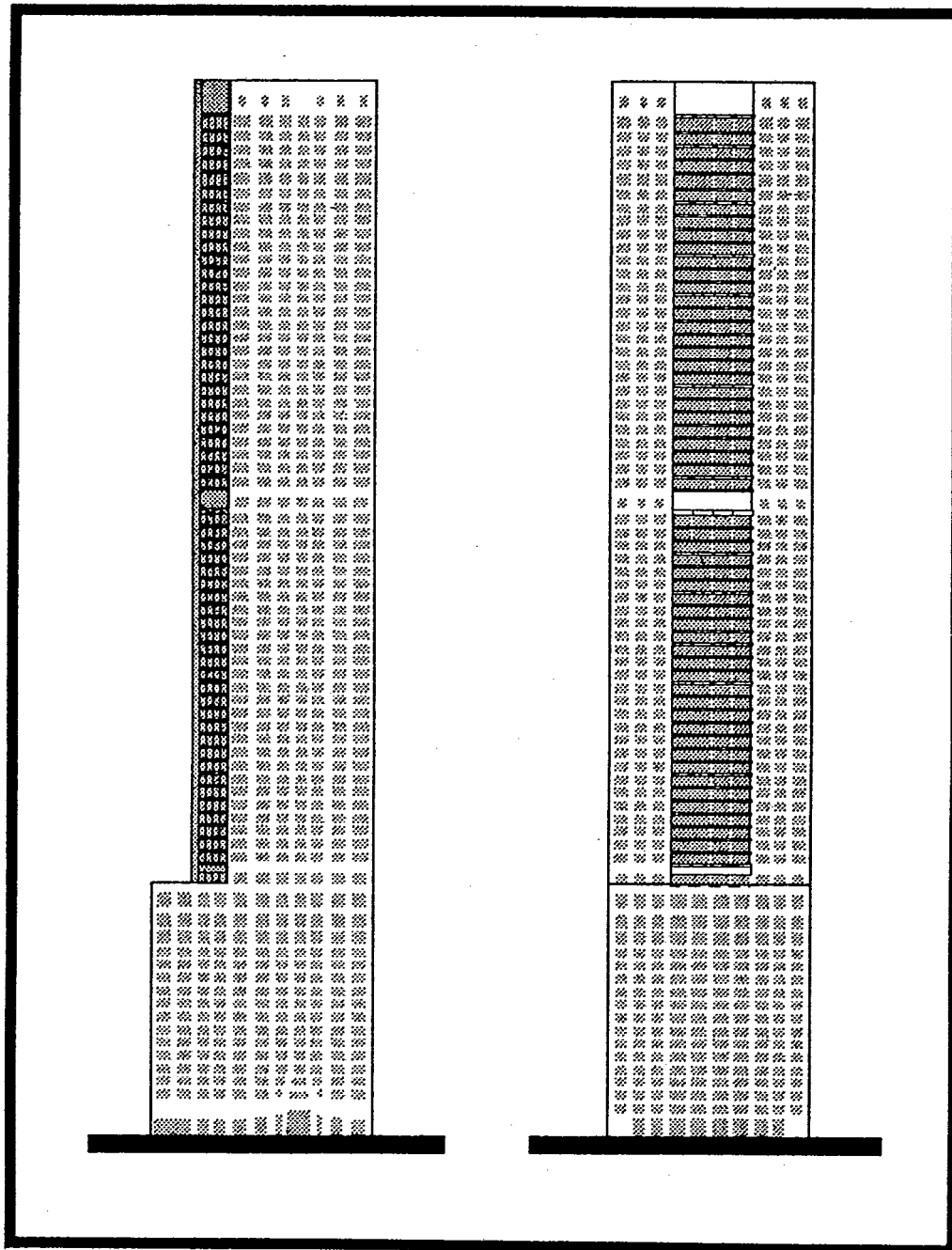


# Long-Term Serviceability of Concrete Structures



Anis Farah  
*Editor*

American Concrete Institute, Detroit



SP-117  
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Printed in the United States of America

Editorial production: Patricia Kost

LIBRARY OF CONGRESS CATALOG CARD NUMBER 89-80359

## PREFACE

This volume includes papers presented at a symposium on long-term serviceability of concrete structures held at the annual convention of the American Concrete Institute, November 1987 in Seattle, Washington. It also includes other papers which were received but could not be presented due to time limitations.

The objective of the symposium was to stimulate discussion and enhance understanding among researchers and practicing engineers involved in work requiring the most up-to-date data and techniques for assessing the long-term serviceability of new and existing structures. The topics covered include rehabilitation; tall buildings; bridges; and earthquake, nuclear, and cylindrical structures.

All thirteen papers in this volume were reviewed in accordance with the ACI review procedure. The cooperation of the authors and the reviewers in meeting a tight schedule made this publication possible.

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Reinforced Concrete  
Cylindrical Storage Structures  
for Aqueous Materials in Cold Climates

by T.I. Campbell, J.C. Jofriet, and W.M. Slater

**Synopsis:** A study of concrete water tanks in the Province of Ontario indicated an unusually high rate of deterioration. The different types of tanks in existence are described. Observed defects and the possible related mechanisms are discussed. Particular attention is directed to freeze-thaw cycles and internal ice formations and methods for estimation of these effects are proposed. Criteria and recommendations for the design of reinforced concrete storage structures in both freezing and non-freezing environments are discussed.

**Keywords:** deterioration; freeze-thaw durability; ice formation; load factors; low temperature; reinforced concrete; structural design; water tanks

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### INTRODUCTION

A study (1, 2), commenced in 1981, of 53 concrete water tanks in the Province of Ontario indicated an unexpectedly high rate of deterioration. It was expected that the life of a water tank should be in excess of 50 years, but most of the tanks studied were less than 9 years old with an average age of about 6 years. Two water tanks had failed prior to initiation of the study.

As shown in Fig. 1, there are essentially three different categories of water tanks: standpipes, elevated tanks and 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 tank. 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.). Ground tanks have a 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 as low head tanks. Within each of above three categories, as listed in Table 1, there are three main structural types,

namely, non-prestressed reinforced concrete, post-tensioned wire wound concrete and post-tensioned concrete with tendons, either bonded or unbonded.

Performance ratings of the 53 tanks were determined by means of surveys using measurement standards (1) for rating the condition of all major components. These surveys indicated that standpipes had the worst performance and ground tanks the best.

This paper describes ten of the main defects found in the tanks. These defects may be attributed primarily to inadequate design criteria and faults in construction. Possible mechanisms causing the various defects are described in general with particular attention directed to the effects of freeze-thaw cycles and internal ice formations. Design recommendations for cylindrical storage containers located in either a non-freezing or a freezing environment are discussed.

#### DEFECTS IN TANKS

The ten main defects revealed in the surveys of the tanks are listed in Table 2. It was found that each tank had its own deterioration peculiarities. However, several types of defects can be associated with individual tank types.

##### Wall Delamination

Vertical wall delamination, or splitting of the concrete in the plane of the wall, has been observed in all types of tanks. The mechanism and causes are similar, but may vary according to the type of construction.

Large areas of vertical delaminations have been observed in non-prestressed reinforced concrete standpipes. The splitting cracks are located at the centre of the tensile hoop reinforcement, thereby separating the wall at these locations into two or three individual cylindrical surfaces. The walls of these delaminated standpipes are radially cracked on vertical lines at about 250mm (10 in.) centres and generally leak when temperatures are above freezing. Delaminations have been observed to grow in freezing weather. Mechanisms by which these delaminations may be formed are discussed later.

Delaminations centred on ungrouted post-tensioning tendon ducts have also been observed. It is assumed that water under pressure gradually penetrates the wall and fills the ungrouted void. On freezing, the confined ice expansion within the void causes both vertical (delamination) and horizontal cracking through the wall. This cracking may cause explosion of a tank when full, or implosion as a tank is being emptied.

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Vertical delaminations have been found in gunited (shotcreted) walls. It is assumed that these cracks are at weak planes where individually placed layers of gunite have not bonded together because of hardening of the first layer before the new fresh layer is sprayed on. Freezing of water which has penetrated into these cold joints can cause cracking.

##### Vertical Cracks in Wall

Vertical cracks have been observed over the entire height of the walls in non-prestressed reinforced concrete standpipes under normal water loads. The width of the cracks and rate of leakage from the cracks depend on a number of factors including, but not limited to, concrete strength at time of filling, tensile stress in the hoop reinforcement and type or lack of waterproofing. While cracking in the lower regions of the wall may be due to water pressure, the vertical cracks in the top regions are thought to have been caused by the expansion of internal ice formations. The effects of internal ice formations are discussed later.

Vertical cracks have occurred at the bottom of a ground tank due to ineffective prestressing caused by restraint against inward movement at the base during construction.

##### Wall/Floor Joint Deterioration

Both sliding and fixed wall-floor joints have exhibited leakage and local deterioration near waterstops. Leakage is caused by poor installation of the waterstop, holes in the waterstop and permeable concrete around the waterstop. Severe deterioration of concrete on the inside of waterstops has been observed. This is caused by freeze-thaw damage of the concrete due to the waterstop acting as a dam, allowing complete saturation of the concrete.

##### Vertical Voids in Shotcrete

Voids outside the vertical reinforcement and under the prestressing wires has been observed in gunite protected tanks. This is caused by poor workmanship allowing the reinforcing bars to act as shields which block the mortar spray and thereby cause the voids.

##### Jack Rod Spalls

Some tanks had been constructed using a slip form method. Hollow pipe rods, known as jack-rods, each approximately 3 m (10 ft.) in length are used to raise the slip form. As construction proceeds, additional lengths of rod are added using threaded solid coupling rods. This results in hollow tubes, interrupted at each joint by a solid rod, spaced at approximately 3 m (10

ft.) centres around the wall and extending the full height of the tank.

Many large conical spalls have occurred at the joints in the hollow jack rods. In some tanks, little attempt was made to seal these jack rods at the top of the tank and water was able to enter the uppermost jack rod section. This water leaks out at the threaded couplings in each jack rod and saturates the surrounding concrete. When water in the jack rod freezes, the ice expansion combined with freeze-thaw action, causes high concentrated forces in the middle of the wall, resulting in explosive conical spalling at the location of coupling with a cone height equal to half the wall thickness. Internal water under pressure can then enter the jack rod at the location of a spall causing even more rapid deterioration at all couplings over the length of a jack rod.

#### Shotcrete Cover Coat Delamination

On some tanks where a cover coat of shotcrete has been applied to the prestressing wires, delamination of the cover coat has occurred where the applied mortar layer is too thin approximately 10 mm (0.5 in.). Cover coat delamination has also been observed in local areas where the spacing of the prestressing wires is too small. Close wire spacing does not allow adequate bonding among the cover coat layers since the smooth wires act as a debonding medium. Thick cover coats, greater than 25mm (1 in.), have not exhibited delaminations.

#### Waterproof Coating Failure

Most failures of the internal waterproof coating have been caused by lack of adequate bond of the coating to the concrete wall. This may be attributed to a number of factors such as the use of poor materials, poor workmanship, poor surface preparation, and application of the coating on damp concrete surfaces, or under environmental conditions of high humidity and low or high temperatures. Many coating failures may be attributed to freeze-thaw action at the interior surface.

#### Shotcrete Cover Coat Cracking

Cracking of the shotcrete cover coat can be attributed to shrinkage as a result of rapid drying and/or insufficient or lack of curing of the young concrete mortar.

#### Cold Joints and Horizontal Cracks

Leaking cold construction joints in jump-formed concrete tanks are common, and are mainly caused by lack of waterstops and/or poor workmanship and preparation at the joints. Cracks at horizontal joints are common and are caused by shrinkage due

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to lack of curing or by temperature differential through the wall. Horizontal cracks resulting from the lack of vertical reinforcement in the wall have been observed at locations of high moment resulting from temperature differential, particularly at a region where a stiff floor slab joins the wall in an elevated tank (21).

### Corrosion of Prestressing Steel

Loss of the protective gunite cover coat from the prestressing wires in wire wound tanks 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 the strands. Lack of grout in ducts has allowed water to enter causing general corrosion of the prestressing steel.

It can be seen that, in general the observed defects in the tanks surveyed are due to inadequate design criteria and/or faults in construction. The influence of a freezing environment on the observed deterioration is apparent. The effects of freeze-thaw cycles and internal ice formations on the behaviour of a tank are discussed below. Methods of repairing defects in water tanks are outlined in Ref. (3).

### STUDIES OF MECHANISMS CAUSING DEFECTS

A number of studies were undertaken in an attempt to explain the causes of some of the observed defects. Two of these studies, namely temperature monitoring and the effects of freeze-thaw cycles, are discussed in Ref. (3), while a third, the effects of internal ice pressure, is reported in Ref. (4). These three studies are described briefly below.

#### Temperature Monitoring

A temperature monitoring system was installed in three tanks, two with insulation and one without, to provide information relating to the climatic influence on the tanks.

Figure 2 shows data obtained from an uninsulated tank during the winter period. Figure 2(a) indicates that temperatures in the north exposure are almost continuously below freezing throughout January, while from Fig. 2(b), it can be seen that freeze-thaw cycling occurs almost daily in the south exposure during the same period.

Further, 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. Water quickly freezes close to the wall of the tank forming an ice ring around the inside of the wall. Temperature variation in the water indicated that an ice cap forms at the top of the tank. Visual observations confirmed the presence of the ice cap throughout the winter period.

Figure 3 shows a graphical record of temperature variation during the winter period for an insulated tank. The insulation consisted of rigid styrofoam sheets covered by corrugated metal cladding. An air gap was left between the styrofoam and the wall of the tank. The significant moderating influence of the insulation is obvious. The temperature fluctuation in the air gap is about one third that of the external ambient temperature. The temperature of the water in the tank is largely independent and isolated from the ambient conditions.

Temperature data obtained from the uninsulated tank indicate that freeze-thaw cycles occur in the wall, particularly in the south exposure. Also it was seen that ice formations, in the form of an ice cylinder on the inside of the wall or an ice cap at the top surface, may exist. The effects of freeze-thaw cycles and the presence of these ice formations and their relations to possible deterioration of the tank wall are now discussed.

#### Effects of Freeze-Thaw Cycles

The surveys revealed that the internal waterproofing system had broken down, allowing water under pressure to penetrate the walls, resulting in saturation of the concrete. Saturated concrete expands when it freezes and a residual expansion or dilation is produced upon thawing (5).

Figure 4 shows a situation where the outside part of a wall of a concrete tank dilates due to freezing and is in compression, while the inside part is unaffected by frost and is in tension. Restraint to this tendency of differential movement induces radial tensile stresses which vary throughout the thickness of the wall as shown in Fig. 5. The maximum tensile radial stress occurs at the interface between the dilating zone and the inner zone of the wall and attains a peak value when the dilating zone occupies one half the wall thickness.

The build-up of radial tensile stress predicted by a mathematical model (3) for a typical tank wall is shown in Fig. 6. The model assumed that 60 cycles of freezing and thawing occur during a two month period each year with frost penetration to the middle of the tank wall. It can be seen from Fig. 6 that tensile stress increases over this two month period and reduces over the next ten months due to relaxation, and that the

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permeability of the concrete is one of the most important parameters. Higher stresses are developed in concrete with higher permeability.

Radial tensile stresses in the order of 2 MPa (290 psi) can develop in high pressure standpipes over a five year period when the concrete permeability is  $8 \times 10^{-12}$  m/s ( $26 \times 10^{-12}$  ft/s) and the dilatancy strain is  $1 \times 10^{-4}$ . The presence of reinforcing steel in the wall induces stress concentrations which can magnify the radial tensile stress in the vicinity of the reinforcement by a factor of 2 to 4. Consequently, tensile stresses large enough to split the concrete in the plane of the reinforcement causing delamination in the wall can be induced.

The radial tensile stresses in the wall will be counteracted by the presence of post-tensioning to a certain degree. A post-tensioned tank therefore would require more freeze-thaw cycles to cause delamination. This supports field observations that reinforced concrete standpipes are more susceptible to delamination than are post-tensioned tanks.

Another mechanism which could result in the generation of considerable hydraulic pressure in the wall of a tank has been termed the hydraulic pressure sandwich. Figure 7 shows sketches representing possible thermal conditions which may result in an unfrozen centre zone in the tank wall. Figure 7(a) indicates a condition where the ambient temperature has been below zero for a considerable period resulting in the formation of an interior ice ring in the tank and frozen concrete throughout the thickness of the wall, followed by a warm period which partially thaws the outer part of the wall. As shown in Fig. 7 (b), the cycle is completed by the ambient temperature returning to below freezing resulting in a thawed central region in the wall, an inward moving freezing front, and an impermeable frozen zone in the interior wall section. As described by Adkins (6), this can result in trapping a lens of unfrozen water which expands upon freezing, causing tensile forces in the tank wall. The location of the unfrozen water layer will determine the plane of failure and resulting delamination, which will be manifested as external or internal scaling.

### Effects of Internal Ice Formations

Ice formations consisting of a floating ice cap and/or an ice tube on the walls of water tanks have been observed. Pressure will be exerted on the wall of a tank as a result of rapid warming and related expansion of such ice formations. Hoop and flexural stresses will be developed in the walls as a result, with the ice cap representing a more severe loading case than the ice tube. A method of computing the stresses induced by an ice cap has been developed by Kong (7) and Kong and Campbell (8). Computation of hoop stress is summarised below.

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Ice is a thermoviscoelastic material in that its behavior under stress is a function of time and temperature. Bergdahl (9) has proposed the following four parameter model, to describe the behavior of ice under pressure:

$$\dot{\epsilon} = \dot{\sigma}/E + BD\dot{\sigma}^n \quad [1]$$

where  $\epsilon'$  is the strain rate,  $\dot{\sigma}$  is the stress rate,  $E$  is the modulus of elasticity,  $D$  is the self-diffusion coefficient and  $B$  and  $n$  are coefficients of viscous deformation. The validity of this model has been established by comparing its predictions with test data (7).

Figure 8(a) shows an ice cap contained within a long cylinder. If creep of the ice is neglected (creep effect is introduced later), and if it is assumed that a linear pressure distribution develops when the ice cap is subjected to an increase in temperature, which varies linearly over the thickness (Fig. 8(b)), the problem reduces to that of a long cylinder subjected to a band of linearly varying pressure around its circumference (Fig. 8(c)). This condition may be analyzed using an extension of the approach suggested by Timoshenko and Woinowsky-Krieger (10) for a cylinder subjected to a band of uniform pressure around its circumference. It can be shown (7) that, within the loaded area, at a location defined by the dimensions  $b$  and  $c$  in Fig. 8, the hoop stress,  $\sigma_h$ , in the wall of the cylinder is given by

$$\sigma_h = \frac{pd}{2t} K \quad [2]$$

where  $K$  is given by

$$K = -(c(e^{-\beta c} \cos \beta c + e^{-\beta b} \cos \beta b - 2)/2\ell + [e^{-\beta c} (\sin \beta c - \cos \beta c - 2\beta \cos \beta c) - e^{-\beta b} (\sin \beta b - \cos \beta b - 2\beta b \cos \beta b)]/4\beta\ell) \quad [3]$$

and  $\beta^4 = 12(1 - \nu_T^2)/d^2t^2$ ,  $\ell$  is the thickness of the ice cap,  $d$  is the diameter of the cylinder,  $t$  is the thickness of the cylinder wall and  $\nu_T$  is the Poisson's ratio of the cylinder material. The maximum hoop stress occurs at a location defined by  $c/\ell \approx 0.667 + 0.01\beta\ell$  and the corresponding value of  $K$  from Eqn. [3],  $K_{\max}$ , is a function of  $\beta\ell$  as shown in Fig. 9.

Similar expressions can be developed (7) for hoop stress in the portions of the cylinder wall above and below the ice cap. Flexural stresses in the wall of the cylinder can also be determined in a similar manner (7).

The pressure,  $p$ , at the top of the ice cap, corresponding to a particular temperature gradient through the ice cap, may be found by considering compatibility of deformations at the location of maximum deformation in the tank wall. The pressure,

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for a particular temperature change, is a function of  $\beta l$  and  $\eta$  as shown in Fig. 10. The parameter  $\eta$  is defined as  $E_I d / E_T t$  where  $E_I$  and  $E_T$  are the moduli of elasticity of ice and tank materials, respectively.

A typical variation of hoop stress in the wall of a tank, as predicted by the above theory, is shown in Fig. 11, which also shows for comparison corresponding variations obtained using the computer program ABAQUS (11) to model the ice cap in the tank but neglecting creep in the ice. An idealized variation of the hoop stress proposed for design purposes is also indicated in Fig. 11. The locations of peak hoop stress and zero hoop stress in the wall of a tank are indicated. The zero stress locations are a function of  $\beta$  and  $l$ .

The magnitudes of the peak hoop stress may be determined using Figs. 9 and 10. Figure 10 may be used to determine the pressure in a cylinder of any material containing an ice cap subjected to a known linear temperature gradient throughout its thickness. The related maximum hoop stress can be determined from Eqn. [2] using the relevant value of  $K_{\max}$  obtained from Fig. 9.

Creep of the ice will reduce the pressure exerted by an ice cap subjected to a rise in temperature over a finite time. The maximum static values of pressure and related hoop stress, determined as described previously, can be adjusted for creep using Fig. 12, which shows the peak pressure developed in the ice as a percentage of the corresponding maximum static pressure which would occur if creep were neglected. Creep effects are dependent on initial temperature, heating rate and  $\eta$ .

Kong and Campbell (8) have shown that, in a concrete tank having a diameter of 7140 mm (23.5 ft.) and a wall thickness of 400 mm (16 in.), a hoop stress in the order of 6 MPa (870 psi) is induced by an ice cap which is heated from a initial temperature of  $-20^\circ\text{C}$  at a rate of  $2^\circ\text{C}$  per hour at the top surface. Such a stress will crack the concrete wall.

### DESIGN CRITERIA AND RECOMMENDATIONS

#### General

From the previous discussions it is clear that no cracks through the wall should be permitted to ensure adequate design life for a tank in a freezing environment. It is recommended that tanks be (a) leakproofed by an interior liner, (b) post-tensioned to provide a residual compression of 1.5 MPa (220 psi) in both vertical and circumferential directions after consideration of all environmental loading and (c) protected

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from freezing by insulation or other methods. Recommendations (a) and (c) are particularly important for non-prestressed reinforced concrete tanks which fared worst in Ontario, as indicated by the survey of 53 water tanks (2). Indeed, the use of unprotected reinforced concrete for standpipes should be discouraged and when used the reinforced concrete wall should be designed to a stringent set of design criteria that take into account all possible loads. A brief discussion of possible design criteria and recommendations for reinforced concrete tank walls follows. This discussion is restricted to actions which result in tensile stress in the circumferential direction.

Design criteria are required to guard against collapse and also reduce to a minimum the development of vertical cracking in the wall in a non-freezing environment. Cracking should be eliminated in the wall of a tank located in a freezing environment.

Based on a cracked section, a design criterion to guard against collapse of the structure can be expressed as:

$$\phi_s R_s > \gamma \alpha_F F \quad [4]$$

in which  $R_s$  = the hoop resistance based on the yield strength of the hoop steel,  $\phi_s$  = strength reduction factor for axial tension,  $F$  = lateral pressure of liquid, bulk solid or ice, or related forces,  $\alpha_F$  = associated load factor and  $\gamma$  = importance factor.

In view of the problems experienced in Ontario, it is clear that special design precautions are necessary in environments with severe frost and alternate freezing and thawing temperatures. Under these conditions, a design criterion to reduce to a minimum the development of vertical tensile cracking in the wall of containers where leakage is to be avoided should state that the strength of the uncracked wall must exceed by some margin the effect of all loads that contribute to this cracking (21).

The Portland Cement Association publication for the design of non-prestressed tanks (12) suggests that the wall thickness of reinforced concrete tanks should be determined by limiting the circumferential tensile stress in the concrete. Many design engineers appear to neglect this concept, and the recommendations of ACI Committee 350 (13) do not make direct reference to this procedure. The maximum tensile stress in the concrete due to hoop tension effects, including lateral wall pressure, shrinkage, creep and thermal gradients can be due to a variety of combinations of these loads. For instance, a sample set of criteria are

$$\phi_c R_c > \gamma \alpha_S S \quad [5]$$

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$$\phi_c R_c > \gamma (\alpha_F F + 0.5 \alpha_S S) \quad [6]$$

$$\phi_c R_c > 0.75 \gamma (\alpha_F F + 0.5 \alpha_S S + \alpha_T T) \quad [7]$$

where  $\phi_c$  = strength reduction factor for concrete in tension,  $R_c$  = strength of concrete in tension,  $S$  = stress due to shrinkage and creep,  $T$  = stress due to temperature gradient and  $\alpha$  = the associated load factors. The factor of 0.5 in Eqs. [6] and [7] is included because shrinkage strains will be recovered partially upon wetting of the concrete. The factor of 0.75 in Eq. [7] would not be appropriate if the load  $F$  always acts at its maximum design value as in a liquid storage tank with a stationary liquid level. It is difficult to generalize concerning all possible load combinations. Each designer should be familiar with the way in which the structure is built, tested, commissioned and eventually used.

The reinforcing steel should be distributed in such a way as to limit crack widths. Reasonable limits on crack widths,  $W_{\max}$ , are those suggested by ACI Committee 350 (13), namely  $W_{\max} < 0.25$  mm for non-corrosive liquids and  $W_{\max} < 0.20$  mm for corrosive liquids.

For the determination of crack width due to direct tension, the work by Broms and Lutz (14) is recommended, namely

$$W_{\max} = \epsilon_s c \sqrt{16 + (s/c)^2}$$

where  $c$  = the cover to the centre of the steel in mm,  $\epsilon_s$  = the strain in the reinforcement, assuming a cracked section, and  $s$  = the spacing of the hoop steel in mm.

Finally, the authors recommend that, for tanks in areas with cold climatic conditions, an upper limit of 1% be placed on the amount of circumferential reinforcement. Such a limitation would indirectly guard against high shrinkage stresses and radial tensile stresses if Eqs. [5], [6] and [7] are not properly applied or neglected. Such a limit would lead to a minimum wall thickness of 575 mm (23 in.) in the lower portion of a 10 m (33 ft.) diameter water reservoir with 30 m (98 ft.) water height reinforced with 400 MPa (60 ksi) steel.

#### Load Factors and Strength Reduction Factors

ACI Committee 350 (13) recommendations for sanitary structures introduced strength design as an alternative to allowable stress design. The load factors presented are those used for buildings (15) but increased by "sanitary durability coefficients" (16, 17, 21) in an attempt to create waterproof structures. This factor is 1.65 for reinforcement in direct tension. The authors consider that the use of these large load

factors, which can exceed 2.5 for hydrostatic pressures, may not result in a durable waterproof structure because very high percentages of circumferential steel may result. This procedure in turn may lead to unacceptability high shrinkage stresses if temperature changes are not considered and reduction due to creep in concrete is neglected (18).

For Eqs. [4], [5], [6] and [7] the authors propose, in the case of liquids, a load factor  $\alpha_p$  of 1.4. For saturated solid materials and for ice, the variability is comparable to typical live loads and a factor of 1.7 is more appropriate. The load factor for stresses induced by temperature gradients and by shrinkage or creep of the concrete should be multiplied by the load factor of 1.4 as recommended in ACI 318 (15).

Equations [5], [6] and [7] do not deal with a life threatening failure. It may be appropriate to use an importance factor,  $\gamma$ , of 0.8, similar to that used for low human occupancy buildings and structures in Canadian design code CAN3-A23.3 (19). The importance factor appropriate for Eq. [4] would generally be 1.0. Certain structures such as potable water storage tanks and cylindrical sanitary structures could be classified as particularly sensitive and be designed with an importance factor of 1.3, equal to the "sanitary exposure" factor of ACI Committee 350 (13).

The design criteria in Eqs. [4], [5], [6] and [7] are expressed in terms of factored strengths. Equation [4] applies to a cracked section analysis and hence the yield strength of the circumferential reinforcing steel applies. The strength reduction factor of 0.9 of ACI 318 (15) should be used.

For Eqs. [5], [6] and [7], a concrete tensile strength is required (ACI Committee 224 (20)). The direct tensile strength is appropriate for the lateral wall pressure,  $F$ , and shrinkage and creep,  $S$ . The modulus of rupture is the applicable strength when considering the differential temperature stress. When combining effects from the temperature load with a uniform stress distribution, an interaction equation is recommended.

ACI 318 (15) has no strength reduction factor for concrete in tension. Considering past experience (12) and allowing a probability of approximately 3-5 percent that cracking will occur, the authors find that a strength reduction factor of 0.85 is appropriate for direct tension and flexural tension.

#### CONCLUSIONS

The following conclusions are based on information obtained from surveys of concrete tanks in the Province of Ontario and from research work related to such structures.

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1. Ten of the main defects and the related mechanisms contributing to the high rate of deterioration of concrete water tanks in the Province of Ontario have been discussed. Most of these defects may be attributed to the effects of a cold region environment on the type of construction used.
2. Rate of deterioration of a concrete water tank is related primarily to water pressure, freeze-thaw cycles (amplitude and frequency), concrete permeability and orientation of the wall section to the sun.
3. Hoop stresses large enough to cause cracking may be induced in the wall of a concrete tank by the expansion of an internal ice cap subjected to an increase in temperature. An approach for estimation of such stresses has been proposed.
4. The design recommendations of ACI of Committee 350 are appropriate for standpipes in non-freezing environments and those where minor frosts occur only occasionally. Design criteria aiming to restrict tensile cracking of the concrete have been recommended for climates that include severe freezing and many cycles of freezing and thawing.

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TABLE 1--NUMBER OF TANKS OF EACH TYPE SURVEYED

| 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               |
| 3           | PTU-S                   | Post-tensioned unbonded - standpipe                    | 4               |
| 4           | PTB-S                   | Post-tensioned bonded - standpipe                      | 4               |
| 5           | RC-E                    | Reinforced concrete - elevated                         | 3               |
| 6           | G-E                     | Post-tensioned wire wound gunite protected - elevated  | 2               |
| 7           | PTU-E                   | Post-tensioned unbonded - elevated                     | 4               |
| 8           | PTB-E                   | Post-tensioned bonded - elevated                       | 8               |
| 9           | G-G                     | Post-tensioned wire wound gunite protected - ground    | 9               |
| 10          | RC-G                    | Reinforced concrete - ground                           | 2               |
| TOTAL       |                         |                                                        | 53              |

TABLE 2--TYPES OF DEFECTS

| Type | Description                        |
|------|------------------------------------|
| i    | Wall delamination.                 |
| ii   | Vertical cracks in wall.           |
| iii  | Wall/floor joint deterioration.    |
| iv   | Vertical voids in shotcrete.       |
| v    | Jack-rod spalls.                   |
| vi   | Shotcrete cover coat delamination. |
| vii  | Waterproof coating failure.        |
| viii | Shotcrete cover coat cracking.     |
| ix   | Cold joints and horizontal cracks. |
| x    | Corrosion of prestressing steel    |

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## NOTATION

|            |                                                   |
|------------|---------------------------------------------------|
| B          | coefficient of viscous deformation                |
| c          | cover to reinforcement                            |
| D          | self-diffusion coefficient                        |
| d          | diameter                                          |
| E          | modulus of elasticity                             |
| F          | lateral pressure                                  |
| K          | coefficient                                       |
| l          | thickness of ice cap                              |
| n          | coefficient of viscous deformation                |
| p          | pressure in ice cap                               |
| R          | strength                                          |
| S          | stress due to shrinkage and creep                 |
| s          | spacing of hoop steel                             |
| T          | temperature or stress due to temperature gradient |
| t          | thickness of wall                                 |
| w          | crack width                                       |
| $\alpha$   | load factor                                       |
| $\beta$    | parameter ( $\beta^4 = 12(1 - \nu^2) / d^2 t^2$ ) |
| $\gamma$   | importance factor                                 |
| $\epsilon$ | strain                                            |
| $\eta$     | parameter ( $= E_I d / E_T t$ )                   |
| $\nu$      | Poisson's ratio                                   |
| $\sigma$   | stress                                            |
| $\phi$     | strength reduction factor                         |

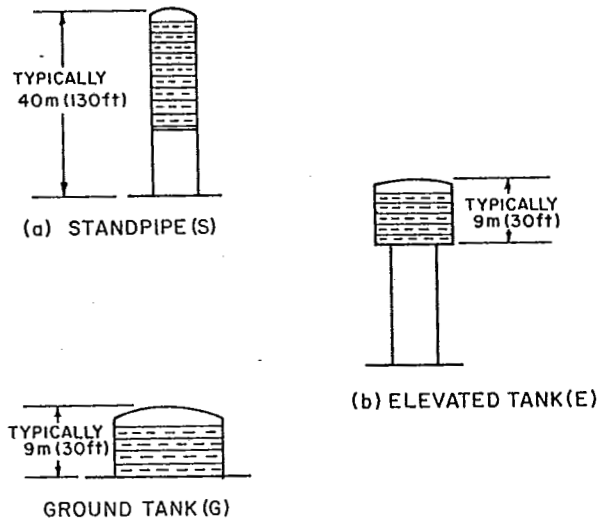


Fig. 1--Types of tanks

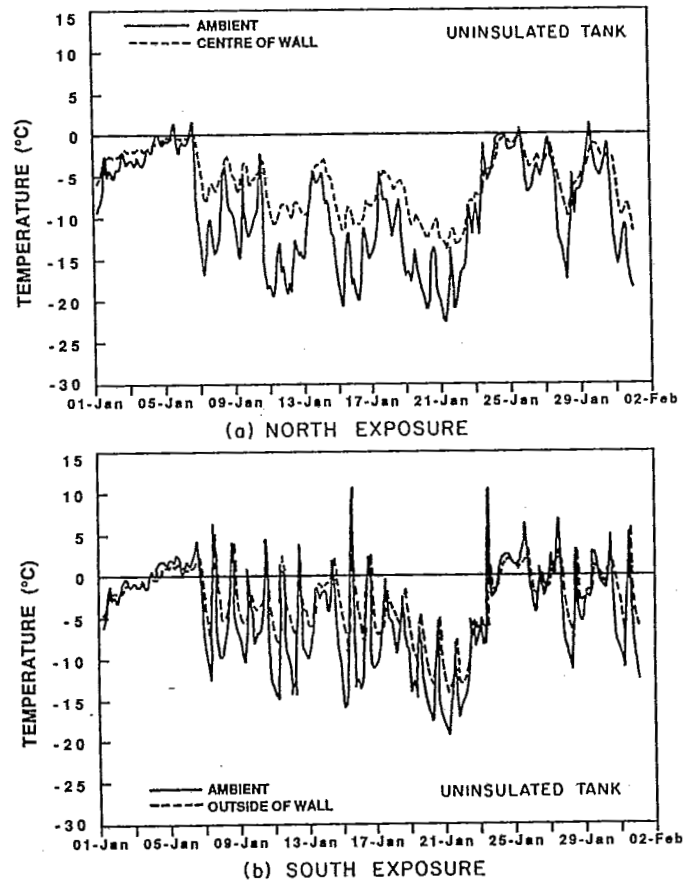


Fig. 2--Temperature variation in uninsulated tank (3)

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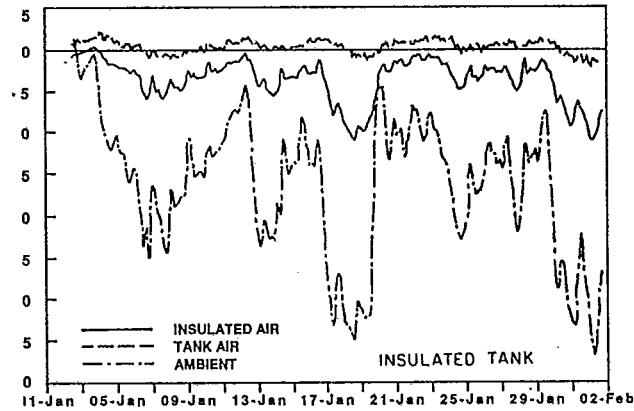


Fig. 3--Temperature variation in insulated tank (3)

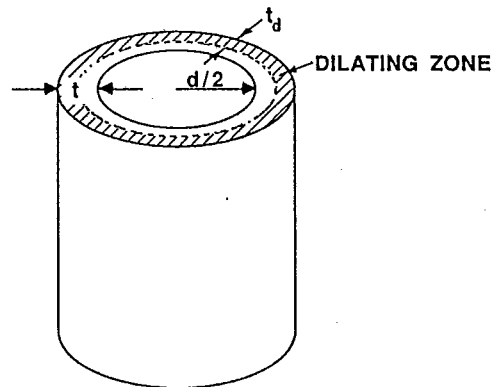


Fig. 4--Location of dilating zone (3)

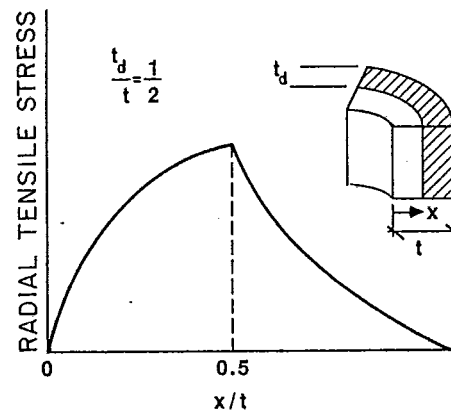


Fig. 5--Distribution of radial tensile stress in wall (3)

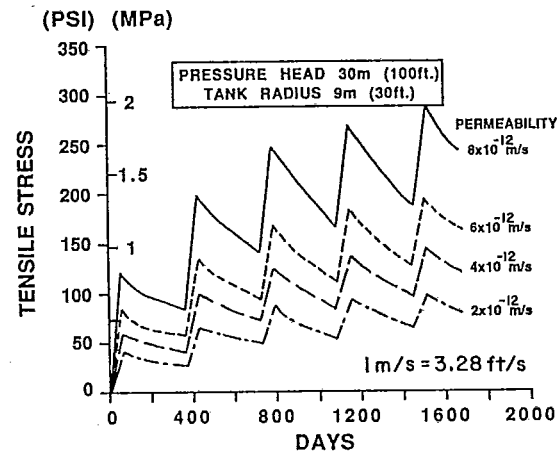


Fig. 6--Variation of tensile stress in wall with time and permeability (3)

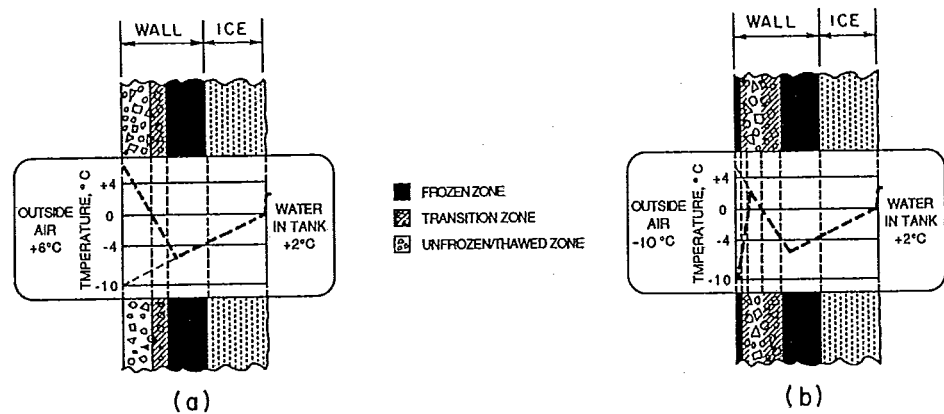


Fig. 7--Thermal profiles in wall of tank (3)

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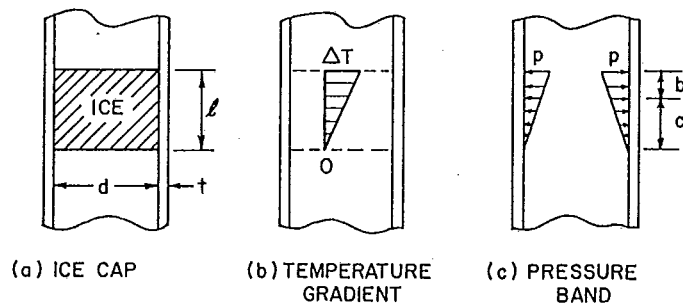
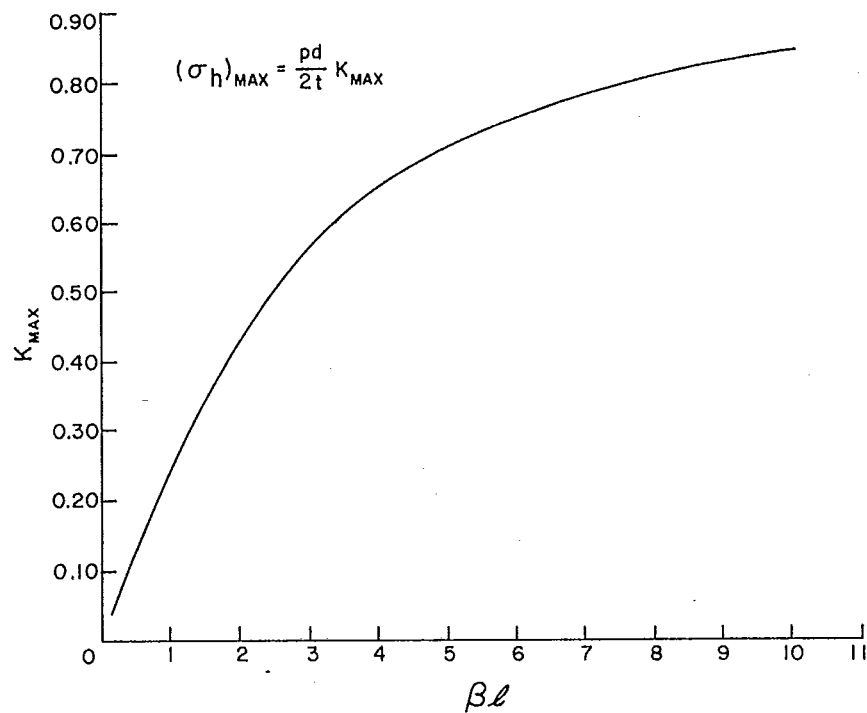
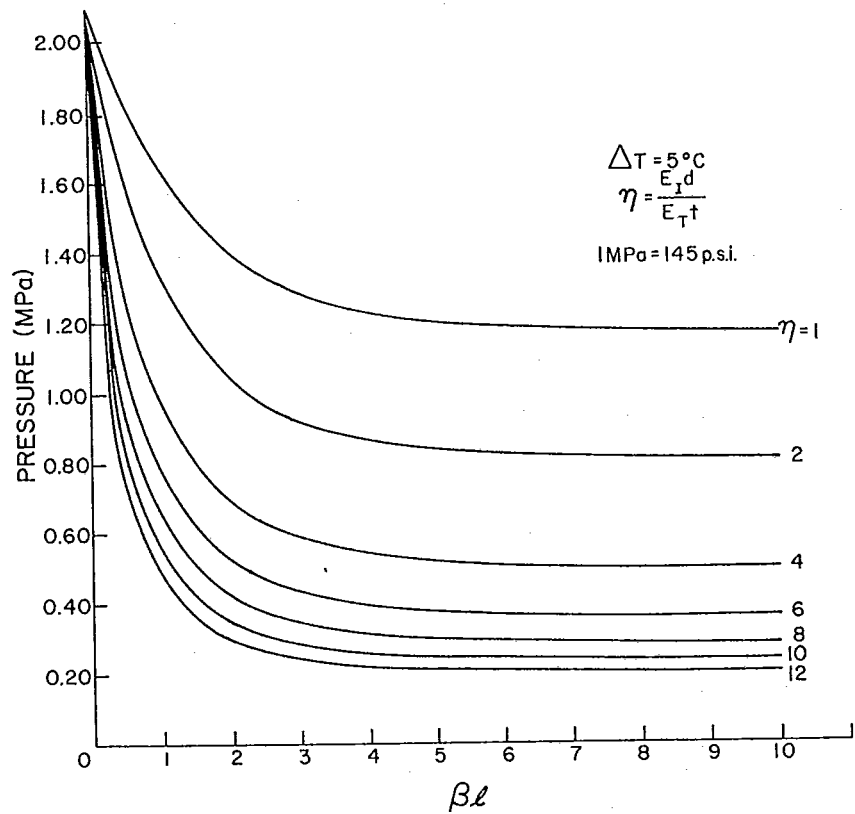


Fig. 8--Temperature gradient and pressure in ice cap

Fig. 9--Relationship between  $K_{\text{max}}$  and  $\beta l$

Fig. 10--Variation of pressure with  $\beta l$  and  $\eta$

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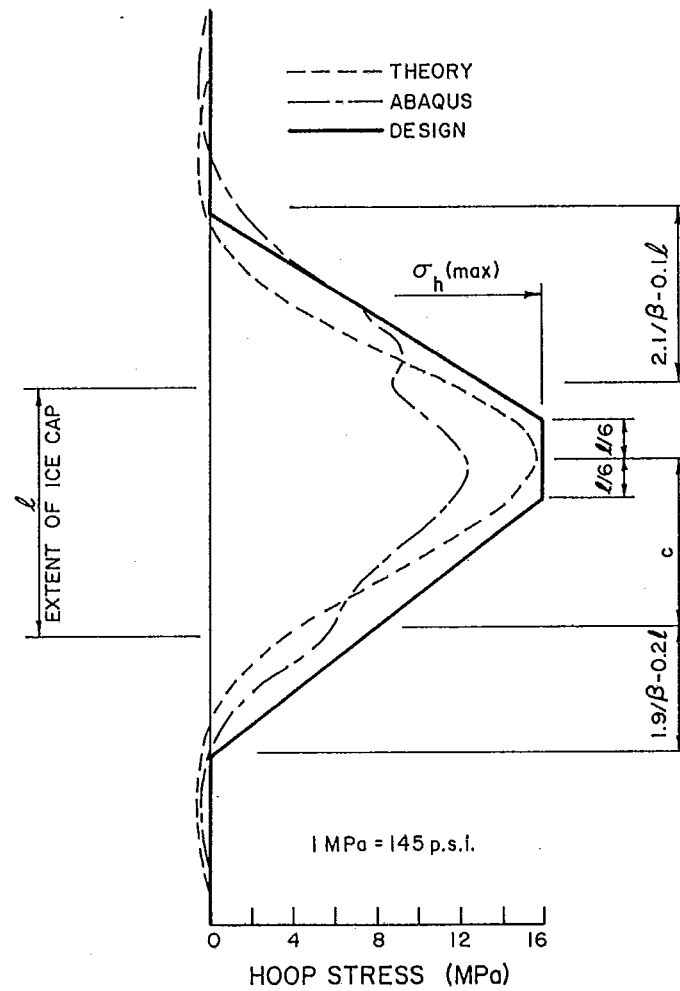


Fig. 11--Variation of hoop stress in wall

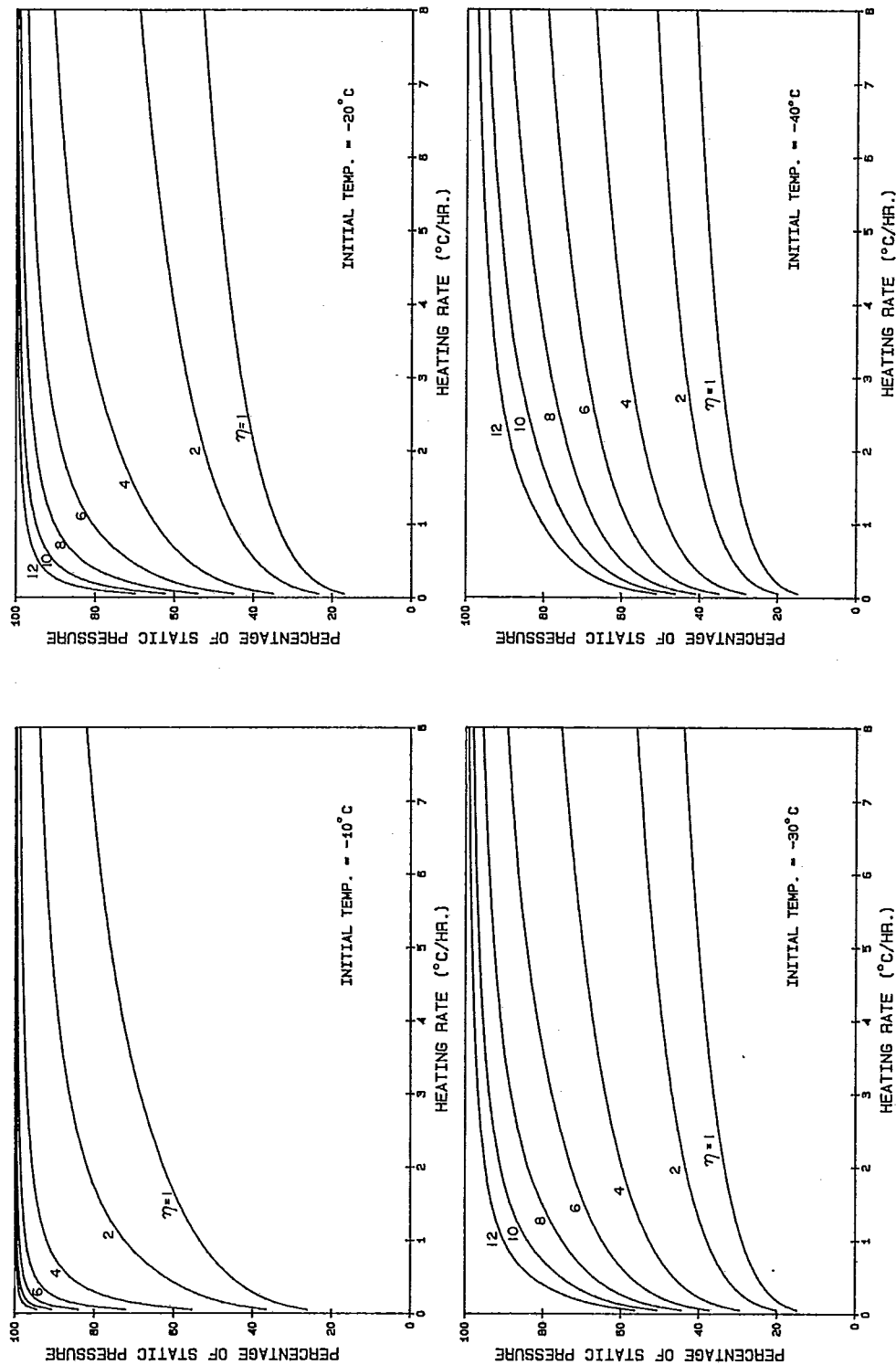


Fig. 12--Influence of creep on static ice pressure

SP 117-2

# Prediction of Temperature Induced Crack Patterns in Cylindrical Concrete Structures

by K. van Breugel and C. van der Veen

**Synopsis:** An analytical procedure is described for predicting the development of vertical cracks in thin-walled and thick-walled cylindrical structures subjected to membrane forces and thermal loads. Sustained, axisymmetrical (thermal) loads and thermal cyclic loading may jeopardize, due to cracking, the serviceability, in this case the tightness, of thin-walled reinforced reservoirs for storage of warm water (25° to 30°C). Non-axisymmetrical thermal loads induced by solar radiation and thermal eigenstresses significantly affect the crack pattern in thick-walled cylinders. Mathematically obtained crack patterns have been compared with field observations: a good agreement between theory and practice could be established from this comparison. On the basis of a reliable prediction of crack patterns, cost-benefit analyses are feasible to weigh crack control measures against possible repair costs in case these measures were neglected. An example of such an analysis shows an initial increase of reinforcement in view of crack control to be preferable to repair (grouting) of cracks.

**Keywords:** chimneys; cracking (fracturing); crack width and spacing; partial prestressing; prestressed concrete; reinforced concrete; reservoirs; serviceability; silos; temperature; thermal gradient; thermal stresses; towers; walls

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## INTRODUCTION

Cylindrical-shaped structures occupy a special place in engineering practice. Silos, reservoirs, towers, chimneys, and protective walls come under this category. These structures have in common that, if above ground, large wall surfaces are exposed to climatic conditions. Climate induced cyclic thermal loads can be quite substantial and can have a great impact on the serviceability of these structures. With respect to serviceability it has been stated, and it will be endorsed here, that not merely cracking, deflections and tightness as such should be considered (1). The associated aesthetic aspects also make out an integral part of the quality of a structure and hence of our increasingly densely-built surroundings.

As far as long-term serviceability is concerned, reliable information on crack development in the design stage of a project could be of great help to designers and owners when initial investments for crack control measures have to be weighed against foreseeable repair costs. This paper describes a method to predict the development of vertical cracks in cylindrical walls subjected to thermal loading, whether or not in combination with a membrane ring force. A brief literature survey will be followed by a description of the loading and of a method for determining the thermal response. Particular attention will be paid to the often underestimated effect of thermal self equilibrating, so called "eigenstresses", on the crack pattern in thick-walled cylinders.

## CRACK PATTERN PREDICTION

For the prediction of crack patterns in hollow cylinders and cylindrical containment structures, very valuable work has been done by Noakowski (2, 3). Tam and Scanton have proposed calculation methods for the prediction of crack patterns in reinforced concrete members subjected to restrained shrinkage deformations (4), whereas Elbadry et al. (5) recently reported on crack pattern prediction in thermally loaded concrete bridges. The response of thick walled cylinders under axisymmetrical thermal loads

has been dealt with by Clarke et al. (6) and England et al. (7). Tanabe et al. (8) pointed to non-axisymmetry of thermal loads induced by solar radiation to which thick-walled shielding structures for nuclear power plants are subjected. He concluded that even 900 mm thick walls might crack due to solar radiation. The effect of thermal eigenstresses on crack pattern development has been discussed several times in connection with box girder bridges (9, 10). The literature on the effect of eigenstresses on crack propagation in thick cylindrical walls, however, is scarce. In this respect, attention is already drawn to the fact that particularly compressive eigenstresses at the surface zones of a wall are quite unpredictable. By suppressing tensile stresses, compressive eigenstresses artificially enhance the magnitude of the moment at which cracking occurs, thus causing higher steel stresses when cracking occurs and consequently wider cracks.

Two cornerstones for a trustworthy crack width prediction are reliable crack width formulae, on the one hand, and a sound description of the structural behaviour in the cracked state on the other. In the analysis to be discussed, a crack width formula based on the bond stress-slip ( $\tau$ - $\delta$ )-relationship between reinforcing steel and concrete has been applied (2). The description of the structural behaviour in the cracked state will have to meet requirements concerning compatibility between deformation due to cracking, on the one hand, and externally imposed deformations, on the other.

#### THERMAL LOADING

Only the temperature distributions in the circumferential and the radial direction will be considered of the basically three-dimensional thermal loading a cylinder might be subjected to.

##### Cross-sectional Temperature Distribution

In the radial direction the temperature profile can be either curved or linear. The thermal loading  $\Delta T(z)$ , measured from a reference temperature  $T_r$ , can be split up into three components, viz. (Fig. 1):

a) Average uniform temperature distribution  $\Delta T_u$  :

$$\Delta T_u = \frac{1}{h} \int_{-h/2}^{h/2} \Delta T(z) dz \quad [1]$$

b) Temperature differential  $\Delta T_b$  :

$$\Delta T_b = \frac{12}{h^2} \int_{-h/2}^{h/2} \Delta T(z) z dz \quad [2]$$

c) "Eigentemperatures"  $\Delta T_e$  :

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$$\Delta T_e(z) = \Delta T(z) - \{\Delta T_u + (\Delta T_b z)/h\} \quad [3]$$

For a linear temperature profile  $\Delta T(z)$ , penetrating to a depth "a" into the wall, the following holds:

$$\Delta T_u = \Delta T a / (2 h) \quad [4]$$

and

$$\Delta T_b = \zeta(a/h) \Delta T \quad [5]$$

where

$$\zeta(a/h) = 3(a/h) - 2(a/h)^2 \quad [6]$$

#### Circumferential Temperature Distribution

Thermal loading in the circumferential direction can be described with the expression (see Fig. 1):

$$\Delta T(\theta) = \Delta T_o + \Delta T^* \cos \theta \quad -\pi/2 < \theta < \pi/2 \quad [7a]$$

$$\Delta T(\theta) = \Delta T_o \quad \pi/2 < \theta < 3\pi/2 \quad [7b]$$

The non-axisymmetrical part of the loading  $\Delta T^* \cos \theta$  is assumed to approximate the effect of solar radiation.

### STRUCTURAL RESPONSE

#### Uncracked State

Axisymmetrical thermal loading--An average uniform temperature loading  $\Delta T_u$  causes radial displacements. If not restrained, as is the case beyond end-effect zones of the cylinder, no hoop stresses will develop.

The temperature differential  $\Delta T_b$  causes bending moments according to (see Fig. 1):

$$M(\Delta T_b) = \frac{\alpha_c \Delta T_b}{h} K (1+\nu) \quad [8]$$

with  $\alpha_c$  = coefficient of thermal expansion and K the flexural rigidity of the wall. The factor  $(1+\nu)$  accounts for the biaxial stress state.

Eigentemperatures  $\Delta T_e(z)$  cause eigenstresses according to:

$$\sigma\{\Delta T_e(z)\} = \alpha_c \Delta T_e(z) E_c (1-\nu)^{-1} \quad [9]$$

Non axisymmetrical thermal loading--Half a cylinder is considered for the analysis of the response to a cosine-shaped thermal loading. This is justified for reasons of line-symmetry. Fig. 2a shows the deformation under a thermal load  $\Delta T^* \cos\theta$  ( $0 < \theta < \frac{\pi}{2}$ ).

For the free deformations at point B ( $\theta = 0$ ) holds:

$$\phi_B(\Delta T^* \cos\theta) = -\int_0^{\pi/2} \frac{\alpha_c \Delta T^* \cos\theta}{h} R d\theta \quad [10]$$

$$\nu_B(\Delta T^* \cos\theta) = \int_0^{\pi/2} \left\{ \frac{\alpha_c \Delta T^* \cos\theta}{h} R (1-\cos\theta) R + \alpha_c \Delta T_u^* R \cos^2\theta \right\} d\theta \quad [11]$$

Deformations caused by cross-sectional forces  $M_B$  and  $N_B$  are determined on the basis of an energy consideration (11) as follows:

$$\phi_B = \frac{R}{K(1+\nu)} \int_0^\pi M_S \frac{\partial M_S}{\partial M_B} d\theta \quad [12]$$

$$\nu_B = \frac{R}{K(1+\nu)} \int_0^\pi M_S \frac{\partial M_S}{\partial N_B} d\theta \quad [13]$$

where  $M_S = M_B - N_B R (1 - \cos\theta)$ .

The distribution of forces in the cylinder wall can be found easily from the equations [10] to [13]. Fig. 2b shows that due to out-of-roundness of the cylinder the maximum moment at point B is about 0.8 times as high as the fully restraint moment would be (0.8 is a rounded-off value; effect of non-axisymmetrical distribution of  $\Delta T_u^*$  is ignored), whereas at point A a moment is yielded with a sign reverse to that at point B. The created membrane forces and associated membrane stresses are generally small in comparison with the bending stresses and will, therefore, be ignored in further analyses.

#### Cracked State

Crack width calculations--The crack width formula used to determine the crack width in the still unstabilized stage of crack formation (Fig. 3) runs as follows:

$$w = 2 \frac{(1+N) d_s \sigma_{s,cr} (\sigma_{s,cr}^{-n} \sigma_{cr})^{\frac{1}{1+N}}}{8 C E_s} \quad [14]$$

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where:

- $w$  =  $2\delta$  = average crack width  
 $d_s$  = nominal bar diameter [mm]  
 $n$  =  $E_s/E_c$  (moduli of elasticity ratio)  
 $\sigma_{s,cr}$  = stress in steel at the crack [MPa]  
 $\sigma_{cr}$  = concrete tensile strength [MPa]  
 $C, N$  = bond characteristics, which define the  $\tau$ - $\delta$ -relationship between reinforcing bars and concrete ( $\tau = C \cdot \delta^N$ ).  
     upper bars :  $C = 0.32 f_{ccm}^{0.28}$ ;  $N = 0.28$   
     general case :  $C = 0.38 f_{ccm}^{0.18}$ ;  $N = 0.18$   
 $f_{ccm}$  = mean cube compressive strength [MPa]

The maximum crack width  $\hat{w}$  is obtained by multiplying the average crack width  $w$  by a factor  $\gamma_s = 1.7$  which accounts for the variation in crack width, and a factor  $\gamma_c$  which accounts for the effects of cyclic loading (if applicable). If more than ten load (moment) alternations have to be expected, the maximum value of  $\gamma_c$  can be determined by (see (12)):

$$\gamma_c = 1 / (1-k) \quad [15]$$

where  $k = 9 \cdot (\sigma_{s,cr}/1000)^3$ . With  $\gamma_c = 1.4$  (corresponding to a steel stress  $\sigma_{s,cr} \approx 320$  MPa) the majority of practical cases are covered.

Crack pattern prediction--How to proceed with the response analysis if cracking occurs will be illustrated by an example of a thin cylindrical wall subjected to an axisymmetric thermal loading. Cracking occurs at a tensile stress  $\sigma_{cr-1}$  when the moment of rupture  $M_{cr-1}$  is reached. Further increase of the thermal loading,  $\Delta\Delta T$ , beyond the temperature at first cracking  $\Delta T_{cr-1}$ , will generate new cracks at an almost constant value of the moment of rupture (Fig. 3). This means that the deformations pertaining to the thermal loading concentrate in the generated cracks. These cracks, of which the widths remain almost constant until a fully developed crack pattern is reached, can be considered as local rotations. For each rotation  $\phi_w$  holds:

$$\phi_w = \frac{w}{d-h_x} \quad [16]$$

in which  $d$  is the effective height of the cross section,  $h_x$  the height of the concrete compressive zone and  $w$  is the crack width. For the number of cracks  $m$  the summarized rotation,  $\sum \phi_w = m \phi_w$ , must correspond with the imposed deformation  $\phi(\Delta\Delta T)_m$  pertaining to the thermal loading  $(\Delta\Delta T)_m$  (see Fig. 3). For this imposed deformation holds:

$$\phi(\Delta\Delta T)_m = \kappa(\Delta\Delta T)_m \cdot 2\pi R \quad [17]$$

$$\text{with imposed curvature } \kappa(\Delta\Delta T)_m = \frac{\alpha_c (\Delta\Delta T)_m}{h}$$

In [17]  $R$  is the radius of the cylinder. For the number of cracks  $m$  at a temperature differential  $\Delta T_m = \Delta T_{cr-1} + (\Delta \Delta T)_m$  follows:

$$m = \frac{\alpha_c (\Delta \Delta T)_m 2\pi R (d-h_x)}{w h} \quad [18]$$

The crack spacing  $\Delta l$  can be calculated as:

$$\Delta l = 2\pi R / m \quad [19]$$

For the crack spacing  $\Delta l_{cr;m,s}$  in a stabilized crack pattern holds (13):

$$\Delta l_{cr;m,s} = 1.5 * w * E_s * (1-N)^{-1} * \sigma_{s,cr}^{-1} \quad [20]$$

The temperature increase  $(\Delta \Delta T)_{m,s}$  at which a stabilized crack pattern is attained, can be obtained from formulae [18] to [20].

Cross-sectional analysis--For the determination of the stresses in the cross section at the stage of unstabilized cracking (Fig. 3), the cross section is assumed to be loaded by the moment of rupture  $M_{cr}$  (with constant magnitude), and a ring force  $N_{00}$  caused by the stored product and/or horizontal prestressing. Bernoulli's hypothesis has been assumed applicable. From the solution of the set of equations resulting from cross-sectional equilibrium requirements the height of the concrete compressive zone  $h_x$ , the steel stress  $\sigma_{s,cr}$ , and the maximum concrete compressive stress at the crack  $\sigma_{c,cr}$  are obtained.

Moment of rupture  $M_{cr}$ --For the cross-sectional analysis the magnitude of the moment of rupture  $M_{cr}$  must be known. It can be obtained from:

$$M_{cr} = W (\sigma_{cr} - \sigma_{ct} - \sigma_e) \quad [21]$$

where  $\sigma_{cr}$  is the concrete stress at cracking,  $\sigma_{ct} = N_{00}/A_c$  the nominal average concrete tensile stress over the concrete cross section  $A_c$  (minus sign in case of compression),  $\sigma_e$  the eigenstress in the ultimate fibre (minus sign in case of compression), and  $W$  the section modulus. The stress  $\sigma_{cr}$  at which the concrete cracks depends on the mode of loading (tension or flexure) and the rate of loading. Values for  $\sigma_{cr}$  can be obtained from Table 1.

In expression [21] the effect of eigenstresses manifests itself at the "resistance side". From this expression it is evident that compressive eigenstresses enhance the magnitude of the moment at which cracking occurs and consequently the steel stresses and the crack widths.

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## THEORY VERSUS PRACTICE

Thin-Walled Reservoir

**Crack pattern**--A reservoir, diameter 18.5 m, height of the wall 2.5 m and wall thickness 0.25 m, contains waste water with a temperature ranging from 25 to 30°C. Given a maximum nominal tensile hoop stress  $\sigma_c = 0.55$  MPa. The actual concrete cube strength is assumed to range from 30 MPa to 40 MPa, values to which concrete grades C22 and C32 would correspond. The wall is symmetrically reinforced with bars  $d_s = 10$  mm,  $\rho_s = 0.157\%$  on each side. Bond characteristics are assumed to be as in the general case (see [14]).

For the C32 alternative Fig. 4 shows, for different rates of loading, the number of vertical cracks in the wall as a function of an axisymmetrical temperature differential  $\Delta T$  over the wall. It is inherent to almost all climate induced thermal loads that tensile stresses are generated at a moderate rate. Cracks should, therefore, be counted according to curve II of Figure 4. After five years in service, on a day when the ambient temperature was about 5°C and the temperature differential  $\Delta T$  over the wall was 20° to 25°C, 34 cracks were counted. In the severe winter of 1987, with ambient temperatures substantially below -20°C during several nights, yielding temperature differentials between 40° to 50°C, almost 80 cracks have been observed. In October 1987, at an ambient temperature of 8°C, 41 more or less open cracks have been counted. In the latter survey the crack width at the outside surface of the wall ranged from almost zero to 0.6 mm. The theoretical average crack width for the moderate loading rate ranged from 0.29 mm to 0.28 mm for grade C22 and C32, respectively (Fig. 5). With a variation factor  $\gamma_s = 1.7$ , a maximum crack width  $\bar{w} \approx 0.5$  mm might have been expected (grade C32). The effect of thermal cyclic loading has been quantified from steel stresses valid for long-term loading cases. For calculated steel stresses of 320 MPa (C22) and 370 MPa (C32), values for  $\gamma_c$  of 1.4 and 1.8 would apply, yielding maximum crack widths from 0.7 mm to 0.9 mm. (Note: the theoretical value  $\gamma_c = 1.8$  is regarded to be on the high side).

**Prevention or repair**--In the period 1985-1987 the largest cracks in the reservoir started to leak. At that time the theoretical height of the concrete compressive zone had decreased from an initial value of 19 mm to 10 mm for the C22 alternative, and from 21 mm initially to 15 mm for grade C32. Assume that within a few years from now 15 cracks will have to be repaired. The total length of these cracks, 2.5 m long each, is 37.5 m. Based on unit prices for grouting of fl. 1.60/m (\$ 0.80/m), the total direct repair costs amount to fl. 6,000.00. Indirect costs, such as process interruption, easily double the repair costs. Possible future repair costs could have been anticipated by increasing the horizontal reinforcement. Fig. 5 shows, for grade C32, the effect on the crack width of increasing the reinforcement ratio two or three times. The mean crack width would decrease to 0.09 mm and 0.05 mm, respectively (moderate rate). Due to lower steel stresses, the effect of cyclic loading on the crack width would be much less:  $\gamma_c = 1.18$  and  $\gamma_c = 1.06$  for  $\rho_s = 0.32\%$  and  $\rho_s = 0.47\%$ , respectively. For the structure in view the extra initial cost for increasing the

horizontal reinforcement two or three times would be, based on steel prices of fl. 2.00/kg (excl. V.A.T.), fl. 1,800.00 and fl 3,600.00, hence significantly less than the repair costs would be.

### Thick-Walled Cylinder

A thick-walled hollow cylinder, diameter 8.5 m and wall thickness 0.650 m, is assumed to be symmetrically reinforced with bars  $d_s = 16$  mm,  $\rho_s = 0.3\%$  and  $0.6\%$ , respectively. The wall is subjected to a thermal loading  $\Delta T$  penetrating the wall to a depth "a", whereas no radially directed loading acts on the wall. Concrete grade C32 has been adopted, together with a cover on the steel of  $c = 30$  mm and bond characteristics as in the general case.

The crack pattern will be investigated for both an axisymmetrical and non-axisymmetrical positive thermal loading, i.e. a temperature rise at the outside surface of the wall relative to the reference temperature  $T_r$  ( $T_r$ : thermally stress-free state).

Axisymmetrical thermal loading--The thermal loading  $\Delta T$  has been split up into the earlier discussed components (Fig. 1). Whereas the cylinder is free to move in radial direction, only the temperature differential  $\Delta T_b$  and the eigentemperatures  $\Delta T(z)$  generate stresses. The compressive eigenstresses at the inner surface of the wall suppress the flexural tensile stresses, thus enhancing the moment of rupture  $M_{cr}$ . In Fig. 6 this moment is given as a function of the depth "a" of the temperature profile. In the bottom part of the figure it has been indicated at which value of  $\Delta T$  the wall is likely to crack (moderate loading rate). Realizing that daily temperature changes of any magnitude penetrate to a depth  $a = 150$  mm to a maximum of 300 mm (14), it will be obvious that cracking of the wall due to axisymmetrical thermal loading is hardly conceivable. For example: On a summer day with a reference temperature  $T_r = 20^\circ\text{C}$ , an ambient temperature,  $T_{am} = T_r + \Delta T_{cr}$ , up to  $35^\circ\text{C}$  would be required for  $a = 300$  mm, and even  $50^\circ\text{C}$  for  $a = 150$  mm. In case cracking would nevertheless occur, the crack widths will be higher the smaller the penetration depth of the temperature profile (Fig. 7). Fig. 8 shows the number of cracks as a function of the thermal loading  $\Delta T$  for several penetration depths "a" of the temperature profile. From the latter two figures it can be seen that ignorance of the eigenstresses, which mathematically means adoption of a penetration depth of the thermal loading equal to the wall thickness (so:  $a = 650$  mm), would incorrectly suggest that a pattern of small cracks might be expected ( $w = 0.04$  mm and  $w = 0.13$  mm for  $\rho_s = 0.6\%$  and  $\rho_s = 0.30\%$ , respectively). In reality, however, and particularly in cases of low reinforcement ratios, the eigenstresses appear to be responsible for the generation of a small number of wide cracks. As will be explained hereafter, non axisymmetrical thermal loads intensify this effect.

Non-axisymmetrical thermal loading--The effect of one-sided solar radiation on the thermal loading and the response of a cylindrical wall can approximately be described by equation (7a, b). The response to such a

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loading will be illustrated by the example for which it holds: reference temperature  $T_r = 20^\circ\text{C}$ , penetration depth of the temperature profile  $a = 200$  mm, and an axisymmetrical part of the thermal loading  $\Delta T_o = 10^\circ\text{C}$ . Moment reduction due to out-of-roundness of the cylinder under the non-axisymmetrical loading is accounted for by a factor 0.8. From the moment distribution under non-axisymmetrical thermal loads, it is obvious that cracking is likely to occur where outside temperatures are highest (Fig. 2b). The temperature rise at which cracking occurs in the non-axisymmetrical case,  $\Delta T_{cr,asym}$ , can be deduced from the cracking temperature in the symmetrical case  $\Delta T_{cr,sym}$ . For  $a = 200$  mm cracking would occur at  $\Delta T_{cr,sym} = 22.6^\circ\text{C}$  (see Fig. 6). It can now be assumed that:

$$22.6 = \Delta T_o + 0.8 \Delta T^*$$

from which follows that  $\Delta T^* = 15.7^\circ\text{C}$ , and  $\Delta T_{cr,asym} = \Delta T_o + \Delta T^* = 25.7^\circ\text{C}$  (Fig. 9). The actual concrete temperature at cracking at the outside surface is then  $45.7^\circ\text{C}$ . Restoring of the original, pre-cracking stress field in the cylinder, but under condition now of the existence of a crack  $w$ , i.e. of a rotation  $\phi_w = w/(d-h_x)$ , requires an increase  $\Delta\Delta T^* \cos\theta$  of the non-symmetrical part of the thermal loading, so that:

$$\phi_w = \phi(\Delta\Delta T^* \cos\theta) = \int_{-\pi/2}^{\pi/2} \frac{\zeta(a/h) 0.8 \Delta\Delta T^* \cos\theta \alpha_c}{h} R d\theta \quad [22]$$

With  $\zeta$  according to [6],  $w = 0.15$  mm (Fig. 7),  $d = 610$  mm and  $h_x = 106$  mm (values from cross-sectional analysis), it follows that  $\Delta\Delta T^* = 3.24^\circ\text{C}$ . With a variation factor  $\gamma_s = 1.7$  and a factor for thermal cyclic loading  $\gamma_c = 1.14$ , (eq. [15];  $\sigma_s = 240$  MPa), a maximum crack width has been calculated of  $\hat{w} = 0.29^s$  mm, whereas for  $\Delta\Delta T^*$  a value of  $6.3^\circ\text{C}$  would then have been found. At temperature rises of  $3.24^\circ$  to  $6.3^\circ\text{C}$  further cracks are likely to be formed.

The foregoing procedure has been carried out for several values of the penetration depth "a" of the temperature profile. The results are diagrammatically shown in Fig. 9. By way of comparison, curve I of Fig. 8 has been inserted, reflecting the crack development in the symmetrical case and without any allowance for eigenstresses. From this figure it is obvious that eigenstresses significantly affect the crack pattern in thick-walled cylinders.

In a reinforced concrete tower, with dimensions similar to the ones discussed above, at first one single dominant crack at the inner surface of the wall has been observed, located at the south side of the structure (structure located at northern hemisphere). In time, two further cracks have been formed.

## DISCUSSION

Analytical procedures for the prediction of temperature induced crack patterns shall meet compatibility requirements concerning stresses and, particularly, deformations. If these requirements are met, reliable

predictions of crack patterns appear feasible, in spite of considerable variation in crack widths observed in practice. A good agreement between theoretical and actual crack patterns has been observed in the case of a thin-walled reservoir due to the fact that many cracks average out the overall crack widths.

Some aspects not dealt with in the foregoing will be briefly considered in the following.

Stress-strain relationship--Linear stress-strain relationships have been used in the analysis. A comparison between the thus obtained results and those obtained on the basis of a non-linear stress-strain relationship of the concrete has revealed that a linear relationship can be assumed in the majority of practical situations, e.g. non-stabilized, crack patterns (18).

Creep effects--Stresses generated by daily temperature changes are hardly affected by creep. Compressive stresses in reservoirs for storage of relatively hot products, however, are quite sensitive to creep. If the stored product is a liquid and in direct contact with the concrete, swelling of the concrete might partly compensate the creep effects. Further research is recommended into the combined effect of creep and swelling on serviceability.

Transitional thermal strain--Transitional thermal strain is defined as a once occurring, temperature- and stress-dependent, irrecoverable strain of the concrete. From relaxation tests carried out by Springerschmid (15), a stress reduction of 30% to 40% relative to linear elastic stresses was observed on heating concrete prisms relaxation from 20°C to 60°C. His findings justify a reduction of theoretical temperature induced stresses in the indicated temperature range by about 20%. This stress reduction can be interpreted as a structure's ability to cope with higher peak temperatures before cracking of the concrete or formation of further cracks occurs. On cooling, however, the concrete is now more likely to crack at the previously heated side. At high peak temperatures even total sign reversal of moments must taken into account (see also (7, 17)).

Tensile eigenstresses--Until cracking, tensile eigenstresses in the inner region of a cylindrical wall are partly suppressed by compressive stresses pertaining to the restrained thermal moment (Fig. 1). If this moment is not fully restrained, e.g. in case of a non-axisymmetrical loading or after cracking, less compression remains to compensate the tensile eigenstresses. This means that further increase of the thermal loading is apt to cause internal cracking of the wall. Crack patterns analysed so far have not revealed internal cracking as indicated here to be of paramount importance. Implicitly the phenomenon considered here has been allowed for by assuming that the moment of rupture can not exceed the magnitude obtained by means of formula [21].

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Maximum concrete compressive stresses--Parameter studies have revealed that low reinforcement ratios are accompanied by high concrete compressive stresses just opposite the crack and make the structure sensitive to creep and to effects of cyclic loading. In this respect and in view of enhanced serviceability, a well-balanced combination of reinforcement and prestressing would be preferable (16).

Wall thickness effect--Procedures for crack width prediction discussed in this paper should not be applied for wall thicknesses exceeding about 800 mm, whereas beyond those thicknesses the Bernoulli hypothesis is no longer valid.

## CONCLUSIONS

- a. Prediction of the development of vertical cracks in cylindrical walls subjected to temperature differentials over the walls appears feasible. The number of cracks increases in time, whereby some cracks tend to grow while others tend to remain small.
- b. A 2.5 m high reservoir, containing hot waste water, initially showed a watertight behaviour at a calculated height of the concrete compressive zone of about 20 mm. When the compressive zone decreased to 10 to 15 mm in time, the cracks started to leak. Information of this kind is most important in view of the development of directives for watertight structures.
- c. From a serviceability point of view as well as from a cost-benefit point of view, initial investments in reinforcement for crack control, possibly in combination with prestressing, are preferable to the repair of cracks.
- d. Partial prestressing is recommended for assuring satisfactory serviceability of thermally loaded cylindrical walls (see also (16)).
- e. In thick-walled cylinders, thermal eigenstresses may be responsible for enhancing the magnitude of the moment of rupture and hence of the steel stresses and the crack widths.

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Table 1 Stress  $\sigma_{cr}$  at which concrete ruptures

| mode of loading                         | loading rate                                                                    | $\sigma_{cr} = \sigma_{cr-1}$ |
|-----------------------------------------|---------------------------------------------------------------------------------|-------------------------------|
| uniaxial tension                        | rapid (minutes)                                                                 | $0.75 f_{ctm,o}$              |
|                                         | moderate (hours)                                                                | $0.62 f_{ctm,o}$              |
|                                         | sustained loading                                                               | $0.50 f_{ctm,o}$              |
| flexure,                                | $\sigma_{cr,fl} = \{0.6 + 0.4 (2p)^{-0.6}\} \sigma_{cr} \quad (< 2\sigma_{cr})$ |                               |
| flexure + normal force                  |                                                                                 |                               |
|                                         | p = height [m] of tensile zone in uncracked state                               |                               |
| $f_{ctm,o}$ = uniaxial tensile strength |                                                                                 |                               |

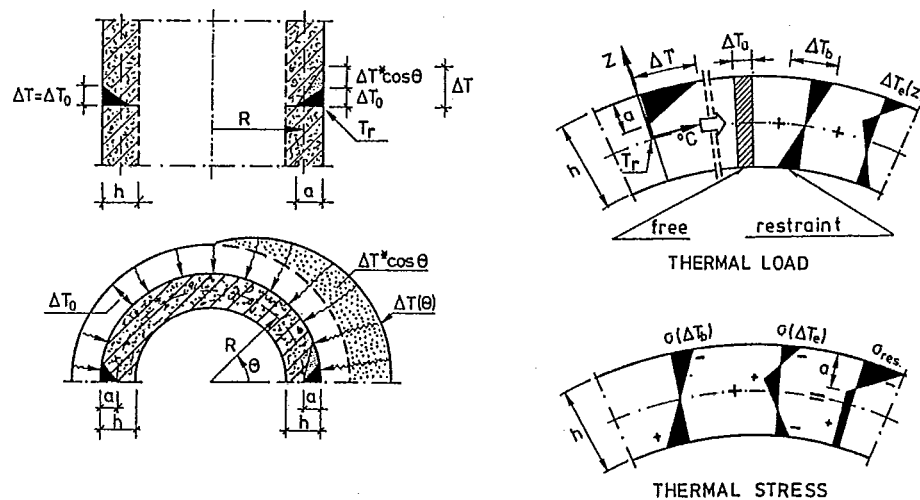
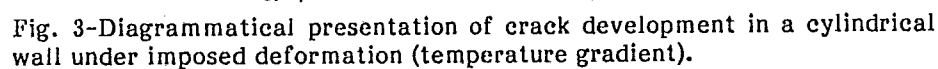
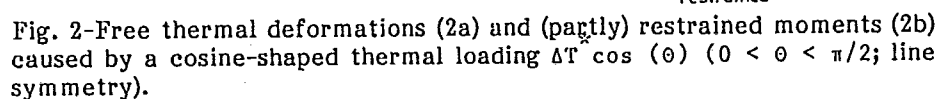


Fig. 1-Circumferential and cross-sectional thermal loading of a cylindrical wall. Bottom/right: restrained thermal stresses.



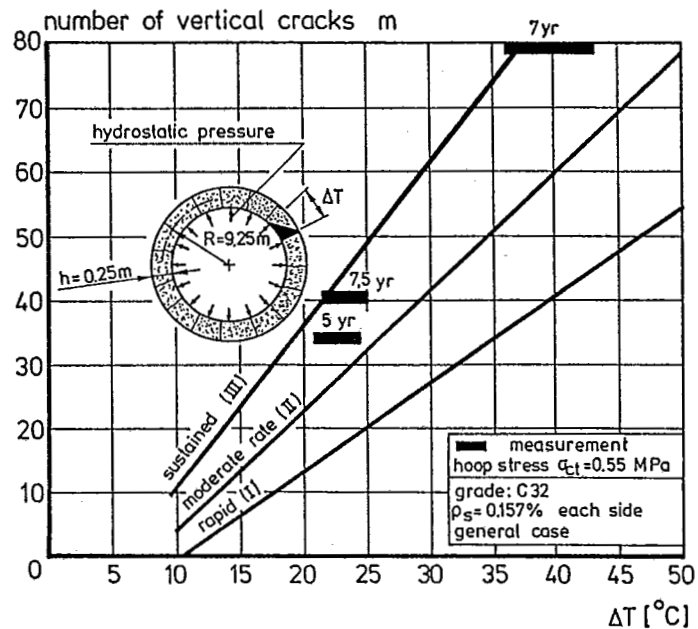


Fig. 4-Number of vertical cracks in a thermally loaded thin-walled reservoir for different loading rates.

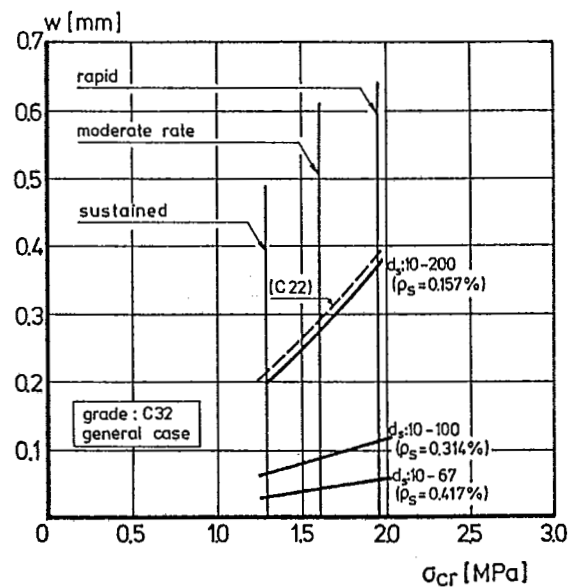


Fig. 5-Average crack width  $w$  in a thermally loaded thin-walled reservoir as a function of the rate of loading for different reinforcement ratios  $\rho_s$ .

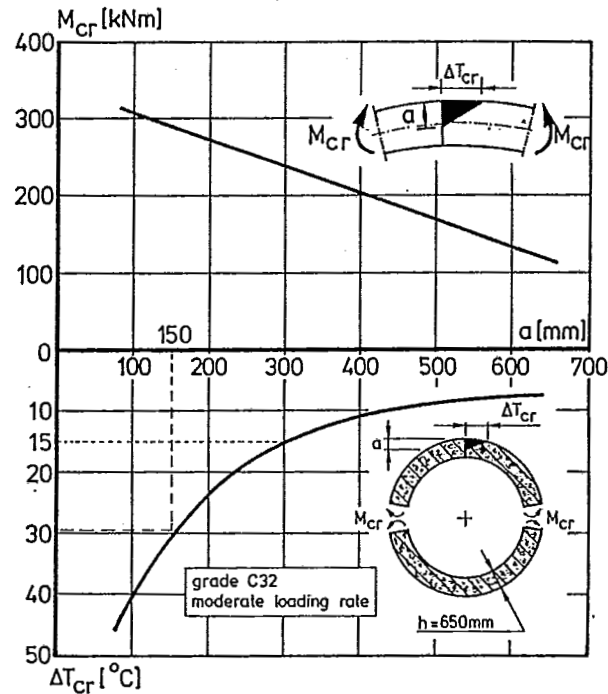


Fig. 6-Modulus of rupture  $M_{cr}$  of a thick-walled cylinder (upper part) and temperature at cracking (bottom part), as a function of the penetration depth "a" of an axisymmetrical thermal load.

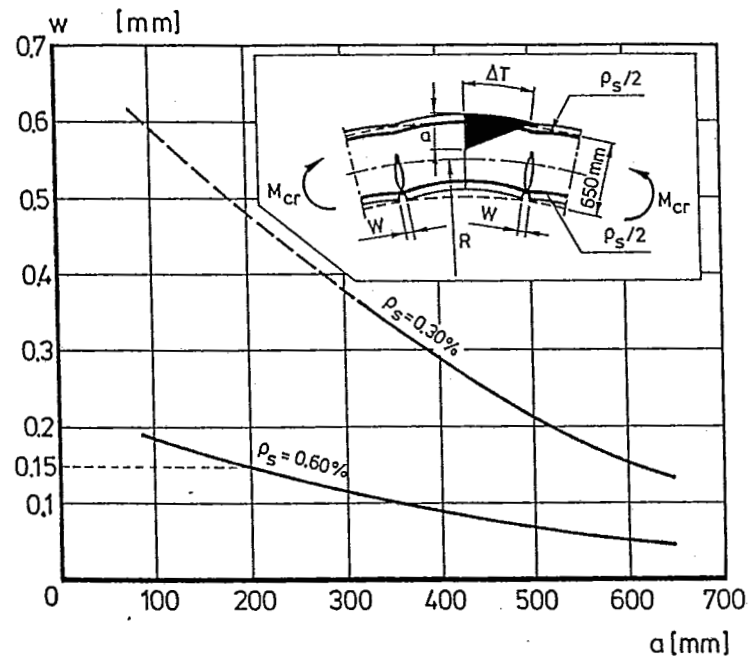


Fig. 7-Average crack width  $w$  in a thick-walled cylinder as a function of the penetration depth "a" for different reinforcement ratios  $\rho_s$ .

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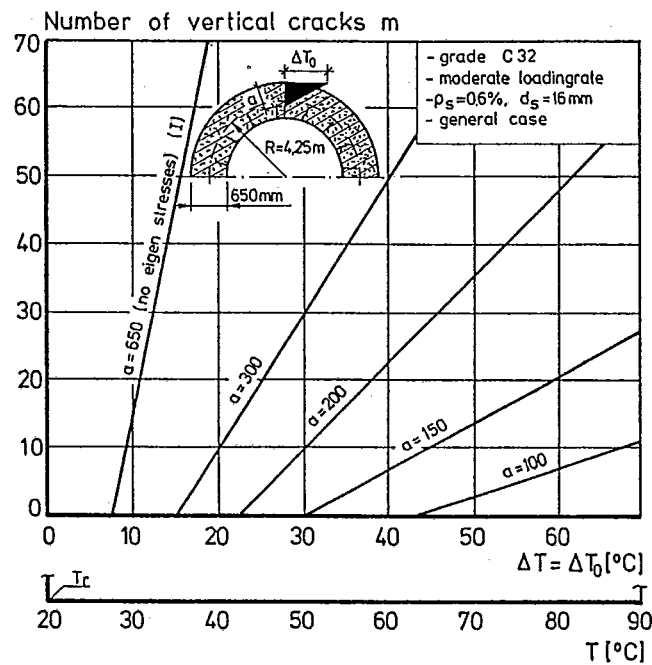


Fig. 8-Crack development in a thick cylindrical wall as a function of a symmetrical thermal loading  $\Delta T_0$  for different penetration depths "a" of the temperature profile.

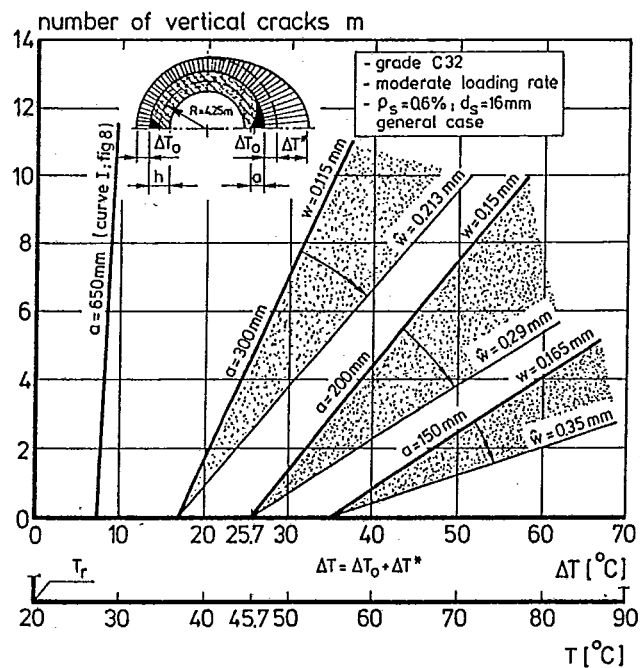


Fig. 9-Crack development in a cylindrical wall as a function of a non-axisymmetrical thermal loading for different penetration depths "a" of the loading.

SP 117-3

# Seismic Design of Concrete Structures based on Serviceability after Earthquakes

by S. Ikeda

**Synopsis:** In seismic zones, severe earthquakes occur definitely within a certain period. However, the important functions of a concrete structure must be maintained after the earthquake. Hence, structures must be designed to fulfill the required safety during the earthquake and serviceability after the earthquake.

The acceptable level of damage can be varied in accordance with the type and importance of the structure. When a reinforced concrete structure suffers significant plastic deformation, residual deformation and large crack opening are remained in the structure after the earthquake, and eventually the functions of the structure are impaired.

A new and rational seismic design method was proposed for the Specification of Reinforced Concrete Design of Japan Society of Civil Engineers. The peculiarity of the concept of the design method is as follows: Seismic design should be performed to fulfill required serviceability after the design earthquake as well as required safety during the earthquake. High magnification factor due to dynamic response was introduced according to actual observation in the earthquakes. Reduction factor referred to the acceptable level of damages remained in the structure after the earthquake was introduced. The importance of design details was emphasized. Furthermore, the influence of axial compressive force on the ductility was pointed out.

**Keywords:** damage; ductility; earthquakes; limit state design; reinforced concrete; safety; serviceability; structural design; structures; yield strength

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## INTRODUCTION

Collapse or failure of reinforced concrete structures can often be seen in severe earthquakes though many of these structures were designed against earthquakes according to their design codes. Furthermore, consequent disasters such as fires or gas explosions may be anticipated. Nevertheless, it is not rational to design a concrete structure against any severe earthquakes without damage because of economy and of balance of safety in the society.

Construction cost can be reduced significantly if some degree of damage by a severe earthquake is accepted on the basis of avoiding loss of lives. Therefore, the seismic design considering ultimate limit state is supposed to be preferable[1,2]. However, in seismic zones, severe earthquakes occur definitely within a certain period and the important functions of a concrete structure must be maintained during and after the earthquake. Hence, structures must be designed to fulfill the required safety during the earthquake and serviceability and performance after the earthquake. This concept of the seismic design of concrete structures was proposed by the author and is coincident with the concept of the long term serviceability of structures.

The Committee on Concrete in the Japan Society of Civil Engineers (JSCE) stipulated a method of seismic design of concrete structures in the Specification of Reinforced Concrete Design in 1986 in which the above mentioned design concept was adopted[3,4]. Here, the basic concept and background of the new design method relating to the Specification of JSCE are described.

## PRINCIPLE OF SEISMIC DESIGN

Concrete structures should be designed corresponding both with safety during earthquakes and with serviceability and performance of the structures after earthquakes. Acceptable degrees of damage of structures due to earthquakes differ from the type and importance of structures. Although required integrity of the structure during and after the earthquake depends on the type of the structures, most of them are expected to be useful after earthquakes. Therefore, it is of primary importance to determine the required performance of the particular structure after the earthquake in the design. In this context, safe and economical structures can be designed by determining the degree of damage due to the earthquake in the level of particular limit