Section G

CONNECTION DESIGN

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This paper is the result of deliberations of the Society's Study Group for the design of STEEL STRUCTURES.

CONTENTS

1. SCOPE

1.1 General

This paper describes various types of connections used for the transfer of seismic loads in steel structures and their method of design under seismic attack. The main areas of work include:

- the connection in a braced structure
- the beam to beam connection (at right angles)
- the holding down connection
- the splice connection (either beam or columns in line)

Design procedures and detailed discussion is confined to the fastener be it weld, bolt or rivet under the action of seismic loadings, i.e. being required to sustain the imposed design loadings as appropriate to the selected performance or failure criteria of the structure.

Exclusions to the material covered by this paper are:

- the beam column connection (covered in Ref.(11))
- power-actuated fasteners

Some comment is made about rivets and turnbuckles. All recommendations in this paper are based on strength design methods.

1.2 Definitions

Connection	-	the entire assemblage of cleats, joints and fasteners at the intersection of two members
Joint	-	the particular item of a connection transferring load from one part to another
Weld	-	a length of weld comprising any of the standard forms transferring load from one part to another
Fastener	-	a prefabricated item which transfers load - either bolts, rivets or turnbuckles
Snug Tight Bolted Connection	-	a connection consisting of bolts working in simple dowel shear or tension and snug tightened as defined by AS 1511:1973
Fully Tightened Bolted Connection	-	a connection consisting of bolts transferring tension forces and/or shear forces by friction or a combination of friction and bearing and fully tightened by 'half turn' method or 'load indicator' method as defined by AS 1511:1973.

2. PHILOSOPHY

2.1 General

The aim of good structural design is to select a method by which the structure may behave during seismic attack and to proportion members size and detail to ensure this behaviour is sustained. The fasteners within connections between members therefore become important in the design process. The design attempts to predict structure

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behaviour by controlling member sizes. However, if the connection fails, i.e. conditions prevail whereby the connection is unable to perform in the way it was intended, completely unexpected behaviour may result which could cause undesirable mechanisms to occur. It is therefore well understood that connections between members should be designed to carry loads in excess of that being carried by the member to retain integrity of the structure as a whole.

Moreover, attention should be given to detail and workmanship to ensure the connection will behave in a predictable manner. The selection of fastener design actions depends principally on the way the structure is expected to respond. Reference (10) discusses the three forms of structural behaviour as being:

(a) fully ductile response
(b) limited ductile response
(c) elastic response

2.2 Connection Influence on Response

Connections between members will affect the performance of the structure. The effectiveness of the connections on response can be gauged from the size and shape of the hysteresis loops generated during cyclic movement and the amount of damping produced. For example, well detailed fully welded connections produce fat, well shaped loops but experience only 1-2 percent elastic damping. Bolted connections, on the other hand, can produce pinched loops and show significantly high elastic damping of 5-7 percent. Moreover, considerable differences in performance are obtained between lapped connections and end plate connections. Bolted connections may also attract more damage during seismic attack and could cause greater overall deformations than their welded counterpart. Consciously allowing energy to be dissipated within a connection will also affect response.

Connections, their method of design, detailing and control of workmanship can therefore, significantly affect the behaviour of the structure and therefore its response under seismic attack. The assessment of dynamic response taking into account performance of the connections and the ultimate choice of structural 'S' factor is fully discussed in Ref.(10).

2.3 Capacity Design and Overstrength of Connections

Erasmus (12) has investigated the mechanical properties of various structural sections and has suggested that yield strengths of mild steel rolled steel sections can vary between considerable extremes. The inference is that designers should be careful when considering the probable yield strength of members.

It is recommended in Ref.(10) that connections should be designed overstrength to the member nominal yield strength both in fully ductile responding and limited ductile responding structures. Therefore, in order to take account of the different levels of expected yield strengths (found by Erasmus) as well as strain hardening the designer should base connection design actions on:

1.5 times the nominal yield strength of the yielding members in fully ductile structures;

1.35 times the nominal yield strength of the yielding members in limited ductile structures.

Moreover, it is recognised that all connections should have some minimum capacity or strength to withstand actions such as shifts in points of contraflexure and unpredictable overall structure deformations. Connections should therefore not be designed for less than those minimum capacity actions described in section 2.7.

2.4 Connections in Fully Ductile Structures

For these cases, the structure is designed to behave in a fully ductile manner and the members chosen and detailed to sustain a particular failure pattern during seismic attack. All members not yielding are subject to capacity design and are required to be over-strength to the yielding member actions as described in section 2.3. Consequently, all connections and fasteners are to be designed for the overstrength member actions.

Irrespective of what values the above actions produce, upper and lower limits of design actions are applicable to any connection (as described in 2.3). The upper limiting value of design action need not be greater than that applicable to the elastic response procedure described in 2.6. The lower limiting value of design action should be the minimum capacity actions described in section 2.7.

2.5 Connections in Limited Ductile Structures

For this case, the structure, is designed to carry a higher seismic load but the stringent requirements of fully ductile behaviour are ignored. Nevertheless, failure failure mechanisms must be chosen and a form of capacity design procedure adopted as described in section 2.3.

Irrespective of what values the above actions produce, upper and lower limits of design actions are applicable to any connection (as described in 2.3). The upper limiting value of design action need not be greater than that applicable to the elastic response procedure described in 2.6. The lower limiting value of design action should be the minimum capacity actions described in section 2.7.

2.6 Connections in Elastic Responding Structures

For these cases, the structure is required to be designed for higher lateral forces and to respond elastically. Nevertheless, it must be recognised that connections must behave in a ductile manner and that some capacity is available in order to prevent catastrophic failure.

For elastic responding structures the connections must be designed for the actions applicable to the elastic design procedure.

2.7 Minimum Design Actions

As described in section 2.3, a minimum connection strength should be provided in order to lessen the chances of premature failure due to unpredictable movements of the structure. It is therefore recommended that the connection be able to sustain the following minimum design actions:

50 percent of the strength of the member in tension or compression ($0.5\ A_sF_y$);

30 percent of the flexural capacity of the member ($0.33\ Z_xF_y$);

15 percent of the strength of the member in shear ($0.15\ A_vF_v$).

The strengths need not be considered as acting simultaneously.

The above requirements for carrying shear and flexural capacity can be relatively easily provided and should apply to all connections. However, the tensile/compressive requirement is severe and need only be applied to those connections likely to be subject to significant tension or compression loads.

For connections which are likely to behave in a brittle manner by way of detail or workmanship, consideration should be given to increasing the design actions on the connection.

2.8 Energy Dissipation Within Connections

It has been common practice for some years for some ductile response to be achieved by energy dissipation within the holding down bolt assembly. Reference (16) covers various aspects of the design and detailing of connections such as these enabling economies to be achieved. A possible development of this concept is to consider some energy being dissipated in investigating overall joint ductilities in the superstructure connections - Wood (14) and Popov and Pinkney (4) in investigating overall connection ductilities have both found that considerable dissipation of energy can be achieved in a well detailed and designed high strength friction grip bolted connection provided proper precautions are taken. Various aspects of detailing which improve the performance of bolted connections are described in section 3.2.

2.9 Flow Chart For Design Procedure

(See top of page)

3. CONNECTIONS

A connection is defined as the entire assemblage at the intersection of members and must transfer the loads defined above without significant distress or brittle failure. The assemblage consists of:

(a) the weld joining member to cleat or member to member
(b) cleats or plates loaded in any plane
(c) bolts in shear or tension

FLOW CHART FOR DESIGN PROCEDURE

A1. Fully Ductile Structure

B1. Limited Ductile Structure

C1. Elastic Response Structure

A2 Design Connection Overstrength
1.5 multiplier to nominal yield strength member actions
Actions need not be Greater than C3
Ensure minimum capacities are achieved C4

B2. Design Connection Overstrength
1.35 Multiplier to nominal yield strength member actions
Actions need not be greater than C3. Ensure minimum Capacities are achieved C4

C2. Upper Limit of All Design Criteria

C3. Design Connection for Elastic Response Member Actions

C4. Connection must not be designed for less than the following proportion of those Separate actions:
50% capacity in tension/compression
30% flexural capacity
15% capacity in shear

3.1 Welding

It is generally accepted and confirmed by experiment (4,5,8 and 9) that properly designed welded connections will perform well during cyclic loading. However, if the fabrication specification is not well written, or if the specification is not met due to poor workmanship, premature failure at very low loads or at low numbers of cycles may occur. The accent is on taking care to ensure that the elements designed are detailed correctly and that the weld is properly applied. NZS 3404 (1) describes various forms of weld failure and Ref.(3) should be consulted for information related to basic welding requirements. A separate paper in this series (13) describes workmanship particularly associated with seismic detailing.

Although not always economical, properly produced full penetration butt welds provide the best means of load transfer provided the designer selects an electrode whose material strength is compatible with the parent metal, i.e. the electrode material strength should be greater but not significantly greater than that of the parent metal (see Table 2, Ref.(3)). This can be of particular concern in partial penetration butt welds where high stresses can occur in the weld metal, but because of small volumes, the comparable parent metal stress is much higher than the allowable value. However.in accordance with normal code requirements, partial penetration butt welds should not be used in areas of stress reversal. It is also important to use a minimum volume of weld metal within a joint, in order to minimise shrinkage stresses which, if high, can assist in inducing lamellar tearing. In this respect the requirements of section 5.4.6 of Ref.(3) regarding limits of weld reinforcement should be noted.

Hydrogen content in the steel and in weld metal can have a bearing on the seismic performance of the material. Normally the percentage is small. However, as the content increases, embrittlement can result particularly with high strength steels. It has been shown that electrodes of low hydrogen content will assist in overcoming the above problem. It is therefore recommended that for connections subject to high ductility demand low hydrogen electrodes be specified.

Fillet welds are an acceptable form of load transfer provided the following rules are followed:

(a) Minimise amount of intermittent welding. Every beginning or end of a run of weld is a potential stress raiser and it is important to keep these discontinuities to a minimum.

(b) Provide welding with throat thickness greater than or equal to half the thickness of the plate being welded.

(c) Effective length of weld is actual length less twice thickness of leg (being twice effective throat thickness).

(d) When using strength design the following stresses in the weld must not be exceeded:
    (i) Direct stress - 0.95 times the specified ultimate tensile stress of the welding electrode.
    (ii) Shear stress - 0.55 times the specified ultimate tensile stress of the welding electrode.

    Note: Both (i) and (ii) are the normal AS1250 code allowances for working stress design divided by 0.6.

(e) Care must be taken in choice of high strength electrodes. If the leg length of weld is small, there is a strong chance that tearing of the parent metal could occur. Therefore in these situations check stresses in the parent metal.

(f) Care must be exercised in specifying the shape of fillet welds as poor workmanship can cause stress raisers leading to brittleness. Refs.(3) and (13) describe some aspects of this.

(g) The end conditions are also very important in the production of a sound weld. Ref.(3) describes requirements for run-on and run-off plates which are necessary in order to reduce stress raisers at critical points of the joint.

3.2 Bolting

The ultimate behaviour of bolted connections under cyclic conditions both in snug and fully tightened modes has not been widely tested. Popov and Pinkney (4) have carried out cyclic tests on a beam-column joint with flange bolts in shear although it is not clear whether or not the bolts were in the fully tightened mode. Nevertheless, the hysteresis loops for the connection were marginally pinched showing the effect of slippage at the faying surfaces. The extent of pinch in the loop was reduced if the holes were drilled 0.4 mm oversize instead of the normal 1.6 mm (USA 3/16" standard). It therefore follows that if the connection racks, it is better for it to move only slightly before going into the bearing mode. The authors also suggested that the effect of "ovalling" of the holes would be reduced if allowable bearing stresses were reduced.

There are a number of factors affecting the design and performance of a bolted connection:

(a) the size of hole and method of production
(b) the conditions of the faying surfaces
(c) whether or not threads are included in the plane of shear
(d) the tightening procedure and design method assumed

These will be dealt with in detail.

3.2.1 The hole

As described above there appears to be advantages to the performance of the fastener by providing a tighter fit to the bolt (i.e. providing hole diameters 0.5 mm greater in diameter than the bolt diameter) and/or reducing allowable bearing stresses under ultimate conditions to F_y. For ductile connections A2 it is recommended that both of these procedures be adopted. For limited ductile connections it is suggested that limiting bearing stresses only is all that is required. (Also refer 3.2.4 (ii) below.)

It has been acknowledged for some time (6 and 7) that punched holes affect the behaviour of connections particularly with respect to plastic deformation. Micro cracking on the exit side of the hole can cause brittle behaviour and is the source of stress raisers. It is therefore recommended that punching without reeming to remove the work hardened steel should not be adopted when requiring a fully ductile response.

3.2.2 The condition of the faying surfaces plays an important role in seismic performance. Wright (5) and Wood (14) found considerable differences in response from fully tightened connections treated with different paint systems. We must be extremely careful in our choice of friction factor during design to ensure the specified factor is achieved.

3.2.3 Threads within or without the shear plane affect the design of bolts on snug and bearing mode tightening procedure method of design. It is generally impractical on most contracts of small or moderate size to specify threads being excluded from the shear plane. Not only is this awkward to inspect, but it is probably very difficult to achieve unless a special order of bolts is made. Specifying and designing for threads outside the shear plane should then be considered only for large works where a special order is made for large numbers of bolts.

3.2.4 Tightening procedure and design method

(i) Fully tightened connections

Fully tightened design provides good performance in seismic conditions. If the connection is required to have no slip during gravity loadings then the connection should be designed in the friction mode accordingly. Under design earthquake and capacity check one can take advantage of the added capacity of the bearing mode. In any event for a fully ductile connection bearing stresses should be limited to F_y or the holes should be tight (as described in 3.2.1). For limited ductile or elastic response structures fully tightened bolting procedures may be used with the stringent requirements for hole size and bearing stress limits relaxed as described below for snug tight bolting procedures.

(ii) Snug tightened connections

Snug tight design has the advantage of being the cheapest alternative per unit of load carried. There has been considerable discussion whether or not snug tightened bolts should be used in connections subject to seismic attack. Secondary effects such as, joint movement or slop, and impact loading caused by slop, are detrimental to the performance of the structure in a minor earthquake. Moreover, structural behaviour becomes unpredictable when large slippage and ovalling of holes occurs in a major earthquake. For these reasons the following guidelines provide recommendations for the use of snug tight bolts:

- for fully ductile design the allowable ultimate bearing stress under strength design is limited to 1.0 F_y and the holes are to be drilled to provide a "tight" (0.5 mm oversize) fit.
- for limited ductile design the allowable bearing stress is limited to 2.1 F_y.
- for elastic design the allowable bearing stress is 3.5 F_y (the normal maximum bearing stress specified by AS1250 for working stress design divided by 0.6).

It should be noted that if normal sized holes are used and/or bearing stresses not limited as described above, significant slip will occur during seismic attach which will increase the connection and overall structure deformation. The designer must make allowances for such increased movement.

3.3 Cleats

With so much emphasis in design guides on the principle sections of a joint being the weld or bolt, the design and detailing of the cleat is often given less importance than its due.

The following local effects on cleats should be checked:

- local effect on M_{yy} - note that small eccentricity of high axial loads on cleats in single shear can cause overstress of the cleat and the weld in bending in the weak direction of the cleat. The cleat and weld accordingly should be checked as a beam column.
- buckling - local buckling of a cleat is similar to a crippling failure in the web of a beam and the normal procedure taking account of the effective length of the cleat in the weak direction is sufficient to check buckling.
- bearing - as described above it is recommended that in some instances bearing stresses at ultimate loads be limited in order to minimise the effect of 'ovalling' of the holes.
- punching, tearing and splitting failures can be catered for by providing a ratio of edge distance to fastener diameter in excess of three.

Design procedures which take account of the above effects are shown clearly in the sample calculations, Appendix II.

3.4 Rivets

Recent tests (15) by the Ministry of Works and Development Central Laboratories have indicated that rivets perform well under cyclic loading. The rivet not only produces a clamping force similar to a fully tightened high strength friction grip bolt but also fills the entire hole. This means that riveted splices in shear may have significantly higher load carrying capacities and considerably less slip than the equivalent high strength friction grip bolt connection. The tests carried out were part of an investigation into the structural adequacy of an existing building and are not expected to be the basis for design rules or recommendations.

3.5 Turnbuckles

Turnbuckles are an accepted form of fastener in bracing provided the following is adhered to:

- turnbuckles are to be used for tension braces only.
- yield must be precluded from the turnbuckle assembly.
- use is restricted to proofloaded turnbuckles.

3.6 Appendices

See Appendix I for Table of Ultimate Loads on Welds and Bolts.

See Appendix II for Sample Calculations.

4. REFERENCES

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(3) NZS 4701:1981, "Metal Arc Welding of Steel Structures", N.Z. Standards Association, Wellington.

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(10) Patton, R.N., "Analysis and Design Methods", Section B, NZNSEE Study Group for the Seismic Design of Steel Structures, Bull. NZNSEE, Vol. 18, No. 4, December 1985,

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(12) Erasmus, L.A., "Mechanical Properties of Structural Steel Sections - Statistical Analysis", Report No. R4-01, N.Z. Heavy Eng. Res. Asso., Manukau City.

(13) McKay, G.R., "Materials and Workmanship", Section K, NZNSEE Study for the Seismic Design of Steel Structures, Bull. NZNSEE, Vol. 18, No. 4, December 1985,

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(16) Powell, S.J., "Holding Down Bolts for Chimneys in Earthquakes", University of Auckland, Dept. of Civil Engineering Report No. 255, November 1980.

(17) Rosenbleuth, E., "Design of Earthquake Resistant Structures", Pentech Press: London, 1980.

APPENDIX I

Tables of Ultimate Loads on Welds and Bolts

The following table gives ultimate loads on fillet welds in kilonewtons per millimeter (kN/mm), equal leg fillets, $\phi = 90^{o}$.

Weld Size (mm)		E41XX Weld Orientation		E48XX Weld Orientation	
Leg	Throat	P_l	P_+	P_l	P_+
4	2.83	0.63	0.78	0.75	0.92
5	3.54	0.80	0.98	0.93	1.15
6	4.24	0.95	1.17	1.12	1.37
8	5.66	1.28	1.57	1.50	1.83
10	7.07	1.60	1.95	1.87	2.28
12	8.49	1.92	2.35	2.23	2.75

NOTES: P_l = maximum ultimate longitudinal load/unit length (kN/mm)
P_+ = maximum ultimate transverse load/unit length (kN/mm)

Table 1.a

Table 1.b (see next page) gives the ultimate loads on High Strength Grade 8.8 Structural Bolts (to AS1511) in Single Shear, bearing stresses limited to F_y = 250 MPa.

APPENDIX II

Sample Calculations

Example 1: Fully ductile response - Eccentrically braced frame

Connection is at the top of a 200UC46 Brace.

Ultimate load in compression under E (S = 0.8) = 600 kN

Section Properties - 200UC46:

Length = 5000 mm r_y = 51.1 mm
F_y = 250 MPa A_s = 5880 mm^2

Design Procedure is A2 - Section 2.9 Notes:

Maximum connection force given by link zone analysis say = 794 kN

Design joint action P' for 1.5(max force), P' = 1191 kN

Check minimum design action P_{min} (tension/compression) as 50% of A_sF_y

P_{min} = 735 kN   Less than P', OK

Try M20 N/TB - Limit bearing stresses to F_y
- threads included in the shear plane
- double shear, k = 2

From Table 1.b Bolt Capacity 92 kN

Try 8 bolts

Bolt group capacity = 1472 kN
Greater than P', OK

For bearing - central plate load per bolt = 149 kN
- two side plates load per bolt = 74 kN

From Table 1.b select
side cleats - thickness = 16 mm
central cleat - thickness = 32 mm

Thickness of web of 200UC46 = 7 mm, therefore require 2-12 mm double plates on UC (see Fig.1a).

Check cleats in compression - Buckling:

Figures 1a, 1b and 1c.

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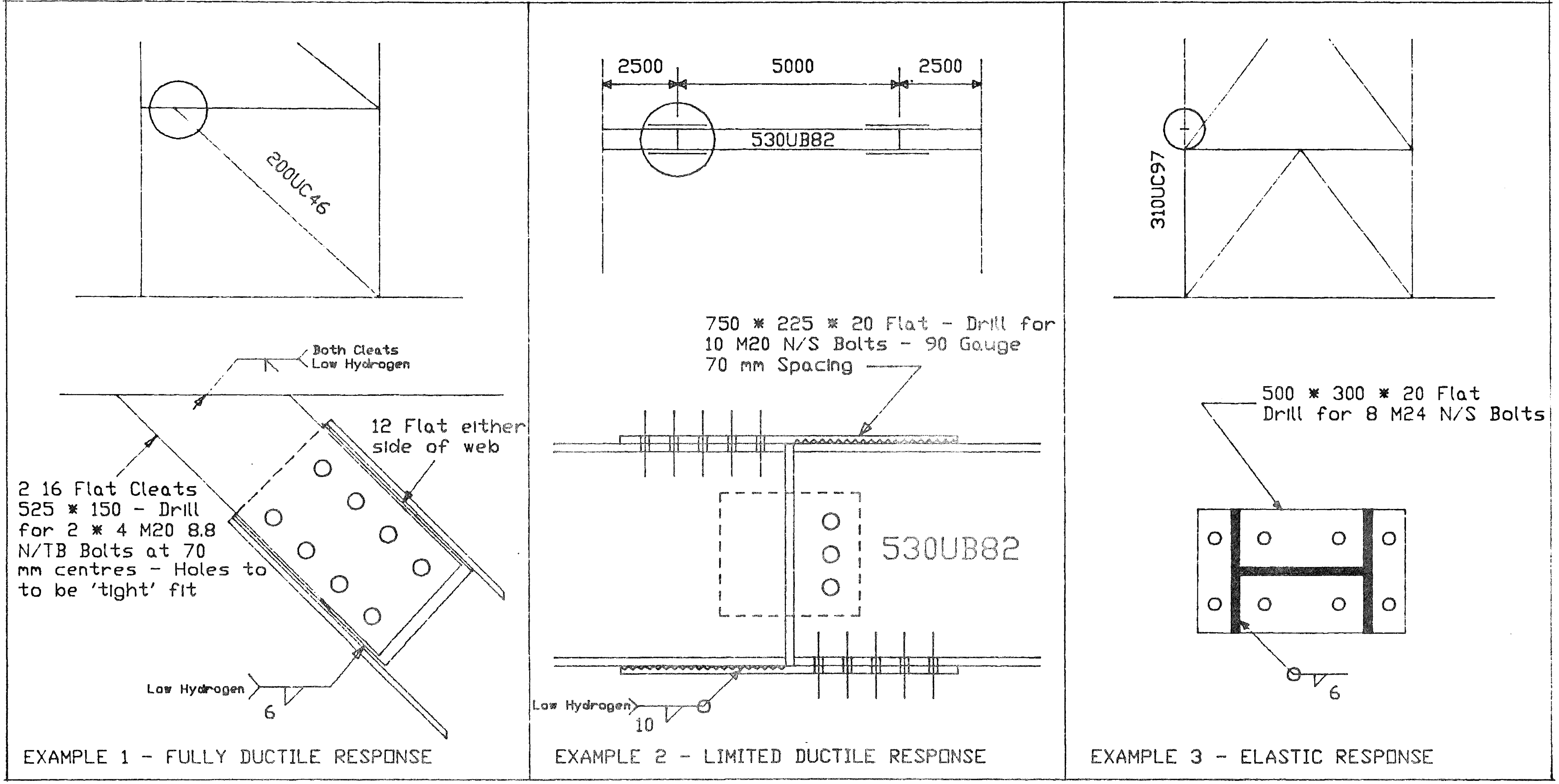
APPENDIX II - SAMPLE CALCULATIONS - SKETCHES



a b c

ULTIMATE LOADS ON GRADE 8.8 BOLTS (kN)															
BOLT SIZE	BOLTS IN SINGLE SHEAR				BEARING kN FOR PLATE THICKNESS (mm)								FULLY TIGHTENED FRICTION MODE	AXIAL TENSION	MINIMUM EDGE DISTANCE e_d (mm)
	SNUG TIGHT		FULLY TIGHTENED BEARING MODE												
	N/S	X/S	N/TB	X/TB	6	8	10	12	16	20	24	32	SF/TF = 0.35		
M20	75	105	92	127	20	40	50	60	80	100	120	160	50	147	60
M22	90	127	111	154	33	44	55	66	88	110	132	176	61	177	66
M24	108	150	132	183	36	48	60	72	96	120	144	192	73	212	72

Table 1.b

Max unsupported length,
say 2/3 of 250 mm = (167 + 60) 0.85
= 193 mm

As a column - 16 Flat Cleat:
r = 0.3D = 4.8 mm

$\frac{l}{r_y}$ = 40 mm, therefore F_{ac} = 142 MPa

F_{ac} (ultimate) = $\frac{142}{0.6}$ = 237 MPa

Actual stress = 248 Mpa
Accept at 5% overstress

Welds - double 16 flat cleats to member have Bevel butt welds full length
- doubler plates to 200UC46

Load in each side plate = 596 kN

Length fillet welds available
= 2 x 350 + 150
= 850 mm in one plate

Required ult. load/mm = 0.701 kN/mm

Select 6 mm fillet weld full profile (minimum for 12 flat)

Note load under E = 600 kN
therefore load per bolt = 75 kN

In double shear, SF = 0.35, M20/TF capable of carrying 100 kN. Therefore this structure does not slip under E load. This condition is not necessarily part of the design but shows that Capacity Procedure may produce this condition.

Example 2: Limited ductile response - Beam splice

See Fig.1b - beam is 530UB82 - S = 3.0

Ultimate moment at the splice under
D + 1.3 L_R + E = 250 kNm

For elastic response S = 6, M_{splice} = 404 kNm

Beam can be considered laterally supported at the ends and midspan only.

Section Properties - 530UB82:

r_y = 43.8 mm

D/T = 40

Z_{xx} = 1800 x 10^3 mm^3

Design Procedure is B2

Capacity of member given by l = 0.85 x 5000
= 4250

l/r_y = 97, therefore F_{bc} = 137 MPa

Ultimate F_{bc} = 137/0.6 = 228 MPa

Therefore ultimate moment capacity = 410 kNm

Capacity joint action M' = 1.35 * Capacity of beam,

M' = 554 kNm

Elastic response design action is 404 kNm - less than M'

Therefore need only design JOINT for 404 kNm

Check minimum requirement 0.3 Z_{xx} F_y = 135 kNm OK

Try Shop Welded/Site Bolted Connection (see Fig.1b)

Force at Flange Face = 404/0.536 = 754 kN

Flange thickness = 13.2 mm

Try 10 M24 N/S Bolts (2 rows of 5)

From Table 1.b through 13.2 mm one M24 N/S bolt carries an ultimate load of 79 kN

Ult. capacity of bolt group = 790 kN, greater than 754, OK

(Note this is limiting bearing stresses to F_y)

Adopt 2 Rows of 5 Bolts in Flange - M24 N/S

Lap plate - try 225 width - drill 2 rows - 90 gauge

Net area required = 3016 mm^2

Try - 20 flat - net area =
(225 * 20) - (2 * 20 * 22)
= 3620 mm^2, OK

Weld - using 10 mm fillet weld, required to transfer 754 kN - length required =
754/1.6 = 471 mm

Actual length = 900 mm, OK - see Fig.1b.

Example 3: Elastic Response - Column Splice (S = 6.0)

Column carries axial load only - under worst combination of conditions maximum ultimate loads are

Compression C = 810 kN
Tension T = 130 kN (S = 6)

Section Properties - 310UC97:

A_s = 12 300 mm^2, F_y = 250 Mpa

Z_{xx} = 1 440 000 mm^3, F_v = 154 Mpa

A_v = 3049.2 mm^2

Design Procedure is C3

Max elastic response load = 810 kN

Check C4 loadings

0.5 $A_s.F_y$ = 1538 kN

0.3 $Z_{xx}F_y$ = 108 kNm

$0.15\ A_V.F_V = 70$ kN

Design splice for axial load = 1538 kN

Try M24 N/S bolts in end plate - say 8 bolts

Max ultimate force in tension = 1696 kN, OK

Weld - length of weld full profile on
310UC97 = 8 * 290
= 2320 mm

Weld capacity required = 0.663 kN/mm

Accept 6 mm fillet weld full profile

Baseplate - by Yield Line theory

Check moment and shear capacity to cover the minimum.

Design actions above - see Fig.1c for final arrangement.