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Innovative Application of Ductile Systems in Seismic Retrofit of Deck-Truss Bridges

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ABSTRACT

For many years deck-truss bridges were designed without due consideration given to seismic loads. Seismic evaluations conducted for number of existing bridges confirm their vulnerability and possibility of sever damage suffered in the event of large earthquakes. The proposed innovative retrofit solution is to use ductile devices as "structural fuses" to protect both superstructure and substructure of these bridges. Ductile panels of EBF, TADAS, and VSL were designed as fuses to replace end and lower-end panels of an 80-m span example bridge. This paper explains sources of seismic vulnerability, the proposed retrofit solution and pertaining design methodology, and summarizes the improvements achieved in seismic behaviour of retrofitted bridge as proposed.

INTRODUCTION

Many steel bridges would suffer severe damage when struck by earthquakes, as evidenced by some recent moderate earthquakes (Astaneh-Asl et al. 1994, Housner et al. 1995, Bruneau et al. 1996). Among all types of steel bridges, deck-truss bridges are particularly vulnerable. Typically, in these structures, the deck is seated on the truss structure, itself supported on abutments or piers. Hence, seismically induced inertia force at the deck acts with a sizable eccentricity with respect to the truss reaction supports, and the entire superstructure is mobilized to transfer these forces from deck to supports. In this type of bridge, lateral-load resisting members were originally designed to resist only wind-induced forces, and are typically found slender; Therefore, they lack enough resistance to the seismic forces, and fail to develop a reliable ductile cyclic behaviour expected in a large earthquake event. Moreover, these spans are often supported on non-ductile substructures.

Recent seismic evaluations of major crossings have confirmed such deficiencies and often proposed to replace and/or reinforce many of the superstructure members and substructure components. This can be expensive, particularly when work is required on the difficultly accessible components of the bridge. While in strengthening approach usually a conservative scenario earthquake is considered for design, there is no guarantee that the actual future earthquake be of lower magnitude than expected, thus, adequacy of added strength would be questionable. In most cases, a base isolation system has been recommended as the retrofit solution for deck-truss bridges. However, while this solution can be considered effective, it can be costly, for it often requires extensive abutment modifications and superstructure changes. In fact base isolation solution may become more expensive than conventional strengthening to implement. (Imbsen, and Liu, 1993; Capron 1995; and Matson, and Buckland, 1995). Moreover, base isolation systems do not appear to be as effective for near-filed events producing high velocity pulses.

The alternative and potentially more economical solution proposed here, is a capacity based seismic retrofit solution to protect both the superstructure and substructure by using ductile steel energy-dissipating devices implemented at judiciously selected locations in the superstructure. Such devices can as structural fuses, reliably dissipate seismically induced energy and prevent yielding in other parts of the structure.

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This paper addresses typical seismic deficiencies in deck-trusses, explains the methodology for the proposed retrofit design using ductile devices, and presents design examples using ductile devices of vertical shear links (VSL), triangular-plate added damping and stiffness device (TADAS) systems, and eccentrically braced frames (EBF), as well as presenting numerical results.

SEISMIC VULNERABILITY OF DECK-TRUSS BRIDGES

Nonlinear time-history analyses of a generic deck-truss bridge were conducted using the program DRAIN-3DX (Prakash et. al. 1994) to identify the location and magnitude of structural deficiencies, and allow full appreciation of how effectively the proposed retrofit strategy can enhance seismic performance. A computer model of the bridge was constructed using information taken from structural drawings supplied by practicing engineers under confidentiality agreements. It has an 80 m span, is 10m tall and 10m wide. The truss panels on all sides are 10m × 10 m in size. The 225 mm thick concrete deck is discontinuous due to the presence of expansion joints at each panel joints (10 m). The bridge is supported by two roller supports bearing on one abutment at each end. Vertical and transverse displacements of the joints are restrained while horizontal displacements in longitudinal axis of the truss is permitted at one end. The truss members are modeled as link elements with elastic buckling behavior in compression, and elasto-perfectly-plastic behavior with 3% strain hardening in tension. Modal analysis of the truss showed the dominant dynamic motion corresponds the first lateral mode of vibration with fundamental period of T= 0.57 sec.

The result of analysis of the bridge subjected to the El Centro 1940 N-S ground motions scaled to a PGA of 0.5g revealed that many end cross-frame braces, verticals, top laterals, interior cross-frame braces, and lower laterals, buckled and yielded, with member ductility demands sometimes in excess of 8. The locations of damage throughout the bridge are identified in the exploded view of the deck-truss shown in Fig. 1.

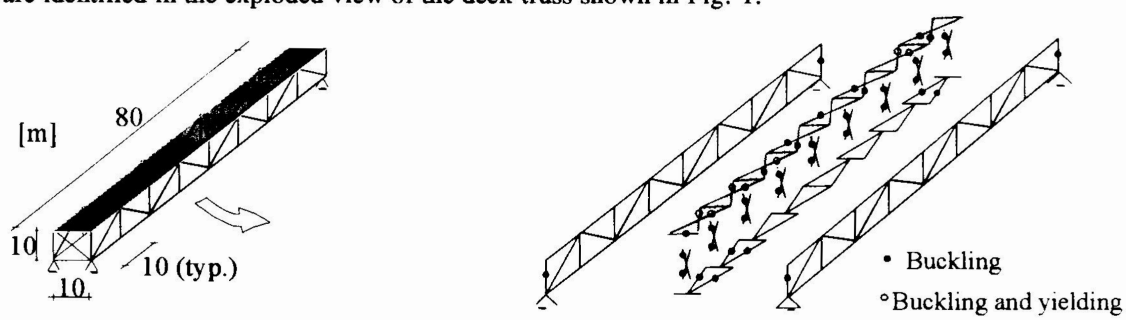


Figure 1. Damaged members of deck-truss

Also, from the displacement history of the end and mid-span joints, It was observed that magnitude of displacement at mid-span is significantly greater than those at the ends. This indicates that a considerable flexibility exist in the top lateral resisting system whereby a greater share of the seismically induced inertia force flowing into the lower laterals through the interior cross-frames (i.e. sway-frames).

Note that for this analysis other factors affecting the seismic performance of the bridge, such as deficiencies in connection of the truss and abutments, are ignored. Thus, more intense damage can be expected in an actual truss bridge.

DUCTILE RETROFIT CONCEPT

Retofit concept is best visualized by 2-D presentation of the truss lateral load resisting system. Fig. 2a shows a 2-D model in which upper beam and lower beam each represents the truss top lateral, and bottom lateral system, respectively, and interconnecting springs representing the panel stiffness of the interior cross-frames. Thus existing lateral load resisting system of a deck-truss consists of two load paths which can interact through interior cross-frames. Having considered a lateral load on the top beam as effect of inertia loads, implementing two ductile yielding devices as fuses at each end of the two beams can limit the magnitude of forces transferred to the lower beam and the interface springs, the top beam, and end support reactions. The implementation of this concept in a bridge is shown in Fig. 2b. It requires conversion of each end cross-frame into a ductile panel having a specially detailed yielding device (i.e. a structural fuse), and conversion of the last lower end panel near each support into a similar ductile panel. It also requires stiffening of the top truss system, which can be achieved by connecting the existing individual concrete deck panels to create a continuous deck. This stiffening has two benefits. First, for a given deck lateral displacement at the supports, it reduces mid-span sway, resulting in lower forces in the interior cross-frames. Second, it increases the share of the total lateral load transferred by the top load path. Interestingly, once the deck is made continuous and well tied to the truss, the in-plane flexural stiffness of the top truss becomes sufficiently large to be modeled as a rigid beam in the 2-D model. This greatly simplifies modelling and obtaining the generalized stiffness for the retrofitted structure.

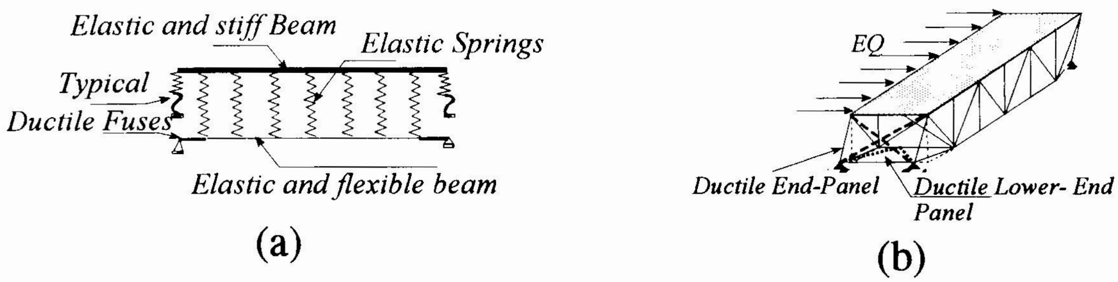


Figure 2. Retrofit concept in a deck-truss. (a) 2-D model of retrofitted bridge; (b) Implementing ductile retrofit in a deck-truss bridge

DESIGN OF DUCTILE RETROFIT DEVICES

Ductile devices to be qualified as structural fuses, must have appropriate stiffness and strength values. General selection criteria developed based on the capacity design are explained below. A more detailed design procedure is given in Sarraf and Bruneau (1998a).

Strength

In a capacity design perspective:

- Strength of the retrofitted end cross-frames must be chosen to ensure that the horizontal transverse force transferred to these panels does not produce buckling of the end-verticals, nor exceed the resistance of the tie down devices. An upper limit for the transverse shear strength of each end cross-frame panel can be determined as the maximum shear corresponding to buckling of end verticals;

- Strength of the retrofitted lower end panels must be chosen to limit the force demand in the interior cross-frames and lower truss. To correlate force demand in interior cross-frames to the lower end shear a method proposed by Sarraf and Bruneau (1998a) is recommended.

- Total strength of both ductile panels combined must not exceed the capacity of substructural elements, such as: bearings and piers, and must be greater that the strength needed to resist wind load.

Stiffness

As dynamic response of the bridge varies with global stiffness, to control deformations in non-yielding elements as well as limiting ductility demand on the ductile device, the following displacement limits are imposed, and later converted to stiffness limit on the ductile panel to prevent excessive ductility demands in the retrofitted panels and excessive drift and deformations in non-ductile components. Generally, the maximum permissible lateral displacement of the deck must not exceed the following:

- Allowable panel displacement without risking end verticals stability in sway mode, or damage to the connections;
- Allowable deformation prior to inelastic distortion of gusset plates, premature bolt or rivet failures, or damage to structural members in lower end panel;
- Drift limit prescribed by the highway bridge codes (optional);
- Allowable deformation limit for the devices used in the ductile panels.

Also, to ensure that yielding in two end and lower end ductile panels occurs simultaneously, It is suggested to keep stiffness of the retrofitted panels proportional to their respective capacities.

SELECTION OF GENERALIZED STIFFNESS AND PERIOD

Using a further simplified 2-D model of the bridge shown in Figure 3, and considering relatively large rigidity of the top truss system resulted from its conversion into a continuous composite concrete deck, the generalized stiffness of the truss bridge as a function of its retrofit panel stiffness is:

$$K_{Global}=2\,(K_{E,S}+K_{L,S}) \tag{1}$$

where $K_{E,S}$ is the stiffness of the retrofitted end cross-frames, taking into account the contribution to stiffness of the braces, verticals, horizontal, and ductile energy dissipation device, and $K_{L,S}$ is lower lateral system stiffness including lower end panel stiffness.

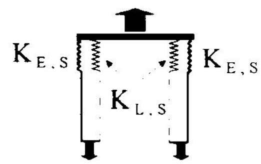


Figure 3. Global stiffness model for deck-truss

The fundamental period for the transverse mode of vibration is given by:

$$T=2\pi\sqrt{\frac{M}{K_{Global}}} \tag{2}$$

Where M is the total mass of the deck. The admissible range of period, T, is determined as follows.

-Determination of Capacity-Based Pseudo-Acceleration:

Having established an upper and lower bound of the total strength, R_{total}, for the retrofitted system, a median value can be chosen as the desired capacity of the system. The term "*Capacity-Based Pseudo Acceleration*", PSa_C, can be defined as the pseudo-acceleration which can cause yielding of an elasto-perfectly plastic SDOF system of mass M. It can be calculated as:

$$PSa_c=\frac{R_{total}}{M} \tag{3}$$

-Dynamic Response and Determination of Period (Global Stiffness) Limits:

The procedure proposed here uses the tripartite representation of the Newmark-Hall design elastic and inelastic response spectra combined with a new line having constant pseudo-acceleration, Psa_C. As shown in Figure 4, for a SDOF system having a given yield strength which can be represented by this line, ductility demand, μ, varies with the period. In the intermediate period range, the ductility demand of systems having a constant strength decreases as the period increases (i.e. as stiffness decreases), while their displacement response increases. Therefore, depending on the permissible displacement of the system and ductility capacity of the system, a range for admissible values of, T, can be found.

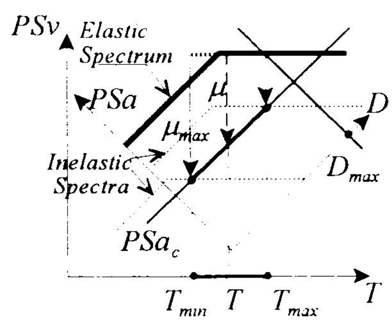
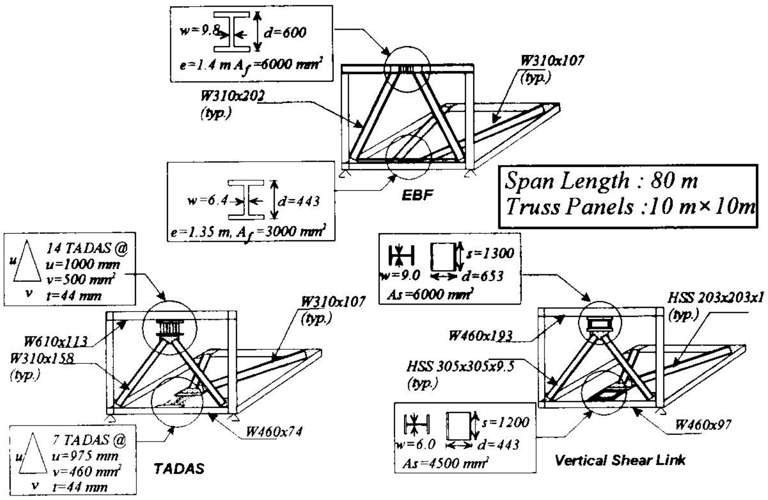


Figure 4. Capacity-Based Pseudo Acceleration line and determining admissible limits for period

Once an admissible range for period, T, is established, using above equations (1) and (2), they can be converted into limits on stiffness of the ductile panels. Subsequently, in addition to the above limit on stiffness, another set of inequalities will be formed to ensure that other requirements pertaining to the design specifications of a particular ductile device are also satisfied. A graphical design approach suggested in Sarraf and Bruneau (1998,b).

Figure 5 shows the details of end and lower-end ductile panels designed for the 80-m span deck-truss example.



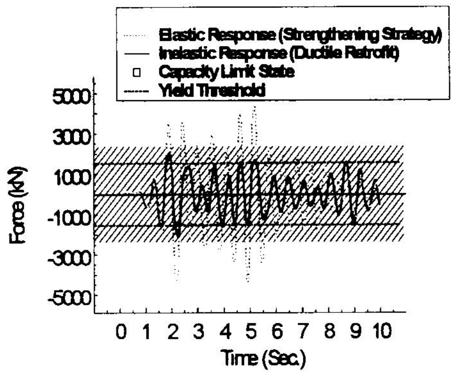


Figure 5. Details of ductile retrofit devices

Figure 6. Horizontal reaction force response of deck-truss to 0.6g El Centro.

RESULTS

A series of nonlinear inelastic time history analyses was conducted using DRAIN-3DX to investigate the improvements made in behaviour of the retrofitted truss. Results for 6 Western United States ground motions scaled to PGA of 0.6 g, indicated an average ductility demand, μ, of 2.68 which is less than the maximum allowable ductility μ_{max} = 3.75 for three retrofits, and a maximum end-panel displacement of 0.054 m which is less than the allowable drift, D_{max} = 0.18 m). As expected, there was no yielding or buckling of truss members, other than those of ductile devices. Moreover, reaction forces at the truss supports did not exceed the allowable force, thus no excessive force was exerted to the substructure. In Figure 7, reaction force history of the retrofitted truss is compared to that of the strengthened truss bridge, behaving elastically.

EXPERIMENTAL PROGRAM

A complete 1/10 scale steel model of 80m-span deck-truss bridge used earlier for evaluation and retrofit has been designed, and constructed in the structures laboratory of the University of the Ottawa. Series of pseudo dynamic tests will be conducted to verify analytical results, observe the actual performance of the retrofitted bridge and evaluate effect of possible factors, such as: Geometric non-linearity, and in-plane and out of plane joint deformations which were difficult to consider in the computer model of the structure.

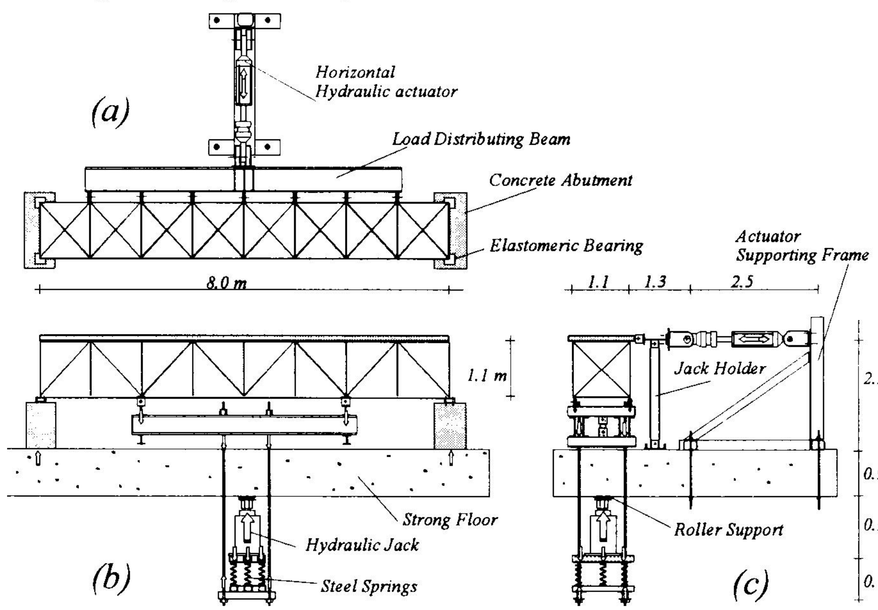


Figure 7. Test set-up and the specimen; (a) Plan view, (b) Vertical loading system, (c) Horizontal loading system

Figure 7 shows an overall view of the bridge model, and the pertaining test set-up. To simulate effect of nonuniform seismic loads at the deck level, an innovative technique was developed and used. To transform a point load applied by one actuator to 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 simulate effect of gravity loads. To allow vertical, lateral and rotational movement of the bridge while maintaining the vertical load, 9 coil springs and rollers are placed between the vertical jack.

CONCLUSIONS

An innovative retrofit solution is proposed based on capacity design principal and requires only minimum structural interventions to implement. Accordingly, a design methodology developed to determine stiffness and strength of ductile retrofit panels and used to design retrofits for an 80-m span deck-truss bridge, using EBF, VSL and TADAS devices as ductile panels. Results of non-linear time history analyses performed for the retrofitted bridges for six earthquakes proved the satisfactory performance of ductile seismic retrofit systems.

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