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PERFORMANCE TESTS OF INNOVATIVE DUCTILE STEEL RETROFITTED DECK-TRUSS BRIDGES

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SUMMARY

Seismic vulnerability assessments of many existing deck-truss bridges often reveals the vulnerability of their substructure and many superstructure members. As an alternative to costly strengthening of many components or seismic isolation, an innovative retrofit strategy developed which requires only replacement of the existing non-ductile end and lower-end bracing with ductile panels. By incorporating these energy dissipating devices acting as fuses, these devices can yield and dissipate energy while protecting both superstructure and substructure. A 27-ft long deck-truss bridge model was designed and constructed at the University of Ottawa and was pseudo-dynamically tested in its as-built as well as retrofitted conditions. Two configurations of Eccentrically Braced Frames, EBF, and Vertical Shear Links, VSL, were used as ductile retrofits, and both performed well. The ductile retrofit devices exhibited a robust hysteretic behavior, dissipated the seismic induced energy and prevented damage in other structural members of the model bridge when it was subjected to scaled El Centro earthquake. This paper describes the retrofit concept, the innovative seismic testing procedure, and the performance of the steel ductile retrofits.

INTRODUCTION

Older deck-truss bridges constructed for decades in North America do not conform to current seismic design codes. These bridges could suffer severe damage in areas struck by major earthquakes (Imbsen and Lui, 1993, and Matson and Buckland 1995) and as evidenced in some recent moderate earthquakes (Astaneh-Asl et al. 1994, Housner et al. 1995, Bruneau et al. 1996). In these structures the deck is supported by truss girders seated on abutments or piers. Lateral inertia forces of the earthquake applied at the deck level have a sizable eccentricity with respect to the truss supports at the end of the lower chords, imposing forces on the entire superstructure members to carry these forces from deck to end supports. Typically, existing lateral load resisting members and their connections are not ductile and are expected to suffer damage in the event of a major earthquake . Particularly, lateral bracings and end and intermediate cross-frames, in older truss bridges which were typically designed for wind forces or stability during construction, cannot be expected to withstand the severe cyclic inelastic deformations expected to develop during large earthquakes.

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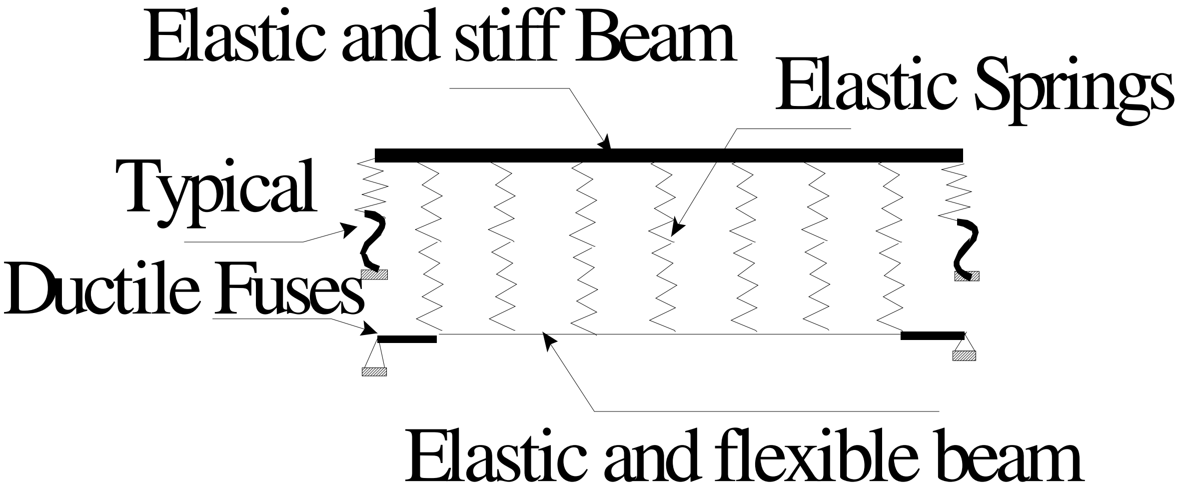
Moreover, these older bridges are often found to be supported on unreinforced masonry or concrete substructures which have a very non-ductile deformation characteristic. Thus, the substructure of such a bridges would also be at high risk of damage during a major seismic event.

DUCTILE SEISMIC RETROFIT

An in-depth coverage of the retrofit concept and development of the design equations based on the predicted inelastic lateral dynamic response of a deck-truss bridge can be found in Sarraf and Bruneau (1998a). The proposed retrofit concept can be best described by a 2-D beam analogy of the 3-D structure of the retrofitted truss bridge. Figure 1(a) shows a 2-D beam analogy in which the upper and the lower beams represent the bridge deck-top lateral assembly, and bottom lateral bracing, respectively. The interconnecting springs represent the stiffness of the intermediate cross-frames. Thus, the existing lateral load resisting system of the deck-truss consists of two load paths which can interact through interior cross-frames. Considering the lateral loads on the top beam as the effect of lateral inertia forces on the bridge, by introducing two ductile fuses at each end of the two beams, the magnitude of forces transferred to all the lower beams, the interface springs, the top beam, and end support reactions will be limited by the capacity of the fuse. The practical implementation of this concept in a bridge is as shown in Figure 1(b) Thus, the proposed ductile retrofit requires conversion of each end cross-frame into a ductile panels having a specially designed yielding device (i.e. a structural fuse), and conversion of the last lower end panel near each support into a similar ductile panel. The stiffening of the top lateral bracing system is also required, which can be achieved by providing composite action between the concrete deck floor beams and the top chords and continuity in the deck system. This stiffening has two benefits: first, for a given lateral displacement at the end supports and deck, it reduces the relative mid-span displacements, resulting in reducing the forces imposed on the interior cross-frames; second, it increases the share of the total lateral load transferred by the top lateral bracing path. Incidentally, assuming the deck is made continuous and integral with the top truss system, the in-plane flexural stiffness of the deck becomes sufficiently large to be modeled as a rigid beam shown in the 2-D beam analogy. This greatly simplifies modeling and development of the generalized stiffness for the retrofitted structure, and designing the retrofits.



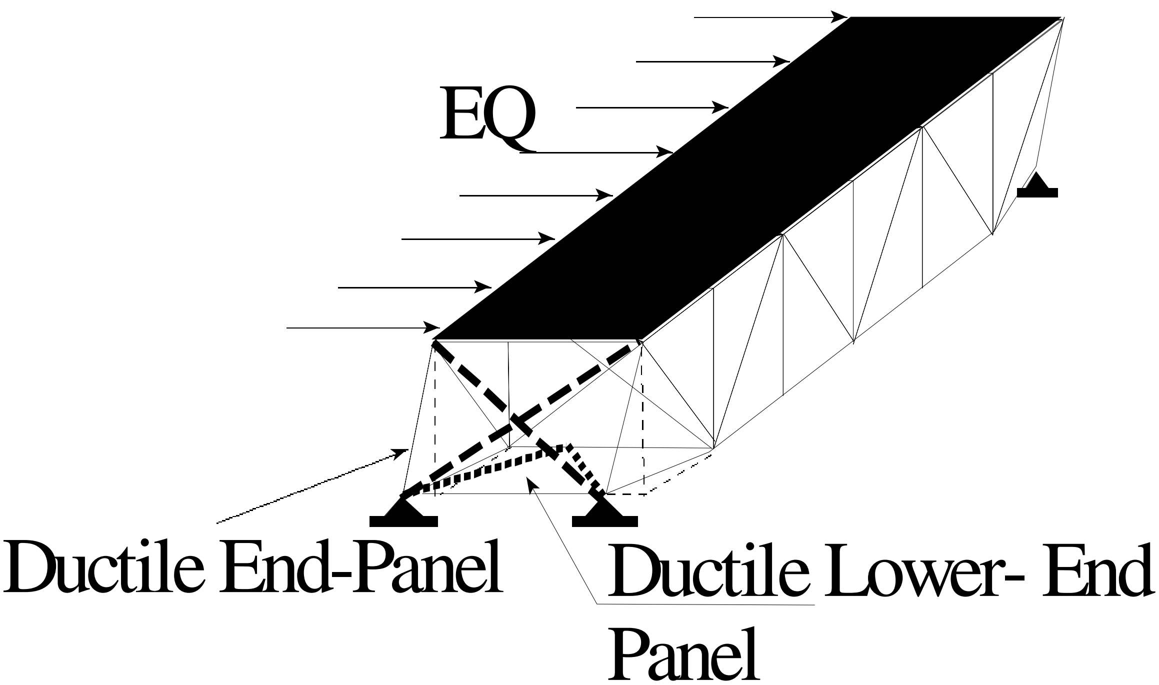


Figure 1. Retrofit concept in a deck-truss. (a) 2-D model of retrofitted bridge; (b) Implementing ductile retrofit in a deck-truss bridge

The procedure for design of the ductile devices is based on two main criteria: strength and stiffness. The yield strength of the ductile panels are selected to be lower than the capacity of the substructure and other superstructure to protect these components. The stiffness criteria established is based on the ductility capacity and drift limits of the superstructure. On the other hand, a very flexible device would result in large lateral displacements of superstructure and possible damage in the adjacent non-ductile members or their joints, on the other hand a very stiff device could have a substantial local ductility demand exceeding their ductility. Using the above criteria and an optimization process the end bracing and ductile components of an eccentrically braced frame and vertical shear links were designed for a 270-ft span deck-truss bridge (Sarraf and Bruneau ,1998b). These ductile devices were also designed and detailed to be used as retrofits for the scale model of a deck-truss bridge.

Analytical models of the prototype truss bridge were generated using DRAIN-3DX program, in which nonlinear behavior of the ductile shear links were modeled. A series of nonlinear time-history analysis was performed for 6 different earthquakes scaled to 0.53 g (El-Centro 1940, Northridge 1994, San Fernando, 1971, at Pacoma Dam, Loma Prieta, 1989, Olympia 1949 and Taft 1952). These ground motions were scaled to generate spectrum compatible motions with an average spectrum which is comparable to the mean-plus-one standard deviation of Newmark-Hall spectrum for PGA of 0.4g. The result of these analyses indicated that other than ductile components which yield and dissipate the induced seismic energy, no other superstructure members suffer damage, and the force response of the substructure does not exceed its capacity limit. Figure 2 shows the history of lateral force response at the end support for El Centro Earthquake as compared with the substructure capacity. Table 1 summarizes the nonlinear time-history results for all 6 earthquakes. Main design parameters to control performance of the device are: distortion angle of the shear links, the global ductility demand and the drift. As indicated in the table, the average global ductility for all 6 earthquakes does not exceed the global ductility capacity of ductile frames qualified for reduction factor of $R=10$ in accordance with UBC. Also, the distortion angle of a shear link as required by AISC-LRFD does not exceed the 9% rotation limit.

Ductile Retrofit System	Distortion Angle, γ (%) Distortion Allowable				Average Global Ductility Demand, μ				End Panel Drift (%)				Requirements Satisfied
	Min	Max	Ave	limit	Min	Max	Ave	limit	Min	Max	Ave	limit	
EBF T=0.49sec	1.5	3.6	2.6	9	1.47	3.8	2.7	3.8	0.3	0.7	0.5	2	Yes
TADAS T=0.8 sec	8	21	14	N/A	1.4	3.4	2.3	3.8	0.9	2.2	1.6	2	Yes
VSL T=0.54sec	1.5	4.9	3	9	1.88	3.5	2.7	3.8	0.5	1	0.7	2	Yes

Table 1 Summary of nonlinear time-history response of retrofitted bridge model to 6 earthquakes (PGA= 0.53g), checked against their permissible limits

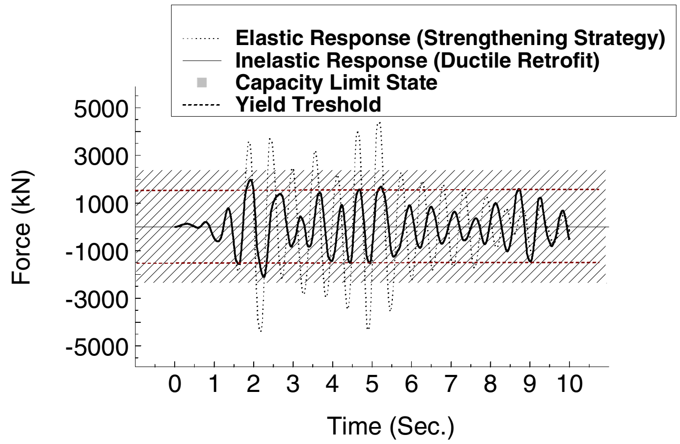


Figure 2 Lateral force history at the end supports limited to the capacity of ductile device

TEST SET-UP AND METHODOLOGY

A complete 1/10th model of 270-ft span steel deck-truss prototype bridge which was used earlier for seismic evaluation and retrofitting, was designed and constructed in the structures laboratory of the University of the Ottawa. Series of pseudo dynamic tests were conducted to verify analytical results, observe the actual performance of the retrofits, confirm that other than ductile retrofit devices no damage occurs in other members of the superstructure, and also to examine the effect of other factors, such as: P-delta effects and out of plane deformations of side diagonals which were difficult to model in the analytical model of the bridge structure.

Figure 3 shows the schematics of the model steel bridge and the test set-up used for the pseudo- dynamic testing. It is 27-ft long, 4-ft wide and 4-ft high. To generate the effect of nonuniform seismic loads at the deck level, an innovative testing technique was developed and used. In this technique a point load applied by one actuator is transformed to a desired distributed forces at the deck level. A load distributing beam is used whose length and stiffness were tuned so that its reaction forces can produce the same lateral deflection in the bridge as that developed in the predicted shape of the vibration. Also, one hydraulic jack positioned vertically is used to apply the gravity loads. To allow vertical, lateral and rotational movements of the bridge as it being push laterally, while maintaining the magnitude of the vertical load, 9 steel coil springs are placed under the vertically positioned hydraulic jack and a set of rollers are placed between the ram and the strong floor. Figure 4 shows a general view of the completed bridge model as-retrofitted and the test set-up components.

The bridge testing program was conducted in two phases. In the first phase, the bridge was tested in its as-built condition where end and lower end panels consisted of concentric bracing members. The free vibration test of the bridge resulted in the measured period of 0.25 sec. In the next stage, the pseudo-dynamic test was performed using the El-Centro ground accelerations scaled to 0.53 g. This magnitude of seismic forces resulted in buckling and yielding of the end cross bracing, bucking of tope lateral bracing and intermediate cross bracing members. This test confirmed the vulnerability of the deck truss members as predicted by nonlinear time-history analysis of the bridge using DRAIN-3DX program.

Damages were not too sever to require any repair or replacement of the members of the superstructure other than modification of the end panel members and retrofitting as planned.

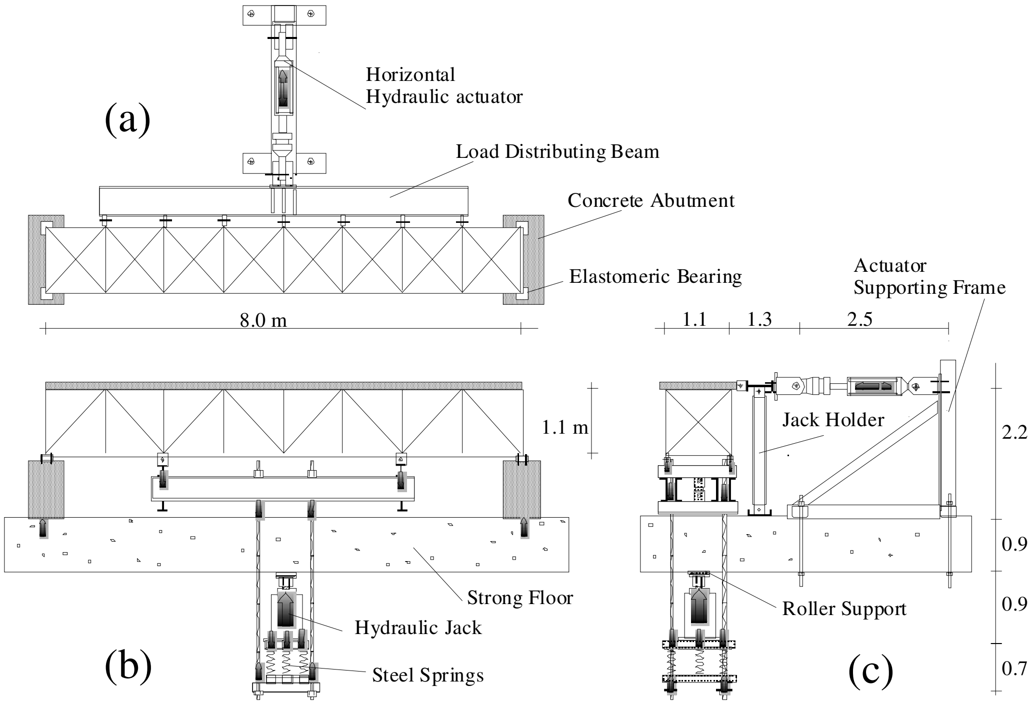


Figure 3 Plan and elevation views of pseudodynamic test set-up.

Figure 4 27-foot long deck-truss model in retrofitted condition and pseudo-dynamic test set-up at the University of Ottawa

DUCTILE STEEL SEISMIC RETROFITS

A specially designed ductile retrofit panel including stiff bracing members and ductile links was used to replace the existing end and lower end panel conventional cross-bracing. Figure 5 shows the details of the vertical link retrofit for the end-panel. The end panel and lower end panel connections were high-strength bolts with minimum hole clearance which were designed to have a negligible slip. Therefore the complete retrofit member assembly could be dismembered after testing the retrofits and the new ductile retrofit assembly would be replaced and tested. Figure 6 shows the end-panel retrofit using vertical shear link.

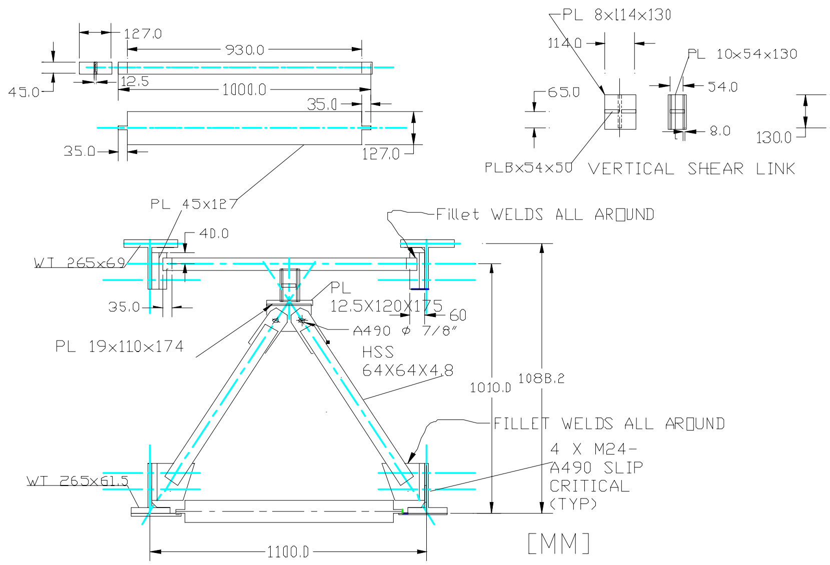


Figure 5 Eccentrically Braced Frame (EBF) retrofit details- End Panel

A 225-mm thick and 1100 mm wide reinforced concrete deck was cast in place. Shear studs were designed to be able to resist both forces in-plane shear force caused by both seismic loads as well as gravity loads. As a result 3/4" (20 mm) dia. welded shear studs at spacing of 250 mm were used, and total of 20 #20 longitudinal steel reinforcement bars were placed in the concrete deck to provided sufficient in-plane bending resistance against lateral loads.

Figure 6 Retrofitted end panels of deck-truss using ductile vertical shear link

PERFORMANCE OF DUCTILE RETROFITS

Initially free vibration tests and cyclic loading tests were performed to determine stiffness and strength characteristic of retrofitted bridge. Fundamental period of vibrations for EBF and VSL retrofitted bridge were measured as 0.26 sec and 0.23 sec, respectively. Cyclic tests resulted in measured yield strength of 450 kN in the EBF retrofit, and 400 kN in VSL specimen. Yielding of both end and lower- end panel devices were detected. The measured yield strength of the retrofits were greater than predicted load of 300 kN due to a number of factors such as: actual yield strength of the steel material, resistance contribution of other components such as connections of the end panel and the last side diagonal members of the truss, as well as a small horizontal component of the applied vertical load. However, despite the additional strength in the devices, no sign of yielding or buckling of the other members of the truss was observed.

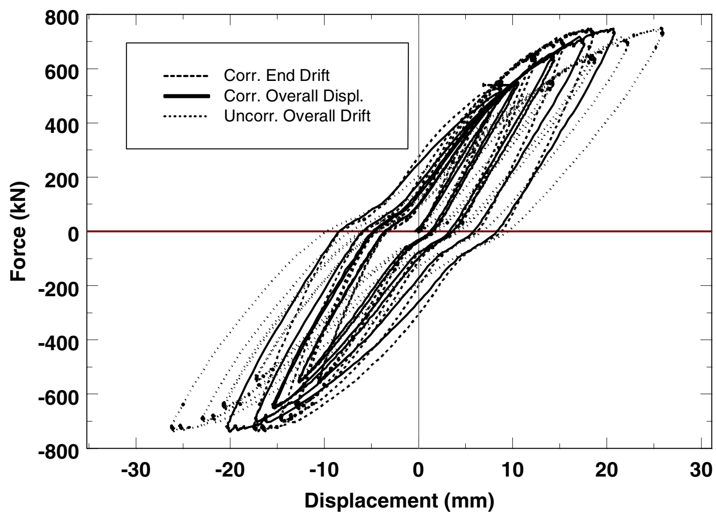


Figure 7 Force-Displacement response for vertical shear link retrofitted bridge

These cyclic tests were followed by pseudo-dynamic tests using El Centro earthquake ground motions scaled to peak ground acceleration of 0.5 g and 0.85 g. Overall response of the bridge was ductile with no damage observed in the cross-frames or lower lateral bracing members, considered the most vulnerable truss members. Similar ductile performance was observed for the same magnitude of El Centro Earthquake when the bridge was retrofitted with eccentrically braced frame. Figure 7 shows the force displacement curve obtained from pseudo dynamic test. The link beams exhibited strain ductilities as high as 12 and exhibited a robust hysteretic behavior (Figure 8).

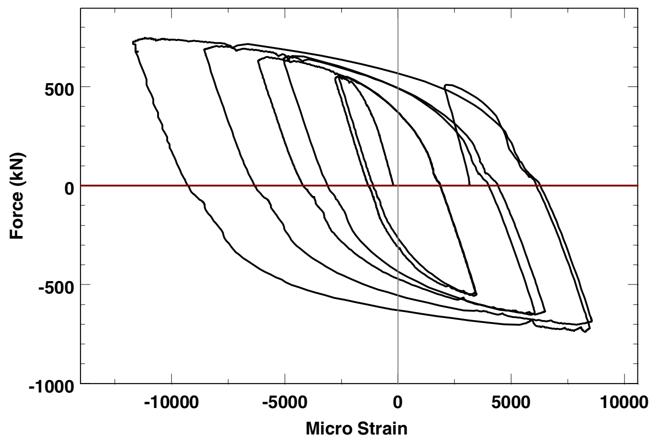


Figure 8 Hysteretic response of Vertical Shear Link retrofit

(a) (b)

Figure 9 Inelastic deformations of ductile retrofits: (a) Eccentrically Braced Frame , (b) Vertical Shear Link

Another important observation is the effect of continuity of the concrete deck and its contribution to the stiffness of top lateral bracing system. Discontinuity of the concrete deck in the as-built condition due to expansion joints does now allow the in-plane stiffness of the concrete deck to contribute to the stiffness of the top lateral system and more uniform distribution of the forces transferred to the intermediate cross bracing members. Casting composite concrete deck as part of retrofit measure also contributed to the stiffness of the top lateral bracing system. This was confirmed by the measurements of the lateral displacements along the deck and comparisons to the lateral displacements of the top chords during the as-built testing where no concrete deck was cast on the top chords. No shear failure of studs or cracking in the concrete was observed.

When 650 kN lateral loads or higher applied, much greater than the designed capacity of 300 kN, only negligible inelastic deformations were observed in intermediate bracing members which were already slightly overstressed during the as-built tests.

After successful completion of the pseudo-dynamic tests, a final cyclic test was performed for each retrofit to measure ultimate capacity of the ductile links. Both specimens exhibited substantial overstrength. Finally, the failure caused by the fracture of the welded connections to the yielding devices in the end panels, which were measured at 800 kN and 740 kN for EBF and VSL retrofitted bridge, respectively, sustaining a global displacement ductility of 3 and 2. Figure 9 shows the shear deformations of the link beam in EBF and VSL retrofits.

CONCLUSIONS

The results of pseudo-dynamic and cyclic tests performed on two different configurations of ductile energy dissipating devices (eccentrically braced frames, EBF and vertical shear link, VSL) used in a 27-ft long seismic retrofitted deck-truss bridge and for the El Centro earthquake scaled to 0.53 g, indicated that such devices can be designed and used as viable alternative seismic retrofit in deck-truss bridges.

The designed devices exhibited considerable cyclic ductility. By yielding and dissipating the induced seismic energy, these devices performed as structural fuses and protected other members of the superstructure. The devices exhibited substantial overstrength, however, which needs to be taken into account when determining the yield capacity of such protective systems to avoid overstressing other superstructural and substructural components.

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REFERENCES

1. Astaneh-Asl, A., B. Bolt, K. McMullin, D. Modjtahedi, S. Cho and R. Donikian.1994. “Seismic Performance of Steel Bridges during the 1994 Northridge Earthquake,” Report number UCB/CE-Steel-94/01, Department of Civil Engineering, University of California at Berkeley.

2. Bruneau, M., Wilson, J.W., Tremblay, R.1996. "Performance of Steel Bridges During the 1995 Hyogoken-Nanbu (Kobe, Japan) Earthquake", Canadian Journal of Civil Engineering, Vol.23, No.3, pp. 678-713

3. Housner, G., Thiel, C. 1995 “The Continuing Challenge: Report on the Performance of State Bridges in the Northridge Earthquake”,Earthquake Spectra, Vol. 11, No 4: 607-636.

4. Imbsen, R., and Liu, W. 1993. “ Seismic Evaluation of Benicia-Martinez Bridge”, Proceeding of the First U.S. Seminar, Seismic Evaluation and Retrofit of Steel Bridges, University of California at Berkeley.

5. Matson, D.D., and Buckland, P.G. 1995. “ Experience with Seismic Retrofit of Major Bridges”, Proceedings of 1st National Seismic Conference on Bridges and Highways. San Diego, California.

6. Sarraf M. and Bruneau M. 1998a.“Ductile Seismic Retrofit of Deck-Truss Bridges. I: Strategy and Modeling”Journal of Structural Engineering, ASCE, Vol. 124, No. 11, November 1998, pp. 1253-1262.

7. Sarraf M. and Bruneau M. 1998b.“Ductile Seismic Retrofit of Deck-Truss Bridges. II: Design Applications” Journal of Structural Engineering, ASCE, Vol. 124, No. 11, November 1998, pp. 1263-1271.