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Standard Seismic Resistant Details For Industrial Tanks & Silos

K N Crawford, Murray-North

STANDARD SEISMIC RESISTANT DETAILS FOR INDUSTRIAL TANKS AND SILOS

by

K N Crawford

A project sponsored by the Earthquake and War
Damage Commission

For the New Zealand Earthquake Research Foundation

Murray-North Limited
Consulting Engineers, Architects, Surveyors,
Planners, Managers

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106 Vincent Street
P O Box 821
AUCKLAND

Telephone: (09) 798-940
Facsimile: (09) 396-676

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Figure 3.5 is courtesy of Motherwell Bridge Pacific and the AO Smith Permaglas glass coated steel bolted tanks.

1. INTRODUCTION

1.1 BRIEF FOR THE REPORT

This report has been commissioned by the Earthquake and War Damage Commission (EQC) for the New Zealand Earthquake Research Foundation to carry out the following :

1. *Investigation of the design and detailing of various anchor chairs, brackets, holding down bolts, stiffeners and the like that form the seismic resistant details for industrial tanks and silos.*

2. *Provide a series of Standard details to cover the most likely sizes and types of tankage including tabulated load and dimensional data for the anchorage fixings.*

1.2 OBJECTIVES OF THE REPORT

The objectives of the report as derived from the brief are as follows :

1. Provide an overview of the existing types of tankage and their typical hold down systems.

2. Review the Current Practices of specifiers, designers and constructors within the industry.

3. Review existing types of anchorages using details in current use.

4. Provide analysis and design recommendations for seismic holding down for the practical use by designers of tankage.

5. Present standard details for hold down of various types and sizes of tank.

6. Provide a set of Specification Notes to accompany the standard details.

7. Review the various methods of anchorage of tank fixings to concrete foundations and provide design recommendations for these.

8. Provide a brief set of outline requirements able to be used by client/ specifier of tankage and silos to cover seismic anchorage of tankage.

9. Provide a set of references that would assist designers requiring further background information and source material.

2. REVIEW OF CURRENT DESIGN PRACTICES

2.1 DESIGN PRACTICE

A distinct New Zealand Standard code of practice for the seismic design of storage tanks (and their holding down details) does not exist at the present time. Current practice is for the design engineer to apply appropriate parts of the following documents :

1. *New Zealand Society of Earthquake Engineers (NZNSEE) "Recommendations for the Seismic Design of Storage Tanks" - December 1986* : Ref [1]

   a state of the art set of recommendations

2. *"Seismic Design of Petrochemical Plants" - Ministry of Works/Ministry of Energy Publication - May 1981 (SDPP)* : Ref [2]

   (specifically written prior to petrochemical installations being installed in the Taranaki area in the 1980's).

3. *NZS4203 : 1984 - Code of Practice for General Structural Design and Design Loadings for Building*s : Ref [3]

   (includes specific requirements for tanks considered as a building)

4. *API 650 : Appendix E (American Petroleum Institute - Seismic Design of Tankage)* : Ref [4]

   (Seismic design rules for thin walled tankage for the Petroleum Industry).

This situation has led to a variety of interpretations by clients and engineers undertaking design of tanks. Often a design brief will clearly state design requirements specifying seismic load and design codes.

In other instances :

- The client requires some generalised seismic compliance be included e.g. NZS : 4203

- After considering the cost and/or risk, the client does not require seismic load resistance

- Bylaw interpretations by Local Authority Inspectors can result in no seismic load resistance being required.

The approaches taken to seismic design by the various parties involved are reviewed in Section 2.2 following.

2.2 REVIEW OF CURRENT INDUSTRY PRACTICES

Owners

Tankage owners range from the dairy industry, food processing and wine industry, grain storage industry, through to the chemical industry, oil and petrochemical industry.

Owners have a varying perception of seismic risk. This is generally driven by the value placed on throughput of contents of a tank or the costs and strategic implications of a system "outage" after a seismic event. It is often also dictated by regulatory authorities in relation to hazards associated with tank contents.

The economic climate of the industry and accepted "industry standards" also have a significant bearing on the seismic detailing practices required by owners.

The cost of seismic detailing of holding down mechanisms for tanks as a proportion of the total cost of the tank is also a significant factor. For example, detailing of necked holding down bolts for a large milk silo can amount to $5,000 in an $80,000 tank as against $2,000 for plain round mild steel bolts.

For a large petrochemical tank the proportion has far less impact as a percentage of the overall costs.

Specifiers

Current practices of specifiers vary according to the level of involvement of professional engineers in the setting of the design brief.

In the absence of clear guidelines current practices appear to be adhoc. This report, in Section 9 attempts to redress this situation by providing the purchaser with a list of essential components of any design brief sent to a consultant or to a "design and build" tank fabricator.

The involvement of a professional engineer to scope a tankage design brief is recommended in all instances to assist the client to achieve a cost effective solution to the hold down of tankage consistent with practical construction details and with the appropriate level of risk.

Clarification of the interface between the tank designer, fabricator and the civil works/foundation designer is also an area that needs to be addressed. Often these are three separate parties. The continuity of a rational seismic design philosophy through each component of a tank design is essential for the satisfactory seismic performance of the tank.

Recommendations for the definition of these roles are made in Section 9.3.

Fabrication Industry

The fabrication industry in New Zealand has a long history of service to tankage industry niches with various proprietary designs.

Seismic design requirements for tankage holddown is an evolving field as indicated by the two documents published in the 1980's, viz SDPP and NZNSEE Recommendations. Fabricators have, in general, adapted proprietary designs to accommodate the more stringent requirements of these documents. Innovative solutions have been developed to maintain a pricing edge in an increasingly competitive environment.

To date the fabrication industry has, in the absence of definitive code requirements, received a variety of specifications for seismic hold-down of tankage from owners and specifiers, in Requests for Quotations and in Tender Document Specifications.

Often the choice of seismic design level, and method of compliance, is left to the designer/fabricator without any review. In other instances a design certificate is requested or certification by an independent third party is required.

From our review of industry design practice it appears the following documents are utilised by a majority of larger fabricators with inhouse design expertise.

1. NZNSEE Recommendations primarily in use for Design Loadings, and tank/foundation interaction, contents slosh effects, etc.

2. SDPP is used for the detailing of necked bolts and for the concept of hierachy of failure and overstrength in design.

Existing practice appears to avoid the requirement to neck holding down bolts unless specifically requested in a specification. This appears to be based solely on the grounds of cost. An elastic strength design procedure is normally adopted with componentry beyond the actual anchor being designed for varying levels of overstrength.

The review encountered the use of ϕ_o = 1.15 to 1.25 as an overstrength factor. The recommendations in Section 4 recommend as essential the use of higher overstrength factors for limited ductile and ductile designs. These recommendations result in nominal additional costs.

Regulatory Authorities

Tankage falls into various categories in terms of requirements for building permit authority. For example:-

1. From the definition of a "building" contained in Clause 1.1 of NZS 1900, Chapter 1: 1985, a tank is considered to be a building, and therefore subject to the building permit approval process, in the following cases:-

   (i) on ground for tanks greater than 23 m^3
   (ii) 1.8 m above GL for tanks greater than 2 m^3
   (iii) 3.6 m above GL for tanks greater than 0.5 m^3

2. Often indoor tankage above ground is deemed as being plant and equipment and therefore not subject to the building permit process. In this instance tankage, particularly that on elevated floors, has the potential to be not subject to building controls, and to be of potential significance in terms of risk in a seismic event if not adequately detailed.

Each type of tankage often has similar significance in terms of seismic risk of failure, yet is treated differently.

As a result of the lack of definition within the codes of practice there is considerable potential for a variable application of regulatory overview for this type of structure.

2.3 ASSESSMENT OF SEISMIC RISK AND HAZARD - CURRENT PRACTICES

2.3.1 General

Tankage contains a variety of contents with varying levels of hazard should a tank fail in a seismic event.

The criteria usually used for a tank is that the tank or silo remain serviceable for storage of contents, for a seismic event of some specified level. Partial or total loss of contents is not acceptable. A hierachy of a tank failure is required to be established that ensures that the potential for loss of tank contents or of significant damage is minimised.

Levels of hazard vary significantly according to the tank contents and the effects of any spillage from a tank failure on the community. Contents vary from toxic flammable contents to nonvolatile, toxic chemicals.

Tanks also contain products that do not represent a high hazard of themselves, but the failure of a tank represents a specific risk or quantifiable cost to the community or the owner. These include such products as :

- contents of high value but low hazard such as cream stored in stainless steel silos

- contents required to be used in the event of a disaster e.g. a potable water supply or firefighting water supply

2.3.2 Seismic Probability and Risk

NZNSEE "Recommendations on the Seismic Design of Storage Tanks" assigns a probability of a seismic event of a given magnitude being exceeded and relates this to the risk of potential loss of the contents.

The probability is based on a "design" code earthquake and a risk factor is then applied that is related to the contents of the tank.

There are three implied levels of earthquake that a structure can be designed to accomodate. The HERA Structural Steel Design Guides Ref [5] describe these as :

(i) serviceability earthquake, resulting in no noticeable structural damage.

(ii) severe earthquake, when some structural damage may occur, but the structure behaves in a predictable controlled manner. For tankage this implies no loss of contents and the primary function of serviceability for storage is maintained.

(iii) extreme earthquake, where considerable structural damage may occur, while the structure is expected to remain standing with no loss of contents.

Recommendation

The design loading used should correspond to a **severe** seismic loading condition. The current design code loading in NZS 4203 : 1984 corresponds to a **severe** earthquake. The return period is approximately 150 years at Risk Factor R = 1.0 corresponding to 0.007 probability of exceedence (Ref. NZSEE Recommendations).

The design probability should be derived from the NZNSEE Recommendations Table 1.1 and seismic coefficient determined accordingly from Equation 2.1.

However the anchorage should also be seismically detailed to be able to withstand the **extreme** seismic event without catastrophic failure.

3. REVIEW OF EXISTING TYPES OF ANCHORAGE

3.1 ANCHORAGE TYPES

A variety of tank anchorage details have evolved. The sophistication of these details and any energy dissipation inherent within each varies with the size and importance of the tank.

Anchorages can be classed as either non-ductile or ductile.

Non-ductile Anchorages

Anchorages are classed as non ductile when the following failure mechanisms can occur :

(a) The anchor bolt is of brittle material and failure after yield is sudden.

(b) Bolt failure occurs over a small number of threads between the base of the nut and the shank of the bolt.

(c) Concrete anchorage failure (e.g. concrete cone failure or anchor pullout) when the anchor bolt over-strength exceeds the capacity of the concrete anchorage.

(d) Anchor bolt over-strength exceeds the capacity of the anchor chair or other fixing to the tank wall. Dependent on the particular detail failing, this may or may not be non ductile.

(e) Thread stripping of the bolt or nut.

Ductile Anchorages

Ductile anchorages will allow considerable post yield extension/deformation without significant loss of strength. This ensures the tank remains serviceable under severe and extreme seismic events. Ductile materials exhibit a well defined yield point and yield plateau and have the capability of sustaining a number of stress reversals (usually greater than 8 number). Ductile and limited ductile anchorages require specific detailing of fixings to ensure that there is a hierarchy of failure.

3.2 EXISTING INDUSTRY PRACTICE

3.2.1 **Smaller Tankage** (less than 100 m^3) e.g. vats, small silos involving :

- Elastic strength used for design of anchorage.
- Mild Steel Grade 250 AS1204 round bar or plain reinforcing bar to NZS3402: 1989 (Gr 300) with ISO Metric course cut threat.

- Grade 4.6/S bolts or High Tensile 8.8/S bolts used as a means of limiting the number of anchor chairs required to resist overturn. (The use of 4.6/S and 8.8/S bolts is discussed in Section 7).
- Angle cleats or Isolated Anchor chairs - (Figs 3.1, 3.2) are used as fixings.
- Use of overstrength factors in design of other fixing components appears to be adhoc. This issue is addressed in Section 4.4.

Other features:

- A dimensional limit of 3650 mm outer diameter is often used for fully shop fabricated mild steel or stainless steel vessels as a limit dictated by road transport regulations.

- Wall thicknesses in small tankage are determined by the minimum in the appropriate code which often is sufficient for seismic loadings also.

- API 650 is applicable in the Pulp and Paper industry and the Petrochemical tankage industry. Wall thicknesses are a minimum of 5 mm. This provides an excess of seismic resistance capacity for this small diameter tankage.

- The lower strake of the tank wall is often designed to NZNSEE buckling requirements with a 15% overstrength factor.

- Corresponding stainless steel silos for the dairy and process industry have wall thicknesses of 2-3 mm. Anchor chairs and cleat connections usually require the addition of doubler plates to the tank wall to ensure the adequacy of the thin wall section.

- Smaller tanks are either supported on short legs, either on ground or elevated, or base supported on a pedestal. Tanks supported on legs are not further addressed in this report.

- The tank configuration consists usually of a tank diameter of up to 3 metres with the floor supported on a sloped concrete pedestal to permit tank drainage. The tank floor is radiussed up to butt weld with the wall. The wall laps with a skirt that is brought up the face of the pedestal. The tank anchorage is by way of cleats or anchor chairs to the foundation slab at the base of the pedestal.

- Bolts are either cast in situ or grouted. The use of mechanical anchorages such as Terrier bolts is also common.

3.2.2 **Larger Tankage** (100 - 300 m^3)

- Limited ductile or fully ductile design for anchorage.
- Mild steel Grade 250 AS1204 round bar or plain threaded reinforcing bar to NZS 3402 : 1989 (Gr 300) is used with ISO coarse cut thread.

- The use of necked bolts appears to be a function of seismic zone and aspect ratio of the tank. It has also developed from specifiers following NZNSEE and SDPP recommendations for necked bolts.
- Industry sources indicate the costs of a necked holding down bolt relative to a Mild Steel round are in the ratio 2.5:1.
- Fully threaded bars have been used on occasion but stock appears to be no longer available.
- Anchor chairs are commonly used. The use of anchor chairs with a continuous ring is common for vessels of high aspect ratio and in high seismic zones, particularly for vessels up to 300 m^3 - (Figs 3.2, 3.3).
- A continuous top ring is also often used as part of a sheathing support to contain insulation.
- An innovative ductile strap detail for a vessel on a skirt has also been reviewed - (Fig 3.4)

3.2.3 Glass-Coated Bolted Steel Tankage

- Proprietary Glass Coated Bolted Steel tanks have primarily been used for auxiliary firewater storage and for silos in New Zealand. They vary in capacity from 14 m^3 to 9000 m^3.
- Plate thicknesses vary from 2.0 mm to 12 mm. The smaller tanks have low wall thicknesses due to the corrosion protection afforded by the baked enamel finish to the plate. Ref [6] describes the analysis and features of such tanks.
- Use of a proprietary lug connection shown in Fig. 3.5 is made for the fixing. Modifications for New Zealand seismic conditions including larger lugs, holding down bolts and the additional fixing of the angle cleat to the bottom strake have been the common practice.

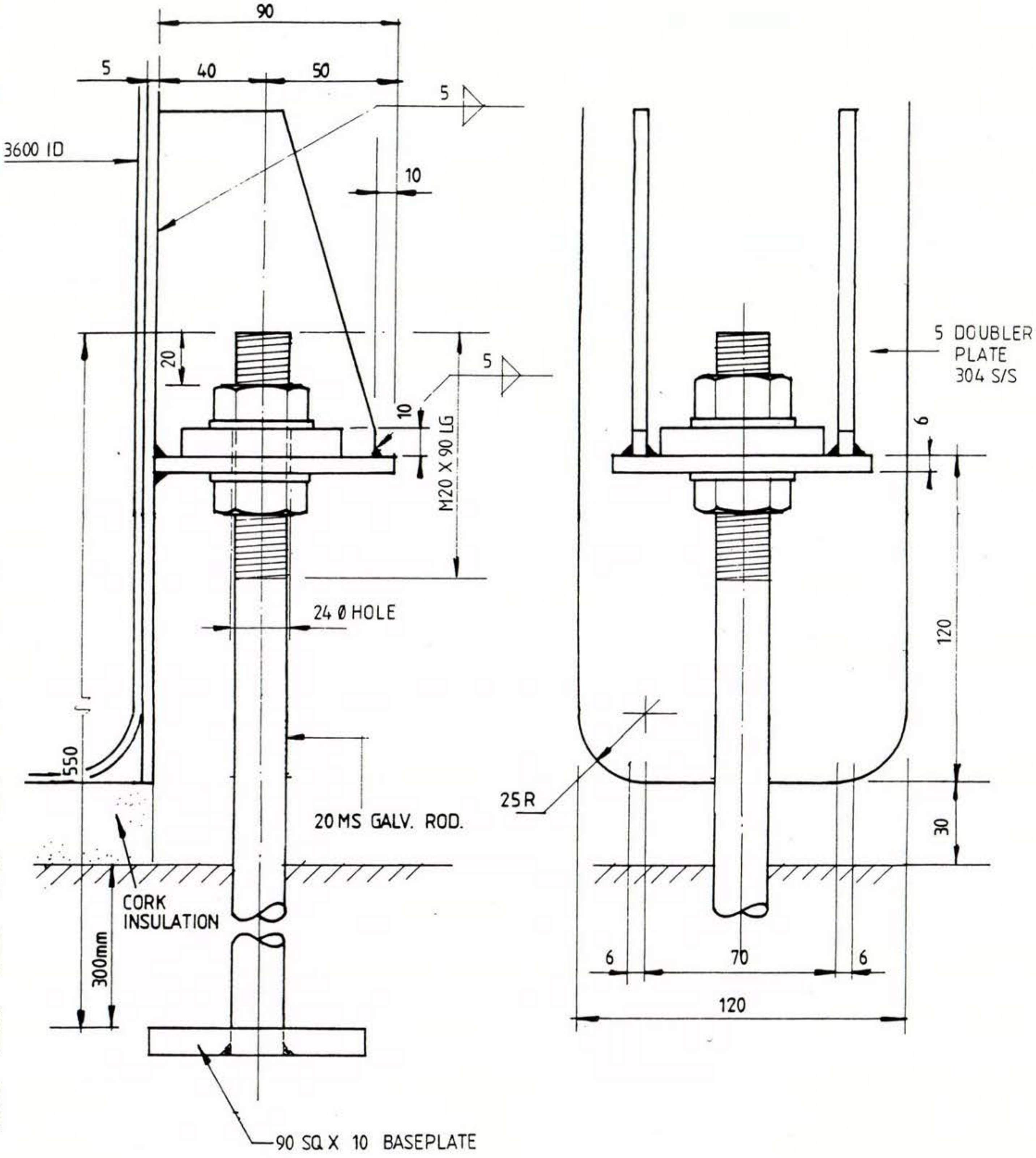


Fig. 3.1 Angle Bracket Detail
60m^3 Stainless Steel Silo
(16 Nº M20 Anchor Bolts)

All Components 304 s/s Unless Noted Otherwise

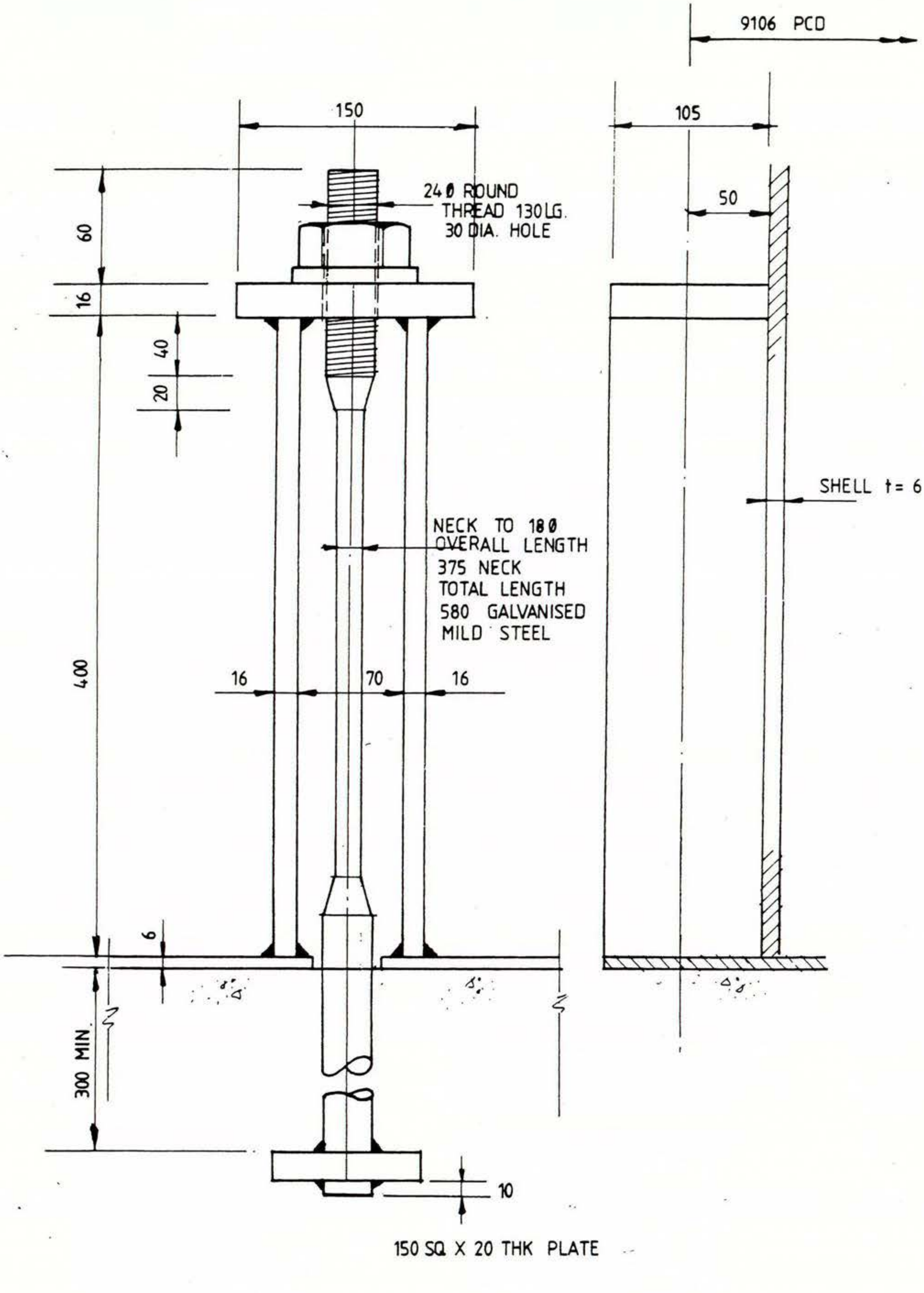


Fig. 3.2 Anchor Chair Detail (Isolated)
500m^3 Stainless Steel
Petrochemical Tank

All Components 316 s/s Unless Noted

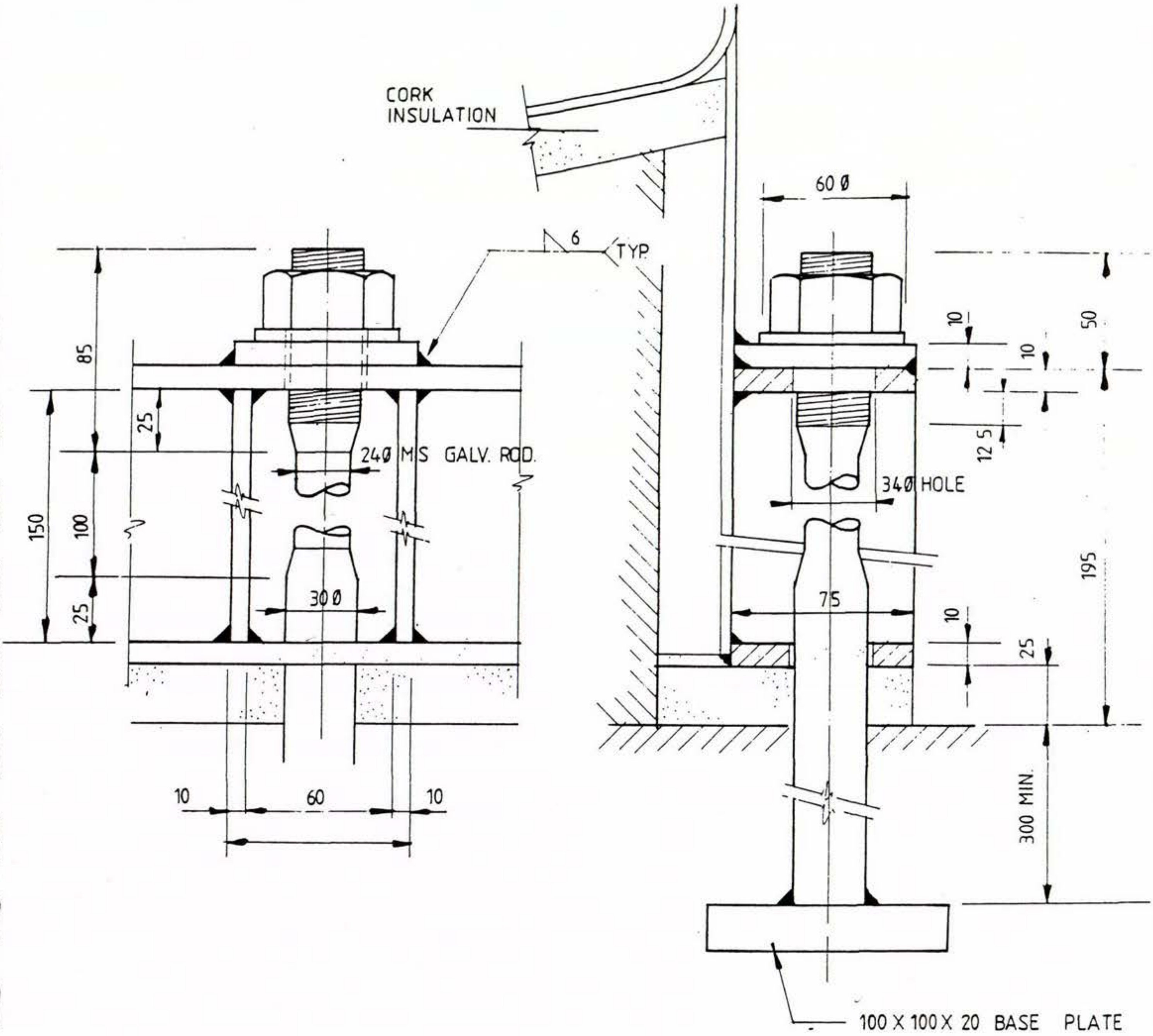


Fig 3.3 Anchor Chair With Continuous Top Ring
250m^3 Whey Silo

All Components 304 s/s Unless Otherwise Noted

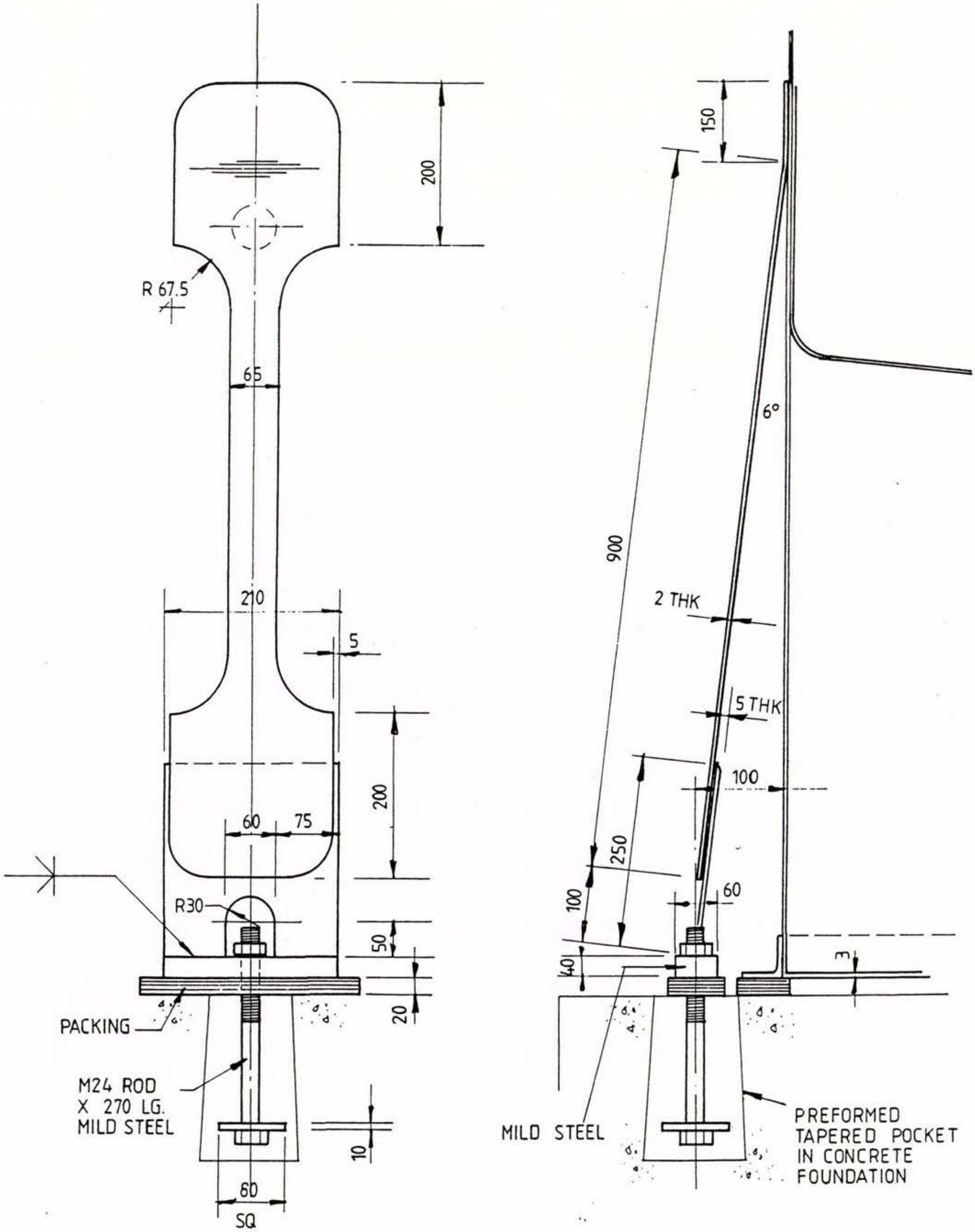


Fig. 3.4 Anchorage Strap Detail
200m³ Stainless Steel Vessel
(Protech NZ)

(All Components 304s/s Unless Noted Otherwise)

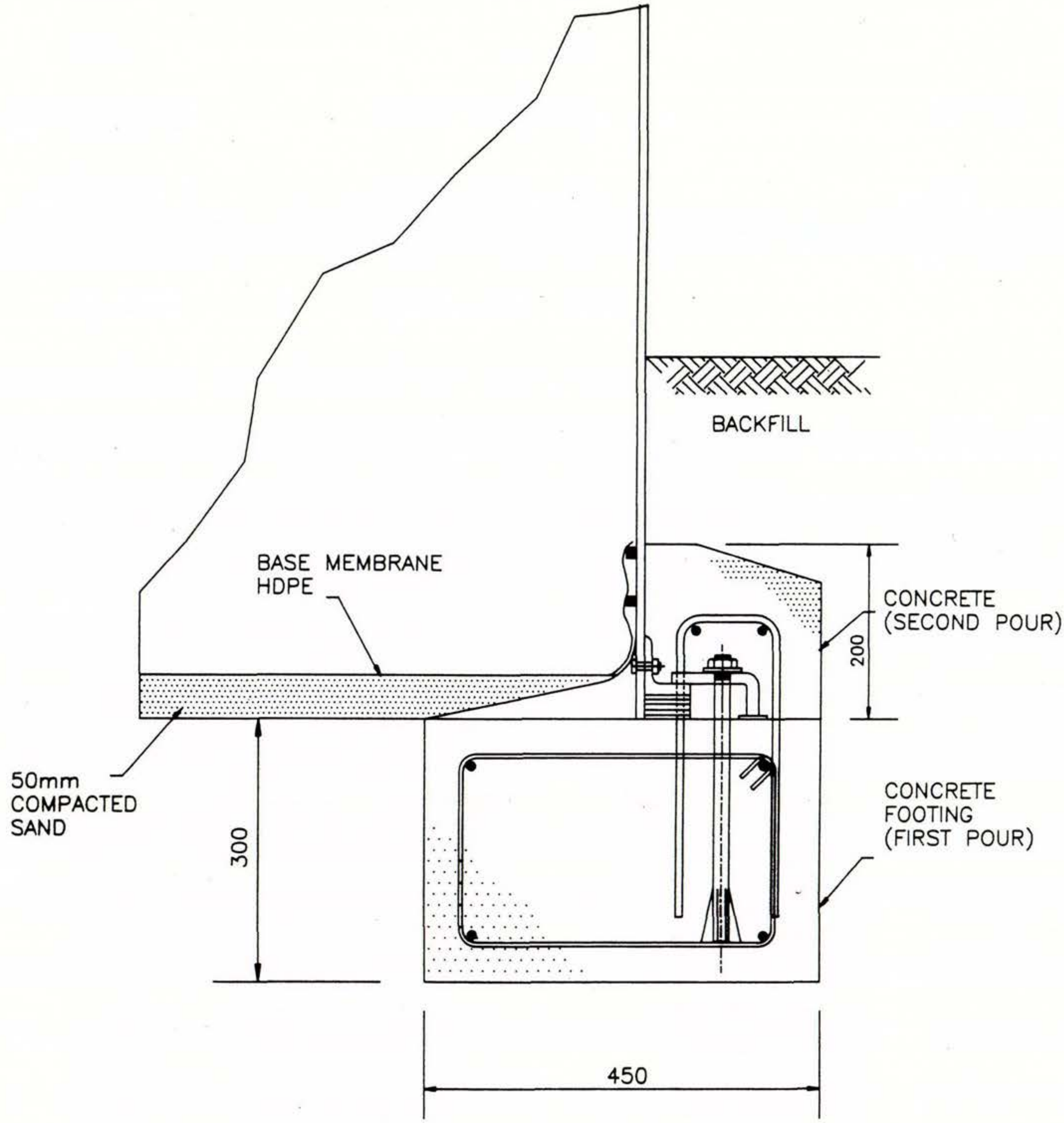


Fig. 3.5 Lugs or Cleats For Proprietary Glass Coated Bolted Steel Tank, (Permaglas Detail).

4. ANALYSIS RECOMMENDATIONS

The procedures required to analyse seismic holddown of tankage are represented in the following flow diagram Fig 4.1.

The analysis recommendations contained in this report pertain only to tanks where anchorage is required.

They do not apply to larger tankage (greater than 300 m^3) of low aspect ratio where an unanchored tank configuration has been shown to provide good seismic performance.

Typical unanchored configurations exhibit :

- $<$ 1:1 aspect ratio (dependent on seismic zone and specific gravity of contents);
- large diameter base (4 m or more);
- appropriately detailed bottom strake, base plate and foundation ring.

4.1 RECOMMENDATIONS

For Anchored Tanks the analysis recommendations are that the NZNSEE "Recommendations for the Seismic Design of Storage Tanks" be used for :

1. Assessment of Risk/Probability of Seismic Events (Table 1.1).

2. Calculation of Earthquake Forces and Overturning Moments. For tank support system ductility $\mu > 1$, the recommendations of Clause 2.12 and C2.12 apply. This involves calculation of period and damping for an equivalent elastic system using equations (C2.41) and (C2.42), to be used in assessing the response of the impulsive modes.

   Note: No reduction in convective forces results from the ductility of the tank support system.

3. Distribution of loads to Anchorages.

Fig C4.2 (b)	for elastic and nominally elastic designs
Fig C4.2 (c)	for limited ductile and fully ductile designs (requires ductile anchor bolts - plain or necked)

4. For design of other fixing components beyond the anchorage, use of overstrength factors as detailed in the following sections are specified for Grade 300 Reinforcing Plain Round Bar complying with NZS3402: 1989. For other materials, overstrength factors are derived from guidance provided in Ref [5].

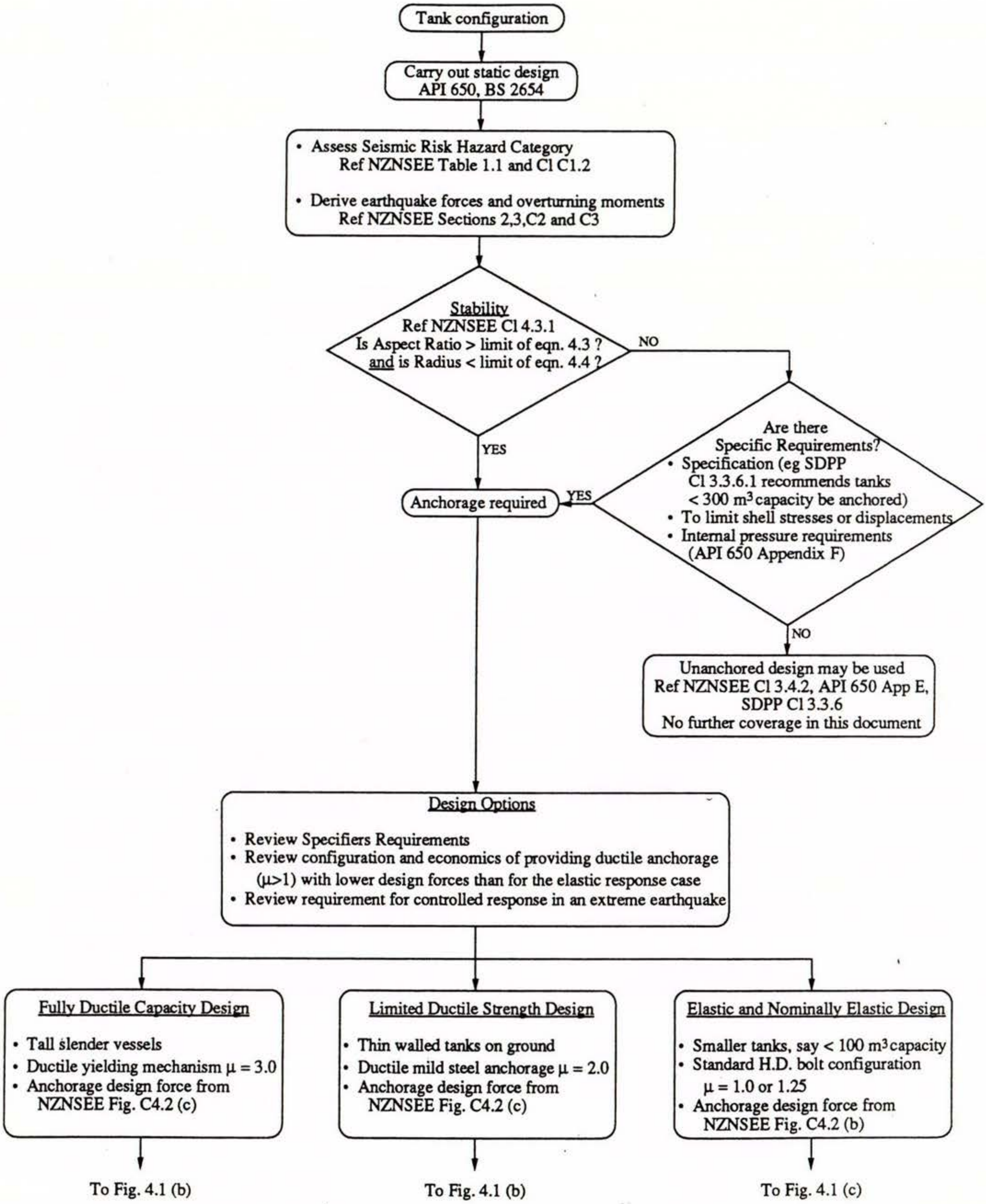


Fig. 4.1 (a)

Fig. 4.1 Seismic Analysis and Design Recommendations

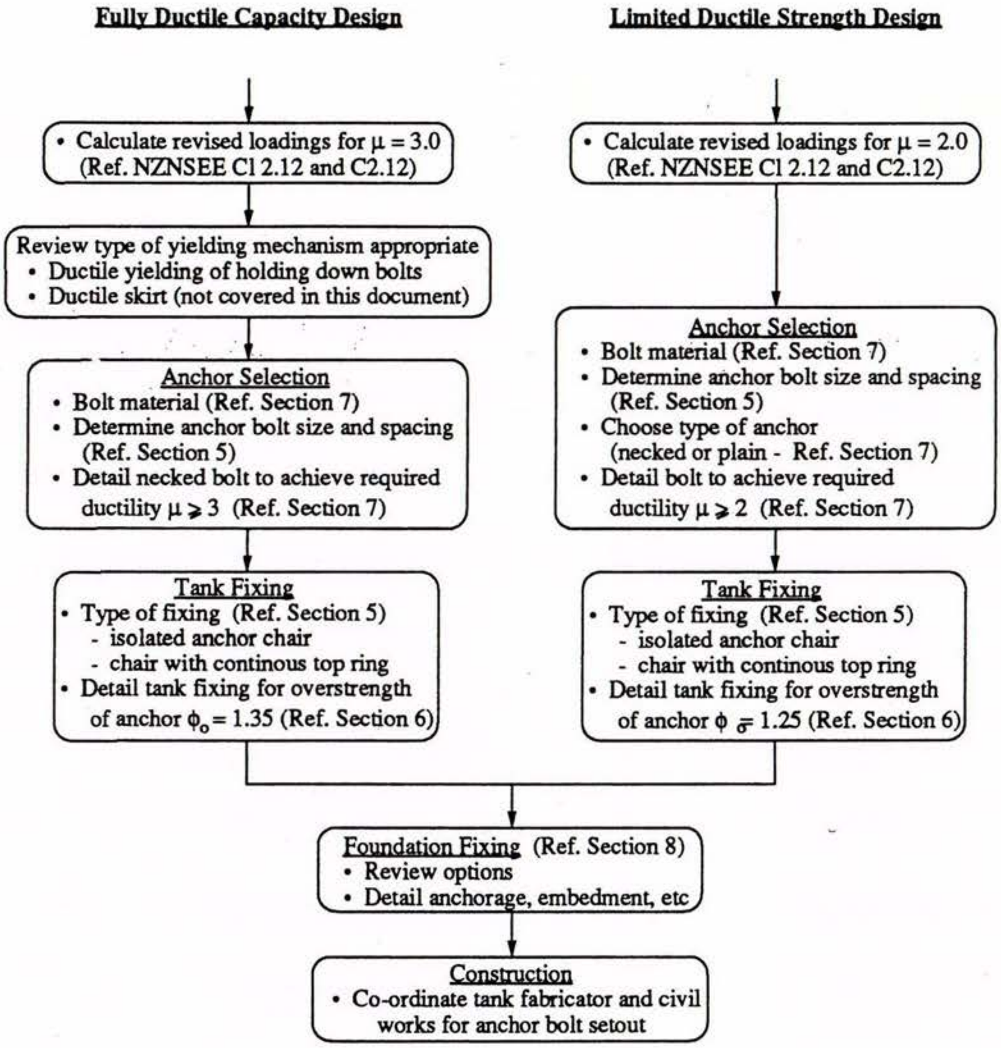


Fig. 4.1 (b)

Fig. 4.1 Seismic Analysis and Design Recommendations

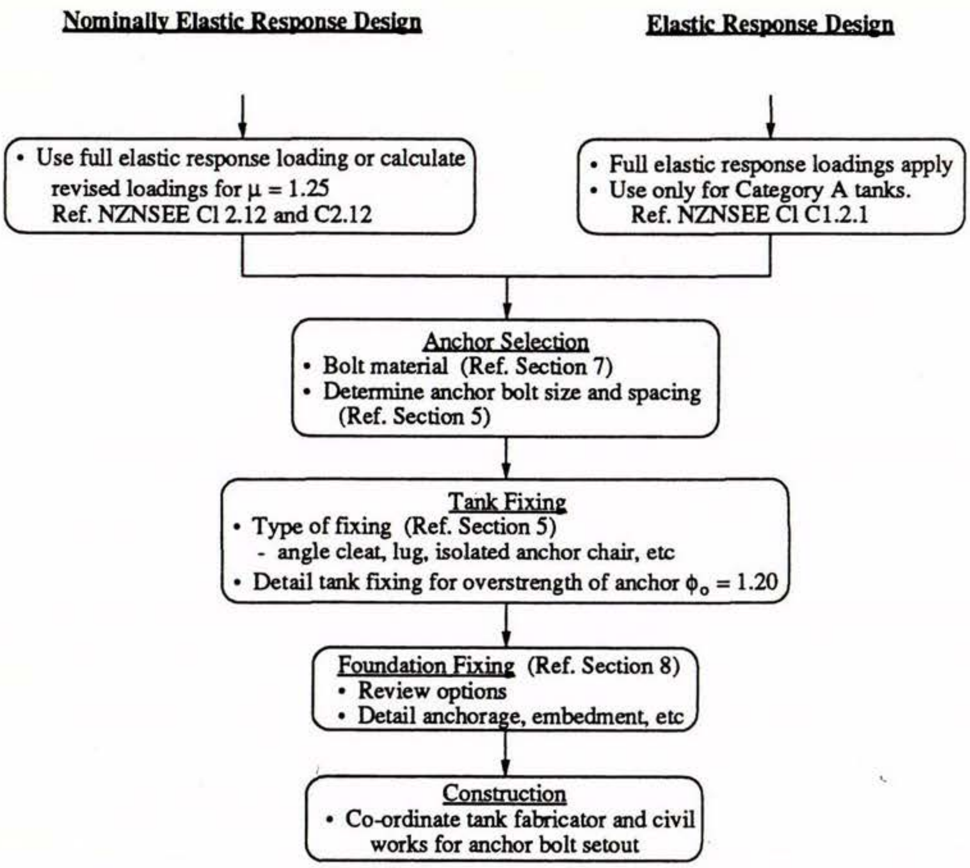


Fig. 4.1 (c)

Fig. 4.1 Seismic Analysis and Design Recommendations

For large vessels, where the number and/or size of bolts derived using $\mu = 1$ is uneconomic, then the use of SDPP design concepts with ductility of $\mu = 2$ or 3 is recommended.

For a ductility of $\mu = 3$ capacity design procedures are proposed using a maximum overstrength factor of $\emptyset_o = 1.35$ for all components beyond the yielding anchorage.

It should be noted that SDPP (Clause C2.2.2) states that the higher the assessed ductility capability for a structure, the lower the earthquake return period for the onset of damage.

The analysis recommendations are based on a rational hierachy of failure being present such that the tank wall and its contents are protected from damage or failure by detailing the holding down bolts for controlled yielding, or for seismic resistance at a higher level of earthquake than the code earthquake.

The designer has three options where a requirement for tank anchorage is determined. The preferred option will be a function of tank size, economics for anchorage, and the specifier's requirements.

The options are :

- Fully Ductile Capacity Design ($\mu = 3$) - intended for tall slender vessels, say taller than 5 metres and of aspect ratio greater than 5:1. (Refer Clause 3.3.1 of SDPP.)

- Limited Ductile Strength Design ($\mu = 2$) - intended for thin walled liquid storage tanks supported by the ground. (Refer Clause 3.3.6 of SDPP.)

- Elastic and Nominally Elastic Design ($\mu = 1$ and 1.25 respectively) will normally be the most economical option for smaller tankage, say less than 100 m^3 capacity and of aspect ratio less than 5:1.

4.2 FULLY DUCTILE CAPACITY DESIGN

The controlled yielding of anchorage systems as the seismic energy dissipation mechanism limits the load that can be transferred to other components.

This is achieved by the concepts of "ductile", "necked", or "yielding" holding down bolts. This has given rise to a number of interpretations. Application of capacity design philosophy has led to ductile yielding of holding down bolts occurring before the possibility of tank wall rupture, and fixing components being designed for an overstrength $\emptyset_o$ factor on the minimum yield load of the holding down bolt.

The SDPP code [Ref 2] makes recommendations for the design of necked holding down bolts for tall slender structures. The intention was that the formulae be applied to tall slender vessels of aspect ratio greater than 5:1 and 5 m high, and to plant such as pressure.vessels having wall thicknesses of the order of 50 mm (i.e. rigid bodies). These would respond to seismic loading in an essentially flexural mode. The principles of necking holding down bolts are **recommended** for fully ductile design of all types of tankage.

Seismic design coefficients based on a ductility factor of 3 for this yielding mechanism are recommended. This requires that sufficient necked length of the yielding bolt be provided. The basis of formulae for the necked length in Ref [2] are as follows:-

(a) steel strain not to exceed 4%
(b) an overstrength factor of ϕ_o = 1.35 (for Grade 300 reinforcing bar material to NZS3402: 1989)
(c) the tension only yielding mechanism is a progressive accumulation of total strain with each load cycle with an ability to dissipate energy through both the yielding of the necked bolt, and the slight uplift of the tank during the tension cycle.

The derivation of seismic coeffients and forces should be to the NZNSEE Recommendations as these represent the current seismic response Spectra and can be related to the proposed NZS 4203 revision.

The support structure ductility of μ = 3 is allowed for by defining an equivalent elastic system from equations (C2.41) and (C2.42) Ref [1] as:-

Impulsive Period T_e/T = 1.23

and Damping ζ_e = ζ + 7.6%

The anchorage force required per metre is derived from Fig C4.2(c) Ref [1] as the equation:-

$$P_{max} = \frac{8Mot}{3\pi D_2^2} - w_t \ (kN/m) \qquad \ldots\ldots\ldots\ldots\ (4.2)$$

4.3 LIMITED DUCTILE STRENGTH DESIGN

The majority of tanks have an aspect ratio of less than 5:1 height/diameter and are considered flexible. Their response to ground motions during an earthquake will be in a combination of shear and flexural modes.

Larger tanks of capacity greater than 100 m^3 generate seismic overturning forces sufficiently large such that anchors designed elastically will often be considered uneconomic.

This is seen in large bolt sizes, close anchor spacing and in the additional thicknesses and detailing required of fixing components.

The use of the ductility of holding down bolt materials to limit seismic forces entering the tank structure, through controlled yielding of the holding down bolts, is therefore an economic proposition.

The holding down bolt can be either a fully threaded bar or plain mild steel round. Both can be necked over a defined length.

In the plain round bolt which is threaded at the top plate of the anchor chair, the yielding usually occurs at the thread. This mechanism is less ductile than a necked shank to a holding down bolt, but can exhibit a limited ductility if properly detailed and installed.

Both the plain round and the fully threaded rod are economic alternatives to the necked shank but of lesser ductility (and hence have an increased risk of failure in the extreme seismic case). A decision on their use should be put into the context of the overall seismic risk of the tank, the effects of loss of contents in an extreme event and the hazard to the community. The petrochemical industry, for example, specifies a necked bolt as standard for such tankage.

The necked bolt has a more predictable ductile yield than a normal bolt. Yielding at first occurs at the bottom most thread and transfers into the necked shank of the bolt. However, it requires careful detailing in the thread to the necked transition, and an adequate length for the necked section, to ensure effective dissipation of seismic energy over a sufficient number of cycles.

The support structure ductility of $\mu = 2$ is allowed for by defining an equivalent elastic system from equations (C2.41) and (C2.42) Ref [1] as:-

Impulsive Period $T_e/T = 1.12$

Damping $\zeta_e = \zeta + 5.9\%$

The anchorage force required per metre is derived from Fig C4.2(c) Ref [1] as:

$$P_{max} = \frac{8Mot}{3\pi D_2^2} - w_t \ (kN/m) \qquad \ldots\ldots\ldots\ldots\ (4.3)$$

An overstrength factor of $\phi_o = 1.25$, for Grade 300 reinforcing material to NZS3402: 1989, to accomodate material variation and a strain hardening factor, is applied to the anchorage yield force to derive loads in tank shell and fixings for both plain and necked types of anchorage. Other materials could require differing overstrength factors. Designers are referred to Ref [5] for guidance.

Recommendation

This report recommends the use of suitably detailed necked holding down bolts on the basis of :

- the more predictable seismic energy dissipation mechanism and lower risk.
- the relatively small cost premium in terms of overall tank cost.
- the ability of tanks involved to survive an extreme seismic event with relatively little damage.

It is recognised that plain round or fully threaded holding down bolts will be appropriate for some industrial applications but the decision should be made by the industry using the basis of performance assessment noted above.

4.4 ELASTIC AND NOMINALLY ELASTIC STRENGTH DESIGN

For smaller tanks of less than 100 m^3 capacity, the minimum practical circumferential spacing and size of holding down bolts will often provide an inherent seismic resistance corresponding to the code earthquake or larger, with the bolts remaining in the elastic range. In such cases, there is no need for necking of holding down bolts.

The SDPP Recommendations in Clause C3.3.1 state :

"For tubular vessels less than 5 m high, the same design principles can be applied though it may be found that a higher seismic lateral loading, even to the extent of designing for fully elastic action (μ = 1), can be easily met in which case the requirements for ductile detailing can be accordingly reduced".

In the case of nominally elastic designs (μ = 1.25), anchor bolts shall be capable of reaching first yield, without loss of load-carrying capacity. That is, anchor bolts must yield before anchor pull-out or concrete cone failure occurs. Fully elastic design and detailing of minor installations (Category A tanks as defined in Clause C1.2.1 Ref [1]) may allow these failure mechanisms, refer Section 8.0.

However, a hierachy of failure for all tank fixing componentry (chairs and tank walls) using a limited overcapacity is **recommended**. This should be based on an overstrength factor of 1.20 x the specified yield load in the bolt, to allow for material variation in the holding down bolt.

Thus, for the majority of smaller tanks of less than 100 m^3, design could be based on elastic response as per NZNSEE "Recommendations for the Seismic Design of Storage Tanks". In the case of a nominally elastic design, the support structure ductility of μ = 1.25 may be allowed for by defining an equivalent elastic system from equations (C2.41) and (C2.42) Ref [1] as:-

Impulsive Period T_e/T = 1.03

Damping ζ_e = ζ + 3.5%

The anchorage force required per metre is derived from Fig C4.2 (b) Ref [1] as:

$$P_{max} = \frac{4Mot}{\pi D_2^2} - w_t \ (kN/m) \qquad \ldots\ldots\ldots\ldots\ldots \ (4.2)$$

Recommendation:

The principles of nominally elastic strength design requiring anchorage yield before failure should generally apply to small tanks, whether designing for μ = 1 or μ = 1.25 actions. However, in the case of Category A tanks as defined in Clause C1.2.1 Ref [1] designed for μ = 1 actions, a lower standard of base anchorage may be acceptable.

5. ANCHOR BOLT AND TANK FIXINGS

5.1 ANCHOR BOLT DESIGN

The selection of an anchor bolt is dependent on a number of parameters, including the design load, the connection details to the tank and the connection to the foundation.

Derivation of the design load is discussed in Section 4. The strength method of design should be used throughout.

The spacing of anchorages around the tank perimeter can be adjusted after the bolt size has been selected, in order to optimise the design. Generally anchor spacings of 1.0 m to 2.5 m are appropriate for medium to larger size tanks. Increasing the anchor spacings will result in greater local stress concentrations at the anchor chair locations. A minimum of 6 anchorages around the perimeter of a tank is recommended, with the maximum spacing being 2.5 metres.

5.1.1 Tension Only

If the shear load on the tank is transferred through the base plate and the holding down bolts are in tension only, then the required bolt size is determined from :

$$|P_u| < P_{us} = \phi_s A_s f_y \quad \text{.................} \quad (5.1)$$

Where
- P_u = design axial tension load (negative for tension)
- P_{us} = dependable strength in tension
- ϕ_s = strength reduction factor for steel (1.0 recommended for ductile anchors, 0.9 recommended for non-ductile anchors)
- A_s = effective tensile stress area of anchor (Refer Appendix 1)
- f_y = maximum specified yield strength of anchor material which should be not greater than:-
  0.70 f_{ut} for ductile anchors
  0.85 f_{ut} for limited ductile anchors
- f_{ut} = specified minimum ultimate tensile strength of anchor (MPa)

5.1.2 Shear Only

Shear loads may be resisted by bearing plate shear friction or anchor shear.

When shear friction is mobilised, dependable shear strength is given by :

$$Q_{uf} = \phi_f \mu (A_s f_s + P_u) \quad \text{.............} \quad (5.2)$$

Where ϕ_f = strength reduction factor = 0.85

μ = coeffient of friction between steel plate and concrete surface

= 0.9 for steel plate fully embedded in concrete

= 0.7 for steel plate bearing directly on concrete surface

= 0.55 for steel plate on grouted plinth

Note that the steel plate must be clean and free of paint. The coefficient of friction μ can be as low as 0.35 if the steel plate surface is contaminated with mill-scale.

When the anchor is loaded in shear, dependable shear strength is given by

$$Q_{us} = \phi_s A_v (0.577 f_y) \quad \ldots\ldots\ldots\ldots\ldots \quad (5.3)$$

where A_v = effective cross-sectional area of anchor for shear (taken as gross area of shank when threads are excluded from the shear plane, and as core area when threads are included).

ϕ_s = 0.9

5.1.3 Combined Tension and Shear

The following interaction formulae are recommended :

$$|P_u/P_{us}| \leq 1$$

$$|Q_u/Q_{us}| \leq 1$$

$$|P_u/P_{us}| + |Q_u/Q_{us}| \leq 1.2 \quad \ldots\ldots\ldots \quad (5.4)$$

5.1.4 Overstrength/Anchorage Fixings

To ensure that anchorage failure is governed by tensile failure of the anchor bolt, anchorage to the foundation and the tank should be designed for the bolt overstrength P_o given by:-

$$P_o = \phi_o A_s f_y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5.5)$$

Where ϕ_o = overstrength factor

The overstrength factor varies dependent on the material used and the choice of ductile, limited ductile or nominally elastic design procedure. This report uses overstrength factors related to Grade 300 plain reinforcing bar complying with

NZS3402: 1989. For guidance on overstrength factors applicable to other materials the designer is referred to Reference [5].

5.1.5 Anchor Bolt Detail

The details relating to thread, corrosion protection and bond length are discussed in Section 7.

(i) Thread

The Effective Tensile Area of the Threaded Anchor is derived from the following formula for ISO metric screw threads (Ref BS 3643).

$$A_s = \frac{\pi}{4} (d - 0.9382p)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5.6)$$

Where p is the pitch of the coarse thread. This varies depending on the nominal diameter, as shown in Appendix 1.

Different relationships apply for threads other than ISO metric.

(ii) Ductile Length

To allow adequate bolt elongation, the minimum section area must continue for a reasonable length.

NZS 3404:1989, Ref [7] recommends that the reduced area necked portion should be not more than 0.80 of the full crossection to ensure yielding occurs in that portion of the bolt.

The machining of necked bolts should be carried out so as to produce a clean scorefree surface to the necked portion.

The minimum length of necked down bolt, should exceed 2-3 times bar diameter or 100 mm whichever is greater.

A method of calculating the required yielding length of necked down bolt is presented in Ref [2]. Clause C3.3.1.

(iii) Anchor Head Detail

Where a stud head end plate (either with nut or welded) is provided it should be designed for the bolt overstrength, P_o. The designer should check for local concrete crushing and for yielding of the plate/stud head. Recommended anchor head details are shown in Fig 5.1.1.

Research described in Ref [8] has shown that for anchors with anchor plates the following geometric relationships will preclude anchor head or concrete crushing failure.

(a) bearing area of the anchor head (excluding the area of the bolt) is at least 1.5 times the area of the bolt evenly distributed around the bolt.

(b) the thickness of the anchor head is at least equal to the dimension from the outer most edge to the face of the bolt.

For bearing plates, as per Fig 5.1.1, bending stress on the effective section should be checked if applicable.

An instance of bolt pullout following anchor head detail failure through bending down during the Edgecumbe earthquake is discussed in Ref [9].

5.2 ISOLATED ANCHOR BOLT CHAIRS

Anchor bolt chairs provide a means of distributing anchorage loads from the anchor bolts to the tank shell. Eccentricity of the bolt causes an outward radial force in the top plate of the chair and local bending stresses in this region. Maximum stress occurs in the vertical direction just above the anchor plate/tank shell joint and is a combination of bending and direct stresses, the magnitude of these stresses being a function of the bolt force and the chair and tank geometry.

For a given bolt force, minimising eccentricity while increasing chair height and width will reduce tank shell stresses.

Isolated anchor bolt chairs are used when holding down bolts are widely spaced, say greater than 500mm. For closely spaced holding down bolts a continuous top ring should be used, as described in Section 5.3.

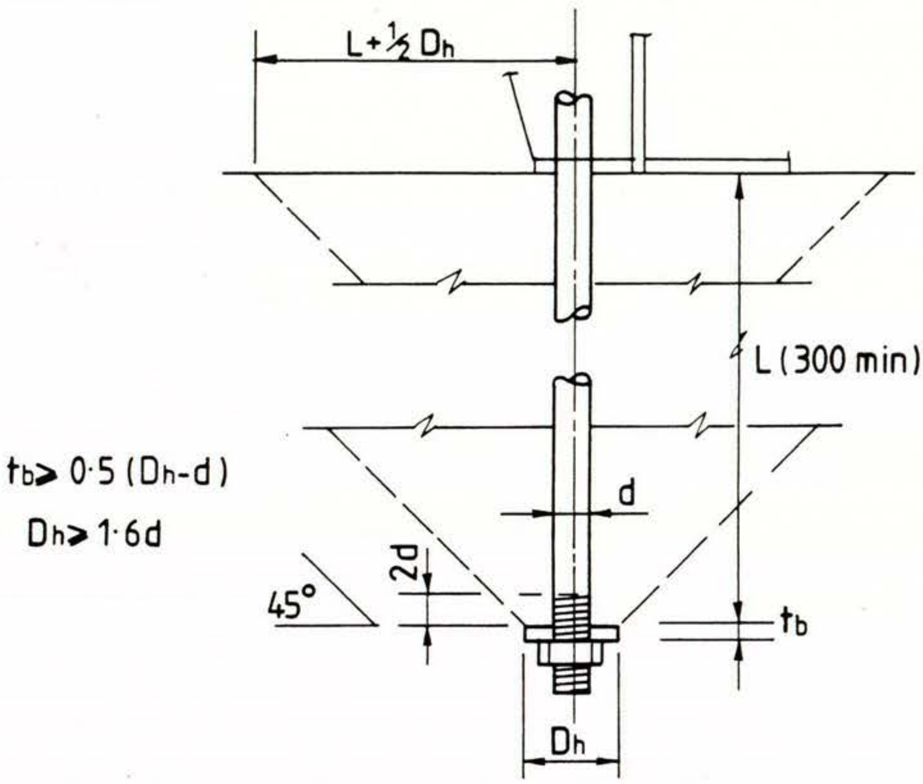
The standard chair details presented in Fig 6.1 have been developed using formulae presented in Ref [10] with some modifications to incorporate capacity design philosophy and in conjunction with other references such as BS5500. These may be refined by more detail finite element analysis in conjunction with testing. It is recommended that the details be tested to optimise these further.

5.3 CHAIR WITH CONTINUOUS TOP RING

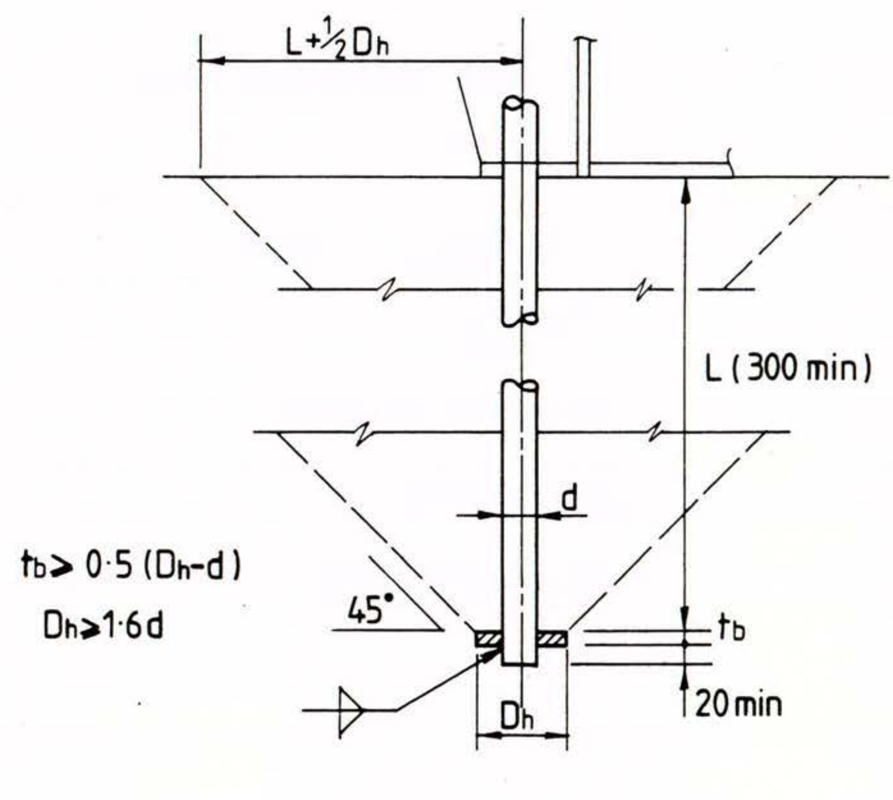
A continuous top ring is provided to resist the moment at each holding down bolt due to the eccentricity of the bolt and shell. This detail is superior to isolated chairs since local shell bending stresses are much smaller. In this case local shell bending stresses are a function of the radial stiffness of the top ring relative to the shell stiffness. A continuous top ring is used when anchor bolts are closely spaced.

The top ring, and baseplate or bottom ring, should be designed for the radial loads as shown in Fig 5.3.1. The tank shell transfers the net effect of these radial loads by shear to the baseplate or bottom ring. Ref [10] recommends that a portion of shell each side of the top plate and within 16 t of the ring may be included for section property calculations.

If a heavy bearing plate is not provided across vertical stiffeners at bolt positions, the top ring should be checked for the combination of radial stress and local bending stress.



(a) Plate & Nut



(b) Welded Plate

Fig. 5·1·1 Anchor Head Details

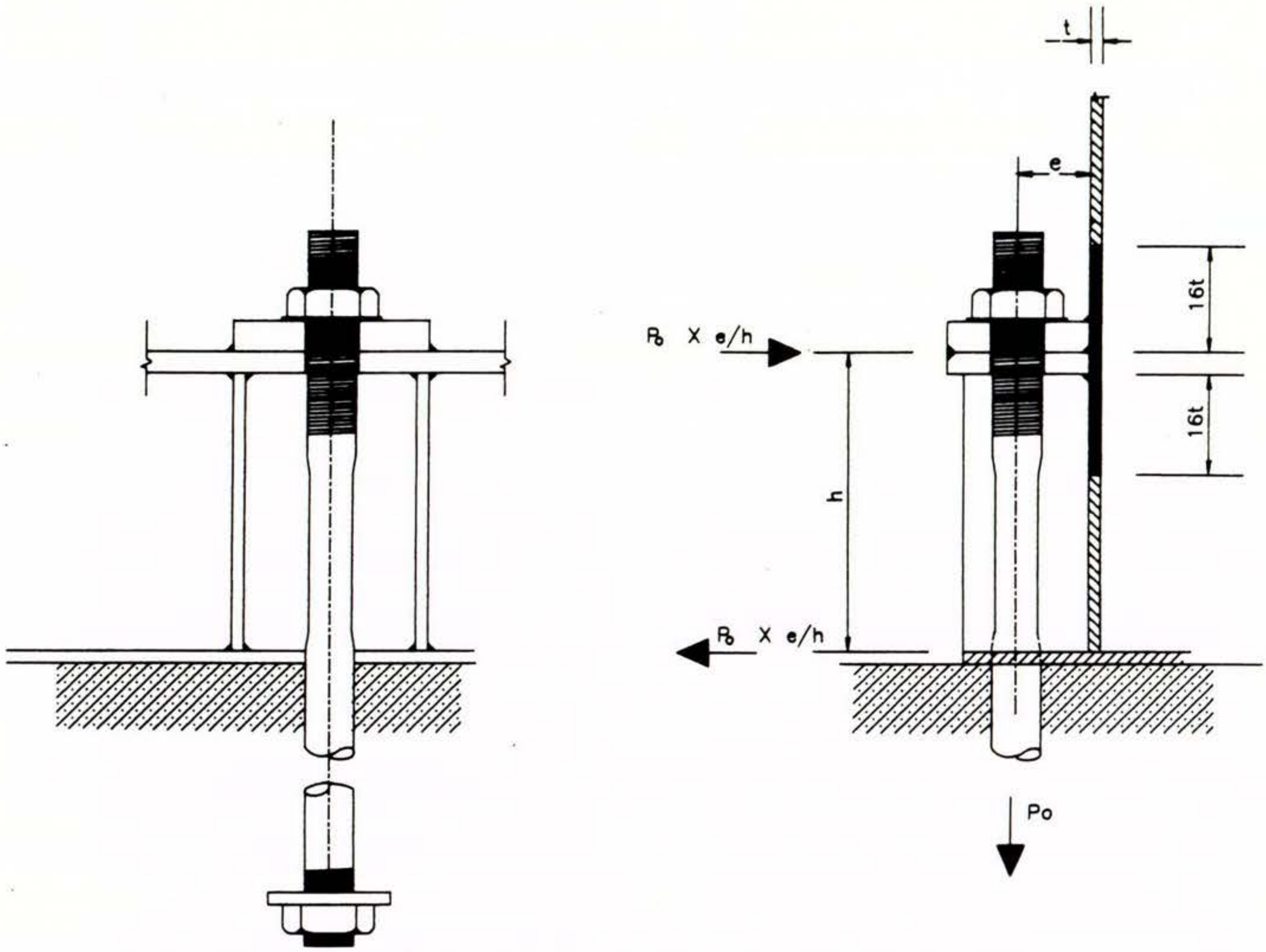


Fig. 5.3.1 – Chair With Continuous Ring At Top

6. STANDARD ANCHOR CHAIR DETAILS

The Tables in Figs 6.1, 6.2 are based on formulae in [Ref 10] and charts in other codes such as BS5500.

Standard anchor chair details for a range of bolt sizes are presented in Figs 6.1 and 6.2, to be read in conjunction with Table 6.1.

The following assumptions have been made :

(a) The bolt is of notch ductile steel from reinforcing plain round bar manufactured to NZS 3402: 1989 with nominal yield stress of 300 MPa.

(b) An overstrength factor of ϕ_o = 1.35 is used for fully ductile structures.

(c) The bolt is necked and the force P_o is derived from the yielding of the shank.

(d) ·Anchor bolt spacing as follows:-

- greater than 500 mm for isolated anchor chairs;
- greater than $(0.01R)^2$ mm for chairs with continuous top ring.

(e) A maximum shell stress of 250 MPa has been adopted. For thin walled tanks, shell doubler plates may be required in the vicinity of an isolated anchor chair. Otherwise, chair dimensions may be increased and the method and formulae of Ref [10] used to check the shell stresses for the altered configuration. Another option is to replace the top plates with a continuous ring (Refer Section 5.3).

(f) The designer requires to carry out a separate analysis to ensure seismic shear is transmitted to the base via either friction on the tank bottom plate or other satisfactory mechanism.

The plate thicknesses, dimensions etc presented in Figs 6.1 and 6.2 may be slightly conservative. If plain holding down bolts are used with the standard chair details, Figs 6.1, 6.2 are entered with the design force P_o without reference to the bolt size. Economy may be achieved by recalculation using the formulae of Ref [10].

The vertical stiffeners should be designed to ensure that adequate resistance to buckling is provided.

Ref [10] gives vertical stiffener geometry limitations of

j	≥ 12 mm
j	≥ .04 (h-c)
jk	≥ 5.7 P_o (j, k, in mm, P_o in kN)

These limits assure a maximum slenderness ratio of 86.6 and a maximum average axial stress of 86 MPa. It is considered that the first condition of $j \geq 12$ mm need not apply to smaller installations, where holding down bolt size is not greater than, say, 20 mm diameter. Analysis and detail design of welds of the anchor chair plate should be carried out using accepted design principles with reference to Ref [10].

STANDARD ANCHOR CHAIR DETAILS

Notation	Description	Basis
a	Top plate width.	
b	Top plate length.	Minimum e + 1.75d AS1250 Table 9.6.2.
c	Top plate thickness.	Refer Ref [10] formulae. Allowable stress 250 MPa, P_o. Top plate "beam width" of f.
e	Anchor bolt eccentricity.	Dimensional limitations.
f	Distance from outside of the top plate to the edge of the holes.	
g	Distance between vertical plates.	Dimensional/welding limitations.
h	Chair height.	- Practical heights. - Tank wall thickness/ stress level within acceptable limits.
j	Vertical plate thickness.	Minimum 12 mm for Bolts > 20 mm dia.
k	Average plate width for Taper plates.	

Notes:

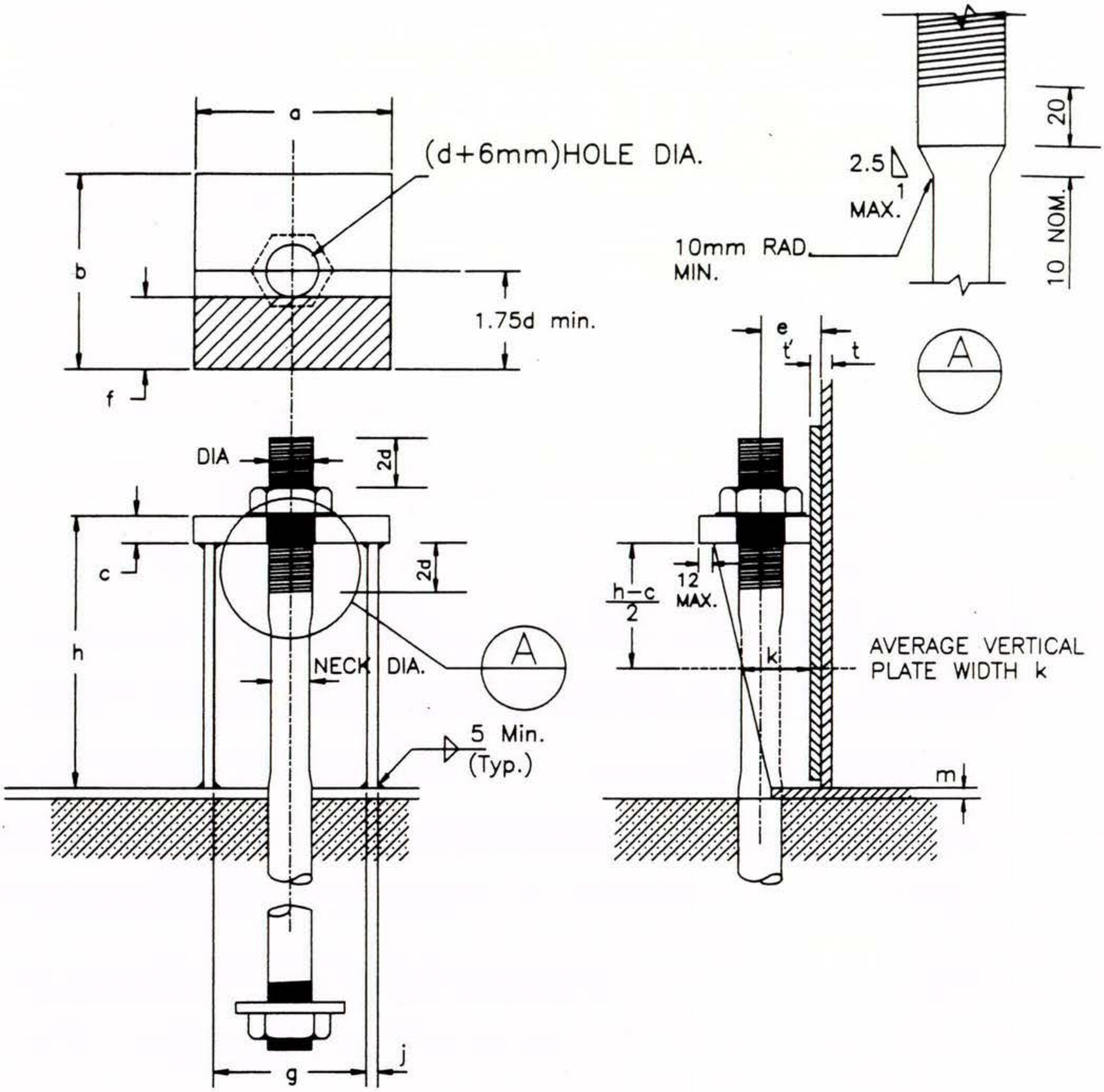
1. Material yield stress 300 MPa based on Reinforcing Round Bar to NZS 3402:1989

2. Design load P_o is derived from the following formulae for ductile design
   $P_o = A_s \times 300 \text{ MPa} \times 1.35\ \emptyset_o$
   A_s = Stress Area - the yielding portion of the necked bolt.
   Refer Appendix 1 for Derivation of Bolt forces

3. The details are valid for tanks of Radii as stated on Fig 6.1, 6.2.

4. A maximum shell stress in bending of 250 MPa has been adopted.

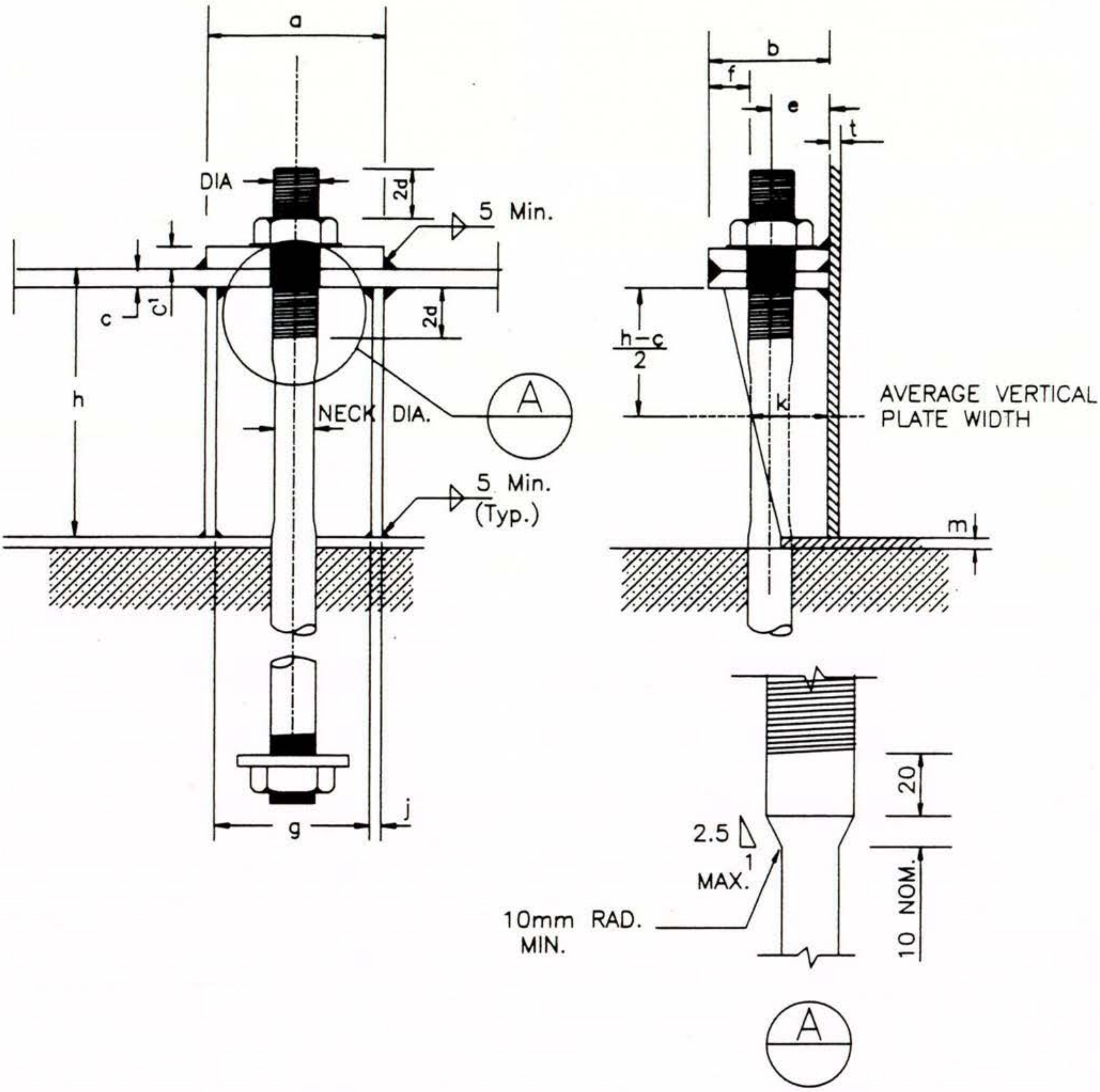
Table 6.1: Standard Anchor Chair Details - Notation and Design Basis (read in conjunction with Figs 6.1 and 6.2)



SIZE (mm)	NECK DIA. (mm)	$A_s f_y$ (kN)	DESIGN LOAD P_o (kN)	e (mm)	a (mm)	h (mm)	b (mm)	HOLE DIA. (mm)	f	c	k	g	j	$t+t'$	$\frac{R}{(t+t')^4}$
M16	12.5	36.8	49.70	35	100	200	76	22	30	12	48	60	10	≥6	≤1.1
M20	16	60.3	81.40	40	120	300	85	26	32	16	53	70	12	≥6	≤0.9
M24	19	85.0	114.70	45	140	300	95	30	35	20	63	80	16	≥8	≤0.4
M30	24	135.6	183.10	50	150	300	110	36	42	25	80	90	16	≥10	≤0.1
M36	29	198.3	267.70	60	170	350	125	42	44	30	106	110	16	≥12	≤0.04

Fig. 6.1 Standard Detail
Isolated Anchor Chair

Note: Table applies for bolt pitch ≥ 500mm



SIZE (mm)	NECK DIA. (mm)	$A_s f_y$ (kN)	DESIGN LOAD P_o (kN)	e (mm)	a (mm)	h (mm)	b (mm)	HOLE DIA. (mm)	f	c^1	c	k	g	j	t
M16	12.5	36.8	49.70	35	100	200	76	22	30	12	10	48	60	10	>4
M20	16	60.3	81.40	40	120	300	85	26	32	12	10	53	70	12	>4
M24	19	85.0	114.7	45	140	300	95	30	35	16	12	63	80	16	>5
M30	24	135.6	183.1	50	150	300	110	36	42	20	16	80	90	16	≥8
M36	29	198.3	267.7	60	170	350	125	42	44	25	20	106	110	16	>10

Fig. 6.2 Anchor Chairs With Continuous Ring

Note: Table applies for bolt pitch $\geq (0.01R)^2$ (R in mm)

Radius R $\leq$ 2400mm

7. MATERIAL SPECIFICATION AND DETAILING NOTES

A number of points of detailing and specification need particular emphasis as essential components to the seismic performance of anchorages. Attention to detailing aspects, and control of materials and workmanship were some of the conclusions from the review of the seismic performance of tankage hold down systems at Edgecumbe cited in Ref [9].

The failures of holding down bolt detailing observed included :

(a) Tensile failure of the bolt initiated at a point of severe localised corrosion.

(b) Tensile failure of the bolt at the beginning of the threaded section (normal cut thread). The report noted that this only occurred in stainless steel holding down bolts, whereas in mild steel holding down bolts considerable ductility was observed, even with normal cut threads.

(c) The nut pulling off due to thread stripping. This was only observed in stainless steel holding down bolts.

(d) Insufficient length of bolt as installed to allow at least 3 complete threads to protrude beyond the tightened nut. In one installation, no bolt was observed to protrude beyond the tightened nut at all and several nuts had only half their threads engaged. This example appeared, however, to have been designed to resist seismic-induced loading and had performed very well.

(e) Failure by cleat tearing due to insufficient edge distance to the bolt.

(f) Excessive cleat distortion due to an apparent lack of design for uplift (seismic-induced tensile) loading.

7.1 DUCTILE MATERIALS

Recommendation
Holding down bolts should be manufactured from notch ductile steel mild steel. Steel of grades that satisfy NZS3404: 1989 Table 12.4.2, Category 1 are acceptable. These require:-

- yield strength less than 360 MPa
- max. ratio of yield to ultimate stress 0.70
- min. length of yield plateau 10 ϵ_y .

Satisfactory materials complying with this specification are listed in Table 7.1.

Steel available in New Zealand ex-stock in the form of round bar is most likely derived from plain reinforcing round ex Pacific Steel with 300MPa yield complying with NZS3402: 1989 material specification.

Steel complying with AS3679 (Grade 250)(formerly AS1204), BS4360/43A, (min. yield stress 275 MPa) may also be used, however this steel is usually only available ex-stock in sizes greater than 36 mm diameter.

Designers should note that overstrength factors could be different dependent on the steel yield for steel grades other than NZS3402/Gr 300. For guidance in the choice of overstrength factors, the designer is referred to Ref [5], Vol 2, in particular Amendment No 3, September 1990 which revises aspects of NZS 3404.

MATERIALS FOR HOLDING DOWN BOLTS		
Design Method	**Design Ductility**	**Materials**
Fully Ductile and Limited Ductile	3.0 - 2.0	• NZS3402/Grade 300 • AS3679/Grade 250 • BS4360/Grade 43A
Nominally Elastic and Elastic	1.25 - 1.0	• NZS3402/Grade 300 • AS3679/Grade 250 • BS4360/Grade 43A • 4.6/S, 8.8/S Bolts • GR 304/316 Stainless Steel • Refer Note.

Note:

1. Proprietary fixings and other details not specifically detailed for anchor yield may be used in minor installations designed by the elastic method. Refer Section 8.2.3.

2. AS3679: 1990 supercedes AS1204.

Table 7.1: Acceptable Materials for Holding Down Bolts

Unacceptable Ductile Materials

1. Only steel with a long yield plateau (e.g. mild steel to AS 3679) will allow stretching of the bolts and formation of a ductile mechanism.

   It should be noted that grade 4.6 (400 MPa UTS) and 8.8 (800 MPa UTS) bolts do not meet this requirement and that actual ultimate tensile strengths of up to 850 MPa and 1250 MPa respectively, could occur. There is also no defined yield plateau.

The use of 4.6/S and 8.8/S bolts are **not recommended** for ductile or limited ductile holding down bolt anchorages. Bolts are currently manufactured in New Zealand from imported materials to AISI Specifications 1025,1010 (4.6/S) and 1315, 5140, 1038 (8.8/S).

2. The manufacture of holding down bolts from "bright steel" often used for its ease of machining is **not recommended** due to its non-ductile material properties. Elongation before fracture is poor being only some 11% [Ref 11].

3. Stainless Steel bolts to Grade 304 or Grade 316 do not exhibit a well defined yield plateau and should also not be used for ductile or limited ductile holding down bolt anchorages. The stress strain curve for 304 grade stainless steel is shown in Appendix 2.

Note:

While the bolts described above are unsuitable for ductile holding down bolt anchorages, they are able to be used in elastic and nominally elastically designed anchorages. Acceptable materials for the different design methods are summarised in Table 7.1.

7.2 BOLT THREAD

For mild steel bolts standard cut coarse ISO metric threads should be used. Bolts should be machined to free fit tolerances.

Testing (Ref. [12]) has demonstrated that fine threads may be more prone to stripping than those of standard coarse thread specification and in that instance the thread failure is of a sudden stripping. The strain at failure for a cut thread is at approximately 7%. In a rolled thread this strain can be up to 25%. However economies of fabrication usually result in the use of coarse cut threads.

At least 3 complete turns of the thread must protrude beyond the tightened nut. A length of 2 x diameter of projected thread is **recommended.**

The transition into the necked portion is an important detail discussed in Section 7.3.

Testing of holding down bolts Ref [12] has shown that threaded bar of notch ductile steel exhibits satisfactory ductile yielding under monotonic tension loading. For a plain threaded bar the failure mechanism is by the thread area initially yielding, and then strain hardening with stress increasing some 10-15%. The shank of the bar then yields and begins to strain harden. The thread then fails in a ductile mode.

Brittle failures have occurred where load is applied over a very short length of thread and the provision of exposed thread between the top plate and the shank or necked shank is recommended.

On this basis, machining down bolts to form a necked length would appear to be less necessary for the elastic design basis.

7.3 NECKING DETAIL

The bolt necking details should be based on the requirements of NZS 3404:1989 Clause 12.12.5 "Notched regions in tension braces for satisfactory ductile performance.

The requirements are stated as follows :

1. Material yield stress not greater than 350 MPa

   Yield plateau not greater than 30 x the yield strain for Gr 250, or 24 x the yield strain for Gr 350.

2. Notched section transition from the full section to be in a slope not exceeding 1 in 2.5. A slope of 1 in 2.5 is **recommended**, as shown on Figs 6.1 and 6.2.

3. Ratio of necked area to full crossection of area is not greater than 0.80.

Testing of necked details has shown that yield initially occurs in the thread followed by strain hardening, the stress increasing some 10-15%, and then subsequent yielding in the necked shank

7.4 BOLT HOLES

Holes, if punched, should be punched 3 mm undersize and reamed.
Hole diameters are recommended to be d + 6 mm Ref [10].

Micro cracks on the exit side of a hole can act as a source of stress raisers and cause brittle behaviour and fracture.

The edge distance for holes in the top and bottom plates of anchor chairs is recommended to be not less than d x 1.75. This corresponds to the requirements of AS1250 for sheared or flame cut edges.

7.5 CORROSION

Adequate corrosion protection should be provided, preferably hot dipped galvanising of all componentry to conform to AS1650.

Anchorages may be subject to a corrosive environment or occur at the junction of dissimilar metals where galvanic corrosion could occur. Special care with detailing is required. Often detailing of stainless steel tankage is such that anchor chairs of mild steel are welded directly to stainless steel walls. A doubler plate of stainless steel is required as a minimum at the shell wall prior to welding mild steel anchor chairs. Preferably the anchor chair should be of stainless steel and the dissimilar material, a mild steel anchor bolt, physically separated from the chair by an inert washer material.

Welds to stainless steel members should be passivated to avoid accelerated corrosion along the weld line.

8. HOLDING DOWN BOLT ANCHORAGE

8.1 INTRODUCTION

Avoidance of a brittle anchorage failure in the concrete foundation is essential in all applications where there is a significant risk to life and/or economic loss in the event of tank failure in an earthquake. In this area, attention to detailing aspects, application of capacity design philosophy and control of materials and workmanship are important. Examples of anchorage failure during the Edgecumbe earthquake are described in Ref [9].

The failures of holding down bolts into concrete included :

(a) Pullout of bolts due to insufficient anchorage length for the bolt.

(b) Pullout of bolts due to both insufficient anchorage length and to the size of the head on the bolt.

(c) Cracking of the foundation slab due to the holding down bolts being positioned too close to the edge of the slab and/or insufficient reinforcement being placed around the bolts to control cracking due to uplift - induced lateral tensile forces in the concrete.

The selection of anchorage details depends on a variety of factors, including :

(a) cost
(b) control of location and tolerances during foundation construction
(c) strength requirements of the designer (seismic loading)
(d) construction sequence and program
(e) performance of previous details
(f) workmanship reliability and quality assurance on site and at the tank fabrication shop
(g) corrosion and site environment
(h) size, function and performance requirements of tank.

Types of details commonly used, grout selection and design methods are discussed in the following sections. This discussion is limited to anchorages to concrete foundations only.

Preformed conical or corrugated ducted pockets are **recommended.** Cast-in Anchorages are also a satisfactory solution <u>but</u> require significant accuracy in set out.

Grouted anchorages in post-drilled or parallel sided preformed pockets (other than pockets formed with corrugated metal ducts) are **not recommended** due to reliance on bond at the grout/concrete interface.

Proprietary mechanical and grouted anchorages are **not recommended** due to the reliance on workmanship and problems of lack of compliance with manufacturers' recommendations. Also the anchor failure mode may be non-ductile. However, if such anchorages are used some outline guidance is provided in Section 8.2.3. A load test of at least 10% of the anchors is **recommended.**

8.2 COMMON TYPES OF HOLDING DOWN BOLT ANCHORAGES

The majority of anchorages in common use can be classified in the following groups.

8.2.1 Cast-in Anchorage (Fig 8.2.1)

This traditional method of structure to foundation connection provides a highly dependable load transfer system. A bolt, stud, threaded bar or threaded socket is cast in the foundation concrete. Bearing plates are used to enhance capacity.

Problems associated with this method include :

(a) Incorrect location of the bolts may render them useless, or at least result in an unsatisfactory anchorage detail.
(b) Close tolerance requirements, although for small diameter bolts a tubular sleeve will allow some movement on site. Insufficient tolerance allows little opportunity for adaptation on site.
(c) Protruding bolts are prone to damage by construction vehicles or during tank installation.
(d) Delay to construction program due to fabrication lead times.
(e) Difficulty in retrofitting and delays if damaged.

Advantages of cast-in anchorages include :

(a) Cost (no drilling/forming/grouting etc).
(b) Reliable, direct connection to the foundation (no reliance on grout workmanship).

The use of a cast-in threaded socket will avoid some of the problems discussed above.

In some cases, bolts are pushed down into position in the wet concrete. Care is needed with the timing and workmanship of this operation, and this practice is **not recommended.**

The designer needs to check for conical pullout, edge distance and bond failure.

8.2.2 Pre-formed Pockets (Fig 8.2.2)

A pocket or blockout is formed in the foundation and a bolt is subsequently grouted in place, usually after the tank is lowered in position.

The pocket may be formed using polystyrene which may later be physically removed, or dissolved by solvent. Alternatively, corrugated metal ducts or conduits may be used. PVC formers that are left in place will not perform satisfactorily. Formed holes with the formers removed normally require over-drilling to remove laitence or weak concrete at the grout/concrete interface.

If the pocket is of conical or truncated pyramid form, reliance on bond at the grout/concrete interface may be avoided.

A conical or truncated pyramid formed pocket or corrugated ducted pocket is the **recommended** detail for a Preformed Pocket.

The use of a high strength expansive grout permits high integrity of the load transfer mechanism. Grout is usually cement based or epoxy mortar. Grout characteristics, hole size and anchor testing requirements are briefly discussed in Section 8.3.

Non shrink, high strength cement grouts are **recommended.**

Advantages of the method include :

(a) ability to cope with large tolerances.
(b) foundation reinforcement is not cut or encountered.

Disadvantages associated with this method include :

(a) the need to ensure the correct location of pockets.
(b) supervision of the grouting operation is required.
(c) the cost of grout to fill the blockout.

The designer should check for conical pull out failure, edge distances, grout/concrete bond and anchor bolt/grout bond if appropriate.

8.2.3 Post-Drilled Anchorage Systems

General

Post-drilled anchorage systems include grouted and mechanical anchors, and many proprietary systems come into this category. They are **not recommended** for important installations as close control of installation methods and procedures (particularly for grouted anchorage systems) is required to ensure that the anchor will perform as expected. Also, the bolts or threaded studs used in proprietary systems are often not of notch ductile steel, and hence the performance of these systems in an extreme earthquake may be unsatisfactory.

In view of the above, it is **recommended** that the use of post-drilled anchorage systems be restricted to smaller tanks (less than 100 m^3 capacity) with Category

A classification as defined in Clause C1.2.1 of Ref [1]. An elastic design basis is essential for such systems.

A nominally elastic design basis may be used in situations where the anchorage is designed to yield the steel anchor before some other more "brittle" failure mechanism occurs - refer Section 8.4.

Advantages of post-drilled anchorage systems include :

(a) Low risk of set-out error
(b) Tolerances more easily achieved
(c) Proprietary systems are available

Disadvantages include :

(a) Cost (of grouted anchorages and good quality mechanical anchorages)
(b) Reliance on workmanship and strict compliance with manufacturer's specification
(c) Anchor location may clash with foundation reinforcement
(d) Any proprietary bolt may not be of notch ductile steel

For proprietary anchor systems the manufacturer's recommendations are required to be followed. The hole depth and diameter is usually specified by the manufacturer, together with anchor capacity, and spacing and edge distance requirements. The generally accepted practice used in the industry is to apply a factor of safety of 4.0 to the average test failure load, to determine a safe working load. Other reduction factors may apply, depending on the situation. A load test, to working load level, of at least 10% of anchors is **recommended.**

Post-drilled and Grouted Anchorages (Fig 8.2.3)

A bolt or threaded bar is grouted into a hole drilled after the foundation concrete has set. The holes may be marked out by lowering the tank in position, reducing the risk of set-out error. The anchor and grout may be a proprietary system with detailed installation instructions supplied by the manufacturer.

Pneumatic or electrically driven masonry drills or diamond drills may be used to form the holes. Masonry drills usually provide a rough surface for good mechanical bond at the grout/concrete interface. However, care must be taken that the drilling method does not produce micro-fractures in the base concrete around the drilled hole which, although not readily apparent, could reduce the grout/concrete bond strength. Diamond drills will also generally produce a sound surface for bonding. However, a polished surface, occasionally produced by diamond drills, may be detrimental to bonding capability and additional surface roughening may be required.

Grout characteristics, hole size and anchor testing requirements are discussed in Section 8.3. Anchor performance is particularly dependent on the development of a good bond at the concrete/grout interface. This may be affected by temperature, the presence of moisture, hole roughness and other factors. For nominally elastic designs the designer needs to check bolt anchorage failure mechanisms as described in Section 8.4. For elastic designs it is normally sufficient to apply an appropriate factor of safety (typically 4.0) to the average anchor failure load as determined by test. Reference [13] provides detailed information on testing and design of a commonly used type of proprietary grouted anchorage system.

Mechanical Anchorages (Fig 8.2.4)

A wide selection of proprietary systems is available. The most suitable anchor in any particular situation should take into account corrosion resistance, performance in cracked concrete, embedment depth, etc. Reference [14] provides background information and formulae for predicting the performance of common types of mechanical anchors. Some particular disadvantages of mechanical anchorage systems include:-

(a) Zinc coating for corrosion protection is often applied by electroplating and coating thickness may not be adequate for long term protection, depending on the environment;

(b) Concrete edge distances and spacing requirements are generally greater than for grouted anchorage systems.

Mechanical anchors are generally simpler to install and easier to handle than grouted systems, and performance of good quality anchors is more predictable.

8.2.4 Lugs or Cleats (Fig 3.5)

In this system a lug or cleat is clamped over the bottom rim of the tank. This system has been successfully used in glass coated steel water storage tanks built of site bolted panels.

Bolts are generally cast in place. Location of the bolts in the radial direction is set out from the centre of the tank. Location set out in the circumferential direction is not critical as alignment with tank fixing cleats or anchor chairs is not required.

Advantages of this method include :

(a) cost
(b) set-out and tolerances easily achieved
(c) a concrete ring beam can provide protection from physical damage and corrosion

Disadvantages include :

(a) bolts may be prone to damage
(b) lack of positive fixing between the lug and the tank, which may allow sliding at the connection. This may be overcome by providing a concrete ring beam which may be designed to provide additional uplift resistance

The designer should check bolt anchorage details and prying actions on the bolt. The lug should be designed for overstrength actions as described for anchor chairs in Section 5.1.

8.2.5 Replaceable Anchors

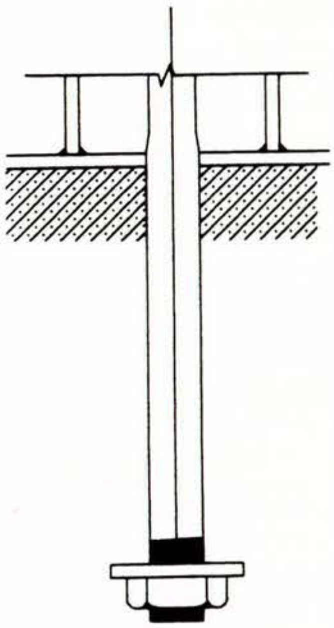
An adaption to cast-in bolts is the use of a cast-in sleeve into which the necked holding bolt is inserted.

This has the following advantages:-

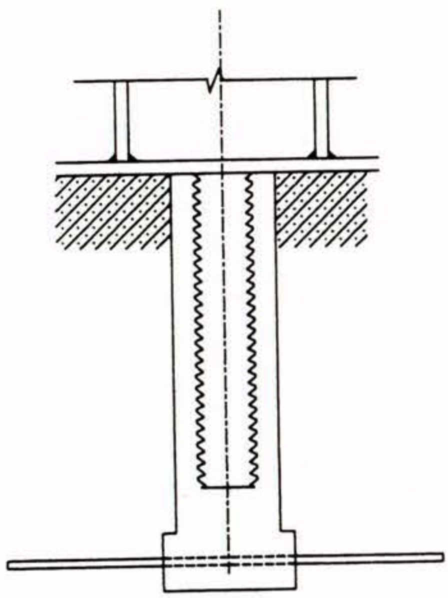
(i) Allows the bolt to be replaceable in the event that it is damaged or has demonstrably yielded in a seismic event;

(ii) Allows the tank to be positioned more easily without upstand bolts to interfere with location operations.

The sleeve itself is anchored by an anchor head cast into the concrete. Attention is necessary to sealing around the perimeter of the sleeve/concrete surface interface and sealing the bolt thread/sleeve interface with densotape to ensure the ability to remove the bolt at a later stage.



(a) Cast-In Bolt



(b) Cast-In Socket

Fig. 8.2.1 Cast-in Anchorage

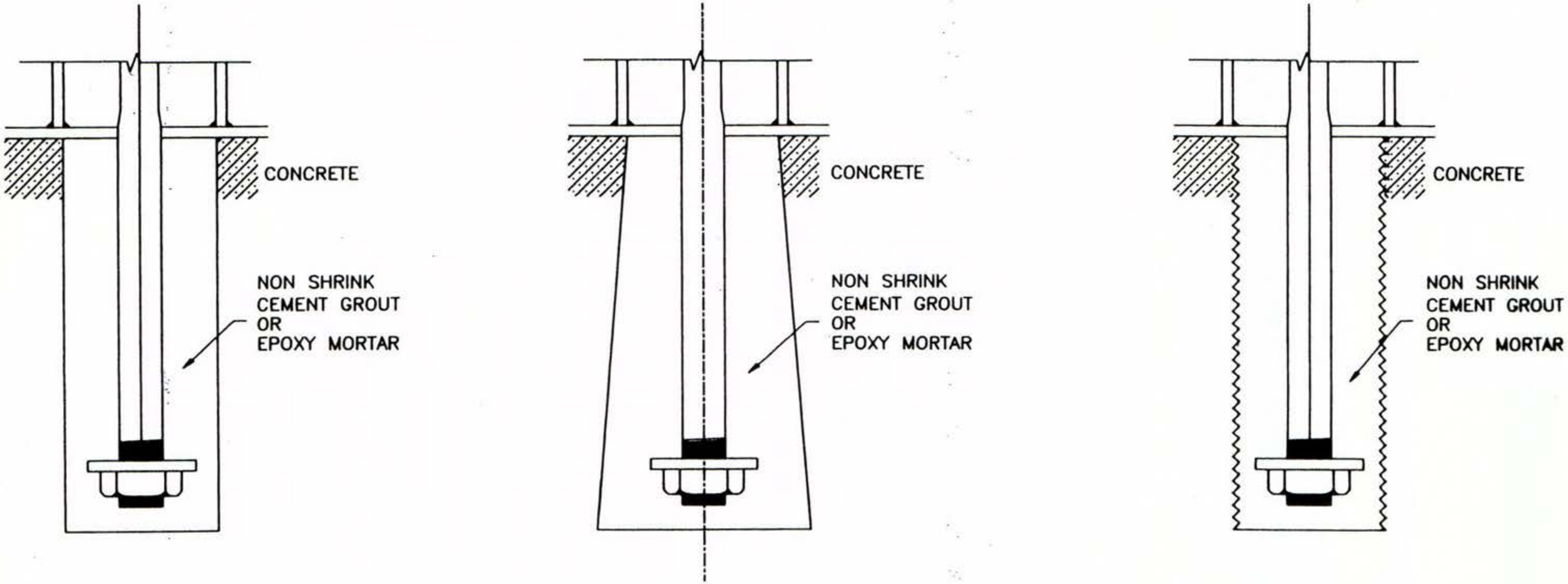


Fig. 8.2.2 Pre-formed Pockets

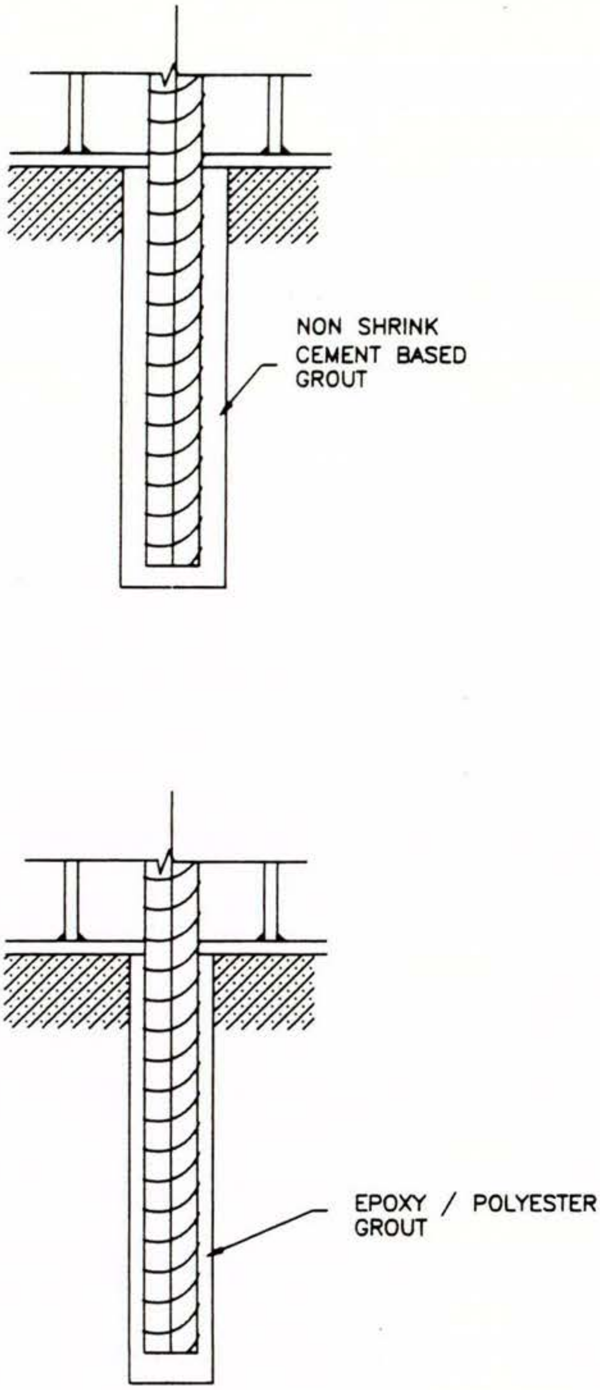
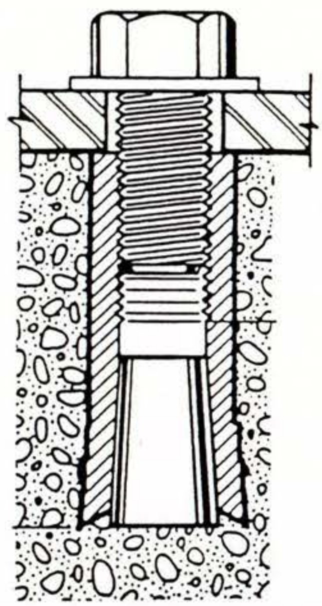
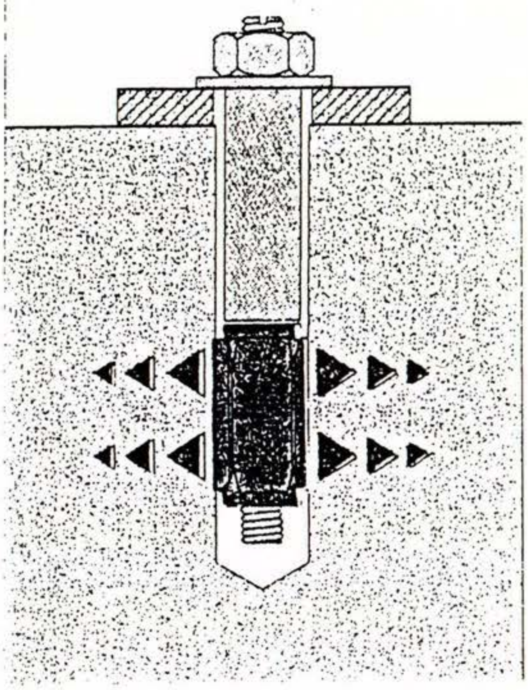


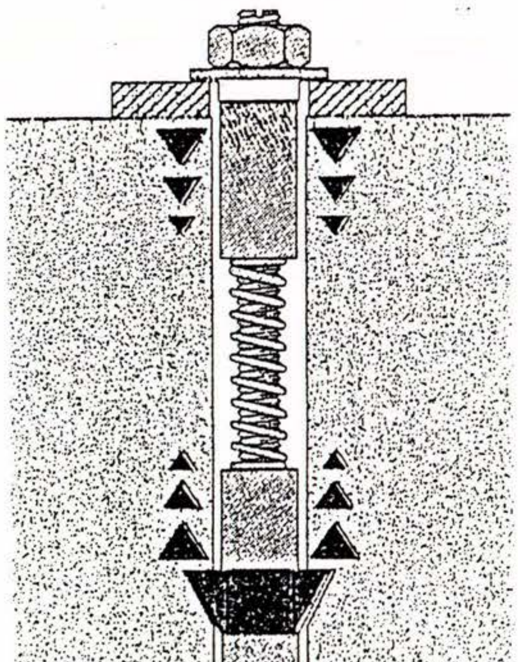
Fig. 8.2.3 Post–Drilled and Grouted Anchorage



SELF DRILLING ANCHOR



'LIEBIG' SAFETY BOLT



UNDERCUT ANCHOR

FIG 8.2.4 Mechanical Anchorages

8.3 GROUTING OF ANCHORS

Anchors may be grouted into preformed pockets or holes drilled after the foundation concrete has set. Reference [15] presents detailed information on grout selection, preparation for grouting and grouting procedures. A brief review only of aspects of grouting of anchors is presented herein.

Grout Characteristics

Suitable grout types include:-

- epoxy and polyester grouts (sometimes referred to as organic, chemical or polymer grouts);
- cement based grouts.

Most polymer grouts formulated for structural applications develop very high compressive strengths in a relatively short time and are capable of developing a stronger bond to concrete than cement based grouts. However, they usually have significantly different thermal expansion and stiffness characteristics from concrete. They come in a wide range of viscosities and strengths and can be sensitive to the conditions in which they are being used. Water and dust in the void will impair the bond and strength to a greater or lesser degree depending on the product. Water can also detrimentally affect the strength of some polyesters in the long term.

Cement-based grouts are easier to use than polymer grouts, and are usually cheaper. Also, chemical and physical properties are more closely matched to the surrounding concrete. Cement-based grouts would generally be more tolerant of damp conditions (during placement) and high in-service temperatures.

Cement-based grouts must be high strength and non-shrink to ensure that forces in the anchorage can be transferred to the surrounding concrete. Non-shrink or expanding grout is required to ensure that the bond and load carrying ability of the grout is not reduced by the presence of shrinkage cracks or entrapped air.

The manufacturer's instructions should be followed in detail when using proprietary grouts.

Hole Size

The optimum grout space, or clearance between anchor and hole, depends on the viscosity and "wetting" ability of the grout, as well as method of placement, and should be specified by the grout manufacturer for each specific application. It may vary from less than 1 mm for low viscosity high wetting polymer grouts, to in excess of 25 mm for cement based grouts.

Typical hole sizes are the anchor diameter plus 25 to 50 mm (ie grout space 12 to 25 mm) for cement grouts.

Anchor Testing

Pull out tests are frequently carried out to prove anchor capacity for a specific application, and may be required to provide basic information on performance of grouts not forming part of a proprietary system, but which are recommended as suitable for grouting of anchorages.

Proprietary anchor systems have, in general, been tested fairly extensively and manufacturer's data normally includes ultimate anchor capacities as well as recommended working loads.

A sampling frequency for load testing of completed anchorages should be established based on number of anchors, importance of the installation, etc. Test load should be limited to about one third the yield strength of the anchor (or say one quarter the ultimate capacity where no defined yield load exists), so as not to damage it for usage.

8.4 ANCHORAGE DESIGN FOR A DUCTILE FAILURE MECHANISM

8.4.1 Philosophy and Failure Mechanisms

Following the capacity design principles for seismic resistance, the designer should ensure that anchorage details will be sufficient to allow the over strength capacity of the anchor to be developed. Failure of the anchorage will then be by ductile yielding of the bar rather than by a brittle pull-out or concrete failure.

Five basic failure mechanisms should be assessed by the designer.

(a) Fracture or yielding of the anchor (Refer 5.1)
(b) Pull-out or excessive slip of the anchor
(c) Cone failure in the base concrete
(d) Splitting of the base concrete
(e) Edge failure for anchors loaded in shear

These are shown in Fig 8.4.1 Concrete Anchorage Failure Mechanisms.

The strength values of materials, rather than working stresses, should be used in assessment of the above failure modes.

8.4.2 Cone Failure in the Base Concrete

The embedment length of an anchor should be sufficient to ensure that the load P_o is achieved before bond or concrete cone failure. An anchor head is often used to provide a positive anchorage point rather than rely on anchor/concrete bond.

The anchorage details should be checked to avoid a pull-out failure of a cone of concrete surrounding the anchor.

The effective stress area, A_e, is the projected area of a 45° conical failure surface. This area excludes the area of the anchor or any stud end or bearing washer.

For a single anchor of head diameter D_h, and length, L, with edge distance greater than embedment length,

$$A_e = \pi L (L + D_h) \quad \ldots\ldots\ldots\ldots \quad (8.1)$$

The Equation:-

$$P_o = \pi L (L + D_h) \times 0.33\, \phi_c \sqrt{f'c} \quad \ldots\ldots \quad (8.2)$$

requires to be satisfied. This is obtained from the review in Ref [16] of various design methods and testing of anchor bolts with heads. It is recommended that L be a minimum of 300 mm. D_h is then defined for a given concrete strength and design force, P_o.

The resistance to cone failure can be affected by a number of factors, although these are unlikely to be relevant to tank anchorages :

(a) close spacing (overlapping cones)
(b) small edge distance
(c) limited foundation thickness for a group of anchors

8.4.3 Splitting of the Foundation Concrete

To avoid splitting of the foundation concrete the criteria of Eligehausen et al Ref [13] may be adapted as follows:-

anchor spacing > $L + D_h$

edge distance > $0.5 (L + D_h)$

where D_h = anchor head diameter

and L = depth of embedment

Cannon et al [8] give criteria for minimum edge distance "n" to avoid tensile failure of concrete due to lateral bursting forces, which may be presented as:

$$\beta P_o = 0.33\ \phi_c \sqrt{f'_c}\ \pi n^2$$

or $$n = 0.6\ [P_o/\sqrt{f'_c}]^{0.5} \text{ for } \beta = 0.25,\ \phi_c = 0.65$$

This expression applies to headed anchors embedded in concrete and would be expected to be conservative for grouted anchors due to the different mechanism of load transfer into the surrounding concrete. However, it is consistent with the edge distance criterion of Eligehausen et al Ref [13] for avoidance of splitting failure. Where anchor spacing parallel to the free edge is less than 2n, the effect of overlapping failure planes on the concrete design strength must be considered.

If the edge distance of the anchor is less than n, reinforcement is required to control tensile failure of the concrete due to lateral bursting forces. Based on accepted practices for prestressing anchorages, spiral reinforcement is recommended Ref [8].

8.4.4 Edge Failure for Anchors Loaded in Shear

For a fully embedded anchor (i.e. concrete tensile capacity exceeds steel tensile capacity), the dependable concrete shear strength is given by:

$$Q_{uc} = 0.5\ \pi m^2\ \phi_c\ 0.33\sqrt{f'_c} \qquad \text{(Ref [8])}$$

where m = edge distance (centre of anchor to concrete face)

Failure in the concrete will not occur under lateral loading if:

$$Q_{uc} \geq A_v f_{ut} \ \ldots\ldots\ldots\ldots \text{(Ref [8])}$$

ie $$0.5\ \pi m^2\ \phi_c\ 0.33\sqrt{f'_c} \geq A_v f_{ut} \ \ldots\ldots\ldots\ldots \text{(Ref [17])}$$

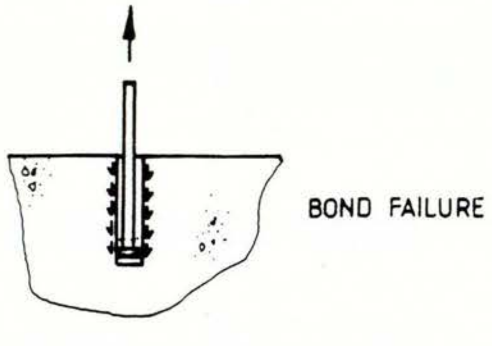
giving $$m \geq 1.7\ [A_v f_{ut}/\sqrt{f'_c}]^{0.5}$$

or $$m \geq 1.5d\ [f_{ut}/\sqrt{f'_c}]^{0.5}$$

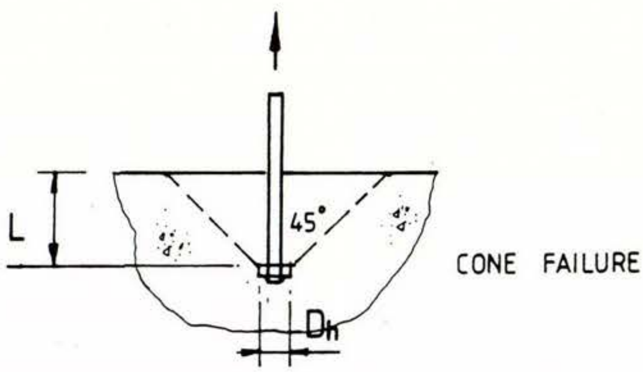
for $\phi_c = 0.65$ and $A_v = \pi d^2/4$

(ie threads excluded from the shear plane)

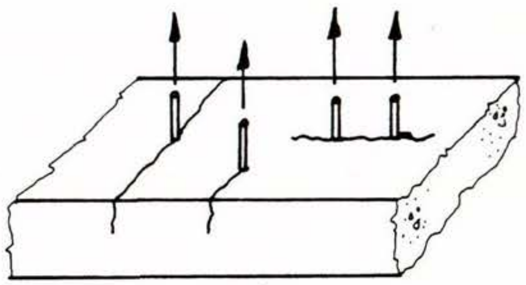
For a laterally loaded anchor, edge distance should never be less than m/3. For edge distances between m/3 and m, reinforcement is required to restrain tensile cracking. Refer Cannon et al [8] and Klingner et al [18] for discussion and details.



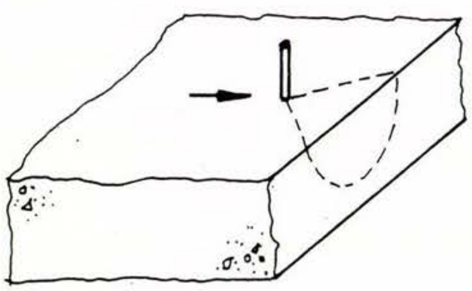
(a) PULL-OUT FAILURE



(b) CONE FAILURE IN THE BASE CONCRETE



(c) CONCRETE SPLITTING FAILURE



(d) EDGE FAILURE

Fig. 8·4·1 Concrete Anchorage Failure Mechanisms

9. OUTLINE SPECIFICATION FOR DESIGN OF STEEL STORAGE TANKAGE (incorporating Seismic Design requirements)

9.1 GENERAL:

This section is intended as a guideline for tankage purchasers to identify the necessary components of a specification to design/build fabricators of tankage to incorporate satisfactory seismic resistance for tankage and to clarify the relevant design responsibilities.

9.2 OUTLINE SPECIFICATION:

Any specification or request for price from a tankage purchaser to a fabricator should specify the following items as a minimum:

1. Design Parameters:

(a) Contents: _________ SG _________ @ _________ °C.

(b) Working Capacity of Tankage:

_________ m^3 @ _________ °C.

(c) Freeboard Requirements:

A freeboard of _________ mm over working capacity or freeboard as calculated by NZNSEE Recommendations for Seismic Design of Storage Tanks, whichever is the larger.

2. Tankage Materials Specification:

(a) Options:

- Mild Steel to minimum standard of AS3679 (formerly AS1204) Gr 250 or equivalent
- Stainless Steel Gr 304, 316
- Other specification

(b) Corrosion Allowance:

Mild Steel *(uninsulated)(usually min 1 mm shell, roof/1.5 mm base).*

3. Tankage Configuration:

(a) Proposed Tank Aspect Ratio *(Height/Diameter)* _________.

Min Diameter _________ Max Diameter _________.

Min Height _________ Max Height _________.

(b) Tank Base Configuration - Tank Drainage Options:

- Conical base *(fall to centre)*;
- Elevated bottom plate, knuckle joint;
- Sloped base *(fall to perimeter)*;

(c) Tank Roof Configuration *(Cone/Dome/Other)*;

(d) Design/Operating Pressure _________ mm WG;

(e) Design Vacuum _________ mm WG;

(f) Nozzle/Sump Data *(Layout Plan of Proposed Nozzle Orientations, etc)*;

(g) Proposed Foundation Type - concrete slab, ringbeam.

4. Design Basis:

(a) Environmental loads - in accordance with NZS4203 : 1984;

(b) Hydrostatic Design - API650 : 1988 or later amendments. *(Fabricator to specify other proposed alternative)*;

(c) Seismic Design Basis:

- NZNSEE - Recommendations for the Seismic Design of Storage Tanks, Dec 1986;

- Identify Seismic Risk Category as follows [Ref [1]]:

A - People at Risk from failure	< 1
Value of Contents at Risk	< $10,000 (1985 index);
B - People at Risk from failure	< 10
Value of Contents	< $200,000 (1985 index);

- Seismic Zone : State location of plant;

- *(Other options include:*

  Seismic Design to API 650 : 1988, Appendix E as modified by MWD SDPP March 1981, specifying return period of earthquake as 150 years minimum).

5. Seismic Resistant Mechanism:

(a) The Seismic Resistant Mechanism shall be in accordance with EQC Standard Seismic Details for Industrial Tanks and Silos, Oct 1990. The Seismic Resistant Mechanism shall be that of *(nominate following options:*

- *Anchored Tank (usually less than 300 m^3 capacity), (Isolated Anchor Chairs or Continuous Ring);*
- *Unanchored Tank (Broadbased Tankage);*

(b) The Design Basis shall be _________ *(Nominate (i) or (ii) or (iii)).* The use of the appropriate overstrength factors nominated in this document for the design of all other elements of the holding down system shall be made.

(i) Elastic or Nominally Elastic Design *(tankage less than 100 m^3 and medium acceptable risk of extreme damage, Category A tanks)*;

(ii) Limited Ductile Design:

(Tankage greater than 100 m^3, of significant importance to the purchaser's process).

(Selection to be based on risk assessment in ensuring the tank survives an extreme seismic event with relatively little damage);

(iii) Ductile *(Tall vessels of aspect ratio greater than 5:1):*

(Important petrochemical installations or with a significant cost of outage in event of failure in a seismic event).

6. Seismic Resistant Materials:

Material selection for holding down bolts shall be in accordance with EQC "Standard Seismic Resistant Details for Industrial Tanks and Silos", October 1990, Section 7.

7. Anchorage of Holding Down System:

The anchorage of the holding down system shall be in accordance with EQC "Standard Seismic Resistant Details for Industrial Tanks and Silos", Section 8, appropriate to be selected Seismic Resistant Mechanism (Refer 5).

9.3 RESPONSIBILITIES OF THE VARIOUS PARTIES

It is recommended that a Professional Engineer be involved in scoping of the tankage design brief to achieve cost effective solutions to holddown of tankage consistent with practical construction details at an appropriate level of risk.

The design responsibilities of the various parties are as follows:-

9.3.1 Tank Designer:

The Tank Design Engineer or the Design/Build Tank Fabricator shall be responsible for the detail design of the tank and holddown systems for all imposed loadings derived from:-

- hydrostatic loads
- internal pressure
- other operating conditions
- environmental loads (wind)
- seismic loads

The Tank Design Engineer or Design/Build Fabricator is responsible for the detail design of anchorage chairs and holding down bolts and anchor heads. The fabricator is responsible for the fabrication of all anchor chairs, holding down bolts and their anchor heads.

The Tank Designer/Fabricator shall supply the design loads and layout of holding down bolts to the Foundation Designer stating the design bases for the holding down mechanism used referring to Sections 4 to 7.

9.3.2 Foundation Designer/Civil Works Contractor:

The nominated Foundation Designer shall be responsible for the design of the foundation to loads nominated by the tank designer. The Civil Works Contractor is responsible for the setout and location of holding down bolts *(supplied by the tank fabricator)* or for the provision of preformed pockets for the insertion of the holding down bolts by the Tank Fabricator.

The Civil Works Contractor is required to site measure the tank and liaise with the tank fabricator to confirm setout and location of holding down bolts.

10. REFERENCES

[1] *NZ National Society for Earthquake Engineering - Recommendations "Seismic Design of Storage Tanks" December 1986* (NZNSEE)

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[3] *Standards Association of New Zealand. 1984. Code of Practice for General Structural Design and Design Loadings for Buildings. NZS 4203. Wellington, New Zealand.*

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[6] *Taylor R.G. "Seismic Design of Bolted Steel Tanks" Local Authority Engineering Vol. 5 No.1 1988.*

[7] *NZS 3404. Parts 1 and 2 : 1989: Steel Structures Code*

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[10] *American Iron and Steel Institute*
"Steel Plate Engineering Data Vol.2 - Useful Information on the Design of Plate Structures", Feb 1979.

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Journal of Structural Engineering Vol. 109 No.9 Sept 1983.

[13] *Eligehausen, R., Mallee, R. and Rehm, G. - "Fixings Formed wtih Resin Anchors", Betonwerk + Fertigteil - Technik, 1984, Heft 10, pp 686-692, Heft 11 pp 781-785 and Heft 12, pp 825-829.*

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[16] *Klingner, R.E. and Mendonca, J.A. - "Tensile Capacity of Short Anchor Bolts and Welded Studs : A Literature Review" A.C.I. Journal 79-27, July-August 1982.*

[17] *Klingner, R.E. and Mendonca, J.A. - "Shear Capacity of Short Anchor Bolts and Welded Studs : A Literature Review", A.C.I. Journal 79-34, September - October 1982.*

[18] *Klingner, R.E., Mendonca, J.A. Malik, J.B. - "Effect of Reinforcing Details on the Shear Resistance of Anchor bolts under Reversed Cyclic Loading", A.C.I. Journal 79-1, January - February 1982.*

11. NOTATION

a	=	top plate width (mm)
A_e	=	effective stress area (mm^2)
A_s	=	effective tensile stress area of anchor (mm^2)
A_v	=	effective cross-sectional area of anchor for shear
b	=	top plate length (mm)
c	=	top plate thickness (mm)
d	=	anchor diameter (mm)
D	=	anchor hole diameter (mm)
D_2	=	bolt pitch circle diameter (mm)
D_h	=	anchor head diameter (mm)
e	=	anchor bolt eccentricity (mm)
f	=	distance from the outside of the top plate to the edge of the anchor hole (mm)
f'_c	=	cylinder strength of base concrete (MPa)
f_s	=	the lesser of f_y and 0.9 f_{ut}
f_{ut}	=	specified minimum ultimate tensile strength of anchor (MPa)
f_y	=	specified yield strength of anchor (MPa)
g	=	distance between vertical plates (mm)
h	=	chair height (mm)
j	=	vertical plate thickness (mm)
k	=	average plate width for taper plates (mm)
L	=	embedment length (mm)
m	=	edge distance of anchor loaded in shear (mm)
M_{ot}	=	total design overturning moment (kN-m)
n	=	minimum edge distance of anchor loaded in tension (mm).
p	=	thread pitch (mm)
P_{max}	=	anchorage force required per metre (kN/m).
P_o	=	over strength tensile capacity of anchor as governed by steel failure (N or kN)
P_u	=	design factored axial load on connection (negative for tension) (N)

P_{uc}	=	dependable tensile capacity of anchor as governed by concrete failure (N)
P_{ue}	=	dependable tensile capacity of anchor as governed by bond failure (N)
P_{us}	=	dependable tensile capacity of anchor as governed by steel failure (N)
Q_u	=	design factored shear load on connection (N)
Q_{uc}	=	dependable shear capacity of anchor as governed by concrete failure (N)
Q_{uf}	=	dependable shear capacity of anchor as governed by shear friction (N)
Q_{us}	=	dependable shear capacity of anchor as governed by steel failure (N)
R	=	radius of tank (mm)
t	=	tank wall thickness (mm)
t_b	=	bearing plate thickness (mm)
T	=	period of vibration (sec)
T_e	=	period of equivalent linear system for yielding structure (sec)
w_t	=	weight of walls and roof at base of wall, per unit circumferential length (kN/m)
μ	=	shear friction coefficient <u>or</u> displacement ductility factor
ϕ_c	=	strength reduction factor for concrete = 0.65
ϕ_f	=	strength reduction factor for shear friction = 0.85
ϕ_o	=	overstrength of anchor
ϕ_s	=	strength reduction factor for steel = 0.90 or 1.0
ζ	=	percent of critical equivalent viscous damping for structural response up to yield
ζ_e	=	percent of critical equivalent viscous damping for equivalent linear structural system

12. APPENDICES

APPENDIX 1

TENSILE

CROSS-SECTIONAL AREAS OF COMMON

ANCHORAGE SIZES

APPENDIX 1

DESIGN FORCES FOR HOLDING DOWN BOLT ANCHORAGES

This table is based on the following premises :

1. Area at necked down portion of bolt.

   = 80% x stress area at thread

2. f_y = 300 MPa for reinforcing round to NZS 3402 : 1989 (ex Pacific Steel)

3. ϕ_o = 1.35 for fully ductile bolt of Gr 300 material. (Note: that other overstrength factors may be applicable to other mild steel materials - refer Ref [5]).

4. Design Anchor chairs using strength method for force from bolt
   = P_o = A neck x 1.35 x 300 MPa.

Bolt Dia Rod Dia mm	Area mm^2	Thread Pitch	Area at Thread (Stress Area) mm^2	Rational Neck Dia' mm	Rational Area mm^2	Neck <u>Area</u> Stress Area	P_o (kN)
16	201	2.0	157	12.5	122.7	.78	49.70
20	314	2.5	245	16.0	201	.82	81.40
24	452	3.0	353	19.0	283	.80	114.70
28	616	3.5	480	22.0	380	.79	153.90
30	707	3.5	561	24.0	452	.81	183.10
32	804	3.5	647	26.0	530	.82	214.70
36	1054	4.0	816	29.0	661	.81	267.70

APPENDIX 2

ANCHORAGE MATERIALS

Typical Stress/Strain Relationships

1. AS1204 Gr 250

2. Grade 304 Stainless Steel

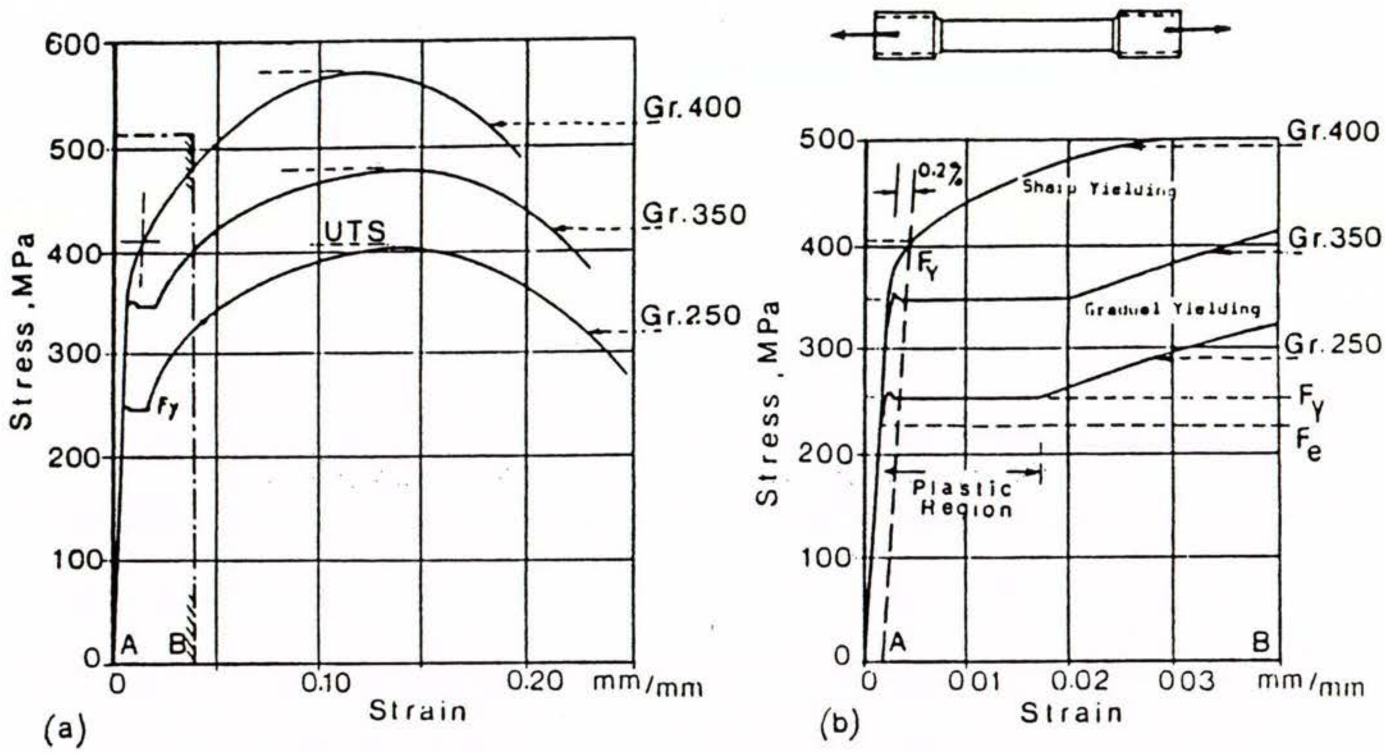

Fig. A 2.1 Typical stress-strain diagrams for steel grades 250, 350 and 400. (a) Complete diagrams. (b) Enlarged portion of the diagram in the region AB.

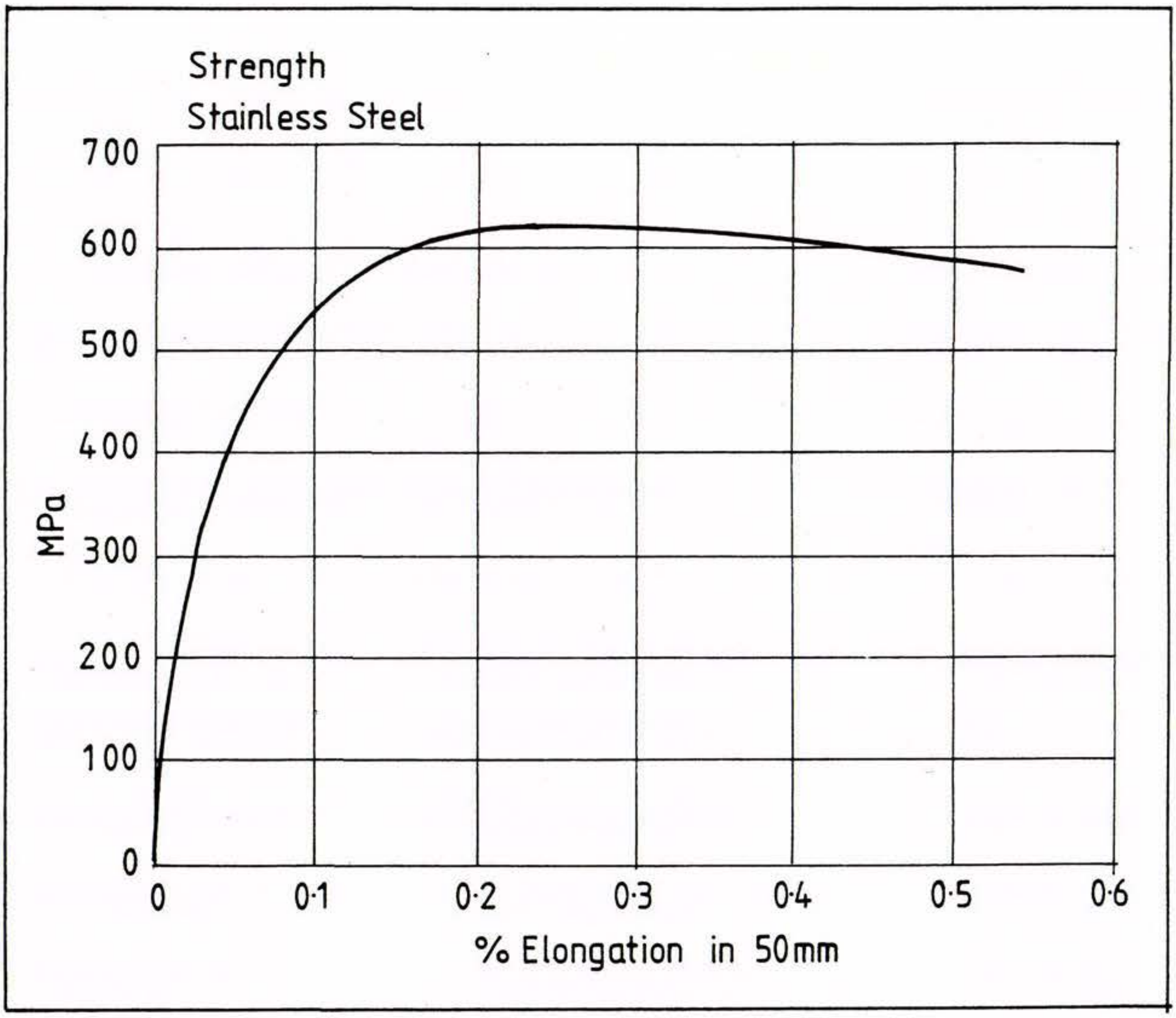

Fig. A. 2.2 Typical Stress / Strain Relationship For Grade 304 Stainless Steel