

INNOVATIVE APPLICATION OF DUCTILE SYSTEMS IN SEISMIC RETROFIT OF DECK-TRUSS BRIDGES

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SUMMARY

For many years deck-truss bridges were designed without consideration of seismic loads. Seismic evaluations conducted for a number of existing bridges confirm their vulnerability, indicating that they may suffer severe damage in the event of a large earthquake. The proposed retrofit solution consists of introducing ductile devices as "structural fuses" to protect both the superstructure and substructure of these bridges. Ductile panels having such fuses are designed to replace the end and lower-end panels of deck-truss bridges. This paper describes the sources of seismic vulnerability, the proposed retrofit solution and design methodology, and summarizes the improvements achieved in seismic behavior for an example deck-truss bridge retrofitted as proposed.

INTRODUCTION

For many years deck-truss bridges were designed without consideration of seismic loads. Seismic evaluations conducted for a number of existing bridges confirm their vulnerability, indicating that they may suffer severe damage in the event of a large earthquake. The proposed retrofit solution consists of introducing ductile devices as "structural fuses" to protect both the superstructure and substructure of these bridges. Ductile panels having such fuses are designed to replace the end and lower-end panels of deck-truss bridges. This paper describes the sources of seismic vulnerability, the proposed retrofit solution and design methodology, and summarizes the improvements achieved in seismic behavior for an example deck-truss bridge retrofitted as proposed.

Recent seismic evaluations of deck-truss bridges have typically revealed the need to replace and/or reinforce many of the superstructure members and substructure components. This can be expensive, particularly when work is required in difficultly accessible parts of the bridge. Approaches that focus on strengthening rather than ductility also require conservative scenarios of future earthquake occurrence, because they cannot guarantee adequate seismic performance beyond the threshold of damage. In most cases, base isolation has been recommended for deck-truss bridges. However, while effective, this retrofit strategy can also be costly as it sometimes require extensive abutment modifications and superstructure changes; in some projects, base isolation proved to be more expensive than conventional strengthening

(e.g. Imbsen, and Liu, 1993; Capron 1995; and Matson and Buckland, 1995).

To provide for an alternative and potentially more economical solution, a capacity based seismic retrofit procedure is proposed here to protect both the superstructure and substructure, by introducing ductile steel energy-dissipating elements at judiciously selected locations into the superstructure. These special ductile elements, detailed to reliably dissipate seismic energy through the development of stable hysteretic behavior, are used as a mechanism to prevent yielding in other parts of the structure, thereby acting as structural "fuses". To achieve this capacity protection in steel deck-truss bridges, the retrofit scheme proposed here requires conversion of the deck slab into a composite slab (as recently done to some bridges in Canada), replacement of the end

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cross-frames by special ductile panels, and replacement of the last lower-lateral braced panels near the piers by similarly conceived ductile panels. These specially designed ductile diaphragms and panels are calibrated to yield before the strength of the substructure is reached, and prior to the development of any undesirable failure modes in the superstructure. Contrary to some of the existing alternatives, retrofit work is only required at easily accessible superstructure locations.

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 consisting of vertical shear links (VSL), triangular-plate added damping and stiffness device (TADAS) systems, and eccentrically braced frames (EBF). Time history results to compare the behavior of retrofitted and non-retrofitted trusses are also presented.

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 along the 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. A fundamental period of vibration of 0.57 sec. in the transverse direction was obtained from modal analysis of this truss.

Analysis of the bridge subjected to the El Centro 1940 N-S ground motion 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. Note that these simple analyses neglect many other deficiencies commonly encountered in deck-truss bridges (e.g. connections unable to develop member strengths) that would require consideration by the engineer in assessing the seismic retrofit needs.

DUCTILE RETROFIT CONCEPT

The proposed retrofit concept is best visualized by a 2-D presentation of the truss lateral load resisting system. Fig. 2a shows a 2-D model in which the upper beam and lower beam represent the truss top lateral, and bottom lateral systems, respectively, and the interconnecting springs represent the panel stiffness of the interior cross-frames. Thus the lateral load resisting system of a deck-truss consists of two load paths linked by the interior cross-frames. To control the magnitude of the seismically-induced lateral load acting on the top beam, ductile yielding devices must be introduced at each end of the two beams. This implementation is illustrated in Fig. 2b. This is practically achieved by converting each end cross-frame into a ductile panel having a specially detailed yielding device (i.e. a structural fuse), and the last lower end-panel near each support into a similar ductile panel. It also requires stiffening the top truss system, which can be achieved by converting the existing deck into 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 of Fig.2. This greatly simplifies modeling and calculation of the generalized stiffness for the retrofitted structure.

DESIGN OF DUCTILE RETROFIT DEVICES

Seismic performance of the retrofitted superstructure varies as a function of the values of stiffness and strength of the ductile end and lower panels. Thus, these values must be carefully determined by the engineer. Any type of ductile energy dissipation system could be implemented in the end panels and lower-end panels of the deck-truss, as long as its stiffness and strength characteristics satisfy the following requirements.

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 than the strength needed to resist wind load.

Stiffness

Some restrictions on stiffness are necessary to prevent excessive ductility demands in the retrofitted panels and excessive drift and deformations in other parts of the superstructure. The engineer must identify the displacement constraints appropriate to specific bridges; these will vary depending on the detailing conditions germane to the particular bridge under consideration. Generally, among those limits of important consequences, the maximum permissible lateral displacement of the deck must not exceed the values at which:

- Unacceptable deformations start to develop in members or connections of the deck-truss, such as inelastic distortion of gusset plates, or premature bolt or rivet failures;
- P- Δ effects causes instability of the end-verticals during sway of the end panel;
- Maximum code drift limits given in the highway bridge codes, if any, are reached (optional);
- The energy dissipating device used in the ductile panels reach their maximum deformation without loss of strength.

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

Highlights of a detailed design procedure, given in Sarraf and Bruneau (1998a; 1998b), follows.

SELECTION OF GENERALIZED STIFFNESS AND PERIOD

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

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where $K_{E,S}$ is the stiffness of the retrofitted end cross-frames, taking into account the contribution to stiffness of the braces, verticals, horizontals, and ductile energy dissipation device, and $K_{L,S}$ is the lower lateral system stiffness including the lower end panel stiffness (Sarraf and Bruneau 1998a).

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

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where M is the total mass of the deck.

Determination of Period (Global Stiffness) Limits

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", P_{SaC} , 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:

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A line corresponding to that constant capacity-based pseudo-acceleration can be drawn on a Newmark-Hall tripartite response spectra, as shown in Fig. 4. For a SDOF system having a given yield strength (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.

Once an admissible range for period, T , is established, it can be converted into limits on stiffness of the ductile panels. Strength and stiffness limits are then used to design the ductile panels. Additional constraints germane to each ductile system or device must also be considered. Many types of structural systems or ductile devices can be used to provide the ductile panels. Vertical shear links (VSL), triangular-plate added damping and stiffness device (TADAS) systems, and eccentrically braced frames (EBF), are a few systems that provide energy dissipation by ductile yielding of structural steel, and that have been used effectively in many building applications. Based on that experience, these three systems were considered here. Interestingly, the first design attempts revealed complications due to the large number of strength, drift, and ductility demand limits that must be respected to achieve the design objectives, and because the seismic load paths and hierarchy of yielding in deck-truss bridges differ from what is assumed in multi-storey frames. A systematic procedure that recognizes the above conditions was developed and found useful, and sometimes necessary, to design the retrofits (Sarraf and Bruneau 1998b). Note that solutions that respect all constraints are not always possible unless the engineer is willing to violate some of the aforementioned constraints. Some types of ductile systems are also considerably simpler to design and are more suitable for this application.

Figure 5 shows the retrofit details of end and lower-end ductile panels designed for a 80-m span deck-truss example. Nonlinear inelastic time history analyses of one such retrofitted bridges, using DRAIN-3DX and 6 Western United States ground motions scaled to PGA of 0.6 g, showed a resulting average ductility demand, μ , of 2.68 (i.e. less than the maximum ductility 3.75 allowed for this type of retrofit), a maximum end panel displacement of 54 mm (again, less than the allowable drift of 180 mm). As expected, there was no yielding or buckling of truss members, other than yielding of the 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 6, the reaction force history of the truss retrofitted using the approach proposed here is compared to that of a truss bridged strengthened to behave elastically.

EXPERIMENTAL PROGRAM

A complete 1/10 scale steel model of the 80m-span deck-truss bridge used in the above example 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 factors that were difficult to consider in the computer model of the structure, such as geometric non-linearity, and in-plane and out of plane joint deformations. Figures 7 and 8 respectively show an overall view of the bridge model and its test set-up, and the specimen during construction. Note that a transfer beam is used to transform the point load applied by one actuator into the desired nonuniform horizontal seismic distributed forces at deck level. Its length and stiffness were selected to produce the same lateral deflected shape of the bridge as that predicted from analyses. Gravity loads are applied by another actuator positioned vertically. Coil springs, rollers and hinges within the gravity load distributing system allow unrestricted vertical, lateral and rotational movement of the bridge under seismic response.

CONCLUSIONS

A capacity-design based solution is proposed for the seismic retrofit of deck-truss bridges. It only requires the introduction of special ductile energy dissipating devices in the end panels and lower end-panels of the bridge superstructure, and thus minimizes structural interventions. A design methodology has been developed to determine the required stiffness and strength of the ductile retrofit panels, and has been used in an analytical retrofit-example for an 80-m span deck-truss bridge, considering EBF, VSL and TADAS devices in the ductile panels. Non-linear time history analyses performed for the retrofitted bridges for six earthquakes demonstrated the satisfactory performance of ductile seismic retrofit systems.

ACKNOWLEDGEMENT

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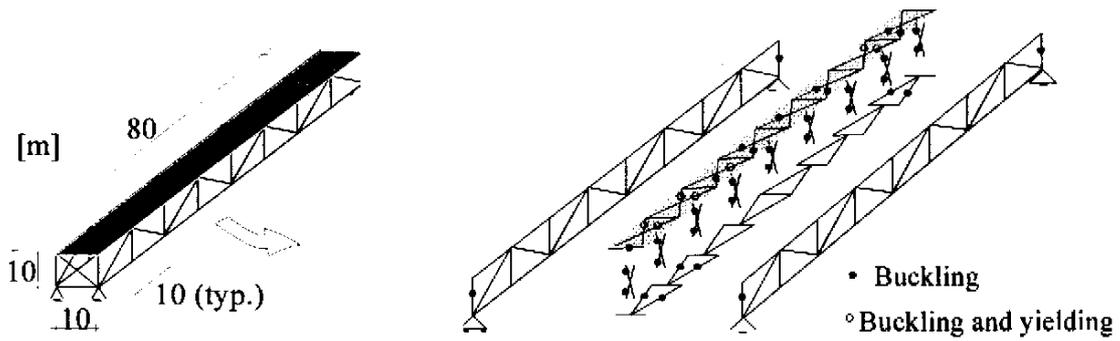


Figure 1: Damaged members in example non-retrofitted deck-truss bridge

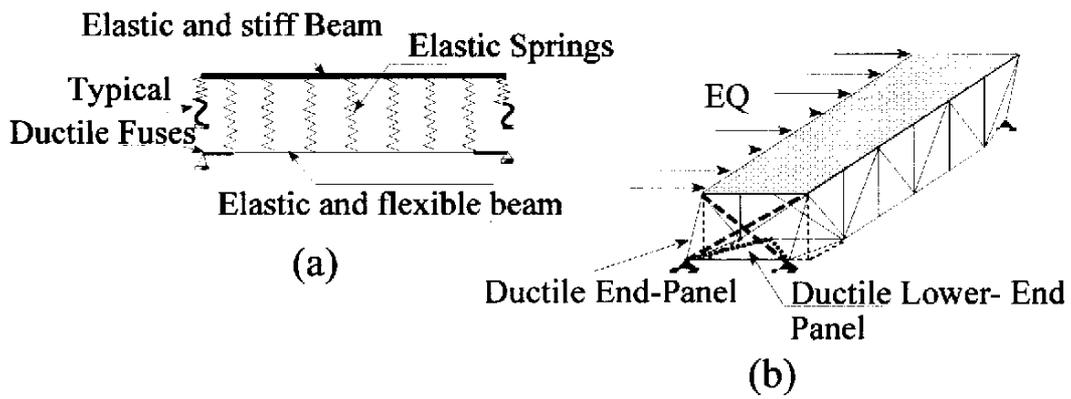


Figure 2: Deck-truss retrofit concept (a) 2-D model; (b) 3-D implementation of ductile retrofit

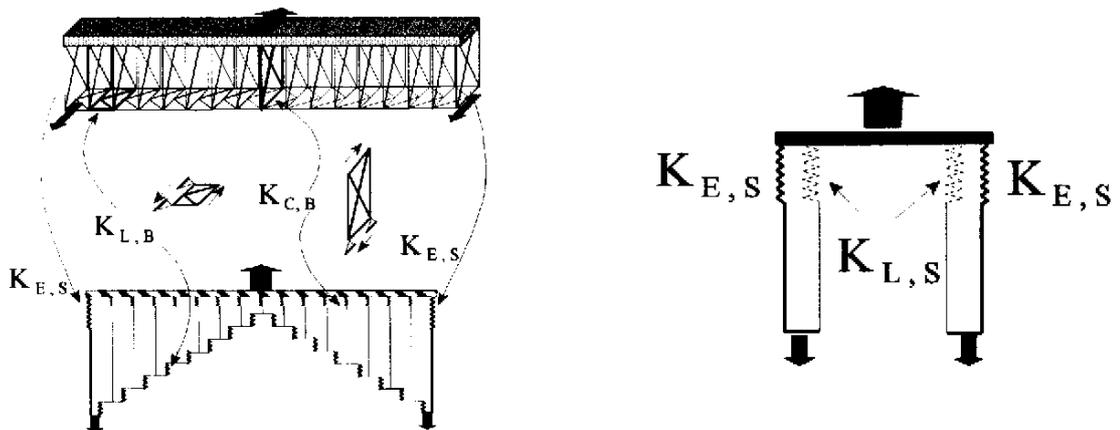


Figure 3: Modeling of deck-truss (a) From 3-D into a 2-D model (left), and; (b) Simplification of the cross brace panel stiffness ($K_{C,B}$) and lower lateral braced panel ($K_{L,B}$) into the lower lateral system stiffness ($K_{L,S}$) (right)

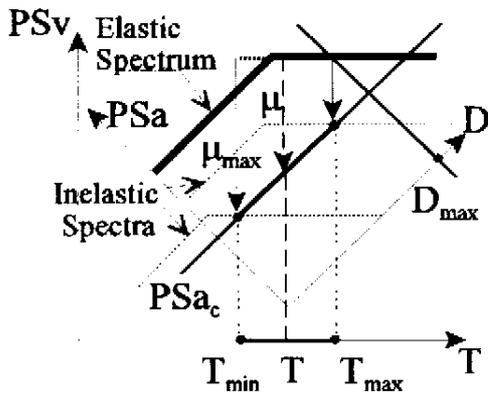


Figure 4: Capacity-based pseudo acceleration line and range of admissible periods

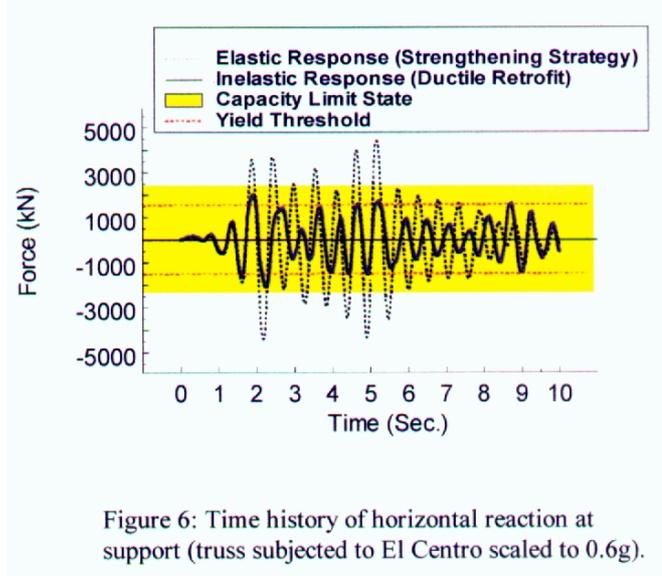


Figure 6: Time history of horizontal reaction at support (truss subjected to El Centro scaled to 0.6g).

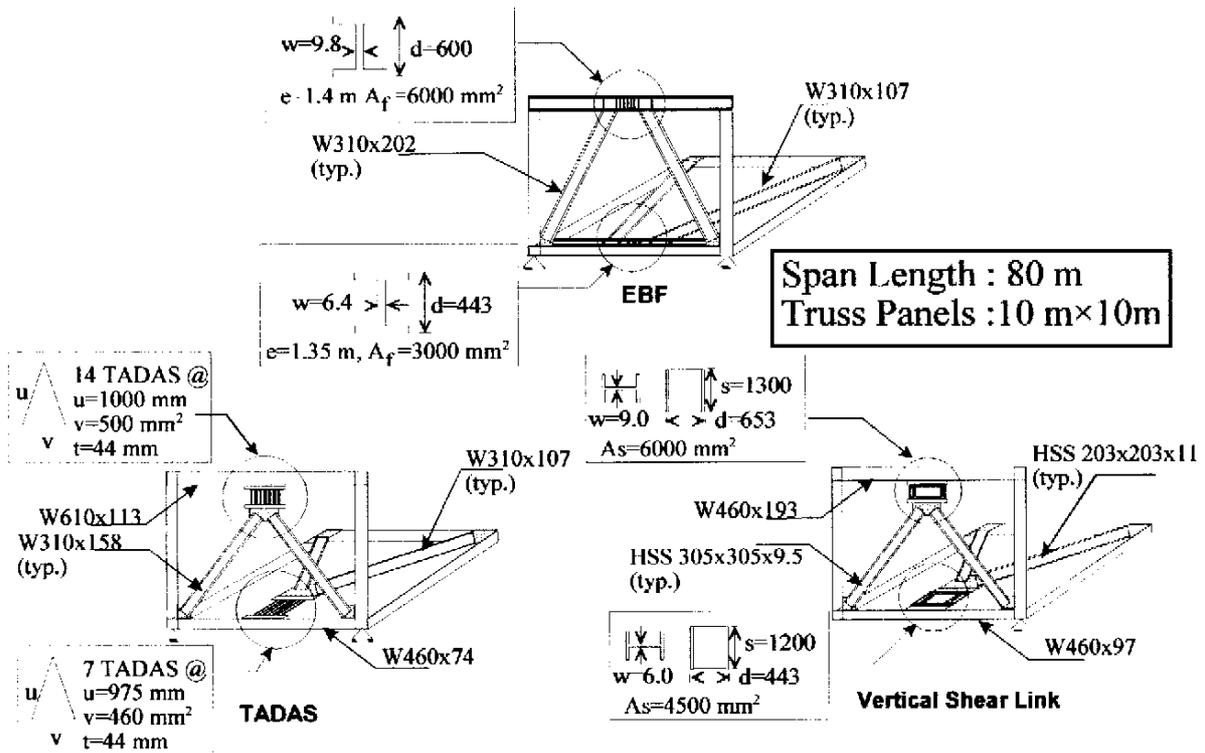


Figure 5. Details of ductile retrofit devices

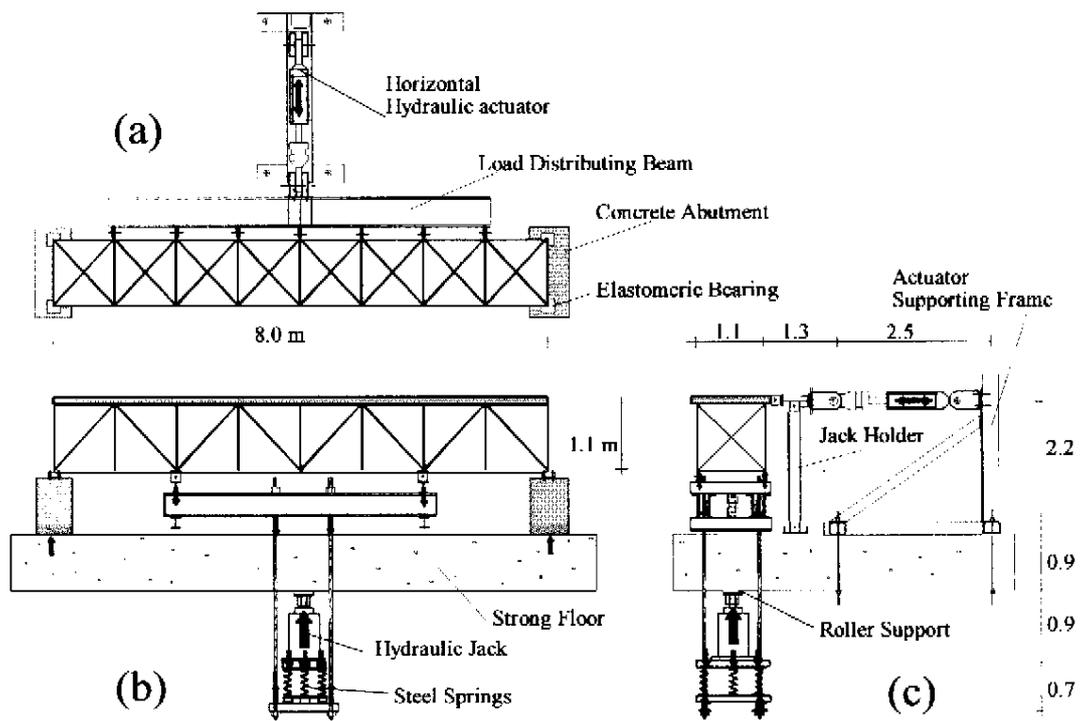


Figure 7. Test set-up and specimen; (a) Plan view, (b) Vertical loading system, (c) Horizontal loading system



Figure 8: Deck-truss 1:10 model (without the ductile retrofit devices) during construction.