



Improvements to seismic design of circular prestressed concrete storage tanks

J.H. Wood

John Wood Consulting, Wellington, New Zealand.

M.J.N. Priestley

European School for Advanced Studies in Reduction of Seismic Risk, Pavia, Italy.

Priestley Structural Engineering

ABSTRACT: Several of the main issues of concern in prestressed concrete tank seismic design are addressed. The most important of these is the application of structural analysis methods for predicting the reserve capacity above the elastic response force levels. Another of the important issues investigated was the influence of soil-structure interaction on the response of both the horizontal and vertical modes of vibration. An analysis procedure for estimating the reserve capacity of prestressed concrete tanks is presented and the method illustrated by the analysis of a tank of typical dimensions. Charts for estimating the influence of soil-structure interaction for the practical range of tank sizes and foundation conditions are also presented.

1 INTRODUCTION

The American Water Works Association (AWWA) codes for prestressed concrete tanks and the New Zealand Society for Earthquake Engineering (NZSEE) guidelines for the seismic design of storage tanks are commonly employed for tank design in North America and New Zealand respectively. These codes and guidelines are currently under revision. The objective of the research summarised in this paper was to provide background to enable these documents to include more rational procedures for considering the effects of the post-elastic response, soil-structure interaction and the combination of the modal responses.

Seismic design of concrete storage tanks is based on allowable stress limits in the tank wall, implying elastic response. However, many design standards, particularly those in North America, allow calculated elastic response force levels to be reduced by a force-reduction factor of typically about 3.0, applied to both impulsive and convective components. Most design standards relate the force-reduction (or ductility factors) for tanks to one or more factors including the structural material, structural form and wall base fixity conditions. There does not appear to be a rational basis for the specified factors and the codes do not indicate how the factors are related to the reserve capacity or how the capacity can be assessed. The high factors used in North America imply a significant reserve capacity above the design force level, which in normal seismic design is attributed to ductility capacity. However, prestressed concrete systems have low energy absorbing capacity, and the mechanism of ductility in tanks is not obvious. Consequently, it has been argued that force-reduction factors should not be applied to the force levels corresponding to the elastic response spectrum.

The first stage of the research reported in this paper was to compare and identify the differences between the North American and New Zealand tank design documents, particularly with respect to the treatment of post-elastic response and force-reduction factors. In the second stage, the specific design issues of structural analysis methods for predicting the reserve capacity above the elastic response

force levels, soil-structure interaction and the methods appropriate for combining modal responses were investigated. Finally, numerical analyses were undertaken to illustrate the importance of the issues for a range of typical tanks.

2 COMPARISON OF DESIGN CODES

2.1 Codes Reviewed

The following codes and guidelines for the seismic design of storage tanks were reviewed:

API 650 (Appendix E): Welded Steel Tanks for Oil Storage

AWWA D115-95: Circular Prestressed Concrete Water Tanks with Circumferential Tendons

AWWA D110-95: Wire- and Strand-Wound Circular Prestressed Concrete Water Tanks

NZSEE 1986: Seismic Design of Storage Tanks. (Guideline document)

NZSEE 1997: Seismic Design of Storage Tanks. (Draft Revision of 1986 Guideline, Whittaker and Jury, 2000)

IBC 2000: International Building Code: Structural Design Provisions

The AWWA codes are only concerned with prestressed concrete tanks. NZSEE 1986 and 1997 and IBC 2000 cover tanks of all construction materials. NZSEE 1997 updates the earthquake load provisions given in NZSEE 1986 but does not cover structural analysis methods or design details. API 650 is only intended to be used for steel tanks, but as with the other documents, the earthquake load derivation procedure is common to all types of construction materials and it was therefore informative to compare it with the others.

All the documents reviewed estimate the earthquake-induced loads on the tank shell using an earthquake ground motion response spectrum and a simplified two mass analogue of shell-fluid dynamic interaction. This simplified model has two lateral modes of vibration. The highest frequency mode is the impulsive mode, which simulates the motion of the portion of the fluid mass that responds in unison with the tank shell. The second or convective mode, simulates the motion of the part of the fluid mass which responds in the fundamental sloshing mode. The AWWA and NZSEE documents require that the lowest frequency vertical mode of vibration be considered to act concurrently with the two horizontal modes.

2.2 Response Spectra and Reduction Factors

The elastic response spectra and force-reduction factors used for converting elastic to inelastic spectra were found to be the components that contributed most to the differences in the earthquake loads calculated using the different provisions and methods. As an initial step in this review, the elastic response spectra from the codes and guidelines were compared without scaling them by any force or damping reduction factors. Although there were significant differences between some of the earthquake response spectra, agreement within the impulsive mode period range (0.1 to 0.6 s) was moderately good except for the case of API 650 where unspecified inelastic response reductions have been made to the elastic spectra. In the convective mode period range (3 to 10 s), the spectral accelerations vary by up to a factor of 3.0 with the AWWA codes lower than the NZSEE guidelines.

The AWWA codes, IBC 2000 and NZSEE 1997 use reduction factors that scale down the elastic response spectra to allow for energy dissipation and reserve strength from inelastic action. NZSEE 1997 also specifies reduction factors that scale down the 5% damped elastic spectra when soil-structure interaction is significant. NZSEE 1986 uses elastic spectra without ductility or force reduction factors. This approach was based on the belief that many tanks did not have good post-elastic behaviour with elastic buckling often limiting the performance of steel tanks and excessive cracking and leakage limiting the useable inelastic behaviour of concrete tanks. The force-reduction factors specified in the AWWA codes are summarised in Table 1.

Table 1. AWWA Force-reduction factors.

Wall Type Description	Force-reduction Factor	
	D115-95	D110-95
Nonsliding base. Fixed joint or a nonsliding hinge.	3.0	2.75
Anchored flexible base. (Elastomeric pad, anchorage with strand cables.)	2.5	4.5
Unanchored and uncontained flexible joint at base.	1.0	2.0
Unanchored and contained flexible base. (Joint contained by concrete curb.)	3.0	Not given

The AWWA D115-95 reduction factors are applied to both the convective and impulsive mode spectral accelerations. However, in AWWA D110-95 the reduction is only made to the impulsive mode spectral accelerations. The general form of the expression for the impulsive mode base shear in both these codes is:

$$V_i = Z I C_i M_i / R_i \quad (1)$$

Where, Z is a seismic zone factor, I is an importance factor, C_i is the elastic spectral acceleration for the impulsive mode, M_i is the impulsive mode effective mass, and R_i is the force-reduction factor.

IBC 2000 specifies force reduction factors of 2 and 3 for reinforced and prestressed concrete tanks with nonsliding and anchored flexible bases respectively. For ground supported tanks, NZSEE 1997 specifies ductility factors related to the tank type and material of construction. Equivalent force-reduction factors for prestressed concrete and reinforced concrete tanks are 1.0 and 1.4 respectively. (These are applied to reduce the 5% damped elastic spectrum in a similar manner to that shown by Equation 1 above for the AWWA codes). Horizontal impulsive mode damping reduction factors for concrete tanks, with D/H ratios of 4 or greater, range from 1.5 for a foundation soil shear wave velocity of 200 m/s to 1.0 for a shear wave velocity of 1000 m/s. No damping reduction is applied to the convective mode.

2.3 Stress Limits

All the reviewed documents specify earthquake loads for the ultimate limit state. In the AWWA codes, the required ultimate limit performance of prestressed concrete tanks is achieved by specifying allowable stress levels based on elastic analysis. These stress levels are generally about 25% greater than permitted for normal service loads. NZSEE 1986 permits both ultimate strength and elastic analysis for prestressed and reinforced concrete tanks. However, the elastic analysis alternative is normally used and for this case the ultimate loads are reduced by a factor of 0.8.

2.4 Base Shears and Overturning Moments

Comparisons of the design actions given by the different response spectra and analysis methods in the codes and guidelines were made by computing the normalised shear and overturning moment (OTM) at the base of the walls of three large prestressed concrete tanks located on either rock or medium to soft stiffness soil profiles, with diameters of 20, 30, and 40m, and diameter over height ratios, D/H, of 3, 4 and 6 respectively. The base shear was normalised by dividing the computed value by the product of the zero period spectral acceleration, A₀, for the rock site category and the total weight of the tank and contents. That is, the normalising factor was the base shear from a perfectly rigid structure located on rock and responding without hydrodynamic action. Similarly, the OTM was normalised by dividing by the product: A₀, x the total weight of the tank and contents x height of the centre of gravity of the contents.

The weight of the tank walls and roof were included in the analyses assuming concrete thicknesses of 200 and 100 mm for the wall and roof respectively. Impulsive mode periods of 0.2 and 0.4s were assumed for the three tanks on rock and the three tanks on the medium to soft soil profile respectively.

The normalised base shears are compared in Figure 1. Similar variations were found for the OTM's. Low values of normalised shear and OTM computed using the AWWA provisions are mainly the consequence of the large structure reduction factors assumed to be 2.75 for D110-95 and 3.0 for D115-95. In D110-95 this factor reduces only the impulsive force component.

The NZSEE 1986 normalised base shear is sensitive to the value of the impulsive period. This is because the unmodified response spectrum used (based on correlations from recorded motions) rises steeply in the 0.0 to 0.4s period range. The high values of normalised base shear result mainly from the use of an unreduced elastic spectrum. NZSEE 1997 uses a modified spectrum with a plateau in the short period range and this eliminates the sensitivity to the impulsive period.

The low values of normalised base shear given by API 650 are the result of the low impulsive mode force coefficient derived from a reduced elastic (or inelastic) response spectrum.

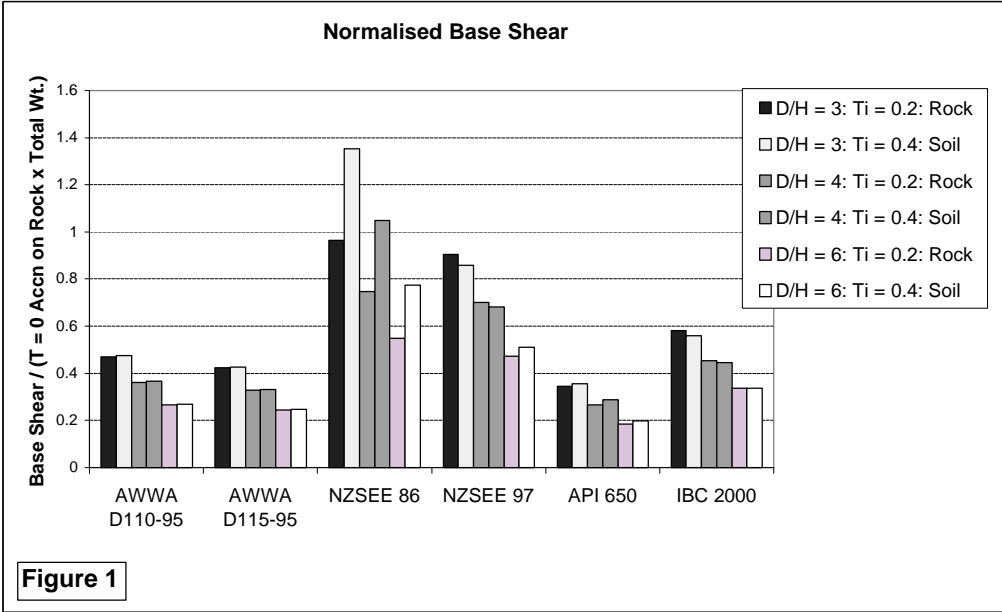


Figure 1

2.5 Hoop Forces

The AWWA codes and the NZSEE guidelines require that the forces from the vertical response of the tank to vertical accelerations be combined with the response actions from the horizontal modes. API 650 and IBC 2000 do not specifically require vertical mode response to be considered. The most significant action of the vertical mode is to increase the hoop forces in the walls. The total earthquake induced hoop force is calculated by taking the SSRS of the impulsive, convective and vertical components. Both the vertical mode and static liquid pressure hoop forces in the walls are axially symmetric and have maximum values around the circumference near the base of the walls. In contrast, the horizontal mode components vary around the wall as a function of cosine of the horizontal angle of position, and the convective mode produces a maximum hoop force near the top of the wall.

Normalised hoop forces in the walls of the prestressed concrete tanks described above were computed using the AWWA codes and NZSEE guidelines. The total earthquake hoop forces were normalised by dividing the computed values by the product of the zero period vertical spectral acceleration for the rock site category and the total weight of the contents. That is, the normalising factor was the total hoop force in the walls generated by vertical mode response of a perfectly rigid structure located on rock without hydrodynamic action. A comparison of the normalised hoop forces is shown Figure 2. The AWWA hoop forces are lower than the NZSEE values, mainly because of the large reduction factors specified in both AWWA codes for the impulsive horizontal mode and the vertical mode.

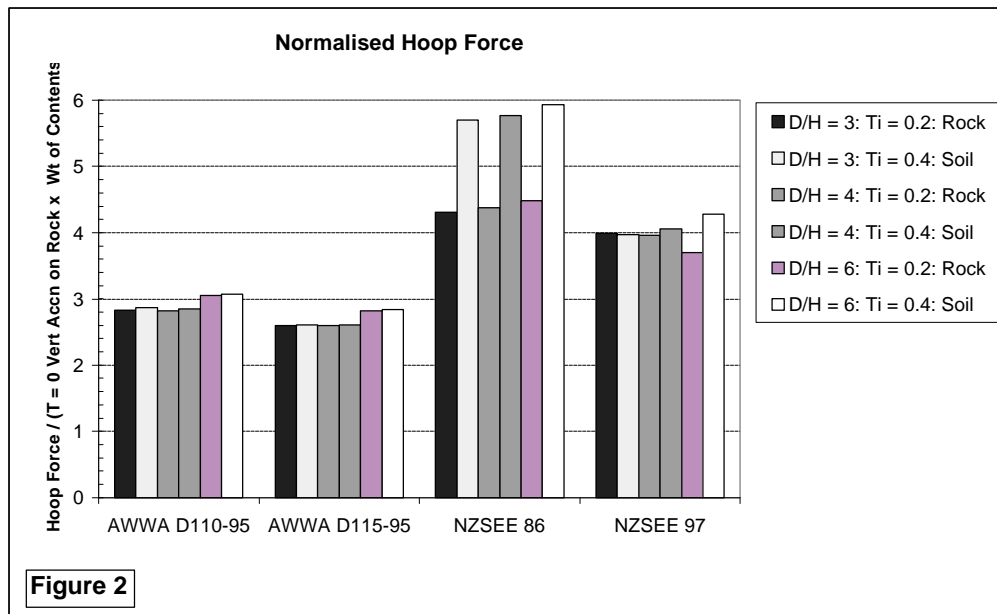


Figure 2

3 COMBINATION OF IMPULSIVE AND CONVECTIVE MODES

The convective base shear component calculated using the AWWA, API 650 and NZSEE 1986 documents was generally less than 30% of the impulsive component. If the impulsive component is added to a convective component of this magnitude by the square root of the sum of the squares (SRSS) method (procedure specified in AWWA and NZSEE documents) then the convective component contributes only about 5% to the total base shear. API 650 uses an absolute summation of the two components, which significantly increases the contribution of the convective component to the total base shear.

Time history analyses were carried out using several input motions with significant long period components to evaluate the various methods of combining the modal responses. The conclusion from this work was that the SSRS method gave the best overall approximation to the total peak combined response as indicated by the time history analyses.

4 FORCE-REDUCTION FACTORS

In order to investigate the question of appropriate force reduction factors, a prestressed concrete tank 40m in diameter and 7.8m high, with a wall thickness of 180mm was designed to the AWWA D115-95 requirements for a site with a peak ground acceleration of 0.4g. A force-reduction factor of 3.0 was applied to the elastic seismic response level. Seismic load combinations governed the hoop prestress design, which was based on zero tension under seismic + water load + prestress. Vertical prestress design was governed by the “tank-empty” case, which induced critical vertical bending moments as a result of unbalanced hoop force. Fifty percent of the hoop prestress was applied with the tank base free to slide, with the remainder applied with the tank base pinned.

It was clear that the design criterion of “zero tension” under design seismic load was conservative. Higher levels of seismic intensity could be supported if cracking was allowed. A reasonable performance limit would be to ensure that circumferential and vertical tendons remained at stress levels lower than the limit of proportionality. This would imply significant cracking under the design level earthquake, but no loss of prestress during the earthquake. After the earthquake, cracks would close up and residual compression would be available under waterload + prestress.

Inelastic analyses of the designed tank were carried out to determine what level of seismic intensity could be carried without violating the above performance criterion. Tank wall behaviour was modeled by inelastic vertical members representing the vertical bending action, and inelastic radial springs

representing the hoop action. Earlier work (Priestley 1978) had shown this simple representation to produce results of similar accuracy to full 3-D finite element modeling. Moment-curvature relationships were developed for the vertical bending response, and inelastic radial force-deflection relationships were developed for hoop action. A modified form of inelastic push-over analysis was carried out using “Ruaumoko” (Carr 1998), where the vector of radial forces corresponding to the seismic pressure were gradually increased.

Results of the analysis are summarized in Figure 3. Typical bi-linear force-displacement response for some of the radial springs are shown in Figure 3(a). Initially, the springs all had the same effective stiffness, but had different strengths, depending on the local level of residual compression stress in the tank wall. Post-cracking stiffness depended on the spacing of the circumferential tendons, and reduced with height up the wall. Moment-curvature response for vertical bending (not shown) was also represented by bi-linear curves.

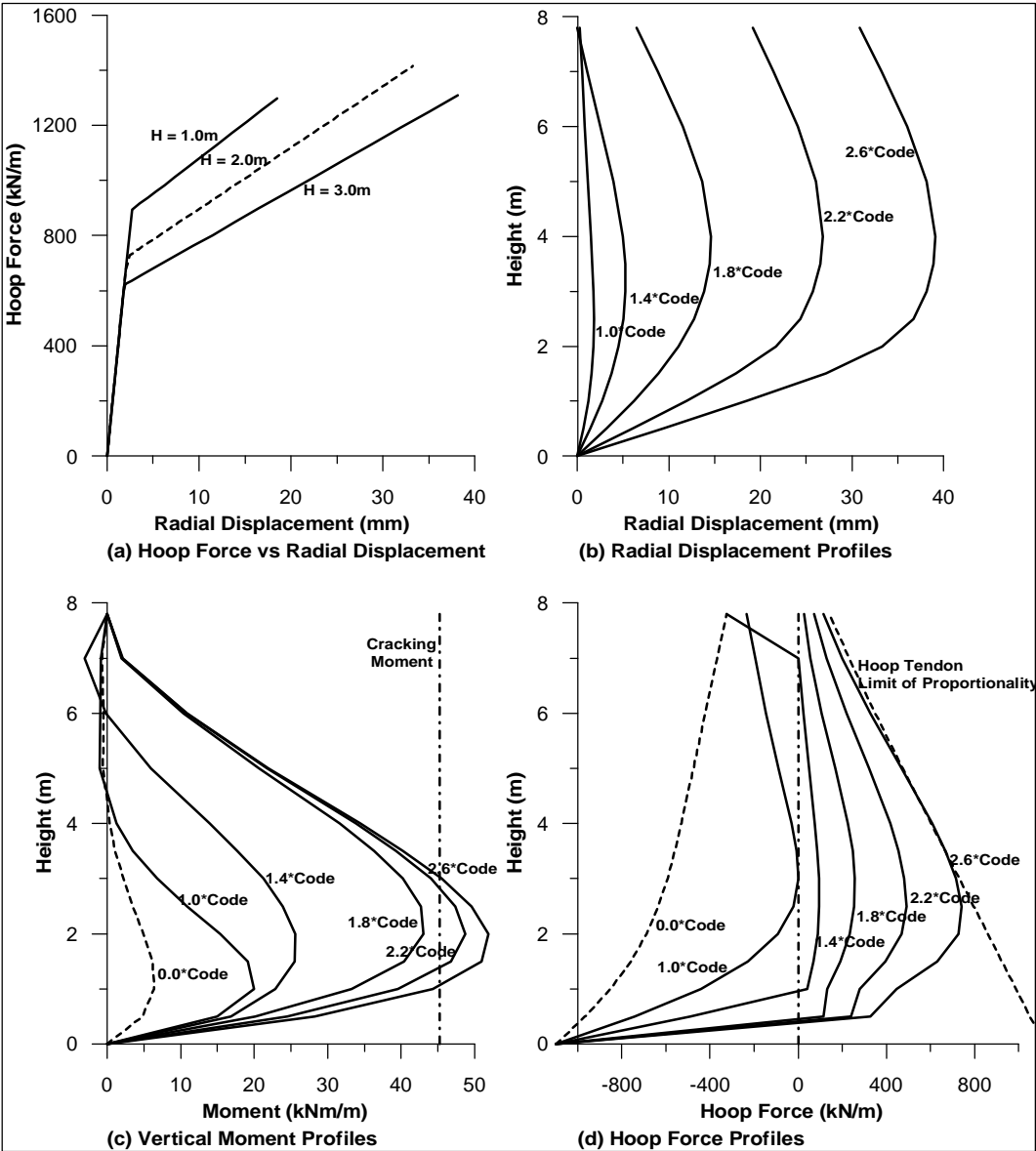


Figure 3 Strength analysis of 40m diameter 7.8 m high tank

Non-linearity of response is apparent in the radial displacement profiles for different levels of seismic intensity, shown in Figure 3(b). It will be observed that not only do the displacements increase greatly after vertical cracking develops at the code seismic force intensity, but the shape of the profile also changes markedly, with much higher relative increases near the top of the wall. This indicates some redistribution of hoop tension up the wall, and more efficient resistance at high pressure levels.

Vertical bending moment profiles are plotted in Figure 3(c) for different seismic intensities. Cracking does not initiate until about twice the code seismic loading, and the extent of the wall subjected to significant bending moment increases markedly as the intensity increases. Again this represents redistribution and an increase in efficiency as cracking is allowed to develop.

Finally, the resultant hoop force profiles are compared in Figure 3(d) with the profile corresponding to the limit of proportionality. The profile 0.0*Code corresponds to the initial "at-rest" condition of water + prestress after losses. It is seen from this figure that the limit of proportionality of tendon force is reached, between 3 and 5m above the base, at 2.6 times the AWWA code force levels.

The results indicated that a considerable reserve of strength existed for this tank above the stress conditions permitted for seismic loads by AWWA D115-95. This increase in resistance was due to redistribution of design actions both between hoop action and vertical bending, and also due to mobilization of larger sections of the wall to high stress levels. It was also clear that considerable ductility existed (see Figures 3(a) and 3(b)). However, this would occur without any significant increase in energy absorption, since the non-linear response was essentially bi-linear elastic. Also, there would be little (if any) reduction in force levels due to period elongation, since the initial period would be short (typically about 0.1 sec), and response would be on the plateau of the response spectrum. It would appear, however, that for a pinned base prestressed tank, the use of a force-reduction factor applied to elastic seismic response levels can be justified. It would also seem that the current value of 3.0 adopted by AWWA for this category of tank is reasonable.

5 SOIL-STRUCTURE INTERACTION

The influence of soil-structure interaction on the earthquake response of circular concrete storage tanks was investigated using a procedure developed by Veletsos and Tang (1990) for steel storage tanks. In their simplified method an equivalent (fixed base) single degree of freedom (SDOF) oscillator was used to represent the impulsive mode of a flexible tank structure interacting with a flexible soil foundation. (Soil-structure interaction has negligible effect on the convective mode response.) The period of vibration and damping of the equivalent SDOF system were derived by firstly computing the response of the tank and flexible foundation system to harmonic forcing. The SDOF period and damping were then obtained by finding the best fit to the transfer function of the flexible base system. The harmonic forcing transfer function for the soil-structure system was evaluated using the solutions given by Jennings and Bielak (1973) for a single storey structure.

5.1 Analysis Parameters

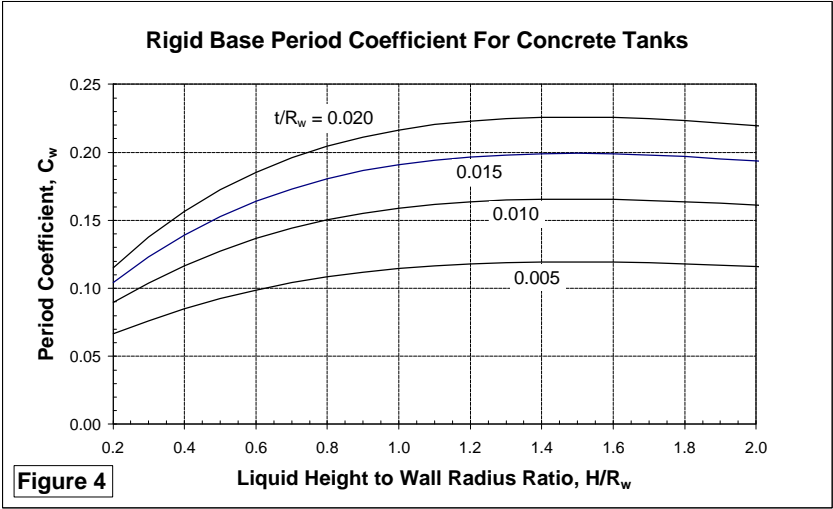
The structural foundation of the tank was assumed to be a massless rigid disk resting on a homogenous visco-elastic half-space. Impedance functions were calculated using the approximate closed-form solutions of Veletsos and Verbic (1973). The hydrodynamic interaction of the fluid on the tank base was incorporated in the analysis using the equivalent rotary inertia coefficients given by Veletsos and Tang (1990). The elastic foundation soil was assumed to have a hysteretic damping coefficient of 0.05, Poisson's ratio of 0.33, a density, ρ_s of 2.0 t/m³ and to have a shear modulus, G_s , uniform with soil depth.

The tank wall was assumed to be elastic with a viscous damping coefficient of 0.02, Poisson's ratio of 0.2 and a density, ρ_w , of 2.45 t/m³. Wall structural stiffness parameters used in the analysis were Young's modulus, E_w , and the wall thickness to radius ratio, t/R_w .

The impulsive mode period for a rigid base was calculated using the following approximate equation from Veletsos (1984):

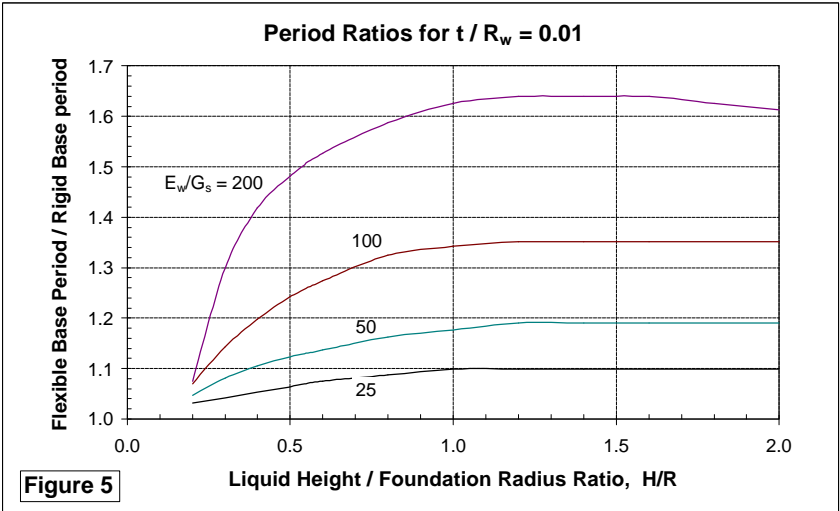
$$T_r = \frac{2pH}{C_w} \sqrt{\frac{r_w}{E_w}} \tag{2}$$

Where H = height of liquid, and C_w is the concrete tank period coefficient which is a function of the H/R_w and t/R_w ratios as shown in Figure 4. Concrete storage tanks will generally have fixed base impulsive mode periods in the range 0.05 to 0.25 s. For example, a tank with $H/R_w = 0.5$, $t/R_w = 0.01$ and H = 10 m has a period of about 0.14 s.

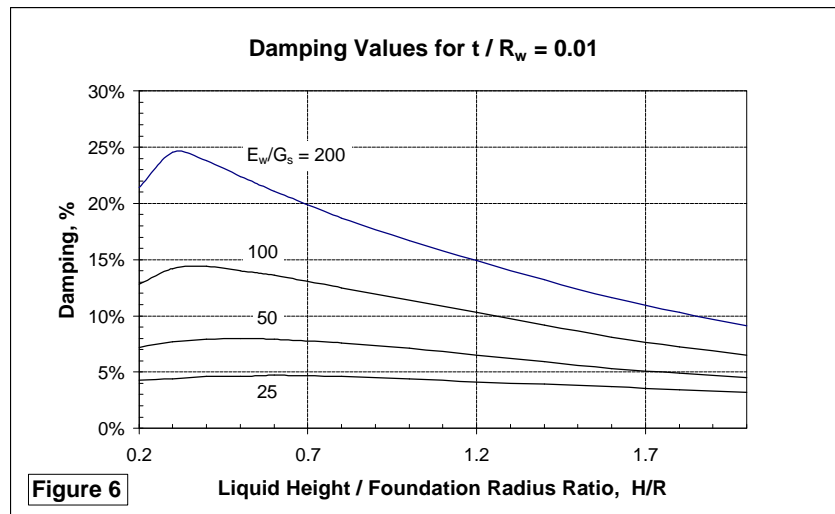


5.2 Analysis Results

The ratio of the impulsive mode equivalent SDOF period for the flexible base system to the rigid base period is shown in Figure 5 for $t/R_w = 0.01$. As indicated in the figure, the period ratio is a function of both the tank H/R (where R is the foundation radius) and E_w/G_s (structure / soil stiffness) ratios. For stiff tanks on soft soils the period ratio is typically in the range of 1.2 to 2.0.



The overall damping in the impulsive mode equivalent SDOF system is shown in Figure 6 for $t/R_w=0.01$. As for the period ratio, the damping is a function of both the H/R and E_w/G_s ratios. For most stiff tanks with $H/R < 1$ and founded on relatively soft soils the overall damping is likely to be greater than 10%. Damping for squat ($H/R < 0.5$) stiff tanks on soft soils may be as high as 20 to 35%.



6 CONCLUSIONS

A more rational approach for specifying force-reduction factors for prestressed concrete storage tanks is required.

The results of the present research have shown that there is a considerable reserve of strength in prestressed concrete tanks above the stress conditions permitted for seismic loads in most codes and guidelines. This reserve strength and damping from soil-structure interaction should be used as the basis for specifying force-reductions in design codes.

7 ACKNOWLEDGEMENTS

Funding from the ACI Concrete Research Council and the California-America Water Company is gratefully acknowledged.

REFERENCES

- Carr, A.J. 1998. RUAUMOKO. Dynamic Nonlinear Analysis, *Department of Civil Engineering, University of Canterbury*, Christchurch, New Zealand.
- Jennings, P.C. & Bielak, J. 1973. Dynamics of Building-Soil Interaction, *Bulletin of the Seismological Society of America*, Vol 63, 9-48.
- Priestley, M.J.N. 1985. Analysis and Design of Circular Prestressed Storage Tanks, *Journal Prestressed Concrete Institute*, Vol 30(4) 64-85.
- Veletsos, A. S. 1984. Seismic Response and Design of Liquid Storage Tanks, *Guidelines for the Seismic Design of Oil and Gas Pipeline Systems*, Technical Council on Lifeline Earthquake Engineering, ASCE.
- Veletsos, A. S. & Tang, Y. 1990. Soil-structure Interaction Effects for Laterally Excited Liquid Storage Tanks, *Earthquake Engineering and Structural Dynamics*, Vol 19, 473-496.
- Veletsos, A. S. & Verbic, B. 1973. Vibration of Viscoelastic Foundations, *Earthquake Engineering and Structural Dynamics*, Vol 2, 87-102.
- Whittaker, D. & Jury, R. D. 2000. Seismic Design Loads for Storage Tanks, *I2WCEE*, Auckland, New Zealand.