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Seismic Design

DUCTILE END-DIAPHRAGMS TO SEISMICALLY RETROFIT SHORT AND MEDIUM SPAN STEEL BRIDGES

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Summary

A seismic retrofit strategy using ductile steel bridge end-diaphragms is presented here. By replacing the existing steel diaphragms over abutments and piers with specially designed ductile diaphragms calibrated to yield before the strength of the substructure (and superstructure for deck-truss bridges) is reached, these can be protected. For slab-on-girder bridges, stiffness of the girders must also be considered, as it has an impact on behaviour. For deck-truss bridges, conversion of the deck slab into a composite slab, replacement of the end sway-bracing panels by special ductile diaphragms, and replacement of the lower-lateral braced panels adjacent to the piers by similar ductile panels are required. In both cases, however, access to the superstructure locations requiring retrofit is relatively easy, compared to the alternative retrofit strategies that may require replacement or reinforcing of piers, bearings and many structural members.

Computer simulations of the dynamic behaviour of the retrofitted bridges subjected to severe ground excitation showed satisfactory performance, and validated the analytical procedure for the proposed retrofit strategy. Experimental results obtained for slab-on-girder bridges indicate that welded ductile diaphragms perform as intended.

INTRODUCTION

Recent earthquakes have demonstrated the seismic vulnerability of steel bridges supported by non-ductile substructure elements. While damage to superstructure components of these bridges is also possible, damage to substructure components such as abutments, piers, bearings, and others have proven to be of far greater consequences, often leading to span collapses. Hence, when existing bridges are targeted for seismic rehabilitation, much attention is paid to these substructure elements. Typically, the current retrofitting practice is to either strengthen the existing non-ductile members, enhance their ductile capacity, replace them by newer elements having more desirable properties, or reduce the force demands on the vulnerable substructure elements using base isolation techniques or other structural modifications. While all these approaches are proven effective solutions, only the base isolation concept recognizes that seismic deficiency attributable to substructure weaknesses may be resolved by operating elsewhere than on the substructure itself.

An alternative seismic retrofit strategy using ductile steel bridge end-diaphragms is presented here. This concept has evolved from earlier research conducted at the University of Ottawa on the seismic evaluation of slab-on-girder steel bridges and on the role of end-diaphragms on seismic performance. By replacing the steel diaphragms over abutments and piers with specially designed ductile diaphragms calibrated to yield before the strength of the substructure is reached, the substructure can be protected. This retrofit concept, so far, has been developed for a common type of slab-on-girder steel bridges, and for deck-truss bridges. The latter case is more complex, as it requires conversion of the deck slab into a composite slab, replacement of the end sway-bracing panels by special ductile diaphragms, and replacement of the lower-lateral braced panels adjacent to the piers by similar ductile panels. However, access to the superstructure locations requiring retrofit is relatively easy, compared to the alternative retrofit strategies that may require replacement or reinforcing of piers, bearings and many structural members.

To date, in this research project, the types of bridges that would most benefit from this retrofit strategy have been identified, the validity of the concept has been demonstrated analytically (using non-linear inelastic analysis), a design procedure has been developed, and the concept was verified experimentally using large scale specimens. Effective ductile diaphragms have been developed using shear panels, TADAS systems, and eccentric braced frames, but other systems are possible using the principles formulated. Full scale ductile diaphragm specimens have been constructed and tested using conventional reversed cyclic inelastic loading as well as pseudo-dynamic testing. This paper summarizes the proposed design procedure, results from the non-linear inelastic analyses, and experimental results.

SEISMIC RETROFIT CONCEPT

The ductile end-diaphragm retrofit strategy was developed in a capacity design perspective, in that all structural inelastic deformations can develop only in some judiciously selected structural elements specially detailed to absorb all seismic energy. The special ductile elements can be seen as "structural fuses", all other elements being "capacity protected". Here, these structural fuses are located into the end-diaphragms of steel superstructures to prevent damage in the non-ductile substructural elements, foundation, and bearings. Therefore, the special end-diaphragms must be designed such that stable hysteretic ductile behaviour will develop and be sustained at a reliable pre-determined load level lower than that the threshold of unacceptable damage of the substructure, as schematically illustrated in Fig. 1.

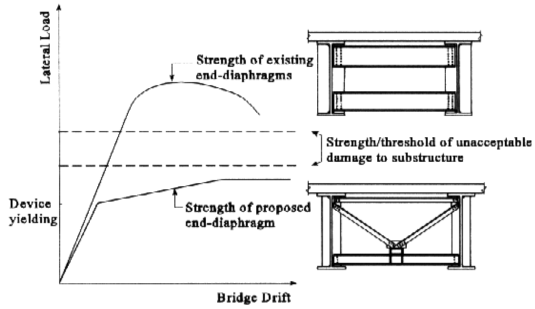


Fig. 1. Inelastic behavior of the proposed ductile end-diaphragms compared to that of existing steel bridges

This retrofit method provides enhanced seismic resistance and substructure protection in the transverse direction of the bridge, and must be coupled with other devices constraining longitudinal seismic displacements. Scope of this study is also limited to slab-on-girder bridges that do not have horizontal wind bracing connecting the bottom flanges of girders as these braces could provide an alternative load path bypassing the special ductile elements - this is not a concern for deck-truss bridges as the retrofit solution accounts for horizontal wind bracing. Note that for slab-on-girder bridges retrofit, bearings stiffeners of the main girders would sometimes have to be trimmed or replaced by narrower ones for reasons described elsewhere (Zahrai and Bruneau 1998).

ANALYTICAL DEVELOPMENT AND DESIGN PROCEDURE

To provide computational efficiency and to allow formulation of a simple design procedure, a simplified 2-D model capturing the essence of the 3-D behaviour of slab-on-girder bridges was developed. The proposed simplified 2-D model consists of the ductile end diaphragm, a stub-length of two girders with their bearing stiffeners and modelled as plane flexural members, a rigid stub of the reinforced concrete deck, and a small mass/spring subsystem located at deck level and introduced to account for the longitudinal generalized mass and stiffness effects.

While the above 2-D model can be implemented directly for computer analyses, its simplicity makes it also suitable for hand-calculations. Indeed, recognizing that the generalized stiffness of the bridge, K^*, the lateral stiffness of the end-diaphragms, K_{ends}, and the lateral stiffness of substructure including abutments, columns, piers and foundations, K_{subs}, are linked together as springs in series, the equivalent stiffness of the entire bridge, K_e, could be written as:

$$K_e = \frac{1}{\frac{1}{K^*} + \frac{1}{K_{ends}} + \frac{1}{k_{subs}}} \quad (1)$$

This study is concerned with single span bridges supported by stiff abutments, and flexibility of substructure is ignored in Eq. 1. Studies considering substructure flexibility are presented elsewhere (Zahrai and Bruneau 1998). Note that, in some instances, the flexural resistance of the girders can potentially contribute to the lateral load resistance of the ductile system, and even to its energy dissipation capability, depending on the relative rigidities of the components of this diaphragm. If that is the case, a tri-linear hysteretic model must be considered, and a more complex relationship between ductility derived using principles of equal energy could be used (Zahrai and Bruneau 1998). Otherwise, the conventional bi-linear hysteretic behaviour and relationship between ductility, μ, and the force reduction factor, R, can be used.

The step-by-step procedure for the design of ductile end-diaphragms requires the following steps: (i) determine basic design parameters (mass, seismic acceleration, geometry, etc.); (ii) calculate generalized mass and stiffness parameters; (iii) determine elastic and inelastic seismic base shear resistance of the diaphragm, considering design criteria and limiting capacity of substructure, and the portion of that shear to be resisted by the energy dissipation device (for trilinear hysteretic systems, this typically require an iterative calculation); (iv) provide proper ductile detailing for the selected energy dissipating device, and design all other structural members and connections of the diaphragm to be able to resist forces 50% greater than that resulting from yielding of the ductile device; (v) check the resulting period of the ductile end-diaphragm, and if different from that assumed in step iii, repeat the appropriate above steps until convergence; (vi) determine the actual force reduction factor, using the appropriate bilinear or trilinear behaviour of the total diaphragm system, and compare with target design value; (vii) repeat until a satisfactory value is obtained for that force reduction factor (usually requires 2 or 3 iterations). Refer to Zahrai and Bruneau (1998) for additional information, examples and nonlinear analyses.

EXPERIMENTAL APPROACH

To experimentally verify the analytical study and predicted structural behavior, a series of cyclic tests were carried out on full-scale specimens, each having a ductile end-diaphragm introduced between two short segments of the girders representative of a 40 m span steel slab-on-girder bridge with four girders. Each specimen consisted of two 0.5 m long segments of WWF1200x333 girders, 2 m center-to-center and having 10 mm thick and 100 mm wide bearing web stiffeners on each side of the web and a 200 mm thick reinforcement concrete deck, connected to top flange of each girder segment by 10 shear studs. The TADAS, EBF and SPS ductile diaphragms were common in that they all had a chevron-braced frame configuration, with a bottom beam, and two double angles as diagonal braces (Fig. 2a). However, each end-diaphragm had a different energy dissipation ductile device.

CAN/CSA G40.21-M 350W structural steel (equivalent to ASTM-A572 Grade 50) was specified for all the steel specimens. Nominal yield and ultimate strengths of 350 MPa and 450 MPa, respectively, were used in designing test specimens. Concrete with a 28 day strength, f_e', of 30 MPa and 10 mm maximum aggregate size was ordered from the supplier.

Design of Test Specimens

All specimens were designed based on the proposed design procedure to resist a PGA of 0.6g (A=0.6), indicative of severe earthquakes. As recommended in the design procedure, only one ductile end-diaphragm panel was considered at each bridge end. Consequently, for the full scale tests, only the two girders connected together by a ductile diaphragm needed to be considered.

For the design procedure, considering the full bridge model, the generalized tributary mass of 71500 kg at each bridge end induced an elastic lateral load, V_{el}, of 1050 kN. According to the codes recommendations (American 1994, Canadian 1995) for most structural frames, a reduction factor, R, of 3 was considered (corresponding to ductility ratio, μ, of 5 per Newmark-Hall's procedure). This gave an inelastic seismic force, V_{ine}l, of 350 kN for each specimen to be considered for design of the ductile devices. Full fixity was assumed during design of the specimens for the connection of bottom flanges, to be more representative of the experimental conditions, although fixity condition at fixed or expansion bearings approach that of a hinge in most field conditions due to large lateral rotations that can develop at the supports.

Instrumentation

All specimens were carefully instrumented for displacement and strain measurements at the points of interest. Some displacements were measured and monitored by three Temposonic Magnetostrictive Displacement Transducers, each with a stroke of 500 mm (20") and accuracy of 0.0077 mm. Also four short Linear Variable Displacement Transducers (LVDTs), each with a stroke of 50 mm (2") and accuracy of 0.0034 mm, were used to take other displacement readings. Micro-Measurements strain gages of the type EP-08-250BG-120 were generally used, except that rosettes of the type CEA-06-250UR-120 were used at a few locations. All specimens were whitewashed to help observe the yielding patterns and their progression.

Loading Approach

Three MTS actuators were used to apply the required forces to the specimens: two for gravity loads and one for lateral loads applied in a displacement-control mode. The applied loads were recorded by means of load cells integrated to the actuators. A constant gravity load of 350 kN (175 kN per girder), was applied to the specimens deck slabs using two vertical actuators. This equivalent load was selected because it created the same P- effects on the specimens and actual bridge. All specimens were tested following the same protocol. Elastic cycles were applied under force control and the inelastic cycles under displacement control. The cyclic displacement history was adopted to consider three cycles at drifts of ±0.25%, ±0.5%, ±0.75%, ±1%, ±1.5%, ±2%, ±3%, ±4%, etc. until failure.

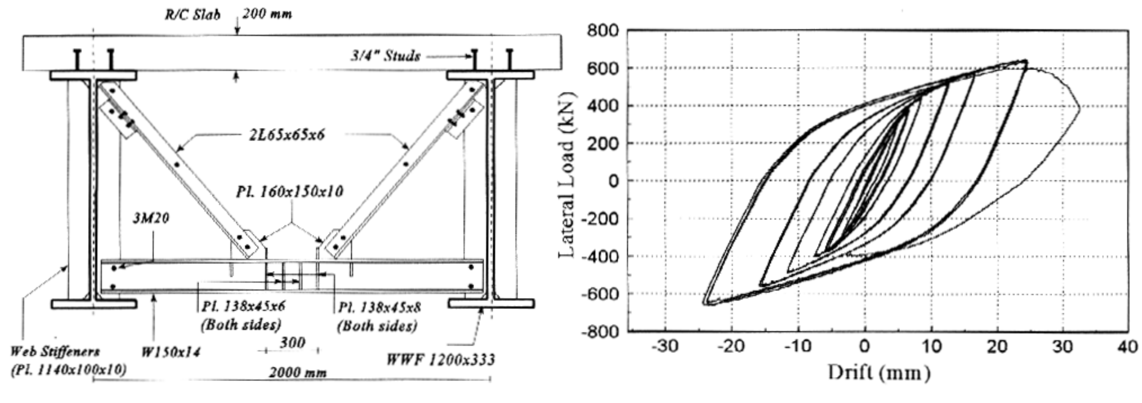
A MTS Testar Controller and a Vishay Data Acquisition System Model 5000 were used in parallel to record the data. The MTS system collected Temposonic displacements, actuators loads and horizontal actuator displacements whenever the specimens deck slabs moved laterally more than 0.1 mm. The Vishay data acquisition system recorded all strain gage and rosette data, the lateral load and all LVDTs displacements every two seconds.

EXPERIMENTAL RESULTS

Eight tests were conducted on the TADAS, EBF and SPS specimens in different situations. Due to paper length constraints, only results from testing of the EBF specimen are presented here. Refer to Zahrai and Bruneau (1998) for the test results of the other specimens. To eliminate the deformations due to slip at the connections, it was decided to weld all members at all connections of the EBF diaphragm.

Fig. 2b shows the hysteretic loops for this experiment. The specimen was subjected to 22 cycles, up to a 30 mm maximum displacement (2.5% drift), at which point the link beam failed due to sudden twist and lateral displacement of the link. As a result of this instability, brittle fracture occurred at the west end of the link flange and buckling developed at east end of the same flange. This failure might have been prevented or delayed had lateral bracing of the link beam ends been installed. Note that this was deliberately not done, and that instruments revealed no significant lateral movements of the link until the sudden buckling that developed at the 2.5% drift.

A specimen without diaphragm, i.e. with two stiffened stub-girders alone, was also tested to failure. Fig. 3a shows the lateral load versus drift hysteretic curves for this specimen. For this specimen without diaphragm, 25 cycles up to a maximum drift of ±96 mm (equivalent to 8% drift) were applied before the specimen experienced severe buckling and fracture of the web stiffeners at all welded connections. Maximum positive and negative lateral load reached 165 and 180 kN, respectively. This illustrates the maximum potential contribution of girders to the total strength of this ductile diaphragm system. In most practical cases, the girder would not have full fixity at its top and bottom.



(a) (b)

Fig. 2. EBF specimen: a) Geometry and member sizes; b) Hysteretic behaviour

Energy Dissipation

The energy dissipation of the specimens under lateral load reversals, was calculated for every displacement cycle up to failure. Hysteretic energy in each cycle, E_H, was determined by measuring the area under the curves for that specific cycle. To provide a better comparison of the hysteretic energies dissipated by the various specimens, these energies per cycle, E_H, are plotted at given drifts, as shown in Fig. 4. The EBF specimen dissipated a large hysteretic energy per cycle up to its failure at about 3% drift, while the specimen without diaphragm dissipated s significantly smaller energy also dissipated a greater cumulative hysteretic energy than the other tested specimens (i.e. bolted TADAS and SPS specimens) whose results are not presented here.

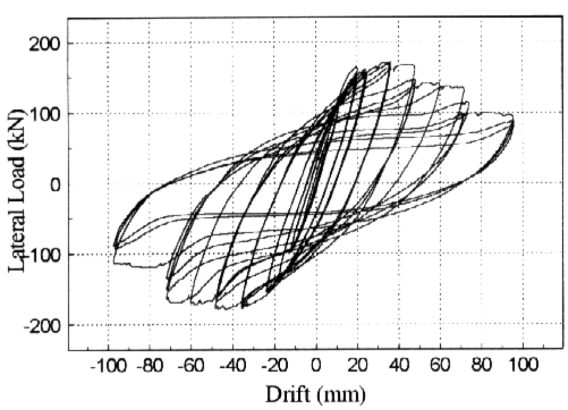


Fig. 3. Behaviour of specimen without diaphragm

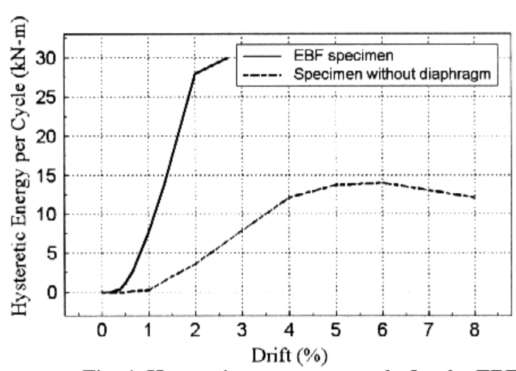


Fig. 4. Hysteretic energy per cycle for the EBF specimen and the one without diaphragm

DECK-TRUSS BRIDGES

Deck-truss bridges were constructed for many decades throughout North America, most notably as approach spans to major crossings. Typically, the deck is seated on the truss structure, itself supported on abutments or piers at its lower end. Hence, the seismically induced inertia forces in the transverse direction at deck level act 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. However, the lateral-load resisting components (i.e. top and lower lateral bracings, interior cross bracings, end-cross bracings)
in these older bridges were designed only to resist wind forces, and would be unable to withstand the severe cyclic inelastic behaviour expected to develop during large earthquakes. Moreover, these spans are generally supported on non-ductile substructures that would also require extensive retrofitting.

Indeed, recent seismic evaluations conducted for deck truss bridges have typically revealed the need to replace and/or reinforce many of the superstructure members and substructure components. While expensive, particularly when work is required in difficultly accessible parts of the bridge, this solution is satisfactory as long as realistic and conservative scenarios of future earthquake occurrence are considered, given that this approach provides no guarantee of adequate seismic performance beyond the threshold of damage. In many cases, base isolation has been recommended for deck truss bridges. However, while very 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. Capron 1995, and Matson and Buckland 1995).

Seismic Deficiencies of Existing Deck-Trusses

To show the location and magnitude of structural deficiencies, and allow full appreciation of how effectively the proposed retrofit strategy can enhance their seismic resistance, a representative example was constructed using information taken from structural drawings supplied by practising engineers under confidentiality agreements and its non-linear dynamic behaviour was investigated.

This example deck truss has an 80 m span, is 10m high and 10m wide, and has a concrete deck of 225 mm thick. The truss panels on all sides are 10m×10 m in size. The concrete deck is discontinuous due to the presence of expansion joints at each panel joints (10 m). The deck-truss is supported on two bearings at each of its end. At these supports, vertical and transverse displacement of the joints are restrained, and horizontal displacements along the longitudinal axis of the truss is allowed at one end. The truss superstructure is taken as seated on abutments (or equivalently on short and stiff piers) and the substructure is not included in the model. Nonetheless, the retrofit strategy proposed here considers the limited substructure strength available and the need to prevent structural damage there. Also, the inadequacy of connections to resist the corresponding seismic loads, as well as other commonly encountered deficiencies, are neglected for first evaluation, but would also be require close scrutiny by the engineer.

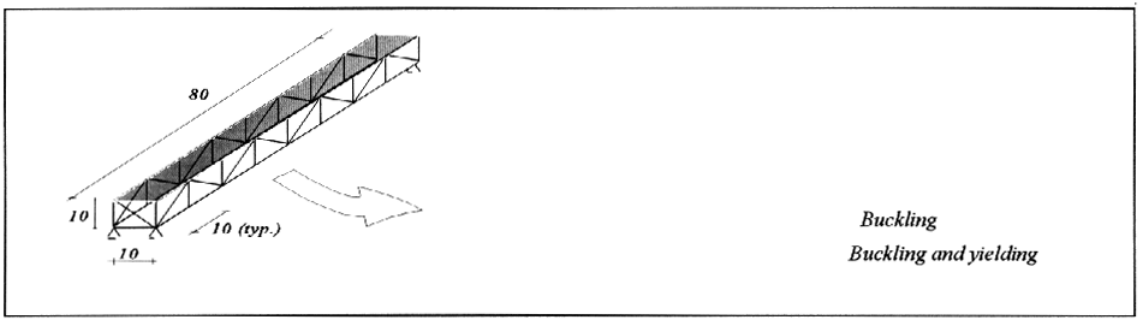


Fig. 5. Example deck-truss and location of damaged members

Non-linear dynamic time history analysis of the deck truss example conducted using DRAIN-3DX revealed that many end cross braces and verticals, top laterals, interior cross braces, and lower lateral braces buckled and yielded when the bridge was subjected to the ElCentro 1940 NS ground excitation (with a peak ground acceleration of 0.34g), with member ductility demands sometimes in excess of 9.

Results also showed significant relative displacement of the mid-span with respect to the Seismic load paths in a deck-truss ends as a consequence of the large flexibility of the top lateral resisting system. Flexibility of the top lateral system translates directly into a greater share of the seismically induced inertia force flowing into the lower path through slender inside bracings. This is due to the fact that the concrete deck, being discontinuous at each expansion joint, cannot provide a significant in-plane stiffness contribution to the top truss system. The locations of damage throughout the bridge are identified in the exploded view of the deck truss shown in Fig. 5. Analyses using other equally severe earthquake excitations (not presented here) revealed consistent damage type and location.

SEISMIC LOAD PATHS IN DECK-TRUSSES AND CONCEPTUAL 2D MODEL

To identify the effective paths of lateral load resistance in deck-truss bridges, determine the critical truss members located along those paths, and investigate how the stiffness of relative structural components impact the flow of loads along these multiple paths, a series of preliminary elastic analyses were conducted using SAP90. These analyses revealed the existence of two dominant load paths contributing to the seismic resistance of deck-truss bridges, as illustrated in Fig. 6. Structural elements along the first path, referred to hereafter as the top load path, consists of top laterals and end-panel, the later also known as the end-diaphragm. The second path of seismic resistance, defined as the bottom load path, is provided by the combined action of the interior cross bracings with the exception of the last braced bay, and the lower laterals. As described in more details in a later section, the seismic resistance of deck-truss bridges also engages to some extent the torsional resistance provided by the trussed box action of the structural system. This torsional resistance was found to depend on the vertical side bracing of the truss, and is treated as lower load path element since the top load path action was found to be insensitive to variations in stiffness of the side bracing.

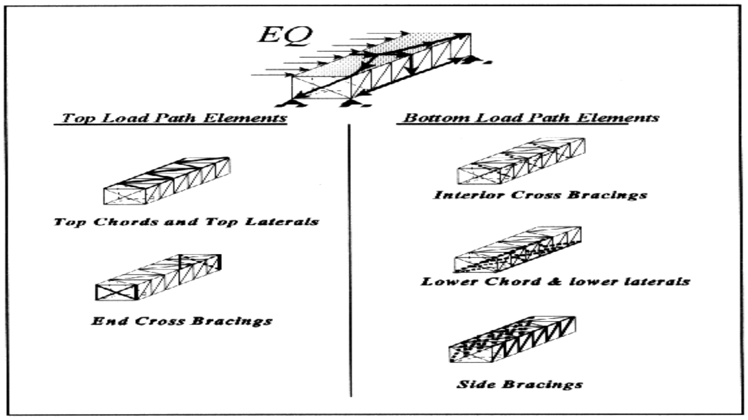


Fig. 6. Seismic load paths in a deck-truss

Furthermore, the lateral loads resisted by individual load paths were found to depend in a complex manner on the stiffness of different structural components along each path, according to an overall system behavior that can be conceptually expressed in a simplified manner by the 2D model shown in Fig. 7a. In this model:

- The top beam simulates the flexural behaviour of the horizontal truss system (top truss), consisting of top chord and top laterals, directly subjected to lateral inertia forces of the deck.
- The lower beam simulates flexural behaviour of the lower chord and lower lateral horizontal truss system (lower truss) subjected to the horizontal component of the loads transferred by the interior cross braces.
- Each spring connecting the top and bottom beam represents the action of a cross-frame.
- The end springs model the end-cross braces, usually larger than the other cross bracing panels, transferring the loads directly to the bridge supports.

This 2-D model is not used for the non-linear analyses reported at the end of this paper, but has proven useful to conceptually visualize the impact of structural changes while developing the retrofit strategy presented here.

SHORTCOMINGS OF STRENGTHENING SOLUTIONS

From the results of non-linear analyses, the engineer may elect to retrofit this seismically deficient truss by strengthening those members which are expected to suffer structural damage. This endeavour would also likely require the strengthening of many connections, bearings, and possibly abutments and piers. One may choose to only strengthen the dominant load path, providing it with enough strength to also account for the additional load it would attract as a result of its increased stiffness once strengthened. However, the results of a series of elastic analyses revealed a number of problems associated with this approach. The conceptual 2-D model described earlier can be used to explain these problems.

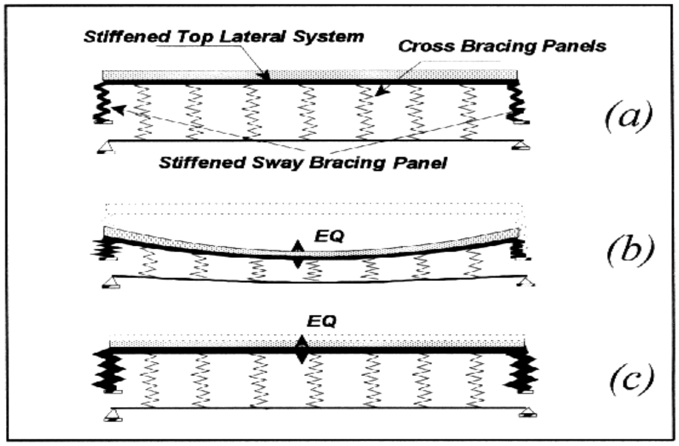


Fig. 7. Conceptual 2-D model of deck truss and strengthening of top load path elements: (a) strengthening of end spring and top beam; (b) Large deformation imposed on interface springs; (c) Highly stiffened top beam and end springs

Consider the strengthening of the top load path. As shown in bold in Fig. 7(a), that strengthening corresponds to strengthening of the top beam (i.e. top truss) and its spring supports (i.e. end-diaphragms). Although some stiffening results from the strengthening of these elements, the flexural deformation of the top beam remains sufficiently large to impose large deformations (and subsequently excessive forces) in the interface springs between the top and lower beams [Fig. 7(b)]. Thus, further strengthening of the top beam and end springs is required merely for the sake of increasing stiffness to reduce the magnitude of the deformations imposed on the lower system and to prevent damage there. This, in turn, also changes the overall stiffness of the structure, reduces its period of vibration, increases the overall inertia forces, all resulting in an increased force demand in the top beam and end springs, which then again require further strengthening. Parametric studies showed that a rather significant stiffening is required to achieve the ideal situation shown in Fig. 7(c) in which no damage develops in the lower load path.

Strengthening the lower load path system alone is even a poorer choice. First, because many of the actual structural members in the lower load path (e.g. cross-frames and lower-laterals) are often small and very slender sections originally designed to satisfy the minimum load-resistance requirements specified by

older codes, and therefore need more significant strengthening; Second, due to the effect of torsional flexibility of the entire deck truss on this load path, even a significant increase in stiffness and strength of these elements a large portion of load leaks to the top load path, unless the side trusses are stiffened as well.

Finally, regardless of other considerations (such as those pertaining to the substructure), strengthening of the deck truss using either approach will only ensure satisfactory performance if future earthquakes do not exceed in severity the seismic risk considered by the structural engineers.

RETROFIT USING CAPACITY DESIGN CONCEPT

A ductile retrofit scheme based on capacity design concepts is an attractive solution to minimize the extent of retrofit activities, and the benefits of this approach have been described in the introduction. Various scheme were considered, and analyses revealed that the best strategy was to locate yielding devices in the end-diaphragm and in the lower end panels. Fig. 8 shows the 2-D model of this concept, with ductile fuses shown in bold. By introducing two ductile yielding device acting as fuses at each end of the span:

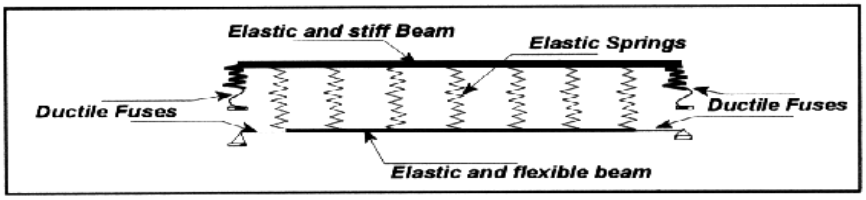


Fig. 8. : Ductile retrofit concept in 2-D model

- The magnitude of the forces transferred to the lower beam and the interface springs is limited by the maximum strength of the fuses at the ends of the bottom beam;
- The magnitude of the forces transferred to the top beam is limited by the maximum strength of the fuses at the ends of the top beams;
- The magnitude of the support reaction at an end of the bridge is limited to the sum of the maximum capacity of the two fuses acting there.

Based on this concepts, retrofit includes conversion of the end cross frame and the last lower end panels to ductile panels, each having a yielding device acting as a structural fuses.

Selection Criteria for Ductile Devices

Seismic performance of the retrofitted superstructure is a function of selected values of stiffness and strength of the retrofitted end and lower panels, and these must be determined by the engineer. Basically, yield capacity of a given device For consistency, use the same type of bullets (spacing, margin, indentation style) as above.must be less than the force corresponding to threshold of damage in interior cross-bracing and lower laterals members, damage in end vertical, tie down devices, or substructure, whichever is the least. On the other hand, the capacity must be greater that the strength needed to resist wind loads. Also, the stiffness of the device must be kept within the limits corresponding to maximum

global ductility limit, in one hand, and maximum displacement limits corresponding to threshold of inelastic distortion, or instability of the end panels, on the other hand, whichever is less.

DESIGN EXAMPLES AND RESULTS

Three different ductile retrofit systems were designed according to the concept explained above. Fig. 9 shows the corresponding details for TADAS, EBF and VSL implemented as ductile fuses at the end and lower end panels. Following the design of each retrofit system, computer models of the retrofitted deck-trusses were generated. Using the Drain-3DX program, non-linear dynamic behaviour of the retrofitted deck-truss were simulated by subjecting each model to 6 different ground motion records of past earthquakes (El Centro 1940, Pacoima Dam 1971, Taft 1952, Olympia 1949 and Loma Prieta 1989), all scaled to 0.6g, to verify if all ductile seismic performance requirements were met, and whether retrofit systems performed as expected. Values for a number of important response parameters tabulated in Table

1. All retrofit designs proved satisfactory.

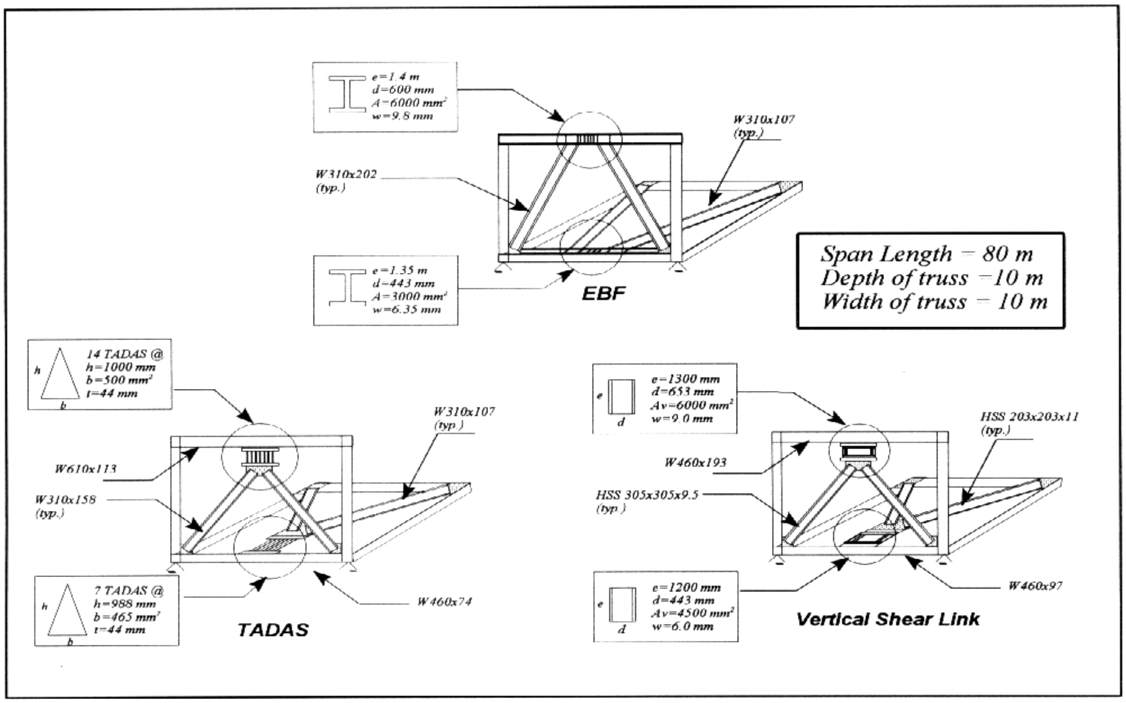


Fig. 9. Details of designed ductile retrofit systems

Table 1. Average displacement and ductility for response to 6 Earthquakes (PGA= 0.6g), compared with maximum limit

Ductile Retrofit System	Distortion Angle, γ (%)	γ_{max} (%)	Global Ductility Demand, μ	μ_{max}	End panel Drift (%)	Maximum Drift (%)
EBF	2.6	9	2.68	3.75	0.54	2
TADAS	N/A	N/A	2.34	3.75	1.59	2
VSL	2.99	9	2.57	3.75	0.75	2

CONCLUSIONS

A ductile diaphragm strategy has been presented for the seismic retrofit of slab-on-girder steel bridges and deck-truss bridges. Per capacity design principle, these ductile diaphragms act as structural fuses calibrated to yield before the strength of the substructure (and superstructure in the case of deck-truss bridges) is reached, thus protecting that substructure (and superstructure) from undesirable damage.

For slab-on-girder bridges, a simple design procedure suitable for hand calculation has been proposed, using a trilinear load-displacement relationship and a strength-versus-ductility relationship based on equal energy concepts, and nonlinear inelastic analyses suggest that the resulting designs will exhibit an appropriate ultimate cyclic seismic behaviour. Results from testing of full-scale specimens illustrate their large initial elastic stiffness, high strength and high capacity to dissipate hysteretic energy. The specimens with EBF end-diaphragms reached a link distortion angle 0.11 rad corresponding to a ductility of 14 before failure.

The specimen without any diaphragm, dissipated some hysteretic energy, but its strength and stiffness degradation occurred early due to buckling of the web stiffeners and fracture of its welds.

For deck-truss bridges, the proposed retrofit strategy requires conversion of the two end cross-frames, and the two lower lateral panels adjacent to the supports, into ductile panels. Following this procedure, stiffness and strength of ductile retrofit panels needed to retrofit a 80-m-span deck-truss were determined. Computer simulations of the dynamic behaviour of the retrofitted deck-truss subjected to severe ground excitation showed satisfactory performance, and validated the analytical procedure for the proposed retrofit strategy.

Although the concept appears sound, further research is desirable prior to implementation. Specifically, experimental validation using 3-D models is desirable, along with parametric studies to determine the range of substructure flexibility for which the retrofit solution remains effective. Finally, some of the constraints imposed here are conservative and could be relaxed if justified by future research.

ACKNOWLEDGEMENTS

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