Design of the Linked Column Frame Structural System – A New Zealand Application

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1.0 Introduction

The Link Column Frame (LCF) system is a brace free hybrid system combining proven seismic load resisting technology; eccentrically braced frames (EBF) with removable links and moment resisting frames (MRF). It was developed to meet the requirement for continued occupancy, or at least rapid return to occupancy after a severe earthquake. This is a departure from the prevailing New Zealand seismic design approach pre the 2010/2011 Christchurch earthquake series which involves designing for controlled damage (energy dissipation) in selected elements of the seismic load resisting system which are typically not rapidly or cost effectively repaired.

Engineers familiar with the design of EBF and MRF systems will readily understand and be able to apply the LCF frame structural system design concepts. The only departure from standard office practice is the requirement to undertake a non–linear push over analysis. As a result, practicing engineers will likely find the design methodology for this system easier to implement than those for some other low damage seismic load resisting solutions.

Useful background information to the LCF system is found in the paper of Dusicka et. al.[1]. It is recommended this is read in conjunction with the present paper which is intended to illustrate the application of the capacity design principles of the Steel Structures Standard (NZS 3404 [2]) to this new system.

2.0 Anatomy of the Linked Column Frame System

The general arrangement of an LCF system, which consists of energy dissipating linked columns (LC) and self-centring moment resisting frames (MRF), is shown in figure 1. The linked column consists of two closely spaced columns, which are connected via bolted replaceable shear links. The linked columns are then connected to one another via an elastically designed MRF. The MRF beams are also gravity load carrying elements and are intentionally designed for low lateral stiffness by having one end of the beams connected with pinned or semi rigid connections.

The column bases should either be pinned or semi-rigid to avoid column base hinging. If the linked column bases are pinned, it is recommended an additional replaceable link is located at the LC base to improve the ground floor frame stiffness.
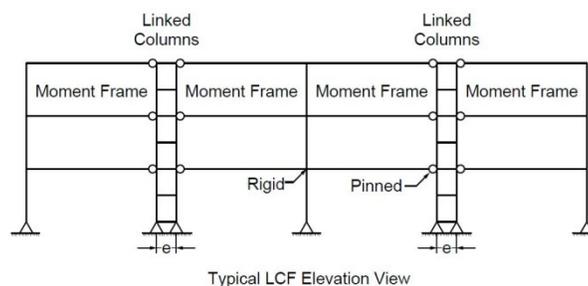

Figure 1: LCF – general arrangement

3.0 The Concept

The LCF system is designed utilising a performance based design approach with an emphasis on rapid return to occupancy after an earthquake event [1].

The lateral response of the LCF system is a combination of the contribution of the LC and the MRF, giving rise to three performance levels (figure 2). These are: elastic, rapid return to occupancy with damage limited to the removable links, and collapse prevention where the MRF is also damaged [1].
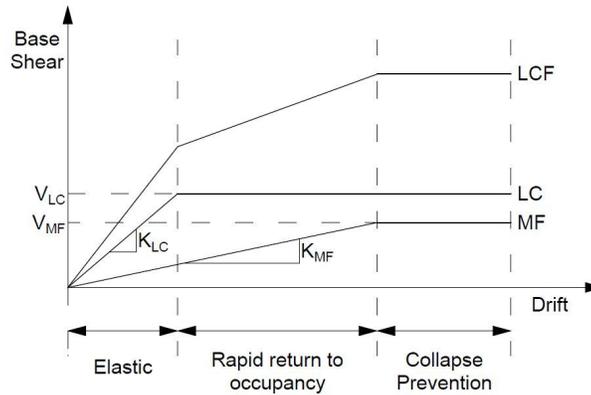

Figure 2: Idealised lateral response [1]

The effectiveness of the rapid return to occupancy is dependent on the relative transition from elastic to plastic response of the LC and MRF. The ideal system is one in which the range of displacements over which rapid return to occupancy occurs is large. A measure of this range is the ratio of the yield displacements (Δ_{LC} and Δ_{MRF}) of the LC and MRF elements,

$$\frac{\Delta_{LC}}{\Delta_{MRF}} = \frac{V_{LC} K_{MRF}}{V_{MRF} K_{LC}} \tag{1}$$

V_{LC} and V_{MRF} are the idealised base shears at the onset of yielding and K_{LC} and K_{MRF} are the stiffness's of the separate components. Ratios of Δ_{LC}/Δ_{MRF} less than one are required to ensure there is a rapid return to occupancy performance level. Ratios of 0.5 or lower are desirable to provide a significant rapid return to occupancy drift range [1].

4.0 Applicability of the LCF System

Owing to its reliance on flexural behaviour, the LCF will tend to be utilised in similar applications to moment resisting frames. Post the main Christchurch earthquake event, many structural engineers are designing buildings for smaller allowable inter-storey drifts than those prescribed in the Loadings Standard [3]. With this in mind, plus the relatively large overturning actions generated by the closely spaced link columns, it is likely the LCF system will be most efficient in the 3-6 storey range. The actual number will be dependent on regional seismicity, soil conditions, the size of the building foot print and the number of LCF bays.

5.0 Design Methodology

5.1 Introduction

Conventional capacity design and analysis procedures for D braced frames with removable links and MRF's are appropriate for LCF structural systems with only minor modification. The only additional step to ensure the rapid return to occupancy objective is appropriately realised, is to undertake a non-linear push over analysis to confirm yielding is limited to the energy dissipating link elements at the design level ultimate limit state earthquake. This may introduce an element of iteration to the design if it is found the elastic moment frame is too stiff relative to the linked column frame and will be subject to yielding under the ultimate limit state inter-storey displacements.

The design methodology presented in this paper for LCF structural systems is based on the seismic provisions in the New Zealand Steel Structures Standard [2] and HERA report R4-76: *Seismic Design of Steel Structures* [4] (which itself is undergoing a full revision of the EBF design provisions).Where design requirements are similar to conventional design approaches, reference will be made to previously published material. The aspects of design that are unique to the LCF system are discussed in greater detail.

5.2 System ductilities and member category

The appropriate structural displacement ductility factors for the energy dissipating LC and the self-centring MRF are limited ductility (μ=3) and nominally ductile (μ=1.25) respectively. The recommendation to treat the elastically responding MRF as nominally ductile is to ensure robustness in the eventuality of an earthquake larger than the design level earthquake. The appropriate member categories are:

LC

Element	Designation	Member category
Link	Primary element	2
Column	Secondary element	3

MRF

Element	Designation	Member category
Beam	Primary element	3
Column	Secondary element	3

5.3 Link Column Frame

5.3.1 Links

5.3.1.1 Introduction

The removable links in LCF systems are identical to those used in eccentrically braced frames with removable bolted links. The principle source of design guidance for removable links is found in Steel Advisor article EQK 1006 [6]. The removable link with moment end plate (figure 3) is well researched and has shown in testing to give similar seismic performance to conventional eccentrically braced frame links in which the collector beam and active link are one continuous element [6].
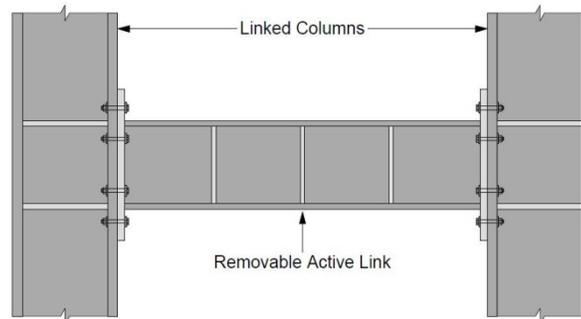
Figure 3: Removable link arrangement

5.3.1.2 Sizing Links

It is recommended link lengths are limited to ensure shear yielding is the mode of energy dissipation.

$$e_{max} \leq 1.6 \, \frac{M_P}{V_P} \qquad (2)$$

The following recommendations are made for sizing the links.

1. Dusicka et.al. [1] identified the most efficient approach to controlling seismic displacements was to increase the link and LC column second moments of inertia. Custom welded links are a good option for decoupling strength and stiffness. This allows designers to tailor the link shear capacity to the demand through careful selection of the web depth and thickness. Link stiffness can be controlled by manipulating the link flange sizes. Such optimisation is not possible for hot rolled sections, even though their cost per kg is cheaper.

2. To ensure fabrication efficiencies, links are sized in bands over multiple floors. Rather than using the NZS 3404 [2] approach of undertaking shear redistribution up to 5%, it is recommended sizing links for the average link shear force in each band. Research has shown [9] the seismic performance of eccentrically braced frames designed using this approach is very similar to that using the NZS 3404 recommendations.

5.3.1.3 Link End Plate Connection

To ensure dependable link connection performance, the removable link end plate connection must be designed for the capacity design derived actions (figure 4). The design and detailing of the removable link end plate connection is covered in Steel Advisor article EQK 1006 [6].
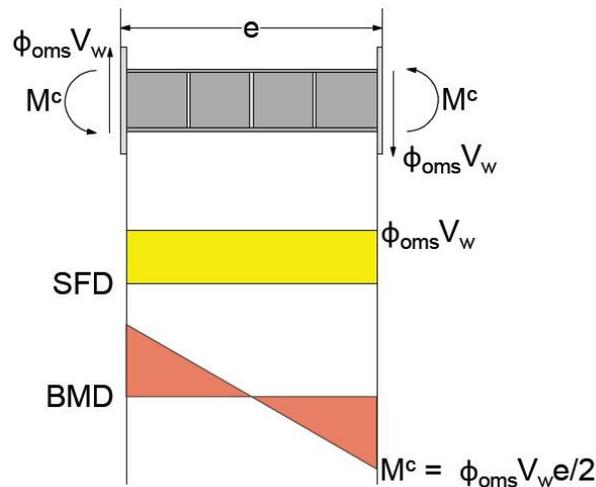
Figure 4: Design actions - removable link

To ensure practical link end plate moment connections, particularly for large link sections, it is recommended limiting e_{link} to ensure $M^c_{link} < (50 - 60\%)x\ \Phi M_{s,Link}$

5.3.1.4 Link Plastic Rotation Limit
Limited ductile (μ=3) LCF system designs that comply with the interstory drift requirements of the Loadings Standard NZS 1170.5 [3] will not exceed the plastic rotation limits in NZS 3404 [2] for shear yielding links. The plastic rotation for a LCF system undergoing 2.5% inter-storey drift is approximately 0.022 radians which is much less than the allowable 0.08 radians.

5.3.2 Columns
5.3.2.1 Member Design
The LCF columns must be designed for the capacity design derived actions associated with shear yielding of the links (figure 5). In addition, the dynamic moment magnification and slab participation factors applicable for a category 2 D braced frame from [2] must be used to scale up the column design actions.
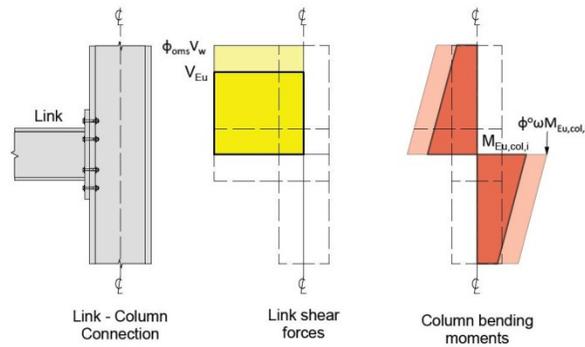
Figure 5: Capacity design derived LC column actions

$$M_{col,i}^c = \phi^o \, \omega M_{Eu,i} \qquad\qquad (3)$$

Where

$$\phi^o = \frac{\phi_{omss} V_w}{V_{Eu}} \qquad\qquad (4)$$

ω is the column dynamic magnification factor from NZS 3404 Clause 12.11.7.4

ϕ_{omss} is the over strength factor incorporating slab   participation factor

5.3.2.2 Lateral restraint

To ensure dependable column performance, the column lateral restraint requirements should comply with the requirements of NZS 3404 [2] clause 12.6.2.2 i.e. $\alpha_m\alpha_s \geq 1.0$ assuming $\alpha_{m,max}=1.75$. This check is made between successive active links.

The top and bottom flanges of the active link must be laterally restrained both ends (NZS 3404 [2]12.11.6). The restraining force is to be 2.5% of the capacity design derived link flange force. In addition to this strength requirement, there is also a minimum stiffness criteria (<4mm from NZS 3404 Clause 12.11.6.2). For the mid height link, the lateral restraint will be provided by the column bent about its minor axis which will easily meet the stiffness requirement.

It is reasonable to apply the restraint force through the column section centroid (figure 6). The justification for this location rather than applying the 2.5% at the column face is the MEP connection bent about its minor axis and the column web stiffeners will possess sufficient flexural strength to ensure the active link effectively acts as a continuous member spanning from column centre line to column centre line. The Steel Structure Standard *Members subject to combined actions* (section 8) provisions must be used to check column biaxial section (8.3.4) and member (8.4.5) capacities.

For most practical column and link sizes, this additional link restraint loading is not anticipated to govern the column design.
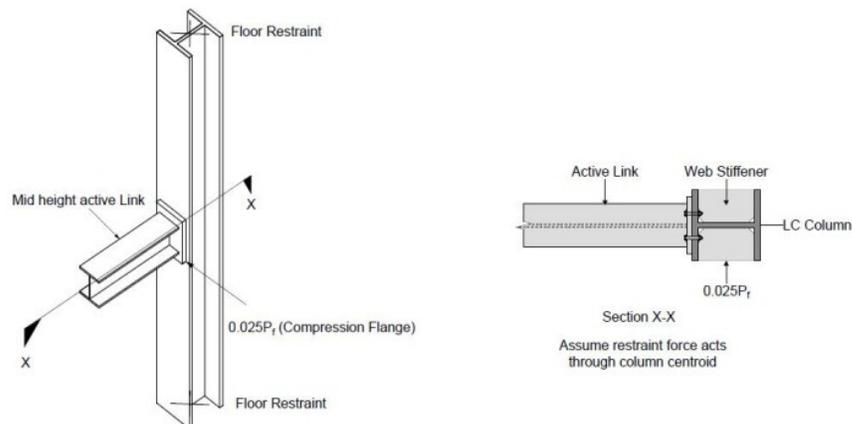

Figure 6: Link restraint force on LC column

5.3.2.3 Axial load limitation

Columns of seismic loading resisting systems are most vulnerable to plastic hinging at their bases if the base connections possess a high degree of fixity and the inter-storey drifts are high. To ensure the low damage performance objectives of the LCF system are realised, column bases should either be pinned or semi-rigid with flexural yielding of the column base suppressed by base rotation that occurs at a lower moment than the column section moment capacity. Provided the base connections are appropriately detailed to prevent column base yielding, the column axial load limitation requirements of NZS 3404 section 12.8.3 are reduced. (Compliance with clause 12.8.3.1(c) is no longer required) [2] while the length for checking compliance with clause 12.8.3.1 (b) is the length between centrelines of adjacent active links.

5.3.2.4 Splices

Splices in conventional seismic load resisting systems must have splices located in the middle third of the story height (NZS 3404 [2]). This is to locate the splice in regions of lower moment demand. In the case of Linked Columns the distribution of column bending moment is very different with two points of contraflexure in the elastic column moment profile (figure 7). In this instance it recommended the splice is located at the column quarter point.
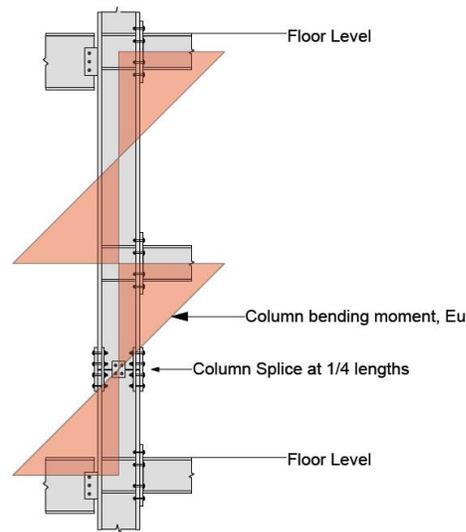
Figure 7: LC column splice location

5.4 Elastic moment frame

5.4.1 Sizing elements

5.4.1.1 Beam

The beams should be designed for the factored gravity loads on the frame (1.2G&1.5Q). The critical section for beam design will be the column face where the moment must be resisted by the bare steel section ie non composite behaviour in the negative moment region. Redistribution of the column face moment is possible, (NZS 3404 [2] section 4.5.4), provided the beam elements meets the lateral restraint, material and section requirements for the appropriate member category. For example if the beam meets the category 3 member requirements, a redistribution of up to 20% is allowable (table 4.5.4.2).

To cover shake down [4], the beams should be able to compositely sustain simply supported the design actions resulting from the load case G&Q.

5.4.1.2 Columns

The column should be sized for the design actions associated with the beam elements developing their overstrength flexural capacities (ϕ_{oms} from table 12.2.8 (1) for a category 3 member [2]), see figure 8. The capacity design derived beam actions are:
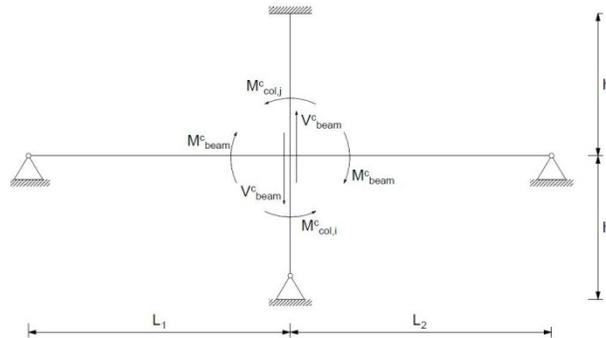
$$M_{beam}^c = \phi_{oms} M_s \qquad (5)$$

$$V_{beam}^c = \frac{M_{beam}^c}{L}$$ (6)

Where

ϕ_{oms} is the beam over strength factor

The out of balance beam actions are apportioned to the column on the basis of the flexural stiffness of the column above and below the level under consideration (figure 8).

**Figure 8:** *MRF column capacity design derived actions*

$$M_{col,i} = \frac{I_i/L_i}{\frac{I_i}{L_i} + \frac{I_j}{L_j}}$$ (7)

Where:

I_i, I_j are the level i and j level column second moments   of inertia
L_i, L_j are the level i and j column lengths

5.4.1.3 Simple Beam Connections

Standard simple connections such as web plate or flexible end plates (figure 9) are to be used for nominally pinned connections between MRF beams and LC columns. It is important these are simple connections to increase the flexibility of the frame. However, depending on how diaphragm shears are introduced into the seismic-resisting system, the MRF will function as a collector system for diaphragm shear forces in which case a FE endplate is better as it can transfer higher compression load from the MRF beam into the LC beam

a) Web Side Plate Connection (WP)   b) Flexible End Plate Connection (FE)
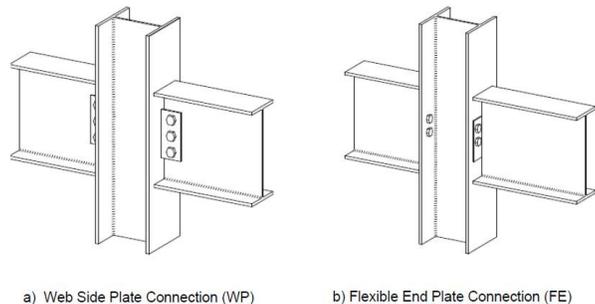
Figure 9: *Simple beam connections [8]*

5.5   Ultimate limit state frame displacement (ULS)

To calculate the ULS frame displacements, a value of μ_d must be calculated using the dual frame provisions of NZS 3404 [2] section 12.13. As an initial approximation designers may assume $\mu_d = 0.9\mu$ i.e. 2.7. This equates to a moment frame to LCF stiffness ratio of approximately 15% i.e.

$$\mu_d = \frac{\mu_{LC} K_{LC} + \mu_{MRF} K_{MRF}}{K_{LC} + K_{MRF}} \qquad\qquad (8)$$

Where μ_{LC} and μ_{MRF} are the structural displacement ductility factors for the linked column frame ($\mu = 3$) and the moment resisting frame ($\mu = 1.25$) respectively.

Once μ_d has been calculated, the Loading Standard [3] ULS displacement provisions can be utilised.

5.6 Pushover analyses
To ensure the low damage criteria for the LCF system is achieved, an incremental non-linear push over analysis is required to ensure no yielding occurs in the elastic moment frame under the ULS frame displacements. Such non-linear analysis may be undertaken using software such as SAP 2000 or ETABS. It is beyond the scope of this paper to provide a step by step push over analysis tutorial, reference should be made to the software user manual. Guidance on modelling of LCF frames for such analyses is given in section 9.0

6.0 Tips for achieving cost effective solutions

1. As an initial estimate of the period (T_1) of the structure, use the empirical equation in NZS 1170.5 C4.1.2.2 for a steel moment resisting frame.

2. Try a LCF column centreline to centreline dimension of 1.2 – 1.5m,

3. Limit the link length to $1.6 M_p/V_p$ to ensure shear yielding

4. Limit link length to ensure the link moment end plate connection capacity design derived action does not exceed 50-60% of the link section moment capacity ($\Phi \cdot _s$). Consider specifying grade 10.9 bolts for heavily loaded MEP connections. This is to avoid the need to use 16 bolt MEP arrangements. Note there is a 3-4 month lead time for grade 10.9 bolts and that they are supplied black steel so will need coating, at least for appearance when visible.

5. Utilise custom welded LCF links and columns to tune the section strength and stiffness requirements to the demand.

6. Size bands of links for an average link shear force, then calculate the system over strength factor to ensure the columns and foundations are not being subject to excess design actions due to a lack of tailoring of link shear strength to demand.

$$\phi_{av}^o = \Sigma \frac{\phi_{oms} V_w}{\Sigma V_{Eu}} \qquad\qquad (9)$$

The range of possible values of over strength (ϕ_{av}^o) will lie between $1.3/0.9 = 1.44$ for a perfectly tailored solution ($\phi_{oms}=1.3$) and that associated with a nominally elastic response ($\mu=1.25$).

For the upper bound value;

$$\phi_{max}^o = \frac{E_{\mu=1.25}}{E_{\mu=3}} = \frac{3}{1.25} = 2.4 \qquad\qquad (10)$$

Once the ϕ_{av}^o value exceeds 1.8-1.9, consideration should be given to having more link sizes, using custom welded links or both.

7. Where rigid beam column connections are required for elastic moment resisting frames, use moment end plate connections. SCNZ Report SCNZ:14, *Steel Connect* [8] has many pre-engineered MEP connections for a range of hot rolled sections. For applications not covered by these pre-engineered solutions, specific design can be undertaken using the design methodology in this document.

7.0 Detailing requirements

7.1 Design of columns for moment connections

The column connection zone of LC and MRF systems must be capable of dependably transmitting the capacity design derived link or beam actions into the column. For column moment connections involving moment endplate connections, the following limit states must be checked (figure 10).
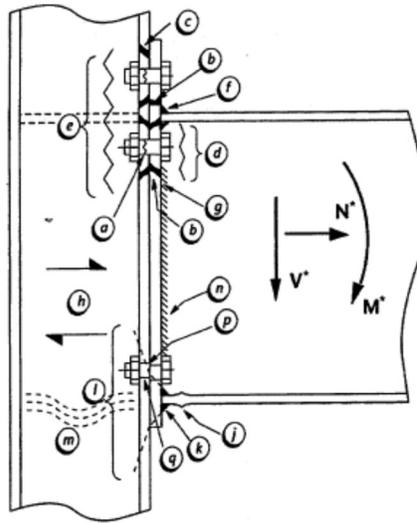
Figure 10: *Column limit state checks for moment-resisting connections [6]*

Where:

Zone	Ref	Checklist Item
Tension	a	Bolt tension
	b	End plate bending
	c	Column flange bending
	d	Beam web tension
	e	Column web tension
	f	Flange to end plate weld
Horizontal Shear	h	Column web panel shear
Compression	j	Beam flange compression
	k	Beam flange weld
	l	Column web crushing
	m	Column web buckling
Vertical Shear	n	Web to end plate weld
	p	Bolt shear
	q	Bolt bearing (plate or flange)

The detailed design checks to satisfy these limit states is covered in HERA report R4-142:2009 *Eccentric cleats in compression and columns in moment resisting connections*. [7]

7.2 Column Bases

To ensure the low damage performance objectives of the system are met, it is important to ensure column bases are either pinned or semi-rigid with the ability to rotate to limit column base moments.

Pure pinned connections are difficult to achieve in practice, they are also undesirable in that the frame stiffness in the lower storey is much lower and therefore makes complying with the inter-storey drift requirements more difficult. To this end a ground level link is used to provide base fixity if a pin based column detail is adopted. An example of such a detail is shown in figure 11. The main disadvantage of a pin connection is the lack of erection tolerance.
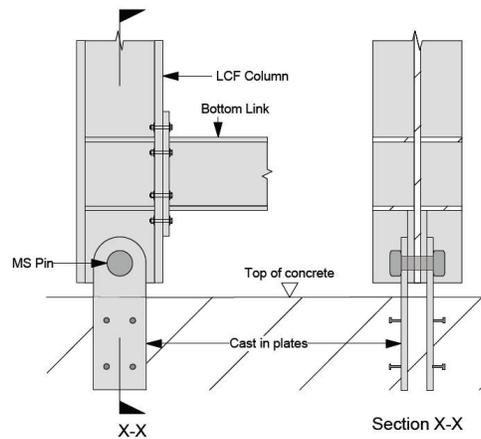
Figure 11: Pinned LC column connection

Even nominally pinned base details, such as that shown in figure 12 will provide base fixity that should be modelled. An equation is presented in section 9 for determining the rotational spring stiffness value appropriate for this type of connection.
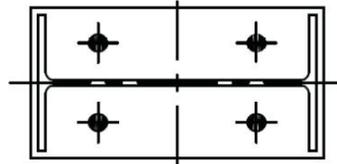
Figure 12: A typical nominally pinned base connection [8]

Semi-rigid base connections have been the subject of research in North America. There is a current research programme in Taiwan testing pinned and semi rigid base connections. New Zealand research into semi rigid base connections is also planned. A nominally pinned base plate shown in figure 12, with extended hold down bolts designed and detailed in accordance with HERA DCB No 56 [10], is expected to develop an elastic rotation of at least 10 milliradians under ULS design seismic load.

8.0 LCF Comparison with conventional Moment Frames

8.1 Introduction

To compare the seismic performance and cost of an LCF system and a conventional ductile moment resisting frame, a comparative study has been undertaken for a model four storey building located in a high seismic zone. This 70x18m foot print building features (figure 13) composite metal deck slabs supported on steel framing. The lateral load resisting system consists of eccentrically braced frames in the cross direction and moment resisting frames or alternatively LCF systems in the longitudinal direction. The seismic system performance data for the lateral load resisting systems is shown in table 1.

| Option	μ	T_i Period (sec)	Max interstorey drift %
LCF	3	1.4	2.1
Conventional ductile MRF	3	1.5	2.2

Table 1: Seismic system data LCF and MRF

For the purposes of this study, type C soil has been chosen and the building is supported on 12m long screw piles with tension and compression capacity as required. The screw pile design and costing were undertaken by Piletech in a previous material comparison study.

8.2 Seismic performance

Both structures were modelled in ETABS using the non-linear pushover capability to confirm the LCF system was behaving as intended and also to monitor hinge location formation. Parameters which were monitored during the comparison included base shear versus drift percentage and hinge formation location (shear link or gravity beam).

The results of the comparison are shown in the roof drift versus base shear plot below:
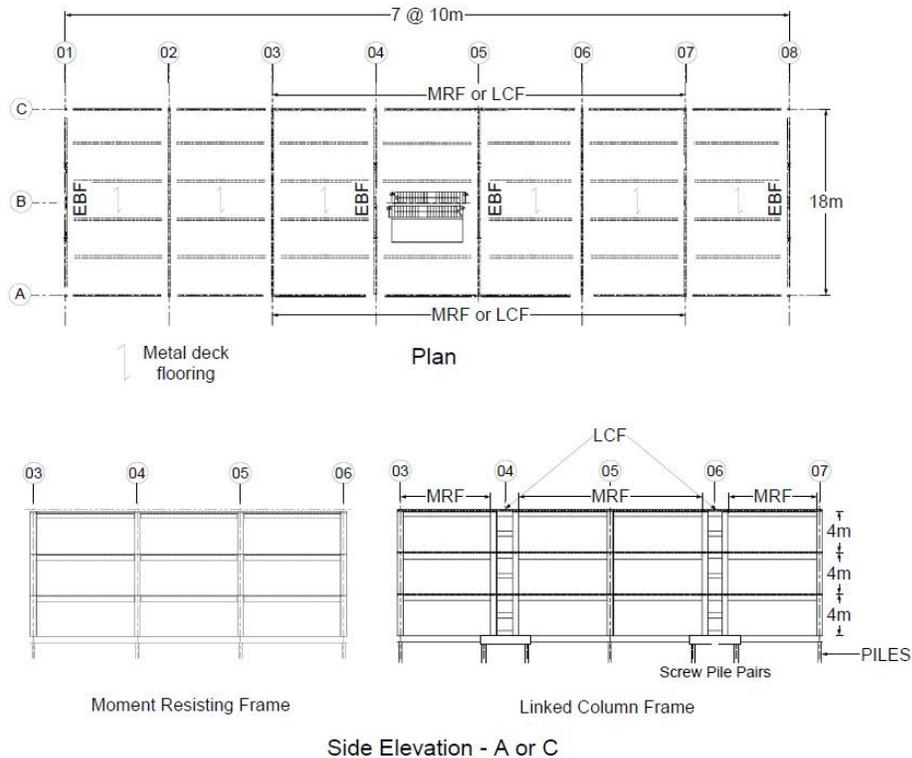
Figure 13: *Four storey model building for comparative study*
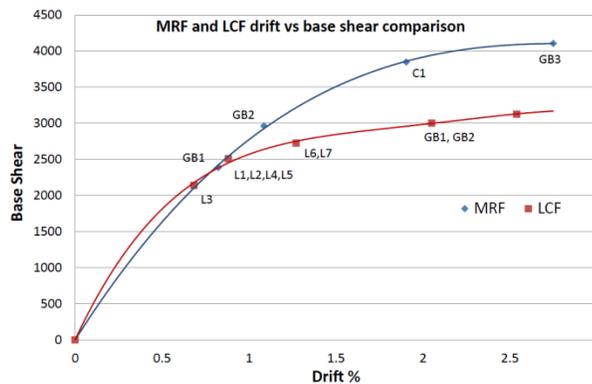
Figure 14: Push over comparison LCF vs ductile MRF

The blue line represents the MRF behavior and the red line represents the LCF behavior. For the MRF, the data points refer to moment hinges forming in either a column at ground level (C1 to C4) or a moment hinge forming in a moment frame gravity beam at either levels 1, 2 or 3 (GB1 to GB3). For the LCF, the data points refer to either a shear hinge forming in the shear links (L1 to L7) up the height of the building, or a moment frame gravity beam at either levels 1, 2 or 3 (GB1 to GB3).

The damage threshold for both framing systems is similar, the first hinge forms in the conventional MRF at about 0.8% drift (base shear 2400 kN) while for the LCF system the first link yields at about 0.7% drift (base shear 2140 kN). The key difference is that the moment resisting frame at this point has sustained potentially costly and difficult to repair damage (level 1 beams) ie no rapid return to occupancy performance level. Conversely the LCF system will continue to sustain readily repairable damage (link removal) until approximately 2.1% inter-storey drift at which point the first moment frame element yielding occurs (GB 1 and GB2).

Based on this particular design, the LCF system has been shown to reach over 2% drift (roof level relative to ground) before permanent damage in a moment frame gravity beam is formed. Up until this point, the structure can be returned to occupancy quite rapidly. Since the moment frame is designed to remain elastic, once the yielded bolted shear links are removed, the building should self-centre. The slab will remain elastic around the LCF region, assisting in the self centering of the building and the baseplate details will also assist in self-centering

8.3 Foundation Requirements
The conventional MRF foundations consisted of a ground beam supported on screw piles located under each column. The LCF system required more substantial   foundations due to large axial forces generated from link overstrength actions. A group of 4 screw piles was used to support each linked column (2 for each column). To reduce the pile loads, a ground beam has been sized to allow the pile group centres to be wider than the linked column centres (figure 13).

8.4 LCF Cost Comparison to a Conventional MRF
Quantities and costs have been calculated for the foundations and super structure for both the conventional MRF and LCF system.

In terms of the superstructure, the LCF system was approximately $95,300 more expensive than the original option MRF system. The total superstructure costs for the base building was $1,256,100.

A typical MRF foundation (screw pile and ground beam) was priced at approximately $6,225 for each column. In comparison, each LCF screw pile group and ground beam cost $24,270 at each linked column location which is approximately $18,000 per column more expensive than the typical MRF foundation. However, there were only 4 LC column locations within the building footprint.

In total the LCF superstructure and foundations cost an additional $167,300 or 7-8% compared to a ductile moment frame alternative. The total construction cost for the 4 storey office building, based on a construction cost of $2200/m^2 [11], is in the order of $11.5 million dollars. The additional cost is therefore approximately 1.5% more expensive than a conventional building. This is in keeping with the oft quoted figure that the premium for low damage solutions is in the order of 1-2% compared to conventional ductile solutions.

The difference in price between a conventional MRF and the LCF system is fully justifiable when one considers the benefits of the LCF, which easily outweighs the additional cost. Also, the ability to quickly repair the LCF and return the building to an operational state could return rental figures larger than the initial capital cost to implement the LCF system.

9.0 Modelling
Two models are required to design LCF structural systems. An elastic analysis (equivalent static or modal response spectrum method) is used to compute member design actions and frame displacements and secondly a non–linear push over analysis is required to confirm yielding is limited to the active links.

<u>Elastic model</u>

Real connections will never be infinitely rigid (fully fixed) or possess no rotational stiffness (pinned). This is recognised in NZS 3404 Clause 4.8.3.4.1 with a maximum stiffness required for a fixed base (subclause (b)) and a minimum stiffness required for a pinned base (subclause (a)). Column base stiffness is best represented by rotational springs. The formulae for these rotational springs are as follows.

$$k_o = 1.67 \; \frac{EI_c}{L_c} \; (Fixed) \hspace{3cm} (11)$$

$$k_o = 0.1 \frac{EI_c}{L_c} \ (Pin) \hspace{3cm} (12)$$

Semi-rigid connections have stiffness's intermediate these two extreme cases. Foundation flexibility will also influence the building seismic performance.

If the moment resisting frame beams possess sufficient numbers of studs to develop composite action (a minimum of 25% composite action is required for deflections), then the appendix N provisions of NZS 3404 [2] can be used to account for composite behaviour.

For the typical case where the studs are stopped a distance 1.5 times the beam depth from the column face, the beam second moment of inertia may be taken as:

I_{comp}=1.2 $I_{bare \ steel}$ \hspace{3cm} (13)

Beam-column joint stiffness can have an impact on frame stiffness. With the NZS 3404 design requirements for relatively high strength panel zones, rigid offsets may be used to represent the beam column connection in conjunction with a rigid offset reduction factor of 0.5. Otherwise panel zone flexibility must be explicity accounted for. The user manual for proprietary analysis software can be consulted for guidance on modelling this effect.

Non-linear push over analysis

The principle modelling difference for a nonlinear analysis is the introduction of shear and moment hinges to potential yielding zones, see figure 15. Proprietary analysis software such as ETABS or SAP 2000 have various default hinge models that can be specified.
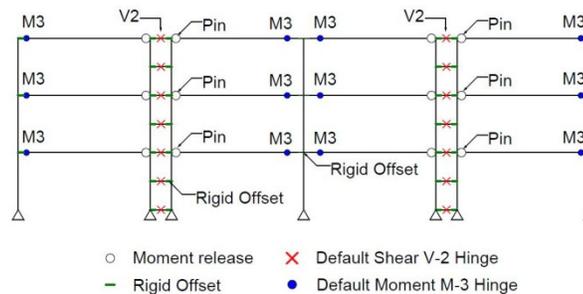
Figure 15: Push over analysis model

If either of these analysis packages is used, type M3 moment hinges can be assigned to column bases and at rigidly connected ends of MRF beam elements. Type V2 shear hinges are appropriate for the shear links. These hinges are located at the ends of rigid offsets to represent the beam-column connection (figure 15).

Care must be taken when assigning material properties. In the seismic assessment of buildings probable or expected strength is used rather than lower characteristic strength which is used for design purposes. The probable strength is more representative of the likely material strength and is therefore recommended for use in a push over analyses. This would equate to specifying a yield stress equal to $\frac{(\phi_{om}+1)}{2} f_y$, where ϕ_{om} is the material variability factor taken from either table 12.2.8(1) or (2), NZ3404 [2] .

10.0 Conclusion
The LCF system is a brace free hybrid of proven structural steel seismic load resting technology (EBF and MRF), that for only a slight premium on the total build cost (approximately 1%), delivers enhanced seismic performance compared to conventional ductile moment resisting frames. It features easy to replace energy dissipating link elements and an elastic moment resisting frame to improve the self centering performance of the structure.

It is a system that will be suitable for low to medium rise construction (3-6 stories), the actual number of stories will be dependent on the size of the building and regional seismicity.

This paper has presented a design methodology for the LCF system for New Zealand application. Engineer's familiar will the design of eccentrically braced frames and moment resisting frames will find the methodology presented in this paper easy to implement.

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