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EVALUATION OF CONVENTIONAL AND SPECIALTY STEELS IN SHEAR LINK HYSTERETIC ENERGY DISSIPATORS

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SUMMARY

The use of shear links as hysteretic energy dissipators in eccentrically braced frames for buildings and major bridges has become an attractive means of reducing the seismic demands imposed by large earthquakes. In certain situations, built-up shear links made from plates are preferred as they allow the designer to overcome some of the restrictions of the rolled wide flange geometries. An experimental study was conducted on four types of shear links, each utilizing a different grade of plate steel including United States conventional and high performance steels (HPS) as well as two types of Japanese low yield point steels (LYP). These steels provided a range of nominal yield strengths from 100 MPa (14.5 ksi) to 485 MPa (70 ksi). The LYP steel in particular allowed for innovative designs of compact shear links without stiffeners. The main objectives of the large-scale experiments were to apply cyclic plastic deformations and determine the deformation capacity, maximum resistance and ultimate failure mode of the shear link components. All of the experiments resulted in ductile hysteretic behavior showing suitability of these steels for shear link dissipators. In links where stiffeners were used, the failure mode initiated with cracks in the web at the stiffener connections and propagated along the heat affected zone leading to progressive tearing and failure at 0.12 *rad* shear rotation. Significant improvement in deformation capacity was found for the LYP links without stiffeners leading to shear deformations up to 0.20 *rad*. The ultimate strength was found to significantly vary among all of the specimens, with overstrength ratios ranging 1.5 to 4.0, the results of which directly affects capacity design of the associated structural components. Similar trends were also observed in the stress-strain response of coupons of the different steels subjected to reverse cyclic axial loading.

INTRODUCTION

Yielding of the web in shear is the primary energy dissipation mechanism of wide flange beams used as shear link components in structural framing systems. This concept has been effectively utilized in eccentrically braced frames (EBFs) for buildings. Primarily based on the research efforts of Popov et al [1-4], the current seismic design provisions for EBFs [5] specify design criteria such as maximum link length, minimum stiffener spacing and also place restrictions on the deformation design demands. The majority of the research to date has concentrated on rolled wide flange beams since building designs could

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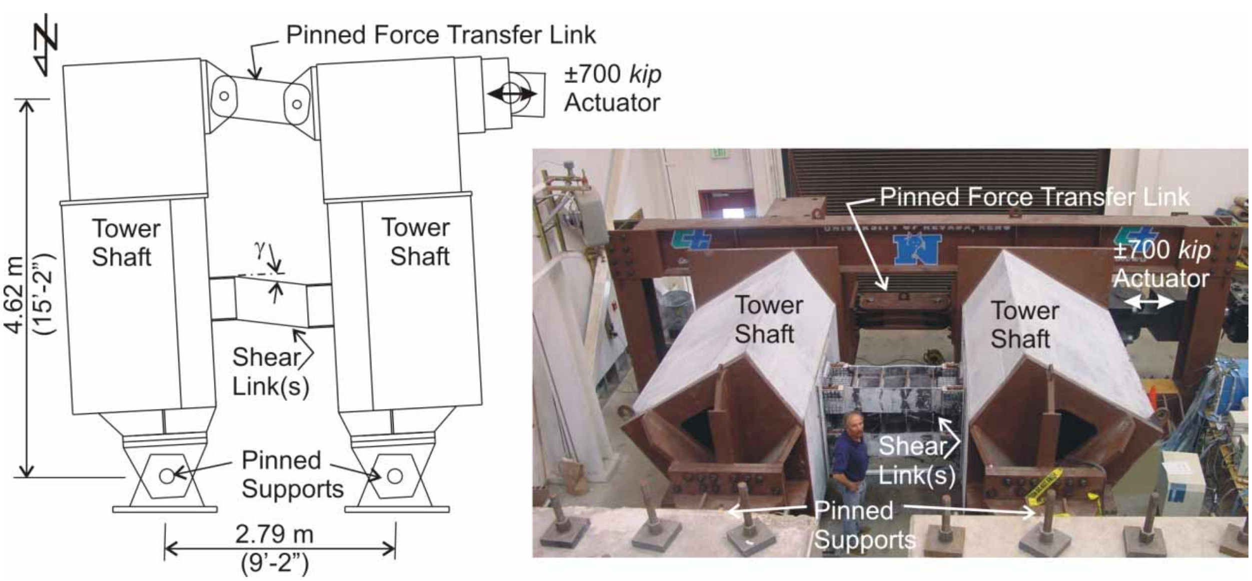
get sufficient capacity from rolled shapes. Recently, the shear links have been used for the retrofit and design of other structures such as long span bridges where the larger scale of these applications have led to the adoption of links that are built-up from plates instead of rolled wide flange beams to meet capacity demands. Two such examples include the EBF retrofit of the Richmond San Rafael Bridge [6] and the use of shear links as fuses in the tower of the new San Francisco-Oakland Bay self-anchored suspension bridge [7, 8].

The use of built-up shear links also allows the optimization of stiffness and strength since the designer has greater freedom with plate sizes and link geometry. This freedom also permits the utilization of new materials to take advantage of the material characteristic properties or to develop innovative design alternatives for these devices. The main objective of this paper is to summarize the results of an experimental investigation of conventional and innovative design concepts of built-up shear link energy dissipators that utilize a selection of conventional and specialty plate steels developed and produced in United States and Japan.

DESCRIPTION OF SHEAR LINK SPECIMENS

Large scale experiments on four different types of shear links were conducted at the University of Nevada, Reno. The test setup shown in Figure 1 was used to impose relative deformation between the tower shafts that was resisted by only the shear links. The aim of the experiments was to apply cyclic deformations of increasing amplitude and determine the deformation capacity, maximum strength and ultimate failure mode of the different designs.



(a) Plan Layout **(b) Photograph of Test Setup**

Figure 1: Test Setup

General Shear Link Layout and Cyclic Deformation History

Initiated by the proof tests of the tower assembly of the new East Span of SFOBB [9], the link layout allowed for replacement after each experiment via bolted end connections. To facilitate this end condition, the general layout of each link consisted of an effective length that was intended to deform plastically and bolted end connection zones that were designed to remain elastic as shown in Figure 2a. The connection zones were designed from higher strength or thicker plates as necessary. The focus of the investigation was on the effective length response, the behavior of which governs the performance of the overall system. As a result, the deformation γ_{eff} was measured independent of the connection zone.

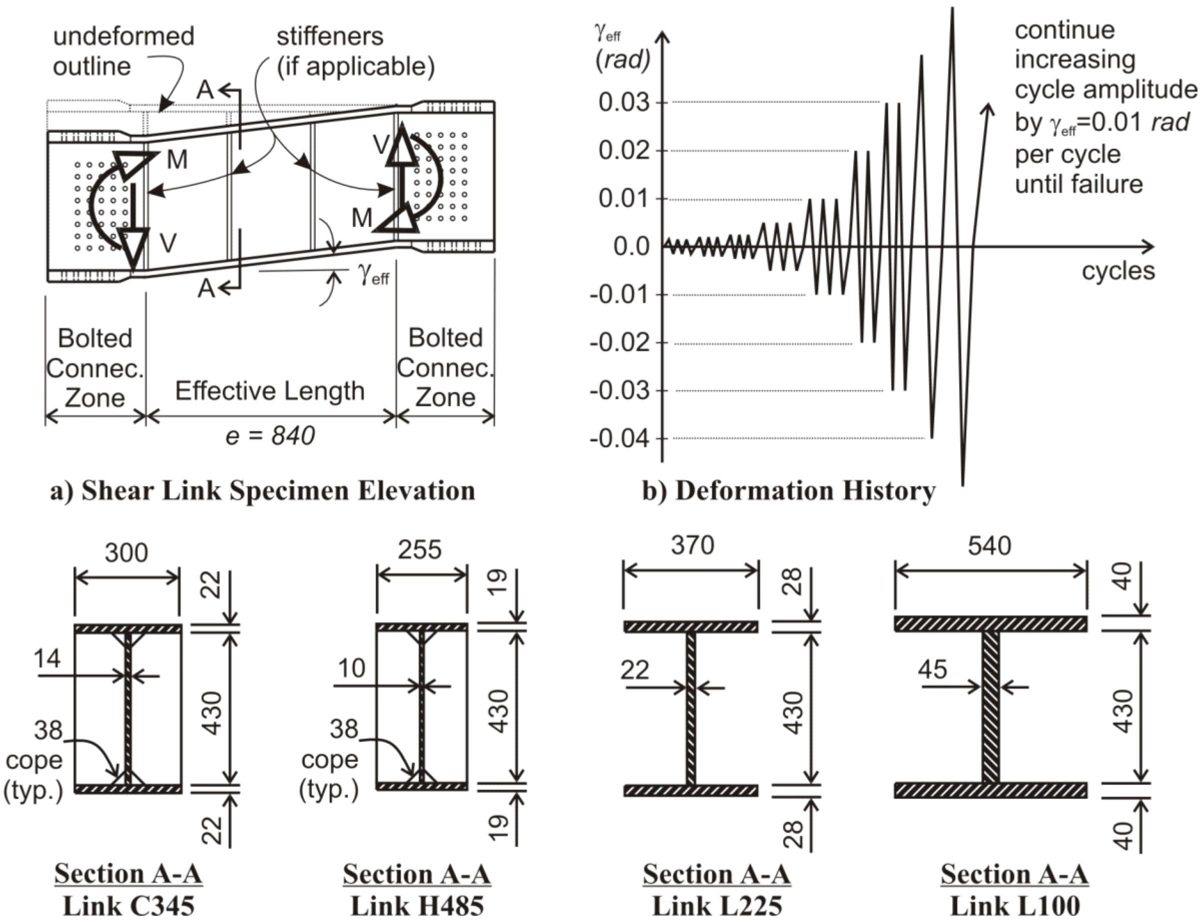


Figure 2: General Layout of Shear Link Specimens (dimensions in *mm*)

Each link was proportioned to have the same nominal shear and moment capacities, i.e. same plastic shear strength V_p and plastic moment strength M_p. These values were calculated using $V_p = 0.6f_y(d\text{-}2t_f)t_w$, where f_y is the nominal yield strength, d is the section depth, t_f and t_w are the thicknesses of flange and web respectively, and $M_p = f_y \cdot z$ where z is the plastic modulus. The overall effective length of 840 *mm* (33.1 *in*) and web depth of 430 *mm* (16.9 *in*) remained the same for all specimens and as a result the cross sectional geometry was changed to achieve the desired component capacities based on the material used. In addition, the flange compactness ratios $b/2t_f$ was also maintained, where b is the flange width. The web to flange plate connection was welded using complete joint penetration, while the stiffeners were fillet welded.

The tests were conducted using the deformation history shown in Figure 2b. This history is based on the AISC recommendations [11], but uses just one cycle per increment starting with the deformation cycle of 0.04 *rad*. Four different types of links were tested, each utilizing a different grade of steel. The different cross sectional geometries are shown as Section A-A in Figure 2 and Table 1 summarizes the nominal design properties.

Table 1: Shear Link Specimen Nominal Properties

Link Type	**C345**	**H485**	**L225**	**L100**
Steel Grade	A709 Grade 345 (50 *ksi*)	A709 HPS 485W (70 *ksi*)	LYP 225 (33 *ksi*)	LYP 100 (15 *ksi*)
Stiffener spacing, *mm* (*in*)	280 (11.0)	168 (6.6)	N.A.	N.A.
Web compactness ratio, h_w/t_w	31	43	20	10
Flange compactness ratio, $b_f/2t_f$	6.8	6.7	6.6	6.8
V_p, *kN* (*kip*)	1245 (280)	1250 (281)	1270 (286)	1160 (261)
M_p, *kNm* (*kip ft*)	1252 (923)	1273 (943)	1290 (951)	1223 (902)

Shear Link from Conventional Grade Steel

Conventional A709 grade 345 *MPa* (50 *ksi*) steel was used to fabricate the effective length of two identical links, which were half scale replicas of the shear link "Type 1" used in the transverse direction of the SFOBB tower [8]. These links were installed in parallel and tested simultaneously to reflect the unique configuration of the bridge tower assembly [9]. This type of shear link will be referred to as Link C345.

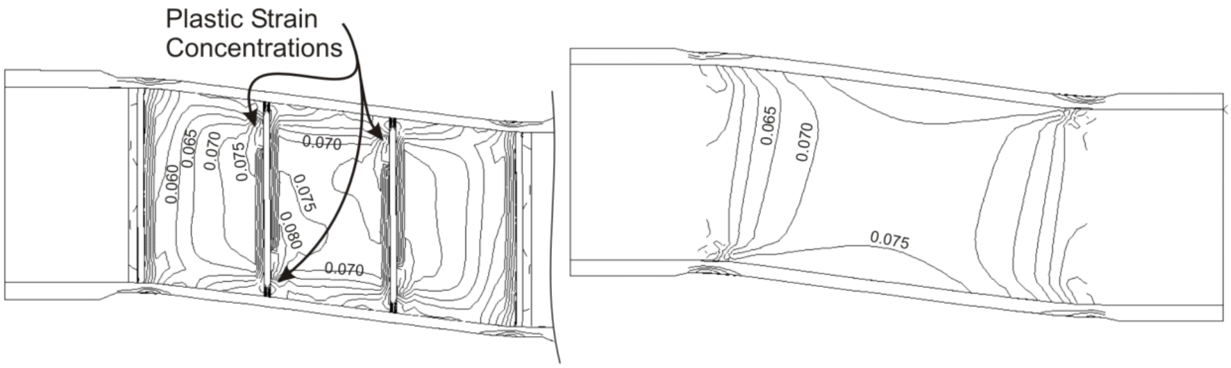
Shear Link from High Performance Steel

High performance steel (HPS) was developed by the joint effort of the American Iron and Steel Institute, the Office of Naval Research and the Federal Highway Administration [12] and is increasingly used for steel plate girder bridges in the United States [13]. This steel has excellent fracture toughness and weldability characteristics, but little is known about its performance under cyclic loads to high yield strains. As a consequence, the AISC Seismic Provisions do not address this grade of steel. It was therefore decided to investigate the applicability of this steel to seismic conditions and a shear link was designed from A709 high performance steel with nominal yield strength of 485 *MPa* (70 *ksi*). The higher strength material resulted in thinner plate dimensions for both the web as well as the flange and also in closer stiffener spacing. This type of shear link will be referred to as Link H485.

Shear Links from Low Yield Point Steels

On the opposite side of the nominal yield strength spectrum to HPS are the low yield point (LYP) steels developed in Japan specifically for seismic applications where high ductility is desired [14]. This grade of steel is produced in two nominal yield strengths; 225 *MPa* (32.6 *ksi*) and 100 *MPa* (14.5 *ksi*). The primary reason for investigating these specialty steels stems from the results of the tests of Links C345 and H485 and the results of a numerical investigation utilizing finite element analyses (FEM).

As will be further discussed in the latter part of this paper, the general failure mode for links C345 and H485 was initiated by cracks that start near the stiffener to web weld. Stiffeners are used to enhance the performance of shear links by delaying web buckling in rolled wide flange beams, which have prescribed web compactness ratios. As illustrated by the results of non-linear FEM analyses of the built-up Link C345 in Figure 3a, equivalent plastic strains were found to locally concentrate in the web at the location of stiffener connection. The equivalent plastic strain demand in the web near the stiffener cope was significantly greater than the plastic strain in the remainder of the web. This location is also potentially vulnerable to crack development due to the presence of welds. Consequently, the heat affected zone and the concentration of plastic strains observed in the numerical model coincide, contributing to the observed failure modes.



(a) Link C345 **(b) Link L225**

Figure 3: FEM Equivalent Plastic Strain Contours for γ_{eff} = 0.10 rad

In a departure from the traditional design methodology of including stiffeners in shear links, an approach was taken to design shear links without stiffeners. The web compactness ratio h_w/t_w was decreased by increasing the web thickness in order to delay web buckling. To achieve the same nominal strength of the overall link, reduced yield strength was needed such that V_p and M_p remained the same. Two link types were developed based on available LYP steel grades and will be referred to as Link L225 and Link L100 for nominal LYP steel yield strengths of 225 *MPa* (32.6 *ksi*) and 100 *MPa* (14.5 *ksi*) respectively. For comparison, the FEM equivalent plastic strain distribution for Link L225 is shown in Figure 3b. The numerical analyses showed the benefit of excluding the stiffeners and reducing plastic strain demand on the web of the link. The heat effects from welding are also reduced on the web within the effective length.

DISCUSSION OF TEST RESULTS

A consistent ductile behavior was recorded for all four types of shear links as shown in the hysteretic responses in Figure 4. Since the focus of the investigation was on the performance of the effective length, the plotted response did not include the flexibility of the bolted connections. However, bolt slippage did result in abrupt reductions in resistance followed by immediate recovery. No pinching or strength degradation was observed in any of the hysteretic loops.

During the experiment on Link C345, where two links were used in parallel, the actuator force capacity was reached in the pull direction after the completion of ±0.07 *rad* cycle. Since additional force capacity remained in the push direction, the actuator underachieved the target in subsequent cycles in the pull direction. Since this did not happen in the push direction, a positive deformation bias occurred in these tests as shown in Figure 4(a). For purpose of comparison with other links, the shear forces plotted in Figure 4(a) have been divided by two to give an average shear per link. All specimens were pushed to ultimate failure without further cycling when significant decrease in resistance was observed.

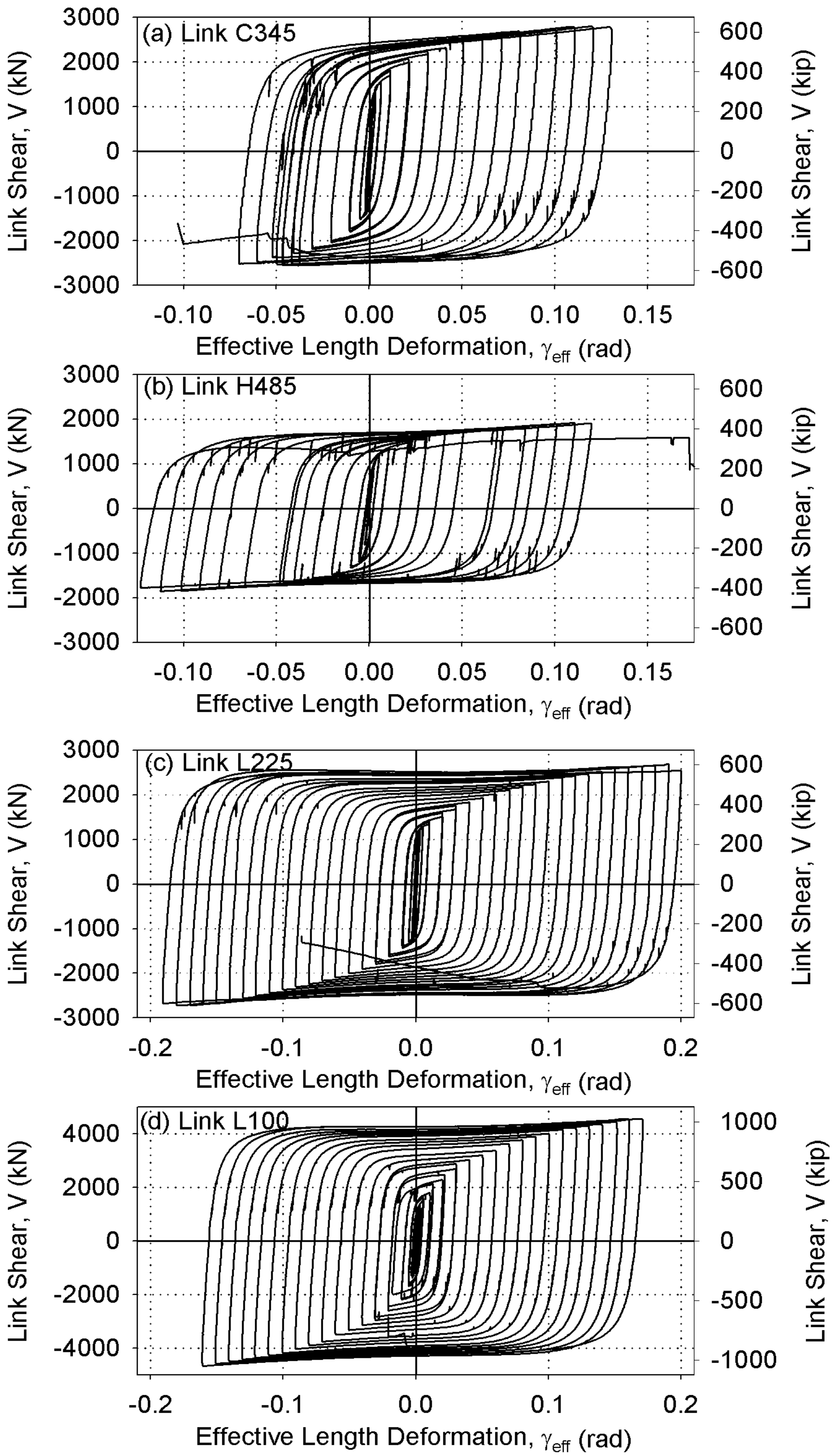


Figure 4: Hysteretic Response of Shear Links under Cyclic Loading

Deformation Capacity and Failure Mode

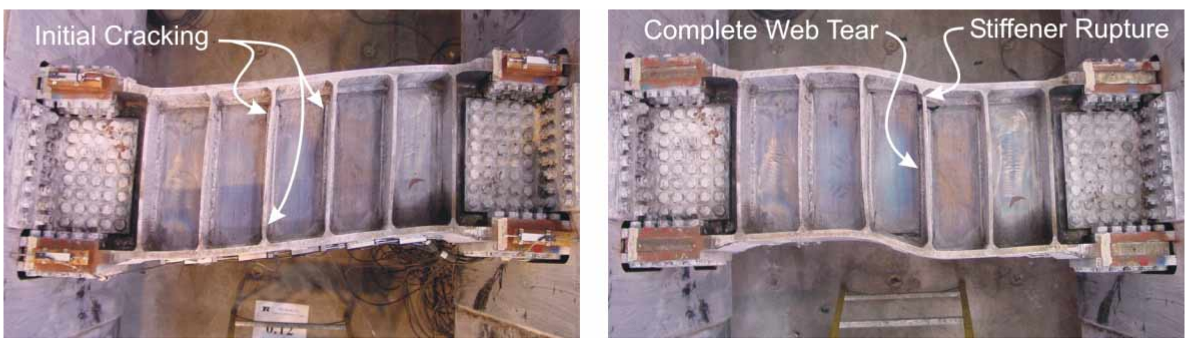
Typically, shear link performance is measured by the achieved deformation capacity in the effective length. The AISC Seismic Provisions recommend the maximum design demand to be below $\gamma_{eff} = 0.08$ *rad*. Maximum recorded capacity of all four types of shear links significantly exceeded this value as summarized in Table 2. The general response can be categorized into two groups of shear links: links in which stiffeners were used and links without stiffeners.

Table 2: Shear Link Performance Summary

Link Type	**C345**	**H485**	**L225**	**L100**
Max. Deformation Capacity, *rad*	0.13	0.12	0.20	0.17*
Max. Shear Resistance, V_{max}, *kN* (*kip*)	2776 (624)	1912 (430)	2715 (610)	4660 (1048)
Overstrength Ratio, V_{max}/V_p	2.23	1.53	2.14	4.02

** maximum deformation capacity not achieved due to premature connection failure*

The links in which stiffeners were used are Links C345 and H485. Both links had similar deformation capacity, demonstrating the potential suitability of HPS steel for seismic applications. The failure mode of the links was also consistent. Cracks were first recorded in the web next to the stiffener weld after 0.04 *rad* cycle. These cracks did not immediately propagate and enlarged only moderately with increased rotational demand. No web or flange buckling was observed. At failure, the cracks propagated from the initial locations and followed the heat affected zone path next to the stiffener welds as shown in Figure 5 for Link H485.



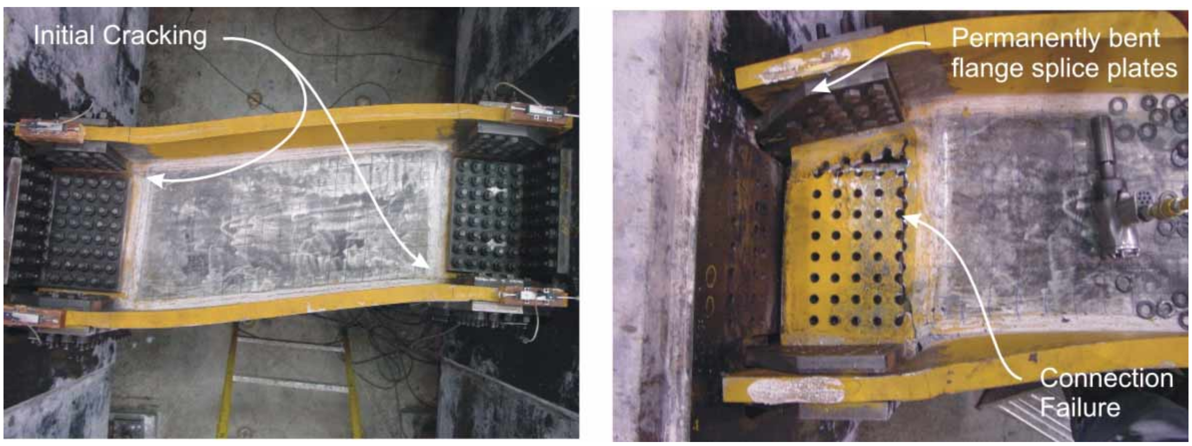
(a) Initial Web Crack Locations **(b) Web Tearing at Failure**

Figure 5: Failure Mode of Link H485

Links L225 and L100, in which stiffeners were not used, utilized the LYP steels. Significant improvement in deformation capacity over the stiffened links was observed for both of these cases. The first cracks occurred in the web in the corners of the effective length after cycles to 0.10 *rad* and 0.09 *rad* for Link L225 and L100 respectively. These cracks did not immediately propagate. For Link L225, out of plane warping of the web was first noticed after cycle 0.14 *rad* was completed and became increasingly prominent with each additional cycle as shown in Figure 6a. Resistance did not decrease and the cyclic deformation continued to follow the prescribed history until the corner of the web started to tear along the ends of the effective length as shown in Figure 6b. The ultimate deformation capacity in Link L100 was actually not achieved due to failure of the bolted connection shown in Figure 7. At the time of the connection failure, the link effective length did not exhibit any signs of significant distress.

(a) Web Warping at γ_{eff} = 0.18 *rad* **(b) Web Tearing Failure**

Figure 6: Failure Mode of Link L225



(a) Deformed Shape at 0.16 *rad* **(b) Connection Failure**

Figure 7: Connection Failure of Link L100

The significant improvement in deformation capacity for Links L225 and L100 was attributed to the combination of using LYP steel and the exclusion of stiffeners from the effective length. The individual contribution of these two factors to the overall improvement cannot be ascertained without additional testing. However, the resulting performance has clearly shown the benefits of this innovative approach of designing compact webs using low yield point steels such that stiffeners can be excluded from the effective length.

Maximum Resistance and Capacity Design

Elements outside of the link must be designed for the maximum forces generated by the shear link, thereby placing significant importance on quantifying the ultimate resistance of the links. As explained in the AISC Seismic Provisions Commentary [5], an overstrength factor of 1.5 has generally been applied to the nominal strength of shear links to determine the design strength of the surrounding components. This factor was based on experimental results of typical rolled wide flange beams.

The built-up shear link specimens considered in this study were designed for the same nominal shear capacity and were found to yield at similar levels of shear demand. However, the ultimate recorded resistance V_{max} significantly varied as summarized in Table 2. The overstrength of each specimen was calculated by normalizing the ultimate resistance to the nominal plastic shear strength V_p. Only Link H485 was found to have overstrength within the expected range of approximately 1.5. The shear link made from conventional steel, exhibited resistance greater than $2V_p$. Link L100 had the highest overstrength, leading to excessive demands on adjacent components and eventual connection failure during the test. The failure of this connection for Link L100 highlights the importance of quantifying the maximum resistance for capacity design purposes.

STEEL CYCLIC PROPERTIES

To investigate the cause of the observed overstrengths, reversed cyclic axial coupon tests were performed to find the stress-strain material characteristics. One coupon for each grade of steel was machined to an effective length of 25 *mm* and diameter of 20 *mm*. Starting with two cycles at ±1%, the axial strain was incrementally increased by 1% after each set of two complete cycles. The resulting hysteretic loops are shown in Figure 8, where the nominal yield strength f_y is highlighted by a horizontal line.

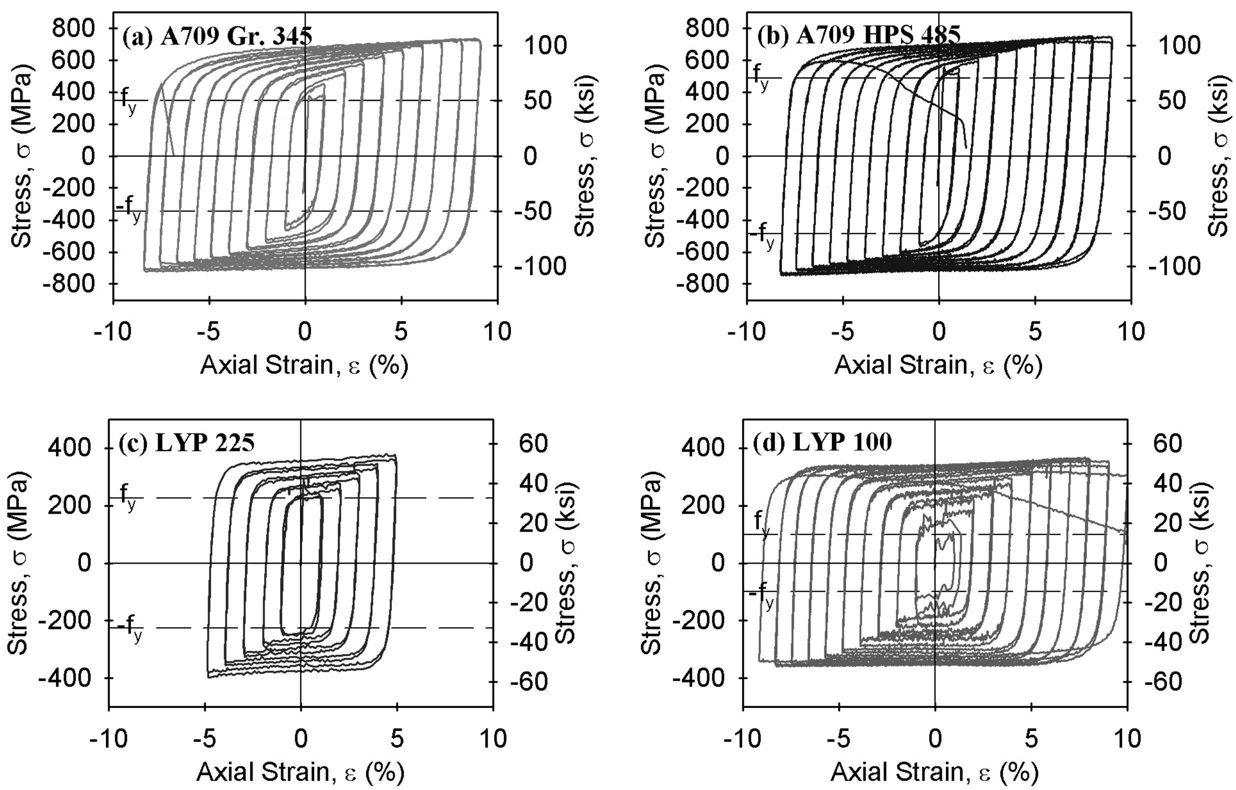


Figure 8: Cyclic Coupon Stress-Strain Behavior under Increasing Axial Strain

When compared to the nominal yield strength, the observed ultimate stress of the coupons showed similar trends to the observed overstrengths in the shear link specimens. The lowest increase was for the high performance HPS 485 *MPa* (70 *ksi*), while the largest was for the low yield point steel LYP 100 *MPa* (14.5 *ksi*) grade as shown in Table 3.

Table 3: Maximum Stress from Cyclic Coupon Tests

Steel Grade	**A709 Gr.345**	**A709 HPS 485**	**LYP 225**	**LYP 100**
Max. Stress, *MPa* (*ksi*)	734 (106)	755 (109)	482 (69.8)	404 (58.6)
Normalized Max. Stress, σ_{max}/f_y	2.1	1.6	2.2	4.0

CONCLUSIONS

The following conclusions can be made based on the results from the large scale experimental investigation using four types of different built-up shear link designs, each incorporating a different grade of steel:

- Shear links built-up from plates can be efficient energy dissipators with high tolerance for inelastic strain, exhibiting ductile hysteretic behavior with deformation capacities that surpass the AISC recommended deformation demand of 0.08 *rad* for EBF links.
- Shear links made from high performance HPS 485 *MPa* (70 *ksi*) steel were shown to have similar deformation capacity and lower overstrength as compared to a link made from the conventional grade 345 *MPa* (50 *ksi*) steel.
- Eliminating stiffeners and using LYP steels for shear links can result in significant improvement in deformation capacity due to the combination of reduced plastic strain demand along with reduction of weldments in the effective length of the link and due to the ductile characteristics of the material itself.
- The overstrength ratio varied significantly among the specimens and in most cases exceeded the expected overstrength for shear links by a wide margin, which without due consideration can adversely affect the capacity design of the remaining components.
- The steel coupons cyclic stress-strain characteristics of the different grades of steel were found to have similar trends to the overstrengths observed in the shear link tests.

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