Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 48, No. 1, March 2015

SUITABILITY OF CFT COLUMNS FOR NEW ZEALAND MOMENT FRAMES

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(Submitted *May 2014*; Reviewed *August 2014*; Accepted *January 2015*)

ABSTRACT

Composite steel-concrete construction uses steel and concrete together to provide the possibility of a system with better performance, and/or lower cost, than using either material alone. This paper firstly subjectively evaluates the advantages and disadvantages of a number of composite concrete filled tubular (CFT) column-connection systems proposed/used around the world in terms of their likely acceptance in moment frames in New Zealand. Then, the cost of a conventional one-way moment-resisting steel frame system is compared with a similarly behaving frame using rectangular concrete filled steel tubular (CFT) columns. It is shown for these studies conducted on one-way frames that composite CFT column construction with beam end-plate connections is generally more expensive than conventional steel column construction.

INTRODUCTION

A steel-concrete composite structural member consists of both structural steel and concrete elements working together. There are many possible element combinations. For example, a concrete slab on a steel beam with mechanical shear connectors allows the slab and beam to resist bending moment as a single unit, increasing the strength and reducing deformations. Steel-reinforced concrete (SRC) columns, comprising a structural steel core surrounded by reinforced concrete, are used when an exposed concrete surface is required. The concrete adds stiffness and protects the column from fire, while the steel provides strength and deformation capacity. Concrete-filled steel tubular (CFT or CFST) columns, of circular or rectangular shape, comprise a hollow steel tube 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 the concrete infill limits steel local buckling.

Connections in composite structural systems differ from conventional connections in steel systems due to different force transfer mechanisms and constructability considerations. Many types of composite connections have been proposed and tested mostly in Europe for non-seismic conditions and in the U.S., China and Japan for seismic conditions.

Even though member and connection studies have been undertaken, there remains a lack of understanding, robust design guidance, and design examples related to member composite action, in terms of strength, stiffness and ductility, and the connections to the members which may be relevant to NZ seismic frames. Also, there may be incorrect perceptions about the costs of composite systems relative to traditional steel construction. For these reasons, design of composite structural systems has not yet become popular in NZ. There is a need to address these issues and to provide appropriate information in order that reasonable decisions may be made about whether or not these composite structures can be used. In order to address this need, this paper seeks answers to the following questions for moment frame structures with composite CFT structural columns:

- What types of composite seismic moment frame beam-to-column connection seem promising for NZ?
- How does the cost of a building with one-way moment frames and CFT columns compare to a similar building with steel columns?
- What other factors are likely to affect the relative economy of frames with and without composite columns?

LITERATURE SUMMARY

The earliest use of steel-concrete composite construction in the USA dates back to the Ward House in Port Chester, NY, in 1877 [1]. As with many early composite systems, this application consisted of steel sections fully embedded in a concrete beam [2]. By the 1930s only the top flange was being embedded, and the slab and beam became separated and tied by shear connectors in the early 1940s. The first rigorous study of steel-concrete composite members in the USA dates to 1908 at Columbia University [3]. Encased composite columns (SRC) originally started as an effort to protect steel columns from fire [4]. Their good performance in the San Francisco Fire of 1907 made encasing steel columns a common practice throughout the USA. It was not until the high-rise boom in the 1960s that concrete-filled tubular (CFT) members became popular. Much of the impetus came from the availability of large built-up square tubes in Japan for increasing seismic resistance [5]. 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 several 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 build up inside the CFT

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column in the event of fire. The same holes are also used to drain residual water before concrete pouring and they must remain unblocked during the life of the structure.

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 which included projects on CFT column systems. Some advantages of the CFT column system have been described by Morino and Tsuda [6] as:

- Local buckling of steel tube is delayed and strength degradation is moderate due to presence of the concrete limiting inward movement of the steel.
- Concrete can develop higher compressive strength due to confining provided by the steel tube.
- Strength degradation of concrete is not as severe as in reinforced concrete columns since spalling is prevented by the steel tube.
- Creep and drying shrinkage of the concrete infill is smaller than for conventional exposed concrete.
- Steel elements in CFT can easily reach full plastification at the outermost section location.
- Concrete improves fire resistance of the steel tube.
- No concrete formwork is required. Reinforcing bars can also be ignored reducing labour, construction time, and cost.
- The construction site is cleaner with less waste.
- The concrete can be separated from the steel tube. Hence both materials can be entirely recycled.

Axially Loaded CFT Members

The compressive strength is less for short rectangular CFT columns since the confining effect is smaller than for a circular CFT column. AISC360 (2010) considers this by using a C2 parameter (average stress for rectangular stress block) equal to 0.95 for circular sections and 0.85 for rectangular ones. Also, for short CFT columns, the axial strength is governed by local buckling of the steel tube while for slender columns global buckling controls. The effect of confinement appears to diminish with member slenderness and this is considered in BS EN 1994-1-1 (2004).

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 design flexural strength may be obtained from plastic design considerations in a similar way to that for reinforced concrete columns [3]. Steel tube confinement may also increase the flexural strength. Empirical methods to obtain the member stiffness are also available (e.g. AISC360, 2010) [7]. These are described later.

Beam-Column Joints and Connections

Several methods, with different costs and performances, have been used to connect beams to CFT columns in moment-frames. A number of these are discussed below. In general, CFT concrete infill resists joint shear deformation resulting in a greater strength that that from the panel zone of a bare steel column. Careful detailing can allow the CFT joint to remain elastic during extreme seismic shaking.

Large scale cyclic tests have been conducted to evaluate the performance of a number of configurations below, but if the beam non-linear action is located away from the joint, using the dog-bone or low-damage connections such as the sliding hinge joint (MacRae et al. 2010) [8], and if capacity design is undertaken to size the column, then CFT column and joint yielding does not occur and monotonic, rather than cyclic, tests may be sufficient to assess the joint behaviour.

Through-Beam Connections

Some testing of beams passing through CFT columns has been undertaken [9, 10]. 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 as shown in Figure 1. These through-beam connections have exhibited stiff and ductile performance.

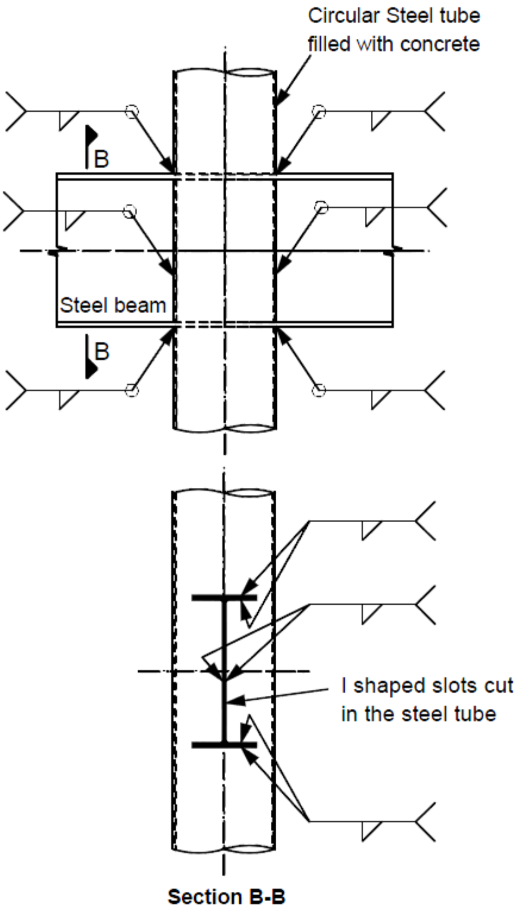

Figure 1: Through-beam connection [9].

Diaphragm Type Connections

Diaphragm type beam-column connections are widely used in Japan, China and Taiwan in two-way and one-way frames (e.g. Toshiyuki and Koji, 2005) [11]. There are several types:

1. Through-diaphragms are used where separate pieces of the column are placed either side of each diaphragm and connected to the diaphragm with full penetration butt welds. This is not only expensive, but it requires special care to keep the column straight and to limit defects which could result in fracture. Robots are sometimes used to make such connections overseas.
2. Internal diaphragms, such as that shown in Figure 2 (top), may also be used, but the electro-slag welding process required for completing the final side requires special equipment and special quality control not currently available in NZ. For both through-diaphragms and internal

diaphragms, a hole is required to place the concrete. The presence of part of a diaphragm inside the tube can interrupt concrete flow and result in poorly compacted concrete beneath the diaphragm when the concrete is placed from the top. The problem is less severe where the high slump concrete is pumped up. This is sometimes used in high rise construction.

3. External diaphragms, such as that in Figure 2 (bottom), are placed on the exterior of the column. They may be more cost effective due to ease of fabrication. However, it may be more difficult to anchor the vertical component of the concrete diagonal compression strut, as described later.

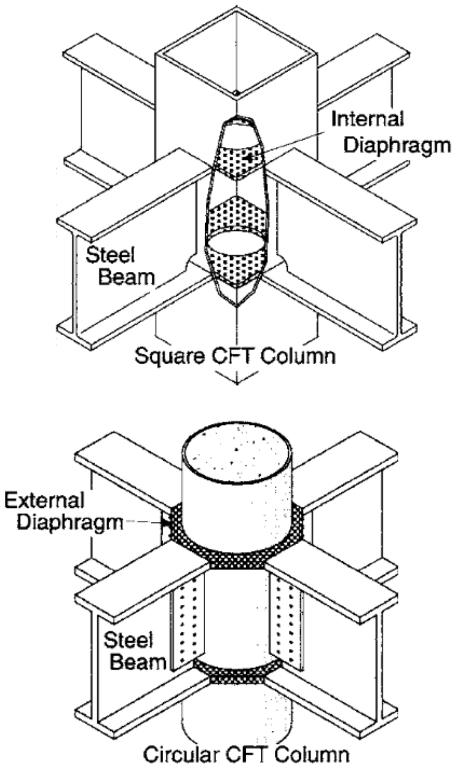

Figure 2: Typical diaphragm type CFT connections [11].

T-Stiffeners

Shin et al. (2004) [12] investigated a T-stiffener connection to a rectangular CFT column. Here beam flange tensile force is transferred through a horizontal plate element, a vertical plate element, and through the web of the rectangular CFT column as shown in Figure 3. The aim was to move plasticity away from the column and the joint. However, the load path was not direct and in most tests failure occurred by weld fracture at the bracket. Here the welding of the bracket to the cold-formed steel bends had led to crack initiation in the tube.

Bolted Moment Endplate Connections

Conventional bolted moment endplate connections for bare steel connections can be used with 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 far side of the CFT column through the bolts. This results in compression on the CFT column steel and concrete rather than having beam flange tension directly to the steel tube skin. For the compression flange, a diagonal compressive strut develops in the joint region concrete. The vertical component of the diagonal strut is resisted inside the column by;

1. friction at the surface between the steel tube and concrete as shown in Figure 4 which depends on the interface conditions,
2. vertical bearing on the ends of the through-bolts, and
3. vertical bearing on any additional mechanical anchors that may exist at or far from the joint.

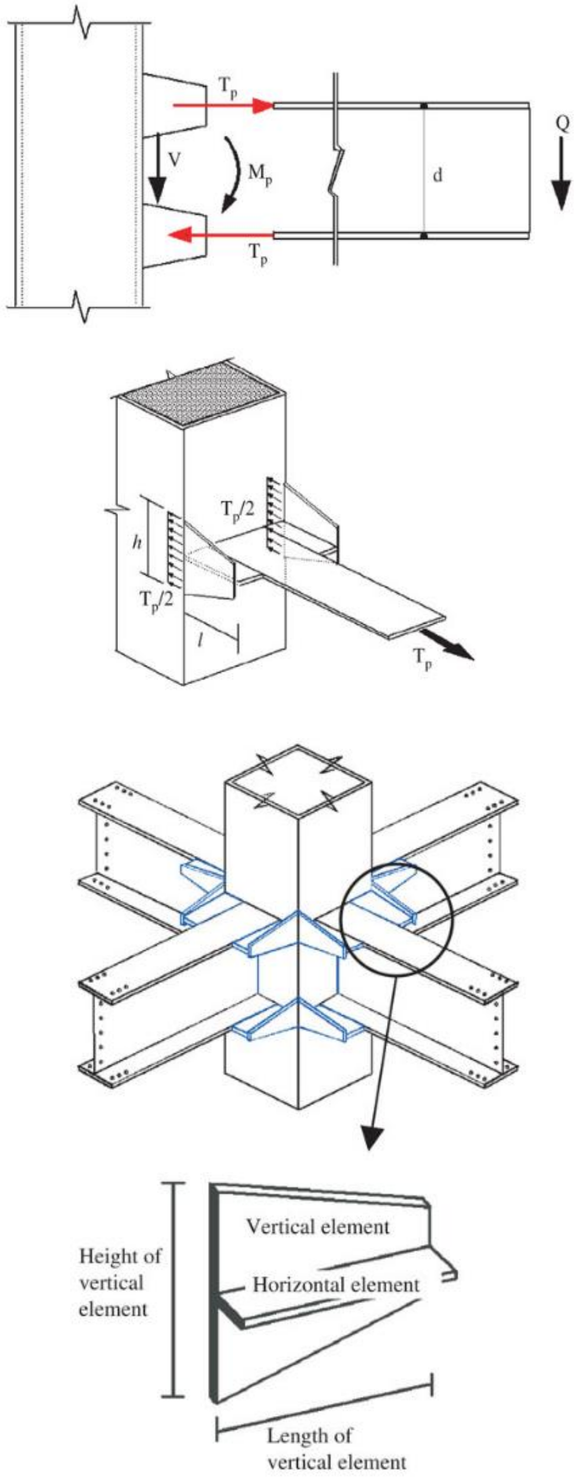

Figure 3: T-stiffener connection [12].

Alternatives to through-bolts (i.e. threaded rods with or without a sleeve passing through the connection) include concrete anchor bolts, blind headed bolts [13], 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, concrete creep and shrinkage may reduce the elastic stiffness, and the strength relies on the concrete tension capacity. Because of this the performance may vary with concrete of different properties.

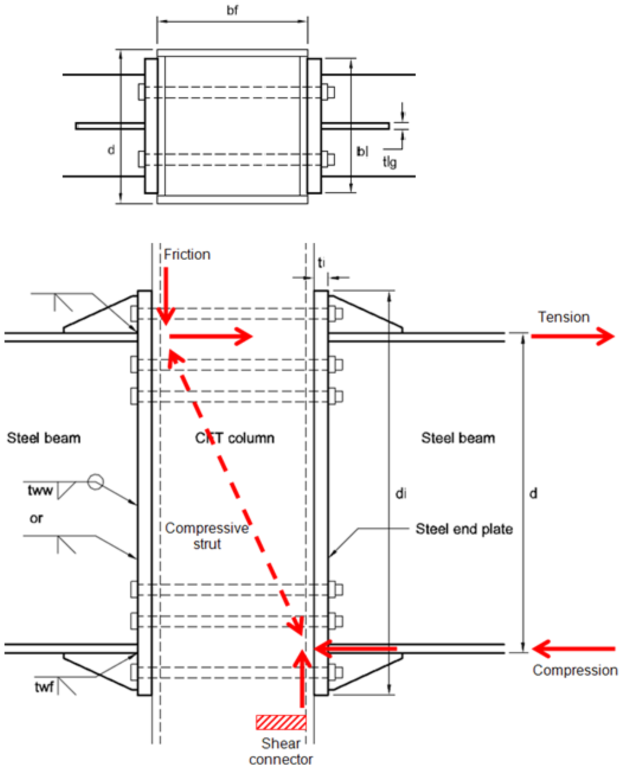

Figure 4: One-way bolted moment end plate connection with through-rods.

In all connections where the concrete is subject to significant stress creep over time can reduce the connection stiffness.

Two way CFT beam-column connections can be constructed by (i) placing bolt holes in both directions at the same level, but curtailing those in one direction and providing a bridge over those in the opposite direction inside the column (e.g. Sheet et al. [14]), or (ii) detailing bolts or rods to be slightly offset so they do not clash inside the tube (e.g. Li et al. [15]). Both of these connection types have behaved well in experiments, although Wu et al. [16] showed that for very thin tube webs, yielding occurred even with concrete infill present.

While a bolted end plate 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 pre-stress across the end-plate column interface may differ at different levels of the frame.

Bolted T-Stub Connections

Bolted T-stub (also called split-tee) 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 with the end plate connection. This type of connection gained significant attention in the US after the 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 different degrees of stiffness and strength depending on the connection element details and they have performed well under cyclic loading [17, 18].

Column Splices

Column splices for CFT members can be designed and built in the same way as for normal steel tubes. Welds, or bolts and splice plates, transfer tension from one tube to another. The concrete infill is not considered to contribute to the tension transfer. The disadvantage of splices in tubular sections is the difficulty of accessing the far (inside) side of the tube.

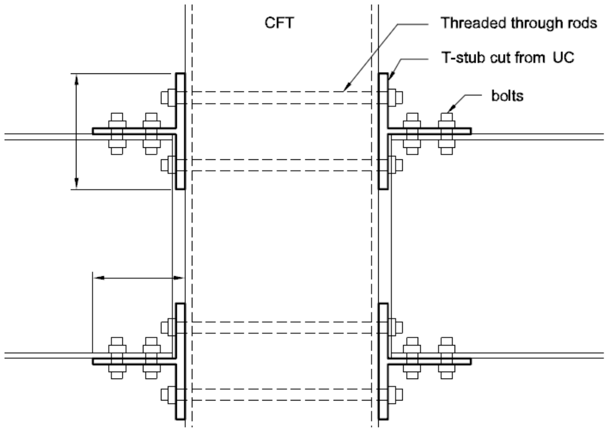

Figure 5: T-stub connection.

Butt welded column splice connections may be made in the shop or on site, and internal backing plates can be used. Bolted splices can be made using internal, or external, splice plates. Possible methods of connecting include welding nuts to the inside of the tube or internal splice plate, using through-bolts which pass through the column, or using a fastener that can be inserted and fastened from one side of the hole such as the AJAX “One Side” bolt. Such a splice is shown in Figure 6.

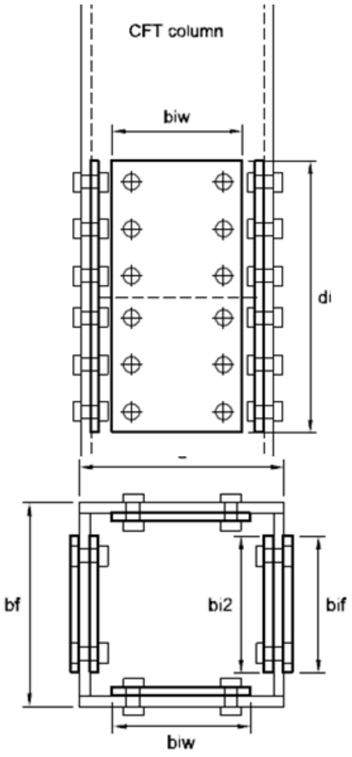

Figure 6: Column bolted splice.

FRAME DESIGN FOR COST ANALYSIS

Bare Steel Frame

The bare steel frame selected for study is modified from the Post-Northridge Design for the Los Angeles site from the SAC steel project. Complete details can be found in Appendix B of FEMA-355C [19].

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 the SAC steel frame and they are summarised in Tables A1 and A2. 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. The plates were welded together.

Section axial strength:

$$\phi N_s = \phi k_f A_n f_y \tag{1}$$

$$\phi N_t = \phi A_g f_y$$

where N_s and N_t are the nominal compressive and tensile strengths; k_f is the form factor taken as 1; A_g and A_n are gross and net cross sectional areas respectively; and $\phi = 0.9$.

Section flexural strength:

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

$$\phi M_s = \phi f_y Z_e \tag{3}$$

Where S is the plastic section modulus and Z is the elastic section modulus.

Moment-axial load interaction equation:

$$M_{rx} = \min \begin{cases} 1.18 M_{sx} \left(1 - \dfrac{N^*}{\phi N_s} \right) \\ M_{sx} \end{cases} \tag{4}$$

Beams

Steel beams sections were obtained from SAC steel frame. The flexural strengths were calculated based on NZS3404:1997 Equations (2) and (3). Table A4 shows the summary of the beam sections.

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 [19]. The bolted moment endplate beam-column connections were designed based on HERA report [20]. 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 steel beams remain the same but the connections used for the rectangular CFT frame were slightly dimensionally modified. If the bare steel columns are replaced with equivalent rectangular CFT columns with the same stiffness then the stiffnesses of all frame members are the same, and the frame period is the same. Also, because the majority of inelasticity was still in the beams, the earthquake response of the frame with both types of columns is similar. Therefore, determination of the best frame may be found based on a cost comparison.

Rectangular CFT Columns

Rectangular CFT strength and stiffness were calculated according to AISC360 [7]. 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:

$$\frac{b}{t} \le 2.26 \sqrt{\frac{E}{f_y}} \tag{5}$$

Section axial strength:

$$\phi_c P_n = f_y A_s + C_2 f'_c A_c \tag{6}$$

$$\phi_t P_t = f_y A_s \tag{7}$$

where P_n and P_t are nominal compressive and tensile strength; A_s is the steel tube area; A_c is the core concrete area; $C_2 = 0.85$ for rectangular and 0.95 for circular sections; and ϕ_c and ϕ_t are 0.75 and 0.9 respectively.

Effective flexural stiffness:

$$EI_{eff} = E_s I_s + C_3 E_c I_c \tag{8}$$

$$C_3 = \min \begin{cases} 0.6 + 2 \left(\dfrac{A_s}{A_c + A_s} \right) \\ 0.9 \end{cases} \tag{9}$$

Flexural strengths of the sections were calculated following the plastic stress distribution method outlined in the Commentary of AISC360 [7] and in Appendix B of Viest et al. [3]. The method assumes a linear strain distribution across the section and elastoplastic 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.

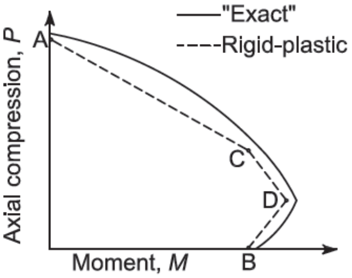

Figure 7: Moment-axial load interaction diagrams [7].

Tensile stress in concrete is ignored. Once the plastic neutral axis is found, the three points (B, C, D in Figure 7) 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 the axial load is zero. Points D and C are obtained by calculating the section moment and axial load capacities assuming that the neutral axis moves to the middle of the section and at the same distance to the other side of the middle of the section.

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. Welding is provided at each corner. 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 8 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. The summary of the CFT columns in the frames designed is given in Tables A5 to A12 of the Appendix.

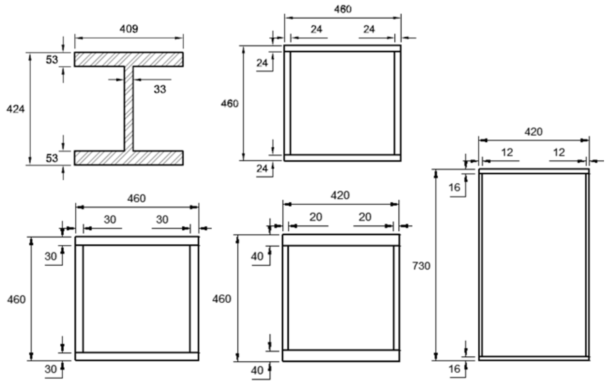

Figure 8: Example of rectangular CFT columns. Base steel case (Top-left), A-EIx (Top-right), A-Mx (Bottom-left), B-Mx (Bottom-middle), C-Mx (Bottom-right).

It was found during the design optimisation phase that when the stiffness of the rectangular CFT section is matched to that of the steel section, the rectangular CFT section tensile strength was reduced. This is because in the rectangular CFT less steel area is needed in the cases considered. However, the impact of smaller tensile strength of a CFT section is small under the normal range of axial load ratios.

Moreover, the flexural strength of a rectangular CFT section that has the same stiffness as a steel section is also less than the strength of the same stiffness steel section.

The overall dimensions of the rectangular CFT section in matching stiffness was not significantly smaller than steel section due to similar requirement of flange thickness and section depth to produce flexural stiffness. This implies that the amount of rentable floor space is also not likely to be significantly increased, and in some configurations it may be decreased. However, the rectangular CFT section has a high weak axis bending capacity allowing more efficient two-way frames.

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. That is EI_{eff} (composite) is the same as EI_{eff} (steel). The strength is relatively less than for the bare steel column.

Design case A-Mx:

The design was also symmetric but the rectangular CFT columns were designed to match the flexural strength in the strong axis of the steel columns at every storey. Flexural stiffness is greater than for the bare steel case and the tensile strength is less due to the smaller steel cross sectional area.

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 was not more than one size increment thicker than the web thicknesses considering the standard plate sizes. The flexural strength in strong axis was set to that of the steel column at every storey. The flexural stiffness is greater than the bare steel case and the tensile strength is less due to smaller steel cross sectional area.

Design case C-Mx:

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

Gravity columns:

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

Beams and Connections

Steel beams and connections for rectangular CFT column frames were the same as for the bare steel frame except for some minor dimension modifications on the connections due to different column sizes.

Alternative Beam-Column Moment Connections Design

Some alternative rectangular CFT beam-column connections were preliminarily designed to compare 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 are no available guidelines. 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 9, so the bolt hole and washer will need to be large enough to allow the anchor bolt to be inserted from the beam side.

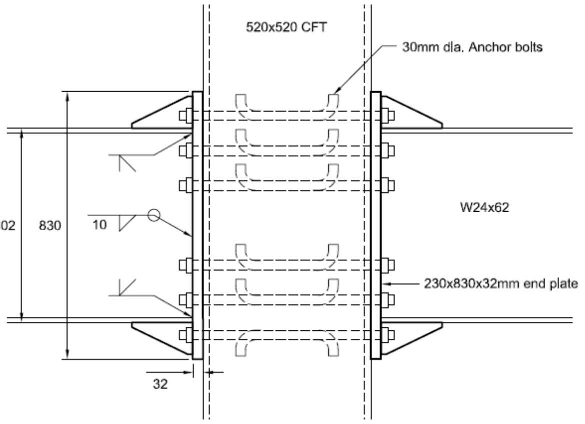

Figure 9: Example of moment endplate detail with anchor bolts.

Alternatively, headed shear studs, welded inside the tube shell, have been proposed as a replacement for through-bolts [21]. 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 inspection with associated cost increment arises,
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 supports are needed until studs are in the hardened concrete.

Through beam connection:

Design recommendations of through-beam connections were proposed by Elremaily and Azizinamini (2001) [10] based on their finite element analyses and experiments on circular CFT. The equations below have been slightly modified for rectangular CFT columns. The column-to-beam moment ratio 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, ϕV_{jh} depends on the beam stub strength, ϕV_{wn} the concrete panel strength, ϕV_{csn} and the tube shell strength, ϕV_{tn} as given in Equations 10-13, where ϕ = 0.7, f_{yw} and f_{yt} = yield strength of beam web and tube, f'_c = concrete compressive strength (MPa), d_c = depth of beam web, t_w = beam web thickness, A_s and A_c = cross sectional area of steel tube and concrete.

$$\phi V_{jh} = \phi\,(V_{wn} + V_{csn} + V_{tn}) \tag{10}$$

$$V_{wn} = 0.60 d_c t_w f_{yw} \tag{11}$$

$$V_{csn} = 1.99 A_c \sqrt{f'_c}\ \text{(MPa)} \tag{12}$$

$$V_{tn} = 0.60 A_s f_{yt} \tag{13}$$

The proposed design equations for weld design between the beam flange and tube shell are given in Equations 14-15, where ϕ_b is 0.9, f_{yf} is the beam flange yield strength, b_f and t_f are the beam flange width and thickness, t_t is the tube wall thickness, and σ_y is the maximum tensile stress in tube wall corresponding to column moment and axial load.

The horizontal force per weld unit length, h, is

$$h = 1.5\left(\frac{0.3\phi_b f_{yf} t_f b_f}{2 b_f}\right) \tag{14}$$

The vertical force per weld unit length, v, is:

$$v = \sigma_y t_t \tag{15}$$

Diaphragm connection:

Empirical equations for calculating the strength of through-diaphragms, $P_{u,int}$, as shown in Figure 10, and external diaphragms, $P_{u,ext}$, as shown in Figure 11 were proposed by Morino and Tsuda [6] using the equations below.

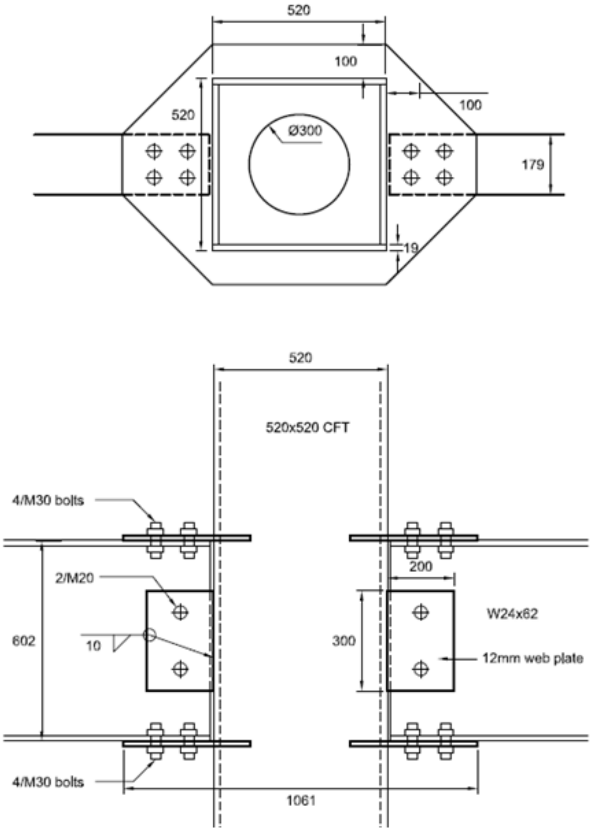

Figure 10: Through diaphragm beam-column joint.

$$P_{u,ext} = 1.42\left(2\left(4t + t_s\right) t F_1 + \frac{4}{\sqrt{3}} h_s t_s F_2\right) \tag{16}$$

$$P_{u,\mathrm{int}} = 1.42\left(D + 2h_s - d_f\right)^2 \left(\frac{B_f t_s}{d_f^2}\right) F_2 \tag{17}$$

Here P_u is the ultimate tensile strength of diaphragm; t is the rectangular CFT thickness; t_s is the diaphragm thickness; F_1 and F_2 is the design strength of steel tubes and diaphragm respectively; d_f is the diameter of the internal opening for concreting; h_s is the diaphragm dimension at corner of rectangular CFT.

The main problem with through-diaphragm fabrication is the steel tube needs to be cut and welded to the top and bottom side of the diaphragm. Complete penetration butt welds are 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.

However, there were weld fracture failures 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.

A variation on the external diaphragm connection is the “flange-bolted joint” proposed by Clifton [22].

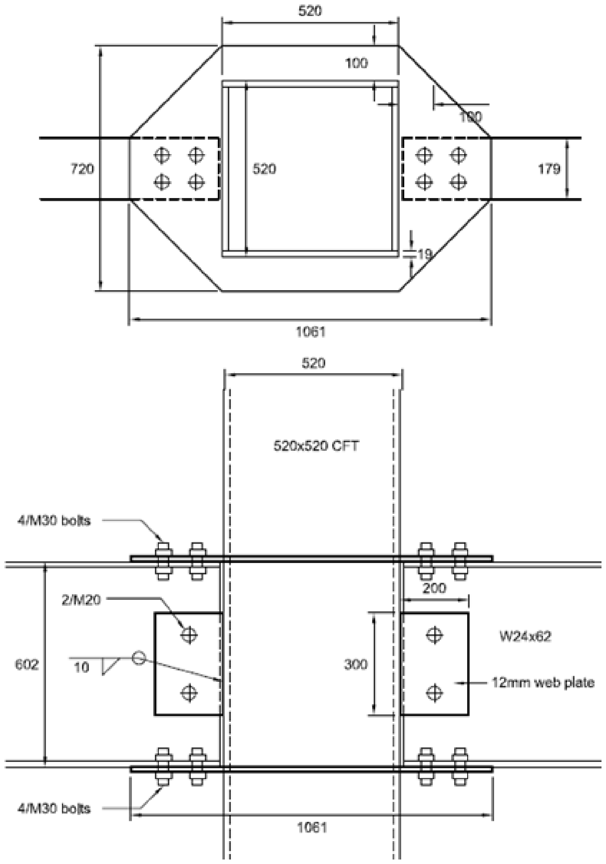

Figure 11: External diaphragm beam-column joint.

T-stub connection:

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

T-stub connection type can be easily fabricated and erected. Using bolts allows position adjustment during erection. No field or shop welding is required. The T-stub connection design for the project is shown in Figure 12. It is also relatively easy to make a low damage connection by fixing the top flange and allowing energy dissipation on the bottom flange. Details of this type are described in [8] and [23] for different types of energy dissipation mechanism.

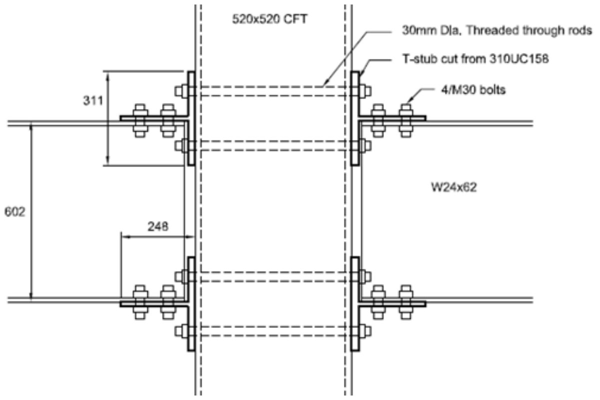

Figure 12: T-stub connection.

Table 1 presents a summary of the relative advantages and disadvantages of the different 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.

FRAME ANALYSIS

Frame Models

Simple 2D analytical models were created and analyses were conducted using the finite element code programme RUAUMOKO [24]. The main 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

Table 1: 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

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, except perhaps at the base, are expected to remain elastic.

Simplified assumptions were made in the modelling process and kept consistent among the two frames. The models were centreline models with all the members 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 [24] was used for all flexural members. The plastic hinge length at each end was approximated at one sixth of member span to ensure that the moment-curvature bilinear rule is consistent with force-displacement relationship. Columns were modelled 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 laterally restrained so out-of-plane deformation cannot occur. Steel columns strengths were obtained from moment-axial load interaction diagrams based on NZS 3404 while the 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 [19]. Gravity loads were derived from seismic masses and evenly distributed to the members proportionately to the tributary floor areas.

Since this paper focuses on system behaviour, frames were designed such that only beam elements and the bases of columns may sufferer significant yield. This and other modelling issues are discussed in [25].

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.

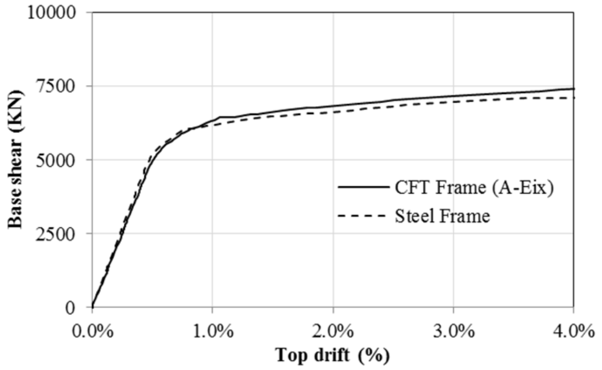

Figure 13: Pushover curves.

The fundamental periods from modal analyses are almost the same, 2.3 and 2.2 seconds for steel frame and CFT frame respectively. As can be seen from Figure 13, the pushover curves of the two frames are similar. The slightly different frame stiffness is due to geometrical differences in column size, beam length and plastic hinge zone characterization. The base shear capacities are 5900 and 5600 kN for steel and CFT frames, respectively. The 5% difference comes from column geometry 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. The 2011 Christchurch earthquake records measured from three stations, Botanical 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 14. 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.

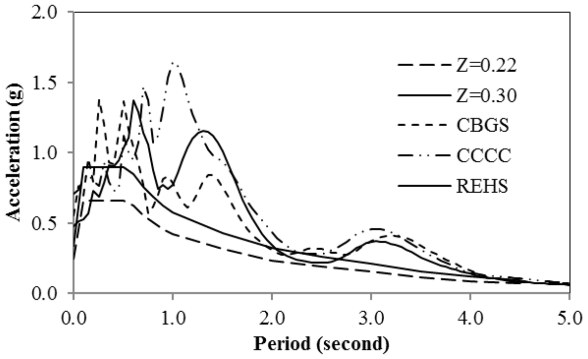

Figure 14: Acceleration spectra.

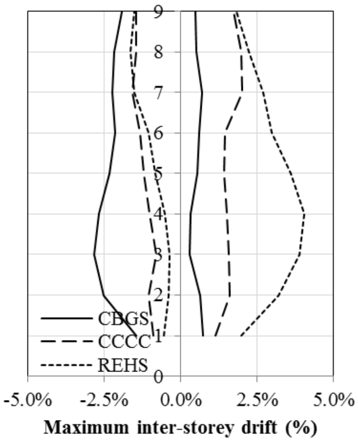

Figure 15: Maximum inter-storey drift of steel frame.

During the frames analyses beam plastic hinges in every storey were activated and most of them occurred before the column bases yielded in flexure. Figures 15, 16 and 17 indicate that the response was not first mode, but the higher modes contributed significantly to the demands. The peak demands in both CFT and steel frames were similar as would be expected given that the stiffness is similar and yielding was dominated by the beams which are identical in both frames.

The steel frame maximum inter-storey drifts demand are slightly higher than, but comparable to, that of the rectangular CFT frame in Figure 17.

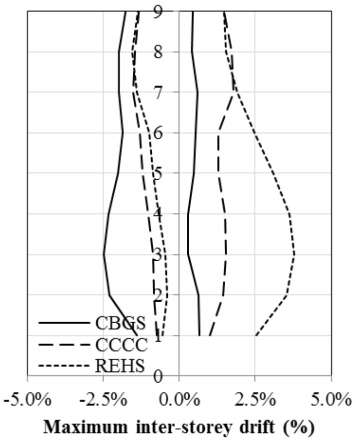

Figure 16: Maximum inter-storey drift of CFT frame.

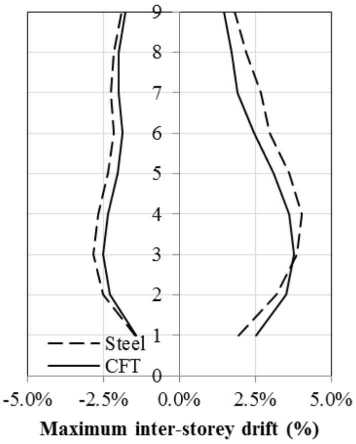

Figure 17: Maximum inter-storey drift envelopes.

FRAME COST STUDY

Since the responses of the frames with similar stiffness are similar, the major parameter affecting whether or not an engineer will select a composite rectangular CFT column is the cost. For the columns designed according to strength, analyses were not conducted, but the column stiffnesses are higher indicating a smaller period. While the design forces are likely to be greater, the beam sizes are generally governed by stiffness, so this means that member sizes would not necessarily increase. Also, the drifts may be slightly smaller.

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 provided by John Jones Steel Ltd, Christchurch, NZ. It should be noted that the size of SAC steel frame chosen for the study was large compared to that commonly used by the NZ steel industry so lower unit costs may be obtained in reality.

Fire proofing costs were also considered. A simple fire design for the bare steel frame was carried out as per NZS 3404 [26] and Clifton [27] with the following assumptions:

- 30 minute fire rating requirement (FRR).
- 3 side exposure for the external side columns.
- 4 side exposure for the internal moment columns and the gravity columns.

It was found that the internal moment columns and the gravity columns did not satisfy the 30 minute fire rating requirement. The extra cost of the required fire proofing for the bare steel columns was estimated according to Rawlinson New Zealand Construction Handbook 2012 [28] using the sprayed gypsum based cementitious Monokote MK6/HY fire proofing product.

The CFT sections met the dimensional requirement of Eurocode [29] and did not need extra fire protection. Therefore, there was no extra fire proofing cost applied to the CFT columns.

The cost study in this paper has been completed on the basis of the material and installation unit costs provided by John Jones Steel New Zealand.

Study 1: Cost of Columns

Average per-metre cost of steel and rectangular CFT columns were derived and summarised in Table 2 for the cases considered in Figure 8. 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.

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 built-up CFT member is significantly affected by the welds. In NZ, sections up to about 400mm square can be formed by bending two plates into C-sections and welding them together at the tips to form a rectangle with two full penetration welds along the length. However, the sizes here are larger than this. Since 4 plates were required, the fabrication method used for the rectangular CFT used four flat plates and fillet welds. Full penetration butt welds could also be used but their cost may as high as ten times the fillet weld cost for the same plate thickness due to inspection costs. Partial penetration butt welds can also be used. Thinner steel plates and fillet welds can be used with a larger section. However, this may reduce the rentable floor space.

Study 2: Cost of a 2D Moment Frame

Cost components of a moment frame are shown in Table 3 and Table 4. It can be seen that the biggest contribution to the frame cost comes from the beams. Columns contribute around 26 to 39 percent while around 10 to 12 percent comes from connections. 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.

Study 3: Cost of 3D Frames (Entire Building)

The last consideration in the cost study was the internal gravity frames. The cost summary of entire building, resulting from the costs of the moment frame only and the gravity frame only, is presented in Table 5. This includes cases when the gravity frame uses steel or composite columns.

Table 2: Cost comparison for moment frame column including fire protection.

Design cases	Average per metre cost components (NZD)			
	Supply cost	Site erection	Concrete material	Total
Steel	$ 337	$ 571	$ -	$ 908
CFT Case A-EIx	$ 803	$ 426	$ 15	$ 1,244
CFT Case A-Mx	$ 1,021	$ 539	$ 14	$ 1,574
CFT Case B-Mx	$ 1,017	$ 539	$ 13	$ 1,570
CFT Case C-Mx	$ 525	$ 350	$ 21	$ 896

Table 3: Cost comparison for one moment frame including fire protection.

Design cases	Cost per one moment frame (NZD)			
	Columns	Beams	Connections	Total
Steel	$ 680,700	$ 1,562,600	$ 293,200	$ 2,536,600
CFT Case A-EIx	$ 932,800	$ 1,562,600	$ 306,000	$ 2,801,300
CFT Case A-Mx	$ 1,180,500	$ 1,562,600	$ 316,400	$ 3,059,500
CFT Case B-Mx	$ 1,177,400	$ 1,562,600	$ 317,200	$ 3,057,100
CFT Case C-Mx	$ 671,900	$ 1,562,600	$ 301,100	$ 2,535,700

Table 4: Relative cost comparison for one moment frame including fire protection.

Design cases	Cost per one moment frame (NZD)			
	Columns	Beams	Connections	Total
Steel	27%	62%	12%	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%

Table 5: Cost comparison for entire frame structure including fire protection.

Design cases	Cost per entire building (frames only) (NZD)			Ratio of bare steel structure
	Moment frame	Gravity frame	Total	
Steel	$ 10,119,200.00	$ 1,519,700.00	$ 11,638,900.00	1.00
with CFT gravity frame				
CFT Case A-EIx	$ 11,205,300.00	$ 2,208,000.00	$ 13,413,300.00	1.15
CFT Case A-Mx	$ 12,238,000.00	$ 2,208,000.00	$ 14,446,000.00	1.24
CFT Case B-Mx	$ 12,228,500.00	$ 2,208,000.00	$ 14,436,500.00	1.24
CFT Case C-Mx	$ 10,142,700.00	$ 2,208,000.00	$ 12,350,600.00	1.06
with steel gravity frame				
CFT Case A-EIx	$ 11,205,300.00	$ 1,519,700.00	$ 12,725,000.00	1.09
CFT Case A-Mx	$ 12,238,000.00	$ 1,519,700.00	$ 13,757,700.00	1.18
CFT Case B-Mx	$ 12,228,500.00	$ 1,519,700.00	$ 13,748,200.00	1.18
CFT Case C-Mx	$ 10,142,700.00	$ 1,519,700.00	$ 11,662,400.00	1.00

The rectangular CFT gravity column is more expensive than its steel counterpart because of the butt weld requirement described previously. In smaller projects, hollow sections can be used to reduce the cost of building up the section. The gravity frame cost is around 11% to 18% and the major cost contribution comes from moment resisting frames. The overall conclusion from Table 5 is the costs of rectangular CFT moment frame structures range from 0% to 18% higher than bare steel structure when it is compared using the same steel gravity frame.

CONCLUSIONS

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 connections 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 material and construction costs. 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, bolted end-plate and external diaphragm connections seem reasonable for rectangular CFTs and external diaphragm connections for circular CFT are preferred. All of these options allow easy site adjustment during erection reducing construction cost.
2. The behaviour of a particular building system with rectangular CFT columns is compared to one with traditional structural steel I-shaped columns. One-way seismic frames with bolted-end plate connections at the end of the beams to the columns are used. 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. The relative costs of the entire building with composite columns ranged between 100% and 124% 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 the butt weld requirement for large section fabrication.
3. For structures in which the columns are expected to carry moment from the 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 which may change the relative desirability of columns with CFT composite construction include different design procedures for strength stiffness and fire, multidirectional loading, architectural space, the need for formwork, different tube construction methods, rolled versus built-up sections and the extent of welding used, the use of low damage connections with composite columns, the use of different beam-column joint details (e.g. bonded vs. unbounded rods in beam end-plate connections).

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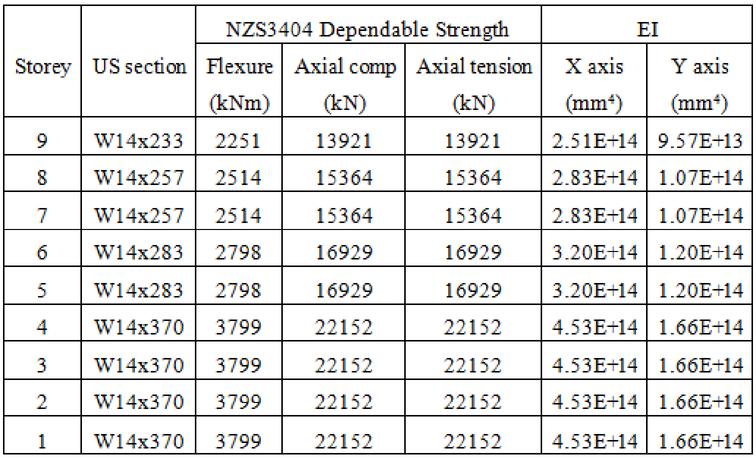
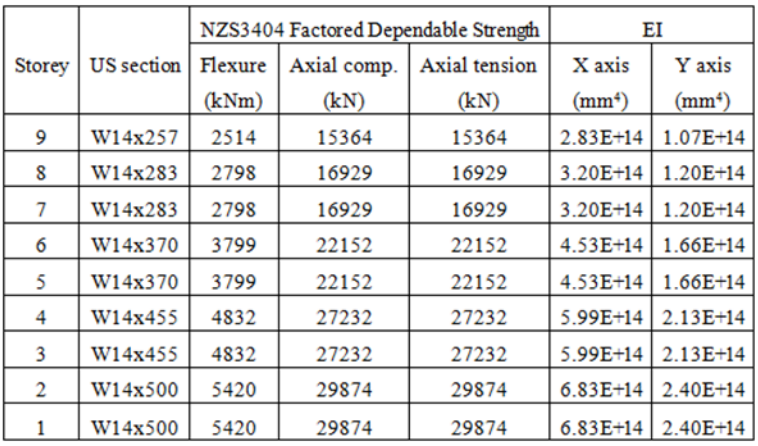
APPENDIX

Table A1: Bare steel moment resisting frame; exterior seismic columns.



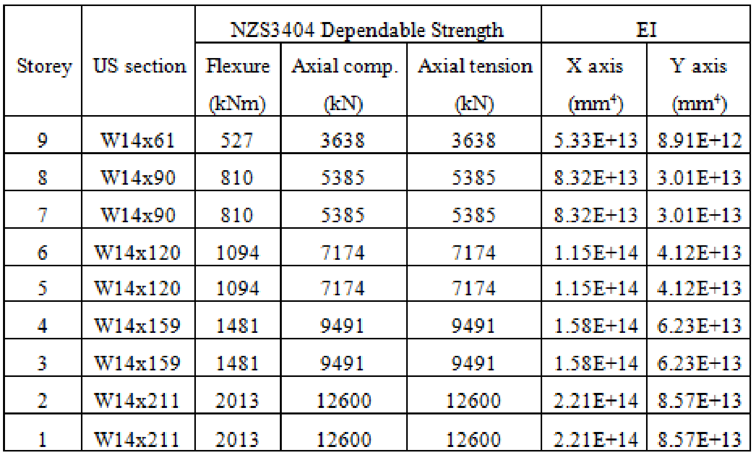
Storey	US section	NZS3404 Dependable Strength			EI	
		Flexure (kNm)	Axial comp (kN)	Axial tension (kN)	X axis (mm⁴)	Y axis (mm⁴)
9	W14x233	2251	13921	13921	2.51E+14	9.57E+13
8	W14x257	2514	15364	15364	2.83E+14	1.07E+14
7	W14x257	2514	15364	15364	2.83E+14	1.07E+14
6	W14x283	2798	16929	16929	3.20E+14	1.20E+14
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 A2: Bare steel moment resisting frame; interior seismic columns.



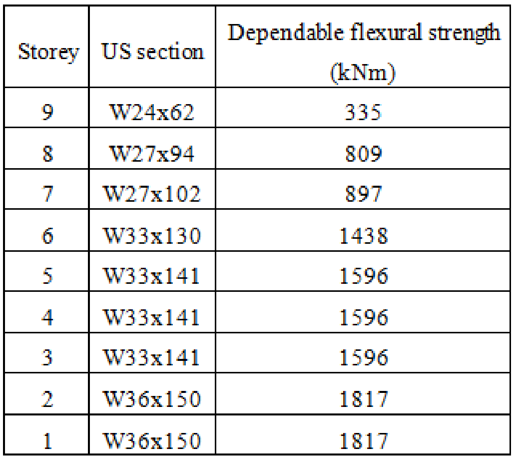
Storey	US section	NZS3404 Factored Dependable Strength			EI	
		Flexure (kNm)	Axial comp. (kN)	Axial tension (kN)	X axis (mm⁴)	Y axis (mm⁴)
9	W14x257	2514	15364	15364	2.83E+14	1.07E+14
8	W14x283	2798	16929	16929	3.20E+14	1.20E+14
7	W14x283	2798	16929	16929	3.20E+14	1.20E+14
6	W14x370	3799	22152	22152	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 A3: 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

Table A4: 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

Table A5: 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 A6: 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

Table A7: 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 A8: 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

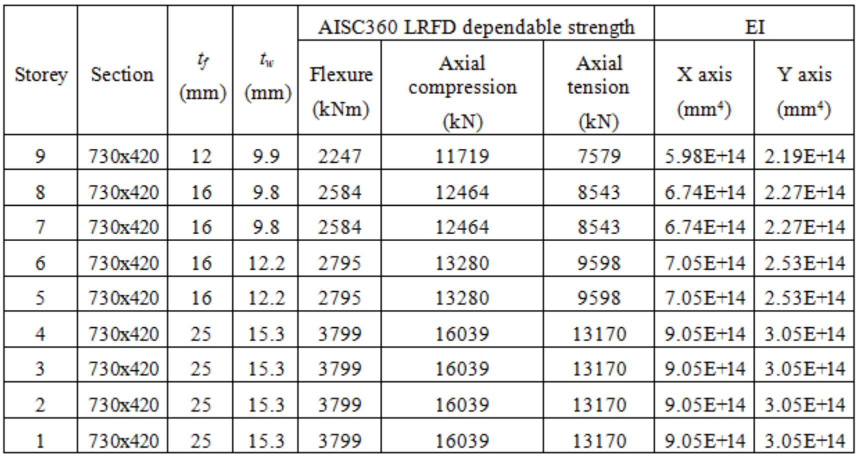
Table A9: Design case B-Mx: Exterior rectangular CFT columns.

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 A10: Design case B-Mx: Interior rectangular CFT columns.

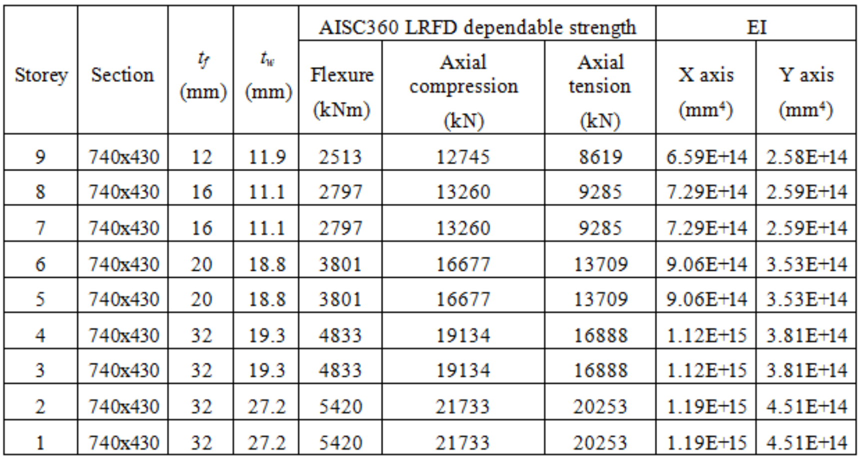
Storey	Section	t_f (mm)	t_w (mm)	AISC360 LRFD factored strength			EI	
				Flexure (kNm)	Axial compression (KN)	Axial tension (KN)	X axis (mm^4)	Y axis (mm^4)
9	520x520	19.4	19.4	2518	14626	12237	4.24E+14	4.24E+14
8	520x520	22	22	2802	15837	13805	4.59E+14	4.59E+14
7	520x520	22	22	2802	15837	13805	4.59E+14	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

Table A11: Design case C-Mx: Exterior rectangular CFT columns.



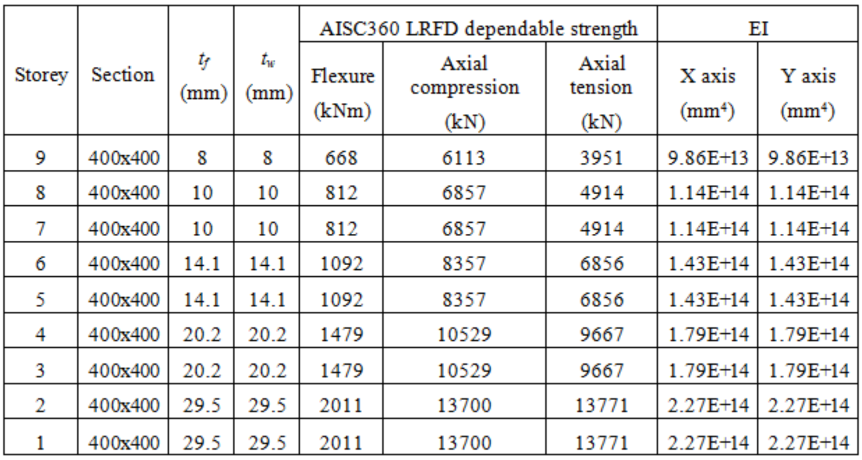
Storey	Section	t_f (mm)	t_w (mm)	AISC360 LRFD dependable strength			EI	
				Flexure (kNm)	Axial compression (kN)	Axial tension (kN)	X axis (mm^4)	Y axis (mm^4)
9	730x420	12	9.9	2247	11719	7579	5.98E+14	2.19E+14
8	730x420	16	9.8	2584	12464	8543	6.74E+14	2.27E+14
7	730x420	16	9.8	2584	12464	8543	6.74E+14	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 A12: 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

Table A13: 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