Section I

COMPOSITE DESIGN

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

2. INTRODUCTION

This paper gives recommendations to assist designers in the design of composite structures to resist seismic loads.

Composite design and construction involves the use of structural steel sections or steel sheets and reinforced concrete acting together to carry the applied loads. It can be applied to some or all of the structural components in a structure but produces its greatest economy in construction when it is applied to those components that normally consist of concrete and steel. Thus in multi-storey steel frame construction the floor slabs and beams are normally made composite whereas the columns consist of steel sections only. Used in this way composite construction is currently the most economic method of multi-storey construction available within New Zealand.

When the initial draft of this paper was being written, in January-February 1985, there was no design information available on composite design to New Zealand design conditions. Thus, to ensure that any seismic design recommendations made were accurate, relevant and practical, a general paper on composite construction had to be written incorporating the seismic design recommendations. The general paper forms the first edition of HERA Design Guide Part 13: Composite Construction and the seismic provisions given in this paper relate directly to this design guide, hereafter referred to as reference 1.

2.1 Scope and Purpose of Paper

The purpose of this paper is to provide designers with an overall understanding of the seismic design requirements for composite construction.

It covers the following categories of composite construction in varying detail:
(a) Floor slabs
(b) Beams
(c) Stub girders
(d) Columns
(e) Beam-Column connections
(f) Shear walls

2.2 Ductility Classification

In line with the classifications of ductility demand given by this study group, recommendations for seismic design will be given for three classes of ductility demand, these being:
(a) Fully ductile response (high ductility demand).
(b) Limited ductile response (low ductility demand).
(c) Elastic response (no or very low ductility demand).

The levels of global displacement ductility demand on the structure, as measured by $\mu = \Delta_u/\Delta_y$, are as follows:

Ductility Demand	μ –	NZNSEE Study Group Category
Elastic	$\mu \leqslant 1.25$	3
Low	$1.25 \leqslant \mu \leqslant 2$	2
High	$2 < \mu$	1

Categories 1 and 2 require a capacity design, category 3 does not. Reference 3 gives details of the analysis and design methods appropriate for each category for composite design.

2.2.1 Choice of Structural Type Factor

The choice of an appropriate structural type factor depends on the amount of elastic damping inherent in the structure, the type of inelastic behaviour that the seismic resisting system will undergo in an earthquake and the degree of inelastic action expected from or allowed in the primary seismic resisting components.

Composite sections detailed for ductility exhibit similar inelastic behaviour to ductile reinforced concrete sections. The moment-rotation behaviour

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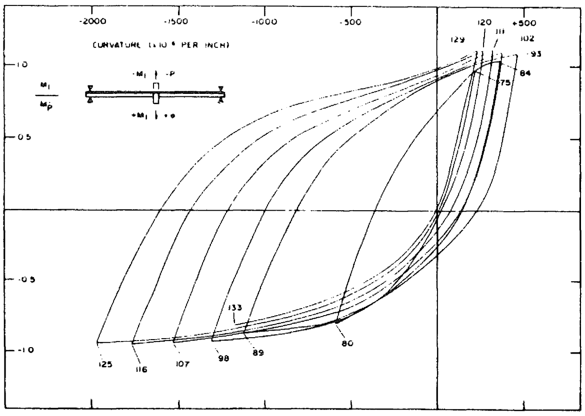


Fig. 1 : Moment-Curvature Hysteresis Loops for Composite Beam (Ref. 4). (Note: the numbers on the hysteresis loops are the numbers of the relevant load cycle).

of a composite beam under negative moment is quite different to that under positive moment due to the concrete slab being in tension in the former case and compression in the latter. Figure 1 shows an example of composite beam hysteresis loops for a composite beam under positive and negative moment. The loops are stable over many load cycles and don't suffer the same degree of pinching and degradation as do the loops for some other materials.

Reference 5 gives a detailed procedure for assigning the appropriate S factor to a given type of seismic resisting steel construction and illustrates its use with the example of a moment-resisting frame. The same method can be applied to composite construction with suitable allowance made for any increase in elastic structural damping due to the reinforced concrete in the composite section.

In the case of a moment resisting frame with mixed bolted splices and welded beam-column connections the elastic damping for the composite beams can be taken as 10%. This gives S factors, for inadequate numbers of plastic hinges, of S = 1.0 for high ductility demand and S = 1.8 for low ductility demand. The material factor, M = 0.8. The definition of inadequate number of plastic hinges is given in NZS 4203:1976. S factors for a structure are determined from consideration of the inelastic behaviour of the structure, the ductility capacity of the yielding regions and the amount of viscous damping. If there are sufficient regions yielding within a structure (i.e. an adequate number of plastic hinges) very poor performance from one region will not seriously affect the structure as a whole. This permits a reduction in the level of design seismic load on the structure. The number of plastic hinges required to allow the reduction and the extent of reduction permitted is defined in NZS 4203:1976.

3.0 COMPOSITE FLOOR SLABS

3.1 Introduction to, and Scope of Section 3

A composite floor slab consists of a concrete slab rigidly connected to a steel beam by means of a shear transfer mechanism.

The concrete slab can be either:
(a) An insitu reinforced concrete slab poured on to conventional formwork.
(b) A precast concrete slab with or without structural topping.
(c) An insitu concrete slab poured on to profiled steel sheet.

Full details of each system are given in Ref. 1. In this paper, seismic design recommendations for the shear connectors are given in Section 4.2, and for ductile composite beam action in Section 4.4. Section 3.2 gives recommendations relating to the transfer of horizontal seismic loads through the composite floor slab into the primary lateral load resisting element(s) of the structure.

3.2 Seismic Provisions for Horizontal Shear Force Transfer

This load transfer occurs through welded or bolted shear studs or through deformed bars welded to the surrounding steel beams and buried in the slab (or topping).

The maximum horizontal shear force required to be transferred is the smallest horizontal force determined from the following two considerations:
(a) The maximum resistance of the primary lateral load resisting system.
(b) The forces arising from the seismic design coefficients, at the floor level concerned, specified by NZS 4203 for parts or portions of buildings clause 3.4.9.

The slab should be able to develop the required diaphragm action without any yielding occurring, otherwise it must be suitably detailed for ductility according to the provisions of NZS 3101, Clause 14.9.

The transfer of this shear force may be achieved by means of one of the following methods, where applicable:
(a) For slabs poured onto formwork or profiled steel decking headed studs welded direct to the steel beam may be used. The ultimate strength per study is given in Ref. 1, based on a capacity reduction factor of 1.0 and is irrespective of the ductility demand on a particular length of member.

These studs may also be used for composite beam action, which will induce shear forces directly additive to the diaphragm seismic shears, using the appropriate load combinations. The critical section for shear is taken along a vertical plane immediately adjacent to the line of the studs.

(b) For slabs consisting of precast units, with or without topping, the shear connections may be achieved by bolting the precast units directly to the steel beams with HSFG bolts used in the /TB or /TF modes. This is illustrated in Fig. 2 and covered in Ref. 2, part 5.4.4.

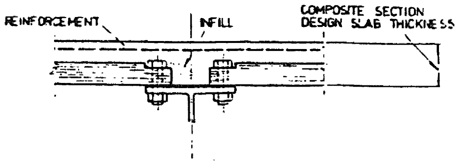


Fig. 2 : Precast Slab With Topping Bolted Directly to Beams (and with Continuity Reinforcement).

The solid plank precast units are better suited to this method than the hollow section units (e.g. Dycore) because their solid cross section is more suitable for resisting compressive bearing stresses and shear forces and their cross-sections are thinner, which allows standard M16 or M20 bolt lengths to be used.

(c) For insitu slabs with an insitu poured topping the seismic shear can be transferred to the supporting beams and/ or columns by means of welded shear reinforcement and direct bearing. The welded reinforcement consists of Grade 275 stirrups or deformed bars welded onto the steel main beams and anchored into the insitu concrete.

This method is only suitable for carrying seismic diaphragm shear forces where no composite beam action is designed for, as the connection is very flexible and would generate excess cracking in the concrete slab under the maximum design service loading if the beams were designed for composite action.

4. COMPOSITE BEAMS

4.1 Introduction to, and Scope of, Section 4

A composite beam consists of a slab, steel beam and shear connectors to provide partial or full composite action between the slab and beam.

The load deflection behaviour of a composite beam with partial or full interaction is compared in Fig. 3(a) with that of a steel beam alone.

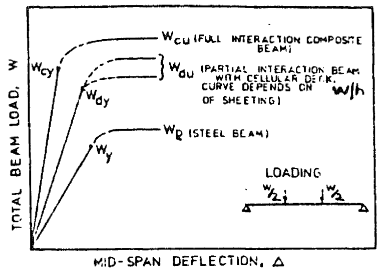
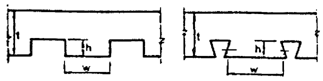


Fig. 3a: Typical Beam Behaviour - Ref. 4.



(a) ribbed profile (b) dovetail profile

t=slab thickness;
w=pan width (average if side sloped);
h=rib height

Fig. 3(b): Sheeting Geometry

Notation:

W_y = yield load, steel beam alone
W_p = plastic moment, steel beam alone.
W_{cy} = load for first yield, fully composite beam
W_{cu} = collapse load, fully composite beam
W_{dy} = load for first yield, beam with cellular decking
W_{du} = collapse load, beam with cellular decking
W = applied load
W,h = cellular deck parameters defined in Fig. 3(b).

Partial interaction composite design involves failure of the section taking place through the shear connectors. It is an established design method in the United States but is not recommended for use in Australia. Furthermore it has an unrealiable inelastic ductility capacity and hence is not recommended in any section subject to low or high ductility demand.

Some steel sheet decking systems have profiles such that the full strength of the shear studs cannot be developed and these systems cannot be used in regions of low or high ductility demand. Reference 1 gives details.

Each shear connector is bearing horizontally against the concrete slab when transferring shear, either due to composite or seismic diaphragm action. This action induces tension in the concrete in the direction perpendicular to the line of shear connectors and this tensile force necessitates the placing of transverse reinforcement in the slab.

Seismic design provisions are given in Section 4.2 for shear connectors, Section 4.3 for transverse reinforcement and Section 4.4 for ductile composite beam design.

4.2 Shear Connections

4.2.1 Introduction

Connectors fall into five classifications: rigid, flexible, bond, rigid plus bond and high strength structural bolts. The only two recommended for general use are the flexible and high strength structural bolt connectors, examples of which are shown in Fig. 4.

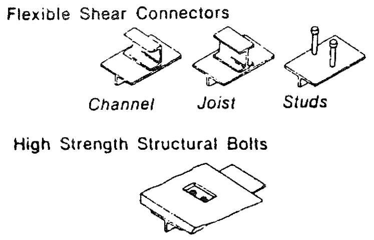


Fig. 4 : Shear Connector Types

These are also the only two classifications of connector about which sufficient experimental information is available to make reliable seismic design recommendations.

4.2.2 Connector Capacity

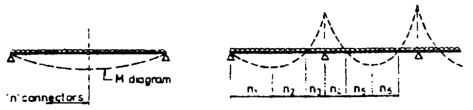
The ultimate capacity of the connectors is the same for all design applications, irrespective of the ductility demand on any section of the member. The reason for this is that the connector capacity is dependent on the concrete being confined and cracked.

There is a need for closer confinement of the concrete with increasing ductility demand. This is achieved by increasing the level of transverse reinforcement through the concrete slab, directly above the beam. Details are given in Section 4.3.

Reference 1 lists the ultimate capacities of the types of connector shown in Fig. 4.

4.2.3 Number of Connectors and Spacing

The required number of connectors may be dependent on the ductility demand, as the connectors have to carry the overstrength capacity of the steel beam in regions of low and high ductility demand. The spacing is also dependent on the nature of the applied load. For uniform loading the requirements for connector numbers are as shown in Fig. 5 and the accompanying equations; for other types of applied load refer to Ref. 1.



$V_h \leq 0.85\ F_c \cdot A_c$

$\leq A_s \cdot F_y \cdot K_1$

n = no. of connectors

$= V_h/q_u$

∴ total no. of connectors for whole span = 2n

In segment n_1, n_2, n_5, n_6:-

$V_{hi} \leq 0.85\ F_c' \cdot A_c$

$\leq A_s \cdot F_y \cdot K_1$

$n_i = V_{hi}/q_u$ (i = 1, 2, 5, 6)

In segments n_3, n_4:—

$V_{hi}' = A_r \cdot fsy \cdot K_1$.

$n_i' = V_{hi}'/ 0.8q_u$ (i = 3, 4)

Fig. 5 : Required Numbers of Connectors

Definitions for Fig. 5

A_s = area of steel beam (mm^2)

F_y = yield stress of steel beam (MPa)

K_1 = 1.0 for no ductility demand design or where the plastic hinges do not occur within the segment under consideration for low or high ductility demand designs

K_1 = 1.35, 1.5 for low, high ductility demand designs where plastic hinges form within the segment under consideration

A_c = effective area of concrete flange = effective (width x thickness) (mm^2)

A_r = total area of longitudinal reinforcing steel in slab contained within the effective flange width at support (mm^2)

f_{sy} = specified minimum yield stress of the longitudinal reinforcing steel (MPa)

q_u = shear capacity of connector from Table 1 in Ref. 1.

For HSFG bolts in the /TF mode the working stress loads must be used as illustrated in Ref. 2, Appendix D.

Reference 1 gives details of maximum and minimum connector spacings for lengths of composite beam subject to no ductility demand. Within plastic hinge zones subject to low or high ductility demand the longitudinal spacing should be no greater than 300 mm or 2 times the slab thickness or 4 times the height of the connector, whichever is the least.

Furthermore, within any plastic hinge zone subject to low or high ductility demand a single line of stud shear connectors must never be placed directly over the steel beam centreline but must be staggered on either side of the centreline to reduce the possibility of longitudinal splitting of the slab.

4.3 Transverse Slab Reinforcement

Premature failure of a composite beam due to longitudinal splitting of the slab is a complex phenomenon which appears to be only reproducible in the laboratory. However it has the potential to occur in practice, especially within regions of low or high ductility demand. Reference 1 gives details of the tests undertaken to study transverse splitting of a slab and gives full details of the transverse reinforcement requirements formulated to prevent its occurrence. The provisions would appear to be conservative and do not relate directly to the cyclic inelastic ductility demands imposed on plastic hinge zones in category 1 and 2 members.

Therefore in the absence of any detailed information and taking into account the inherent conservativeness of the transverse reinforcement, the following design provisions have been made:

(a) the provisions given in BS5400:Part 5, as described in full in Ref. 1, are suitable for regions of a beam not subject to inelastic ductility demand.

(b) for low and high ductility demand regions the concrete contribution to longitudinal splitting resistance is decreased by 25% and the maximum bar spacings for bottom reinforcement reduced to one half of the spacings allowed in (a).

Limited numbers of tests have been undertaken on composite beams with slab transverse reinforcement conforming to these recommendations and the inelastic behaviour has been very satisfactory. It is planned to undertake a testing programme in New Zealand in the near future that will verify and hopefully reduce these requirements for transverse reinforcement.

When composite slabs are formed by pouring concrete onto profiled steel decking, where the decking profile is designed to ensure full composite action with the concrete (e.g. Dimond Hi-Bond), the steel decking can be considered as transverse reinforcement.

The area of transverse reinforcement required is not great, even within high ductility demand plastic hinge regions, and the steel decking will usually provide more than enough transverse reinforcement.

Reference 1 also gives full details for placing and curtailing transverse reinforcement and gives a minimum required area of reinforcement for all composite beams.

4.4 Ductile Composite Beam Design

This section provides recommendations for ductile composite beam design in each of the following four zones along a beam:

(a) Positive moment within the span (zone 1 in Fig. 6)
(b) Positive moment at the supports (zone 3 in Fig. 6)
(c) Negative moment at an interior support (zone 2 in Fig. 6)
(d) Negative moment at an exterior support (not shown in Fig. 6).

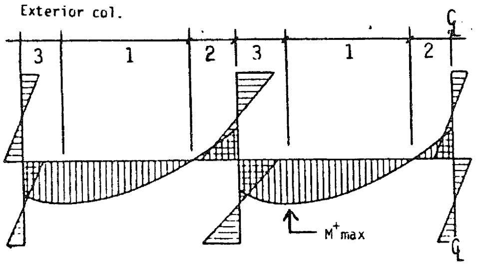


Fig. 6 : Zones for Connector Spacing

4.4.1 Positive Moment Within the Span (Zone 1, Fig. 6)

The ductility capacity of a composite section under positive moment can be determined through the use of the ductility factor as given by Eq. 4.1. This parameter is derived in Ref. 6 and verified by tests in Refs. 6 and 7.

$$\psi = \frac{0.72 f'_c B_c \varepsilon_u (D_s + D_c)}{A_s F_y (\varepsilon_u + \varepsilon_{sh})} \quad \text{for } f'_c \leqslant 30 \text{ MPa} \qquad \ldots \quad (4.1)$$

where ψ = ductility factor
f'_c = concrete 28 day cylinder strength
B_c, D_c = slab effective width, slab depth
A_s, D_s = steel beam cross-sectional area, depth
ε_u = ultimate concrete compression strain at extreme fibre
ε_{sh} = steel beam maximum strain at commencement of strain hardening = $10\varepsilon_y$
ε_y = steel beam strain at first yield.

A value of $\psi = 1.0$ means the steel beam will just commence strain hardening when the concrete slab ultimate strength is reached (this can be considered the composite beam equivalent of the balanced steel ratio, ρ_b, in reinforced concrete construction). Reference 7 recommends a minimum value of $\psi = 1.4$ to ensure ductility for plastic design.

An appropriate relationship between the displacement ductility demand μ and ductility factor ψ has been established in Ref. 1. Hence it is shown that the conservative values of ductility factor for sections required to undergo low and high levels of ductility demand are 1.8 and 2.2 respectively. These ductility factors are easily obtainable in a typical midspan composite beam section subjected to a positive moment.

The overstrength factors for this section are as obtained from Ref. 3 for the grade of supporting steel beam used.

4.4.2 Positive Moment at a Support (Zone 3, Fig. 6)

Under seismic (and wind) lateral loading a positive moment may well form adjacent to the column face, as shown in zone 3, Fig. 6. The width of concrete slab effective at this position is traditionally taken as the column flange width unless spreader plates are employed. Tests conducted on such regions show that the ultimate moment capacity can be predicted based on the effective width of slab equal to the column flange (Ref. 8). However this leads to a section which is often heavily over-reinforced, using reinforced concrete terminology with failure taking place primarily by crushing of the concrete slab. Tests (Ref. 8) have shown that the section has considerable inelastic ductility but that severe slab degradation occurs under positive moment action unless the slab is well confined, and even then the performance of the concrete slab is poor.

Because of slab crushing, under inelastic cyclic loading, the beneficial effect of the slab in resisting positive moment at the support is quickly lost and the heavy damage to the slab adversely affects the anchorage of longitudinal slab reinforcement running past the column, thus reducing the negative moment capacity of the section.

Where composite beams form part of the primary seismic resisting system for a structure they must be capable of stable inelastic cyclic action. There are two possible methods of achieving this. The first is to not use composite action near the supports. This is most economical where the beam is gravity dominated or deflection controlled. In this method the midspan section of the beam is made fully composite, taking advantage of the increased moment capacity and decreased deflection resulting from composite action. The shear connectors are then curtailed a suitable distance away from the supports and the support capacity determined using the steel beam alone. Where the shear connectors are thus curtailed extra connectors should be placed beyond the point of curtailment so that double the required design shear transfer capacity is provided. These connectors should extend 600 mm or twice the depth of steel beam, whichever is the greatest, beyond the point of curtailment. This prevents progressive splitting along the shear connectors from the ends inwards under shears higher than expected from the nominal design strength of the beam at the supports. The second method is to use composite action at the supports for resisting both positive and negative moment and to ensure that the concrete remains intact under applied positive moments and provides the necessary ductility. This involves the implementation of a new design procedure based on the ductility factor concept raised in Section 4.4.1.

The ductility factor provides a measure of the ductility capacity of a composite beam section under positive moment. In Section 4.4.1 it was stated that ductility factor values of 1.8 and 2.2 respectively are required to provide composite sections (under positive moment) with low and high ductility capacity.

These factors are based on the slab concrete being unconfined, with an ultimate concrete compression strain of $\varepsilon_u = 0.003$. However concrete confined to the requirements given in Ref. 9 can dependably sustain an ultimate compression strain of 0.004 (Ref. 10). If this value is used in Eq. 4.1, the ductility factors required for low and high ductility demand are 1.4 and 1.7 respectively.

The theoretical concrete effective widths required to obtain ductility factors of 1.4 and 1.7 are greater than the width of the column face, and can be obtained directly from a rearrangement of Eq. 4.1. The theoretical effective width times the depth of concrete in compression, times the ultimate concrete compression stress, gives the total compression force required to be carried in the slab. The width of concrete in direct contact with the column face is designed to carry its share of this compression force and the remainder is carried by compression reinforcement placed near the top of the slab. This reinforcement is butted up to the column face to ensure transfer of compression force into the column, but is not fixed to the column thus ensuring it has no tensile capacity.

The volume of concrete directly in contact with the column face, which carries this compression reinforcement, is then confined according to the requirements of Clause 6.5.4.3.(b) of Ref. 9 in order that the very high concrete compression strains may be dependably carried.

Thus the concrete slab will behave in a ductile manner under positive inelastic moment with failure occurring by yielding in the steel beam. Figures 7 and 8 give details of the compression and confining reinforcement layout with the full design procedure set out in Ref. 1.

4.4.3 Negative Moment at an Interior Support (Zone 2, Fig. 6)

The calculation of the moment capacity of this section is clearly laid out in many references, including Ref. 2. The compression zone is in the steel beam and tension reinforcement consists of continuous slab reinforcement and part of the top of the steel beam. Hence no concrete is involved in the section moment capacity.

The section can be very ductile, with the only limits on ductility being imposed by lateral and local buckling of the supporting steel beam flanges and web in the compression zone. This is intuitively obvious and has been experimentally confirmed by tests, both cyclic inelastic and monotonic inelastic tests (Refs. 2, 4, 8, 11, 12).

The requirements for compactness of the section flange are the same as for a steel beam acting on its own. However from considerations of section internal equilibrium, the tensile force created by the slab reinforcement must be resisted by a longitudinal compression force in the steel section. This enters the steel beam through the shear connectors and introduces a compression force into the beam web. The interaction of axial load and bending in the web must be accounted for in a composite beam as it is in a steel section. Reference 13 provides a means of accounting for this interaction and this is incorporated into Table 1.

Table 1 gives the requirements for section compactness and/or web stiffening requirements for UB ,UC sections,in steel grades 250, 350, 450 for sections subjected to no, low or high ductility demand support negative moments. These are taken from the above references plus Ref. 14 (Ref. 14 gives the high ductility demand limits and reasons for them).

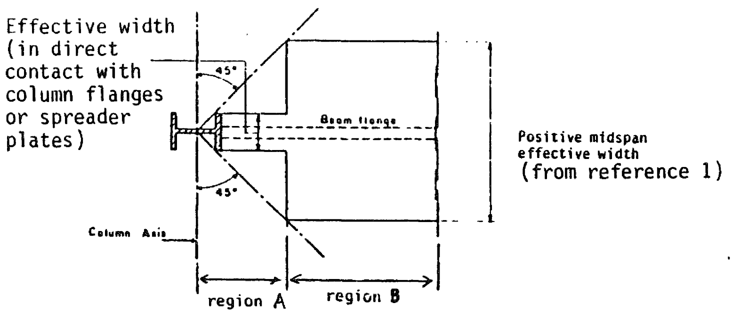


Fig. 7 : Effective Width of Positive Moment Regions at Support

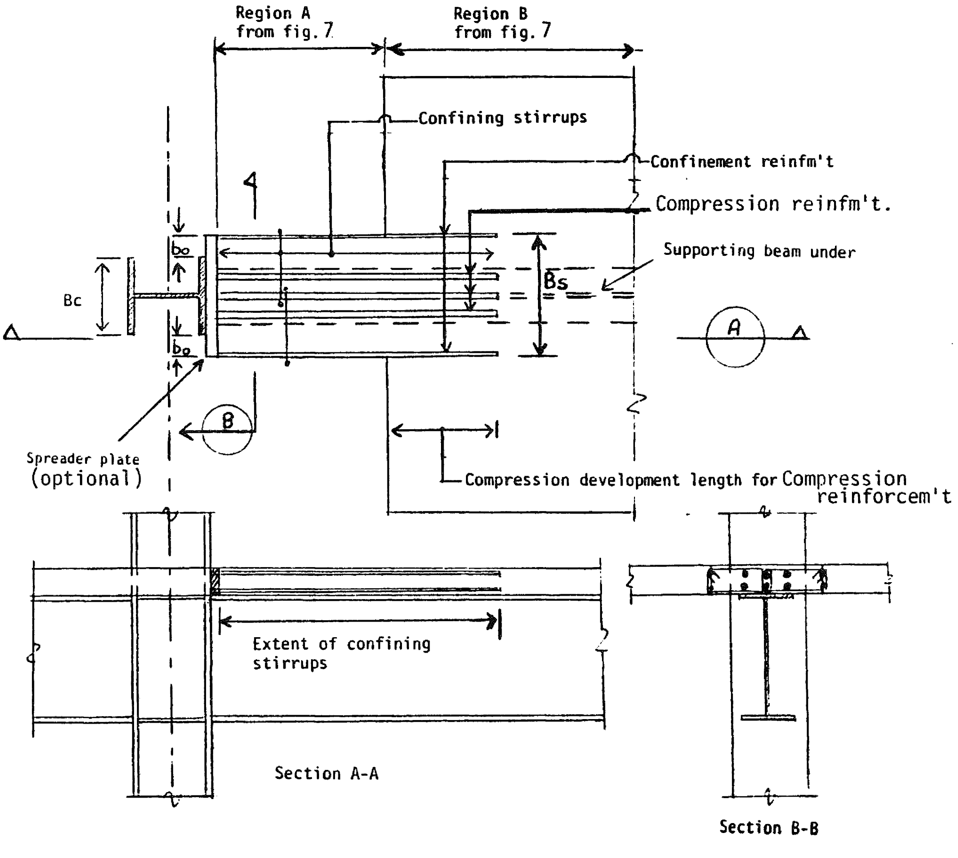


Fig. 8 : Compression Reinforcement in Composite Positive Moment Section at a Support

Notes: (B_c = width of column)

(B_c + 2bo = width of spreader plate)

The overstrength factors for this section are as obtained from Ref. 3 for the grade of supporting steel beam used.

Table 1 : Limitations on Web and Flanges of Unstiffened Steel Beam Used in Composite Construction in Negative Moment Regions

Steel Grade	Yield Stress (MPa)	Ductility Demand on Section		
		No or very low (ref. notes 1,2)	Low (ref. note 2)	High (ref. note 2)
250 (AS1204)	250	$\frac{b_1}{T} = 16.2$ $\frac{d_1}{t} \leqq 81(1-1.4\beta)$ for $0 \leqq \beta \leqq 0.33$ $\frac{d_1}{t} = 43$ for $\beta > 0.33$	$\frac{b_1}{T} = 8.6$ $\frac{d_1}{t} \leqq 71(1 - 1.4\beta)$ for $0 \leqq \beta \leqq 0.28$ $\frac{d_1}{t} = 43$ for $\beta > 0.28$	$\frac{b_1}{T} = 7.6$ $\frac{d_1}{t} \leqq 71(1-1.4\beta)$ for $0 \leqq \beta \leqq 0.28$ $\frac{d_1}{t} = 43$ for $\beta > 0.28$
350 (AS1204)	350	$\frac{b_1}{T} = 13.7$ $\frac{d_1}{t} \leqq 69(1-1.4\beta)$ for $0 \leqq \beta \leqq 0.33$ $\frac{d_1}{t} = 36$ for $\beta > 0.33$	$\frac{b_1}{T} = 7.3$ $\frac{d_1}{t} \leqq 60(1-1.4\beta)$ for $0 \leqq \beta \leqq 0.28$ $\frac{d_1}{t} = 36$ for $\beta > 0.28$	$\frac{b_1}{T} = 6.4$ $\frac{d_1}{t} \leqq 60(1-1.4\beta)$ for $0 \leqq \beta \leqq 0.28$ $\frac{d_1}{t} = 36$ for $\beta > 0.28$
55 (BS4360)	430	$\frac{b_1}{T} = 12.1$ $\frac{d_1}{t} \leqq 62(1-1.4\beta)$ for $0 \leqq \beta \leqq 0.33$ $\frac{d_1}{t} = 33$ for $\beta > 1.33$	Steel with $F_y = 360$ MPa not permitted in Low ductility demand region	Steel with $F_y = 360$ MPa not permitted in high ductility demand region

Definitions for Table 1:

1) b_1/T and d_1/t are as defined in AS 1250

2) $\beta = \frac{\phi_o A_r F_{sy}}{A_s F_y}$

$\beta \leqq \frac{0.4}{\phi_o}$ is recommended for maximum efficiency (Refs. 12,2)

where ϕ_o = overstrength factor, as defined in Section 4.4.3.
A_r = area of continuous slab reinforcement within slab effective width at supports
F_{sy} = yield stress of slab reinforcement
A_s = area of supporting steel beam
F_y = yield stress of supporting steel beam

Notes:

1) The requirements of this column of Table 1 are for no ductility demand but full section capacity required. If the applied moment maximum is less than 70% of the ultimate moment of resistance at the support and the appropriate elastic checks are carried out, then it would appear reasonable to relax the above limits. Reference 14 provides some guidance on this.

2) The limiting web d_1/t ratios given in Table 1 are quite severe. They can, however, be doubled if a stiffener is placed at midheight on the web in accordance with Fig. 9 and the associated notes.

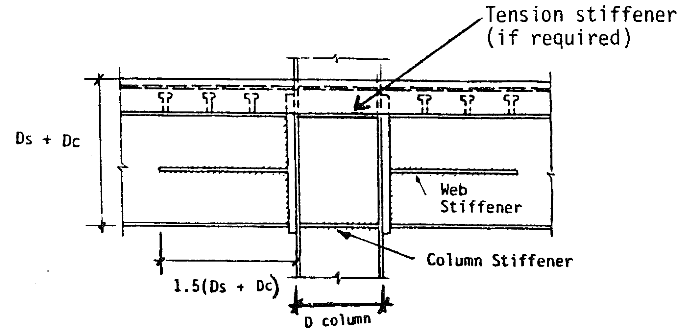


Fig. 9 : Web Stiffener Placement

Requirements for Web Stiffener in order that the Web (d_1/t) ratios given in Table 1 may be doubled:

1. The stiffener(s) must extend the distance shown in Fig. 9 from the support and be located at the midheight of the beam.
2. The stiffener(s) may be placed on one or both sides of the web as required provided the stiffness requirements of 3 below are met.
3. The stiffener(s) must be proportioned as for compression elements and the cross section should be chosen so that the moment of inertia about the beam Y-Y axis, taken about the beam web centreline is the same as for one beam flange I_{yy} taken about the beam web centreline.
4. The stiffener to beam web weld should be capable of transmitting the full squash load of the stiffener. It can be an intermittent fillet weld for sections not subject to inelastic ductility demand but must be continuous including around the exposed end of the stiffener for regions of sections subject to low or high ductility demand.

   The overstrength factors for this section are as determined by the grade of slab reinforcement, i.e.

   ϕo = 1.25 for grade 250, low ductility demand.

   ϕo = 1.40 for grade 380, low ductility demand.

   Above x 1.15 for high ductility demand.

4.4.4 Negative Moment at an Exterior Support

If the slab reinforcement at the support is anchored in such a manner that its full tensile strength can be provided at the support then the moment capacity can be taken as for an interior support acted upon by a negative moment. This anchorage can be into the external ring beam , in which case the beam must transfer the tensile load from the reinforcement into the column in bending and shear. Alternatively the reinforcement can be anchored to a rolled section placed against the external face of the column which carries the reinforcing load in shear and bending and transmits it by bearing load to the column external face.

If the slab reinforcement cannot development its full tension load the capacity of the section is as given by the steel section alone and the appropriate design rules apply.

4.4.5 Design for Vertical Shear

The seismic and non-seismic design procedures for vertical shear are identical and covered in detail in reference 1.

5. COMPOSITE BEAM-COLUMN CONNECTIONS

5.1 Introduction to, and Scope of Section 5

There are three classes of connection commonly used in composite construction. These are flexible or simple support connections, semi-rigid connections and (fully) rigid connections. Fig. 9 gives an example of a rigid connection and Fig. 10 an example of a semi-rigid connection.

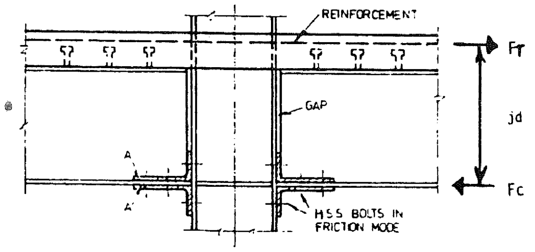


Fig. 10: Semi-Rigid Connection

The design of the beam to column connections is one of the starting points of the overall design. Until a decision has been made as to whether the structure is to be braced or unbraced, and if braced whether simply supported or partial fixity connections will be used (unbraced structures require rigid connections) it is impossible to analyse the frame or determine the beam moments.

References 1 and 5 provide guidance on the appropriate choice of connection.

Section 5 of this paper addresses only the seismic design considerations for each class of connection, in sections 5.2, 5.3 and 5.4 respectively for flexible, semi-rigid and rigid connections.

5.2 Flexible or Simple Support Connections

These cannot carry seismic moments directly but will subject to an induced moment under seismic loading due to the deformation of the overall structure. The likely inelastic deformations of the overall structure should be determined and hence the rotation imposed on the joint. Then the available rotation capacity of the joint should be checked to ensure that the separation between bottom flange and column face is adequate.

5.3 Semi-Rigid Connections

As with simple connections the only effect seismic loading has on these connections is the imposition of a joint rotation and the associated moment. The connection should be checked under this rotation to ensure its integrity and gravity load carrying capacity are not compromised. The degree of moment developed in a semi-rigid joint can be considerable, as can be appreciated from Fig. 10, where the moment F_T.jd can equal or exceed the plastic moment capacity of the steel beam alone. Under seismic induced joint rotation this will result in unbalanced moment transfer into the column. This unbalanced moment will induce high shear strains into the column panel zone which may be substantially higher than the shear strains imposed by gravity loads alone.

These may lead to panel zone shear failure unless the panel zone is adequately designed to carry the imposed shear as outlined in section 5.4.1.

5.4 Rigid Connections

The design procedure for these connections under seismic loading is similar to that for non-seismic loading. However the connections may connect beams which form the primary lateral load resisting elements for the structure and as such are subject to low or high ductility demand in the plastic hinge regions.

In this situation the connections must be designed to transmit the overstrength forces from the composite beam into the column. The overstrength forces are determined from section 4.4 of this paper or section 2.9 of reference 17, whichever are the greatest.

The connection itself must be designed and detailed to ensure satisfactory behaviour under the applied loads, for the relevant level of structure ductility demand. Furthermore it must be able to cope with a plastic hinge forming in the beam immediately adjacent to the connection under certain combinations of vertical and seismic lateral load. Reference 17 provides guidance on welding and bolting procedures suitable for the three classes of ductility demand and reference 16 provides more detailed guidance for bolted moment resisting endplate connections connecting beams subject to low or high ductility demand.

5.4.1 Joint Panel Zone Behaviour

In rigid unbrace frame structures the correct design of the joint panel zone is of great importance.

The panel zone is that area of column web bounded by the upper and lower surfaces of the connecting beam - i.e. in Fig. 9 it is the area of column web of length (Ds + Dc) and width Dcolumn. It is stressed primarily in shear due to unbalanced beam moments under seismic loading.

The behaviour of the panel zone of a composite and bare steel joint is quite similar and both are described in reference 18. Fig. 11 shows this behaviour clearly. In a normal elastic analysis of an unbraced frame the joints are assumed rigid and structure deflections calculated. The joints however possess some flexibility, mostly due to the flexibility of the panel zone. Provided the shear strains do not increase above $Fy/(\sqrt{3} \times G)$ (refer Fig. 11) the additional flexibility in the joints is small and is counteracted, in terms of lateral deflections and the production of additional P-Δmoments, by the analysis being based on the member length between centre-line intersections. If however the panel zones reach a general state of yield the structure lateral deflection increases markedly and strength decreases.

Reference 16 gives the appropriate procedure for non-composite panel zones and reference 18 the procedure for composite panel zones to ensure that the joint shear strains do not exceed $(Fy/\sqrt{3}\ G)$. The internal actions on the column from a composite beam section are shown in Fig. 12 and are somewhat different to those from a steel beam alone. Reference 1 describes how the composite joint design procedure takes account of these forces in the panel zone design.

It is important that the concrete in composite panel zone regions is adequately confined, especially if members framing into the joint are subject to low or high levels of ductility demand adjacent to the joint. The basic concept is to provide confinement by means of intersecting U bars, inserted through holes (predrilled) in the beam web, as shown in Fig. 13. Reference 1 gives two detailed procedures for ensuring adequate confinement of the panel zone concrete, one for joints with no members subject to inelastic ductility demand (i.e. all members are category 3 in section 2.2) and a more stringent procedure for joints carrying members subject to low or high ductility demand adjacent to the joint.

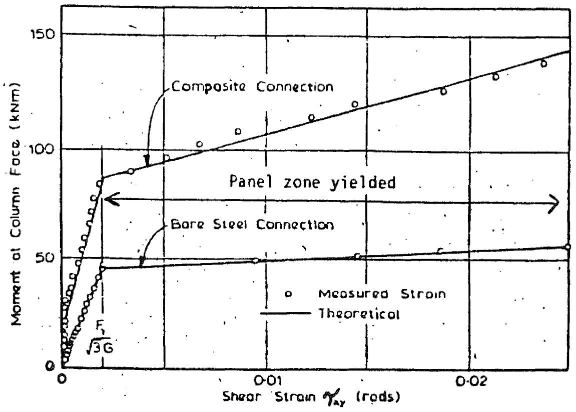


Fig. 11 : Stress Strain Curves for Panel Zone

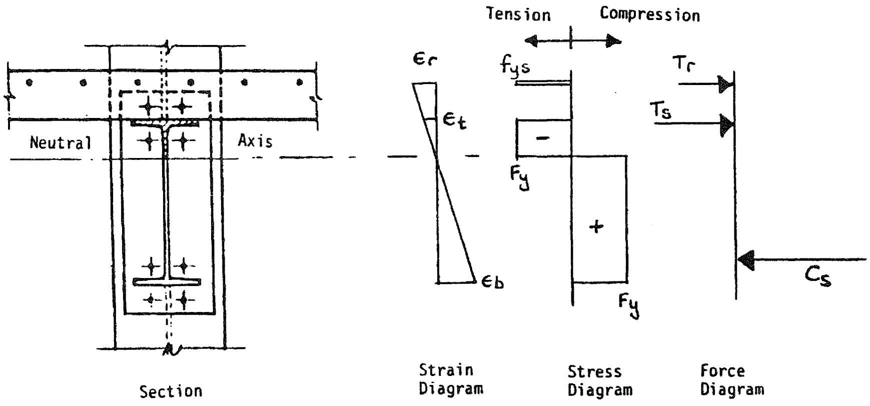


Fig. 12 : Internal Actions on Composite Sections Subject to Negative (Ultimate) Moment

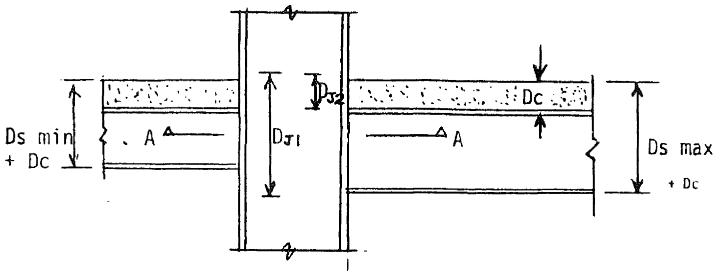


Fig. 13(a) : Joint Confinement Concept (from reference 1)

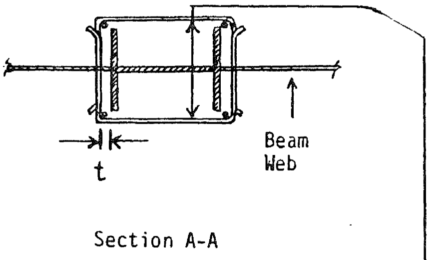


Fig. 13(b) : Intersecting deformed U-bars as shown inserted through holes predrilled in beam web, extending 50 mm above and below the maximum joint depth (Ds max + Dc).

6. STUB GIRDERS

6.1 Introduction to, and Scope of, Section 6

Stub girders arose from the increasing need to span large distances with the least possible depth of beam whilst maintaining freedom to pass services through the beam without having to resort to cutting holes in the web. The general concept of a stub-girder system and its interaction with building services is shown in Fig. 14.

Reference 1 and the associated references called up therein provides general design details including a means of undertaking a rapid preliminary design for member sizing purposes and a more comprehensive design. Reference 20 provides an extremely detailed coverage of stub girder floor systems, but has not been fully evaluated by the author at this time to assess its suitability for New Zealand design. It is based on Canadian Limit State Design procedures so should be at least broadly compatible with New Zealand strength design procedures.

Thus references 1 and 20 provide detailed design guidance and section 6.2 of this paper gives some general seismic guidance on the design of stub girders.

6.2 Seismic Design Considerations

Stub girders can be made fixed or simply supported at the supports. For fixity at interior supports the slab reinforcement is made continuous across the supports whilst at exterior supports it is anchored to the column in the same manner as for a normal composite steel beam at an exterior support.

Such rigid connections are suitable for category 3 stub girders under seismic loading (very low ductility demand on girder). In theory a stub girder can be designed and detailed to provide low and high ductility demand hinge capacity, however no experimental work has been undertaken on this to date.

Stub girder beams, incorporating rigid connections, in a moment resisting frame will almost always be gravity dominated. Hence it is very likely that they will form only negative moment plastic hinges. They can be used in low-rise structures with the hinges designed to occur in the columns, at least in the direction of the stub-girder. This does not involve any new concepts in seismic design to implement.

Alternatively they can be used to carry vertical loads in a braced frame, with simply supported connections used - i.e. the support conditions as shown in Fig. 14 used in design. The designer must check that the connections can fulfil their load carrying function under the imposed joint rotations likely to occur during an earthquake.

7. COMPOSITE COLUMNS

7.1 Introduction to, and Scope of, Section 7

Many steel columns are encased in concrete, however few, if any, of these are proper composite columns. A fully composite column involves strain compatibility between the steel section and surrounding concrete, with the section capacity under axial load and moments being derived from integral action of both materials. The amount of longitudinal and especially transverse stirrup reinforcement required for this is very much greater than that normally used in concrete encased steel column design.

However, as concrete encasement can alter a steel column's behaviour, then all cases of concrete encasing are covered in this paper with respect to seismic considerations and in reference 1 where full details or appropriate references are given.

There are three cases to consider. The first is a steel section encased solely for fire resistance purposes. This involves minimal stirrups and no longitudinal reinforcement. The second case is a column encased with the surrounding concrete used to increase the sections resistance to local and lateral buckling. The third case is full composite action achieved between the steel section and surrounding concrete. Seismic design aspects of these three cases are covered in sections 7.2, 7.3 and 7.4 respectively.

7.2 Concrete Encasement to Provide Fire Protection Only

When concrete encasement is not used to increase the section strength or resist local or lateral section buckling, but to provide resistance against fire, then the reinforcement within the concrete need be sufficient only for the purpose. Typically this reinforcement will consist of steel binding wire of not less than 2.3 mm thickness or mesh heavier than 0.48 kg/m^2 at a maximum spacing of 150 mm.

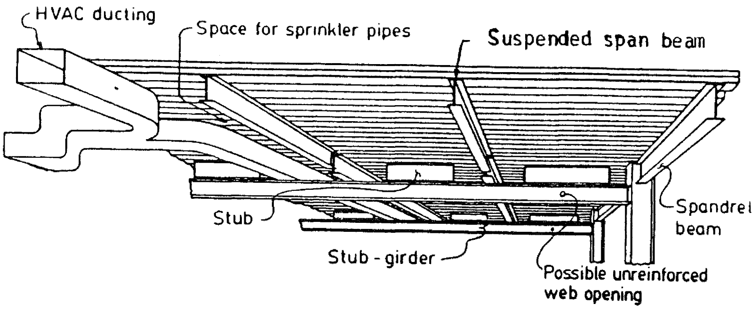


Fig. 14 : Stub Girder System showing Interaction with Building Services

Where plastic hinges are designed to form in the columns, either as primary energy absorbing mechanisms or as a last resort under overloading from capacity design, the effects of concrete encasement with this level of reinforcing can be ignored. In the case of encasement of primary hinge positions the concrete will commence spalling at compression strains of 0.0018 (reference 22). Hence spalling will commence before the steel section obtains its plastic moment. The concrete is effectively unconfined at the very low level of reinforcement required for fire protection and will separate from the steel section without increasing the section capacity to any extent. The overstrength factors applied to the steel plastic hinges will cover any increase in strength due to concrete encasement. In case of potential plastic hinge zones arising from overstrength forces generated by the primary hinging members any additional section strength will delay hinging and hence be beneficial. The concrete encasement will not affect the ductility of the steel section plastic hinge.

7.3 Concrete Encasement for Fire Protection Used to Increase Section Buckling Resistance

Encasing the steel section, provided the encasement is reinforced to the minimum required level (in excess of that given in section 7.2), increases its resistance to lateral and local buckling.

AS1250:1981 covers the maximum permissible design strengths and concrete reinforcement requirements in clause 6.2 for columns and clause 5.6 for beams. The reinforcement should comply with NZS 3402P: 1973 instead of AS1302 otherwise the requirements can be used directly for working stress design. For strength design the maximum loads and/or stresses should be multiplied by $^1/_{0.6}$.

The provisions do not provide a section capable of full composite action. The maximum encased section moment capacity is still that of the uncased section neglecting lateral buckling. The maximum compression load can be increased above that of the uncased section and may exceed the squash load under strength design. However the concrete confinement cannot withstand any significant ductility demand without severe spalling and once spalling occurs the stirrups cannot contain the enclosed concrete adequately. Hence loss of compression concrete under low or high ductility demand will occur, with subsequent reduction of the section alone and reduction of the axial load carrying capacity to that of the squash load on the steel section.

Therefore when an encased section, designed to the requirements of AS1250 clause 6.2 for columns or clause 5.6 for beams, is expected to form a plastic hinge at a given position under seismic loading the following points should be considered in the design:

1. The section between plastic hinge zones can be designed using the encasement to increase the lateral buckling capacity and increase the axial load capacity as required.

2. Within the plastic hinge zone the section design must be based on the capacity of the steel section alone. The normal compact section rules for uncased steel beams will apply to prevent premature local buckling of the section within the plastic hinge zone when the cover is lost under seismic loading.

3. The minimum amount of stirrup confinement required may increase the section capacity under inelastic cyclic loading for at least the first few load cycles. This increase may well be greater than the overstrength capacity of the steel section alone. For sections forming the primary lateral load resisting elements in a structure subject to capacity design the stirrup spacing should be at 300 mm crs over the hinge zones (use a hinge length of 1.5 x (section depth).

This will ensure immediate spalling of the concrete under inelastic action. For sections other than the primary elements in a capacity design, where plastic hinges may form under severe overload, the ductility of a compact steel section is not impaired by encasement.

7.4 Full Composite Design

7.4.1 Introduction

The calculation of section capacity for a composite column follows exactly the same procedure as for a reinforced concrete column and is covered in references 1 and 22.

Any composite column forms part of an overall structure and the design of the column must take account of the effect of the surrounding structure. An uncased steel section design to AS1250:1981 takes into account the following:

(i) Effect of inplane restraint of the column through the calculation of the column effective length.

(ii) Effect of lateral-torsional and local buckling due to applied moments, axial load, in determining the moment capacity.

(iii) Effect of lateral buckling under the axial load, dependant on the slenderness ratio.

(iv) Magnification of any applied forces on the column dependant on the ratio of the actual compressive load to the Eular buckling load.

A composite column is subject to all the above except that the effect of (ii), (iii) and (iv) are much less significant on a composite column than they are on a bare steel column. This is due to the increased stockiness of a composite column, and the absence of thin, highly stressed and unrestrained sections within the column cross-section.

Limitations on slenderness ratios for reinforced concrete columns are given in NZS 3101: Part 1 Clause 6.5.2. If these limits are adhered to for composite columns then the problem of (lateral) buckling - (ii) and (iii) above - of the column under compressive load and/or moment is eliminated. However allowance for magnification of any applied forces on the column (dependant on the ratio of the actual compressive load to Eular buckling load) is not covered by simply adhering to the slenderness ratio limitations given in Clause 6.5.2 of NZS 3101. The effective length about each principal axis must be determined and the resulting slenderness ratio assessed as stocky or slender for each principal axis. If the column is thus determined to be slender then a moment magnification procedure must be applied to the column design, for both braced and unbraced columns.

The procedures given in NZS 3101 for reinforced concrete column design can be adapted to composite column design, however they have their limitations. A comprehensive moment magnification procedure is given for braced columns, however no guidance of any sort is given for moment magnification in unbraced columns. This is despite the fact that an unbraced column which has a relatively slender cross section (i.e. just complies with the dimensional requirements of NZS 3101) will attract a significant degree of moment magnification due to slenderness effects. Also composite columns can often be subjected to biaxial bending and there is no design procedure in NZS 3101 to explicitly cover this.

The recently produced United Kingdom limit state codes present a design procedure which incorporates column effective length determination, moment magnification and biaxial bending. Reference 1 provides details of this procedure, which is taken from reference 23.

One of the fundamental requirements of full composite action is that no slip occurs between the structural steel section and surrounding concrete core. Any tendency for slip to occur between the two materials can be prevented by means of confining stirrup reinforcement, placed in sufficient quantities at close centres.

For general composite column design reference 1 gives recommendations for the quantity and spacing of stirrups required to achieve full composite action.

Section 7.4.2 of this paper covers the seismic recommendations for full composite column design.

7.4.2 Seismic Considerations

The calculation of section capacity follows the same procedures for seismic loading as for non-seismic loading. However in an unbraced frame the determination of the column effective lengths and column design actions needs some clarification and the transverse reinforcement requirements within potential plastic hinge zones must be given. These are now covered in turn:

(a) Effective Lengths and Moment Magnification

The determination of exact effective lengths in an inelastically responding frame is very difficult, however determination of the effective lengths should be undertaken according to the normal method given in appendix E of AS1250, modified to account for composite action as detailed in reference 1, section 6.4.1.

The effective length thus determined can be used to determine the extent of moment magnification required due to column slenderness effects. If, for any composite column, overall column dimensions are such that equations 7.1 are satisfied, the column can be considered stocky and no moment magnification due to slenderness effects need be considered.

$$\ell_x/b_x \leq 10 \text{ and } \ell_y/b_y \leq 10 \quad (7.1)$$

where: ℓ_x, ℓ_y = effective length about each principal axis

b_x, b_y = column width about each principal axis

(b) Determination of Column Design Actions

The column design moments can be based on the moments resulting from a linear elastic analysis, factored at each end of the column by the appropriate beam overstrength ratios. The dynamic magnification factor given in section C3.A5 of reference 10 need not be applied to composite columns just as it need not be applied to bare steel column design.

However the axial load reduction factors given in section C3.A6 and table C3.A2 of reference 10 should be applied. The shear forces should be based on the overstrength column moments divided by the clear storey height.

(c) Transverse Reinforcement Requirements

In potential plastic hinge zones the amount of transverse reinforcement required is given by the following NZS 3101 equations:

(a) For reinforcement consisting of spirals or circular hoops - the greatest volumetric ratio given by equations 6.22, 6.23 or 6.3 with $\phi = 0.9$.

(b) For reinforcement consisting of rectangular stirrups (hoops) - the greatest total area given by equations 6.24, 6.25 or 6.4 with $\phi = 0.9$.

The maximum spacing within the plastic hinge zone should comply with clause 6.5.4.3 a(3) for spirals or circular hoops and clause 6.5.4.3 b(3) for rectangular stirrups. With rectangular stirrups the requirements for supplementary cross ties every 200 mm can be ignored. However the yield force requirement of clause 6.5.4.3 b(6) should be met.

In the zone adjacent to the plastic hinge the requirements of clause 6.5.4.3 d should be followed.

Away from this region the transverse reinforcement can be placed according to section 6.4.2.3 of reference 1.

(d) Longitudinal Reinforcement Requirements

The diameter of additional reinforcement placed within the section should comply with the requirements of NZS 3101:1982

8. COMPOSITE SHEAR WALLS

Steel shear walls have been used to provide additional seismic strengthening of existing buildings. Reference 25 describes one application to an existing reinforced concrete hospital building, giving details of the design and installation of walls.

They are more expensive than reinforced concrete shear walls in terms of material costs only but can be placed in the interior of a building with minimal disruption to the existing structure and services.

One design problem with the steel plated shear walls is the prevention of out of plane elastic buckling of the steel panels prior to the ultimate strength of the wall being developed. A factor of safety of 4 against elastic buckling was considered necessary in reference 25 due to uncertainties in panel distortion due to welding, initial imperfections. Because of this the panels required edge stiffening at relatively close centres.

One way of preventing out of plane buckling is to confine the steel plates in a reinforced concrete sandwich. This method was used in a new building constructed near San Francisco. The details are given in reference 26.

The steel plates were designed to take the total shear force in the wall. The concrete cover, varying from 130 to 380 mm on each side of the plate provided additional stiffness and prevented out of plane buckling. It was reinforced to the extent of being able to carry comparable strains to the steel plates without loss of confinement. Figs 15 and 16 are taken from reference 26 and show details of the composite shear wall. The dimensions have been converted to metric units.

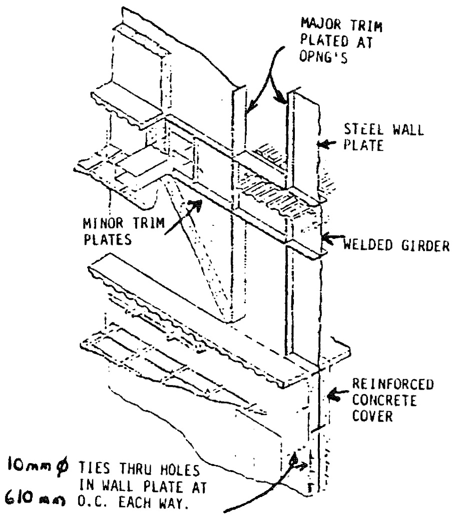


Fig. 15 : Elevation of Plated Wall

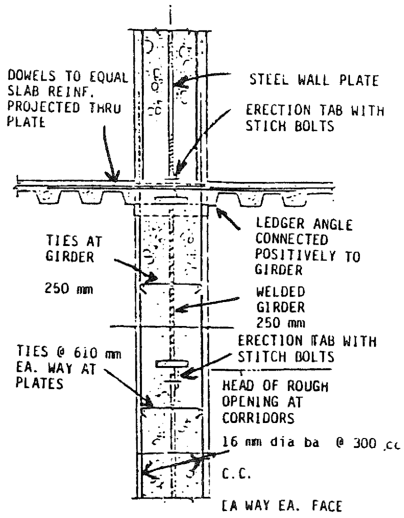


Fig 16 : Section Through Plated Wall

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