SESOC DESIGN GUIDE - ANCHOR BOLTS FOR STEEL STRUCTURES

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SUMMARY

This paper has been prepared by SESOC for the guidance and assistance of its members. It deals with anchor bolts (historically called 'hold down bolts') that are cast into reinforced concrete foundations in order to 'anchor' steel structures to those foundations. The guide is of a general nature, explaining the manner in which anchor bolts behave and therefore should be designed, the materials and fabrication methods that should be specified and used in their manufacture, and common problems with design, manufacture and installation. The basic principles of structural action are described, with only a small number of relevant equations. Because of the infinite variety of foundation types, reinforcing arrangements and edge distances, it is not possible to provide 'equations' to cover all situations. However, if the fundamental actions are understood and allowed for in the design of the bolt embedment and the foundation, a dependable anchorage can be achieved.

1.0 INTRODUCTION

1.1 General

This design guide, "Anchor Bolts for Steel Structures" provides guidance on the design of steel anchor bolts cast into reinforced concrete foundations in order to anchor steel structures to the foundations through the transmission of tension and/or shear forces.

As anchor bolts straddle the interface of reinforced concrete and structural steel, they have tended to be omitted from both types of structural design standard in most countries, and good design guidance is difficult to find. This is despite the fact that the interface is present in nearly all steel structures. Anchor bolt assemblies are often on the critical path, and the accurate installation of anchor bolts is a critical operation that must be carried out with care.

In addition to providing information regarding anchor bolt design actions, this guide also provides comprehensive information on the range of materials that are available in NZ for the fabrication of anchor bolt assemblies, how to detail anchor bolt assemblies in ways that aid accurate construction and avoid common problems, and warnings against certain bad practices.

Special anchor bolts such as those with upstand rolled threads or reduced shanks used for storage tanks, designed as tension yielding energy absorbers under seismic loading, are not covered specifically in this guide. Nor are special installations such as bolts located at the rocker supports of bridge girders and crane runway girders, where long bolt projections are necessary to allow the bolt to flex with rotation of the support. Similarly, design of anchor bolts for fatigue loading is excluded, although some general advice is given on the qualitative fatigue performance of different thread types, and on the pretensioning of stress bars used to anchor large, wind exposed monopoles and the like.

The design of steel baseplates, and shear keys, is integral with the design of anchor bolts. However specific reference on the design of these is beyond the scope of this guide. This should not adversely affect the understanding of the material within.

1.2 Bolt and Material Terminology and References

With regard to the 'grade' of steel used in anchor bolts, the following terminology is used in this guide:

'Property Class'	-	used to describe a commercially available bolt or fully threaded stud manufactured in accordance with the Standards listed below (or similar),
'Grade'	-	used to describe plain (round) steel that may be machined into bolts and studding. When using Standards such as those listed below to assess the requirements for machined bolts, such as matching nut grades, the Engineer shall treat the machined steel as the equivalent Property Class, based on its mechanical properties.

PAPER CLASS & TYPE: GENERAL REFEREED

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Some New Zealand and Australian/New Zealand Standards that are relevant to the design and specification of anchor bolts are:

AS/NZS 1111 ISO metric hexagon commercial bolts and screws.

AS/NZS 1112 ISO metric hexagon nuts, including thin nuts, slotted nuts and castle nuts.

AS/NZS 1252:1996 High-strength steel bolts with associated nuts and washers for structural engineering.

AS 1214-1983 Hot-dip galvanised coatings on threaded fasteners (ISO metric coarse thread series).

AS 4291.1-2000 Mechanical properties of fasteners made of carbon steel and alloy steel,
Part 1: Bolts, screws and studs.

AS/NZS 4291.2:1995 Mechanical properties of fasteners – Nuts with specified proof load values – Coarse thread.

(Refer to Section 6.0 for the complete list of references for this guide).

2.0 DISCLAIMER

Considerable effort has been made and reasonable care has been taken to ensure the accuracy and reliability of the material contained in this guide. This guide must be used by suitably trained and experienced structural engineers, in conjunction with sound engineering principles, and current information on material availability and properties. All materials must be used strictly in accordance with the manufacturers' recommendations. SESOC and the authors of this guide make no warranty or guarantee in connection with this guide, and shall not be held liable or responsible for any loss or damage resulting from the use of this guide.

3.0 PROBLEMS OBSERVED WITH THE DESIGN, FABRICATION AND INSTALLATION OF ANCHOR BOLT SYSTEMS

3.1 Problems Observed

(a) Incorrect design procedures,

(b) Inadequate anchorage and concrete edge distance in relation to the design actions,

(c) Specification of material grades, bolt sizes and bolt geometries that cannot be obtained, or fabricated as designed,

(d) Anchor bolt assemblies that make no allowance as to how they can be accurately installed, and held in place during the placement of concrete,

(e) Clashes between anchor bolt assemblies and reinforcement in foundations,

(f) Inappropriate use of drill-in anchors,

(g) Poor installation of drill-in anchors,

(h) Incorrect fabrication, including cutting the wrong threads,

(i) Inaccurate placement – incorrect location, incorrect levels, incorrect offsets, incorrect orientation, movement during concrete pours,

(j) Contractors neglecting to cast in anchor bolts, and in some instances preferring to not cast in anchor bolts, hoping to drill and epoxy fix straight rods after the concrete has been poured,

(k) Site damage to threads,

(l) Inappropriate welding or bending of high strength bolts and bars.

Proper thought and detailing at the design stage, and good structural drawings and specifications, can alleviate many of these problems.

3.2 Incorrect Design Procedures

Anchor bolts are often incorrectly designed as if the baseplate is part of a steel to steel connection, with bolts in combined tension and shear, using the traditional interaction equation for shear and tension, namely, in NZS 3404 nomenclature,

$$\left(\frac{V_f^*}{\phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\phi N_{tf}}\right)^2 \leq 1.0$$

The bolt shear design capacity, ϕV_f, and the interaction equation itself, are for bolts in bearing mode in a *steel to steel connection, not* for bolts embedded in concrete.

3.3 Unacceptable Bad Practices

These include:

(a) Tack welding or welding of nuts, washers, bearing plates and locating bars to Property Class 8.8 and other high strength anchor bolts.

Property Class 8.8, AISI 4140 and similar high strength material *cannot be welded, tack welded, or subjected to adverse heat effects,* such as allowing molten metal to fall on it. These materials are simply not weldable in practical building construction terms. The arbitrary specification of 'preheat' and/or 'low-hydrogen electrodes' does not make high strength material weldable.

Tack welding is especially bad – it is not "just a tack weld." The low heat input of a tack weld often results in rapid quenching and crack formation in non-weldable, high strength materials.

(Strictly speaking, AISI 4140 type material, and the similar Property Class 8.8 material, can be welded, but the preheat temperatures are near 500^0 C, special electrodes are required, and the weldment must be post-weld heat treated, quenched and annealed to regain its yield strength and ductility. Special welding procedures must be developed, and proven before use).

(b) Over-cutting bolt threads to accommodate a hot dip galvanising layer. (Refer Section 5.3).

(c) Bending of threaded sections to form U and L bolts.

(d) Bending of high strength material to form U and L bolts. Property Class 8.8, AISI 4140 and similar high strength material should not be bent.

(e) Poor bolt setout, both in plan and in elevation. The wrong setout in plan is bad enough, but when bolts are set too low, proper extension of the bolts is difficult, or impractical.

Table 1 shows some common bad practices in diagrammatic form. The actual bad practices are marked with an asterisk.

Table 1a. Examples of bad practice

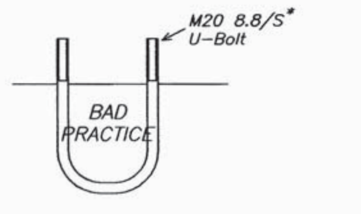
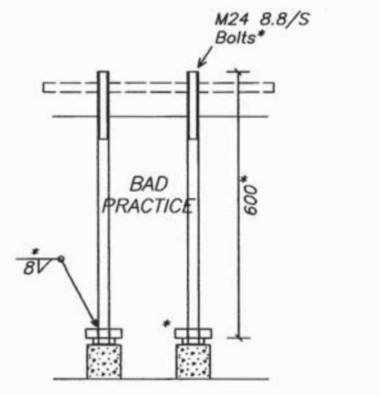
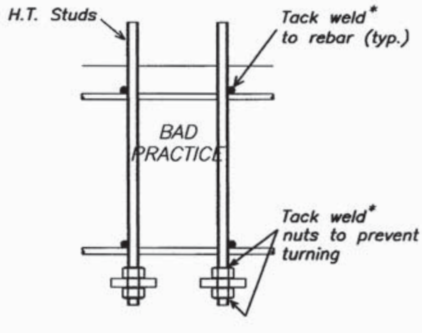
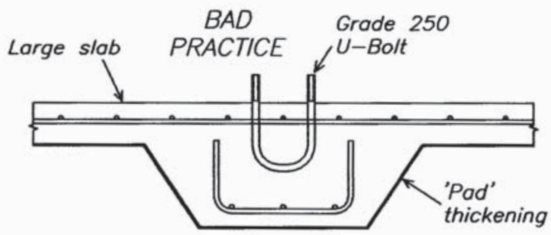
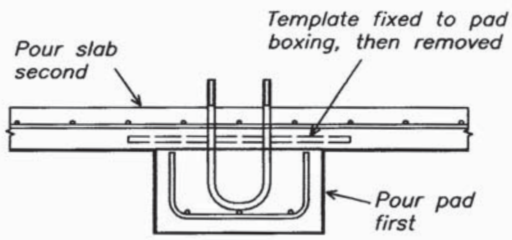
BAD PRACTICE	COMMENTS
1. 	* Property Class 8.8 and other high strength material must not be bent. Property Class 8.8 plain round is not generally available to fabricate such bolts.
2. 	* Property Class 8.8 bolts this long are not available. Property Class 8.8 and other high strength material must not be welded. Property Class 8.8 plain round is not generally available to fabricate such bolts. Use fully threaded 8.8 studding, AISI 4140, etc. Although the top template is rigidly located, and the bolts are supported underneath, there is nothing to stop the bottom of the bolts moving around during the concrete pour. Use steel flats to connect the bolts in both directions, then tie these flats to the reinforcing cage.
3. 	* Property Class 8.8 and other high strength material must not be welded or tack welded. Bolts cannot be accurately located by fixing directly to reinforcing.

Table 1b. Examples of bad practice

BAD PRACTICE	COMMENTS
4. 	* How can the U–Bolt be expected to be located accurately, especially after concreting? It is much better to box and pour the pad first, with longer U–bolts firmly held in position as they are cast in. Pour slab in a second operation, thus: 

4.0 DESIGN METHOD FOR ANCHOR BOLTS WITHOUT SHEAR KEYS

4.1 Design Actions

The design forces and moments acting on the baseplate shall be those arising from the various Ultimate Limit State (ULS) load cases, including seismic overstrength where appropriate, that are given in the NZ Structural Design Actions standard AS/NZS 1170:2002 - 2004 [6].

Clearly, the design actions must make full allowance for variable loading, in particular all governing wind directions, and reversing seismic loads. (This guide differs from the recommendations of DCB-56 [15] in that this guide acknowledges that under seismic loads, the maximum actions of shear, axial force and bending moment may not occur at the same instant.)

Consistent with NZ practice, the design strength of the anchor bolts in resisting these ULS actions is determined by multiplying the nominal strength by an appropriate strength reduction factor.

4.2 Design Methods - General

Steel baseplates with anchor bolt assemblies behave very much like reinforced concrete sections when carrying bending moments and/or axial loads from the structure above. Bending moments are resisted by the anchor bolts acting in tension at an appropriate lever arm from the corresponding area of mortar and concrete in compression. Just as in reinforced concrete sections, net tensile axial loads on the baseplate increase the bolt area required to resist a particular bending moment, and net compression loads reduce the bolt area required to resist the same bending moment.

It is in the transmission of shear through baseplates and anchor bolts to the foundations that anchor bolts may behave differently to shear transfer mechanisms associated with reinforced concrete.

In this guide, in the absence of shear keys welded to the underside of the baseplate and embedded in the foundations, or embedded baseplates, the following modes of shear transfer are recommended:

(a) For non-seismic 'static' type loading, such as dead, live and wind, where dependable compression forces are present due to axial loads and/or bending moments, shear forces may be transferred through friction.

(b) For non-seismic 'static' type loading where there is insufficient compression due to axial load or bending moment to transmit the shear by friction, the entire shear force shall be transferred through dowel action with full allowance taken of the resulting bending moments in the anchor bolts.

(c) For seismic loading, shear forces should be transferred through dowel action, with full allowance taken of the resulting bending moments in the anchor bolts.

(d) For baseplates fixed to the foundation through the use of pretensioned stress bars, for example at the base of large monopole communication towers, shear forces may be transferred through friction, if and only if the bars do not yield under the load case considered.

The rationale for recommending these various modes is as follows.

For many years now, New Zealand engineers have been familiar with the concept of shear friction as a mechanism to carry shear across interfaces in reinforced concrete. Shear friction has been codified in all editions of NZS 3101, with one equation covering any level of axial load, from axial compression, to none, through to axial tension. Various types of interface are allowed for, including as-rolled clean steel to concrete.

However, recent testing by HERA [14] on composite beam shear stud behaviour has shown that, in the absence of an exterior compression force, shear transfer between concrete and a steel plate will be by dowel action, because the interface is too smooth to generate the clamping force from the reinforcing implied in the shear friction equation. This is especially so under cyclic seismic loading.

Under seismic loading, bending moments, shear forces and axial forces may develop at the base of columns. However, except for simple symmetric single storey structures, these design actions are unlikely to occur at the same time. Consequently the supportive compressive stress resulting from the design moment cannot be counted on to provide friction to resist the design shear force. Additionally, as a consequence of the over sizing of the holes in the baseplate, the design shear force cannot be resisted by all the anchor bolts without some sliding of the base plate. Therefore it is recommended that, in lieu of shear keys, anchor bolts be considered to resist seismic shears by dowel action.

There are some significant differences between the recommendations given in this guide, and the behaviour of Embedded Items, Fixings and Secondary Structural Elements as defined in NZS 3101:2006 [2] Sections 17 and C17 as follows.

(i) Many large anchor bolt assemblies will be designed to 'lap' with longitudinal reinforcing bars in the foundations, and will not rely on the tensile strength of concrete for their embedment.

(ii) Sections 17 and C17 of NZS 3101:2006 consider shear from plates that are fixed directly to the supporting concrete, whereas structural baseplates usually bear on a bed of drypack or grout mortar, with relatively small 'edge' distances between the centre of a bolt and the adjacent mortar edge.

(iii) Under lateral loading when there is no dependable compression component acting from the column onto the baseplate, dowel action can cause significant bending moments to develop in the bolts due to the vertical distance between the baseplate and the foundation concrete.

4.3 General Information and Requirements

(a) 'Thick' Baseplates and Prying Action on Bolts

With the exception of annular baseplates such as the one shown in Table 3c Item 6, it is usual to assume that the baseplate is 'thick,' and acts in single curvature where it projects outside the perimeter of the column. (In the context of this Guide, a "thick" baseplate will be defined as sufficiently thick so that it "cantilevers" beyond the base of the column in single curvature allowing the bolt tension to be calculated directly from overall base actions. A calculation procedure to obtain a suitable thickness is outlined in DCB 56 [15]).

It should be noted, that if a relatively thin baseplate is used, the reverse curvature in the baseplate results in a significant increase in bolt tension forces. [3] (p.18).

(b) Size of Holes and Requirements for Oversize Holes

In accordance with NZS 3404:1997 Section 14.3.5.2.2, baseplates may have holes that are up to 6 mm greater than the anchor bolt diameter. Where holes are 3 mm to 6 mm oversize, a special plate washer with minimum thickness of 4 mm must be used under the nut.

Note that Section 14.3.5.1.3 is the only instance where NZS 3404:1997 allows the flame cutting of a bolt hole, even for 'site rectification'. Where it is necessary on site to enlarge any holes to be greater than 6 mm oversize, the oversize hole must be covered with a specifically designed washer fabricated from steel plate. The washer shall have a 2 mm oversize hole to accommodate the anchor bolt (3 mm oversize for bolt diameters greater than 24 mm). The washer shall be thick enough to resist all reactions from the nut, and, if used to transmit shear, shall be site welded to the baseplate as per the Engineer's instruction.

(c) Embedment of Anchor Bolts and Concrete Resistance to Shear Forces

The design of the anchor bolt embedment and transfer of the baseplate actions to the foundation structure should be based on first principles and sound engineering judgement. It is important to ensure that the bolt embedment and the foundation reinforcing geometry allow the internal forces necessary for equilibrium to develop. Depending on the shape of the anchor bolts, the anchorage of the bolts can be achieved in three ways – bearing on an end plate, development as a hooked reinforcing bar, or development as a deformed reinforcing bar.

In general, the ungalvanised shank of round bars threaded at the end should not be relied upon for anchorage by friction forces, because they are often covered in machining lubricant or oil. When considering bearing on an end plate where there is significant side cover, the standard bearing provisions of NZS 3101 do not apply. Instead refer to NZS 3101:2006 Sections 17 and C17.

NZS 3101:2006 Sections 17 and C17 contain extensive information for determining the pullout resistance and concrete shear resistance of relatively 'shallow' anchor bolts in unreinforced and lightly reinforced concrete.

The embedment and concrete shear resistance of 'deeper' anchor bolts in more heavily reinforced concrete foundations can be designed in a similar fashion to conventional reinforced concrete.

The Strut and Tie Method is extremely useful both qualitatively and quantitatively for checking the transmission of the baseplate and anchor bolt forces into the foundation.

(d) Damage to Anchor Bolts Under Seismic Loading

As stated earlier, proper anchor bolt design and detailing seems to be a subject much neglected by the authors of Standards and text books. Yet damage to anchor bolts is a common occurrence during strong seismic events. For obvious reasons, anchor bolts are very difficult to replace when damaged.

For this reason, in Chile, where very large earthquakes occur on a very regular basis, anchor bolts for large structures must be removable 'hammer head' bolts, or the bolts must be designed for fully elastic (μ =1) seismic actions. Therefore, be generous when sizing anchor bolts, particularly when the structure is important, and especially in the more seismically active parts of the country.

In New Zealand, it is recommended practice for rod bracing members with turnbuckles that a minimum length of threaded section (100 mm or so) remain in tension after the turnbuckle is done up. This is to ensure that yielding under seismic loading is not confined to a very short length of threaded section. A similar minimum length of threaded section of anchor bolt capable of yielding in tension can easily be achieved with conventional baseplate and anchor bolt arrangements. A length of threaded section in the upper concrete zone of the foundation can be debonded with tape.

(e) Pre-tensioning of Anchor Bolts for Fatigue Loading

The anchor bolts of some structures, such as large poles and circular towers used to support communications or lighting equipment in windy environments, are subjected to cyclic loading that can cause fatigue failure.

In addition to careful detailing of the bolts themselves, to avoid any stress raisers such as welds in highly stressed areas, the risk of fatigue failure of anchor bolts can be significantly reduced by pre-tensioning them.

Good practice in Britain is to calculate the tension in the anchor bolts due to the 'fatigue load case,' including all prying effects, and then apply a pretension force to the anchor bolts that is one-third greater than the tension calculated for the fatigue load case.

Pretensioned anchors suitable for fatigue loading are most likely to be very high strength stressbars with rolled threads, supplied by specialist manufacturers of prestressing products.

Note that where high tensile anchors are required to be pretensioned, a hardened washer must be placed between the nut and the baseplate (or plate washer if present).

(f) Bolt Threads and Cross-Sectional Areas

The cross section properties most relevant to anchor bolts are the nominal shank area (A_o), the minor diameter (or core) area (A_c), and the tensile stress area (A_s).

The tensile strength of bolts is governed by the threaded section, which has a tensile strength equal to the product of the material ultimate tensile stress and 'the tensile stress area, A_s'. The formula for calculating the tensile stress area is described as "an approximation only, which has been found to correlate reasonably with test results for steels having substantial ductility."

In a typical steel to steel bolt bearing connection, the shear strength of the bolt is dependent on the cross-sectional area of the bolt at the interface of the plates, that is, whether threads are 'included' or 'excluded.' Dowel action in anchor bolts usually means that shear forces must be carried through the threaded section, therefore the shear strength is governed by the 'core area, A_c.'

The following information is given for guidance only, and should be checked against the relevant Standards, particularly when the bolt diameter is in excess of 36 mm. [13].

For coarse metric threads relevant to building structures,

Core area: $$A_c = \frac{\pi(d_1)^2}{4}$$

Tensile stress area: $$A_s = \frac{\pi(d_s)^2}{4}$$

Nominal shank area: $$A_o = \frac{\pi}{4}(d_f)^2$$

where:

A_c = minor diameter area (core area), mm^2
A_s = tensile stress area, mm^2
A_o = nominal plain shank area, mm^2
p = pitch of threads, mm
d_f = nominal bolt diameter, mm
d_1 = basic minor diameter = (d_f – 1.22687 p), mm
d_s = d_f – 0.9382p, mm.

For the typical range of bolt diameters, coarse metric thread pitches are:

Nom. Dia.	Pitch
12 mm	1.75 mm
16 mm	2 mm
20 mm	2.5 mm
24 mm	3 mm
30 mm	3.5 mm
36 mm	4 mm

(g) Rolled versus Cut Threads

Both "rolled" and "cut" coarse metric threads are suitable for typical anchor bolts for structures subjected to 'static' and seismic loads as under direct tensile loading up to bolt fracture, cut threads have virtually identical strength to rolled threads.

Similarly, with pretensioned bolts under fatigue loading, whether the thread is rolled or cut makes no difference (although, in most cases, a proprietary high tensile stress bar with a rolled thread would be used).

(It is only in non-pretensioned fatigue applications, such as for short anchor bolts for machines, that the beneficial effect of the grain structure of rolled and/or forged threads has a noticeable effect).

(h) Grout, Mortar and Drypack

Once the baseplate has been correctly levelled through the use of steel shims or levelling nuts, the gap between the underside of the baseplate and the foundation can be filled with non-shrink cementitious mortar or grout, or, for minor structures, with drypack. "Drypack" may not support the baseplate if it is poorly installed, so judgement must be used when defining a "minor" structure. (It should be noted that the use of levelling nuts is NOT recommended by DCB-56 as they prevent the compression induced on the baseplate by bolt tensioning being transferred to the concrete as intended. The tensioning of the bolts assists in creating a baseplate connection that is as rigid as practicable).

Whether grout, mortar or drypack is used, some form of temporary (timber) dam around the void is required to either contain grout, or to provide a firm surface against which the mortar or drypack can be rammed. Engineers and draftsmen often show on their drawings a 45^o bevel to the side of the mortar bed, projecting down from the edge of the baseplate. Unless there is a specific reason for wanting this bevel, show a simple vertical mortar face at the edge of the baseplate and a better result will be more easily achieved.

A wide variety of proprietary non-shrink cementitious grouts are commercially available. Most can be mixed to a variable consistency; mortar consistency for small baseplates that can have the mortar effectively packed in, or grout consistency for large baseplates or where the gap to be filled is small. Refer to the relevant product catalogues for the properties of specific grouts.

Note that large grouted baseplates require air bleed holes. (DCB-56 suggests that a 50 mm diameter hole is suitable for every 0.5m^2 of base area. It also suggests that if it is intended to place grout through these holes, the diameter should be increased to 100 mm). Some commentators suggest these DCB recommendations are a little excessive. The effect of these holes on the strength of the baseplate must be taken into account.

With regards to economy, it is not necessary to 'over specify' the type of non-shrink cementitious grout. Clearly, the extremely high strength grout used for NMB reinforcing splices would not be needed except for an exceptionally heavily loaded column. Similarly, for most applications, epoxy grouts and mortars are not required. These are costly and more difficult to work with, and their fire resistance must be checked.

Drypack is a more traditional mortar made in the following manner. One part of cement is uniformly mixed with 2½ parts of clean, sharp sand passing a BS No. 14 sieve. Small quantities of water are added and thoroughly mixed in until the mortar can be formed into a ball that will hold its shape, but no more. When the ball of drypack is squeezed, no water is released, but the hand is wet. The drypack is then rammed in place using a hardwood packer hit with a hammer. The quality of drypack in place is directly dependent on the quality of workmanship used in mixing and placing it. Because of this, drypack should only be used for less important and less heavily loaded structures. **The compressive strength of drypack** mixed per the above can be taken as **20 MPa**, but no more. (Note, specialists workers under well supervised conditions can produce drypack with greater strengths, however the recommendation of BCSA [3] is that 15 MPa is to be used in the absence of "special control".).

It should be noted that the recommendation of DCB-56 is that the compressive strength of the grout (when used) should be greater than the compressive strength of the adjacent concrete, that is,

$f'_m > f'_c$

Consequently, the use of drypack will, in most circumstances, not comply with this recommendation and should alert the designer as to whether the structure is sufficiently “minor” to allow drypack to be specified.

Non-shrink cementitious grout mixed to mortar consistency, and drypack, should only be used on relatively small baseplates, and only where sufficient gap exists for proper placement. It is recommended that drypack is not to be used when a plan dimension of the baseplate exceeds 400 mm. For gaps less than 20 mm, a non-shrink grout mixed to a flowable consistency will have to be used instead.

When inspecting the work, poorly mixed and placed drypack can often be detected by picking at the edges with a large steel nail.

(i) Minimum Design Actions on Anchor Bolts

The minimum design shear force shall be equivalent to 0.05 times the maximum design compression axial load acting on the column, as per HERA Structural Connections Guide R4-100 [17]. This is consistent with AS/NZS 1170.0, Cl. 6.2.3. (Note this guide is at variance with R4-100 which does not allow consideration of bending in anchor bolts).

The designer should check that the anchor bolt arrangement also satisfies the torsional restraint requirements assumed in the design of the column.

4.4 Design Strength of Anchor Bolts

(a) General

The design strengths described in this section relate to the anchor bolt itself. The design approach to the anchorage of the anchor bolts in concrete shall be determined in accordance with first principles and relevant references such as NZS 3101. For plain concrete this concerns embedment, anchorage bearing, shear resistance, side cover and bolt spacing effects etc. For reinforced concrete, a “strut and tie” analysis may be required.

The design strengths given in this section are for a single anchor bolt, with the following properties:

d_f = nominal bolt diameter, mm

A_c = core area, mm^2 (refer Section 4.3(f))

A_s = tensile stress area, mm^2 (refer Section 4.3(f))

p = pitch of threads, mm

f_y = bolt steel yield stress, MPa (must comply with Section 2.1.1 of NZS 3404:1997)

f_u = bolt steel ultimate stress, MPa (must comply with Section 2.1.2 of NZS 3404:1997)

ϕ = strength reduction factor for a bolt,

Z_e = plastic section modulus of bolt, mm^3, based on the equivalent diameter, d_s, corresponding to the tensile stress area. For solid round sections, Z_e is limited to 1.5 Z, the elastic modulus.

In accordance with NZS 3404, and Section 4.3(f),

$$\phi = 0.8 \qquad d_s = d_f - 0.9382\,p \qquad Z_e = \frac{1.5\pi\,(d_s)^3}{32} = 0.147(d_s)^3$$

(b) Design Shear Strength

$$\phi V_n = \phi\, 0.62\, f_u A_c$$

(c) Design Tension Strength

$$\phi N_t = \phi\, f_u A_s$$

(d) Design Bending Strength

$$\phi M_s = \phi\, f_y Z_e$$

(e) Interaction Equations

If V*, N* and M* are the ULS actions on a single anchor bolt for a given load case, it is required that:

$$\frac{V^*}{\phi V_n} \leq 1.0$$

$$\frac{N^*}{\phi N_t} + \frac{M^*}{\phi M_s} \leq 1.0$$

$$\frac{V^*}{\phi V_n} + \frac{N^*}{\phi N_t} + \frac{M^*}{\phi M_s} \leq 1.2$$

4.5 Design Method – Anchor Bolts Without Shear Keys – Non Seismic Load Combinations – Shear Transfer by Friction in Mortar Compression Zone

(a) This method applies to ‘static’ loading where the actions on the baseplate result in a bearing compression zone on the underside of the baseplate, through which shear can be transmitted by friction.

(b) The baseplate is assumed to be 'thick' (refer Section 4.3(a)), and the mortar/concrete is assumed to act like a reinforced concrete section, with the anchor bolts acting as tension reinforcement. The compression stress block characteristics are taken to be as per NZS 3101. (In this guide, unconfirned concrete has been assumed, but allowance for confinement may be made).

(c) At the ULS, the Design Friction Factor between the underside of the baseplate and grout/mortar is determined in a manner similar to NZS 3101.

Design friction factor = $\phi\mu_f$

Strength reduction factor ϕ = 0.75

Friction factor μ_f = 0.90 where the base plate is recessed below the concrete surface,

= 0.70λ otherwise,

where λ = 0.70 for non-shrink cementitious mortar or grout,

= 0.55 for drypack.

These λ values provide design friction factors consistent with Japanese and British practice.

Strictly speaking, the underside of the baseplate should be clean, as-rolled steel. However, adequate friction should develop with painted undersides, given the interlock with the sand aggregate of the mortar.

(d) The design method for shear transfer by friction, for both 'pinned' and 'moment resistant' baseplates under uniaxial loads, is:

(i) Assess the governing ULS load case(s), and develop an initial baseplate geometry.

(ii) Based on simple equilibrium of vertical forces and bending moment, determine the zone of mortar in compression, the resultant compression force, and the resultant bolt tension. (Strains may need to be checked to validate any assumptions made as to the stress levels at any point).

(iii) Check that the shear resistance, equal to the product of the Design Friction Factor and the resultant compression force, exceeds the applied shear force.

(iv) Design the anchor bolt assemblies to resist the tension forces.

(v) Check that the baseplate has sufficient strength and stiffness, particularly in bending, to be able to transmit the various actions and reactions concerned.

(vi) The nominal shear stress in the grout due to shear shall be equal to or less than v_{max}, which is the smaller of 0.2 f'_c or 8 MPa. (A re-interpretation of NZS3101:2006 Cl 7.5.2 [2]).

$$\frac{V^*}{\phi A_{cv}} \leq 0.2 f'_c \quad and \quad 8 MPa$$

For rectangular sections and major axis shear:

A_{cv} = bd = effective shear area
b = width of baseplate
d = effective depth, from extreme compression fibre to centroid of tension force

(vii) Repeat Steps (i)-(vi) until a satisfactory design is produced.

If, for any of the ULS load cases, the resultant compression reaction is too small to resist the applied shear force through friction, an alternative mechanism, such as dowel action, will have to be used for that load case (or add a shear key, or set the baseplate in a grouted pocket).

(viii) Note that, in a fashion similar to reinforced concrete:

- A large axial compression force acting in conjunction with a small moment will result in an eccentric reaction that does not require anchor bolt tension for equilibrium,

- A large bending moment with negligible axial load will result in equal total bolt tension forces and compression bearing forces. Additional axial compression load will decrease the resultant total bolt tension force, whereas additional axial tension load will increase the resultant total bolt tension force.

(e) This method of shear transfer by friction is illustrated in the following Figures 1 and 2 whereas Figure 3 shows where shear friction cannot be used because of insufficient compression.

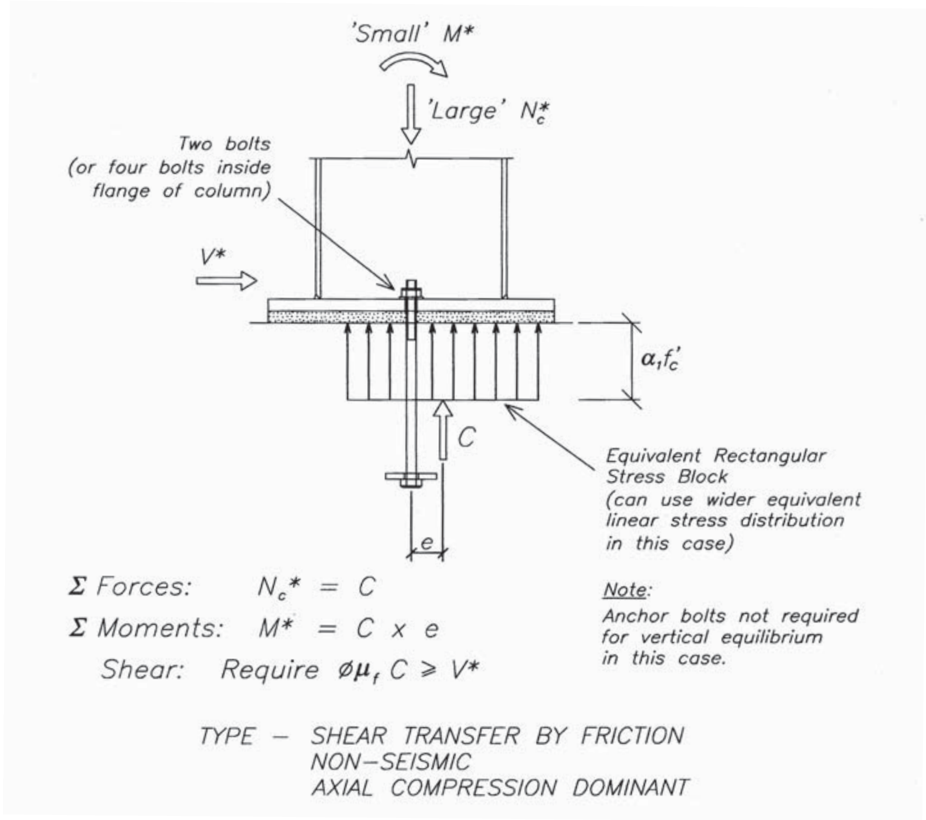


Figure 1.

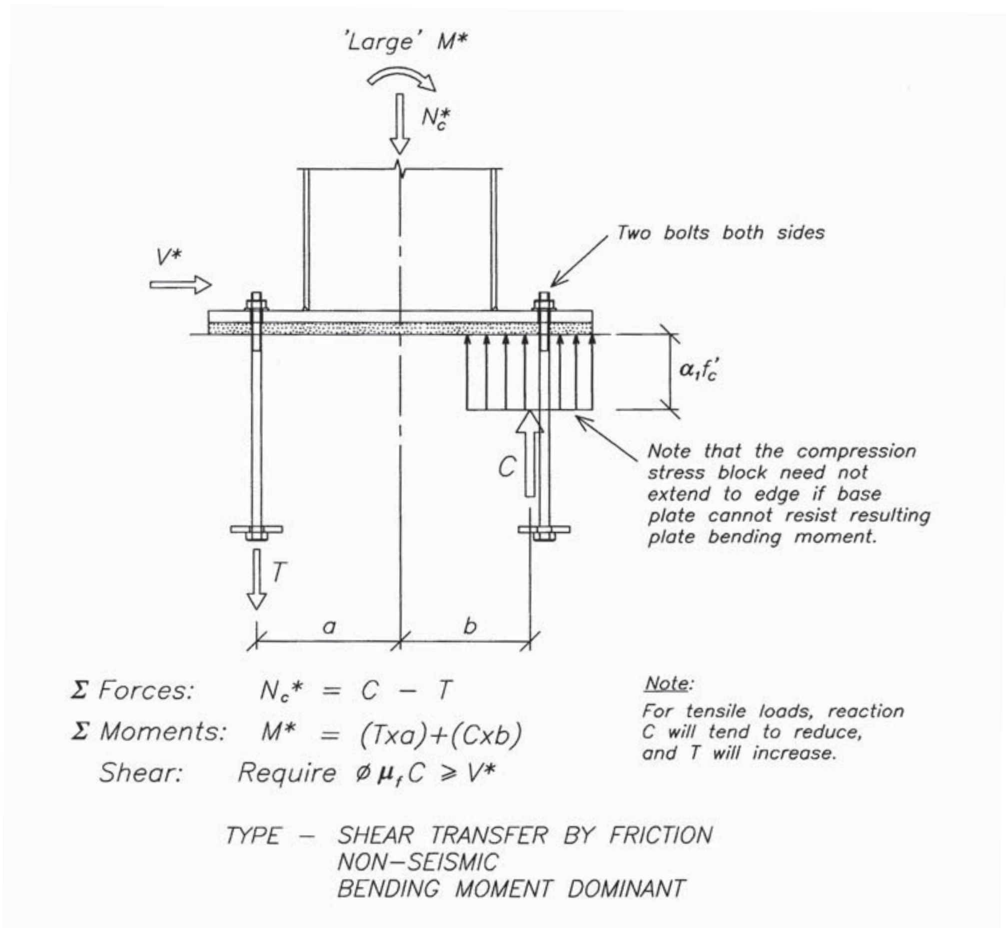


Figure 2.

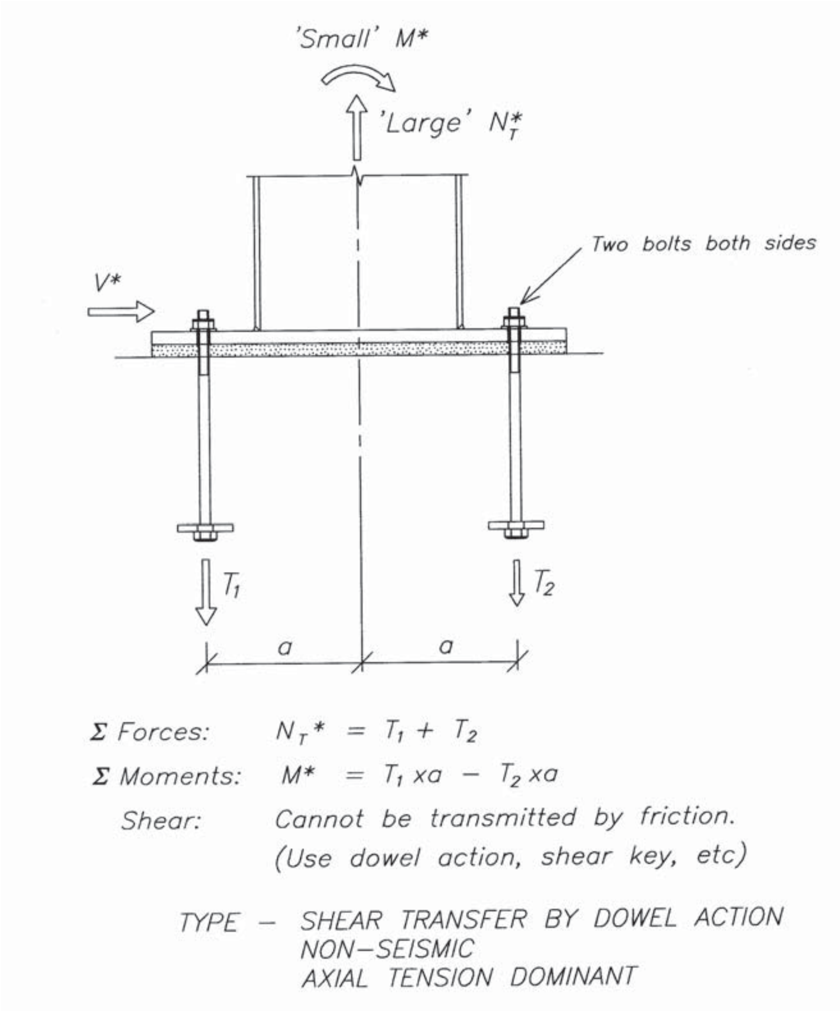


Figure 3.

4.6 Design Method – Anchor Bolts Without Shear Keys – Shear Transfer by Dowel Action of Anchor Bolts

This method applies for ULS seismic loading, and for 'static' type ULS loading where there is insufficient compression reaction to provide the required friction resistance to shear. For these situations the design approach is to assume:

(a) All of the shear is to be resisted by bolts through dowel action. However, because of mortar edge distance considerations, the shear distribution between bolts may be non-uniform.

(b) The baseplate is assumed to be 'thick' (refer Section 4.3(a)), and the mortar/concrete is assumed to act like a reinforced concrete section, with the anchor bolts acting as tension reinforcement. The compression stress block characteristics are taken to be as per NZS 3101.

(c) At the ULS, the dowel action in the anchor bolts should be accounted for as per Section 4.7.

(d) The design method for shear transfer by dowel action, for both 'pinned' and 'moment resistant' baseplates under uniaxial loads, is:

(i) Assess the governing ULS load case(s), and develop an initial baseplate geometry, including initial anchor bolt sizes.

(ii) Based on simple equilibrium of vertical forces and bending moment, determine the zone of mortar in compression, the resultant compression force, and the resultant bolt tension. (Strains may need to be checked to validate any assumptions made as to the stress levels at any point).

(iii) Calculate the shear capacity of the bolts, and the bolt bending moments caused by dowel action, in accordance with Section 4.7.

(iv) Confirm the adequacy of the anchor bolts under combined shear, tension and bending moment.

(v) Check that the baseplate has sufficient strength and stiffness, particularly in bending, to be able to transmit the various actions and reactions concerned.

(vi) Repeat Steps (i)-(v) until a satisfactory design is produced.

(vii) Note that, in a similar fashion to reinforced concrete:

- A large axial compression force acting in conjunction with a small moment will result in an eccentric reaction that does not require anchor bolt tension for equilibrium,

- A large bending moment with negligible axial load will result in equal total bolt tension forces and compression bearing forces. Additional axial compression load will decrease the resultant total bolt tension force, whereas additional axial tension load will increase the resultant total bolt tension force.

(e) This method of shear transfer by dowel action is illustrated in Figures 3 and 4.

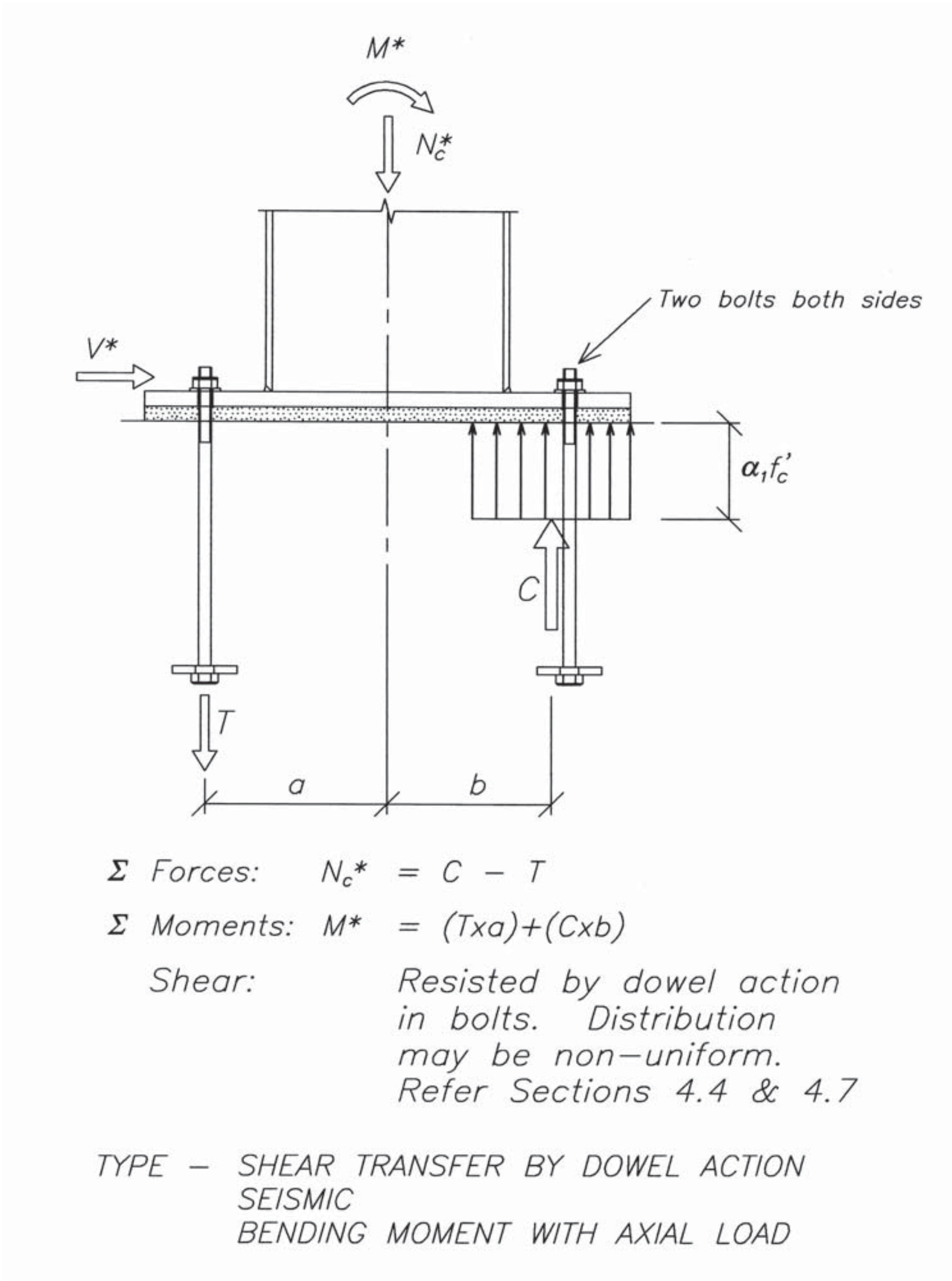


Figure 4.

4.7 Dowel Action on Bolts and Resulting Bending Moments

The principles of the behaviour of anchor bolts transmitting shear forces through dowel action are illustrated in the following Figures 5-8.

Each bolt behaves like a pile under lateral load, subject to bending moments caused by the action and reaction of the eccentric shear force. The eccentricity between the action and the reaction, and hence the magnitude of the bending moment in the anchor bolt, is dependent on the magnitude of the shear force and the strength of the materials, the thickness of the baseplate and the mortar, the direction of loading, and whether the mortar adjacent to an anchor bolt is thick enough in plan dimension to resist the shear force, and the size of the holes in the baseplate, and the washer details.

Clearly, the distribution of the shear force on a baseplate to the anchor bolts acting as dowels depends on the stiffness, geometry and the load carrying capacity of each bolt. This is expanded upon in Section 4.8.

For baseplates transmitting shear through dowel action of the anchor bolts, the following methods of distributing the shear force between equal size bolts are appropriate (refer Fig. 5):

(i) For a given direction of shear force, if the dimension ‘x’ from the centre of the “back” bolts to the adjacent edge of the mortar is at least $6d_f$ [3], then the mortar can be relied upon to resist the shear. Other wise, the shear resistance of the mortar shall be ignored.

(ii) For a given direction of shear force, if all bolts can reliably bear against the mortar, the shear force will be carried equally by all bolts. Given that the “front” bolts will usually be subject to tension forces as well, up to 30% redistribution of shear from “front” to “back” bolts is allowed.

(iii) If the “back” bolts have insufficient edge distance to bear reliably on the mortar, either:

- Distribute shears between bolts based on equal displacements of fixed headed cantilevers of varying heights, or

- Ignore the “back” bolts

(iv) If some or all of the holes in the baseplate are cut grossly oversize (more than 6 mm), then *all* bolts should have close fitting plate washers fitted and site welded to the baseplate. The shear forces can then be distributed in accordance with the above, but with the consequential greater eccentricity between shear force and reaction. (Refer Figure 8 (c),(d) – g as shown).

Referring to Figs. 6, 7 and 8, the design bearing stress for the steel baseplate bearing against the bolt is simply $\phi \times 3.2 f_u$, where $f_u = \min(f_{u\ plate}, f_{u\ bolt})$. Given that $\phi = 0.9$, this gives a design bearing stress of $2.9 f_u$.

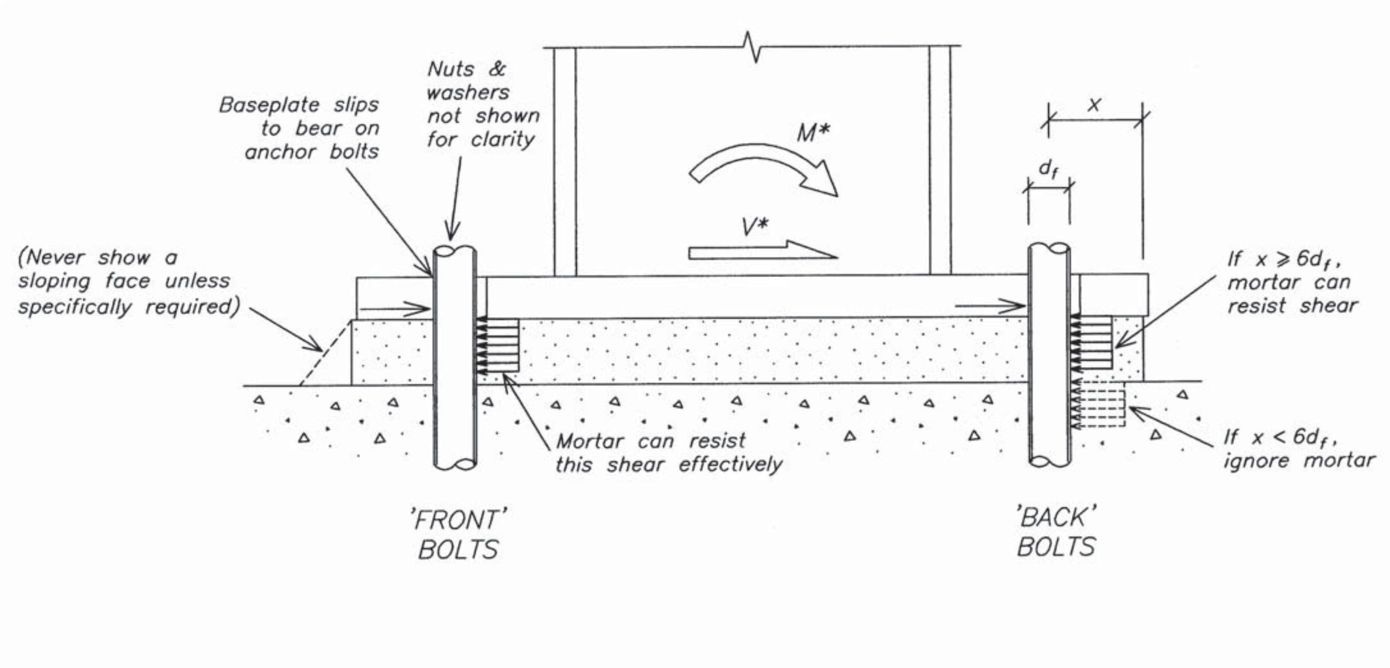


Figure 5. - Problems with Lateral Bearing of Bolts on Mortar

For lateral bearing of the bolt against mortar, similar to British practice as described above, if the edge distance in the direction of shear (x) is greater than or equal to $6d_f$ the design bearing stress is $2f_m'$.

If the edge distance in the direction of shear (x) is less than $6d_f$ the mortar is ignored.

The design lateral bearing stress against concrete has been determined from a back analysis of the bearing stresses developed by headed shear studs in solid slabs at the design shear load. Taking into account the confinement that may or may not be available from the baseplate and mortar, the following design lateral bearing stresses apply:

Edge distance (x) in direction of shear $\geq 6d_f$, the design bearing stress is $4f_c'$.

Edge distance (x) in direction of shear $< 6d_f$, the design bearing stress is $2f_c'$.

As stated previously the engineer must ensure that the foundation has sufficient size, edge distance and reinforcing to resist these shear forces.

These methods assume uniaxial actions, there is no torsion moment on the baseplate, and all bolts are of the same strength and diameter. Appropriate modifications to the method can be made to account for biaxial actions, torsion and varying bolt strength and size, based on first principles.

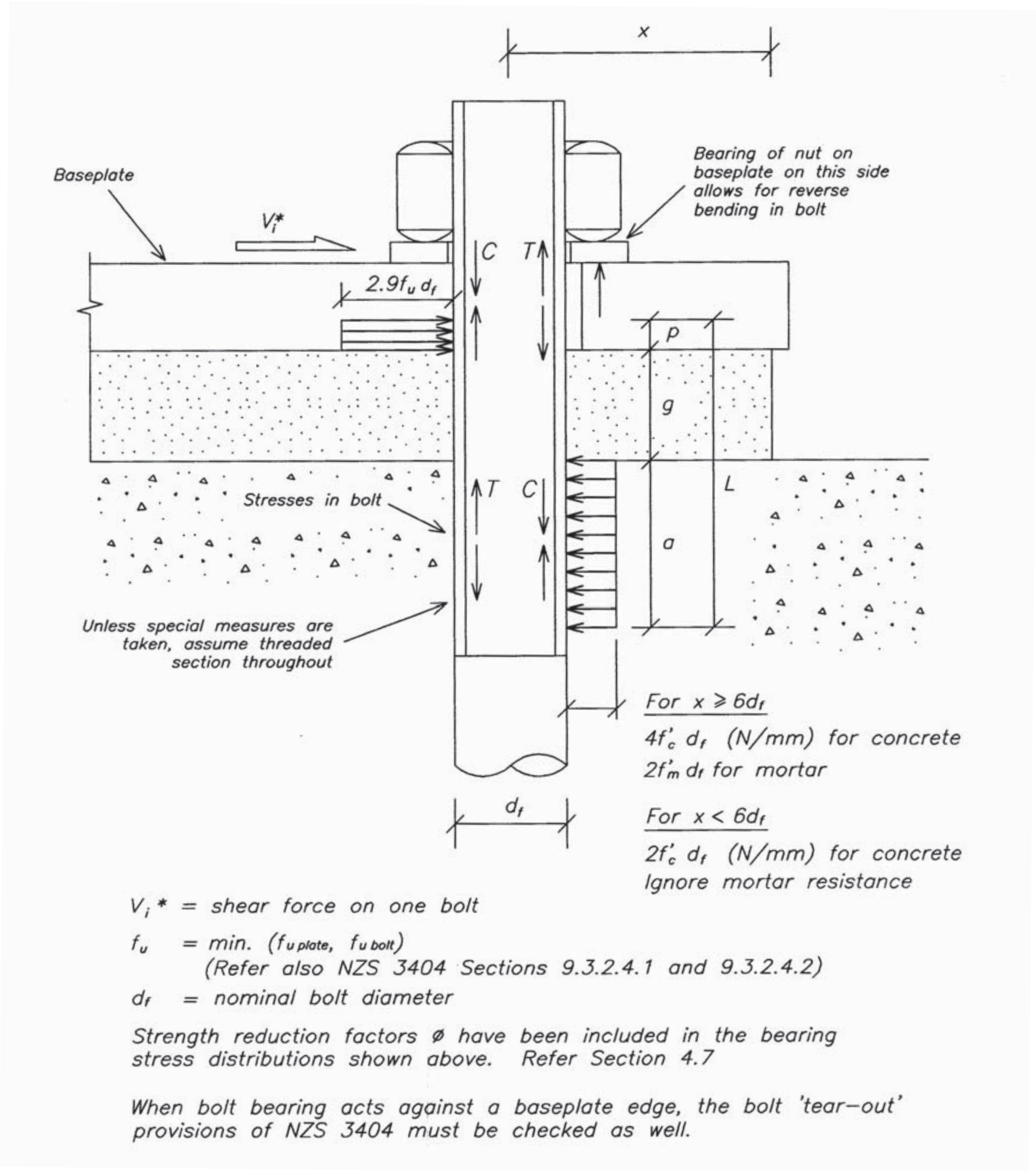


Figure 6. Detail of Bolt Bearing on Baseplate and Concrete, and Bolt Bending

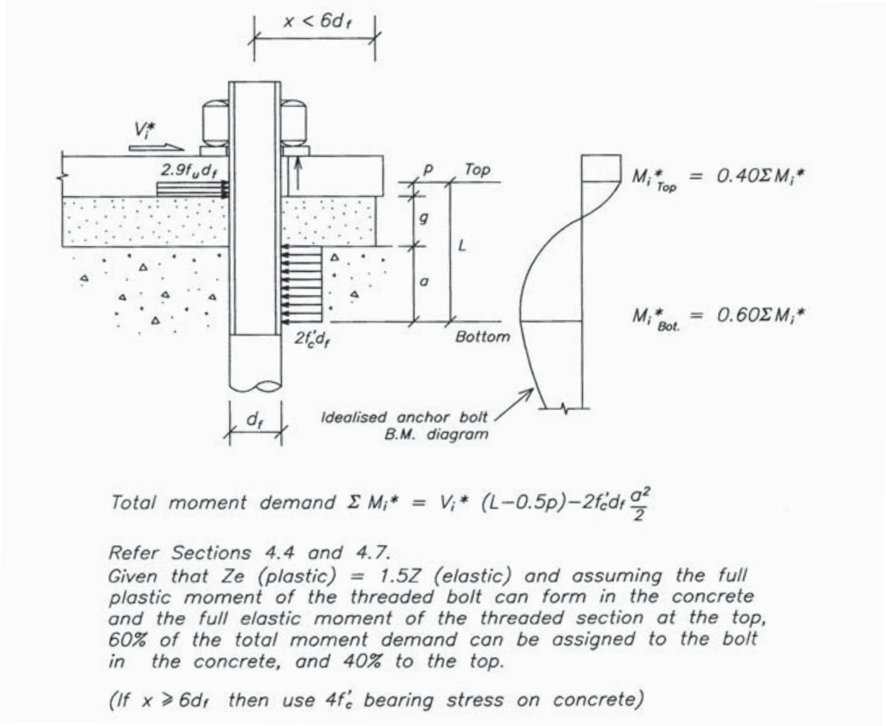


Figure 7. Calculation of Bending Moment in Anchor Bolt

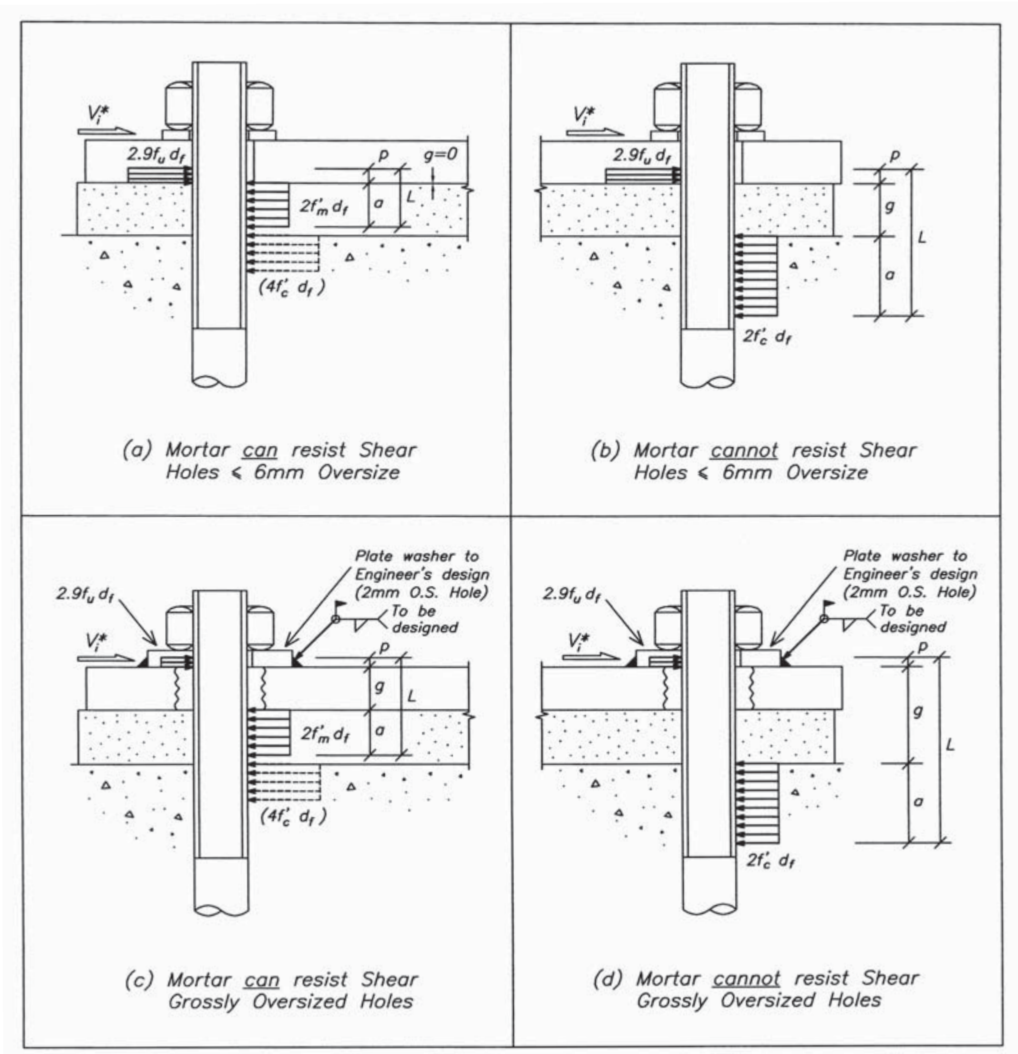


Figure 8. Determining the Gap 'g'

4.8 Compatibility of Displacements for Various Mechanisms

As with all structural systems, the distribution of load to various elements must be based on force equilibrium, but also stiffness, and compatibility of displacements.

The engineer must ensure that the distribution of the load to each anchor bolt assumed in the design is consistent with these principles.

For example, in one very heavily loaded baseplate, a large number of large diameter anchor bolts were found to have insufficient strength to resist a very large tension and horizontal reaction from the seismic resisting bracing. A shear key was added to resist the 'excess' of shear that the bolts could not. However, in this case the shear key, because of its greater stiffness, would carry all of the shear force until it fractured, leaving the anchor bolts overloaded under the full shear force.

Figure 9 illustrates one compatibility issue involving a laced strut. The bottom strut solution can be used to distribute the shear to where dependable friction can resist it.

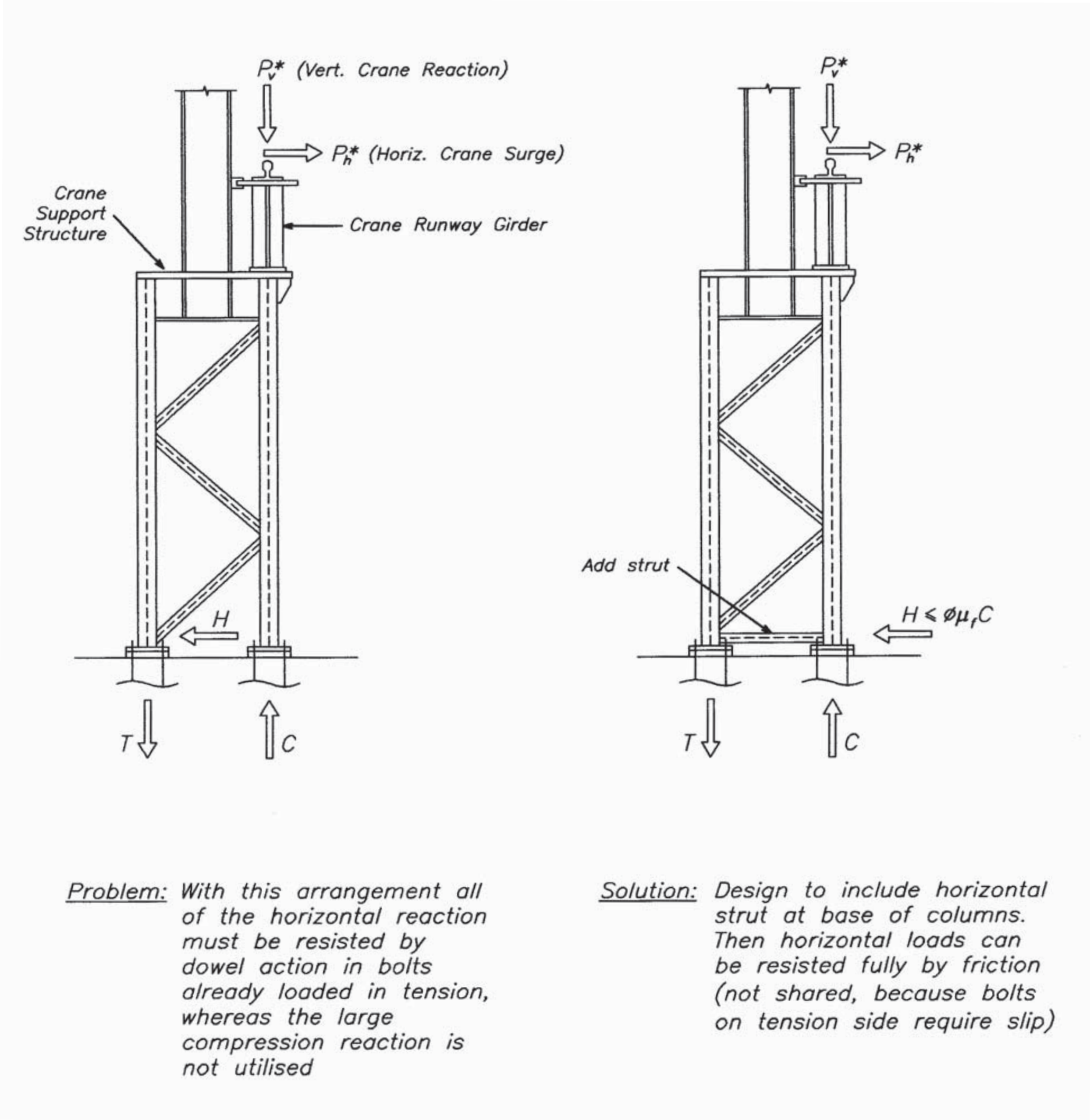


Figure 9. Example Illustrating Need to Consider Compatibility of Displacement in Bolts

4.9 Design Suggestions – Shear Transfer at the Base of Special Structures by Friction Due to Pretensioned Stress Bars

(a) For certain special types of structure, such as large cantilever pole communications towers exposed to high wind speeds, the magnitude of the tension forces due to bending that act at the base may require the use of large numbers of very high tensile strength bars.

To improve the service performance of such a baseplate arrangement, particularly for fatigue loading, it is standard practice to pretension these high tensile bars.

This pretensioning serves as a good test of their anchorage, and also provides a dependable clamping force that can transfer shear through friction.

Refer Sections 4.3(e), Table 2d Item 7, and Table 3c Item 6 and [16].

Note that large pole towers may have full baseplates similar to those shown for the columns in Sections 4.5 and 4.6, but they are more likely to have annular baseplates as shown in Table 3c Item 6.

Full circular baseplates behave in a similar fashion to the baseplates shown in Sections 4.5 and 4.6, and it can be seen that bending moments due to tension and compression forces acting on the baseplate outstands can be resisted by the continuous baseplate.

However, annular baseplates are much more complex. As can clearly be seen in Table 3c Item 6, significant prying actions are inevitable, and the thick baseplate cannot simply 'cantilever' off the much thinner, and already highly stressed, wall section.

For annular baseplates, the full three dimensional behaviour of the ring, and the finite size of all compression zones must be considered. Refer to HERA DCB-65 [16] for comprehensive guidance.

Note that for both full circular baseplates, and annular baseplates, stiffeners must be placed close to anchor bolts to be effective. Also, the proper design of stiffeners such as those shown in Table 3 Item 6, especially with regard to the forces they exert on the wall section, can be very complex. One simplifying and strengthening detail is to add an external annular ring at the top of the stiffeners, so that stiffener bending moments can be resolved into a couple, each force resisted by its own ring.

In the absence of net axial tension loads the total shear that can be transferred by this type of baseplate at the ULS is simply the product of the design friction factor, as given in Section 4.5(c), and the pretension force that remains after all losses. The capacity of the mortar and concrete to transmit these shear forces should be checked in a similar manner to Section 4.5 (vi), using a rational assessment of the available 'shear area.'

5.0 FABRICATION AND INSTALLATION OF ANCHOR BOLTS

5.1 Materials Suitable for Anchor Bolts

Indicative information on the various materials that are suitable for use as anchor bolts is shown in Table 2, subject to the restrictions noted. The designer should always check on the precise range of materials and sizes available at the time of each project, including delivery times and precise material properties. Note that for most steel grades, the thicker sections have slightly lower yield stresses, due to their slower cooling rates during casting and rolling. Similarly, some higher strength materials may have variable strength through the cross section, with higher strength material at the surface. This material may be cut away during machining, reducing the average yield strength of what remains.

Ensure when specifying that anchor bolts and nuts be machined from high strength material that the material has notch toughness and elongation at least as good as Property Class 8.8 material.

Clearly, in the large majority of projects in NZ, anchor bolts fabricated from commercially available Property Class 4.6 bolts and studding or mild steel rounds, will be the norm.

In general, as noted in Table 2, threaded sections should not be bent, and bolt material with yield stresses above approximately 400 MPa should not be bent, welded, tack welded, or damaged with direct heat or hot metal. All welding must comply with AS/NZS 1554 [19].

When engineers have to assess existing structures, particularly when changes to loads or layout are proposed, they commonly find that drawing records are available, from the owner of the structure, or the Territorial Authority, but specifications and the like cannot be found. Therefore, the structural drawings for new designs should include a brief summary of floor design loads and the steel grades used in the structure, including the material used to fabricate the anchor bolt assemblies.

Where high strength material that should not be welded is used, measures should be taken to ensure that anyone attempting future modifications cannot miss this requirement.

Not all of the steel grades mentioned may be hot dip galvanised. Refer Section 5.4 for guidance.

The following terminology is used in this section:

'Property Class' is used to describe a commercially available bolt or fully threaded stud manufactured in accordance with the Standards listed in Section 1.2 (or similar),

'Grade' is used to describe plain (round) steel that may be machined into bolts and studding.

When using Standards such as those listed in Section 1.2 to assess the requirements for machined bolts, such as matching nut grades, the Engineer shall treat the machined steel as the equivalent Property Class, based on its mechanical properties.

Table 2a. Material suitable for anchor bolts

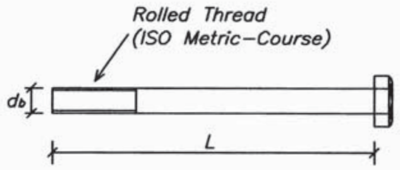
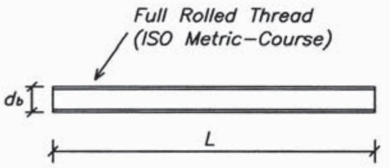
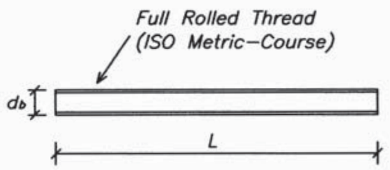
MATERIAL	WARNING	NOTE
1. 'Commercial' Bolts Property Class 4.6 		Weldable f_y = 240 MPa f_u = 400 MPa Typically, $d_b \leq$ M64 $L \leq$ 400mm Note that for PC4.6 bolts, the nuts are typically Class 5. Check weldability if nuts are to be welded (typ.)
2. Property Class 4.6 Studding 	No bend	Weldable f_y = 240 MPa f_u = 400 MPa Typically, $d_b \leq$ M36 $L \leq$ 3.0m
3. Property Class 8.8 Studding 	No weld No bend	f_y = 640 MPa f_u = 800 MPa Typically, $d_b \leq$ M36 $L \leq$ 1.0m

Table 2b. Material suitable for anchor bolts

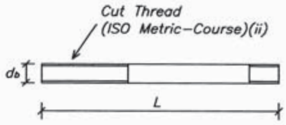
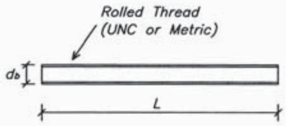
MATERIAL	WARNING	NOTE
4. Mild Steel Rounds 		Grade 300 round, up to 90mm dia. and 6m long, is available from Australian mills. Grade 250 round, up to 8" dia. (203mm) and larger, is available from Australian and Asian mills. Note: (i) Check availability of commercial nuts, otherwise these will have to be fabricated. (ii) Consider use of American UNC threads for very large diameters – nuts may be available ex–USA.
5. B7 Studding & 2H Nuts 	No weld No bend	B7 studding is one of several grades of high strength studding similar to AISI 4140 (see below) used in the petrochemical industry. Available in New Zealand from Croucher & Crowder Engineering Services in Hawera, and EDL Fasteners Ltd. Typical sizes available: UNC ⅜" x 1.6m UNC ½" to 2" x 3.65m M16, M20 & M24 Stud material grade: ASTM A193 Grade B7 To 2 ½": f_u = 860 MPa f_y = 730 MPa (0.2% Proof Stress) To 4": f_u = 790 MPa f_y = 660 MPa (0.2% Proof Stress) Nut material grade: ASTM A194 Grade 2H

Table 2c. Material suitable for anchor bolts

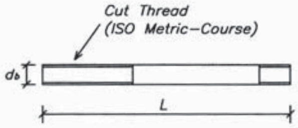
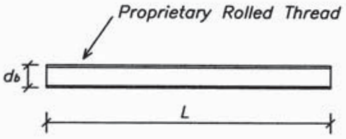
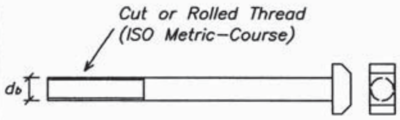
MATERIAL	WARNING	NOTE
6. AISI 4140 rounds 	No weld No bend	Extensively used throughout the world in a wide variety of industries. Available in very large sizes (d_b >500mm, L ≥ 8m) worldwide, but NZ stocks more limited. AISI 4140 is a medium–carbon low–alloy hardenable steel that has a martensitic microstructure, which is tempered to improve ductility and toughness. As a consequence, the UTS and Yield Strength can vary, but are of the order of 860 MPa and 720 MPa respectively. Check strength throughout the thickness of the cross–section – is surface stronger than core? As regards conventional structural fabrication techniques, AISI 4140 steel cannot be welded or bent. Refer Sections 5.3 and 5.4 regarding issues with hot dip galvanising. Refer Section 5.5 as to why advice on sizing of nuts made from AISI 4140 material, as given in HERA DCB No.51 and No.52 should not be followed.

Table 2d. Material suitable for anchor bolts

MATERIAL	WARNING	NOTE
7. High Tensile Stressbars 	No weld No bend No HD Galvanizing	Typically, $d_b \leq$ 73mm $f_u \cong 1000$ MPa $f_y \cong 800-900$ MPa (0.1% P.S.) High tensile fully threaded stressbars are manufactured by prestressing specialists such as VSL, BBR, Macalloy and Dywidag. Stressbars are particularly suited to anchoring large, wind exposed circular towers and monopoles, where large tension forces due to base moment are present, and significant pre–tensioning is required to prevent fatigue failure (refer Section 4.3(e))
8. Hammer Head Bolts 	Check weldability	These are a special type of 'removable' anchor bolt designed to be fitted within unfilled tubes and sockets in foundations to anchor main building columns and large pieces of plant, especially in severe seismic situations, in combination with double baseplates. (Refer Table 3b Item 4) After an earthquake, the exposed section of thread can be inspected for elongation and damage. If required, the anchor bolts can be removed and replaced. One disadvantage is the relatively large rectangular holes that must be formed in the baseplates to allow for the hammer head end to be removed. Hammer head bolts are manufactured in Germany, typically with f_y =235 MPa. There is no reason why they could not be forged and machined from stock bar in NZ (check patents).

5.2 Anchor Bolt Assemblies

All anchor bolts should be cast into reinforced concrete foundations, except for minor structures and structural modifications to existing foundations, where drill in fixings may be appropriate.

If polyester, epoxy or similar adhesives are to be used, the designer must note that:

(a) The recommendations and specifications of the supplier are to be complied with.

(b) Proper drilling and thorough clean out of the holes is essential. This may be achieved by brushing the holes with a stiff nylon brush then having them blown out with clean compressed air.

(c) Polyester and some other adhesives (including some common 'capsule' systems) cannot be used if the concrete is green. Ensure that there will be time for adequate curing of the concrete before the use of a chemical anchor.

(d) For most modern chemical anchors, the differences between the coefficients of thermal expansion of the chemical anchor and the concrete are not expected to create any adhesive issues. There may have been difficulties in the past, so the designer is reminded to consider this issue.

(e) Polyesters are thermoset polymers and will soften when heated. These may not provide sufficient anchorage during a fire. Check the allowable temperature range for the adhesive you have chosen and ensure that it is suitable.

(f) Ensure that creep is not an issue and that the adhesive is capable of sustaining long term loads and/or dynamic shock loads as appropriate. (Note that the collapse of precast concrete 'ceiling' panels in the Boston 'Big Dig' tunnel was due to the inability of the epoxy adhesive to sustain long term loading.)

(g) Adhesive anchors may not be suitable for use in cracked concrete.

There are numerous instances recorded in NZ where, due to poor on site installation, adhesive fixed rods and bars have been pulled out by hand and expansion type anchors have slid out of their holes under very low load. Good workmanship is essential.

5.3 Bolt Threads

Bolt and nut threads are designed to enable fracture of the bolt through the tensile stress area of the bolt before nut fracture or thread stripping occurs.

A close examination of the relevant bolt and nut standards will show that for a given bolt material, the matching nut material will be 'stronger,' but 'softer.' That is, the ultimate tensile stress of the nut will be higher than that of the bolt, but the hardness will be less.

This higher nut strength is to ensure the hierarchy of failure given above, but the nut material must be softer to allow it to flow as the nut is tightened to give uniform bearing on each thread interface.

5.4 Hot Dip Galvanising (When Required)

(a) If corrosion protection in the form of hot dip galvanising is required, then care must be taken that steel embrittlement associated with the galvanising process does not occur. There are two forms of embrittlement that are most relevant to anchor bolt assemblies, *hydrogen embrittlement and strain age embrittlement*.

For all materials stronger than Property Class 4.6 or Grade 300, check with the material supplier's and the galvaniser's technical staff that the steel may be pickled and hot dip galvanised, and any precautions or additional treatments that may be required.

(i) Hydrogen embrittlement is caused by the presence of hydrogen atoms within the crystal lattice structure of the steel. In the usual galvanising process, hydrogen is absorbed by the steel from the acid bath during the pickling (cleaning) process.

Steels with an ultimate tensile stress in the order of 1000 MPa and higher, or an equivalent surface hardness of 30 Rockwell C or higher are considered to be most susceptible to hydrogen embrittlement. Hence, Property Class 8.8 bolts may be hot dip galvanised, but stronger material like Property Class 10.9 or ASTM A490 cannot.

AISI 4140 material is slightly stronger than Property Class 8.8. With cut threads, it may be hot dip galvanised, but with rolled threads, which have hardened material at the root of the thread, it cannot. However, given that anchor bolts fabricated from AISI 4140 material will typically also have their nuts fabricated from the same material, and given the issues raised in (b) below, and Section 5.5, a corrosion protection system for AISI 4140 other than hot dip galvanising should be used.

(ii) Strain age embrittlement is associated with strain that results from plastic deformation, or cold working, in other words. The ageing process is a function of temperature, and occurs very rapidly at the 450-460^{o}C temperatures of the galvanising process. Severe cold working of steel in general can be caused by hole punching, tight radius bending or rebending

With regard to anchor bolts:

- Do not bend anchor bolts with f_y > 400 MPa
- Do not bend threaded sections
- If using U, J or L shaped bars as anchor bolts, bend after galvanising.
- Use bend radii larger than normal code minima,
- Consider hot bending of the bars,
- Anneal at 650-815^{o}C prior to galvanising.

(b) A close examination of the relevant bolt and nut standards will show that when the bolt and nut are galvanised, the thickness of the galvanising layer is accommodated in the following manner:

(i) the bolt thread remains unchanged,

(ii) the nut thread is over-tapped to allow for the zinc layer,

(iii) the galvanised nuts are made from a slightly higher strength material than plain nuts, to account for the reduced nut thread size.

Ensure that the fabricator does not overcut the bolt thread to accommodate the zinc layer if the bolts are to be galvanised.

Except for very large bolts, hot dip galvanising of bolts, nuts and washers should be done using spin galvanising. The fasteners are put in a basket before being dipped in the molten zinc bath, and are then spun to remove excess zinc from the threads.

For large bolts, excess zinc can be cleaned off with a tapping nut, etc.

5.5 Washers, Nuts and Lock Nuts

(a) When holes in baseplates are drilled well oversize, or are further enlarged on site to aid erection, the washers used under the nuts of anchor bolts will be 'plate' washers fabricated from steel plate. Refer Figure 8.

Should high tensile anchor bolts be required to be pretensioned to any significant degree, such as to resist fatigue loading, additional hardened washers are to be located between the nut and plate washer.

(b) An inspection of manufacturers' tables of bolt and nut geometry and properties, for Property Classes 4.6 and 8.8 will show that, as mentioned above, the nuts are stronger (but softer) than the bolts, and that for the same size bolt, Property Class 8.8 nuts are larger than Property Class 4.6 nuts.

Typically, Class 5 nuts are supplied with Class 4.6 bolts. The ratio of Class 5 UTS to Class 4 UTS equals 1.30. Also, for Property Class 8.8 bolts, the ratio of nut UTS to bolt UTS equals 1.30.

Therefore, if machining nuts from similar material to the bolt (which may occur with very large bolt sizes), ensure that the nut material is no harder than the bolt material, and the following allowance should be made in the nut geometry:

(i) For Grade 250 to Grade 450 steel, scale up the nut sizes for Property Class 4.6 bolts (nut width and height) by the ratio (≥ 1):

$$\frac{1.30}{(f_{u\,Nut} / f_{u\,Bolt})}$$

(ii) For AISI 4140 steel, scale up the nut sizes for Property Class 8.8 bolts (nut width and height) by the ratio (≥ 1):

$$\frac{1.30}{(f_{u\,Nut} / f_{u\,Bolt})}$$

(c) Class 8.8 nuts up to M42 and M48 may be able to be sourced ex-stock in Australasia. Class 8.8 nuts can be used with anchor bolts fabricated from AISI 4140, although the bolt strength is somewhat higher than that for Class 8.8 bolts.

Consideration could also be given to sourcing large A325 nuts with UNC threads from the US. (A325 is equivalent to Property Class 8.8).

(d) Note that the advice given in HERA DCB No.51 *and* No.52 regarding the sizing of nuts for AISI 4140 material *should not be followed*, because it does not make allowance for the ratio of UTS of the materials concerned.

(e) Where there is a possibility of nuts loosening in service, but pretensioning is not appropriate, heavy duty spring washers may be located between the nuts and plate washer, or additional lock nuts should be used.

5.6 General Comments on Detailing, Installation and Set Out

The correct design and installation of anchor bolts is one part of the construction process that is not accorded the importance it deserves, yet anchor bolts are difficult to rectify should anything go wrong. However, proper thought at the design stage can alleviate many construction problems, and the time that contractors should spend in properly locating anchor bolts, and checking their position before they are cast in, is a very sound investment. Having to 'correct' baseplates to accommodate incorrectly cast in anchor bolts is an activity steel fabricators are all too familiar with, but wish to avoid.

If anything can go wrong with the installation or set out of anchor bolts, it most likely will. Therefore, it is best to simplify anchor bolt layouts as much as possible.

Wherever possible, ensure that all bolt groups are centred on grid lines, or well clear of grid lines. No matter how well detailed and drafted, bolt groups with small offsets from grid lines tend to be cast onto grid lines, or on the wrong side. Avoid bolt groups with non-symmetric arrangements; they may well be cast in the wrong way round. Avoid bolt groups that have almost the same bolt centres in each direction; their installation may well be out by a quarter turn.

At the design stage, give careful thought as to how the anchor bolts are to be supported securely in their correct position during concrete pours, and how they are to avoid clashing with the reinforcing steel, and vice versa.

Also give consideration at the design stage as to how the structural columns are to be set level and vertical prior to the grouting of the baseplate. Are the columns to be seated on central steel shims, or must the mortar thickness accommodate levelling nuts, and their adjustment?

We suggest that the Engineer include in the job specification a clause similar to the following:

'After the anchor bolts are installed and firmly held in position, but prior to the pouring of the foundations, the Contractor shall have the anchor bolt positions surveyed by a suitably qualified surveyor experienced in this type of construction. The survey information shall be provided to the Engineer and the structural steel Sub-Contractor. All anchor bolts that are positioned outside the tolerances for this project (as per this specification and the contract drawings, or, in lieu of this, NZS 3404) shall have their position corrected prior to the pour, or alternative corrective measures shall be taken, as approved by the Engineer in writing before the pour.'

Table 3 contains useful examples and information on a range of anchor bolt types.

Table 3a. Useful Examples & Information

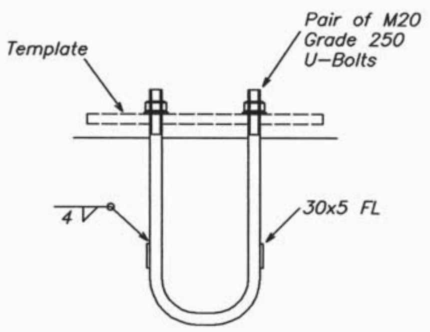
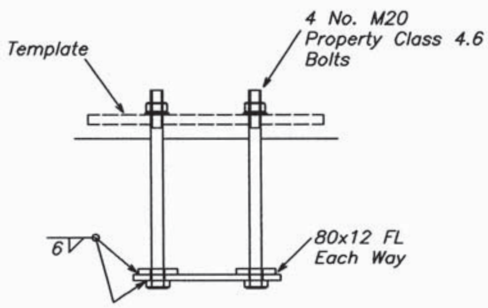
USEFUL EXAMPLES & INFORMATION	COMMENTS
1. A Typical U–Bolt Arrangement 	The flats welded between the pairs of U–bolts (clear of the bends) ensure a rigid assembly that can be accurately held in place during the concrete pour through the use of a template fixed to formwork. Ensure bend radii comply with minimum standard requirements. If steel is bent before hot dip galvanising, increase bend radii to avoid embrittlement – refer HD galvanising standards.
2. A Typical Straight Anchor Bolt Arrangement 	The pairs of flats welded between the bolts are sized to ensure proper mechanical anchorage through bearing on the concrete. They also hold the bolts in a rigid assembly that can be accurately held in place during the concrete pour through the use of a template fixed to formwork.

Table 3b. Useful Examples & Information

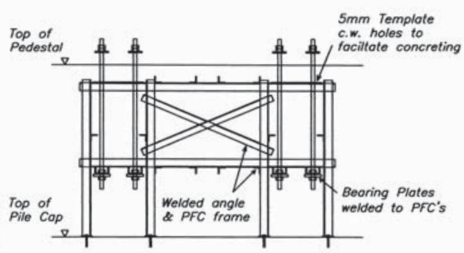
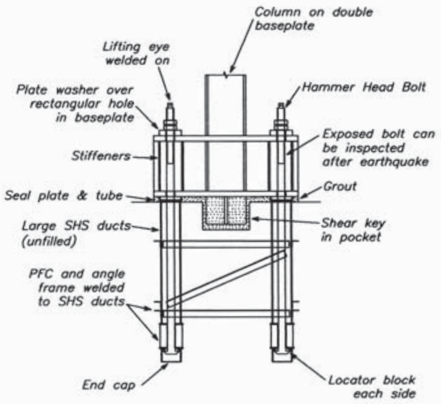
USEFUL EXAMPLES & INFORMATION	COMMENTS
3. Bolt Frame for Heavy Industrial Column 	The steel frame allows the anchor bolts to be accurately located relative to each other, and the entire assembly can be accurately and firmly fixed on site.
4. Hammer Head Anchor Bolts & Double Baseplate for Heavy Industrial Column 	The double baseplate used in conjunction with the hammer head bolts allows for relatively easy bolt replacement should they be stretched or otherwise damaged in an earthquake.

Table 3c. Useful Examples & Information

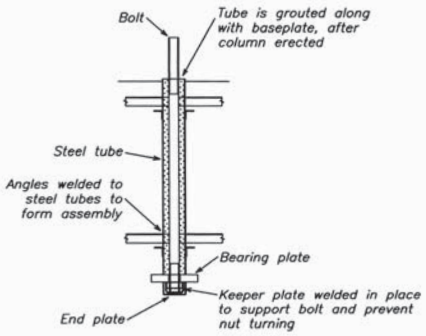
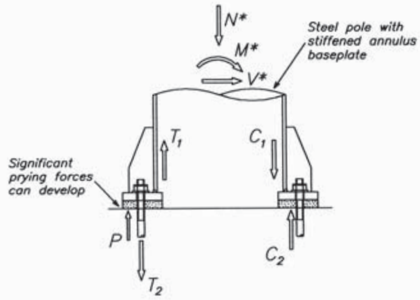
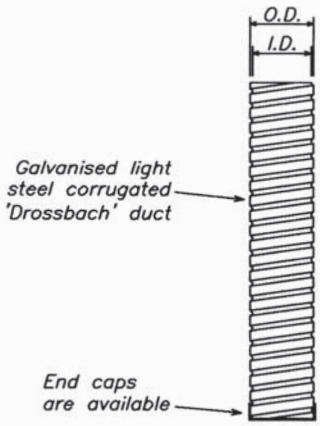
USEFUL EXAMPLES & INFORMATION	COMMENTS
5. Tube Around Heavy Bolt Allowing Some Adjustment During Column Erection 	Tubes such as this are often requested on large industrial projects, to allow adjustment during the erection of the columns. The amount of adjustment is limited, and is no substitute for correct drawings, accurate site setout and rigid fixing in place.
6. Annular Steel Pole Baseplates 	Annular baseplates like this are relatively complex to analyse properly. Significant prying forces can develop. Refer DCB–65 for design guidance. Refer Section 4.3(e) regarding the pre–tensioning of anchor bolts to resist fatigue loading.

Table 3d. Useful Examples & Information

<table>
<tr><th>USEFUL EXAMPLES & INFORMATION</th><th>COMMENTS</th></tr>
<tr><td>

7. <u>'Drossbach' Ducting</u>



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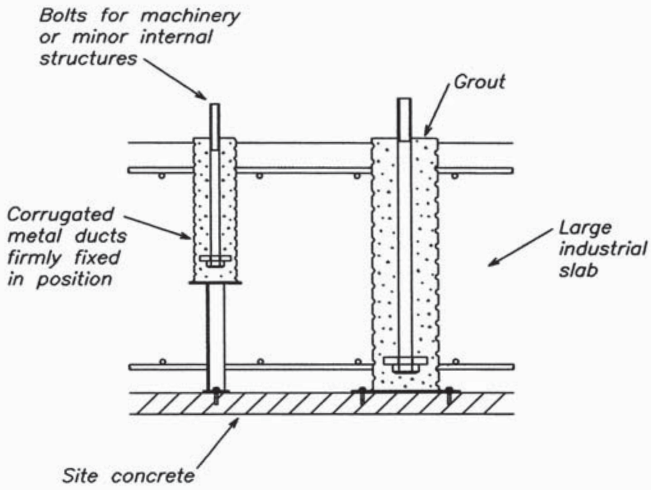
Corrugated ducts fabricated from thin galvanised steel are stocked by most prestressing specialists, for use as ducts for prestressing cables.

Diameters up to 114mm O.D. are usually stocked.

</td></tr>
<tr><td>

8. <u>One Application for Drossbach Ducts – Machinery Bolts in Large Industrial Floors</u>



</td><td>

One advantage of this approach is that all of the anchor bolts for one piece of machinery can be attached to the machine, and the bolts are located precisely with the alignment of the machine. Grouting of the ducts occurs as the machine base is grouted.

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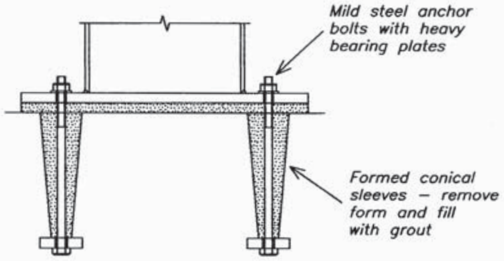
</table>

Table 3e. Useful Examples & Information

USEFUL EXAMPLES & INFORMATION	COMMENTS
9. Sleeved Anchor Bolts 	Conical sleeves can be formed using timber pieces or waxed cardboard tubes. These allow some adjustment after the concrete pour. Project cones above concrete to allow removal. Refer [20] for additional information. However, solvent dissolved polystyrene should not be used – toxic to environment. Steel bearing plates must be large enough to engage concrete outside formed cone and thick enough for the resultant bending. Clearly, the concrete in the formed cone will easily pull out.

6.0 REFERENCES

1. NZS 3101:1995 Concrete Structures Standard, Standards New Zealand.

2. NZS 3101:2006 Concrete Structures Standard, Standards New Zealand.

3. Joints in Steel Construction – Moment Connections, The Steel Construction Institute, in association with The British Constructional Steelwork Association Ltd.

4. Design Standard for Steel Structures 1979, The Architectural Institute of Japan.

5. NZS 3404:1997 Steel Structures Standard, Standards New Zealand.

6. AS/NZS 1170:2002 Structural Design Actions, Standards New Zealand.

7. AS/NZS 1111 ISO metric hexagon commercial bolts and screws.

8. AS/NZS 1112 ISO metric hexagon nuts, including thin nuts, slotted nuts and castle nuts.

9. AS/NZS 1252:1996 High-strength steel bolts with associated nuts and washers for structural engineering.

10. AS 1214-1983 Hot-dip galvanised coatings on threaded fasteners (ISO metric coarse thread series).

11. AS 4291.1-2000 Mechanical properties of fasteners made of carbon steel and alloy steel, Part 1: Bolts, screws and studs.

12. AS/NZS 4291.2:1995 Mechanical properties of fasteners – Nuts with specified proof load values – Coarse thread.

13. Bolting of Steel Structures – Third Edition, Australian Steel Institute (then Australian Institute of Steel Construction).

14. DCB- 55 HERA, Shear studs in composite construction: concepts involved in determining the shear stud design shear capacity, April 2000.

15. DCB-56 HERA, Design concepts for Moment-resisting column baseplate connections in steel frame seismic-resisting systems, June 2000.

16. DCB-65 HERA, Design of Circular Bolted Flange Annulus Connections, Dec. 2001.

17. R4-100, HERA Structural Connections Guide.

18. NZS3109:1997 Concrete Construction, Standards New Zealand.

19. AS/NZS 1554 Welding, Standards New Zealand.

20. Steel Advisor ERC1004 - Sleeved Holding Down Bolts - 29 June 2009, SCNZ.