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2nd Progress Report for

American Institute of Steel Construction
and Oregon Iron Works

DEVELOPMENT OF LINKED COLUMN FRAME LATERAL LOAD RESISTING SYSTEM

by

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SUMMARY

The research on linked column frame system includes investigations in global system response, detailed component analysis and plans for physical tests. Thus far, promising results have been obtained at the global level especially for the 3 story tall buildings. The target performance levels have been achieved and the potential for rapid return to occupancy illustrated. In the past year, the investigation focused on the suitability of the linked column concept for a 9 story tall building. This aspects needs more attention as the linked column was found to have less influence for these taller structures within the cases considered. Additional work is also required in looking at the dynamic response via time history analyses.

As an alternative to conventional EBF shear link designs, the component analyses concentrated on investigating a link that utilized a sandwich of steel and an elastic core to provide the needed web stiffenering effect. Numerically, this was shown to have significant potential in this past year and further analyses are planned with refined material models.

Preparation for the experimental tests had been initiated with re-design and construction of the reaction frame in the lab. Acquisition of material and fabrication of the test specimen is still needed and will be more aggressively pursued without the reliance on donations as had been the case thus far.

The research project has been ambitious and exciting by pursuing system and component modeling while also aiming to conduct laboratory experiments. Progress in the past year had been less rapid, but overall progress had been made on all fronts and the indicators are positive toward achieving the objective of rapid return to occupancy using the link column frame system concept while also investigating other related novel ideas such as the sandwich web shear links.

SPONSORS AND PARTNERS

The research was initiated with support from the American Institute of Steel Construction (AISC) and has evolved to include Oregon Iron Works (OIW), a fabricator local to Portland, Oregon. With their sponsorship, matching funds were obtained from Oregon Metals Initiative (OMI) and had significantly broadened the research project. The support from all of the sponsors and partners is acknowledged and appreciated.

1.0 LATERAL SYSTEM ANALYSES

1.1 Introduction

The concept development and layout details of the linked column frame (LCF) lateral system concept had been outlined in last year's progress report. The work had been ambitiously broken into three interrelated segments; global system development, critical component investigation and laboratory tests. The research continued along these three paths during the past year, though at a slightly decelerated pace due to a graduate student premature departure. This report summarizes the results of that effort and outlines tasks for the upcoming year.

1.2 Numerical Model Layout

During the first year, the viability of the LCF system had been illustrated to achieve the goals of rapid return to occupancy performance level for three story structures through the introduction of replaceable yielding links. System pushover analyses were pursued to adopt a similar approach to 9 story frame. The LCF frame was adopted to a 9 story building originally designed as SMRF as part of the SAC Project. One of the considered layouts is shown in Figure 1. The building was square in plan and included a basement level that provided lateral restraint at the first level. The lateral load resisting system consisted of two perimeter frames.

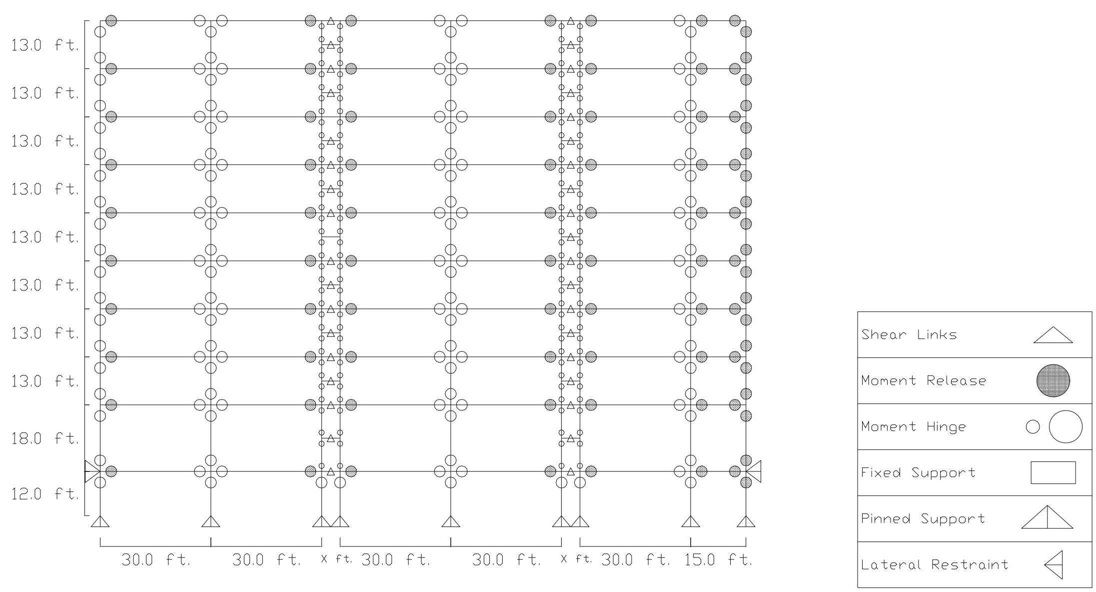


Figure 1: LCF 9 story Model with 2 Linked Columns

Same structural members were used as in the SAC SMRFs and are listed in Table 1. The replaceable links were designed based on AISC Seismic Provisions for EBF links. The link properties were varied to investigate the strength and link length influence on the overall behavior. Nominal 36 ksi steel was assumed for the links with the realization that without additional restrictions in typical design drawings, A36 often has significantly higher yield strength. The properties of considered links are summarized in Table 2. All links were designed as short shear dominated. In each system analysis, the same links were used as any optimization was left to be conducted at a later stage of this research.

Table 1: Frame Structural Member Sizes

Story/Floor	Columns	Girders
-1/1	W14 x 500	W36 x 150
1/2	W14 x 500	W36 x 150
2/3	W14 x 455	W36 x 150
3/4	W14 x 455	W33 x 141
4/5	W14 x 398	W33 x 141
5/6	W14 x 398	W33 x 141
6/7	W14 x 283	W30 x 116
7/8	W14 x 283	W30 x 116
8/9	W14 x 257	W27 x 94
9/Roof	W14 x 257	W21 x 62

Table 2: Considered Link Properties

Link #	Depth d (in)	Web Thickness t_w (in)	Flange Width b_f (in)	Flange Thickness t_f (in)	Length (in)	$1.6\frac{M_{CE}}{V_n}$	Q_{CE} (kips)	Δ_y (in)
1A	25.2	0.55	12.9	1.34	60	103	267	0.147
2A	"	"	"	"	72	103	267	0.192
3A	"	"	"	"	84	103	267	0.246
4A	"	"	"	"	96	103	267	0.311
5A	40	"	"	"	60	111	443	0.134
6A	40	0.33	"	"	60	169	266	0.128
7A	70	"	"	"	60	130	799	0.125
8B	70	0.185	"	"	60	299	269	0.120

Static non-linear pushover analyses were performed on 2 dimensional models using Sap 2000 Nonlinear 10. All models included member plastic behavior and non-linear geometry. The yield strength used for all beams and columns was 57.6 ksi as specified in the SAC report. The yield strength for all links was 36 ksi. A detailed panel zone was not modeled since the focus of the research was not on beam-column behavior, but instead beam lengths that did not account for the column depth were used to approximate the added flexibility of the beam column connections. Half of the seismic mass was lumped at each floor because the building consists of two perimeter lateral force resisting frames. P-delta effects were accounted for by a P-Δ column, which carried half the interior gravity load. The P-Δ column was attached to the frame with axially rigid members that did not contribute to the lateral force resistance of the system. All moment and shear hinge properties were defined using FEMA 356. All beam and column moment hinges were defined at the beginning and end of each member. Link shear hinges were defined at the center of each link. Also all beam and column elements used a strain hardening ratio of 3%.

Lateral loads were distributed throughout the height of the building and were based on equivalent lateral forces. Seismic floor masses as well as lateral forces were distributed and applied at the nodes. Gravity loads were defined using the International Building Code, in which the load combination called for a 20% increase in dead load and 50% of the design live load was used in all analyses.

1.3 Results of Pushover Lateral Response

The initial investigation of the new lateral frame system began with a 2 linked-column LCF system with fully restrained beam ends (2LC-R). Also the initial link lengths investigated were 60 inches. When both ends of the beam were fully restrained the system elastic stiffness and ultimate strength exceeded that of the SMRF. The onset of beam yielding occurred at a roof drift 0.97 % and the links plastically deformed at 0.43% drift. In an effort to delay plastic deformation in the beams, each beam had only one end restrained and the other pinned (2LC-P). The first links became plastic at a drift value of 0.79% and the first beam yielded at 1.27%. By restraining one end of the beam plastic deformation of the beam was delayed by about 0.30% drift. The system strength and stiffness were found to be highly dependent on the fixity of the beams to the columns and can be seen in Figure 2. The initial stiffness of the system was calculated to be 139 k/in when both ends of the beams were restrained, however when only one end was restrained the stiffness reduced to 72 k/in. The maximum base shear also decreased from 2,581 kips to 1,651 kips.

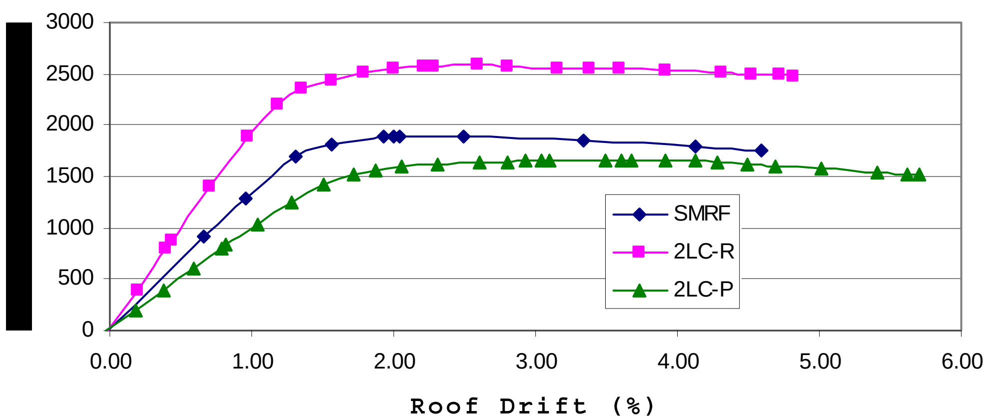


Figure 2: Pushover Response of LCF with Fully and Partially Restrained Beam Ends

To quantify the stiffness contribution of the LCF systems components, linked columns were decoupled from secondary moment frames. Analyses revealed that the linked columns in the 2LC-P model contributed 25.2% to systems stiffness, compared to 13.1% for the 2LC-R model. Strength was evaluated at a drift of 2% and the linked columns were observed to resist 26.6% and 16.5% of the systems base shear for the 2LC-P and the 2LC-R systems respectively. Since the linked columns primary purpose is to resist lateral forces its essential that they contribute more to the pushover response of the system than the secondary moment frame. In an effort to increase the contribution of linked columns, one additional linked column was added to the system and the results are shown in Figure 3. These models are referred to as 3LC-P models. By adding an additional linked column the contribution of the secondary moment frame decreased and as desired the contribution of the linked columns increased. The linked columns in the 3LC-P model were found to contribute 44.9% to the systems stiffness. Also, it was found to resist 50% of the systems base shear. Though the contribution of the linked columns increased the overall stiffness of the system slightly decreased to 68.2 k/in. By adding additional linked column, the secondary stiffness from the moment frame parts of the system reduces the desired effect of restoring stiffness for the overall system. Consequently the general arrangement of LCF for 9 story buildings requires additional investigation.

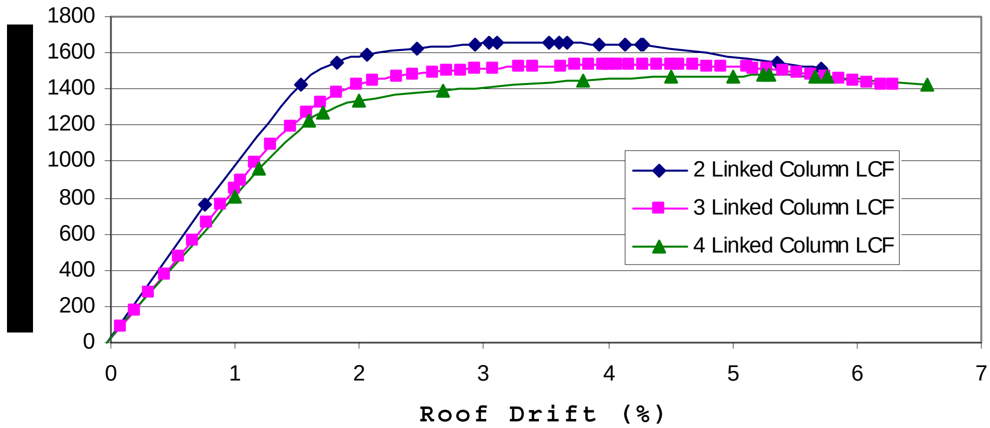
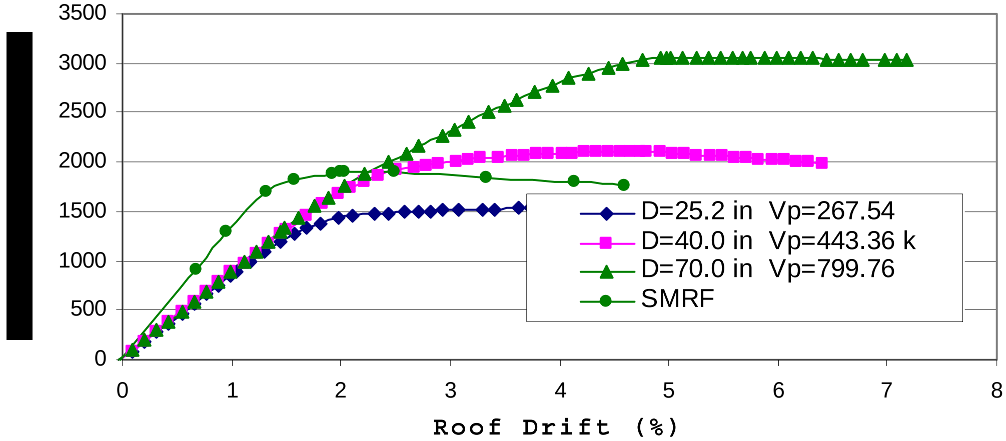


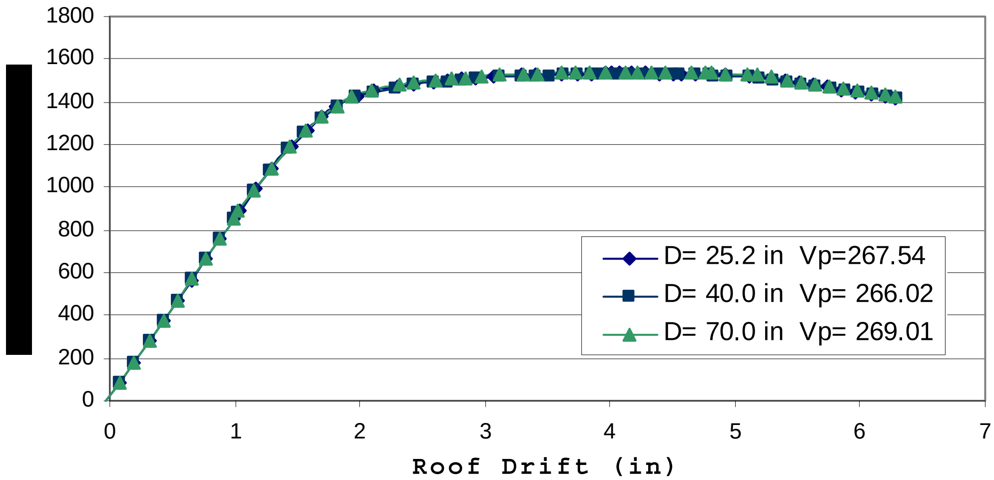
Figure 3: Pushover Influence of Adding LCF

Part of the project included investigating the variation of link shear strength (V_p) and link depth (D). During this part of the investigation V_p was varied by only changing the depth of the link. It was found that the shear strength of the link did not influence the elastic stiffness of the system. However, the base shear of the system and onset of link plasticity were influenced by link shear strength variation. Figure 4a shows as V_p increased the base shear resistance of the system increased. Links with higher shear strengths were found to plastically deform at higher drift values. For shear strengths of 268, 443, and 800 kips, the maximum base shears were 1533, 2104, and 3058 kips, respectively. Also, respectively, the first link to yield occurred at 1.04%, 1.5%, and 2.93% drift.

Another investigation involved keeping V_p constant while varying link depth. This investigation was performed to illustrate the impact on stiffness of the system while not affecting the overall strength. V_p is a function of the links geometry, so the web of the links were manipulated to keep a constant V_p since the depth of the member varies. In this investigation varying the depth of the link had no affect on the elastic stiffness of the system, strength of the system or onset of link yielding. The systems elastic stiffness remained 68 k/in. The base shear of the system remained at approximately 1530 kips and the links initially became plastic at a roof drift of 1%. The similarity of system response is shown below in Figure 4b. These results indicate that link strength more so than the geometry influences the system global response.



(a) Link Depth and Strength Variation



(b) Link Depth Variation While Maintaining Similar Link Strength

Figure 4: System Response for Varying Link Properties

To control drift demands, a higher elastic stiffness was desired. So, in an attempt to obtain a higher elastic stiffness the lengths of links were increased. During this part of the investigation it was determined that the stiffness of the system significantly depended on the length of the link. As link length increased the elastic stiffness of the system increased, as well as the linked columns contribution toward the system stiffness. Link lengths of 5, 6, 7, and 8 ft were evaluated to determine the affect of varying link lengths. The strength of the links was not changed. Figure 5 below shows the change in the systems pushover response as link lengths increase. For link lengths 5, 6, 7, and 8 ft, the elastic stiffness's are 68, 79, 89, and 100k/in. In order to achieve stiffness approximately equal to that of SAC's SMRF the link length of 3LC-P system must at least 8 ft.

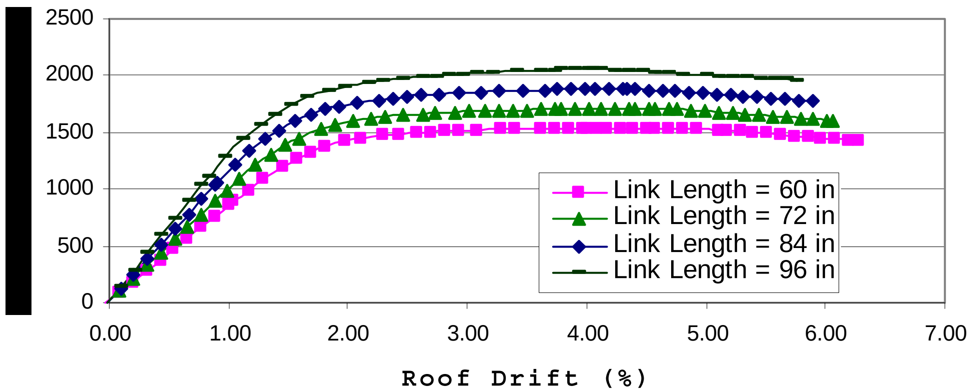


Figure 5: Variation of Link Length

System response of a 9 story building with linked columns was investigated using pushover analyses. The system was shown to remain ductile, with system layout that achieved first link yielding followed by beam column mechanism as intended in the three performance levels targeted by the LCF system . However additional work needs to be done on investigating the 9story buildings because the linked column contribution and the overall stiffness were found to be lower than in the 3 story buildings. Also, further refinement of the performance needs to be addressed along with investigations using time history analyses.

2.0 REPLACEABLE LINK MODELING

Stiffeners are required in most cases to constrain the web because past research has shown that web buckling can result in undesirable hysteretic behavior and premature failure of these components. However, web stiffeners can introduce other issues. The welding associated with web stiffeners can change the characteristics of the web metal, which is relied upon for the ductile behavior of the beams and could therefore influence the performance of this critical component. Also, fabricating the beams with many stiffeners or with small stiffener spacing could result in increased cost and reduced welding accessibility during fabrication. Although a conventional shear link design could work for the LCF system, an alternative is being pursued in having shear links without stiffeners through the use of sandwich webs.

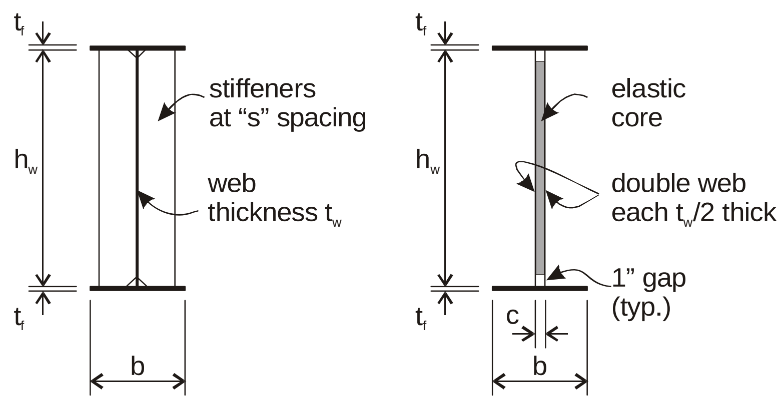


Figure 6: Sandwich Web Link Cross-section

The design concept consists of two steel plates with an elastic core between them, forming a sandwich web as shown in Figure 6. The objective of the numerical simulation was to ascertain the feasibility of using sandwich web in order to enhance the cyclic performance of unstiffened web links subjected to shear dominated plastic deformations. The numerical model was developed using Abaqus finite element software, with the meshed model illustrated in Figure 7. The steel was assigned 50 ksi yield strength and was idealized with bi-linear stress-strain behavior of 2% elastic kinematic hardening. The core was assumed elastic, with a modulus of elasticity E_c. Geometric non-linearity was also taken into consideration throughout the analyses.

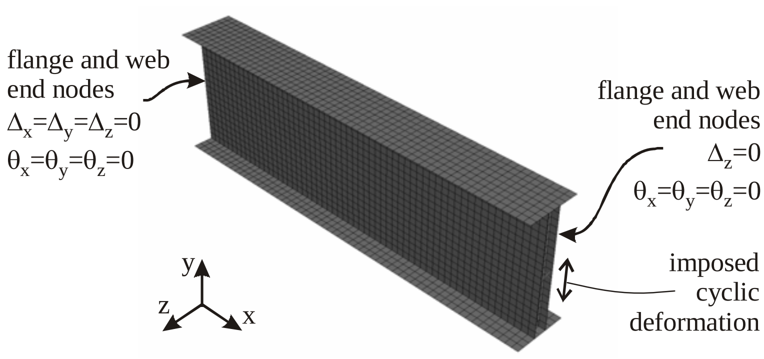
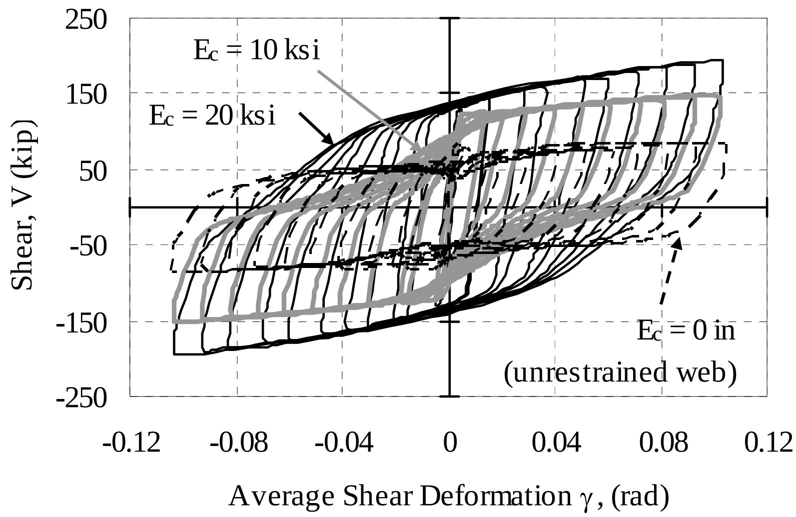


Figure 7: Meshed FEA Model and Boundary Conditions

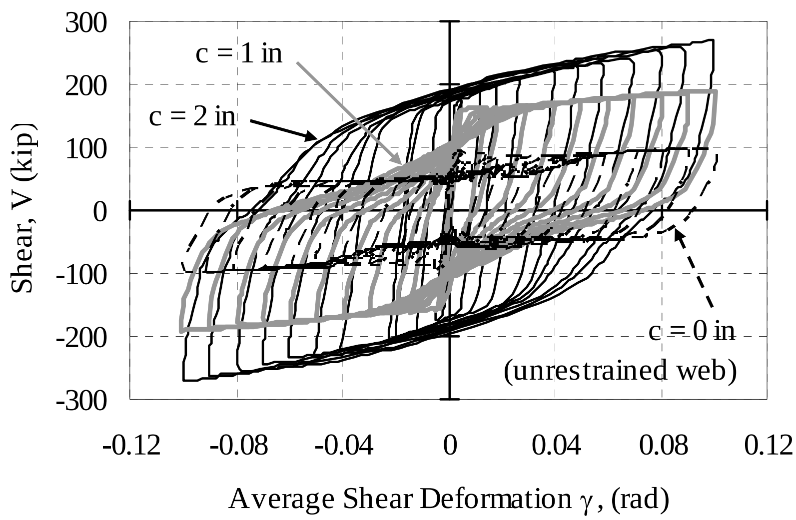
The cyclic deformation was imposed at one end of the link and analyses conducted using non-linear material and large deformation considerations. The imposed displacement history was reversed cyclic, increasing in deformation at each cycle. The amplitudes followed the recommendations for testing eccentrically braced frames and utilized the link average shear deformation γ = 0.005 rad, 0.01 rad with increasing amplitude thereafter by 0.1 rad. Unlike the recommendations for physical testing, the analyses did not include repeated cycles at each amplitude because little additional value would be gained toward the study objective. Unlike physical tests, the analyses do not specifically account for non-linear cyclic hardening of the material, nor include any failure criteria. A single cycle per amplitude was therefore used for the analyses to aid the computational efficiency.

Analyses were conducted to average shear deformation values of γ = 0.10 rad in all cases, however it should be stressed that these numerically obtained results did not specifically model any failure modes. The focus of the study was to study the effect of using an elastic core between two thin webs to provide a continuous restraint along the web surface. The effects of core elastic modulus and thickness were studied.

The cyclic behavior of link 18" deep link (B18) of 1 in core thickness is shown in **Error! Reference source not found.** with increasing values of the core elastic modulus. The hysteretic response is significantly improved even for core elastic modulus values that are a small fraction as compared to steel, which is usually assumed to be 29,000 ksi. For E_c = 10 ksi, the initial strength increased to 92% of the plastic capacity. The hysteresis were pinched at reversals, which was especially evident at large shear deformations. But, the effectiveness of the core in improving the cyclic response was demonstrated at relatively low values of the core elastic modulus. Similar response was observed for B24 and was even more favorable for thicker cores. The cyclic behavior of link 24" deep link (B24) of 10 ksi core elastic modulus is shown in **Error! Reference source not found.** with increasing values of core thickness. The hysteretic behavior improved with increasing core thickness or an effective separation of the double web. The plastic shear strength was nearly reached at 96% of V_p for 1 in core thickness. The pinching of the hysteresis following reversals was not as severe for the larger core thicknesses of 2 in. Similar improvements in behavior were observed for link B18. From these analyses, the thickness of the core has a significant influence on improving the cyclic behavior of links subjected to inelastic shear dominated deformations.



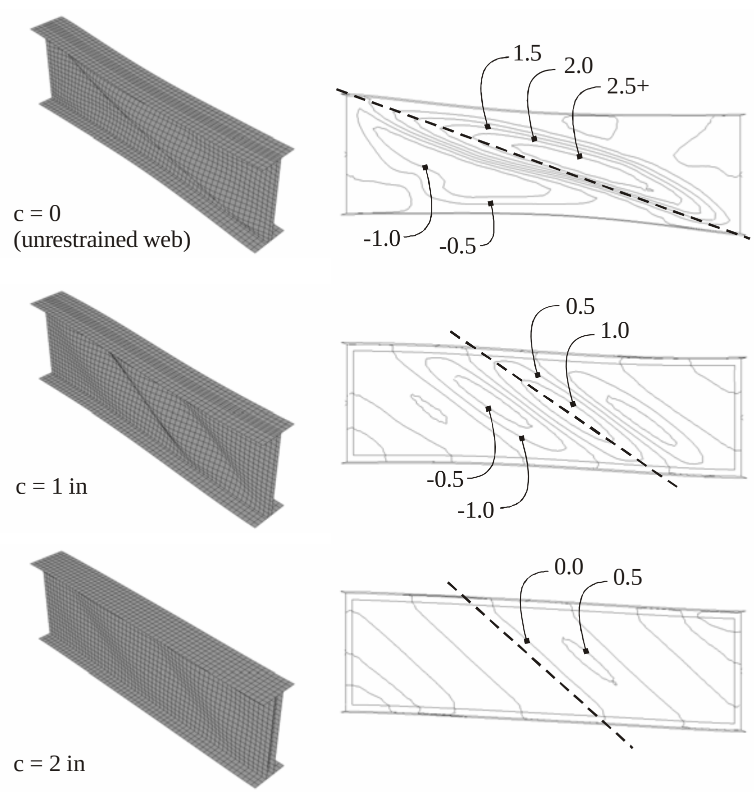
(a) Link B18 with varied values of elastic modulus



(b) Link B24 with varied core thickness

Figure 8: Cyclic Response of Sandwich Web Links

In terms of the link behavior, the web in the unrestrained web case buckled prior to developing the full shear strength capacity. Under increasing deformations, a tension field developed from opposite corners of the web. An illustrative snapshot is shown for B18 at an average shear deformation of $\gamma = 0.05$ rad in Figure 9a for the deformed shape and Figure 9b for the magnitudes of web out of plane deformations. The dashed line was drawn in to illustrate an approximate direction of the line of action of the tension field. Once the tension field was formed, the link was able to maintain some level of strength, at least in the numerical models. With increasing core thickness, the slope of the line of action increased due to the improved web out of plane resistance, facilitating the link in attaining its plastic shear capacity without the use of discrete stiffeners. In addition to the web distress, the severe out of plane deformations of the web in the unrestrained case resulted in forcing of the flanges towards each other. The distorted web of the unrestrained web was not able to carry the shear as effectively.



(a) Deformed Shape (b) Out of Plane Web Deformation Contours (in)

Figure 9: Sandwich Web Influence on Web Deformations

The finite element analyses indicate a significant potential for links with sandwich webs to form an effective alternative to the conventional stiffened plate shear links. In order to further study this novel concept, improvements to the numerical models are needed. The elastic core was assumed to be purely elastic, however a material model more representative of a potential material is needed to more accurately represent the behavior.

3.0 LINK-COLUMN ASSEMBLY LABORATORY TESTS

Two novel ideas had been developed during the course of this research project; the link column frame system that relies on the performance and replaceability of links to achieve rapid return to occupancy and the sandwich web as an alternative to the conventionally stiffened shear link. Both of these concepts will be investigated using physical experiments.

On the replaceability aspect, the objectives of the physical tests are to investigate the cyclic behavior of bolted links between two columns. Conventional shear links in EBFs are formed from continuous beams and the behavior of bolted yielding element had not been widely investigated. At the same time, having the ability to replace the links will allow the study of a number of different link configurations, including the ones with sandwich webs. The objectives in testing the links will be to find the cyclic behavior, quantify the failure modes and the ultimate strengths of the link components.

The test setup had been outlined in the first year and some progress had been made toward assembly of the setup near the end of the year. The lateral reaction system had been re-designed to accommodate a recent equipment addition to the laboratory, a long stroke hydraulic actuator, that will be used for these tests. The reaction frame part of the test setup had been fabricated, delivered and is now being assembled. An attempt was made at having the test specimen (linked columns and links) donated, however those efforts had not yet been realized. In the next year, the acquisition of the columns and replaceable links will be put out for bid and more aggressively pursued. It is expected that physical testing would be carried out over the summer months.

4.0 Upcoming Research Activities

Next year, the research effort will concentrate on further developing two of the promising aspects that had been developed thus far; the global analysis of the three story model and sandwich web link concept. Both of these have aspects that not only contribute knowledge to this particular research effort, but can also be adopted in other steel design and construction applications.

Tasks associated with the global system development will include:

- further refining the 3 story model – The underlying premise of the analyses were the adoption of SMRF members. In LCF, the linked columns were found to contribute significantly toward the lateral response leaving the opportunity to decrease gravity member sizes from those used in SMRF, yet still achieve the intended performance levels. The LCF building members will be re-designed without the legacy SMRF and the steel requirements compared.
- time history analysis – Analyses in the development of the frame system consisted of non-linear pushover. With the refined LCF model, non-linear time history analyses will be conducted to investigate the dynamic performance.
- 9 story building – although some successes had been reached in the analysis of the taller building, the results had not been as positive as for the 3 story building. Some of the issues such as the lower contribution of linked columns to the overall response and potentially large column axial forces at the foundation, are bound to exist in structural systems such as the special shear walls. Further investigation into the viability of LCF for taller buildings will be made by looking at some of the alternate lateral systems in these types of buildings.

The currently employed software platform (SAP2000) will be abandoned for the time history analyses due to the inherent modeling limitations. Two different software tools are being currently evaluated for adoption in this work; Perform3D and OpenSees. The analyses will be conducted with the help of a new graduate student.

Tasks associated with the component analyses of the link will include:

- sandwich web links – the concept of using elastic cores to provide some level of restraint had shown promise and the next steps in its investigation requires analyses with more representative material models. Options of physical materials for the elastic core will be proposed and
- connection details – one of the items not addressed thus far is the link details of transition from zones of plastic strains to the elastic end of the connection. Connection details had been proposed in the first progress report and those will be modeled for evaluating effectiveness.

Tasks associated with the experimental part of the investigation will include:

- completion of the reaction frame – Part of the reaction frame is now being assembled in the lab along with incorporation of a new actuator. The load transfer to the strong floor still needs to be completed as is the specimen end (i.e. the linked column frame with bolted links) of the test setup.
- assemble test setup – Abandon the pursuit of donated materials and put out to bid the linked column part of the test setup so that activities in the lab can continue to progress toward testing during the summer months.
- test linked column frame subcomponent – Assemble components and conduct tests on linked column with conventional as well as with sandwich web links.