Composite Steel-Concrete Construction for New Zealand

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ABSTRACT: Composite steel-concrete construction uses steel and concrete together to obtain a system with better performance, and/or lower cost, than using either material alone. This paper evaluates the advantages and disadvantages of a number of composite structural systems which have been are proposed/used around the world in terms of likely cost and performance, (ii) likely situations for composite construction in New Zealand are specified, and (iii) a comparison of the application of a conventional steel moment resisting one-way frame system, with identically similarly performing composite one using rectangular concrete filled steel tubular (CFT) columns is made considering design details and cost. It is shown that for the studies conducted on one-way frames, composite CFT column construction with beam end-plate connections was generally more expensive than conventional steel column construction.

INTRODUCTION

A steel-concrete composite structural member contains both structural steel and concrete elements which work together. There are many combinations between structural steel and concrete. For example, a concrete slab on a steel beam with mechanical shear connectors allows the slab and beam to resist bending moment together. Steel-reinforced concrete column (SRC), comprising a structural steel core surrounded by reinforced concrete, is used when an exposed concrete surface is required and when concrete is to protect the steel core from fire. In a concrete-filled steel tube column (CFT or CFST) the hollow steel tube is filled with concrete, with or without reinforcing bars. Here, the steel element contributes tensile capacity, provides confinement to concrete elements, and reduces concrete shrinkage while concrete element prevents steel from premature local buckling and fatigue.

Connections in composite structural system differ from conventional connections in steel system due to different force transfer mechanism and constructability. There are many types have been proposed and tested in many countries, mostly in the U.S., China and Japan. For moment-frame structures, these connections can be categorised into beam-column connections and column splices.

While many studies have been undertaken in the past, in general there is still a lack of understanding of the composite action (strength, stiffness and ductility) of members and connections in seismic frames, and robust design guidance and examples both in NZ and overseas. Also, there may be the perception that a composite system may have a higher overall construction cost than traditional steel construction. For these reasons, design of composite structural systems is not yet become popular in NZ. In order to address some of the issues described above, this paper seeks to address the following questions for moment frame structures with composite CFT structural columns:

- What types of composite seismic moment frame beam-to-column connection are commonly used overseas or seem promising for NZ?

- How does the cost of a moment frame with CFT columns compare to a similar frame with steel columns?
- What other factors, not included in the study above, are likely to effect the relative economy of frames with and without composite columns?

LITERATURE SUMMARY

The first study of steel-concrete composite members began as early as 1908 at Columbia University (Viest, Colaco et al. 1997). The combined material strength was not appreciated in the early days and the design concept considered two individual materials by either conservatively neglecting the contribution from one or another or by adding them separately. An early composite beam system that gained popularity was a concrete slab on steel beam with mechanical shear connectors. Later, other composite forms including concrete filled steel tube (CFT) construction where concrete is placed in a hollow steel member, reinforced-concrete steel (RCS) construction with RC columns and steel beams, and construction with steel-reinforced concrete (SRC) columns (ref), became popular. SRC columns involves with steel members surrounded by concrete. This paper concentrates on CFT construction.

In high rise building construction, CFT construction allows efficient rapid staged construction. Steel tubes are usually erected on construction site, and connected to beams before concrete is placed inside the tubes. Steel workers can erect the steel tube frame a few levels ahead of the concrete work. The steel tubes alone are designed to carry dead and construction loads associated with the construction sequence.

Steel tubes of CFT columns must be provided with vent holes at the bottom to avoid steam pressure built up inside the CFT column in the event of fire. The same hole is also used to drain residual water before concrete pouring. Corus (2002) recommended two-20mm diameter holes diametrically opposite to each other, at top and bottom of every storey heights are sufficient. Attention is to be paid to ensure the holes are not blocked by subsequent construction.

Concrete filled steel tubular (CFT) construction

Studies on CFT in Japan began as early as 1960. Also, significant studies were conducted as part of the fifth phase of U.S.-Japan Cooperative Research Programme in 1993 (Goel, 1998) which included projects on CFT column systems. Some advantages of the CFT column system have been described by (Morino and Tsuda 2003) as:

- Premature local buckling of steel tube is delayed and strength degradation is moderate due to the restraining effect from concrete.
- Concrete can develop higher compressive strength due to confining effect from the steel tube.
- Strength degradation of concrete is not so severe due to spalling is prevented by steel tube.
- Creep and drying shrinkage of concrete infill is smaller than conventional exposed concrete.
- Steel element in CFT is well plastified due to the outermost location in the section.
- Concrete improves fire resistance of the steel tube.
- No concrete formwork or reinforcing bars are required. Hence labour, construction time, and cost are reduced.
- Construction site is cleaner and produces less waste.
- Concrete in CFTs can be easily crushed and separated from steel tube. Hence both materials can be entirely recycled.

Axially loaded CFT members

The compressive strength of short CFT compression members is higher than squash load of concrete column due to confinement provided by steel tube. An ineffective confining area in a rectangular CFT column makes the confining effect smaller than for a circular CFT column. AISC360 (2010) considers this by using a C_2 parameter (average stress for rectangular stress block) equal to 0.95 for circular sections and 0.85 for rectangular ones. This difference is not reflected in NZS3101 (2006). Also, for short CFT columns, the axial strength is governed by the local buckling of the steel tube while for slender columns global buckling controls.

An initial axial stiffness of a CFT member can be approximated from the sum of stiffness of two materials. However, if differential longitudinal shrinkage of the concrete occurs, then the stiffness will tend to become closer to that of the steel tube alone.

Beam-column CFT members

The effect of the steel tube providing confinement may increase the flexural strength of the CFT member. Therefore the composite flexural strength is greater than the combined strength of the individual materials combined. A circular CFT column also gains benefits from confinement to a greater extent and has more ductility than a rectangular section.

Beam-column joints and connections

Several methods have been used to connect the beams to the columns in moment-frames (e.g. Alostaz, 1996). CFT joint details can have different levels of fabrication difficulty and stiffness properties. In general, welding the beam directly to the tube skin is quite flexible, while passing a beam through the CFT column is quite rigid.

Concrete infill usually increases the strength of joint panel zone compared to that for a bare steel column alone. This allows a CFT joint to be designed to remain elastic during extreme seismic shaking. It is also possible to locate locations of beam non-linear action away from the joint. This non-linearity may be a result of beam yielding, or it may be due to low-damage connections which are easily used again after a major earthquake event. Designing low-damage connections in the beam away from the joint and protecting the joint by capacity design, can result in very desirable behaviour. Furthermore, since the joint does not yield, monotonic rather than cyclic tests are required to assess the joint behaviour.

Through-beam connections

Some testing of beams passing through CFT columns has been undertaken (Deirlein 1988; Elremaily and Azizinamini 2001). Here the steel I-shape beam passes through the CFT column through a pre-cut I-shape slot in the tube and it is welded to the tube. These through-beam connections are very stiff and ductile.

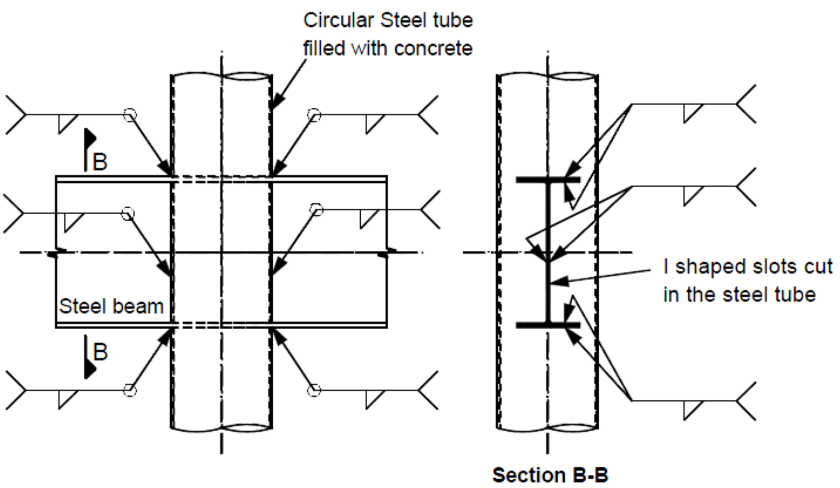
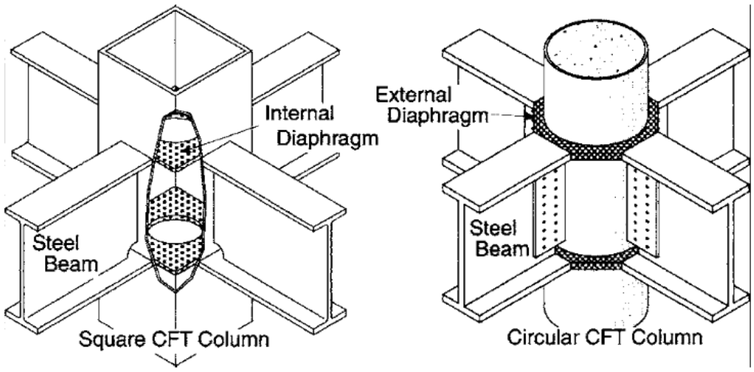


Figure 1. Through-beam connection (Elremaily and Azizinamini 2001)

Diaphragm type connections

Diaphragm type beam-column connections are widely used in Japan and Taiwan in two-way and one-way frames. Often through-diaphragms are used where full penetration butt welds are required in the column tube either side of each diaphragm. This is not only expensive, but it also provides more opportunities for defects to result in poor behaviour. Internal diaphragms may also be used, but electroslag welding process for completing the final side requires specialist equipment not currently available in NZ and very good quality control for good performance. While it is used overseas (e.g. Taiwan), its performance is not always good. External diaphragms are much more cost effective and do not require the same weld quality, but they do not provide the connection with the concrete that the other methods have. Both internal or through-diaphragms require a hole to allow concrete pouring. The presence of diaphragm inside can interrupt concrete flow and result in poorly compacted concrete under the diaphragm. The problem is not so severe in high rise construction where concrete pump-up method and high slump concrete are used. On the other hand, external diaphragms may be less efficient in terms of force transfer mechanism but that can be offset by the ease of fabrication (Shin, Kim et al. 2004).

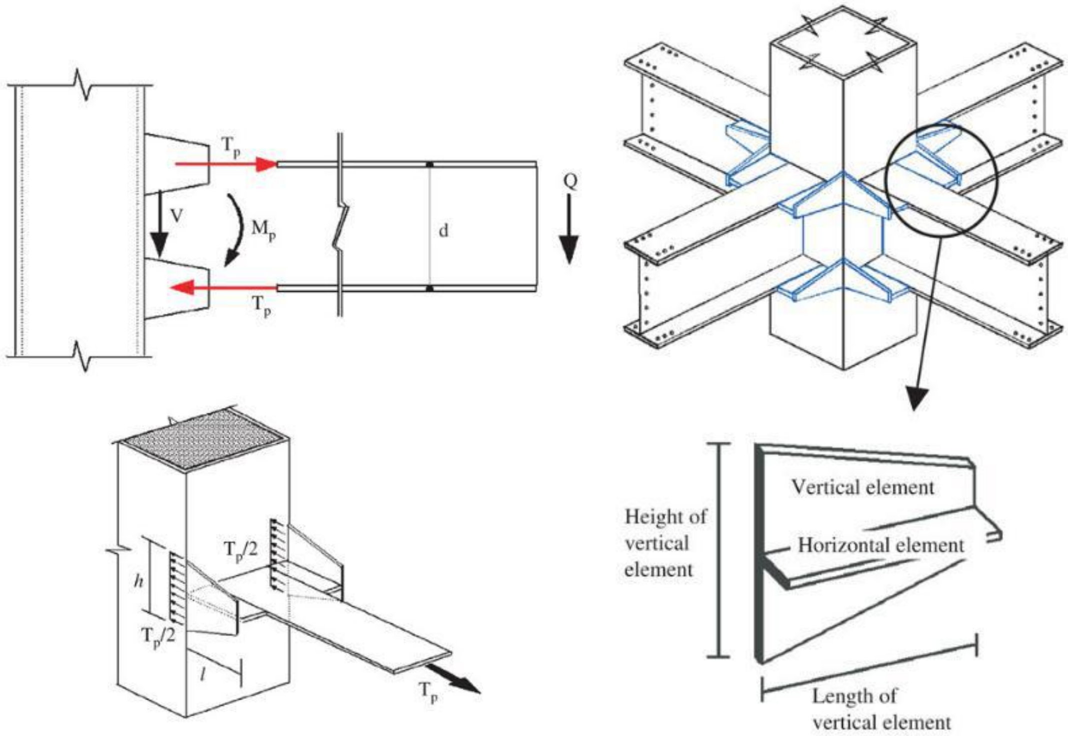


(a) Internal Diaphragm (b) External Diaphragm

Figure 2. Typical diaphragm type CFT connections (Toshiyuki and Koji 2005)

T-stiffeners

(Shin, Kim et al. 2004) investigated a T-stiffener connection to a rectangular CFT. Here beam flange tensile force is transferred through a horizontal plate element, a vertical plate element, and through the web of the rectangular CFT as shown in Figure 3. The aim is to move plasticity away from the column and the joint(Shin, Kim et al. 2004)(Shin, Kim et al. 2004). However, the load path is not direct and in most of the tests failure occurred by fracture at the welding to the bracket.



(a) Forces (b) Configuration

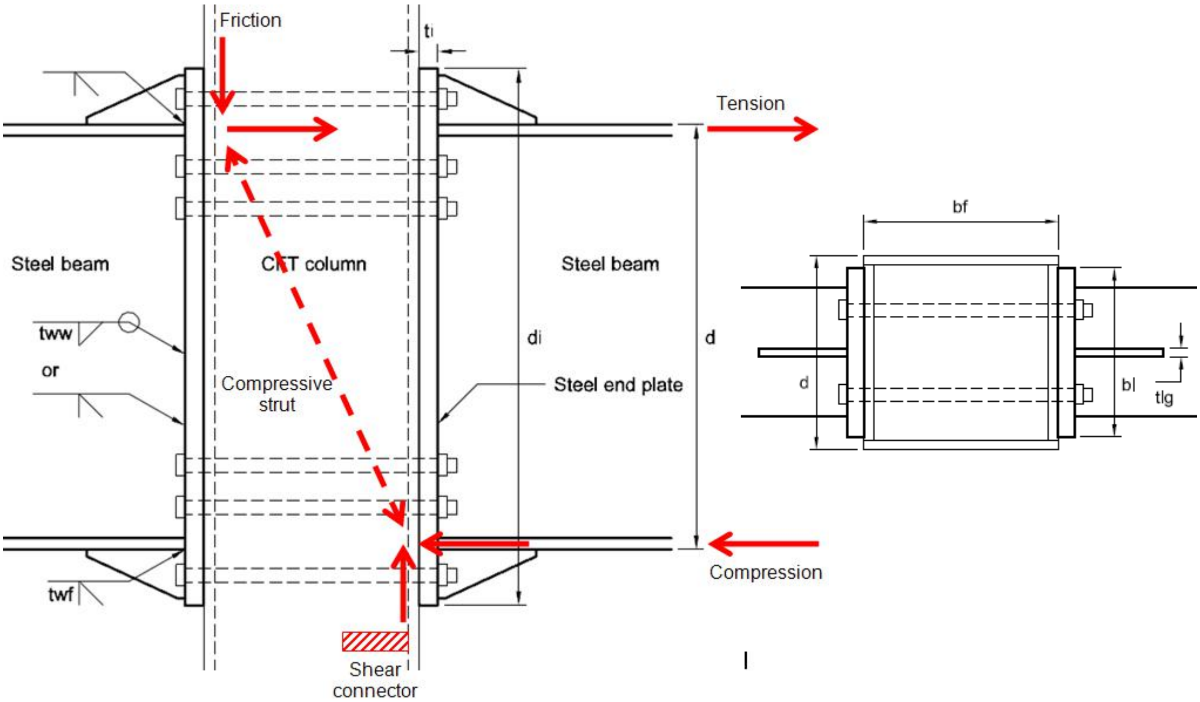
Figure 3. T-stiffener connection and force transfer mechanism (Shin, Kim et al. 2008)

Bolted moment endplate connections

Conventional bolted moment endplate connections for bare steel connection can be applied to CFT connections by replacing bolts with through-bolts (i.e. rods passing through the column). The purpose is to transfer beam flange tension to the other side of CFT through the bolts and become compression on the CFT and filled concrete, rather than transferring beam flange tension directly to the steel tube skin on the connection side. For the case of flange compression, diagonal compressive strut develops in the filled concrete in the joint region to transfer the compressive force to the other side. The vertical component of the diagonal strut is resisted by friction at the surface between the steel tube and concrete as shown in Figure 4. Additional mechanical anchors or shear studs, acting together with the through-bolts, may also be used to resist the vertical force from the diagonal strut especially if the tube inner surface is smooth.

Alternatives to through-bolts (i.e. threaded mild steel rods with or without a sleeve passing through the connection) include concrete anchor bolts, headed shear studs welded inside the tubes, etc. While these do not pull directly on the tube, and have behaved well in a number of experiments (e.g. Goldsworthy, 2011); they rely on tension in the concrete and performance may vary with concrete of different properties.

Two way CFT beam-column connections can be designed by detailing bolts or rods to be slightly offset so they do not clash inside the tube.



a) Elevation Showing Joint Strut ***b) Plan***

Figure 4. One-Way Bolted Moment Endplate Connection with Haunches and Through-Rods

This sort of connection behaved well in experiments (E.g. Li et al., 2009). (Wu, Chung et al. 2005) showed that for very thin tube webs on the steel tubes, panel zone yielding occurred even when concrete infill was present.

While conceptually this type of connection is easy to specify and construct, contractors generally make the beams a few millimetres shorter than specified, so that they can adjust the actual length and get a good fit with shims. If the columns are stiff, the prestress across the end-plate column interface may differ at different levels of the frame.

Bolted T-stub connections

Bolted T-stub connections, such as that shown in Figure 5, eliminate the need for welding the beam flange to the endplate or column flange. It also provides greater construction tolerances than the end plate connection. This type of connection gained significant attention in the US after Northridge earthquake event when a number of weld fracture failures occurred in beams welded directly to columns. T-stub connections can be designed to have either fully-rigid or semi-rigid behaviour by proportioning strength and stiffness of the connection elements relative to the beam.

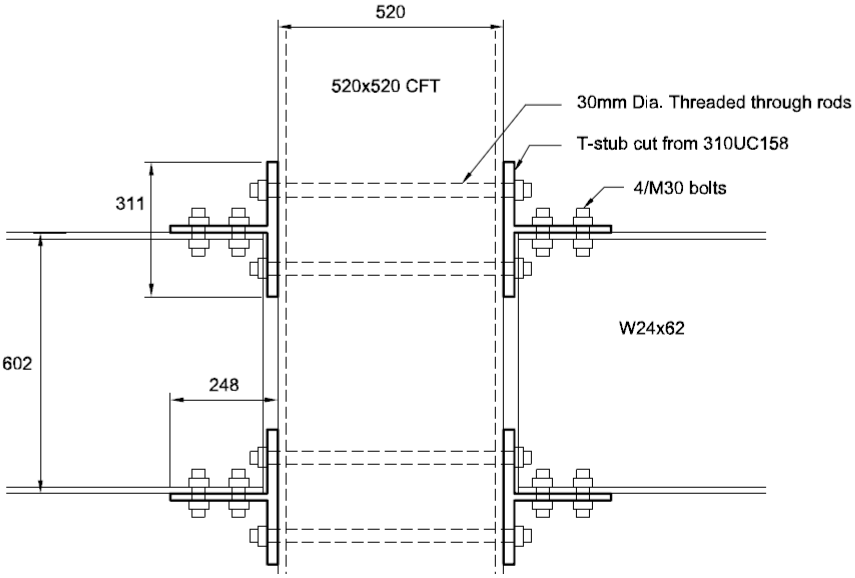


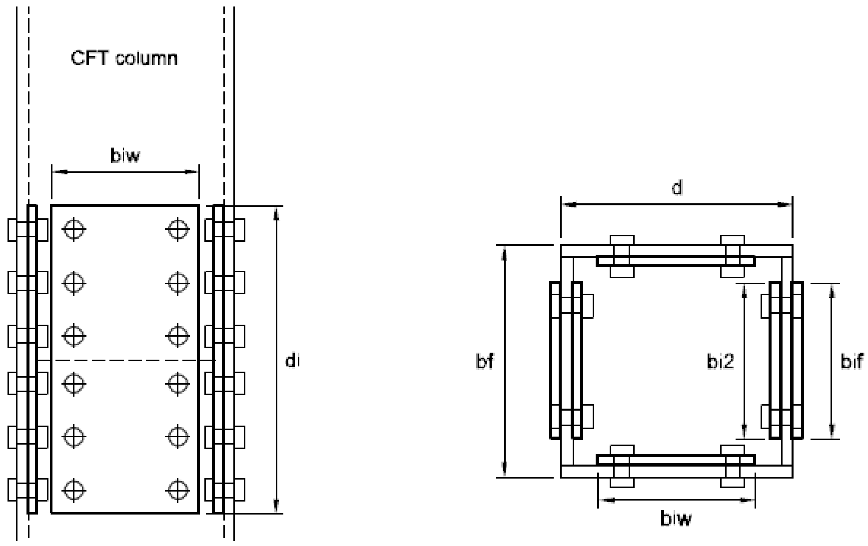
Figure 5. T-stub connection

Column splices

Column splices for CFT members can be designed and built in the same way as for normal steel tubes. Bolts and splice plates transfer tension from one tube to another tube. Filled concrete is not considered to contribute to the tension transfer. The disadvantage of splices in tubular sections over that in open sections is the difficulty of installing backing plates and bolts in large section.

One simple technique is to tack weld the plate with one steel tube before connecting to the other tube. However, often further bolt hole alignment adjustment is needed. Common practice is "forcing", by shaking or hitting, the tube until the holes are aligned and the bolts can be inserted. The backing plate with tag weld can be broke off.

The bolts for CFT splices can be through-bolts where only external splice plates are used. However, when internal backing plates are required, a fastener that can be inserted and fastened from one side of the hole can be used. For example, the AJAX "One Side" bolt (AJAX 2012) can be used to fasten external and internal splice plates.



a) Elevation b) Plan

Figure 6. Column Bolted Splice

FRAME DESIGN

Bare steel frame

The bare steel frame selected for study in this report is modified from Post-Northridge Design for Los Angeles site from the SAC steel project. Complete details can be found in Appendix B of (FEMA-355C 2000).

The office building has a square layout with perimeter moment resisting frames on all sides. Each side has five bays of 9.1m (30’). All interior frames are considered to be part of the gravity load carrying system. The frame is modified to have nine storeys and no basement. The storeys heights are 4m (13’) except 5.5m (18’) on the ground floor. Yield stresses of steel are 350MPa (50ksi) and 252MPa (36ksi) for columns and beams respectively. A concrete compressive strength of 30MPa is assumed for the CFT columns.

Columns

Steel wide-flange column sizes are those for SAC steel frame and they are summarised in Table 1 and Table 2. The axial and flexural strengths including interaction between them were calculated based on NZS3404:1997 provisions. There was no strength reduction due to buckling or out-of-plane effects due as they were rather stocky. All section plate elements were compact.

Table 1. Bare Steel Moment Resisting Frame. Exterior Seismic Columns

Storey	US section	NZS3404 Dependable Strength			EI	
		Flexure (kNm)	Axial comp (kN)	Axial tension (kN)	X axis (mm^4)	Y axis (mm^4)
9	W14x233	2251	13921	13921	2.51E+14	9.57E+13
8	W14x257	2514	15364	15364	2.83E+14	1.07E+14
7	W14x257	2514	15364	15364	2.83E+14	1.07E+14
6	W14x283	2798	16929	16929	3.20E+14	1.20E+14
5	W14x283	2798	16929	16929	3.20E+14	1.20E+14
4	W14x370	3799	22152	22152	4.53E+14	1.66E+14
3	W14x370	3799	22152	22152	4.53E+14	1.66E+14
2	W14x370	3799	22152	22152	4.53E+14	1.66E+14
1	W14x370	3799	22152	22152	4.53E+14	1.66E+14

Table 2. Bare Steel Moment Resisting Frame. Interior Seismic Columns

Storey	US section	NZS3404 Factored Dependable Strength			EI	
		Flexure (kNm)	Axial comp. (kN)	Axial tension (kN)	X axis (mm^4)	Y axis (mm^4)
9	W14x257	2514	15364	15364	2.83E+14	1.07E+14
8	W14x283	2798	16929	16929	3.20E+14	1.20E+14
7	W14x283	2798	16929	16929	3.20E+14	1.20E+14
6	W14x370	3799	22152	22152	4.53E+14	1.66E+14
5	W14x370	3799	22152	22152	4.53E+14	1.66E+14
4	W14x455	4832	27232	27232	5.99E+14	2.13E+14
3	W14x455	4832	27232	27232	5.99E+14	2.13E+14
2	W14x500	5420	29874	29874	6.83E+14	2.40E+14
1	W14x500	5420	29874	29874	6.83E+14	2.40E+14

Table 3. Bare Steel Gravity Columns

Storey	US section	NZS3404 Dependable Strength			EI	
		Flexure (kNm)	Axial comp. (kN)	Axial tension (kN)	X axis (mm^4)	Y axis (mm^4)
9	W14x61	527	3638	3638	5.33E+13	8.91E+12
8	W14x90	810	5385	5385	8.32E+13	3.01E+13
7	W14x90	810	5385	5385	8.32E+13	3.01E+13
6	W14x120	1094	7174	7174	1.15E+14	4.12E+13
5	W14x120	1094	7174	7174	1.15E+14	4.12E+13
4	W14x159	1481	9491	9491	1.58E+14	6.23E+13
3	W14x159	1481	9491	9491	1.58E+14	6.23E+13
2	W14x211	2013	12600	12600	2.21E+14	8.57E+13
1	W14x211	2013	12600	12600	2.21E+14	8.57E+13

- Section axial strength:

$$\emptyset N_S = \emptyset k_f A_n f_y \tag{1}$$

$$\emptyset N_t = \emptyset A_g f_y$$

Where N_S *and* N_t are nominal compressive and tensile strength; k_f = form factor = 1; A_g *and* A_n are gross and net cross sectional areas; and ∅ = 0.9

- Section flexural strength:

$$Z_e = \min(S, 1.5Z) \tag{2}$$

$$\emptyset M_S = \emptyset f_y Z_e \tag{3}$$

Where S = plastic section modulus; Z = elastic section modulus; and ∅ = 0.9

- Moment-axial load interaction equation:

$$M_{RX} = 1.18\, M_{SX} \left(1 - \frac{N^*}{\emptyset N_S}\right) \leq M_{SX} \tag{4}$$

Beams

Steel beams sections were obtained from SAC steel frame. The flexural strengths were calculated based on NZS3404:1997 Equations (2) and (3). Table 4 shows the summary of the beam sections.

Table 4 Bare Steel Frame Beams also Used in CFT Frame

Storey	US section	Dependable flexural strength (kNm)
9	W24x62	335
8	W27x94	809
7	W27x102	897
6	W33x130	1438
5	W33x141	1596
4	W33x141	1596
3	W33x141	1596
2	W36x150	1817
1	W36x150	1817

Connections

Bolted endplate connections, such as are commonly used in New Zealand, were assumed for the SAC steel frames even though these were different from the fully welded connections in the original SAC structure ((FEMA-355C 2000). The bolted moment endplate beam-column connections were designed based on HERA report (Hyland, Cowie et al. 2003). The other connections (i.e. gravity beam-column connections, column splices, etc.) were designed using basic force transfer mechanisms.

Rectangular CFT frame

The frame with rectangular CFT columns was designed to be comparable with the bare steel frame. The building layout was identical. The bare steel columns are replaced with equivalent rectangular CFT columns. The steel beams remain the same but the connections used for the rectangular CFT frame were slightly dimensionally modified. Apart from the columns, the differences between bare steel and CFT frame were kept to a minimum so clear comparison of the cost from rectangular CFT columns can be made.

Rectangular CFT columns

Rectangular CFT strength and stiffness were calculated according to AISC360:2010. All the steel plate elements slenderness of the rectangular CFT sections was compact according to the CFT provisions. There was assumed to be no strength reduction due to buckling or out of plane effects. Concrete tensile strength was neglected.

- Plate compactness limit

$$\frac{b}{t} \leq 2.26\sqrt{\frac{E}{f_y}} \qquad (5)$$

- Section axial strength

$$\emptyset_c P_n = f_y A_S + C_2 f'_C A_C \qquad (6)$$

$$\emptyset_t P_t = A_S f_y \qquad (7)$$

Where P_n *and* P_t are nominal compressive and tensile strength; A_S = steel tube area; A_C = core concrete area; C_2 = 0.85 for rectangular and 0.95 for circular; $\emptyset_c$ *and* $\emptyset_t$ are 0.75 and 0.9.

- Effective flexural stiffness

$$EI_{eff} = E_S I_S + C_3 E_C I_C \quad (8)$$

$$C_3 = 0.6 + 2\left(\frac{A_S}{A_C + A_S}\right) \leq 0.9 \quad (9)$$

Flexural strengths of the sections were calculated following the plastic stress distribution method outlined in the Commentary of AISC360:2010 and in Appendix B of (Viest, Colaco et al. 1997). The method assumes a linear strain distribution across the section and elasto-plastic behaviour of the steel. The plastic neutral axis is first calculated by assuming all steel reaches the yield stress, F_y, in tension or compression, and concrete reaches compressive stress of $0.85\,f'_C$. Tensile stress in concrete is ignored. Once the plastic neutral axis is found, the three points (B, C, D in Figure 5) on moment-axial load interaction curve can be calculated. Point B can be found from calculating the section moment capacity at the plastic neutral axis, while axial load is zero. Points D and C are obtained by calculating section moment and axial load capacities assuming neutral axis moves to the middle of the section and at the same distance to the other side of the middle of the section.

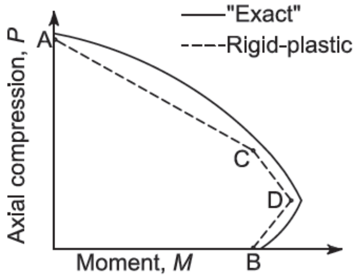
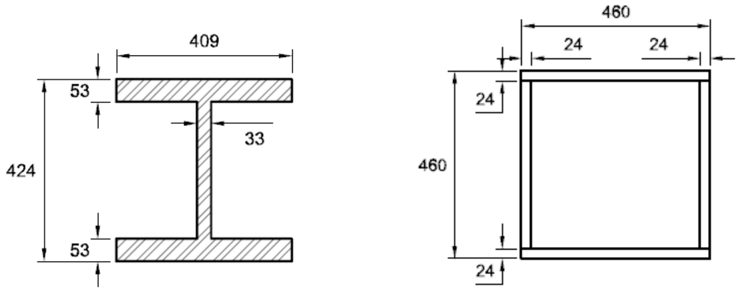


Figure 5. Moment-axial load interaction diagrams (AISC360 2010)

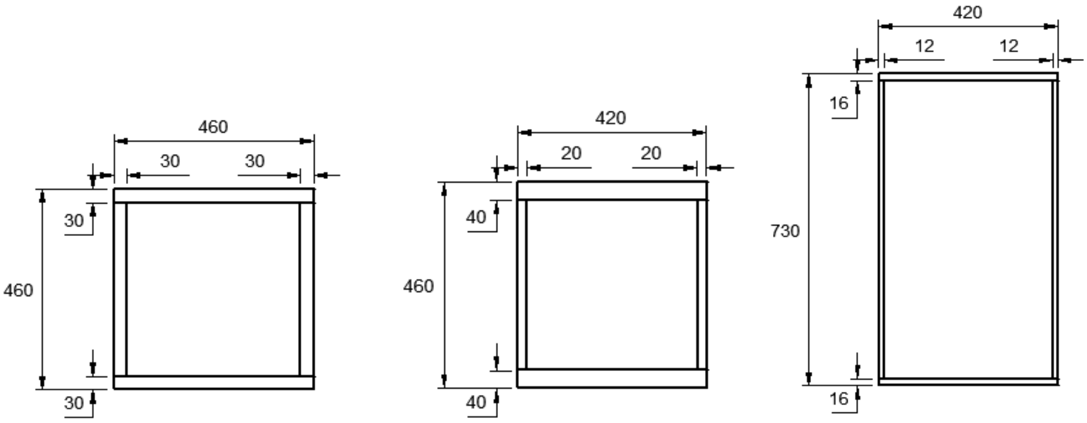
Rectangular CFT design cases and discussions

Four different design cases were considered as there is not necessarily one unique best CFT solution. For all cases, the plate elements were made compact. For each design case, the CFT columns have the same exterior dimensions throughout the entire height of the frame and plate thicknesses become thinner at higher storeys.

Figure 6 shows typical concept of rectangular CFT design cases. A thick flange plate allows smaller section depth design while large rectangular CFT section allows thinner plates to be used.



(a) Base steel case *(b) A-EIx*



(c) A-Mx *(d) B-Mx* *(e) C-Mx*

Figure 6. Example of Rectangular CFT Columns

It was found during the design optimisation phase that when rectangular CFT section is matched the stiffness with the steel section, the axial strength of rectangular CFT is always less. The reason is the steel cross sectional area required to produce the same flexural stiffness as the steel section is less than the steel section because in rectangular CFT, the steel element is placed at outermost of the section hence contribute very efficiently to section flexural stiffness. The axial strength, especially in tension, is smaller as a result of smaller steel sectional area.

Moreover, flexural strength of rectangular CFT section that has the same stiffness as steel section is also less than the strength of the steel section. The flexural strength of rectangular CFT section is governed by concrete compressive strength which is less than steel compressive strength.

Overall dimension of the rectangular CFT section in matching stiffness design was not significantly smaller than steel section due to similar requirement of flange thickness and section depth to produce flexural capacity. However, rectangular CFT section can be easily designed to have identical biaxial bending capacity which makes moment connections on the weak axis possible unlike in wide-flange steel section. As a result, the design of lateral resisting system layout is less restricted.

It has been claimed in some publications that architectural space can be increased by replacing bare steel columns with CFT columns. However, this benefit was not found in this study, because the study compared CFT columns with steel columns in strong axis only. Significant space gain is expected to be resulted from adopting CFT member for the entire structure which can gain benefit from biaxial properties of CFT columns.

- Design Case A-EIx

The rectangular CFT columns were designed to have the same properties in two perpendicular axes by using square tubes with the same thickness all around. The exterior width of rectangular CFT columns was kept to the same as the depth of bare steel column at ground level. The rectangular CFT columns are designed to have the same flexural stiffness in strong axis as the steel columns at every storey, for example EIx.eff (composite) = EIx (steel). Strength is relatively less than bare steel column from Table 1.

Table 5 Design Case A-EIx: Exterior Rectangular CFT columns

Storey	Section	t_f (mm)	t_w (mm)	AISC360 LRFD dependable strength			EI	
				Flexure (kNm)	Axial compression (kN)	Axial tension (kN)	X axis (mm^4)	Y axis (mm^4)
9	460x460	16.3	16.3	1669	11088	9113	2.51E+14	2.51E+14
8	460x460	19.7	19.7	1959	12491	10929	2.83E+14	2.83E+14
7	460x460	19.7	19.7	1959	12491	10929	2.83E+14	2.83E+14
6	460x460	24	24	2311	14234	13185	3.20E+14	3.20E+14
5	460x460	24	24	2311	14234	13185	3.20E+14	3.20E+14
4	460x460	42.4	42.4	3636	21284	22310	4.53E+14	4.53E+14
3	460x460	42.4	42.4	3636	21284	22310	4.53E+14	4.53E+14
2	460x460	42.4	42.4	3636	21284	22310	4.53E+14	4.53E+14
1	460x460	42.4	42.4	3636	21284	22310	4.53E+14	4.53E+14

Table 6 Design Case A-EIx: Interior Rectangular CFT columns

Storey	Section	t_f (mm)	t_w (mm)	AISC360 LRFD dependable strength			EI	
				Flexure (kNm)	Axial compression (kN)	Axial tension (kN)	X axis (mm^4)	Y axis (mm^4)
9	520x520	10.5	10.5	1479	10379	6741	2.83E+14	2.83E+14
8	520x520	12.7	12.7	1748	11443	8118	3.20E+14	3.20E+14
7	520x520	12.7	12.7	1748	11443	8118	3.20E+14	3.20E+14
6	520x520	21.5	21.5	2748	15605	13504	4.53E+14	4.53E+14
5	520x520	21.5	21.5	2748	15605	13504	4.53E+14	4.53E+14
4	520x520	33.8	33.8	3995	21169	20706	5.99E+14	5.99E+14
3	520x520	33.8	33.8	3995	21169	20706	5.99E+14	5.99E+14
2	520x520	41.9	41.9	4737	24673	25241	6.83E+14	6.83E+14
1	520x520	41.9	41.9	4737	24673	25241	6.83E+14	6.83E+14

- Design Case A-Mx

The design employed biaxial concept the same as previous case but the rectangular CFT columns were designed to match the flexural strength in strong axis of the steel columns at every storey. Flexural stiffness is greater than bare steel case and tensile strength is less due to smaller steel cross sectional area.

Table 7. Design Case A-Mx: Exterior rectangular CFT columns

Storey	Section	t_f (mm)	t_w (mm)	AISC360 LRFD dependable strength			EI	
				Flexure (kNm)	Axial compression (kN)	Axial tension (kN)	X axis (mm^4)	Y axis (mm^4)
9	460x460	23.3	23.3	2255	13952	12821	3.14E+14	3.14E+14
8	460x460	26.6	26.6	2515	15270	14526	3.41E+14	3.41E+14
7	460x460	26.6	26.6	2515	15270	14526	3.41E+14	3.41E+14
6	460x460	30.4	30.4	2803	16761	16455	3.71E+14	3.71E+14
5	460x460	30.4	30.4	2803	16761	16455	3.71E+14	3.71E+14
4	460x460	44.9	44.9	3796	22191	23484	4.68E+14	4.68E+14
3	460x460	44.9	44.9	3796	22191	23484	4.68E+14	4.68E+14
2	460x460	44.9	44.9	3796	22191	23484	4.68E+14	4.68E+14
1	460x460	44.9	44.9	3796	22191	23484	4.68E+14	4.68E+14

Table 8 Design Case A-Mx: Interior rectangular CFT columns

Storey	Section	t_f (mm)	t_w (mm)	AISC360 LRFD dependable strength			EI	
				Flexure (kNm)	Axial compression (kN)	Axial tension (kN)	X axis (mm^4)	Y axis (mm^4)
9	520x520	19.4	19.4	2518	14626	12237	4.24E+14	4.24E+14
8	520x520	22	22	2802	15837	13805	4.59E+14	4.59E+14
7	520x520	22	22	2802	15837	13805	4.59E+14	4.59E+14
6	520x520	31.8	31.8	3802	20285	19561	5.77E+14	5.77E+14
5	520x520	31.8	31.8	3802	20285	19561	5.77E+14	5.77E+14
4	520x520	43	43	4833	25139	25844	6.94E+14	6.94E+14
3	520x520	43	43	4833	25139	25844	6.94E+14	6.94E+14
2	520x520	50	50	5423	28049	29610	7.58E+14	7.58E+14
1	520x520	50	50	5423	28049	29610	7.58E+14	7.58E+14

- Design Case B-Mx

The rectangular CFT columns were designed using the overall dimensions of the steel columns at the ground level. The flange thicknesses of rectangular CFT section were kept not more than one size thicker than the web thicknesses in most cases. The flexural strength in strong axis is then matched with steel column at every storey. Flexural stiffness is greater than bare steel case and tensile strength is less due to smaller steel cross sectional area.

Table 9 Design Case B-Mx: Exterior rectangular CFT columns

Storey	Section	t_f (mm)	t_w (mm)	AISC360 LRFD factored strength			EI	
				Flexure (kNm)	Axial compression (kN)	Axial tension (kN)	X axis (mm^4)	Y axis (mm^4)
9	460x420	25	25.5	2255	13895	13202	3.05E+14	2.63E+14
8	460x420	32	23	2510	14670	14205	3.38E+14	2.62E+14
7	460x420	32	23	2510	14670	14205	3.38E+14	2.62E+14
6	460x420	40	20	2794	15572	15372	3.73E+14	2.60E+14
5	460x420	40	20	2794	15572	15372	3.73E+14	2.60E+14
4	460x420	50	48	3799	22328	24116	4.58E+14	3.87E+14
3	460x420	50	48	3799	22328	24116	4.58E+14	3.87E+14
2	460x420	50	48	3799	22328	24116	4.58E+14	3.87E+14
1	460x420	50	48	3799	22328	24116	4.58E+14	3.87E+14

Table 10 Design Case B-Mx: Interior rectangular CFT columns

Storey	Section	t_f (mm)	t_w (mm)	AISC360 LRFD factored strength			EI	
				Flexure (kNm)	Axial compression (KN)	Axial tension (KN)	X axis (mm^4)	Y axis (mm^4)
9	520x520	19.4	19.4	2518	14626	12237	4.24E+14	4.24E+14
8	520x520	22	22	2802	15837	13805	4.59E+14	4.59E+14
7	520x520	22	22	2802	15837	13805	4.59E+14	4.59E+14
6	520x520	31.8	31.8	3802	20285	19561	5.77E+14	5.77E+14
5	520x520	31.8	31.8	3802	20285	19561	5.77E+14	5.77E+14
4	520x520	43	43	4833	25139	25844	6.94E+14	6.94E+14
3	520x520	43	43	4833	25139	25844	6.94E+14	6.94E+14
2	520x520	50	50	5423	28049	29610	7.58E+14	7.58E+14
1	520x520	50	50	5423	28049	29610	7.58E+14	7.58E+14

- Design Case C-Mx

In this case, the thinner plate elements were used and allow the depth of rectangular CFT column to be more than the depth of steel columns. The flange thicknesses of rectangular CFT sections were kept not more than one size thicker than the web thicknesses in most cases. The flexural strength in strong axis is then matched with steel column at every storey. Flexural stiffness is significantly greater than bare steel case because the deeper section design. However, the tensile strength is less than the bare steel column due to smaller steel cross sectional area.

Table 11 Design Case C-Mx: Exterior rectangular CFT columns

Storey	Section	t_f (mm)	t_w (mm)	AISC360 LRFD dependable strength			EI	
				Flexure (kNm)	Axial compression (kN)	Axial tension (kN)	X axis (mm^4)	Y axis (mm^4)
9	730x420	12	9.9	2247	11719	7579	5.98E+14	2.19E+14
8	730x420	16	9.8	2584	12464	8543	6.74E+14	2.27E+14
7	730x420	16	9.8	2584	12464	8543	6.74E+14	2.27E+14
6	730x420	16	12.2	2795	13280	9598	7.05E+14	2.53E+14
5	730x420	16	12.2	2795	13280	9598	7.05E+14	2.53E+14
4	730x420	25	15.3	3799	16039	13170	9.05E+14	3.05E+14
3	730x420	25	15.3	3799	16039	13170	9.05E+14	3.05E+14
2	730x420	25	15.3	3799	16039	13170	9.05E+14	3.05E+14
1	730x420	25	15.3	3799	16039	13170	9.05E+14	3.05E+14

Table 12 Design Case C-Mx: Interior rectangular CFT columns

Storey	Section	t_f (mm)	t_w (mm)	AISC360 LRFD dependable strength			EI	
				Flexure (kNm)	Axial compression (kN)	Axial tension (kN)	X axis (mm^4)	Y axis (mm^4)
9	740x430	12	11.9	2513	12745	8619	6.59E+14	2.58E+14
8	740x430	16	11.1	2797	13260	9285	7.29E+14	2.59E+14
7	740x430	16	11.1	2797	13260	9285	7.29E+14	2.59E+14
6	740x430	20	18.8	3801	16677	13709	9.06E+14	3.53E+14
5	740x430	20	18.8	3801	16677	13709	9.06E+14	3.53E+14
4	740x430	32	19.3	4833	19134	16888	1.12E+15	3.81E+14
3	740x430	32	19.3	4833	19134	16888	1.12E+15	3.81E+14
2	740x430	32	27.2	5420	21733	20253	1.19E+15	4.51E+14
1	740x430	32	27.2	5420	21733	20253	1.19E+15	4.51E+14

- Gravity columns

Rectangular CFT gravity columns are designed to have the same flexural strength as the bare steel gravity columns at the axial load level of 20% of the axial capacity.

Table 13 Rectangular CFT gravity columns

Storey	Section	t_f (mm)	t_w (mm)	AISC360 LRFD dependable strength			EI	
				Flexure (kNm)	Axial compression (kN)	Axial tension (kN)	X axis (mm^4)	Y axis (mm^4)
9	400x400	8	8	668	6113	3951	9.86E+13	9.86E+13
8	400x400	10	10	812	6857	4914	1.14E+14	1.14E+14
7	400x400	10	10	812	6857	4914	1.14E+14	1.14E+14
6	400x400	14.1	14.1	1092	8357	6856	1.43E+14	1.43E+14
5	400x400	14.1	14.1	1092	8357	6856	1.43E+14	1.43E+14
4	400x400	20.2	20.2	1479	10529	9667	1.79E+14	1.79E+14
3	400x400	20.2	20.2	1479	10529	9667	1.79E+14	1.79E+14
2	400x400	29.5	29.5	2011	13700	13771	2.27E+14	2.27E+14
1	400x400	29.5	29.5	2011	13700	13771	2.27E+14	2.27E+14

Beams and Connections

Steel beams and connections for rectangular CFT frame were the same as for the bare steel frame except some minor dimension modifications on the connections due to different member sizes. This allows a clear comparison on the cost difference from rectangular CFT columns to be made.

Alternative beam-column moment connections design

Some alternative rectangular CFT beam-column connections were preliminarily designed to discuss their constructability to the conventional bolted moment endplate connection. The design was based on the available literature and based on simple force transfer mechanism where there is no publication available. The cost and constructability associated with each type are discussed.

Moment end plate connection with through threaded rods or anchor bolts

Long-through bolts in moment endplate connections can be replaced with threaded mild steel rods or concrete anchor bolts. The through-type connectors are preferred due to flange tension can be transferred as compression on the opposite side of the tube. The following issues affect the economy or performance when concrete anchor bolts are used;

1) connection and beam members need temporary support for erection before concrete filling, which adds extra cost and complexity to the construction;
2) anchor bolts need to be long enough and overlap with the opposite bolts to develop sufficient stress in concrete, so it may be hard to secure the bolts in the position during concreting;
3) the anchor bolts often have hook end, as shown in Figure 7, so the bolt hole and washer will need to be large enough to allow the anchor bolt to be inserted from the beam side.

Alternatively, headed shear studs, welded inside the tube shell, have been proposed as a replacement for through-bolts. Application of internal shear studs were studied (Alostaz and Schneider 1996). A piece of the tube shell was cut off and shear studs were welded on the inside face of the cut-off piece. The piece of shell was then welded back to the tube so the shear studs were inside. The connection presents the following fabrication difficulties;

1) the weld required is likely to be complete penetration butt weld because it is subjected to high column axial load demand so the need for cost increment and inspection arise,

2) temporary supports are needed for the case of site weld or in the case that the tube's strength alone is not enough to carry construction loads so the supports are needed until studs are in the hardened concrete.

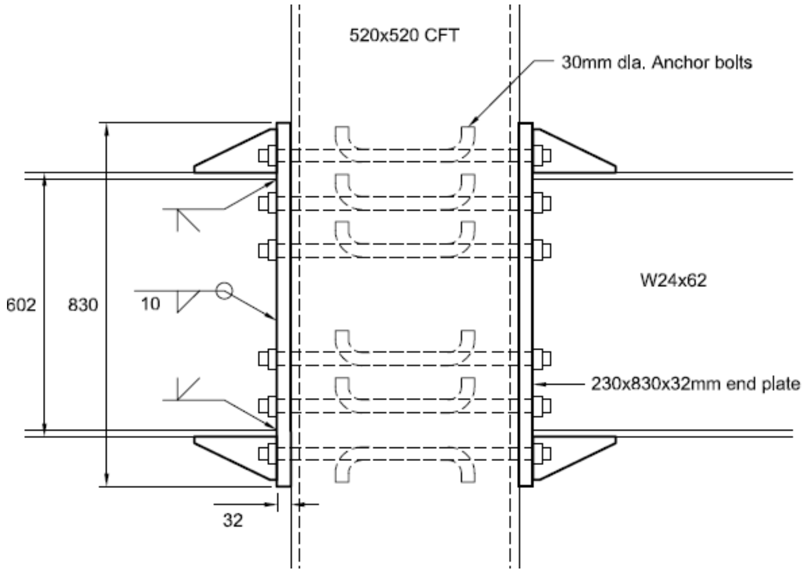


Figure 7 Example of moment endplate BCJ detail with anchor bolts

Through beam connection

Design recommendations of through-beam connections were proposed by (Elremaily and Azizinamini 2001) based on their finite element analyses and experiments on CCFT. The equations have been slightly modified here for rectangular CFT columns. The column-to-beam moment ratio is recommended by Elremaily and Azizinamini is 1.5 for full penetration butt welds and 2.0 for fillet welds to avoid failure at the column. The proposed design equation for the dependable joint shear strength, $\emptyset V_{jh}$ depends on the beam stub strength, V_{wn} the concrete panel strength, $\emptyset V_{csn}$ and the tube shell strength, V_{tn} as given in Equations 10-12, where $\emptyset$ = 0.7, f_{yw}, f_{yt} = yield strength of beam web and tube, f'_C = concrete compressive strength, d_c = depth of beam web, t_w = beam web thickness, A_s, A_c = cross sectional area of steel tube and concrete.

$$\emptyset V_{jh} = \emptyset(V_{wn} + V_{csn} + V_{tn})$$

$$V_{wn} = 0.6\, f_{yw} d_c t_w \tag{10}$$

$$V_{csn} = 1.99\sqrt{f'_C}\, A_c \tag{11}$$

$$V_{tn} = 0.6\, f_{yt} A_S \tag{12}$$

The proposed design equations for weld design between beam flange and tube shell are given in Equation 13-14, where $\emptyset_b$ = 0.9, f_{yf} = yield strength of beam flange, b_f, t_f = beam flange width and thickness, t_t = thickness of tube wall, σ_v = maximum tensile stress in tube wall corresponding to column moment and axial load.

$$h = 1.5\left(\frac{0.3\emptyset_b f_{yf} t_f b_f}{2b_f}\right) \qquad \text{Horizontal force per weld unit length} \quad (13)$$

$$v = \sigma_v t_t \qquad \text{Vertical force per weld unit length} \quad (14)$$

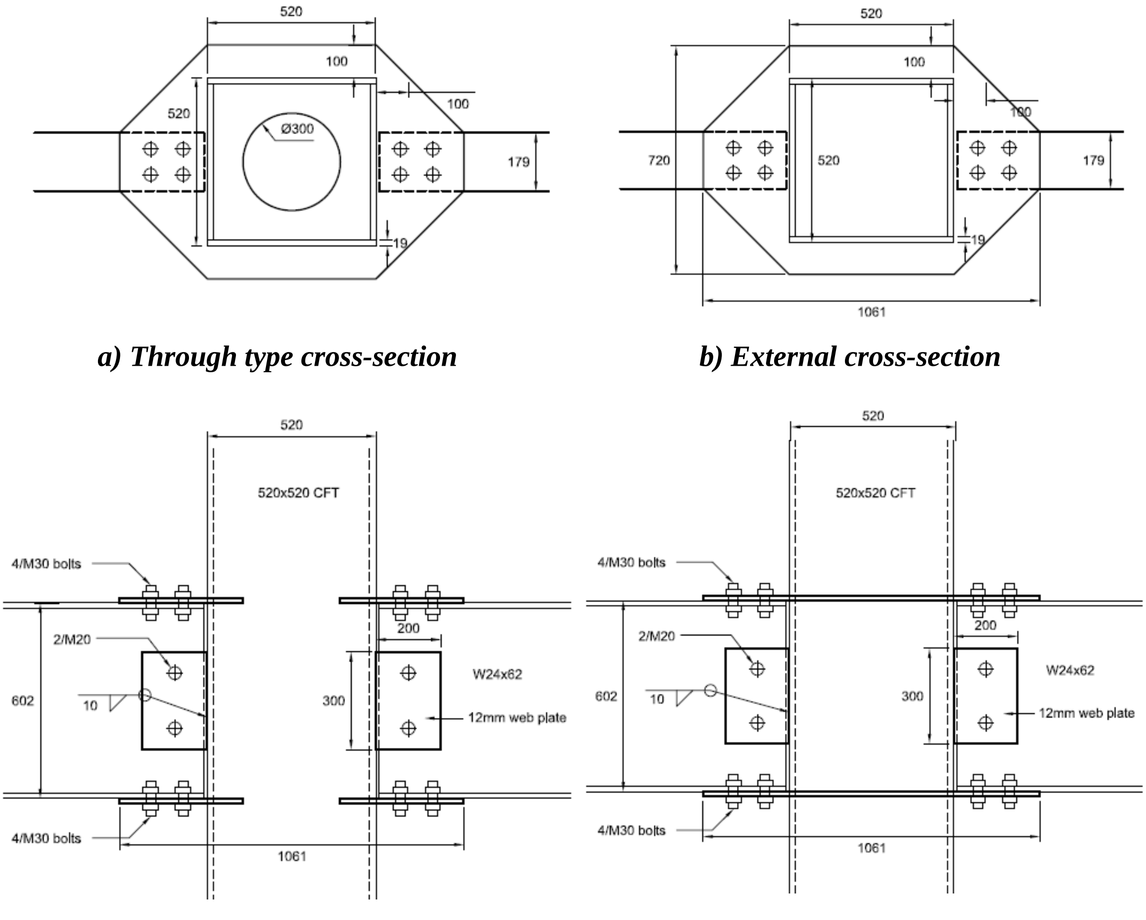
Diaphragm connection

Empirical equations for calculating ultimate tensile strength of external, $P_{U,ext}$, and through diaphragms, $P_{U,int}$, were proposed by (Morino and Tsuda 2003).

$$P_{U,ext} = 1.42(2(4t + t_S)tF_1 + \frac{4}{\sqrt{3}}\, h_S t_S F_2) \qquad (15)$$

$$P_{U,int} = 1.42(D + 2h_S - d_f)^2 \frac{B_f t_S}{d_f^2}\, F_2 \qquad (16)$$

Here P_U = Ultimate tensile strength of diaphragm; t = thickness of rectangular CFT; t_S = thickness of diaphragm; F_1, F_2 = design strengths of steel tube and diaphragm respectively; d_f = diameter of internal opening for concreting; h_S = diaphragm dimension at corner of rectangular CFT.



a) Through type cross-section ***b) External cross-section***

c) Through type elevation ***d) External elevation***

Figure 8. Diaphragm Beam-Column Joint Details

The main problem with through-diaphragm fabrication is the steel tube needed to be cut and welded to the top and bottom side of the diaphragm. Complete penetration butt weld is usually required in this situation due to high load transferring demand in the column, hence making the fabrication cost much higher. These types of connection are common in Japan where fabrication quality control is very stringent. However, there were some examples of weld fracture failure of this type of connection in Kobe earthquake. Moreover, the distance between top and bottom diaphragms is needed to be fabricated accurately to avoid excessive tolerance or unfitting when erecting the beam.

T-stub connection

T-stub connections may be designed to be rigid or with flexibility. Several force transfer mechanisms were considered in the design including; 1) shear in flange bolts, 2) yielding in T-stub stem, 3) bending in T-stub flange, and 4) tension and prying effect in through bolts.

T-stub connection type has been appreciated by the NZ steel industry in its ease of fabrication and erection. Using bolts only increases ability to adjust the position during erection. No field or shop welding is required. It is also relatively easy to make a low damage connection here by fixing the top flange and allowing energy dissipation on the bottom flange. Details of this type are described in MacRae et al. (2010) and MacRae (2011) for different types of energy dissipation mechanism.

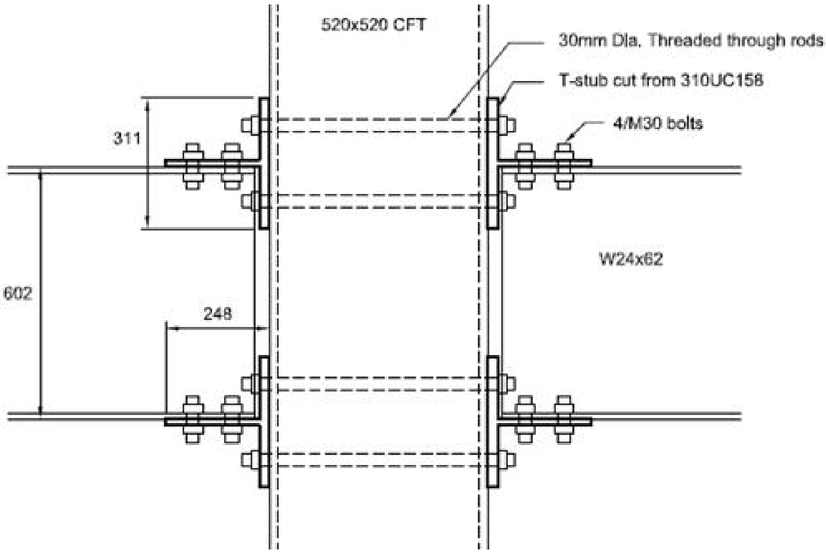


Figure 9 T-stub connection

Table 14 presents a summary of the relative advantages of the different moment frame beam column joint connection types. All of the connections can be used in low damage construction by moving the location of energy dissipation away from the joint in the beam itself. In some cases, the low damage connection may be incorporated as part of the connection itself. Providing low damage connections beside any of the beam-column-joint details listed below will result a system with excellent performance.

Table 14. Summary of Relative Advantages of Different Beam-Column Joint Details

Type	Advantages	Disadvantages
Moment endplate	• Familiar design concept to designers • Wide choices of fasteners • Can be used for 2-way frames • No site welding required	• Need of shimming for construction tolerance • Poor fastener choice can be inefficient and expensive
Through-beam	• Strong and ductile • Low damage type connection easily incorporated away from the joint • Construction tolerance allowed	• Site weld usually required at tube • Concrete compaction problem around through beam • Not suitable for 2-way frames

Diaphragm	• Efficient force transfer mechanism • Suitable for 2-way frames • Construction tolerance allowed • No site weld required • Easy to make low-damage detail	• Complex shop weld requirement for internal stiffeners, but not for external stiffeners • Concrete compaction quality control for internal stiffeners
T-stub	• Rigidity can be tuned by proportioning connector mechanisms • No weld required, bolt only • 2-way connection easily designed • Construction tolerance allowed • Easy to fabricate and erect • Easy to make low-damage detail	• Complicated design

FRAME ANALYSIS

Frame models

Simple 2D analytical models were created and analyses were done using finite element code programme, RUAUMOKO (Carr 2008). The purpose of the analyses was to confirm the similarity in the seismic behaviour of the two frames. The rectangular CFT design case A-EIx was chosen for analysis and comparison because it was designed to the same stiffness as the steel frame. Since the column stiffness is identical, the seismic performances of the frame are similar, therefore allowing the two frames to be compared by cost only. Since the design of members in most frames is governed by stiffness, rather than strength, any difference in column strength does not significantly affect the behaviour as the columns are expected to remain elastic, except perhaps at the base.

Simplified assumptions were made in the modelling process and kept consistent among the two frames. The models were centreline models which all the members were connected at the nodes. Joint panels were excluded based on the assumption of negligibly small joint panel deformations. Interaction between the frames and slab was neglected. A fictitious column was modelled and pin connected to the frames to induce lateral force increment to the frames due to P-Delta effect.

Giberson one component model, which lumped plasticity at both ends, (Carr 2008) was used for all flexural members. The plastic hinge length at each end was approximated at one sixth of member span to ensure moment-curvature bilinear rule is consistent with force-displacement relationship. Columns were model as beam-column members which interaction between moment and axial load was taken into account. Bilinear hysteresis rule with 0.5% post-yield resistance was employed.

Strengths of the steel beams for both frames were calculated based on NZS 3404:1997 assuming the beams are fully braced so out-of-plane effect does not occur. Steel columns strengths were obtained from moment-axial load interaction diagram based on NZS 3404 while AISC 360 procedure was used for rectangular CFT columns. Details of moment-axial load interaction can be found in the previous sections.

Seismic masses used in the analyses were extracted from SAC frame design information (FEMA-355C 2000). Gravity loads were derived from seismic masses and evenly distributed to the members proportioned to tributary floor areas.

Pushover analysis

A monotonic lateral linear force distribution was applied to the frames. P-Delta effect was not considered due to a limitation of RUAUMOKO which does not permit negative post-elastic stiffness for fixed force distributions. The pushover analyses were terminated at the top displacement of 4% drift.

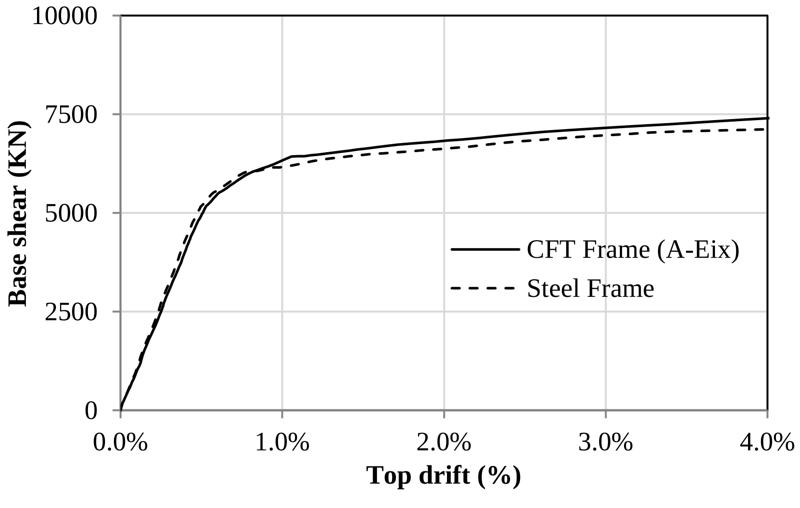


Figure 10 Pushover curves

The fundamental periods from modal analyses are almost the same, they are 2.3 and 2.2 seconds for steel frame and CFT frame respectively. It can be seen from Figure 10, the pushover curves of the two frames are similar. Slightly different frame stiffness is because geometrical difference in column size, beam length and plastic hinge zone. The base shear capacities are 5900 and 5600 kN for steel and CFT frames. The 5% difference comes from column geometrical difference and slightly smaller CFT columns strength in the stiffness-matching design case (case A-EIx). In conclusion, the result confirms two frames have comparable capacities.

Inelastic time history analysis

The two frames were analysed in the real earthquake events. 2011 Christchurch earthquake records measured from three stations, Botanic garden (CBGS_N89W), Cathedral College (CCCC_N64E), and Resthaven (REHS_S88E), were used to excite the structures. The records were unscaled and are compared with NZS1170.5 design acceleration spectra in Figure 11. The 0.22 and 0.30 hazard factors (Z) represent the code provision before and after 2011 Christchurch events. The acceleration demands from the records are close to the design spectra at the structure periods.

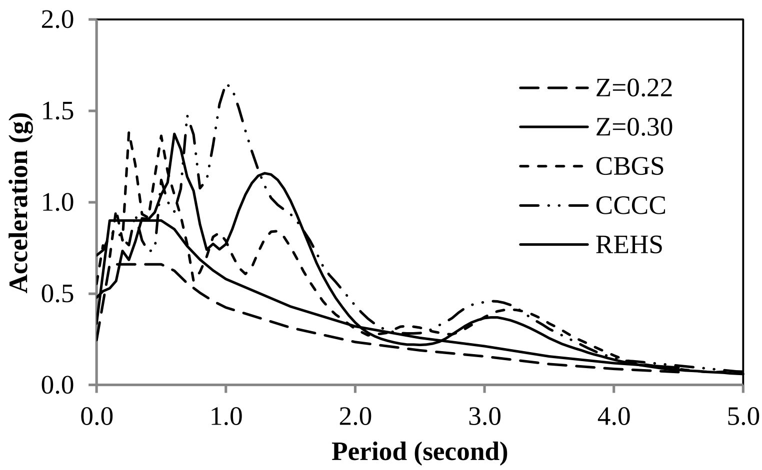


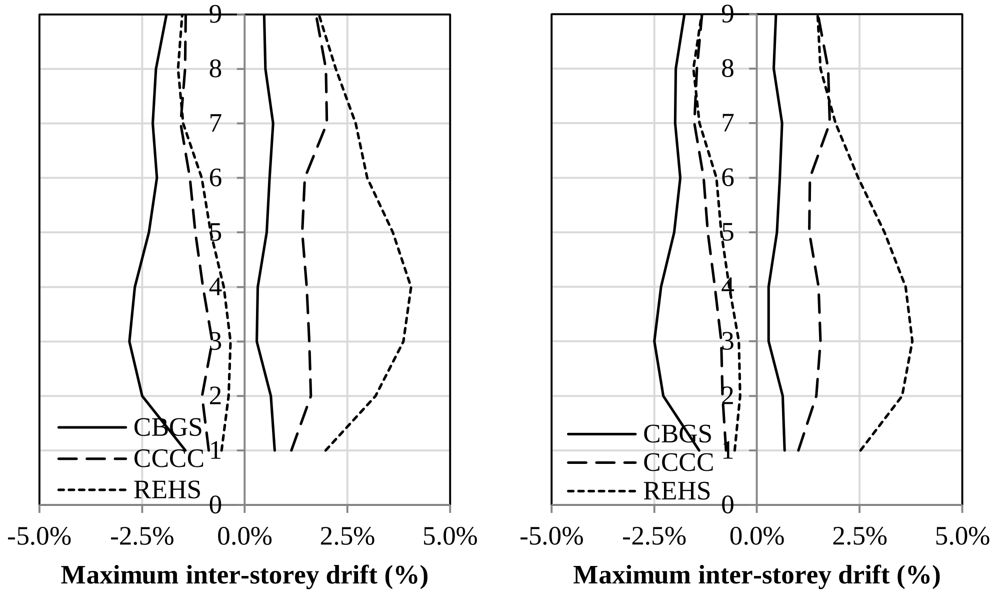
Figure 11. Acceleration spectra

The frames behaved satisfactorily during the analyses. Beam plastic hinges of every storey were activated and most of them occurred before the column bases yielded in flexure. The higher mode effects and asymmetries of the earthquake records were clearly noticed. The results in (a) Steel frame (b) CFT frame

Figure 12. Maximum inter-storey drift

and Figure 13 reflect such phenomena.

The maximum inter-storey drifts demand of the steel frame is higher than rectangular CFT frame. However, the simple analytical models were not expected to capture the difference and further detailed models and analyses are recommended if the drift results are important. The result in Figure 13 is sufficient for this project to confirm the comparable behaviour of the steel and composite frames.



(a) Steel frame

(b) CFT frame

Figure 12. Maximum inter-storey drift

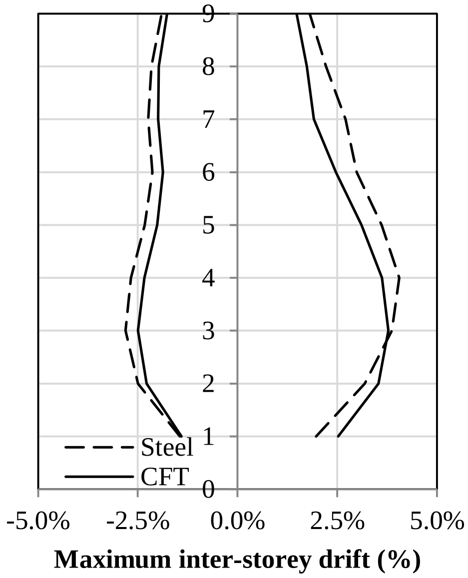


Figure 13. Maximum inter-storey drift envelopes

FRAME COST STUDY

Since the responses of the frames are similar, the major parameter affecting whether or not an engineer will select a composite rectangular CFT column is the cost. The construction cost estimate of the 5 design cases, base steel frame and four rectangular CFT frames, were carried out. The cost estimate was based on bare frame structures and without slabs and non-structural components. The unit prices, material and fabrication, were kindly provided by *John Jones Steel Ltd*, Christchurch, NZ. It should be noted that the size of SAC steel frame chosen for the study was relatively large compared to what is commonly used by the NZ steel industry. As a result, the cost per square metre or per storey may not within the common NZ range. Future studies about composite construction in NZ should take into account an appropriate scale of projects to suit the NZ industry.

Study 1. Cost of columns

Average per-metre cost of steel and rectangular CFT columns were derived and summarised in Table 14 for the cases considered in Figure 6. The cost of bare steel column was considered from built-up section. Rectangular CFT designs are not competitive in term of cost based on this study except design Case C-Mx. The supply rate, material and shop-fabrication, of rectangular CFT columns were significantly higher than the rate of built-up I-section steel columns. The reason is mainly due to the expense of butt welds used in built-up rectangular CFT fabrications used. Typical NZ construction only uses fillet welding for welded steel columns.

The weights of steel plates were similar in all designs except Case C-Mx. Hence the site erection rates per ton were also similar for all columns. The concrete material and installation cost was less than 3 per cent.

Table 14. Average cost components per metre of column

Design cases	Average per metre cost components (NZD)			
	Material & Fabrication	Site erection	Concrete material	Total
Steel	$ 328.06	$ 570.59	$ -	$ 898.65
CFT Case A-EIx	$ 802.97	$ 425.77	$ 14.94	$ 1,243.68
CFT Case A-Mx	$ 1,020.89	$ 539.14	$ 14.04	$ 1,574.06
CFT Case B-Mx	$ 1,017.32	$ 539.44	$ 13.05	$ 1,569.81
CFT Case C-Mx	$ 524.91	$ 350.17	$ 20.82	$ 895.90

When CFT columns are used in medium and high rise buildings the required steel tube sizes are likely to be larger than NZ market-available sizes of RHS or CHS. Hence built-up CFT sections are common. The cost of a CFT member significantly comes from welds. Preferable built-up technique for a rectangular CFT is using four flat plates and fillet weld. The cost of fillet welding is commonly accepted but however, in a very large rectangular CFT section butt weld may be required to allow the built-up section to develop sufficient confinement and sustain construction loads. The cost of complete penetration butt welding can be as high as ten times the fillet weld for the same plate thickness due to inspection costs. A partial penetration butt weld can be designed as a solution compromising between strength and fabrication cost. An alternative solution uses thinner steel plate and fillet welding. However, a larger column cross section is needed which may reduce the available space of the floor.

Study 2. Cost of a 2D moment frame

Cost components of a moment frame are shown in Table 15 and Table 16. It can be seen, the biggest contribution to the frame cost comes from beams. Columns contribute around 26 to 39 per cents while around 10 to 15 per cents come from connections. Table 16 also provides the view of what cost component should be targeted if the overall cost reduction is to be achieved.

The column costs are in the same order as per-metre cost. Beam components are the same for all designs. Although the connections were specifically designed to different column sections, the costs of the connections are almost the same for all designs. The cost of the connection in this study includes column base plates, splices and beam-column joints. It should be noted that the rectangular CFT connections were designed to be similar to what used in the steel frame. The available low-damage connections for rectangular CFT were not taken into account.

Table 15 Cost comparison for one moment frame

Design cases	Cost per one moment frame (NZD)			
	Columns	Beams	Connections	Total
Steel	$ 673,987.81	$ 1,562,610.96	$ 405,756.86	$ 2,642,355.62
CFT Case A-EIx	$ 932,760.52	$ 1,562,610.96	$ 305,960.09	$ 2,801,331.57
CFT Case A-Mx	$ 1,180,544.33	$ 1,562,610.96	$ 316,354.15	$ 3,059,509.44
CFT Case B-Mx	$ 1,177,354.13	$ 1,562,610.96	$ 317,166.33	$ 3,057,131.42
CFT Case C-Mx	$ 671,924.70	$ 1,562,610.96	$ 301,135.01	$ 2,535,670.68

Table 16 Cost comparison for one moment frame

Design cases	Cost per one moment frame (percentage)			
	Columns	Beams	Connections	Total
Steel	26%	59%	15%	100%
CFT Case A-EIx	33%	56%	11%	100%
CFT Case A-Mx	39%	51%	10%	100%
CFT Case B-Mx	39%	51%	10%	100%
CFT Case C-Mx	26%	62%	12%	100%

Study 3. Cost of 3D frames (Entire building)

The last task of cost study was to take the internal gravity frames into account. The cost summary of entire building, resulting from the costs of the moment frame only and the gravity frame only, is presented in Table 17. This is done for the cases when the gravity frame uses steel columns only, or uses composite columns.

Table 17 Cost comparison for entire frame structure

Design cases	Cost per entire building (frames only) (NZD)			Ratio of bare steel structure
	Moment frame	Gravity frame	Total	
Steel	$ 10,569,422.49	$ 1,496,032.12	$ 12,065,454.61	1.00
with CFT gravity frame				
CFT Case A-EIx	$ 11,205,326.28	$ 2,207,958.14	$ 13,413,284.42	1.11
CFT Case A-Mx	$ 12,238,037.74	$ 2,207,958.14	$ 14,445,995.88	1.20
CFT Case B-Mx	$ 12,228,525.69	$ 2,207,958.14	$ 14,436,483.83	1.20
CFT Case C-Mx	$ 10,142,682.71	$ 2,207,958.14	$ 12,350,640.85	1.02
with steel gravity frame				
CFT Case A-EIx	$ 11,205,326.28	$ 1,496,032.12	$ 12,701,358.39	1.05
CFT Case A-Mx	$ 12,238,037.74	$ 1,496,032.12	$ 13,734,069.86	1.14
CFT Case B-Mx	$ 12,228,525.69	$ 1,496,032.12	$ 13,724,557.81	1.14
CFT Case C-Mx	$ 10,142,682.71	$ 1,496,032.12	$ 11,638,714.82	0.96

The rectangular CFT gravity column is more expensive than its steel counterpart. The reason is because of butt weld requirement which has been described previously. In smaller project hollow sections can be used to reduce the cost of building up the section. The percentage of gravity frame cost is around 11 to 18 and the major cost contribution comes from moment resisting frames. The overall conclusion from Table 17 is the costs of rectangular CFT moment frame structures range from 4% lower to 14% higher than bare steel structure if compare using the same steel gravity frame.

CONCLUSION

This paper describes previous work on the experimental performance of connections to composite columns in one-way seismic frames, compares the seismic response of frames where the columns have similar properties to conventional response to conventional construction, describes a cost comparison of the composite column frames with traditional steel column frames, and the constructability of different systems is discussed. It was shown that:

1. Significant studies have been conducted on a range of composite column to steel beam connection for moment frames. These include, bolted end plate connections, through-beam, diaphragm, and T-stub connections. Those most promising for NZ are those which have both a low materials and construction cost. Through-beam connections seem effective for one-way frames. Here, Christmas-tree type construction may be used where a short beam is to be placed through the column and all welding conducted in the factory. Then, in the field, beams may be bolted to the beam stubs using low damage seismic connections. However, in case of two-way columns, T-stub and bolted end-plate connections for rectangular CFTs and external diaphragm connections for circular CFT are preferred. All of these options allow easy site adjustment during erection and this reduces the construction cost. T-stub connections are also quick to build as no welding is required. Low-damage connection details can be included easily into a T-stub connection.
2. The behaviour of a particular building system with rectangular CFT columns is compared to one with traditional structural steel I-shaped columns. The seismic frames are one way and in both cases the bolted end plate connections were used at the beam ends to the columns. The stiffness/strength properties of the columns in both cases were similar, so that the seismic performance of both frames was similar. Since the response was similar, the difference was cost. Here, the relative costs of the entire building with composite columns ranged between 96% and 120% of the cost of the traditional steel frame depending on the CFT column design. CFT column cost per linear metre is, in general, higher than structural steel column cost, mainly due to butt weld requirement for large section fabrication. Moreover, column and connection cost contributions are less than the cost contribution from steel beams.
3. For structures in which the columns are expected to carry moment from frames in two orthogonal directions it is expected that buildings with composite columns may be significantly more economical than those with non-composite columns. Also, different connection types may also influence the cost. Further considerations of the following may change the relative desirability of columns with CFT composite construction. These include modifications to design procedures, multidirectional loading, usable architectural space considerations, the need for formwork, different tube construction methods, the use of low damage connections with composite columns, and different beam-column joint details (e.g. bonded vs. unbounded rods in beam end-plate connections), etc.

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