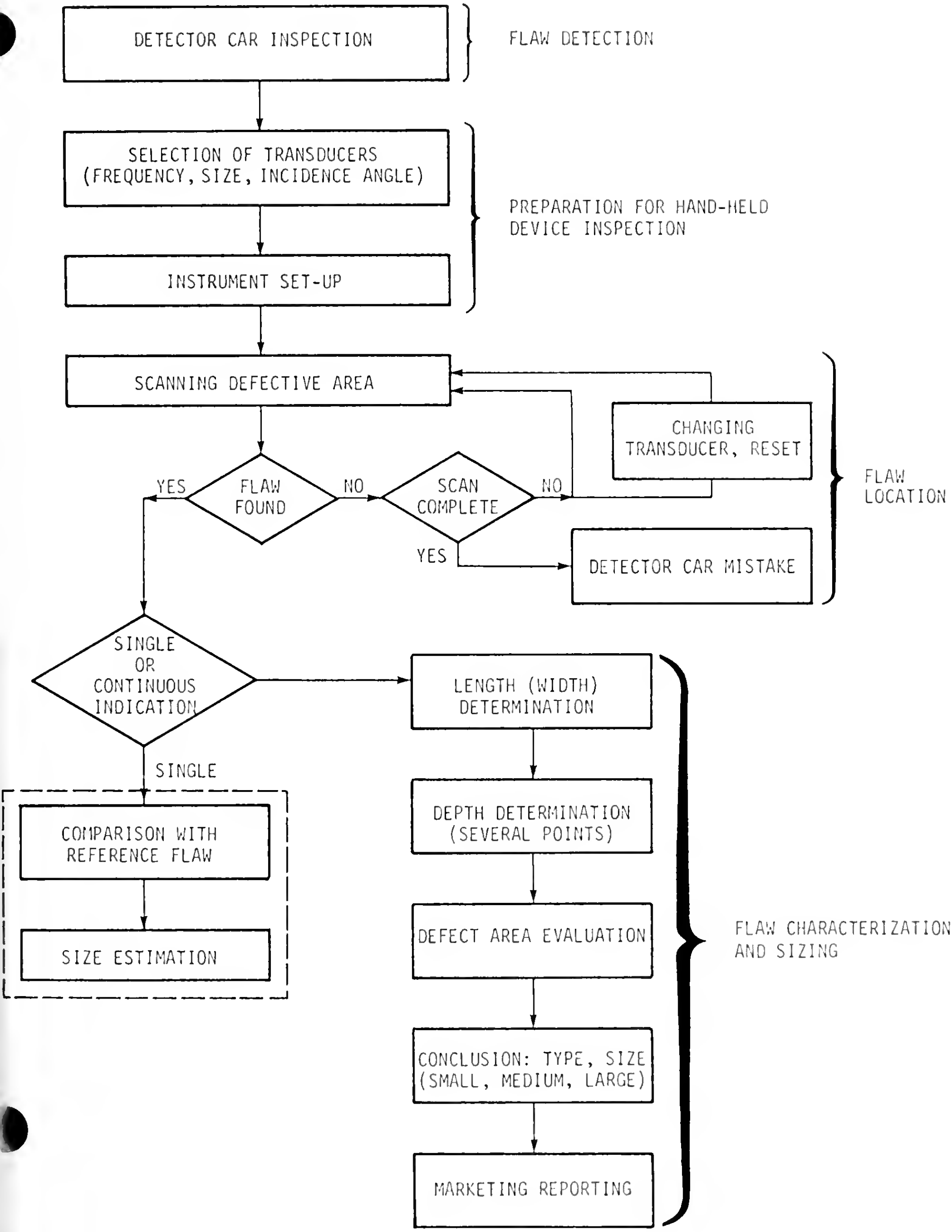


Figure 1 - Simplified Flowchart of Rail Hand-Held Device Inspection.

A typical dependency of the relative echo-amplitude versus defect size is shown in Fig. 2. It is obvious that a response from a large defect ($b/a \geq 3$) is equal to the response from the infinite plane. Practical techniques for sizing of such large flaws usually consist of scanning a flaw, finding the maximum response, and locating the flaw edges by moving the probe until the signal amplitude vanishes or decreases to some predetermined level N, e.g., 6, 12, 20 db (Fig. 3). Dimensions of defects determined by these techniques are conditional and depend on the orientation, shape, and type of the flaw. There are numerous other factors that affect the accuracy of the size determination (Table 1).

It is conventional in the inspection of rail to classify transverse defects as small, medium, or large, denoting separation of 1-20%, 20-40%, and 40-100% the rail head, respectively. As a result of the influence of different factors, errors in the size determination and misclassification of the defect size can occur. For transverse defects in rail, this can be represented probabilistically by the curves in Fig. 4. Here, $P_{sm}(S)$, $P_{med}(S)$, $P_l(S)$ are probability density functions of the classifications of defects as small, medium, large, respectively when actual size is S. If α_{ij} is the probability of misclassifying a flaw of class i into class j, errors of size misclassification can be determined from the following formulae:

$$\alpha_{12} = \int_{20\%}^{40\%} P_{sm} ds \qquad (3)$$

$$\alpha_{13} = \int_{40\%}^{S_3} P_{sm} ds \qquad (4)$$

$$\alpha_{21} = \int_{S_1}^{20\%} P_{med} ds \qquad (5)$$

$$\alpha_{23} = \int_{40\%}^{S_4} P_{med} ds \qquad (6)$$

$$\alpha_{31} = \int_{S_2}^{20\%} P_{lg} ds \qquad (7)$$

$$\alpha_{32} = \int_{20\%}^{40\%} P_{lg} ds \qquad (8)$$

These errors can be used as estimates of the inspection accuracy. Probability density distributions P_{sm}, P_{med}, P_{lg} would idealy be obtained from *in situ* NDE measurements and subsequent destructive determination of actual flaw size. Estimates of the distributions may be possible by comparing typical *in situ* NDE measurements with more careful and exhaustive NDE measurements on the same defects. Improvement of sizing procedures should be oriented to suppress these errors as much as possible.

SURVEY

A questionnaire on flaw characterization and sizing of defects in rails was prepared by Failure Analysis Associates (FAA) and distributed by the Association of American Railroads (AAR) to several railroad inspection teams. The purpose of this survey was to collect information on techniques used with hand-held inspection devices for characterization and sizing of defects in rails. Analysis of the information obtained in response to the questionnaire is presented below.

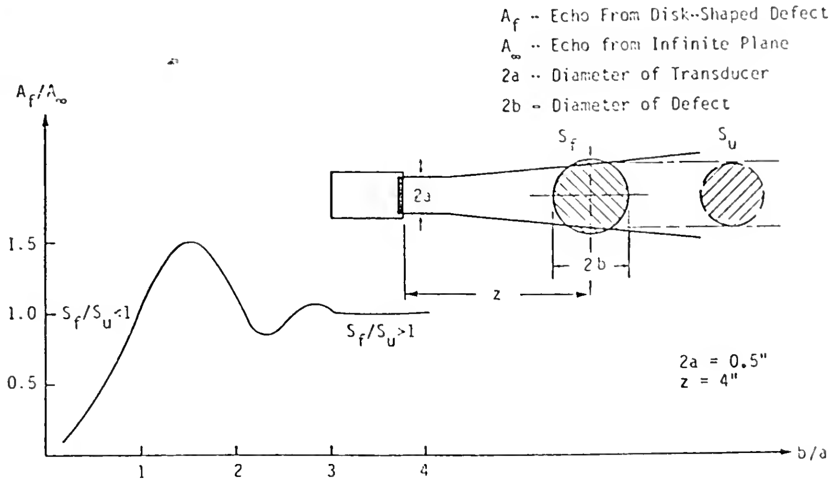
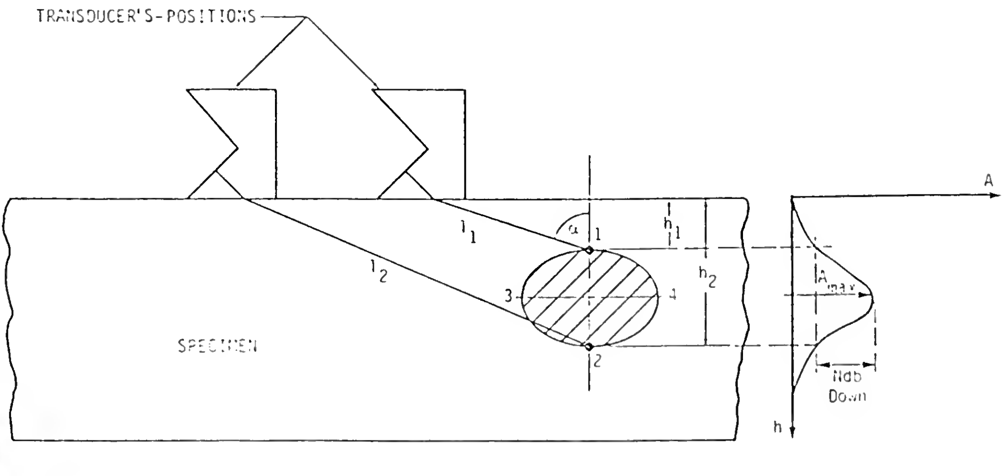


Figure 2 - Relative Signal Amplitude v.s. Relative Defect Size.



$h_1 = l_1 \cos \alpha$

$h_2 = l_2 \cos \alpha$

Figure 3 -- Scanning and Sizing a Flaw.

TABLE 1

FACTORS AFFECTING ACCURACY OF THE FLAW SIZE CHARACTERIZATION

TEST SPECIMEN	TYPE, SHAPE, AND ORIENTATION OF FLAW SURFACE CONDITIONS, ATTENUATION
CALIBRATION	ACCURACY OF DETERMINATION OF TRANSDUCER PARAMETERS; ANGLE, INDEX POINT, CALIBRATION OF DISTANCE SCALE SENSITIVITY SET-UP EDGE THRESHOLDS (NDB DOWN)
INSPECTION	SCANNING SPEED AND INDEX (INCREMENT) ACCURACY OF THE TRANSDUCER LOCATION ON THE RAIL
ENVIRONMENTAL CONDITIONS	AMBIENT TEMPERATURE HUMIDITY PRECIPITATION
HUMAN FACTORS	TRAINING SKILLS PHYSICAL CONDITIONS (TIREDNESS)

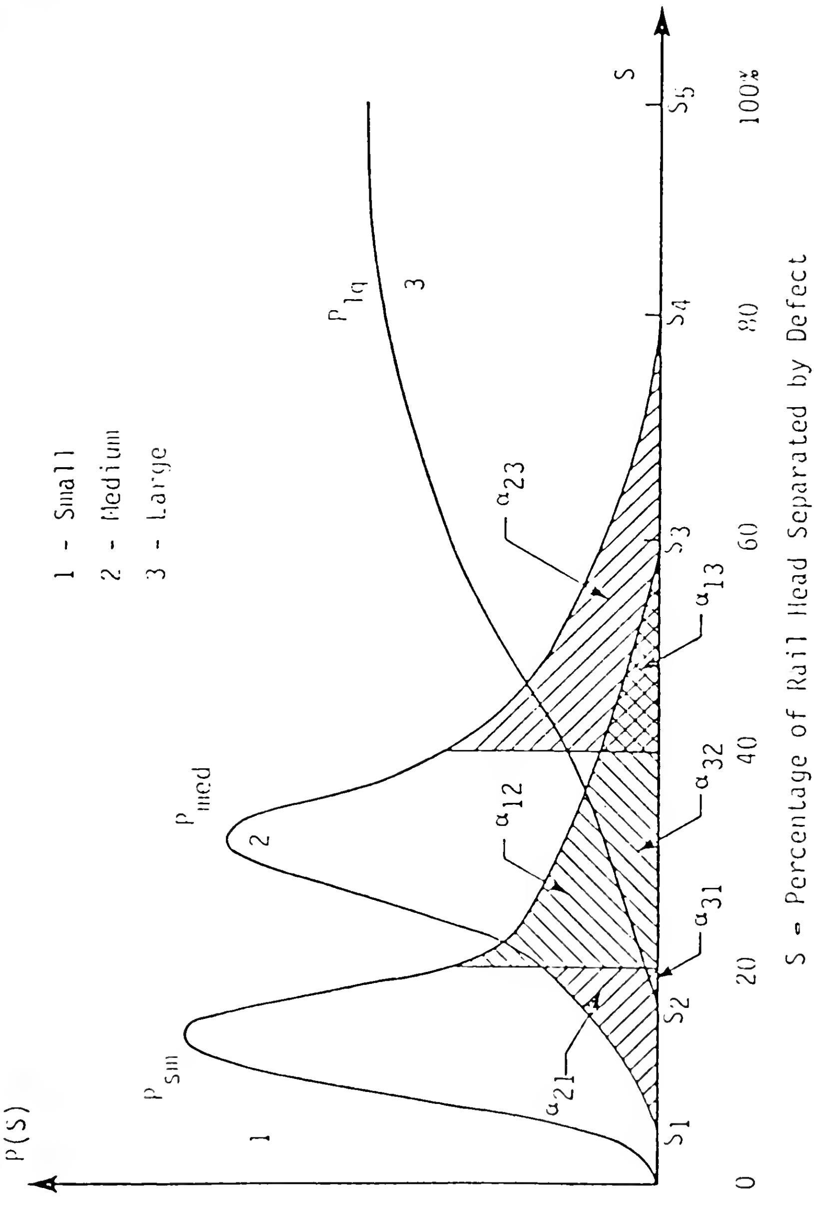


Figure 4 .. Probability Density Distribution P(S) of Sizing Transverse Defects.

Answers to the questionnaire were received from six (6) inspection teams, designated as teams A, B, C, D, E, F, as summarized in Appendix A. All teams exclusively use ultrasonic testing as a nondestructive technique for the ground flaw characterization. (Team B has also indicated magnetic technique, but evidently they use it in the detector car instrumentation.) Different types of ultrasonic flaw-detectors are in use. Most instruments are portable flaw-detectors manufactured by Krautkramer-Branson, Inc. (Instruments USM-2, USL-31, ULS-32, USL-38). Portable instruments manufactured by other companies (UJ-Automation Industries, RS 702-Magnaflux) are also in use.

Ultrasonic transducers being used are of different types and manufacturers. Straight beam transducers are implemented for detection and evaluation of preferably horizontally oriented defects. Also, double probes are in use. The most important are angle beam transducers with angle from 45 to 76°. Most transducers operate at 2.25 MHz frequency. Transducers have dimensions from 0.25" up to 1", mostly 0.5" Dia. A transducer with variable frequencies (1, 2.25, 5 MHz) and angles (10°, 45°, 70°) was developed at the Southern Pacific detector car laboratory[3]. This transducer allows detection and characterization of defects of various orientations, types, and locations and is very promising for practical use.

Calibration techniques sometimes employ such reference reflectors as the rail base, rail edge, indications from joints, etc. Other reference reflectors and specimens are in limited use. Answers related to minimum sizes of detectable flaws for major defects in rails show that there are no clear standards or requirements on testing sensitivity. Also, there are no written procedures for ground inspection and flaw characterization in rails. Development of such standards and requirements may increase reliability and accuracy of the inspection.

ANALYSIS OF SIZING PROCEDURES AND RELATED EXPERIMENTS

Different sizing techniques employed in ultrasonic testing[(2), (48)] have been considered to develop an improved feasible procedure for rail inspection. Such techniques as flat bottom hole (FBH) calibration, 6 db and 20 db dropping and vanishing echo procedures have been analyzed. A series of laboratory experiments has been also carried out with specimens containing real transverse and artificial defects for this purpose. Results of these investigations are presented in this section.

Flat Bottom Hole Calibration

Although calibration on flat bottom holes (FBH) is primarily used for the sensitivity selection and sizing of small defects, this technique is also used to determine the profile of continuous flat defects. A defect is scanned at the sensitivity level corresponding to the response from FBH of small diameter (e.g., ⅛" or ¼"). The extreme points of the defect are considered to be determined by the positions of the transducer corresponding to the signal dropping below the FBH level. The accuracy of this technique strongly depends on the transducer beam profile and geometrical characteristics of the defect. This technique can provide an accuracy of about FBH radius by using narrow beam transduecers with the beam direction perpendicular to the defect surface.

Applicability of the FBH calibration technique for rail inspection was tested on a rail specimen containing a transverse defect, Fig. 5. One of the edges of this specimen was cut at a 70° angle and ten flat bottom holes with diameters 1/16", 3/32", and ⅛" were manufactured. Dependencies of the signal amplitude vs. FBH diameter and its distance from the transducer are shown in Fig. 6. Signal amplitudes are expressed in db corresponding to attenuation readings. Signal amplitudes have a sharp decrease with increased distance because of the divergence of the ultrasonic beam and attenuation in the material. Compensation for this decrease can be accomplished by means of an amplitude distance correction (ADC), a correc-

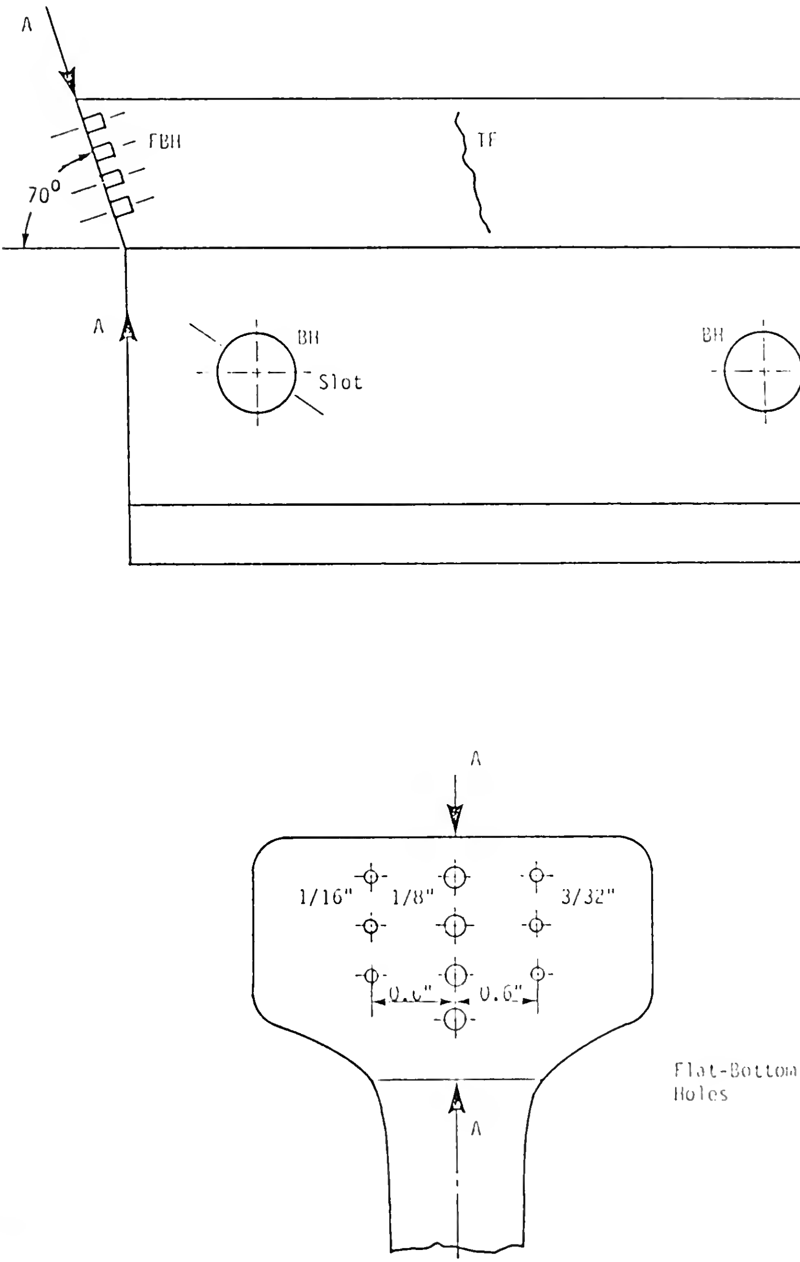


Figure 5 - Rail Specimen with Natural and Artificial Defects.

tion that results in approximately the same signal, independent of transducer distance from the flaw, and allows more convenient evaluation of defect sizes. Such a correction (3-point gain control of the instrument "Sonoray" 303) was used to obtain an ADC curve on the screen of the flaw detector, Fig. 6. This curve was supposed to represent the threshold level for the extreme points of a flaw. Building of such a curve by indications from the reference FBH is time-consuming because the operator has to pick a FBH, maximize its response, and repeat these steps several times while using 3-point gain control to equalize the responses from the FBH for different distances.

An alternative for this procedure is the use of automatic selection sensitivity concepts[9] but this requires implementation of the computer technology that is not practical for the fast ground inspection of rails, in this case.

Although calibration on FBH for sizing of continuous defects is not efficient, it can be used for the size characterization of small defects.

Dropping Techniques

As the probe is moved from a position where the entire ultrasonic beam intercepts a flaw to a position where only part of the beam intercepts the flaw, the echo signal diminishes. This fact is useful in locating flaw edges.

The 6 db dropping technique is based on the underlying assumption that the echo signal will fall by one half while the probe is being moved from the position corresponding to the maximum response to the position where the axis of the beam is brought into line with the edge of the flaw.

This technique should be quite accurate for large defects, particularly if their edges are long and almost straight and the reflection surface is smooth enough and perpendicularly oriented to the ultrasonic beam. For real defects such as transverse fissures and detailed fractures, the ratio N=6 db (2 times) is not realized in most cases because of the random orientation and shape of the defects, their rough surface, and curvature of the rail head.

To obtain a more realistic ratio N for sizing of transverse defects, artificial defects (slots) were made in the stell specimens, Figs. 7, 8, 10, and variation of the echo signal from the slot was monitored when the transducer was moved relative to the slot edge.

Figures 7-10 illustrate dependencies of the signal on probe position. Although the slot surfaces were quite smooth (surface roughness 32 inch), signal drops ΔA corresponding to the probe position where its axis crosses the edge of the flaw are greater than 6 db and have a range 8-10 db in most cases. For real flaws, expected signal drops might be even higher because of the reasons mentioned above.

The method of sizing a flaw by sensing its outline with the edge of the ultrasonic beam was used first for sizing fatigue cracks in railway locomotive axles in Britain[4]. The most accurate measurements by means of this method were achieved by using a 20 db drop from the maximum response and accounting for the ultrasonic beam profile. The detailed analysis of this technique is presented in Ref.[4]. This technique requires meticulous measurements of the beam profile by using small targets. The accuracy of the procedure depends on relative dimensions of the flaw and the ultrasonic beam. For larger defects, a more appropriate ratio might be 15 or 10 db rather than 20 db if the beam profile is obtained by interrogation of the small targets. Procedures utilizing the beam profile measurements and 20 db drop technique is very time-consuming, as was shown by laboratory measurements of the beam profile by reflection from side drilled holes in rail specimens. Also, these measurements showed that echo signals from side drilled holes were of low level relative to echos from transverse defects and cannot be used as reference reflectors.

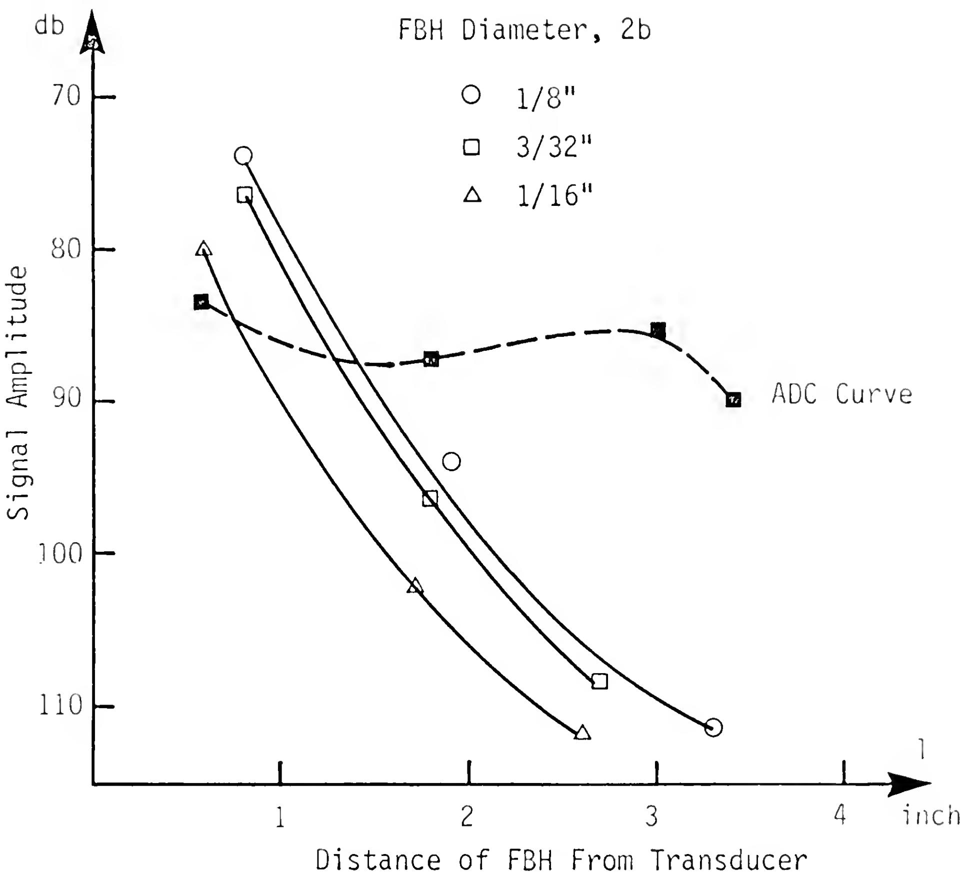


Figure 6 - Dependencies of Signal Amplitude from Flat Bottom Hole (FBH) on Distance and Size.

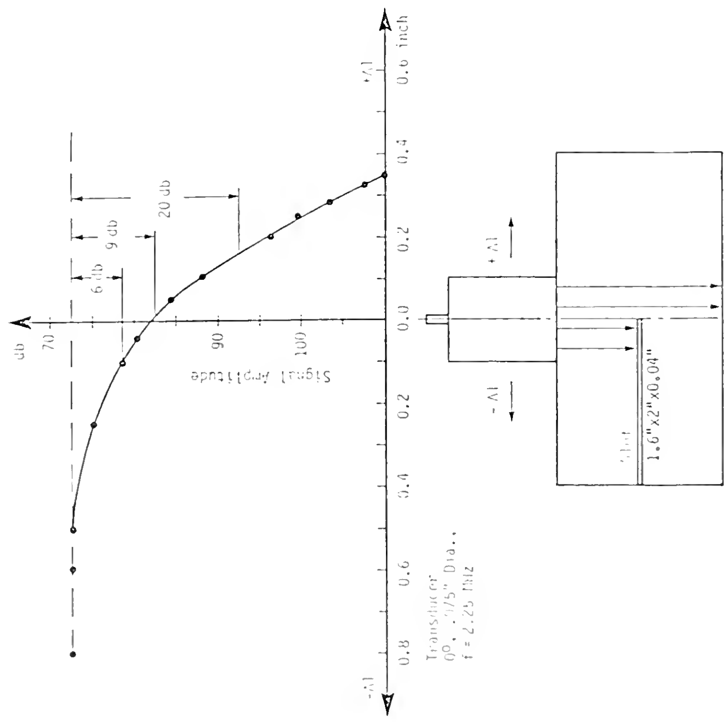


Figure 7 - Variation of Echo Signal From Slot in Steel Specimen.

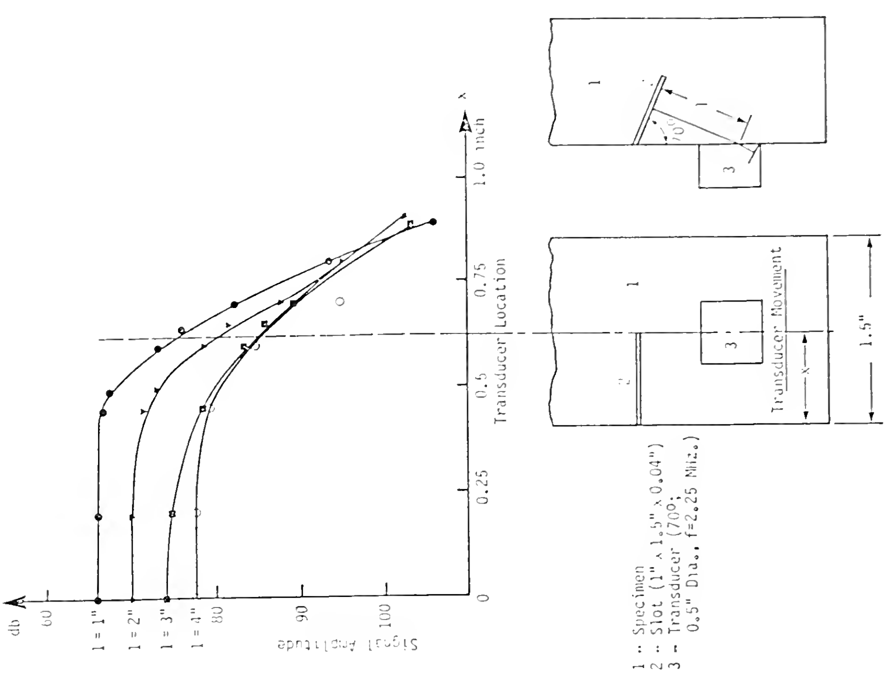


Figure 8 - Variation of Echo Signal From Slot From Lateral Movement of the Transducer.

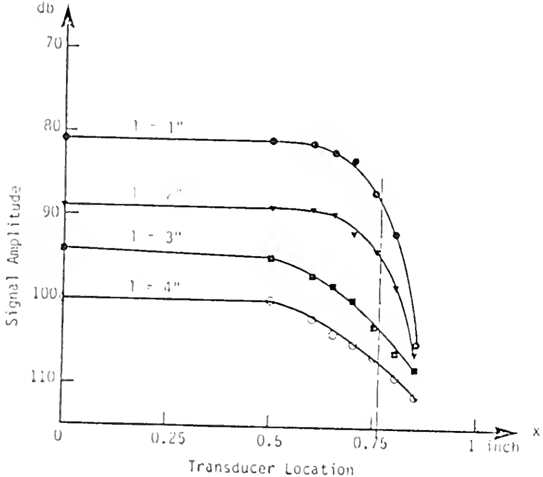


Figure 9 - Variation of Echo Signal From the Slot (Fig. 8) 1" x 1.5" x 0.04" From Lateral Movement of the Transducer (0.25" Dia.; f = 5 MHz.)

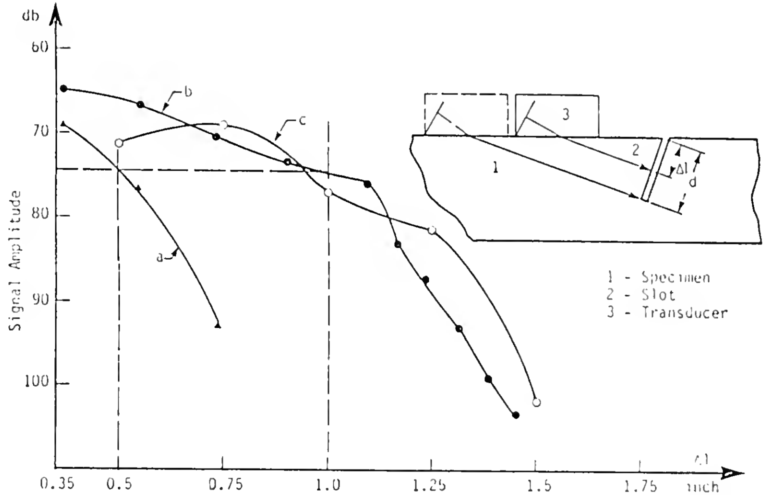


Figure 10 - Variation of Echo Signal From the Slot From Axial Movement of the Transducers.

(a) β = 70°; 2a = 0.5"; f = 2.25 MHz; d = 0.5"

(b) β = 70°; 2a = 0.5"; f = 2.25 MHz; d = 1"

(c) β = 45°; 2a = 0.5"; f = 2.25 MHz; d = 1"

β - Refraction Angle

2a - Transducer Diameter

f - Frequency

d - Depth of Slot

Vanishing Echo

An approximation of the defect size can be obtained by a vanishing echo technique. This practice is frequently used, particularly for ground rail inspection. When utilizing this technique, the instrument sensitivity is set by the maximum response from the detected flaw or by the benign reflection from some surface on the part be tested. (In rails, such reference reflectors are usually the side and end corners of the rail.) Echo signals from such reflectors were measured on the laboratory specimens. These measurements showed a significant variation of echo amplitudes depending on particular spots. Simulation of this procedure for measurement of the transverse defect in the laboratory specimen (Fig. 5) revealed much uncertainty in the determination of the defect outline, especially of its height (vertical dimension) as a result of these uncertainties. Also, uncertainties in the determination of the sensitivity level by reflection from the rail corner may cause missing a flaw, which is more dangerous than oversizing.

Back-wall Echo (BWE) Calibration

Sizing procedures based on the signal dropping techniques include determination of the maximum signal from the flaw. If the defect is large enough and the interrogative beam is perpendicular to its surface, the maximum signal might be equal or close to back-wall echo, i.e., response from the infinite plane.

Measurements of BWE and maximum signals from the transverse defects confirmed this assumption. Therefore, BWE is the most appropriate candidate for the reference signal and can be used for set-up of the testing sensitivity. Manufacturing of the reference block for obtaining BWE is much easier than that of other types of reference specimens. A length of rail of 7-10 inches can be used for this purpose, Fig. 11. Usage of the rail specimen as a reference block provides identical acoustical coupling conditions for both calibration and inspection. One specimen can be used to calibrate transducers of two different refraction angles, as shown by the calibration block in Fig. 11. Ends of the rail head were cut at angles of 70° and 45° for transducers with refraction angles of 70° and 45° respectively. Such a specimen can be made for other possible refraction angles such as 80° and 60°.

When calibration is performed, the amplitude distance correction (ADC) controls of the flaw-detector are employed for equalizing BWE at different distances. A typical ADC curve is shown in Fig. 12. Also, several curves corresponding to various N db down levels are displayed. These levels are used for determination of the defect border.

In Fig. 13, profiles of the transverse fissure in the rail specimen (Fig. 5) are shown for different N db down levels and flaw detectors used. Also in this figure, a profile obtained by the vanishing echo technique combined with the corner signal set-up is shown. In the last case, apparent dimensions of the defect seem to be enlarged compared to the techniques believed to be more accurate.

Qualitative evaluation of results of the analyses and experiments with different calibration techniques for sizing of the transverse defects are presented in Table 2. These analyses and experiments allowed development of recommendations for improvement of detecting and sizing transverse defects in rails.

RECOMMENDATIONS FOR SIZING OF TRANSVERSE DEFECTS IN RAILS

Main steps of the recommended procedure are outlined in Fig. 14. This procedure was employed for profiling of the transverse defects in rails in the Elvas-Sacramento section of the Southern Pacific Railroad, October-November, 1979, and in rail specimens at Federal Railroad Administration (FRA) Transportation Test Center (TTC), Pueblo, Colorado, January,

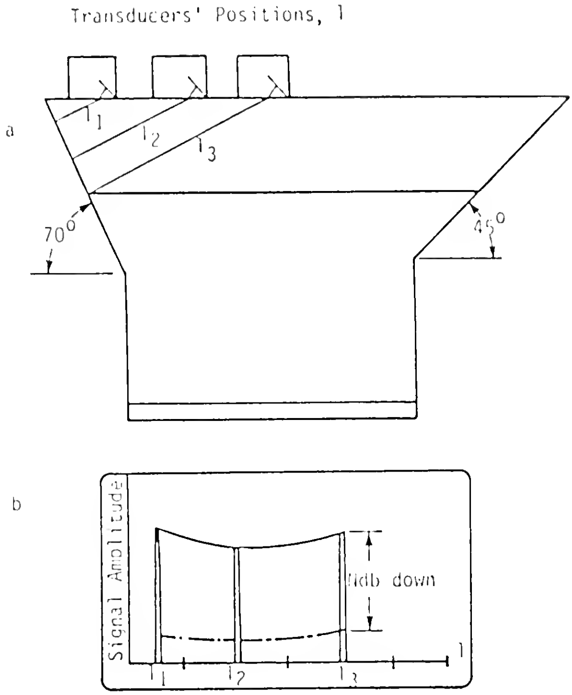


Figure 11 - Rail Calibration Block (a) and Calibration of the Flaw Detector Scale (b).

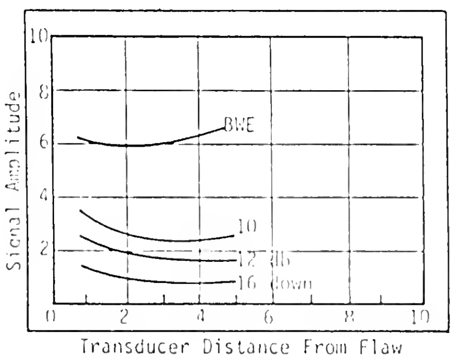
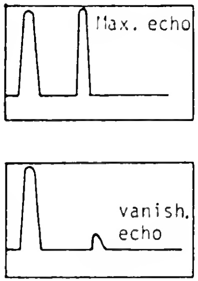
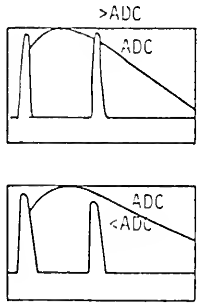
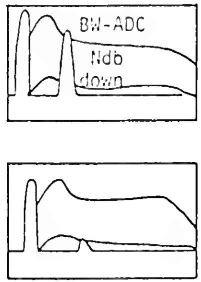
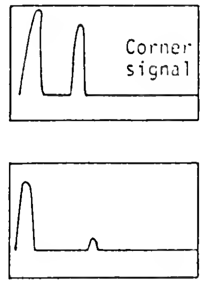


Figure 12 - ADC Curves on the Flaw-Detector Scale for Transducer 70°, 0.5" Dia., f = 2.25 MHz.

Table II - Calitration Techniques for Sizing Transverse Defects.

Technique	Vanishing Echo	FBHE	BWE	Corner Signal
Set-up	Maximum signal amplitude -vanishing echo	Flat bottom hole echo	Back wall echo and Ndb drop	Signals from side and end of the rail - Signal drop N or vanishing echo
Pictures on the screen of flaw detectors				
Advantages	Ref. blocks are not needed	Standardized set-up	-good approximation for transverse defects -simple manufacturing of reference blocks	Ref. blocks are not needed - Simple set-up
Disadvantages	low reliability of flaw detection and size estimation	-time consuming -need manufacturing FBH -possible variations in material properties	-possible variations in material properties	-low accuracy of size determination -probability of missing flaw may be significant

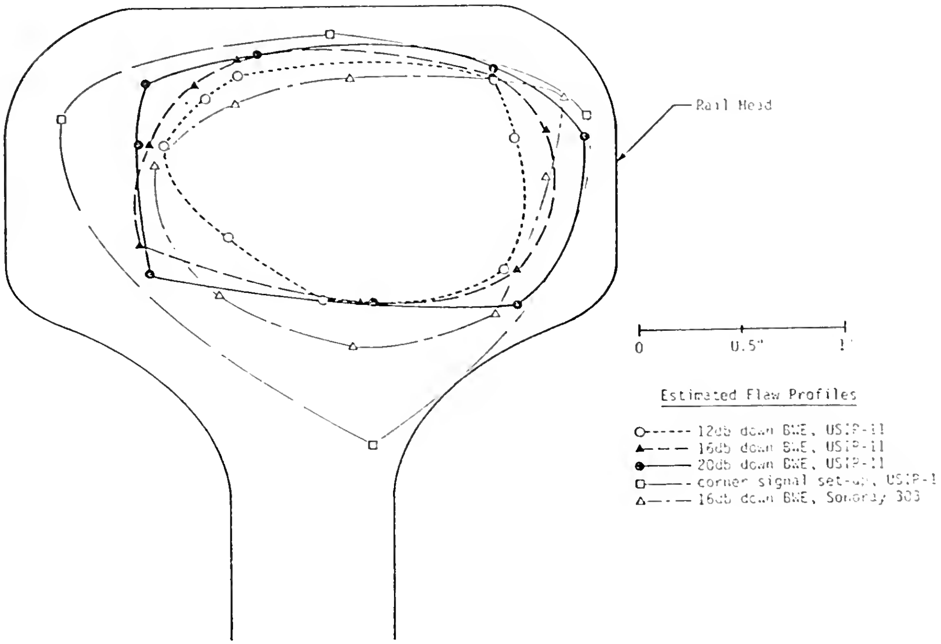


Figure 13 - Sizing of the Transverse Defect in a Rail Specimen.

SET UP DISTANCE RANGE OF THE FLAW DETECTOR ON SHEAR WAVES

SET ADC AND N_{DB} DOWN CURVES

CHECK ATTENUATION OF THE REFERENCE BLOCK AND THE RAIL TO BE TESTED

DETECT A FLAW

LOCATE FLAW'S BORDERS IN 4 - 8 DIFFERENT POINTS - (DISTANCE L AND TRANSDUCER'S POSITION X, Z)

CALCULATE FLAW SIZE

Figure 14 - Main Steps of the Procedure Developed for Flaw Size Determination (Transverse Defects).

1980. These tests showed the feasibility of the procedure that was developed, but revealed additional features that have been taken into account and included in the recommendations described in the following step by step procedure.

Calibration of the Distance Range

Calibration of the flaw-detector consists of distance calibration on shear waves and plotting of the amplitude-distance correction curves (ADC) on the screen of the flaw detector. Calibration of the distance range can be carried out on the standard reference block IIW, Fig. 15a. A transducer is set on the standard and moved to obtain the maximum response from the circumferential reflector C. The initial pulse must be set at the zero point of the distance scale. Adjusting the distance control, the echo must be set at the distance point equal to:

$$l' = l + l_w' \qquad (9)$$

where l is the length of the metal path in the specimen and l_l' is the equivalent path of the ultrasonic pulse in the transducer wedge.

$$l_w' = l_w \frac{C_{s2}}{C_{l1}} \qquad (10)$$

where l_w is the geometrical path (fig. 15a) C_{l1}—is the velocity of the longitudinal waves in the wedge material, C_{s2}—is the velocity of the shear waves in steel.

The final step in the distance calibration is adjusting a position of the echo pulse (Fig. 15b) at the distance point equal l (Fig. 15c). This can be done by means of the smooth delay control of the flaw detectors.

Set-up of the ADC and N db Down Curves

This set-up should be carried out by means of the reference rail specimen, Fig. 11a, with the edge cut at the angle corresponding to the beam angle of the transducer. For this operation, the flaw detector must contain 3-point swept gain control (flaw detectors "Sonoray" 303, USM-2MT, USIP-11, USL-38). By using this control, amplitudes of the echoes can be approximately equalized at three positions of the transducers (e.g., at points $l_1 = 0.5''$, $l_2 = 2''$, $l_3 = 4''$).

Tops of these indications should be marked on a transparent scale bonded to the instrument screen for the particular type of the transducer and connected with a smooth curve. All settings of the controls affecting gain and distance calibrations should be recorded.

After building the ADC curve as described above, the N db down ADC curve must be set, Fig. 11b. This can be done by introducing additional attenuation N db and marking of the indication heights at the corresponding points l_1, l_2, l_3. A reasonable value of N db drop was determined to by 16 db. If reference blocks are used for the gain calibration, ultrasonic attenuation of the tested rails and reference blocks should be compared by using the normal probe and comparing echoes from the rail bases. If the difference between attenuations is not greater than 0.25-0.3 db/inch, no correction should be made. Otherwise, attenuation differences should be taken into account.

Flaw Detection

Flaw detection as a part of the hand held device rail inspection is carried out for the verification of the indications obtained by the detector car. Detection is followed by the

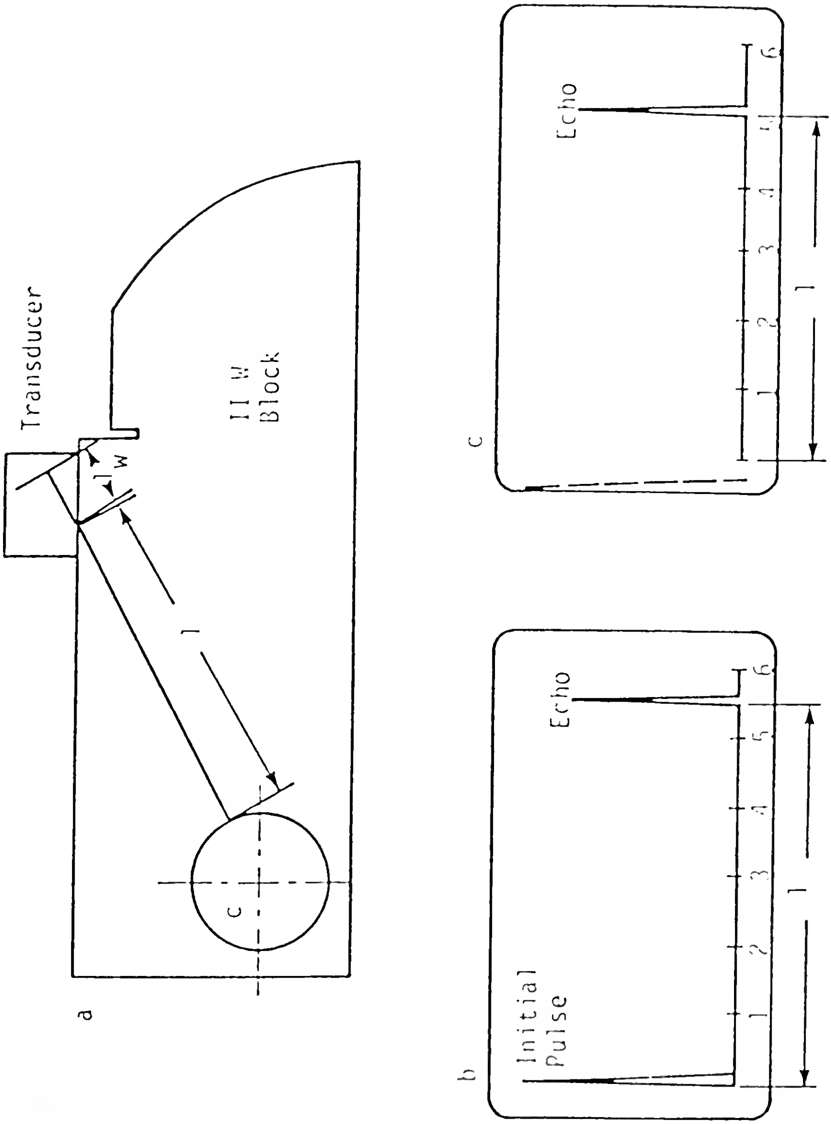


Figure 15 - Distance Calibration.

further flaw characterization and sizing. Ultrasonic inspection must be accompanied by careful observation that contributes to the determination of the flaw type, its location and dimensions.

Flaw Characterization and Size Determination

After a flaw is found, the maximum amplitude of the indication must be determined as well as coordinates of the corresponding point (1_M, z_M, x'_M), Fig. 16. If the maximum amplitude of the indication A_m is 3 or more db lower than the BW echo level A_B from the reference block, transducers with different angles must be tired (e.g., 80°, 60°, 45°). The optimum angle beam transducer is the one which provides the maximum response. This means that the beam of such a transducer is perpendicularly oriented to the defect area.

It is possible that some sections of a large defect may have different orientations. In this case, different transducers should be used for interrogation of these sections with optimum orientation of the ultrasonic beam.

If, for the optimum transducer, the difference between ADC level and maximum response $\Delta A_m = A_b - A_m \leqslant 3$ db, the defect should be characterized as a continuous one. If this difference is greater than 3 db but scanning of the defect with different transducers reveals its continuous character (i.e., the ultrasonic signal does not decay significantly for probe movement within a distance approximately equal to the probe dimensions, or several maxima appear within this distance) the instrument sensitivity should be increased in magnitude ΔA_m. This action may compensate for a difference between the actual defect orientation and orientation of the available ultrasonic probe. If, after scanning with different transducers, it is determined that the defect is not continuous and the difference $\Delta A_m > 3$ db, the defect should be evaluated as a small flaw. The technique recommended for sizing small defects are different from that for continuous defects. The two techniques are described in the following sections.

Continuous Defect

In the case of the continuous flaw, coordinates of several extreme points of the defect must be determined (6, 8, at least 4), (Fig. 6). Depending on the instrument used, a data format might vary slightly. Consider a data collection procedure implemented for the flaw detector "Sonoray" 303, set up as it was described above. Assume also that the orientation of the defect investigated does not vary significantly. After a flaw point M with the maximum response is found (Fig. 16b), the flaw should be scanned to get an idea of how large it is. Then, several extreme points are to be interrogated. Metal distance 1, transverse coordinate z', and axial coordinate x' are to be recorded for each point. (If instrument USL-38 is used, a flaw depth h and surface distance x_s can be determined instead of distance 1.) These coordinates 1, z', and x' are intermediate. For plotting a defect profile and determination of its size and location, the coordinates of interest are computed:

the flaw depth

$$h = 1 \cos\beta \tag{11}$$

transverse coordinate

$$z = z' + \frac{d}{2} \tag{12}$$

the axial coordinate

$$x = x' - 1 \sin \beta \tag{13}$$

a

b

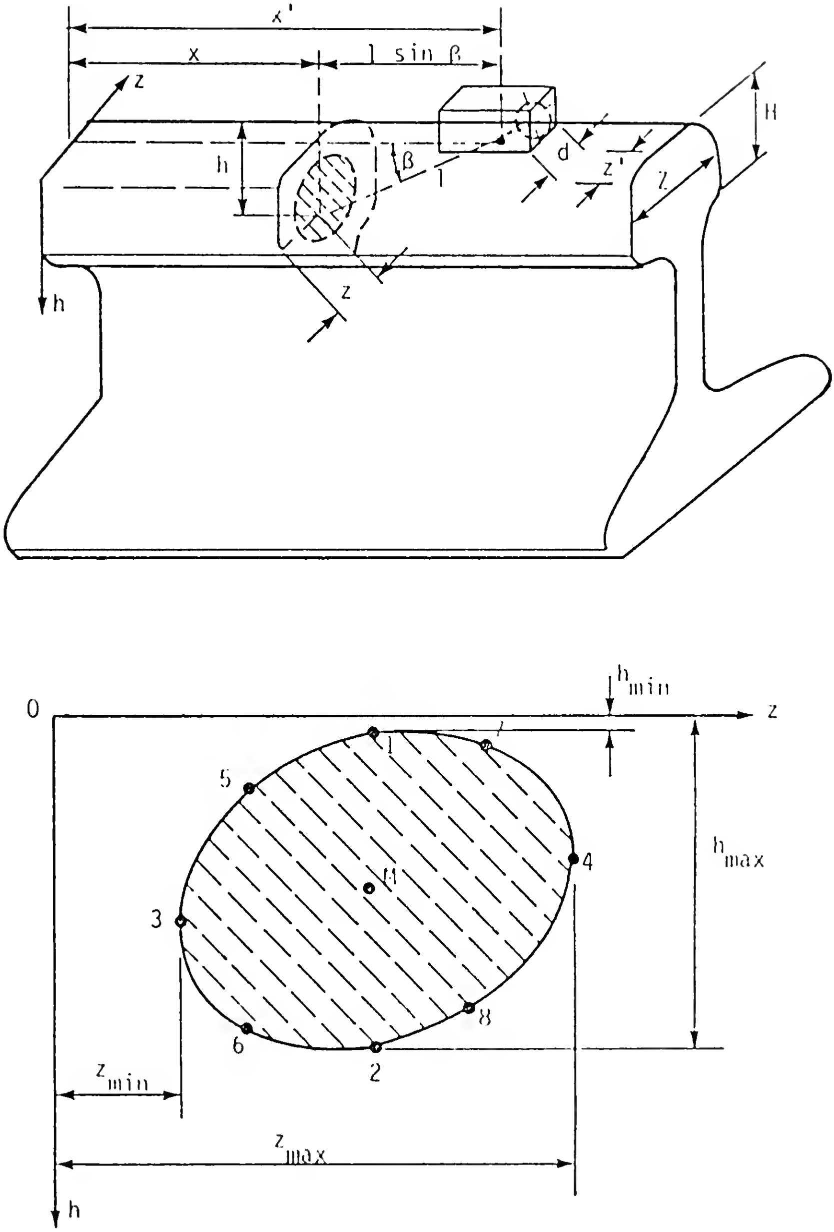


Figure 16 - Determination of Flaw Coordinates.

where β is the angle of ultrasonic refraction, and d is the width of the transducer wedge.

For defects of the transverse fissure type, a percent size can be approximately computed from the formula

$$S(\%) = \frac{\Delta z \cdot \Delta h}{Z \cdot H} \times 100 \qquad (14)$$

where $\Delta z = z_{max} - z_{min}$, $\Delta h = h_{max} - h_{min}$

are differences between maximum and minimum coordinates z and h, respectively, Z is the head width, and H is the head height.

For large transverse defects with borders close to the top surface and sides of the rail, flaw sizes can be underestimated because of two factors. The first factor is a dead zone of the transducer, limiting characterization of subsurface defects because of noises following the initial pulse. The second factor is the curvature of the rail edges that worsens acoustical coupling of the transducer and the rail. To partially overcome these factors and improve the accuracy of the near-surface flaw area examination, transducers with minimum dead zones and small dimensions should be used. Also, defects' borders in the transverse direction can be more accurately determined with angle beam probes located on the side surface of the rail[10]. In the case of detailed fractures which usually initiate from the rail surface, visual information must supplement ultrasonic measurements, and a flaw size can be determined from flaw profile plotting, accounting for results of visual observation.

Use of A Computational Device

Determination of flaw coordinates and size requires some computations that take time. This time can be reduced if inspection is assisted by an automatic calculating device. Also, possible computational errors can be eliminated by this means. The programmable calculator HP-41C has been applied for this purpose. Use of the hand-held computational device allows determination of a flaw size in real time of the inspection. Test results and the profile of the detected flaw can be printed out by means of the portable printer. A flow-chart of the program developed for computing the size of a continuous flaw with symmetrical or quasi-symmetrical shape is shown in Fig. 17. Also, a program was developed for profiling such a flaw. These programs utilize a numerical input of data. Future work to make the technique fast enough for general field use calls for development of a system with direct transfer of inspection data from the flaw detector to the computational device (analog to digital conversion). Also, expanding of programmable techniques for sizing flaws with different shapes is planned.

Small Defects

Small defects can be evaluated by using the distance-gain-size (DGS) method modified for angle beam transducers. The method of DGS (see diagram, Fig. 18) is based on use of graphic dependencies between relative variables: amplitude gain G (signal response), distance between transducer and reflector related to near zone of transducer l/ln, and defect size related to transducer size b/a. By knowing 2 of these variables, e.g., gain G and distance l/l_n, it is possible to determine the third variable, relative defect size.

Originally, the DGS diagram was developed for contact straight-beam transducers and disk-shaped defects[2], but it can also be applied for angle-beam transducers by using some approximations and transforming a 2-medium "transducer-tested material" system into a mono-medium system.

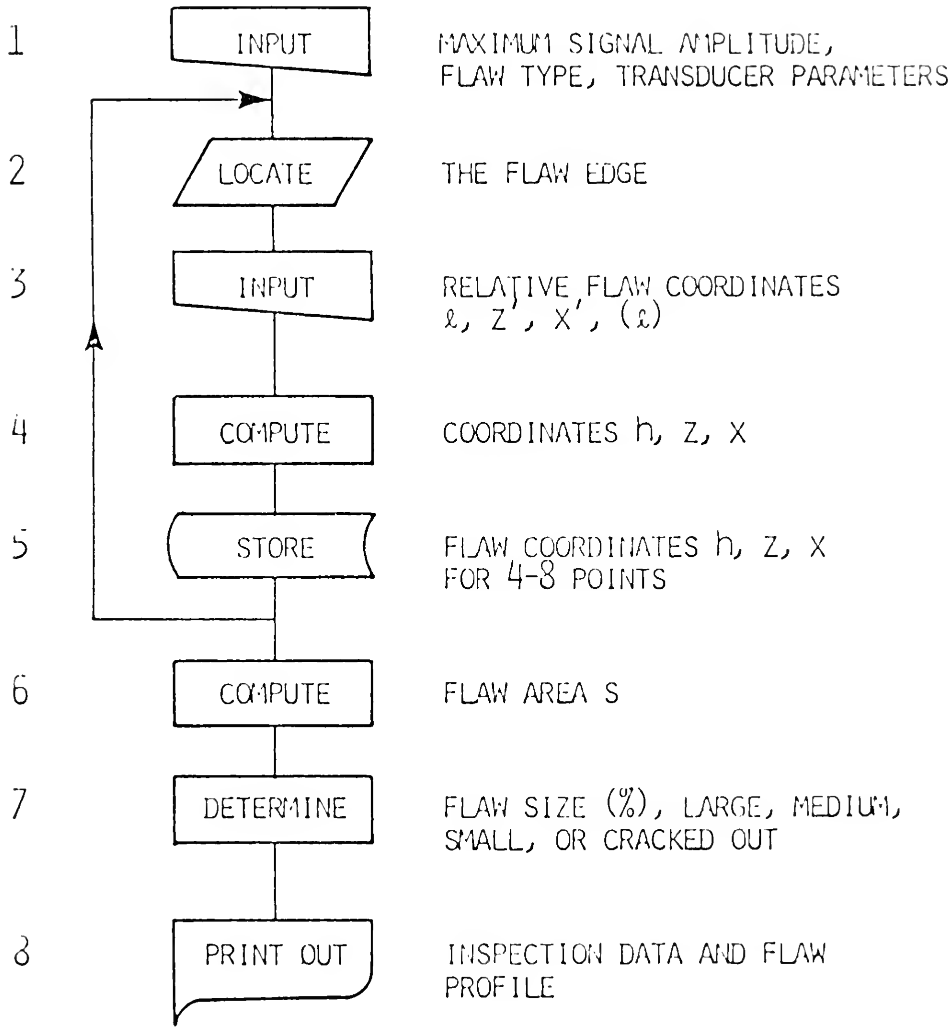


FIGURE 17 - FLOW CHART OF A PROGRAM FOR CALCULATION OF CONTINUOUS DEFECT SIZE.

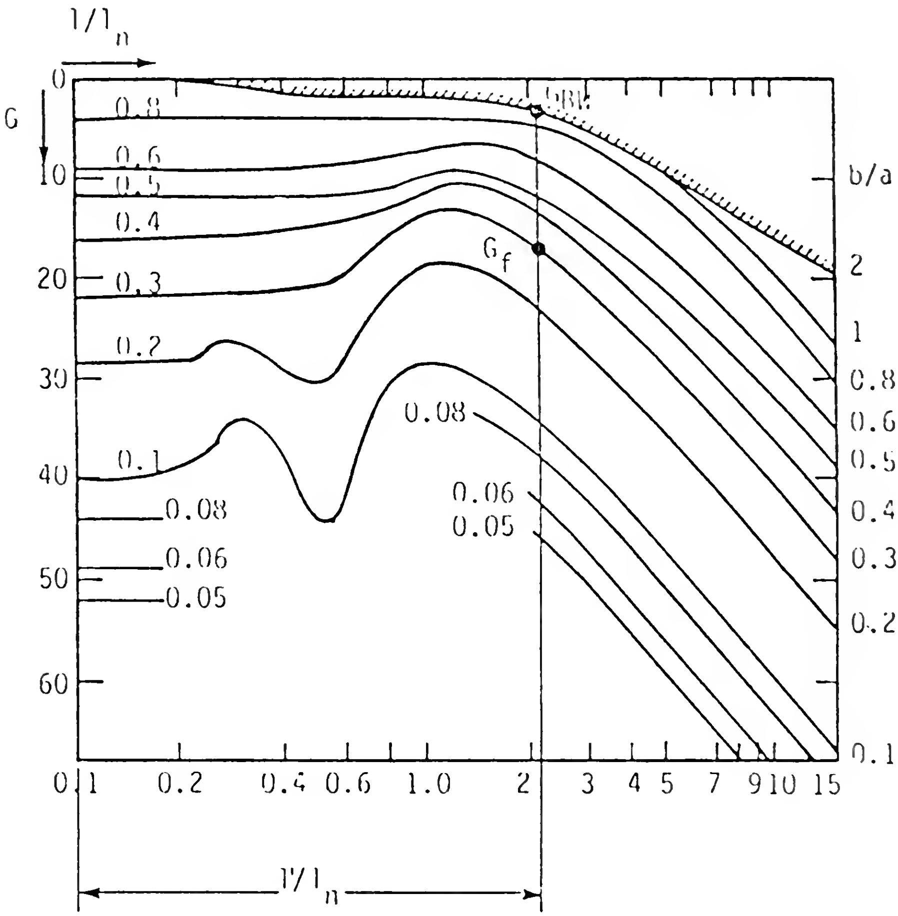


Figure 18 - DGS Diagram.

The following procedure can be carried out for sizing a "small" defect by using the DGS method.

1) Compute the near zone l_n of the transducer and distance l''_w for the equivalent mono-medium system:

$$l_n = \frac{a^2\cos\beta}{\lambda\cos\alpha} \tag{15}$$

$$l''_w = n l_w \frac{\cos\beta}{\cos\alpha} \tag{16}$$

Here 2a is diameter of the transducer,
α is an angle of the incidence,
β is an angle of the refraction,
l_w is geometrical path in the transducer wedge,
λ is a length of the ultrasonic wave in the tested material
$n = \frac{\sin\alpha}{\sin\beta}$ is the refraction coefficient
(This step may be done before measurement.)

2) Measure the maximum amplitude of a signal from the flaw A_m, metal distance l and amplitude of BW echo on the rail reference specimen A_{bw} at the distance l

3) Compute equivalent distance

$$l' = l + l_w \tag{17}$$

and relative distance l'/l_n

4) Compute gain factor

$$G_f = G_{bw} + (A_m - A_{bw}), \tag{18}$$

where G_{bw} is the gain factor determined from the DGS diagram (Fig. 18) for BWE and relative distance l'/l_n

5) Determine ratio b/a from the DGS diagram for the relative length l'/l_n and gain factor G_f

6) Calculate equivalent diameter of the detected flaw

$$2b = (b/a) \times 2a \tag{19}$$

Variation of Size Characterization Performance

The recommended procedures for size characterization can provide greater accuracy of defect sizing than is now achieved during regular inspection. However, the completeness of their performance should depend on the kind of the inspection and specific requirements. If the accuracy of size determination is a major requirement, all the procedures should be performed completely as recommended. In the case of routine inspection when the time for characterization of defects is limited and precision in sizing is not required, some variations or abbreviations of the recommended procedures seem to be reasonable, as follows:

*Detection of a flaw in the mode of maximum sensitivity is to be suggested.
*Flaw coordinates can be determined only in four extreme positions of the probe (front, back, left and right-most).
*A single flaw can be qualified as "small" without its size evaluation.

CONCLUSIONS

1) Sizing of defects in rails is influenced by many factors such as orientation, shape, and type of flaw; accuracy and mode of the ultrasonic equipment calibration; and training and experience of ultrasonic inspectors.

2) A survey on the flaw characterization and sizing of defects in rails has indicated the necessity for improvement of the ground rail inspection, its standardization and possible automation.

3) Modelling of ultrasonic inspection for transverse defects has shown that the maximum response from a continuous defect can be approximated by the echo from an infinite plane perpendicularly oriented to the direction of the ultrasonic beam. As a result of this conclusion, a simple reference block manufactured from the rail has been proposed and implemented for the sensitivity calibration.

4) Analysis and laboratory experiments of "dropping" techniques used for sizing of transverse defects have established a reasonable value of the signal drop representing the flaw border threshold.

5) Field inspections have shown the feasibility of the procedure that was developed and revealed additional inspection features that have been included in the preceding *Recommendations for Sizing of Transverse Defects in Rails.*

6) Accuracy and efficacy of the flaw size characterization in rails can also be improved with the assistance of a hand-held programmable calculator for computing an apparent flaw size and plotting its profile.

7) Results of the analysis and recommendations may be useful for development of specifications or standards on ground rail inspection with hand-held ultrasonic devices.

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ACKNOWLEDGEMENT

The authors would like to thank the Association of American Railroads for the support of this work. Also, help of the Southern Pacific Railroad and the FRA Transportation Test Center is deeply appreciated.

APPENDIX A
RESULTS OF SURVEY ON FLAW CHARACTERIZATION AND SIZING OF THE DEFECTS IN RAILS

ITEM	TEAM A	TEAM B	TEAM C	TEAM D	TEAM E	TEAM F
1. Nondestructive techniques being used for flaw characterization						
a) Ultrasonics (UT)	Yes	Yes	Yes	Yes	Yes	Yes
b) Magnetic (MT)	No	Yes	No	No	No longer	No
c) Eddy-current (ECT)	No					No
d) Other (specify)	No					No
2. Flaw detectors (instruments being used)						
a) Brand		USM-2	Magnaflux PS702	USM-2, USL-31, USL-32, 35, 38	USM-2	1) Sperry, UJ 2) USM-2
b) Manufacturer	Ry Co.	KBI*	Magnaflux Corp	KBI	KBI	1) Sperry Div. of Automation Industries 2) KBI
c) Evaluation of instrument Satisfactory	Yes	Yes	Yes	Yes	Yes, but not entirely	Yes
d) List any drawbacks			Miniature toggle switches fail	Need for operator training and experience with test procedures	1) Not weatherproof 2) LMO connectors not satisfactory in long terms	—
e) List needed improvements			CRT definition could be better	To make testing completely independent of operator skills**	1) Weatherproofing 2) Use BNC connector	—

*KBI—Krautkramer-Branson, Inc.
**USL 35 and 38 provide some relief in this respect.

ITEM	TEAM A	TEAM B	TEAM C	TEAM D	TEAM E	TEAM F
3. Transducers						
a) Type and manufacturer of ultrasonic transducers	Railway Company		Sperry, KBI (Aero-tech), DAPCO, Magnaflux	Krautkramer D-Series-KB Aerotech α and γ series	DAPCO Lead Zirconate	1) Sperry 2) Aerotech-KBI
b) Straight Beam	Yes	Yes	—	Single element and T/R types	Yes	Yes, 0°
c) Angle Beam—What is the angle	Variable from 45 to 70°	70°	75°-70°	37, 45, 60, 70°	75°, 75°	75° and 70
d) Frequency	2.25 MHZ	2.5 MHZ	2.5 MHZ	2.0, 2.25, 4.0 MHZ	2.25 MHZ	2.25 MHZ
e) Dimensions	.5″ dia.	.5″	½″-¾″ round 0° (½″ × ½″)-(½″ × ⅝′), 70°-45°	0.5″-1.25″ dia. (0.25″ × 50″) – 0.75″ × 1.0″	0°-½″ Dia. 45°-75°-½″ × ¾″	¼″ × ¼″ ½″ Dia.
f) Types and main characteristics of non-ultrasonic transducers (sensors)	None	—	None	—	—	—
4. Surface conditions and coupling						
a) How do you prepare the rail surfaces for testing?	—	None, except under exceptional circumstances	No preparation	Remove any loose scale	Wetting agent only	Apply liquid
b) Type of coupling agent for UT	Water with additives	Water (+ methanol in winter), diesel fuel for handcheck	Water	Lubricating oil, proprietary couplants	Summer: water with some water-soluble oil Winter: water with ethylene glycol (auto anti-freeze)	Water, in extreme cold weather, use SAE 10 motor oil
5. Calibration techniques						
a) Do you use back echo from the rail base as a reference signal for UT?	Yes	Yes	Yes	Feasible only with normal beam probes.	Yes	Yes
b) Do you use other reference signals?	No	—	Signals from upset metal in welded rail	Signals from 5 mm side drilled hole in rail head, 25 mm below top and from standard test blocks	No	Test block

ITEM	TEAM A	TEAM B	TEAM C	TEAM D	TEAM E	TEAM F
c) Type or description of standards (reference blocks) being used for calibration	Some prepared rail samples	None	None	II W block ASTM E 428 distance amplitude blocks	Gain and gate settings adjusted the problems related to false indications	Special
6. Minimum size of detectable flaws						
a) Please briefly describe how you evaluate the type of a detected flaw	Standard classification–small, medium, large	By screen display on hand check scope considering characteristics of probe being used	Based on current-voltage drop, visual inspection of defective rail correlation with broken rails, ultrasonic indication and location of the transducer	By maximum signal and distance from surface based on calibration of instrument	By initial readout indication, visual inspection and ultrasonic handcheck	—
b) What are minimum sizes of flaws which have been reported for major defects in rails?						
Transverse Fissure (TF)	5%	10%	All sizes	2 mm dia	Under 5%	UT reject size 10% of head
Detail Fracture (DF)	3%	10%	"	2 mm dia	That applies to most defects	"
Compound Fissure	Small	30%		UT response is difficult to evaluate		"
Engine Burn Fracture	Intermediate	50%				"
Head and Web Separation (HSW)	3"	1½"		≥ 1"		"
Vertical Split Head	6"	6"		Depth 1.5 mm can be detected		"
Base Fracture	None					"
Other Defects (Specify)	Split web, oxyacetylene welds, electric flash butt welds, thermite welds, fillet rocks, bolt hole breaks	BHC+	BHC, piped rail	HSW, BHC-3 mm		BHC

ITEM	TEAM A	TEAM B	TEAM C	TEAM D	TEAM E	TEAM F
6. c) What sizes are rejectable for major types of flaws		All flaws detected are rejecte	Determined by the FRA Track Safety Standards (12/'72)	TF-8-40 mm DF-6-10 mm		
7. Sizing techniques						
a) Do you use any written procedures for sizing?	None	None	None	BHC-6-12 mm DGS method is recommended	No	No
b) How do you define flaw limits using UT?	db reference is denied	6 db	Vanishing echo	A signal greater than the calibration level	None	Interpretation of C.R.T. display
8. Please report data about correlation between size of flaws nondestructively evaluated and their confirmation by subsequent destructive tests		Although the size evaluation can be misleading, predicted sizes are quite close to actual ones	No data available	Flaws evaluated by the DGS method are never smaller than indicated, and are often many times larger	The sizes of TF's are given too large	Generally not correlated
9. Comments						
Have you any other comments regarding flaw evaluation and sizing in rails?	None	None	Flaw type evaluation is especially difficult with angle beam ultrasonics, and sizing is only educated guessing at best.	Operator skills are highly variable. Some are incompetent. Training and practice are obviously necessary. Qualifications to ASNT standards is suggested as a minimum.	Most railroads are too much pressured to get more miles of track tested than to be worried about actual size of rail defects. We cannot spend much time on each verification event. There are too many tape indications to check.	We are obligated to determine flaw size in order to classify the defect by FRA Track Safety Standards. In order to maintain rail testing production at an acceptable level, an approximation of flaw size must be made quickly. The presence of a defect, regardless of size, requires remedial action. (But, FRA rules specify different remedial actions depending on flaw size.)

Ballast Research

by Ernest T. Selig*

INTRODUCTION

The type and condition of ballast and subgrade are key factors in the performance of railroad track structures. During the service life of the track, permanent strains accumulate in the substructure, causing permanent deformation which is visible as deterioration of surface and line. Whether the structural deficiency is in the ballast, the subgrade, or the track superstructure (cross-tie and up), or whether the track degradation is caused by an overloading of the normal traffic-carrying capacity of the track, the correction is usually affected by reworking the ballast. However, this reworking in turn changes the ballast physical state and leaves it prone to increased deformation and hence, further track settlement.

Specifically, the mechanical properties of ballast that are involved in the ballast-compaction/track-performance problem are defined by the physical state of the ballast. Physical state is a function of 1) the in-place density, and 2) the ballast particle index properties, such as size, shape, angularity, and hardness.

The in-place density of ballast is a result of a compaction process. Typically, the resulting initial density is created by maintenance tamping, and subsequent density changes result from train traffic as well as environmental factors. Experience has shown that tamping does not produce a high degree of compaction, and there is clearly little geometry control in achieving compaction by train traffic. Therefore, consideration has been given to additional compaction during maintenance using vibration methods.

A research project was undertaken to learn more about ballast compaction. Questions to be considered included:

1. How much can ballast be compacted?
2. What degree of compaction can be achieved by various mechanical processes involved in track construction and maintenance operations and by train traffic?
3. How much additional compaction can be achieved by altering normal processes or by adding new processes such as those involving compactors?
4. What are the best means to achieve compaction?
5. What are the benefits of providing additional compaction in comparison to the effort and cost involved?

Preliminary research revealed: 1) contradictory opinions as to the extent of compaction associated with maintenance and traffic, 2) little quantitative data from which to draw conclusions, 3) an uncertainty as to the relative importance of immediate and long-term benefits of crib and shoulder vibratory compaction, and 4) a need for methods to measure and assess compaction.

Ballast materials possess different particle physical and chemical properties. Many standard laboratory index property tests, such as those for abrasion resistance, absorption, shape and soundness, are currently utilized to quantify and categorize the relative merits of these different ballast types. However, an individual index property test does not by itself provide a direct indicator of expected field performance of the ballast matrix in the track structure under traffic loading and environmental conditions. This situation is particularly evident from the differences of opinion as to what values of what index properties represent the best ballast.

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Thus, presently, research is still needed to determine how to classify ballast in relation to its effect on track performance.

The purpose of this paper is to provide some answers to the questions about ballast compaction and the physical state of ballast in track based on the results of recent research. The influencing factors will be examined and implications of the research results to track performance and maintenance practice will be discussed.

BALLAST PHYSICAL STATE MEASUREMENTS

The lack of suitable methods of measuring the degree of ballast compaction became evident after a review of the state-of-the-art of ballast compaction. Therefore, effort was devoted to developing methods that could be used to measure the physical state of ballast in the field. After considering possible alternatives, the three most promising methods were selected for study. These methods are as follows:

1. The ballast density test (BDT), which determines the in-situ density as a direct measure of compaction (Refs. 1, 2). Conceptually, this test involves determining the volume of a membrane-lined hole excavated in the ballast by carefully measuring the amount of water required to fill the hole. The density equals the weight of ballast removed divided by the volume of the hole.

Used in conjunction with this measurement is a reference density test, which provides the means to assess the relative amount of ballast compaction achieved in the field (Ref. 2). This test involves compacting samples of ballast in a 12-in.-diameter by 12-in.-high steel container with a special rubber-tipped drop hammer. The reference density is essentially the maximum density achieved with this technique.

2. The plate load test (PLT), which determines the vertical ballast stiffness as a measure of the effect of compaction on the ballast physical state (Refs. 2, 3, 4). A 5-in.-diameter steel plate is seated on the ballast using gypsum plaster. The plate is then loaded vertically and the measured contact pressure per unit plate settlement is taken as an index of ballast stiffness.

3. The lateral tie push test (LTPT), which determines the resistance offered by the ballast to an individual tie displaced laterally as an indirect measure of the physical state and compaction (Ref. 2). In this test, the tie is displaced perpendicular to the rail after removing the fasteners so that the tie is disconnected from the rail and carries no vertical force other than its own weight.

In this paper, results with all three of these methods will be used to illustrate the effects of maintenance and traffic on ballast properties. These results have been obtained in a number of field tests representing a variety of track and maintenance conditions, as well as in laboratory investigations.

DENSITY MEASUREMENTS

Density measurements were obtained in a variety of field situations. The observed trends have been summarized in Fig. 1. The four typical track conditions represented are: 1) after initial tamping during new construction or complete ballast undercutting, 2) after compaction immediately following tamping, 3) after accumulation of traffic, and 4) after maintenance tamping.

Density is represented in Fig. 1 as percent compaction, which is the ratio of measured density to the laboratory reference density, expressed as a percent. The smooth boundary of the container used for the reference density test gives a density value that is systematically lower than the field measurement method gives, even though the actual density state in the two cases may be identical. Thus a percent compaction greater than 100 does not necessarily mean that the reference density test produces less compaction than occurs in the field.

The crib density measurements were obtained with essentially a full crib. Thus the crib density values represent ballast density between adjacent ties above the level of the tie bottom. The under-tie ballast density values were obtained in the tie-bearing area after emptying the cribs adjacent to a tie and then carefully removing the tie.

The initial tamping condition (Fig. 1a) represents a newly ballasted crib. The density along the crib was the lowest of any situation. Although the center density appeared to be slightly higher than the tamped zone, the density was relatively uniform along the crib. The density was greatest in the tamped zone under the tie. Apparently when the ballast is very loose, the vibratory tamping operation will densify the ballast. The density was lowest in the center under the tie, where the ballast loosely falls in during the track raise.

After vibratory crib and shoulder compaction (Fig. 1b), the density is increased significantly in the crib near the rail where compaction is applied. A slight increase in density beneath the tie near the rails may also occur. However, no significant density change appears to occur in the center of the track.

In some cases, the uniformity in the measured ballast density from one tie to another appeared to be improved by compaction, compared to the tamped-only conditions. As Hardy (Ref. 5) asserted from a series of field tests at a Canadian National line, the uniformity of ballast density distribution along the track could be one of the important benefits of using crib and shoulder compaction.

Traffic was observed to produce the greatest amount of compaction (Fig. 1c). Substantial increase in density occurred at all locations. The crib density was uniform and relatively high, apparently as a result of traffic vibration and perhaps cyclic loading from the sides of the ties. The density under the tie increased to a high level, particularly near the rail seat, where the tie-ballast contact pressures were greatest. However, the center zone under the tie also was compacted by the traffic loading. Although the level of compaction did not reach the same percent as near the rails, the greatest increase in density occurred under the center of the tie where the density was initially the lowest before traffic.

The effects of maintenance tamping on the ballast density change can be seen in Fig. 1d. Compared to the measurements after traffic, ballast tamping in a track previously subjected to traffic consistently loosens the ballast layer regardless of location of measurements. The density decrease was quite significant in the rail area, almost totally eliminating the compaction achieved by the traffic after initial tamping. Even in the center where no insertion of tamping feet was made, the ballast density is shown to have consistently been reduced, although by the smallest amount of any location.

The amount of density decrease from maintenance tamping appears to be dependent on various factors such as ballast type and condition, and also on the amount of raise during tamping. The greater the raise, the greater the density reduction.

PLATE STIFFNESS MEASUREMENT

Ballast stiffness was measured in the field using the plate load test in track conditions similar to those for the density test. The results are summarized in Fig. 2. The stiffness was represented by the ballast bearing modulus B_k, defined as the plate load per unit area required to displace the plate downward by a specified amount, which is usually 0.1, 0.2 or 0.3 in., divided by this displacement.

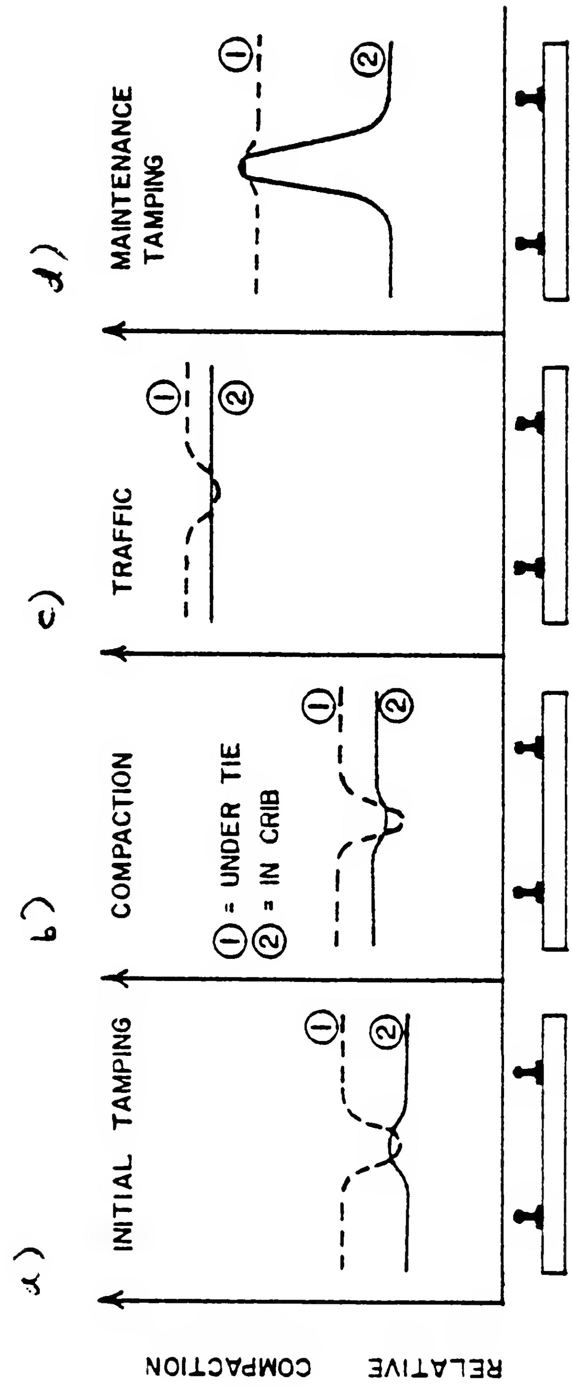

RELATIVE COMPACTION
a)
INITIAL TAMPING
1
2
b)
COMPACTION
1 = UNDER TIE
2 = IN CRIB
1
2
c)
TRAFFIC
1
2
d)
MAINTENANCE TAMPING
1
2

Figure 1

The trends in Fig. 2 are similar to those for density. After initial tamping, the bearing index was low (Fig. 2a). The lowest values were in the center of the track. In the tamped zone under the tie, the ballast stiffness was much greater than in the crib, where the ballast was left in a loose state after withdrawal of the tamping tools.

Crib compaction approximately doubled the bearing modulus in the crib where compaction was applied (Fig. 2b). A smaller increase occurred under the tie in the same area. This increase is believed to be partly caused by a ballast density increase under the tie and partly by increased lateral confinement from the compaction accomplished in the ballast directly below the crib. In the center of the track, which is away from the zone of compaction, little change in the ballast stiffness resulted from crib and shoulder compaction.

The addition of traffic greatly increased the ballast stiffness under the tie in the area near the rails, and also significantly increased the stiffness throughout the crib (Fig. 2c). A much less pronounced increase took place in the center beneath the tie. However, in time as center binding develops, the bearing stiffness will undoubtedly increase to a much higher level beneath the tie center.

Results are not shown for the condition following maintenance tamping. However, the observations for density indicate the expected trends. Tamping will decrease the bearing stiffness greatly in the zone of tamping penetration in the crib. The stiffness will also decrease in the tamped zone beneath the tie. The greater the raise, the greater will be the stiffness reduction beneath the tie.

Ballast bearing modulus is correlated in Fig. 3 with percent compaction. Ballast stiffness increases with density, but at an increasing rate. This trend indicates that small increases in density after some compaction has occurred that might be hard to detect with the density test, might still result in a significant stiffness increase.

Both the density and bearing stiffness values in the center of the track depend heavily on the track traffic and maintenance history. Results measured in the center of the track after any particular maintenance cycle do not necessarily represent the effects of that operation, because tamping and compaction are not performed in this location. A small raise will not result in much reduction in ballast stiffness and density in the center of the track. However, a large raise will result in a big decrease because under the tie, the ballast is deposited loosely by rolling and sliding, and in the crib, the replacement ballast needed is also deposited in a loose state.

The in-situ plate load test (PLT) has previously received very little attention for measuring the relative changes in the ballast physical state. However, Peckover (Ref. 6) utilized the PLT as a means of evaluating the effectiveness of different track maintenance operations. His plate size and test procedures were similar to those described in this paper. In general, the trends observed by Peckover are consistent with those shown in Fig. 2.

The density and plate test data were examined to estimate the amount of traffic required to produce the increase in ballast density and stiffness equivalent to that achieved during maintenance by crib and shoulder compaction. Unfortunately, insufficient data are available to clearly show the rate of change of ballast physical state with traffic, and field data variability is large. In addition, the results are a function of conditions prior to maintenance. However, the data suggest that roughly 0.1 to 0.2 MGT of traffic would produce the same change as the compaction. Contradictions to this conclusion do exist. For example, Birman and Cabos (Ref. 7), using nuclear density measurements, observed that the initial density difference before traffic, which had resulted from different ballast compaction methods, still remained preserved even after significant traffic.

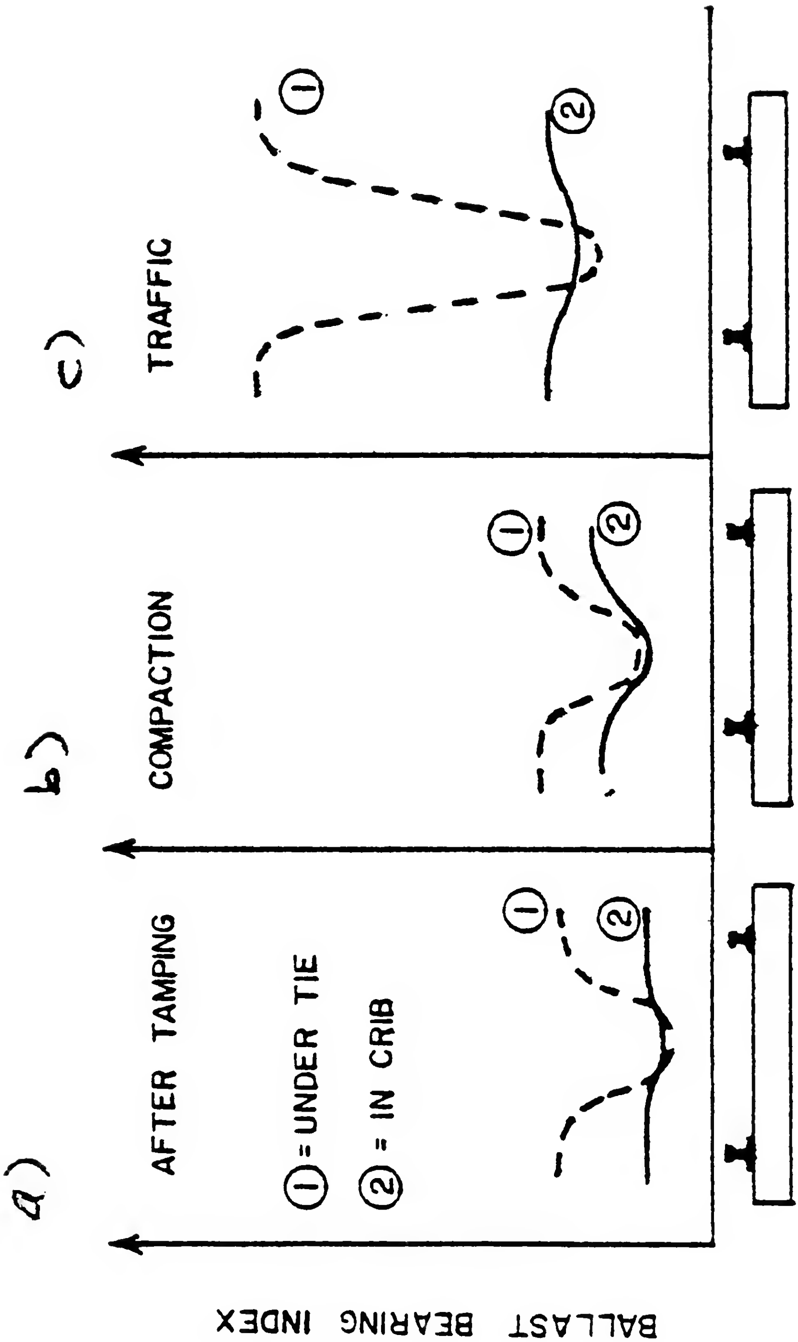

a)
AFTER TAMPING
① = UNDER TIE
② = IN CRIB
b)
COMPACTION
c)
TRAFFIC
BALLAST BEARING INDEX

Figure 2

LATERAL TIE RESISTANCE

The lateral tie push test (LTPT) has been widely used to evaluate ballast conditions and infer lateral track stability. The parameter used as an index is the force required to displace the tie a specified amount, typically 0.04 to 0.25 in. (1 to 6.4 mm).

Representative lateral force-displacement curves are compared in Fig. 4 for the conditions of tamping only, and crib and shoulder compaction following tamping. Resistance to displacement increases at a decreasing rate in both cases. However, the important observation is that compaction always significantly increases resistance above that present after tamping.

Proper interpretation of the LTPT requires an understanding of the factors contributing to the resistance. The three components of resistance provided by the ballast are the base, the crib, and the shoulder. An example of relative magnitude of each component is shown in Fig. 5 based on laboratory tests (Refs. 2 and 8). The lateral resistance is given as a ratio of an arbitrary value for comparison, because the actual values depend on the track condition and the specified lateral displacement.

The base resistance represents the force to overcome frictional resistance with a normal force equal to the tie weight. The friction coefficient will, of course, depend greatly on the roughness of the tie bottom. This component of resistance will increase with the tie weight (Fig. 5a).

The crib resistance is caused by frictional resistance on the sides of the tie under a normal force caused by lateral ballast pressure. The crib component increases directly with the depth of ballast in the crib (Fig. 5b). The illustration in Fig. 5b represents tamped-only ballast. The ballast pressure on the sides of the tie is greatly increased by compaction. Thus after compaction, the crib contribution will be even greater. However, vibration of the ballast or movement of the tie can relieve this pressure over a period of time, and thus diminish the effect of compaction observed immediately after maintenance.

The shoulder resistance also increases directly with the height of the shoulder above the tie bottom and with the shoulder width (Fig. 5c). Compaction of the shoulder will increase this resistance further.

In the example given in Fig. 5, the base contribution to resistance, assuming a wooden tie, accounts for only about $\frac{1}{7}$ of the total resistance. The crib accounts for $\frac{4}{7}$, i.e., more than half of the total, and the shoulder accounts for $\frac{2}{7}$. The lateral resistance under traffic will be much greater than for the unloaded tie because the base component will increase substantially without change in the crib and shoulder components. Thus the percent increase in lateral resistance from compaction should be much less under traffic load than shown in Fig. 5.

The lateral resistance after tamping also depends on the amount of raise during tamping. According to Fig. 6 (Ref. 9), as the raise increases, the loss in lateral resistance increases relative to the undisturbed track after traffic. This result takes place for compacted as well as tamped-only conditions. The Fig. 6 also shows that crib compaction increases lateral resistance and shoulder compaction further increases the resistance.

With the application of traffic, neglecting crib and shoulder compaction, all three components of resistance will increase (Fig. 7). Thus the total lateral tie resistance will increase and the percent of total contribution from the crib will probably decrease. However, it is important to remember that these conclusions are for an unloaded track. For a loaded track, the percent contribution from the crib will be entirely different, and greatly reduced.

All available relevant data were evaluated in order to estimate the traffic equivalent of crib and shoulder compaction on lateral tie resistance (Refs. 10 to 15). Variability of the data

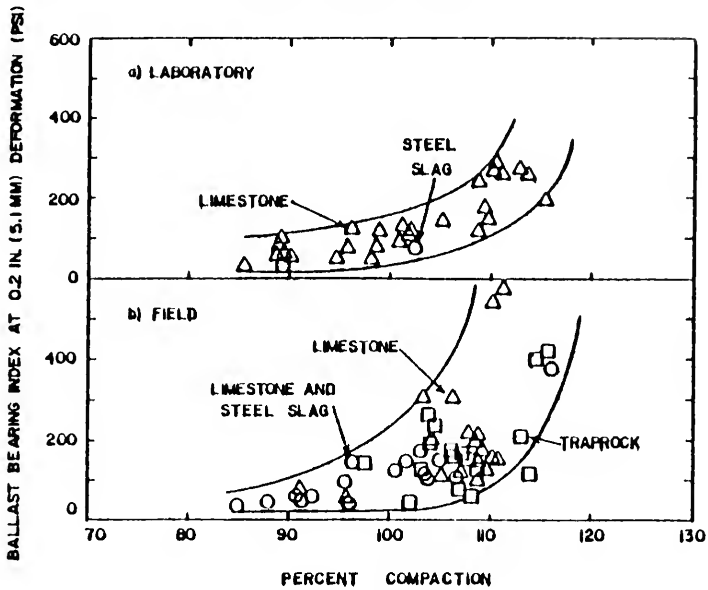


Figure 3

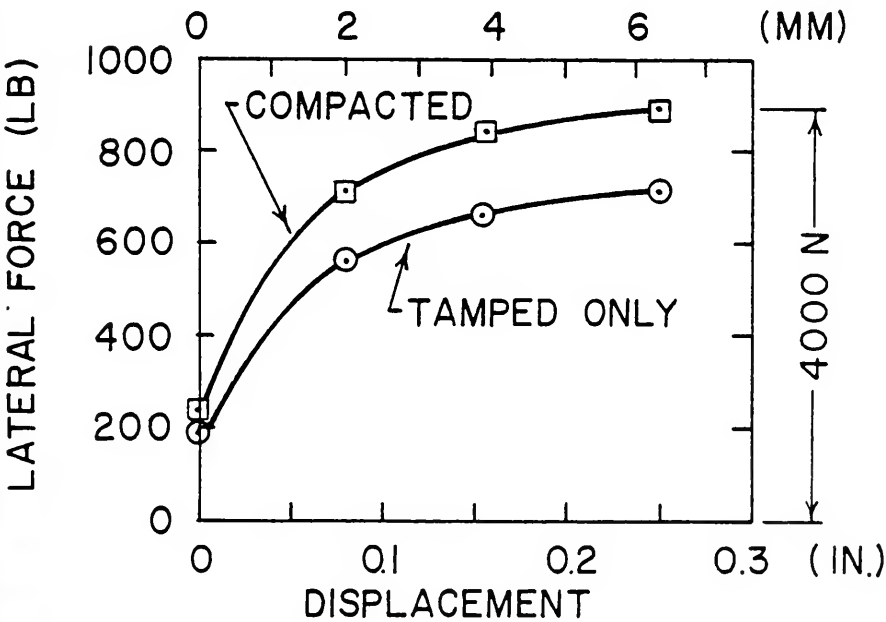


FIGURE 4 EFFECT OF CRIB AND SHOULDER COMPACTION ON LATERAL TIE RESISTANCE FOLLOWING MAINTENANCE TAMPING FOR A WOOD TIE IN LIMESTONE BALLAST

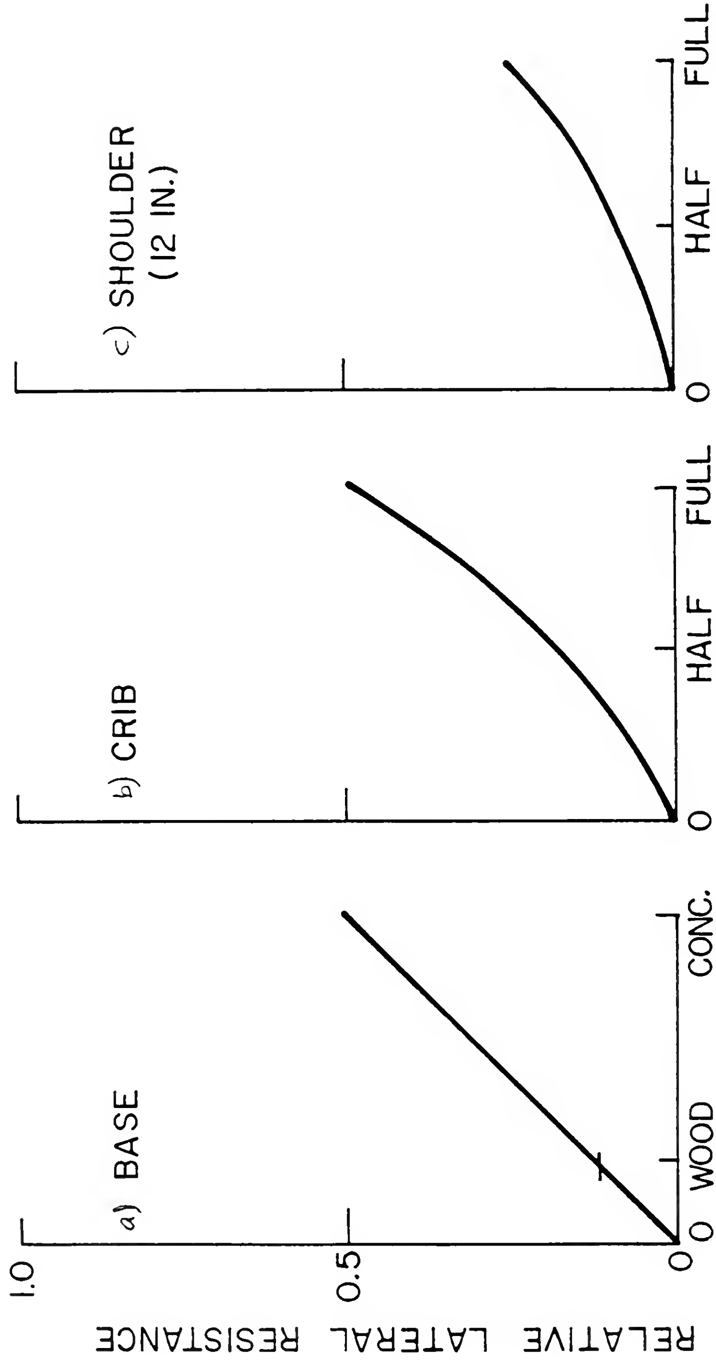

a) BASE
b) CRIB
c) SHOULDER
(12 IN.)
RELATIVE LATERAL RESISTANCE
1.0
0.5
0
0
WOOD
CONC.
0
HALF
FULL
0
HALF
FULL

Figure 5

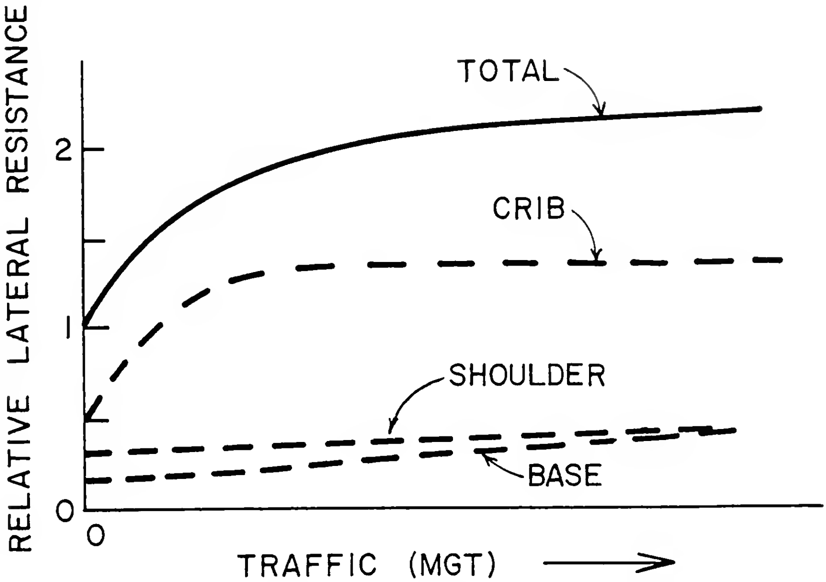


Figure 6

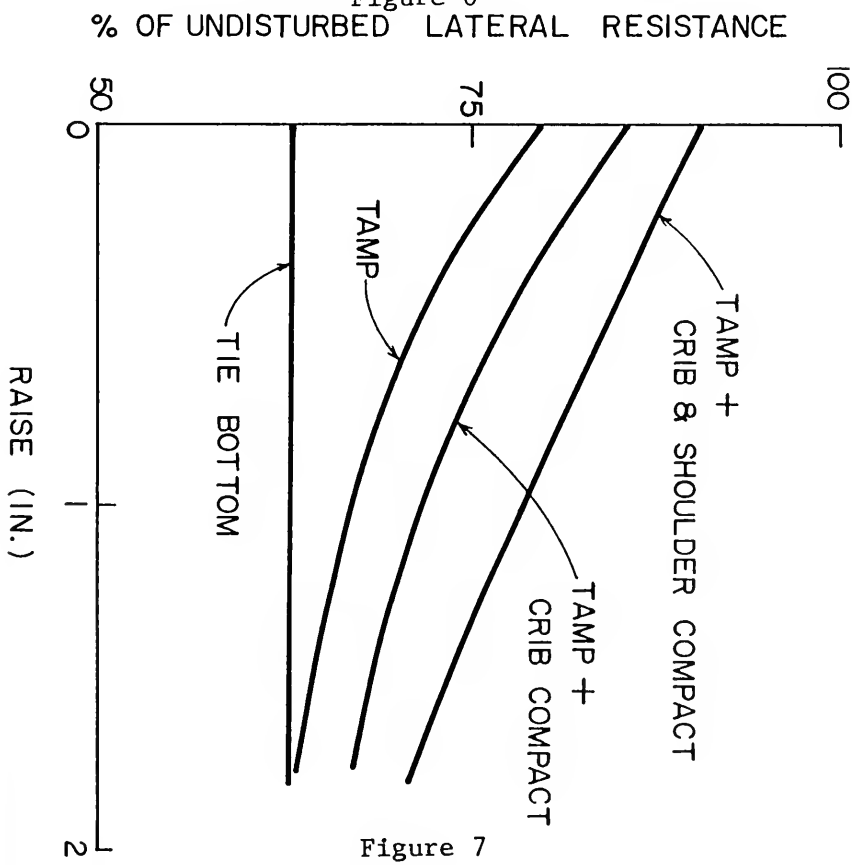


Figure 7

and the wide range of track conditions represented prevent precise conclusions from being drawn. However, a reasonable interpretation is believed to be represented by Fig. 8. Crib and shoulder compaction increases lateral resistance by an amount that is equivalent to about 0.2 MGT traffic. After 2 MGT, the lateral resistance is about the same whether or not compaction was applied at the time of maintenance. After about 20 MGT traffic, the lateral resistance stabilizes.

BALLAST STRAIN AND IMPLICATIONS FOR MAINTENANCE

Further insight into the track behavior is given by the ballast and sub-ballast permanent strain development with accumulated traffic (Ref. 14). The general trends are illustrated in Fig. 9. Vertical compressive strain accumulates in the ballast at a diminishing rate until tamping is done. At that point, extension strain is caused by the raise, after which a new settlement cycle begins. The subballast and subgrade are not affected by tamping, so they do not show the extension from maintenance. The development of center binding is evident in Fig. 9 because the strain beneath the rail is greater than in the center.

From the observed field behavior of ballast presented in this paper, track settlement can be expected to increase with traffic and the amount of raise, as shown in Fig. 10a.

The cost of tamping is also likely to increase with the amount of raise as illustrated in Fig. 10b. However, by allowing settlement to increase and hence require higher raises in maintenance, the period between maintenance cycles can be lengthened. The net effect of these factors is the relationship in Fig. 10c, which shows an optimum raise which gives the minimum average annual cost. If numerical values are obtained for Figs. 10a and 10b and annual traffic figures are known, a value for the optimum raise can be estimated. This has not yet been done. The important point, however, is that such an optimum exists, i.e., deferring maintenance beyond a certain point will actually cause an increase in the cost per year, even though the number of years between maintenance operations is increased.

Valuable insight into the relationship between the key parameters and track performance could be gained by proper assessment of field experience. This case study approach requires the following inputs:

1. Characteristics of the ballast and subgrade materials and a description of the track structure.
2. Maintenance history, including type of maintenance performed, frequency of maintenance, and condition of track before maintenance.
3. Description of general environmental conditions, which can be adequately determined from available weather bureau data.
4. Traffic history in terms of annual tonnage and approximate mix of individual car weights.
5. A method to integrate the above factors to determine the relative tendency for track settlement.

Of the first four inputs, an adequate knowledge of the ballast and subgrade conditions is the only part that is difficult to accomplish. Progress has been made in development of a prediction model to provide the fifth input (Ref. 16). This model should be useful for maintenance planning and track design.

SUMMARY AND CONCLUSIONS

A study was carried out to obtain information on the physical state of ballast in track and its influence on track performance. Methods of measuring the physical state were developed and then used to investigate a wide variety of track conditions.

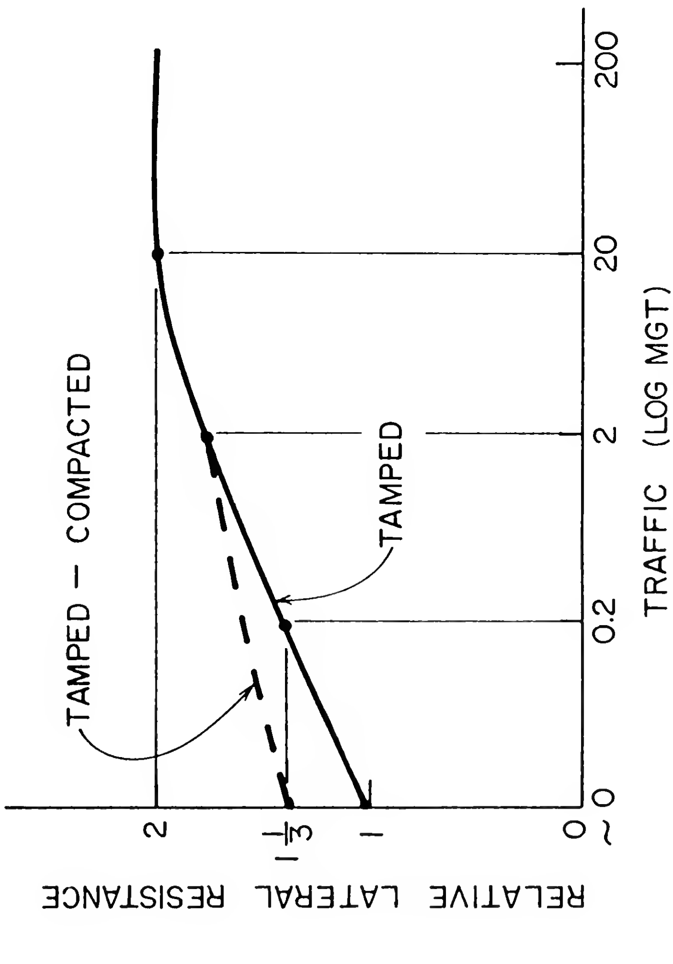


Figure 8

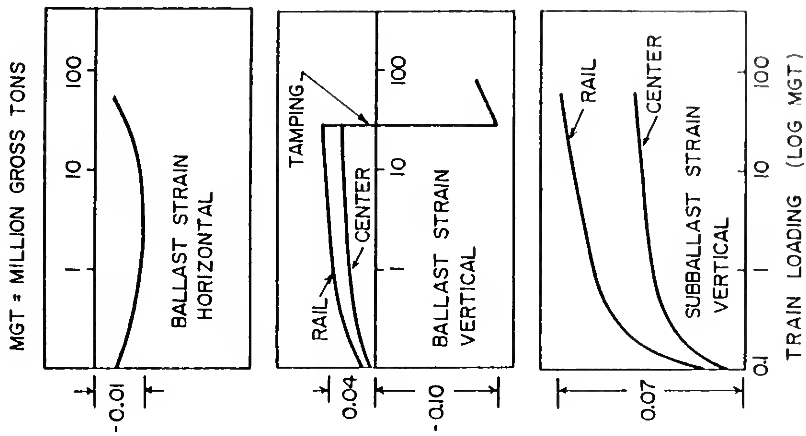
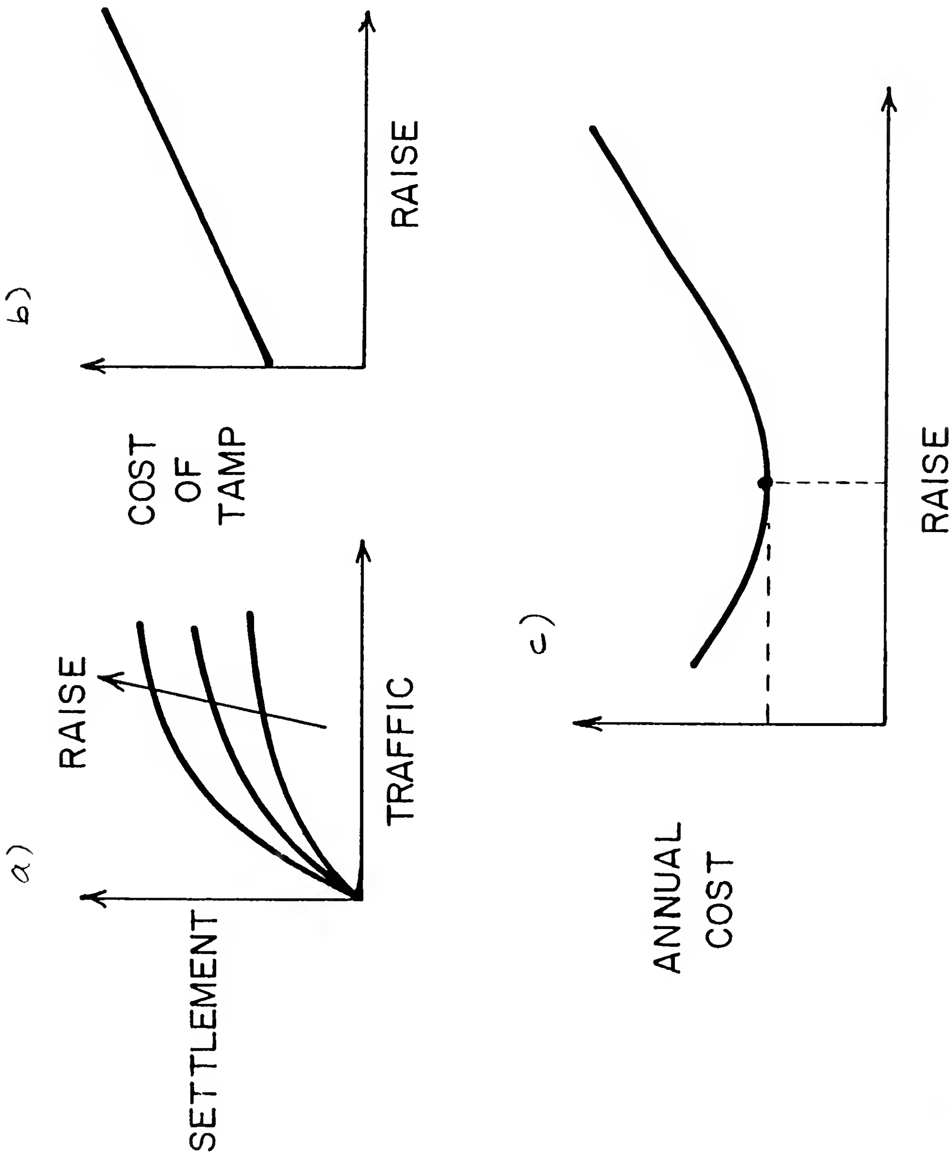


Figure 9


a)
SETTLEMENT
RAISE
TRAFFIC
b)
COST
OF
TAMP
RAISE
c)
ANNUAL
COST
RAISE

As might be expected, no evidence was found that the tamping operation has any significant effect on the subballast and subgrade performance. However, the influence of traffic was felt even down into the subgrade. Cyclic stresses from the succession of train axle loads caused cumulative vertical strains which produced a contribution to settlement from the subgrade as well as from the ballast and subballast layers.

No subballast and subgrade measurements were available in tests in which crib and shoulder compaction were involved. However, based on experience with compaction, as well as the observations associated with tamping, no significant effects on the subballast and subgrade behavior are expected from surface crib and shoulder compaction with available machines.

Newly deposited ballast conditions occur with such maintenance operations as undercutting, sledding, and new construction. The ballast density is usually lowest for these conditions. As a result, the density is usually increased under the tie by tamping. Under the center of the tie, as well as in the crib, the ballast will be as loose.

The primary effect of the crib and shoulder compactor is in the crib at the locations of application of the compacting plates (near the rail). When a crib and shoulder compactor is used following tamping of newly deposited ballast, the ballast density in the crib near the rail increases significantly. A small increase may also occur under the tie near the rail.

Traffic significantly increases the amount of compaction in the crib and under the tie after tamping and crib and shoulder compaction. Traffic-induced compaction under the tie is greater than with tamping and crib and shoulder compaction. Compaction caused by traffic can even be greater in the crib. A valuable objective of further research would be to find out how to produce traffic-induced level of compaction as part of the maintenance operation so that the track will be stable before the application of traffic, rather than be stabilized by traffic.

Traffic degrades track by creating permanent settlement, which is generally manifested by irregularities in surface and line. However, at the same time, the ballast is compacted by the action of traffic and its properties will be improved unless the ballast degrades by mechanical breakdown or pumping from infiltration of fines and moisture. Thus, maintenance tamping is required periodically as part of the process of reestablishing surface and line. Unfortunately, this tamping also loosens the ballast from its state after traffic. The amount of disturbance is directly related to the amount of the raise. The greater the raise, the lower the ballast density becomes.

The research results show that crib and shoulder compaction increases the ballast stiffness, both under the tie and in the crib, compared to the effects of tamping only. The percentage of stiffness increase, of course, is greatest in the crib, where tamping is applied. The stiffness increase under the tie may be a result of greater ballast confinement from crib compaction rather than the change in physical state of the ballast under the tie.

Crib and shoulder compaction was also observed to consistently increase the lateral tie resistance compared with the resistance following tamping only. However, the percentage increase measured with the LTPT is believed to be much greater than the corresponding increase in track lateral resistance, which would develop under train load with the ties fastened to the rail. Furthermore, this increased resistance may only be temporary.

One of the accepted benefits of crib and shoulder compaction of ballast is to reduce the slow order time for traffic imposed because of the reduced lateral stability after tamping. The results suggest that the compaction process provides the equivalent of about 0.2 million gross ton (MGT) of train traffic in stabilizing the track. Thus immediate operation of trains at the speed otherwise permitted after 0.2 MGT of slow orders could be initiated immediately. Low traffic density lines might benefit more than higher density lines, since the period of time for slow orders would be much longer on the low density lines.

Slow orders should be more critical in maintenance operations involving undercutting and ballast replacement because the ballast bed will generally be much looser. Normally, the compaction process follows the final tamping-surfacing-lining operation with fully ballasted cribs. However, consideration should be given to compaction before the cribs are filled in order to provide a greater depth of penetration of the compaction effect into the ballast below the bottom of tie. The benefits of this approach might be particularly useful for the reballasted track. Test results have shown that the lateral tie resistance is greater when compaction is done on a fully ballasted crib than on a partially filled crib. To conclude from this that compaction is most effective using full cribs may be misleading because in the LTPT measurements, a substantial part of the lateral resistance comes from the crib when the tie is unloaded. However, the bottom of the tie provides a much greater proportion of the resistance for loaded ties under train traffic. Possibly the benefits of compaction would be increased by a sequence which first provides compaction with low cribs, followed by crib filling and a second application of compaction. Further studies are needed to evaluate this possibility.

Another important application of crib and shoulder compaction is in conjunction with spot maintenance where loosening of the ballast has occurred around only some of the ties. The use of crib and shoulder compaction will reduce the physical state difference in the ballast between the disturbed cribs and the undisturbed cribs.

By far the most significant benefit of using crib and shoulder compactors that has been expressed by railroad users is associated with maintenance that must be done during hot weather. For a variety of reasons, track maintenance involving tamping may be impossible to defer until a sufficiently cool period. In such cases, immediate stabilization is a very important safeguard that even slow order traffic cannot provide. Economically, the benefits of crib and shoulder compaction would be greatest if a substantial lengthening of the maintenance life of the track were to result. This would occur either if the rate of settlement were reduced by compaction or the uniformity of settlement were improved by compaction. Unfortunately, the available field information is not adequate to conform this benefit, although intuition suggests that it must be true to a certain extent.

In general, the results of this study suggest that:

1. The addition of crib and shoulder compaction to the maintenance operation is beneficial,
2. Crib and shoulder compaction with existing machines and procedures does not restore the ballast to the same degree of stability that is provided by traffic, and
3. The full potential of compaction of ballast, including the optimal use of the vibratory compaction parameters, has not yet been reached.

Continued research on ballast physical state and its relationship to track maintenance and track performance is needed. This effort will lead to an improved methodology for predicting track degradation. An important and much needed benefit will also be to determine how to classify ballast in relation to its influence on track performance.

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Principal Investigator for the study was Ernest T. Selig, Professor of Civil Engineering. Assistance in technical direction of the work was provided by Tai-Sung Yoo, Research

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ANNUAL LUNCHEON

Luncheon Address

by J.R. Nelkirk*

Thank you very much Buck, officers and members of AREA, guests, ladies and gentlemen. It is indeed a pleasure for me to attend another AREA annual meeting. My last one was in 1973.

As Buck Durham mentioned to you, my background in railroading is in operations—my first job was in the engineering department—followed by several in M/W. I have always been grateful for those early experiences. I look back on those M/W jobs as *"character builders."* I know of very few areas of railroading that require the long hours, physical effort and dedication as do the jobs in engineering and maintenance, jobs that quite often must be performed under adverse weather conditions. I am pleased to be here as I have always felt at home around engineering and maintenance people.

I feel certain that the major portion of your time in Chicago has been and will be spent on standards, the latest engineering applications, committee reports, approving or disapproving proposals before your various sections. Let me re-phrase that to, "I hope most of your time has been directed to those efforts." I am not a civil engineer but I do not have to be one to know that this is where it all starts in railroading. This is where the action is and the success or failure of our industry itself lies more in this area than in any other—area is spelled A.R.E.A. For that reason, I want to take just a few minutes of your time today to steer you in another direction—a direction in which I feel we must dedicate a greater portion of our attention—and that is to choosing the people who must turn the decisions you make here this week into reality. The particular group of people I want to focus on are the managers and supervisors.

They must be your leaders, the ones who are out front and "make things happen." I will assume that as you leave here this week that most of you are "cocked and primed" to return home and launch into your maintenance full blast. Your *material*, *machinery*, and *labor forces* have all been approved and signed off on by your management. You have decided this week if there will be additions or deletions to any of your major guidelines, and you are ready to go. So in the next week or so, you will place all of this in the hands of these *people* I'm focusing on. They will ultimately decide whether or not your plans are fulfilled. Your game plan is set—you must now hand the ball to someone else and hope he runs like hell with it.

If I were to ask you individually how you found these people, I would get a great variety of answers. Since time certainly will not permit that, I'd like to pass along some ideas on the qualities I think you should look for in the search for your man and how to recognize him when he shows up.

An often overworked term is dedication. Certainly our man must be dedicated to his company and to the tasks before him. I would like to take a few minutes and play you a tape which epitomizes the way many people view the *dedication*, *loyalty*, and *desire to excel* of most managers and employees in the rail industry's engineering and maintenance of way sections.

This in no way implies that Gunga Din worked in M/W. However, I do think you'll agree, that's dedication. At least that was the definition as viewed by my first section foreman. And perhaps Gunga Din has jogged your memory to some particular incident in your career when you had to hang tough and blow that bugle. But *times and attitudes are rapidly changing*. Employees today are not the same as they were 10 years ago, as a matter of fact, not even 5 years ago. In many cases, their *educational level is much higher*, their *life styles and goals* are

*Vice President—Administration, Norfolk & Western Railway.

different. Also, I do believe in many cases our employees' *work ethic remains quite high*, but they also *want to know why* and they want to be *part of the team* with some hope of career advancement. This can very easily be a *positive or negative* situation. The *managers or supervisors* we select as leaders will determine the success or failure of these attitudes.

Successful managers provide the energy for successful businesses. Although *machines, technology,* and the *physical plants* are of primary importance, they still do not make the difference between success and failure.

The difference is, and always will be, people. In my opinion *only certain people* make things happen, generate action, and move industry forward. They are unusual individuals who possess an *iconoclastic* dissatisfaction with static situations, the deep urge to improve all things, and the unrelenting drive to achieve more than their fellow employees.

These are the movers and doers in virtually any field of endeavor, and perhaps the most important role of any *chief engineer, regional manager* or *division engineer*, etc., is to single them out from the crowd and having done so, convince them that their organization will provide a framework for their personal accomplishment and growth. To single them out from the crowd is a very difficult task because—

At first observation, potentially successful managers appear to have no *physical, behavioral, educational* or *age* specifications. One can only imagine the dilemma that's created when a civil engineer is given an assignment with no specifications.

The individuals who lead corporations forward come from different backgrounds. They are of varying degrees of intelligence and physically range from strong and handsome to frankly homely (Buck, this lets you out!) Their personalities can be either exuberant or introverted, and their methods of achieving results are infinite in variety.

Good managers do not necessarily have to be creative; creative ideas have little intrinsic value in industry. It is only when ideas are implemented by determined individuals that they become meaningful at all. John Derek did not invent a "10"—namely Bo Derek, now the latest rage of world film and media. The difference is that he created a walking, breathing empire while the rest of us swivel our eyeballs.

Ideas often must be drawn from others who have a talent for creative thought, yet may not be doers and achievers themselves. But we do not want our man to be completely without ideas of his own.

Our managers must have strong desires to please themselves. He is inspired by an inner necessity to know the truth and to attain goals.

Tying in closely with self-image, good managers have more entertaining friends than the average. Good managers are more curious—more aware. While they are not necessarily of superior intelligence, they seem especially wide-awake to current events and happenings. They *work hard, live hard,* and *waste little time*. They have energy to burn and burn it freely. Having fun is important so they waste little effort on merely appearing to have fun.

That *drive to accomplish*, the *urge to compete, ambition, determination, pride*—these are only facets of the overriding characteristic that sets super managers apart. That characteristic can be summed up in one word—and that key word is *hunger.* A lot of really intelligent people know almost everything yet understand almost nothing. Intelligence, at least as it relates to our rail industry, is difficult to define. IQ tests are no sure measure of the kind of intelligence a successful manager needs. Intelligence can conceal almost any weakness—except the lack of hunger, the paramount essential.

Humor, tact, persuasiveness—all of these are extremely valuable in managers. They need these characteristics to overcome resistance to change.

Sometimes good managers become so engrossed in their missions that they tend to cause personality conflicts. Tactless and abrasive managers, however, are less likely to achieve positive results than those who are willing to sell their ideas and create action with tact, humor and persuasiveness.

It takes courage to attempt new things in any business. Good managers take risks. They must have the courage to shed their old skin and risk the tender, new one underneath. In this regard, a competent manager is like a growing caterpillar. A poor manager never exposes himself to risk, never sheds an old skin—and never grows. And more importantly, *he never blows a bugle.*

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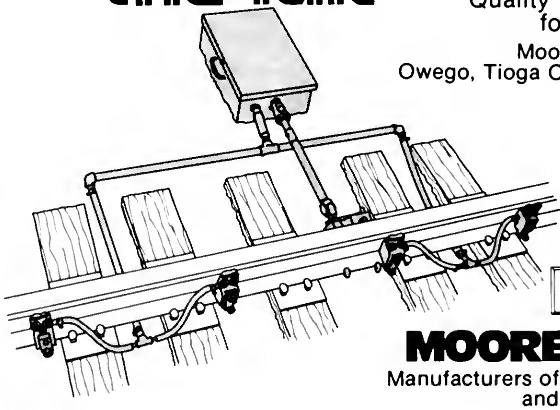

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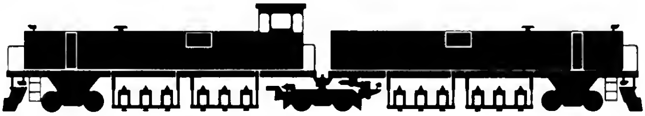

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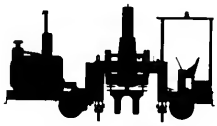

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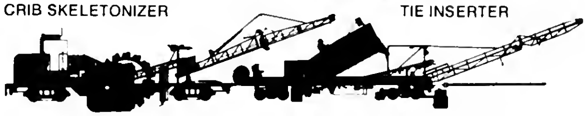

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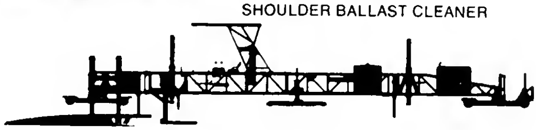

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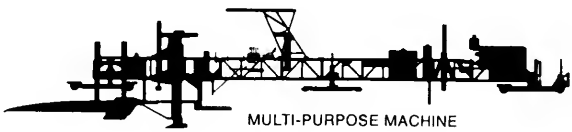

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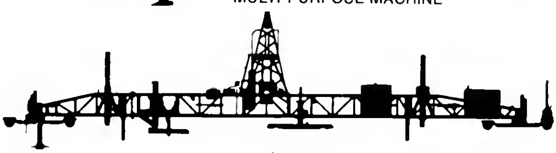

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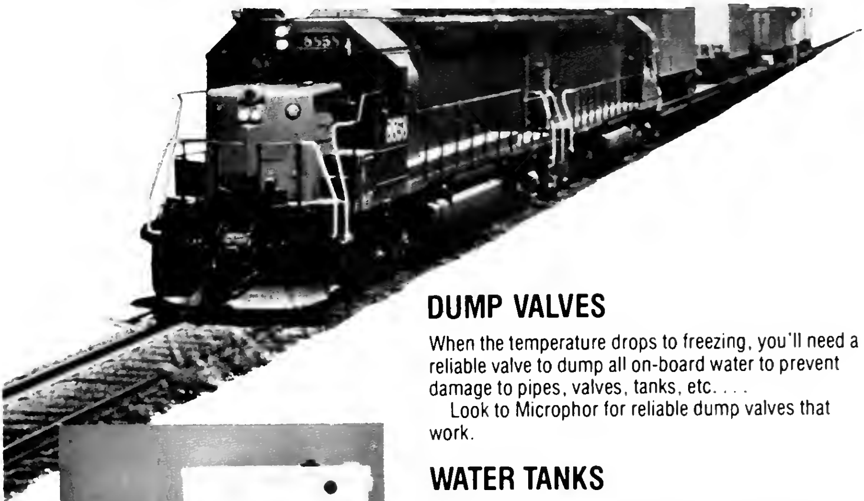

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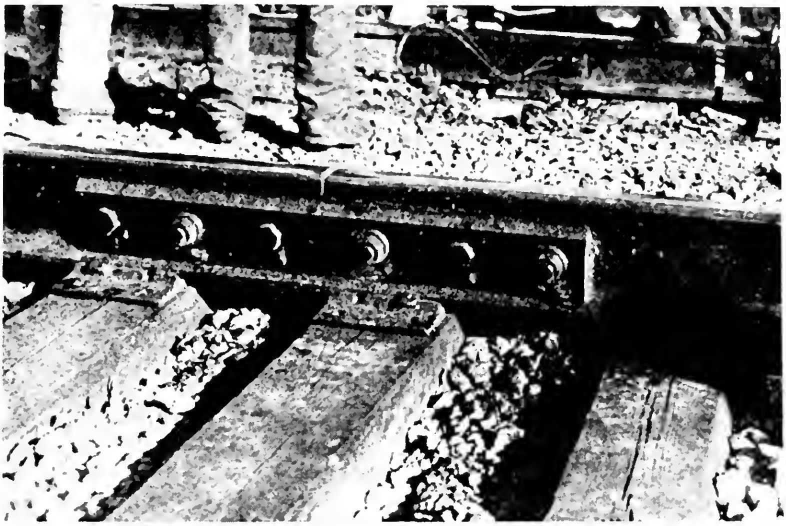

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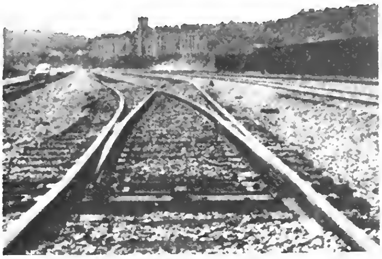

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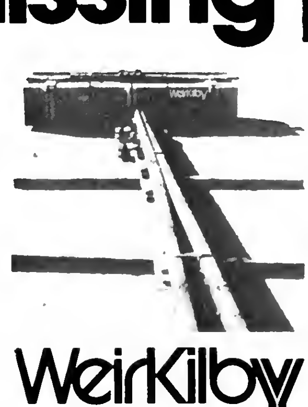

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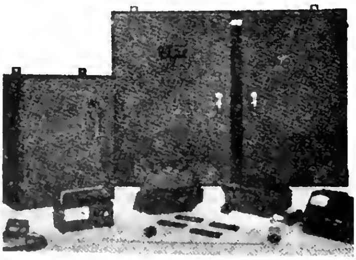

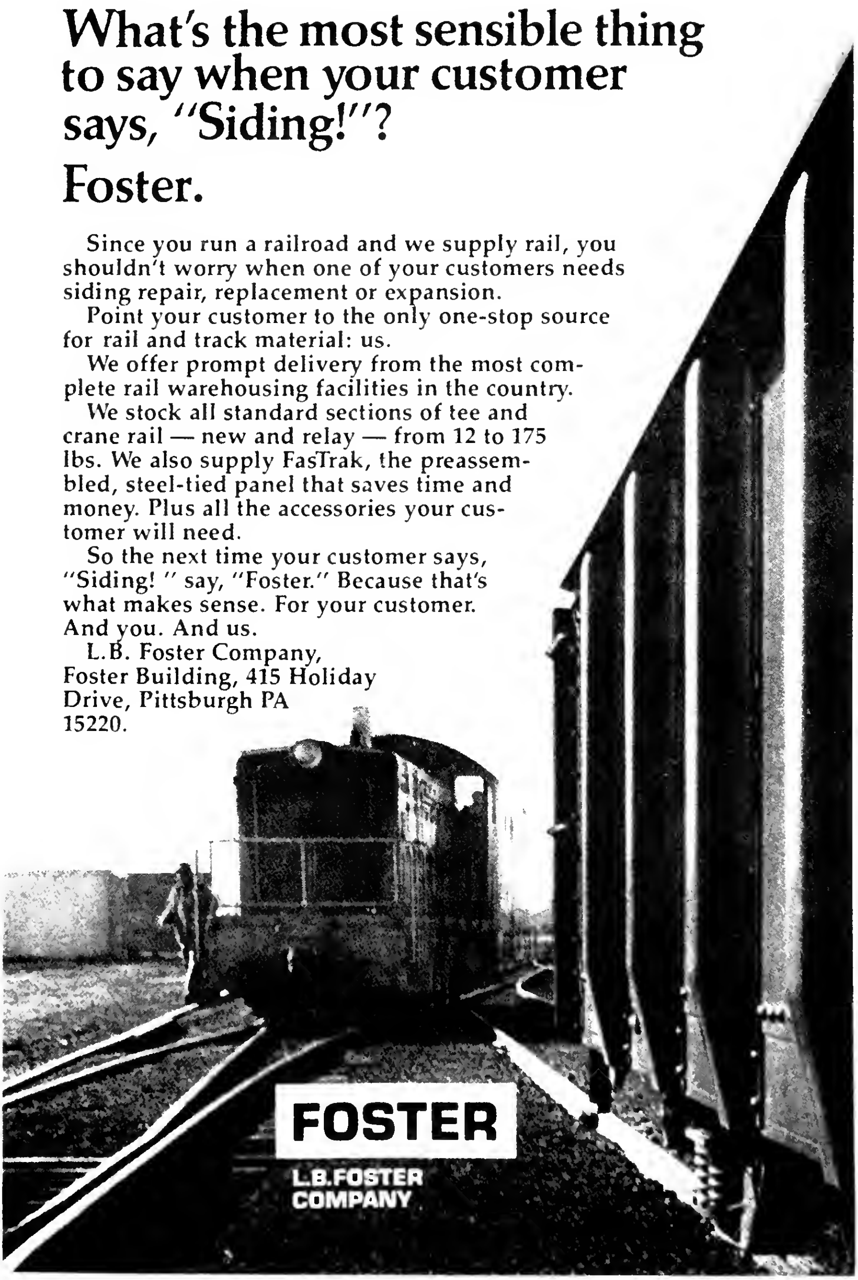
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INSTALLATION OF OFFICERS, FINAL FEATURE, AND ADJOURNMENT

Installation of Officers

PRESIDENT DURHAM: There is only one feature remaining on the program of the 1980 meetings. However, before proceeding with this feature I would like to install the newly elected officers at this time. This is a short but important and impressive ceremony and will let each of you become more acquainted with the officers you have elected.

Before doing so, however, I want to take this opportunity to thank all who contributed to the work of our Association during the past year and to the success of this Conference. The AREA has had an extremely successful and productive year in spite of many demanding challenges.

There are so many to whom I personally am indebted that I cannot possibly name them all here, but I do want to express my personal appreciation for the splendid cooperation of our officers and directors, our committee chairmen and the active committee members, and all others who contributed in any way to the success of the 1979-1980 Association year. I also want to extend our thanks to those persons both inside and outside the AREA and AAR who prepared and delivered the outstanding and timely features at this Conference.

I especially want to express my sincere appreciation, and the appreciation of the Association, to our headquarters staff for the manner in which they have conducted the affairs of the AREA during the past year, while moving the headquarters from Chicago to Washington. Their attention to the multitude of details in the planning and execution of the Association's activities and programs, and their efforts in returning the important, world-wide used AREA publications to a normal schedule under the very difficult circumstances caused by the move, have been invaluable to the Association, the Board of Direction, and to me. In addition, they have taken on the new duty of handling many technical matters in Washington, which proved very valuable with the work involving the FRA's proposed revisions to the track safety standards. They deserve our congratulations and support.

The Conference Operating Committee, under the direction of its chairman, Vic Hall, Santa Fe, did a fine job in connection with operating this Conference in cooperation with the Association's staff.

These well-planned and well-operated Conferences do not just happen. Other than our past presidents, few members are in a position to know the multitude of details handled by the staff and the operating committee relative to our Conference, and how easily things could go awry if it were not for their diligence and dedication.

I join Ms. Cynde Kraft, Membership and Conference Coordinator, in thanking all of those ladies who, with Mrs. Cerny, Mrs. Rougas, Mrs. Montgomery, and Miss Susan Rich, Secretary to the Executive Director, gave so generously of their time in assisting with the functions of our Conference planned expressly for our spouses. You have our grateful appreciation.

(Applause)

PRESIDENT DURHAM, (CONTINUING): To make room for new officers and directors a certain number of them complete their service on the Board of Direction each year.

I want to thank each member of the board for his counsel, advice and support, and especially those members who, having completed their term of office, are retiring from the governing body of the Association. The close of this technical conference completes the services on the board of Past President Burt Worley, Vice President-Material Recovery & Disposition, Milwaukee Road. The AREA Constitution provides that past presidents will remain on the Board for two years after completion of their term as president. We are deeply indebted to Mr. Worley for his long and outstanding service to our Association and I am sure he still will be called upon for counsel and advice as important matters come up.

(Applause)

PRESIDENT DURHAM, (CONTINUING): Other members of the AREA Board of Direction completing their term of service are these directors:

R.F. Beck, Chief Engineer, Elgin, Joliet & Eastern Railway
D.J. Bertel, Chief Engineer-Maintenance, Missouri Pacific Railroad
J.T. Sullivan, Chief Engineer-Design & Construction, Conrail
C.E. Webb, Assistant Vice President-Engineering, Southern Railway

These men have served our Association well in their official capacity on the Board, and I want to express our deep appreciation to each of them.

Will Messers. Beck, Bertel, Sullivan, and Webb please stand and permit us to show our appreciation for their service.

(Applause)

PRESIDENT DURHAM, (CONTINUING): It is now my privilege and pleasure to install the new Directors you have elected for the ensuing year. As I read your name, please come to the Speaker's Table and take a place at my right.

(President Durham shakes the hand of each new Director as he ascends to the Speaker's Table)

From the East District

B.J. Gordon, Chief Engineering Officer, Conrail
J.C. Hobbs, Chief Engineer, Richmond, Fredericksburg & Potomac Railroad

From the West District

J.R. Masters, Director-Engineering, Burlington Northern, Inc.

From the South District

H.L. Rose, Assistant Vice President, Maintenance of Way & Structures, Southern Railway

Gentlemen, I welcome you as Directors of the American Railway Engineering Association. These are offices of high honor and responsibility you are assuming. I hope you will enjoy your service on the Board of Direction and will bring much value to its deliberations. Congratulations. You may be seated.

Our newly elected Junior Vice President is Bill Glavin, Vice President-Administration, Grand Trunk Western Railroad

Mr. Glavin, will you please come forward?

PRESIDENT DURHAM, (CONTINUING): Mr. Glavin, I congratulate you upon your election as Junior Vice President and your continuing service on our Board of Direction.

The new Senior Vice President is Jim Brent, Director-Operations Planning, Chessie System.

Mr. Brent, will you please come to the platform?

Mr. Brent, I congratulate you on your advancement to Senior Vice President and continued service on the governing body of our Association. I know that you will discharge this greater responsibility with distinction.

You and Mr. Glavin will make a splendid team of vice presidents. Please be seated.

Before installing our new president, who will then complete the remainder of this 79th Technical Conference I ask if there is any unfinished or further business to come before this meeting?

(At this point, Past President D.V. Sartore asked for the privilege of the floor and presented President Durham with the AREA plaque)

After the presentation, President Durham made an appropriate response.

PRESIDENT DURHAM: Is there any further business at this time?

If not, I will continue. Our new President is Mike Rougas, Chief Engineer, Bessemer & Lake Erie Railroad. To accord Mr. Rougas proper recognition, I have requested Past Presidents T.B. Hutcheson and W.J. Hedley to escort him to the platform at this time.

(President-Elect Rougas was escorted to the platform, and as he reached it President Durham shook his hand.)

PRESIDENT DURHAM, (CONTINUING): Mr. Rougas, I congratulate you upon your election to the highest position of honor in the American Railway Engineering Association. I share the confidence which has been placed in you by our membership and it is with great pleasure and satisfaction that I turn over the responsibility of AREA President to you. I now proclaim you President of the American Railway Engineering Association.

I share the confidence which has been placed in you by our membership and it is with pleasure and satisfaction that I turn over the responsibility of President to you.

In doing this, I want to present you with this lapel pin which is the official emblem of our Association. I am sure you will wear it with equal pleasure to yourself and honor to the Association.

(Past President Durham then pinned the emblem on President Rougas and retired, while President Rougas assumed the podium.)

(President Rougas responded, and then continued as follows.)

PRESIDENT ROUGAS, (CONTINUING): Since we will adjourn this Conference after our final feature has been completed, I would like to ask at this time if there is any further business to come before this meeting?

(At this point in the program, Bessemer & Lake Erie Railroad interrupted the proceedings and requested the privilege of the floor to present President Rougas with a gavel which had been prepared for him by his friends and associates on the Bessemer & Lake Erie.)

CASE STUDIES OF TIMBER BRIDGES' PROBLEMS CAUSED BY UNIT TRAINS

by Art Fish

The title of this presentation could be "Unit Trains vs Timber Bridges" because it is definitely an adversary situation.

According to some of the people I have talked to and some of the evidence I have gathered there seems to be a magic number somewhere between ten and twenty million gross ton miles. Below that number the wooden bridges seem to be able to withstand unit train traffic. Above that number these structures suffer the types of damage that we are going to examine. As the tonnage goes up the amount and the severity of the damage increases even more rapidly.

What is a unit train? It didn't take long to find a number of different opinions. One thing for sure they are nothing new. Maybe they called them "drags" but for many years the coal-hauling roads have been moving long trains of heavy cars, or in other words, unit trains.

A freight traffic man once said that the concept of the unit train was to run a whole trainload of one commodity using one waybill instead of issuing a separate waybill for each car. That idea saved the paperwork but it did nothing to protect the track and the bridges from the pounding of the unit trains.

Finally I said to myself, self, since you are giving the presentation why not propose a simple definition? Let us define a unit train as a long string of cars, all of them with the same wheel spacing and uniform heavy wheel loads. If you must get technical we can add the motive power and the markers. A unit train is thus quite different from the more conventional train which can include heavy cars but generally is a mixture of cars of different lengths and gross weights.

During the preparation of this paper I consulted with my friend Joe. Now Joe is no ordinary bridge engineer but he always has a solution. "Joe," I said, "think about an ordinary light traffic branch line. They operate conventional trains and load a few cars of ties in the spring. Some of the local sawmills load a few cars of chips. Usually the whole operation is carried on at a rather leisurely pace."

"On this line they have some very ordinary wooden bridges like this one. Well, does this look a little more ordinary? Under what could be called normal traffic these bridges will last for years, even with minimum maintenance."

"So what's the problem? Well they are planning a large openpit mining operation complete with unit coal trains. How do you convince the boss you are going to need extra money to look after the bridges that are going to be taking a real beating under that kind of traffic?"

"Heck," says Joe, "its just like the failure of the new bridge I built last year." Oops, that's not Joe's bridge. Actually it is a logging road bridge but it makes a great place for drying fire hoses. This is the bridge that Joe built, a plank from one bank of the creek to the other. When Joe takes some of his product to town he walks the plank and it does everything a bridge should do.

This would be the normal traffic pattern. Conventional trains with their mixture of cars present few problems for a timber bridge that has been properly built and maintained.

Every so often the boys in town have a big party. When Joe hears about it he grabs all the product he can handle and hits the bridge on the dead run. The plank just flexes more than usual and moves a little closer to town.

Think of this as the odd time when the crew starts to abuse the speed limit in order to get home for a special occasion. A well built and maintained timber bridge can absorb the occasional big impact or overload without serious or lasting damage. So why did Joe's bridge fail?

Joe won some money in a lottery and a whole army of guys showed up at his place. They wanted to sell him something, or they wanted a donation for a pet project, or they just wanted a handout. Joe persuaded them to leave by turning loose his dogs. With that kind of incentive they all hit the bridge as fast as they could go and as close together as possible. Under the rhythmic pounding of all those feet the plank collapsed into the creek. Unit trains set up severe vibrations of definite frequencies and these do the damage to the structure.

Conventional trains usually operate with similar consists in both directions. Loaded unit trains usually operate in one direction, the empties being returned in the opposite direction. This tends to produce a longitudinal movement of the structure. With their uniform wheel spacing and loadings the unit trains when running at a constant speed set up far more regular and stronger vibrations than conventional trains.

On one short branch line that runs through a low lying area, the ground is mostly river silt with large pockets of peat. There is also a high water table so the subgrade is usually rather mushy. There had always been maintenance problems but when they started operating unit trains of grain and potash more problems developed. The formal reports stated that track tie life was being substantially reduced. What the Roadmaster said was that they were just tearing the hell out of his ties.

As part of the effort to find the cause of this damage a movie camera was set up to record the movement of the track. As had been expected the unit trains did set up a very definite oscillation. One disturbing finding was that at a certain speed the oscillations were of much greater magnitude. To keep damage to a minimum the trains were restricted to speeds below that critical speed for this stretch of track.

When a unit train passes over a wooden bridge does it stop creating vibrations? Not very likely. The frequency could change because of the difference in the rigidity of the bridge deck and the subgrade but they will be there and doing their best to exploit any weakness in the structure.

The inspection of a bridge must start on the approaches. This has always been an area for controversy. The bridgemen complain that because the trackmen don't tamp the ties properly the bridge gets damaged. The trackmen say there would be no problem if the bridgemen would only build a proper ballastwall.

The vibrations set up by the unit trains can cause some settlement or compaction behind the bulkhead depending on the nature of the subgrade material. If the material is just compacted then a sag develops in the approach. Without compaction the material will be forced down and tend to spread out. If the ballast wall cannot withstand the extra pressure caused by the longitudinal movement of the subgrade material it will start to move toward the bridge.

As soon as the ballast wall contacts the ends of the stringers the same longitudinal pressure is applied to them. Unless the stringers are securely fastened to the cap they can easily move off the dump bent cap. This reduces the bearing area, increases the stress in the remaining contact area and results in earlier crushing of the cap and the stringers.

When the stringers are secured to the caps they can force the caps off the piles. This also reduces bearing areas, increases stresses and speeds up crushing of the cap. Once in a while a drift pin that is moving with a cap will split a pile or widen an existing split.

When both the caps and stringers are firmly attached to the piles the pressure can result in the tipping of an entire bent. Periodically we will find a bent further along the bridge that will put up resistance to this longitudinal movement. Under these conditions the stringer chord will move laterally and the deck will look like it is suffering from a sunkink.

A bulkhead that has decided to move can also come in direct contact with the cap and/or piles of the dump bent. The end result is the same set of problems.

Wood subject to compression across the grain can only withstand a fraction of the stress it can resist when the load is applied with the grain. Bridge and track ties are stressed across the grain and usually very close to their ultimate strength. Any crushing or plate cutting of a tie transfers more load to its neighbours making them more susceptible to damage. At any location in a timber member where there is loading across the grain the inspector must be alert for the first sign of crushing. Just as with ties, any unequal bearing within a group of similar members soon results in damage to the adjacent members. If this sequence is not stopped it will progress until the entire structure is wracked.

In one long stretch of a main line there was a mixture of open deck timber bridges and steel bridges with large wooden ties. The bridge people knew pretty well how long these ties and trestles would last under normal traffic and climatic conditions. They used this knowledge when planning their long range programs.

Then came the unit trains and some of the older ties literally started falling apart. There were some rather panicky moments until the necessary replacements were made. The only explanation seemed to be the rhythmic pounding of the unit trains. This dramatic reduction in expected tie life led to a stepped up inspection of the wooden bridges. These inspections confirmed that the trestles were being damaged to the point that their service life would be shortened by the unit trains. A replacement program was developed and now after ten years is almost complete. During those ten years there have been many careful bridge inspections followed by very prompt action to reinforce or help any weak spots found.

One of the most critical areas of a trestle is the area of contact between the bottom of the cap and the top of the pile. Under the most favourable of circumstances the bottom of the cap is stressed very close to its maximum resistance. If a pile is not bearing fully and evenly on the cap the increased stress results in the pile cutting into the cap. It is imperative that full bearing is maintained in this critical area.

The live load from the cap is not divided equally, the interior piles carry far more of the total load than do the outer or batter piles. The percentage of the load that each pile carries is a function of the spacing of the piles and the size of the cap. There have been many cases, under unit train traffic, where the interior piles have cut deeply into the cap.

It is difficult to illustrate this type of damage without removing the cap to take its picture. However, the problem is readily apparent if the inspector will take a close look at the bottom of the cap. The effect of this damage on the cap is the same as if the interior piles were not bearing. The cap is supported mostly by the batter piles. The additional stress will quickly force these piles into the cap. While this is going on the cap is subjected to greater than normal flexing. If there is a slight defect in the cap a horizontal shear failure can quickly develop. Particularly in open deck bridges where the stringers are chorded together the additional flexing of the cap will cause additional movement of the stringers and this in turn will quickly exploit any weakness in the stringers resulting in horizontal shear failures.

One method of providing extra support for the cap to reduce the type of damage is to install cross caps under the cap supporting them with posts and sills or extra piles.

What happens if there is an internal defect in the cap? It is not unusual to find internal defects in caps that appear to be perfectly sound. The inspector must find these defects because they seriously weaken the cap. It took a lot of time and effort to get the local termites to drill this neat hole. The cap is being squeezed between the stringers and the piles and just cannot handle the load. Unfortunately, failure can occur suddenly and the results can at times be rather messy.

Some studies have been made of the vibrations induced by unit trains in masonry bridges and rock tunnels. Frequencies of 50 to 70 Hz with lateral components of even lower frequency have been recorded. These approximate the frequencies of earthquake tremors. These vibrations could well be the reason why the occasional pile starts to pump. When the pile is moving up and down under traffic it is not carrying its share of the load. This starts the familiar routine of transferring loads, increasing stresses, additional flexing and more rapid deterioration of the structures.

You would not plan to use a unit train as a pile driver but on occasion they sure act like one. On one branch line all of the timber bridges had been renewed in kind a few years before the unit trains started to operate. The piles had apparently been well driven using a diesel hammer because they had stayed in place under the then normal traffic. The rhythmic vibrations of the unit trains apparently convinced some of the piles that they needed more penetration. Some of them started going down and a few caps got broken in the process.

We have not mentioned frame bents but they have similar problems. Areas subject to crushing are the bottom of the cap and the top and bottom of the sill where the posts and piles bear on these members. These bents are often less resistant to tipping. If they are set on mudblocks any wet or unstable soil can churn under traffic and allow the bent to settle.

The bridge engineer probably has a great deal of knowledge and experience. Even after careful inspections he cannot accurately predict all the damage that will be done or just when it will happen. The only way to minimize damage by unit trains is to make frequent and careful inspections. Corrective action must be taken promptly when a potential trouble spot is located.

I do not want to appear negative about wooden bridges because they are really very remarkable structures. The fellow that designed it may never have thought about strings of hundred ton capacity cars but they are carrying that kind of traffic—a real tribute to wooden bridges.

Effect of Unit Trains on Concrete Railroad Bridges

by Tim Christenson

I would like to thank all of you for being here and can only assume that I am the last speaker to ensure I do not talk beyond the allotted time, as I am known to do. Mr. Fish, the previous speaker, has told you what a unit train is, the types of additional loads, impacts, vibrations, and forces they exert on bridges. He also explained what effects the unit trains have on timber bridges, and I am sure all of you would agree that unit trains do have a detrimental effect on timber bridges and the track system, requiring additional maintenance attention.

I am going to try to explain that in addition, unit trains have a detrimental effect on many of the existing concrete structures supporting rail traffic. In most cases, properly designed and constructed reinforced concrete slabs, spans, piers and abutments handle unit train traffic very well. The problem is that a large percentage of the concrete railroad structures were built over 50 years ago. Many of these were built with little or no reinforcing steel, and in many cases the steel was not properly placed. There was little quality control in the concrete mix, which was usually by hand with on site aggregate and water.

The foundations of these early structures were sometimes built with spread footings, or upon timber grillage. Borings of the underlying soils were rare, if non-existent.

With this background of early concrete structures, we will look at some of the effects unit trains have on them.

The unit trains impart excessive impact and vibration to the concrete structure. In the case of these early structures with insufficient or no steel reinforcement, cracking of the concrete occurs.

The vibrations and pounding effect of unit trains may cause settlement and consolidation of the underlying soils. This sometimes creates gross separation of the previously monolithic member. Once this has occurred, additional loadings are placed on the foundation soil because of the inability of the member to spread the load evenly across the area.

At age plus 50 years, even the best of concrete may show signs of weathering. This is caused by cycles of wetting and drying, or freezing and thawing. The cracks are then propagated by the hammering of unit trains, and in some cases may divorce large sections of the member.

Aggregates were in the past considered inert chemically. In the case of alkali present in cement or mix water, certain rocks do react with them, creating an expansive by-product. These expansive reactions create internal stresses which crack the concrete members and reduce the strength of it.

As we all know, every construction crew works according to its own standards and methods. In the case of the crews that built certain structures, a special technique was used in the early 1900's. This technique resulted in the center portions of the members being of what appears to be "no fines" concrete, surrounded by a 4 to 6 inch shell of good concrete. This yields a concrete which can easily be overloaded with today's unit trains.

We all agree that unit trains may tend to tip timber bridges in the direction of the loaded train traffic, but they can also tip or push concrete structures. In the case of one concrete deck slab, the slab had moved a couple of inches, creating considerable damage to the pier top.

Unit trains seem to take their toll on concrete bridges with small or loose bearing plates. For the concrete to support the loads of these trains, the steel to concrete contact must be even and secure to minimize any pounding or air hammering effect. Adjustable bearings of moveable spans require special attention under unit train traffic.

Many of the loads particular to unit trains are transmitted to the concrete substructure through the anchor bolts, and in some cases they can bend, break, or loosen, which instigates cracks in the concrete. Once the anchor bolts have become loose, a pumping action can occur, and this, together with water and sand, causes the bearing plates to wear into the concrete bridge seat. This also occurs under short span steel bridges with rigid bearing plates. The rotation at the supports is not allowed for, and to accommodate the deflections the bearing plates actually rock to supply the rotation. The plate works itself into the concrete surface until in some cases the steel beams, or stringers, may bear directly on the concrete, causing shear failures in the concrete or cracks in the steel.

What will satisfactorily support "normal" train traffic may not always withstand the effects of unit trains, and special attention must be given to the substructure, bearing areas, and spans, if a safe and useful structure is to be maintained.

Adjournment

PRESIDENT ROUGAS: The feature just presented completes the technical sessions of our Conference this year. I also want to express the appreciation of the officers, directors, staff and the other members of the Association to our committee chairmen, subcommittee chairmen and the active committee members for their work during the past year, and for their published reports and papers in the AREA Bulletin.

PRESIDENT ROUGAS: You will be interested to know that the registration for the 79th Annual Technical Conference of the American Railway Engineering Association and the 1980 Annual Meeting of the Engineering Division, Association of American Railroads is as follows:

Men	1007
Ladies	137
Total Registration	1144

PRESIDENT ROUGAS: The officers, directors and staff of the Association sincerely hope that you have gained much from the information and knowledge dispensed during the past two and one half days. We solicit your comments and suggestions—preferably addressed to Executive Director Cerny at Association Headquarters.

PRESIDENT ROUGAS: Before I adjourn this Conference, I would like to remind all members of the Board of Direction and Engineering Division General Committee, including the members just released and the members just installed, and all the members of the AREA Conference Operating Committee, that we will have a joint luncheon together in the Wabash Parlor on the third floor of this hotel immediately following the adjournment of this meeting.

PRESIDENT ROUGAS (CONTINUING): Before closing these meetings of the AREA and Engineering Division, is there any further business to come before the meetings?

PRESIDENT ROUGAS: If not, I shall use this beautiful gavel which has been presented to me and declare the 79th Annual Technical Conference of the American Railway Engineering Association and the 1980 Annual Meeting of the Engineering Division, Association of American Railroads, adjourned.

AAR ENGINEERING DIVISION SESSION

REMARKS BY DIVISION CHAIRMAN, L. A. DURHAM, JR.*

During the morning session, I spoke to you on behalf of the American Railway Engineering Association. I now have the privilege to welcome you to the 1980 Annual Meeting of the Engineering Division of the Association of American Railroads.

The Annual Meeting of the Engineering Division of the AAR is always a most productive and enlightening session of our technical conference. We have the pleasure of hearing from some of our top AAR officers and reports from the Research and Test Department of the AAR.

We have continued close relationship with the Operating-Transportation Committee and feel our participation has been beneficial to both the O-T Committee and the Engineering Division. Executive Director L. T. Cerny and the chairman of the Engineering Division attend the meetings to represent our organization.

The relationship between the Engineering Division of the AAR and AREA has been reviewed periodically by past presidents, and I believe it is well to again review that relationship for the benefit of new members. The governing body of AREA consists of a Board of Direction, numbering seventeen, who are elected by the membership. The governing body of the Engineering Division of AAR is a General Committee appointed by the Vice President, Operations and Maintenance Department, of the AAR. The General Committee consists of the seventeen members of the AREA Board of Direction, plus an additional four representatives selected from railroads not represented on the AREA Board. These four representatives are chosen to insure a broad representation throughout the United States and Canada.

AREA, with all of its technical expertise in developing recommended standards and specifications, cannot make its recommendations binding on any railroad. However, the Engineering Division—AAR, acting through its General Committee, can make its recommendations binding with proper approvals.

The American Railway Engineering Association is a voluntary professional association of engineers and other professionals who work with railroads but are not directly employed by railroads. The AREA is concerned with matters affecting railway fixed plant, with the publication and maintenance of standard reference manuals which cover design and performance standards, specifications and recommended field practices. AREA provides officers for and direction to the affairs of the Engineering Division of the AAR. The Engineering Division is responsible for policy and administration in matters of the fixed plant and receives its assignments from the Vice President-Operations and Maintenance of the AAR.

The July 1, 1979, relocation of the headquarters staff and offices of both the AREA and Engineering Division-AAR to AAR headquarters in Washington will strengthen the relationship between the two organizations and their relationships between these organizations and other departments and divisions of AAR.

The Research and Test Department of AAR is concerned with and has a responsibility for the conduct of research necessary to enable our railroads to improve safety and operations. The Research and Test Department of AAR now has operations in Chicago, Washington and Pueblo.

*Chief Engineer, Norfolk & Western Railway.

There is an increasing need for safety research and analysis by R&TD-AAR to respond to legislative, regulatory, and research activities of the Federal agencies. This response requires close liaison with Federal agencies based in Washington. For this reason, much of the safety research analysis formerly conducted in Chicago has been transferred to Washington and the newly created Safety Research Division.

The AREA Board of Direction in its meeting in December 1979, approved the formation of a FAST Liaison Committee, to be chaired by W. S. Autrey. Several AREA committees already have informal contacts with FAST, and the formation of this committee should open all channels of communication between FAST and AREA. Such cooperation will result in benefits to all of our railroads.

Another committee has been established, namely, the AAR Research and Test Department Committee on Track Maintenance Research, to conduct developmental research for track maintenance planning. At the conclusion of the research phase of this study, the results will be evaluated by the Engineering Division—AAR for suitability and appropriateness to the railroad industry.

Full interaction between the staffs of AREA, Engineering Division—AAR, and the Research and Test Department—AAR, will progress research needs and translation by the AREA and the Engineering Division of valuable information into design and performance standards, specifications, recommended practices, etc., in AREA manuals and other publications.

I am quite optimistic over the future of AREA, the Engineering Division, and the Research and Test Department—AAR.

Thank you.

Remarks by A.W. Johnston*

EXECUTIVE DIRECTOR CERNY:

As you know, Bill Johnston is my boss in the A.A.R. Engineering Division, and he asked me to deliver the speech he had prepared for this occasion in his absence. Mr. Johnston was unable to be here today because he and Mr. Dempsey are defending the interests of the railroads at Senate hearings on proposed amendments to the Federal Railroad Safety Act of 1970. Some of the proposed amendments, about which I do not have the time to get into detail, would be catastrophic to the railroads if not changed.

Please remember as I read these words that they are those prepared by Bill Johnston for delivery here and represent his views as an official of the Association of American Railroads.

A year ago I appeared before this group to apprise you of the impending transfer of AREA and our Engineering Division from the homey atmosphere of Chicago to the often unfriendly climate of Washington. As you are aware, the transfer was completed in July of last year.

I was rather new on my job there at that time. And, as I recall, there was considerable concern on the part of most of you about the move to your new home. As it turned out, I believe you will agree it was a most prudent decision.

I think the wisdom of the move has been borne out by developments since that time. The day has long passed in railroading when each particular discipline can afford to think of its area as independent of the others. The different disciplines must act as a system.

Having the Engineering Division and AREA in Washington has resulted in the improvement of the Division's ability to work with other divisions of the AAR—in particular, the various joint engineering and mechanical projects now underway.

For example, research proposed by the Engineering and Mechanical Divisions on rail and wheel profiles and the request for research into rail which will last 800 million gross tons on tangent track under 263,000 pound cars, is another area where this cooperation produces results.

But the most striking example of all, perhaps, is the role you played in the railroad industry's reply to the Federal Railroad Administration's proposed track standards.

For those who might not be familiar with this case, I would like to point out that when the proposed revisions to the track standards were published in the Federal Register last September, commentary on the proposals and a skeleton of suggested changes were promptly issued by the Engineering Division.

This was followed by the formation of a committee that worked on a formal reply and alternate proposals.

By having the Engineering Division headquarters staff in Washington where it could work directly with the AAR's Legal Department and the Research and Test Department, we were able to develop an excellent response to the FRA proposals—proposals which could have cost the railroads as much as $858.7 million in one-time costs and $63.5 million in annual costs, rather than the $20.2 million in one-time costs and $3.5 million in annual costs estimated by the FRA.

*Vice President-Operations and Maintenance, Association of American Railroads.

We have since heard comments that this was one of the finest presentations ever made before the FRA, and the presentation by your incoming president, Mike Rougas, was commendable in every respect. I am confident this team effort was instrumental in the FRA decision to cancel the proposed amendments to the FRA regulations containing track standards in March.

Where AREA is concerned, I'm sure that some of you were most apprehensive about its continued independence once located in Washington—in close proximity to AAR headquarters.

Not only has AREA retained its independence, the fact is, it has prospered in terms of financial health, increased membership and an all-time peak in the sale of its publications.

More importantly, perhaps, the difference between the AREA and the AAR has been more sharply delineated than it was before.

A study undertaken by the AAR through its Research and Test Department clearly affirms the value of AREA as an independent organization. The input of its non-railroad members continues to be of extreme value. And the expertise of AREA members and committees is indispensable to the railroad industry.

We are all united in a desire to see railroading continue to progress. And that it is doing—in almost every way.

One area in which we have made great advances, for example, is that of safety. In 1979, the number of train accidents—including derailments—went down by 12 to 15 percent compared with the previous year in spite of all-time record traffic levels. This was a factor in the continued reduction of the ratio of loss and damage claims payments to freight revenues.

We are carefully watching certain problem areas—these, involving the Environmental Protection Agency—concern the continued use of creosote as a wood preservative and the means of disposing of old ties containing creosote. They also concern noise regulations . . . and an increase in railroad liability exposure under a proposed Emergency Response Act . . . this act, pushed by various environmental interests, seeks to set up response procedures and a system of liability funding for response to environmental mishaps.

The industry contends that the proposed response procedures would interfere with the railroads' ability to respond to accidents and clean up after spills. It also opposes the increase in railroad liability which involves the establishment of a "superfund" to pay accident claims.

The proposed Emergency Response Act . . . the ultimate solution of the creosote and tie-disposal problems . . . and noise regulations, many of which are not really applicable to the railroad industry . . . can result in staggering new costs to an already hard-pressed industry, just at a time when we are beginning to correct many of the problems that have plagued us for decades . . . and just at a time when the industry is beginning to show significant progress in many areas.

Ton miles, which were at an all-time record level last year, are currently running more than 11 percent ahead of the 1979 pace, in spite of recession talk.

Coal traffic is up by nearly 30 percent in carloadings . . . grain by better than 45 percent in spite of the Russian embargo . . . and piggyback traffic by 4.4 percent, possibly on the way to its fourth straight record year.

Mr. Briggs has already discussed, this morning, the industry's improving financial situation, so I won't go further into that subject at this time.

Suffice it to say that today we have a truly progressive and dynamic industry . . .

We see it in our revitalization program, now in its fifth year . . . and in the restructuring movement now going on through mergers and consolidations along the lines originally suggested by a Congressionally-directed study in the 1920s and reiterated by study-after-study since that time.

What we see today is an adjustment to changed conditions and circumstances by the industry, now that it is possible to get a proposed merger through the Interstate Commerce Commission in a reasonable period of time, thanks to provisions of the 4-R Act.

In addition to the mergers in various stages of the process at this time, we have the liquidation of the Rock Island and reorganization of the Milwaukee Road under the bankruptcy laws . . . this does not mean that we will see an irresponsible, wholesale abandonment of tracks and rail freight customers. Far from it.

The Department of Transportation reports that it has received 17 bids that would pick up 75 percent of the track mileage and 88 percent of the freight traffic of the Rock . . . and offers that would preserve 77 percent of the Milwaukee's mileage and 96 percent of its traffic, when joined with the core system proposed by the trustee.

When you consider that these two roads are centered in the area of the country where railroad overbilling was most prevalent. Such reductions constitute a thoroughly justifiable "slimming down" which has long been needed for the health, or even the survival, of large segments of the industry in the Midwest.

Where *deregulation* is concerned, the railroad industry is awaiting further action by Congress.

The industry stands by a compromise bill known as Senate Bill # 1946, as it was reported by the Senate Commerce Committee late last year. This bill is threatened by an amendment which—if adopted—would put a lid on coal rates thus, in effect, "re-regulating" a large segment of rail traffic.

If this amendment is adopted, the railroad industry will oppose the overall bill.

Of course, we don't know what is going to happen there—or in the case of *competitive equity* which Mr. Briggs discussed this morning, but many members of Congress—and particularly some of the newer members—seem to recognize the fact that the plight of the railroads in recent years was in large part a result of past government policies favoring the other modes—at the expense of railroads.

We hope and believe that this recognition will lead to more equitable policies in the future.

That is the goal of the industry's renewed drive for competitive equity. That is the fight we face today. And I'm sure you will be hearing more about it in the future.

I thank you.

Remarks by G. H. Way*

Mr. President, Members, and Guests: Bill Harris, the Research and Test Department of the AAR, and I wish you a productive and congenial meeting. Like Bill Dempsey and Bill Johnston, Bill Harris is confined to Washington because of the congressional hearings on railroad safety. He asked me to give you his best wishes.

The older we get the faster time flies and the more it seems we have to do in the time we have. It seems like only yesterday that we gathered here at the Palmer House for the 1979 Annual Meeting and Technical Conference. And wasn't it only just before that that I stood here in 1978 explaining that Bill Harris could not be with us because of the congressional hearings on railroad safety? Have we made any progress? Are we even working on the real problems? In 1978 the railroad industry was under violent attack by the newspapers and on television because of an unprecedented series of derailments of hazardous materials. We think we have taken significant steps to reduce the potential for such accidents. Headshields, insulation, and shelf couplers have been found effective deterrents and have been installed on a large portion of the tank car fleet. The number of catastrophic derailments has been drastically reduced and yet, what do we read in the newspapers: in the *Washington Star* a series entitled "Our Unsafe Railroads" with headlines reading "Unrepaired Railroad Beds Often Lead to Derailments" and in the *Chicago Tribune* an article condemning land grants, one more time, and taking the opportunity once again to chastise the industry for reaping profits while letting its track deteriorate. It seems that since there was not a single fatality as a result of a hazardous material release in 1979 that our friends in the press, having nothing better to write about, have taken to speculation about how bad disasters would be if they did occur. I don't begin to understand the motivation behind such things. I suppose they always were this way and always will be. Twenty years ago I was sent to Philadelphia from Altoona with specific instructions from the Chief Engineer and General Manager to keep the railroad's name out of the Philadelphia Bulletin. And, at about the same time, listening to a lamenting trainmaster complain that if he had a pair of wheels off the track within twenty miles of Wheeling, West Virginia, that it made the front page of the Wheeling News-Register. Suffice it to say that the railroad industry has and has had an image problem. On a recent flight to Atlanta a rather intelligent fellow passenger's first question to me was, "what business was I in?" The second question was, "why are railroad tracks in such deplorable condition."

I have come to believe that the real problem we face has its roots in a general public distrust of technology. People don't tend to believe in what they don't fully understand. Then, let there be a succession of incidents such as tank car explosion in Waverly, the Three Mile Island Incident, a DC-10 crash, and a computer malfunction that ties up Washington Metro Subway, and we have a public outcry for legislation, regulation, and certification to prevent further crimes against the public. The public understands greed and "rip-offs." It is not generally well enough informed to understand technological limitations and malfunctions. Hence, broken rails or high L/V ratios become translated into exploitation by a big business which in the simplified public perception would prefer huge profit margins to repairing track.

While I can honestly challenge the public's criticism of railroads for making too much money (after all, the industry's rate of return is less than my passbook savings account pays), I must agree that in many respects the fruits of technology in recent years have been less than outstanding. There are many reasons that technological productivity has suffered. The problems are more complicated, and the constraints are more severe. Henry Ford and Isaac Watts didn't have EPA emission standards to meet in designing their automobiles and steam engines.

*Assistant Vice President, Research & Test Dept., Association of American Railroads

Furthermore, the public has been spoiled by success and fails to remember the many failures that preceded the major technological accomplishments of the past. Nonetheless, even given the problems we technologists face today, I believe our general performance is subject to some legitimate criticism. Categorize it as sloppiness, carelessness, imprecision, inattention to detail, or whatever, I perceive today cases of trends toward an acceptance of work quality and performance in many areas of technical endeavor such as communication, transportation, and industrial production that was not acceptable thirty, twenty, or even ten years ago. The *Washington Star,* in its unsafe railroads series, calls attention to a rail joint under which four new ties had been installed but never spiked. The author pointed out that experts agreed that the situation did not yet present a safety hazard. The newspaper inspected the joint over a three month period and the defect was never reported by either FRA or railroad inspectors. With heavy rail and on tangent track, I agree that the situation didn't constitute a danger to the traveling public, and I won't have considered the matter newsworthy—but it is symbolic of a kind of carelessness which we in this profession simply cannot afford. After all, budget constraints, revenue shortfalls, nor low ROI are not a satisfactory excuse for not spiking ties which you have just installed. I see research reports prepared by very bright, advanced-degree-holding professionals, containing misspelled words and poor grammar, which to my mind reflect the same kind of fault as the unspiked joint. There seems today to be an increasing impatience, a false urgency, which gets in the way of true craftsmanship. There seems to be a preoccupation with grand ideas at the expense of detail and accuracy. I have discussed my concern about these attitudes with a number of my colleagues, and we all agree that engineering academia must share in the blame. In our view, too many teachers of engineering have been intimidated by scientists. Science is pure. Engineering is contaminated by economics, or worst yet, commercialism. We next find engineering faculty and curricula beginning to imitate those of pure science. Just as has happened in English Departments where everyone wants to teach literature and no one has an interest in grammar, we find no one teaching surveying or drafting in prestige schools of engineering. These non-prestige subjects are more and more relegated to two-year certificate programs. Yet, it is the very orderliness, the appreciation of precision, and the understanding of measurement, and the theory of error that these subjects teach so well which many of us find suffering in today's technology.

While engineering depends on pure science, it is no more subordinate to it than it is to economics or business on which it also depends. Engineering is dedicated to putting the fruits of science to work for the good of society. There can be few more noble purposes for any discipline. That process, however, is very different from that of pure science. It is a "bottom line" kind of discipline. In engineering it matters little if an elegant theoretical solution is found if the mundane calculations and measurements associated with it are careless or if the detailed refinements and dirty hard work which are always required to implement it go undone.

As railway engineers, our purpose is to make available to society the safest most effective transportation at the least real cost. AREA, as our professional association, has an important role in that process. So, it occurs to me, does the AAR, its R&T Department and Engineering Division, the FRA, and so does the Transportation Research Board. As an employee of the AAR and a member of AREA, I have upon occasion been confused by the interrelationships between all of these institutions. Bill Harris and I asked Tom Hutcheson to make a study of this question about a year ago. Tom was and is uniquely qualified to make such a study. He is a past president of AREA, a past chairman of the Engineering Division, a current member of TRB, and has dealt with the Research and Test Department all through his railroad career. In addition, he is a superb railway engineer, has had both supervisory and executive level responsibility on a major carrier, and has a deep concern for the engineering profession and the railroad industry.

His study was completed in January, and I believe some of his conclusions are significant, bear importantly on the matters I have been addressing, and are worth repeating. Tom pointed out the not-so-obvious and often overlooked differences between these institutions and in their respective goals and purposes, especially between AREA and the AAR's Engineering Division and Research and Test Department. Tom Hutcheson has a very clear understanding of AREA as a voluntary professional association of private individuals whose primary aim is the advancement of knowledge pertaining to location, construction, operation, and maintenance of railways, i.e., railway engineering. The Association of American Railroads, on the other hand, is an association of railroad companies. The AAR represents those companies in matters relating to operations, maintenance, safety, research, economics, finance, accounting, law, legislation, and such other matters that require joint handling. The AAR is beholden to its member railroads. The AREA reflects the needs and wishes of individuals. There should be no confusion nor competition between AAR and AREA. There should be no superiority or hierachy between them. They are separate and distinct organizations serving different constituents. Yet there does remain some confusion between these institutions and it works to the disadvantage of both. AREA's credibility as a technical and professional association is damaged when it is viewed as an arm of an institution obvious in its advocacy of the railroads. Similarly, the very advocacy of AAR is weakened if it must work through a volunteer and unprejudiced professional association.

The responsibility for enhancing the qualifications, prestige, state of knowledge, and professionalism of railway engineers rests largely with AREA and while a number of us may be affiliated with both AAR and AREA, it is the AREA we look to for leadership in this area.

Just as the AREA, legitimately, only has an indirect interest in the prosperity of the private railroad industry, the AAR's Engineering Division and the Research and Test Department only have an indirect interest in the esteem in which railway engineers are held within the industry and by society in general.

So, if there is, as I perceive, a demise in railway engineering's recent accomplishments, reputation, and voice within the industry, I lay a portion of the blame at the feet of academia. I feel, however, that we ourselves must accept a large part of it as well. It is practicing railway engineers after all who set their own specifications and standards through AREA. It is practicing railway engineers who let academics set curricula and don't protest sufficiently loud when they see surveying or oral or written english dropped. It is practicing railway engineers who don't always adequately demonstrate the essentiality of their profession to railroading's survival. And lastly, it is practicing railway engineers who hire and fire and in the last analysis are responsible for the real training of their subordinates.

A good deal of the responsibility for turning public opinion with respect to railway technology resides with us. To a large extent, the situation is just like Pogo once described, "We have met the enemy and he is us." Public opinion of railway technology can be turned around, but only we can do it. It will require that we demand and provide no less than excellence in engineering and maintenance.

I thank you.

UPDATE ON RESULTS FROM THE FAST TRACK FAST TECHNICAL GROUP PORTION

by J. R. Lundgren*

Introduction

Mr. President, AREA Members and Honored Guests, it is indeed a pleasure for me to represent the FAST Technical Group at your conference. I thank you for the privilege.

In the limited time we have at our disposal, I would like to concentrate my remarks in two areas: First, a brief review of those track experiments which are currently in place at FAST. Second, we will explore a sampling of the track loadings which result from the operation of loaded 100-ton design car equipment on a well-maintained track such as FAST.

Experiments:

Let us first turn our attention to a tally of the track experiments currently underway. Several of these have been reported on in detail at previous conferences, so we shall not dwell on them for long. Figure 1 shows the layout of the specially-constructed 4.8 mile loop which is divided into 22 sections where specified combinations of track components and structures are installed for testing. It contains 2.2. miles of tangent, 0.4 miles of 3° curve, 0.3 miles of 4° curve, and 1.1 miles of 5° curve; the remaining 0.8 miles being transitional spirals. Figure 2 is a listing of the current track experiment titles and the corresponding sections of the loop which contain system or component evaluation tests in support of them.

The Ballast Experiment is being run in Sections 3, 17, 18, 20 and 22. Track surface and line degredation with traffic are being studied for a number of different ballast types, gradations and depths.

The Bonded and Insulated Joint Experiment has sample joints in Sections 1, 11, 16, 17, 21 and 22 for the evaluation of maintenance requirements.

Concrete ties and fasteners are being evaluated in Sections 17 and 22. Section 22 currently supports a comparative evaluation of the performance of a concrete tie track system with a wood tie track system. The Section 22 concrete tie assembly is a replication of the Northeast Corridor Improvement Project's track structure from traprock ballast on up.

The rail-highway crossing protection equipment in Sections 9 and 10 will be used to study electrical and mechanical equipment performance and maintenance under high-frequency use.

The Rail Corrugation Experiment being run in the curves of Sections 3, 13 and 17 will measure corrugation growth with traffic. The optimization of grinding procedure and frequency, as well as exploratory investigation into the causative factors affecting corrugation initiation and growth, are our basic objectives.

Our series of rail metallurgy wear investigations is continuing in Sections 3, 13, 17 and 22 with the introduction of additional domestic and foreign alloy and heat-treated rails. We do collect some fatigue failure information as well, although our sample sizes may limit statistical significance.

The welded rail end batter study arose out of industry interest in primary and secondary

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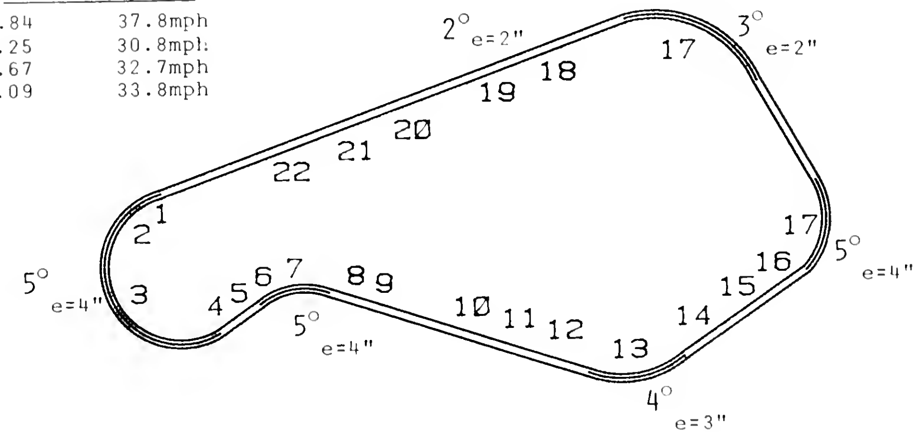
FAST

WHEEL	CW	CCW
A	INNER	O
B	OUTER	I

degree	e_b	e_a	45 MPH e_{ub}	balance speed
2	2.84	2	0.84	37.8mph
3	4.25	2	2.25	30.8mph
4	5.67	3	2.67	32.7mph
5	7.09	4	3.09	33.8mph



WRL TEST SECTIONS

FIGURE 1

FAST TRACK EXPERIMENTS

BALLAST	SECTION 3, 17, 18, 20, 22
BONDED & INSUL. JOINTS	SECTION 1, 11, 16, 17, 21, 22
CONCRETE TIES & FASTENERS	SECTION 17, 22
GRADE CROSSING PROTECTION	SECTION 9, 10
RAIL CORRUGATIONS	SECTION 3, 13, 17
RAIL METALLURGY	SECTION 3, 13, 17, 22
WELDED RAIL END BATTER	SECTION 3, 17, 22
WOOD TIES & FASTENERS FASTENING SYSTEMS	SECTION 7
WOOD TIES & FASTENERS COMPONENTS	SECTION 2, 3, 4, 9, 10, 19
TURNOUTS	SECTION 1, 9, 10, 21
MAINTENANCE OF WAY	SECTION 1 THROUGH 22
LUBRICATION	SECTION 3, 7, 13, 17 (LUBRICATORS IN 2, 5, 14, 18)

FIGURE 2

batter occurring at welds in CWR. There are some indications from earlier FAST test results that weld profile or hardness differentials may initiate corrugation formation.

The 5° Curve in Section 7 supports the ongoing evaluation of wood tie fastener systems. Evaluation is based on gage retention, resistance to rail rotation and tie plate cutting.

Various wood tie types, tie plate material and size, as well as fastener component evaluations are run in Track Section 2 (plastic tie plates), Section 3 (reconstituted and vertical laminated ties), Section 4 (18" plates), Section 9 (reconstituted, dowel laminated and horizontal laminated ties), Section 10 (elastic spikes), and Section 19 (comparison of hardwood and softwood ties).

A new evaluation of #20 AREA-design turnouts is scheduled to be placed in Track Sections 1, 9, 10 and 21 this summer. Our primary interest is in the wear evaluation of individual components and in the maintenance requirements of design variations.

Maintenance data is collected on all track sections and is used to support the individual tests for a given section, as well as for cross-comparisons; section to section. Several specific maintenance procedure evaluations are under consideration.

The lubrication sub-experiment is carried out in order to provide control on the type and quantity of lubrication applied to the FAST Track. Our goal for all track-wear experiments is to provide known, uniform levels of lubrication, consistent with experiment objectives.

Wheel Loads:

We now turn to a review of the loads 100-ton cars impart to the FAST Track. For this discussion, we will limit our observations to the wheel-rail loads occurring at the vehicle track interface as measured by a strain-gaged wheelset or by strain-gages applied to the rail. FAST, by the way, does have additional instrumentation for recording tie loads and ballast loadings.

Our primary emphasis will be on the lateral loads occurring in curves, with some review of the vertical wheel-rail loads. For this discussion, we will omit mention of longitudinal loads. They, of course, can be critically important under circumstances of high temperature and heavy braking, whether air or dynamic.

The track loadings we will examine were obtained during a short-term experiment on FAST, run during the summer of 1979. The experiment was titled the "FAST Wheel-Rail Loads Test". Two methods were used to obtain the vertical and lateral loads occurring between wheel and rail. A useful means of obtaining wheel-rail loads for many varied track conditions is the strain-gaged wheelset of which an example is shown in Figure 3. Unless more than one wheelset is available or those available are switched from vehicle to vehicle, as was done at FAST, there is the obvious limitation of the loads being vehicle-specific.

The instrumented wheelset was run under various 100-ton cars at several different speeds around the FAST Loop (Figure 4). Nominal operating speed on the FAST Track is 45 mph. Our mean time speed is on the order of 42 mph. The 3° Curve has a balance speed of approximately 31 mph; the 4° Curve, 33 mph; and the 5° Curves, 34 mph. The FAST Train generally operates over these curves with an unbalance of superelevation ranging from between two to three inches.

The test train is shown approaching one of the instrumented track sites of Section 7 in Figure 5. The second method of obtaining wheel-rail loads utilized strain gages supplied to the rail web and base as shown in Figure 6. The strain-gaged rail has the advantage over the instrumented wheelset of being able to readily record data on a great number of vehicles. However, it has the limitation of evaluating those vehicles' performance at a single point in

FIGURE 3

Instrumented Wheel Set and ALD under Radial Truck

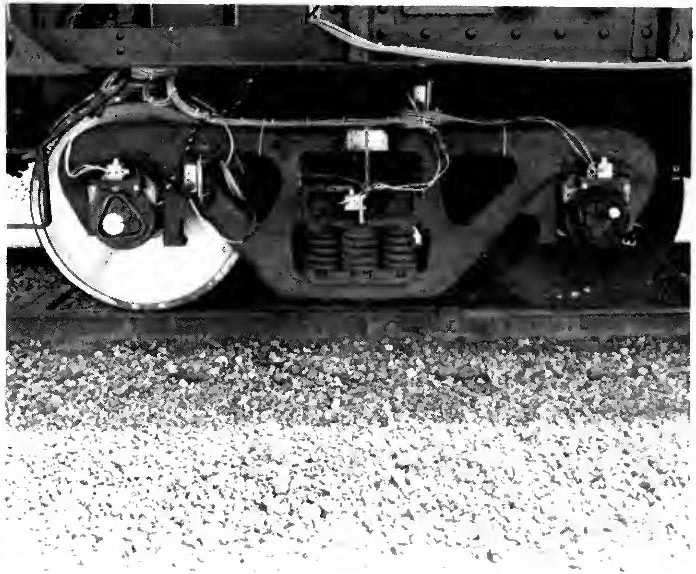


FIGURE 4

Instrumented Wheel Set on Car #47

the track and is subject to local alinement or surface conditions and the vehicle response generated by them.

Figure 7 shows one of the instrumented rail sites in Track Section 7. The strain gages on the rail have had protective covers installed. The additional equipment shown is a CP Rail laser measurement system loaned to FAST for determining wheelset angles of attack as they pass over the strain gaged rail locations. Figure 8 shows the instrumented wheelset passing over a trackside measurement location.

The instrumented wheelset records for a 100-ton car moving at 45 mph through the pair of back-to-back 300 foot 2° spirals in Section 19 is shown in Figure 9. The "A" wheel is on the low rail, the "B" wheel is on the high rail. Vertical loads are generally above the 30,000 lb. level for the high rail, generally below the 30,000 lb. level for the low rail. The lateral loads on both wheels pick up as the spiral is entered and diminish as they exit 600 feet later. The lateral load tends to be somewhat higher and more erratic on the high rail or "steering" wheelset. Note the momentary peak of 20,000 lbs. about 120 feet into the spiral. Loads above the zero lines are downward if vertical, outward if lateral. Thus, both wheels on the axle are tending to push the rails outward or "spreading the gage" on the curve. This may lead to dynamic gage-widening under the heavier lateral loads, or if the track is in less than optimum condition, to cumulative gage-widening through yielding of the tie-fastener assembly.

A similar plot of the traces for the instrumented wheelset is shown in Figure 10 for the wheelset moving through the turnout side of a No. 20 turnout from frog to point. For this particular turnout, FAST Section 16, significant lateral force peaks on both wheels are occurring ahead of and behind both the frog and the point. Again, under our operating conditions, peak lateral loads are in the 20,000 lb. range.

Figure 11 shows the wheelset passing through the tangent side of a No. 20 turnout, FAST Section 14. Although not as prolonged nor as high, there are significant lateral-force impacts at irregular intervals. As you are all aware, turnouts are not noted for their smooth riding qualities in either the lateral or vertical direction unless meticulously maintained. This is graphically illustrated by the instrumented wheel-set traces we have viewed.

Turning briefly to the trackside or strain-gaged rail indications, Figure 12 displays a histogram of the percentage of lateral-wheel loads recorded at a series of 2000 lb. intervals. The lateral loads shown are for the high-rail location in Section 7, reverse 5° curve, recorded during a series of FAST runs on June 18, 1979. The peak-lateral loads are shown according to their source as indicated by axle location. Leading axles tend to produce a larger percentage of the higher lateral loads.

Figure 13 presents similar data for the low-rail loads measured at another location in Section 7. There are some 2000–4000 lb. loads pulling the low rail inward under the action of trailing axles, but again, the lead axles are contributing high-lateral outward loads up to 20,000 lbs.

The distribution of vertical wheel loads as measured by the instrumented rail in Section 7 is shown in Figure 14. Many of the vertical loads coming onto the track fall into the 35,000 lbs.-and-up range with a small percentage exceeding 45,000 lbs.

During the Wheel-Rail Loads Test, cars with various lading levels were run. Figure 15 portrays the variation in mean-lateral loads as a function of static vertical wheel loading as measured by the wayside instrumentation in Section 7 on the lead axles of the leading trucks. The measurements on the 5° Curve illustrate a significant rise in lateral force levels on both high and low rails as car lading increases from the empty condition to a 10% overload for the 100-ton equipment.

FIGURE 5

Consist running tests, Section 7- Approaching Angle-of-Attack Device

FIGURE 6

Strain gauge installation in Section 3, Tie 0482

FIGURE 7

Wayside - Angle-of-Attack measuring

Device, Section #7

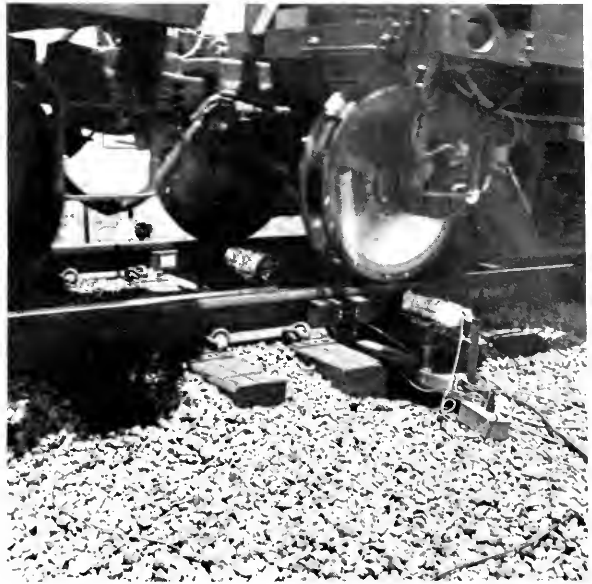


FIGURE 8

Close up of instrumented wheel, running

through Angle-of-Attack Device, Section #7

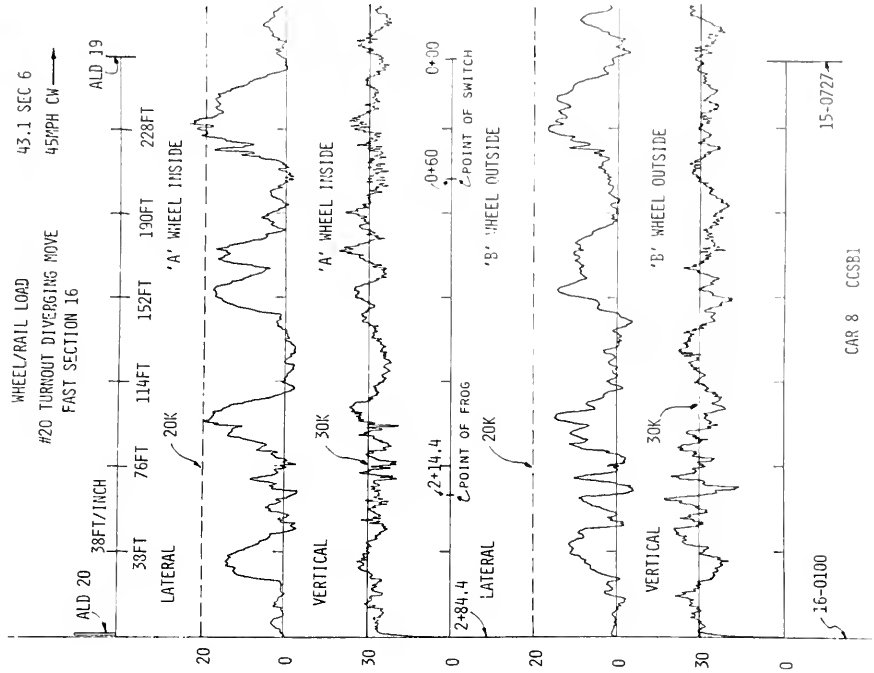


FIGURE 10

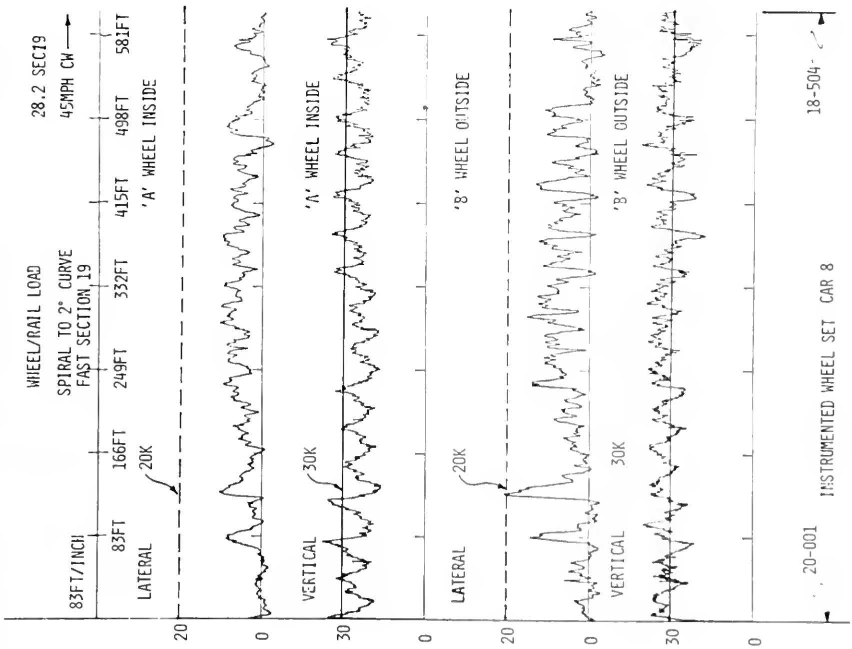
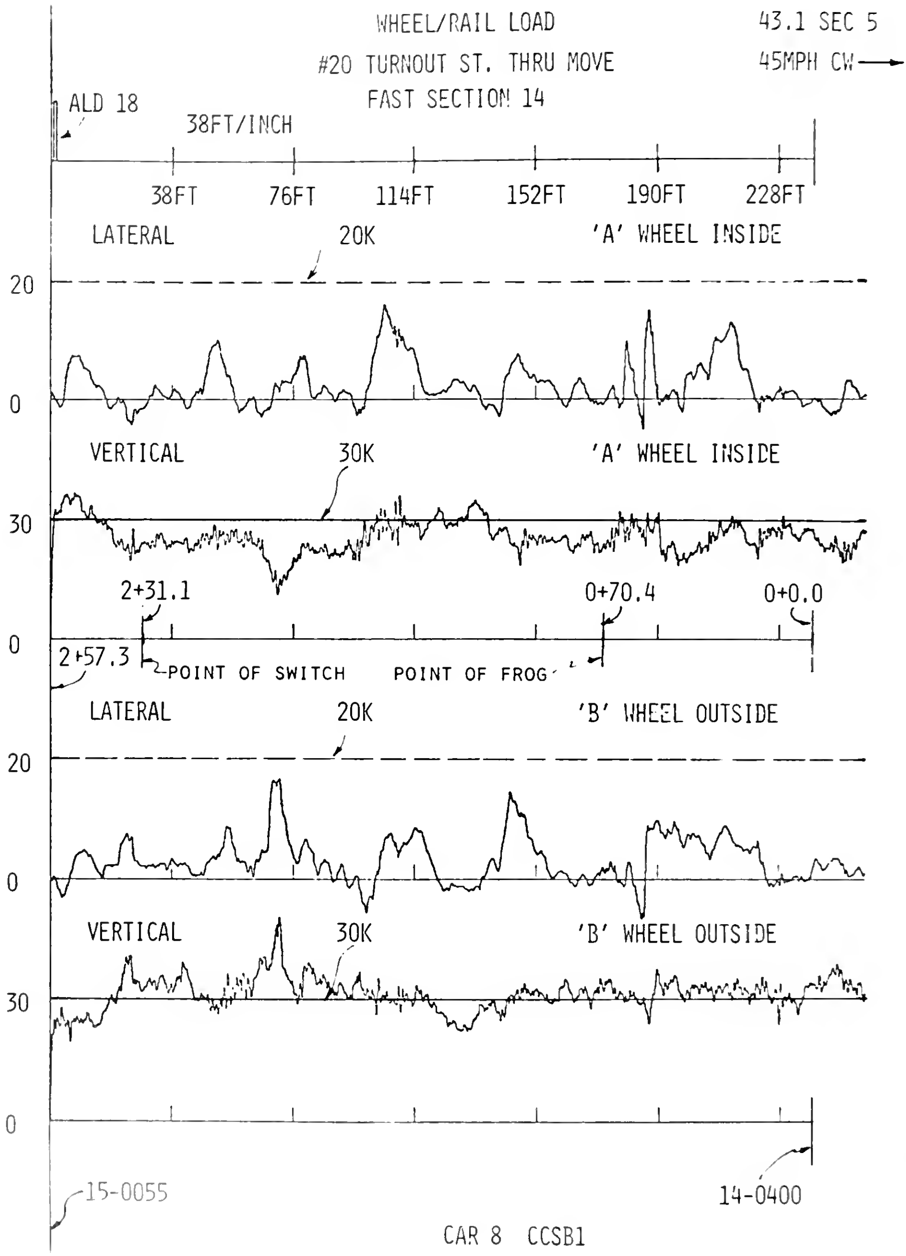


FIGURE 9



WHEEL/RAIL LOAD
#20 TURNOUT ST. THRU MOVE
FAST SECTION 14
43.1 SEC 5
45MPH CW
ALD 18
38FT/INCH
38FT
76FT
114FT
152FT
190FT
228FT
LATERAL
20K
'A' WHEEL INSIDE
20
0
VERTICAL
30K
'A' WHEEL INSIDE
30
2+31.1
0+70.4
0+0.0
0
2+57.3
POINT OF SWITCH
POINT OF FROG
LATERAL
20K
'B' WHEEL OUTSIDE
20
0
VERTICAL
30K
'B' WHEEL OUTSIDE
30
0
15-0055
14-0400
CAR 8 CCSB1

FIRGURE 11

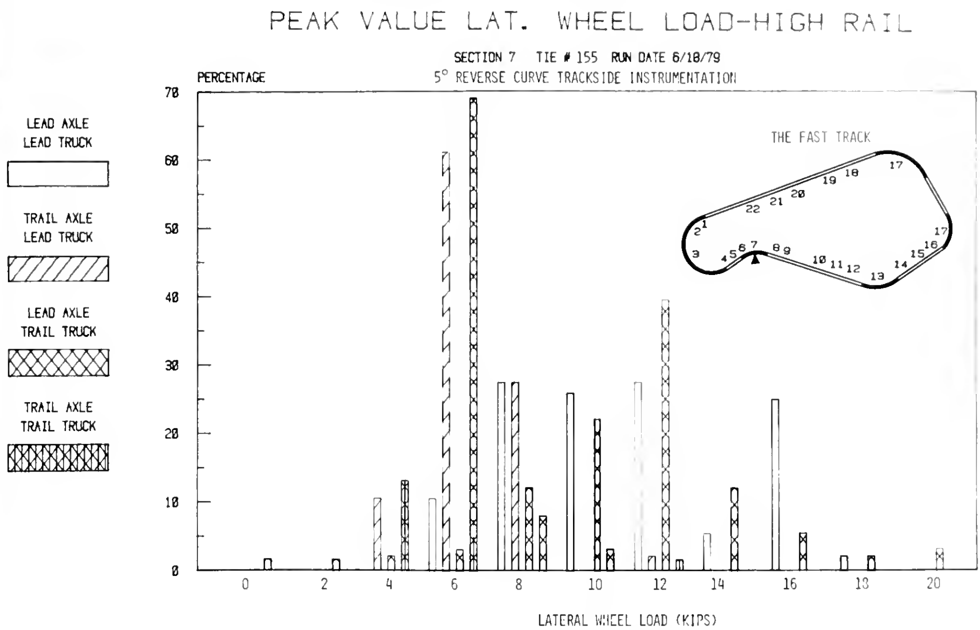


FIGURE 12

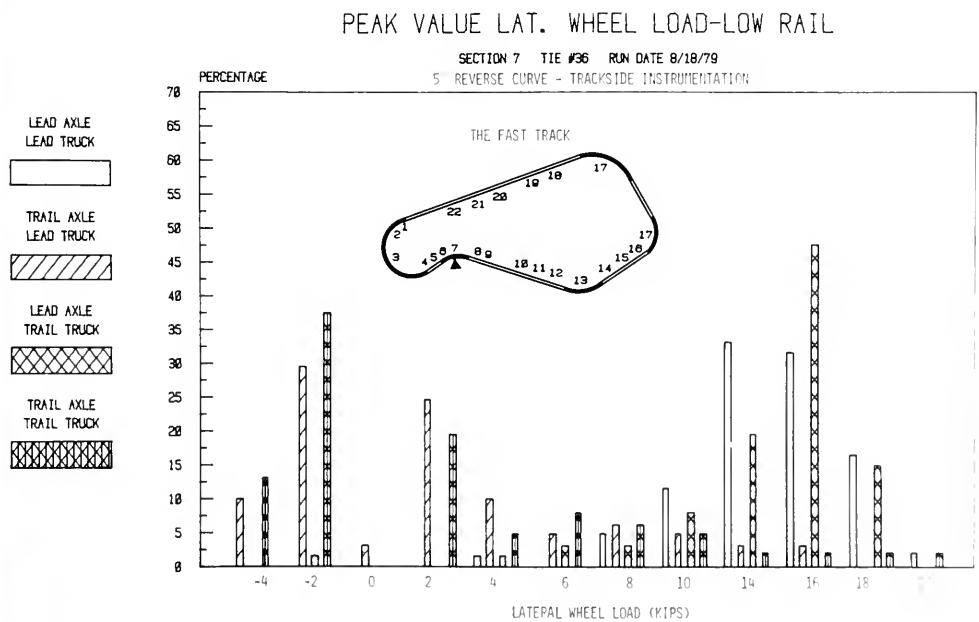


FIGURE 13

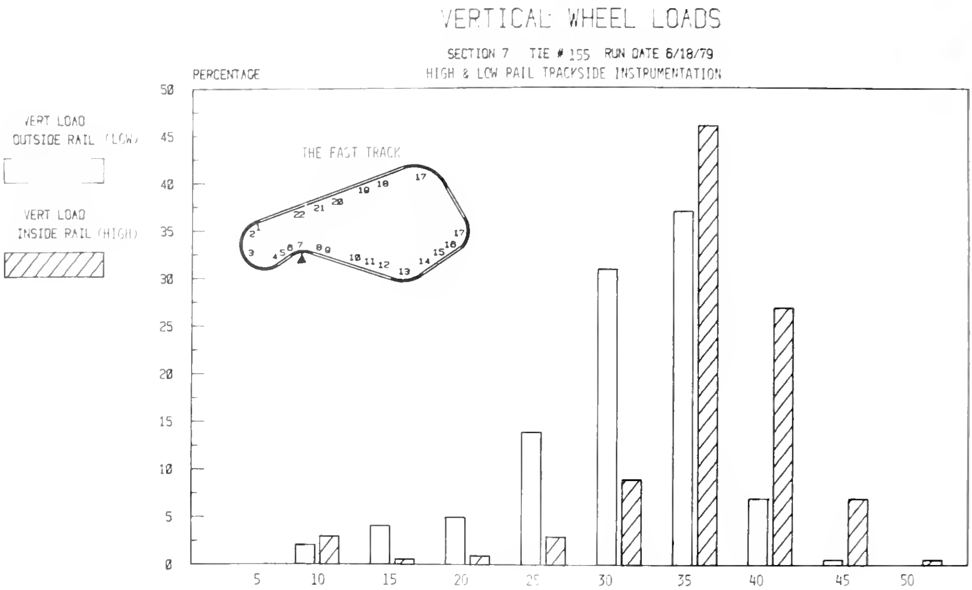


FIGURE 14

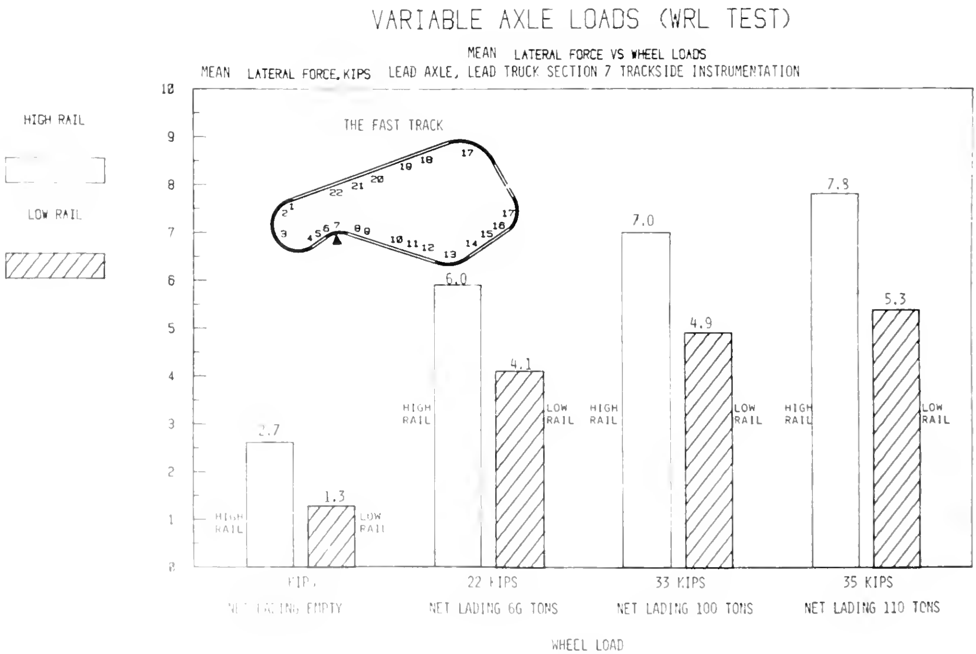


FIGURE 15

The balance of our wheel-rail load discussion returns to a presentation of loads indicated by the instrumented wheelset. Some indication of the change in mean lateral load level with degree of curvature is shown in Figure 16 for a car operated at the head end of the FAST Consist. A similar plot is shown in Figure 17 for a car operated at the rear end of the consist. One should bear in mind that the varying levels of draft and buff force in the train and the operation on grades or in reverse curves may have a significant influence on the load levels generated, particularly for a car operated at the head end. Generally speaking, mean lateral load levels in the 14,000–16,000 lb. range can be anticipated for our operating conditions.

A similar plot of mean lateral forces for the high rail in Figure 18 shows the variation between clockwise and counterclockwise movements of a car located at the head end of the consist.

An attempt to determine the significance of curve lubrication resulted in the plot shown in Figure 19. The 5° Curve in Section 17 did experience some increase in lateral load levels. Since the unlubricated run had been preceeded by lubricated testing, it is likely that considerable track lubrication remained on the rail and wheels. The relationship between lubrication, angle-of-attack and lateral force levels needs further investigation before quantitative conclusions can be drawn.

The effects of running speed on wheel-rail loads developed in FAST Track Section 3 are shown in Figure 20. The shift of the vertical load from inner to outer rail is evident as we pass from speeds under equilibrium to speeds over the balance speed for the curve. This car at the head end experiences a decrease in the level of lateral loads with increase in speed.

A similar plot for a different car operated at the rear end of the consist is shown in Figure 21. For this case, the transition to a near-balance condition on vertical loads for inner and outer wheels occurs above the balance speed. There is little variation in the lateral load levels with speed. Apart from the position in train effect, there is an indication that the characteristics of individual cars such as spring deflection, truck wear and clearances, side bearing type and condition, and roll angle play a significant role in the levels of vertical and lateral loads generated under a given set of operating conditions.

The instrumented wheelset was used to sample the performance of one radial or self-steering freight car truck in comparison with a conventional truck. The results are tabulated in Figure 22 for the various curves on FAST. The radial truck exhibits a decrease in lateral forces in all the curves. There is no significant change in the vertical loads.

An additional plot, Figure 23, shows the maximum vertical and lateral loads for the runs made above and below the balance speed of the 5° curve in Section 17. The one sigma bandwidth about the mean peak loads is shown as a shaded area. Corresponding plots for the L/V ratios are shown to the right. At 45 mph, peak vertical loads on the high rail may be expected to run near 45 kips with 30 kips on the low rail. At 30 mph, the force levels reverse, low rail to 45 kips, high to 30 kips. Lateral-force peaks exhibit a similar behavior pattern with anticipated peaks of 20 kips on the high rail and 20 on the low rail at 45 mph. At 30 mph, the low rail receives the higher load levels. Peak L/V ratios for these operations generally fell in the range of 0.3 to 0.8.

A variation in the presentation of the load data is shown in the sample percent exceedence plot shown in Figure 24. Lead wheelset lateral loads on the high rail for these consist locations are shown.

Maintenance Effects

What are the effects of these load levels on the track structure and track maintenance

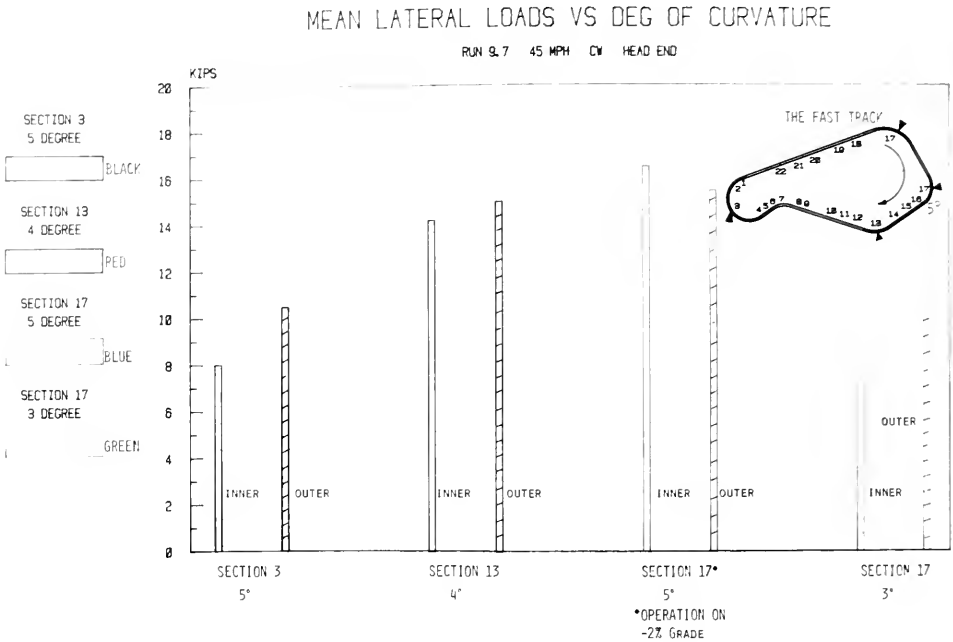


FIGURE 16

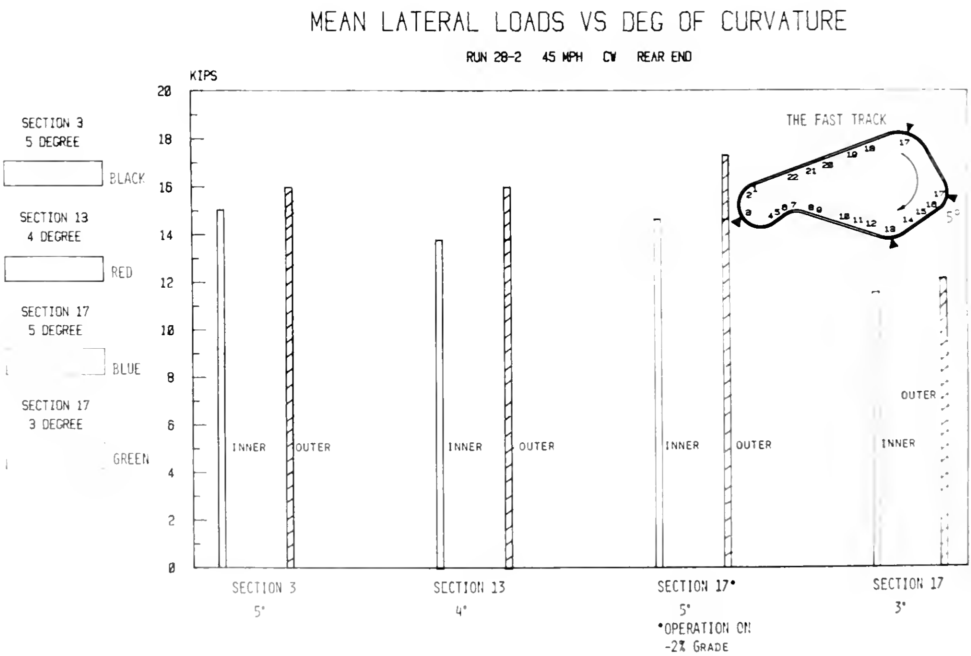


FIGURE 17

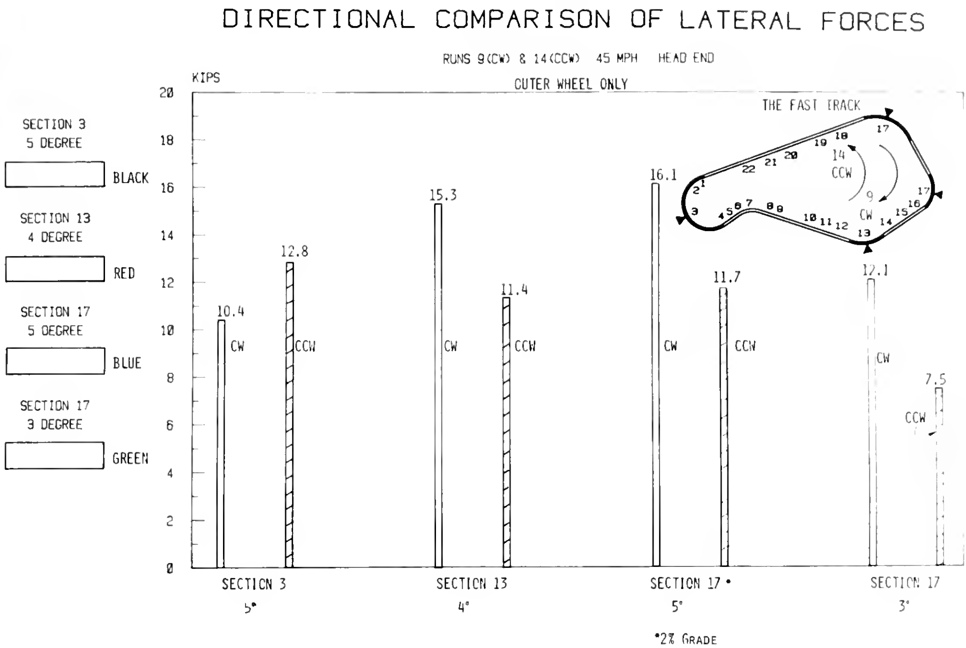


FIGURE 18

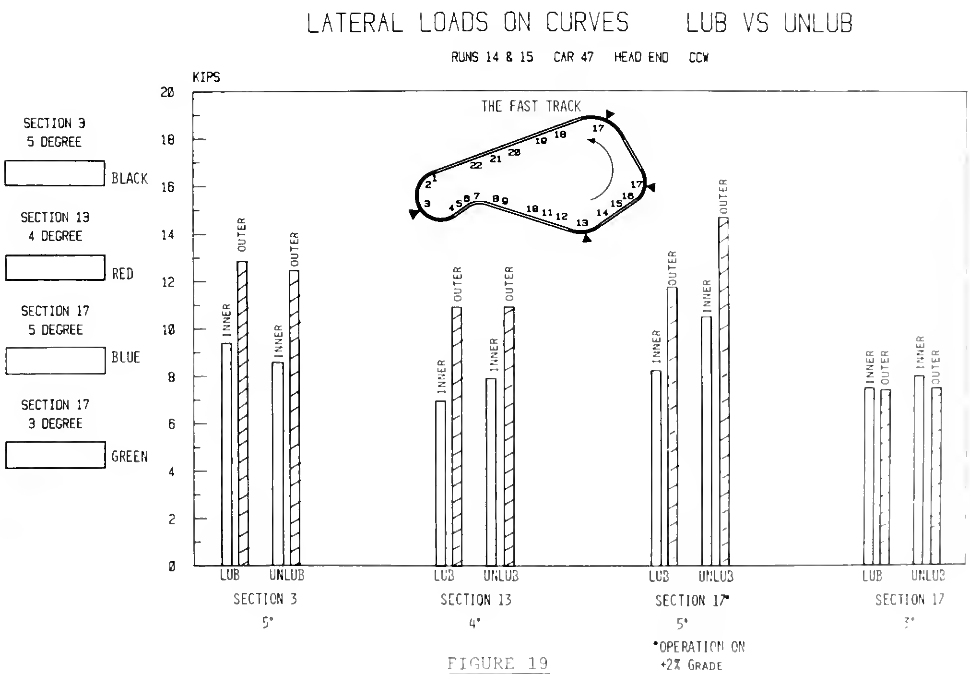


FIGURE 19

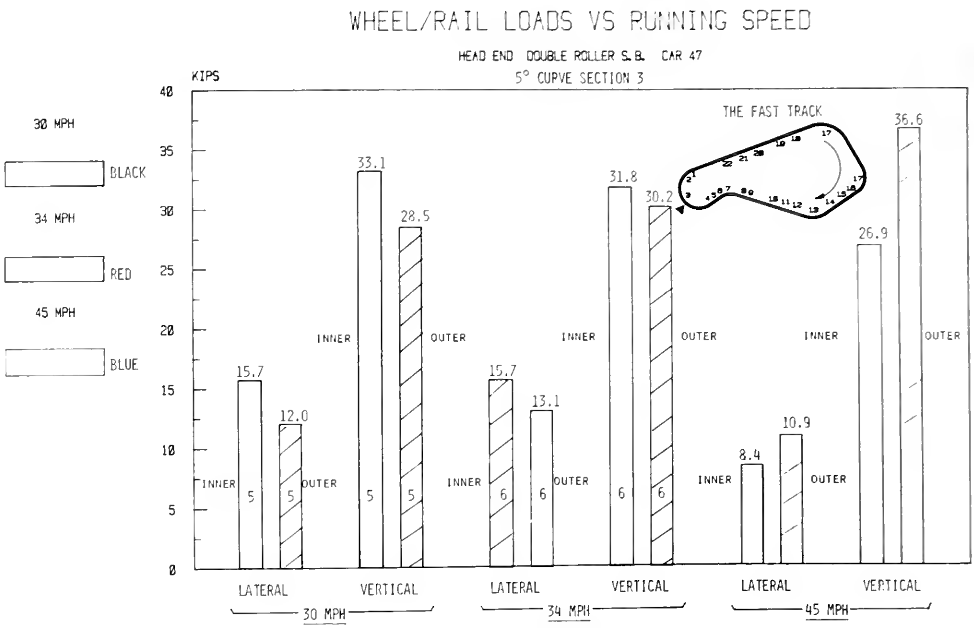


FIGURE 20

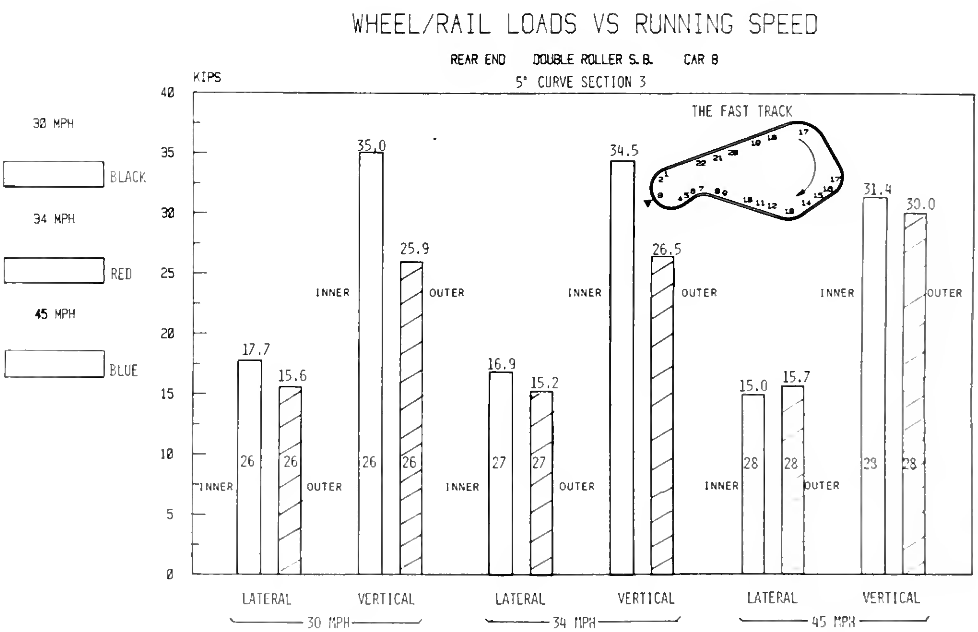


FIGURE 21

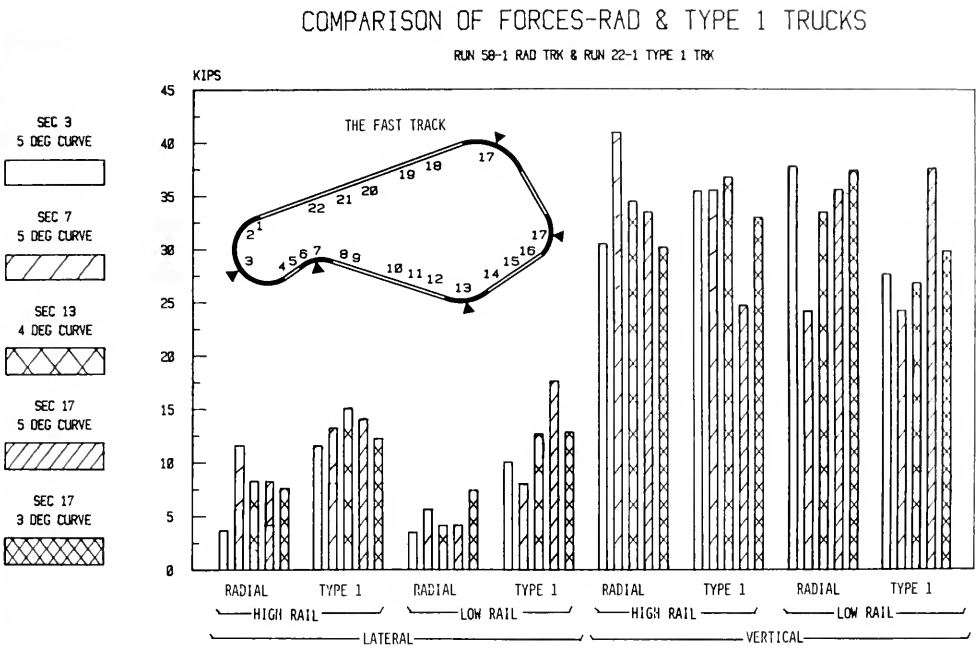
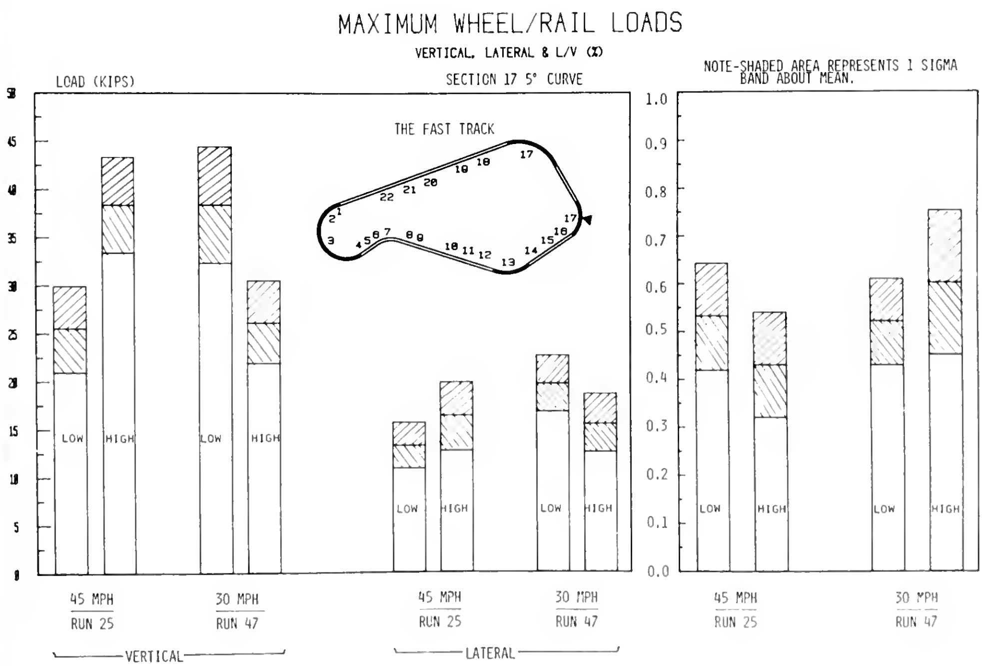
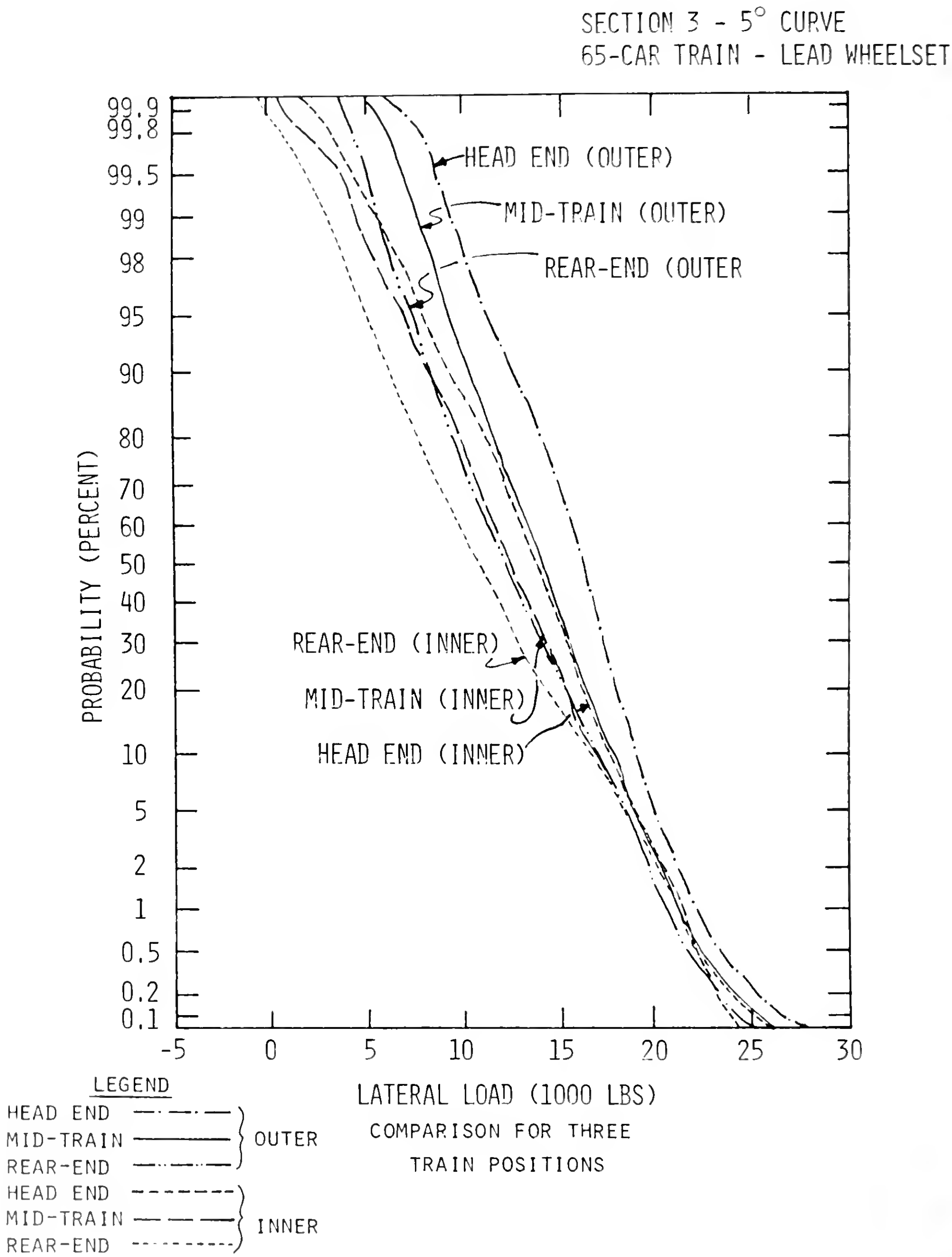


FIGURE 22

FIGURE 23





SECTION 3 - 5° CURVE
65-CAR TRAIN - LEAD WHEELSET
HEAD END (OUTER)
MID-TRAIN (OUTER)
REAR-END (OUTER
REAR-END (INNER)
MID-TRAIN (INNER)
HEAD END (INNER)
PROBABILITY (PERCENT)
99.9
99.8
99.5
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
-5
0
5
10
15
20
25
30
LATERAL LOAD (1000 LBS)
COMPARISON FOR THREE
TRAIN POSITIONS
LEGEND
HEAD END
MID-TRAIN
REAR-END
OUTER
HEAD END
MID-TRAIN
REAR-END
INNER

FIGURE 24

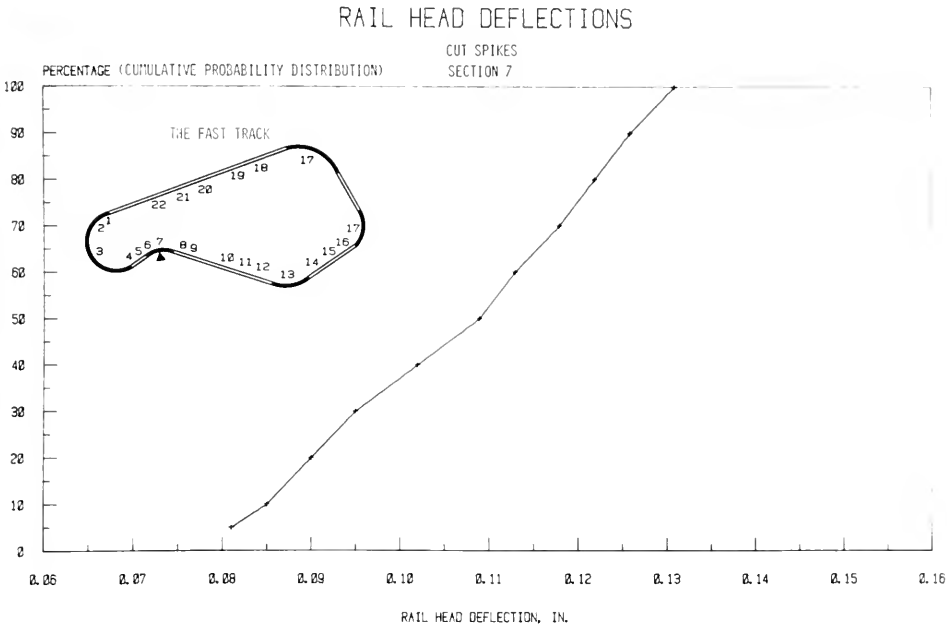


FIGURE 25

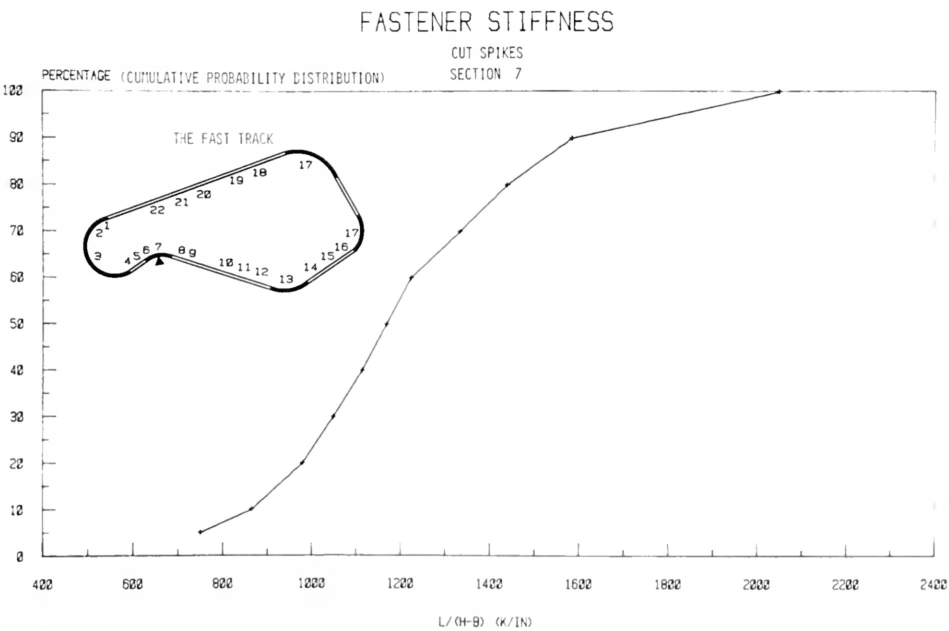


FIGURE 26

requirements? Some insight may be obtained by looking at several measures of track performance.

A sampling of the rail-head deflection data taken on the cut spikes in Section 7 is shown in Figure 25. At the time this sample was taken, the rail displayed a range of dynamic-deflection values from 0.08 inches to 0.13 inches. The deflection data has been combined with lateral loads to show the fastener system resistance to lateral movement (Figure 26). With increased traffic accumulation, one anticipates a shift in the deflection measurements to higher values with a resulting decrease in the lateral stiffness as the track "loosens up".

The change of mean-dynamic gage with traffic is shown in Figure 27. Over a 160-MGT period, mean dynamic gage widening increased by about 0.3″.

As an example of the fastener distress which may occur, Figure 28 shows the cumulative number of spike replacements which were required over the life of the screw spike test run during the second fastener evaluation series on the 5° curve. This particular installation utilized only two spikes per plate, an inadequate design.

One of the immediate effects of heavy wheel loads for the track engineer is their impact on maintenance requirements. Not only the amount of traffic, but also the type of train traffic is significant. As we have seen on various plots, axle loadings associated with 100-ton equipment produces significant lateral curving forces, as well as vertical loads. The effects of operating a train of loaded 100-ton equipment can be judged by the maintenance demands imposed on the FAST test sections. The maintenance history plots shown are for sections 3, 11 and 19 (Figures 29, 30, and 31 respectively). In viewing any of these samples, there is a readily-apparent increase in the maintenance effort (average manhours per MGT for various MGT accumulation levels) required to maintain track performance as it ages under 100-ton traffic.

Conclusion:

In conclusion, I am hopeful this brief discussion of limited data from the FAST experience has been of value. One should bear in mind that the preliminary data was gathered under FAST operating conditions. However, the trends and indications of load levels are indicative of what one might expect to find in a revenue-service environment. Variations in axle loads, train speeds, grade, curvature, equipment design and equipment maintenance are among some of the variables which may influence load levels on any particular section of track.

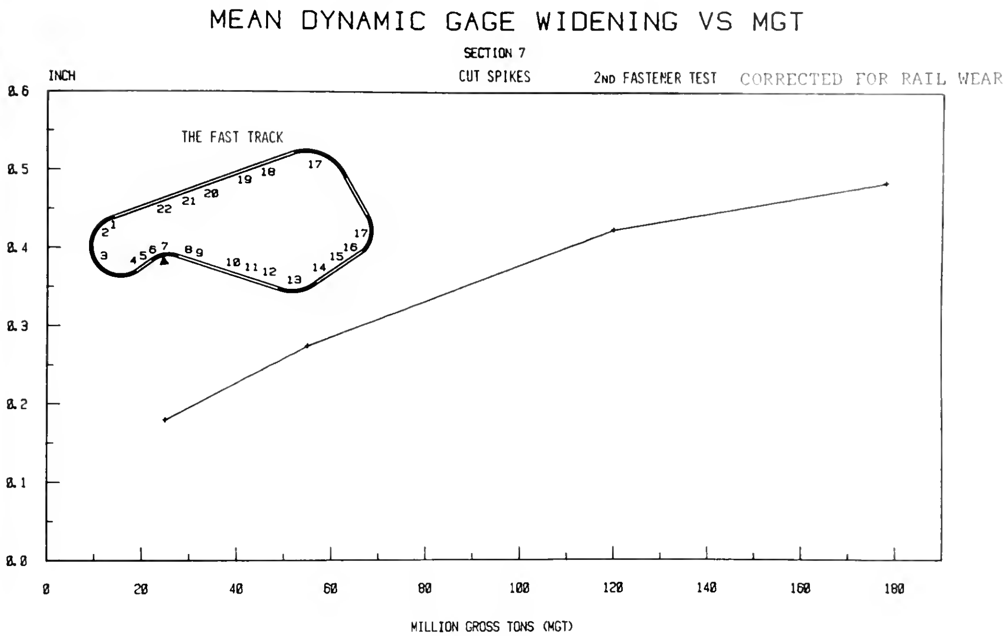


FIGURE 27

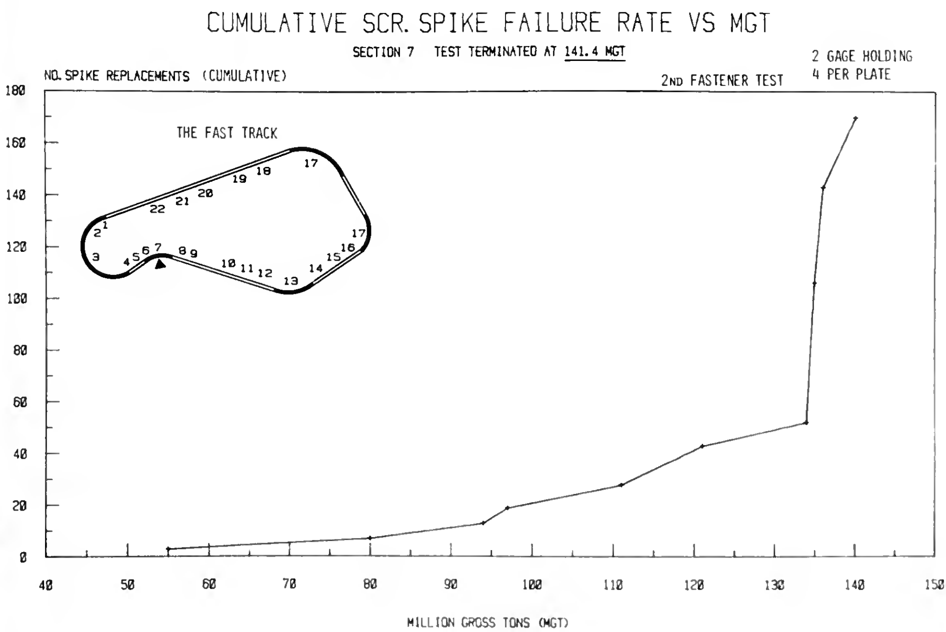


FIGURE 28

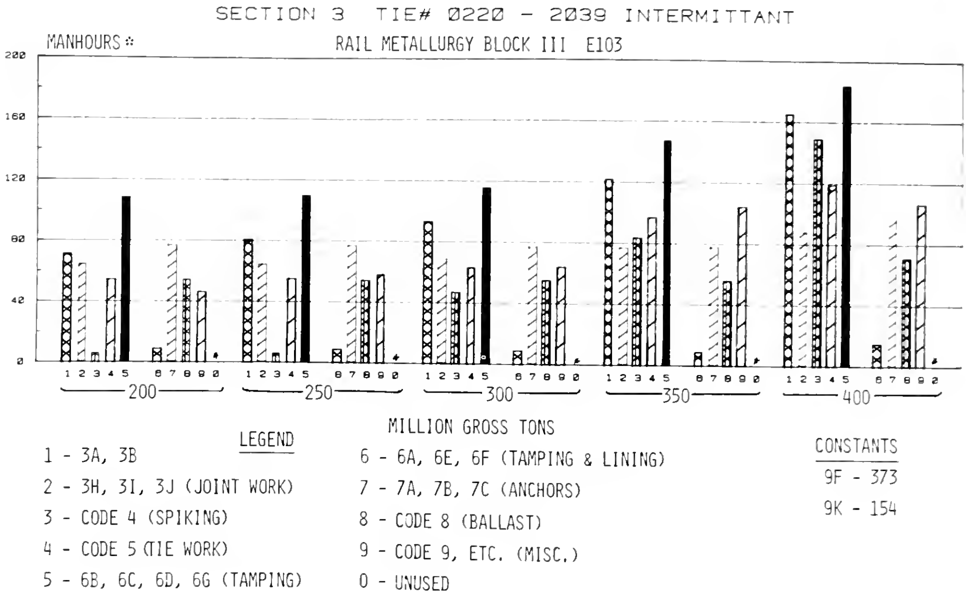


FIGURE 29

*Average Manhours per MGT for various MGT accumulation levels

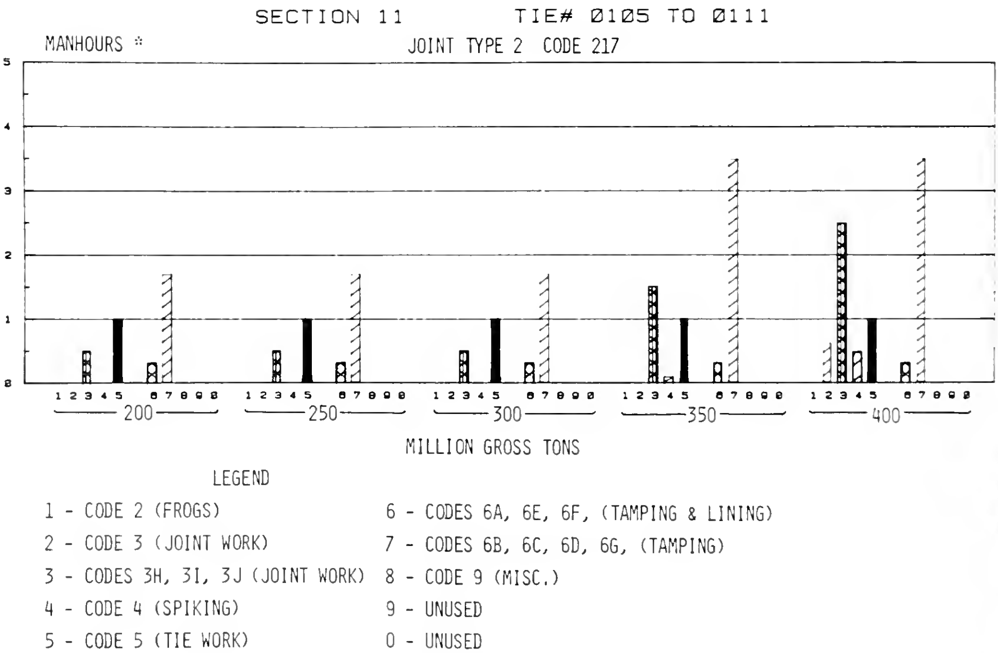
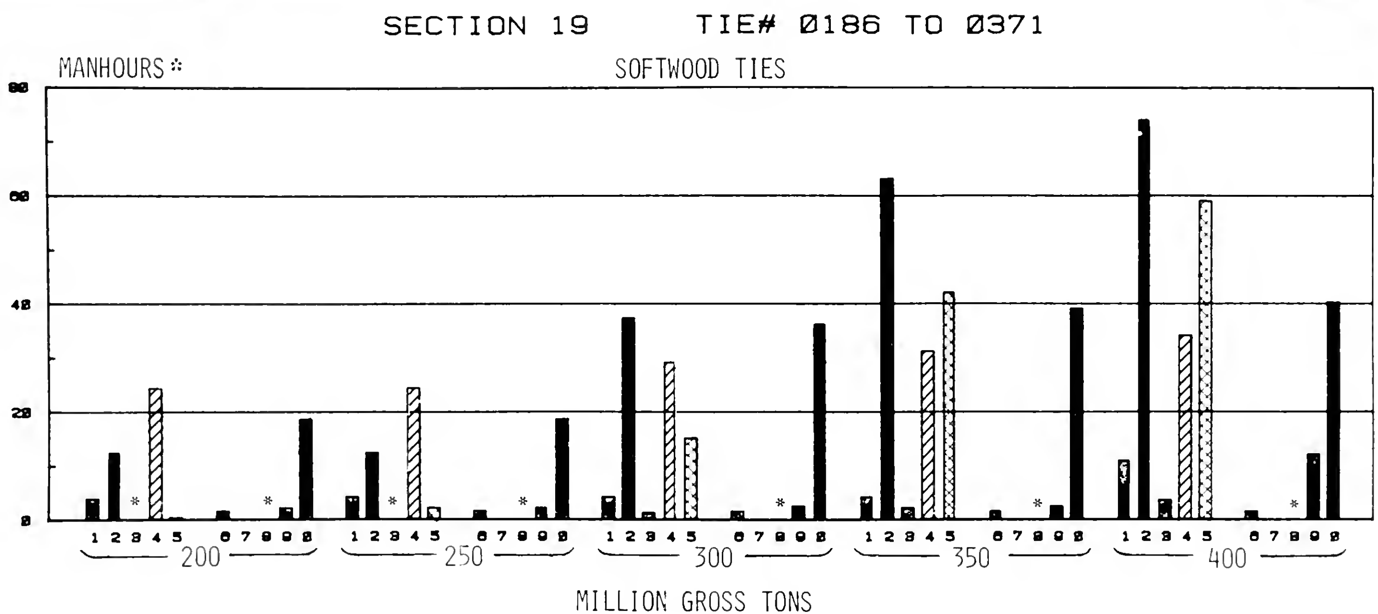


FIGURE 30

*Average Manhours per MGT for various MGT accumulation levels



LEGEND

1 - 3A, 3B (JOINT WORK)
2 - 3H, 3I, 3J (JOINT WORK)
3 - CODE 4 (SPIKING)
4 - CODE 5 (TIE WORK)
5 - 6B, 6C, 6D, 6G (TAMPING
6 - 6A, 6E, 6F (TAMPING & LINING)
7 - 7A, 7B, 7C (ANCHORS)
8 - 7D TO 7N (FASTENERS)
9 - CODE 8 (BALLAST)
0 - CODE 9 ETC. (MISC.)

FIGURE 31

*Average Manhours per MGT for various MGT accumulation levels

Rail Behavior under 125 Ton Cars on Monongahela Railway

by L. T. Cerny*

315,000 lbs. on 4 axles! 125 net tons of load! With all of the problems that railway engineers have had with 263,000 lbs. on 4 axles, the very thought of the 315,000 lb. cars carrying 125 tons might make us cringe. But today, unit trains of 140 of these cars have been rolling from south of Pittsburgh to Detroit for over 10 years now. In the recent past, research studies, primarily those by Allan Zarembski as reported in A.R.E.A. Bulletin 673 have shown a way to predict rail life under such loads, thus permitting the analysis which can indicate the overall economics of these cars.

It seems fairly obvious that the track maintenance cost per ton hauled is higher for 100 ton cars than 70 ton cars, and is higher for 125 ton cars than 100 ton cars. But it may be that the increased weight is beneficial to railroading from an overall standpoint as long as the budgets are provided for the appropriate track and bridge work that is needed. If the cars are merely put on a track without an increase in the track maintenance budget, there will be major trouble ahead.

The design of these nominal 125 ton cars indicate one possible solution to the car weight problem. This car weighs only 53,800 lbs. and carries over 130 tons, so it would seem that a similar design could weigh less than 50,000 lbs. if it was carrying only 100 tons. This would indicate that it might be possible to build a 100 ton car with the gross weight on rail of something like 246,000 lbs. The reason that the A.R.E.A. began this study last July was because these Detroit Edison unit trains are composed exclusively of 315,000 lb. cars and has been in operation for about 10 years on a line used exclusively by these unit trains. On the southerly portion of this run near the mines, the last 20 miles were constructed new for this train, and the last 144 miles have seen mostly 125 ton car unit trains since that time.

The train operates from the mines near the West Virginia-Pennsylvania border about 70 miles south of Pittsburgh and then runs via the Monongahela Railway, Pittsburgh and Lake Erie Railroad and Conrail to Monroe, Michigan, where the coal is unloaded at a generating plant of the Detroit Edison Company.

The normal train consists of 140 125 ton cars, 6 units (two of these being cut into the train about two-thirds of the way back) and a caboose, totalling about 23,000 tons, making these the heaviest trains in the world as far as regular movements are concerned. Wheel diameter is 38 inches. Total tonnage to date has been a little over 50 mgt. Calculations by the methods indicated by Dr. Zarembski would predict rail life on tangent track under this type of load exclusively of about 185 mgt, with replacement necessary due to fatigue prior to that caused by rail wear. On curves, however, which are as sharp as 10°, rail has been changed out at under 50 mgt. The new line has ruling grades against loads of 0.5% and almost continuous curvature, much of it in the 6-10° range. It appears that the life of rail on tangent track under 125 ton cars is approximately half that of under 100 ton cars, and that the life of rail under 100 ton cars is less than one-half of that under 70 ton cars.

It thus appears from the rail life standpoint that the incremental cost increase due to additional tonnage per car is no more damaging in the 100 to 125 ton range than it is in the 70 to 100 ton range, and again, as indicated by Allan Zarembski in Bulletin 673, the funda-

*Executive Director, AREA and Engineering Division, Association of American Railroads.

Figure 1—Detroit Edison Unit Train with 140 4-Axle 315,000 lb. gross weight cars (1980 photo).

Figure 2—Route of Unit Train of 315,000 lb. cars using jointed relay 112 lb. rail over 30 years old (1980 photo).

Figures 3 and 4—scenes on Route constructed south of Waynesburg, Pa. in 1968 for use of unit trains of 315,000 lb. cars (1980 photos).

mental breaking point in rail behavior wherein fatigue becomes more important than wear occurs just above the axle load created by 220,000 lbs. on 4 axles, the nominal 70 ton car (although some coal hoppers of 220,000 lbs. carry as much as 83 tons). While the southern end of the line has been built exclusively for these trains, the majority of the route for this train is over trackage which is used for other general railroad trafifc. This involves the trains going over many switches and other special trackwork that have nothing to do with the operation of this train, and, in some cases, although the maximum speed of this train is limited to 25 mph, it is traversing superelevations designed for passenger trains, such as the commuter operation on the P & LE west of Pittsburgh. In one way, this train could be referred to as the "Concorde" of railroading, in that it is functioning, but most authorities would say it was beyond the economic limits of present technology. However, it presents a very fundamental question of direction. Ever since railroading began over 150 years ago, the trend has been towards larger and heavier equipment. If we are now to say that the 100 ton car is as heavy as equipment should get, we are advocating a fundamental change in this direction. As technologies in other modes improve, such a change in philosophy could mean that railroading would not keep competitive.

My personal experience includes cars designed for, and operated at an axle load of 136,000 lbs. (which is equivalent to a 544,000 lb. car on 4 axles). These cars were used on the EJ & E Railway, with 115 lb. rail, although at slow speeds and with extremely low rail life. The basic problem of heavier axle loads appears to be in the rail itself, and the A.R.E.A. has advocated further research in this area.

One of the mysteries of this investigation was a stretch of 112 lb. rail from 1946-1947 with 4-hole joint bars, which has remained in excellent condition under this traffic. While the records of previous service on this rail could not be located, it apparently was changed from a heavy-duty main line from somewhere on the present Conrail system, and may indicate the benefits of work hardening. Heat-treated 115 lb. rail has held up as well or better than the 112 lb. rail used, but lubrication is certainly a factor that must be considered.

I should note that it is reported that the Black Mesa and Lake Powell has returned to loading its cars to 122 tons after a period of reducing this loading to 100 tons. Dan Stone will now get into more of the details of his study of the rail on the new line used by the Detroit Edison trains.

COMPARISON OF RAIL BEHAVIOR WITH 125-TON AND 100-TON CARS

by D. H. Stone*

INTRODUCTION

Rail lines which haul a single commodity in cars of uniform size can be used as laboratories, within which the effects of wheel load on rail life may be evaluated. The above condition is found at the Facility for Accelerated Service Testing (FAST) at Pueblo, Colorado, and on many mining railroads throughout the world. Table 1 gives a listing of such railroads and the size of cars used. Thus, the effects of wear, defect formation, and plastic flow on rail life, as a function of car weight or axle load can be evaluated. Each of these effects will be discussed independently.

WEAR

Before comparing wear data as it affects rail life, one must choose a limit of wear which represents the useful life of the rail. Hay [1] states that the wear limit of a rail is usually accepted as 25 percent of the head area. Figure 1 shows the limit on head wear accepted by seven iron ore railways to be 25 to 30 percent [2]. The 30 percent value has some standing in law, since the Internal Revenue Service has ruled, in a case involving the Chesapeake and Ohio Railway Company [3], that this value may be used in a formula to determine the monetary value of relay rail.

If, for the purpose of this report, a value of 25 percent head wear is used as the limit of wear life, the wear on the Waynesburg Southern (WS), with its 318,000 pound cars, may be compared to other railways using cars of lower gross weights. Figure 2 compares the wear rates of the Waynesburg Southern with those of the Quebec, North Shore, and Labrador Railway, which carries iron ore in cars of 254,000 to 264,000 pound gross weight [4]. Also included for comparative purposes are three data points from the Mt. Newman Railway of Australia.

Notice that the limited Mt. Newman data fits well with the equivalent QNSL data, even though climatic conditions are radically different between the two railroads. (The Mt. Newman Railway operates through the Pilbara Desert of Northwest Australia). Table 2 shows that the wear rates range from 5.5 times, in the case of tangent track, to 1.2 times, in a 4 degree curve (437 m radius), greater for rail under 125-ton cars than under 100-ton cars. This effect is shown quite clearly in Figure 2. In fact, the difference in rail wear rate for 125-ton cars seems to be decreasing as curvature increases with the wear rates equivalent to the 100-ton cars in curves over 5 degrees (350 m radius). It is interesting to note that the effect of high-strength** rail is to cause the rail to wear at approximately the same rate as standard rail in a curve with 2 degrees less curvature. Thus, one might construct a hypothetical curve for rail wear of high-strength rail under 125-ton cars. This is indicated by the dashed line in Figure 2, which shows the theoretical head losses to be well above those measured for 100-ton cars on both standard and high-strength rail. This projection would lead one to expect that high-strength rail would give no more than a 50 percent increase in life under 125-ton cars, rather than the experienced 250 to 300 percent increase in life under 100-ton cars.

*Director Metallurgy, Research & Test Dept., Association of American Railroads.

**High-strength rail is herein defined as described in the AREA Specification entitled, "Rail," Chapter 4, 1979 version, namely rail of 321 to 388 Brinell Hardness, whether heat-treated or alloy.

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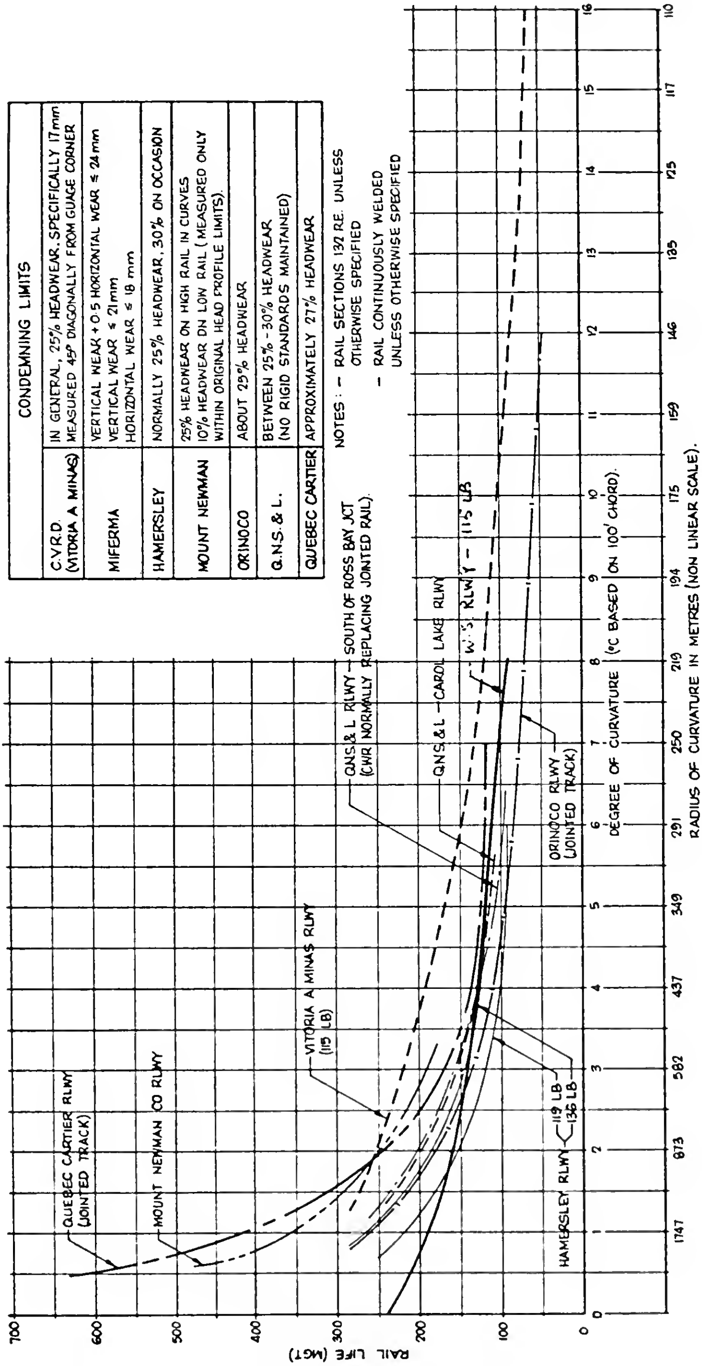

Figure 1 Rail Life as a Function of Curvature (from Reference [2] with WS Data Added)

TABLE I

FREIGHT CAR CHARACTERISTICS ON SELECTED RAILROADS

Railroad	Maximum Gross Car Weight lb. (kN)	Wheel Diameter in. (m)
Waynesburg Southern	315,000 (1400)	38 (0.965)
FAST (Pueblo, Colorado)	263,000 (1170)	36 (0.914)
Quebec, North Shore, and Labrador	266,000 (1184)	36 (0.914)
Mt. Newman (Australia)	263,000 (1170)	36 (0.914)
Hammersly (Australia)	263,000 (1170)	36 (0.914)
CVRD (Brazil)	200,000 (890)	36 (0.914)?

TABLE 2

RAIL WEAR vs CURVATURE FOR 100 AND 125 TON CARS

Curvature (Deg.)	100 ton (Std. Rail)*	Rail Wear (mm^2/100 MGT) 100 ton (H.S. Rail)**	125 ton (Std. Rail)*	Ratio 125-ton/100-ton
Tangent	50		263	5.4
1°	100		325	3.3
2°	200		375	1.9
3°	350		424	1.2
4°	410	160	474	1.2/3.0
5°		280	523	1.9
6°		380	573	1.5
7°		500	622	1.2

*Standard Rail
**High Strength Rail

When the above data is plotted as wear life (based on 25 percent head wear), it can be compared to the rail life curve of other railroads. Figure 1 is such a plot, which shows that the rail life on the Waynesburg Southern (with 125-ton cars) is reduced to 2.7 times of that of railroads with 100-ton cars, depending upon the individual railroad at low degrees of track curvature.

When rail wear particles from the Waynesburg Southern were examined by scanning electron microscopy, the particles were found to be in excess of 500μm (1.3×10^{-5} in.) in diameter, as shown in Figure 3. Corresponding wear particles from the British Columbia South line of the Canadian Pacific, which result from the passage of 100-ton cars, have been shown to be approximately 100μm (2.5×10^{-6} in.) in diameter by Kalousek and Bethune [6]. Note that the ratio of volumes of typical rail wear particles falls within the same range as the ratio of the wear rates between the QNSL and WS, as shown in Figure 2.

PLASTIC FLOW

Plastic flow of the steel in a rail head towards the gage side of the rail has two effects. First, material is continuously being transferred towards the passing wheel flanges, where it can be sheared off and lost. Secondly, shelling and transverse failures (detail fractures) are known to originate within the areas of plastic flow. Marich, et. al. [7] have shown that the plastic flow observed in rail steels can be reproduced in the laboratory by means of cyclic compression tests. The results of these tests, for standard and high-strength rails, respectively, are shown in Figures 4 and 5. If the surface contact stress is calculated by dividing the calculated contact areas into the corresponding car load, the surface contact pressures may be calculated and compared with the values shown in Figures 4 and 5. The surface contact pressure for a 263,000 pound car on 36 inch diameter wheels is 113.8 ksi (784.6 MPa), and that for a 315,000 pound car on 38 inch wheels is 117.8 ksi (812.2 MPa). It is of interest to note that for standard rail the curve for a 120 ksi (827.4 MPa) stress shows an increasing deformation rate with the number of applied loads, whereas the 100.0 ksi (689.5 MPa) load curve shows a slow steady increase in deformation rate, under the same condition. This is not the case, however, for high-strength rail, which has a yield stress in the range of 130 ksi (896.3 MPa), and is thus fairly resistant to plastic flow at stresses lower than 130 ksi (896.3 MPa).

DEFECT FORMATION

Traditionally, the dominant criteria for the replacement of rail in main line track has been either excessive end batter or head wear. The increasing use of continuously welded rail has significantly decreased the occurrences of rail end batter. Thus, rail head wear remains as the dominant replacement criterion. However, with the increasing traffic loads, and especially the increasing wheel loads, that the track structure is being called upon to support, the development of fatigue "defects" in the rail is emerging as a major replacement criterion for main line tangent track. In fact, current and proposed safety criteria now emphasize the detection of fatigue defects as they develop in tangent track. Thus, it appears that, in many instances, fatigue is also an important criterion for replacing rail in service.

Recently, two methods of predicting the rates of defect formation in rails have been developed. One method, developed by Zarembski [8, 9, 10], enables predictions of the fatigue failure rates of rail to be made. Because the requirements for this analysis involve only the car weights and rail size, the life prediction calculations may be made in the absence of actual service defect data. However, it does predicts a rail fatigue life, which represents the failure mode for about 3 percent of the rails in service, and is thus valid for comparative purposes, but not for the determination of defect rates. Figure 6 shows the results of this analysis when applied to 100-ton and 125-ton car unit trains, and compared with actual load spectra obtained

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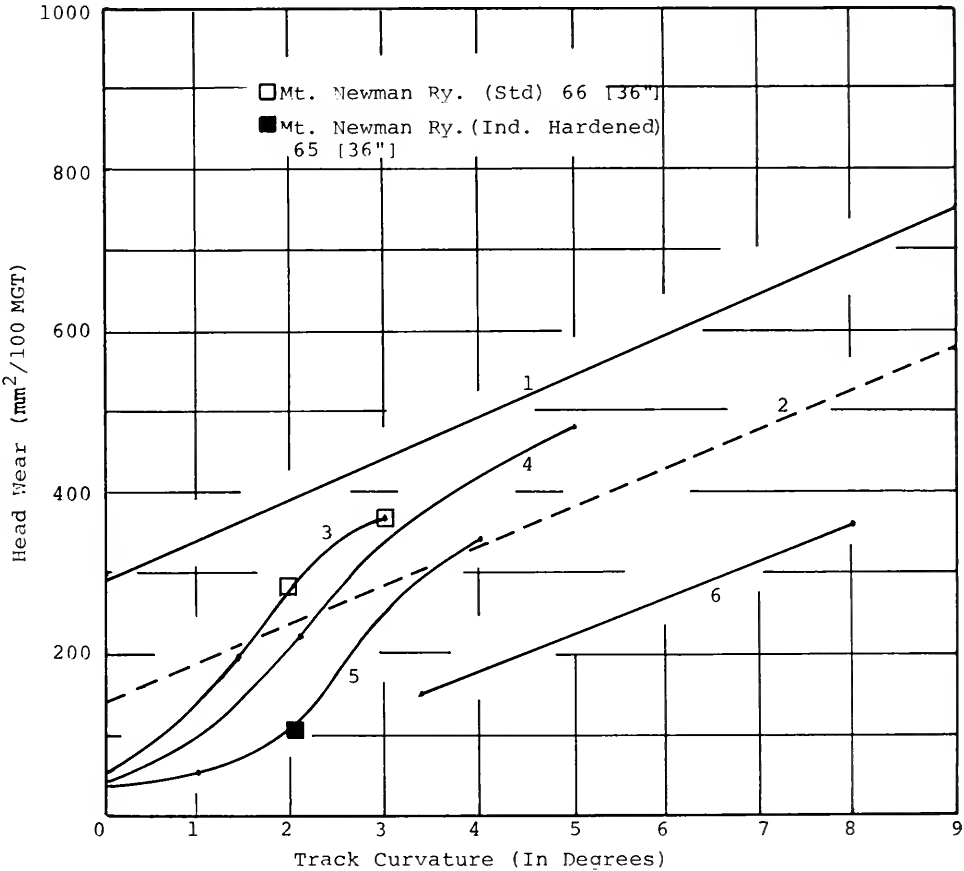

Figure 2. Comparative Rail Head Wear Rates on Railways Using 100 and 125-Ton Capacity Cars.

1. W.S. (std. rail) 78.75 kip axle load (38 in. wheels)
2. Hypothetical curve for high strength rail wear under 125-ton cars
3. QNSL (std. rail) 62-66 kip axle load (36 in. wheels)
4. QNSL (std. rail) 62 kip axle load (36 in. wheels)
5. NL Ry. (std. rail) 65.6 kip axle load (36 in. wheels)
6. QNSL (F. heat treated rail) 62-66 kip axle load (36 in. wheels)

gure 3 Scanning Electron Micrograph of Wear Particles from the Waynesburg Southern Railway, X200.

TABLE 3
WEAR LIFE OF VARIOUS RAIL SECTIONS UNDER 125-TON CARS ON TANGENT TRACK, BASED UPON DATA FROM THE WAYNESBURG SOUTHERN RAILWAY

Rail Section	Head Area (in^2)	Head Area (mm^2)	Life Based on 25% Wear (MGT)
115 RE	3.91	2520	240
132 RE	4.42	2850	271
136 RE	4.86	3135	298
140 RE	5.00	3225	307

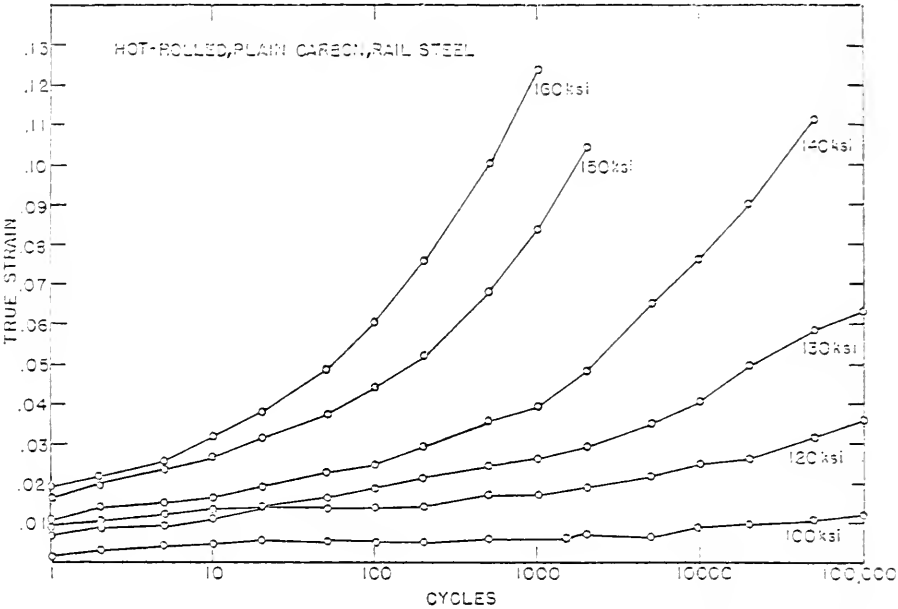

Figure 4 Cyclic Deformation Behavior of Standard Carbon Rail Steel [7].

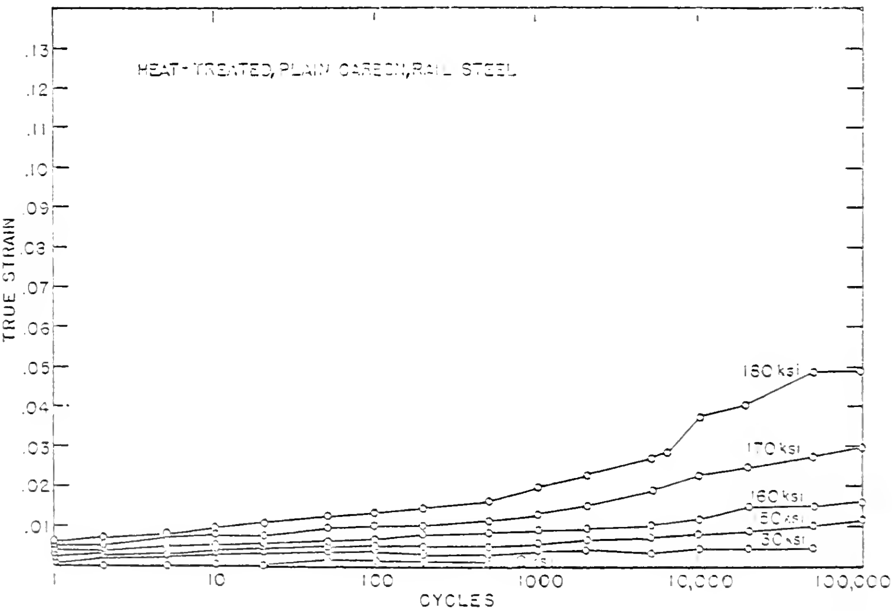

Figure 5 Cyclic Deformation Behavior of Fully Heat-Treated Rail Steel [7].

from a test section on the Union Pacific Railroad. Note that for 115 pound rail, a rail life of 170 MGT is predicted for unit trains of 125-ton cars, as opposed to a life of 330 MGT for unit trains of 100-ton cars.

The second method, developed by Stone and Besuner [11, 12], relies upon actual defect data to predict defect occurrence rates. Upon analyzing rail defect data from six main line test sections, it was found that the data made an excellent fit to a Weibull distribution, and this could be used to calculate defect rates. To date, there have been six fatigue defects located in the heads of rails on the Waynesburg Southern, and the adjoining Ten Mile Run (TMR) branch of the Monongahela Railroad, which have each accumulated 49 MGT of 125-ton car traffic. Figure 7 shows a plot of percent of rail failure *vs* MGT for the WS-TMR track, and for one of the original Santa Fe railroad test sites, which had 119 pound rail. Although the Santa Fe test site experienced the passage of a small percentage of 100-ton car (0.6%), it was primarily subjected to less than 70-ton cars (95.3%). Notice that for the 315,000 pound cars (on the WS), 0.01 percent of the rails (the first failure) occurred at 30 MGT, whereas on the line using lighter cars, no defects were formed below 100 MGT of traffic. This 100 MGT formation period for rail defects has been found to be consistent over normally-operated U.S. railroads. If the data for 315,000 pound cars is extrapolated beyond 50 MGT, and if the average rail life on the Waynesburg Southern-TMR should reach 100 MGT, this analysis predicts that the percentage of failed rails would reach 0.35 percent (approximately 40 cumulative defects). Over the time period from 90 to 100 MGT, about 12 defects would be expected, which is approximately 100 times the current rate of one per 10 MGT. The two-parameter Weibull distribution for the 315,000 pound cars on 115 pound rail may be represented by the Equation:

$$\mathrm{PD(MGT)} = 1\text{-}\exp\,[-(\mathrm{MGT}/570)^{3.9}] \qquad \text{Equation 1}$$

where PD(MGT) is the probability a rail will have a defect before undergoing a particular MGT of usage. The defect occurrence or failure rate, λ(MGT), can be calculated by the Equation:

$$\lambda(\mathrm{MGT}) = 6.97 \times 10^{-11}\,(\mathrm{MGT})^{2.9} \qquad \text{Equation 2}$$

Finally, it should be remembered that wear behavior can have both direct and indirect effects upon the stresses, and hence upon the rate of defect formation and their growth, through a size reduction of the rail [13].

DISCUSSION AND CONCLUSIONS

With respect to rail, there are two changes that can be made to improve its resistance to wear, plastic flow, and defect formation. These are to (1) increase the rail section size, and (2) to employ high-strength rail.

Increasing the section size will not affect the wear rate, but since there is a greater head volume in a larger rail, there is more material to be worn away to reach the point of 25 or 30 percent head wear. For example, Table 3 tabulates the wear data from Figure 1, and shows that the use of 136 or 140 pound rail would increase the rail life by approximately 25 percent (approximately 60 MGT). It should be noted; however, that a 136 pound rail is 18% heavier than 115 pound rail, so that there would be a corresponding increase in cost for the heavier rail.

With respect to plastic flow, there would probably be no advantage in using heavier rail, because this is a problem associated with contact pressures, which are independent of rail size.

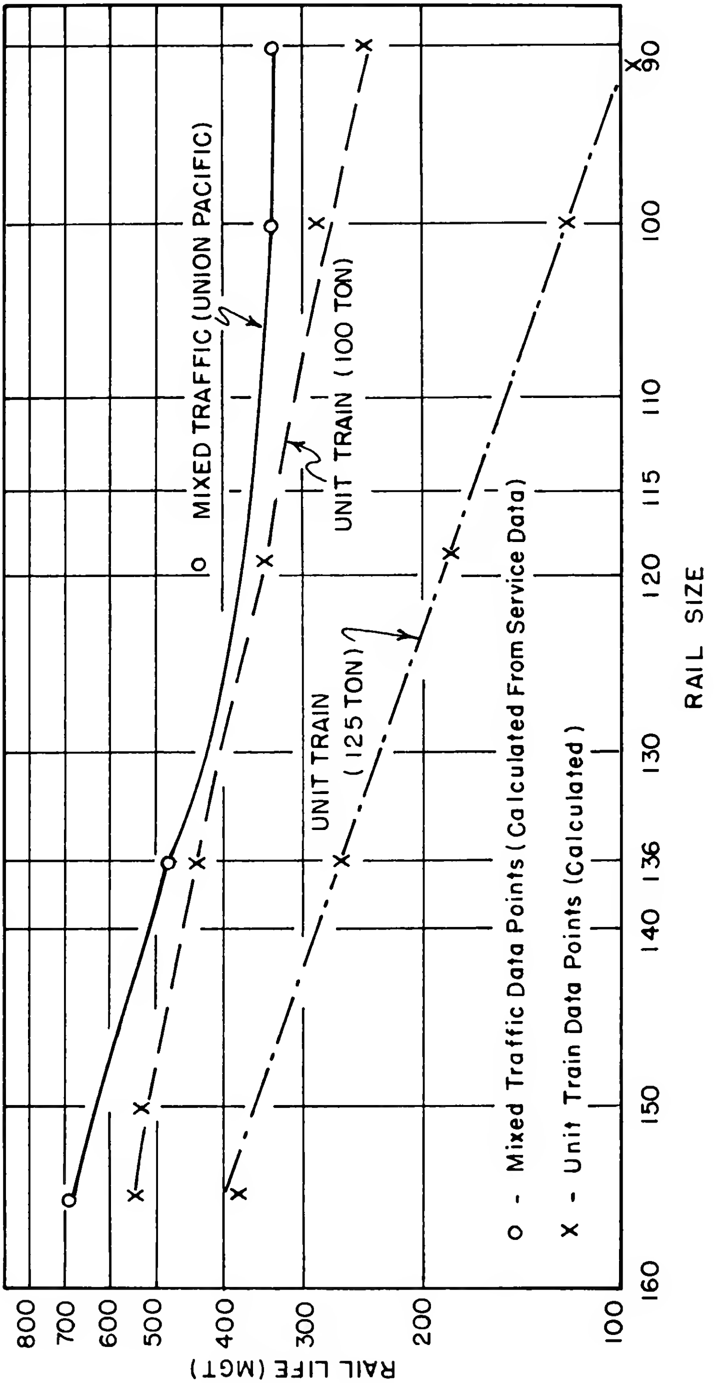

FIGURE 6. EFFECT OF INCREASING TRAFFIC LOADINGS

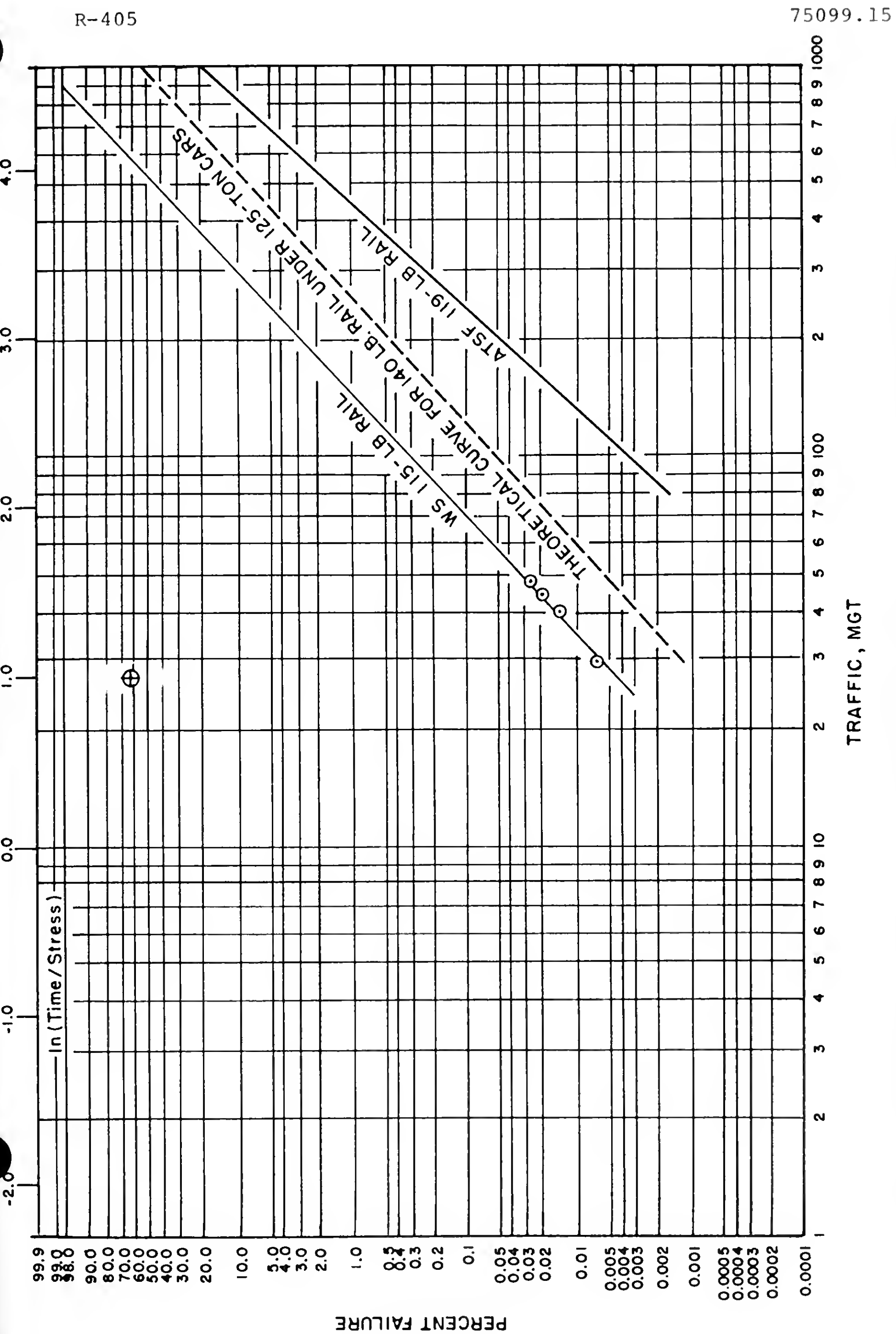

FIGURE 7. PERCENTAGE OF RAIL FAILURE VERSUS TRAFFIC (in MGT) FOR THE WAYNESBURG SOUTHERN — TEN MILE RUN BRANCH OF THE MONONGAHELA RAILROAD AND ONE SANTA FE RAILROAD TEST SECTION.

The use of high-strength rail would significantly decrease the amount of wear, since improvements in wear lives of from three to ten times that of standard rail have been observed under normal traffic conditions. The use of high-strength rail under 315,000 pound cars may not show the dramatic improvement that has been previously discussed, but there appears to be the potential for increasing the rail life in tangent track by a factor of approximately 1.5. In addition, it may be feasible to use high-strength rail in all tracks carrying 125-ton car rather than only in curves of 3 or 4 degrees (582 or 437 m radius), which is currently-accepted railway engineering practice.

The use of high-strength steel in a heavier rail section would probably double the rail life under 125-ton car service conditions.

It would also appear that the use of high-strength rail would substantially reduce the amount of plastic flow.

With respect to rail defects, a larger rail section would reduce the number of defects, by lowering the stress levels. Because the section modulus and moment of inertia of 115 pound rail are 72 and 68 percent, respectively, of the corresponding values of a 140 pound rail, it follows that, on an average, the bending stresses in the 115 pound rail, as determined from beam-on-elastic-foundation theory, should be approximately 116.3 percent of the corresponding 140 pound rail values. The ratio of the defect life has been shown [11] to be related to the fourth power of the following ratio:

$$\frac{\text{Defect Life}_{140}}{\text{Defect Life}_{115}} = [1.163]^4 = 1.67 \qquad \text{Equation 3}$$

The dashed line in Figure 7 gives the theoretical defect probability *vs* tonnage for 125-ton cars, when 140 pound rail is used in place of 115 pound rail.

Unfortunately, there exists a lack of defect data for high-strength rail. However, the number of cycles to failure is a function of the endurance limit of the rail steel, and the ratio of the endurance limit of high-strength steel to standard steel is approximately 1.45. Experience on the railway systems in the Soviet Union has shown the defect occurrence rate to be extended by a factor of 1.5 [14]. Since Soviet rail metallurgies are virtually the same as those in the U. S., the above estimates should be valid for use.

Therefore, the use of a high-strength, heavy-section rail would probably reduce the incidence of rail defects from 125-ton cars to a level comparable with that of general freight traffic on standard rail of an equivalent section size.

ACKNOWLEDGEMENT

The author would like to thank various members of the Industrial Engineering Department of the Pittsburgh and Lake Erie Railroad, under the direction of Mr. Henry Farnsworth, for providing the raw data on rail wear and rail defects which were used in the preparation of this report.

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Track Related Performance Guidelines and Economic Analysis of High-Capacity Covered Hopper Car Designs

by M. B. Hargrove*

Ladies and Gentlemen, I am pleased to have this opportunity to discuss the economic benefits associated with the use of high performance/high cube covered hopper cars in the transportation of bulk commodities. Keith Hawthorne and Brad Johnstone have already defined what the car must do in terms of dynamic performance; I hope to shed some light on how much it is worth. Ultimately, the acceptance and wide-spread use of a dynamically improved car will be determined by the economic benefits from using such cars in providing transportation service in the North American environment.

Before presenting the body of the analysis, I would like to acknowledge the assistance of an Ad Hoc Subcommittee.[1] Without their advice, experience, and technical expertise, this analysis would not have been possible. In addition, members of the Canadian Institute for Guided Ground Transport (CIGGT) staff and the AAR Technical Center staff provided technical support in exercising the models used in the analysis.

Today's presentation is intended to serve two objectives.

First, to describe an economic evaluation methodology to evaluate the net economic benefits resulting from the use of dynamically improved equipment in providing transportation service.

Second, to provide an evaluation of the system economics of a conservatively designed, high performance car, which minimally meets the specified performance goals, in a limited range of operating scenarios. It is not intended to provide specific economic benefit estimates for all responsive designs in all operating environments.

The major emphasis is on the comparison of a high performance/high cube covered hopper car to the current, conventional 100-ton covered hopper car. A limited comparison with the lighter 80-ton car is presented to indicate the relative economics of the three alternatives in the areas of rail, fuel, accident, and car maintenance. This analysis does not consider line and yard congestion costs, switching costs, or other route-specific costs which can be expected to vary with changes in car capacity; therefore, it is not intended to be a comprehensive analysis of the choice between 80-ton and 100-ton equipment in a specific service.

Components of the System Economics

The major components of system economics of utilizing freight cars in providing rail transportation service are shown in Figure 1. For the economic analysis of the high performance/high cube covered hopper, the best available approach to modelling each of these components has been identified, and benefit estimates developed on a component-by-component basis. These benefits are aggregated to a net annual benefit associated with the provision of 2.5 million ton-miles of service per car per year (25,000 miles carrying 100 net tons with an equivalent empty back-haul). The incremental capital investment justified by these benefits is then computed.

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*Research & Test Dept., Association of American Railroads.

[1]Ad Hoc Subcommittee includes R. E. Ahlf, ICG; R. F. Beck, EJE; T. S. Guins, AAR; B. Johnstone, AAR; W. R. Martin, SOU; and M. D. Roney, CIGGT; with technical support provided by A. M. Birk, CIGGT; R. W. Lake, CIGGT; W. S. McEwan, AAR; and A. M. Zarembski, AAR.

DYNAMICALLY STABLE, BULK COMMODITY CAR

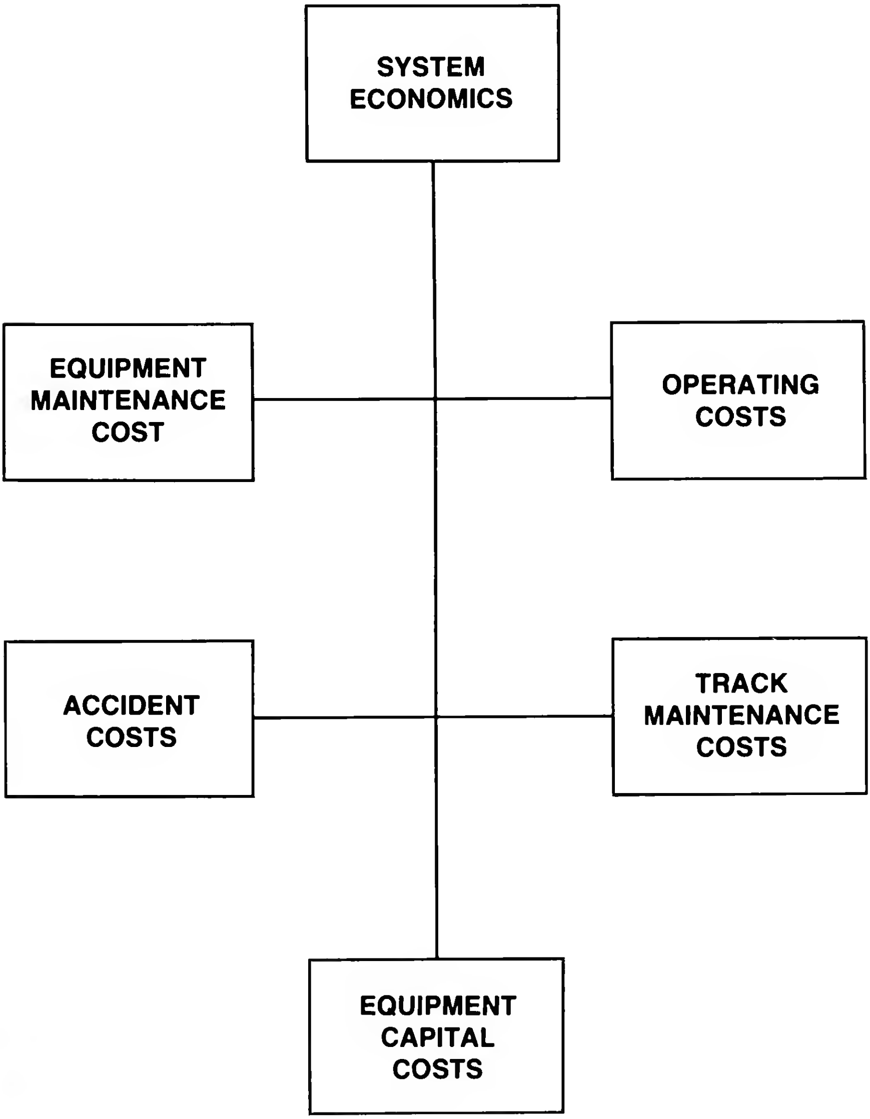

Figure 1

An overview of the approach used in modelling each of the major benefit areas, the assumptions made in the analysis, the estimated benefits under the alternative operating scenarios, and an evaluation of the sensitivity of the benefit analysis to variation in key assumptions to be discussed. The technical discussions of the modelling logic, data sources, and model calibrations will be issued in a three-volume set of reports to be made available through TTD, Task 1.

Alternative Cars Evaluated

Covered hoppers of three different general specifications are evaluated in this study. The high performance/high cube covered hopper of 100-ton capacity is evaluated against a base case of a 100-ton capacity covered hopper of current design. A lighter car alternative represented by an 80-ton covered hopper having current design features is also evaluated against the 100-ton base case and alternative high performance 100-ton car specification in terms of rail, fuel, accident, and car maintenance costs.

The base case 100-ton covered hopper is assumed to have 263,000-pound gross weight with 63,000-pound tare weight with two, two-axle conventional three-piece trucks with 36-inch wheels resulting in 33 kip static wheel load. The conventional light car alternative is assumed to have a gross weight of 220,000 pounds with a tare weight of 60,000 pounds resulting in a load capacity of 80 tons. The light car has two, two-axle conventional three-piece trucks with 33-inch wheels resulting in a 28 kip static wheel load. Roller bearings are assumed for all cars. The specifications of the dynamic inputs for these conventional cars were developed from an instrumented wheel set test conducted at the FAST loop.

The high performance/high cube covered hopper is assumed to have a 263,000-pound gross weight with a 63,000-pound tare weight and 100-ton load capacity. It is assumed to have two, two-axle radial trucks, or 36-inch wheels with an improved suspension system resulting in a 33 kip static load and a 50 percent reduction in lateral and a 25 percent reduction in vertical dynamic loads. This specification is a conservative representation of the high performance/high cube covered hopper car minimally meeting the performance goals in the performance specifications. More radical approaches involving lower tare through the use of alternative materials, higher net to tare or lower static loads through the use of span bolsters, three-axle trucks, or larger wheels are not assumed in this analysis, although their potential areas of benefits and costs are noted. The development of innovative car designs should not be discouraged by the conservative design option considered by this analysis; however, a detailed economic analysis of each design using the methods outlined in this report must be undertaken if such design proposals are submitted.

Operating Environments Considered

The economic benefits associated with improved car design depend on the service utilization rates and on the nature of service provided. This report presents results for a car providing 2.5 million ton-miles of service per year, or 25,000 loaded car miles with a 100-ton load. An equal amount of empty car mileage (backhaul) is assumed in each analysis. This level of utilization of 50,000 car miles per year is somewhat greater than the present covered hopper fleet average, but substantially below the utilization rate of equipment in unit train operations. This level of utilization is representative of a well-managed, grain service pool.

The important dimensions of the type of service provided include the typical speed of movement, the degree of curvature and gradient of the route, and the quality of the track. In the analyses presented, the typical operating speed is assumed to be 45 miles per hour which is representative of moderate speed, mainline freight service on many raods in grain territories. The dynamic augments measured at FAST are representative of operation over track maintained to high surface and alignment standards far exceeding the norm for track seeing

45 mph service; therefore, the dynamic forces in operating over typically maintained track can reasonably be assumed to be higher and the economic benefits to dynamic force reductions to be greater. Unfortunately, the data required to calibrate the models over the full spectrum of speeds of operation and standards of surface and alignment are not presently available; however, the conservative bias introduced into the economic analysis through the use of the FAST dynamic data should be recognized since the actual dynamic augments in operation over normally maintained track would be greater.

Three alternate route specifications having variations of gradient and curvature are evaluated including the following:

—A tangent route consisting of 100 percent less than ½ degree curvature in level territory,
—A moderate route consisting of 85 percent less than ½ degree, 7.5 percent of ½ degree to 2 degree, 5 percent of 2 degree to 5 degree, and 2.5 percent of greater than 5 degree curvature in low gradient territory, and
—A severe route consisting of 61.5 percent less than ½ degree, 5.4 percent of ½ degree to 2 degree, 21.1 percent of 2 degree to 5 degree, and 12.0 percent of greater than 5 degree curvature with moderate gradients.

These routes were selected as representative of the operating environment of most bulk commodity movements in the U.S. rail system. The tangent and severe routes represent actual track segments of approximately 100 miles in length, while the moderate route, based on the opinions of industry people knowledgeable in the area of track, is a typical North American route.

ECONOMIC EVALUATION OF ALTERNATIVE CARS

Evaluation of Track Maintenance Impacts

For an evaluation of the impact of alternative cars specifications on track maintenance costs, the engineering model developed by the Canadian Institute for Guided Ground Transport (CIGGT) was identified as the appropriate tool. This model recognizes the parameters affected by the car design including the lateral and vertical components of the dynamic and static forces, and the curving behavior of tracks as key inputs to the component life and maintenance cycle modelling process. The CIGGT model has had previous success in predicting variations in rail wear associated with alternative route and traffic composition specifications. Substantial effort was expended to increase the ability of the existing CIGGT models to deal with the non-rail portions of track maintenance cost and to incorporate the fatigue limits of rail life into the rail life cycle estimation.[1] The CIGGT model and the additional development performed in conjunction with this project are presented in detail in a forthcoming report.

The logic of the CIGGT rail wear model is to start from the point of contact between wheel and rail; define both the static and dynamic forces present including the vertical, lateral, longitudinal, and creep components; and to model the effects of these forces as functions of the physical properties of the materials present. The variations in forces are traced through the rail to the tie and fastener system to the ballast and subgrade. Separate models to relate maintenance cycles to the forces induced by the passage of equipment are developed for rails, ties and fasteners, and ballast and subgrade.

[1]The rail fatigue model used was reported in A. M. Zarembski and R. A. Abbott, "Fatigue Analysis of Rail Subject to Traffic and Temperature Loading," Heavy Haul Railways Conference Proceedings, Perth, Western Australia, September 1978.

To allow a meaningful comparison among the alternative car specifications without conducting an excessive number of evaluations, certain parameters which affect component lives or maintenance cycles and the economic consequences of these variations were held constant in this study.

—A traffic density of 15 MGT per year involving 50 percent fully loaded movements and 50 percent empty returns with all traffic in cars of the specified design,
—Track constructed of 136-pound per yard continuous welded rail (CWR) of standard carbon metallurgy on wood ties; and
—Track support characterized by a track modulus[1] of 2,000 pounds per inch per inch representing moderate ballast and subgrade conditions.

The choice of 136-pound per yard CWR as a base was made due to the increasing use of heavier, welded rail in the U.S. rail system. The environment that will exist in the period 1985 and into the first decade of the twenty-first century is relevant to the economic value of this car design, thus increasing use of welded rail is assumed. Some analysis of the economics of operation over 115-pound per yard CWR was conducted, but at a traffic density of 15 MGT per year, the economic benefits associated with the alternative car specification were quite similar.

The equivalent annual benefits (in terms of track maintenance cost savings) of the operation of a high performance and an 80-ton conventional covered hopper are presented in Table 1 broken down by degree of curvature of track. Separate benefits are shown for the three major component areas along with the estimated component lives for rails and ties. These equivalent annual benefits (EQAB) represent maintenance cost savings per mile of track of the specified curvature class. These costs must be combined with the route curvature specification and the equipment specifications to calculate the track savings per car; however, at this point several interesting points can be observed, including:

First, in the less than 2 degree curvature areas, the dominant rail failure mode, fatigue, is primarily responsive to reductions in static vertical loads and then to reductions in vertical dynamic augments;

Second, in the greater than 2 degree curvature areas, the dominant rail failure mode is expected to be wear; and all the maintenance costs become sensitive to the lateral steady-state curving forces and reductions in lateral dynamic augments producing greater relative benefits to the high performance car; and

Third, in the less than 5 degree curvature areas, the importance of the "other track maintenance costs" and the sensitivity to vertical loads make the 80-ton conventional car the equipment choice which produces the lowest track maintenance expenditure per mile per year.

To convert the benefits per mile of track to annual track maintenance benefit per car per year, one must first calculate the number of car miles of operation required to produce 15 MGT on a mile of given curvature track. For example, for a 100-ton car (263,000 pounds loaded, 63,000 pounds empty) the average movement is 81.5 gross tons (50 percent loaded, 50 percent empty). Thus, the accumulation of 15 MGT requires 184,049 car movements per year, thus the annual track maintenance benefits can be distributed per car mile per year by dividing the equivalent annual benefit per mile of track by the required miles of operation. Given the specified annual mileages per car of 50,000, the benefit per car mile can be converted to a benefit per car per year when the car is operated on a given curvature of track by multiplying by the annual car mileage. For example, on tangent track, the equivalent annual benefit in the operation of high performance, 100-ton cars would be $583 per year per

[1]Track modulus is the load on one rail to deflect one tie one inch divided by the tie spacing in inches.

TABLE 1

ESTIMATED ANNUAL TRACK MAINTENANCE BENEFITS (COST SAVINGS) PER MILE OF TRACK BY CLASSES OF DEGREE OF CURVATURE

CURVATURE—CLASS LIMITS	TANGENT	0-2 DEG.	2-5 DEG.	5-8 DEG.
Rail Life (in years)				
100-Ton (Base Car)	347.9	347.9	193.6	128.1
100-Ton—High Performance	399.2	399.2	399.2	375.8
80-Ton—Conventional	642.2	642.2	267.5	171.9
Rail—Advantage Over 100-T Base Car ($ per year per mile)				
100-Ton—High Performance	$ 231	$ 231	$1,666	$3,230
80-Ton—Conventional	$ 825	$ 825	$ 894	$1,246
Ties—Life (in years)				
100-Ton (Base Car)	24.8	22.7	16.8	15.9
100-Ton—High Performance	25.0	24.8	20.1	19.3
80-Ton—Conventional	26.5	24.8	20.1	19.3
Ties—Advantage Over 100-T Base Car ($ per year per mile)				
100-Ton—High Performance	$ 13	$ 147	$ 389	$ 437
80-Ton—Conventional	$ 112	$ 147	$ 386	$ 437
Other Track Maintenance— Advantage Over 100-Ton Base Car ($ per year per mile)				
100-Ton—High Performance	$ 339	$1,071	$1,576	$1,656
80-Ton—Conventional	$1,493	$1,845	$2,564	$2,683
Total Track Maintenance— Advantage Over 100-Ton Base Car ($ per year per mile)				
100-Ton—High Performance	$ 583	$1,449	$3,631	$5,293
80-Ton—Conventional	$2,430	$2,817	$3,884	$4,366

mile of track × 50,000/184,049 = $158.38 per car. Table 2 shows the track benefits per car in each of the curvature classes.

The calculation of equivalent annual benefit per car for 80-ton cars involves an additional transformation to adjust for the fact that additional car miles are necessary to produce the 15 MGT per year of traffic. The average 80-ton car movement is 70 gross tons (220,000 pounds loaded, 60,000 pounds empty) thus, 214,285 car miles are required to produce the track benefits. At 50,000 miles per car per year, the equivalent annual track maintenance benefit per year is given by the annual track mile benefit per 15 MGT divided by 214,285 car miles per year times 50,000 miles per car per year. For example, in tangent track, the equivalent annual benefit for 80-ton car is $2,470 × 50,000/214,285 = $576.33. These results for the 80-ton conventional car are also shown in Table 2.

These benefits per car per year in given curvature classes are converted to route-specific track maintenance cost savings by weighting the savings in each curvature class by the percentage of the route in the specified curvature classes. The route-specific track maintenance benefits per year per car for the three trail routes are given in Table 3. For example the moderate curvature route's track maintenance benefits are calculated as follows: $158 × .85 + $394 × .075 + $986 × .05 + 1,438 × .025 or $250 for the high performance 100-ton covered hopper car.

In summary, there appear to be substantial reductions in annual track maintenance expenditures associated with the operation of high performance/high cube covered hopper cars. The reductions are not as large as can be obtained by using 80-ton conventional cars, especially in low curvature track, due to the importance of vertical static loads in the formation of fatigue defects leading to rail removal.

Accident and Lading Damage Costs

A second major area of potential benefit in the design of a high performance/high cube covered hopper is in reducing the costs associated with accidents, particularly derailments, and with the associated lading loss. Because of the nature of the commodities typically carried by high cube, covered hopper cars, no benefit is estimated for the improved ride quality of the high performance car in reducing lading damage in transit.

The current state-of-the-art in the analysis of the relationship between the static and dynamic behavior of freight cars and the accident rates in specified operating environments has not progressed to the point where precise estimates of the economic value of given decrements in lateral and vertical forces can be made. A current study of the Safety Division of the Research and Test Department, AAR has examined the relationships between freight car type and size and their involvement in train accidents. This provides the basis for evaluating the expected benefits associated with improving the behavior of covered hopper cars, and those parts of the study relevant to the analysis of covered hopper cars is included in the forthcoming report.

Overview of Analytic Procedures

The AAR study uses the FRA accident report file for the years, 1975-1978, along with descriptive data from the Universal Machine Language Equipment Register (UMLER) and the 1976 one percent Waybill sample to establish the exposure in terms of loaded car miles and the accident frequencies by car type. This allows the evaluation of train accident probabilities per million loaded car miles by car type. The product of the mean accident costs (appropriately inflated to include the lading loss and damage, train delay and congestion costs, and environmental and other cleanup costs) and the accident probability for operating 25,000 loaded miles per year in the given car type gives the expected annual accident costs in the specified car type. The differential in the expected annual accident costs among different car types is the estimated benefit to the less accident-prone car.

The accident record of the high performance/cube covered hopper car cannot be determined from historic data, but given the dynamic performance goals, the assumption that it

TABLE 2

ESTIMATED ANNUAL TRACK MAINTENANCE BENEFITS (COST SAVINGS) PER CAR BY CLASS OF DEGREE OF CURVATURE

CURVATURE—CLASS LIMITS	TANGENT	0-2 DEG.	2-5 DEG.	5-8 DEG.
Equivalent Annual Benefit in Track Maintenance Over 100-Ton Base Car ($ per car per year)				
100-Ton—High Performance	158	394	986	1,438
80-Ton—Conventional	576	657	906	1,019

TABLE 3

ESTIMATED ANNUAL TRACK MAINTENANCE BENEFITS (COST SAVINGS) PER CAR BY SPECIFIC ROUTE

TYPE OF ROUTE	STRAIGHT	MODERATE CURVATURE	SEVERE CURVATURE
Equivalent Annual Benefit in Track Maintenance Over 100-Ton Base Car ($ per car per year)			
100-Ton—High Performance	158	250	499
80-Ton—Conventional	576	610	703

TABLE 4

ESTIMATED ACCIDENT RATES FOR THREE COVERED HOPPER CAR DESIGNS

CAR DESIGN	ACCIDENTS 1975-1978	LOADED CAR MILES 1975-1975-1978 (IN MILLIONS)	ACCIDENTS PER 25,000 LOADED CAR MILES
CONVENTIONAL—100-TON	670	2301.16	.00727
CONVENTIONAL—80-TON	251	437.81	.01433*
HIGH PERFORMANCE—100-TON			.00444**

*Primarily cars built before 1968 without supplemental snubbing.
**Estimated on the basis of the average for entire fleet.

should be no worse that the average car in the existing rail fleet appears to be a conservative assumption which is used in this evaluation.

One additional factor that must be considered in this analysis, is the radical change in the derailment rate of conventional 100-ton covered hoppers following the requirement of supplemental snubbing devices for cars constructed after 1967. The inclusion of pre-1968 cars in the data base for establishing the accident rates for conventional 100-ton cars would produce an estimate which is not representative of the current design; therefore, these pre-1968 cars have been excluded. The lighter covered hopper cars, almost all built before 1968, are not required to have supplemental snubbing, and their accident rates are quite high; however, for a meaningful comparison with cars of current design, an accident rate per loaded car-mile identical to the 100-ton, supplementally snubbed car is assumed.

Summary of Findings

The number of accidents, estimated numbers of car miles, and estimated probability of accident per 25,000 miles of loaded operation are shown in Table 4.

The resulting expected accident costs computed as follows:

100-ton—Conventional

.00727 × \$111,378[1] = \$811 per car per year.

80-ton—Conventional

.00727[2] × \$111,378 × 1.2[3] = \$973 per car per year.

100-ton—High Performance

.00444 × \$111,378 = \$495 per car per year.

The accident benefits for the high performance car is estimated to be \$316 per year when compared to the conventional 100-ton cars. The current 80-ton hopper is revealed to be a high accident cost car resulting in a loss (negative benefit) of \$162 per car per year.

Based on this analysis, the high performance/high cube car is economically beneficial if the accident rate can be lowered to the level of the average car in the fleet; a rather conservative performance goal. The current 80-ton car is shown not to be an attractive alternative; however, it appears that improvements in the design of lighter cars should be attempted if this alternative is seriously considered in the future, and the accident rate used in this analysis assumes such improvement.

Car Maintenance Costs

The analysis of the potential benefits resulting from the development of the high performance/high cube 100-ton car was analyzed using the Car Maintenance Cost Data Base (CMC) developed by the Research and Test Department of the AAR. This data base contains the detailed repairs both on-line and off-line along with on-line and off-line mileages for approximately 200,000 cars in the fleets of four Class I railroads. This data base was analyzed for differences between 80-ton and 100-ton conventional cars in maintenance costs, and the

[1]Represents the FRA mean accident costs for line-haul accidents in 1978 multiplied by 2 to reflect the lading loss, train delay, and environmental and other cleanup costs not included in the FRA data inflated at 10 percent to 1980 dollars.

[2]The accident rate for current design 100-ton cars is used for estimating the risks associated with modern design 80-ton equipment including supplemental snubbing.

[3]The 1.2 reflects the increased number of car miles required of 80-ton cars to provide the equivalent transportation service.

individual repair items were analyzed for categories which would be anticipated to be improved by the lower dynamic loadings of the high performance car. A detailed description of the CMC data base and the analysis performed are included in the forthcoming report.

The comparison of the maintenance costs of 100-ton versus 80-ton cars is complicated by the differences in the ages of the 80-ton and 100-ton fleet, and the systematic variation in maintenance costs over the typical life cycle of a freight car. The analysis was performed by overlaying the annual maintenance expense curves as a function of car age, assuming that both the 100-ton and 80-ton car will have the same shape with age, but that their levels may differ.

The results of this analysis are that there appear to be no measurable difference per car mile between conventional 100-ton and conventional 80-ton covered hoppers; however, because of the greater number of 80-ton car miles required, there is an increase of $497 per year per equivalent car carrying 2.5 million net ton-miles per year for the use of 80-ton cars.

For the high performance/high cube 100-ton car, potential maintenance savings in the area of wheel wear, primarily in the flange with a small amount of rim wear reduction, have been identified. Although no specific modelling has been done of the relationship between wheel flange and rim wear as a function of lateral and vertical dynamic forces, an approximation that the wheel wear differentials are proportional to the rail wear differentials was used to estimate equivalent annual benefits through improved dynamic behavior to be $8 in the straight track environment, $48 in the moderate curvature environment, and $144 in the severe curvature environment per car compared to the current 100-ton covered hopper.

Fuel Cost Savings

The use of high performance/high cube cars will produce fuel savings due to the improved curving behavior resulting in lowered train resistance in curves. The 80-ton car has an increased fuel requirement due to the increased number of car-miles required to provide the equivalent 2.5 million net ton-miles of service per year and the increased tare weight involved. The Train Operations Simulator (TOS) as developed by the Track-Train Dynamics Program was the model identified as allowing the evaluation of the fuel requirements for operations over specified routes.

TOS was used to evaluate the operation of an 80 car train of 100-ton conventional covered hoppers, an 80 car train of high performance/high cube covered hoppers, and a train of 100 cars of 80-ton conventional covered hoppers over approximately 100-mile routes of straight, level track and severe curvature, moderate grade track. Runs were made in both directions loaded and empty, the fuel consumption calculated and the differences per mile were extrapolated to the equivalents of 2.5 million net ton-miles per year. At a fuel cost of $.85 per gallon in 1980, the resulting savings for high performance–100-ton cars is estimated to be $138 per car per year in severe curvature routes, and none in tangent territory. The moderate route is extrapolated to be $25 per car per year. The 80-ton car is evaluated as costing an extra $265, $308, and $327 per year per equivalent car load capacity in straight, moderate curvature, and severe curvature environments, respectively. The details of the TOS model and the analysis performed are included in the forthcoming report.

Economic Evaluation

The component benefits are brought together, and an evaluation of the net economic advantage in terms of equivalent annual benefits are expressed in Tables 5, 6, and 7. The variation in operating environments is shown to have a substantial effect on the net economic advantage of the 100-ton high performance/high cube car, with the severe environment producing the maximum advantage. The 80-ton conventional design is shown to produce greater savings in track maintenance, but to have higher annual costs in the other areas considered.

Also, 80-ton cars can be anticipated to have higher switching costs and line-haul and yard congestion costs which may be significant in certain services.

The warranted investment values in Tables 5, 6, and 7 represent the incremental amounts that can be invested in a car of 100-ton capacity and meet a 20 percent corporate hurdle rate for a current dollar internal rate of return (IRR).[1]

For example, in Table 5 the improved 100-ton car is shown to have a net annual after-tax savings of $570 per car which would produce a 20 percent internal rate of return on an investment of $8,504. The 80-ton car is shown to produce a net loss of $146 per annual operation of the equivalent load capacity of one 100-ton car, which justifies the investment of $2,177 less per equivalent load capacity of one 100-ton car or ($2,177 × (80/100)) $1,742 less per 80-ton car considering only track, fuel, car maintenance, and accident costs; not switching and congestion costs which may be significant for the 80-ton alternative.

If any amount less than this incremental amount is spent on the car, the IRR would be above 20 percent. At this point the incremental cost of the improved car is not known, so that an exact IRR or pay-back period cannot be calculated.

Conclusions

At this point, the importance of the specific evaluations presented here should be considered as the following:

—First, demonstrating the methodology to be employed in evaluating specific design proposals,

—Second, showing that no single value can represent the value of improvements in car design; that the value is highly dependent on the operating scenerio, and

—Third, there would be a major economic benefit to an innovative design which could lower the static loading on the track while maintaining the operating economics of the 100-ton car.

Hopefully, this study along with the performance guidelines will help stimulate the development of a technically and economically desirable high performance/high cube covered hopper car.

[1]Assuming a 10 percent long-term inflation rate and 9 percent risk premium and time discount factor.

TABLE 5
COST COMPARISONS WITH CURRENT 100-TON CAR SEVERE CURVATURE–MODERATE GRADE ENVIRONMENT

COST CATEGORY	IMPROVED 100-TON CAR ANNUAL SAVINGS[1]	IMPROVED 100-TON CAR WARRANTED INVESTMENT[2]	CURRENT 80-TON CAR ANNUAL SAVINGS[1]	CURRENT 80-TON CAR WARRANTED INVESTMENT[2]
TRACK	$ 259	$3,864	$ 366	$5,461
ACCIDENT	$ 164	$2,447	($ 84)[3]	($1,253)
CAR MAINTENANCE	$ 75	$1,119	($ 258)	($3,849)
OPERATING (FUEL)	$ 72	$1,074	($ 170)	($2,536)
TOTAL	$ 570	$8,504	($ 146)	($2,177)

TABLE 6
COST COMPARISONS WITH CURRENT 100-TON CAR MODERATE CURVATURE—LEVEL ENVIRONMENT

COST CATEGORY	IMPROVED 100-TON CAR ANNUAL SAVINGS[1]	IMPROVED 100-TON CAR WARRANTED INVESTMENT[2]	CURRENT 80-TON CAR ANNUAL SAVINGS[1]	CURRENT 80-TON CAR WARRANTED INVESTMENT[2]
TRACK	$ 130	$1,940	$ 317	$4,730
ACCIDENT	$ 164	$2,447	($ 84)[3]	($1,253)
CAR MAINTENANCE	$ 25	$ 373	($ 258)	($3,849)
OPERATING (FUEL)	$ 13	$ 194	($ 160)	($2,387)
TOTAL	$ 332	$4,954	($ 185)	($2,759)

TABLE 7
COST COMPARISONS WITH CURRENT 100-TON CAR STRAIGHT, LEVEL TRACK ENVIRONMENT

COST CATEGORY	IMPROVED 100-TON CAR ANNUAL SAVINGS[1]	IMPROVED 100-TON CAR WARRANTED INVESTMENT[2]	CURRENT 80-TON CAR ANNUAL SAVINGS[1]	CURRENT 80-TON CAR WARRANTED INVESTMENT[2]
TRACK	$ 82	$1,223	$3,009	$4,476
ACCIDENT	$ 164	$2,447	($ 84)[3]	($1,253)
CAR MAINTENANCE	$ 4	$ 60	($ 258)	($3,849)
OPERATING (FUEL)	$ 0	$ 0	($ 138)	($2,059)
TOTAL	$ 250	$3,730	($ 180)	($2,685)

[1]In after-tax dollars assuming a 48 percent maginal tax rate.

[2]In before-tax dollars assuming a 9 percent discount rate (inflation free), a 10 percent inflation rate, a 20 percent hurdle rate, a 48 percent marginal tax rate, and a 10 percent investment tax credit.

[3]( ) Indicates negative savings (higher costs).

MEMOIRS

H. B. CHRISTIANSON
1892—1979

Hilmar Barman Christianson, a Director of AREA 1953-55, died December 19, 1979, in California at age 87. A Minneapolis native, he was graduated in engineering from the University of Minnesota in 1915, worked a year for the Northern Pacific and a year for Soo Line before enlisting for service in World War I. Then Chris began a long career with Milwaukee Road, serving as division engineer from 1925 in Iowa, Montana, Wisconsin and Illinois, and assistant chief engineer from 1950. An important special assignment was to prepare the Milwaukee-St. Paul route for the high-speed Hiawatha trains introduced in 1936.

In 1942 Chris entered World War II as Lt. Colonel. After military railway training in Louisiana he served 33 months in the southwest Pacific, first assessing Australian railway defense capability, then participating in progressive assaults and landings on New Guinea and Luzon.

After retiring from Milwaukee Road in 1957, Chris joined Tippets-Abbett-McCarthy-Stratton as chief railroad engineer locating an ore railway in French Equatorial Africa, now Congo. Leaving the tropics for the sub-artic, he next served as deputy project engineer constructing the 194-mile Quebec Cartier Mining Railroad in Canada. Later he served TAMS in New York City as design consultant on the Mt. Newman Railway in Western Australia.

Chris was a life member of ASCE, past director of American Railway B&B Association, past president of M. of W. Club of Chicago, and registered professional engineer in IL. A member of AREA since 1928, he served highway committee #9 1946-48, yards and terminals committee #14 in 1948, track committee #5 1949-59, and continuous welded rail committee 1951-59 as its first chairman.

Marvis Zink Christianson, a native of Texarkana AR and his wife for 59 years, resides at 5718 Ravenspur, Rancho Palos Verdes CA 90274, Others surviving are a sister Gundrun Grassel of Denver CO, a son H. B. Christianson, Jr. of Baltimore MD, daughters Margaret Mauermann of San Diego CA and Bonnie DeMos of Palos Verdes Peninsula CA, ten grandchildren and three great-grandchildren.

H. B. Christianson, Jr.

FRED AUGUST HESS
1901—1980

Fred August Hess, retired Maintenance Engineer of the Indiana Harbor Belt Railroad, died at Rolling Meadows, Illinois, on February 17, 1980. Married to Emily Kirchner on June 7, 1924, who survives him, he is also survived by two daughters, both married, now Jean E. Rust and Nanette Feld. Mr. Hess was born in Chicago on June 28, 1901, and after attending local schools, pursued Civil Engineering studies at Armour Institute of Technology (Chicago) graduating with the class of 1923. He immediately began his railroad career with the New York Central Railroad as draftsman, advancing through the years to several positions in the engineering echelon to the position of Maintenance Engineer (IHB) which he held at the time of his retirement in 1964.

Mr. Hess became a member of the American Railway Engineering Association in 1944, was appointed to its Committee 14—Yards and Terminals—in 1946 and attained Life Membership in 1964. He was an active member of the Committee until his death, making invaluable contributions to the progress of its works, chaired numerous subcommittee activities, and served as Committee's Vice Chairman (1950-53) and Chairman (1953-56). He was elected a Member Emeritus in March 1967. His contributions to the railroad industry and A.R.E.A., his technical approach to problems and his interest in the progress of the engineering art were characterized by his keen analytic mind. These factors with his pleasant fellowship will be missed by his associates and friends.

Committee 14

KENNETH E. HORNUNG
1911-1980

Kenneth E. Hornung, retired, Chicago, Milwaukee, St. Paul and Pacific Railroad, died in Northwest Memorial Hospital on April 3, 1980, after a brief illness.

Mr. Hornung was born June 2, 1911 at Dubuque, Iowa and attended Iowa State University for three years, then transferred to the University of Minnesota where he graduated receiving a BA in Architecture.

In 1937 Ken joined the Chicago, Milwaukee, St. Paul and Pacific Railroad as Architectural Draftsman. He became Chief Draftsman in 1945 and was elevated to Architect in 1947, the position he retained until 1973 when he was promoted to Assistant Chief Engineer Structures, the position he held until his retirement in June, 1976.

Mr. Hornung was elected to membership in the American Railway Engineering Association July 19, 1948 and became a member of Committee #6 in 1949. He served as Vice Chairman of this Committee in 1959 and Chairman in 1961, and continued his membership through 1979.

For 39 continuous years, Ken worked for the Milwaukee Road with headquarters in the Chicago Union Station. He has resided in Highland Park, Illinois for many years where he was very active in the Highland Park Presbyterian Church.

Mr. Hornung is survived by his wife, Toddy, their son Kenneth, Jr. of Los Gatos, California, and one grandchild, Brian.

Committee 6

AUDITORS REPORT

February 13, 1980

American Railway Engineering Association
2000 L Street N.W.
Washington, D.C. 20036

We have examined the balance sheet of American Railway Engineering Association as of December 31, 1979 and the related statement of income and expenses for the year then ended.Our examination was made in accordance with generally accepted auditing standards and accordingly included such tests of the accounting records and other auditing procedures as we considered necessary in the circumstances.

In our opinion, the accompany balance sheet as of December 31, 1979, fairly presents the financial condition of American Railway Engineering Association in conformity with generally accepted accounting principles.

O'Neill & Gaspardo
Certified Public Accountants

AMERICAN RAILWAY ENGINEERING ASSOCIATION
BALANCE SHEET
DECEMBER 31, 1979

ASSETS		
Current Assets		
Cash in National Bank of Washinton Checking	$ 2,428.10	
Cash in National Bank of Washington Savings	55,453.00	
Cash in Northern Trust Co. Chicago Savings	3,250.99	
Cash in Northern Trust Co. Chicago Certificates of Deposit	100,000.00	
Petty Cash	50.00	
Investment—U.S. Treasury Bonds	28,000.00	
Investment—U.S. A. Treasury Bonds	4,000.00	
Accounts Receivable—Advertising	205.86	
Accounts Receivable—Dues	2,850.00	
Inventories	29,410.00	
Prepaid Postage	701.83	
Total current Assets		$226,349.78
Fixed Assets		
Furniture & Fixtures		1,935.24
		$228,285.02
LIABILITIES		
Current Liabilities		
Expenses Payable		$ 10,663.35
Balance—January 1, 1979	$167,242.03	
Net Increase of Income over Expenses Year ended December 31, 1979	50,379.64	
		217,621.67
		$228,285.02

AMERICAN RAILWAY ENGINEERING ASSOCIATION COMPARATIVE STATEMENT OF INCOME AND EXPENSES

YEAR ENDING DECEMBER 31,	1979	1978
INCOME		
Dues	$ 80,126.96	$ 76,804.37
Student Affil. Fee	114.00	88.00
Publication—Bulletins	7,262.85	7,586.39
Proceedings	1,436.00	2,799.35
Manual	102,402.27	45,354.98
Trackwork Plans	14,621.71	10,518.25
Misc.	670.00	657.65
Advertising	9,193.96	9,616.57
Interest	15,119.56	5,495.07
Conference	10,872.90	19,493.35
Misc. Income	8,534.87	2,005.70
Total Income	$250,355,08	$180,419.68
EXPENSES		
Salaries	$ 51,041.76	$ 49,788.64
Payroll taxes and fringe benefits	15,089.54	12,976.41
Retirement benefits	3,152.28	3,152.28
Travel expenses	3,555.31	1,626.42
Professor expense	8,755.52	7,921.26
Printing costs—Bulletins	43,493.79	20,017.71
Manual	33,256.10	41,400.26
Newsletter	1,005.10	2,241.36
Miscel. Stationery	6,792.49	4,886.91
Rent	570.00	1,140.00
Shipping costs	9,137.08	9,386.54
Telephone	105.82	261.89
Supplies	1,286.01	518.57
Accounting & Auditing	600.00	500.00
Conference costs	7,435.11	15,083.16
Misc. and extraordinary expenses	14,699.53	2,605.98
Total expenses	$199,975.44	$167,507.41
NET INCOME FOR PERIOD	$ 50,379.64	$ 12,912.27

AMERICAN RAILWAY ENGINEERING ASSOCIATION DECEMBER 31, 1979

NOTES TO BALANCE SHEET

Federal Form 990 was prepared in conformity with this report.

Inventory values were reviewed with Mr. L. Cerny and his staff and the following valuation appeared reasonable:

Manual Binders 1180 @ $8.50	$ 10,030.00
Chapter 4600 @ 2.00	9,200.00
Portfolios 160 @ 20.00	3,200.00
Portfolios Supplements & binders	1,180.00
Manual Supplement	3,600.00
Bulletin Binders	1,000.00
Miscellaneous	1,200.00
	$ 29,410.00

Deceased Members

ASTRUE, C. J. (M '27, L'54)
Retired Assistant Chief Engineer, Southern Pacific Transportation Co., San Rafael, CA

BIRKENWALD, E. S. (M '42, L '67)
Retired Assistant Chief Engineer, Bridges, Southern Railway, Atlanta, GA

BOESSNECK, W. O. (M '30, L '59)
Retired Office Engineer, Erie Railroad, Westlake, OH

BOGLE, R. H., Jr. (A '61)
President, R. H. Bogle Co, Alexandria, VA

BUTLAND, A. N. (M '59)
Retired Chief Engineer, British Railways Board, Surrey, England

CARTER, J. N. (M '69)
Manager Roadway & Maintenance, Norfolk & Western Railway, Roanoke, VA

CHRISTIANSON, H. B. (M '28, L '57)
Retired Special Engineer, Chicago, Milwaukee, St. Paul & Pacific Railroad, Rancho Palos Verdes, CA

CLARK, M. D. (M '41, L '70)
Retired Chief Engineer, Piedmont & Northern Railway, Charlotte, N. C.

CUDWORTH, A. G. (M '51, L '77)
Retired Engineer, Denver & Rio Grande Western Railroad, Sun City, AZ

DOWNEY, W. N. (M '44, L '69)
Retired Engineer of Design, Seaboard Coast Line Railroad, Lawrenceburg, KY

EARTHMAN, W. B. (M '36, L '72)
Retired Assistant Engineer, Canadian National Railways, Quebec, Canada

GRAHAM, E.A. (M '47, L '75)
Retired Chief Engineer, Colorado & Southern Railway, Denver, CO

HAIRE, C.C. (M '23, L'52)
Retired Valuation Engineer, Illinois Central Railroad, Long Beach, CA

HAMILTON, W. H. (M '29, L '61)
Retired Chief Engineer, Montour Railroad, Edgeworth, PA

HESS, F. A. (M '44, L '70)
Retired Maintenance of Way Engineer, Indiana Harbor Belt Railroad, Bridgman, MI

HORN, S. R. (M '73)
Supervisor of Machinery & Automotive Equipment, Grand Trunk Western Railway, Durand, MI

HORNUNG, K. E. (M '48, L '77)
Retired Assistant Chief Engineer—Structures, Chicago, Milwaukee, St. Paul & Pacific Railroad, Highland Park, IL

INSANA, D. P. (M '68)
Principal Engineer—Public Improvement Projects, Conrail, Philadelphia, PA

JOHNSTON, A. V. (M '47, L '75)
Retired President, CANAC Consultants Ltd., St. Thomas, Ontario

KILCOYNE, J. F. (M '63)
Office Engineer, Atchison, Topeka & Santa Fe Railway, Chicago, IL

KUEHNER, R. E. (M '49)
Engineer Planning, Norfolk & Western Railway, Stow, OH

MCALLISTER, J. F. (M '65)
Assistant Division Engineer, Southern Pacific Transportation Co., Los Angeles, CA

MOSS, L. W. (M '26, L '59)
Retired, New York Central, Mt. Carmel, IL

PRICE, T. E. (M '44, L '69)
Retired Engineer, Maintenance of Way, Canadian Pacific Railway, Vancouver, B.C.

PRINKALNS, G. L. (M '71)
Assistant Manager Welding & Reclamation Plants, Conrail, Cornwell Heights, PA

RUST, J. A. (M '44, L '74)
Retired Chief Engineer, Design & Construction, Southern Railway, Atlanta, GA

SCHRADER, L. F. (M '58)
Assistant Division Engineer, Conrail, Chicago, IL

SHEARER, M. J., Jr. (A '69, M '77)
Assistant Superintendent, Terminal, Louisville & Nashville Railroad, Louisville, KY

THAYER, M. A. (M '70)
Assistant Engineer Track, Bessemer & Lake Erie Railroad, Greenville, PA

VERCELOTE, D. L. (M '64)
Industrial Development Agent, Elgin, Joliet & Eastern Railway, Joliet, IL

WECCHEIDER, H. J. (M '29, L '61)
Retired Engineer Maintenance of Way, Erie Railroad, Youngstown, OH

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