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ALUMINIUM SHEAR-LINK FOR SEISMIC ENERGY DISSIPATION

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SUMMARY

An aluminium shear-link is developed for earthquake resistant structures. It is just an I-shaped beam of low yielding aluminium alloy to be strategically placed in various structural systems to dissipate seismic energy. The aluminium beam is designed to yield in shear to limit the maximum force due to lateral loads transmitted to primary structural members. Shear yielding of aluminium is very ductile and large inelastic deformations (about 10% strain) are possible without tearing or buckling. Moreover, shear yielding of the web maximises the amount of material participating in the plastic deformations without a high concentration of plastic strain. Also, replacement of shear-links is easy after an extreme seismic event. Numerical studies clearly show the effectiveness of aluminium shear-links for two structural systems: Concentric Braced Frames (CBFs) and Truss Moment Frames (TMFs). Shear-link systems demonstrated more uniform distribution of story drifts, reduced base shear, and a larger energy dissipation capacity per unit drift

INTRODUCTION

I-shaped beams of low yielding ductile alloys of aluminium designed to yield in shear mode when suitably placed can limit the maximum lateral force transmitted to primary structural members. They function as a metallic yielding device (“fuse”) and can dissipate significant energy. These links can also be viewed as damping devices which dissipate earthquake induced energy through inelastic deformations (metallic hysteresis), thus minimising (or eliminating) the energy dissipation demand on the primary structural members. These properties make aluminium shear-links attractive for both new buildings and upgrades to existing structures.

The shear yielding of low alloy metals such as aluminium has been observed to be very ductile and large inelastic deformations (about 10% strain) are possible without tearing or buckling of the member. Lower yield strengths allow the use of thicker webs in I-shaped links that reduces the problem of plastic web buckling. This web shear yielding maximises the amount of material participating in plastic deformation and gives much more uniform strain than in flexural yielding elements, resulting in a large amount of energy dissipation before the material fractures. Also, the significant strain hardening present in Aluminium alloys lead to resist more lateral loads after the first yield in a shear-link, causing additional deformations to be absorbed by links in other storeys of a multi-storeyed structure. Thus it reduces concentration of inelastic deformation in a particular storey leading to an undesirable “soft-storey” failure mechanism [Rai & Wallace 1998].

The objective of the paper is to describe the inelastic cyclic behaviour of the shear-link as a seismic energy dissipator in CBFs and TMFs. Design and hysteretic properties of the shear-links are derived from shear testing of medium scale (1:4) models of links. A design methodology for SLBF and SLTMF systems is developed. Static “pushover” and simulated earthquake type loadings are then conducted utilising the SNAP-2DX computer program [Rai et al. 1996] to compare their behaviour to that of OCBF and TMF systems, respectively.

HYSTERETIC BEHAVIOUR OF ALUMINIUM SHEAR-LINK

Figure 1 shows a typical shear-link which was subjected to a cyclic shear loading of strains up to 0.2 and its shear stress—shear strain hysteretic behaviour. The 3003-0 aluminium alloy of the link sustained large plastic deformations without tearing and was found superior to alloy 6061-0. First yield was typically observed at 0.002

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strain and at a stress of 0.722 times the 0.2% offset yield stress $\sigma_{0.2}$ of the material. The link strain-hardened during subsequent cyclic loading and achieved an average stress of 1.866 $\sigma_{0.2}$ in 0.2 strain cycles. Stable hysteretic loops were observed up to 0.1 strain and degradation in strength following the Bauschinger effect was observed in 0.2 strain cycles. Severe panel buckling was observed at this stage and specimens appeared distressed with deformed stiffeners, but retained good load carrying capacity. Transverse stiffeners help form a cyclical diagonal tension field by developing a Pratt truss action, thereby achieving stable hysteresis behaviour.

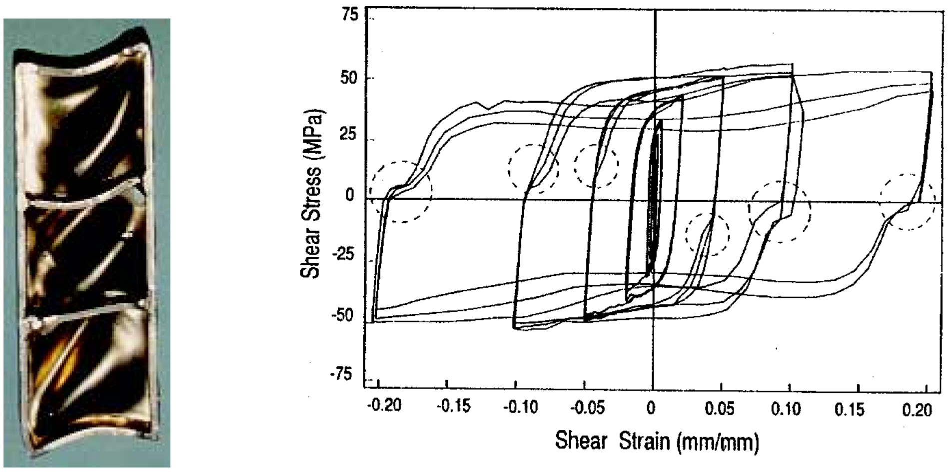

Figure 1. Yielded shear-link and its measured shear stress—shear strain hysteretic response

PROPORTIONING OF SHEAR-LINKS

Shear-links are designed on the basis of two limit states corresponding to strength and ductility demands of the design level and maximum credible earthquakes. The horizontal area of the web of the shear-link is calculated by dividing the shear by a "design" shear stress of the link corresponding to a limiting value of shear strain. This shear stress for a given shear strain γ in a shear-link is given by a power equation: $\tau_{avg,max} = 2.6 \cdot \sigma_{0.2} \cdot \gamma^{0.2}$, where $\sigma_{0.2}$ is the tensile yield stress of the material (3003) corresponding to 0.2 strain. The maximum allowable shear strength of the shear-link is assumed to correspond to 0.2 shear strain, and a shear higher than this is assumed to represent the failure of shear links. Experimental data suggest that an upper bound estimate of the maximum shear capacity of the link, i.e., $\tau_{max} = 1.88 \cdot \sigma_{0.2}$. Shear force in a link is shear stress τ times A_w, the horizontal web area of the link (*l* times t_w). Similarly, shear deformation Δ of a shear-link is related shear strain γ as $\Delta = \gamma \cdot d$, where *d* is the depth of the I-shaped shear-link. Design shear strain for a shear-link can be taken to correspond to the allowable storey drift, but should not exceed 0.1 strain because links showed excellent load carrying capacity and hysteretic behaviour below this strain level.

Stiffeners on both sides of the web are required to prevent early plastic buckling of the web and ensure a ductile shear failure of the web. Transverse stiffeners must be provided at each end of the link. Intermediate stiffeners should be provided at regular intervals so that the maximum web deformation angle during cyclic excursion $\gamma_b = 9.37 \cdot k_s / \beta^2$ where β is web depth-to-thickness ratio and k_s is a buckling coefficient as defined below:

$$k_s = \begin{cases} 5.6 + 8.98/\alpha^2 & for(\alpha \le 1) \\ 8.98 + 5.6/\alpha^2 & for(\alpha \ge 1) \end{cases} \tag{2}$$

where α is aspect ratio, which is defined as the ratio of stiffener spacing to the clear depth of the link beam. For design purposes γ_b can be taken as twice of γ_d the web deformation angle for the design load. The expression for γ_b is obtained from an analysis of web buckling data of aluminium shear-links. To allow shear-links to maintain their post-buckling capacities, each transverse stiffener is proportioned to avoid local buckling and remain effective after the web buckles to support the tension field as well as to prevent the tendency of flanges to move towards each other. Therefore, stiffeners must meet stiffness and stability checks [Rai & Wallace 1998]. Intermediate stiffeners can be avoided if the web depth-to-thickness ratio β of link is less than 20.

A SIMPLIFIED MODEL OF ALUMINIUM SHEAR-LINK

A simple non-degrading hysteretic model is assumed to represent the shear force-displacement behaviour of shear-links. The model operates on a bilinear skeleton relation between shear force and link displacement which can be exclusively defined by the following three parameters: yield shear force, the first stiffness and the second stiffness. The first stiffness is taken as the secant stiffness corresponding to a shear strain of 0.002 at which the general yielding of the shear-link specimens was observed. At this stage the average shear stress was 0.722 $\sigma_{0.2}$ and can be taken as the "yield" shear stress. This value is multiplied by the horizontal web area of the shear-link to give yield shear force. The second stiffness is computed so that the maximum shear force allowed occurs at a shear strain of 0.2. The second stiffness is very small in comparison to the first stiffness but is large enough in absolute terms to ensure satisfactory frame response, as will be demonstrated in following sections. Element 7 of the SNAP-2DX represents this simple hysteretic behaviour of shear-links.

SHEAR-LINK BRACED FRAME (SLBF) SYSTEM

One application of the aluminium shear-link is to improve chevron type Ordinary Concentric Braced Frames (OCBFs). The shear-link is sandwiched between the tops of the diagonal braces and a beam from the floor above, as shown in Figure 2. The shear-link is designed to yield at a lateral force less than that required to buckle the compression brace, eliminating the severe loss of story strength and stiffness due to compression brace buckling. Another advantage of this Shear-Link Braced Frame (SLBF) system is that the floor beam continues to carry gravity loads even after link collapse (Figure 2). The horizontal area of the web of each shear-link was obtained by dividing the story shear by the value of shear stress corresponding to a limiting value of shear strain. This shear strain can be taken to correspond to the allowable story drift but can not exceed 0.1 strain. The depth of the shear-link is typically 1/10 – 1/12 of story height.

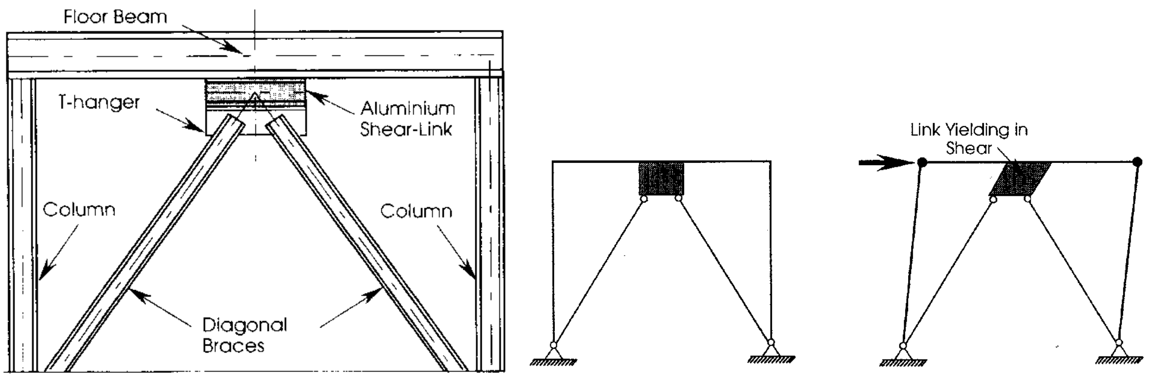

Figure 2. Schematic diagram and typical collapse mechanism of the SLBF system

Design of SLBF System

The braced frame system is designed for a four story office building for the UBC Seismic Zone 4 on the soil profile S2. The typical framing plan and elevation of a braced bay of the building is shown in Figure 3. Two braced frame systems are designed to resist the Code required base shear equal to 15.64% of its seismic weight. The design story shears for the SLBF were kept the same as the ones used for the OCBF to facilitate a direct comparison. Calculations related to the proportioning of shear-links are shown in Table 1. The design shear strain of 0.06 corresponds to allowable inter-storey drift of 0.5%. As the shear-link web thickness ratio (11.33) is less than 20, intermediate stiffeners are not required; only end transverse stiffeners of thickness 25 mm and combined width of 270 mm are provided. Once the shear-links were proportioned, the braces were designed for loads equal to the maximum strength of the shear-links. However, in this study, the braces and other members of the SLBF were kept the same as in the OCBF system to facilitate a direct comparison.

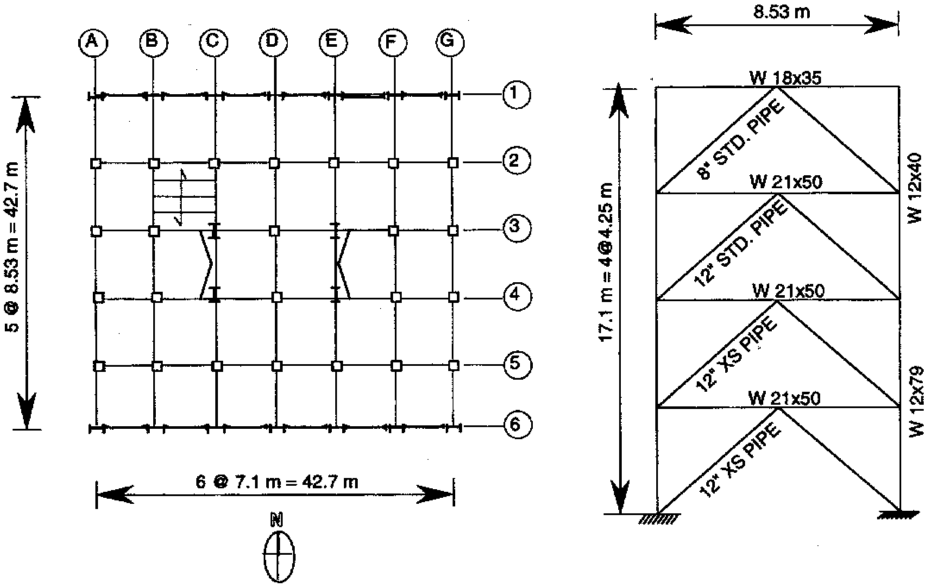

Figure 3. Plan of the study building and OCBF members designed for the UBC seismic forces

Table 1. Proportioning of shear-links for SLBF system

Storey	Design storey shear (kN)	Design shear strain	Design shear stress (MPa)	Equivalent AISC W-section	Length, depth and web thickness			Shear strength provided (kN)
					l (mm)	d (mm)	t_w (mm)	
First & second	1833	0.06	52.1	W 12x170	1525	355.6	24.4	2460
Third & fourth	1023	0.06		W 12x170	815	355.6	24.4	1310

Seismic Performance of SLBF system

Seismic performance of the SLBF system was studied numerically using the SNAP-2DX non-linear analysis program and compared to a conventional system. SNAP-2DX uses member-to-member modelling in which one-to-one correspondence exists between the model elements and structural members. Hysteretic force-deformation properties are input for each element. The frame members were grouped into beam-columns (element 2), braces (element 9) and shear-links (element 7). Chords of the truss girder and columns were modelled as element 2; web members were modelled as element 9, whereas shear-links were represented by element 7.

Four different ground motions were used in the time-history response analyses. Recorded ground motions of Miyagi-Ken-Oki (1978), El Centro (1940), and Northridge (1994) earthquakes were scaled to a peak ground acceleration (PGA) of 0.26*g*, 0.38*g*, and 0.33*g*, respectively, to match the Code assumed “intensity” of a design level earthquake for this structure. El Centro and Northridge earthquake records have small low frequency contents, whereas the synthetic accelerogram has a long duration with a rich frequency content and extremely high energy content. Miyagi has a narrow band of frequency around 1 Hz with an intermediate energy content

The OCBF developed a soft story mechanism, whereas the SLBF behaved as expected with only shear-links undergoing from the Miyagi excitation as shown in Figure 4. When one of the braces buckled and its load capacity dropped in the OCBF, the resulting unbalanced vertical component of brace forces overstressed the floor beam, causing plastic hinges and seriously weakening the frame. The OCBF experienced a drifting type of response primarily due to a mechanism in its first story, which was avoided in the SLBF by the high stiffness and strain-hardening of the shear-links. Moreover, the low yielding SLBF system attracted less base shear than the OCBF, though the reduced stiffness of a “partially” yielded system resulted in rather large floor displacements.

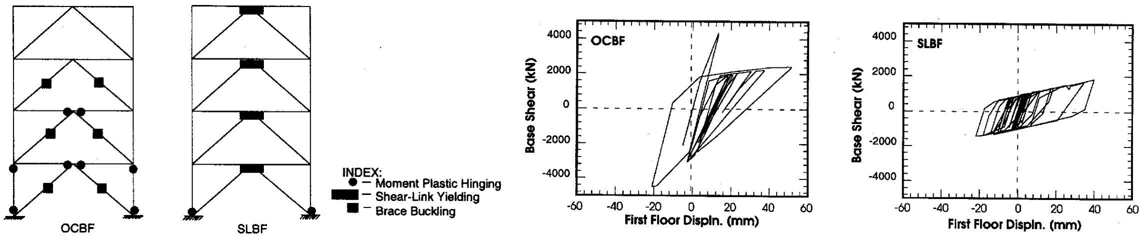

Figure 4. Inelastic activities for Miyagi and hysteretic response for El Centro motion

Inter-story drift and story shear response envelopes for both systems are compared in Figure 5 for all ground motions. Storey drift responses clearly show nearly uniform distributions of deformation for the bottom stories of the SLBF building, as opposed to the concentrated damage of OCBF building at the first story. However, only for the synthetic accelerogram case, shear-links at the third stories were subjected to shear strain of 0.24 mm/mm, in excess of the maximum suggested value of 0.2. The maximum base shear resisted by the OCBF is in the range of 2.8 to 3.0 times the design base shear, while it is only 1.2 to 1.7 for the SLBF.

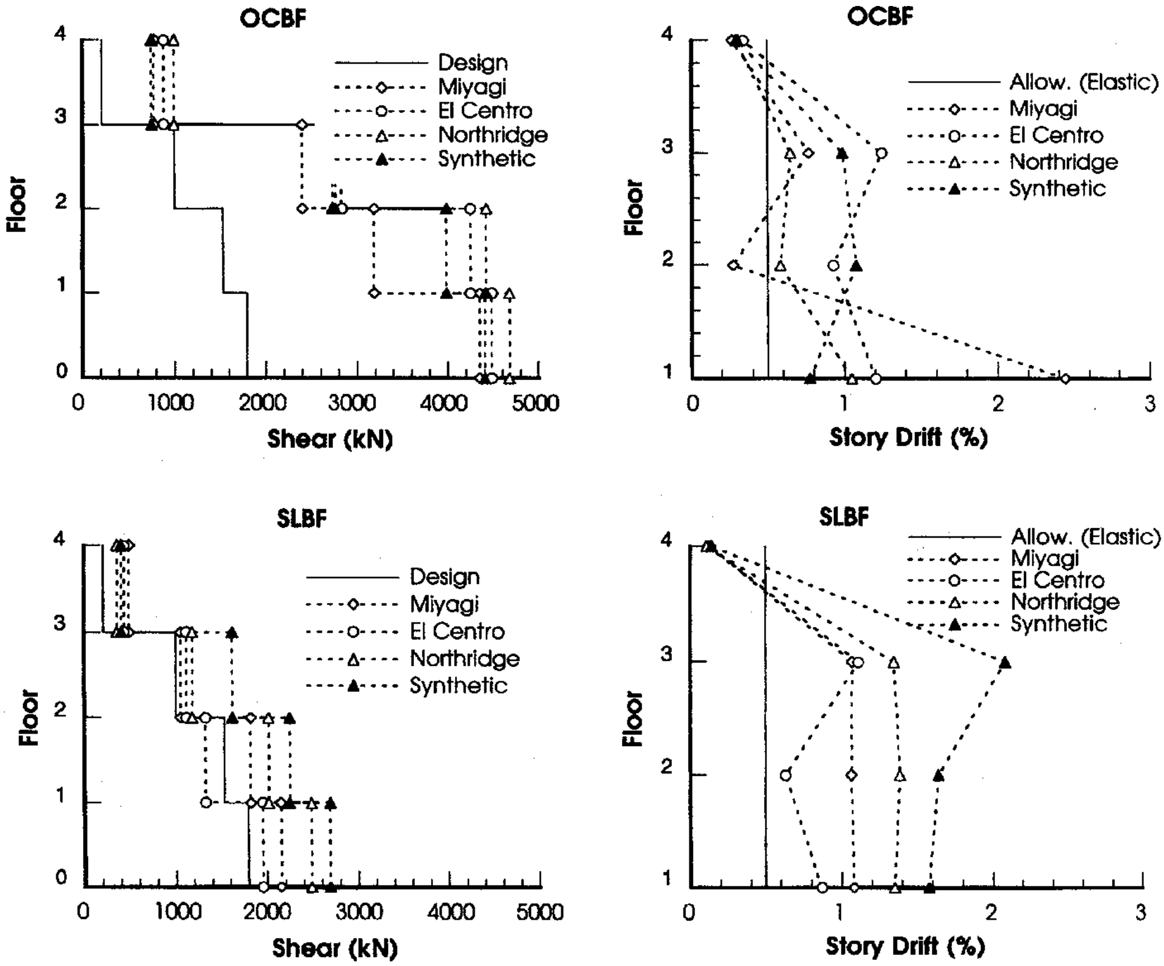

Figure 5. Response envelopes of braced frame systems: storey drifts and storey shears

SHEAR-LINK TRUSS MOMENT FRAME (SLTMF) SYSTEM

In another application, Aluminium shear-links are found to significantly enhance the seismic energy dissipation of Truss Moment Frames (TMFs). Conventionally, TMFs are designed as weak column—strong girder and can be detailed as Moment Resistant Frames (MRFs). Such a design philosophy did not yield satisfactory results in

Mexico City earthquake (1985). A different strong column-weak girder yield mechanism was chosen for the design of Shear-Link TMF (SLTMF). As shown in Figure 6, the link is placed between the horizontal vertices of diagonals of adjacent panels at the centre of the truss girder, where the resultant vertical shear due to lateral loads and gravity is maximum. This system prevents the buckling and yielding of diagonals and all the inelastic deformations are confined to shear-link only [Rai & Prasad 1998].

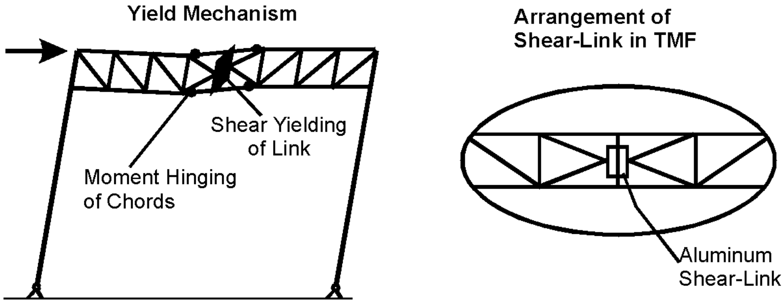

Figure 6. Schematic diagram and typical collapse mechanism of the SLTMF system

Design of SLTMF System

The design concept is based on the limit state approach in which a yield mechanism involving most of the inelastic deformations in the shear-link is assumed for the TMF at ultimate loads. For this mechanism, the frame is solved for member forces by methods of plastic analysis. The factored seismic loads are resisted by vertical shear in the shear-link and chords of the special segment, i.e., the panel which contains the shear-link. First, section sizes for shear link and chord members of the special segment are selected. All members outside the special segment including columns are designed to remain elastic under forces generated by strain hardened shear-links and plastic hinges of chords of special segment. This maximum amplified vertical shear V_{ss} developed in the special segment is given by the following relation:

$$V_{ss} = 3.4\left(M_s / L_s\right) + 0.11 EI \left(L - L_s\right) / L_s^3 + R_{max} \tag{3}$$

where M_s and EI is flexural strength and stiffness of chord members, respectively; L is span length and L_s is 0.9 times of the length of the special segment; and R_{max} is the maximum shear strength of strain-hardened shear-link which can be taken to correspond to a maximum strain of 0.2.

The design philosophy essentially is the same as for the Special Truss Moment Frames (STMFs) [Goel et al. 1998]. The special segment is kept within the middle one half length of the truss girder and its length is kept between 0.1 to 0.5 times its span. Also, the length to depth ratio of any panel of TMF should fall between 0.67 and 1.5. In the fully yielded state, the special segment develops its vertical shear strength through shear yielding of the link and moment hinging of the chord members. The shear contribution of chords can be taken as 25% of the required vertical shear and the remaining 75% is resisted by the shear-link.

The example building for SLTMF is one storey industrial building with plan dimensions of 36 m by 90 m as shown in Figure 7. Six frames on column lines 2, 3, 5, 6, 8 and 9 are lateral force resisting TMFs in the N-S direction. The other frames are designed to resist only gravity loads. The design base shear for the N-S TMFs is 9.3% of the seismic weight. The calculations related to sizing of the shear-link are shown in Table 2. Since web thickness ratio β (40) is more than 20, intermediate stiffeners are required besides the end transverse stiffeners.

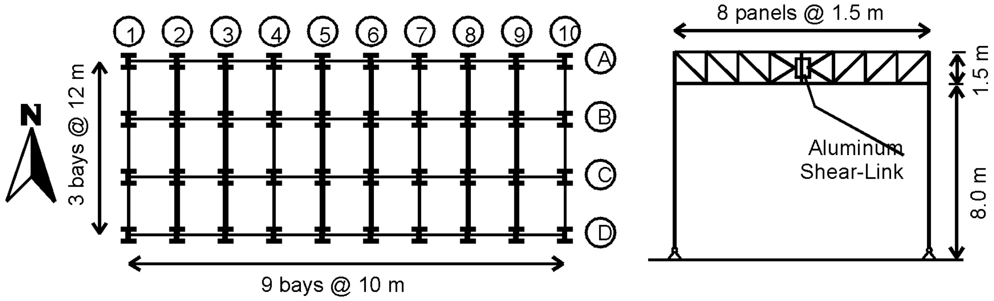

Figure 7. Plan of the study building and elevation of a typical bay in the N-S direction

Seismic Performance of SLTMF System

Static pushover analyses were carried out to determine ultimate lateral capacities and collapse mechanisms of structural systems and results are shown in Figure 8. In the SLTMF yielding of the shear-link was observed at a base shear of 31.0 kN (0.21% storey drift) which is 2.9 times the design base shear. A 45% reduction in the initial lateral stiffness (1.55 kN/mm) was noted after the yielding of shear-link as the second stiffness of the shear-link was low compared to its first stiffness. However, the system reached the maximum allowable strength of shear-link at a storey drift of 1.08% with inelastic activities contained in the shear-link only. In comparison, the first inelastic activity in the STMF was noted at 46.2 kN (0.37% storey drift) when the buckling of X-diagonal was observed. However, no appreciable reduction in stiffness (1.32 kN/mm) of the frame was observed and it continued to carry load until X-diagonals started yielding in tension at a peak base shear of 116.5 kN (0.97% store drift). After buckling and yielding of X-diagonals, the system developed a mechanism at 1.81% storey drift following moment hinging at the ends of the chords of the special segment.

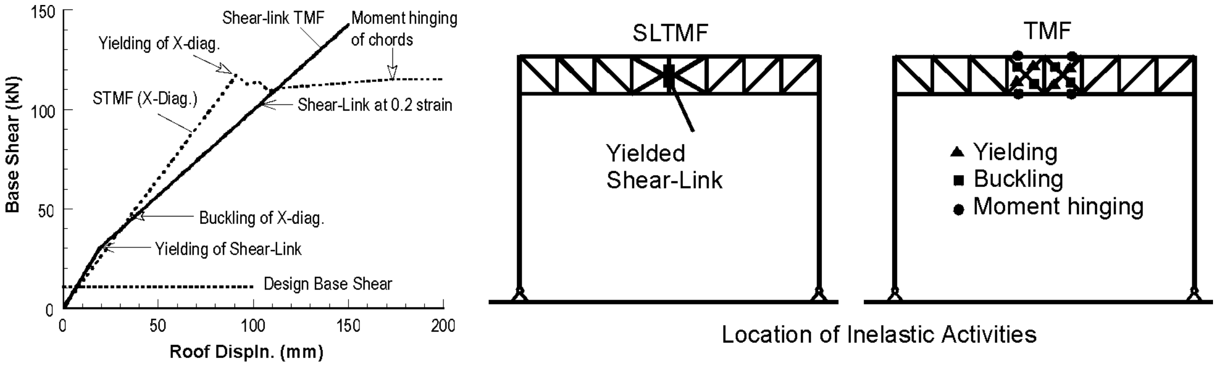

Figure 8. Pushover curve for STMF and SLTMF and the location of inelastic activities

Four different accelerograms of higher intensity were used for the TMFs: Miyagi-ken-Oki (0.51g), El Centro (0.347*g*) and two records Sylmar (0.84*g*) and Newhall (0.59*g*) of Northridge earthquake. 5% elastic response spectra of El Centro ground motion has spectra ordinates similar to the design spectrum for the range of periods which are significant to the study frames, i.e., about 0.7 s for the elastic frame to 1.0 s for the yielded frame. However, the other ground motions will result in demands much higher than implied by the design spectra. Miyagi has a narrow band of frequency around 1 Hz with an intermediate energy content. The Sylmar and Newhall motions recorded during Northridge (1994) earthquake are considered as “catastrophic” due to their very high velocities of longer durations, indicating greater ground displacements.

In Figure 9, base shear and roof displacement time histories for Miyagi ground motion excitation is shown. A drifting response for STMF was noted after the formation of a yield mechanism when the system lost its strength and stiffness, whereas the SLTMF maintained the stiffness and strength without any permanent drift due to excellent strain-hardening property of the shear-link.. Pinching and degrading hysteretic behaviour was observed for STMF and very little energy was dissipated by buckling and yielding of X-diagonals. SLTMF reduces the seismic energy input and therefore causes low ductility demand on energy dissipating elements.

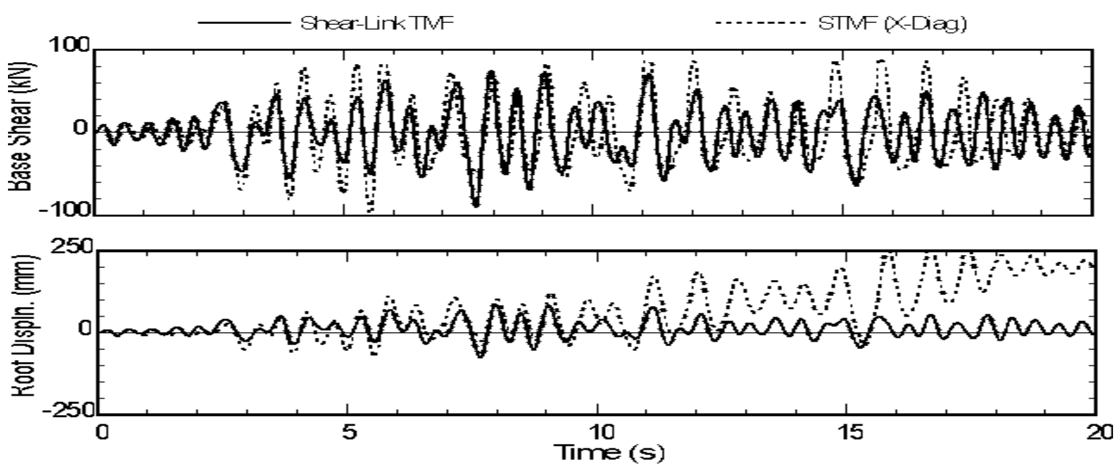

Figure 9. Comparison of time history response of SLTMF and STMF for Miyagi ground motion

Some response envelopes for both systems are compared in Table 2. Base shear as well as storey drift for Shear-Link TMF in general is less compared to STMF (X-Diag.) frame. The Miyagi motion is especially damaging for these systems because its energy is a narrow banded around 1 Hz which is close to the fundamental frequency of the yielded frames. Shear strain in links exceeded the maximum suggested value of 0.2 only in the case of catastrophic ground motion of Newhall. However, for design level El Centro ground motion, the maximum shear strain is close to the design shear strain (0.1).

Table 2. Response envelope values of TMF system

Ground motion	Base shear (kN)		Storey drift		Shear strain in link (mm/mm)
	SLTMF	STMF	SLTMF	STMF	
EL Centro (0.347g)	52.83	92.7	0.44	0.98	0.08
Miyagi (0.51g)	88.7	98.3	0.72	2.82	0.14
Newhall (0.59g)	116.4	140.1	0.99	1.79	0.17
Sylmar (0.84g)	132.47	137.5	0.92	1.34	0.19

CONCLUSIONS

Aluminium shear-links of soft alloy (3003-0) demonstrated excellent stiffness and energy dissipative capacity over a wide range of strains. They have excellent strain hardening behaviour, which is desirable in avoiding the soft storey problem. Transverse stiffeners restrain the pinching of hysteresis loops, delay the onset of plastic web buckling, and help shear-links achieve very large inelastic shear strains up to 20%.

The aluminium shear-links proved to be very effective and reliable in dissipating large amounts of energy for both braced frames and truss moment frames. A methodology to design the proposed shear-link has been developed together with analytical studies which establish its suitability under earthquake type loads. Results show the shear-link systems, SLBFs & SLTMFs, have high initial stiffness, with increasing strength at larger drifts. In general, dynamic response to various earthquake records resulted in lower values for the SLBF. In addition to these advantages, the shear-link equipped structure had a reduced base shear, a more uniform distribution of story drifts and a larger energy dissipation capacity per unit drift. Other advantages of shear-link include easy link replacement after an extreme earthquake and the ease of tailoring different link strengths to particular stories by adjusting link lengths.

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