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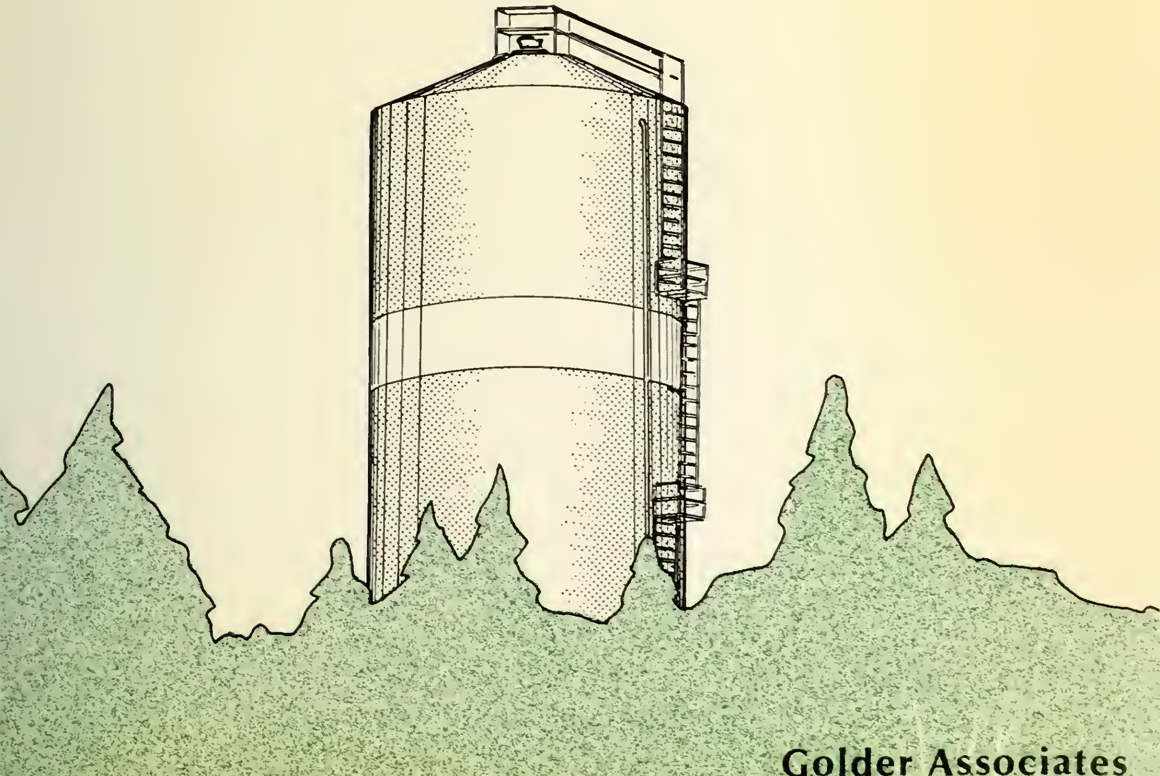
DETERIORATION AND REPAIR OF ABOVE GROUND CONCRETE WATER TANKS IN ONTARIO, CANADA

REPORT TO

ONTARIO MINISTRY OF THE ENVIRONMENT



Golder Associates

W. M. Slater
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September, 1987

DETERIORATION AND REPAIR OF ABOVE GROUND CONCRETE WATER TANKS IN ONTARIO, CANADA

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REPORTS PREPARED UNDER THE APPLIED RESEARCH PROGRAMME FOR THE MINISTRY OF THE ENVIRONMENT

Evaluation of Waterproof Coatings for Concrete Water Tanks.
Mackenzie, K., (Dalhousie Materials), Slater, W.M.,(W.M. Slater & Associates Inc.) and McGrenere, P., (Knox Martin Kretch) (Editing). Preliminary, 1985.

Freeze Protection for Above Ground Concrete Water Tanks in Cold Regions.
Aldworth, G., Staton, R., (MacLaren Engineers), and Slater,W.M., (W.M. Slater & Associates Inc.). Preliminary, 1985.

Temperature Monitoring, Ontario Concrete Water Tanks.
Grieve, R. (Golder Associates). May, 1984 and February, 1986.

Ice Loading in Elevated Water Tanks.
Campbell, T.I. and Kong, W.L., (Queen's University). April, 1986.

FOREWORD

This report began as a study of the problems associated with the deterioration of some of the 53 above ground concrete water storage tanks built in Ontario during the period 1956 to 1980. It was initiated and funded by the Ministry of the Environment (MOE) as part of the concrete water tank rehabilitation programme which was supervised by it's Project Engineering Branch

Concrete tanks are structurally straightforward systems but, being exposed to severe environmental conditions, appear to suffer a rate of deterioration which has greatly reduced the expected life of the structures. The damage ranges from heavy surface spalling and cracking to delamination and eventual failure of the structure. The study showed that the prime factors identified as determining the rate of concrete structure deterioration were the number of freeze-thaw cycles, temperature amplitudes and frequencies, concrete permeability, hydrostatic pressure, location, the effect of steel reinforcement embedments, and internal ice formations.

Since construction defects, as well as the prime factors listed above, have a dominant affect on the accelerated rate of deterioration of concrete water tanks in Ontario, remedial solutions such as repair of joints and honeycombing, applying waterproof coatings, insulation, and replacement by steel tanks, were proposed.

The study was limited to addressing already established problems existing in the 53 pre-1981 structures. Many of the concrete tanks had site specific problems, so basic research into such areas, for example, as the design of special concrete mixes to improve new concrete tank service lives to, say, fifty years was not carried out.

The objective of the study was to be directed towards seeking rehabilitation solutions for existing structures in order to achieve a life expectancy of at least 25 more years. The applied research programme, therefore, was focussed mainly in the direction of repair, rather than on basic concrete research.

In spite of the emphasis in the study to seek remedial solutions to the various problems associated with the rapid deterioration and failure of concrete water tanks in Ontario, a study of the mechanisms which caused some of the observed rapid failures was considered to be most important in seeking repair solutions. Numerous field observations of apparently unique and hitherto unreported and undocumented types of failure in reinforced and prestressed concrete water tanks, required some attempt to find a scientific explanation for the causes of the problems.

The mechanisms of concrete dilation and delaminations, as well as the effect of internal ice, air entrainment, thermal differential strains and strain rates, and other deterioration factors are described in the report, but no laboratory or proof tests have been carried out to date.

Since the various factors and mechanisms may act concurrently, and have not been described in technical literature, it is recommended in the report that basic research be carried out in these areas in the future to try to quantify the deterioration identified.

A brief section in the report gives the principal results of the temperature monitoring of three concrete tanks, one uninsulated and two insulated, from three separate locations in central, south-west, and north-west Ontario.

The main objective of the monitoring programme was to measure the thermal history of an uninsulated tank, and by comparison, measure the effectiveness of insulation systems developed as possible engineering solutions to the concrete tank deterioration problem by reducing both the number and temperature amplitude characteristics of freeze-thaw cycles on the tank walls. The detailed graphical data is presented in a separate report prepared for the MOE by Golder Associates in February 1986, entitled "Temperature Monitoring, Ontario Concrete Water Tanks". Useful information such as the number and frequency of freeze-thaw cycles, differences between exposed solar and shaded quadrants, temperature differentials and rate of temperature change, temperature external to and within the concrete walls, etc. has been reported and plotted in that report. Full analysis of this data and its general aspects is not within the scope of this report.

The main conclusion of the report is that, without adequate protection of permeable concrete from direct contact with water and elimination of cyclic freezing in the tank wall, above ground concrete water tanks will continue to deteriorate rapidly.

The report discusses the various methods used to repair different types of concrete tanks and gives recommendations for assessment and analysis of repair systems.

Although corrosion of metals is not a serious problem in concrete water tanks, some corrosion of tendons and other metallic components has occurred. The report gives examples of this type of deterioration, and the remedial methods used in the rehabilitation programme.

Interim guidelines have been prepared for the design and construction of new concrete water tanks in Ontario based on the experience gained during the rehabilitation programme, and from the applied research carried out. The guidelines recommend that internal waterproofing and external insulation be used as the primary protection against the deterioration of above ground concrete water tanks in Ontario.

ACKNOWLEDGEMENTS

The work described in this report is the result of the efforts of many individuals who assisted with various tasks from tank inspection and testing through to review and drafting of the report.

The members of the Applied Research Group wish to acknowledge the help, encouragement, and support offered by the following Ministry of the Environment personnel: L. Benoit, R.K. Brown, R.G. Crawford, J.C.F. MacDonald (retired), W.C. Ramsden, P. Rostern, M. Toza, O. Wigle, and T. Wright.

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1.0 INTRODUCTION

1.1 General

In recent years it has become evident that concrete water tanks located in various regions of Ontario suffer distress.

In 1981 a study was undertaken on behalf of the Ministry of the Environment by W.M. Slater & Associates Inc. to identify the nature and extent of the tank deterioration.

The study revealed (Ref. 1 and 2) that a large number of the water tanks inspected had deteriorated significantly over a short period of time. Although it was expected that the life of a water tank should be in excess of 50 years, most of the tanks studied were less than 9 years old, with an average age of about 6 years. It was noted that two water tanks had structurally failed.

Photo 1-1 *Collapsed tank* (DUNNVILLE)

The main conclusion of the study was that, in general, the rate of deterioration was unexpectedly high and, if not arrested, would lead to structural failure of the tanks. Based on this potentially dangerous situation, it was recommended that a repair programme be initiated immediately.

1.2 Description

During the preliminary surveys and subsequent remedial work to the tanks it was found that little data was available on the types of materials used, construction records or test results of materials. It was, therefore, decided that a central system of acquiring data and providing transfer of technical knowledge gained was essential to the successful outcome of the repair programme.

1.3 Applied Research

Concurrently with the repair programme, a programme of applied research was initiated in order to provide an understanding of the factors leading to water tank deterioration so that current remedial measures and the design of structures in the future would have the advantage of this knowledge. Golder Associates undertook the examination of the physical mechanisms of tank deterioration and collection of the information obtained from the remedial works. Other studies carried out under the Applied Research Programme were directed towards waterproof coatings, freeze protection (both of the concrete tank and the stored water), ice loadings, and temperature monitoring.
(see page (i))

1.4 Rehabilitation Programme

An important part of the tank rehabilitation programme in the period 1981 to 1984 was to record any noted tank defects, establish causes where possible, obtain samples of defects and subsequent repair materials, and record and monitor the quality and effectiveness of repair methods, materials and applications. Prime consultant for the programme was W.M. Slater & Associates Inc. Golder Associates provided an overview of the materials used and specialized testing. Design and resident inspection for each repair was carried out by various consultants who were responsible for the supervision and management of the repairs.

Photo 1-2 *Tank implosion. (SOUTHAMPTON)*

To provide a degree of uniformity in inspections, a daily inspection sheet was developed by Golder Associates to be completed by each resident field inspector recording observations on ambient conditions, type of work being undertaken by the contractor, visual estimate of quality, and samples obtained. As the remedial programme proceeded, general patterns of deterioration with respect to tank types, methods of construction, construction defects and the like became evident.

Photo 1-3 *Detailed view of imploded section. (SOUTHAMPTON)*

It is the purpose of this report to illustrate the typical defects occurring in concrete water tanks in Ontario and to present a mathematical model of the mechanisms of deterioration observed which are described in sections 4 and 12. The report summarizes the conclusions reached from the applied research and proposes reasons for the rapid rate of deterioration resulting in a service life of under 10 years for many concrete water tanks in Ontario. It describes the repair methods developed for the various defects observed, proposes recommendations to improve the performance and service life of existing tanks, and makes recommendations for the design and construction of future concrete water tanks in Ontario.

1.5 Above Ground Water Tank Types

Ontario concrete water tanks incorporate many different and individual design features; however, there are essentially three different categories, namely:

- standpipes
- elevated tanks
- ground tanks

Standpipes are cylindrical structures up to 46 m (150 ft.) high and 7 to 9 m (25 to 30 ft.) in diameter. Large storage capacity and high internal water pressure in the lower portion of standpipes are distinct features of this category of tanks. (See Figure 1.1).

Elevated tanks have a smaller storage capacity but are capable of providing high operating head with relatively low internal head, approximately 10 m (30 ft.). Water pressures on the walls of ground and elevated tanks are about half the pressure present in standpipes.

Ground tanks have low operating head, about 10 m (30 ft.), but have a large capacity due to their diameter, up to 30 m (100 ft.). These tanks are constructed at the ground level and can be classified

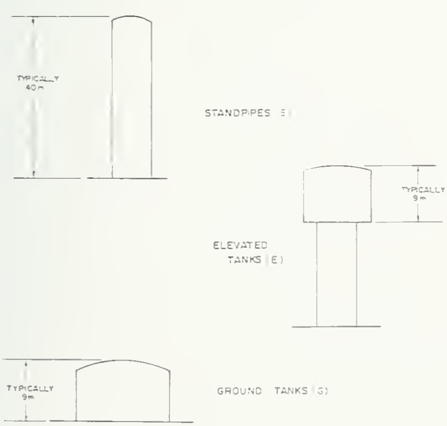


Figure 1.1 *Categories of Ontario concrete tanks*

TABLE 1/1

Number of tanks of each type constructed

Type Number	Abbreviated Designation	Construction Method Description	Number of Tanks
1	RC-S	Reinforced concrete-standpipe	8
2	G-S	Post-tensioned wire wound gunite protected - standpipe	9
[illegible]	PTU-S	Post-tensioned unbonded - standpipe	4
[illegible]	PTB-S	Post-tensioned bonded - standpipe	4
	RC-E	Reinforced concrete - elevated	3
6	G-E	Post-tensioned wire wound gunite protected - elevated	2
[illegible]	PTU-E	Post-tensioned unbonded - elevated	4
8	PTB-E	Post-tensioned bonded - elevated	8
[illegible]	G-G	Post-tensioned wire wound gunite protected - ground	9
[illegible]	RC-G	Reinforced concrete - ground	2
	TOTAL		53

as low head tanks. Within each category of above ground tanks built in Ontario, there are three main structural types as listed in Table 1/1. (see also fig.5.1 page 5-7)

1.6 Performance Rating

The performance rating of different types of above ground tanks were determined in the survey of tanks carried out by W.M. Slater & Associates Inc. (Ref. 2). Table 1/2 presents the average rating of tanks by category based on data presented in that report.

It can be seen that standpipes were given the poorest performance rating. The performance rating of tanks within the same category varies and data can be found in Ref. 1 where the rating system developed by W.M. Slater & Associates Inc. is described.

TABLE 1/2

Tank performance rating by category (1981)

Rank	Tank Category	No.	Performance Rating (0-9) Scale*
1	Ground	9	8.0
2	Elevated	26	6.9
3	Standpipe	10	5.9

* A rating of 9 represents a sound tank.

2.0 DETERIORATION OF ABOVE GROUND CONCRETE WATER TANKS

2.1 General

There were 11 main defects revealed in the survey of concrete water tanks. These are summarized in Ref. 1 and are listed in Table 2/1 below, in order of diminishing priority and structural importance,but not necessarily cost.

TABLE 2/1
DEFECT RATING

1. Wall delamination.
2. Vertical cracks in wall.
3. Wall/floor joint leaks and wall deterioration.
4. Vertical voids in shotcrete.
5. Spalls caused by jack-rods left in walls.
6. Cover coat delamination and debonding from prestressing wires.
7. Waterproof coating delamination and debonding.
8. Cover coat shrinkage cracking.
9. Cold joints and horizontal cracks.
10. Corrosion of prestressing wires.
11. Corrosion of post-tensioning tendons.

Between 1976 and 1986, approximately 50 concrete water tanks were inspected in detail and were partially or fully repaired, including the demolition and replacement of 11 concrete tanks by steel tanks. During this process, it was found that each tank has its own deterioration peculiarities; however, several types of defects can be associated with individual tank types.

2.2 Observations

2.2.1 External Wall Delamination

Vertical wall delamination was considered to be the most serious concrete defect found in concrete water tanks, and varied in form according to the type of construction used and whether the structure was prestressed or not.

Examples of delaminations associated with the various types of construction are described below:

.1 Post-Tensioned Unbonded (PTU)

Two standpipes which failed at DUNNVILLE and SOUTHAMPTON, exhibited wall delamination on the centre lines of tendons which apparently filled with water and then froze causing splitting of the wall. The standpipes were re-built in steel.

.2 Post-Tensioned Bonded (PTB)

Investigations of walls at leaks revealed serious delamination of the walls at CASSELMAN, RED LAKE and PICKLE LAKE elevated tanks. The delaminations were on the centre lines of the post-tensioning ducts and occurred mainly in the bottom of the tanks. At PICKLE LAKE, delaminations occurred in the presence of major horizontal cracks caused by ice thrust. In some cases, the ducts were observed to be ungrouted at the delamination locations.

.3 Post-Tensioned Wire Wound Gunite Protected (G)

Many isolated delaminations were noted during repairs to the FENELON FALLS, BADEN, and other similar wire wound tanks. These external delaminations could also be associated with internal jack-rod spalls, a defect discussed in greater detail later in this report. At VAL CARON, a large diameter wire wound post-tensioned tank with a gunite cover coat, it was found that small external delaminations were associated with leakage through form-ties.

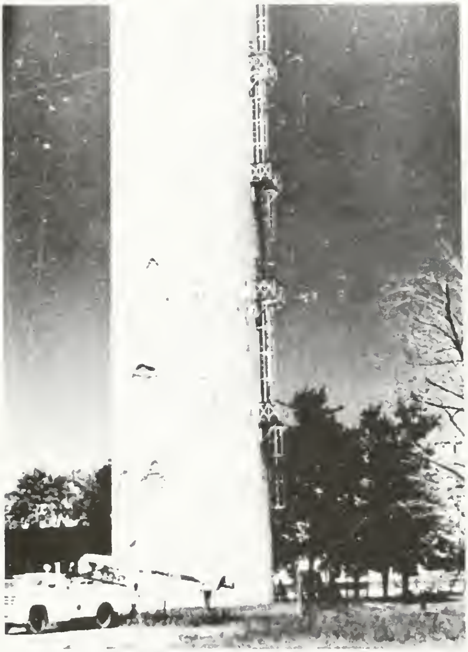


Photo 2-1 *Typical deterioration at bottom of reinforced concrete standpipe (WATFORD)*

At AMHERSTBURG and CHELMSFORD major external delaminations had occurred at locations where there were defects in the waterstop between the wall and floor of the tanks, as well as at other minor locations.

At WOODVILLE, a post-tensioned wire wound gunite standpipe, a series of delaminations occurred around the tank at the level of the manhole. Further detailed inspection revealed that, due to the presence of the manhole, the post-tensioning wires had been omitted from this region of the tank (see Photo 2-8).

.4 Reinforced Concrete Standpipes (RC)

The areas of delaminations found in reinforced concrete standpipes were much greater than in the tanks constructed of prestressed concrete.

Photo 2-1 and Photo 2-2 show the south face of the WATFORD water tower and illustrate typical deterioration of the lower section of a tank. Delamination occurred at the level of the reinforcing steel with little or no corrosion of the steel.

Similar delamination was found during repair of the ALVINSTON tank where large areas of external and internal delamination generally occurred on the side of southern exposure. One interesting observation from this tank was that an internal epoxy resin coating had been applied to the lower 6m (20 ft.) of the tank and all major delamination had occurred above this level.

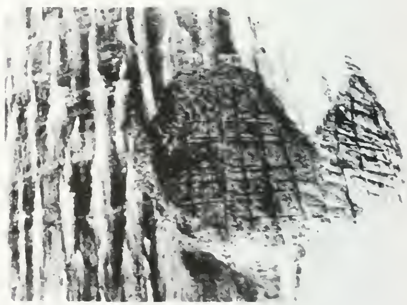


Photo 2-2 *External wall delamination (WATFORD)*

A third reinforced concrete standpipe also exhibited extensive wall delamination (CAMLACHIE) and was eventually replaced by a new tank. Photos 2-3 and 2-4 show the major spalled areas. As with the other reinforced concrete tanks, no corrosion of the reinforcing steel was observed.

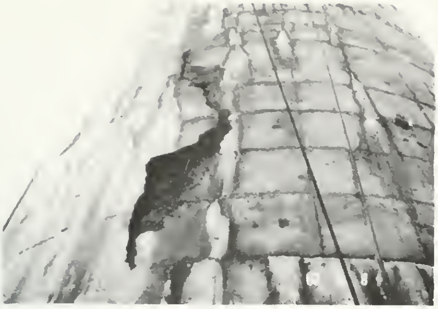


Photo 2-3 *External wall delamination (CAMLACHIE)*

Cores taken through the tank walls indicated fracture of the concrete at both the external and internal reinforcing steel (Photo 2-5).

Photo 2-4 *Fracture at reinforcing steel (CAMLACHIE)*

The CAMLACHIE tank shown in Photo 2-3 was monitored during the winter of 1982-83 prior to intended repair. It was observed that the majority of fracturing occurred during the spring of 1983, sometimes with explosive force accompanied by noise and vibrations of the structure.

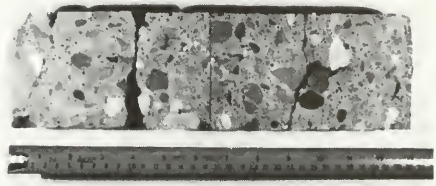


Photo 2-5 *Core section showing fracture at external and internal steel (CAMLACHIE)*

After the detailed tank condition survey (Figure 2.1) taken during December 1982 and after the subsequent damage of the 1982-83 winter, a rehabilitation design was completed. It was decided, however, to replace the tank with a steel structure.

Because of the severity of delaminations found in most reinforced concrete standpipes and the high cost of repairs, strengthening, insulation and cladding, the following additional standpipes and elevated tanks were, or will be demolished and replaced by steel tanks - ARKONA, CALLANDER, MARKDALE, PITTSBURGH, WEST LORNE and WYOMING.

ARKONA tank, a reinforced concrete standpipe was inspected internally in summer 1985 after dewatering. Delaminations similar to the CAMLACHIE and WATFORD tanks were found at the bottom of the southern exposure. Cores taken in

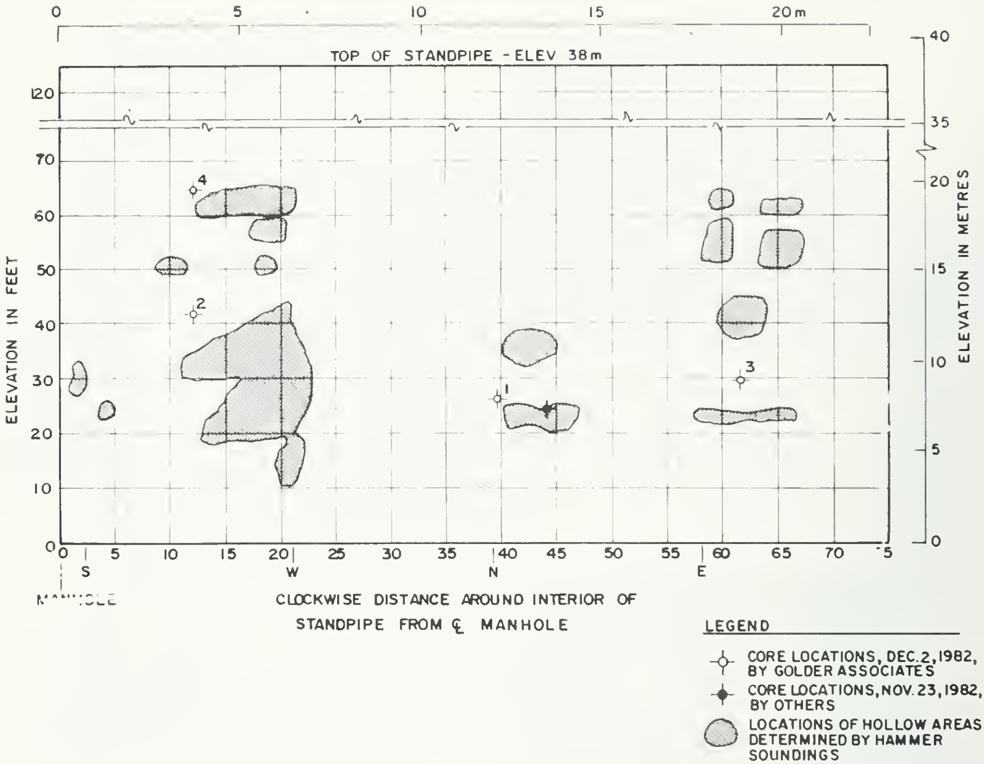


Figure 2.1 *Delamination survey of standpipe (CAMLACHIE)*

these regions confirmed that the tank wall had delaminated at the plane of the external steel. In addition the concrete surrounding the steel was almost totally disintegrated.

In an effort to obtain a sound core for compressive strength testing, the ARKONA tank was sounded throughout its lower circumference. An area was selected at the east exposure just outside of a location considered to be delaminated. Due to the presence of a waterstop, the core was extracted in two sections. The waterstop was found to be defective at that location, however both core segments were found to be sound. It was noted that the inside hoop steel was well bonded to the concrete, but the outside hoop steel was completely debonded from the surrounding concrete and was free to move because of considerable enlargement of the interface surrounding the reinforcing steel. Detailed examination of this interface revealed that the debonding was not uniform around the reinforcing steel. The concrete interface towards the inside of the tank wall mated perfectly with the steel but there was a gap of about 2 mm surrounding the section of reinforcing steel facing the outside tank wall.

Due to the geometry of the gap the more usual causes of debonding such as lack of cleanliness of the reinforcing steel or excessive bleed water during concrete placement were considered unlikely. Since serious delamination of the tank wall had been revealed in other cores it was considered possible that the condition of this core was a result of the deteriorating mechanism and was representative of a tank wall shortly before delamination (see Photos 2-6 and 2-7).

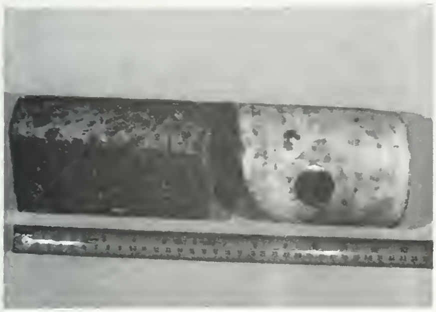


Photo 2-6 *Core through tank wall (ARKONA)*

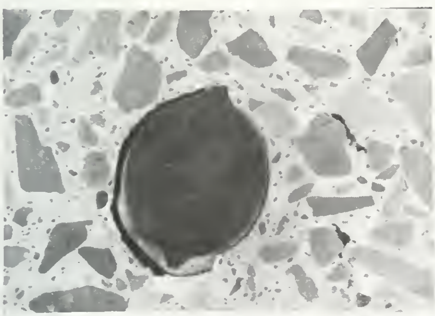


Photo 2-7 *Close-up of expanded steel concrete interface (ARKONA)*

.5 Conclusions

Based on the above observations it was concluded that, in general, where the internal water under pressure was allowed to penetrate to the external surface, external delamination was likely; furthermore, reinforced concrete standpipes particularly in the southern lower portions of the tank, appeared to be more susceptible to external delamination than other tank types including elevated prestressed concrete tanks. In comparing standpipes, it appeared that post-tensioning reduced the likelihood of external delamination except where internal spalling had reduced the wall thickness. It was concluded that the physical factors involved were the availability of water, pressure, and southern exposure, with radial compression being a mitigating factor.

Photo 2-8 *External delamination in wire wound post-tensioned tank (WOODVILLE)*

2.2.2 Internal Wall Delamination

Several different categories of internal delamination were observed. The categories noted were extensive delamination to the depth of the internal reinforcing steel, localized spalling where the depth of cover to the reinforcing steel was shallow, internal surface delamination, conical spalls associated with jack-rod couplings, and at locations adjacent to vertical unbonded post-tensioning tendons.

As with external delamination, major areas of deep internal delamination were invariably associated with

Photo 2-9 *Internal delamination (ALVINSTON)*

reinforced concrete standpipes. Again, little or no corrosion of the reinforcing steel was noted (Photo 2-9).

2.2.3 Localized Spalling

Localized spalling had occurred at many tanks and could not be associated with any tank type in particular, however, it was noted that in the majority of cases the localized spalling was associated with a shallow depth of cover to the reinforcing steel or with individual aggregate particles (Photo 2-10).

Photo 2-10 *Localized spalling (BADEN)*

One interesting defect was found at the BADEN tank where two construction gloves had been accidentally buried in the tank wall at separate locations. In both cases deterioration was extensive and deep around the gloves (Photo 2-11). In another tank (VAL CARON), localized spalling occurred at vertical joints and could be associated with leakage at the water-stop (see Photo 2-12).

Photo 2-11 *Deterioration caused by construction glove (BADEN)*

In the majority of tanks, the internal coating had deteriorated away from the wall (Photo 2-13). Examination of core sections revealed that in most cases the surface of the concrete was fractured to a depth of between 5 mm and 15 mm In some cases the deterioration was so extensive that the remnants of the coating could only be detected by microscopic examination.

Photo 2-12 *Damage caused by water seepage at water-stop (VAL CARON)*

Deterioration of this nature did not appear to be associated with tank type but seemed to be a function of coating type and quality (see report to Ministry of the Environment (1985) titled "Evaluation of Waterproof Coatings for Concrete Water Tanks" where this subject is discussed more fully).

Photo 2-13 *Coating deterioration and a jack-rod spall (WOODVILLE)*

2.2.4 Jack-Rod Spalls

Some tanks had been constructed using a slip form method. Hollow pipe rods, known as jack-rods, each approximately 3 m in length are used to raise the slip form. As construction proceeds, additional lengths are added using threaded coupling rods. This results in hollow tubes at approximately 3 m centres extending the full height of the tanks and interrupted at each joint by the solid rods.

Photo 2-14 *Jack-rod spall at threaded coupling (BADEN)*

In the tanks inspected to date, little attempt was made to seal these jack-rods; the effectiveness of the seal between the roof and the tank walls was apparently relied on to prevent the ingress of water. Inspection has shown that this assumption is untrue and that, in most cases, water was able to enter the uppermost jack-rod section. This water leaks out at the threaded coupling, saturating the surrounding concrete.

Observations revealed that these tanks experienced internal spalls and that the spalls invariably occurred exactly at the coupling between the individual rods (Photo 2-14). The spalls were always conical in shape, and although a small amount of totally disintegrated concrete was usually present at the apex, the spalled concrete was normally intact without any sign of disintegration (Photo 2-15). In general, as many as 20 to 30 of these spalls could be present in the tanks constructed using this system (see Figure 2.2).

Photo 2-15 *Intact spall recovered from bottom of tank (WOODVILLE)*

2.2.5 Cover Coat Shrinkage and Cracking

On tanks where a concrete cover coat had been applied to the prestressing wires, external delamination was not a serious problem, however, the majority of these tanks had many cracks principally in the horizontal direction. (see Photo 2-16 and Photo 2-17).

Photo 2-16 *Typical horizontal cracks and leachate stains occurring at a prestressed standpipe (WOODVILLE)*

Photo 2-17 *Typical cracks and leachate stains occurring at a prestressed ground tank (VAL CARON)*

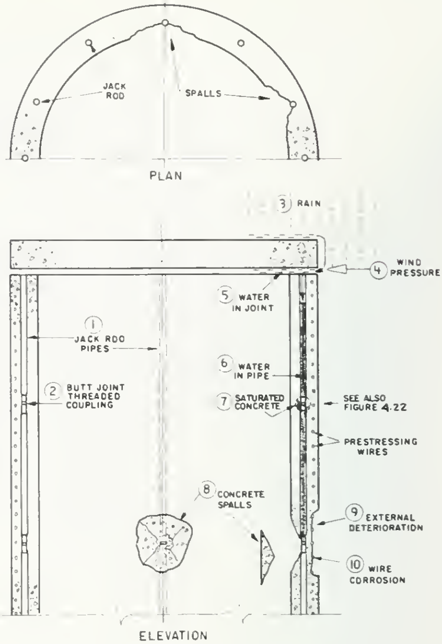


Figure 2.2 *Jack-rod spalls*

In most instances these external horizontal cracks could be traced through to the inside wall. Core sections taken through this type of defect sometimes indicated that freeze-thaw damage was present at a microscopic level while in other instances the crack had no deterioration. The general rule observed was that where leaks or leachate stains were present on the outside, then internally fractured walls were likely.

2.2.6 Quality of Concrete in Water Tanks

As stated in the Portland Cement Association's publication, "Design and Control of Concrete Mixtures", one of the greatest advances in concrete technology was the development in the mid-1930's of air-entrained concrete. The principal reason for using intentionally entrained air is to improve concrete's resistance to freezing and thawing. However, there are additional beneficial effects of entrained air in both fresh and hardened concrete. Known benefits are improved workability, resistance to de-icers, and sulphate resistance. The latter two factors may be explained by the reduction in water demand for a given cement content due to the improved workability and which leads to a reduction in the water cement ratio and, hence, produces a less permeable cement matrix.

However, the application of "gunite" is not ideal for air entrainment and, as, shown in Table 1/1 a number of tanks have, in the past, used this method.

Unfortunately, original quality control data obtained during the construction of the tanks utilizing this method are not readily available; however, it is understood that normal quality control tests were made and were consistent with the Ministry and other relevant standards specified at that time. Typically, for normal concrete placement methods, the requirements were for a 28 day cylinder strength in excess of 3000 psi and, where normal concrete placement methods were utilized, an entrained air content of 5 to 7 percent was specified. Table 2/2 gives details of typical concrete quality obtained by core sampling several of the water tanks during remedial works. The results of tests for compressive strength indicate that, in general, the compressive strength of the concrete considerably exceeded the required standards of the day. As stated above, some concrete tanks were constructed using air entrained concrete. Other tanks due to the concrete application method, were not air entrained. Although in most cases the air entrained concrete contained less air than desirable, nevertheless, most of the examples given are within the limits of "acceptable quality" as judged by the Ministry of Transportation and Communications Bridge Deck Rehabilitation manual (Ref. 3). Thus, it can be stated that the presence and pattern of deterioration was not consistent with the lack, or otherwise, of air entrainment. Furthermore, it was considered that, since most of the concrete standpipes exhibited external deterioration at the bottom of the tanks it would be a remarkable coincidence if all poor quality concrete was placed in the lower sections of these tanks and all durable concrete was placed in the uppermost sections.

Photo 2-18 *Deterioration of reinforced concrete standpipe (ALVINSTON)*

TABLE 2/2 Summary of typical water tank core test results

Tank Name	Compressive Strength (MPa)		Air Content (%)	Spacing Factor (mm)	Remarks
	Design	Tested			
Arkona	28.0	45.7	3.0	0.281	Cast Concrete
Rockwood	35.0	84.5	*	*	Shotcrete
	*	61.6	*	*	
Odessa	35.0	26.6	4.1	0.342	Cast Concrete
	*	*	2.5	0.214	
Woodville	32.0	78.6	*	*	Cast Concrete
	*	61.4	*	*	
Amherstburg	32.0	41.4	1.2	0.124	Shotcrete
Hespeler	32.0	88.0	*	*	Shotcrete

2.2.7 Summary of Observations

Based on the results of these surveys the following generalizations can be made with respect to the walls of concrete tanks.

1. All tanks exhibiting general external deterioration have an ineffective internal coating.

2. Tanks exhibiting isolated external deterioration have an internal defect which allows seepage of water toward the exterior tank surface.

3. High pressure tanks exhibit more deterioration than low pressure tanks.

4. Reinforced concrete standpipes exhibit the most severe external deterioration.

5. External deterioration tends to be oriented towards the south exposure (solar quadrant).

6. There is little galvanic corrosion of reinforcing steel in areas of exterior and interior wall delamination.

7. Atmospheric corrosion of prestressing wires has occurred where the concrete cover coat has delaminated and separated from the wall.

Apart from the last two points relating to corrosion, it can be seen that freezing and thawing is the consistent underlying phenomenon related to deterioration of concrete water tanks in Ontario. In some cases, defects which may be relatively harmless in other structures, significantly alter the durability of the tank. In other cases, structural types and construction methods appear to have a significant bearing on the useful life of the tank.

To help explain these observations, published research on the freeze-thaw durability of concrete was examined, the temperature histories of tanks were monitored and a conceptual model describing progressive freeze-thaw deterioration was developed. These subjects are discussed in the following sections.

3.1 General

Although most of the parameters required for this study were available in published research, the literature did not reveal data of adequate detail to assist in providing an understanding of the climatic influence on water tanks. Consequently in January 1983, as part of the on-going applied research, the Ministry of the Environment initiated the installation of a temperature monitoring system at ROCKWOOD, an uninsulated concrete water tank. A similar system was installed at ALVINSTON, the first concrete tank to be insulated as part of the tank rehabilitation programme.

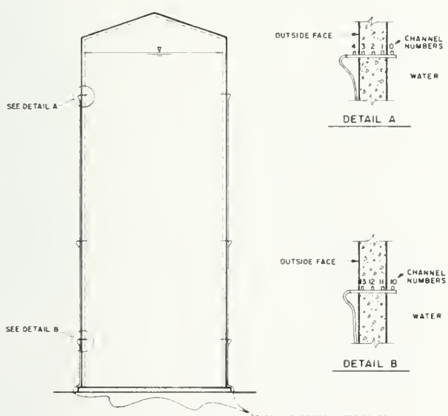


Figure 3.1 *Location details of temperature instrumentation(ROCKWOOD)*

The temperature monitoring sensors were installed in January 1983 prior to completion of the ALVINSTON insulation. To study the effects of a northern environment, a monitoring system was installed during a 1984 rehabilitation to EAR FALLS, a tank with the first combined heating, circulating and insulation systems installed for freeze protection.

3.2 Temperature Instrumentation

The ROCKWOOD temperature monitoring system consisted of a total of 55 temperature sensors and was designed such that temperature variations with respect to height and solar exposure could be analyzed. The assembly consisted of four main cables at each cardinal point. Each cable had an array of sensors at the 20 m (60 ft.), 10 m (30 ft.), and 3 m (10 ft.) levels. The two uppermost arrays consisted of five separate sensors embedded into the tank wall such that the inside sensor was in the water, three sensors were evenly spaced inside the tank wall, and one sensor was exposed to the atmosphere. The lower array (10 ft. level), consisting of four sensors, did not have the external sensor (see Figure 3.1).

The system was linked to data logging and computer equipment which was set to monitor the temperature of the sensors at two hour intervals.

The instrumentation at EAR FALLS and ALVINSTON was of similar design, but with some of the sensors placed in the air gap between the insulation and the outside concrete to check actual heat losses and accuracy of heat-loss calculations (see Figures 3.2, 3.3, 3.4 and 3.5).

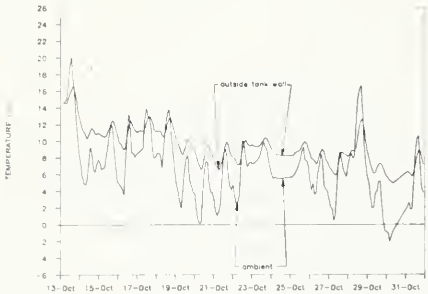


Figure 3.2 *1983 autumn data at north side of an uninsulated tank (ROCKWOOD)*

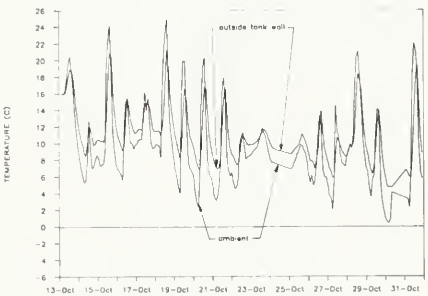


Figure 3.3 *1983 autumn data at south side of an uninsulated tank (ROCKWOOD)*

Figure 3.4 *1984 winter data on north side of an uninsulated tank (ROCKWOOD)*

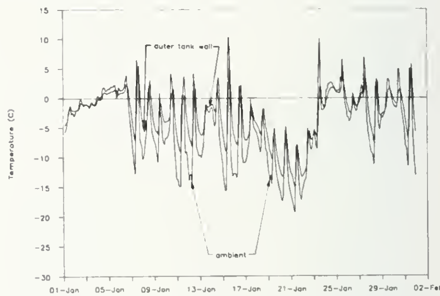


Figure 3.5 *1984 winter data on the south side of an uninsulated tank (ROCKWOOD)*

3.3 Test Results

Figures 3.2 to 3.7 illustrate typical data from each of the instrumentation systems.

Figures 3.2 and 3.3 are graphical records of typical Autumn data comparing the north with the south aspects of the ROCKWOOD tank. The water supply is obtained from an underground source and has a steady inlet temperature of approximately 7°C. Figures 3.4 and 3.5 are graphs of typical data obtained at these locations during the winter period. Analysis of the data revealed the following points.

- Temperatures in the North exposure are almost continuously below freezing throughout January. Freeze-thaw cycling occurs almost daily in the South exposure during the same period.

- There is a substantial difference in daily temperature fluctuation between North and South ambient temperatures. This difference is reflected in the concrete wall temperature. The amplitude of the ambient temperature and at 25 mm (1 in). into the wall surface at the South exposure is approximately 20°C and 15°C, respectively. Corresponding amplitudes for the North exposure are 7°C and 5°C, respectively. These differences in amplitude account for the significant difference in the number of occurrences of freeze-thaw cycling in the South exposure when compared to the North exposure.

- Freezing and thawing regularly takes place on the inside of the tank wall and has a considerable influence on the water temperature close to the tank wall.

- Temperature records of the tank water obtained during sub-zero ambient conditions indicate that the tank water provides very little buffering effect and suggests that little water circulation takes place inside the tank. Sensors at the centre of the water in the tank indicate a steady reduction in water temperature (throughout the period).

- Sensors in the water close to the tank wall indicate that the water quickly freezes at that region of the tank forming an ice ring around the wall of the tank. It should be noted that for the period examined, water close to the North wall froze 4 days prior to the water at the South wall.

- The sensor in the water at the 20 m (60 ft.) level showed that an ice cap forms at the top of the tank to a considerable depth (greater than 3 m). Visual observations confirmed the presence of the ice cap and its presence was noted throughout the winter period.

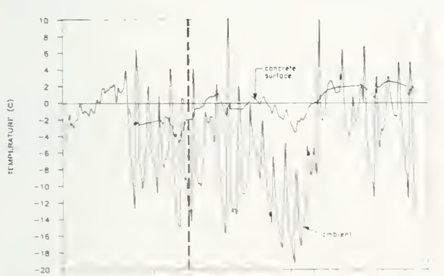


Figure 3.6 *1984 winter data at an insulated tank in Southern Ontario (ALVINSTON)*

Figures 3.6 and 3.7 are graphical records from the insulated ALVINSTON and EAR FALLS tanks where, in both cases, water is from a surface source.

Figure 3.6 shows that the insulation has a significant moderating influence on the air space where minimum temperatures are roughly one third of the minimum external ambient temperatures. It should be noted that the construction of the insulation was completed on this tank on January 12, 1984. (shown as a vertical line in figure 3-6).

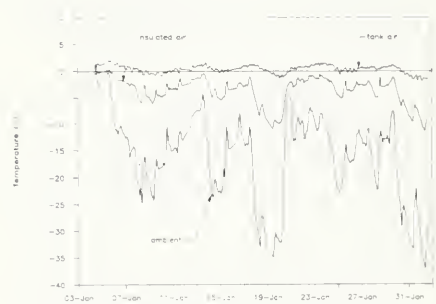


Figure 3.7 *1985 winter data at an insulated tank in Northern Ontario (EAR FALLS)*

Figure 3.7 is a graphical record of the EAR FALLS tank during January 1985. Again, the data illustrates the significant moderating influence of the insulation on the air gap. It was recorded that the temperature of the tank water is largely independent and isolated from the ambient conditions.

(Detailed records from which the above noted observations were obtained are given in an Applied Research Report Titled "Temperature Monitoring of Ontario Concrete Water Tanks" issued by the MOE in April 1986).

3.4 Freezing in Concrete

Distilled water exposed to atmospheric conditions will freeze at 0°C. The freezing point of water, however, is lowered with reduction in purity and increasing pressure. In concrete there is a considerable range of pore sizes. Research by Helmuth (Ref. 4) demonstrated that supercooling is more likely to occur than freezing unless it is seeded by a catalyst in a process called nucleation.

The smaller the pore size, the more difficult it is for nucleation to occur. For this reason, at any point during cooling below 0°C, all the water occupying cavities smaller than a certain size remain unfrozen. The lower the temperature the greater becomes the quantity of frozen cavities. The formation of ice in the pores of concrete can occur over a considerable temperature range. However, research has demonstrated that for practical purposes freezing of the pore water begins at around -2°C and is essentially complete at around -4°C (ref. 5, and 6).

Another consideration of temperature reduction in this range occurs when the concrete is saturated. As each cavity size is frozen, the passage of water through it is prevented.

The significance of the above phenomena is that freezing in the pores of wet concrete is not a simple event but occurs throughout a range of temperatures below 0°C. As stated above, it can be considered that the majority of freezing is completed at -4°C and depending on the degree of saturation, is accompanied by a reduction in effective permeability. When all pores are frozen, therefore, the effective permeability of the concrete approaches zero starting on the outside face of the structure.

3.5 Seasonal Wall Conditions

The temperature monitoring of the various tanks noted above has revealed that significant seasonal wall conditions can develop depending on such factors as inlet water temperature, the ratio of daily water used with respect to tank volume, tank exposure conditions and geographical location. Where these factors are favourable, namely high inlet temperatures, high tank turnover and sheltered exposure conditions, (e.g. ground tank) then ice is less likely to form on the inside of the wall. However, these conditions are rarely the case. For example, at VERNER, an elevated tank, a 2 m (6 ft.) ring of ice was observed around the perimeter of the inside of the wall. At ROCKWOOD, an unbonded tendon standpipe, a 1 m (3 ft.) ring of ice was observed. Divers measured the thickness of ice occurring during January, 1984, at CASSELMAN, an elevated tank, and reported the thickness of ice to be 300 mm (12 in.) around the walls and 150 mm (2 in.) at the bottom of the tank. Tests at PORQUIS JUNCTION, a reinforced concrete tank, revealed that due to a low daily requirement compared to volume, the effective tank volume was reduced by 75% by the massive formation of ice inside the tank. These observations and many others have shown that ice cap, wall ice and floor ice, often estimated at several hundred tons, regularly form on the inside of most tanks during winter and are often present until the middle of May.

The formation of ice on the internal walls of tanks has several disadvantages such as deterioration of the internal coating due to physical movement of the ice, prevention of internal tank inspection for a large portion of the year due to the danger of falling ice and a reduction in the operating volume of the tank. However, the formation of ice on the internal wall can also produce significant changes in the thermal profile of the tank wall which can have serious consequences on its durability and is, therefore, worthwhile considering in detail.

3.5.1 Thermal Conductivities

To estimate the thermal profile of the various wall conditions the following thermal conductivities were used.

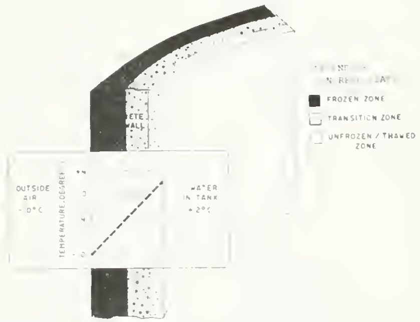
- Steel
  43.0 W/°C^{-m2}
- Concrete (Wet)
  1.81 W/°C^{-m2}
- Ice
  2.16 W/°C^{-m2}
- Water
  0.60 W/°C^{-m2}
- Air
  0.024 W/°C^{-m2}
- Styrofoam
  0.029 W/°C^{-m2}

3.5.2 Autumn - Winter Condition

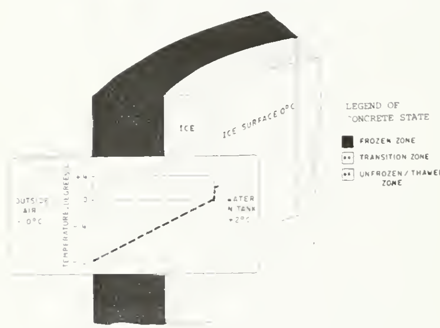
Figure 3.8a is an example of an average temperature which regularly occurs between autumn and mid-winter when the minimum 24 hour ambient temperature is -10°C. During this period, ice has not yet formed on the inside of the wall and therefore the tank water maintains the inside of the wall above zero (typically 2°C). Thermal calculations demonstrate that for this condition the maximum depth of total freezing (-4°C) is about half the wall thickness.

3.5.3 Winter - Spring Condition

Figure 3.8b is an example of an average temperature profile which can occur after the development of a ring of ice on the inside surface of the tank and typically occurs between mid-winter and early spring. In this case, a significant thickness of ice has formed on the inside surface of the concrete. The presence of this ice prevents any buffering effect from the internal water



a) *Ice free condition (autumn - winter)*



b) *Ice ring present inside the tank*

Figure 3.8 *Typical seasonal wall conditions*

and, therefore, the entire tank wall can be frozen to a point where freeze-thaw cycling can occur throughout the thickness of the wall.

The temperature data for January 1984 obtained from ROCKWOOD illustrates the change of temperature profiles as the ice ring is formed and is given on Figure 3.5. The graph shows that the water close to the south wall began to freeze around January 17th. Prior to this, the temperature of the outside of the wall was cycling between -10°C and +5°C and the inside face was consistently above zero. After January 17th, the temperature of the water fell below zero and ice formed. At that point, the entire wall temperature was consistently below zero and temperature cycles occurring in the wall are clearly seen.

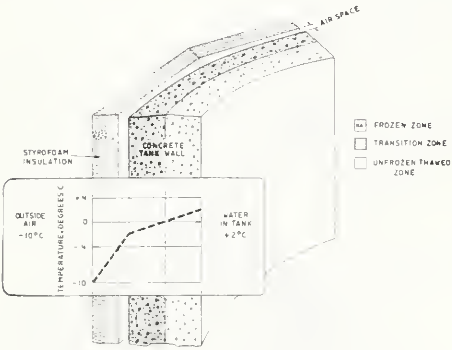


Figure 3.9 *Typical thermal profile after insulating*

3.5.4 Insulated Tank Winter Profile

A typical thermal profile through the wall of an insulated tank is given in Figure 3.9 which illustrates the beneficial effect of the insulation when the ambient temperature is -10°C. At that ambient temperature the air gap temperature is approximately -2°C and the water in the smaller pores remains unfrozen in the outer layer of the concrete. Field inspections made during winter conditions revealed that internal ice formations were essentially eliminated, thus improving the thermal profile through the tank wall and preventing deterioration of the internal coating by freeze-thaw cycling on the inside wall. In addition it should be noted that the insulation reduces the amplitude of the daily thermal cycle considerably - a significant additional benefit.

3.6 Daily Thermal Cycle

Figures 3.10 and 3.11 are graphs giving details of the thermal history of the ROCKWOOD tank between January 14 and January 17, 1984. They illustrate typical thermal cycles occurring approximately 50 mm (2 in.) away from the outside of the tank wall (ambient) and approximately 25 mm (1 in.) into the concrete surface and demonstrate the substantial thermal difference between the North and South exposures.

Figure 3.10 *Daily temperature cycles showing continuous freezing at the North side of the tank*

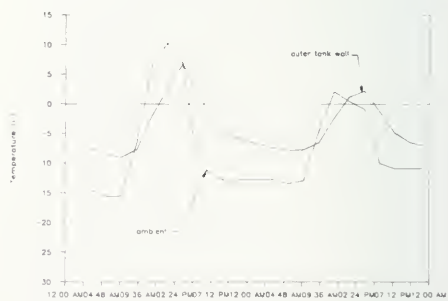


Figure 3.11 *Daily temperature cycle showing daily freeze-thaw cycles at the South side of the tank*

At the North exposure, the maximum ambient temperature adjacent to the wall tends to be consistently below zero. Daily ambient temperature fluctuations are about 7°C, with corresponding concrete temperature changes of about 4°C to 5°C, and is similar to shaded ambient conditions. At the South exposure the temperature fluctuation is significantly different. It can be seen that the daily amplitude of ambient temperature change is about 20°C, with a corresponding amplitude of about 15°C inside the concrete, from +7°C to -8°C.

3.7 Critical Ambient Temperatures

Examination of the detailed temperature history of the North and South faces of an uninsulated tank has demonstrated that the above noted differences are consistent throughout the year. During all seasons, the South exposure receives considerable additional solar energy which increases the daily temperature amplitude when compared to the North exposure. However, the data demonstrates that under certain ambient temperature conditions, it is this additional solar energy received during the winter which greatly increases the number of freeze-thaw cycles at the South exposure.

Based on the data, the daily amplitude on the South face of the wall during winter is about 20°C and daily freezing and thawing cycles can occur where the shaded ambient conditions are between -5°C and -15°C and clear sunny conditions prevail.

At the North face of the structure, the corresponding daily temperature amplitude is about 7°C; consequently, the critical range of shaded ambient conditions for freeze-thaw cycling to occur is between -7°C and -5°C; this may occur infrequently.

The temperature monitoring of an uninsulated tank has revealed that, there is a difference in the daily temperature amplitude between the South and North exposures which accounts for the difference in the number of freeze-thaw cycles experienced by concrete at the South exposure. It can be seen therefore that the majority of freeze-thaw cycles is a result of solar radiation and occurs when the background ambient temperature is within a critical range below zero, and solar radiation thaws the concrete. In Southern Ontario it appears that the critical temperature is between -5°C and -15°C at the South exposure and between -5°C and -7°C at the North exposure.

Comparing Southern Ontario with other regions, it is likely that these basic environmental conditions are less prevalent in either more Northerly or more Southerly climates where the shaded ambient conditions are likely to be either above or below this critical range. Considering these conditions for other climates, it is also likely that even where ambient conditions are within this critical range, reduced solar radiation due to the prevalence of cloud cover will reduce the amplitude of the daily cycle and consequently reduce the critical temperature range. This aspect of the data may help explain why concrete deterioration is so prevalent in Southern Ontario.

4.0 INVESTIGATION OF THE FREEZE-THAW FAILURE MECHANISM IN CONCRETE

4.1 General

There can be several causes of premature deterioration of concrete, some chemical in nature or some as a result of severe climatic conditions. With the exception of the presence of chlorides, most chemical agents have to be present in significant concentrations in order to produce rapid deterioration. Since water tanks are being used to store drinking water and elaborate precautions are made to exclude concentrated chemicals, it was inferred early in the study that the deterioration observed was not chemical in nature.

Many concrete structures which are subjected to extremes of weathering exposure will deteriorate unless protected. Structures frequently exposed to saturation by water, followed by cycles of freezing and thawing, deteriorate more rapidly than others. Such structures include concrete pavements, bridge decks, curbs and gutters, spillway floors and drainage channels.

Concrete water tanks may incorporate some system to prevent water from contact with concrete. Coating breakdown or an inadequate coating can expose the concrete surface to water saturation. This can lead to severe deterioration of concrete under sub-zero temperatures (Ref. 7). The following quotation from this source provides an insight into the nature of the problem:

"The effect of freezing on concrete tanks constitutes a problem of considerable magnitude. Concrete has some porosity; and when it is subjected to high water heads, the water permeates throughout these pore structures. It is easy to see the effect this will have when the excess water is subjected to freezing temperatures. An expansion of the water during freezing will start a slow breakdown of the concrete. The action is known as spalling, and unless it is arrested, the entire tank may end up as an unusable water holding facility. At best, expensive repair bills can result."

The survey of water tanks carried out by W.M. Slater & Associates Inc. revealed that the protective coating in a large number of tanks was ineffective. Exposure of concrete to water under high hydraulic head combined with freezing temperatures undoubtedly creates conditions of dangerous exposure. Although deterioration of concrete in these conditions is a distinct possibility, it is far from evident that conventional measures such as air entrainment of concrete will be able to prevent it.

While it is technically possible to eliminate conditions of severe exposure by providing an unconditionally impermeable liner, e.g. an internal steel shell (Ref. 8) or by providing insulation to the tanks (Ref. 8), such drastic protective measures add significantly to repair or construction costs and should be considered only after gaining a detailed knowledge of the deterioration mechanisms, their rates, and factors which affect them.

It is the purpose of this study to explain the mechanisms of concrete water tank deterioration and to identify factors which affect the deterioration rates in order to provide a rational basis for evaluation of repair and design alternatives.

4.2 Literature Review

The literature on concrete water tanks is not extensive and the main emphasis is on the structural design of tanks, their construction and repair methods (Refs. 8, 9, 10, 11, 12). A record of 19th century water tank failures is contained in the monograph "Stand Pipe Accidents and Failures" by Prof. W.D. Pence who, in 1894, collected data on such occurrences from the earliest known, to the time of publication. It is interesting to note that out of 45 accidents with steel standpipes presented in the book *"23 were total wrecks, 14 were slightly damaged and 8 were slightly injured. As far as determined, the cause of the accident was in 22 cases due to water; in 11 cases water and ice; 11 were reported due to the wind, while a number of cases were from failure of foundation"*. Several more cases were reported in Ref. 9 which dates back to 1910 and from which the above quotation of Prof. Pence's work was taken. Since tanks in that era were not constructed using concrete, most of the accidents described are not directly relevant to modern concrete tanks. However, the sentiments expressed can be associated with current concerns and provide echoes from the past.

The history of modern concrete water tanks dates back to 1908, when the first U.S. patent on prestressed concrete tanks was taken. An interesting history of the evolution of modern water tanks is described in Ref. 7. This recent monograph is apparently one of the few publications which clearly states that concrete water tanks are problem structures in freezing conditions.

Review of modern literature revealed that much of the research on freeze-thaw deterioration of concrete relates to studies of the material aspects of this phenomena. A very significant portion of this work is either directly related to road structures like pavements and bridge decks or is derived from observations on the performance of these structures. Little research has been reported for the freeze-thaw durability of concrete under constant hydrostatic loading. To our knowledge, above ground concrete water tanks have not previously been a subject of a study concerned with deterioration mechanisms under freeze-thaw conditions.

4.3 Some Factors Affecting the Freeze-Thaw Durability Of Concrete

The following remarks were made by T.C. Powers in 1966 when introducing his paper on "Freezing Effect in Concrete". These appropriately state the dilemma when considering the action of freezing and thawing under hydraulic pressure.

"Specimens of concrete kept continually wet on all surfaces by spraying or immersion usually become damaged or destroyed when they are cooled well below the normal freezing point of water; if the period of soaking is long, nearly all concretes so exposed, air entrained or not, cannot withstand freezing. On the other hand, concrete structures having at least one surface exposed to air continually show extremely various behaviour, from total failure, usually localized, to apparent immunity to freezing effects" (Ref. 13).

Examined from this point of view, the concrete tank represents long term immersion; however, one face is also continually exposed to the air. It appears, therefore, that several factors are involved in the durability of concrete water tanks and not just the simple expedient of adding appropriate air entrainment.

4.3.1 Saturation of Concrete in Water Tanks

(i) Unsaturated Concrete

Saturation of concrete can occur in natural conditions of weathering exposure (Ref. 15). It is normally expected, however, that the external concrete walls of a structure will not become saturated. The major difference between water tanks and other types of structures is that without an appropriate waterproofing system, the applied hydraulic pressure will force water through the pores of the concrete, and result in saturation of the concrete walls.

The rate at which water will flow through concrete is a function of its porosity which depends on the size, distribution and continuity of the pores. Research has shown that these factors are a function of mix design, hydration, curing, construction techniques and the like.

(ii) Saturated Concrete

Calculations based on D'Arcy's law have shown that in the lower portion of a typical high pressure head tank, saturation may be achieved between 2 and 4 years and is illustrated in Figure 4.1. This rate of saturation will of course be increased by the presence of normal defects such as horizontal and vertical cracks.

It is only after the concrete becomes saturated to a critical level, above 90%, that the action of freezing and thawing, which was previously harmless, starts its destructive action. It can be demonstrated that even the highest quality of concrete will achieve this state under the pressure conditions prevailing in a typical concrete standpipe.

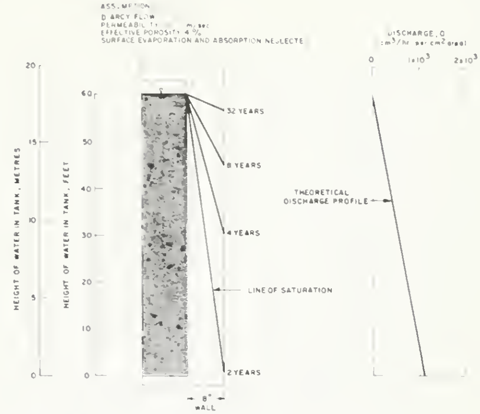


Figure 4.1 *Theoretical saturation zone and seepage discharge*

4.3.2 Hydrostatic Pressure and Evaporation

As stated elsewhere, entrained air bubbles cannot effectively protect saturated concrete from the development of freezing and thawing induced pore pressures. Where one face of the concrete is exposed to air, then, under low hydraulic pressure, the evaporation process can reduce the level of saturation and, therefore, may reduce the freeze thaw deterioration.

For a typical concrete water tank structure the factors which determine the rate of water entering the pores of the concrete are the permeability of an internal coating, the presence of cracks and fissures in the wall and the pressure of the water acting on the wall. This last factor indicates that, if the other factors are uniform throughout the height of the tank, the degree of saturation (or perhaps more accurately the depth of saturation within the wall) increases with hydrostatic pressure and is at a maximum at the bottom of the tank. Thus, it is likely that the effects of evaporation are negated by the presence of a sufficiently high hydrostatic pressure at the bottom of high head tanks.

4.3.3 Air Voids

From the point of view of saturation, the presence of air voids represents an increase in porosity, and therefore, in theory, air entrained concrete should saturate more readily. However, as Verbeck (Ref. 14) points out in his discussion on concrete porosity, the presence of air voids has other beneficial factors such as increased workability and reduced segregation, which tend to improve the overall homogeneity and, consequently, decrease the permeability of the concrete compared to its non-air entrained counterpart.

The main function of entrained air bubbles in the cement paste is to prevent the development of internal hydraulic pressure. This pressure can be produced by the physical increase in the volume of ice compared to water. However, it can also result from the development of osmotic pressure due to the presence of alkali in the concrete.

The water entering the pore space of the concrete due to external hydraulic pressure will contain a considerable quantity of dissolved alkali. As the water in the larger pores drops below zero some of it freezes. Since it is only the water that freezes, the effect is to increase the concentration of the alkali at that pore space. At other regions, the space is smaller and therefore has not yet begun to freeze. This water, containing dissolved alkali of low concentration will tend to move to the zone of highly concentrated alkali contained in the larger, partially frozen pores, creating osmotic pressure.

The presence of entrained air greatly reduces the hydraulic pressures generated by either source. In the case of hydraulic pressure generated by physical expansion of the ice the air bubbles act as relieving reservoirs. Osmotic pressure is also relieved by the air bubbles since any water containing dissolved salts and exuding into the bubble will immediately freeze. If the bubble is not full, then, although the dissolved alkali is concentrated by the effect of freezing, no osmotic pressure can occur due to the discontinuity of the fluid.

Based on the details of these two mechanisms, namely, hydraulic pressure and osmosis, it can be seen that in addition to the importance normally given to the quality, size, and spacing of the entrained air, it is equally important that the air bubbles are not filled or even partially filled with moisture since the more moisture there is in the air bubble the less protection it can impart to the cement paste surrounding it.

As pointed out by T.C. Powers, another consideration with respect to air entrainment is the fact that some 60% to 70% of the concrete mass cannot be air entrained, namely the aggregate particles. Research has indicated that concrete, provided it is made with saturated aggregates and maintained at a high level of saturation prior to test, immediately fails a standard freeze-thaw test.(Ref. 13, and 17).

Under high hydrostatic pressure conditions, the presence of air voids has an effect on the permeability of the concrete, however, when these voids are filled with water they cannot prevent the development of pore pressures during freeze-thaw cycles and may aggravate the condition.

4.3.4 Affect of prestress on permeability

In addition to the factors previously described, placement conditions and the presence of stress affect the permeability (see Figure 4.2). Mills (Ref. 16) found that the permeability of concrete was greater in a direction parallel to bleeding and that the

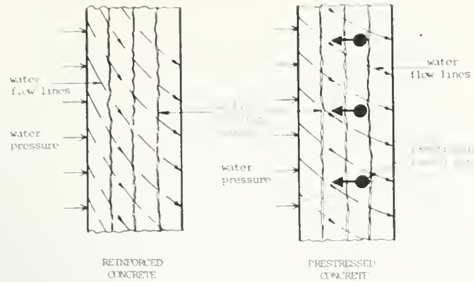


Figure 4.2 *Relationship between bleed water paths, water pressure flow and stressing force in water tank walls.*

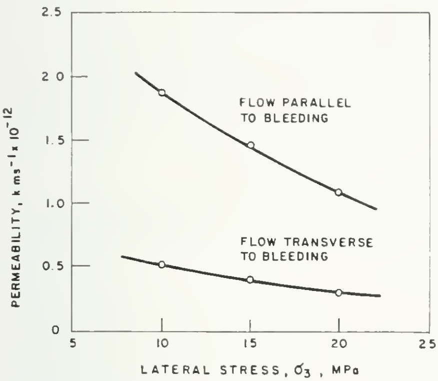


Figure 4.3 *Permeability as a function of confining stress (ref. 16)*

presence of lateral stress reduced permeability (see Figure 4.3). In concrete water tanks, therefore, water flows in the least permeable or horizontal direction and circumferential prestressing reduces the permeability. Typical values of permeability are given in Table 4/1. (ref. 19)

4.4 Action of Freezing Temperature on Concrete

The durability of concrete structures exposed to moisture and freezing temperatures has been a major concern to engineers for a long period of time.

Table 4/1
Typical values of permeability of concrete used in dams

Cement content	(lb/yd)	Water:cement ratio	Permeability 10^{-12} m/s
[illegible]	263	.69	8
[illegible]	254	0.74	24
[illegible]	235	0.75	35
[illegible]	376	0.46	28

Although microscopic mechanisms of frost action are not entirely understood, the major features of concrete behaviour under freezing temperatures are well established.

- Dry concrete is not affected by frost.

- Wet, highly saturated concrete expands when it freezes.

The expansion of concrete under sub-zero temperatures is frequently attributed to an increase in specific volume of water during ice formation (Ref. 17). Dilation of concrete is then attributed to high porewater pressures which can be created when excess water is forced out of freezing areas causing disruption of the internal structure and overall expansion of concrete. From the point of view of this mechanism, the degree of pore saturation is a major factor affecting concrete behaviour under freezing conditions.

Brittle porous materials such as concrete, rock and bricks all appear to exhibit similar behaviour under cyclic freezing and thawing in a saturated condition. Figure 4.4 is an example of laboratory freezing and thawing of clay bricks where the dimensional changes which can occur in these types of materials under one cycle of saturated freeze-thaw conditions are shown as a solid line. (Ref. 18).

As the specimen is cooled to 0°C, normal contraction occurs. At a temperature of approximately -4°C, the specimen undergoes rapid expansion. At around -12°C to -15°C, the maximum expansion is reached and normal thermal contraction is reinstated. During the thawing cycle the reverse pattern is observed. The specimen expands at a slightly greater rate than the rate of contraction until it reaches a maximum at 0°C. At that point, ice within the specimen melts and thawing shrinkage occurs. Depending on such factors as permeability and pore size, the amount of shrinkage is normally less than the amount of expansion and consequently produces a residual or non-elastic expansion after each cycle.

Also plotted on the right hand axis of figure 4.4 is the time history during freezing and thawing. The dashed line (time v's temperature) shows that the entire cycle is completed within 1 hour and that both during the freezing cycle, and also, during the thawing cycle, a delay occurs at -4°C and 0°C respectively, indicating that heat is being absorbed or given off and indicates that the water is changing state at these temperatures.

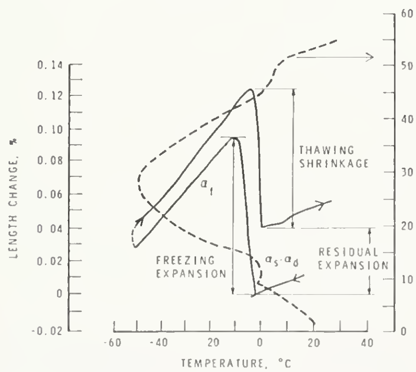
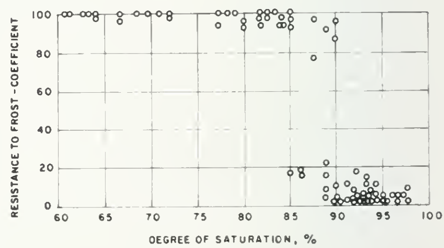


Figure 4.4 *Typical experimental length change results from thermomechanical measurement (ref. 18)*

Figure 4.5 illustrates that concrete with a low degree of pore saturation is immune to cyclic freezing whereas similar concretes which have a high degree of pore saturation become highly susceptible to frost damage. The critical saturation level at which dilation occurs appears to be about 85% or above.

Apart from the degree of saturation, the magnitude of dilation depends on a variety of factors such as strength of concrete, permeability of concrete, presence of air entrainment and type of aggregate. Figure 4.6 illustrates the influence of some primary factors on the magnitudes of dilation.



$$\text{RESISTANCE TO FROST COEFFICIENT} = \frac{\text{DILATION}}{\text{DILATION AT 100 \% SATURATION}} \times 100$$

Figure 4.5 *Correlation between degree of saturation and resistance to freezing and thawing (ref. 14)*

Research has established a relationship between the magnitude of residual expansion and the total expansion occurring during a freezing cycle. Figure 4.7 illustrates this relationship for bricks and indicates a high degree of correlation for both durable and non-durable bricks.

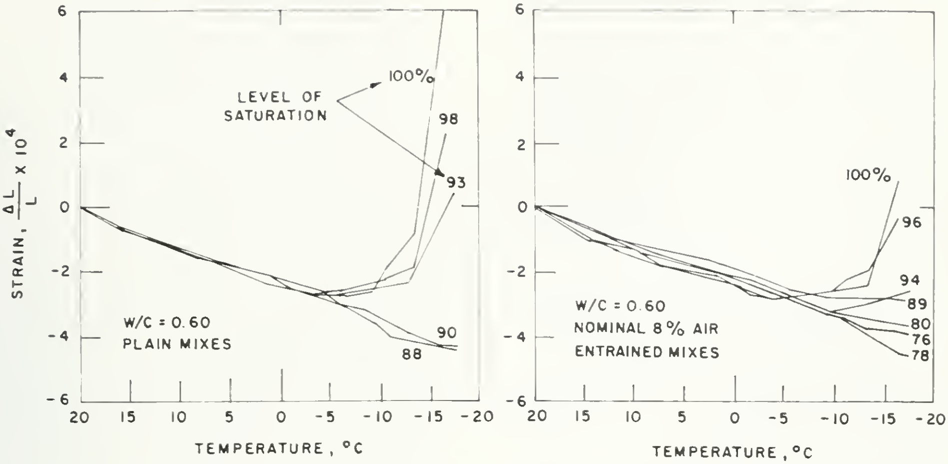


Figure 4.6 *Relationship between temperature and dilation for a plain and air entrained concrete at various levels of saturation (ref. 19)*

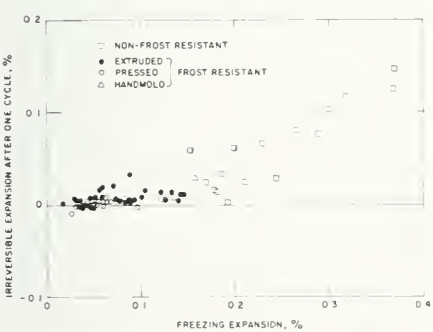


Figure 4.7 *Relationship between irreversible expansion and freezing expansion of frost resistant and non-frost resistant bricks (ref. 18)*

Investigations of this nature have also

Investigations of this nature have also been done for concrete. MacInnis and Whiting (Ref. 20) established that a similar pattern of dilation occurs during freezing of saturated concrete. A total of 112 samples cut from concrete paving slabs together with laboratory prepared specimens were tested. The entrained air contents of these samples ranged between 0 and 6.5%. The saturation procedure was one hour of applied vacuum (75 cm mercury), in a dry condition; one hour of vacuum in a submerged condition, and three days of saturation under atmospheric pressure. The samples were subsequently conditioned into 5 sets representing varying degrees of saturation, i.e., 100%, 88%, 72%, 47% and 30%, and subjected to 5 freeze- thaw cycles. Length change measurements were made during these cycles.

Figure 4.8 shows a typical length change pattern for a fully saturated specimen and indicates a similar behaviour pattern to that for saturated brick samples; dilation commences at about -4°C and on completion of the cycle a residual expansion remains. MacInnis reports that no dilation occurred on the other sets of specimens with a reduced degree of saturation. For the saturated specimens, the average dilation per cycle was 2.2×10^{-4} for the air-entrained concrete and 1.8×10^{-4} for plain concrete.

This research by MacInnis corroborates other researchers who report that there is a critical degree of saturation required for dilation to occur. Additionally, it suggests that under the saturation procedure used the entrained air voids may be filled, rendering them ineffective against frost action. It is argued that this is also likely to occur under hydrostatic pressure; this would be consistent with the deterioration of concrete water tanks in their lower section regardless of the presence or magnitude of entrained air.

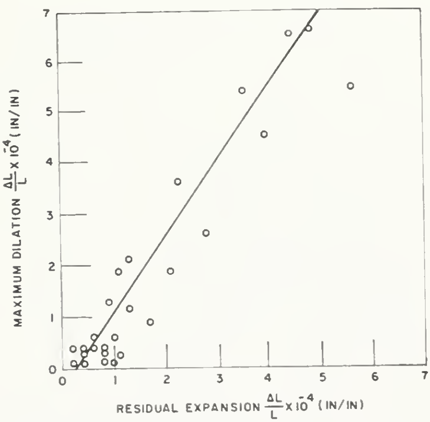


Figure 4.8 *Maximum dilations v's residual expansions after one freeze - thaw cycle for "fully" saturated specimens (ref. 22)*

Some investigators have carried out research on the physical changes that occur at a microscopic level during the freezing and thawing cycle. Using hardened pastes of plaster-of-paris, Marusin (Ref. 23) demonstrated that a single freeze-thaw cycle was accompanied by expansion (dilation), an increase of water absorption, a decrease in strength and an increase in the average pore size as shown on Figure 4.9

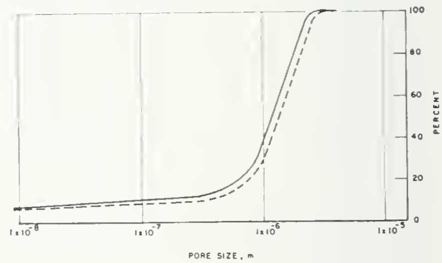


Figure 4.9 *Pore size enlargement after a single freeze - thaw cycle*

A schematic diagram of the sequence of events is given in Figure 4.10 and demonstrates that the residual expansion results in a reduction in the degree of saturation.If no additional water is introduced into the concrete it seems reasonable to assume that there will be little additional dilation with further freeze-thaw cycles since the concrete now has a reduced degree of saturation. Small additional expansion may occur until the concrete is below the critical saturation level. However if more water is introduced into the concrete, its original degree of saturation may be achieved and therefore another freeze-thaw cycle will produce more dilation. From Figure 4.10 it can be seen that the volume of water required to restore the original level of saturation is equal to the volume of residual expansion.

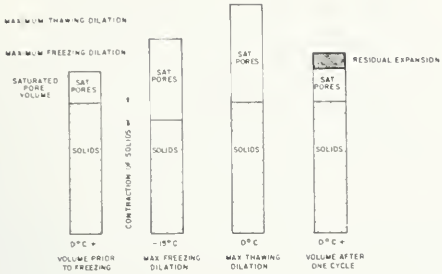


Figure 4.10 *Schematic diagram of a freeze - thaw cycle*

With respect to damage caused by freezing and thawing, this concept distinguishes between effective and non-effective cycles. Some freeze-thaw cycles will not be effective in contributing to damage if concrete previously dilated is not re-saturated prior to the onset of the next freezing cycle. Although research has demonstrated that freezing and thawing expansion can occur in one hour (Figure 4.4) the governing factor in the continued effectiveness of freeze-thaw cycles to cause damage is the rate of water supply during the thaw period.

4.4.1 Summary

From the data on the physical changes occurring during a single freeze-thaw cycle of saturated concrete, it can be concluded that

- Freezing expansion occurs at approximately -4°C.

- Freezing expansion ceases at approximately -15°C.

- Additional expansion occurs during the thawing cycle.

- The entire cycle can occur within a period of 1 hour.

- A residual expansion remains after completion of one cycle and is directly related to the total expansion.

- There is a critical degree of saturation at approximately 90% above which significant dilation occurs.

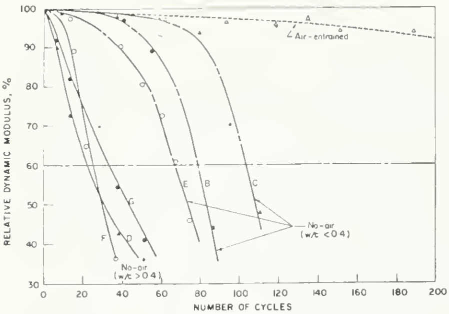
- A freezing and thawing cycle permanently enlarges the size of the individual pores.

4.5 Standard Freeze-Thaw Tests

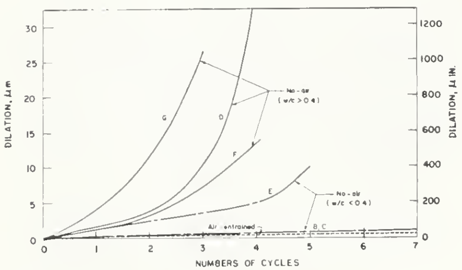
To illustrate the failure mechanism further, it is worthwhile considering the two standard ASTM procedures for testing the durability of concrete under freezing and thawing cycles, and their results.

In the two Standard ASTM procedures (designations C671 and C666) saturated concrete is subjected to freeze-thaw cycles.
The major difference between the two tests is the frequency of freezing and thawing. In the ASTM procedure, C671, concrete is kept in water for two weeks between cycles, while in procedure C666 concrete is not allowed to "rest" and freeze- thaw cycles follow one another within a 2 to 4 hour period. Although damage is assessed differently in these procedures, in the slow cycle test method where specimens are allowed to "rest" in water, the concrete is generally considered damaged after far fewer cycles than those in the rapid method; typically the comparative ranges are 1 to 10 cycles and 40 to 300 cycles, respectively (Figure 4.11a and b).

The point is that the "rest" in water is detrimental because additional water can enter the pores at this stage. This is precisely the condition of the saturated concrete walls of a water tank subjected to freezing and thawing.



a) *Relative dynamic modulus (ASTM C666)*



b) *Dilation - cycle (ASTM C671)*

Figure 4.11 *Typical ASTM freeze-thaw results*

The tests described reflect two possible types of concrete exposure to frost and water:

- conditions in which the time between freeze-thaw cycles is insufficient to re-saturate concrete (ASTM designation C666 - "Resistance to Rapid Freezing and Thawing")

- conditions in which re-saturation of concrete occurs between freeze-thaw cycles (ASTM designation C671 - "Critical Dilation of Concrete Specimens Subjected to Freezing")

In both cases, concrete can be damaged, but in the condition of re-saturation, damage occurs in a smaller number of cycles.

The reason for the drastic effect of "rest" periods in water is that after reducing its degree of saturation in a single freezing cycle the concrete then absorbs more water and becomes re-saturated. The initial loss of saturation occurs due to dilation which is microscopically translated into an increase in average pore size.

For the next cycle to be as damaging as the previous one, this extra volume of pores must be filled with water. Two weeks "rest" between cycles appears to be sufficient to re-saturate normal concrete under atmospheric pressure but the time is dependent on the coefficient of permeability.

This evidence of the difference between standard test methods supports the concept that re-saturation of the concrete during thawing is the critical factor which affects the rate and extent of damage due to frost action, permeabilities being constant.

It can be understood that thawing during winter conditions will occur more frequently in the walls of water tanks between the south-east and south-west quadrants where the direct rays of the winter sun are sufficiently warm to thaw the concrete after freezing at night. This concept suggests that if re-saturation is possible during a thaw, then rapid and severe damage could occur. It is therefore important to examine the time taken to re-saturate dilated concrete so that the rate of deterioration of concrete tanks can be estimated.

4.6 Rate of Re-Saturating Dilated Concrete

The re-saturation period is the time during which the volume of water re-fills that portion of the concrete pores equal to the volume increase due to the residual expansion.

To provide a simple model, the rate of water influx into concrete pores under a given head was assessed using D'Arcy flow assumptions. Figure 4.12 illustrates re-saturation rates for typical permeabilities of good to excellent quality concrete (2 to 8 x 10^{-12} m/s)

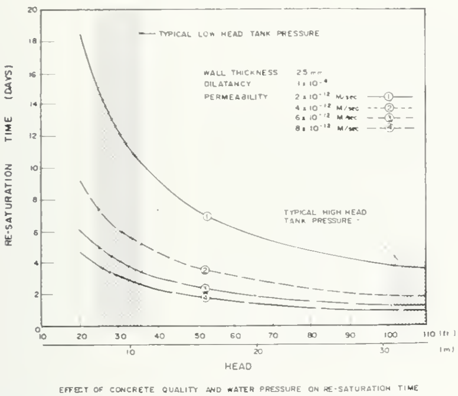


Figure 4.12 *Effect of concrete quality and water pressure on re-saturation time*

It can be seen that at a pressure head of 33 m (100 ft.) excellent quality concrete takes approximately 4 days to re-saturate but good quality concrete can re-saturate within 1 day.

This illustrates the reason for the divergence in the rate of deterioration with respect to concrete quality. However, it is clear that concrete with permeabilities below the illustrated range will deteriorate at the same rate for the same pressure head since they can all be re-saturated within the period prior to the next freezing cycle. Conversely concrete with lower permeability will deteriorate at a slower rate. It should be noted that the permeabilities used for this illustration are representative of good to excellent quality concrete.

Another point illustrated in the graph is that tanks with a low pressure head and built with good quality concrete will not re-saturate daily and therefore will be less affected by freeze-thaw cycles.

Damage to concrete during freezing and thawing results in general deterioration of its mechanical properties i.e. reduction of strength and stiffness. This degradation is conventionally viewed as ultimately leading to spalling, frequently observed on the outer layers of the affected region. The phenomenon of surface spalling, easily detectable visually, has masked other mechanical and more serious consequences of freezing and thawing in concrete water tanks in Ontario which cannot be so readily observed.

The next section examines development of internal stresses related to differential movements created by dilation of concrete.

4.7 Stresses in Concrete Due to Frost Induced Expansion

After each freezing cycle a saturated specimen dilates resulting in a residual expansion. Continuation of this process results in breakdown of the matrix and general disintegration of the concrete as the pore walls expand and ultimately fail.

In practice however, a concrete member can be fully saturated and only frozen to a limited depth. In this circumstance, part of the concrete member is dilating while the remainder is unaffected by frost. Under this condition, the dilating concrete is restrained and will produce reaction forces, the magnitude being dependent on the geometry of its constraints and on the magnitude of unconstrained dilation.

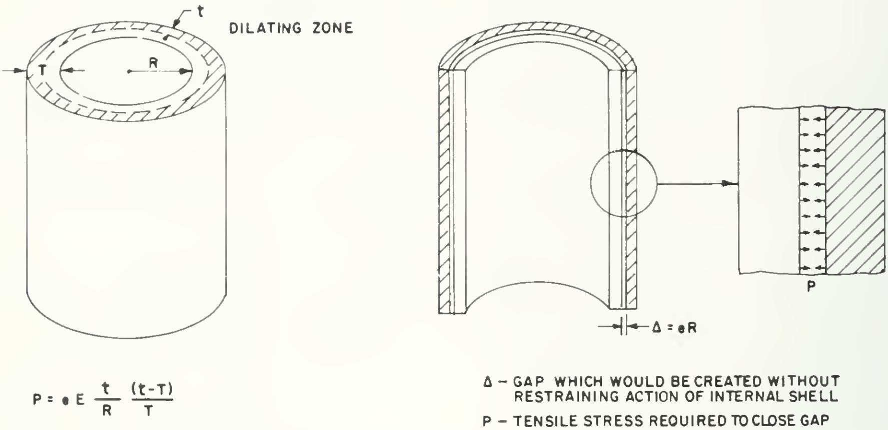


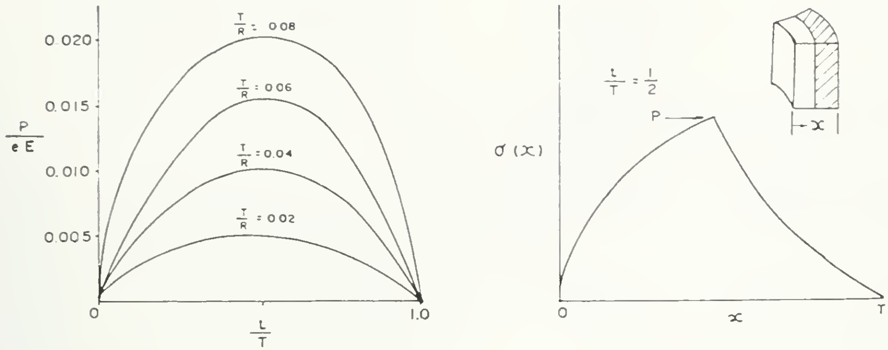
Figure 4.13 *Theoretical model of tank stress development*

In mechanical terms, the resulting stresses are of the same nature as thermal stresses induced by temperature increase. An analogous situation occurs when a thick epoxy resin topping is applied over a concrete base. On heating and cooling, the epoxy resin, having a different thermal coefficient compared to concrete expands and contracts more, resulting in reaction forces which break the bond between the two layers. Since the magnitude of concrete dilation is roughly equivalent to a thermal expansion in the range of 10°C - 30°C, stresses induced by freezing and thawing cycles can be significant.

Due to the permanent and irreversible nature of the non- elastic volumetric changes caused by frost action, stresses created in constrained concrete will not be relieved immediately after the cycle (as is the case in a temperature cycle). Some slow relief due to creep may be expected, but if freeze-thaw cycles follow closely one after another and the time between cycles is sufficient to permit re-saturation of the concrete, stress accumulation will occur eventually reaching the rupture limit of concrete. Thus, frost related stresses can be much more dangerous than temperature related stresses.

4.7.1 Tensile Stresses in Tank Walls

Stresses in dilating concrete can be induced not only by the presence of localized external constraints but also due to the restraining action of different parts of a structure.



a) Maximum radial stress v's position of dilating zone

b) Distribution of radial stress along the wall

Figure 4.14 *Distribution of stress in tank wall*

Consider a situation when the outside part of a concrete tank wall dilates due to freezing, while the internal part of the wall is not affected by frost (Figure 4.13). The dilating outer shell will tend to increase in diameter and pull away from the non-dilating part of the wall. This tendency to differential movement will result in tensile stress occurring in the entire wall. The tensile stresses developed can be calculated as follows:

$$P = eE\,\frac{t\,(t \quad - T)}{R \quad T}$$

where e = strain due to dilatancy

E = Young's modulus of concrete

T = wall thickness

R = tank radius

t = thickness of dilating concrete

$\sigma_{(x)}$ = radial stress

P = $\sigma(x)$ maximum

Calculations for various ratios of tank radius to wall thickness and depth of freezing zone are given in Figure 4.14a and indicate the effect of tank radius on the development of the induced stresses. Figure 4.14b illustrates that the maximum stress occurs at the interface of the dilating and non-dilating zones when this interface is at the centre of the wall. In this case, since t = 0.5T, tensile stress (P) can be calculated as follows:

$$P = \frac{eE}{4} \cdot \frac{T}{R}$$

Using this simplified formula, with typical parameters for good quality concrete, stress occurring during one freeze- thaw cycle of a saturated concrete tank with freezing to half the wall thickness is as follows:-

Strain due to dilatancy (e). . . . . 1×10^{-4}

Young's modulus of concrete (E). . . . . 28 GPa (4×10^6 psi)

Tank radius (R) . . . . 5m (15 ft.)

Wall thickness (T) . . . 25 cm (10 in.)

Average modulus of rupture of concrete. . . 245 kPa (35 psi)

Induced tensile stress due to single cycle . . . 35 kPa (5 psi)

The apparent modest stress induced in one cycle of stress application can nevertheless accumulate successively under repeated cycles of freezing and thawing until the modulus of rupture is reached. The mechanism attains a maximum rate under the following conditions:-

- re-saturation occurs in the period prior to the next freezing cycle; and

- insufficient time is available between freezing cycles to permit stress relaxations due to creep.

Compare the laboratory induced freeze-thaw cycle (Figure 4.4) with the records from the tank monitoring study. They are quite similar. It can be seen that typically, on a daily basis, the temperature of the south side of the tank is reduced below -4°C, the temperature at which dilation initiates, then continues to reduce towards -15°C the temperature at which maximum freezing dilation occurs. It peaks above 0°C where thawing and residual expansion is accomplished.

Based on the temperature data from the ROCKWOOD experiment, it seems reasonable to assume that cycles of alternate freezing and thawing would occur on a daily basis during January and February of each year. With this information, together with published data and the previously given formula for tensile stress development, a theoretical model of stress accumulation was constructed.

4.7.2 Model of Tensile Stress Accumulation

To assess the rate of tensile stress build-up under cycles of freezing and thawing, a mathematical model incorporating the following environmental assumptions was developed.

i) Cycles of freezing and thawing follow daily for two months every year, i.e. 60 daily cycles of freezing and thawing per annum.

ii) In each cycle, frost penetrates to the middle of the tank wall.

iii) Concrete is exposed to water under pressure continuously applied to the inside face of the tank wall.

iv) Dilation of concrete takes place only when concrete becomes re-saturated in the period between cycles, i.e. cycles occurring in the period when re-saturation is in progress are "inactive".

v) Tensile stress relaxation occurs continuously. Data on stress relaxation in concrete are taken from Ref. 21.

Computations were performed using direct computer simulation of time events. The model assumed that the tank wall was initially saturated and that dilation would occur in the first cycle. Using D'Arcy's law, the time taken to re-saturate the additional volume of pores created by previous expansion was calculated. If re-saturation was completed in the period between two consecutive cycles then further dilation was considered to occur causing extra tensile stress in the tank wall.

If the time between cycles was insufficient to re-saturate the concrete then effective freezing cycles were omitted until such time as the water in the concrete could be replenished.

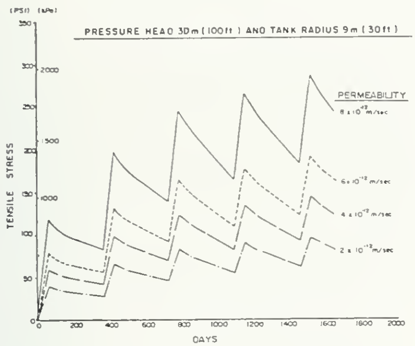


Figure 4.15 *Permeability and tensile stress for 30m (100 ft.) head*

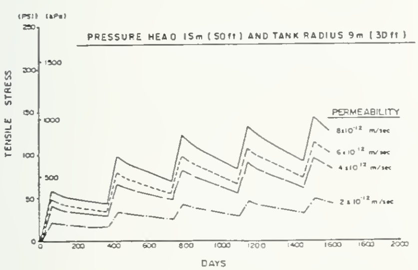


Figure 4.16 *Permeability and tensile stress for 15m (50 ft.) head*

The resultant stress was accumulated for a period of 60 cycles. At that point it was assumed that no further freezing and thawing cycles would occur and the accumulated stresses would be gradually reduced due to creep for the remaining part of that year. This process was continued until the concrete had built up sufficient tensile stress known to result in fracture of the concrete.

The results of the simulation are given in Figures 4.15 and 4.16 for various concrete permeabilities at pressure heads of 30 m (100 ft.) and 15 m (50 ft.), respectively, and illustrate their influence over a five year period. In all cases the following parameters were used:

- strain due to dilation . . . 1×10^{-4}
- wall thickness . . . . . . 25 cm (10 in)
- tank diameter. . . . . . . 9 m (30 ft.)

The overall trend of the graphs demonstrate the steady accumulation of stresses through the winter period and their partial relaxation during the rest of the year due to creep resulting in a "ratchet" effect. Also illustrated is the dramatic influence of the internal water pressure on stress accumulation, since at high pressure heads, practically every freeze-thaw cycle produces dilation and associated stress. In low internal pressure tanks, the stress accumulation is not nearly so acute because the re-saturation rate is less than the rate of freezing and thawing.

These effects relate to the geometry of the tank, however, the graphs also illustrate that the permeability of the concrete is one of the most important governing factors. It should be noted that as previously described the permeabilities were deliberately selected as representing good to excellent quality concrete.

It can be seen that radial tensile stress in high pressure standpipes can accumulate dangerously over a five year period to a level of tensile stress which would fracture concrete.

It should be noted that due to the internal water pressure, this radial tensile stress is superimposed on a compressive stress of about 140 kPa (20psi) which exists in the middle of a tank wall. The accumulation of tensile

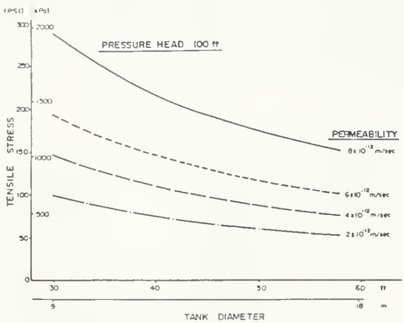


Figure 4.17 *Five year accumulated tensile stress related to tank diameter for 30m (100 ft.) head.*

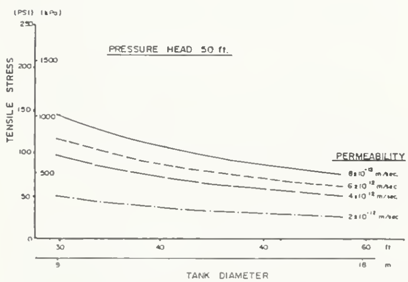


Figure 4.18 *Five year accumulated tensile stress related to tank diameter for 15m (50 ft.) head.*

stress with time will surpass this compressive stress and the net tensile stress can reach a level which is sufficient to cause the fracture of concrete.

Perhaps one surprising feature of the model is the influence of the tank radius on the accumulation of stress. For example, under a 30m (100 ft.) head of water an accumulated stress of 2.1 MPa (300 psi) would occur in five years in a 10 m (30 ft.) diameter tank whereas only 1.1 MPa (160 psi) tensile stress would accumulate under the same head in a tank of 20 m (60 ft.) diameter. The relationship between tank diameter and tensile stress for various permeabilities is illustrated in Figures 4.17 and 4.18.

This particular influence, perhaps masked by other factors, has not been noted in practice, however, the model corroborates many of the observations previously mentioned. For example the high pressure head of a standpipe results in rapid deterioration. Also, construction defects which allow the water access to the external surface effectively amounts to a reduction in the wall thickness and hence reduced flow path resulting in faster re-saturation. Under these circumstances a localized section of a low pressure tank could behave like a high pressure tank since the reduced flow path allows the remaining concrete to re-saturate more rapidly. Another feature of the model is that all tanks will suffer some stress accumulation regardless of the quality of the concrete because the permeability of good quality concrete is insufficient by itself to prevent re-saturation, particularly under high pressure heads. This feature also highlights the importance of the internal coating, which, if sufficiently impermeable, will reduce the re-saturation rates.

One major difference which has been observed in the field is that reinforced concrete standpipes deteriorate by delamination more rapidly than prestressed concrete standpipes. To understand the possible reasons for this it is necessary to examine the distribution of the stresses as they accumulate in the wall of a tank.

4.7.3 Distribution of Stresses

Figure 4.19 illustrates typical forces acting on a reinforced concrete wall when there is stress accumulation due to dilation of saturated concrete. In the example given, half the wall thickness is subjected to freezing and thawing and is restrained by the remaining half of the wall section.

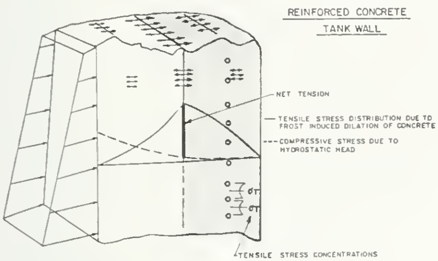


Figure 4.19 *Distribution of stresses in tank wall section under increasing linear water pressure.*

As can be seen there is a compressive stress acting on the wall due to the hydrostatic pressure on the internal face of the wall. This pressure is maximum at the internal concrete surface and is zero at the external surface. Due to the restraint provided by the internal half of the wall section,reaction forces are developed between the dilating and non-dilating zones. The distribution of stress throughout the wall due to this reaction can be calculated as described previously.

These effects represent a general picture of stress accumulation and do not consider the presence of reinforcing steel in the area of tensile stress accumulation. The presence of reinforcing steel results in local stress re-distribution which may eventually lead to tensile stress concentrations if the bond between the reinforcing steel and the concrete is destroyed.

The example on Figure 4.20 is designed to illustrate the possibility of reinforcing steel debonding when saturated concrete is dilating due to cyclical freezing and thawing. Tensile stresses in the concrete-steel interface are created by the tendency of the dilating material to move away from the reinforcement. Concrete-steel bond will restrain the movement of dilating concrete and therefore tensile stresses will develop.

Calculations show that the interface tensile stress induced in one freezing cycle is 1.2 MPa (170 psi). This stress is based on a dilatant strain of 1.0 x 10^{-4}. A strain twice this value will result in debonding of the reinforcement in a single freeze-thaw cycle. (see appendix B)

This phenomenon has been observed in cores taken in the walls of reinforced concrete tanks as shown in Photo 4-1. The debonding of the reinforcing steel will effectively create cylindrical openings in concrete and will induce stress concentrations in the matrix between the reinforcing steel. Depending on the diameter of the steel and the spacing between bars, the stress concentration can be between 2 to 4 times the general level of stress.

Photo 4-1 *Dilation of concrete and subsequent debonding of reinforcing steel*

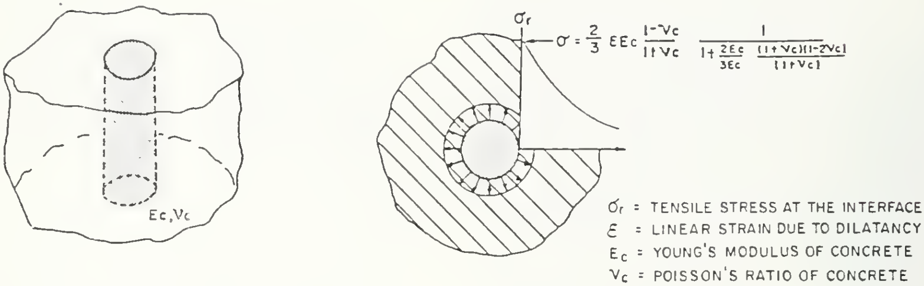


Figure 4.20 *Illustration of model used to examine debonding of reinforcing steel*

Consequently, although stress accumulation may be greater near the centre of the wall, fracture at the level of the reinforcing steel is more likely.

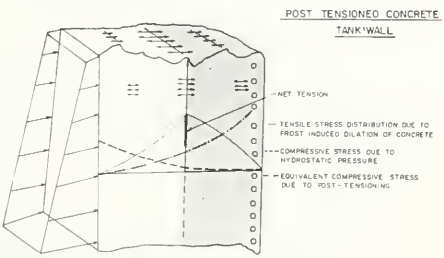


Figure 4.21 *Distribution of stresses in post-tensioned tank wall section*

Figure 4.21 is a diagram of a similar situation in a post-tensioned tank. In this case, significant radial compressive stress caused by post-tensioning offsets the magnitude of tensile stresses accumulating due to freezing and thawing. The post-tensioned tank therefore would require considerably more freeze-thaw cycles to accumulate sufficient net tension to fracture concrete at the centre.

The above theoretical models appear to support the field observations that reinforced concrete standpipes are more susceptible to delamination due to freezing and thawing than are post-tensioned tanks where the radial compression of the post-tensioning wires or tendons reduce or eliminate the tendency to cause tensile fracture. Post-tensioning may also be beneficial since it produces a lateral stress which, as previously mentioned, reduces the permeability of the concrete (Ref. 16).

4.7.4 Summary

In the above analyses, concrete tank walls were assumed to be uniformly saturated and subjected to uniform freezing temperatures. This rather simplistic set of conditions appears to be adequate to explain general trends in the deterioration of concrete tanks as follows:-

- Standpipes deteriorate more rapidly than elevated or ground tanks since the internal pressure in standpipes is sufficient to re-saturate concrete on a daily basis.

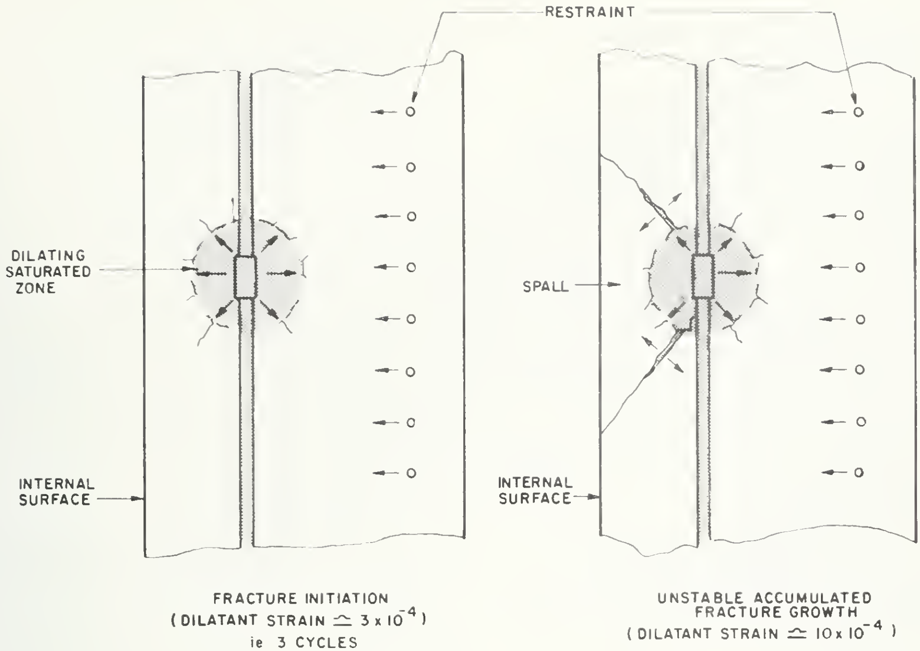


Figure 4.22 *Formation of jack-rod spalls*

- Post-tensioned tanks do not externally delaminate to the same extent as reinforced concrete standpipes due to the presence of radial compression.

- Under high internal pressure conditions, normal high quality concrete may not be sufficiently impermeable to eliminate effective freeze-thaw cycles by substantially reducing the rate of re-saturation.

- Defects and cracking in tanks can reduce the effective wall thickness and allow a rate of re-saturation which may induce delamination even in low pressure tanks.

4.8 Spot Saturation

In the previous analyses, concrete tank walls were assumed to be uniformly saturated and subjected to uniform freezing temperatures. This rather idealized set of conditions is adequate to analyse general trends in tensile stress build-up under freezing cycles, but is unable to predict random patterns of concrete deterioration accompanied by spalling.

In actual service conditions, concrete tank walls may not be saturated in all areas, due to, for example, the partial presence of protective coating, local variations in quality, permeability, and porosity or non uniform exposure to external moisture. In practice, therefore, spot saturation is the rule rather than the exception.

The presence of a saturated zone dilating under freezing conditions can result in a considerable internal stress due to the restraining action of the non-saturated and, therefore, non-dilating area.

Depending on the geometry of the dilating zone and its proximity to the concrete surface, dilation can be accompanied by tensile stresses resulting in spalls. Due to an almost unlimited variety of geometrical possibilities and the difficulties in estimating the resulting stresses, the precise mechanisms of spalling cannot be readily identified.

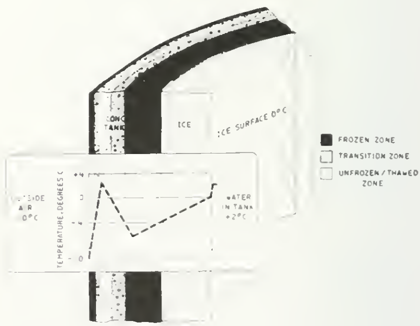
One case of increased restraint that merits detailed explanation, is the case where a regular pattern of internal spalling invariably occurs at the coupled joints between jack-rod sections.

4.8.1 Jack-Rod Spalls

As previously described, jack-rod spalls occur where the hollow rods used for slip forming have not been filled with grout. Inspection showed that typically 20 to 30 spalls occurred in each of the tanks constructed using this method and they invariably occurred at the jack-rod junction (see Figure 4.22).
An attempt has been made to explain spalls by ice formation within the hollow jack-rods. Although this mechanism could account for some observed vertical splitting along the length of the tube, it could not explain the regular occurrence of spalls at the couplings.

The observed pattern of spalls is attributed to saturation of concrete in the area of the jack-rod couplings. It is reasonable to assume that water would leak from the couplings and would create a zone of saturated concrete which would dilate on freezing. The concrete outside the zone would provide a restraint to dilation and tensile stresses outside of the dilating zone would be developed. (see Figure 4.22).

a) *Thermal profile produced by frozen internal wall section*



(b)*Thawed zone trapped between two frozen zones*

Figure 4.23 *Schematic development of thawed zone in wall*

Calculations show that a dilating zone of about 25 mm radius (1 in.) will initiate local fracture when the dilating strain is

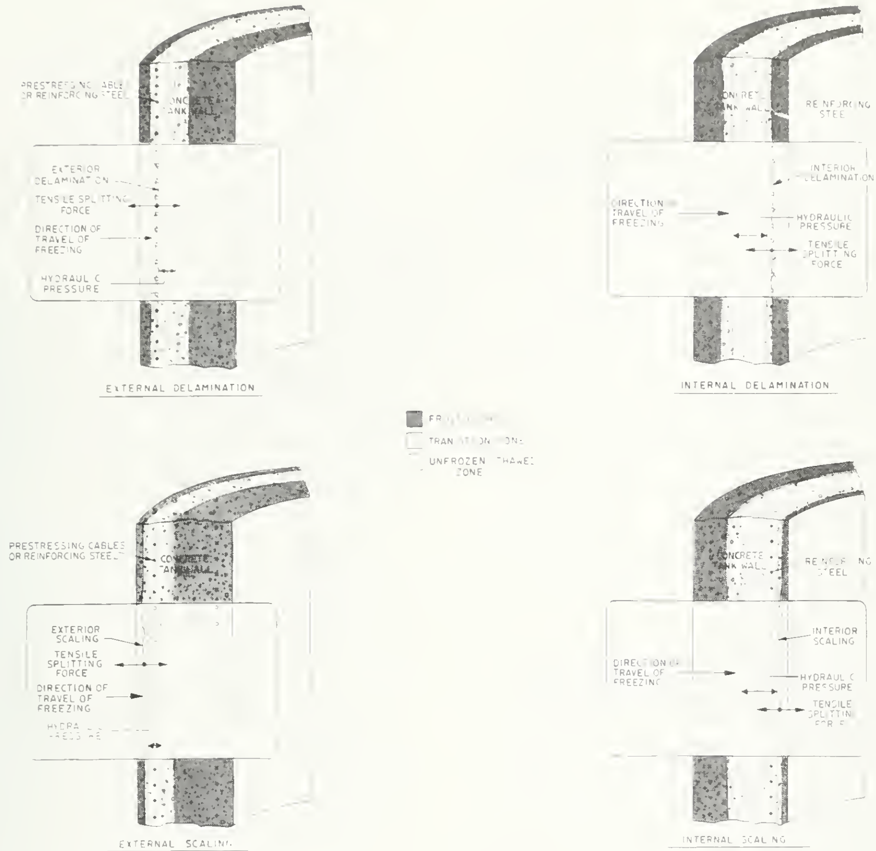


Figure 4.24 *Nature of wall damage at location of unfrozen zone*

about equal to the limiting tensile strain of concrete (3 x 10^{-4}). Accumulated dilatancy of this order of magnitude can be reached in several freeze-thaw cycles (typically 3 cycles where the dilatancy strain is 1 x 10^{-4}). Subsequent dilation cycles will result in fracture propagation to the concrete surface. Calculations show that this would occur when an accumulated dilatancy of about 10 x 10^{-4} has been achieved in the area near the coupling.

4.9 Hydraulic Pressure "Sandwich"

Another mechanism can be envisaged which could result in the generation of considerable hydraulic pressure and, therefore, produce the types of deterioration witnessed in many of the concrete water tanks. This mechanism requires the formation of an ice ring inside the tank which allows the concrete wall to freeze throughout its thickness. As previously discussed, this usually occurs during mid-winter to spring in the majority of tanks. Since this mechanism may well be additive to the previously discussed dilation mechanism, it may help explain why an increase in the extent of deterioration is often more noticeable during spring than during the autumn.

As stated in Section 3.4, when saturated concrete is frozen, one of the main effects is to reduce the effective permeability. Subsequent freezing and thawing cycles occurring at the exterior of the tank wall can push a zone of unfrozen water ahead of the freezing front which may be accommodated until it reaches the frozen layer. At that point, due to the reduced permeability, a hydraulic pressure may be generated which could be sufficient to rupture the concrete matrix. This mechanism has been duplicated in the laboratory by Adkins.'(ref. 25)

Figures 4.23a and b, are sketches representing thermal conditions which can result in an unfrozen centre zone. The sketch in Figure 4.23a indicates that the ambient temperature has been below zero for a considerable period resulting in the formation of interior ice in the tank and frozen concrete throughout the thickness of the wall. This is followed by a warm period which partially thaws the outer part of the wall. The cycle is completed by the ambient temperature returning to below freezing (in this case -10°C) resulting in a thawed centre region, an inward moving freezing front, and an impermeable interior wall section.

The increasing hydraulic pressure of the pore water trapped in the decreasing space between two frozen layers, can cause horizontal tensile forces in a tank wall. The location of the unfrozen water layer will determine the plane of failure and delamination splitting which will be manifested as external or internal scaling or delamination.

Based on this mechanism, the type of visible damage occurring depends on the extent of the thawing period compared to the freezing period, since that determines the location of the unfrozen central section. Figure 4.24 illustrates the nature of the damage which could occur depending on the position and geometry of the unfrozen section at the onset of the freezing cycle.

4.10 Conclusions and Recommendations

Based on the review of historical and current practices, field observations, and the concepts and mathematical models described, the following conclusions were presented to the task force and the Ministry.

- The detailed surveys confirmed the initial work of W.M. Slater & Associates Inc. that the deterioration of the Province's concrete water tanks was widespread.

- Tank inspections showed that where internal coatings existed they could not provide the required reduction in permeability to prevent saturation and consequent deterioration due to freezing and thawing cycles.

- Due to their high internal hydraulic pressure, standpipe structures, and in particular reinforced concrete standpipes, are prone to deterioration by delamination.

- Small defects which are perhaps harmless in other types of structures, allow ingress of water into tank walls and can initiate delamination even in elevated and ground tanks subjected to freezing conditions.

From the above conclusions it was recommended that:

1. Methods of providing insulation to inhibit the effect of freeze-thaw cycles should be studied.

2. Waterproofing barrier systems for potable water storage tanks should be studied.

3. Condition surveys and quality control procedures should be initiated as they are of the utmost importance during tank rehabilitation. They provide detailed information on the repairs required and ensure that necessary quality of materials and workmanship is carried out.

These recommendations were accepted and acted upon by the Ministry. Item 1 and 2 form the basis of other reports referenced on page (i). Condition surveys, repair methods and quality control are discussed in the next sections.

5.0 REPAIR AND REHABILITATION OF CONCRETE WATER TANKS IN ONTARIO

5.1 General

Repair is defined as the restoration of defects in a structure to the intended original state. Rehabilitation is defined as repair and upgrading to a new desired state, such as designing for increased seismic forces, and insulation and waterproofing requirements. The tank repair and rehabilitation programme was more complex and difficult than the investigation and diagnostic stage, because of the following factors:-

- Exposure to height (safety)
- Harsh environment
- Limited construction season
- Limitation of suitable contractors, specialists, materials
- Need for temporary storage and pumping to ensure continued operation of a municipality's water supply
- Limitations of condition survey techniques
- Project management of short term contracts
- Short lead time for the development of methods of repair
- Need for development of efficient and safe work stages

Many of the tanks inspected in the period 1981 to 1982 had already been unsuccessfully repaired, some of them several times before the rehabilitation programme started. Reasons for the ineffective repairs arose from the lack of understanding of the real causes of failure. The main causes of failure were an appreciation of the limitations of the materials used, the need for careful surface preparation and the provision of proper environmental conditions. Leakproofing cold joints and deteriorated concrete at cracks formed during slip forming is difficult and costly; using bonded coatings alone is not always adequate without removal and replacement of the deteriorated concrete before surfacing and coating. Complete reconstruction of the bottom of walls has been necessary at locations where water has flowed past or over water-stops and where it has frozen on the inside.

5.2 Design of Repairs

5.2.1 General

Remedial measures can be put into four categories as follows:-

- Upgrading of structural integrity where necessary;
- Repair of deteriorated concrete;
- Containment of water (leakage);
- Prevention of saturation and freezing.

It should be noted that the last item regarding waterproof coatings and ice protection is not fully discussed in this report since it forms the basis of other reports.

5.2.2 Structural Evaluation

.1 Loading

Loads may be placed in two categories, namely applied loads and environmental loads as follows:-

Applied Loads	Environmental Loads
• water pressure (a) • post-tensioning (a) • wind (b) • seismic (b)	• thermal (b) • shrinkage (b) • creep (b) • ice (c)

The loads may be described as a) well-defined b) defined, or c) ill-defined. All these factors, if not handled with care, result in stress accumulations causing cracking of concrete, deterioration of waterproofing and severe leakage rendering the tank unserviceable in a relatively short time.

Water pressure is directly related to the head of water in the tank, while the specified prestress is applied at the construction stage. Thus both these loadings are well- defined. Wind and seismic loads at specified locations in the Province of Ontario are defined in relevant codes (Ref. 26) and thus the levels of such loads for design purposes are defined.

Present design approaches seem to treat environmental loading on water tanks in a somewhat cursory manner. Some aspects are either underestimated or are not considered at all. A substantial amount of work carried out in New Zealand (Ref. 27) on thermal effects shows that these are generally underestimated due to the neglect of direct solar radiation. Thermal gradients up to 30°C have been recorded. Shrinkage effects are analogous to thermal effects and can be treated using an equivalent temperature gradient (ETG) approach. The ETG is a function of the wall thickness and the boundary conditions of the walls. Creep is generally beneficial in that it tends to reduce displacement induced stress. However, it also reduces the effective prestress in a prestressed tank. While such loadings are defined in codes, the levels to be considered in design may be underestimated.

Ice loading in tanks is not considered in North American approaches but is considered in some other countries (Refs. 28, and 29). Most past work on ice pressures has been related to ice sheets on lakes and reservoirs and little information on its effects in water tanks is available. Evidence of ice formations in tanks in Ontario exists (Ref. 2). These ice formations may be in the form of a plate near the top of the tank, a cylinder on the inside walls, a slab built up at the domed floor of a tank or in the cracks in the wall of the tank.

Photo 5-1 *Internal wall ice at bottom of tank after demolition in late spring*

Loading from ice formations is most critical when the ice heats up, since the coefficient of thermal expansion of ice is approximately five times that of concrete. Work in Finland (Ref. 28) shows, that the rate of heating is a significant parameter, and that pressures of up to 250 kPa (35 psi) can be generated in a tank by a plate of ice which warms from -20°C to 0°C in four hours. Cracking of an ice plate due to differential temperature and subsequent freezing forming ice in the resultant cracks can develop significant pressures 700 kPa (100 psi) as shown by work in Sweden (Ref. 29).

Environmental loading on elevated water tanks has been studied at Queen's University with particular emphasis on the effects of ice. This work has been reported by T.I. Campbell and W.L. Kong in "Ice Loading in Elevated Water Tanks" dated April 1986.

.2 Analysis

Methods of analysis may be classified as (a) simple, (b) adequate, and (c) refined. In a simple analysis a tank may be modelled as a thin-walled cylinder, while in an adequate analysis use can be made of charts (Refs. 12, 30, 31 and 32) for the computation of the stress resultants. A refined analysis can be made using classical approaches (Ref. 33) or a finite element model (Ref. 34). All methods have their use but should be used with caution.

Reinforced concrete tanks have usually been designed by taking care of the straight hoop forces with the horizontal ring reinforcing. This arrangement, for water load only, would be suitable if the wall at the bottom of the tank were as free to expand, as it is for most of the tank's height.

However, in many cases the tank wall is rigidly connected to a heavy slab or foundation mat. This restraining effect completely changes the stress pattern at this location. The hoop forces are reduced to near zero and their role is replaced by significant vertical bending moments and horizontal shear forces. These stresses rapidly taper off further up the wall and in most cases, for all practical purposes, can be neglected except in the vicinity of the wall slab junction where their influence is significant. The problem can further be complicated by inadequate vertical reinforcing.

In uninsulated water tanks in Ontario with large height to diameter ratios and cold surface water sources, ice formation inside the tanks is a common occurrence. The influence of ice pressure, especially at rigidly restrained structural modes must be investigated.

Additional problems can be expected due to temperature differences through the concrete wall thickness. These differences can be rather large, considering the cold water inside in combination with warm outside air together with solar radiation. The significant bending moments in both the vertical and horizontal direction have to be added to the other effects already discussed.

Fast filling of a new tank with cold water during a hot summer may cause thermal shock and cracking.

Post-tensioning is applied on the empty tank, causing larger than water load stresses in reverse. In places of restrained freedom of movement these stresses can crack the concrete before the tank is filled up with water. Design analysis will very seldom deal with this potential problem.

In some elevated tanks the post-tensioning has been applied in stages, while the structural system of the tank was changing during the course of the construction. For example, the cylindrical wall of the tank and the tension ring is constructed first and partially post-tensioned, followed by building of the domed bottom and roof and the completion of the remaining post-tensioning.

It has been noted in cases investigated, that considerable temporary bending moments are created in locations without sufficient reinforcing to take care of them. The cracks later appearing at such places might have developed in these early stages.

A finite element model shows that a standpipe behaves like a thin-walled cylinder except in the region of the wall-floor intersection where steep moment gradients occur in the wall. Thus, refined methods of analysis need only be carried out in this region provided relevant boundary conditions are incorporated into such a model. Generally, charts are not readily applicable to standpipes since most charts cover only tanks with floors at grade level and having aspect ratios outside those of a standpipe. However, they may be used provided the designer has a proper understanding of the overall behaviour of this type of tank. The classical approach (Ref. 33) is applicable only to tanks with radial symmetry. Thermal loading which is not symmetrical due to effects of solar radiation, wind etc, can only be analyzed by a refined model. However, the cost of a refined model can be high and such models should be used with discrimination.

5.3 Repair Methods Developed

Repair methods were developed, and suitable materials evaluated, in the following main areas:

Structural

- Vertical crack control
- Replacement of corroded prestressing steel
- Concrete spalls
- Delaminated prestressed wall removal

Leakproofing

- Steel Liners
- Epoxy injection
- Caulking

Waterproofing

- Internal coatings (non-toxic, odourless, tasteless)
- External corrosion protection for prestressing wires
- Surfacing materials and procedures

External stage post-tensioning hardware and methods have been used for replacement of corroded prestressing wires in the wall and may be used for vertical crack control.

Ice prevention systems such as insulation, circulation mixing, and heating are now being used in conjunction with structural repairs in order to correct cracking caused by the expansion force from internal ice formations, or where these formations are excessive inside the tanks.

5.4 Tank Repair Methods

5.4.1 Condition Surveys

Prior to designing a repair or rehabilitation programme it is essential that a condition survey is undertaken to determine the nature and extent of deterioration. The main components of the survey are visual inspection and exploratory inspection. Record photographs should be obtained at all stages of the survey. The methods used are mapping, light jack hammering, coring and laboratory analysis.

The first aspect of visual inspection is to determine the location of external leaks, spalls and cracks. It has been found that this is best accomplished when the tank is filled. These defects should be noted on a plan indicating the cardinal points and if possible a point of reference such as the manhole.

The next stages are accomplished after emptying the tank and consist of external and internal delamination mapping and are accomplished using hammer sounding.

It is important that potentially defective areas are thoroughly inspected in detail to examine the depth of degradation. Some of the tank repairs completed required further remedial works within the maintenance period, which sometimes could be attributed to lack of detailed inspection at these particular locations.

Based on current experience, the presence of the following possible defects should be verified in addition to general defects observed:

- All internal coatings should be examined for depth of fracturing on the inside concrete surface - obtain cores and examine by microscopy.

- Reinforced concrete standpipes should be examined for depth of external and internal delamination - cores obtained from outside and inside the tank.

- Gunited tanks should be examined for presence of loosely compacted concrete and delamination between layers (from the shotcrete process). Take cores in suspect areas.

- Post-tensioned, unbonded tanks should be examined for the presence of water leakage into the tendon ducts - look for cracks, water leakage and possible corrosion.

- All tanks should be examined for the presence of vertical and horizontal cracks indicating possible structural weakness caused by ice formation or thermal movement or insufficient prestress.

- All construction joints should be examined for water infiltration and possible deterioration through the section.

- Obtain external and internal cores to the depth of the waterstop, particularly where leakage is suspected. Verify in areas not showing leakage.

- Determine if jack-rods have been used in construction and verify if they are completely filled with grout.

- Locate presence of possible voids in tank wall created by improper construction methods.

5.4.2 Surface Preparation

The first step in all coating rehabilitation work is to remove all unsound concrete using light chipping hammers and sand or grit blasting. Water jetting was successful in some cases in removing the internal coating and fractured internal concrete surface. If reinforcing steel is present then it can be cleaned using mechanical wire brushing and sandblasting.

5.4.3 Delaminations and Spalls

Where the concrete is removed to a depth greater than 75 mm, welded wire fabric reinforcing is included to provide mechanical bond. This is accomplished by drilling eye inserts and installing ring fasteners. The formwork is then placed over the patched area and grout poured in an entry port. Since only small quantities of materials are required, proprietary brands of pre-mixed grout containing bonding agents are normally used.

At locations where the depth of removal is less than 75 mm, a latex bonder is first applied and a stiff mortar is trowelled in.

Curing of all of the above repairs is achieved using wet burlap over the patched area.

5.4.4 Crack Repair

Prior to the determination of the method of crack repair, an assessment of the overall pattern of cracking has to be made. In some tanks vertical and horizontal cracks were somewhat isolated and cores indicated sound concrete within the vicinity of the crack. In such cases the procedure used was to rout out the crack and fill it with a sand filled epoxy mortar. In some tanks, parts of the inside surface are occasionally highly fissured. Dealing with each crack separately was judged to be uneconomical and therefore the approach taken was to trowel a surface layer of epoxy mortar over the fissured area. This is usually done in conjunction with the provision of an overall surface layer as described in detail in the report on coatings evaluation.

Photo 5-2 *Surfacing tank wall prior to application of coating.* (HESPELER)

One difficult area to design repairs for is at joints. Typically, joints occur between the floor and wall, occasionally there are also vertical joints. Examination of these joints has shown that, in many cases, the water has penetrated the waterstop and that concrete deterioration is common at these locations.

5.4.5 Waterproofing

As discussed in previous sections of the report, it is important that the water is contained within the structure and is not allowed to be in close proximity to the external surface, as may occur due to improper design or defective workmanship.

Photo 5-3 *Trowelling latex modified mortar. (HESPELER)*

Both bonded coatings and steel liners have been used for waterproofing.

Where coatings have been applied, it was found difficult to provide an intact film over the rough surface of the tank wall. Consequently, the internal walls of tanks undergoing this treatment were surfaced with a layer of either modified latex or epoxy mortar.

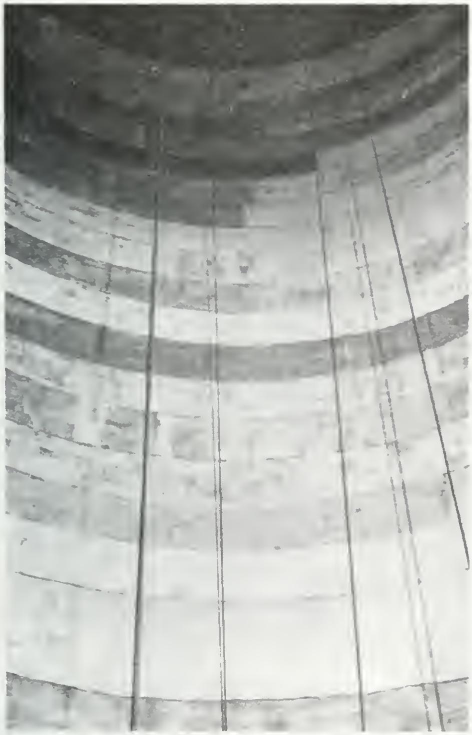


Photo 5-4 *Completed surfacing prior to coating application.* (HESPELER)

5.5 Typical Repair Systems for Various Concrete Tank Types

5.5.1 Concrete Tank Types

The different categories, designations and construction methods used in building concrete tanks in Ontario are given in Figure 1.1 and Table 1/1. These are summarized in Figure 5.1. The various construction methods, structural types and forms of prestressing resulted in a variety of defects and forms of deterioration requiring individual repair systems to be adopted for each tank type. These are described in the following pages with respect to each of the tank types repaired in the rehabilitation programme to the end of 1986.

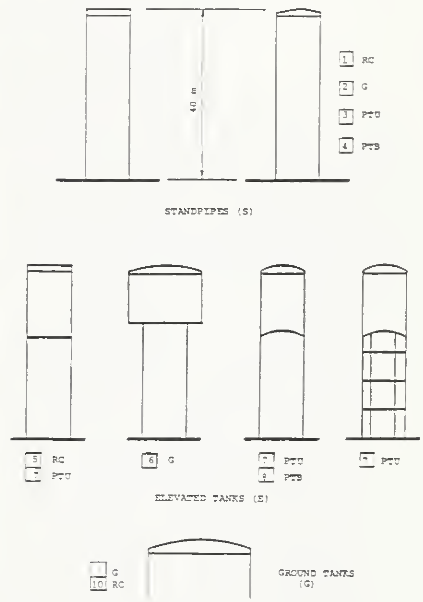


Figure 5.1 The 10 *Concrete tank types in Ontario (see Table 1/1 page 1-3 for description and abbreviations)*

5.5.2 RC-S Type Tanks

Two (2) standpipes of this type, namely WATFORD and ALVINSTON were repaired, and five (5) have been or will be replaced with new steel standpipes since the repair cost of the badly delaminated tanks were equal or greater than the cost of a new steel standpipe.

Leaking active vertical (hoop tension) cracks at 300 mm(1 ft.) approximate spacing, apparently static horizontal cracks and defective construction joints at 600 mm (2 ft.) lifts, had contributed to rapid saturation with resultant wall delamination and massive external spalling in these tanks. The remedial methods were to remove and repair the delaminated concrete, in one case,

Photo 5-5 *Installation of post-tensioning anchors. Note leaks at construction joints.* (ALVINSTON)

by casting on a new 200 mm (8 in.) thick exterior reinforced concrete wall bonded to the old, and closing or controlling the vertical cracks from opening by external post-tensioning. Internal waterproofing was carried out using a bonded epoxy coating.

One standpipe (ALVINSTON) was supplied from a surface water source. The temperature of the inlet water was found to be marginally above freezing during winter, resulting in considerable ice formation in late winter and possible damage due to ice thrust forces. Consequently, as part of the remedial works the tank was the first in Ontario to be insulated and clad with pre-finished steel. Additionally, as part of the applied research programme the performance of the ALVINSTON tank was monitored with temperature transducers hooked up to a data logger and computer.

Photo 5-6 *Installation of external post-tensioning tendons.* (ALVINSTON)

5.5.3 G-S Type Tanks

Eight (8) standpipes of this type have been repaired or replaced up to the end of 1986, namely BADEN, CHESLEY, FENELON FALLS, GRAVENHURST, HESPELER, L'ORIGNAL, WINGHAM and WOODVILLE.

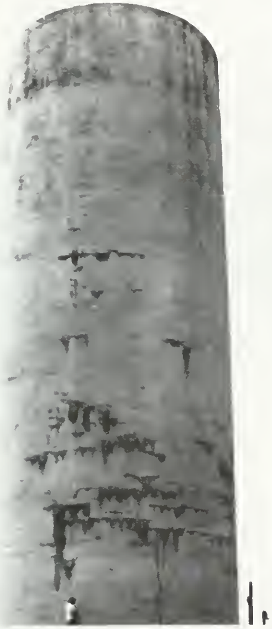


Photo 5-7 *Typical horizontal cracking of a G-S type tank.* (L'ORIGNAL)

These types of tanks usually exhibited external leakage with incipient and actual wire corrosion at locations where the cover coat had spalled. The majority of these defects were attributed to the presence of internal jack-rod spalls. Additionally, wall damage and horizontal micro- cracking caused by slip forming and poor concrete placement was present at some of the areas where the cover coat had spalled.

Repairs were effected by removing deteriorated internal coatings and

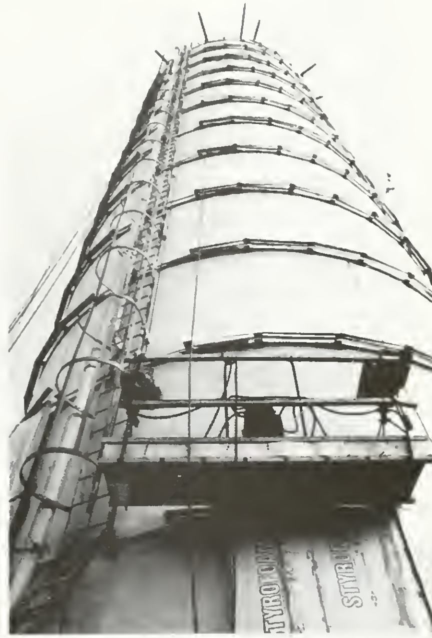


Photo 5-8 *Ring support structure for insulation and cladding.* (BADEN)

locating and grouting up the jack-rods in the wall. In later repairs these rods were more precisely located using radiographic methods. Significant extra costs were incurred due to the variety and extent of wall preparation required after removal of old (mainly cementitious based) coatings. Bonded epoxy coatings applied in 2 or 3 coats to a total thickness of 15 to 35 mils were used as the waterproof coating on 6 standpipes. In some cases a surfacing mortar was applied before the epoxy waterproofing to fill bugholes and provide a smooth surface for the coatings.

Initial problems were encountered in applying effective waterproof epoxy coatings in the first rehabilitation contracts. These problems were condensation of moisture on the walls, pinholes in the coating, and failure to remove or seal and leakproof weak, porous, and micro-cracked substrates. Condensation was prevented by introducing environmental controls and dry heat. Pinholes were eliminated by using near 100 per cent solids epoxies (solvent free) and using either acrylic modified cement or epoxy mortars as a parging or surfacing material to fill all surface holes, and provide a smooth surface layer for the coating. A positive air pressure was applied in some tanks.

Photo 5-9 *Standpipe prior to repair (BADEN)*

Concrete deterioration of the walls in this type of tank is sometimes difficult to identify due to the presence of the "confinement" of the prestressing compressive force. This compressive force limits typical external delamination found in reinforced concrete standpipes but can result in a highly microfractured concrete section. For example, in one of the repaired tanks, a 5 m high band

Photo 5-10 *Standpipe after insulation (BADEN)*

Photo 5-11 *Standpipe after insulation (WOODVILLE)*

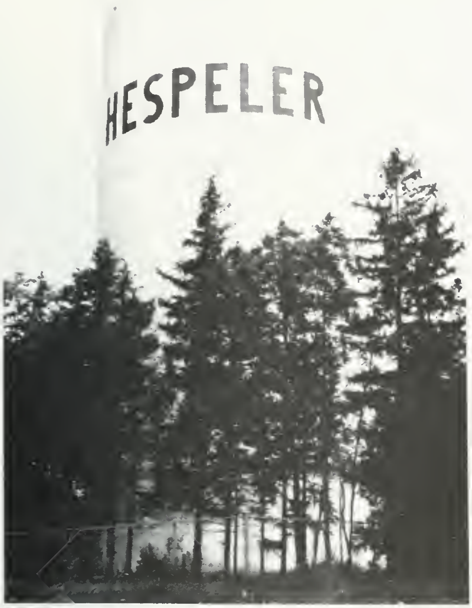


Photo 5-12 *Standpipe prior to repair* (HESPELER)

Photo 5-13 *Standpipe after insulation (HESPELER)*

of microfractured concrete was not identified as a particular problem during surface preparation. The sub-strata, after scabbling and sand blasting appeared to be of suitable quality for epoxy coating and this was completed. Later, in service, small pin-hole leaks formed in the coating and allowed water into wall micro-fractures. Several months later leachate stains were discovered on the outside of the tank. The source of the problem was identified only after extensive laboratory analysis of core samples, and remedial solutions were developed.

It is probable that the initial cracks, having a vertical spacing of between 50 and 250 mm were caused by problems with the slip forming process and that subsequent cyclic freezing produced the fractures within the matrix.

Photo 5-14 *Applying coat of MMA to exterior of standpipe (BADEN)*

5.5.4 PTU-S Type Tanks

GLENCOE standpipe, which is a twin to one which failed at DUNNVILLE, has been repaired by adding external post-tensioning, grouting all the jack-rods left in the wall after slipforming was completed (located using radiography) and internally coating with a bonded epoxy. It was later insulated and clad. Two (2) failed standpipes were replaced with new steel standpipes.

Photo 5-15 *External post tensioning (GLENCOE)*

5.5.5 PTB-S Type Tanks

One large standpipe of this type in Northern Ontario, EAR FALLS, leaked at the construction joints at the top of each jump-formed lift. The rehabilitation method adopted was to install a steel liner, grout between the liner and existing concrete wall to provide a leakproof system, install exterior insulation and cladding and provide a mixing and heat boosting system.

5.5.6 RC-E Type Tanks

One tank (BRECHIN) was repaired by taking the roof off and installing a fabricated internal steel liner. The gap between the concrete wall and the liner was grouted with portland cement grout to provide corrosion protection to the back face of the steel. The inside face of the steel was coated with vinyl paint. External post-tensioning was added to control vertical cracking and insulation and cladding, for freeze protection. Another tank of this type was demolished and replaced with a steel standpipe (PITTSBURGH).

Photo 5-16 *Steel liner being installed. Note external post-tensioning* (BRECHIN)

Photo 5-17 *Tank after rehabilitation including strengthening by post-tensioning, installation of new steel liner, and insulation and cladding. (BRECHIN)*

5.5.7 G-E Type Tanks

The only two (2) elevated tanks of this type built with gunite (shotcrete) walls, namely, AMHERSTBURG, and CHELMSFORD, have required extensive repairs.

Photo 5-18 *Internal maintenance inspection of liner paint system after one year in service (BRECHIN)*

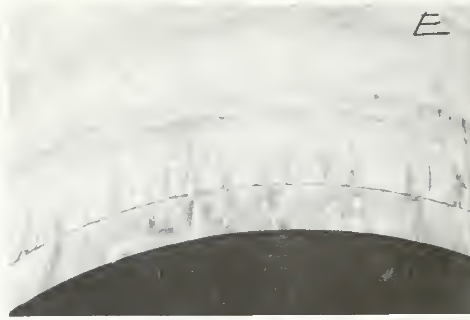


Photo 5-19 *Vertical cracking at base of wall due to ineffective prestressing. This was caused by inadequate provision for inward movement between internal thrust ring and wall.(CHELMSFORD)*

Emergency strengthening repairs were required to keep them in service because of concrete delamination, prestressing wire corrosion and breakage, resulting in concern for public safety. The leakage leading to the wire corrosion in one case was due to the presence of shotcrete rebound and improper water-stop installation resulting in serious wall delamination 40 mm (1 1/2 inch) deep.

Both tanks required major wall repairs - in one case an 2.5 m x 1.8 m (8 ft. x 6 ft.) section was cut out of the prestressed wall. The walls of both tanks were strengthened with external post-tensioning and their horizontal floor thrust blocks and rings were structurally upgraded to resist earth-quake forces.

Internal waterproofing was carried out in both tanks using Tapecrete latex modified cement slurry with fabric reinforcement. Additionally, a mechanically anchored partial liner of PVC coated nylon fabric was installed in one tank, together with back-up drains to substitute for the defective waterstop. Some wall leaks re-appeared in one of the tanks.

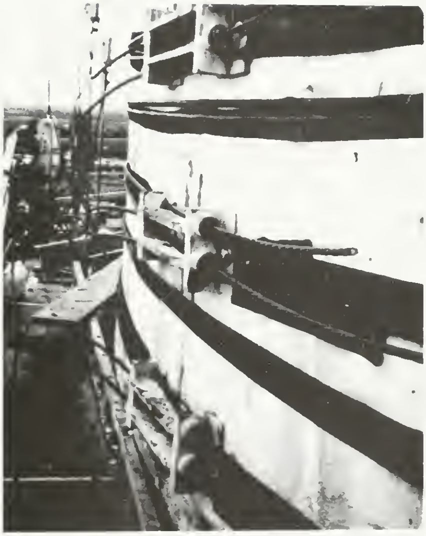


Photo 5-20 *External post-tensioning added to compensate for lack of prestressing in wall.* (AMHERSTBURG)

At one of the tanks many vertical voids which occurred between the original 6 mm (1/4 inch) thick cover coat, wires, and reinforcing steel were filled. Subsequently a 20 mm (3/4 inch) thick latex gunite was applied over the wires as corrosion protection. In the other tank a white MMA exterior coating was applied over the thin cover coat to give the wires added corrosion protection.

New sliding aluminum hatches with a folding access ladder, to allow lifting above winter ice, were added in one tank. In the other tank, a plastic sky dome was installed replacing an area of poor concrete and acted both as a repair and to allow more light into the tank.

Both tanks were insulated and clad in 1986 and 1987,respectively.

Photo 5-21 *Deteriorated internal thrust ring. Note thrust block against wall stopping movement.* (CHELMSFORD)

Photo 5-22 *Repair of thrust ring.* (CHELMSFORD)

Photo 5-23 *Completed repair. Note external tensioning at base of tank. (CHELMSFORD)*

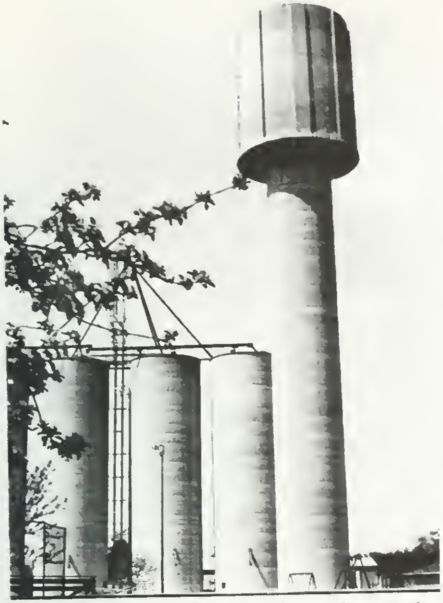


Photo 5-24 *Tank after leakproofing and before insulation.(BRIGDEN)*

Photo 5-25 *Steel support system for insulation and cladding.* (BRIGDEN)

5.5.8 PTU-E Type Tanks

One (1) elevated tank of this type, with hoop and vertical unbonded tendons, has been repaired. The BRIGDEN tank exhibited leakage at the floor, the wall/floor joint, and some wall dampness, which was repaired in two stages. An interior flexible joint sealant was installed by cutting a groove in the floor and bonding the sealant to the wall. The entire circumferential band surrounding the floor joint was given two coats of epoxy to waterproof the joint.

The first stage was completed prior to the winter of 1983, and included temporary repairs to the damp buttress recesses holding the unbonded tendons and to leaks in the tank roof.

During the second stage, completed the following spring, poor quality concrete mortar was removed from all the post-tensioning anchor recess pockets, which revealed a number of slack anchorages due to failure of five of the unbonded prestressing tendons. Detailed investigation demonstrated that the strands had broken as a result of stress corrosion. The corrosion was caused by water entering the anchorages through the porous mortar and into plastic tubes sheathing the strands which were inadequately protected by corrosion inhibiting grease. The inhibitors in the grease may have been rendered inactive or the grease itself may have been displaced by the infiltrating water. (See Section 6 for more details). The failed strands were removed, replaced with new regreased strands, and the recess pockets were filled with a dense non-shrink reinforced mortar.

Photo 5-26 *Completed rehabilitation of tank.* (BRIGDEN)

During the leakage test for the stage 2 repairs, dampness was again observed at the wall/floor joint which had been sealed during the previous stage 1 repair. This included removal and replacement of areas of deteriorated surface mortar.

Investigation revealed that although the surface mortar was "sound", deteriorated concrete existed underneath and close to the waterstop situated at the centre of the wall, and was hidden by the mortar. All unsound material was removed, except adjacent to the vertical post-tensioning dead end anchorages, and further repairs were carried out to correct the original fault of a poorly installed waterstop. The bottom of the wall was re-constructed using epoxy pea gravel and sand mortar.

It was considered that the inferior material at the centre of the wall was the end product of an improperly installed waterstop which allowed the mortar to saturate and whose structure had subsequently been destroyed by freezing and thawing.

This tank was insulated and clad as the final stage of rehabilitation.

One other tank of this type was demolished and replaced by a new steel tank (VERNER).

Photo 5-27 *Delaminated exterior at source of leak after removal of fractured concrete. (CASSELMAN)*

5.5.9 PTB-E Type Tanks

Eight (8) tanks of this type were constructed. The 1981 study indicated that these tank types had one of the highest performance ratings, exhibiting the least amount of visible deterioration. Consequently, their repair was scheduled for the latter stages of the programme. After the 1981 external inspection, three (3) tanks developed serious deterioration problems and were repaired earlier than anticipated.

In 1984/85 winter the CASSELMAN tank suddenly developed a small wall leak adjacent to an ungrouted post-tensioning duct. Detailed investigation revealed that the wall was delaminated at that region of the tank. The remedial solution adopted for this tank was to demolish the walls and roof of the tank, and to construct a new steel tank on the existing base.

Photo 5-28 *Delaminated wall. Inspection revealed ungrouted post-tensioning ducts at this region.* (CASSELMAN)

The PICKLE LAKE tank was noted to be badly cracked and delaminated by internal ice forces after the 1982/83 winter. The remedial solution designed for this tank was to construct a steel liner inside the existing concrete tank and to insulate the space between the steel and concrete walls.

Although RED LAKE exhibited little external deterioration, the 1985 internal inspection revealed considerable delamination of the inside walls of the tank. A remedial solution similar to the PICKLE LAKE repair system was selected for this tank.

The remaining five (5) tanks of this type have exhibited only minor deterioration. The rehabilitation solutions adopted for these tanks have included the application of cementitious or epoxy coating to the inside concrete wall, and the installation of external insulation and cladding.

5.5.10 G-G Type Tanks

Two (2) wire wound gunite ground tanks, namely BARRY'S BAY and ORILLIA, were repaired because of excessive leakage - one through the floor, the other through the walls. One of the tanks which had extensive floor cracking was sealed and Tapecrete was applied over the floor area and around the entire region of the wall/floor joint. The other tank, a large municipal tank, was waterproofed with the Tapecrete fabric and slurry system.

Photo 5-29 *Reservoir before repair* (PRESTON)

Photo 5-30 *Reservoir after repair* (PRESTON)

The smaller of these tanks, BARRY'S BAY, exhibits an ice formation in late winter and will be insulated and clad.

5.5.11 RC-G Type Tanks

CAMPBELLFORD tank was internally coated using a cementitious slurry and fabric method and was insulated and clad.

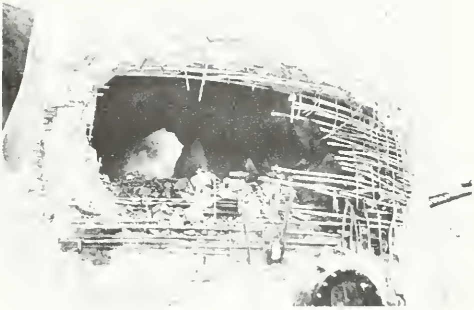


Photo 5-31 *Tank demolition - cutting hole at base. Note ice still inside empty standpipe.* (CALLANDER)

Photo 5-32 *Standpipe toppled.* (CALLANDER)

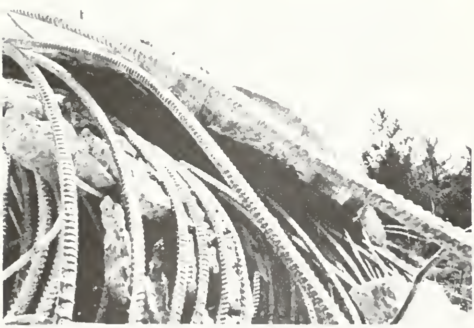


Photo 5-33 *Reinforced concrete tank wall after toppling. Note no corrosion of steel and delamination of concrete from steel.* (CALLANDER)

5.6 Quality Assurance and Measurement

As stated at the beginning of this report, field observations soon indicated that the Ontario water tanks were located in a very severe environment. Small defects in the tanks, perhaps insignificant in other types of structures, have resulted in rapid deterioration of the tank or at least a significant part of the tank. It was, therefore, considered that quality assurance of the remedial works was of the utmost importance to assure success of the repairs. Resident inspection of the remedial work was completed on a 100 per cent basis. The inspection was supplemented by routine testing and occasional specialist advice and testing.

During the repair programme, several quality assurance issues arose. These related to humidity, concrete wall temperature, surface preparation, coating thickness and bond strength measurement.

In order to check that sand or grit-blasted surfaces were prepared to the desired roughness, test patches were prepared and compared with a selected grade of sandpaper with grit size (ALO 80). This is reported in the coatings report.

Sites were issued with sling psychrometers and rotary bi-metal thermometers. It was found that the rotary thermometers were inadequate to measure air or wall temperatures with sufficient accuracy. However, a rapid response surface electronic thermometer proved successful.

Coating thickness was initially measured using wet thickness combs and gross volumetric measurement. Where coatings had solvents or were applied on rough substrates, the wet thickness gauges proved to be inadequate. Additionally, on rough surfaces, although measurement of the consumption of gross quantity of materials accurately reflected the average thickness, microscopic analysis of core samples showed that in many areas the coating was extremely thin. Consequently, a non-metallic coating thickness gauge was introduced. This test essentially involves scratching the coating down to the concrete interface with a cutter of known grooving angle. Measurement of the dimensions of the groove, and hence coating thickness are obtained using an inbuilt measuring microscope. It was found that measurements of the minimum coating thickness (the specified measurement) could be determined rapidly by the site inspector.

Measurement of bond strength either of the cementitious based repair materials or the coating itself was an important consideration, and therefore, during the course of the 1983 programme, test methods were developed which could be used to assist the repair programme. A modification to the Lok-test apparatus was constructed. The Lok-test is essentially a hydraulic ram which exerts a pull normal to the test surface. To test the repair materials, a 50 mm (2 in.) diameter diamond core drill was used to make a cut extending beyond the repair material. A steel disc was attached to the surface using rapid setting epoxy resin and was pulled in direct tension using the modified Lok-test apparatus.

To test the bond strength of the coating to the sub-strata, a similar technique was used. However, for this test, the initial core cut was not required.

Additional quality assurance procedures were required during the grouting of steel liners. Due to the use of thin liners it was necessary to avoid high fluid pressures occurring in the grout which could buckle the liner. One procedure used, was to balance the fluid pressure by filling the tank with water at a similar rate to the rate of grouting.

Time domain reflectometry was used to monitor the grout and water levels. This technique uses guided electrical pulses and is sensitive to the medium surrounding the wire. Wires were positioned to the height of the liner at cardinal points and connected to an oscilloscope at ground level. Using a series of switches it was possible to monitor the grout and water level by observation of the oscilloscope at ground level. This system also enabled control of pumping rate and sampling for percent bleeding and compressive strength to be accomplished at one location.

The above practical techniques were found useful to avoid some problems encountered in the tank repair programme. High humidity and low wall temperatures have initiated the formation of condensation, with resultant blistering of the coating, after one year of service. Cold wall temperatures have made the application of epoxy resin coatings difficult due to a substantial increase in viscosity at low temperatures. Rough surfaces have produced coatings with an uneven profile with the consequent necessity to apply additional coats. The systematic application of some of the above techniques, therefore, cannot be overemphasized and will result in benefits to both the contractor, and the consultant in addition to assuring an acceptable repair.

6.0 DETERIORATION OF METALS IN CONCRETE WATER TANKS

6.1 Introduction

6.1.1 Role of Metals In Concrete Tanks

The main part of the report is focussed on the deterioration of concrete as a material, and on the reasons for the need to rehabilitate defective reinforced and prestressed concrete water tank structures. Metals, however, especially steel, provide important and necessary structural components of the concrete tanks. These metals, as explained in this section, can corrode, and in some cases, critically weaken the structure. In addition to the steel reinforcing bars and post-tensioning tendons cast in the concrete wall itself, prestressing wires or strands are also wound around the walls and protected with gunite shotcrete. This steel reinforcement provides the principal structural tensile reinforcement for concrete tanks; however, many other important concrete tank components, vital to operations, are fabricated from metals, mainly steel and aluminum.

Some of these other metal components are access manways and covers, roof beams, decks, hatches and covers, elevated tank floor beams, air vents, internal and external ladders and landings, safety rails, inlet and outlet pipes and valves, floor drains and covers, overflow pipes and supports, air-craft lights, and more recently, elements of freeze protection systems such as mixing units, temperature sensors, and roof hoists. All these tank appurtenances are subject to deterioration and must be maintained in good and safe working condition in order that a concrete water tank can be operated satisfactorily, inspected, and maintained efficiently and safely.

6.1.2 Deterioration Sequence

An important point which has resulted from the inspection of many tanks during the Ministry rehabilitation programme is that the inspections of leakage and concrete deterioration have generally, incidentally, led to the discovery of the corrosion and deterioration of metal components in concrete tanks. Corrosion has been observed in critical components in tanks in service for less than 8 years. Since these components are often vital for the safe operation of the tank, it is important that the problem is rectified as soon as possible. In a planned maintenance programme, the condition of all metal components should be inspected in a separate scheduled programme.

6.1.3 Importance of Construction Process

Another important factor which must not be ignored, is the process by which post-tensioning is installed in the concrete walls. The steel tendons are placed in steel or plastic tubes so they can stretch during tensioning. Water entering through defects in these hollow tubes can freeze and expand, cracking the concrete section. In some cases, this can lead to sudden leakage problems and even tank failure. This is neither a direct concrete material, nor a metal deterioration problem, but a problem resulting from the construction process itself.

6.1.4 Summary

On a cost comparison basis, corrosion of steel and deterioration of other metals has not been as serious a problem in concrete tanks in Ontario to date, as has been the problem of concrete deterioration in the freezing environment. Recent observations, however, indicate that the corrosion problem is increasing and cannot be ignored in a rehabilitation and maintenance programme.

6.2 Corrosion of Steel Wall Reinforcement

6.2.1 Need for Reinforcement

Steel reinforcement provides the tensile strength required at ultimate and service loads to resist the applied loads from the retained water, wind and earthquake, as well as environmental loads, in all concrete water tanks. Concrete, being a brittle material, cracks at a very low tensile strain. Sufficient steel reinforcement, therefore, must cross an incipient or actual crack to resist the load and control the crack width. Alternatively, sufficient prestress must be supplied by post-tensioning tendons to pre-compress the concrete to balance and resist the tensile load, and for leaktight construction, prevent the concrete from cracking. Concrete and steel materials must, therefore, be combined in the construction process to result in a load resisting structure.

6.2.2 Types of Reinforcement

Four types of steel reinforcement consisting of two grades - reinforcing steel and prestressing steel, have been used in the construction of the 53 concrete tanks in Ontario, and are listed in Table 6/1.

All 3 types of post-tensioning use high tensile prestressing wires on either an individual basis (G), or as strands of 7 wires (PTB and PTU). All prestressed concrete water tanks, including the G type, include some ordinary reinforcing steel, as well as tendons, in the concrete wall.

TABLE 6/1

Types of steel reinforcement

No. of tanks built	Tank designation	Description
12	RC	Reinforcing bars
12	PTB	Post-tensioned tendons (bonded)
9	PTU	Post-tensioned tendons (unbonded)
20	G	Post-tensioned wires, wound and gunite protected
53 Total		

6.2.3 Detection of Corrosion

The mechanism of general corrosion of steel resulting from the development of an electrical battery with an anode, a cathode, and an electrolyte, is described in many texts and will not be described here, except where special circumstances pertaining to concrete tanks occur. Half-cell measurements using equipment consisting of a copper/copper sulphate cell can, in some circumstances, detect the level of corrosion activity of steel within a concrete wall from electrical potential readings. "Hot spots" where electrical potential readings over 350 milli-volts are read can lead to locations where active steel corrosion within the concrete may be occurring. However, this method must be used with caution to survey the corrosion state of prestressing wires. In the walls of prestressed tanks, steel reinforcing bars in vertical and horizontal directions, as well as vertical steel jack-rod pipes, may exist in the wall in addition to the prestressing wires. Readings of corrosion "hot spots" in the wall can lead to conclusions that the initial prestressing wires are corroding, when in fact, investigations by chipping into the wires and the wall may indicate that the wires are bright and corrosion free on the outside, but that the corrosion is taking place at an unimportant jack-rod coupling near the inside face of the wall. It is, therefore, important to combine half-cell investigations with visual examination of corrosion at "hot-spots" by removal of concrete, before conclusions are made as to what type of steel is corroding in the wall, and the actual location and state of the corrosion.

In most cases of steel corrosion observed in concrete tanks, except in the case of unbonded tendons, signs of corrosion are usually evident, and can be seen as rust, pits or rust stains on the concrete surface. Where the concrete is delaminating, atmospheric corrosion is sometimes observed after removal of the concrete.

6.2.4 Protection of Steel by Quality Concrete

It has been stated previously that general corrosion of steel reinforcement in concrete has not been a serious problem in the concrete tanks inspected except at locations where there is leakage and dampness, exposure of the steel to the atmosphere, or at a visible concrete delamination, spalling or porous area of concrete. Good quality concrete with a maximum water/cement ratio of about 0.50 and an adequate portland cement content, provides a protective passive alkali environment for steel reinforcement, prestressing wires and strands. The pH (hydrogen ion level) on the O (acid) to 14 (alkali) scale, with 7 being neutral, should be a minimum of 12 in concrete in order to provide permanent protection of steel against corrosion. The pH value of poor quality and porous concrete can be reduced by carbonation (CO_2), acid rain, and an aggressive environment so that the original protective passive property of the alkali concrete material is lost, and the corrosion of steel can start.

The widespread corrosion and concrete delamination caused by chlorides from de-icing salts as experienced in concrete bridge decks and parking garages in recent years, is noticeably absent in concrete water tanks in Ontario.

6.3 Observation and Repairs

6.3.1 Reinforced Concrete Tanks (Type RC)

Corrosion of the reinforcement in reinforced concrete (RC) tanks which are non-prestressed has been negligible. Even where external spalling of the concrete has occurred causing the steel to be exposed for years, the general corrosion has not been serious (WATFORD, CAMLACHIE, ALVINSTON, etc.).

Examination of reinforcement adjacent to vertical delaminations 1 mm wide in the walls of concrete tanks under permanently saturated conditions has revealed no sign whatsoever of corrosion of the steel with approximately 30 mm of concrete cover (ALVINSTON, CAMLACHIE).

Examination of the reinforcement from a large elevated reinforced concrete tank demolished after less than ten years in service (PITTSBURGH) revealed that there was no corrosion and that the steel was in an "as new" condition.

All observation confirms that a protective cover of about 25 mm of good concrete provides satisfactory corrosion protection for reinforcement during the normal expected service life of 30 to 50 years of a concrete water tank (PRESTON - 30 years old).

6.3.2 Post-tensioned Bonded Tanks (Type PTB)

.1 Description

Post-tensioned bonded tendons in Ontario concrete water tanks consist of a single 16 mm diameter 7 wire prestressing strand placed in 30 mm diameter corrugated ducts or sheaths manufactured from plain bright steel strip.

After tensioning the strands, protection against corrosion is carried out by injecting a neat portland cement grout, containing an expansion admixture, into the ducts with a pressure pump, so they are filled with alkali material with a minimum pH of 12.

The grout then hardens, thus effectively bonding the strands in the corrugated ducts within the structure like normal hi-bond reinforcement in concrete material.

Hoop or circumferential tendons are generally 180°, (half circle) with steel anchorages at the ends installed in external buttresses or internal pockets. The strands are locked off after tensioning (and before injection) in tapered holes in the steel anchorages, with tapered wedges, and with machined teeth. Straight vertical tendons are incorporated in some PTB tanks.

.2 Problems (See Figure 6.1)

Ungrouted ducts

Problems can occur later if the ducts are not properly filled with portland cement grout by injection after tensioning the strands. As described elsewhere, the empty ducts may fill with water some years later and freeze in winter, causing expansion and cracking of the wall (EAR FALLS, CASSELMAN).

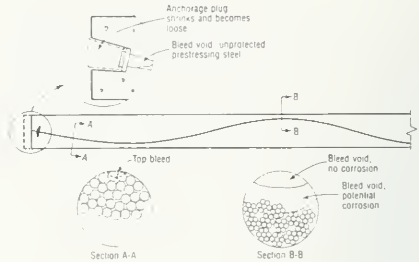


Figure 6.1 *Bonded tendons. Bleed void, corrosion of unprotected prestressing steel. Concrete anchorage pocket plug shrinks (large circle, upper left) and becomes loose. Poor bond or non-expansive mortar permit aggressive materials access to anchorage and prestressing steel, likewise with improperly bonded and anchored exterior-end anchorage (shown at left end of prestressing steel) protection. (ref. 35)*

This can lead to increased leakage in the spring followed first by corrosion of the sheet steel ducts and then corrosion of the unprotected and vital prestressing strands themselves. The deterioration process here, as described elsewhere in the report, is similar and occurs in 3 stages. First the initiation stage consists of the steady breakdown of the waterproofing system followed by concrete saturation and finally leakage into the empty ducts. The second active stage consists of the filling of the ducts with water, freezing, cracking of the concrete section and subsequent leakage of water to the outside. Exposure to air, together with moisture, initiates the start of corrosion of the duct and prestressing strand. The third and final stage is the loss of steel section and prestressing force, eventual failure of the strands or wires and loss of strength of the tank wall section.

Investigations under the spalled concrete fill in the vertical tendon anchorage recesses at the roof level of one standpipe (MILLBROOK) revealed severe corrosion of the strands and wedge anchorages. Some strands had been completely eaten away down into the corroded wedges to the point where anchorage of the strand tendon by grout bond was essential, because the steel anchorage had failed. The severity of the aggressive corrosion into the anchorage indicated that calcium chloride, or a similar corrosive material, may have been used in the mortar to either speed up hardening of the mortar or prevent it from freezing.

.3 Repair

Diagnosis of the strand corrosion problem (a) described above, by investigating at the leak locations, must be made early so that the corrosion process can be stopped before significant damage is done. This can be accomplished by filling the ducts with protective epoxy or cement grout before the loss of section and the state of corrosion of the wires of the strands becomes serious from pitting, or if stress corrosion occurs.

If strands are seriously corroded or have failed, it may be necessary to strengthen the tank with added external post-tensioning tendons, as well as leakproofing the tank. In this case the designer must check that the wall is not overstressed in the empty tank state by too much additional prestress.

6.3.3 Post-tensioned Unbonded Tanks (Type PTU)

.1 Description

Post-tensioned unbonded tendons in Ontario concrete water tanks consist of single 13 or 16 mm diameter 7 wire prestressing strands, coated with protective water-resistant grease charged with rust inhibitors, and pushed into black polyethylene tubes with walls about 2.5 mm thick. The air space between the coated strand and the inside of the tube may be about 2 mm. Steel anchorages are installed at the ends of each 180° or 360° hoop tendon to lock off the strands with tapered wedges with machine teeth in tapered holes in the anchorages. Straight vertical tendons are added in the walls of many PTU tanks. If either the strand or an anchorage fails, the total unbonded tendon fails and is lost to the structure.

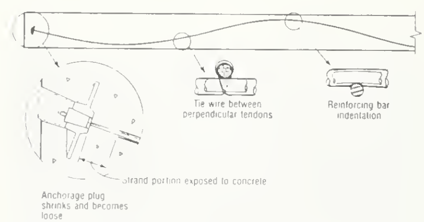


Figure 6.2 *Unbonded monostrand. The anchorage plug shrinks and becomes loose. Poor bond and non-expansive mortar permit aggressive materials access to anchorage and prestressing steel. Strand portion is exposed to concrete because no physical connection is made between the sheath and anchorage. At stressing end, this portion of tendon is pulled through intimate concrete closure when stressed. Tie wire between perpendicular tendons causes local indentations in sheaths which tend to shear off when tendons are tensioned. Reinforcing bar indentation causing hard point that tends to shear off when tendon is tensioned.*

.2 **Problems** (See Figure 6.2)

Two problems exist with unbonded tendons, firstly that of corrosion of the strands which is the subject of this section; and secondly, the filling of the air spaces in the plastic tubes with water under pressure, and freezing, especially of vertical tendons. This may contribute to the rupture of the wall causing complete and sudden failure of tanks (DUNNVILLE and SOUTHAMPTON) which is described elsewhere in this and other reports.

Five (5) of the 116 unbonded strand tendons were found to have failed by stress corrosion cracking during the rehabilitation of the seven year old BRIGDEN elevated concrete water tank. For a typical failure see Photo Nos 6-1, 6-2, 6-3 and 6-4. The sequence of events were presumed to be as follows:

Photo 6-1 *Poorly protected post-tensioning anchorage allowing water to enter.* (BRIGDEN)

- water from external precipitation penetrated to the tendons through the poor quality and porous concrete fill in the anchorage recess pockets.

- stress corrosion cracking across some wires started at the non-metallic inclusions in the steel surface.

- longitudinal brittle fractures in the wire started at the stress corrosion crack sites as a result of bending stresses on the wire due to the wall curvature.

- tensile brittle fractures of the remaining uncorroded wires occurred similar to that expected from an increased load on the remaining wires.

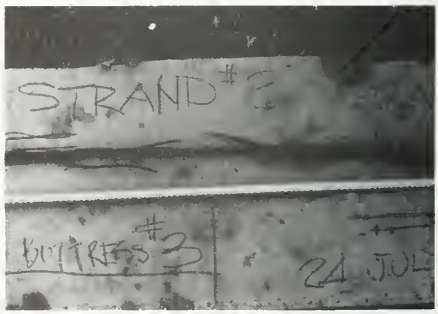


Photo 6-2 *Detailed view of strand showing corroded wires.* (BRIGDEN)

.3 **Repair**

All anchorages were exposed by removing the covering mortar from the recesses. Lift-off load tests were carried out on all but 4 of the 116 strands in the tank with specially developed tools and post-tensioning jacks to a load of 70 per cent of the guaranteed ultimate strength of the strands which was 17 per cent above the design load of the strands (60 per cent ultimate). This action resulted in obtaining a proof test of the total hoop strength of the tank at that time and identifying that all 5 strands failed from corrosion and were no longer capable of supplying the proof test force slightly above the required design strength of each strand.

Photo 6-3 *Typical longitudinal fracture of one of the wires of a strand. Lower break shows some elongation which was not typical of breaks observed. Some grease is present on the wire.* (BRIGDEN)

Techniques were developed for removing the 5 strands, either already failed or failing to meet the proof test load, and replacing them with new strands coated with grease in the existing plastic ducts.

Finally the anchorages and recesses were sand-blasted, mesh reinforcement fastened with drilled inserts was installed, and a high quality latex concrete mortar was used to fill the recess pockets to seal the ends of all tendons and prevent further water leakage into the ducts.

6.3.4 Gunite Protected Tanks (G Type Tanks)

.1 Description

The sequence of construction of post-tensioned wire wound (G type) tanks is different to that of PTB and PTU types described previously, where the strand post-tensioning tendons inside ducts are placed first, then cast into the concrete walls. After the tank wall is completed and when the concrete reaches a specified minimum compression strength, normally 24 to 28 MPa, the tendons are tensioned and anchored. This is carried out in a specified order of stressing so that the wall is not cracked by the applied initial prestressing forces during the post-tensioning process.

Photo 6-4 *Start and progression of a typical fracture. Fracture starts at the surface of the wire then progresses longitudinally going deeper into the wire as it progresses. Note corrosion of wire near the bottom of picture. An example of the second type of fracture (transverse) can be seen in the wire on the right of the strand.* (BRIGDEN)

In the G type tanks, the wire tendons are tensioned as they are pulled through a die and wound continuously around the already constructed concrete tank wall after it reaches the required minimum strength, usually 31.5 MPa. The walls of ground and elevated G type tanks are normally constructed of shotcrete concrete (gunite) mortar. All standpipes in Ontario, except one, were constructed using slipformed concrete. This method uses vertical jack-rod pipes, coupled every 3 m (10 feet) height in the walls, on which the forms are continuously raised by hydraulic jacks during the concreting process. The jack-rod pipes left in the walls have caused serious concrete deterioration problems in many tanks because of internal spalling at the coupling points due to the pipes filling with water and then freezing, causing explosive forces on the concrete. This deterioration is described elsewhere in this report.

The hoop post-tensioning of G type concrete tanks is carried out using No. 8 gauge high tensile prestressing wire of 4.1 mm diameter drawn through a die to 3.6 mm. The initial wire stress is 980 MPa reducing to a final effective stress of 735 MPa after all losses. After the post-tensioning of each layer of wires is completed, they are covered with a thin 2-3 mm wash layer of gunited on concrete mortar (shotcrete). The number of layers of prestressing wires placed on top of previous layers is a function of the hoop force from the wires required to resist the applied water load, and any other design loads on the tank. A maximum of five (5) designed layers has been observed. Additional layers to the designed number have been applied in some tanks (L'ORIGNAL and FENELON FALLS) because of problems during post-tensioning, or with the quality of the wire.

After the completion of winding on the final exterior layer of wire, the wires are protected with a final coat of the pneumatically applied concrete gunite mortar. This cover coat thickness is usually specified as either a minimum of 20 or 25 mm.

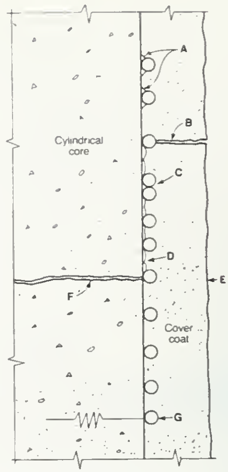


Figure 6.3 *Flaws leading to corrosion of wire wrapped circular pipes and tanks.*

A) sand pockets or voids which permit the passage of electrolytes and oxygen.
B) uncontrolled structural cracks, permitting the environment access to the prestressing steel.
C) Bundled wires or strands which can result in a continuous void.
D) Delaminated cover coat which is generally connected with a circumferential crack exposing the backs of the prestressing steel.
E) Inadequate cover coat which is less than 15 mm or pervious, permitting access of environmental contaminants.
F) Cracks or honeycomb in cylinder core giving access of contaminated material to prestressing steel.
G) Electrical connection of prestressing steel to other metal components.
Note that electrolyte has to be present for corrosion to occur

.2 **Problems** (See Figure 6.3)

Two elevated G type tanks were built in Ontario, at AMHERSTBURG, and CHELMSFORD. Both these tanks developed severe wire corrosion problems at the bottom of the tank walls for

Photo 6-5 *Delaminated cover coat exposing broken prestressing wires (AMHERSTBURG)*

heights of 2 m above the floor slab and required major repairs, including strengthening by external post-tensioning, after about ten years in service.

Seven broken wires were found at the AMHERSTBURG tank and a further number badly corroded in an area 2 m x 2 m adjacent to a major leak at a wall delamination or split and where the cover coat had fallen off, exposing the wires. (see Photo No's 6-5, 6-6) The original cover coat was only about 5 mm thick over the wires.

A similar area of corroded wires, to that described above, was located at CHELMSFORD, when a large area of de-bonded cover coat which had bulged 75 mm outward, was removed. The de-bonding had exposed the wires to the elements probably soon after the tank was filled, judging by the severe state of the corrosion, and other observations. Vertical cracks in the tank indicated that strengthening was required.

A number of broken or severely corroded wires have been found in three (3) G standpipes, generally at locations of local leakage and where the thin

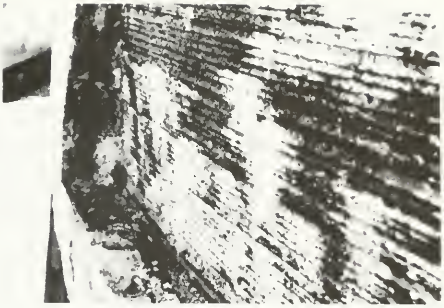


Photo 6-6 *Corroded prestressing wires under delaminated cover coat (CHELMSFORD)*

cover coat has de-bonded and fallen off the vertical face exposing the wires.

Figure No. 6.3 shows flaws leading to corrosion of wires of wire wrapped tanks (after Schupack).

.3 Repair

Two external post-tensioning procedures have been used for strengthening tanks with corroded or failed circumferential prestressing wires, both using special anchorages developed for tanks.

The first method was to install greased strands in black polyethylene tubes around the tank. The second method was to place bare strands around the tank with the stressing anchorages near the broken wire. This assures that the maximum force is applied near the break in wires, because of the steel/concrete friction loss around the tank. The strand is then protected from corrosion by spraying on a cover of gunite.

Care must be exercised in the design, specifications, and the application of post-tensioning not to overstress the tank wall so that it cracks in the empty state.

Photo 6-7 *Corroded access tube (PRESCOTT)*

Corrosion of prestressing wires in G type tanks from exposure to external moisture and air through a cover coat which is too thin (much less than the recommended 20 mm minimum), presents a difficult maintenance problem. If corrosion is general, widespread and well advanced, or a number of broken wires exist, strengthening by adding external post-tensioning may be possible. If the corrosion is minor and sporadic, extra protection to the wires by adding surface coatings can be carried out. An extra gunite layer, preferably a 20 mm additional thickness of portland cement mortar, modified with latex, or a methyl methacrylate (MMA) coating, which is less costly, have been used. The MMA coating has been developed so that an attractive solid white gloss finish can be obtained in 2 coats (with some touch up) enhancing the appearance of the repair, which is important in the repair of concrete tanks because of their high profile. (see photos 5-23 and 5-30)

Photo 6-8 *Corroded roof truss (HESPELER)*

Photo 6-9 *Completed roof repair (HESPELER)*

6.4 Deterioration Of Metal Components

6.4.1 Steel Access Tubes in Elevated Tanks

.1 Problem

Three (3) elevated concrete tanks in Ontario, AMHERSTBURG, CHELMSFORD and BRIGDEN, are constructed with internal steel tubes containing access ladders to the roof, or into the tank, and provide support for the roof. These steel tubes are primary tank components and must be protected from corrosion so that their strength and leaktightness are maintained, or the tanks will fail to retain water.

Photo 6-10 *Aluminium ladder exhibiting severe pitting corrosion (BRIGDEN)*

.2 Repair

Severe corrosion and pitting of the central steel access tube in contact with the water occurred in the BRIGDEN tank after six years in service. Protection was carried out by removing the rust and preparing the steel by grit blasting and re-painting with a five coat vinyl paint system as specified for new Ministry steel water tanks.

6.4.2 Aluminum Ladders in Water

.1 Problem

Severe, deep, and widely distributed pitting of the internal aluminum ladder below the water line was observed in the BRIGDEN tank described above, with a central corroded steel access tube.

Investigation of the corrosion showed that the pits were initiated on the surface of the aluminum because of the central presence of iron on the surface. Iron was present in the water because of the corroding central steel tube. Small particles of iron set up localized galvanic cells which resulted in pitting corrosion over 10 mm deep in places.

.2 Repair

As it was virtually impossible to completely stop the advanced pitting corrosion in the aluminum it was recommended that the ladder be replaced. It was decided to replace the ladder with one fabricated from steel and hot-dip galvanized for protection.

6.4.3 Recommendations

- There have been reports of certain types of aluminum corroding in chlorinated water.

• If aluminum is used, it is recommended that it be an aluminum alloy such as Alclad 6061-T4 or T6 temper.

• Some authorities are now specifying fiberglass rather than metal ladders in water treatment and sewage plants.

6.5 Metal Appurtenances on Concrete Tanks

6.5.1 Description

A description of miscellaneous metal components in concrete tanks is given in paragraph 6.1.1 of this section. These metal components include main structural members, manhole covers, access ladders, safety lights and piping, and they are fixed to the concrete tank with metal fastenings.

Photo 6-11 *Corroded steel manway cover and bolts.* (WATFORD)

6.5.2 Observations

• In some cases, the main component may be in satisfactory condition but the fastenings, cast or drilled in the concrete, such as inserts, are corroded, often because they have not been protected satisfactorily against corrosion or because they are different to or are an inferior metal to the appurtenance. In the case of ladders and landings, this corrosion of fastenings can lead to unsafe conditions.

• Working or critical parts of appurtenances such as hatch and manway hinges, threaded retaining bolts for manways, keyholes in locks, etc., are often badly corroded causing delays in entry for inspections and maintenance work, and costly replacement if threaded studs cannot be removed, for instance. (see Photo 6-11)

• Steel mesh screens in cylindrical aluminum air vents installed to prevent entry of insects and birds into the tanks are almost invariably severely corroded because of the bi-metallic contact.

6.6 Summary and Conclusions

• The two most common metals used in concrete water tank construction are steel and aluminum. Steel is installed inside the concrete in the form of reinforcing bars and as post-tensioning tendons. Appurtenances on tanks such as external and internal ladders, landings, hatches, vents etc. are normally aluminum or galvanized steel.

• Exposed metals on concrete tanks must be protected and maintained, in order to arrest or prevent deterioration.

• Examples of severe corrosion of steel reinforcement are rare, but have occurred where the steel was exposed.

• Hi-tech methods for the detection of corrosion "hot-spots", such as half cell voltage measuring, should be validated by visual examination. The reason for this visual examination is because steel pipe jack-rods may be in close proximity to each other or touching in the wall. Severe corrosion activity in a jack-rod pipe, for example, may be read inaccurately as being in the prestressing wires, which is erroneous.

• Dense, high quality concrete provides good protection for steel reinforcement in concrete water tanks, providing the cover exceeds 25 mm. The cement content must be sufficiently high in order to maintain a pH value of 12 or more. This alkali environment protects the steel from corroding.

• Corrosion of post-tensioning wires and strands will occur if the high tensile prestressing steel is unprotected by grease containing corrosion inhibitors (PTU tanks), by portland cement grout injected into the ducts (PTB tanks), or by a minimum of 20 mm thickness of high cement content gunite cover coat (G tanks) and is exposed to the atmosphere and external or leakage water from the tank.

• Stress-corrosion can cause brittle tensile failures in unbonded prestressing strands without rust signs, because the steel fails inside a plastic tube.

• Detection of corrosion of post-tensioning tendons is sometimes difficult because they are hidden from view. Start inspections near leakage points.

• Concrete tanks weakened by corroded internal or wirewound tendons may be strengthened by external post-tensioning.

• Steel access tubes in concrete tanks may be primary structural elements and must be inspected and maintained in a corrosion free condition.

• Severe pitting corrosion of aluminum has been observed in a concrete tank with a corroding steel access tube.

• Many instances of the corrosion of tank appurtenances have been observed, especially fastenings and vital working parts, which need costly replacement when not maintained.

6.7 Recommendations

• The metal parts of concrete tanks often form primary structural elements and must be inspected and maintained on a regular basis.

• Post-tensioning tendons exposed to external water or tank leakage are vulnerable to weakening and failure due to corrosion and must be maintained and protected from this environment and all leakage or infiltration of water to prestressing steels must be stopped.

• Aluminum ladders and fittings should not be used in chlorinated water, unless the metal is proven to be a corrosion resistant type alloy in that environment.

7.0 CONCLUSIONS

7.1 Introduction

The principal conclusions in this report relevant to the deterioration of concrete tanks, result from the five year study of the deterioration of 53 uninsulated concrete tanks in Ontario built since 1956, but mainly since 1970. The study was initiated by the Ministry of the Environment in 1981 and includes the period up to the end of 1986. It incorporates findings from the applied research programme and references listed, as well as from investigations, inspections, repairs, and rehabilitation of the tanks. Eleven tanks have been, or will be replaced. The total rehabilitation programme in this five year period has cost over $15 million, including engineering, research, development, remedial repairs,waterproofing, replacement, insulation and cladding, freeze protection, construction contracts, and temporary storage.

The study was started in 1981 because of the sudden failure of two new municipally owned concrete standpipes 45 m high, the first in 1976 at DUNNVILLE, the second in 1980 at SOUTHHAMPTON and because of reports from various sources at that time of widespread deterioration and leakage of many of the other concrete tanks in Ontario with as little as 5 years service. This led to general concern for the condition of the tanks, their service life, and for the safety of the public. No above ground concrete water tanks have been constructed in Ontario since 1980.

7.2 Concrete Deterioration Mechanisms

7.2.1 General

A major factor in the extent of deterioration observed in above ground concrete water tanks in Ontario is the type of construction used, e.g. bonded or unbonded post-tensioning tendons, jack-rods left in the tank wall,inadequate gunite cover coat, cold joints produced during jump forming etc. This, combined with a cold region environment has resulted in catastrophic reductions in expected service life.

7.2.2 Internal Ice

Cold region environments can result in detrimental internal ice formations in uninsulated water tanks with low water turnover. Ice formations inside concrete tanks can cause significant hoop pressures on vertical walls subjected to rapid temperature rises. Pressures of 0.7 MPa have been measured in ice caps due to expansion caused by a rise in temperature. These pressures are capable of splitting a concrete tank wall. Tanks with upward sloping walls reduce ice pressure effects.

7.2.3 Freeze-Thaw With Pressurized Water

Freeze-thaw action on permeable concrete, saturated by water under pressure, can result in rapid failure of the concrete microstructure. Studies of this failure mechanism show that dilation or expansion of the water-filled micro pores occur on freezing at about -4°C. Part of this expansion on thawing has been demonstrated in this report to be non-elastic and irreversible. At the start of the thaw cycle, the ice may expand further, before melting. The greater expanded volume of the pores in the microstructure may then be filled with additional water under pressure which on freezing causes a further expansion.· The accumulation of this ratchet effect of permanent residual dilation and associated incremental strains, results in the initiation of the failure of the matrix microstructure when the tensile strength of concrete is reached.

7.2.4 Rate of Deterioration

Observations of concrete deterioration indicate that the rate of deterioration in concrete water tanks is related primarily to the water pressure,freeze-thaw temperature amplitudes and frequencies, concrete permeability and the orientation of the wall section to the sun. The latter observation corroborates the postulation that it is temperature amplitudes and frequency which determine the rate and extent of deterioration. Additionally, temperature monitoring demonstrated that the amplitude and frequency of temperature changes were similar throughout the height of the tank. Since the deterioration was consistently at the lower portions of the tank this suggests that the rate of freezing in a single cycle is not as significant as previously considered. Dilation occurring in densely reinforced wall sections has resulted in progressive delaminations in reinforced concrete standpipes in cold regions. Air entrainment bubbles or voids, while reducing damage in other building structures, may contribute to damage under freeze-thaw conditions in high pressure water tanks.

7.2.5 Freezing in Wall Voids

Water under pressure, freezing in wall voids and cracks, can result in confined ice pressures measured as high as 21 MPa producing spalling and delamination of concrete water retaining structures. Voids can occur under horizontal reinforcing, in ungrouted metal post-tensioning ducts, in unbonded tendons, and inside hollow coupled vertical jackrod pipes left in the wall after slipforming. Deterioration of the concrete is accelerated at leakage points.

7.3 Expansion Joints

Poor installation of plastic or rubber waterstops in expansion joints can result in leakage and costly repairs. Waterstops can act as dams in the wall to permeating water; concrete deterioration due to freeze-thaw action then often occurs on the interior face where the concrete is saturated by water under pressure.

7.4 Corrosion of Prestressing Steel

Losses of the protective gunite cover coat from wire wound prestressing wires has resulted in general corrosion and wire failure causing loss of circumferential prestress. This prestress loss results in vertical cracking of the tank wall. Poorly filled external post-tensioning anchorage recesses have permitted water to enter horizontal unbonded post- tensioning tendons resulting in stress-corrosion failure of strands. Metal ducts accidentally flattened during concrete placing has prevented the insertion of post-tensioning tendons. Lack of grout in ducts has allowed water to enter and freeze, causing cracking and delamination of the concrete wall, and general corrosion of the prestressing steel.

7.5 Repair Methods

7.5.1 General

To be effective, the cause of the problems must be analyzed thoroughly before the repair is designed. Inspection and repair techniques expose people to the dangers of heights and high winds. Repairs are difficult, require specialist contractors and inspectors, and must be carried out within Ontario's limited construction season.

7.5.2 Bonded Waterproofing Coatings

One method of preventing water freezing in the saturated permeable concrete and voids is to apply a bonded waterproofing coating on the inside of the concrete wall. The most effective coating materials used to date have been 100 percent solids epoxies which are non-toxic, tasteless and odourless. The epoxy coating has been applied in 3 coats on a dry grit-blasted surface. Good bond (2Mpa minimum) is essential. Strict environmental control during epoxy application and curing is essential. A surfacing subcoat 3 mm thick must be used on rough concrete to avoid pinholes in the epoxy. Latex and epoxy mortars have proved to be good surfacing materials with adequate bond. It is difficult to completely prevent pinholes from forming in epoxy coatings without pressurizing the tank slightly during epoxy application. Water-filled blisters have been observed in epoxy coatings during the one year warranty inspections. These can be repaired satisfactorily, however in some cases the blisters were not repaired where it was considered they were stable.
Any caulking used, must be durable under water pressure and resistant to chlorine during tank disinfection operations.

7.5.3 Steel Liners

Interior steel liners have been used successfully. Two methods of constructing steel liners inside concrete tanks have been used. One method employs bolts fastened to the concrete wall to provide temporary support of the steel plates, prior to welding. The other method is to construct a freestanding steel tank inside the concrete tank without using any mechanical connections to the concrete wall. A portland cement grout having a pH greater than 11.5 provides protection of the steel against corrosion on the unpainted outside face. The steel liner face in contact with the water is protected against corrosion by applying a bonded coating.

Where large roofs such as domes exist, access holes have been cut so that the steel liner plates can be loaded inside.

In three elevated concrete water tanks, insulation has been placed between the steel liner and the inside of the existing wall, before grouting. External insulation and cladding has also been used on one other elevated tank and one standpipe where interior steel liners were installed.

7.5.4 Plastic Liners

A partial plastic liner has been installed as a cut-off over a defective wall floor expansion joint. The liner was a tough woven nylon fabric coated with PVC, and has performed satisfactorily for 5 years. Recent inspection has revealed that the liner has now become somewhat stiffer and some holes have had to be repaired.

Full height hypalon and polythene liners have been investigated, but to date have not been considered a practical or economical solution.

7.5.5 External Post-Tensioning

External post-tensioning has been carried out to control vertical hoop cracks in reinforced concrete standpipes, and to strengthen tanks initially post-tensioned by the wire winding process and where corrosion failure of the wires had taken place. Special anchorages were developed for single and twin strand tendons wound around circular tanks. Stage post-tensioning of the hoop tendons is necessary to control cracking of thin walls during the application of prestress.

On one project, water entering tubes has caused 3 percent tendon failure after seven years exposure to the elements. The failure from corrosion was apparently because of an inadequate original coating of protective grease on the strands.

7.6 Freeze Protection

7.6.1 General

Freeze protection systems have been developed and installed in concrete tanks in cold regions to attempt to eliminate internal ice formations, reduce the possibility of freeze-thaw cycling in the concrete, and to prevent freezing from taking place in the walls of standpipes constructed using unbonded tendons. Temperature sensors in the water and walls have been used to compare actual performance against heat loss predictions.

7.6.2 Insulation and Cladding Systems

A system developed to insulate circular tanks consists of rigid Styrofoam SM sheets and corrugated pre-painted metal cladding to form a polygonal external wall separated from the concrete tank wall by an air gap. The system is removable so that regular exterior inspections of the concrete exterior may be performed.
Measured performance of the external insulation system with an air gap shows a dampening down of local temperature fluctuations in the wall and a reduction factor of about 3 for the ratio of ambient to air gap temperature (e.g. at a temperature of -30°C the air gap temperature is -10°C). Differences between the theoretical temperatures determined from heat loss calculations and the actual recorded temperatures are attributed to air leakage in the system; however, harmful freeze-thaw cycling and internal ice formations have been eliminated.

7.6.3 Mixing and Heating Systems

In extreme cases, external booster water heaters and internal eductor jet mixing units activated by external pumps are installed in the system.

7.6.4 Air Gap Heating

An air gap booster heating or guard ring system, external to the tank, has been developed to prevent concrete wall surface temperatures of existing tanks constructed with unbonded post-tensioning tendons, or where leaks and wall defects have been difficult to repair, from falling below freezing.

7.7 Tank Types Not Recommended

Two types of above ground water tank construction are not recommended in freezing environments. These are prestressed concrete tanks post-tensioned with unbonded tendons, and reinforced concrete standpipes.

8.1 Introduction

Although no above ground concrete water tanks have been constructed in Ontario since 1980, there may be reasons in the future for them to be constructed, such as the need for competition with steel because of the future high cost or shortage of steel. These recommendations are directed at describing how to try to build concrete water tanks in Ontario in the future, without undergoing the problems experienced in the past.

8.2 Design & Construction of New Concrete Water Tanks

8.2.1 Codes

Codes and guidelines for the design and construction of above ground water tanks in Ontario and Canada generally do not specifically address the important freeze-thaw deterioration problems described in this report and do not include specifications to ensure a satisfactory performance for the design service life of the structure in the Ontario environment. Such codes should be developed. General guidelines, recommended to be followed where applicable, are as follows:-

CAN3-A23.3-M84
: Design of Concrete Structures for Buildings.

CSA S474 Part 4
: CSA Preliminary Standard for the Design, Construction and Installation of Fixed Offshore Production Structures.

ACI Committee 344
: Report on Recommendations for Design and Construction of Circular Prestressed Concrete Tanks.

8.2.2 Interim Guidelines

Interim guideline recommendations of minimum requirements for the design and construction of new above ground water tanks in Ontario (prepared by W. M. Slater & Associates Inc.) have been included as Appendix 'A' in this report.

The main requirements in the guidelines are as follows:-

- Design and construct for a service life of 50 years with minimum maintenance.

- Concrete permeability shall not exceed 0.25×10^{-12} m/s, as measured first by test mixes and confirmed by cores taken from the actual tank.

- Concrete tanks shall be leakproofed by the installation of a liner or the interior shall be coated with an approved waterproof coating. A one year extended warranty against future leakage shall be required after a satisfactory leakage test.

- Concrete water tanks shall be biaxially post-tensioned with external or grouted tendons to give a reserve compression of 1.5 MPa in horizontal and vertical directions after consideration of all applied and environmental loads.

- Concrete water tank structures, and the stored water, shall be protected from freezing by insulation, air gap heating, or other methods. No internal ice formations will be permitted. Carry out heat loss calculations and assess the requirement for either a passive or an active ice prevention system.

8.3 Maximum Head

The maximum head of new concrete ground or elevated tanks should be 12 m. No standpipes shall be constructed.

8.4 Technology Transfer

The principal conclusions and recommendations in the tank applied research reports should be made available to the engineering profession, consultants, and operations personnel, by means of reports, seminars, papers, etc. so that repairs underway, and future concrete tank design and construction, and maintenance, might benefit from the work.

8.5 Durability

Further development is required in design, construction, and materials to optimize the economy of a new breed of insulated concrete tanks in the following areas:

- concrete mix designs and methodology to result in minimum construction defects and a minimum coefficient of permeability.

- design for better concrete placement

- improved wall/floor joint details

- development of a thin (3 mm thick) stainless steel liner (no maintenance) waterproofing system for concrete tanks using normal concrete mixes.

8.6 Further Applied Research

- The models presented in chapter 4 of this report should be verified under laboratory conditions. Development of a laboratory model will enable greater understanding of the rate of concrete deterioration under varying hydrostatic pressures and saturated freeze-thaw conditions. It will also permit preventative measures such as concrete permeability, liners, coatings, penetrating sealers and imperfections in these types of barriers to be evaluated accurately.

- The potential to passively maintain the tank inlet water at temperatures greater than 2°C should be investigated. Possible sources of heat gain would be the source water and the feeder pipe system. Seasonal fluctuations at these locations should be monitored to optimize system design and minimize the energy costs accrued during the service life of the tank.

- Cost effective and less disruptive methods of internal inspection of water tanks should be developed in order to allow regular inspection and minimize maintenance costs. Remotely operated underwater vehicles equipped with video cameras may allow inspections to be carried out without having to empty tanks and disinfect after the inspection has been completed.

- Early warning of any undesirable or potentially dangerous conditions should be made available to operators. Where possible these should be remotely sensed and controlled to avoid the necessity of manual inspection during severe winter conditions. Currently this may require the operator to climb 45 m on external ladders at temperatures below -25°C. Continuous monitoring of water inlet and outlet, concrete wall and the temperature of other significant locations as well as viewing internal ice and wall coating conditions etc. could be made available in the water plant by installing equipment developed from monitoring technology already used in the tank rehabilitation programme.

9.0 LIST OF REFERENCES

1) Slater, W.M., & Associates Inc. Interim Report on Ontario Concrete Water Retaining Structures". Ministry of Environment of Ontario, Project WTS-001, 1982

2) Slater W.M., "Concrete Water Tanks in Ontario". Proceedings, Canadian Society of Civil Engineering 1983 Annual Conference, June 1-3, 1983

3) Ministry of Transportation and Communications of Ontario. "Bridge Deck Rehabilitation, Manual", Part One, Downsview 1983.

4) Helmuth, R.A., "Capillary Size Restriction on Ice Formation in Hardened Cement Pastes", Proceedings, Fourth International Symposium on the Chemistry of Cement, Washington, D.C., V.2, 1960.

5) Fagerland, G., "The Significance of Critical Degrees of Saturation at Freezing of Porous Brittle Materials". ACI Publication SP-47, American Concrete Institute, Detroit, 1975.

6) Beaudoin, J.J. and MacInnis, C., "Dimensional Changes of Hydrated Portland Cement Mortar Due to Slow Cooling and Warming". ACI Publication SP-47, American Concrete Institute, Detroit, 1975.

7) Kays, W.B., "Construction of Liners for Reservoirs, Tanks, and Pollution Control Facilities", John Wiley & Sons, 1977

8) Gray, W.S., "Reinforced Concrete Water Towers, Silos, and Gantries". Concrete Publications Limited, 1953

9) Hazlehurst, J.N., "Towers and Tanks for Water Works",John Wiley & Sons, 1910

10) Creasy, L.R., "Prestressed Concrete Cylindrical Tanks",John Wiley & Sons, 1961

11) Manning, G.P., "Concrete Reservoirs and Tanks", Concrete Publications Limited, 1967

12) Gray, W.S., "Reinforced Concrete Reservoirs and Tanks",Concrete Publications Limited, 1954

13) Powers, T.C., "Freezing Effects in Concrete" ACI Publication SP-47 American Concrete Institute, Detroit, 1975.

14) Verbeck, G., "Significance of Tests and Properties of Concrete and Concrete - Making Materials" Chapter 18. ASTM Publications STP 169B, 1978.

15) Carrier, R.E. and Cady, P.D., "Moisture Distribution in Concrete Bridge Decks and Pavements" ACI Publication SP-47, American Concrete Institute, Detroit, 1975.

16) Mills, R.H., "The Permeability of Concrete for Reactor Containment Vessels", Report 84-01 University of Toronto, July, 1983.

17) Powers, T.C., "A Working Hypothesis for Further Studies of Frost Resistance of Concrete". ACI Journal, Proceedings, V. 41, No.3, February 1945.

18) "Frost Durability of Clay Bricks - Evaluation Criteria and Quality Control" - Proceedings No.8 CBAC/DBR 1984

19) Neville, A.M., "Properties of Concrete" Pitman Publishing Company, 1974.

20) MacInnis, C. and Whiting, J.D., "The Frost Resistance of Concrete Subject to a De-icing Agent", Cement and Concrete Research, Vol.9, 1979.

21) Neville, A.M., "Creep of Concrete: Plain, Reinforced and Prestressed", North-Holland Publishing Co., Amsterdam, 1970.

22) Khalil, S.M., Ward, M.A., and Morgan, D.R., "Freeze-Thaw Durability of Non Air-Entrained High Strength Concretes Containing Superplasticizers", Proceedings, First International Conference on Durability of Building Materials and Components, Ottawa, Canada, August, 1978.

23) Marusin, S., "The Effect of Variation in Pore Structure on the Frost Resistance of Concrete, Cement and Concrete Research, Vol. 11, 1981.

24) Litvan, G.G., MacInnis, C., and Grattan-Bellow, P.O., "Cooperative Test Program for Precast Concrete Paving Elements", Proceedings, First International Conference on Durability of Building Materials and Components, Ottawa, Canada, August, 1978.

25) Adkins, D.F. Laboratory Duplication of Surface Scaling. Concrete International Vol 8 No. 2 February, 1986

26) National Building Code of Canada, 1985.

27) Wood, J.H., Adams, J.R. "Temperature Gradients in a Cylindrical Concrete Reservoir", Proceedings of the 6th Australasian Conference on the Mechanics of Structures and Materials, Christchurch, N.Z. 1977.

28) Tuomioja, M., Jumppanen, P. and Rechardt, T., "Jaan Lujuudesta ja Muodonmuutoksista" (The Strength and Deformation of Ice), Rakennustekniikka, 1973:1 pp 233-246.

29) Bergdahl, L., "Thermal Ice Pressure in Lake Ice Covers", Report A2, Department of Hydraulics, Chalmers University of Technology, Sweden, 1978.

30) Reynolds, C.E., "Reinforced Concrete Designer's Handbook", Concrete Publications Ltd., London, 1961.

31) Ghali, A., "Circular Storage Tanks and Silos", Spon Ltd., London, 1979.

32) Portland Cement Association, "Circular Concrete Tanks without Prestressing".

33) Markus, G.Y., "The Theory and Computation of Circular Symmetrical Structures".

34) Hibbitt, Karlsson and Sorensen, Inc., "ABAQUS - Structural and Heat Transfer Analysis Program".

35) Schupack, M., 1982. Protecting Post-tensioning Tendons in Concrete Structures, Civil Engineering - ASCE.

ACI Committee 344. 1970. Design and construction of circular prestressed concrete structures. ACI Journal, September, pp/ 657-672.

ACI Monograph No. 3, 1966, "Freezing and Thawing of Concrete - Mechanisms and Control", Detroit.

Aldworth, G., Staton, R., (MacLaren Engineers), and Slater, W.M., (W.M. Slater & Associates Inc.). Preliminary, (1985). "Freeze Protection for Above Ground Concrete Water Tanks in Cold Regions".

ASCE Civil Engineering. 1981. Structural failures. New York, December, p. 44.

Beeby, A.W. 1978. Cracking: what are cracking limits for? Concrete, London, July, p. 3.

Campbell, T.I., and Kong, W.L., (Queen's University). April, (1986). "Ice Loading in Elevated Water Tanks".

Carpenter, C.H. 1982. Constructing and maintaining distribution storage structures. AWWA Journal, Vol. 74, No. 11, Denver, Col., pp. 581-583.

Cedolin, L., Poli, D., Ioro, I. 1983. Experimental determination of the stress-strain curve and fracture zone for concrete in tension. Proceedings of the International Conference on Constitutive Laws for Engineering Materials, University of Arizona, Tucson, Arizona, pp. 393-398.

Code of practice for concrete structures for the storage of liquids. 1978. Part I, design based on "resistance to cracking" approach, Wellington, N.Z.

Crowley, F.X. 1976. Maintenance problems and solutions for prestressed water tanks. AWWA Journal, Denver, Co., November, pp. 579-585.

Frederking, R., Sinha, N.K. 1977. Ice action on wharf at Strathcona Sound. Proceedings of Fourth International Conference on Port and Ocean Engineering under Arctic Conditions, Memorial University of Newfoundland, St. John's, pp. 707-717.

Gold, L.W. 1958. Some observations on the dependence of strain on stress for ice. Canadian Journal of Physics, Vol. 36, Ottawa, Ontario, pp. 1265-1275.

Gold, L.W. 1960. The cracking activity in ice during creep. Canadian Journal of Physics, Vol. 38, No. 9, Ottawa, Ontario, pp. 1137-1148.

Gold, L.W. 1965. The initial creep of columnar-grained ice. Part I: Observed behaviour. Part II: Analysis. Canadian Journal of Physics, Vol. 43, Ottawa, Ontario, pp. 1414-1422 and 1423-1434.

Gold, L.W. 1966. Observations on the movement of ice at a bridge pier. Proceedings of a Conference on Ice Pressures against Structures at Laval University, Quebec City, pp. 135-141.

Gold, L.W. 1966. Elastic and strength properties of fresh-water ice. Proceedings of the Conference on Ice Pressures against Structures at Laval University, Quebec City, pp.13-23.

Grieve, R. (Golder Associates). May, (1984) and February, (1986). "Temperature Monitoring, Ontario Concrete Water Tanks".

Harper, W.B. 1982. Inspection, painting, and maintaining steel water tanks. AWWA Journal, Vol. 74, No. 11, Denver, Col., pp. 585-587.

Hassett, R.W. 1980. Design and construction of water storage facilities. Strand Associates, Inc., Madison, Wisconsin.

Hertzberg, L.B., Westerback, A.E. 1976. Maintenance problems with wire-wound prestressed concrete tanks. AWWA Journal, Denver, Col., December, p.44.

Krajcinovic, D., Selvaraj, S. 1983. Constitutive equations for concrete. Proceedings of the International Conference on Constitutive Laws for Engineering Materials, University of Arizona, Tucson, Arizona, pp. 399-406.

Krausz, A.S. 1963. The creep of ice in bending. Canadian Journal of Physics, Vol. 41, No. 1, Ottawa, Ontario, pp. 167-177.

Krausz, A.S. 1966. Plastic deformation of fresh-water ice. Proceedings of the Conference on Ice Pressures against Structures at Laval University, Quebec City, pp. 5-12.

Krausz, A.S. 1981. Adfreeze strength of model piles in ice. Canadian Geotechnical Journal, Vol. 18, No. 1, pp. 8-16.

Kronen, H., Anderson, J.H. 1982. Concrete exposed to cryogenic temperature, Nordisk betong, Norway, pp. 13-17.

Larrabee, R.D., Billington, D.P., Abel, J.F. 1974. Thermal loading of thin-shell concrete cooling towers. ASCE Journal, Structural Division, December.

Mackenzie, K., (Dalhousie Materials), Slater, W.M., (W.M. Slater & Associates Inc.) and McGrenere, P., (Knox Martin Kretch) (Editing). Preliminary, (1985). "Evaluation of Waterproof Coatings for Concrete Water Tanks".

Perkins, P.H. 1978. Concrete structures: repair, waterproofing and protection. Reprinted. Applied Science Publishers Ltd., Barking, Essex, England, 302 p.

Pitkanen, A. c. 1980. Roihuvuori water tower, Helsinki, Finland. Publication unidentified, 2 p.

Priestley, M.J.N. 1976. Ambient thermal stresses in circular prestressed concrete tanks, ACI Journal, October.

RILEM (Reunion internationale des laboratoires d'essais et de recherches sur les materiaux et les constructions). 1981. Materials and structures. Properties of set concrete at early ages-state-of-the-art report, Vol. 14, No. 84, Paris, France, pp. 399-449.

Schupack, M. 1981. How to inspect and evaluate prestressed concrete tanks. Public Works, September, pp. 88-89.

Schupack, M., Suarez, M. 1981. Some recent corrosion embrittlement failures of prestressing systems in the United States. ACI Journal, Vol. 27, No. 2, Chicago, Illinois, pp. 38-56.

Slater, W.M. 1975. Stage post-tensioning-a versatile and economic construction technique. PCI Journal, Vol. 20, No. 1, Chicago, Illinois, pp. 14-27.

Slater, W.M. & Associates Inc. 1982. Study of concrete elevated water tanks. Part 1, Analysis and evaluation of defects using a numerical coding system. Part 2, Interim guideline recommendations. Report 3 (for review). Project WTS-001, Toronto Ontario, 111 p.

Timoshenko, S., Woinowsky-Krieger, S. 1959. Theory of plates and shells. McGraw-Hill.

Vessey, J.V., Preston, R.L. 1978. A critical review of code requirements for circular prestressed concrete reservoirs. FIP 8th Congress, London.

11.0 APPENDIX A

INTERIM GUIDELINE RECOMMENDATIONS OF MINIMUM REQUIREMENTS FOR THE DESIGN AND CONSTRUCTION OF NEW ABOVE GROUND CONCRETE WATER TANKS IN ONTARIO

11.1 Introduction

This report has concluded that under high pressure, the permeability of normal structural concrete is insufficient by itself, to eliminate the deterioration of the walls of a concrete water tank indefinitely, in the environmental conditions prevailing in Ontario. It is, therefore, essential that either the concrete is prevented from attaining a high degree of saturation, or that cyclic freezing and thawing is eliminated.

Based on the experience gained to date, reduction of severe freezing and thawing by means of external insulation is the more positive protection. It is less dependent on the site and seasonal conditions, and is less costly than the application of waterproof coatings and steel liners. A second, but important benefit is the elimination of the internal formation of ice round the tank walls. In addition, small deviations from specification do not have radical consequences to the expected life of the tank. Since the insulation is external to the tank structure, it can be rectified without undue interruption to normal operations. Conversely, protecting a tank by means of an internal coating only, is less assured, since it is difficult to install, may have a limited life, and will not prevent the internal formation of ice. Internal repairs result in costly delays to normal operations.

The following interim guidelines will result in concrete water tanks which have higher initial, but reduced maintenance costs than have been experienced. It is the intention of the guidelines to recommend standards which will ensure the construction of a tank which will not deteriorate during its expected service life of fifty (50) years (minimum), and should not require costly maintenance. It is recommended, therefore, that alternate designs should be compared on the basis of initial and maintenance costs.

11.2 Scope

These recommendations are not "stand-alone" and are limited to the primary design and construction requirements of new concrete water retaining structures only, and exclude details of appurtenances and the system, except where noted.

11.3 References

The recommendations are additional to and take precedence over the requirements of the following listed codes, standards, and references:

- Province of Ontario
  Building Code. O. Reg. 419/86
  August 6, 1986.

- The National Building Code of Canada (1985).

- Supplement of NBC (1985).

- CSA Standard CAN3-A23.1-M77, CAN3-A23.2-M77
  Concrete Materials and Methods of Concrete Construction
  Methods of Test for Concrete.

- CSA Standard CAN3-A23.3-M84. Design of Concrete Structures for Buildings.

- ACI Standard 318-83 Building Code Requirements for Reinforced Concrete.

- ACI Committee 344 Report on the Design and Construction of Circular Prestressed Concrete Structures (1970).

- ACI Committee 350 Report on Concrete Sanitary Engineering Structures (1977).

- AWWA Standard for Welded Steel Tanks for Water Storage ANSI/AWWA D100-79.

- Ministry of Environment Standard Specifications.

11.4 Design

11.4.1 Design Philosophy

In addition to the strength requirements of the specified codes, standards and references listed in section 11.1.3 for Applied Loads, design for the following conditions shall be carried out:

- Environmental loads (ice, thermal differential, etc.)

- Durability for the actual freeze-thaw environment.

- Provide a service life goal with minimum maintenance, of fifty (50) years (minimum).

11.4.2 Tank Design Requirements

.1 Insulation

Concrete tanks shall be protected from freezing and shall be insulated and clad on the exterior to reduce heat loss and prevent significant internal ice formation.

.2 Ice Prevention

- An ice protection system shall be designed to prevent the tank water from freezing during the worst winter conditions anticipated during the service life of the structure (50 years).

- Actual tank heat loss is dependent on the quality of the construction and air tightness of the insulation, therefore, heat loss calculations shall be made assuming a minimum tank water temperature of 2°C, unless more accurate data is available and based upon temperature monitoring results of similar systems and materials.

.3 Inspection

The cladding and insulation shall be demountable to allow exterior inspections and maintenance.

.4 Mixing/Heating

- Mixing and booster heating of the water in some tanks with low turnovers may be necessary to supplement the heat loss reduction from the insulation for short periods during severe winters.

- The provision of heated air in the air gap between the insulation and the concrete is an alternative to the above where it is important that the concrete surface temperature does not fall below 0°C.

.5 Prestressed Construction

All concrete water tanks shall be of prestressed construction.

.6 Prestress Design

Concrete water tanks shall be post-tensioned in the vertical and horizontal directions with the following reserve compression after service loads are considered:-

- Reserve Compression (Hoop)

The minimum final (after losses) reserve hoop (circumferential) prestress shall be 1.5 MPa (214 psi).

- Reserve Compression (Vertical)

The minimum final (after losses) reserve vertical prestress shall be .5 MPa (71 psi).

- Post-tensioning

Post-tensioning shall be carried out using internally bonded (grouted) tendons, or external tendons and hardware permanently protected against corrosion.

.7 Concrete Wall Thickness

The minimum thickness of the concrete wall shall be 200 mm (8 in.).

.8 Waterproofing

Concrete water tanks shall be maintained permanently leaktight using impermeable concrete, bonded waterproof coatings, steel liners or other liners of impermeable material proven to be durable in chlorinated water.

.9 Coatings

- Waterproof coatings shall be 100 per cent solids epoxies or a modified cementitious slurry with fabric and shall be applied to all concrete in contact with water.

Epoxy coatings shall be applied on dry concrete in a minimum of 3 coats (each coat 6 mils minimum) on top of surfacing mortar when the original surface exceeds a critical roughness or exhibits too many surface pin holes.

- The permeability of the concrete for walls to be waterproofed with bonded coatings shall not exceed 2.5×10^{-13} m/s tested at 90 days.

.10 Liners

- Where plain or stainless steel is used as an internal waterproof liner it shall be 5 mm (3/16 inch) minimum thickness. Where plain steel is used, it shall be protected on the face in contact with the water by an MOE approved single component epoxy or vinyl paint system.

- The space between the liner and the concrete wall shall be filled with an MOE pre-approved cementitious grout having a minimum pH value (hydrogen ion value) of 12.

11.5 Construction

11.5.1 Concrete Quality

- The concrete mix, wall design, and construction shall result in a crack free and leaktight wall. No cold joints will be permitted without waterstops being installed.

11.5.2 Slip Forming

- Jack-rod pipes used for jacking up the forms shall be removed and the wall void formed by the pipes grouted up from the bottom to the top, or the pipe sections shall be completely grouted individually from bottom to top.

- Locations of the jack-rods shall be recorded on as-built shop drawings.

- Details of grout vents and methods of grouting jack-rods shall be approved by the authority before construction of the tank.

- Horizontal micro-cracking caused by binding of the slipform shall be prevented, or if they occur they shall be repaired by routing out and filling with epoxy mortar.

11.5.3 Jump Forming

- Pour heights shall not exceed 2.5m (8 ft. 2 inches).

- Form vibrators clamped to the forms shall be used in addition to internal vibrators.

- Waterstops shall be installed at the top (and bottom) of each lift.

11.5.4 Vertical Waterstops

- Only plain, de-greased, mild steel waterstops shall be used.

- Minimum dimensions shall be 200 mm (8 ins.) for the base and 150 mm (6 ins.) high elsewhere, and 6 mm (1/4 in.) thick.

- Vertical joints in steel waterstops shall be continuously lap welded (fillet).

11.5.5 Concrete Joint Preparation

- Construction joints shall be sandblasted to remove all laitance, before placing fresh concrete.

11.5.6 Concrete Curing

- Continuous wet curing and covering of the concrete with burlap or tarpaulins shall be carried out for a minimum of 7 days after stripping the forms.

11.6 Quality Assurance And Tank Performance Testing

11.6.1 Leakage Testing

- A 3 day leakage test shall be carried out and any leaks or damp spots repaired and retested, if necessary, before applying waterproof coatings or exterior insulation.

11.6.2 Final Inspection of Structure

- An external maintenance inspection to prove that the tank is leaktight and sound, and an internal inspection to prove that the waterproof coating or liner is satisfactory, shall be made after the tank has been in continuous service for a period of between 9 and 12 months.

- Any deficiencies shall be repaired and the tank re-inspected after a further equal period in service.

11.6.3 Heat Loss and Ice Prevention

- External ambient temperatures, temperatures in the air gap between the insulation and the wall, and the tank water temperatures shall be measured during an extreme cold period of the first winter of tank operation.

- Verify that the insulation is airtight, and that the freeze protection system is functioning satisfactorily, according to the design and heat loss calculations.

- Sensors installed in the locations described above are recommended for temperature monitoring and control of water temperature.

- Inspect top surface of water at wall in late winter to observe that significant ice formation(s) does not exist.

11.7 Security And Safety

11.7.1 Security Fence

- Tanks should be enclosed within a perimeter security fence to prevent vandalism to the cladding, and insulation (fire), climbing by unauthorized persons, etc., and to provide protection from falling objects, ice, etc., and during maintenance or repair operations.

11.7.2 Safety

- New concrete water tanks should be located at a minimum distance of twice (x 2) their height from public thoroughfares or at least three times (x 3) their height from occupied buildings. This will increase the safety of the public during demolition or during a major disaster causing collapse or toppling of the tank.

11.8 Miscellaneous And Appurtenances

- Aluminum. Permanent aluminum ladders shall not be used in contact with chlorinated water. Use galvanized steel or fibre glass.

- Inlet/Outlet Pipes. Separate inlet and outlet pipes to provide some natural mixing of the tank water.

APPENDIX B

DILATION EXPANSION OF SPHERICAL REGION WITHIN A LARGER SPHERE

A confined saturated spot dilating under freezing will result in a considerable internal pressure which can cause fracture of concrete. The magnitude of resulting stresses will strongly depend on geometrical dimensions and the shape of a saturated zone. A variety of qualitative effects and magnitudes of stresses caused by expansion of confined zones can be investigated by considering expansion of a spherical region confined within a sphere of a larger diameter. Due to the symmetry of the problem, an analytical solution for stresses and deformations is possible.

It is physically clear that the expansion of an inner sphere within a larger sphere will result in a compressive lateral stress accompanied by tensile hoop stresses. Maximum tensile stress will be at the interface of the dilating and non-dilating zones. If the tensile stress is large enough, brittle fracture will be initiated in concrete. Further increases in the dilation will cause fracture propagation. Due to the symmetry of the problem, the fractured zone can be assumed to be spherical. In general, there could be three different zones (Figure 12.1):

- Dilating zone - radial and hoop compression (zone A)
- Fracture zone - radial compression, zero hoop stress (zone B)
- Elastic zone - radial compression, hoop tension (zone C)

Stresses and displacements in each of the above zones can be determined by solving equations of static equilibrium and geometrical compatibility. The location of the interface between fractured and elastic zones can be found from the condition that the tensile strain in the elastic zone does exceed the limiting tensile strain at fracture. Figure 12.1 illustrates various features of the solution.

Pressure at the interface of the dilating and non-dilating zones is proportional to dilatant strain and increases with the thickness of the non-dilating material covering the dilating region (Figure 12.1a).

Fracture at the interface is initiated at a certain dilatancy (Figure 12.1b). The more confined the dilating zone is, the larger the dilatancy required to initiate fracture.

Further increase in dilatancy causes fracture propagation. When dilatancy reaches a certain "critical" value, the process of fracture propagation becomes unstable and the entire sphere is ruptured (Figure 12.1c).

The dilatant strain which causes the rupture increases with the thickness of non-dilating material (Figure 12.1d).

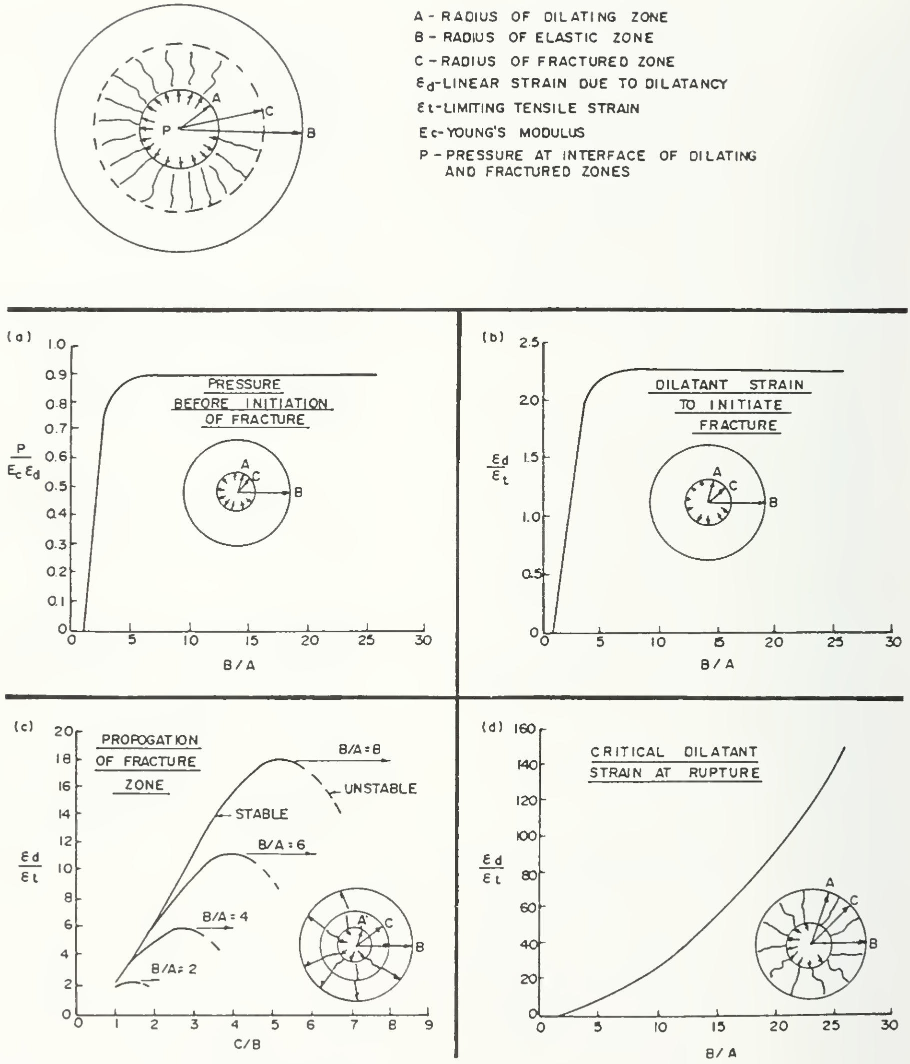


Figure 12.1 *Rupture of a brittle sphere due to dilation*

13.0 APPENDIX C

TERMS OF REFERENCE

The terms of reference included as Appendix 'A' in the agreement between the Ministry of the Environment and W.M. Slater & Associates Inc. for "The Investigation of Elevated Concrete Water Retaining Structures" dated June 24, 1981 were as follows:

1) Inspect approximately 30* concrete water retaining structures in all six MOE Regions to establish their condition, define deficiencies, recommend remedial measures and develop standard methods for repairs where warranted.

2) Study available material on record for each structure inspected and relate the findings to the condition of the structure at the time of inspection.

3) Prepare a report on each structure investigated and inspected including the method of effecting permanent repairs together with associated costs.

4) Make necessary arrangements for local co-ordination of investigations including testing of materials and any other analyses.

5) Prepare design and maintenance guidelines including standard methods of repairs and inspections.

6) Record all findings and submit a "General Study Report"in a format suitable for general distribution.

7) Identify structures requiring urgent repairs before the onset of winter 1981 and recommend courses of action to protect such structures.

8) If required, prepare specifications for tendering repairs and perform the supervision of construction related thereto.

9) Act as expert witness for the Crown, if required.

Explanatory Note

In response to items 4, 5, 6, 7 and 8 in the above terms of reference, the report on "Immediate Research Needs" dated October 20, 1982 was prepared. Funds were subsequently made available to carry out applied research in the areas of structural effects of freezing, ice (freezing) prevention, and leakproofing. This report records the observations made and the applied research carried out in the area of freeze-thaw failure mechanism. It was later decided to expand the terms of reference to include sections on the deterioration of metals, and the repair of concrete tanks.

* Provincially owned at the time of construction