# Seismic Performance of the Wall Foundation Joint in Concrete Reservoirs

WORKS
Consultancy Services

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## TABLE OF CONTENTS

# LIST OF FIGURES

# 1. INTRODUCTION

Works Consultancy Services Ltd (WORKS) submitted a proposal to the Earthquake and War Damage Commission (EQC) to investigate aspects of precast concrete reservoir design. The proposal was prompted by an apparent weakness in the wall/foundation joint of some existing structures identified by WORKS through its extensive background and experience in reservoir design and strengthening. EQC accepted the proposal and commissioned WORKS to undertake the study.

WORKS first identified the need to provide for shear restraint at the wall/foundation interface under seismic loading in a study (reference 1) undertaken in 1974/75. The findings of that work along with code recommendations were presented in a paper (reference 2) at the 1975 Technical Conference of the New Zealand Concrete Association (then called NZ Prestressed Concrete Institute). Subsequently these provisions were included in DZ 3106:1976, the draft code of practice for the design of concrete structures for the storage of liquids and more recently in the latest version of that code, NZS 3106:1986.

While the concept of seismic shear restraint was sufficiently recognised and accepted to justify code inclusion, there remained an element of scepticism within the engineering profession as to the theoretical validity of the concepts involved. The purpose of this study is to provide sufficient theoretical evidence to allay this scepticism and to produce design guidelines and recommendations which will improve the seismic safety and integrity of both new and existing concrete reservoirs.

## 2. SCOPE OF STUDY

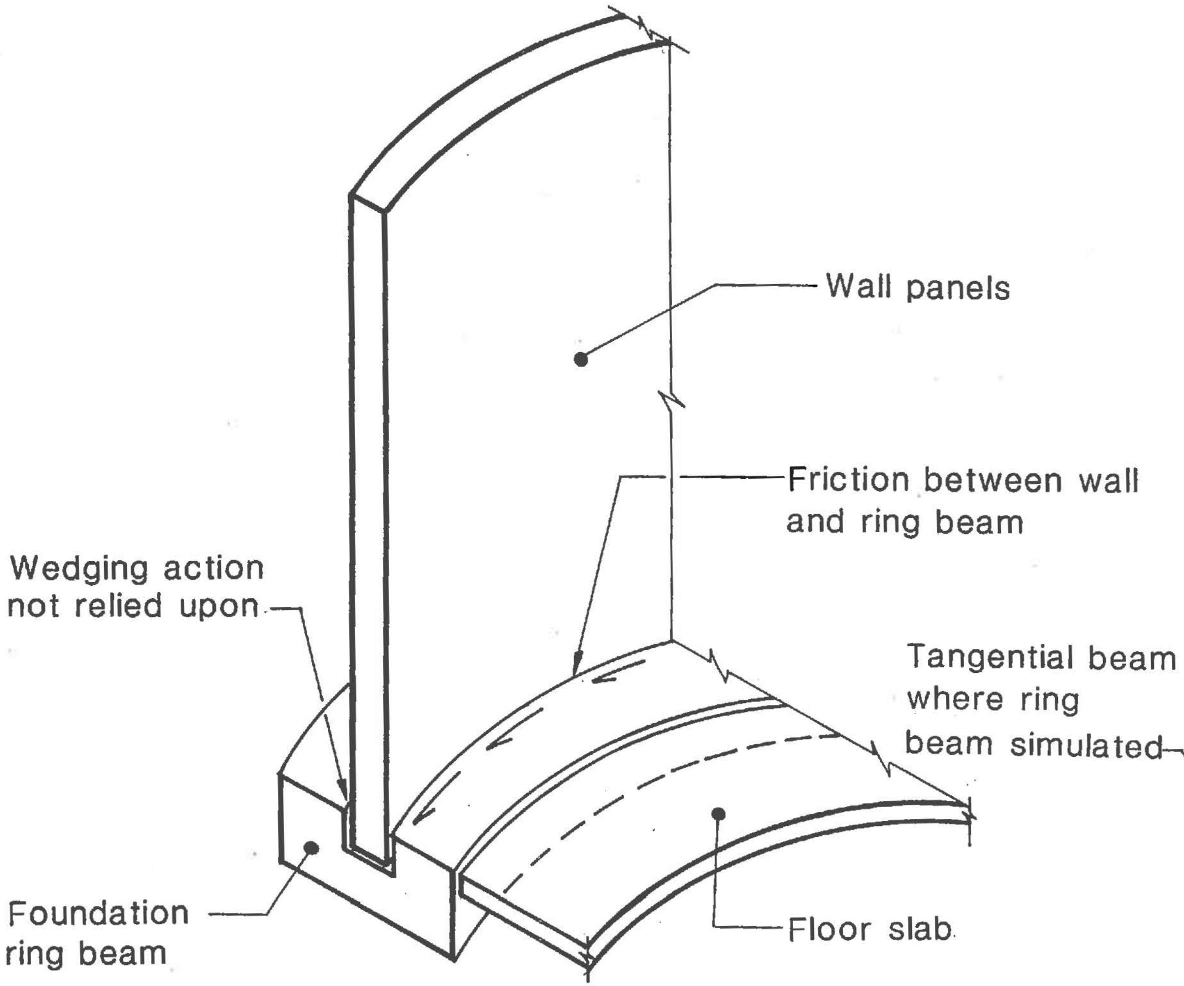
The main purpose of this study is to verify the theoretical validity of shear transfer between the walls and foundations of circular concrete reservoirs under seismic load conditions. In many older reservoirs, bases of walls simply sit in a slot in the footing with no mechanical attachment. A typical base detail is shown in Figure 1A. Because of complex structural action of the wall in the slot under earthquake loading, much higher hoop stresses than predicted by normal design methods may be induced in the lower part of the wall in some reservoirs.

Post 1975 when WORKS first identified this aspect, most engineers in New Zealand included horizontal dowel bars from the bottom of the precast walls into the foundation ring beam. These appear to provide good shear transfer between the two elements. Such a detail is illustrated in Figure 6.

A finite element computer model was developed to simulate the processes and mechanisms involved. Stresses calculated using traditional manual analysis of simple static load cases were used to validate the computer model. This model was then used to investigate seismic stresses in the reservoir wall for a range of joint configurations and load transfer mechanisms.

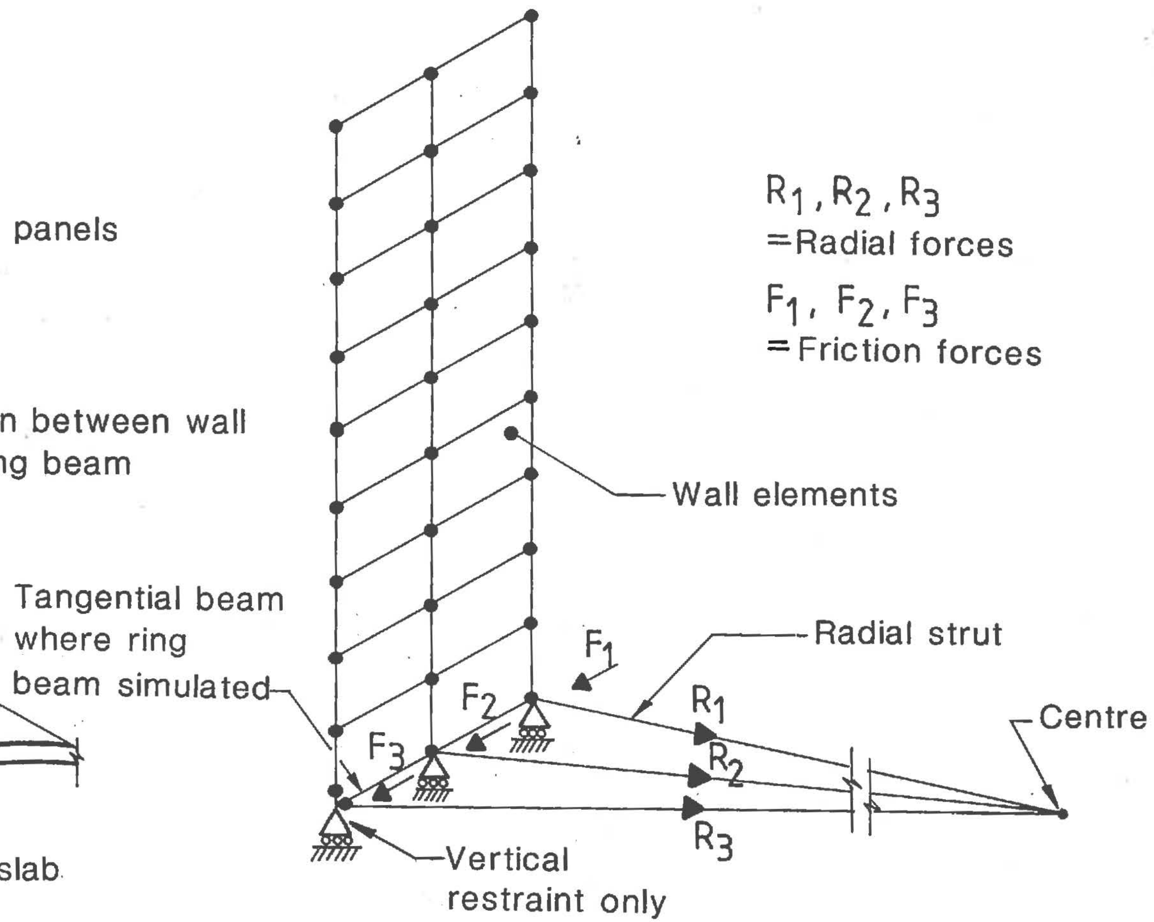
In the course of examining shear transfer at the wall/footing joint, investigations identified another potential problem area. It was found to be common practice for the floor slab to be cast separate from the foundation ring beam with no reinforcement connection between them.

This study reveals that unless the ring beam is substantially reinforced and thoroughly connected to both the wall and the floor slab, potential deficiencies exist for transfer of seismic forces resulting in excessive steel stresses in both the walls and the foundations.




TYPICAL STRUCTURE PRE 1975 NEW ZEALAND PRECAST RESERVOIR

FIGURE 1A




$F_i = R_i \mu$

where i=1 to 80 and $\mu$ = Coefficient of friction

IDEALISED COMPUTER MODEL

FIGURE 1B

FIGURE 1: RESERVOIR MODEL

A study of seismic stresses and load transfer mechanisms in the total reservoir-fluid-ground system has been carried out. Stresses computed for a number of different scenarios show that particular attention needs to be given to connection at the foundation level. Design methods routinely used in the past and even some in current use may not be sufficient to ensure sound seismically resisting structures. Guidelines and recommendations are provided to avoid these weaknesses.

The investigation involved the following activities:

(i) A calculation of seismic stress levels in the walls of three existing reservoirs with assumed sound base connections was carried out using standard hand analysis techniques and design practices;

(ii) Hand analyses of eight existing reservoirs with inadequate base connections (slot fixing) were carried out to establish critical stresses at the base of the walls under seismic loads. Friction effects of the sides of the slot were included in the analysis by integrating force resultants around walls. Walls were assumed to slip in the tangential direction when the friction coefficient $\mu$ reached 0.7;

(iii) A finite element computer model was set up to check results in (ii) above and to provide greater insight into stress distributions;

(iv) The finite element computer model was extended to examine the case where the walls are thoroughly connected to the foundation ring beam but the ring beam is not connected to the floor slab;

(v) Methods were developed to design new reservoirs according to sound principles and to carry out remedial work on existing reservoirs.

This study considers only circular reinforced and precast concrete reservoirs. The study does not relate to rectangular reservoirs. Buried reservoirs were not specifically considered although many of the findings apply.

## 3. ANALYSES USING THE NEW ZEALAND RESERVOIR CODE NZS 3106:1986

A total of three reservoirs were analysed fully for seismic effects using the New Zealand Code of Practice for Concrete Structures for the Storage of Liquids NZS 3106:1986. Whereas analyses in the next sections concentrate on identified deficiencies in reservoirs this section covers the total seismic performance of reservoirs which are thoroughly fixed at their bases. The object of this section was to confirm that reservoirs were generally satisfactory from a seismic viewpoint in all respects apart from the base fixing detail.

NZS 3106:1986 provides clear guidelines for seismic load cases. Based on the assumption of a pinned wall base with no tangential slippage, tank wall stresses have been computed for three existing reservoirs. The resulting stresses, listed in Table 1 (numbers 1, 3 and 6), are not significantly in excess of code requirements of NZS 3106:1986, ranging from complying to 125% of the specified maximum allowable. In all cases a risk factor of 1.6 (refer NZS 3106:1986) has been considered appropriate for municipal reservoirs. The risk factor of 1.6 prescribes an increase in seismic load of 60% over levels set for less critical applications.

The selection of reservoirs assumed to be fully pinned at the base (numbers 1, 3 and 6 in Table 1), were found to nearly meet code requirements. Catastrophic failure of the wall is not likely to occur even though some slight overstress is present. In other words, if adequate load transfer is provided from the reservoir walls into ground, then a typical reservoir in New Zealand Zone A is likely to perform very satisfactorily in terms of the code. The reasons why inadequate base fixing will prevent this otherwise good behaviour in many reservoirs is discussed in the following sections.

## TABLE 1 SEISMIC EVALUATION OF EXISTING RESERVOIRS

| | Reservoir Location | Capacity | Seismic Zone<br><br>NZS4203 19.. | Description | Year of Construction | Wall in slot detail | Seismic hoop stresses at bottom of wall as per NZS 3106 | Seismic analysis results other then identified base fixing detail* | Comments |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Nelson | 5700 $m^3$ | A | Precast, pretensioned and post-tensioned | 1972 | Yes | 179% | 122% of NZS 3106 seismic load case | Appears in good condition |
| 2 | Nelson | 2300 $m^3$ | A | Reinforced | 1961 | Yes | 108% | Meets NZS 3106 requirements | Appears in good condition |
| 3 | Palmerston North | 4500 $m^3$ | A | Precast, pretensioned and post-tensioned | 1974 | Yes | 161% | 125% of NZS 3106 seismic load case | Appears in good condition |
| 4 | Queenstown | 4500 $m^3$ | A | Precast, pretensioned and post-tensioned | 1972 | Yes | 225% | Full seismic analysis not carried out | Not site inspected |
| 5 | New Plymouth | 4500 $m^3$ | B | Precast, pretensioned and post-tensioned | 1962 | Yes | 150% | Full seismic analysis not carried out | Not inspected on site |
| 6 | Gisborne | 9500 $m^3$ | A | Cast insitu post-tensioned | 1962 | Yes | 160% | Meets NZS 3106 seismic load case | Good condition |
| 7 | Auckland | 4500 $m^3$ | C | Precast, pretensioned and post-tensioned | 1965 | Yes | Meets code requirements | Full seismic analysis not carried out | Not inspected on site |
| 8 | Invercargill City | 6000 $m^3$ | B | Precast, pretensioned and post-tensioned | 1971 | Yes | 153% | Full seismic analysis not carried out | Not inspected on site |

Notes:
1) *Reservoirs analysed assuming an ideal pinned base
2) Percentages given are as a proportion of allowable stress

# 4. HAND ANALYSES OF SLOT FIXING

Many older precast concrete reservoirs in New Zealand (pre 1975) do not have a complete connection between walls and foundations. Instead of providing for shear transfer to the foundation around the wall, the wall panels simply sit in a slot (Refer Figure 1A).

Traditional hand analysis methods were used to calculate excess stresses which arise under seismic load due to inadequate connections at the base of the wall. A total of eight reservoirs were analysed, located from Invercargill to Auckland (see Table 1).

The following mechanisms were assumed for hand calculations:

(1) The wall rests on a hard rubber strip approximately 20 mm thick such as RB 200 or other flexible compound, i.e. vertical contact forces do not restrict tangential sliding of the wall in the slot. It is normal practice to seat walls on a flexible compound to facilitate movement during stressing.

(2) The friction in the tangential direction resulting from the contact force perpendicular to the base of the reservoir wall is effective in resisting wall sliding. A coefficient of friction of 0.7 was assumed and this results in a significant contribution to seismic strength of the reservoir.

(3) Panel surfaces are relatively smooth where embedded in the slot and that no wedging action occurs. While there will undoubtedly be some wedging action, it would not be considered good engineering practice to rely on it.

(4) Seismic loadings are as specified in NZS 3106:1986.

(5) When a very high hoop tension develops in the wall, prestressing strand stretches and can carry no more load, i.e. is limited by ultimate tensile stress but is assumed not to fail (rupture). Under seismic load applied to a typical wall-in-a-slot type reservoir, this occurs at about 5 MPa applied concrete hoop stress. When additional load is applied after the prestress hoop force reaches its capacity, all additional load is carried by hoop compression on the opposite side of the reservoir.

Results in Table 1 show hoop stresses ranging up to 225% of allowable and significant vertical bending stresses are also present. These stresses represent serious overstress under seismic conditions, much higher than the designer would have envisaged.

There is a significant likelihood of tensile failure of a reservoir wall because of high local stresses. Loss of contents is considered a high probability in the event of a major earthquake. This failure (loss of contents and permanent deformation) of seven of the eight reservoirs analysed is likely even though the walls would be strong enough to survive a large earthquake if they were adequately fixed.

## 5. COMPUTER ANALYSES

A computer model was set up to simulate stresses in a typical reservoir. A small section of the idealised computer model is shown in Figure 1B where restraint and boundary conditions are illustrated.

An existing reservoir in New Zealand Seismic Zone A (NZS 4203) was selected for the computer modelling and analyses. The reservoir, located in Gisborne, has a storage capacity of 9500 $m^3$ and is illustrated in Figure 2. It was designed in the 1960s as an early prestressed concrete design. Although walls were cast insitu they were separated from the foundation and cast in a slot. In many respects the reservoir is typical of a modern precast concrete reservoir.

A complete plot of the finite element grid is shown in Figure 3.

Equivalent pseudo static loadings were taken from NZS 3106:1986 "Code of Practice for Concrete Structures for the Storage of Liquids" as follows:

(i) A zone A seismic region was selected.

(ii) This resulted in an impulsive force coefficient of 0.64.

(iii) The convective coefficient was 0.09.

(iv) Pressure distributions from the NZNSEE document "Seismic Design of Storage Tanks" were used. These pressure distributions are illustrated in Figure 4.

(v) The weight of the walls and roof was included with the impulsive load case.

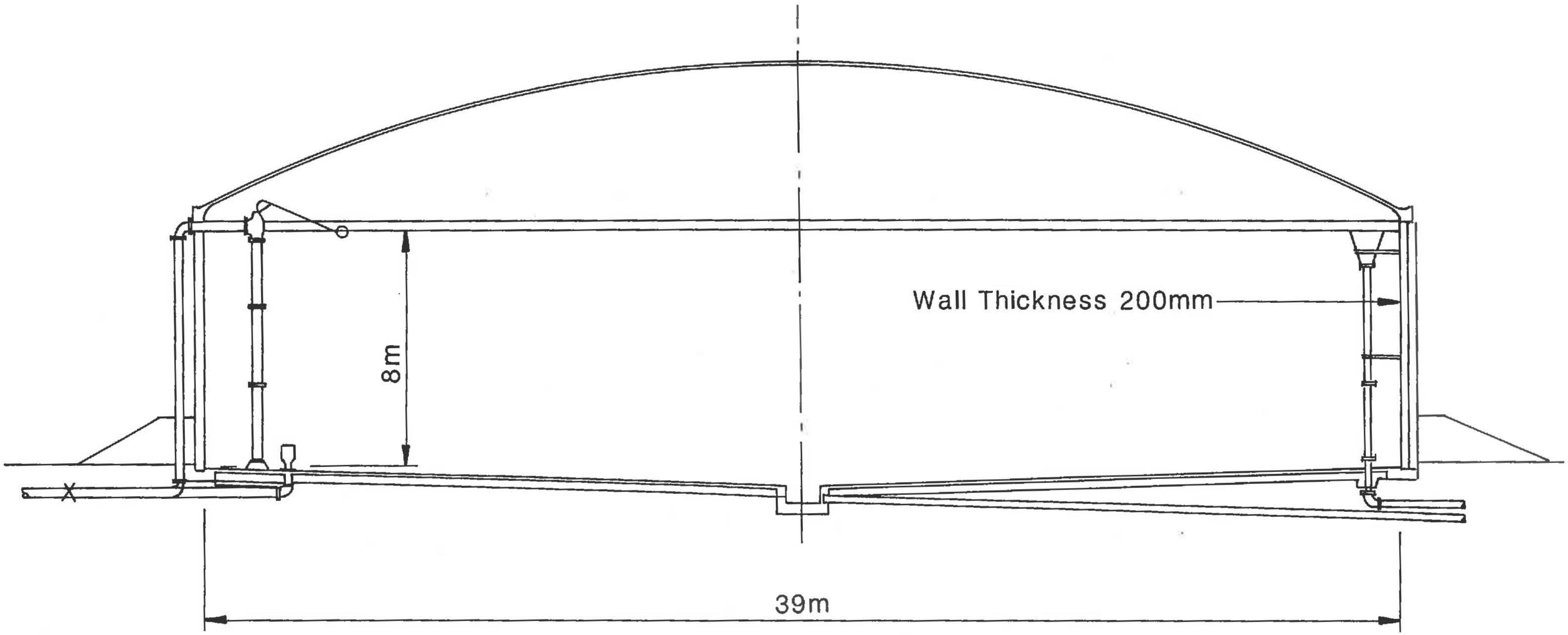



FIGURE 2: 9500 $M^3$ ZONE A RESERVOIR USED FOR ANALYSIS

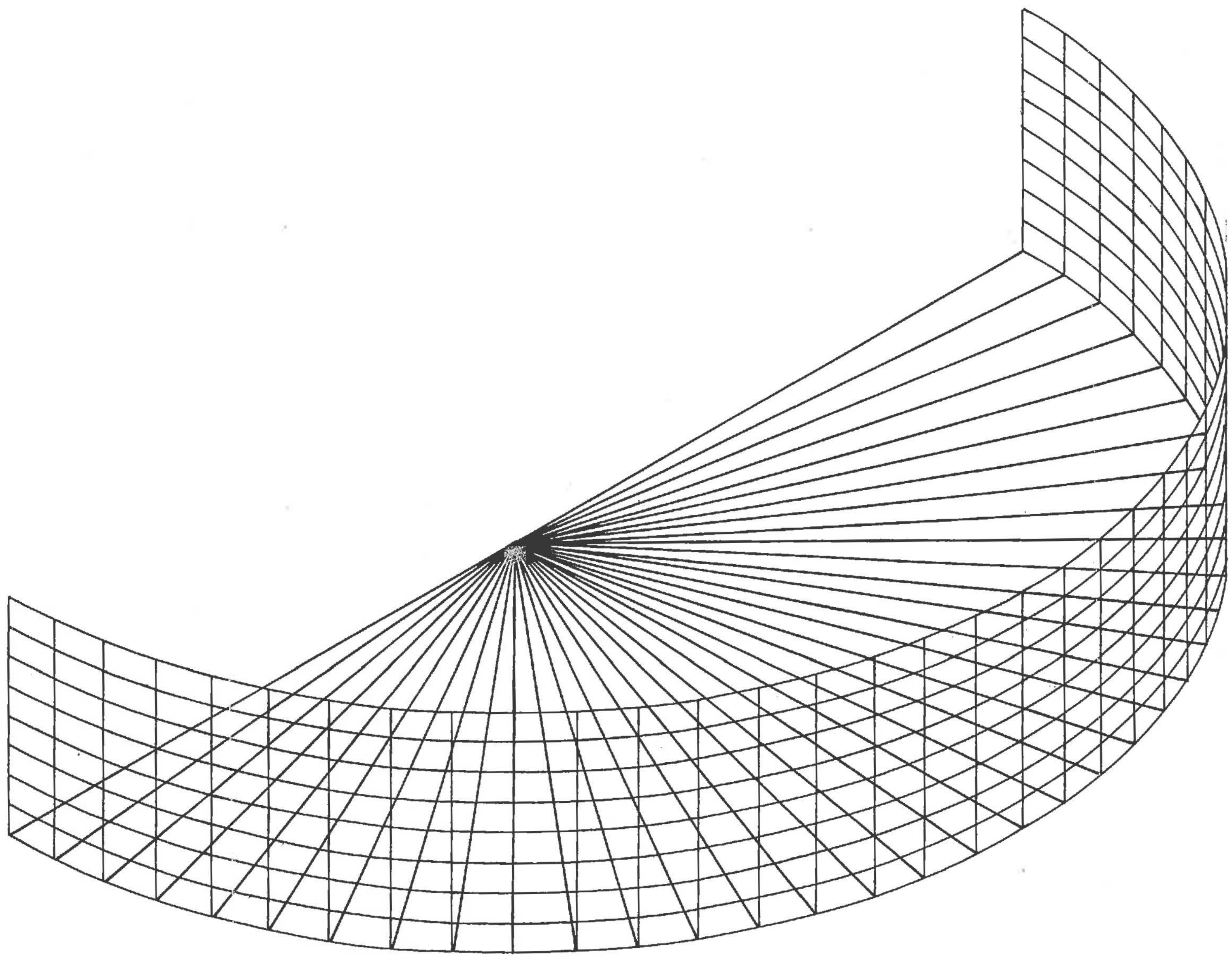
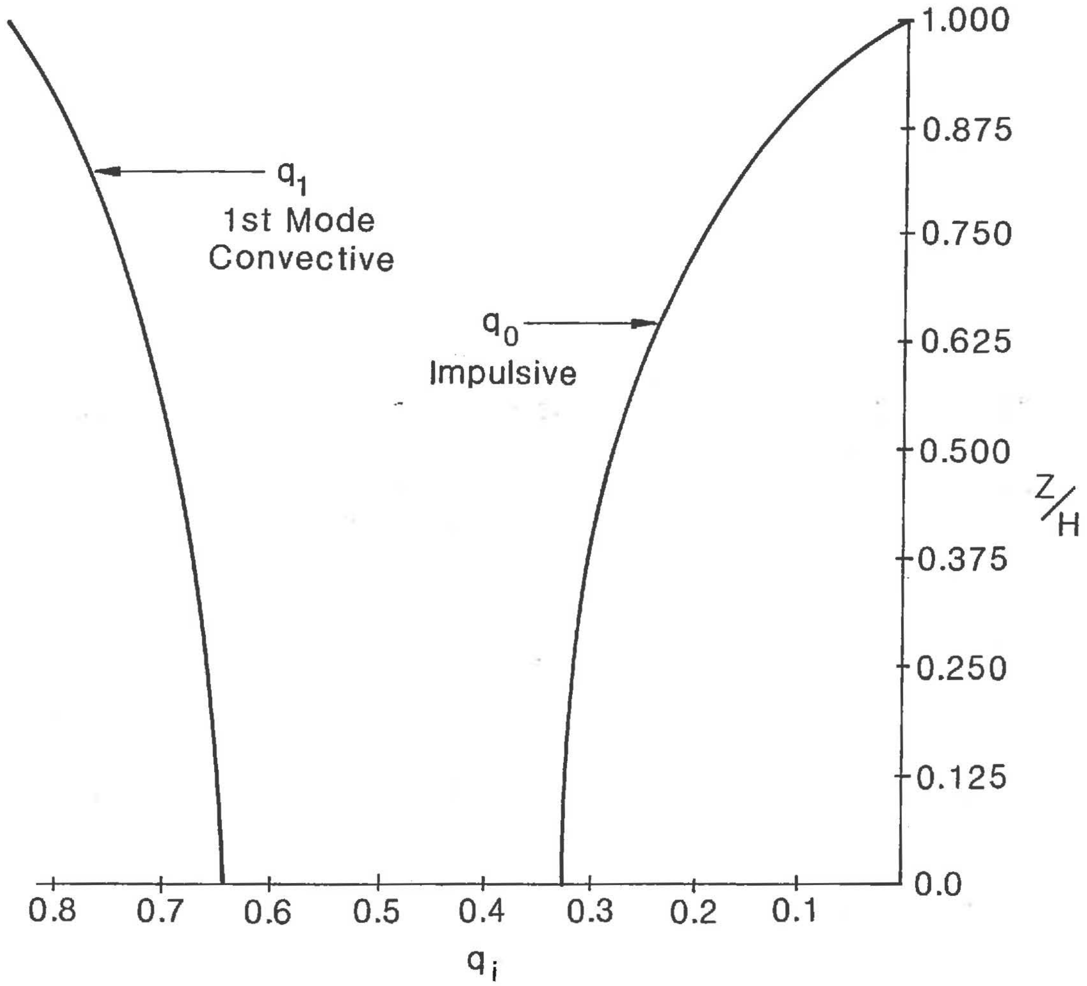


FIGURE 3: COMPUTER FINITE ELEMENT MESH




IMPULSIVE – $P(Z,\theta) = q_0(Z)\ C_h(T_0)\ \gamma_\ell\ R\ \text{Cos}\theta$

CONVECTIVE – $P(Z,\theta) = q_1(Z)\ C_h(T_1)\ \gamma_\ell\ R\ \text{Cos}\theta$
(1st MODE)

Where – $q_0(Z)$ and $q_1(Z)$ are plotted above

– $C_h(T_0)$ = impulsive acceleration at period $T_0$

– $C_h(T_1)$ = convective acceleration at period of first mode $T_1$

– R = tank radius, $\theta$ = angle around tank

– $\gamma_\ell$ = liquid unit weight

FIGURE 4: IMPULSIVE AND CONVECTIVE LOADINGS

(vi) Separate analyses were carried out for impulsive, connective and vertical load cases. Stresses for these three cases were combined using the SRSS (square root of the sum of the squares) method.

(vii) Stresses induced by hydrostatic loads may be added to seismic stress results although this was not done for the results presented in this report.

Consisting of hybrid plane stress - plate bending finite elements, the model was able to simulate the following seismic load cases:

CASE 1: Pinned Base

This case is the typical theoretical model assumed in reservoir analysis. The base of the wall while fixed radially and tangentially provides no moment restraint. By way of illustration, the wall in Figure 1B has a pinned base in this case.

CASE 2: Slot Fixing (Frictionless)

This case represents the situation where the base of the wall sits in a frictionless slot in the foundation ring beam. It is an idealisation of typical pre 1975 reservoirs where a complex transfer of seismic loads occurs. The physical situation is illustrated on Figure 1A while a portion of the computer model is shown in Figure 1B. In this case friction forces F1, F2, F3 in Figure 1B are zero.

CASE 3: Slot Fixing (With Friction)

Case 3 assumes slot fixing as in 2 above except that contact friction is computed and incorporated in the model. This is thought to more accurately reflect the actual situation of pre 1975 reservoirs. Friction forces in Figure 1B (F1, F2, F3) were calculated assuming a coefficient of friction of 0.7 and using iteration of the computer model to converge on a nonlinear solution.

CASE 4: Unconnected Slab (Frictionless)

In this case the wall is rigidly fixed to the annular foundation beams but with the foundation beam not mechanically attached to the floor slab. This is an idealisation of many reservoirs being constructed in New Zealand at the present time. Friction between the floor slab and the annular ring beam was ignored.

To simulate the case the following were included in the analysis:

(i) Beam elements attached to the base of the wall were used to represent the foundation ring beam;

(ii) Concrete to soil friction under the ring beam (average soil types assumed);

(iii) Passive pressure on side of ring beam mobilised with radial movement of the foundation;

(iv) Radial contact forces between the annular foundation beam and the floor slab.

CASE 5: Unconnected Slab (With Friction)

This case was as for Case 4 except friction between the foundation ring beam and the floor slab was incorporated.

The load transfer mechanisms for Cases 4 and 5 are illustrated in Figure 5. Most of the sliding resistance under seismic load is provided by contact of the floor slab on ground developing friction under the weight of water. Relatively little resistance is provided by passive pressure on the side of the ring beam or friction under the ring beam. As a result, loads must be transferred from the ring beam into the floor slab by compression contact, placing the base of the wall and ring beam into tension.

A coefficient of friction of 0.7 was assumed at the side of the ring beam where it is in contact with the slab. Assumptions were made on friction and passive resistance of the annular foundation and their contribution towards resisting to total lateral forces on the reservoir system. However these resistance mechanisms were found to be small in comparison with that provided by the floor slab with a weight of water on it.

As in Case 3 above, friction between the foundation ring beam and the slab assists in transferring the load. Nevertheless hoop stresses at the bottom of the wall and in the annular foundation beam are higher than ever envisaged by the designers.

In practice, where there is no mechanical connection between the side of the ring beam and the outside edge of the slab, the "trailing edge" is likely to separate from the floor slab, ceasing to transfer load (see Figure 5).

As a result, increased load is applied to the "trailing edge" ring beam (diametrically opposite). The ring beam and base of the wall have limited capacity in tension and the stresses are likely to be high for the NZS 3106 seismic design case. As the reinforced concrete ring beam can carry only about 2 MPa equivalent tension, clearly this is an undesirable situation.

An investigation was carried out to determine what influence the sequence and timing of wall stressing and base fixing had on tension forces/stresses in the foundation and walls of the reservoir. Two construction types considered were:

(i) Type 1: walls are free to slide during prestressing and creep is allowed to occur for seven days before casting the connection between walls and foundation ring beams. In this event about 40% of the effective prestress is applied with the base free to slide and 60% with it pinned (i.e. the effect of creep after pinning the base results in an effect equivalent to about 60% of the prestress being applied whilst the base is pinned). Calculation, using classical circular reservoir theory on the reservoir selected showed that the resulting prestress in the ring beam was 2.9 MPa over the gross 0.387 $m^2$ area;

(ii) Type 2: walls are free to slide while 50% of prestress is applied and pinned for the remaining 50% of prestress. This sequence of stressing results in 3.9 MPa prestress on the foundation ring beam. An evaluation of the limited beneficial effect of this construction sequence is presented in the next section.

## 6. COMPUTER ANALYSIS RESULTS

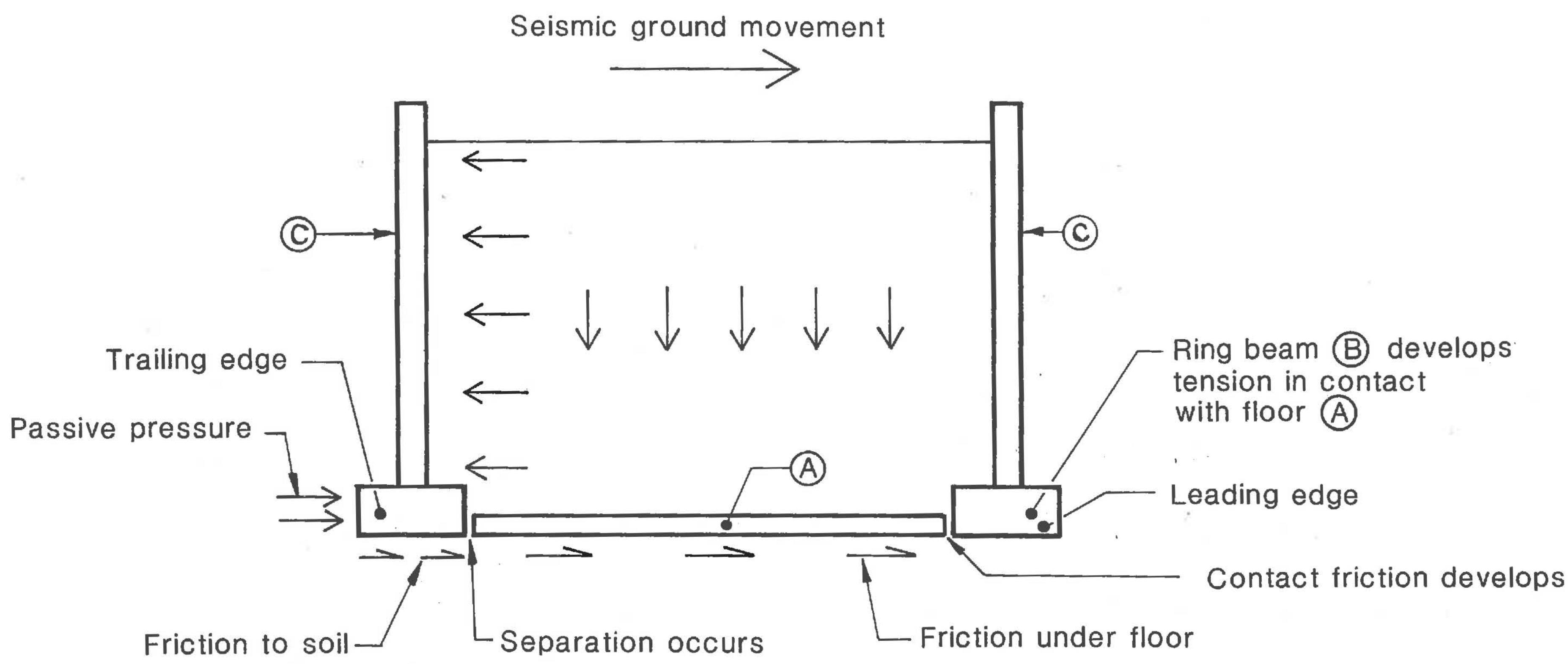
Results of the computer analyses are reported for each of the five cases outlined in the previous section.

CASE 1 : Pinned Base

Wall stresses at the leading edge (i.e. on the diameter in the direction of earthquake movement) calculated by manual methods and by computer analyses, are plotted in Figures 7 and 8.

Hydrostatic analyses carried out by hand and using the computer model compare very closely giving confidence in the model.

A significant variation in wall hoop stresses and wall bending moments calculated with different assumptions arises on Figures 7 and 8. The lines on these figures designated "Case 1" show seismic stresses for a pinned base wall using the seismic load distribution given in Figure 4. These are considered to be the theoretically correct distributions (used for all analyses in this report). The lines designated "EFI" on Figures 7 and 8 are based on a trapezium distribution suggested in NZS 3106:1986 "Code of Practice for Concrete Structures for the Storage of Liquids" to simulate the theoretically correct distributions. Although the discrepancy between lines "Case 1" and EFI" on Figures 7 and 8 is of concern since it is being used by practising designers, it was outside the scope of this study.




1) Majority of seismic restraint is provided by floor slab A friction developed by the weight of water.

2) Annular ring beam B is securely connected to wall panels C

3) Annular ring beam B is not mechanically connected to floor slab A

FIGURE 5: SCHEMATIC OF MODERN NEW ZEALAND PRECAST RESERVOIR

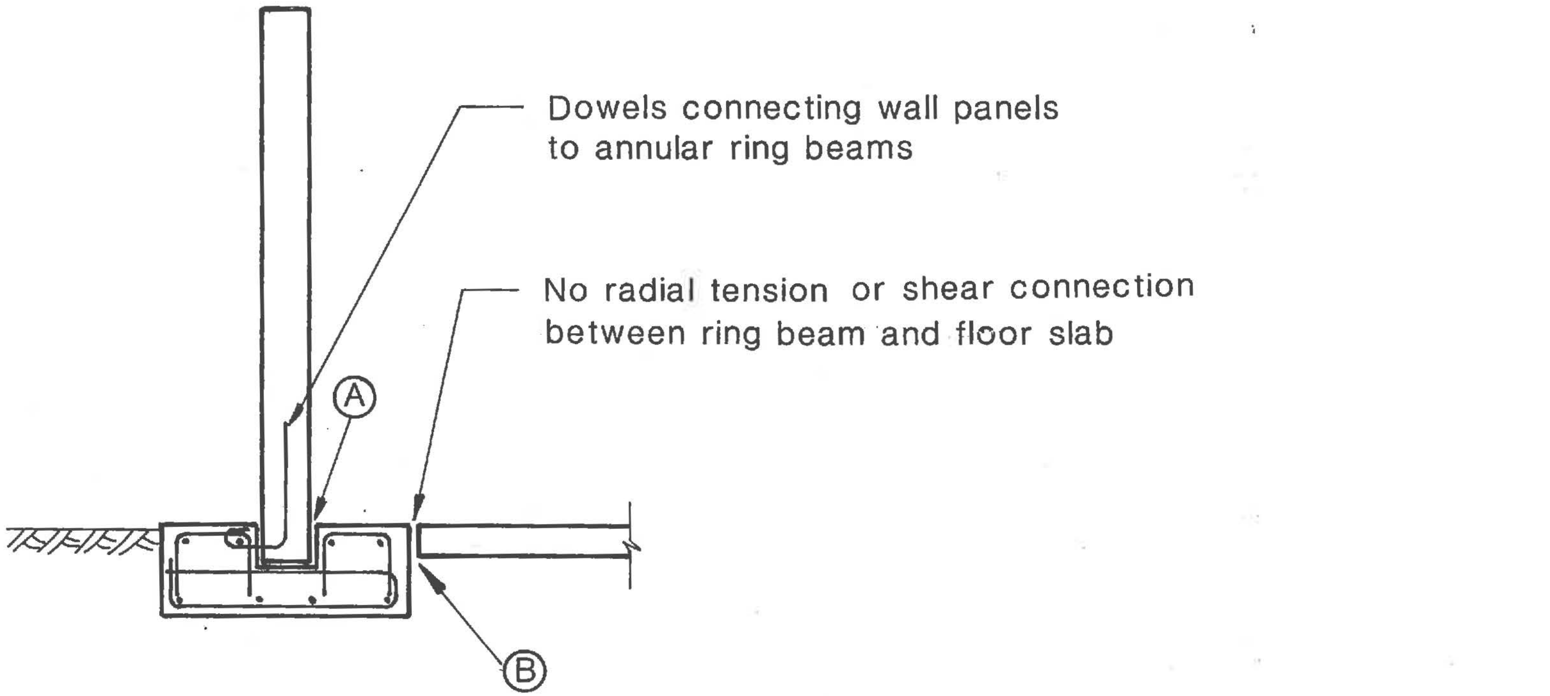



FIGURE 6: TYPICAL DETAIL OF

MODERN NEW ZEALAND PRECAST RESERVOIR

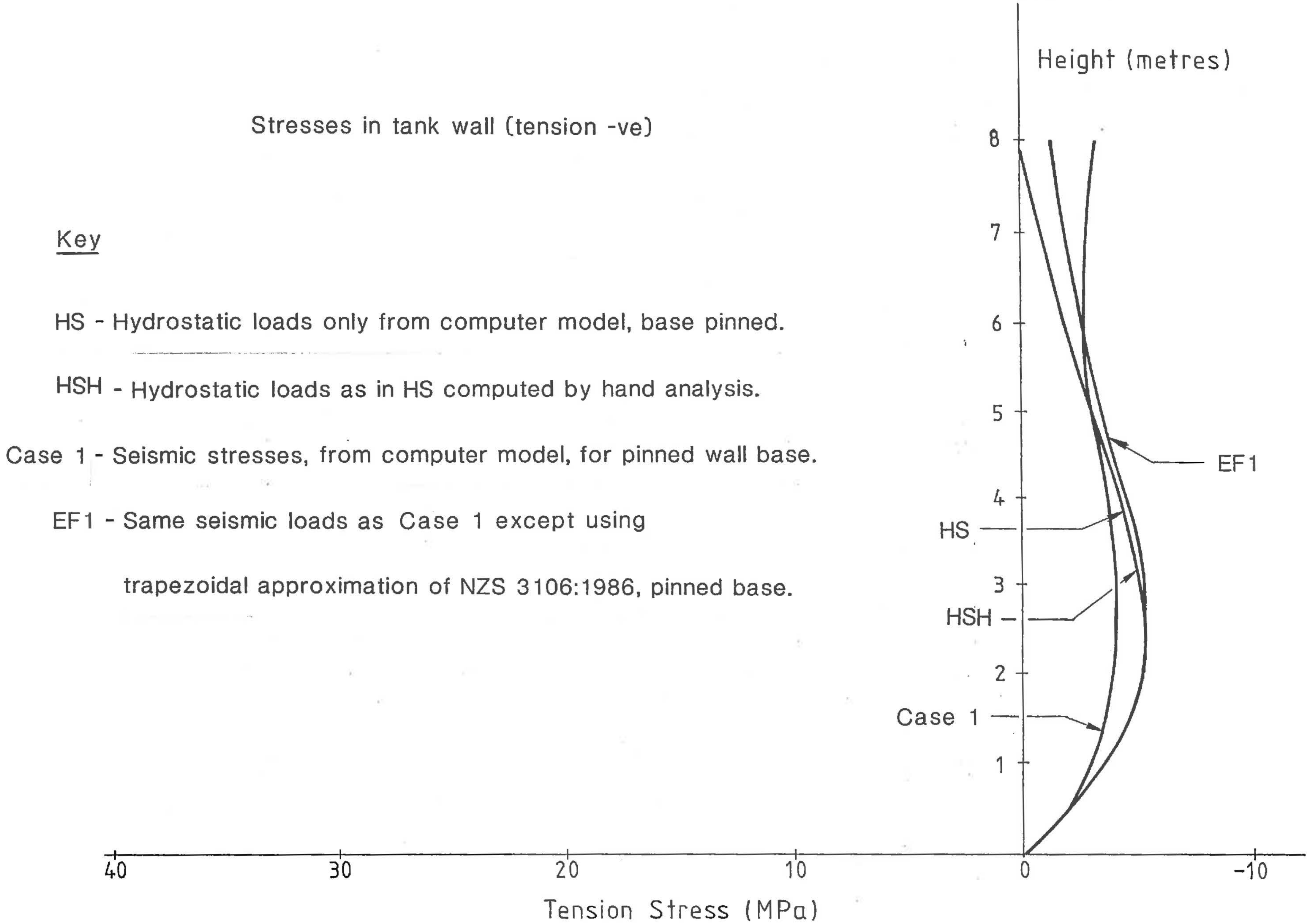



FIGURE 7: MAXIMUM WALL HOOP STRESSES (CASE 1)

Key

HS - Hydrostatic stresses only from computer model, base pinned.

HSH - Hydrostatic stresses from hand calculation, base pinned.

Case 1 - Seismic stresses from computer model, base pinned.

EF1 - Same seismic stresses as Case 1 except using trapezoidal approximation of NZS 3106:1986, pinned base.

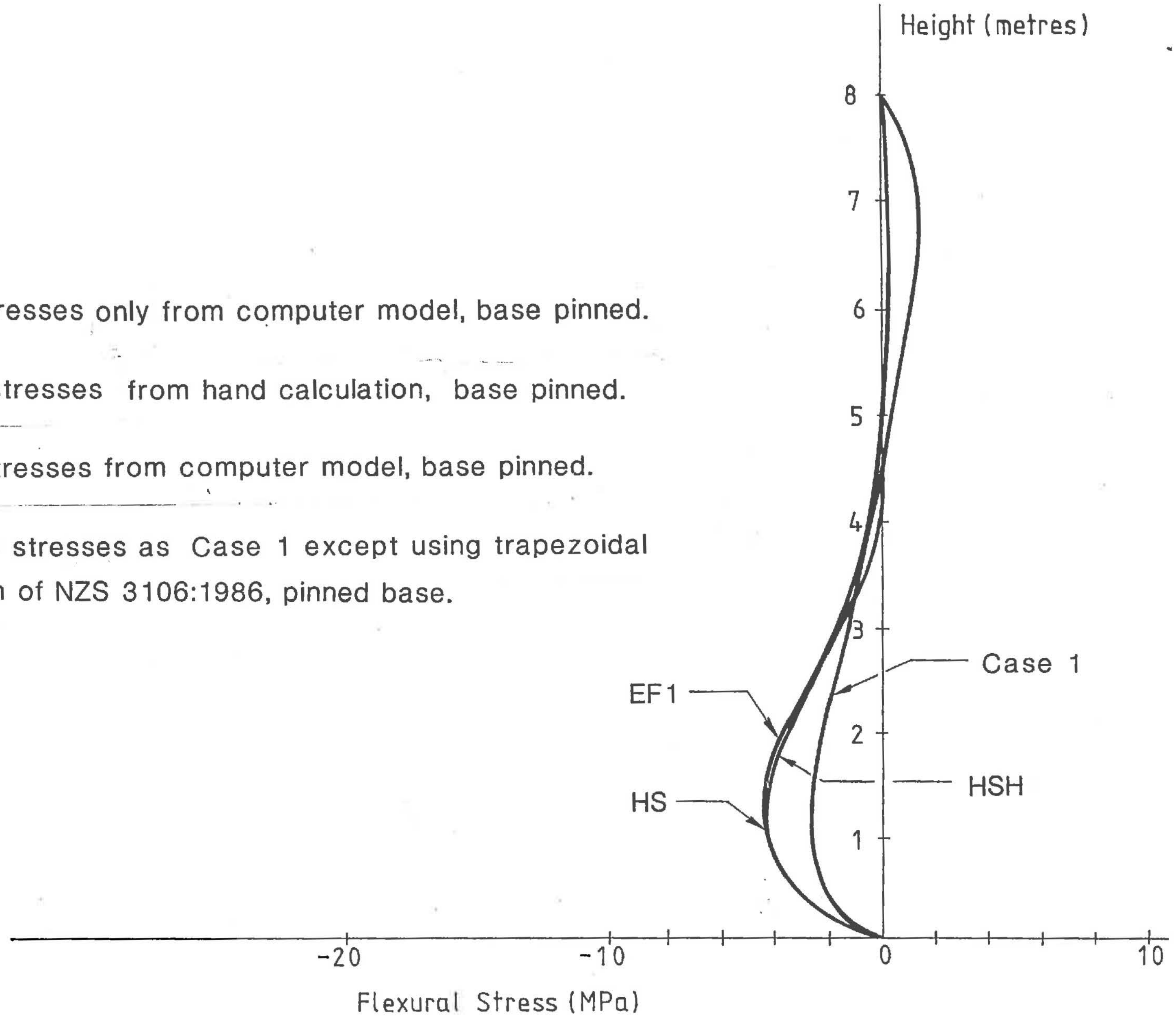



FIGURE 8: VERTICAL BENDING STRESSES IN WALL (CASE 1)

CASE 2 : Slot Fixing (Frictionless)

Hoop stresses at the leading edge as calculated from the computer analysis are given in Figure 9. The results of the manual analysis whilst not shown, are significantly higher. The model assumed for the computer analysis of this case, underestimates stresses. The model was not used in a nonlinear fashion to simulate tension rupture of the walls. In the model it is assumed that the reservoir can carry the full tension load with tensile and compressive stresses of 25 MPa on diametrically opposite sides of the reservoir. In reality the tensile capacity of the prestress is limited to around 5 MPa. To achieve overall equilibrium, the strand will stretch at about the load level and stresses redistribute to the compressive face where they rise to 45 MPa. This effect was not simulated by the computer model. Instead, the reservoir was judged to fail when hoop tensions exceeded the design hoop prestress of 5 MPa. It is worth noting that hoop tensile stresses reach about 18 MPa. Computer hoop stresses computed on the same basis as the hand analyses (permitting stretching of tendons at their ultimate capacity) are likely to be higher than those shown for Case 2 on Figure 9.

Vertical bending stresses are shown in Figure 10.

CASE 3 : Slot Fixing (With Friction)

Friction between tank walls and sides of the slot has a significant beneficial effect in reducing maximum wall stresses. A coefficient of friction of 0.7 was assumed and five iterations were required to converge on the solution. Although there will be variation in the coefficient of friction the selected coefficient demonstrates the effect. Again as in Case 2, hoop compressive stresses are likely to be about double computer results if stretching of tendons is allowed for.

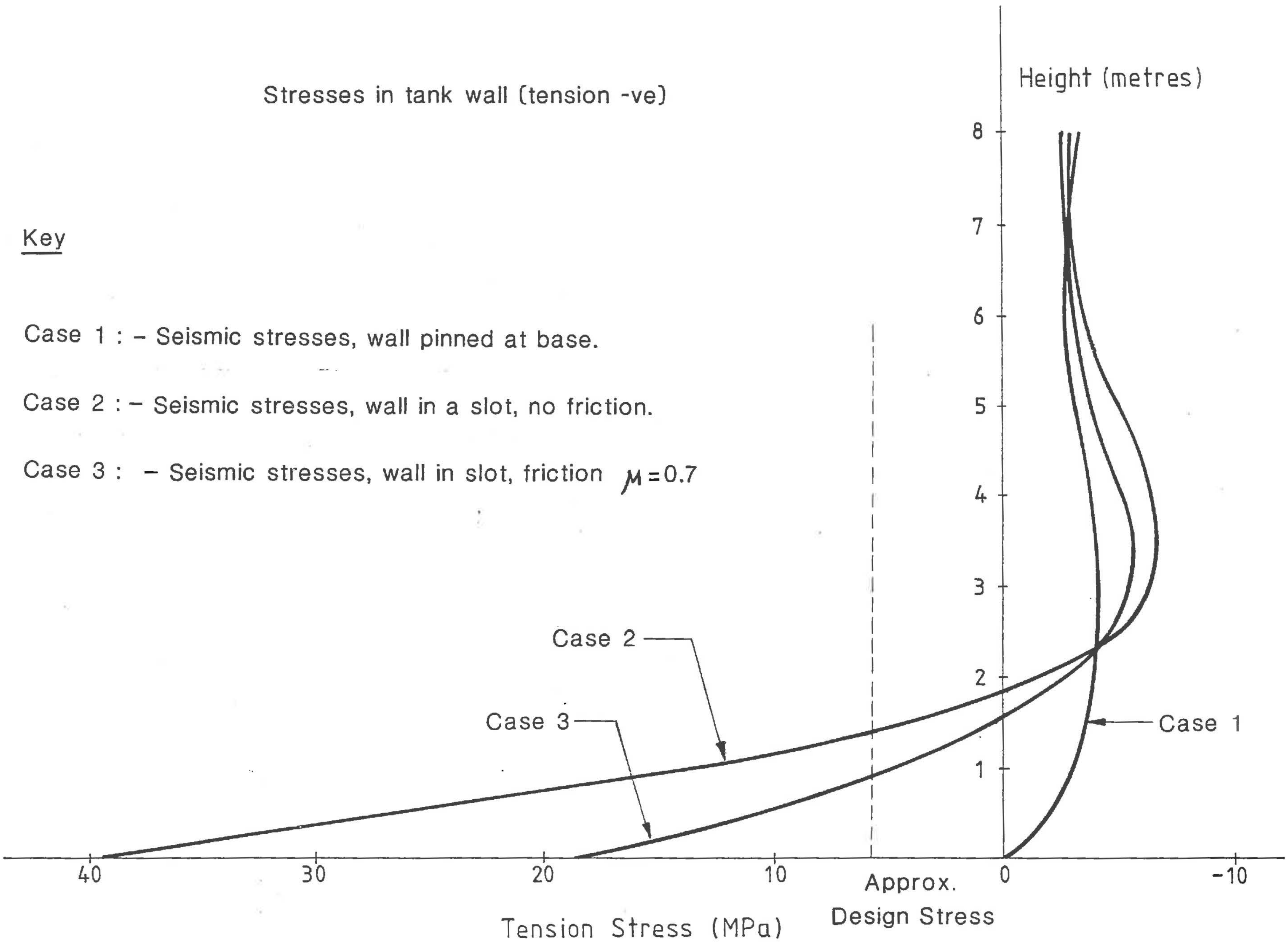



FIGURE 9: MAXIMUM WALL HOOP STRESSES (CASES 1, 2 AND 3)

CASE 4 : Unconnected Slab (Frictionless)

The analysis showed very high forces in the annular foundation beam. Unless very high quantities of reinforcement are present large strains and substantial cracking are predicted. A substantial separation of the ring beam and the floor slab occurs on the "leading edge". Results are shown in Figure 11.

CASE 5 : Unconnected Slab (With Friction)

Results of Case 5 shown in Figure 11 are similar to Case 4 except stresses are lower. These stresses are still too high exceeding code limits for hoop tension. For type 1 construction method from Section 5 (normal practice where creep results in some prestress in the ring beam) the hoop prestress of the ring beam and wall is likely to be about 15% of applied loads. A design approach should be pursued to reduce these high stresses.

Even with the Type 2 construction method from Section 5 (i.e. 50% of prestress applied with the base of the wall pinned), prestress on the ring beam is only of the order of 20% of the applied seismic tensions.

This second construction option is considered undesirable because the prestress in the lower part of the wall is reduced. Large seismic hoop tensions can arise in this region as shown on Figure 11.

It is concluded that the base connection of reservoir walls and ring beam should be specifically designed taking into account the flow of seismic forces from walls into foundations.

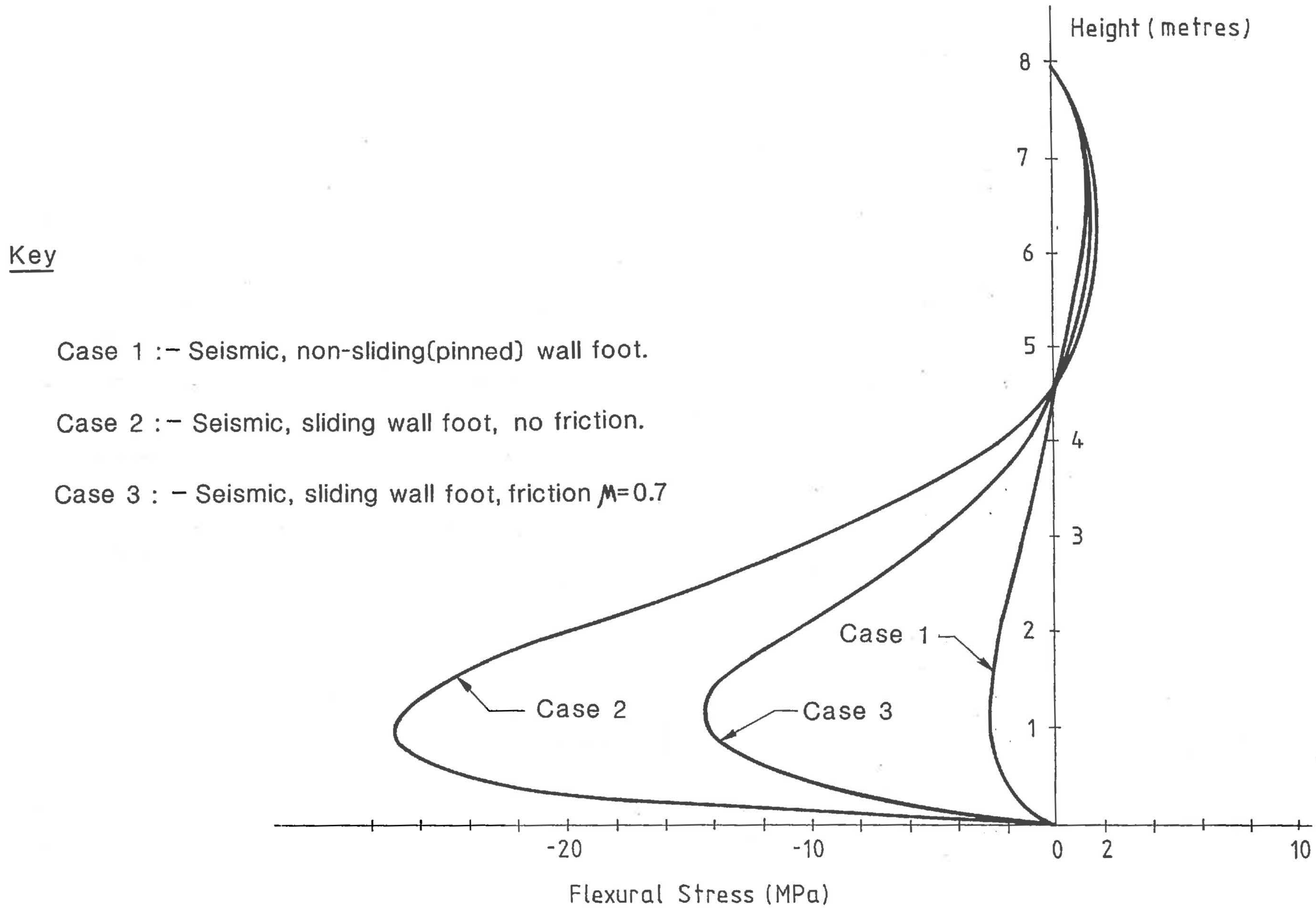



FIGURE 10: VERTICAL BENDING STRESSES IN WALL (CASES 1, 2 AND 3)

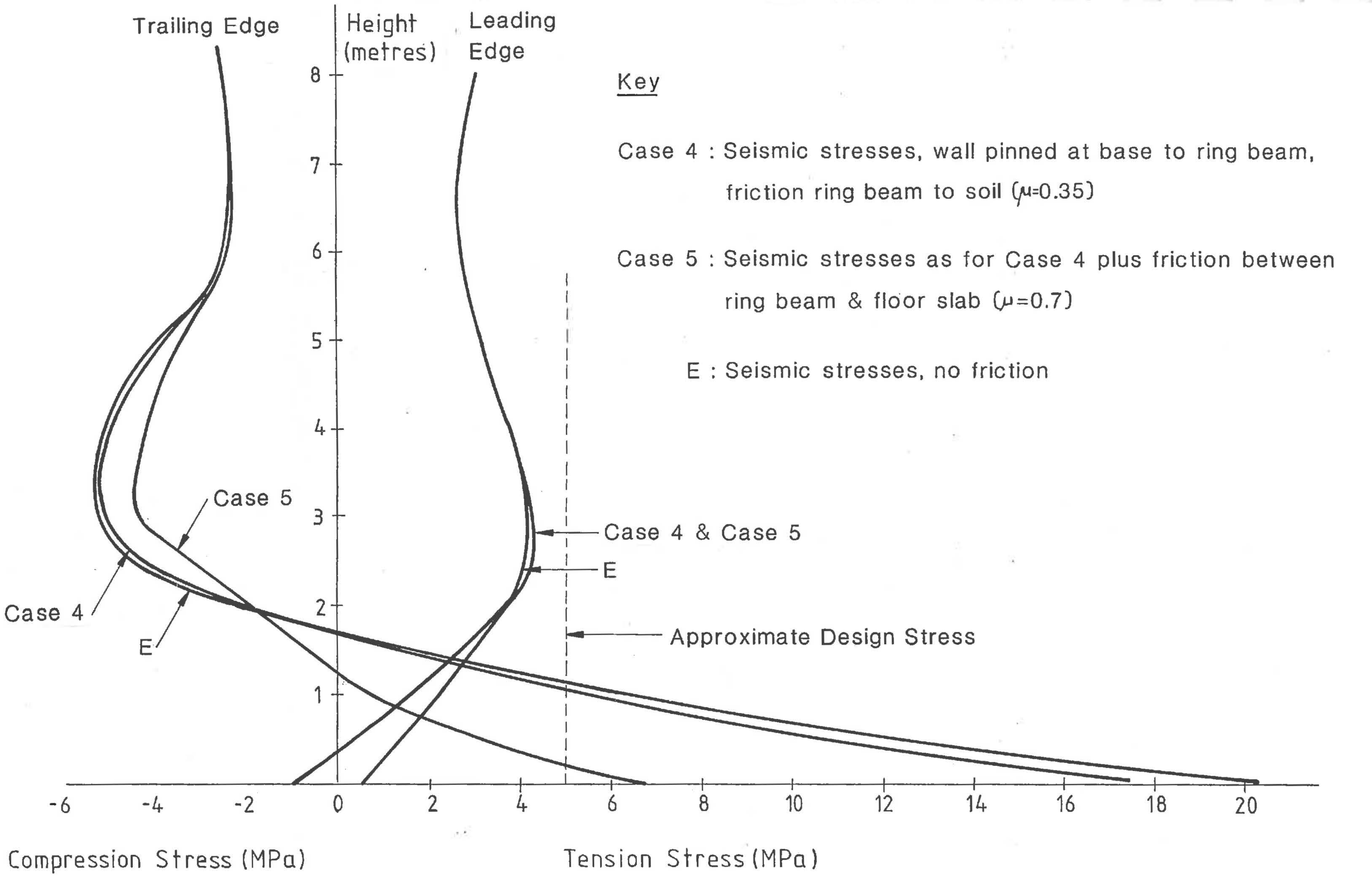



FIGURE 11: MAXIMUM WALL HOOP STRESSES [CASES 4 AND 5]

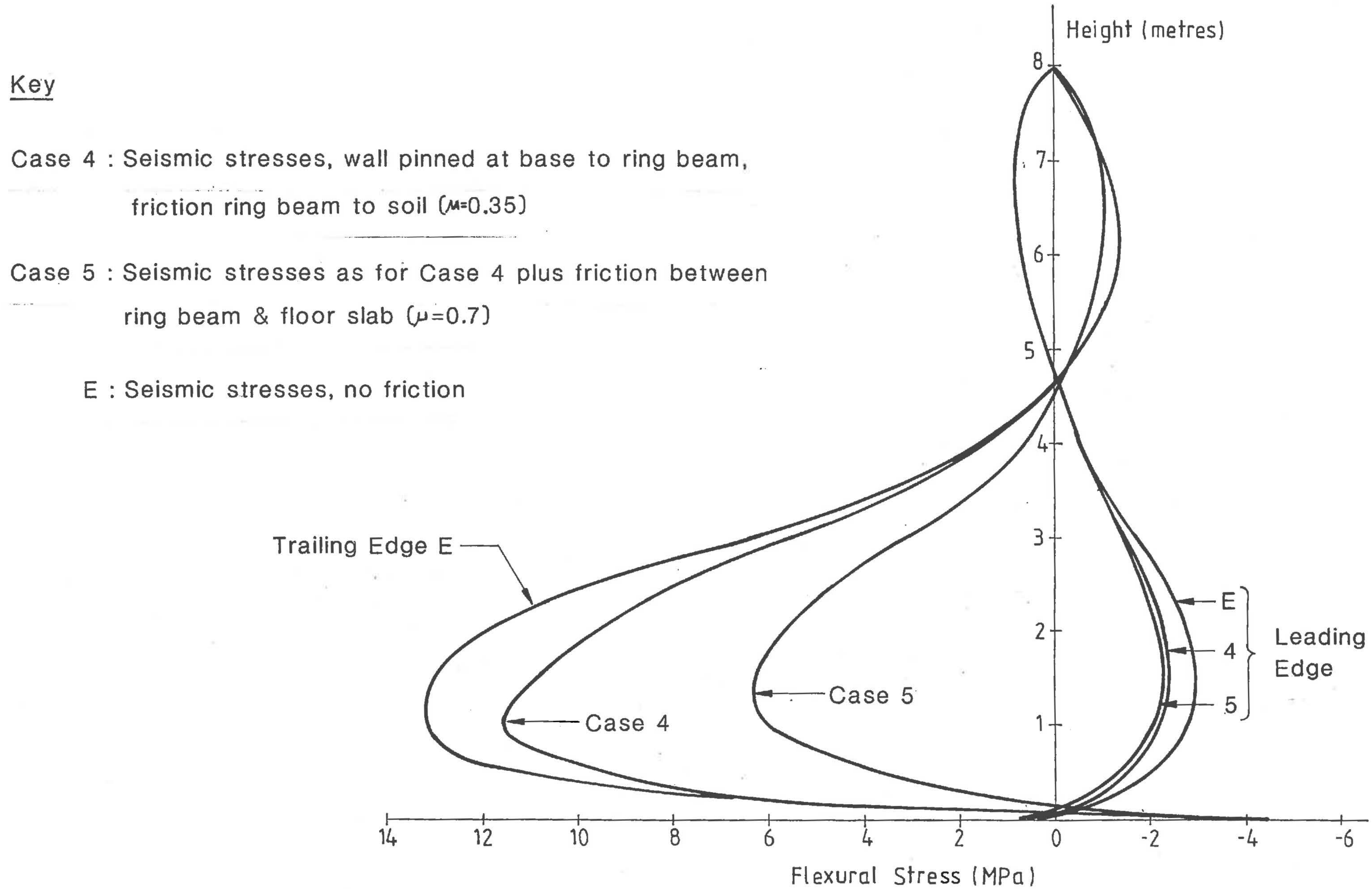



FIGURE 12: VERTICAL BENDING STRESSES IN WALL [CASES 4 AND 5]

Seismic overturning moments were calculated under seismic loads giving an uplift wall line load of 60 kN/m. The restoring force per metre around the wall, taking the weight of walls, pilasters and roof is approximately 62.5 kN/m. Although there was no overturning problem in this case, a reservoir with columns to support part of the roof weight or one with a higher height to diameter ratio will require holding down of the wall.

## 7. REMEDIAL MEASURES FOR EXISTING RESERVOIRS

A reduced life expectancy has been predicted for typical older circular prestressed concrete reservoirs in New Zealand. Technical reasons for this reduction, which relate only to the wall-base detail have been described previously. The solution described here brings the critical wall-foundation interface up to full current seismic design standards.

The strengthening system consists of steel plates epoxy bonded in position on the outside of the reservoir as illustrated in Figure 13. The steel plates transfer membrane forces in the reservoir wall efficiently, directly into the foundation. This is equivalent to providing steel reinforcement between the wall-base and the foundation.

The use of epoxy adhesives avoids the necessity for drilling into thin reservoir walls and interfering with prestress or reinforcement. Epoxy adhesive is a proven civil engineering material and bonding of concrete and steel is well established. For example, applications have included epoxy bonding plates to the underside of bridge beams to increase load capacity.

Workmanship and quality control, especially on preparation of bonded surfaces and application of epoxy, however, are critical.

Secondary effects on the steel plates such as temperature movements and torsional stresses need to be taken into account to ensure a satisfactory design.

This remedial approach has now been applied to two large reservoirs and a number of others are under study.

Costs of strengthening have been established by tender and the following results have been obtained:-

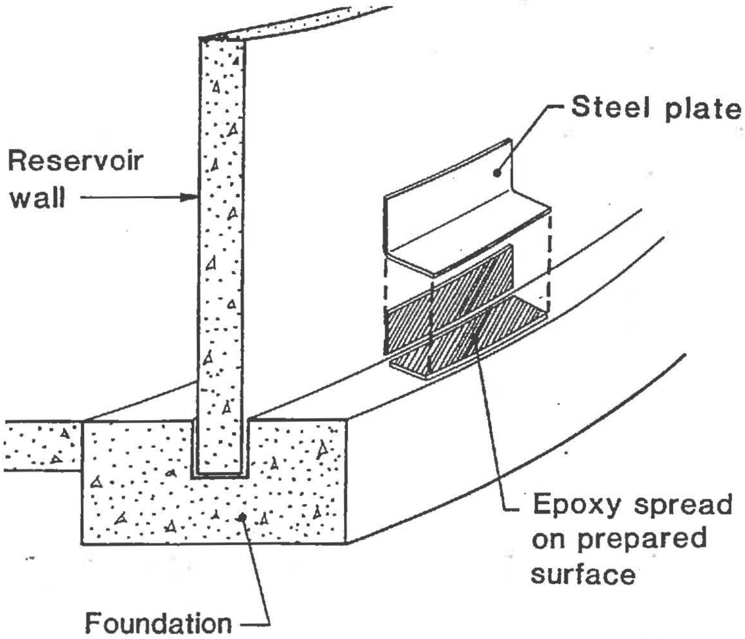



FIGURE 13a: EXPLODED VIEW OF STRENGTHENING PLATE INSTALLATION

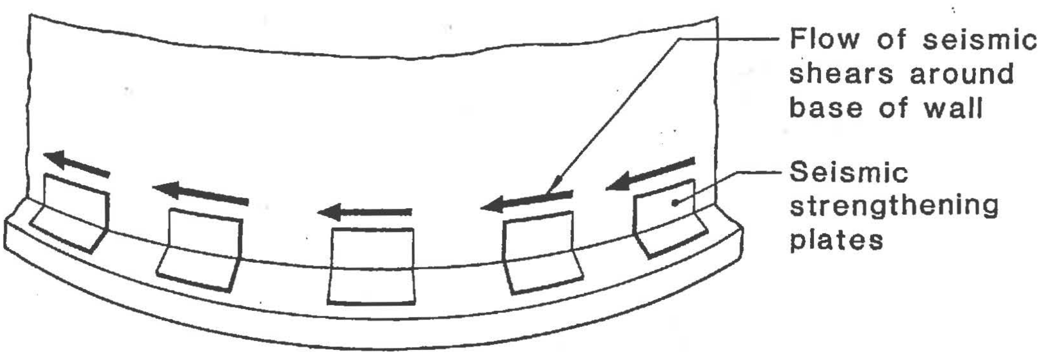



FIGURE 13b: VIEW OF TANK SHOWING FLOW OF LOAD THROUGH SEISMIC STRENGTHENING

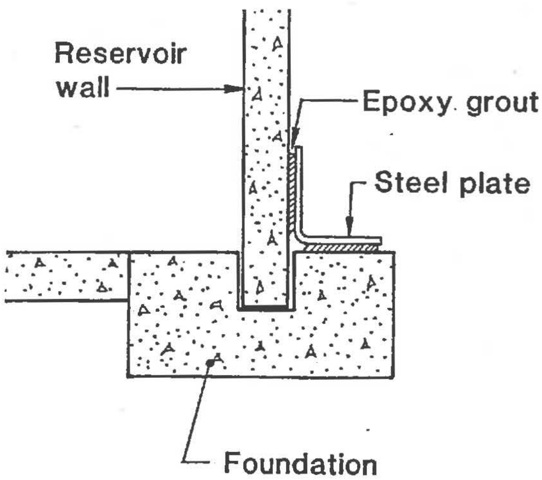



FIGURE 13c: SECTION THROUGH WALL SHOWING STRENGTHENING PLATE

| Reservoir Size | Seismic Zone | Wall in Slot | Full Seismic Strengthening at Base | Approximate Contract Cost Exclusive of GST |
|---|---|---|---|---|
| 9000m$^3$ | A | YES | YES | $17,000 |
| 4600m$^3$ | A | YES | YES | $16,000 |

The seismic upgrade is expected to reduce the risk of wall rupture due to the base fixing or of the reservoir sliding off its foundations during a major earthquake.

Benefits are evaluated in Section 9.

A potential weakness still exists if the foundation ring beam has inadequate tensile reinforcement or if the ring beam is not connected to the floor slab.

## 8. DESIGN RECOMMENDATIONS FOR NEW RESERVOIRS

While the need for detailing seismic connections between walls and foundations is now recognised (NZS 3106:1986) it would appear that the seismic design of large reservoirs has not been widely understood in sufficient depth to design the ring beam-slab connection adequately in all cases. Clearly particular attention is required in this area and recommendations should be incorporated into codes and/or design guides.

Such recommendations should include the following instructions for designers:

(i) Provide continuous reinforcement between floor slab and ring beam. Preliminary calculations have shown that it is feasible to supply sufficient reinforcement to transfer the full shear load in the reservoir walls. A design difficulty with this approach is the tensile stress induced in the floor slab on filling the reservoir. Specific design is required to avoid cracking.

(ii) Alternatively, design dowels for shear transfer between floor slab and ring beam such that cracking of the floor slab is not a critical issue. This detail is illustrated in Figure 14. Again preliminary calculations have shown that it is feasible to provide sufficient reinforcement to transfer all seismic loads.

(iii) Specifically design the sequence of hoop prestress such that some clamping of the ring beam on the foundation is provided. As described in Section 5, Case 5 this is likely to be only partially effective. Specific design is required to establish that the mechanism is effective.

Because it is possible to achieve full transfer of loads across critical interfaces with options (i) and (ii) these solutions eliminate the high stresses reported in this study.

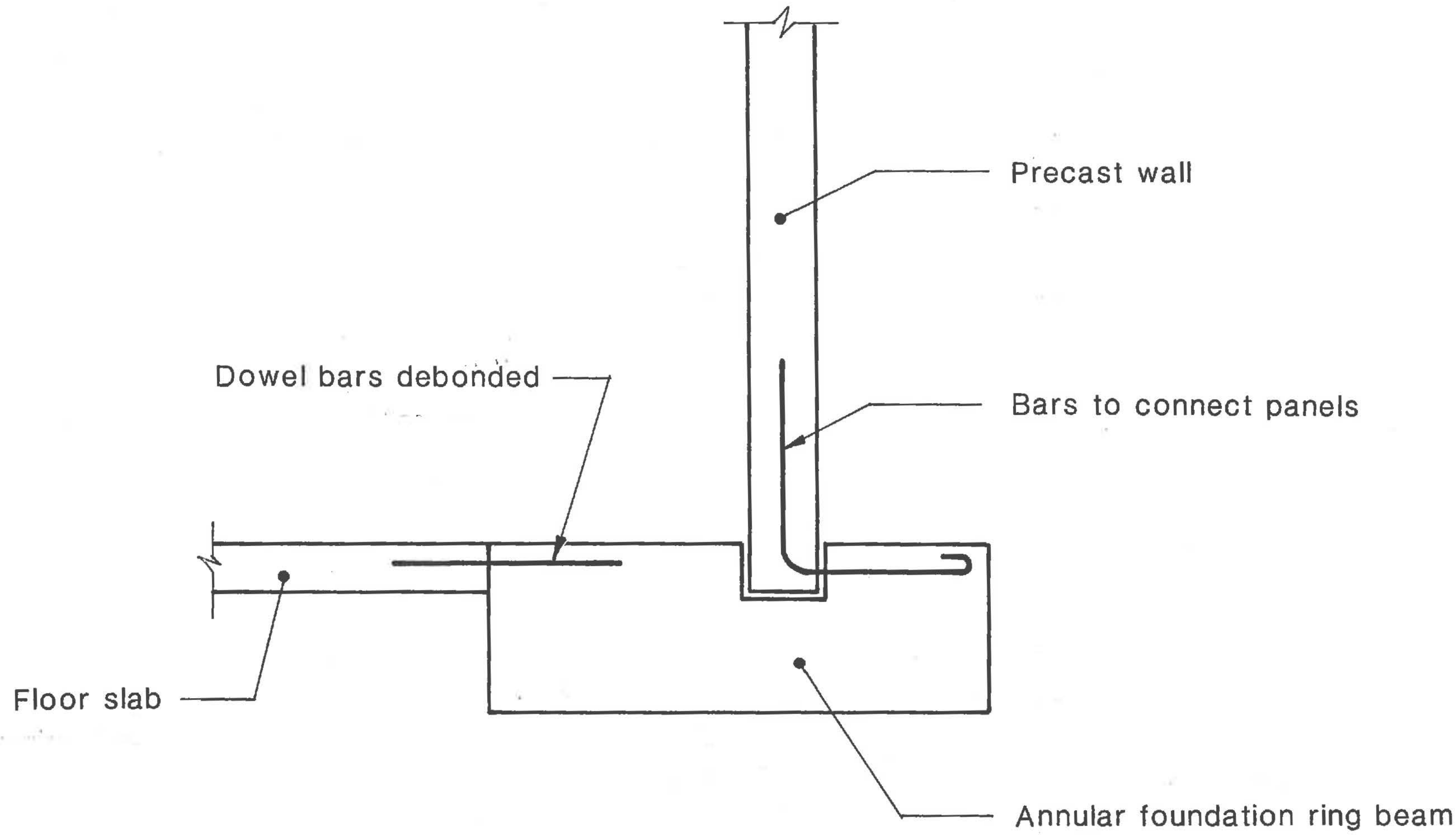



FIGURE 14: PROPOSED DETAIL FOR DESIGN OF PRECAST PRESTRESSED RESERVOIRS

## 9. ECONOMIC ANALYSIS

The aim of this project is to analyse the extent of seismic damage that might be suffered by some existing reservoirs that are seismically substandard and to develop remedial solutions for reducing damage potential.

An important part of this exercise is to determine the optimum level of mitigation. Obviously it is not worth carrying out remedial work unless the expected reduction in damages exceeds the actual cost of carrying out the remedial work.

The expected cost of seismic damage is given by the area under the damage cost vs probability of occurrence curve (Ministry of Works and Development, 1983). Graphically this is illustrated in Figure 15. The expected saving in damage resulting from remedial strengthening is given by the shaded area.

An economic analysis was carried out for one of the reservoirs referred to in Section 7. The reservoir, located in Palmerston North, is of circular, precast concrete construction with a storage capacity of 4600 $m^3$.

Mean return periods (years) for varying earthquake intensities occurring at the Palmerston North location (Smith and Berryman, 1986) are presented in Table 2 along with the estimate of seismic damage which is likely to be suffered at each intensity level. Calculated stress levels were compared with sustainable stresses to derive the extent of damage expected at each earthquake intensity.

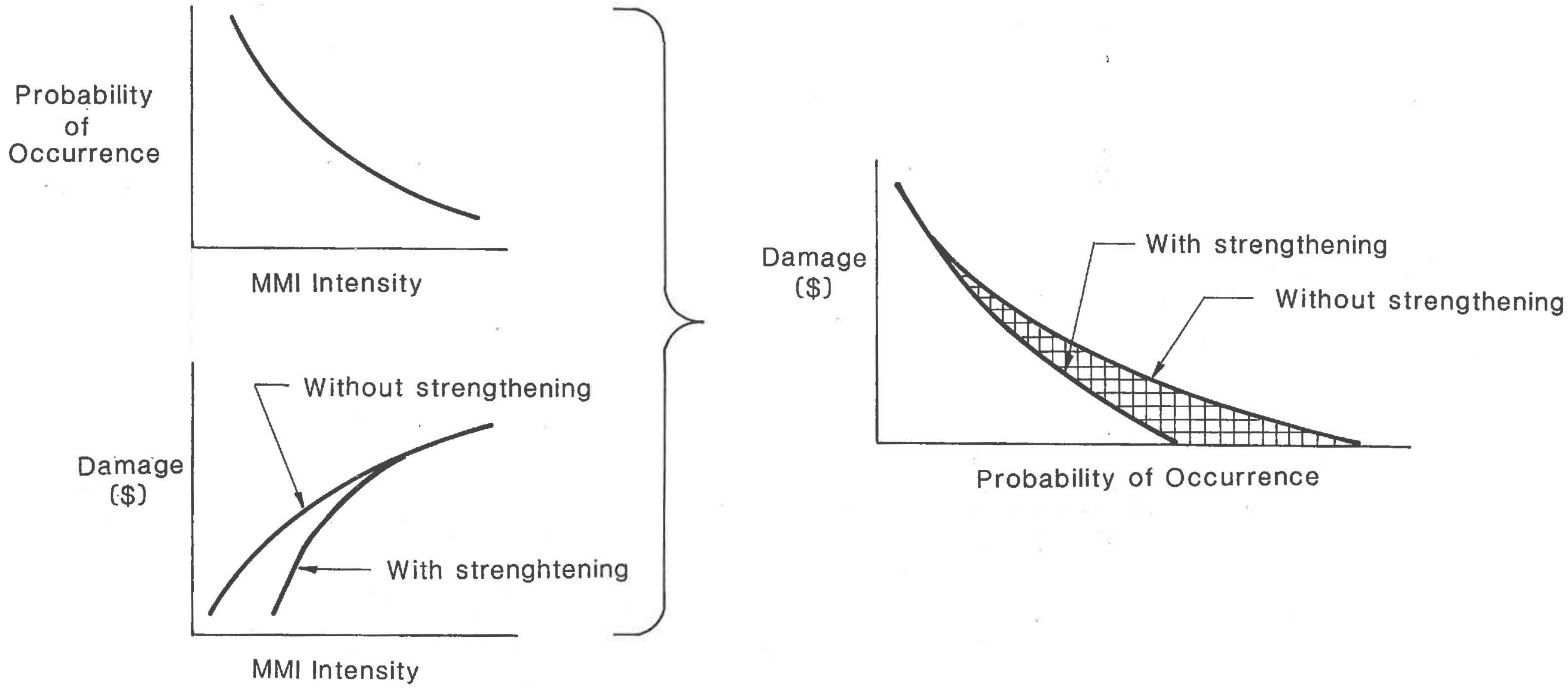



FIGURE 15: GENERALISED RISK MITIGATION PROCEDURE

| SEISMIC INTENSITY (MMI) | RETURN PERIOD (Years) | COST OF DAMAGE ($) | |
|---|---|---|---|
| | | Without Strengthening | With Strengthening |
| VII | 22 | - | - |
| VIII | 70 | 40,000 | - |
| IX | 225 | 120,000 | - |
| X | 833 | 800,000* | 80,000 |
| XI | 3125 | 800,000* | 800,000* |

Table 2 : Seismic Damage

* Total replacement cost for a 4500 $m^3$ reservoir and adjacent pipework.

The proposed remedial strengthening work is expected to result in an annual saving in damages of $3,000, which equates to an equivalent present value of $30,000 when discounted over the life of the structure.

This is only the saving in direct damages to the reservoir itself. There are also indirect costs to society resulting from disruption to the water supply. A generally accepted rule of thumb is that indirect costs will be at least double the direct damage costs.

In this situation we are attempting to assess the incremental benefits from seismic strengthening of the reservoir. In the event of a large and damaging earthquake there is likely to be disruption to the water supply from a variety of sources such as damage to pipework and pumps etc. This disruption may occur regardless of seismic strengthening of the reservoir itself. Consequently it is appropriate to attribute only partial contribution of indirect damages to the reservoir itself.

For the risk analysis it has been assumed that seismic strengthening of the reservoir would reduce indirect costs by a quarter.

i.e. saving in indirect damages = 50% of direct damage costs

In other words total savings = 1.5 direct savings = $45,000.

This potential saving of $45,000 from a reduction in seismic damage, unlike the $16,000 upfront cost for the strengthening work, is not guaranteed or certain. It does however on average represent the most likely expected saving and as such, with a benefit cost ratio of around 2.8, is a sound economic investment.

Application of these seismic provisions to new reservoirs is even more compelling. Similar savings to those given by seismic strengthening of existing reservoirs can be achieved but at a fraction of the cost, with a benefit cost ratio of around 10.

## 10. CONCLUSIONS

Computer modelling techniques have been used to carry out a detailed examination of the seismic performance of circular concrete reservoirs.

A large number of circular prestressed concrete reservoirs in New Zealand do not comply with current codes with respect to seismic wall/base fixing. They are typically satisfactory in every other respect. Supplies of potable and fire fighting water may be lost at a time when they are most necessary.

As a result of the greater understanding obtained, improved recommendations have been provided for the refurbishment of existing and design of new reservoirs. Although complex analysis methods have been necessary to identify and define the problem simple analyses and practical design details should result in improved seismic performance of reservoirs.

Recommendations have been made for the improved design of new reservoirs, and for the seismic strengthening of existing reservoirs with the wall-in-slot base fixing.

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