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Multi-hazard resistant steel plate shear wall bridge pier concept

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ABSTRACT: 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 are considered in this research since they are present as a minimum for some locations). This underscores the need to develop an innovative design concept for bridges from a multi-hazard perspective in which all hazards and their sometimes conflicting design solutions are considered from the onset. This is done here, focusing on a multi-hazard resistant pier composed of steel plate shear walls. Following the development and design of a Steel Plate Shear Wall (SPSW) box pier concept that considered each hazard by use of simplified analyses, advanced nonlinear finite element analyses were conducted to verify and validate its behavior. The proposed SPSW box pier system has adequate ductile performance and strength for each of the hazards considered in this research.

1 INTRODUCTION

Bridges are built in a variety of locations, many of which are susceptible to multiple extreme hazards. New York City and South Carolina, for example, are regions susceptible to hurricanes and earthquakes, and bridges in all regions are susceptible to vehicle collisions and blasts. In fact, numerous bridges have been damaged by various extreme hazards (Keller & Bruneau 2009). This exposure and vulnerability of bridges to multiple hazards underscores the need to introduce multi-hazard principles into bridge design. Bridge piers in particular are vulnerable elements whose damage could lead to bridge closure or even total collapse as a result of any of the aforementioned hazards. Therefore, the objective of this research is to develop an innovative design concept for bridge piers from a multi-hazard perspective.

The concept formally referred to as multi-hazard engineering has recently surfaced 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 (Ettouney et al. 2005).

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 (FEMA 2004). 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. (2005) 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.

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 (Bruneau 2007). 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.

Given that designing a bridge pier from a multi-hazard perspective is a wide-reaching proposition, the scope was narrowed by focusing on developing the pier system with concepts from steel plate shear walls (SPSWs) design. 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, although unknown at this time, should be capable of resisting multiple hazards. Hazards considered here included earthquakes, vehicle collisions, tsunamis, and blast. Note, however, that SPSW concepts, while already implemented in buildings, have never been incorporated into bridges, presenting an additional challenge.

2 BRIDGE PIER CONCEPT DEVELOPMENT

In considering the seismic hazard, comparable resistance from the piers in each of a bridge's principal directions was desired while at the same time being redundant enough to sustain gravity loads and maintain its integrity after occurrence of any of the other hazards. Additionally, a design that had aesthetic appeal was sought. To visualize and explore various concepts, a generic pier bent and superstructure (at first, one composed of reinforced concrete girders, but ultimately one with steel plate girders) was chosen. Figure 1 illustrates possible retrofit concepts (a & b) and new construction concepts (c–e) as they would appear looking longitudinally down a bridge. Arbitrary sizes for the cap beam and columns were chosen and were not designed for the purpose of this preliminary investigation. Figures 1a & 1b show the pier bent with SPSW assemblies that could be inserted as either retrofit solutions or

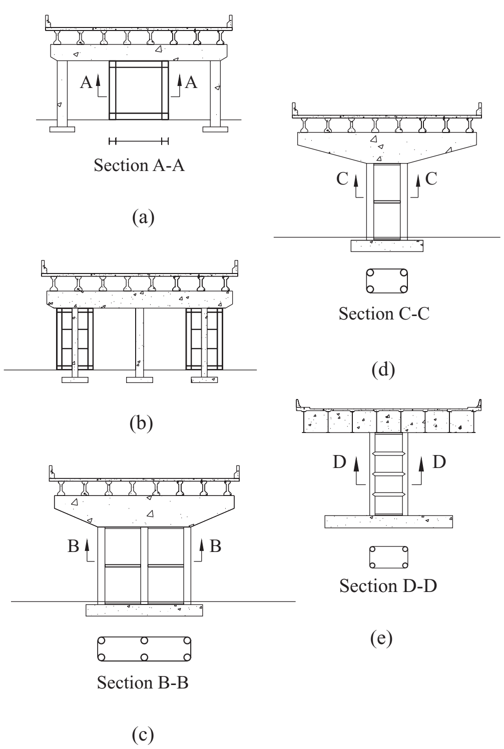


Figure 1. Progression of multi-hazard resistant SPSW bridge pier concept.

new construction measures, and Figures 1c–1e show the superstructure completely supported by SPSW assemblies, which would be implementable into new bridges. Note that foundation and connection details were not developed for these preliminary concepts.

Keller & Bruneau (2009) describe short-comings of concepts a & b by discussing specific aspects of these concepts that fail to adequately coalesce favorable design features for each hazard into a single multi-hazard solution. This, in addition to the limited freedom of design for retrofits and the potential difficulties of anchoring SPSW assemblies to existing bridge piers, shifted the focus to concepts for implementation into new bridges where the pier is completely composed of steel in the form of a SPSW box assembly (c–e) aimed at providing significant redundancy and comparable strength in a bridge's transverse and longitudinal direction. Note from the sections in these figures (B–B, C–C, and D–D) that the vertical boundary elements (VBEs) are hollow circular tubes. The use of tubes was preferred over the use of wide flange shapes, as is typical with SPSWs (e.g. Section A–A in Figure 1a) due to their cross-sectional symmetry about any axis. Similarly, the horizontal boundary elements (HBEs) are hollow circular tubes. Note that the elevations in Figure 1 show the pier without plates attached, thus revealing the boundary frame, when in fact the pier's boundary frame is wrapped with plates that are assumed welded to the HBEs and VBEs allowing the inside to remain dry.

After due consideration of each concept's benefits, the four-column box pier concept was retained as worthy of further development. In addition to its seismic resistance in each direction, which can be adjusted by simply changing plate thickness, the plates are anticipated to be sacrificial for the other hazards. This is an important point considering the premise behind multi-hazard design is to be conscious of design solutions with favorable features for one hazard that may be detrimental for other hazards. The plates, in particular, are an important feature to any SPSW system for seismic resistance, but at the same time provide surfaces that collect pressure loads from other hazards (e.g. tsunamis and blast). This undesirable design feature provided for seismic resistance, however, is not destructive for the other hazards if the plates are indeed sacrificial without consequence to the boundary frame.

Figure 1e illustrates the final concept that was developed in this research where the pier is attached to a pier cap that is integral with the bridge superstructure, which was found to be advantageous. Figure 2 illustrates what the total bridge system would look like considering a prototype bridge with the final pier concept integrated into it. This continuous three-span steel plate girder prototype bridge was adapted from a seismic design example developed for the Federal

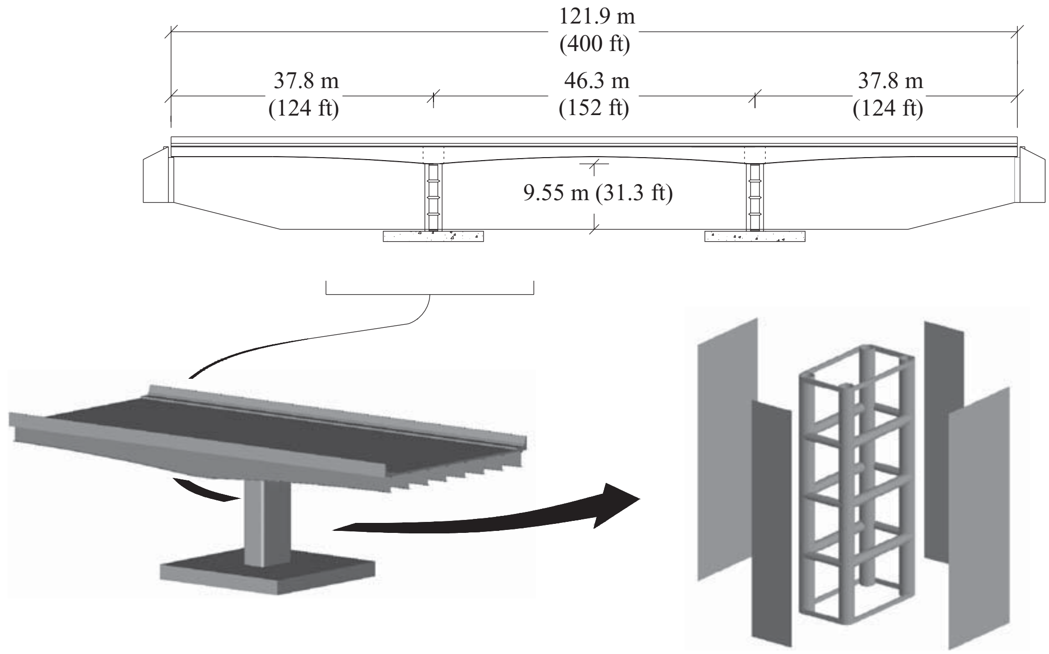


Figure 2. Final multi-hazard resistant SPSW bridge pier concept as it would appear integrated into the three-span prototype bridge.

Highway Administration (Mast et al. 1996). 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.

The pier's plan dimensions are 3708 mm (146 in) transversely × 1880 mm (74 in) longitudinally (centerline to centerline of VBEs), and its total height is 9376 mm (369.08 in) with three intermediate HBEs spaced equally at 2344 mm (92.27 in).

3 ASSESEMENT OF PIER TO MULTIPLE HAZARDS

3.1 *Earthquake*

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 systems for seismic hazards are available in codes and design guides.

For the purpose of this research, 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 (2007) 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 bridge 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 each direction, the top and bottom of the piers was assumed rigidly attached to the pier cap and foundation, respectively. Note that since each pier is identical, only one was designed and is elaborated upon.

Design relied on use of nonlinear pushover analysis with the commercially available structural analysis software SAP2000 (2007). Linear frame elements representing the boundary frame and "tension-only" strips representing the plates were used, as is commonly done when analyzing SPSWs (Sabelli & Bruneau 2006). The strips were connected to the centerlines of the boundary elements and thereby neglected the eccentricity of connecting the plates tangentially to the boundary frame, which was found to be of no consequence for the final design. It was assumed that the horizontal members at the top and bottom of the pier would be continuously connected to the pier cap and foundation, respectively. This condition was achieved by modifying their properties and making them rigid. The boundary condition at the pier's top to the pier cap was achieved using "diaphragm" and "equal" constraints.

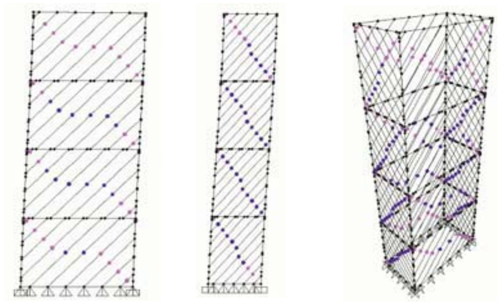
Plastic hinging was only allowed at the ends of the boundary frame members and within the strips representing the plates. Hinging in the boundary frame was modeled using discrete nonlinear "Fiber P-M2-M3" hinges displaying elastic-perfectly plastic behavior, and in the plate strips using discrete "Axial P" hinges at the strips' centers also exhibiting elastic-perfectly plastic behavior. The steel assumed for the tubular sections was A500 Gr. B ($\sigma y = 290$ MPa (42 ksi)) and the material assumed for the plates was A36 ($\sigma y = 248$ MPa (36 ksi)) steel.

Critical loading for the VBEs was thought to occur when the pier was pushed simultaneously (or bi-directionally) in the transverse and longitudinal directions until plates on each side of the pier had yielded and when all boundary frame hinging had occurred. This would constitute the system's ultimate behavior and collapse mechanism. Therefore, in designing the pier, pushover analyses were performed by pushing the top of the pier laterally with a resultant lateral load at 45 degrees to either of the bridge's principal directions until all hinging had occurred. Following each pushover analysis 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.

Figure 3 shows the model used in designing the pier deformed after being pushed to its ultimate capacity. Figure 3a shows the transverse side of the pier, Figure 3b shows the longitudinal side, and Figure 3c shows a 3-D view.

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 (ABAQUS 2004). Each



(a) (b) (c)

Figure 3. SAP2000 model following a pushover analysis.

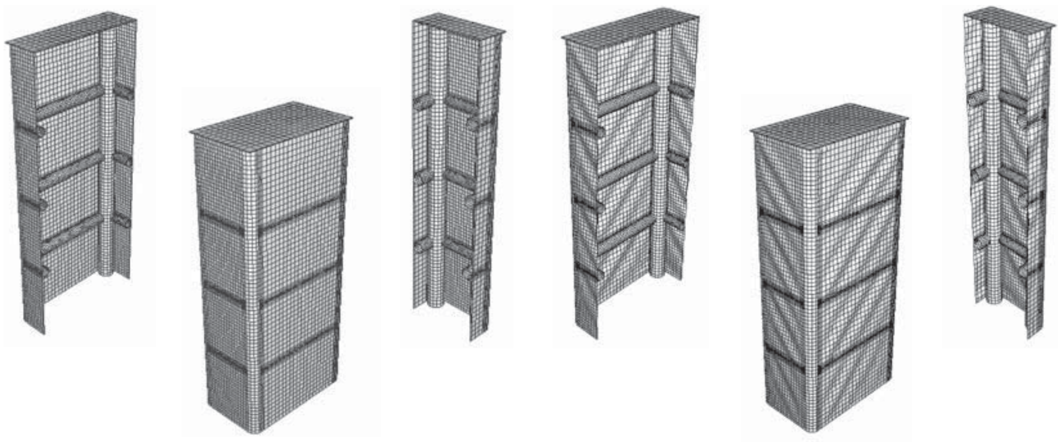
component of the pier (i.e. VBEs, HBEs, and plates) was modeled with deformable shell parts. One additional part, a plate modeled as a deformable solid having an arbitrary thickness of 25.4 mm (1 in), was used in modeling the pier cap boundary. This part was eventually assigned a constraint making it rigid. Each part, exclusive of the pier cap plate, was individually meshed with quadratic quadrilateral S8R elements. The pier cap was meshed with quadratic hexahedral C3D20R.

To compare the behavior of the pier to that observed in simplified analysis and design, bi-directional pushover analyses were performed. Figure 4 shows the model of the pier before and after an analysis was carried out. Figure 4a shows the pier prior to loading and Figure 4b shows the final stage of behavior of the pier following the pushover. Notice that the plates develop tension field action, as is typical of SPSW systems.

3.2 *Vehicle collision*

Although detailed results are not presented here due to space constraints, pier design also considered the vehicle collision by way of statically applying a 1780 kN (400 kip) concentrated load at 1200 mm (4 ft) above the ground, per AASHTO (2007) requirements, to a VBE in a linear elastic analysis. Because it is unknown at this point how to account for the plates in simplified design and analysis they were conservatively neglected. Simplified analysis and design found this hazard to be of no consequence to the pier system.

Advanced, finite element analysis was used to assess the impact the plates have on the global behavior of the system to this hazard, and it was found that the plates did in fact serve in resisting load in a way similar to how they resist the seismic hazard. As was found in simplified analysis and design, this hazard was of no consequence to the pier. Thus, loading was



(a) (b)

Figure 4. Finite element model showing (a) the pier in its undeformed state; and (b) the pier after being laterally loaded to its capacity (scale factor = 1).

further increased from that specified by AASHTO (2007). In doing so, it was evident that the bottom plates were engaged the most and that they mitigated deformation in the boundary frame by developing tension field action.

3.3 *Tsunami*

Loads associated with tsunamis and the manner in which they can be represented in design were obtained from FEMA 55 (FEMA 2000) and the City and County of Honolulu Building Code (CCH) (CCH 2000). Simplified analysis and design considered tsunami loads corresponding to an event having a 3 m design stillwater depth with water flow having a computed design velocity of 10.8 m/s (35.4 ft/s) perpendicular to the bridge's deck. The load cases involved (1) surge forces and debris impact forces, and (2) hydrostatic, hydrodynamic and debris impact forces. The resulting tsunami loads were applied statically in a linear analysis to the model used for seismic design, but with the strips representing the plates removed. The load collected by the plates was accounted for with a simplified iterative decoupled analysis in which they were assumed to resist load through large inelastic deformations. That load was then transferred to the boundary frame elements via plate edge reactions. This simplified analysis predicted that some plates would yield but that the boundary frame would remain essentially intact and would not develop plastic hinges based on evaluation of internal member forces following each analysis.

Further analysis with finite element modeling capable of capturing the coupled behavior between the plates and the boundary frame confirmed the behavior of the pier for this hazard. Four analyses, each only considering hydrostatic and hydrodynamic pressure applied to the plates, were conducted, each varying the water height on the pier. The worst case considered flowing water as high as to the underside of the bridge deck (i.e. the pier's full height). It was found that the plates would undergo yielding making them sacrificial but that the boundary frame would remain essentially intact, as was the conceptual design intent at the onset of design.

3.4 *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 (4228 psi) and a reflected impulse of 9.7 MPa-msec (1407 psi-msec). This load was conservatively applied uniformly over the bottom plates and over the bottom (up to the first HBEs) of the VBEs.

The plates (reduced to single degree of freedom oscillators) were analyzed by combining the iterative procedure used in the plate analysis for tsunamis with the principle that the kinetic energy imparted to the plates by the blast's impulsive loading was absorbed by the plates' internal work. This 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.

The VBEs were investigated as first being subject to full, simultaneous yielding of attached plates and second to blast pressures acting on the tubes themselves without the plates attached. 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 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 behaviors when the pier is locally subjected to larger pressures loads, and in a manner that simulated the likely failure sequence of pier elements, the plates being assumed to fail first. The first analysis considered adjacent first story plates and adjoining VBE to be increasingly simultaneously loaded statically to determine the impact of plates being loaded simultaneously with the VBE. Even after the plates were pushed far into the inelastic region the VBE remained elastic leading to the conclusion that the plates could fail without being destructive to the VBE.

Load was then applied in two different analyses to a VBE from the pier's exterior (i.e. pushing on the VBE) and from within the pier (i.e. pulling on the VBE). When pushing the VBE, the tube's wall was the weak link failing first from a collapsed wall which unveiled a possible design consideration; that is, the need to consider local deformation in the wall of hollow tubes when subjected to blast loads at close-in ranges.

Ultimately, this study uncovered the possible 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 behavior.

4 CONCLUSION

Being mindful of the demands characteristic of each of the hazards chosen for this research, a SPSW box bridge pier consisting of a four-sided tubular boundary frame wrapped with steel plates was developed. Following the design of the pier with simplified methods, subsequent and more refined analyses were employed to further assess the pier's behavior when subject to the demands of multiple extreme hazards. The design was found to satisfactorily resist all hazards. Specifically, the system resisted the seismic hazard through tension field action in the plates over the pier's height; the plates were found to mitigate deformation and therefore damage in the pier for vehicle collisions applied to the boundary frame; the plates could be considered sacrificial for the tsunami hazard relieving the boundary frame from taking any load aside from load directly applied to it and the capacity of the plates; the pier appeared to resist the blast hazard satisfactorily by considering the plates to be sacrificial, baring local failures of the hollow boundary frame tubes, which were found to be susceptible to local failure under larger compressive loads.

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