STRUCTURAL FUSES AND CONCRETE-FILLED STEEL SHAPES FOR SEISMIC AND MULTI-HAZARD RESISTANT DESIGN

Michel Bruneau[1], Samer El-Bahey[2], Shuichi Fujikura[3] and David Keller[4]

SUMMARY

Bridges are built in a variety of locations, many of which are susceptible to multiple extreme hazards (earthquakes, vehicle collisions, tsunamis or storm surges, and blasts as a minimum for some locations). In addition, they must be built to achieve the objectives of both accelerated bridge construction (ABC) and rapid return to service following a disaster. Meeting some or all of these demands/objectives drives the development of innovative multi-hazard design concepts. This paper presents recent research on structural fuses and concrete-filled steel shapes strategies developed for this purpose. The structural fuse concept considered here for seismic resistance was developed and experimentally validated for implementation in a composite multi-column pier using double composite rectangular columns of Bi-Steel panels. Experimental results from another series of tests on the blast resistance of concrete-filled-steel-tubes support the blast resistance of the concept. In parallel, the development and design of a conceptual multi-hazard resistant steel plate shear wall box pier concept considered each of the four aforementioned hazards by use of simplified analyses for design, and of advanced nonlinear finite element analyses to confirm that the proposed steel plate shear wall box system provides adequate ductile performance and strength for each of the hazards.

INTRODUCTION

The emergence of new design objectives in bridge engineering always provides new opportunities to re-examine past design practices and explore the potential benefits of various alternative design solutions. Three such new performance requirements are considered here. First, the need for Accelerated Bridge Construction (ABC) solutions intended to minimize construction time and thus the inconvenience to the users of the road network, given that traffic congestion (due to construction delays or other sources) have been conclusively demonstrated to translate into major losses to modern economies. Second, the need for seismic design solutions that allow rapid repair and near-immediate return to service, as bridges decommissioned for long periods of time following disasters translate into major direct and indirect losses to society. Third, the need for multi-hazard solutions – recognizing that bridges are often built in locations susceptible to multiple extreme hazards (earthquakes, vehicle collisions, tsunami or storm surges, and blasts as a minimum for some locations). Meeting some or all of these constraints drives the development of innovative multi-hazard design concepts.

This paper presents recent research on structural fuses and concrete-filled steel shapes strategies developed for the purpose of meeting the above performance requirements for bridges. The structural fuse concept considered here for seismic resistance was developed and experimentally validated for implementation in a composite multi-column pier using double composite rectangular columns of Bi-Steel panels. Although Bi-Steel panels are already known for their blast performance, experimental results from another series of tests on the blast resistance of concrete-filled-steel-tubes provide additional evidence in support of the blast resistance of the concrete-filled shapes in bridge pier applications, as contrasted with other conventional seismically designed piers. In parallel, the development and design of a conceptual multi-hazard resistant Steel Plate Shear Wall (SPSW) box pier concept considered each of the four aforementioned hazards by use of simplified analyses for design, and of advanced nonlinear finite element analyses to confirm that the proposed SPSW box system provides adequate ductile performance and strength for each of the hazards. Together, these studies validate and verify the effectiveness of structural fuses and concrete-filled shapes for multi-hazard resistant design.

STRUCTURAL FUSE FOR SATISFACTORY SEISMIC PERFORMANCE

The concept of designing some sacrificial members, dissipating the seismic energy while preserving the integrity of other main components, is known as the structural fuse concept [1-4]. However, for a true structural fuse analogy [e.g 3, 4], the sacrificial elements should be easily replaceable,

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allowing the rest of the structure (that remained elastic) to return to its plumb condition after the fuses are removed. Here, in that perspective, a structural fuse concept is proposed in which structural steel elements are added to the bridge bent to increase its strength and stiffness, and also designed to sustain the seismic demand and dissipate all the seismic energy through hysteretic behaviour of the fuses, while keeping the bridge piers elastic. Several types of structural fuses can be used and implemented in bridges; the focus in this paper will be on using two types of structural fuses.

First, an innovative Steel Plate Shear Link (SPSL) is introduced. The proposed SPSL shown in Figure 1 consists of a steel plate restrained from out of plane buckling using an encasement and an unbonding material. The steel plate is designed to yield in shear (reaching 0.6Fy) for the purpose of dissipating seismic energy.

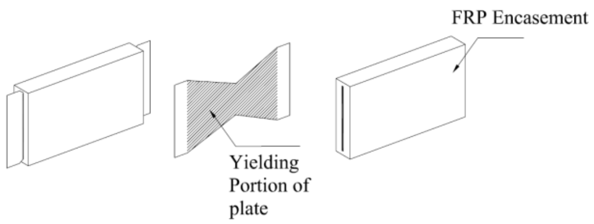


Figure 1: Proposed link sketch.

Three types of plastic mechanisms can develop in links regardless of the shape of the cross section. The type of the plastic mechanism developed depends mainly on the link length in which links can be categorized into:

- Flexural links (pure flexural yielding) developing full plastic moment hinges, M_p, at the ends of the links and developing a shear force less than the full plastic shear force, V_p, whereby energy is dissipated by flexural plastic rotation.
- Shear links (pure shear yielding) developing the full plastic shear force, V_p, over the entire length of the link with moments at the ends less that the plastic moment reduced to account for the presence of shear, M_p^r, whereby energy is dissipated by shear plastic distortion.
- Intermediate links which are links yielding in both flexure and shear using the Von Mises yield criteria assuming that one yielding mode develops after the other mode strain hardens.

Various experimental studies have been done on links by previous researchers and it was found that shear links exhibit the most stable and ductile cyclic behaviour [5-7]. The ultimate failure mode for shear links is inelastic web shear buckling, which can be delayed by adding vertical stiffeners [5]. For the proposed link, the web shear buckling is overcome by wrapping the steel plate with unbonding material and surrounding it by an encasement.

An assumed stress distribution for a shear link is shown in Figure 2 from which the plastic shear and plastic moment can be calculated as:

$$V_p = \frac{F_y}{\sqrt{3}} t y_0 \tag{1}$$

$$M_{pr} = F_y y_1 t (y_0 + y_1) \tag{2}$$

where V_p is the plastic shear force for section A-A, M_{pr} is the reduced plastic moment due to the presence of shear force for section B-B, and F_y is the yield stress of the plate.

The balanced length, e^*, from which the transition of behaviour occurs from flexural to shear can be calculated using simple free body diagram equilibrium as:

$$e^* = \frac{2y_o}{\sqrt{3}\tan^2\theta}\left(1-\sqrt{3}\tan\theta\right) \tag{3}$$

while the balanced link angle, θ^*, can also be calculated using free body diagram equilibrium and the geometry of the link as:

$$\tan^2\theta^* + \frac{2y_0}{e}\tan\theta^* - \frac{2y_0}{e\sqrt{3}} = 0 \tag{4}$$

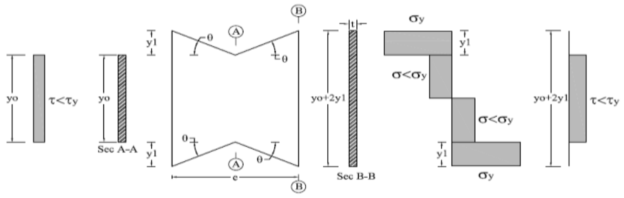


Figure 2: Assumed stress distribution in mid and end plate.

Second, Buckling Restrained Braces (BRBs) are utilized as structural fuses. The BRB consists of a steel core encased in a steel tube filled with concrete. The steel core carries the axial load while the outer tube, via the concrete provides lateral support to the core and prevents global buckling. Typically a thin layer of material along the steel core/concrete interface eliminates shear transfer during the elongation and contraction of the steel core and also accommodates its lateral expansion when in compression (other strategies also exist to achieve the same effect). This gives the steel core the ability to contract and elongate freely within the confining steel/concrete-tube assembly. A variety of these braces having various materials and geometries have been proposed and studied extensively over the last 10-15 years [8-15]. A summary of much of the early development of BRBs which use a steel core inside a concrete filled steel tube is provided in [16], and since the 1995 Kobe Earthquake, these elements have been used in numerous major structures in Japan [17]. The first tests in the United States were conducted in 1999 [18]. Figure 3 shows a schematic mechanism of the BRB.

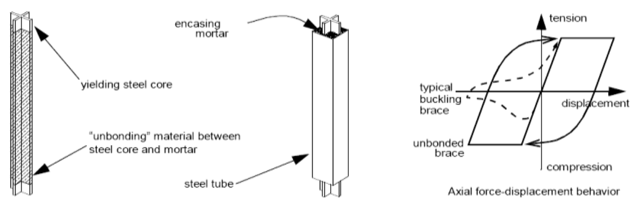
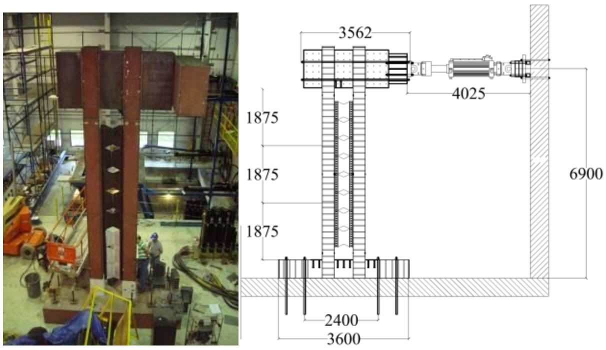


Figure 3: Schematic mechanism of the BRB [19].

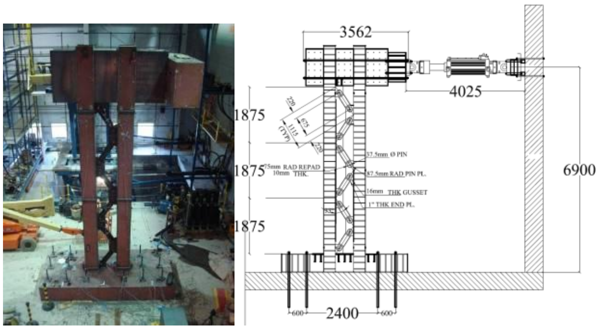
EXPERIMENTAL SETUP, INSTRUMENTATIONS AND LOADING PROTOCOL

A series of quasi-static cyclic tests has been performed using the recommended Applied Technology Council (ATC) loading protocol of ATC 24 [20] on a proposed twin column segmental bridge bent, utilizing the SPSLs and BRBs as a series of structural fuses between the columns. The columns used for the experiment consisted of segments of Bi-Steel sections [21] which is a system of double skin steel–concrete–steel high performance rapid erect panels. These panels are composed of steel plates connected by an array of transverse friction welded shear connectors and filled with concrete. This system could be beneficial when strength or speed of construction is of vital importance. Column sections were stacked over each other and connected by welding. A 1.5 scale for the geometric properties of the specimen was chosen based on the limitations of the Structural and Earthquake Engineering Simulation Laboratory (SEESL) at the University at Buffalo and other considerations regarding the availability of the Bi-steel sections in particular, the maximum height of the SEESL strong wall is 30 ft, so the maximum height of the specimen was set to be 25 ft. Two static actuators available at SEESL each with a capacity of 400 kips were used applying the horizontal force to a transfer beam from which the load is then transferred to the specimen. Figures 4, 5, and 6 show general views of the tests utilizing SPSLs, BRBs and the bare frame respectively, while figure 7 shows a plan view cross section detail of the BiSteel columns utilizing the SPSLs as structural fuses.



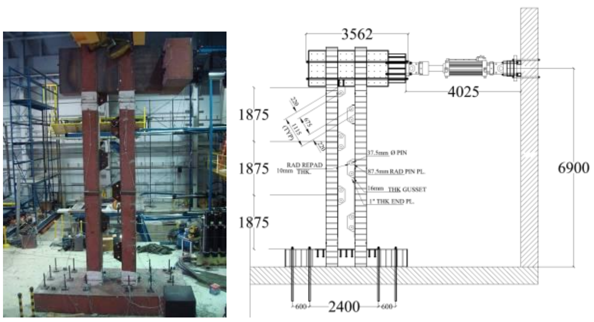
(a) *(b)*

Figure 4: ***Experiment setup (a) General view of the experiment, (b) Bridge pier with SPSLs.***



(a) *(b)*

Figure 5: ***Experiment setup (a) general view of the experiment, (b) bridge pier with BRBs.***



(a) *(b)*

Figure 6: ***Experiment setup (a) General view of the experiment, (b) Bare bridge pier.***

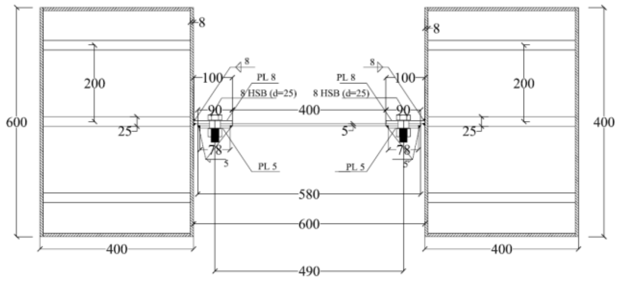


Figure 7: ***Columns cross section details (Plan view cross section).***

Instrumentation for this experimental project has been designed to measure global response of the frame, and local performance of the links and braces. Global response of the structure in terms of displacements was obtained from string-pots installed at different levels from the base to the top of the frame. Optical coordinate tracking probes (Krypton sensors) were also distributed on the columns up to their mid heights (due to camera range constrains) to measure displacement response at specific points. Seismic response of the columns was obtained from strain gages installed at critical points (top and bottom of each column), to determine whether these columns remain elastic during the test, recalling that one of the objectives of this experiment is to assess the effectiveness of the structural fuse concept to prevent damage in columns. Axial deformations of the BRBs were measured with String-Pots installed in parallel with the braces and connected to the gusset-plates. To measure strains in the SPSLs, 30- 60 degree rosettes were installed at the midpoint of a few critical links. To ensure that no slippage or uplift occurs in the base, horizontal and vertical transducers were installed at its four corners.

EXPERIMENTAL RESULTS

For the first specimen with the SPSLs, loading was performed up to a drift level corresponding to the onset of column yielding to ensure that energy dissipation was through the SPSLs, then testing continued until fracture occurred at the base of both columns. This specimen reached a ductility ratio of 4 and 100 mm top displacement (1.5% drift) without any sign of plastic deformation in the columns, Figure 8 shows the hysteretic behaviour at that level of drift. Signs of local buckling started to occur at the west column at 125 mm top displacement (1.8% drift) as shown in Figure 9, and the same column fractured at 160 mm top displacement (2.3% drift) and the load dropped almost 33% as shown in Figure 10.

For the second specimen with BRBs, loading was performed up to a drift level corresponding to the onset of column yielding (1.5%); also a ductility of 4 was reached, and no signs of plastic deformation were observed for both columns. The BRBs exhibited stable hysteretic behaviour. Figure 11 shows the hysteretic behaviour for one of the BRBs installed (3rd from top) plotted against the total system force. A small amount of slippage occurred due to the pin connection of the BRBs. Hysteretic behaviour for the specimen with BRBs is shown in Figure 12.

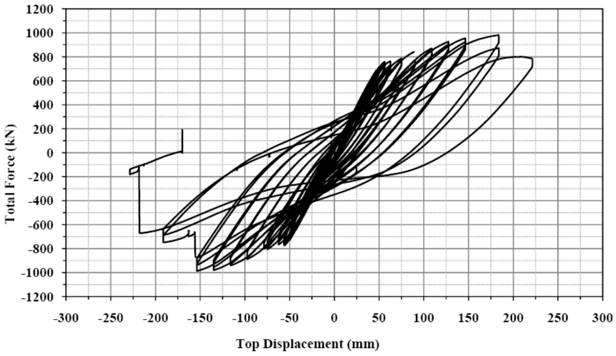


Figure 8: ***Hysteretic behaviour for column utilizing SPSLs at the onset of column yielding.***

Figure 9: ***Local buckling of west column (West Side) at 1.8% drift.***

Figure 10: ***Fracture of west column (Northwest Corner) at 2.3% drift.***

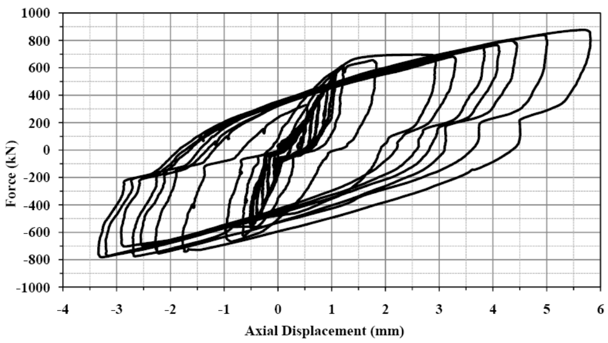


Figure 11: ***Total lateral force vs axial BRB displacement hysteretic curve for BRB3 (3rd from top).***

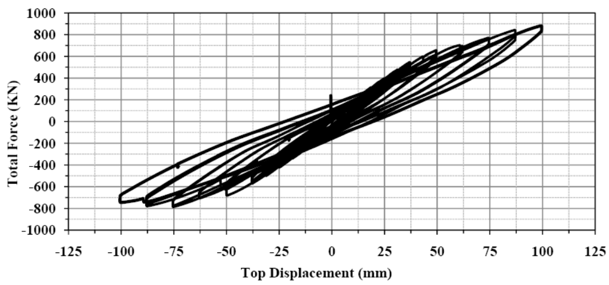


Figure 12: ***Hysteretic behaviour for column utilizing BRBs at the onset of column yielding.***

OBSERVATIONS

All specimens tested in this experimental program exhibited stable force-displacement behaviour, with little pinching of hysteresis loops until the significant accumulation of damage at large drifts. All specimens performed well, behaving elastically at small displacements and exhibiting stable hysteretic behaviour as the seismic energy was dissipated through the structural fuses. Adding the fuses increased both the stiffness and strength of the bare frame about 40% and increased the amount of energy dissipated by the frame. Further analysis is underway to investigate the results of this experimental program.

BLAST RESISTANCE OF CONCRETE-FILLED STEEL SHAPES

There are some similarities between seismic and blast effects on bridge structures: both major earthquakes and terrorist attacks/accidental explosions are rare events that can induce large inelastic deformations in the key structural components of bridges. However, a design to resist one hazard does not automatically provide resistance against the other hazard – which can easily be demonstrated by case studies beyond the scope of this paper.

A review of several different structural configurations of bridge piers and potential bridge bent systems was conducted to identify systems deemed most appropriate in meeting the objectives of multi-hazard design. It was found that concrete-filled steel tubes (CFSTs) can be used as multi-hazard bridge piers capable of providing an adequate level of protection against collapse under both seismic and blast loading, and with member dimensions not very different from those currently found in typical highway bridges. These CFST columns are smaller than the typical 914 mm (3 ft)-diameter reinforced concrete pier column, but expected to perform significantly better under blast loads. This type of structural member was deemed likely to be accepted in practice (and incidentally is helpful in fulfilling the objective of accelerated construction). This structural configuration was therefore selected for experimental verification of its blast resistance (seismic performance of such columns had already been demonstrated by researchers, such as Bruneau and Marson [22]).

A series of blast experiments on 1/4 scale multi-hazard bridge piers was performed by Fujikura *et al.* [23, 24]. Piers were CFST columns with different diameters [D = 102 mm (4 inch), 127 mm (5 in) and 152 mm (6 in)], connected to a steel beams embedded in the cap-beam and a foundation beam. The bent frame was braced in what would correspond to the bridge longitudinal direction at the level of the cap-beams. A reaction frame was built for this purpose. Blast tests showed that CFST columns of bridge pier specimens exhibited a satisfactory ductile behaviour under blast loading as shown in Figure 13-a. The foundation connection concept applied in this experiment allowed to develop the composite strength of CFST column under blast loading.

Note that for comparison, another blast test series was conducted to examine the blast resistance of ductile reinforced concrete (RC) bridge piers [D = 203 mm (8 in)] and non-ductile RC bridge piers retrofitted with steel jackets [D = 213 mm (8- 3/8 in)] that are designed according to current seismic knowledge and that are currently applied in typical highway bridge designs. Out of that test series, standard RC and steel jacketed RC columns were not found to exhibit a ductile behaviour under blast loading, failing in direct shear at their base rather than by flexural yielding as was the case with CFST columns (see a test result of a RC column in Figure 13-b). Furthermore, this non-ductile failure occurred for a much smaller blast pressure than used for the comparable CFST

[25]. Reinforced concrete details by current seismic codes and steel jacketing, known to be effective to provide satisfactory seismic performance, were thus shown to be ineffective for the blast loading cases considered.

(a) *(b)*

Figure 13: ***(a) CFST column (D = 127 mm) after the test; (b) RC column after the test.***

MULTI-HAZARD SPSW BOX-PIER CONCEPT

The concept formally referred to as multi-hazard engineering has recently emerged as a new interest in the field of civil engineering. It addresses the anticipated cost implications of growingly complex structures required to resist the sometimes conflicting demands of multiple hazards [26]. A true multi-hazard engineering solution is a concept that simultaneously has the desirable characteristics to protect and satisfy the multiple (contradicting) constraints inherent to multiple hazards [27]. It calls for holistic designs that encompass all hazards in an integrated framework, and that provide optimized, single cost/single concept solutions rather than a collection of multiple design schemes.

Favorable features for design against one hazard may inevitably be unfavorable for other hazards, thus lending mismatched design solutions to the multi-hazard dilemma. Such conflicting design aspects are well illustrated elsewhere [28]. To make a design that is beneficial for one hazard while at the same time avoiding the possibility of making the structure vulnerable to other hazards, a system's approach to design must be undertaken. Such an approach necessitates designers to be knowledgeable of multiple hazards, and to consider the numerous and sometimes contradicting demands from the multiple hazards at the onset of the design process such as to avoid foreseeable mismatched design solutions. Ettouney *et al.* [26] provide a list of benefits for considering a multi-hazard approach, some of which include: potential for economic designs and constructions, a more accurate estimation of inherent resiliency of systems, a more accurate treatment/estimation of life cycle cost of systems, and a more accurate analysis of systems.

Given that the objective of this research, designing a bridge pier from a multi-hazard perspective, is a wide-reaching proposition, the scope was narrowed by focusing on developing a pier system that incorporated concepts from SPSW design. Hazards considered here included earthquakes, vehicle collisions, tsunamis, and blast. A system incorporating SPSWs was sought because of their ductile nature, because of the redundancy they offer, and because they are easy to repair. Such qualities of SPSWs make them a resilient structural system that suggested at the onset of this research that they should be capable of resisting multiple hazards. However, SPSW concepts, while already implemented in buildings, have never been incorporated into bridges, which posed an additional challenge.

In considering the seismic hazard, adequate resistance in each of a bridge's principal directions was desired while at the same time being capable of sustaining gravity loads and maintaining its integrity after occurrence of any of the other hazards. Additionally, a design that had aesthetic appeal was sought. Various concepts were explored [29] before eventually converging on the four-column box pier solution shown in Figure 14. The continuous three-span steel plate girder prototype superstructure was adopted from a seismic design example developed for the Federal Highway Administration [30]. For this research, the pier cap, which was made integral with the superstructure, is integral with the SPSW pier system, which was found to be advantageous. Also note that the pier assembly was made reasonably narrow in the longitudinal direction to reduce the plate surface area subject to wave loads arising from surging water transverse to the bridge's deck.

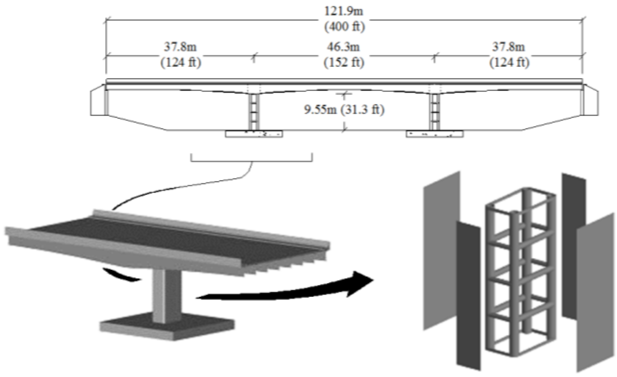


Figure 14: ***Final multi-hazard resistant bridge pier concept.***

The pier's plan dimensions (centerline-to-centerline of vertical boundary elements (VBEs)) are 3,708 mm (146 in) transversely (i.e. perpendicular to the bridge spans) x 1,880 mm (74 in) longitudinally (i.e. parallel to the bridge spans), and its total height is 9,376 mm (369.08 in) with three intermediate horizontal boundary elements (HBEs) spaced equally at 2,344 mm (92.27 in).

ASSESMENT OF PIER TO MULTIPLE HAZARDS

Earthquakes

In general, the system was designed for a given seismic hazard and then analyzed for the other hazards. This was only possible because of the multi-hazard approach taken in conceiving a concept at the onset. The seismic hazard was also used as the starting point of the detailed design because proven methods for the design and analysis of SPSW for seismic hazards are available in codes and design guides.

For the purpose of design, in accordance with AASHTO [31], the seismic acceleration coefficient was chosen to be 0.20 placing this bridge in seismic performance zone III, the bridge was classified as "regular", and its importance classification was chosen to be in the AASHTO category of "other bridge". The response modification factor, R, was chosen to be 5, and based on recommendations from AASHTO (Article 3.10.5.1) when the soil profile is unknown, the site coefficient was chosen to be 1.2. In analysis, movement of the superstructure in the longitudinal and transverse direction was assumed to be resisted by the two piers acting in parallel, the superstructure was assumed to be rigid, and it was assumed that there would be sufficient space for movement at the abutments so that the piers could develop their ultimate strength (the abutments were assumed to offer no resistance). In both directions, the top and bottom of the pier was assumed rigidly attached to the pier cap and foundation, respectively.

Design relied on use of nonlinear pushover analysis performed with SAP2000 [32]. Beam-column elements representing the boundary frame, and "tension-only" strips representing the plates, were used as is commonly done for SPSW design [33]. Plastic hinging was allowed only at the ends of the boundary frame members. Hinging was modeled using discrete nonlinear "Fiber P-M2-M3" hinges displaying elastic-perfectly plastic behaviour placed at the ends of the boundary frame elements, and using discrete "Axial P" hinges at the strips' centers also exhibiting elastic-perfectly plastic behaviour. The steel assumed for the tubular sections was A500 Gr. B (Fy = 290 MPa (42 ksi)) and the material assumed for the plates was A36 (Fy = 248 MPa (36 ksi)) steel.

Critical loading was assumed as occurring if the pier were to be pushed simultaneously (or bi-directionally) in the transverse and longitudinal directions, where all strips in the perpendicularly oriented plates yield. The design was then checked to ensure that hinges had formed only in the intended locations, that the members were not shear critical, and that the assumed stiffness in the transverse and longitudinal direction (used, with the reactive mass, to compute the seismic demand on the pier required for sizing the plates) matched that of the design. This approach was iterated until a satisfactory design was converged upon.

The final boundary frame design consisted of VBEs having an outer diameter of 609.6 mm (24 in) with a wall thickness of 46.0 mm (1.812 in), longitudinal HBEs having an outer diameter of 323.9 mm (12.75 in) with a wall thickness of 12.7 mm (0.5 in), and transverse HBEs having an outer diameter of 406.4 mm (16 in) with a wall thickness of 21.4 mm (0.843 in). The transverse plates were each 1.588 mm (0.0625 in) thick, and the longitudinal plates were each 3.175 mm (0.125 in) thick.

This design was further assessed with non-linear finite element modeling using the graphical interface program ABAQUS/CAE [34]. Figure 15 shows the model of the pier both prior to and following a pushover analysis being carried out. Notice that the plates buckle in compression and develop tension field action, as is characteristic of SPSW systems. However, the steel plates in this case would not act as true fuses; while their replacement is possible and relatively easy, the frame would not necessarily bounce back plumb as the boundary frame is expected to experience plastic hinging, per design intent. Small residual drifts may nonetheless be not so conspicuous and thus acceptable to some Departments of Transportation.

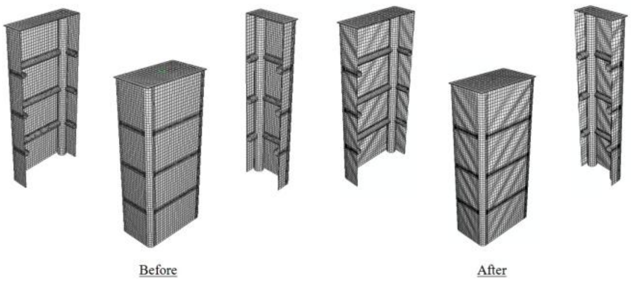


Figure 15: ***Finite element model before and after the pushover analysis.***

Vehicle Collision and Tsunami

Although detailed results are not presented here due to space constraints, the pier's design also considered the vehicle collision hazard by way of statically applying a 1,780 kN (400 kip) concentrated load at 1,200 mm (4 ft) above the ground, per AASHTO requirements, to one of the VBEs in a linear elastic analysis. Not being captured in simplified analyses, advanced, finite element analysis was used to assess the impact the plates have on the global behaviour of the system to this hazard, and it was found that the plates aided in resisting load in a way similar to how they resist the seismic hazard – through the development of tension field action (Figure 16).

Tsunami preliminary design considered loads that were obtained from FEMA's Coastal Construction Manual [35] and the City and County of Honolulu Building Code (CCH) [36], and assumed an event corresponding to a 3 m design stillwater depth with water flow having a computed design velocity of 10.8 m/s (35.4 ft/s) in the direction perpendicular to the bridge's deck. Design considered the following two load cases: (1) surge forces and debris impact forces, and (2) hydrostatic, hydrodynamic and debris impact forces. While the plates were expected to yield in response to being loaded, the boundary frame was expected to remain undamaged.

Further analysis with a finite element model similar to that used in analysis of the seismic and vehicle collision hazards, considered only hydrostatic and hydrodynamic forces, but for four different water depths; the fourth depth very conservatively considered the pier to be fully submerged (Figure 16). It was found that (even for the fourth load case) while the plates did yield and act as sacrificial elements for this hazard, the boundary frame was observed to remain stable and not develop any plastic hinges following each finite element analysis, per conceptual intent at the onset of design.

Blast

In initial design, the plates and VBEs were assessed separately in a decoupled analysis being subject to a blast load having a peak reflective pressure of 29.2 MPa (4,228 psi) and a reflected impulse of 9.7 MPa-msec (1,407 psi-msec). Design considered this load to act uniformly over the bottom plates and the bottom (up to the first HBEs) of the VBEs; these elements would have the least standoff to an explosion occurring at the base of the pier and would therefore be the most severely loaded.

Simplified analysis revealed that the plates would likely offer little resistance against the threat considered and would thus be sacrificial assuming the boundary frame remained stable. Accordingly, the VBEs of the system were assessed to validate this assumption. It was found that the VBEs would be sufficiently strong to resist the loads imposed by simultaneous yielding of attached plates. Likewise, it was found through a separate SDOF flexural analysis that the VBEs would also likely remain elastic if subject to the design blast loads acting over their own surface.

Nonlinear static analyses were also conducted in an effort to uncover unanticipated behaviours when the pier is locally subject to larger pressures loads, and in a manner that simulated the likely failure sequence of pier elements, the plates being assumed to fail first. Of primary concern was how the VBEs would behave under large compressive forces, so the finite element analysis applied uniform pressure loading over the bottom quarter of one of the VBEs (Figure 16). Ultimately, this study uncovered the potential need to locally reinforce the cross-sections of any hollow structural shape, and that the VBEs could undergo significant flexural deformations without apparent consequence to the pier's global behaviour. As such, a revised and final multi-hazard concept suggests the use of concrete-filled steel tubes instead of hollow ones. The design concept remains identical otherwise.

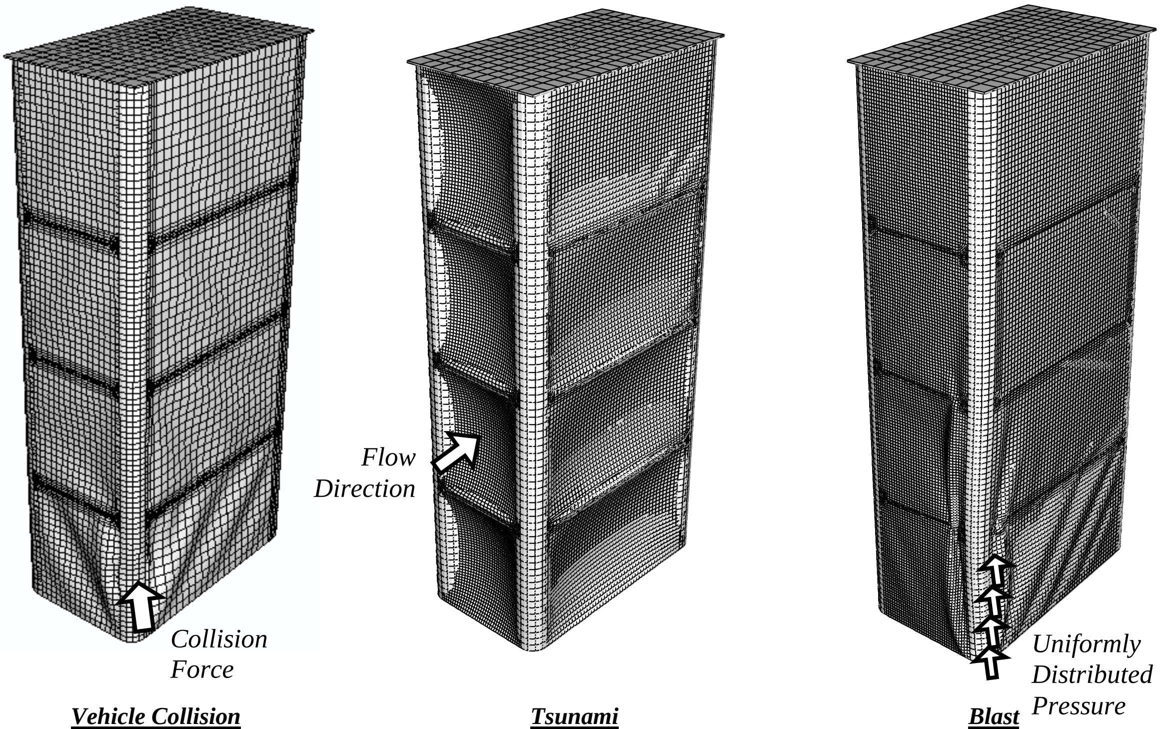


Figure 16: Finite element model following analysis for vehicle collision, tsunami and blast.

CONCLUSION

This paper investigated the ultimate behaviour of structural fuses and concrete-filled steel shapes strategies developed to meet a number of emerging performance demands in bridge engineering. In particular, a structural fuse concept implemented in a composite multi-column pier using double composite rectangular columns of Bi-Steel panels was shown to provide satisfactory seismic performance while facilitating post-earthquake repair and being compatible with the goals of accelerated bridge construction. Testing showed the enhanced blast resistance that concrete-filled shapes can provide over conventional seismic-only ductile design of piers having comparable strengths. Advanced nonlinear finite element analyses validated a SPSW box pier concept as one possible approach to achieve a multi-hazard resistant bridge pier, and suggested that concrete-filled steel shapes may be necessary in such applications.

The results obtained demonstrate the effectiveness of implementing structural fuses and concrete-filled steel-shapes in a bridge application to provide multi-hazard resistance.

ACKNOWLEDGMENTS

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