ALUMINIUM SHEAR LINK-AN ENERGY DISSIPATION DEVICE FOR TRUSS MOMENT FRAMES

A DISSERTATION

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By

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CANDIDATE'S DECLARATION

I hereby declare that the work which is being presented in this dissertation titled, **"ALUMINIUM SHEAR LINK—AN ENERGY DISSIPATION DEVICE FOR TRUSS MOMENT FRAMES"**, in partial fulfillment of the requirements for the award of degree of MASTER OF ENGINEERING IN EARTHQUAKE ENGINEERING, with specialisation in **STRUCTURAL DYNAMICS**, submitted in the Department of Earthquake Engineering, University of Roorkee, Roorkee, India, is an authentic record of my own work carried out during the period from July 1998 to January, 1999, under the supervision of **Dr. DURGESH C. RAI** , Lecturer, Department of Earthquake Engineering, University of Roorkee, Roorkee.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree.

Dated:January 29th, 1999

Place: Roorkee

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This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ABSTRACT

ALUMINIUM SHEAR LINK—AN ENERGY DISSIPATION DEVICE FOR TRUSS MOMENT FRAMES

Truss Moment Frames (TMF) are widely used in large span Industrial and commercial buildings. However, their poor seismic performance in the past has limited their use in very high seismic regions due to their low ductile potential. Recent research at University of Michigan, Ann arbor, USA introduced a modified system," Special Truss Moment Frames(STMF)" which utilizes the weak girder-strong column yield mechanism. The inelastic activity of these frames is confined to a particular segment which is known as Special Segments, where the resultant vertical shear due to lateral loads is high. Out of these framing system energy dissipation is more in Vierendeel configuration. But the system has low initial stiffness due to lack of diagonal web member where the energy dissipation in X-diagonal configuration is somewhat poor.

A superior energy dissipation can be achieved from a TMF fitted with Aluminium shear links as an energy dissipator. Aluminium shear links are nothing but I-shaped beams designed to yield in shear mode and dissipates energy through its shear yielding (metallic hysteresis).The web members of truss girders will be arranged in either "K"or in diamond shape (<>) and shear links will be placed between horizontal vertices braces of adjacent panels.

A methodology is developed to design Shear-Link TMF. The seismic performance of Shear-Link TMF is compared with STMF(X-diagonal) for both monotonic as well as dynamic loads under simulated ground motions. The Shear-Link TMF demonstrated reduced energy input, base shear, storey drift and a large energy dissipation capacity per unit drift.

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INTRODUCTION

1.1 GENERAL

Industrial and commercial structures like warehouses, shopping malls, office buildings require large and clear areas unobstructed by columns. The economy in weight of the building can be attained by keeping large spacing for columns and having large spans for the trusses. In these structures main truss girder supports the secondary triangular roof trusses running in the perpendicular direction. Skylight may be provided by raising the height of the middle panel and providing the glazing portions. A general layout of an industrial building is shown in figure 1.1. Steel open-web truss moment frames are frequently used for these structures to resist lateral loads due to earthquakes and to support gravity loads.

Steel structures are generally preferred due to their high strength to weight ratio, large ductility, easy fabrication and erection. Open-web joist framing systems are preferred than solid-web framing system. Some advantages of the open-web framing system over the solid-web framing system are:

- Simpler detailing for moment connections,
- Ability to provide maximum ceiling height so that ducts and pipes can be installed through web openings, and

- Most effectively used for commercial and office buildings which contains large spans because of their light weight.

1.2 SEISMIC RESISTANT STEEL STRUCTURES

Earthquake resistant structures are usually designed to resist lateral forces resulting from representative major earthquakes that might occur near the site of the structure in their life time. Depending upon the importance of the structure they are usually classified as 1) Ordinary structures 2) Special structures. Ordinary structures are designed for demands due to moderate seismic activity which has a less probability of exceedence. Special structures are designed for seismic demands imposed by an extreme ground motion which has a less probability of occurrence.

Lateral force resisting elements are those which are going to resist the lateral forces caused by earthquakes. Three of the most commonly used lateral force resisting elements in buildings are: Moment Resisting Frames (MRF), Concentric Braced Frames (CBF) and Shear walls, of which CBF, MRF are solid web framing systems. However, in case of large span structures, for advantages as mentioned above, open web framing system like Truss Moment Frames are frequently used.

Truss Moment Frames (TMF) can be designed to resist lateral forces due to wind and earthquakes in large span buildings. Uniform Building Code (ICBO 1994) suggested seismic resistant truss moment frames to be designed either as Ordinary Moment Resisting Frames (OMRF) or Special Moment Resisting Frames. In OMRFs inelastic activity is not controlled whereas SMRF are designed such that inelastic activity is kept out of the truss girder. The design philosophy of SMRFs are based on "weak column-strong girder" (*WCSG*) yield mechanism. But such frames yielded unsatisfactory results during the 1985 Mexico City earthquake. The observed load-displacement hysteretic behaviour of OMRFs is generally poor with large abrupt drops in strength and stiffness caused by the buckling and early fracture of diagonal web members of the truss girders.

Recent research at the University of Michigan (Goel et al. 1990) resulted in the development of a system which is based on the "strong column-weak girder" (*SCWG*) yield mechanism and is referred as Special Truss Moment Frame (STMF). In this framing system, inelastic activity of truss girder is allowed in specified panels of the truss girder, known as Special Segments. The inelastic activity is confined to these segments only. All the remaining members outside the special segments behave elastically under factored design loads. This framing system can be configured in two ways: 1) STMF with X-Diagonals and 2) STMF with Vierendeel segment, as shown in figure 1.2.

The Vibration of buildings can be reduced by two major methods. One method adjusts stiffness whereas other improves damping performance. The performance of buildings can be improved by fitting the building with a mechanism or a device that absorbs seismic energy. The supplemental damping techniques may be classified into following four types:

- Designs using a viscous material like an oil damper,
- Designs using a viscoelastic material, and
- Designs using friction.
- Designs using metallic hysteresis.

Aluminium Shear-Link is one such element which utilises the metallic hysteresis for enhanced seismic resistance of building components. It is nothing but an I-shaped aluminium beam which is designed to yield in shear mode. It restrains the lateral force transmitted to the primary structural members and provides the significant energy dissipation potential. Energy dissipation capacity of STMFs (X-Diag.) can be further improved by application of energy dissipation devices like Aluminium Shear-Link. An arrangement of shear-link in the TMF is shown in figure 1.3.

1.3 OBJECTIVES OF THE STUDY

Special Truss Moment Frame (STMF) with Vierendeel configuration dissipates more energy than STMF with X-Diagonals, but the system has less initial stiffness. Superior energy dissipation can be achieved from Truss Moment Frame (TMF) fitted with Aluminium Shear-Link as an energy dissipation device. The objective of this research programme is to develop a design methodology of Truss Moment Frame system with aluminium shear link as an energy dissipator, i.e., Shear-Link TMF and evaluate its seismic performance. This research programme involved analysis of single storey study building consisting of Shear-Link Truss Moment Frame by using SNAP-2DX computer program (Rai et al. 1996). The computed response of this frame was compared with that of STMF (X-Diag.). These results confirm the feasibility of the system as well as its usefulness and effectiveness in controlling the structural response under simulated ground motions. Similarly, a four storey study building consisting of Shear-Link TMFs for its vertical lateral load resistance was analysed to evaluate its seismic performance and to verify its ability to avoid the "soft storey" problem.

1.4 ORGANIZATION OF THE DISSERTATION

A brief overview of seismic resistant large span steel structures is presented in this Chapter. Earlier studies and research related to truss moment frames, and aluminium shear-link for energy dissipation are reviewed in Chapter 2. Design considerations for the proposed system, i.e., Shear-Link Truss Moment Frame are given in Chapter 3. Description of the study buildings, modelling of frame for SNAP-2DX numerical analyses are narrated in Chapter 4. Step-by-Step illustration of design procedure for Shear-

Link TMF is presented in Appendix for a single storey study building. The results of numerical evaluation of the proposed system are discussed and compared with the STMF (X-Diag.) system in Chapter 5. Summary and conclusions of this study are given in Chapter 6.

CHAPTER 2

RELATED RESEARCH: STMF AND SHEAR-LINK

2.1 TRUSS MOMENT FRAMES

Since 1990, Goel and his students at the University of Michigan have been involved in an extensive research on a new type of seismic resistant open-web truss framing system, known as Special Truss Moment Frame (STMF). The full and stable hysteretic behaviour of STMF is better than the hysteretic behaviour of Ordinary Moment Resistant Frames (OMRF). STMF allows inelastic activity in some panels of the truss girder of the frames. The inelastic activity is limited to a particular segment called as "Special Segment", and all the remaining members of the truss frame behave elastically. This segment is generally placed near the mid-span where the vertical shear due to gravity loading is small and the resulting shear due to lateral load is maximum. Uniform Building Code (ICBO 1994) specifies a value of 10 to R_w for STMFs.

Plastic design approach is suggested for the design of STMF, because of the predetermined yield mechanism. The required strength of the special segment can be calculated by the plastic analysis under factored (ultimate) gravity and lateral design force combination. All members of the truss which are out of the special segment are designed to such a strength so that they can provide the strength needed to develop the ultimate shear strength of the special segment. These outside members are expected to behave elastically, so that their sections need not be compact and any truss configuration can be used outside the special segment. All the members of the frame including the special segment are designed to remain elastic under other design load cases. This new concept is based on the "strong column-weak girder" combination which is favoured for multi-storeyed construction. Depending on the type of configuration of the special segment, these frames are divided into two categories, as mentioned in the previous chapter:

- STMF with X-Diagonals and
- STMF with Vierendeel Segment

Typical configurations for these frames are shown in figure 1.2.

2.1.1 STMF WITH VIERENDEEL CONFIGURATION

In the experiments conducted by Basha & Goel (1994), the concept of STMF was extended to fully open Vierendeel type special ductile segments located near the middle half of the truss girder. These open panels can be useful where more open space for larger ductwork is desired. In this system the dissipation of energy was achieved through the development of plastic hinges at the ends of chords of the special segment. The observed yield mechanism, and full and stable load-displacement hysteretic behaviour of this kind of STMF is shown in the figure 2.1.

2.1.2 STMF WITH X-DIAGONALS

In the case of X-diagonal segment the ultimate strength of the special ductile segment is provided by tension yield and post buckling compressive strength of the X-diagonals and the energy dissipation in the structure is primarily through the buckling of X-diagonals. Braces in the special segment are designed as tension only members. Figure 2.1 shows the yield mechanism of STMF with X-diagonals and the typical load-displacement response of full-scale test sub-assemblage experiments conducted by Itani & Goel (1991).

Moment hinging at the ends of chords of the special segment completes the collapse mechanism following yielding and buckling of X-Diagonals.

Energy dissipation is superior in Vierendeel type special segment but the initial stiffness of the system is low due to the lack of diagonal braces. On the other hand, the energy dissipation by diagonal braces through buckling and yielding is somewhat poorer than due to plastic hinging of chords in Vierendeel configuration. Retrofitting of yielded X-diagonal after an extreme seismic activity is easy, but in case of Vierendeel configuration repair of chord members is relatively cumbersome as floors are directly supported on them.

2.2 ALUMINIUM SHEAR-LINK AS AN ENERGY DISSIPATOR

Aluminium shear-link is an I-shaped beam designed to yield in shear mode to limit the amount of lateral force that is transmitted to the primary structural members, and it also works as an energy dissipation device absorbing seismic energy through its inelastic deformations. Shear-link is designed to such a force so that it starts yielding prior to the buckling of braces or any other member of the structure. The aluminium alloys are chosen for shear-link because of its low yield strength which enables the use of thicker webs reducing the problem of web buckling. The shear yielding of web maximises the amount of material participating in plastic deformation without excessive localisation of strains. On the contrary the flexural yielding causes gradual plastification of the section and high concentration of strains. As a result, in shear yielding through relatively uniform distribution of plastic strains, a large amount of energy can be dissipated before the material fractures (Rai & Wallace 1998). The significant amount of strain hardening in aluminium alloy reduces the undesirable 'soft storey' problem by enabling all the links in a multi-storey structure to participate in energy dissipation.

The shear stress vs. shear strain hysteretic behaviour of shear link and a typical shear-link which was subjected to a cyclic shear loading of strains up to 0.2 is shown in the figure 2.3. The 3003-O aluminium alloy of the link sustained large plastic deformations without tearing. The first yield was observed at 0.002 strain and at a stress of 0.722 times the 0.2% yield stress $\sigma_{0.2}$ of the material. The link strain hardened during subsequent cyclic loading and achieved an average stress of $1.88\sigma_{0.2}$ in 0.2 strain cycles. Stable hysteretic loops were observed up to 0.1 strain and a degradation of the strength was observed at this stage. Specimens appeared distressed with deformed stiffeners, however, they retained a good load carrying capacity. The addition of end stiffeners to the link beam performs 'pratt truss' action between the flanges, stiffeners and web, which helps in achieving stable hysteretic behaviour through the formation of cyclical diagonal tension field. A yielded specimen is shown in figure 2.3(a) where the development of 'buckles' in panels formed by stiffeners can be seen. Stiffeners delay the initiation of plastic web buckling and improve the post buckling behaviour of the link (Rai & Wallace 1998).

CHAPTER 3

DESIGN OF SHEAR-LINK TRUSS MOMENT FRAME

3.1 ALUMINIUM SHEAR-LINK IN TRUSS MOMENT FRAMES

Aluminium shear-link can be effectively used to enhance the seismic energy dissipation of Truss Moment Frames (X-Diag). They will not only retain the stiffness provided by diagonal members in the truss girder but also improve the energy absorbing capacity. The energy dissipation in this framing system is by the shear yielding of the Aluminium shear-link. This shear-link is designed in such a way that shear yielding of link commences prior to the buckling or yielding of any other member of the frame. The link is placed between the horizontal vertices of diagonals of adjacent panels. Web members of the truss girder can be arranged either '**K**' or in diamond (<>) shape. Figure 3.1 Shows the arrangement of shear-link in TMF and its expected yield mechanism under lateral loads (Rai & Prasad 1998).

Strong column-Weak girder (*SCWG*) yield mechanism was chosen for the design of Shear-Link TMF. All the inelastic activity of the frame was allowed at the centre of the truss girder, i.e., 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. Replacement of link is easy after an extreme seismic event. Moment hinging at the ends of chords of the special segment are expected at the collapse in addition to the yielding of link under lateral loads.

3.2 DESIGN METHODOLOGY FOR SHEAR-LINK TMF

The design concept is based on the Limit State approach in which a pre-selected yield mechanism is assumed for the TMF at ultimate loads. The frame is detailed in such a way that most of the inelastic deformations are contained in the shear-link only. For the selected yield mechanism, frame is solved for the member forces by methods of plastic analysis. The link should not yield under factored gravity loading. Pattern loading should be considered within the bay also as the bay spans are large and considerable variations in design forces are likely due to spatial distribution of loads. The factored seismic loads are resisted by vertical shear in the shear-link and chords of the special segment. All members outside the special segment including columns are designed to remain elastic under forces generated by strain hardened shear-links and plastic hinges in chords of the special segment.

3.2.1 DESIGN FORCES

Shear-Link TMF is a strong column-ductile truss girder frame system for which the response reduction *R* of the IS:1893-1996 Draft Code can be taken as 5 for computing lateral forces. The frame is to be designed using the Plastic Design (PD) methods as per IS:800-1984 (BIS 1984). Following load combinations need to be considered:

(a) 1.7(D+L)	(Elastic design)
(b) 1.7(D+E)	(Plastic design)
(c) 1.3(D+L+E)	(Plastic design)
(d) 1.3(D+L)+V_l	(Ultimate mechanism)

Load combinations (a), (b) and (c) are required to provide the sufficient strength under factored Dead (D), Live (L) and Earthquake (E) loads. All members of the frame are designed to remain elastic for load case (a). Plastic activity in the special segment including link is permitted for load combinations (b) and (c) . The primary objective of load case (d) is to keep the members outside the special segment elastic when shear-link and chords of the special segment reach their ultimate strengths. The term V_l represents the maximum strength which shear-link and chords of the special segment can attain including the effect of strain hardening and material overstrength.

3.2.2 SPECIAL SEGMENT

The size of shear link and chords of the special segment are based on the maximum vertical shear due to load cases (a), (b) and (c). Pattern loading is to be considered on the truss girder to get a maximum value of shear. Vertical shear from the load case (a) is assumed to be resisted solely by the shear link in elastic regime and there should not be any yielding in the link. Shear-link is designed to carry 75% of the required vertical shear in the girder. Shear contribution of chords can be taken as 25% of required vertical shear.

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. The length to depth ratio of any panel in the special segment is kept between 1.5 and 0.67. Splicing of chord members are to be avoided within the special segment. The top and bottom chords of the special segment must have identical sections and in the fully yielded state must provide at least 25% of the required vertical shear strength.

Axial stresses in chords must be kept below $0.4f_y$. The width to thickness ratio (b/t) of chord members should not be greater than $0.31\sqrt{(E/f_y)}$, where **E** is Young's modulus and f_y is yield stress of steel. The ends of the special segment should be laterally braced since yielding of chords are expected to occur.

The diagonals in the special segment are designed to such a strength so that the axial forces in diagonals of the special segment must be less than their buckling loads, even after the link attains its maximum strength.

3.2.3 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. Shear-link is assumed to contribute 75% of vertical shear in the special segment. The horizontal area of the web of the shear-link is calculated to this shear. The web area was obtained by dividing the vertical shear by the shear stress of the link corresponding to a limiting value of shear strain. This shear stress $\tau_{ave,max}$ for a given shear strain γ in a shear-link is given by following expression (Rai &Wallace 1998):

$$\tau_{ave.,max} = 2.6\ \sigma_{0.2}\ \gamma^{0.2} \tag{3.1}$$

where $\tau_{ave,max}$ is average peak shear stress and $\sigma_{0.2}$ is the tensile yield stress of the material (3003) corresponding to 0.2 strain.

Shear force R and shear deformation Δ of a shear-link is related to shear stress τ and shear strain γ as follows:

$$R = \tau\ A_w \tag{3.2}$$

$$\Delta = \gamma\ d \tag{3.3}$$

where A_w is the horizontal web area of the link (l times t_w) and d is the depth of the I-shaped shear-link.

Above power relation (Equation 3.1) is based on the experimental data up to 0.2 shear strain and can not be used for large strains. Hence 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. Based on the experimental data given by Rai & Wallace (1998), the maximum shear capacity of the link can be taken as 1.88 times the tensile yield stress $\sigma_{0.2}$ of the material, i.e.,

$$\tau_{max} = 1.88\ \sigma_{0.2} \tag{3.4}$$

Limiting shear strain can be obtained corresponding 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. In other words 'design' shear stress to proportion shear links is obtained from equation 3.1 at design shear strain γ_d which is given as

$$\gamma_d = \delta/d \leq 0.1 \tag{3.5}$$

where δ is allowable storey drift and d is the depth of the shear-link.

Stiffeners to the link beam on both sides of the web are required to prevent an early plastic buckling of the web and to ensure a ductile shear failure of the web. Transverse stiffeners may be provided at each end of the link and, if required, intermediate stiffeners should be provided at regular intervals so that the link satisfies the following equation

$$\gamma_b = 9.37\ (k_s/\beta^2) \tag{3.6}$$

where β is web depth-to-thickness ratio and k_s is buckling coefficient as defined below:

$$k_s = \begin{cases} 5.6 + 8.98/\alpha^2, for(\alpha \leq 1) \\ 8.98 + 5.6/\alpha^2, for(\alpha \geq 1) \end{cases} \tag{3.7}$$

where α is aspect ratio, which is defined as the ratio of stiffener spacing a to the clear depth of the link beam $(d_w = d\text{-}2t_f)$ where t_f is the thickness of the flange. Intermediate stiffeners can be avoided if the web depth-to-thickness ratio β of link is less than 20.

To allow shear-links to maintain their post-buckling capacities, each transverse stiffener is proportioned to avoid the possibility of local buckling of the stiffener which must 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.

The stiffness check is satisfied if the moment of inertia of stiffener about its centroidal axis parallel to the web is not less than $J\xi a(t_w^3)$, where

$$J = \{ [2.5/(a/d_w)^2] - 2 \} \geq 0.5 \tag{3.8}$$

where a is the distance between transverse stiffeners and ξ is an amplifying coefficient to account for post-buckling strength and a value of 4 is suggested for an open section stiffener. For stability check, the stiffener is designed as a compression strut for an axial force equal to 0.3 times the factored shear load. The stiffener thickness t_s should not be less than t_w, and width should be $(b_f/2 - t_w)$, where b_f is flange width of the link beam (Rai & Wallace 1998).

3.2.4 MEMBERS OUTSIDE THE SPECIAL SEGMENT

All members and connections of Shear-Link TMF outside the special segment are designed to withstand the maximum amplified vertical shear developed in the special segment. The amplified vertical shear in the special segment is as follows

$$V_{ss} = 3.4\Omega\left(\frac{M_{nc}}{L_s}\right) + 0.07EI\left(\frac{L - L_p}{L_p^{\,2} L_s}\right) + R_{max} \tag{3.9}$$

where $L_s = 0.9\, L_p$, and L_p is length of the special segment; Ω is over strength factor, suggested as 1.1; M_{nc} and EI are flexural strength and stiffness of chords of the special segment and R_{max} is the maximum shear strength of strain hardened shear-link and is given as

$$R_{max} = 1.88\ \sigma_{0.2} A_w \tag{3.10}$$

where A_w is vertical web area of the shear-link, i.e., length l times thickness of the web t_w. The first two terms in equation 3.9 represent the vertical shear corresponding to strain hardened plastic hinges in chords of the special segment (Rai et al. 1998).

CHAPTER 4

MODELLING OF SHEAR-LINK TRUSS MOMENT FRAMES

Seismic performance of the shear-link TMF as described in previous chapter is studied numerically using SNAP-2DX program for non-linear analysis of planar structural frames (Rai et al. 1997). The performance is compared with STMF (X-Diag). The STMF was designed as per the requirements of UBC96 (ICBO 1996), however, the design seismic forces were kept same as the ones used for shear-link TMF. The following sections describe modelling of structure using SNAP-2DX and the description of study buildings.

4.1 DESCRIPTION OF STUDY BUILDINGS

4.1.1 STUDY BUILDING 1

A single storey storage building is assumed to be located in the Draft IS:1893-1996 (BIS 1996) seismic zone V (Z = 0.36) on the soil site. In the plan, the building is 90 m long in the E-W direction (9 bays @ 10 m) and 36 m (3 bays @ 12 m) wide in the N-S direction. As for elevation, the height of the roof is taken as 9.5 m. The typical framing plan of the building is shown in figure 4.1. Six frames on column lines 2, 3, 5, 6, 8 and 9 are lateral force resisting truss moment frames in the N-S direction . The other frames are designed to resist only gravity loads.

The calculation of design base shear for the North-South TMFs is summarised in Appendix. For the above building both Shear-Link TMF and the STMF (X-Diag) are designed for a comparison purpose. The members chosen for the STMF (X-Diag) as per UBC 96 (ICBO 1996) and members of the Shear-Link TMF are given in table 4.1. The size of the shear links were determined as discussed in the previous chapter. Step-by-step design procedure is provided in Appendix. In case of outer bays the design vertical shear in the special segment is calculated by assuming the truss girder to be fixed at interior end and simply supported at the exterior end. In case of interior bays it is assumed simply supported at both ends. The response of the interior bay frame was compared for both the systems.

4.1.2 STUDY BUILDING 2

A four storey building using Shear-Link TMF is selected for the second example. It has plan dimensions of 48 m by 36 m for all four stories. The building is divided into 6 bays in both N-S, and E-W directions. The basic components of the building are as follows:

- Seven frames in the N-S direction
- Seven frames in the E-W direction
- Secondary steel joists spanning the transverse direction between the transverse frames and simply supported in the longitudinal direction and
- A steel deck and concrete flooring system.

The four moment resisting longitudinal frames (A, C, E and G) resist lateral forces and provide lateral stability of the building in that direction. The interior frames (B, D & F) support only gravity loads for their tributary areas. Four frames spanning in the N-S direction of the building along column lines 1, 3, 5 and 7 are Shear-Link Truss moment Frames, while others are gravity frames.

In this design example, the focus is on frames spanning in the N-S direction. A typical interior frame in longer direction is subjected to equivalent earthquake lateral forces, in addition to gravity loads transferred by the secondary steel joist. Figure 4.2. Shows the plan of the building and the elevation view of a typical interior bay frame in the N-S direction of the building. Each floor height is taken as 4.5 m. Each truss girder is divided into 8 segments of 1 m length. The seismic force calculations for this frame are shown in table 4.2. Two sizes of links are used in this framing system. One large size link (450x10 mm) is provided for the first and second floors and another small size link (400x5 mm) is provided for the third and fourth floors. Properties of both the links are presented in table 4.3 and other members that are selected for this framing system are shown in table 4.4, same frames are adopted for all floors but they can be reduced for top stories. The depth of the truss girder is 1 m and length of the special segment is 2 m. Shear-link is placed at the centre of the truss girder connecting vertices of diagonals of the special segment. This frame's response is studied for dynamic loading.

4.2 MONOTONIC ANALYSIS

To determine the ultimate lateral load capacity and collapse mechanism, generally monotonic (Static Pushover) analysis is preferred. In this method frames are subjected to incremental monotonic loads in the code assumed storey distribution of loads at all loading steps. When executing the program (SNAP-2DX), a number of data files are automatically created, of which the main output file includes a status of various members of the force at every load step. By inspecting the output file the sequence of yielding in the frame can be determined.

4.3 DYNAMIC ANALYSES

The objective of this analysis is to evaluate the performance of the Shear-Link TMF system under various strong ground motions and to study the response of the framing system in comparison with STMF (X-Diag.) for earthquake type loads. Choosing a representative earthquake is difficult, for each earthquake has a different duration of strong shaking as well as amplitude and frequency content. Consequently, there is no assurance that the intensity of shaking during an earthquake will not exceed the maximum intensity level which the structure was originally designed to resist. Therefore, it is generally recommended to use more than one acceleration record in studying the dynamic response of structures.

4.3.1 GROUND MOTIONS

Four different ground motions are taken for this study in time history analysis. They are Miyagi-Ken-Oki (0.51g), El Centro(0.347g), two records of Northridge earthquake (Sylmar, 0.84g and Newhall, 0.59g). Figure 4.3(a) shows a comparison of Draft IS:1893-1996 Soil site design spectrum (5% Damping) to the spectra of above mentioned ground motions. El Centro earthquake record is almost similar to Draft IS:1893-1996 design spectrum for soil site (5% damping). Miyagi record has a narrow band of frequency around 1 Hz with an intermediate energy content. Both ground motions of Northridge earthquake represent seismic demands far greater than other motions over period range of interest. Fourier spectra of these ground motions are shown in figure 4.3(b). Seismic mass of the frame is lumped at columns of the frame. The damping matrix is taken proportional to the mass matrix only and a damping value of 2% of critical is included in the analyses.

4.4 MODELLING IN SNAP-2DX

SNAP-2DX is a computer program for dynamic non-linear analysis for two dimensional frame structures. The program has the capability to perform static pushover analysis under displacement or load control. It can also perform time history dynamic analysis with constant as well as variable time step. The program generates some output files for post processing. These files provide the storey shears, overturning moments at each storey, storey displacements, storey drifts, recoverable strain energy and hysteretic energy dissipated at each storey and by each element group.

This program utilises the member to member modelling approach, i.e., one-to-one correspondence exists between the elements of the model and the members of the structure. The force deformation properties of the elements are described in terms of hysteretic behaviour expressed by a force-deformation pair controlling the behaviour of the element. The frame members are grouped in to beam-column (element 2), bracings (element 9) and shear-links (element 7). The chords of the truss girder and columns are modelled as beam-column elements and the web members of the truss girder as bracing elements.

A bilinear non degrading type of hysteretic behaviour is assumed for shear-links which can be defined by the following three parameters:

- Yield shear force R_y,
- First stiffness K_1,and
- Second stiffness K_2,which is expressed as percentage of K_1.

Figure 4.4. shows the bilinear hysteretic model considered for shear links. 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 yield stress is taken as 0.577 times the tensile yield strength $\sigma_{0.2}$. This value is multiplied by the horizontal web area A_w of the

shear-link to give R_y. The second stiffness can be easily computed from the fact that the maximum shear force allowed at the shear strain 0.2 marks the end-point of the second linear branch of the primary curve. The second stiffness is very small as compared to the first stiffness but is large enough in absolute terms to ensure satisfactory frame response.

4.4.1 MODELLING USING EQUIVALENT BEAM METHOD

Modelling of multi-storey multi-bay frame results in a large number of nodes and elements which means a large number of degrees of freedom leading to very large structure stiffness matrix. Preparation of input file for such frames is cumbersome and need extra care. It also needs more time for the execution of the program. Therefore a simple, mathematical model for this frame, as shown in figure 4.5, is considered which reduces the I/O effort of analyses. The concept of the idealised mathematical model of the frame is as follows (Rai et al. 1997):

Elastic segments of the truss girder (outside Special segment) are replaced with simple beam elements having equivalent stiffness, I_e in the plane of deformation as defined below:

$$I_e = 2f\left\{I_0 + A(d/2)^2\right\} \tag{4.1}$$

where I_0 and A, are the moment of inertia about axis of bending and area of chords, respectively, and d is the depth of the truss girder. From the earlier studies by many investigators, the suggested value of f ranges between 0.6 and 0.7.

Similarly, the moment of inertia of an equivalent column of the one bay frame model is to be taken by using the following relation

$$I_{1\text{-}bay} = \frac{\sum I_{n-bay}}{2n} \qquad (4.2)$$

and for shear-links

$$K_{1\text{-}bay} = \frac{\sum K_{n-bay}}{2n} \qquad (4.3)$$

where n is the number of bays in the complete frame.

To preserve the properties of chords of the special segment and other members of the special segment they are kept as original. Rigid elements are used at column-to-girder connections as well as at girder-to-special segment connections. A comparative response of complete 3-bay frame and equivalent single bay frame for the study building 1 are shown in figure 4.6. Pushover curves for both the frames are almost same. There is a little difference in case of dynamic response. This difference is due to the fact that for equivalent single bay frame average properties for shear-links were used which made this frame to yield a little earlier than the complete 3-bay frame. An early initiation of inelastic activity in equivalent bay frame changed its stiffness properties which affect its dynamic response.

CHAPTER 5

RESULTS AND DISCUSSION

5.1 STUDY BUILDING 1

5.1.1 MONOTONIC ANALYSES

The relationship between the base shear and roof displacement for the interior bay frame of STMF (X-Diag.) and Shear-Link TMF is shown in figure 5.1. Shear-Link TMF showed a better performance over the STMF (X-Diag.). Yielding of shear link was observed at a base shear of 44.1 kN (0.21% of storey drift). After the yielding of shear-link, 42.8% reduction in the initial stiffness (1.75 kN/mm) was noted. However, the shear-link attained its maximum allowable strength (R_{max}) at a base shear of 147 kN, i.e., at 1% of storey drift. At this stage all the inelastic activity of the frame was observed in the shear-link only and system was still capable to resist more base shear with further increase in lateral displacements.

In comparison, the first inelastic activity was noted in the STMF (X-Diag.) at 46.2 kN (0.37% storey drift) when the buckling of X-diagonals was observed. There was no significant reduction in the stiffness of the system after buckling of X-diagonals and it continued to carry more load until X-diagonals started yielding in tension at a peak base shear of 116 kN (0.97% storey drift). The STMF lost its strength and stiffness at 1.81% of storey drift forming complete yield mechanism, i.e., moment hinging at the ends of chords of the special segment along with yielded and buckled X-diagonals in the special segment.

5.1.2 TIME HISTORY ANALYSES

Base shear response of both the Shear-Link TMF and STMF (X-Diag.) is shown in figure 5.2 for all four different ground motions studied. Shear-Link TMF attracts less base shear when compared to STMF (X-Diag.). The input energy of the system is also less for Shear-Link TMF. Figure 5.3 shows the roof displacement response of the Shear-Link TMF and STMF (X-Diag.). These responses were obtained under full action of gravity loads. STMF (X-Diag.) showed a drifting response at the end of 20 s of Miyagi record with permanent drift after the formation of yield mechanism. But the Shear-link TMF retained its strength and stiffness without any permanent drift due to excellent strain hardening properties of the Aluminium shear-links.

Results from the other ground motions also affirmed the better performance of Shear-Link TMF over the STMF (X-Diag.). Shear-Link TMF also showed a little permanent drift for catastrophic ground motions, Sylmar (0.84*g*), and Newhall (0.59*g*). However this drift was much smaller than those observed for the STMF (X-Diag.). If they were designed for the forces expected from extreme motions such as Sylmar and Newhall, one would not get permanent drift for Shear-Link TMF.

Base shear vs. roof displacement hysteretic plots for both the frames are shown in figure 5.4. Pinching and degrading hysteretic behaviour is observed for STMF where a little energy is dissipated by buckling and yielding of X-diagonal. On the other hand, full and stable hysteretic loops are observed for Shear-Link TMF where all the energy is dissipated by the shear-link only. Energy time histories for both the frames are shown in figure 5.5. Shear-Link TMF attracted less seismic energy than the STMF (X-Diag.), however, in both the frames most of the energy is dissipated by the hysteresis of yielding elements. Shear-Link TMF allows low ductility demand on energy dissipating elements due to less seismic energy input.

5.2 STUDY BUILDING 2

5.2.1 TIME HISTORY ANALYSES

Figure 5.6 shows the base shear response of the equivalent single bay four storey Shear-Link TMF. Frame attained complete yield mechanism at the end of 20 s Sylmar record. All the links of the frame yielded and participated in the energy dissipation. Moment hinging at the ends of chords of special segment of first storey frame was noticed at the end of this ground motion. The system showed a good response for El Centro ground motion. Inelastic activity for El Centro ground motion was only limited to shear-links. Figure 5.7 shows the floor displacement response for each storey of the frame. Hysteretic response of storey shear vs. storey drift for each storey are shown in figure 5.8, which shows stable and full hysteretic loops as expected. Energy time histories are shown in figure 5.9 along with the energy absorbed in each storey. Most of the energy is dissipated by the shear-links alone. In case of Sylmar ground motion moment hinging was noticed at the ends of chords of the special segment of the first and second floors. No moment hinging was observed in case of El Centro ground motion.

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 SUMMARY OF THE STUDY

Aluminium shear-link has many desirable attributes for the enhanced seismic resistance of Truss Moment Frames. It allows the superior energy dissipation without any changes in the strength and stiffness of the truss moment frames. A methodology for designing these Shear-Link TMFs is developed and illustrated. Limit State Design procedure is used to design this system. The factored seismic loads are resisted by the vertical shear in the special segment, mostly by shear-link. The design begins by selecting the size of shear-link to resist the resulting vertical shear in the special segment. Inelastic activity of the frame is confined to the shear-link and chords of the special segment. All members outside the special segment including columns are designed to remain elastic under factored design loads.

Three bay single storey Shear-Link Truss Moment Frame of the study building was analytically investigated for monotonic as well as dynamic loading using SNAP-2DX computer program. Shear-Link Truss Moment Frame showed excellent results with less floor drifts, large energy dissipation and large lateral strength in comparison to the STMF (X-Diag.). Equivalent single bay frame model was also discussed to reduce the I/O effort and execution time of the computer program. Selected response envelopes for STMF (X-Diag.) and Shear-Link TMF are presented in table 6.1 for all ground motions used in this study. Base shear and storey drift for the Shear-Link TMF are less when compared to STMF (X-Diag.). In case of catastrophic ground motions shear-links attained shear strain nearer to the maximum suggested value of 0.2. The maximum shear stress in the link is less than the design shear strain (0.1) in case of design level El Centro ground motion.

Response envelopes of four storey equivalent single bay frame are presented in table 6.2. This frame also performed well both for monotonic as well as for dynamic loads. Excellent strain hardening characteristics of the aluminium reduces the undesirable soft storey problem by allowing all the links in a multi-storey structure to participate in energy dissipation.

6.2 CONCLUSIONS

The major findings and conclusions from this study can be summarised as follows:

1. Low alloy metals such as Aluminium can be used for large plastic deformations.

2. Aluminium shear-links have very ductile shear yielding and can dissipate large amount of energy effectively and reliably even at large strains (up to 20% shear strain).

3. Energy dissipation capacity of Truss Moment Frames can be improved significantly with Aluminium shear-links as an energy dissipator.

4. Aluminium Shear-Link not only retains the lateral strength and stiffness of the system and also reduces the energy transfer into the primary structural members.

5. Truss Moment Frames equipped with shear-links showed significant reduction in
    - Seismic energy input,
    - Base shear, and
    - Storey drifts.
6. Excellent strain hardening characteristics of Aluminium reduces the undesirable soft storey problem, by enabling all the links in a multi-storey structure to participate in energy dissipation.
7. Shear-Links can be easily replaced after an extreme seismic event and can be deployed in existing Truss Moment Frames for enhanced seismic resistance.

6.3 SUGGESTIONS FOR FUTURE STUDY

The numerical studies clearly indicates superior performance of Aluminium shear-links as a seismic energy dissipator for STMFs. Studies should be undertaken for its experimental verification. A sub-assemblage study as carried out for the verification of STMFs can be very helpful in assessing its effectiveness. Shake Table studies should be attempted at a later stage to study various system parameters that can not be obtained from sub-assemblage tests. It should be noted that the shear yielding of low alloy metals can be exploited in many ways. Recent developments in low yielding alloys of steel promises many interesting applications of shear yielding devices for earthquake resistance. These steel alloys will eliminate problems arising due to presence of dissimilar materials in a structure, for example, aluminium shear-links in steel girder.

APPENDIX

APPENDIX

STEP-BY-STEP DESIGN PROCEDURE FOR SHEAR-LINK TRUSS MOMENT FRAMES

A.1 General

Various steps involved in the design of lateral load resisting Shear-Link TMF is illustrated in this Appendix for the study building 1. The general description of the building is provided in Chapter 4.

A.1.1 LOADS

Gravity Loads

Dead Load:

Roof:

Framing	= 250 N/m²
Corrugated Steel deck	= 150 N/m²
Total	**= 400 N/m²**

Exterior Walls:

Versa Wall	= 175 N/m²
Wind girt	= 90 N/m²
Columns	= 90 N/m²
Misc.	= 15 N/m²
Total	**= 370 N/m²**

Live Load:

Live Load on the roof = 1000 N/m²

Seismic Loads

Seismic weight of the Building, W = 0.4X36X90 + 2X90X0.37X9.5/2

= 1612.35 kN.

Base Shear, $V = \frac{(Z/2)(S_a/g)}{(R/I)} W$ (IS:1893-1996 Draft)

Z=0.36 (Zone V)

$I = 1.0$

$R = 5$ (SMRF)

T= 0.085 $(h)^{0.75}$ = 0.46 s

S_a/g= 2.5 (Soft soil and 5% Damping)

Therefore, V= 145.11 kN.

Base Shear for each frame =145.11/6 = 24.18 kN.

Add for accidental torsion the maximum shear for each frame is

$$V_t = 1 + \frac{0.1\text{x}36\text{x}35}{2[35^2 + 25^2 + 5^2]} V = 1.034 \text{ V}$$

Maximum design base shear for each frame = **25 kN**

A.2 INTERNAL FORCES IN THE TRUSS GIRDER

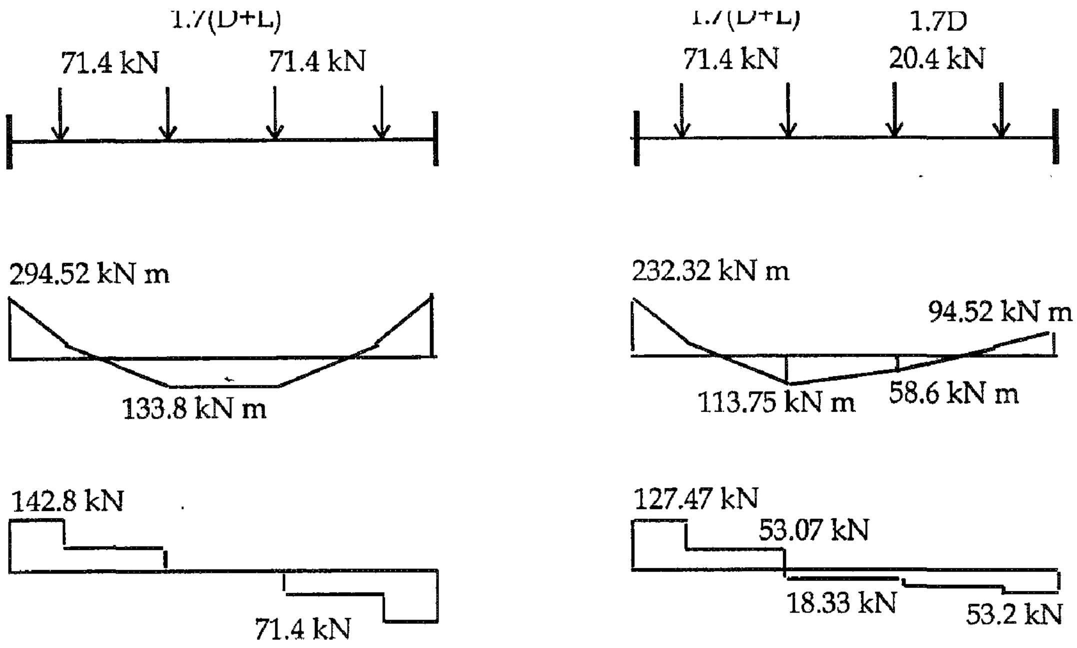
Vertical shear in special segment due to load combinations 1.7(D+L) and 1.3(D+L+E) is calculated by simple approximate analysis using appropriate boundary conditions at either ends of the truss girder. Pattern loading is considered within the bay which leads to two load cases, namely, balanced and unbalanced cases.

Pattern dead load, PD = 0.4x12x10/4 = 12 kN

Pattern live load, PL = 1.0x12x10/4 = 30 kN

A.2.1 Load case(a) 1.7(D+L):

Shear force and bending moment diagrams for the truss girder are shown above for two major load cases. Maximum vertical shear in the special segment due to unbalanced load case is 18.33 kN.

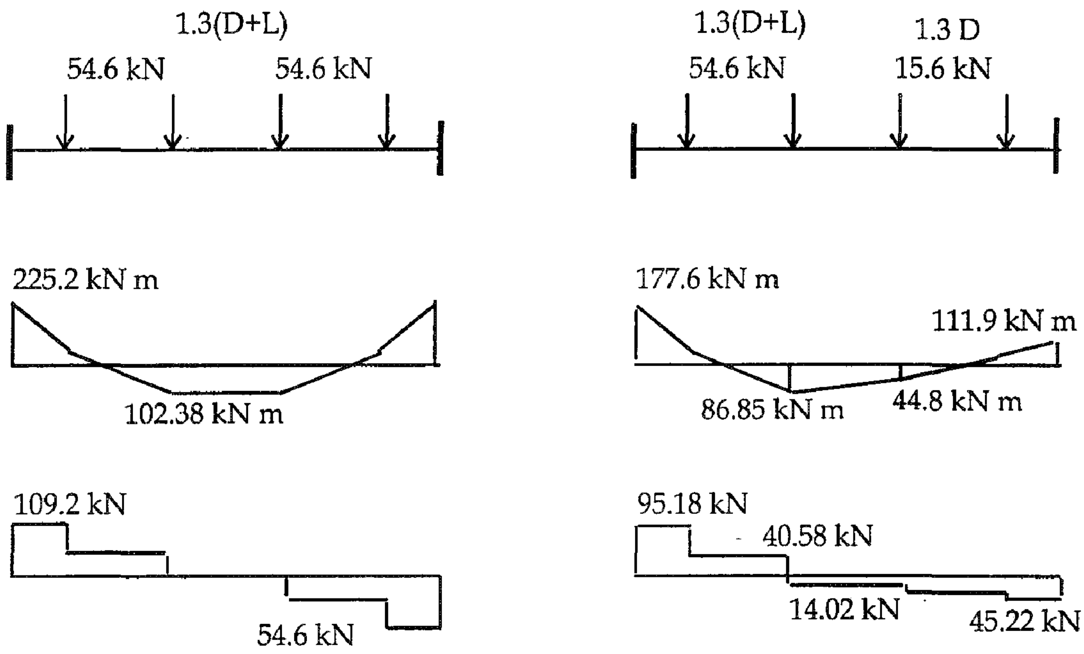


Balanced Case Unbalanced Case

Figure A.1 Internal forces in truss girder due to 1.7(D+L)

A.2.2 Load case(c) 1.3(D+L+E):

Vertical shear in special segment is determined by superposing two load cases 1.3(D+L) as above and 1.3E. The shear due to 1.3E is determined by considering the equilibrium of forces on an equivalent one bay frame as shown in figure A.3.



Balanced Case Unbalanced Case

Figure A.2 Internal forces in truss girder due to 1.3(D+L) for both ends fixed

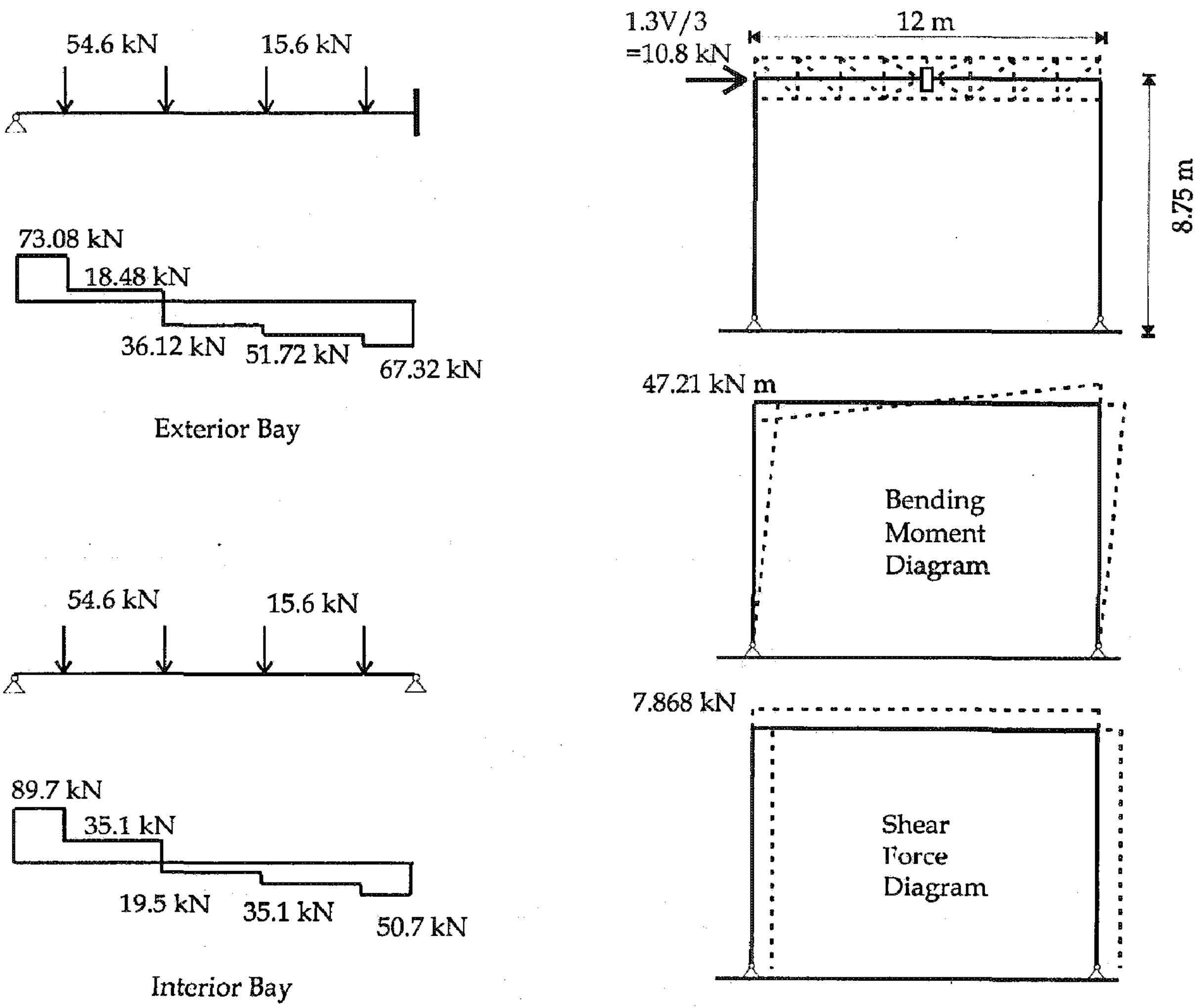


Figure A.3 Internal forces in truss girder due to 1.3(D+L) and 1.3E

To get a rough estimate of the maximum vertical shear in a typical bay of the truss girder various end boundary conditions should be considered. For interior bays a simple support conditions at both ends yields satisfactory results whereas one end hinged and the other end fixed boundary conditions is suitable for outer bays.

The maximum vertical shear in the truss girder is obtained from the load 1.3(D+L+E), i.e.,

$V_{req} = 36.12+7.868 = 44.0$ kN

A.3 SPECIAL SEGMENT

A.3.1 Design of Shear-Link

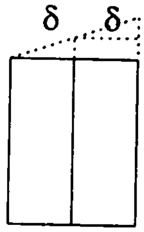
Shear yield stress of link (3003-O Aluminium) $\tau_y = (1/\sqrt{3})\ \sigma_{0.2}$

$\tau_y = 20.323\ \mathrm{N/mm^2}$

Shear-Link is designed for 75% of $V_{req} = 31.7$ kN

Shear area required = 31.7x1000/20.323 = 1560 mm^2

Providing link size as 350x5 (*l times* t_w) of area = 1750 mm^2



$R_y = A_w\ \tau_y = 35.56$ kN

$R_{max} = 1.88\sigma_{0.2}\ A_w = 115.8$ kN

$\delta = \theta\ L = (0.004)\ (6000) = 24$ mm

Assuming two links on either side of the vertical web member as shown in figure below, the δ for each link equals 12 mm.

Therefore, depth of the shear-link, $d = \delta/\gamma = 12/0.1 = 120$ mm

The properties of shear-links which are required for SNAP-2DX modelling can be calculated as follows:

$K_1 = R_y/\Delta_y = 35.56\mathrm{X}1000/0.0094 = 3783$ kN/cm, where $\Delta_y = (\tau_y/G)d$

Shear modulus $G = 26000$ MPa

$K_2 = (R_{max}-R_y)/(\ \Delta_{max}-\Delta_y\ ) = 33.56$ kN/cm, where $\Delta_{max} = 0.2d$

$\beta = d/t_w = 24 > 20$

Therefore, intermediate stiffeners of 10 mm thick are provided @120 mm c/c which meet the requirements of stability and stiffness.

A.3.2 Design of Diagonals

The buckling strength of diagonals should be at least equal to the maximum strength of shear-link, i.e.,

$2\ P_{br}\ Sin\theta = R_{max.}$, where $\theta = 28.53^0$

which gives $P_{br} = 121.23$ kN

Provide ISA 60X60X8 mm

$A = 896\ \mathrm{mm^2}$ $\qquad r_{min} = 18.0$ mm $\qquad l/r = 87.26$

$\sigma_{ac} = 93$ N/mm^2

Therefore, compressive strength of X-diagonals, P_{ac} =141.68 kN > P_{br}

Tensile strength of the X-diagonals, P_{st}= 190.4 kN

A.3.3 Design of Chords

Chords are designed as beam-Column elements as per IS:800-1984

Chords are designed to resist a shear equal to 25%of V_{req} =10.57 kN

Design moment for chords = 25% V_{req} $(L_{ss})/4$ = 7.9 kN m, where L_{ss} is length of the special segment = 3.0 m

Generally, chords share less than 25% of vertical shear V_{req}. As a result chords can be designed to a less strength, however, one must excercise care as small size section will decrease lateral stiffness.

Provide 2 ISA 55x55x8mm (back-to-back) for chords.

A = 1636 mm^2	M_p = 5.3 kN m	P_y = 409 kN
I_{xx} = 90x10^4 mm^4	Z_p = 31173 mm^3	

Check:

b/t = 55/8 = 6.875 < $0.31\sqrt{(E/F_y)}$ OK

Axial force in chords = M/(depth of girder)=86.85/1.5=57.9 kN.

P/P_y = 57.9x1000/409 = 0.141<.15 OK

$P_e = \pi^2 E\, A_S/(L/r_y)^2$= 96.51 kN

$\lambda_o = P_y/P_e$ =2.06

$(0.6+0.4\beta)/\lambda_o$ = (0.6+0.4x0.516)/2.06 >P/P_y OK

A.4 MEMBERS OUT SIDE THE SPECIAL SEGMENT

A.4.1 Amplified Vertical Shear (V_{ss})

The maximum vertical shear in the special segment V_{SS}' at the maximum deformation includes the effect of strain hardening and expected yield strength of special segment, i.e.,

$$V_{ss} = 3.4\Omega\left(\frac{M_{nc}}{L_s}\right) + 0.07EI\left(\frac{L-L_p}{L_p{}^2L_s}\right) + R_{max}$$

M_{nc} is flexural strength of chords

$M_{nc}=1.18\ M_P[1-(P/P_y)]$ = 1.18x5.3x(1-0.141) =5 .36 kN m.

L_P = Length of the special segment = 3.0 m

$L_S = 0.9\ L_P$ =2.7 m.

$$V_{ss} = 3.4 \times 1.1\left(\frac{5.36}{2.7}\right) + 0.07 \times (2 \times 10^5) \times 440\left(\frac{12000-3000}{3000^2 2700}\right) + 115.8$$

= 7.42+2.51+115.8 = 125.73 kN.

A.4.2 Total Vertical Shear in the Special Segment

Vertical shear in the special segment due to lateral loads V_l is obtained as follows

$$Vss = \mp V_l + V_g \Rightarrow V_l = \pm V_{ss} + V_g$$

where Vss is a constant since it depends on maximum deformation in the special segment (it can be +ve or -ve depending on the direction of lateral force). Different gravity load cases result in different values of V_g, and consequently different V_l. To calculate these forces under load combination 1.3 (D+L)+V_l, gravity loads and lateral loads are applied separately and then the two cases are superimposed. Both balanced and unbalanced cases of gravity loads are combined with lateral loads acting on the frame in either direction.

For balanced case:

- Lateral loads applied from left to right $V_l = V_{ss} - V_g$
- Lateral loads applied from right to left $V_l = V_{ss} + V_g$

For unbalanced case:

- Lateral loads applied from left to right $V_l = V_{ss} - V_g$
- Lateral loads applied from right to left $V_l = V_{ss} + V_g$

A.4.3 Chords of the Truss Girder

Load Case (a):

$F_{chord}=M/d=294.52/1.5=196.35$ kN

Load Case (b):

$F_{chord}=225.2+47.206/1.5=181.62$ kN

Load Case (c):

$F_{chord} = F_{cg}+F_{cl}$

F_{cg} = Max. force in chords outside special segment due to gravity loads

= 225.2 kN

F_{cl} = Max. force in chords outside special segment due to V_l loads

$= (0.5L)V_l/d = 6\text{x}161.85/1.5$

Therefore, F_{chord} =797.5 kN.

Hence, Design load F_{chord} =797.5 kN

Provide 2ISA 80x80x12mm

A =3562 mm^2 I_{xx} = 203.8x10^4 mm^4

M_p=16.57 kN m P_y = 890.5 kN.

Check:

$b/t = 80/12 = 6.67 < 0.31\sqrt{(E/F_y)}$ OK

A.4.4 DIAGONALS

F_{br} = Vertical shear/Sinθ and θ=45^{o}

Load Case (a):

F_{br}= 142.8/Sin45 =202.0 kN

Load case (b):

F_{br} = (109.2+7.868)/Sin45=165.56 kN

Load case (c):

$F_{br} = F_{cg}+F_{cl}$ = (109.2+161.85)/Sin45 = 383.30 kN

Hence, Design Load F_{br} = 383.0 kN

Provide ISA 100x100x12 mm

A= 2259 mm^2 r_{min}= 30.3 mm

$l/r_{min} = 70$ $\sigma_{ac} = 112\ N/mm^2$

$P_{br} = 430.1$ kN $P_{st} = 480.04$ kN.

Check:

$b/t = 100/12 = 8.33 < 0.31\sqrt{(E/F_y)}$ OK

A.4.5 Verticals

$F_{ver} = F_{dia}\ Sin\theta$ $\theta = 45^{O}$

Load Case (a):

$F_{ver} = 142.8$ kN

Load case (b):

$F_{ver} = (109.2+7.868) = 117.07$ kN

Load case (c):

$F_{ver} = F_{cg}+F_{cl} = (109.2+161.85) = 271.05$ kN

Hence, Design force $F_{ver} = 271.05$ kN

Provide ISA 75x75x10mm

$A = 1402\ mm^2$ $r_{min} = 22.6$ mm

$l/r_{min} = 66.37$ $\sigma_{ac} = 115.6\ N/mm^2$

$P_{br} = 275.6$ kN $P_{st} = 297.9$ kN.

Check:

$b/t = 75/10 = 7.5 < 0.31\sqrt{(E/F_y)}$ OK

A.4.6 Columns

Interior Column:

Load case (a):

$M = 86.02$ kN m

$P = 124.47 + 59.13 = 183.6$ kN

Load case (b):

$M = (177.6-111.8) + 2(47.21) = 160.132$ kN m

$P = 95.18 + 45.22 = 140.4$ kN.

Load case (c):

$M = (177.6\text{-}111.8) + 2\text{x}6\ (161.85) = 20008$ kN m

$P = 140.4$ kN.

Exterior column:

Load case (a):

$M = 294.52$ kN m

$P = 142.8$ kN

Load case (b):

$M = 225.52 + 47.21 = 272.43$ kN m

$P = 95.18 + 45.22 = 140.4$ kN.

Load case (c):

$M = 177.7 + 6\text{x}161.85 = 1148.8$ kN m

$P = 257.03$ kN.

Provide ISMB 600 + 320x10 mm plates.

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TABLES

Table 4.2 Lateral design seismic force calculations for study building 2

Calculation of design base shear

Base shear, $V = \frac{(z/2)(Sa/g)}{(R/I)} W$

$Z = 0.36$ (Zone V)

$I = 1.0$ (office building)

$h = 18$ m

Period $T = 0.085\ (h)^{0.75}$ $T = 0.085\ 18^{0.75} = 0.743$ s

Therefore $S_a/g = 1.75$ (soil site) from

$R_w = 5$

Therefore, $V = \frac{0.36x1x1.75}{10} W = 0.063\ W$

Dead load: Roof = 2.0 kN/m^2 and Floors = 4.0 kN/m^2

Live load: Roof = 1.5 kN/m^2 and Floors = 3.0 kN/m^2

W= 2.0x36x48+(4+.25x3)35x48 = = 28080 kN

V= 1769.04 kN

$V_{\text{per frame}}$= 442.25 kN

Add accedental torsional shear = $1 + \frac{0.1x48x18}{2[18^2+6^2]} = 0.12$

Total shear =1.12 V= 495.32 kN

Vertical Distribution of design seismic shears

Level	h_x (m)	w_x (kN)	$\frac{w_x h_x^2}{\sum w_x h_x^2}$	F_x (kN)	V_x (kN)
Roof	18	96	0.325	160.98	160.98
3	13.5	228	0.434	214.97	375.95
2	9.0	228	0.193	95.60	471.55
1	4.5	228	0.048	23.77	495.32

Table 4.3 Proportioning of Shear-Link for study building 2.

Storey	Design vertical shear (kN)	Design shear strain γ_d	Design shear stress (Mpa)	Size of shear link (mm)		
				l	d	t_w
First and second	69.75	0.1[a]	20.323 [b]	450	160	10
Third and fourth	32.25	0.1[a]	20.323 [b]	400	160	5

a) *The shear strain correspond to limiting storey drift of 0.4%*
b) *The design stress corresponds to yield stress of 3003-O Aluminium alloy* $\tau_y = 0.57\,\sigma_{0.2}$, $\sigma_{0.2} = 35.2$ *MPa*

Table 4.4 Members selected for the four storey building frame (study building 2).

Special segment			Out side the special segment			Columns
Shear-Link	Diagonals	Chords	Chords	Diagonals	Verticals	
450x10 mm	ISA 80x80x10 mm	2 ISA 75x75x10 mm	2 ISA 150x150x 18 mm	ISA 130x130x15 mm	ISA 110x110 x15 mm	ISMB 600+320x 40 mm plates

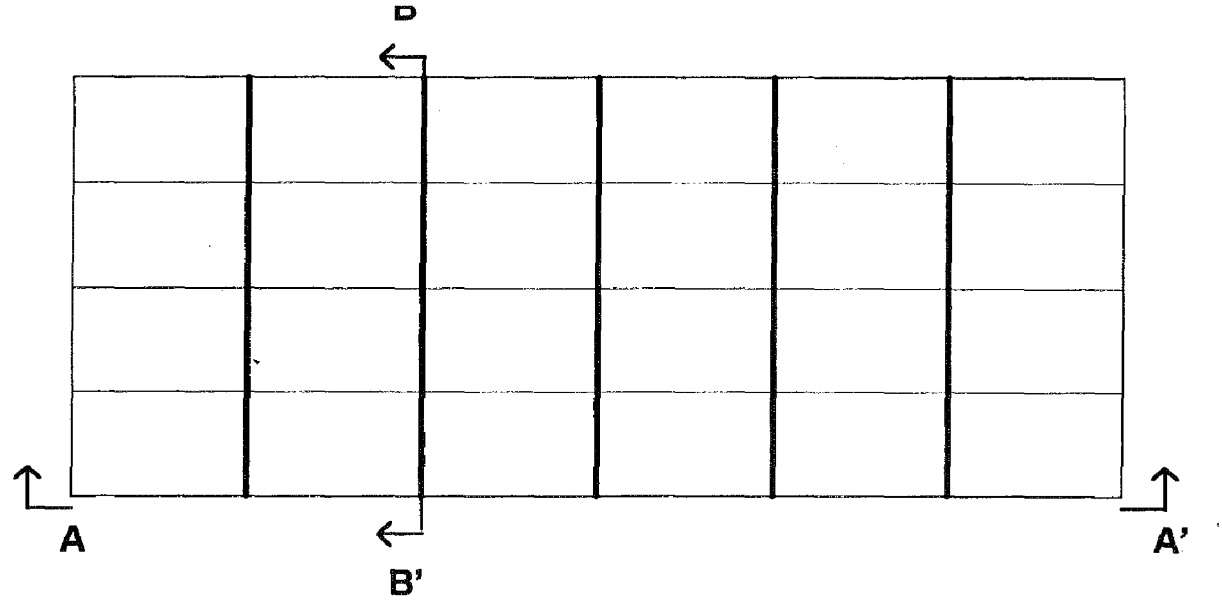
Table 6.1 Response envelopes for STMF(X-Diag.) and Shear-Link TMF of study building 1.

Ground motion	Base shear (kN)		Storey drift		Shear strain in link (mm/mm)
	Shear-Link TMF	STMF (X-Diag.)	Shear-Link TMF	STMF (X-Diag.)	
EL Centro (0.347g)	52.83	92.7	0.443	0.98	0.08
Miyagi-Ken-Oki (0.51g)	88.7	98.3	0.719	2.82	0.143
Newhall (0.59g)	116.4	140.1	0.99	1.79	0.17
Sylmar (0.84g)	132.47	137.5	0.915	1.34	0.186

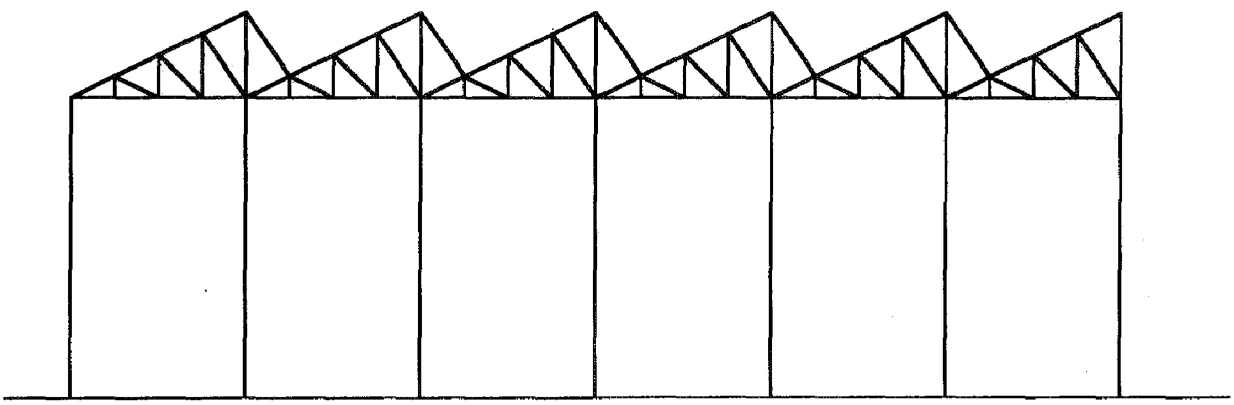
Table 6.2 Response envelope values for study building 2.

Floor	Storey shears (kN)			Rel. Storey drifts (%)		
	Design	El Centro (0.347g)	Sylmar (0.84g)	Design	El Centro (0.347g)	Sylmar (0.84g)
4	26.83	49.64	91.168	0.40	0.080	0.20
3	89.49	142.57	238.37	0.40	0.15	0.31
2	168.1	212.18	325.85	0.40	0.23	0.45
1	250.6	246.31	395.56	0.40	0.37	0.68

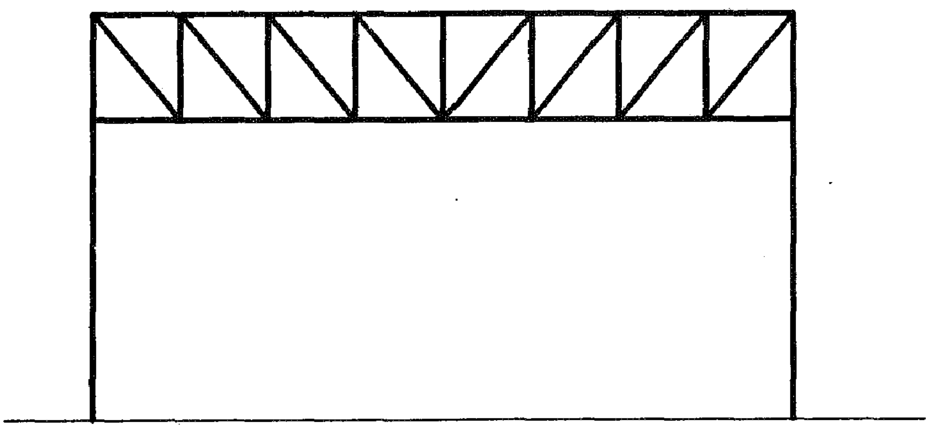
FIGURES



PLAN

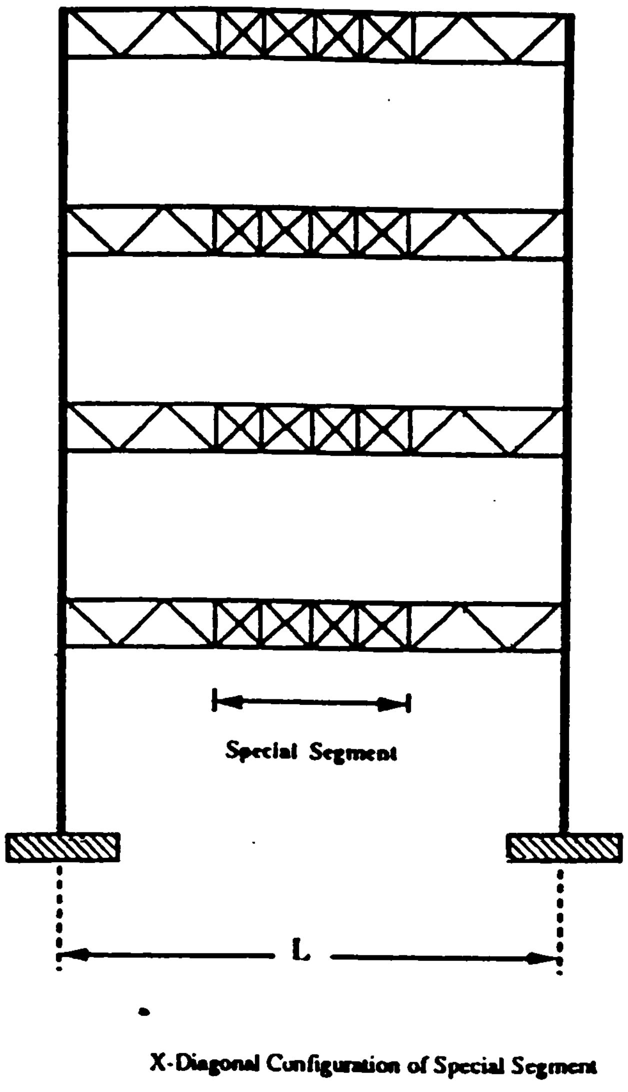


A-A': Roof Truss



B-B': Open Web Truss Moment Frame

Figure 1.1. General layout of a large span industrial structure



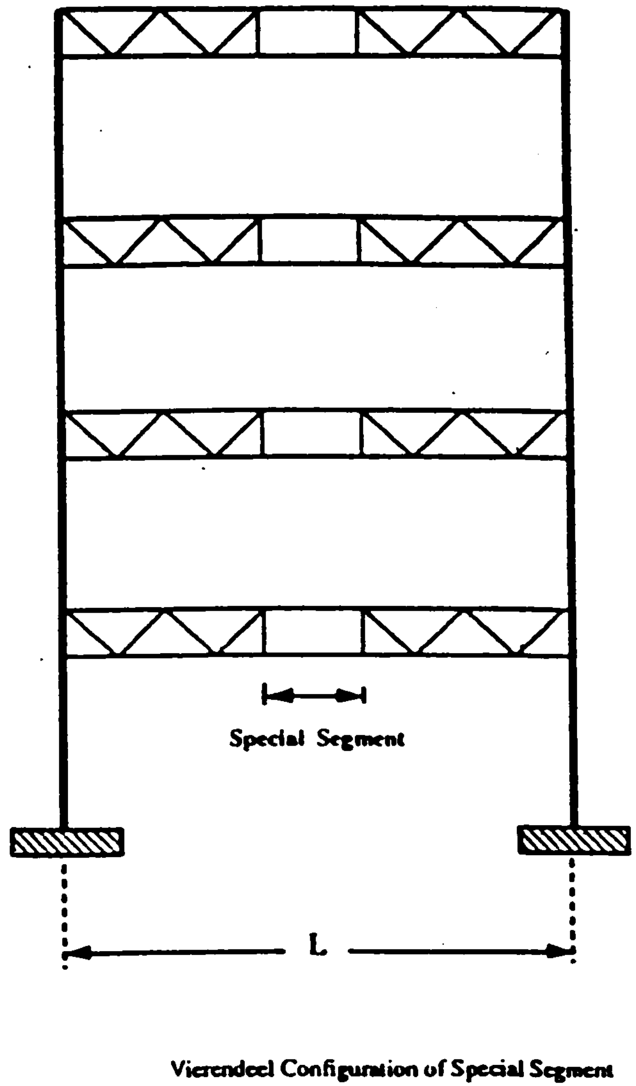


Figure 1.2 Special truss moment frames (STMF)

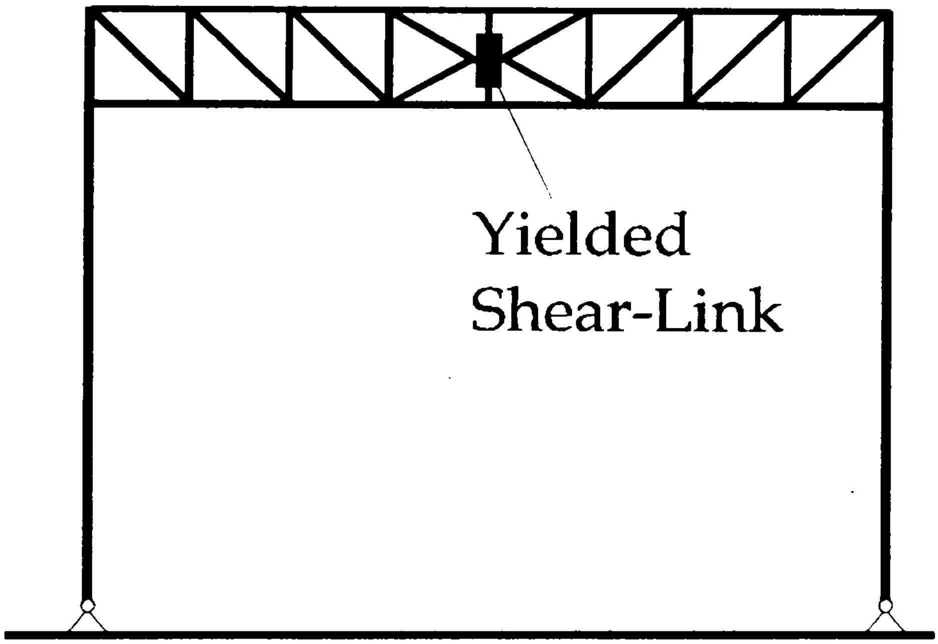


Figure 1.3 Shear-Link Truss Moment Frame

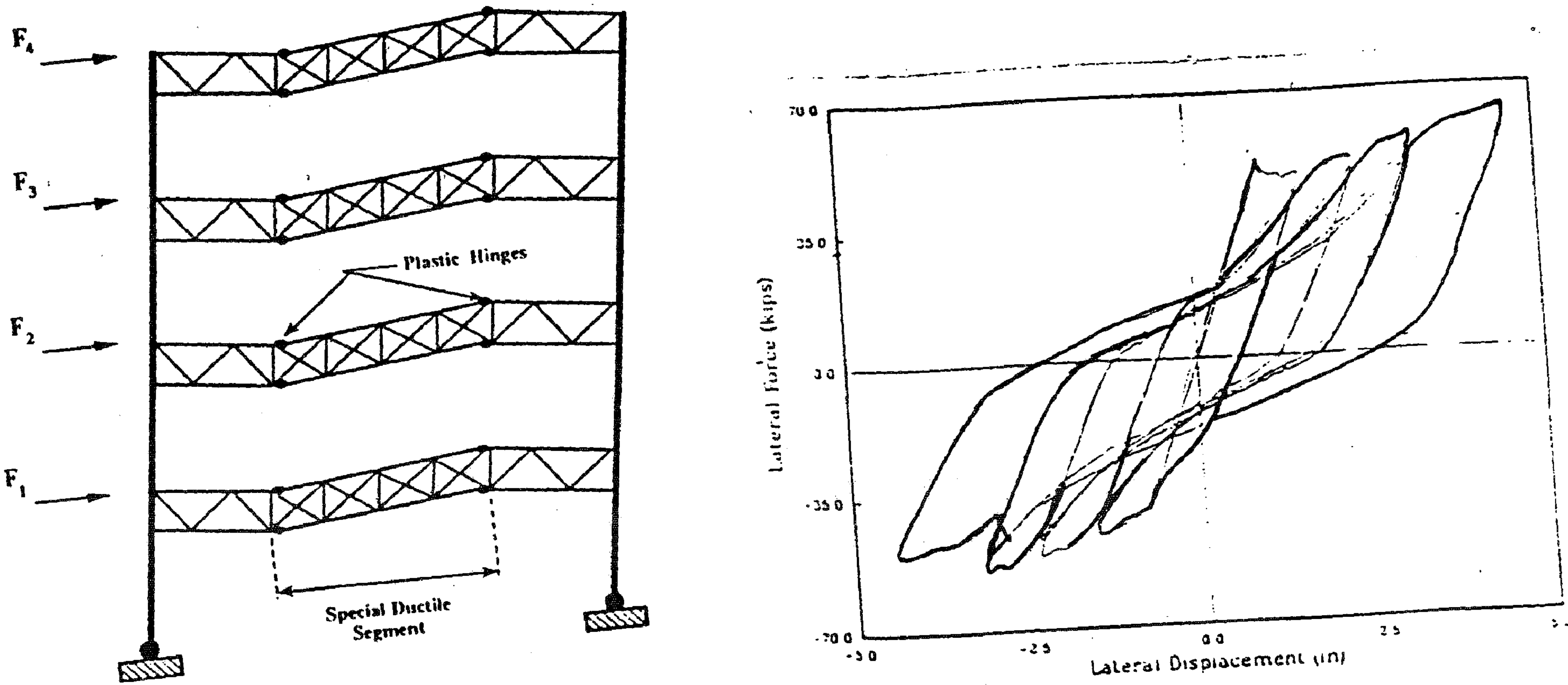
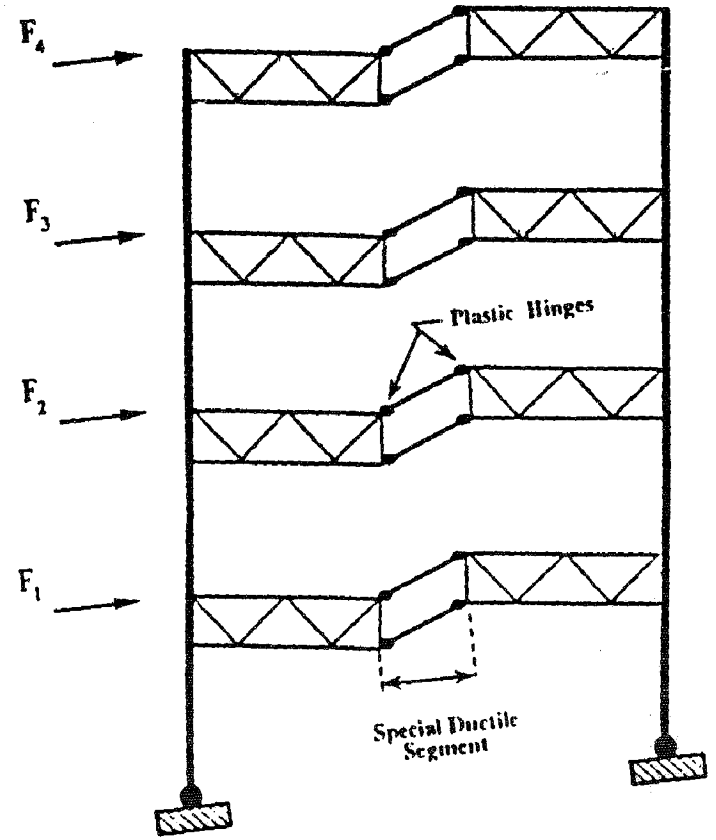


Figure 2.1 Yield mechanism and typical hysteretic loops of STMF with X-Diagonals (Itani & Goel 1991)



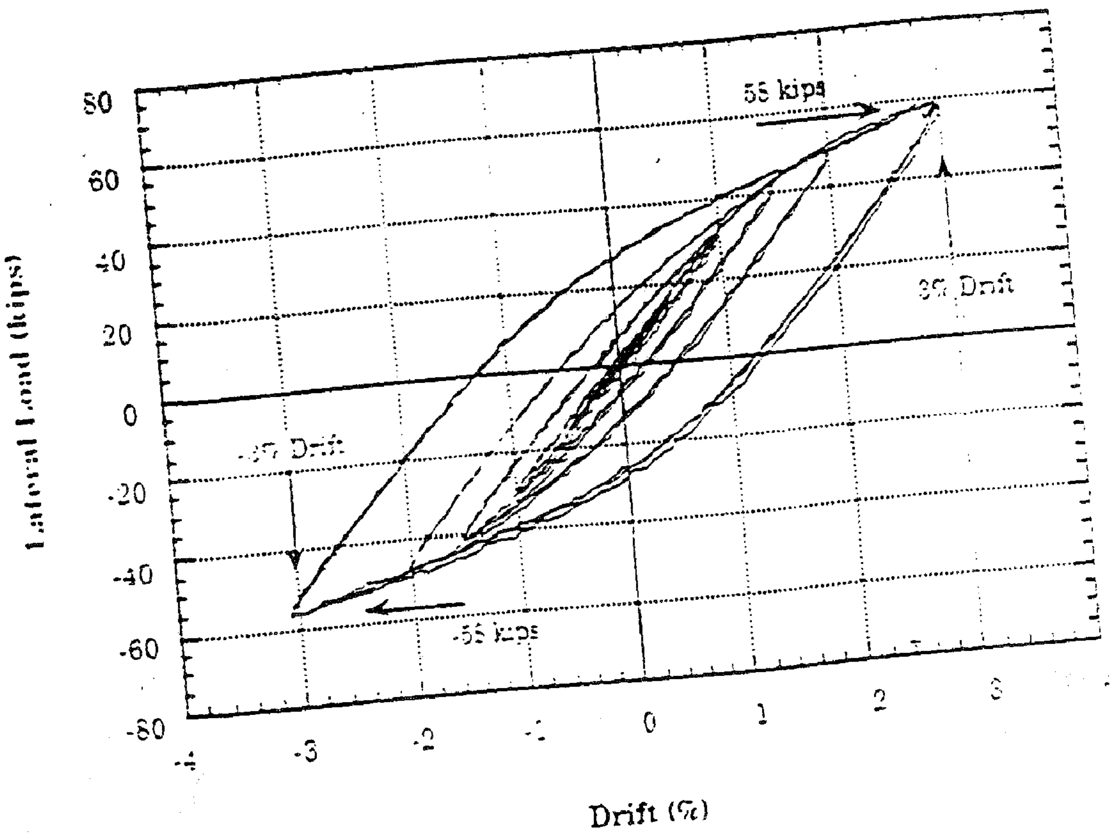
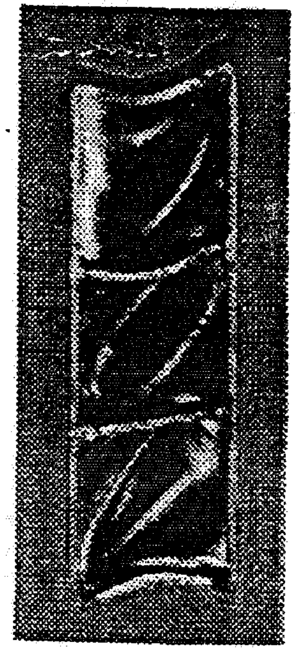


Figure 2.2 Yield mechanism and typical hysteretic loops of STMF with Vierendeel segment (Basha & Goel 1994)



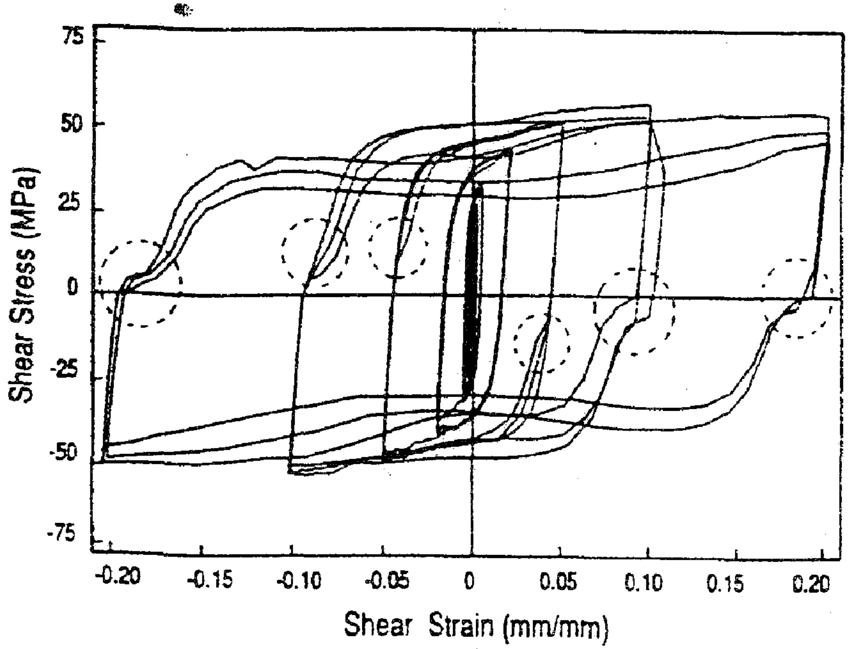


Figure 2.3 Specimen of yielded shear-link and its hysteretic loops (Rai & Wallace 1998)

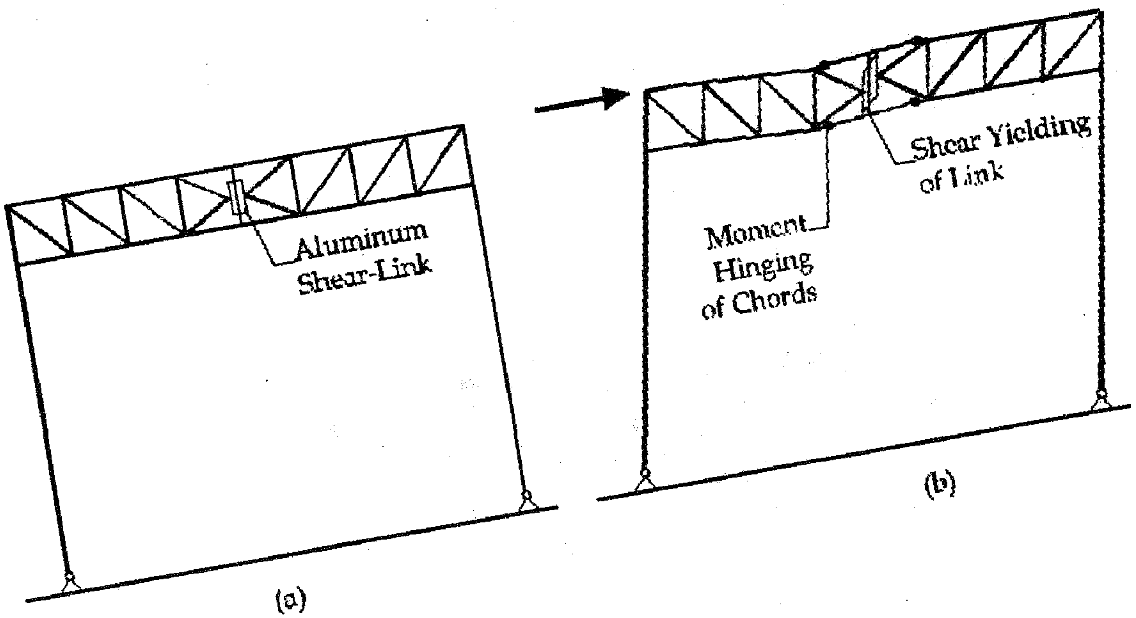
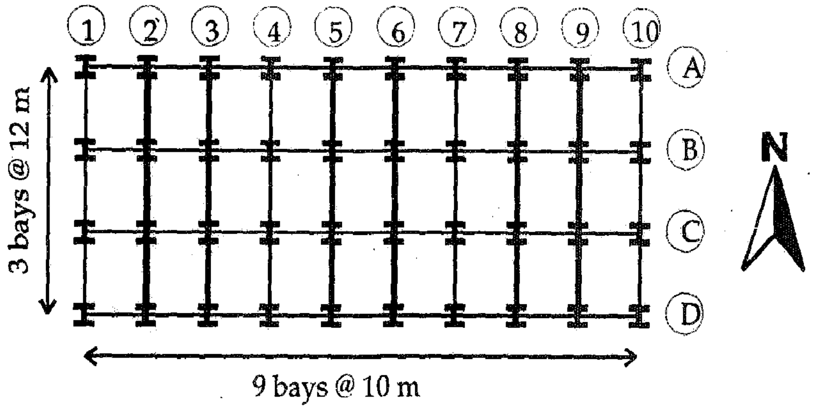
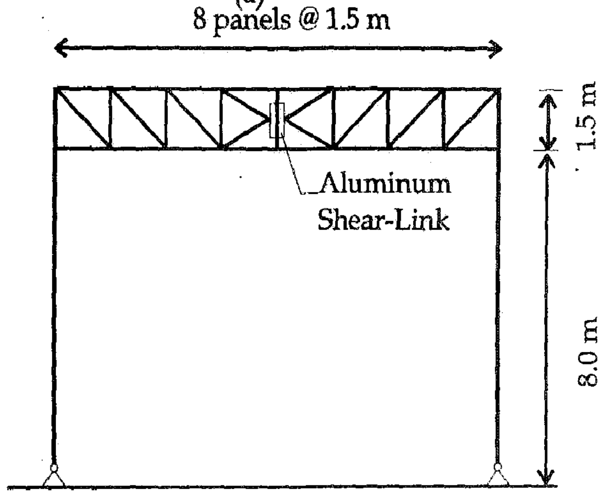


Figure 3.1 (a) Shear-Link Truss Moment Frame and (b) its expected yield mechanism (Rai & Prasad 1998)



(a)



(b)

Figure 4.1 (a) Plan of study building 1 and (b) elevation of interior bay frame

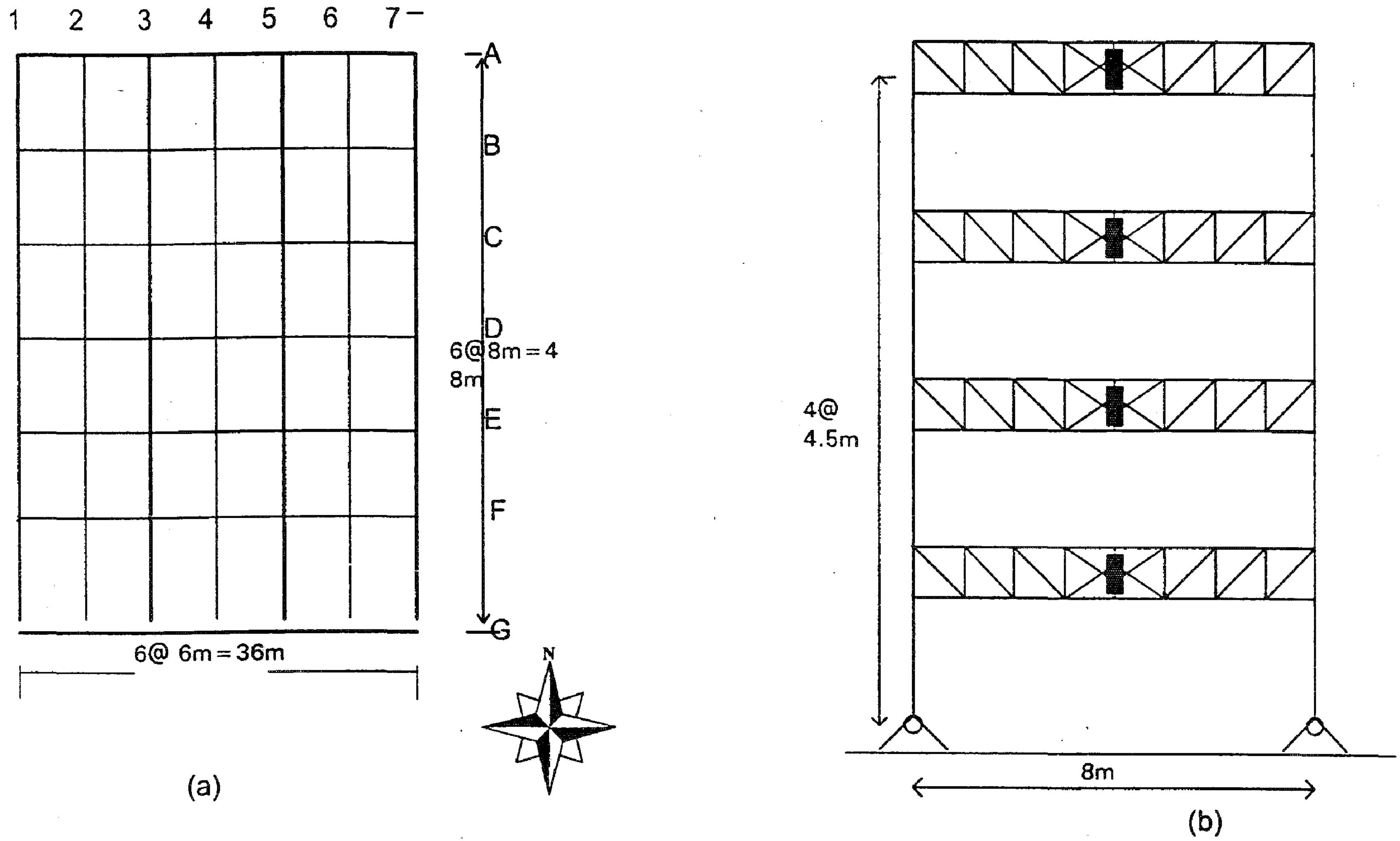


Figure 4.2. (a) Plan of study building 2 ,(b) Elevation of typical bay frame in N-S direction

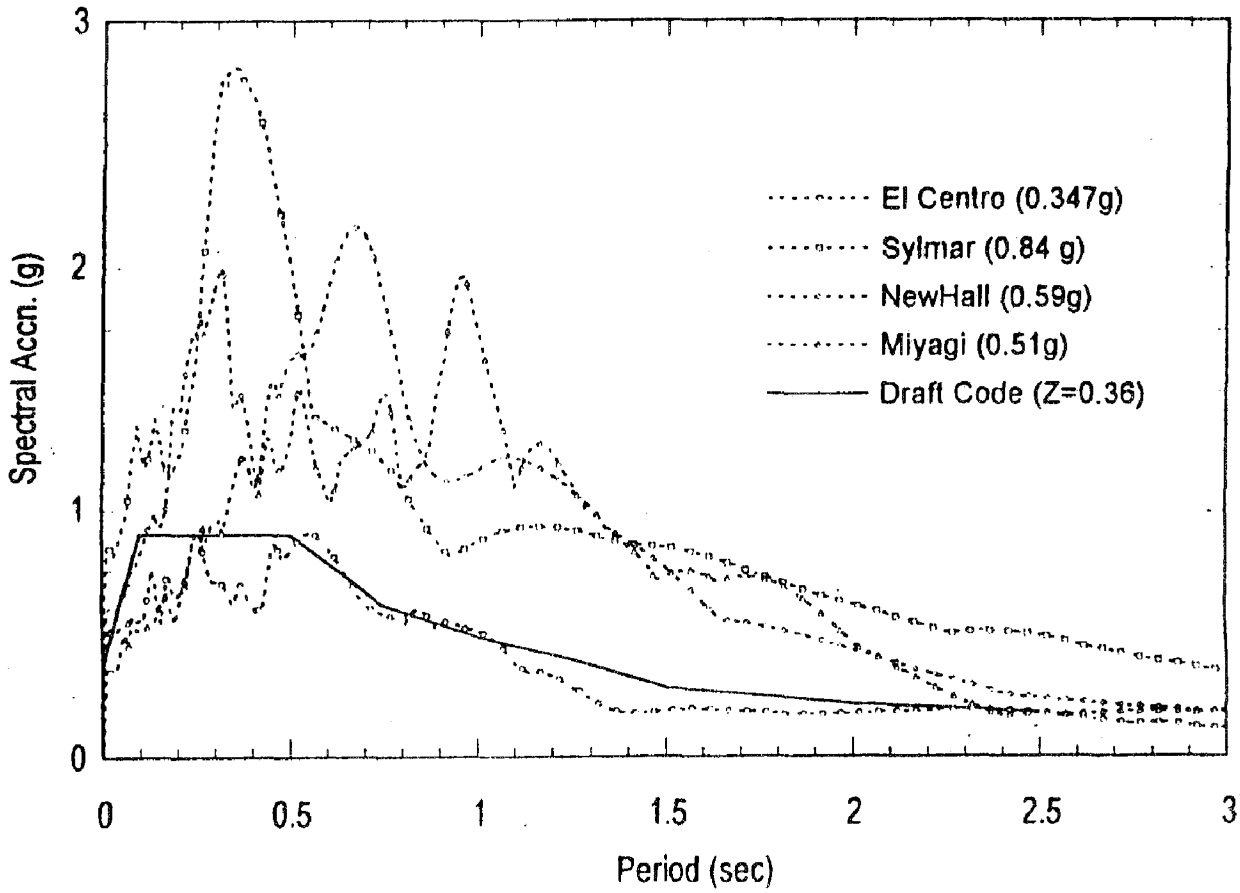
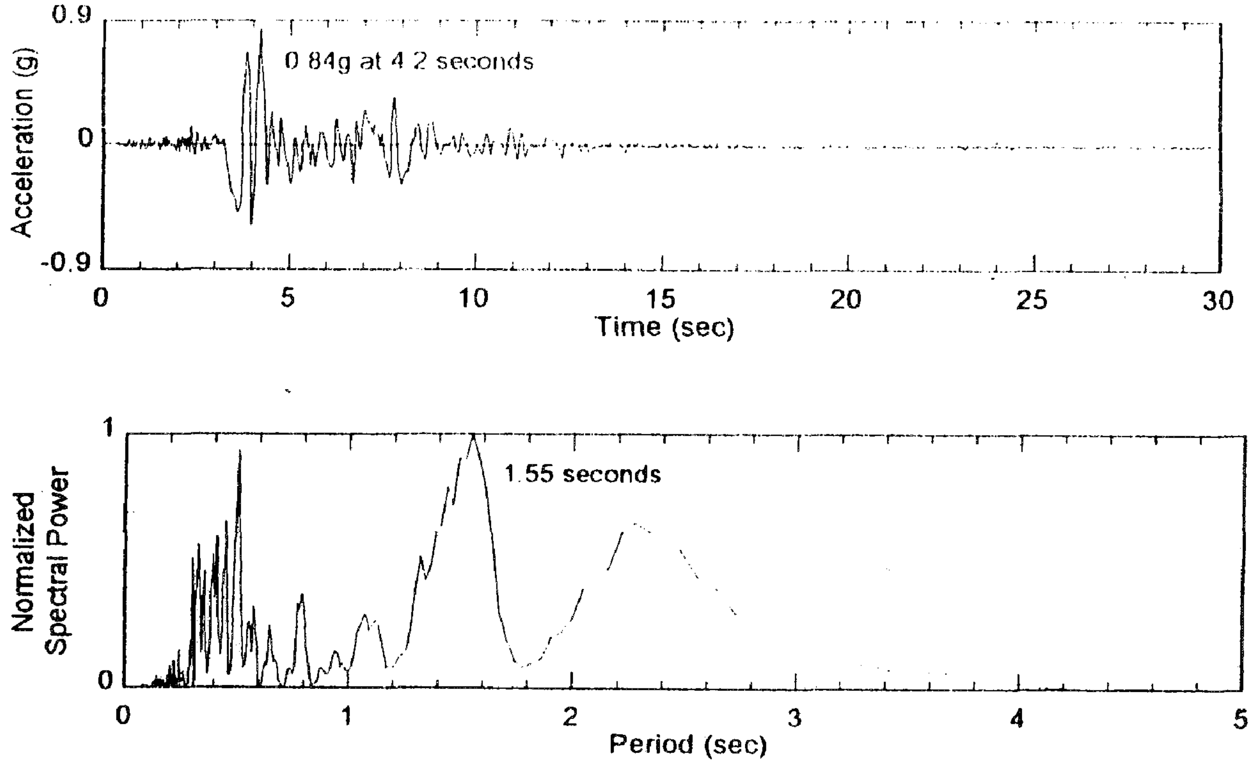
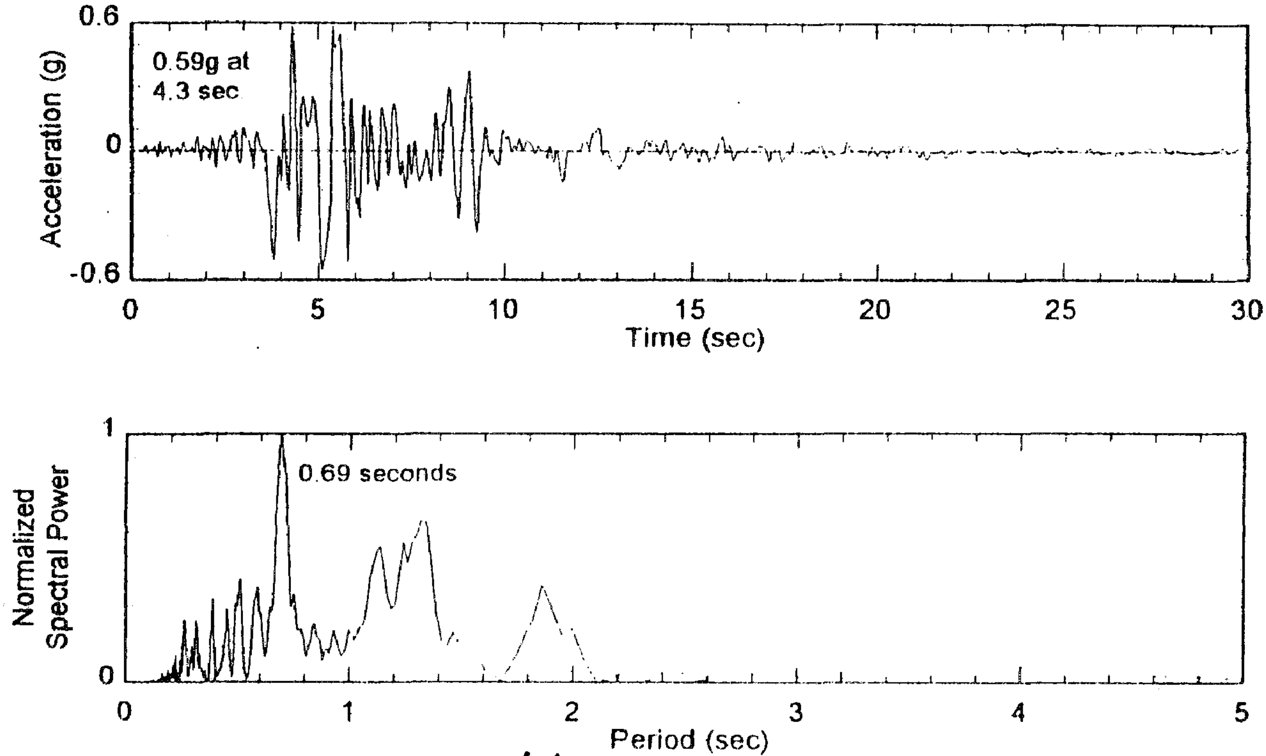


Figure 4.3 (a) Acceleration resptra of ground motions used in this study



(a)



(b)

Figure 4.3 (b) Time histories and Fourier spectra of (a) Sylmar and (b) Newhall ground motions

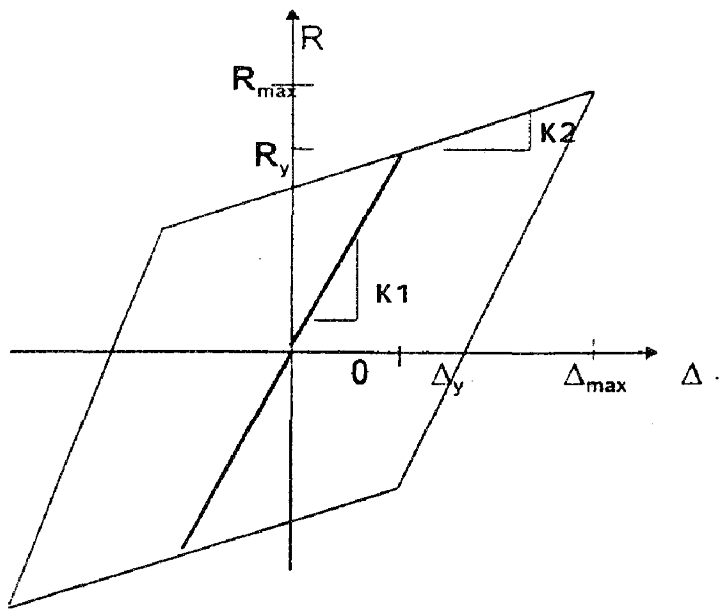


Figure 4.4 Bilinear hysteretic model of shear-link used in the study

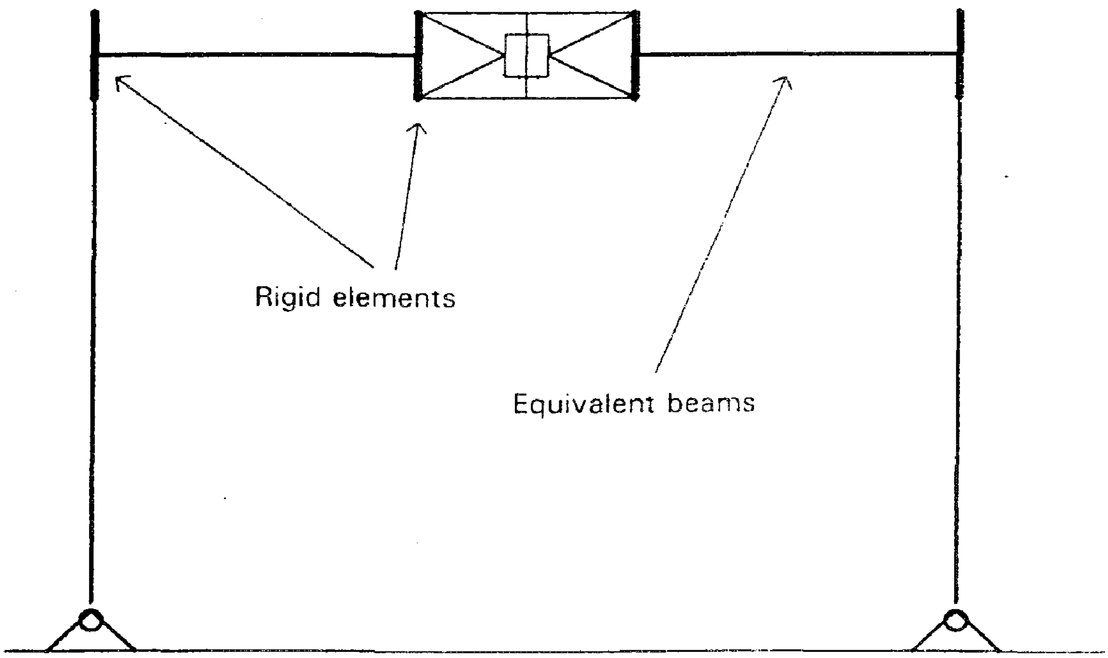
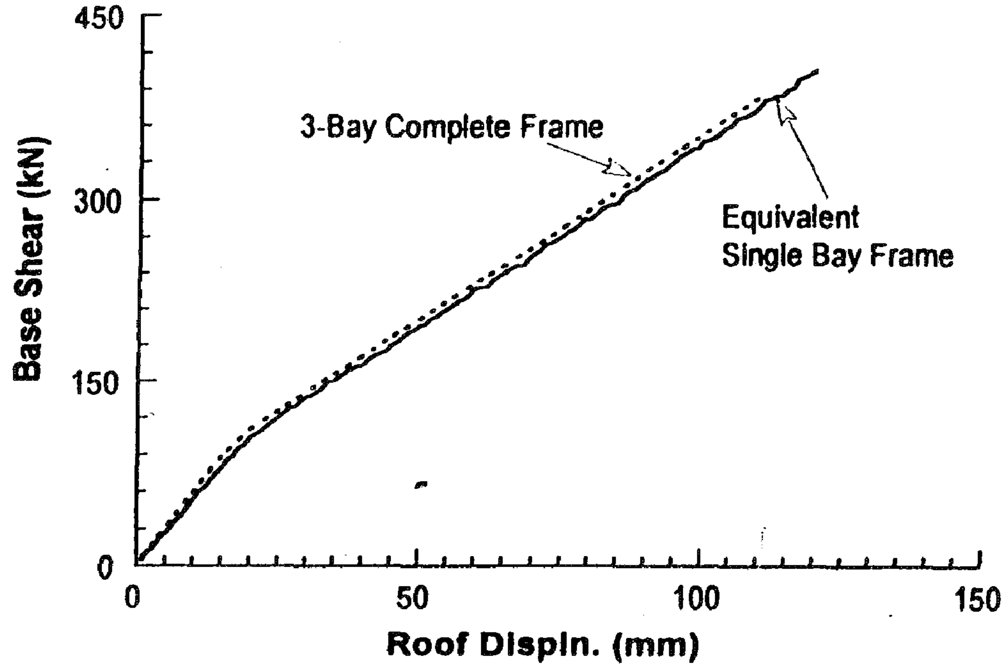
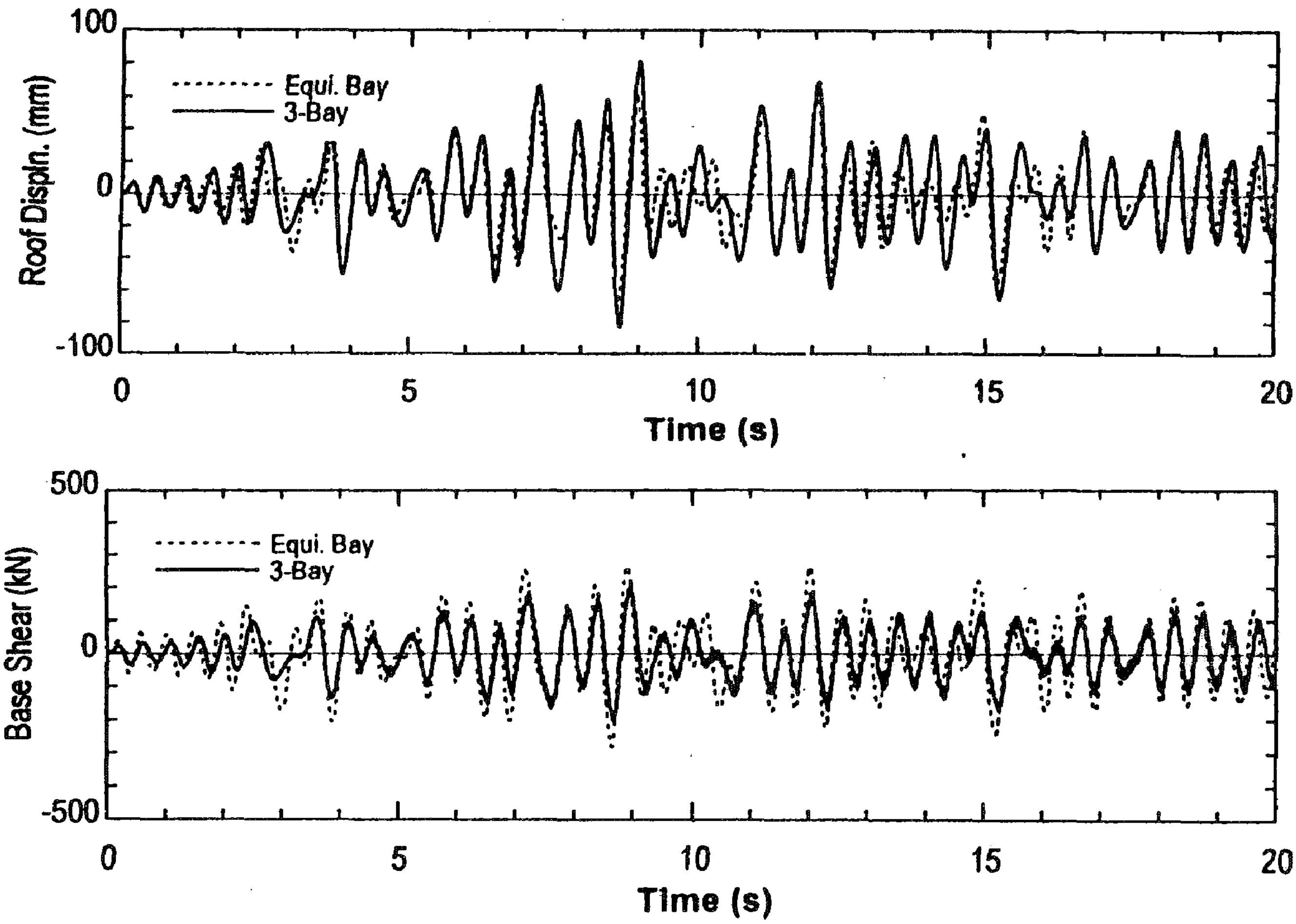


Figure 4.5 Equivalent single bay model of Shear-Link TMF

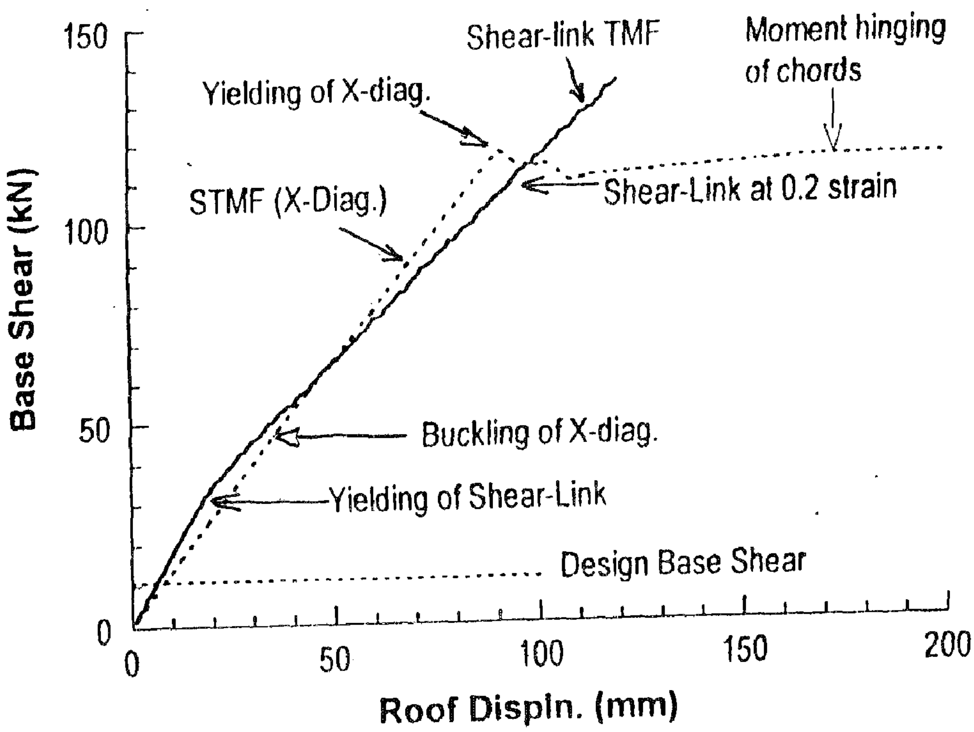


(a)



(b)

Figure 4.6 Comparative response curves for 3 bay complete frame and equivalent single bay frame model (a) Static Pushover curve and (b) Time history response



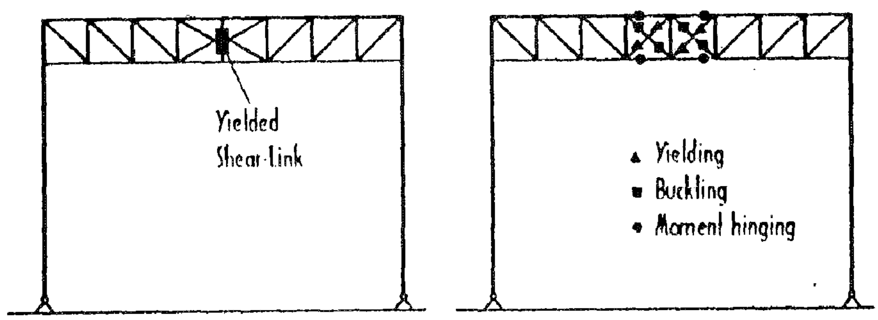


Figure 5.1 Pushover curve for Shear-Link TMF and STMF(X-Diag.)

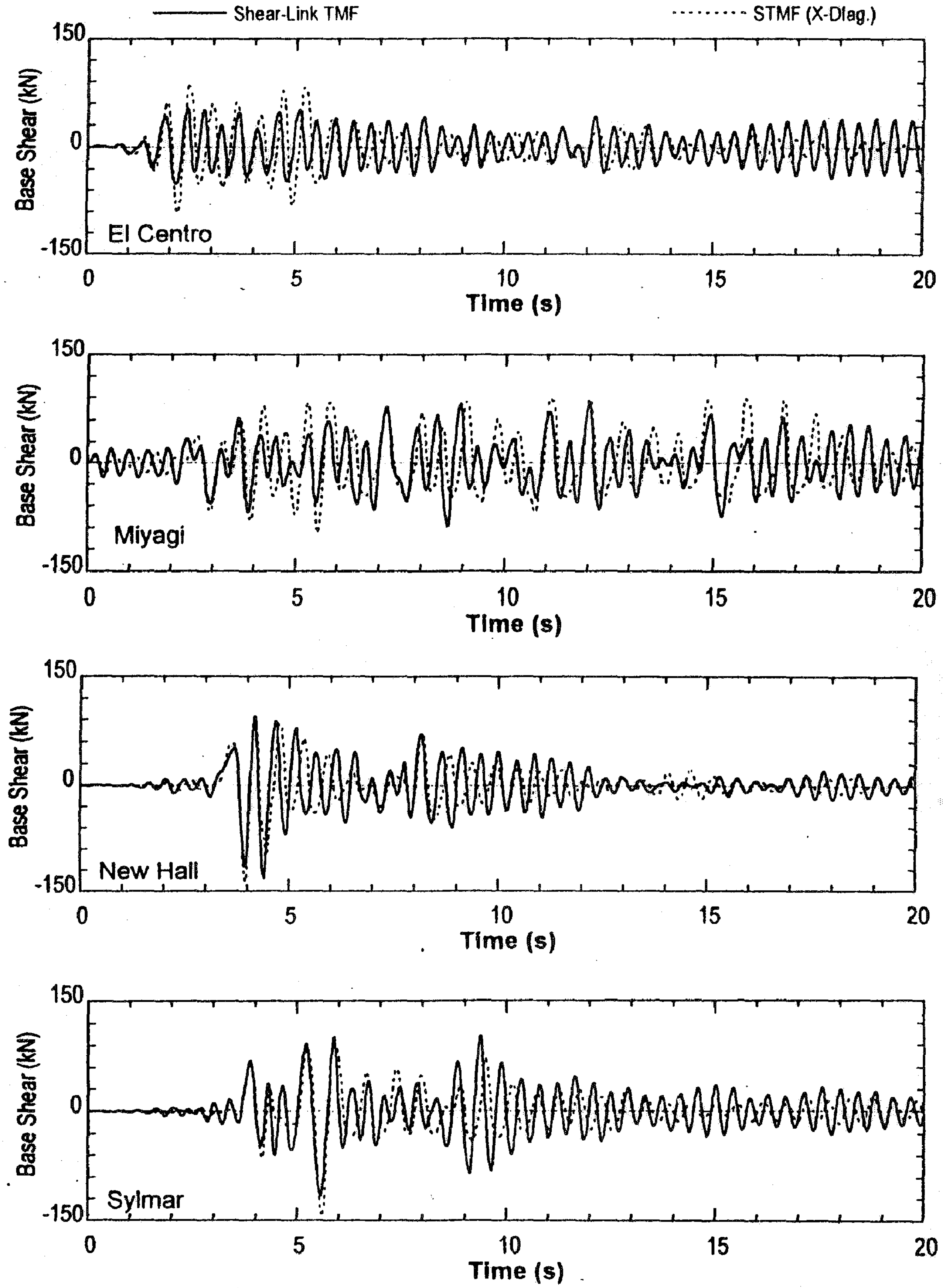


Figure 5.2 Base shear response of Shear-Link TMF and STMF (X-Diag.)

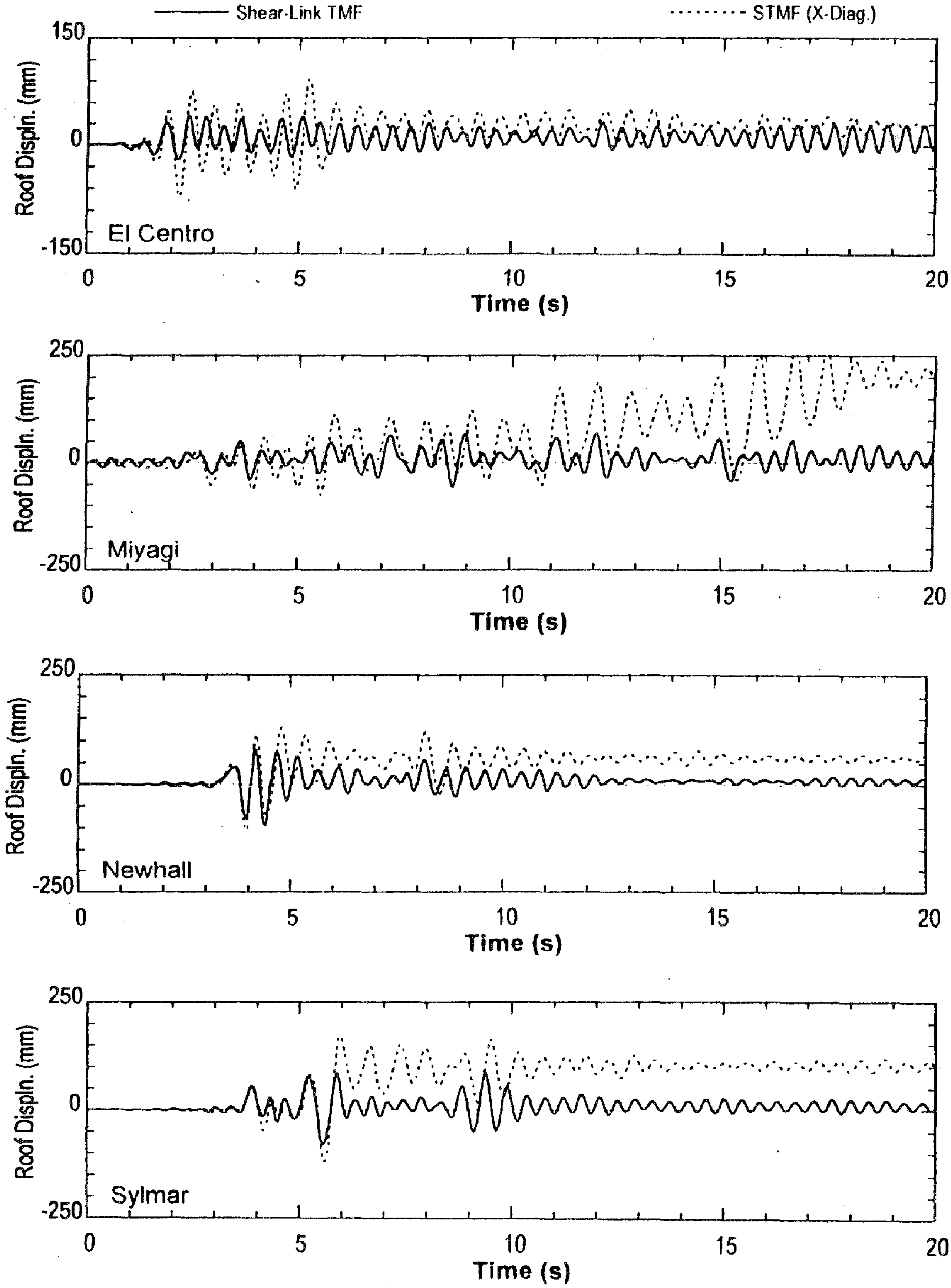


Figure 5.3 Storey displacement response of Shear-Link TMF and STMF (X-Diag.)

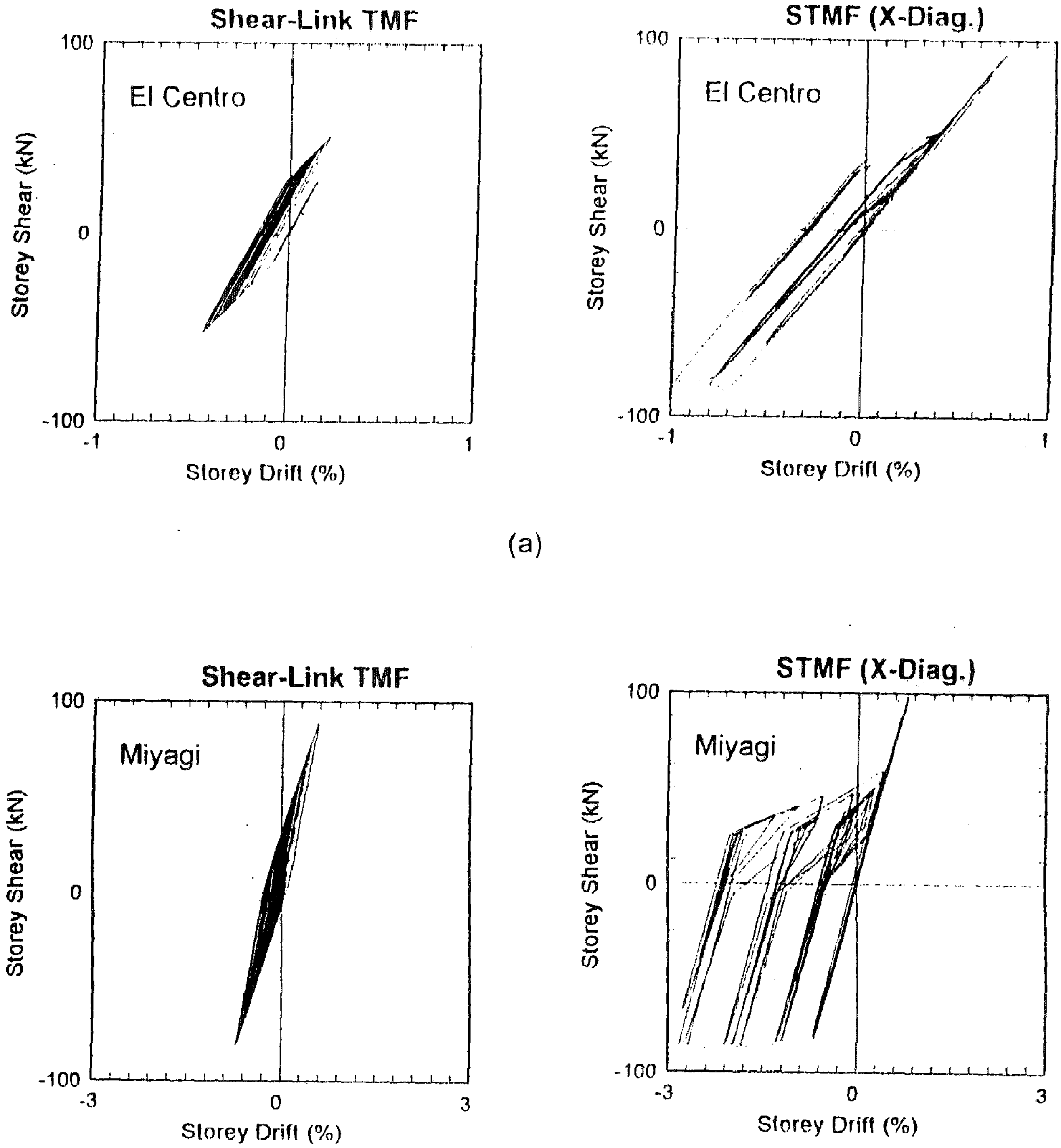
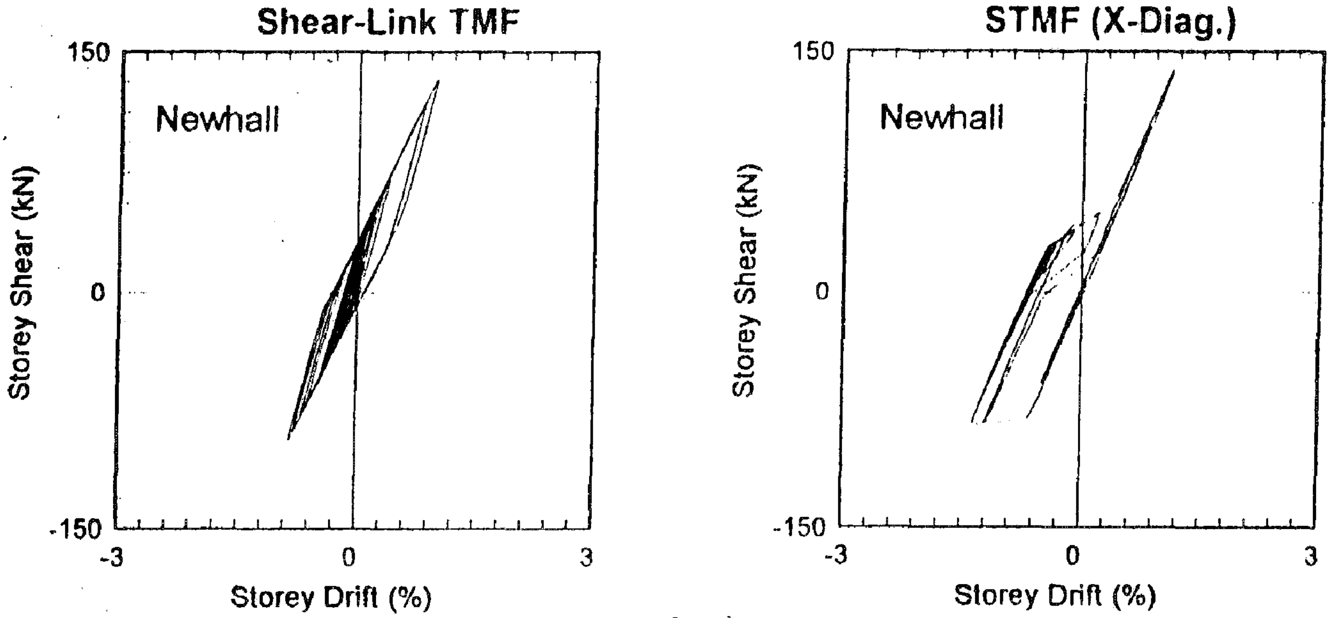
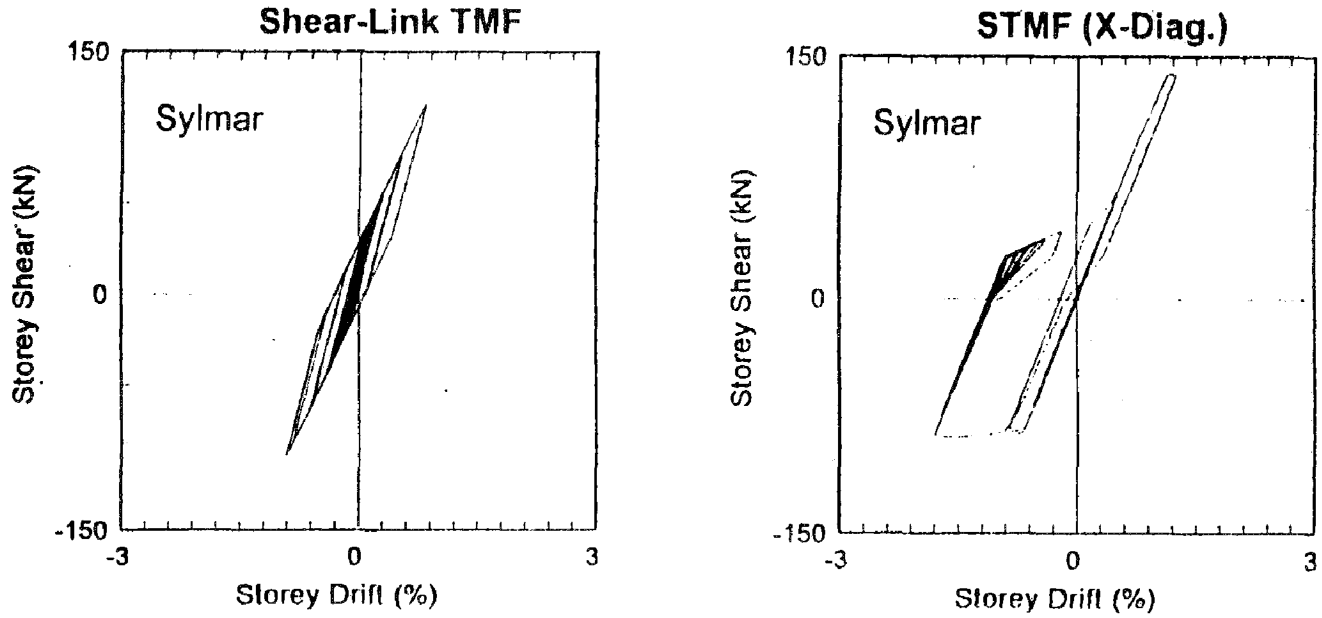


Figure 5.4 Hysteretic response of Shear-Link TMF and STMF (X-Diag.)



(c)



(d)

Figure 5.4 Hysteretic response of Shear-Link TMF and STMF (X-Diag.)

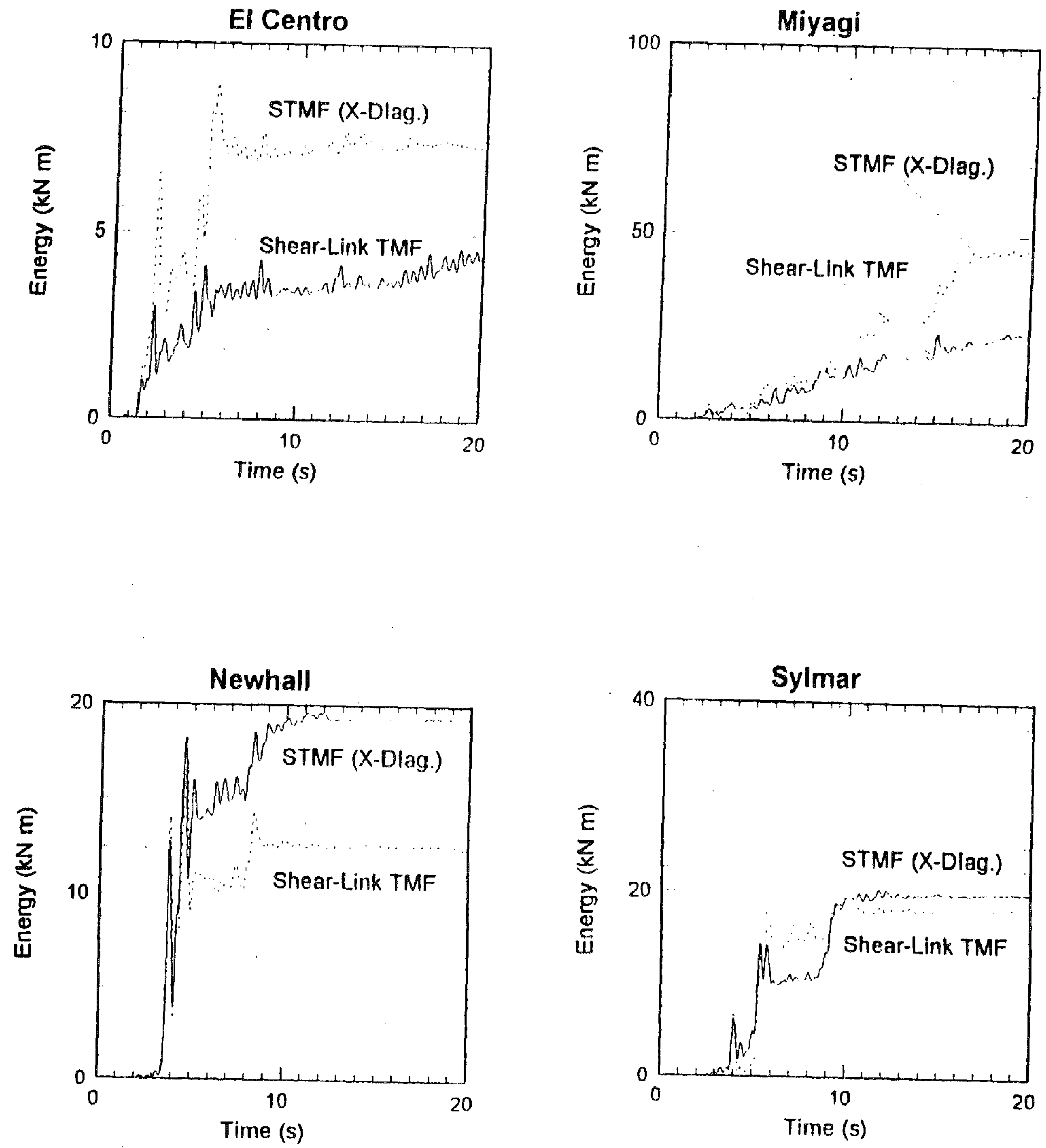


Figure 5.5 Energy time histories of Shear-Link TMF and STMF (X-Diag.)

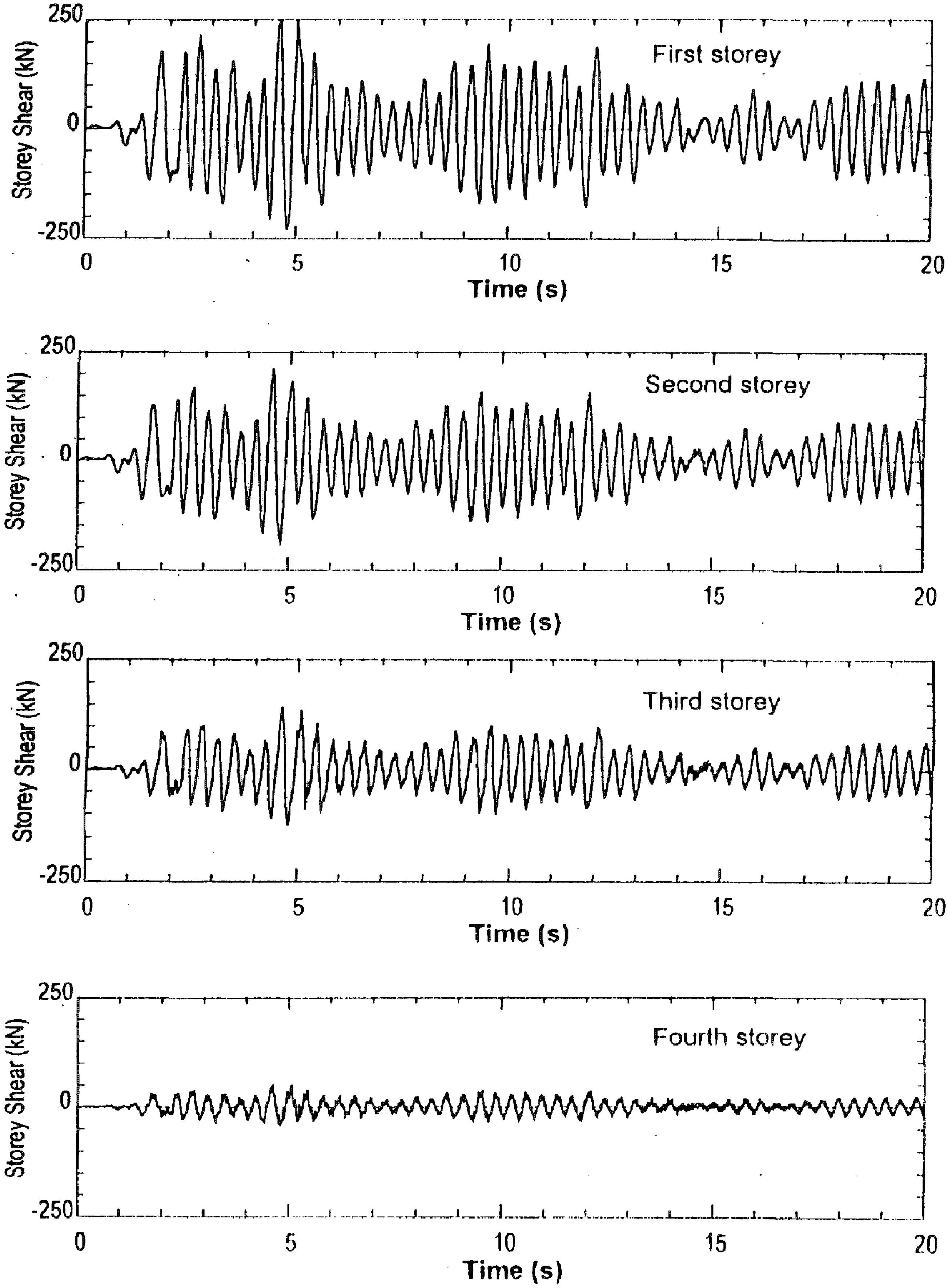


Figure 5.6 Storey shear response of four storey Shear-Link TMF for El Centro ground motion

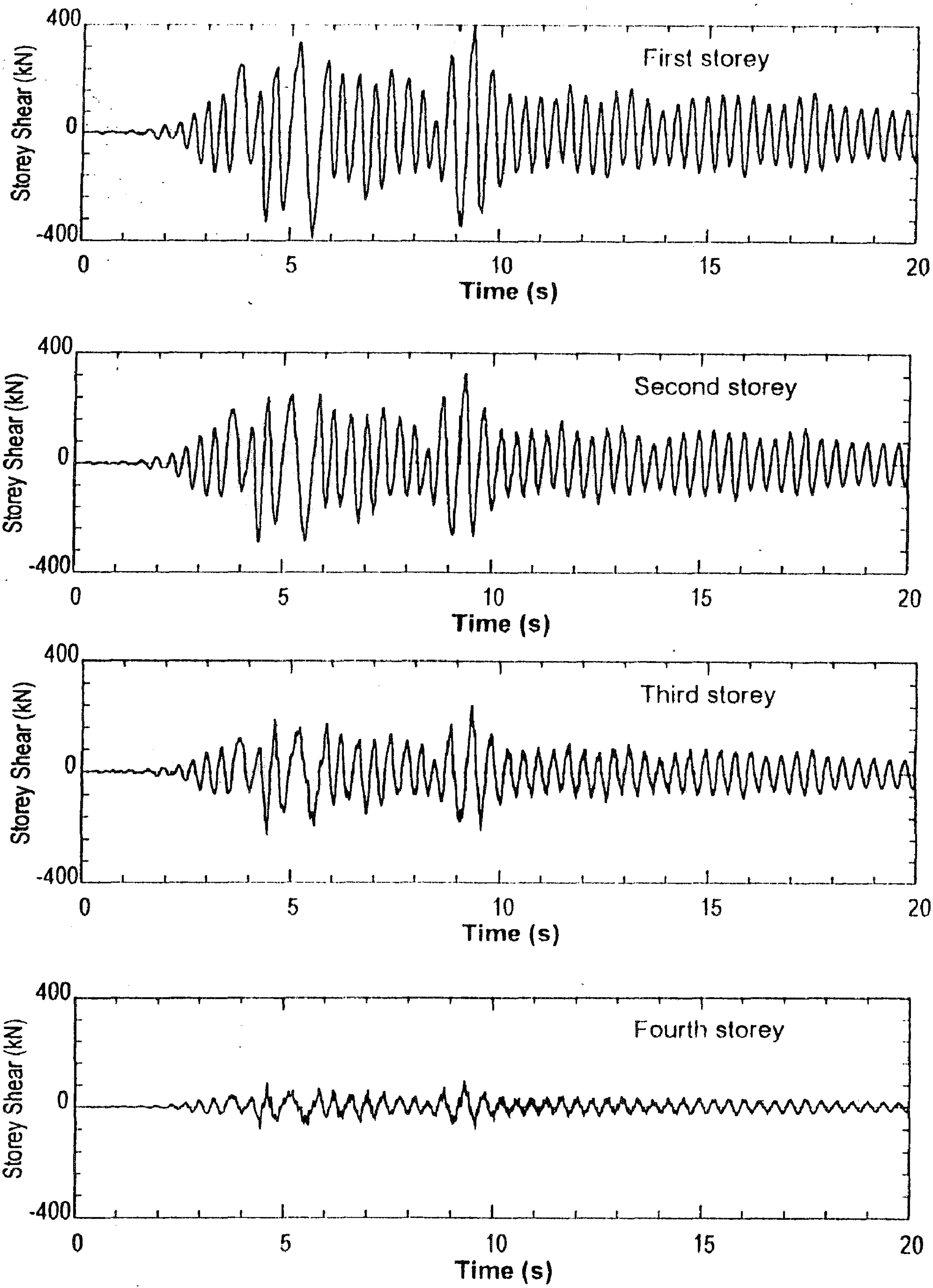


Figure 5.6 Storey shear response of four storey Shear-Link TMF for Sylmar ground motion

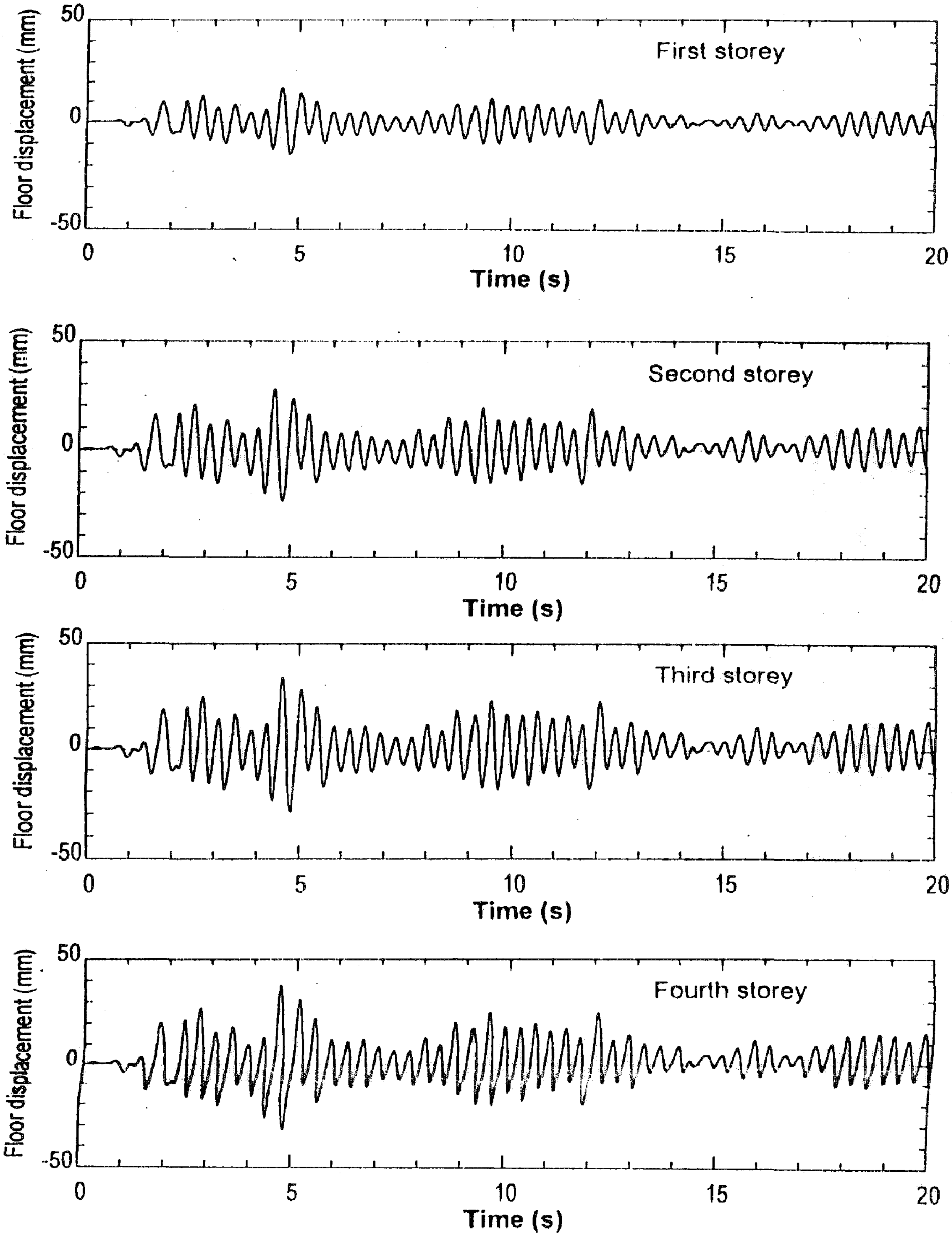
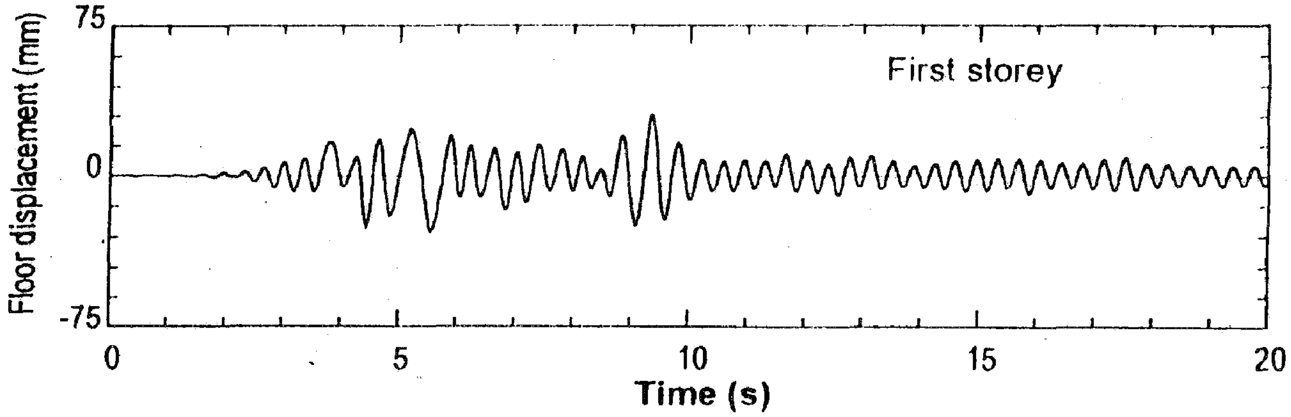
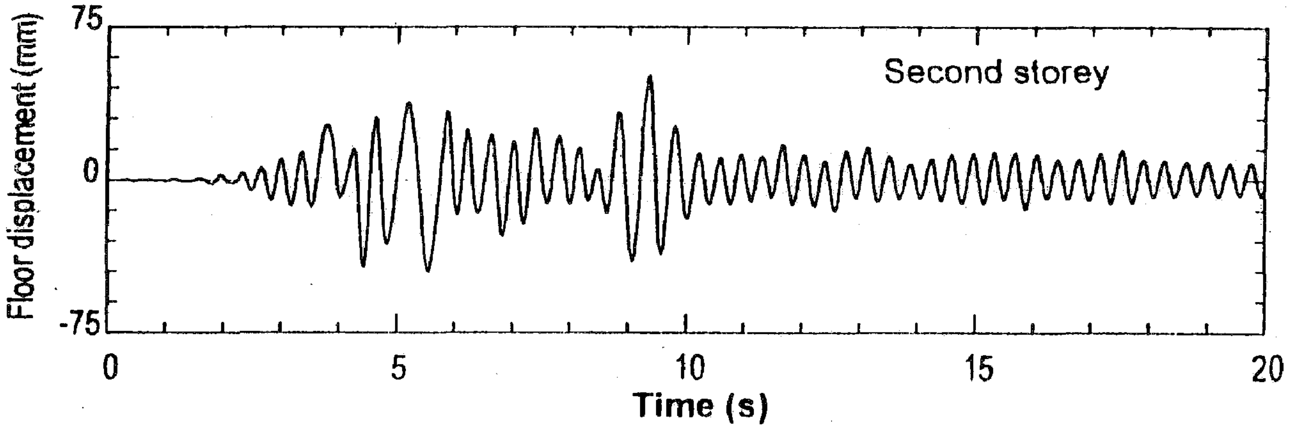
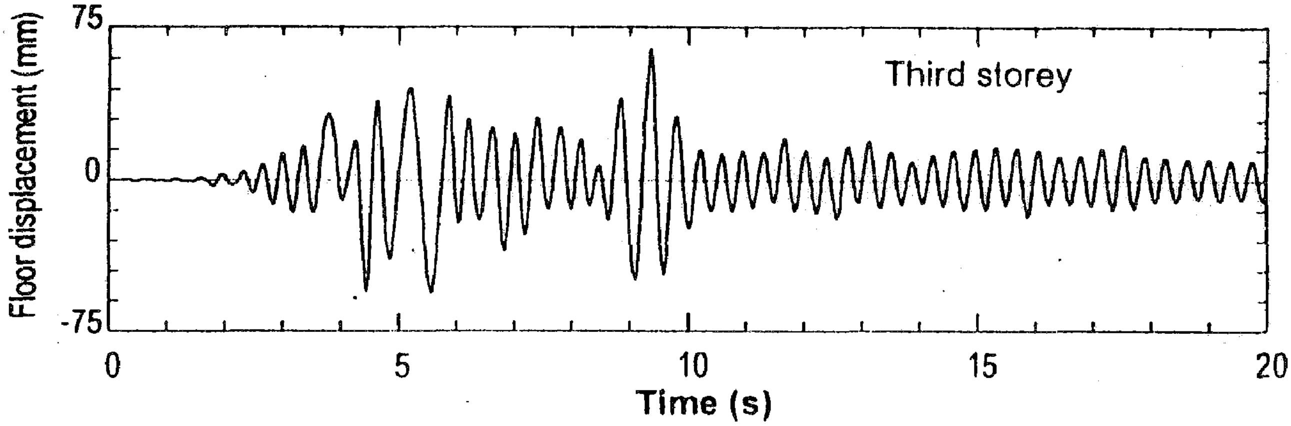


Figure 5.7 Storey displacement response of four storey Shear-Link TMF for El Centro ground motion







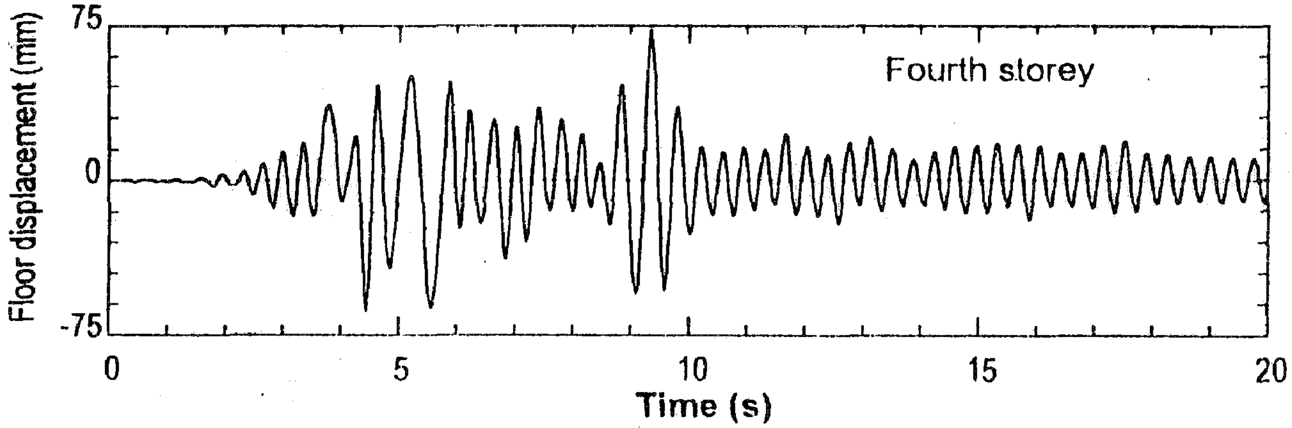


Figure 5.7 Storey displacement response of four storey Shear-Link TMF for Sylmar ground motion

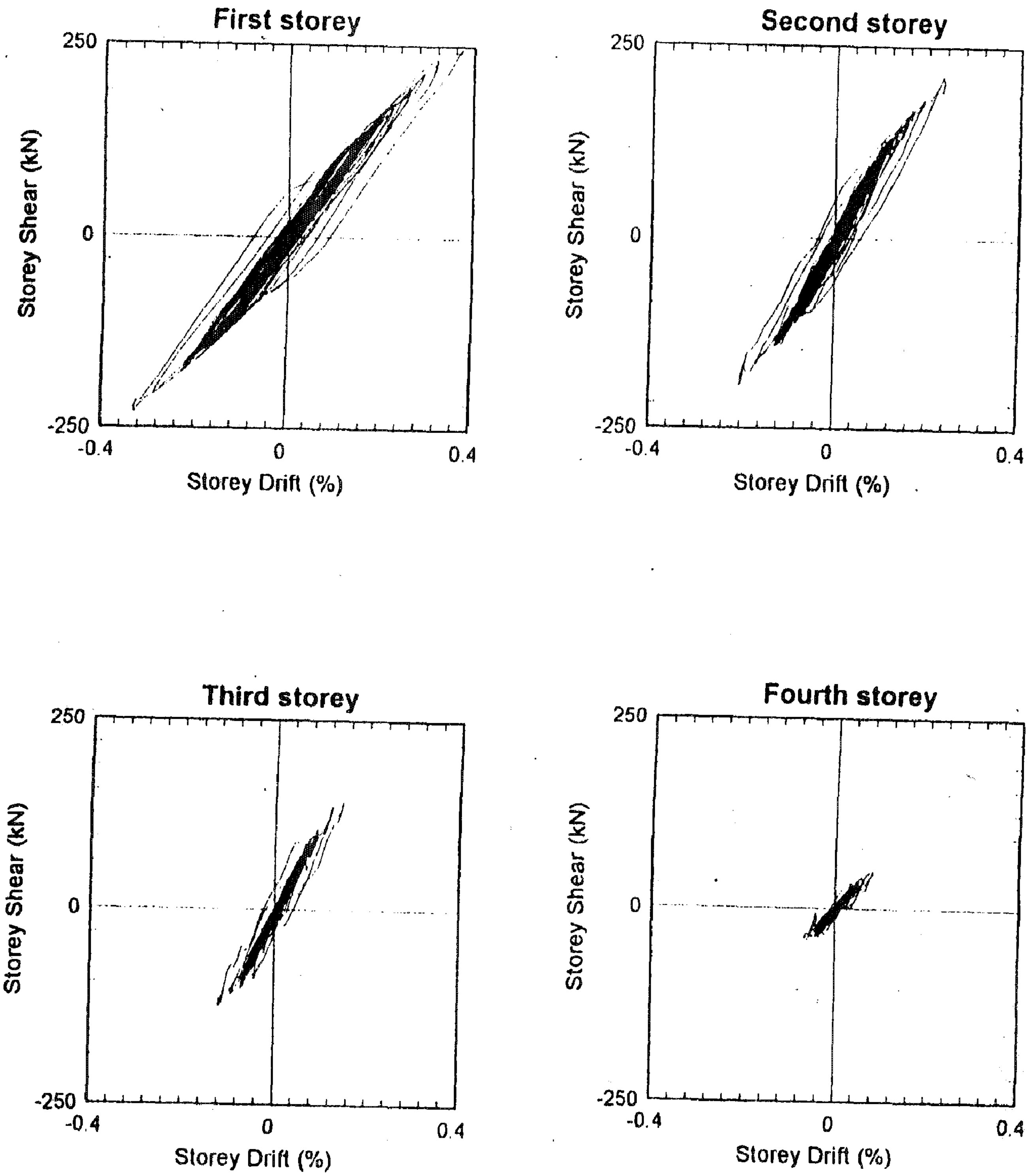


Figure 5.8 Hysteretic response of four storey Shear-Link TMF for El Centro ground motion

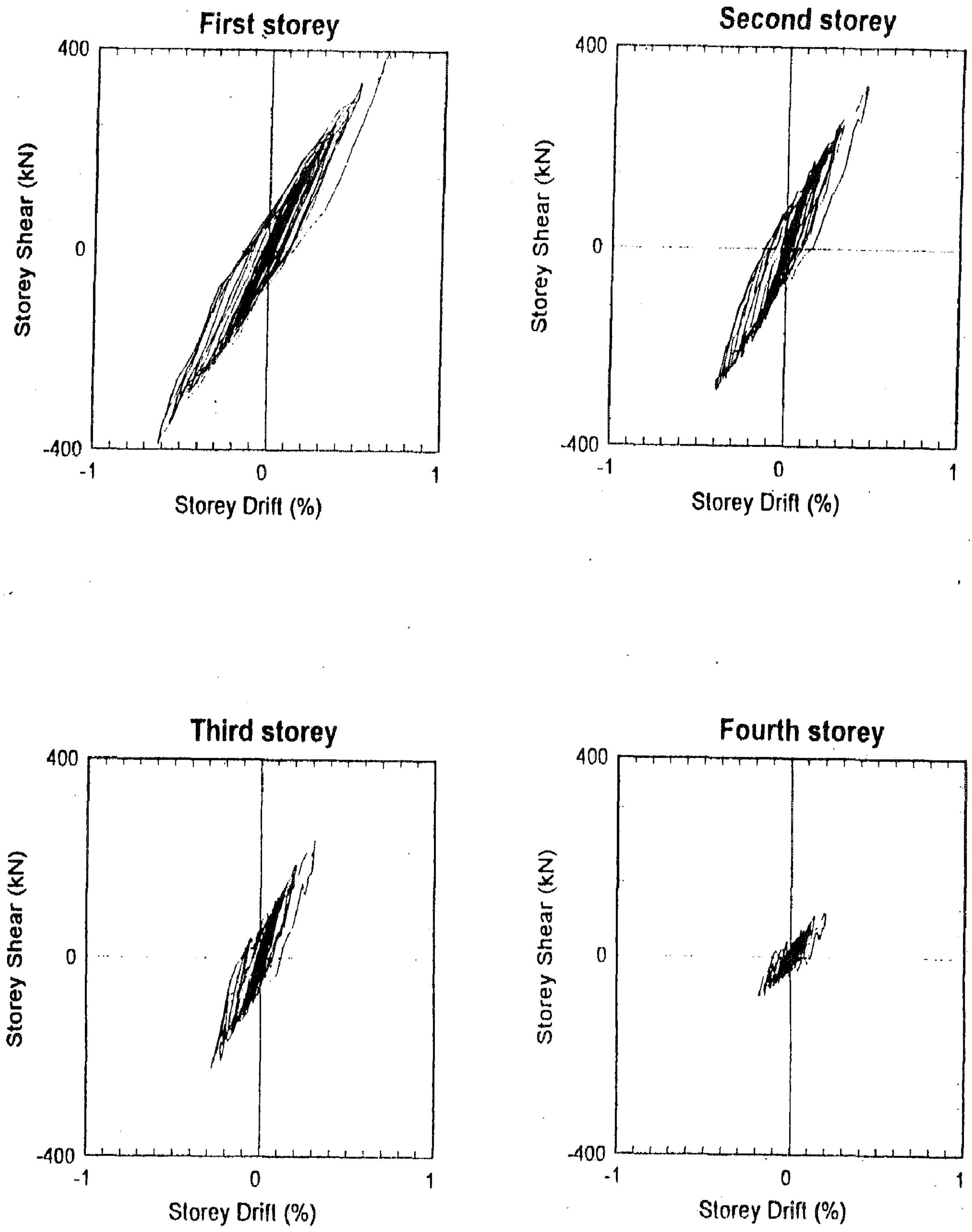


Figure 5.8 Hysteretic response of four storey Shear-Link TMF for Sylmar ground motion

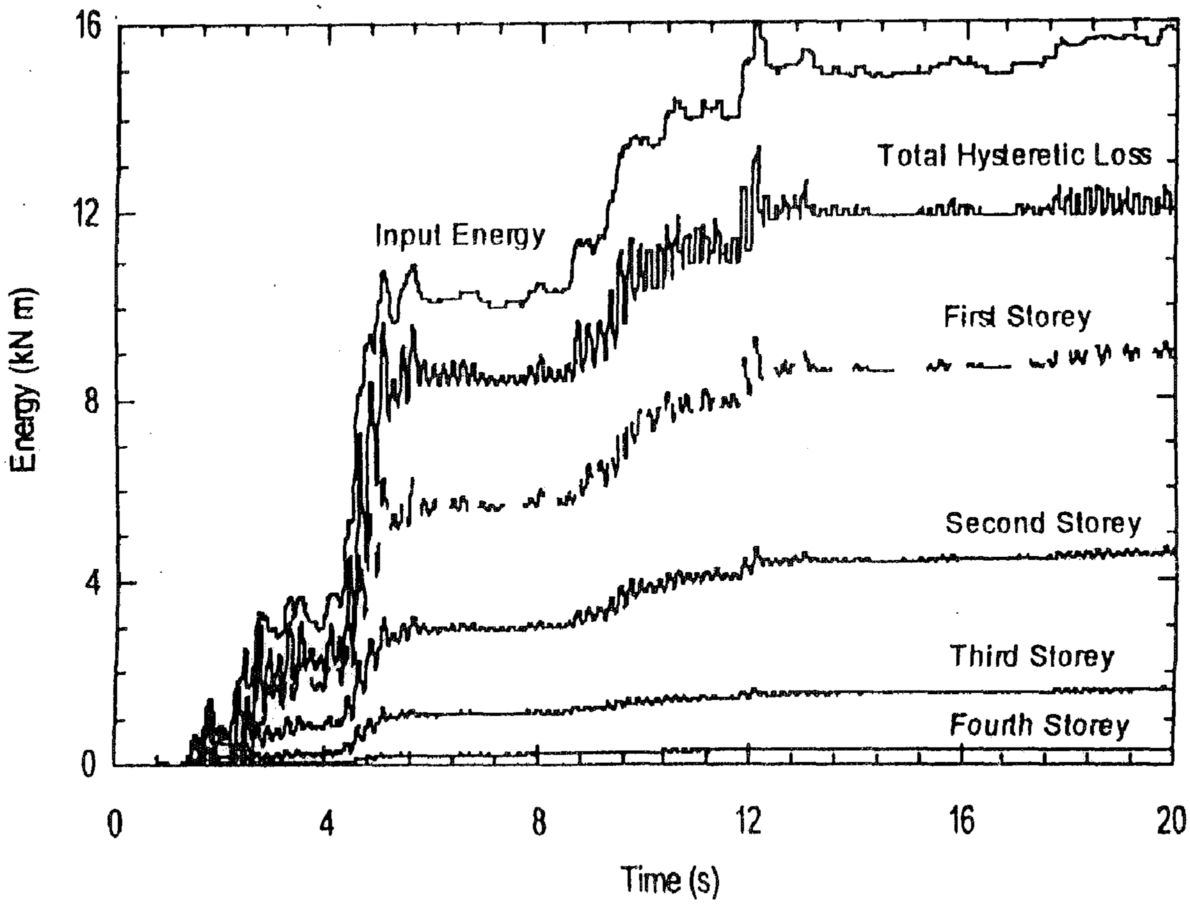


Figure 5.9 Energy time histories for four storey Shear-Link TMF for El Centro ground motion