13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1703

DAMAGE ESTIMATION OF A SELF-CENTERING PRECAST CONCRETE BRIDGE PIER SYSTEM USING A PERFORMANCE-BASED ASSESSMENT METHODOLOGY

Won K. LEE[1], Sarah L. BILLINGTON[2], and J. Matt ROUSE[3]

SUMMARY

A precast segmental concrete bridge pier system is being investigated for use in seismic regions. The proposed system uses unbonded post-tensioning (UBPT) to join the precast segments and has the option of using a high performance fiber-reinforced cement-based composite (HPRFCC) in the precast segments at potential plastic hinging regions. The UBPT is expected to facilitate self-centering of the columns (i.e. cause minimal residual displacements) after cyclic loading and allow only a low amount of hysteretic energy dissipation. The HPFRCC material is expected to add hysteretic energy dissipation and damage tolerance to the system. Large-scale experiments on columns using the proposed system were conducted and are briefly reviewed. An overview of the performance-based framework developed by the Pacific Earthquake Engineering Research Center is presented, and an assessment of the potential damage in the proposed system using the performance-based methodology is presented.

INTRODUCTION

To ensure post-earthquake serviceability of bridges, attention has been drawn to the development and implementation of innovative materials and the use of self-centering systems for improved seismic resistance. An example enhanced-performance system made of precast, post-tensioned segmental concrete for improved post-earthquake serviceability is currently under investigation. The proposed system uses unbonded post-tensioning (UBPT) to join the precast segments and has the option of using a high performance fiber-reinforced cement-based composite (HPFRCC) in the precast segments at potential plastic hinging regions (Figure 1). The UBPT is expected to facilitate self-centering of the columns (i.e. cause minimal residual displacements) after cyclic loading and will provide a small amount of hysteretic energy dissipation. The HPFRCC material provides significant damage tolerance to the system as well as an additional small amount of hysteretic energy dissipation.

The development of performance-based design guidelines for seismic design can facilitate the implementation of this enhanced performance system. However, in order to ensure proper

[1] Graduate Researcher, Stanford University, Stanford, Calif. Email: wonlee21@stanford.edu
[2] Clare Boothe Luce Asst. Professor, Stanford Univ., Stanford, Calif. Email: billington@stanford.edu
[3] Graduate Researcher, Cornell University, Ithaca, New York. Email: jmr19@cornell.edu

implementation, performance-based assessment of this system must be conducted and should then be compared with the predicted performance of traditional reinforced concrete systems. The proposed system is currently being assessed using the performance-based earthquake engineering (PBEE) approach developed by the Pacific Earthquake Engineering Research (PEER) Center (Deierlein & Moehle [1]). This paper focuses on the methods of estimating damage in the proposed system, for use in overall performance evaluation using the PEER-PBEE approach.

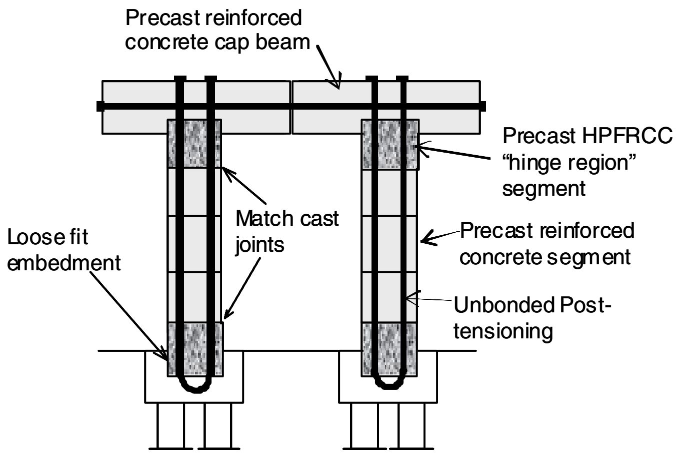


Figure 1 Schematic of proposed self-centering, precast concrete bridge pier system for seismic regions.

BACKGROUND

The proposed self-centering precast concrete system is an extension of a post-tensioned concrete bridge pier system previously studied for non-seismic areas (Billington *et al*. [2]). For seismic applications, unbonded post-tensioning is proposed. Unbonded post-tensioning exhibits nonlinear-elastic response under cyclic loading and results in a lower hysteretic energy dissipation capacity than systems with bonded post-tensioning. Previous experimental and analytical research has shown that unbonded post-tensioned systems do in fact provide some hysteretic energy dissipation (through concrete cracking and crushing) and as expected, reduce residual displacements (e.g. Priestley & MacRae [3], Ikeda [4], and Kwan & Billington [5],[6]).

The integration of segments made with HPFRCC material for the plastic hinging regions of the segmentally precast concrete bridge pier system is expected to result in increased energy dissipation and a high damage tolerance of large cyclic displacements through evenly spaced micro-cracking over the HPFRCC segment. The HPFRCC material investigated for this application exhibits strain hardening behavior in uniaxial tension and is composed of Portland cement, water, silica fume or fly ash, fine sand, and a low percentage by volume (roughly 2%) of randomly oriented polymeric fibers. This material exhibits multiple, fine cracks upon loading in tension as a result of steady-state cracking (Li & Leung [7]). The HPFRCC displays higher tensile ductility, tensile (strain) hardening behavior and energy dissipation than traditional concrete and many fiber-reinforced concrete materials (Figure 2). A review of the micromechanics-based design of the HPFRCC investigated here is given in Li [8].

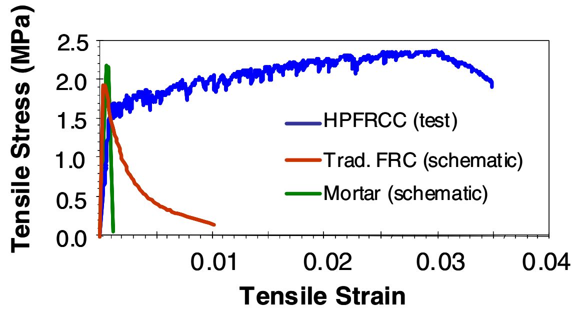


Figure 2 Uniaxial tensile response of HPFRCC compared with other cement-based materials.

The benefits of the proposed enhanced-performance system are being evaluated using the PEER-PBEE methodology. This methodology for performance-based earthquake engineering encompasses four main steps: hazard analysis, structural analysis, damage analysis, and loss analysis. Each step is handled on an individual basis and in a probabilistic fashion. It is assumed that each of these is a discrete Markov process, meaning that none of the steps is conditioned on the other. It is for this reason that the assessment can be broken up and analyzed individually. All of the steps are then brought together to provide an assessment of overall system performance in terms of monetary losses, downtime, and casualties. A complete description of this approach is given in Deierlein & Moehle [1]. The four steps are briefly summarized as follows:

1. Hazard Analysis – Given a site location and structural design, calculate the annual frequency with which a given seismic Intensity Measure (IM), e.g. spectral acceleration, will exceed certain levels (expressed with a hazard curve).
2. Structural Analysis – Given the IM and the structural design, perform numerical simulations on models of the structure to determine resulting Engineering Demand Parameters (EDPs), which are measures of the structural response to the given IM. EDPs include such values as drift ratios, floor accelerations, and plastic hinge rotations.
3. Damage Analysis – Given the EDP, determine the probability that a structural component or system will experience a certain level of damage. The levels of damage are defined by damage measures (DMs) particular to the system under consideration. DMs should be selected such that they can be assessed by observation in the field (e.g. spalling for reinforced concrete).
4. Loss Analysis – Given the levels of damage sustained, calculate measures of performance termed Decision Variables (DVs), which can be used by owners of a structure for decision making. Decision variables are typically in terms of monetary losses, structure or facility downtime, and casualties.

In the on-going research on self-centering precast segmental concrete bridge piers, the two areas of focus are the structural analysis and damage analysis portions of the assessment. The process of conducting the damage analysis is discussed in this paper, beginning with a brief presentation of the performance of the proposed system as measured in large-scale cyclically loaded experiments.

LARGE-SCALE EXPERIMENTS

An experimental study of the cyclic behavior of a precast, post-tensioned, self-centering bridge pier system was recently completed. The piers were segmentally precast and joined with unbonded post-tensioning strands. Four out of six specimens used HPFRCC in the potential hinge regions for enhanced performance under cyclic loading. Selected details and results are given here and a full report can be found in Rouse [9]. The goal of the pier system again is to achieve minimal residual deformations (self-centering) while ensuring some ductility and hysteretic energy dissipation in an unbonded system.

Six roughly half-scale segmental piers (specimens) including cap blocks and foundations were built and tested. Each pier had a clear height of 3.7 m and a 0.46 m square cross-section. The objectives of the experiments performed were: 1) to assess the reversed cyclic load response of segmentally precast concrete bridge columns reinforced with vertical unbonded post-tensioning; 2) to evaluate the use of various mix designs of HPFRCC in place of concrete in the hinge regions of the bridge columns; and 3) to evaluate the necessity of seismic detailing of transverse reinforcement when HPFRCC is used in place of concrete in the bridge columns.

The specimens were subjected to a combination of a nominally constant axial dead load and cyclic, quasi-static lateral loads deforming the piers in double curvature. To achieve the rotational restraint at both the top and bottom of the piers while allowing lateral translation of the cap, two pier specimens were tested simultaneously as illustrated schematically in Figure 3. For testing purposes the piers were oriented horizontally to slide laterally on sheets of Teflon. Figure 4 shows one set of specimens prior to testing.

Each pier consisted of a foundation block, four column segments and a cap segment. The column segments at the top and bottom were embedded in pockets in the foundation and in the cap. In the proposed system, none of the mild reinforcing bars cross the precast segmental joints. Only the unbonded post-tensioning was continuous in the column and it was designed to remain elastic at ultimate loads. Table 1 lists the variables of the specimens.

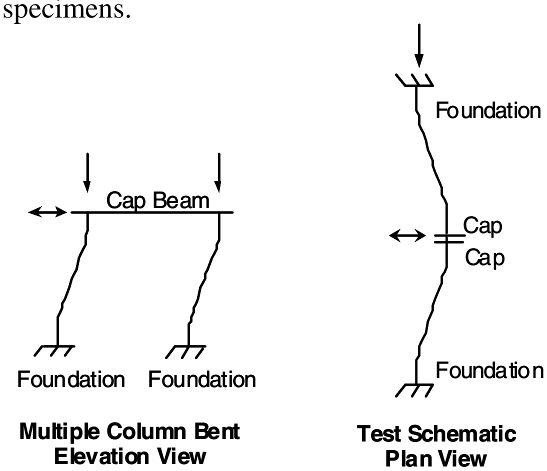


Figure 3 Prototype and test set-up.

Table 1 Experimental Variables

	Hinge Segment Material	Hinge Segment Reinforcement	Hinge Segment Length
Specimen 1	Concrete	Shear & Shrinkage (light)	1.067 m
Specimen 2	Concrete	Seismic (heavy)	1.067 m
Specimen 3	HPFRCC (PVA fibers*)	Shear & Shrinkage (light)	1.067 m
Specimen 4	HPFRCC (UHMWPE fibers**)	Shear & Shrinkage (light)	1.067 m
Specimen 5	HPFRCC (PVA fibers)	Seismic (heavy)	1.067 m
Specimen 6	HPFRCC (PVA fibers)	Shear & Shrinkage (light)	0.864 m

**PVA = Polyvinyl Alcohol*
***UHMWPE = Ultra High Molecular Weight Polyethylene*

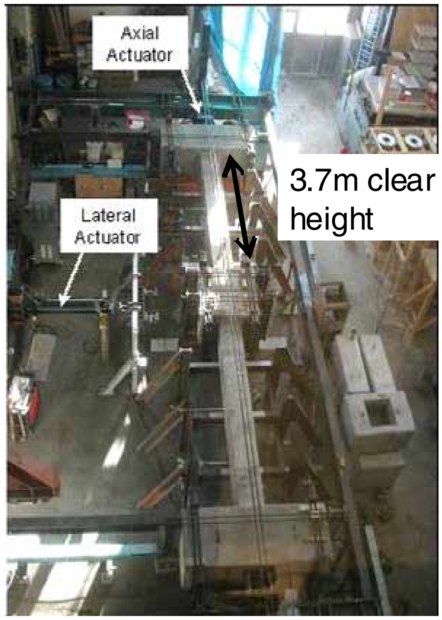


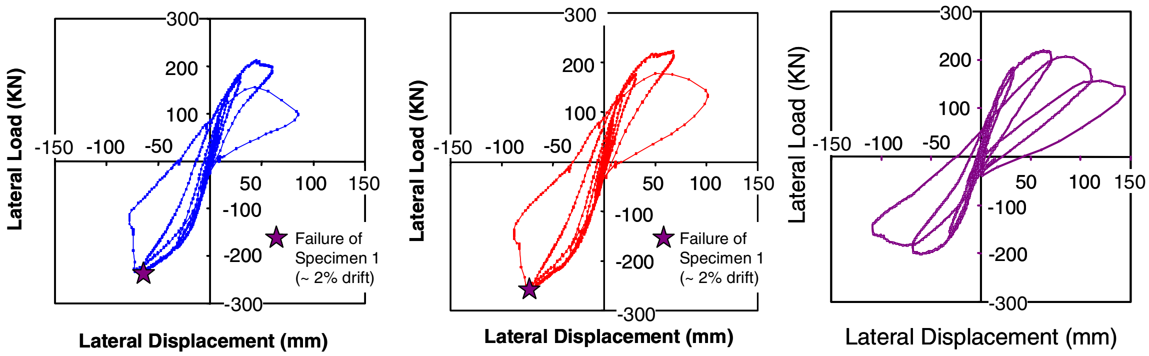
Figure 4 Two specimens prior to testing.

Selected Experimental Results

Among numerous findings were that the all-concrete system with seismically detailed transverse reinforcement was more brittle than an equivalent system using HPFRCC segments without seismically detailed transverse reinforcement. The HPFRCC system did not spall, whereas the all-concrete system had severe spalling. All of the specimens generally failed due to excessive opening of the construction joint closest to the foundation or cap beam stub, where the only continuous reinforcement was the unbonded post-tensioning. All of the specimens had low residual displacements on the same order. In no case did any of the strands in the specimens experience over 80 percent of their theoretical yield load. Complete results from the six specimens are presented and discussed in Rouse [9].

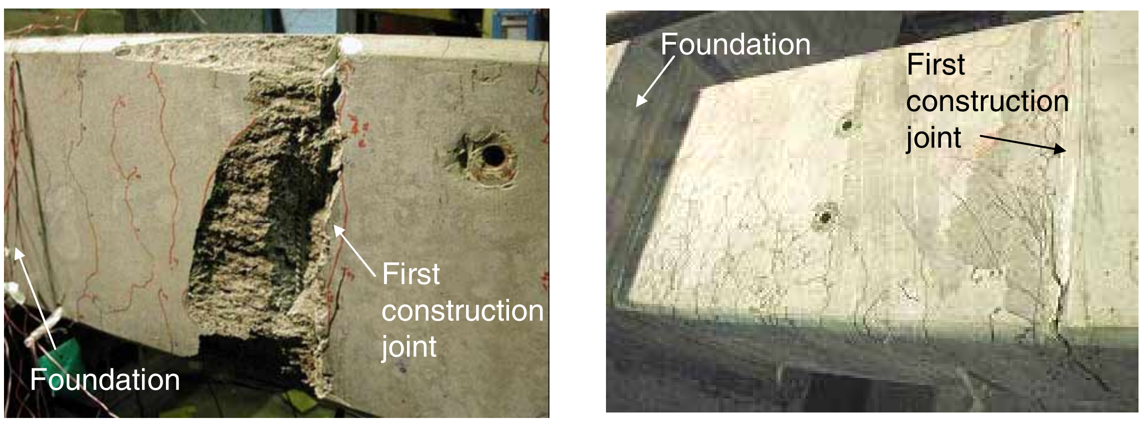
The load-deflection behavior for Specimens 1-3 is shown in Figure 5. The all-concrete piers (Specimens 1 and 2) reached peak lateral load at a lateral drift of 2.1% at which time the more lightly reinforced specimen (Specimen 1) experienced a sudden, catastrophic failure. Longitudinal reinforcement buckled, the cover spalled and a small amount of crushing of the core was observed. Cracking was distributed through the first segment ("hinge" region), and failure occurred at the first construction joint, which was 762 mm above the base of the column, and through which no mild steel reinforcement passed. This peak drift of 2.1% represented the maximum drift at which the dead load of the pier could be sustained. Because Specimens 1 and 2 were attached in this test set-up, it was not possible to continue testing Specimen 2 once Specimen 1 failed. However as shown in Figure 6a, Specimen 2 was at the onset of failure as observed by the severe spalling and initial core crushing just as Specimen 1 failed.

Specimen 3, which contained HPFRCC, also reached its peak lateral load at a lateral drift of 2.1%, but rather than failing suddenly, it exhibited a much more controlled post-peak softening behavior. Specimen 3 was able to sustain full axial dead load to lateral displacements in excess of 3.4% drift (at 60% of its peak lateral load capacity). Similar to Specimens 1 and 2, Specimen 3 formed large localized cracks at the unreinforced construction joint. However, Specimen 3 showed much finer, well-distributed cracking in the hinge regions than the all-concrete specimens (1 and 2). Figure 6b shows the local region of impending failure of Specimen 3 just prior to termination of the test (beyond 3.4% drift).



(a) Specimen 1 (b) Specimen 2 (c) Specimen 3

Figure 5 Lateral Load-Displacement Response of Specimens 1, 2 and 3.



(a) Specimen 2 at failure of Specimen 1 (b) Specimen 3 at 3.4% drift

Figure 6 Damage near peak drifts for piers with (a) concrete and (b) HPFRCC segments.

The specimens with HPFRCC reached higher drift levels than the concrete specimens because the HPFRCC maintained its structural integrity rather than spalling, had much less severe compression softening behavior than normal concrete, and provided better confinement for the mild longitudinal steel.

DAMAGE ASSESSMENT USING PERFORMANCE-BASED FRAMEWORK

In order to quantify the benefits of the proposed enhanced performance system, a comparison is being made between the self-centering system and a traditional reinforced concrete bridge pier system using the performance-based earthquake engineering assessment methodology described earlier. The current focus is on studying the self-centering UBPT bridge pier system in the areas of structural analysis (IM to EDP) and damage analysis (EDP to DM). Presented in this paper is the approach being taken for the damage analysis, i.e. the relationships between various calculated engineering demand parameters and observable damage measures.

Approach to Damage Estimation

The relationship between the engineering demand parameters and damage measures for a structural system is represented by a fragility curve. The fragility curve shows the probability of being in a given damage state or greater given a certain level of the engineering demand parameter. The development of fragility curves for structural or non-structural components is typically based on available data from experimental testing or observed damage from previous earthquakes. For example, Berry and Eberhard [10] developed fragility curves relating drift (a demand parameter) to the damage states of concrete spalling and longitudinal reinforcing bar buckling for reinforced concrete bridge columns based on an extensive database of experimental results from 104 cyclic column tests. Aslani and Miranda [11] are developing fragility relationships for slab-column connections in reinforced concrete buildings, also based on experimental results. Unlike the two previously mentioned studies, which are both based on existing experimental data, there is very little existing experimental data for UBPT columns from which to develop fragility curves for the proposed enhanced performance system. It is for this reason that the fragility curves are being developed based primarily on finite element simulations.

To develop fragility curves analytically for the proposed UBPT system, several issues need to be considered. For example, sources of uncertainty such as material properties and design parameters must be incorporated into the fragility curves. Damage measures for this new system must be identified, as many will be different than the damage states typically used for reinforced concrete bridge columns due to the differences in behavior between the two systems. Finally, it must be shown that the occurrence of these damage states can be accurately predicted by simulation. Several types of simulation could be used such as non-linear static pushover analyses and dynamic time-history analyses. Each of these issues is described in more detail below.

Incorporation of Uncertainty

The fragility curves for the damage analysis display the probability that a structural component or system will be in a certain damage state or higher given a level of an engineering demand parameter. The uncertainty in the response of the system to the EDP arises from numerous sources, some of which include variations in material properties and quality of construction. The first sources of uncertainty being incorporated into the fragility curves for this study are material and design uncertainty. Using mean and standard deviation values for various material parameters (e.g. compressive strength and elastic modulus of concrete, yield strength of mild steel, etc.) and various design parameters (e.g. column dimensions, reinforcing ratios for mild reinforcing and post-tensioning, etc.) numerous simulations on a prototype bridge pier are being performed using Monte Carlo analyses. First-Order Second-Moment (FOSM) methods being developed by Baker and Cornell [12] are also being investigated as a more efficient means of incorporating uncertainty into the fragility curves.

Identification of Damage Measures

As previously mentioned, the damage measures for the UBPT system will be different than those used for traditional reinforced concrete systems. Various researchers have proposed different sets of criteria to define states of damage in reinforced concrete bridge columns (e.g. Hose *et al*. [13]), where the damage states correspond to different levels of action that must be taken in terms of repair or replacement of part or all of the system. Some damage states, such as residual displacement, will be applicable to both the traditional systems and the proposed system. In this case, the two systems can be compared directly on the fragility curve level. Other examples of damage measures include cracking and spalling of concrete. However several of the damage states for reinforced concrete will not apply to the proposed system, such as spalling of concrete cover, as the HPFRCC material is self-confining and does not spall. Furthermore, buckling of longitudinal reinforcement is unlikely because the mild reinforcement is in short lengths within each segment and again, the HPFRCC is self-confining and does not spall. Damage measures unique to the self-centering system include loss of prestress and opening of segmental joints.

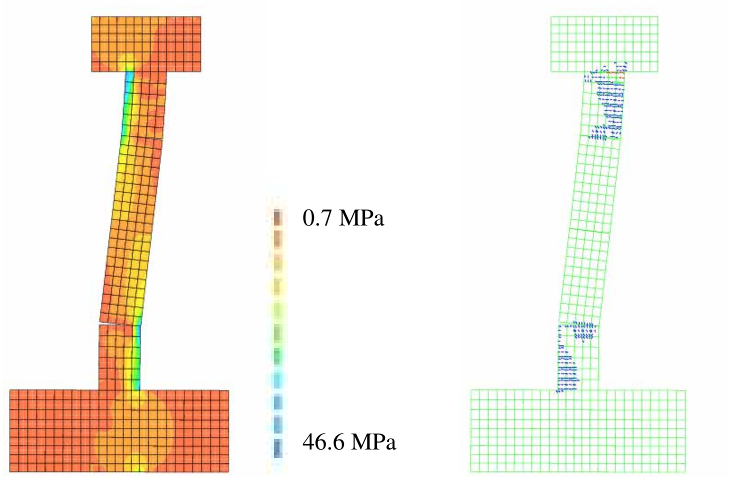
The benefits of the proposed system will become more apparent when the damage states found only in traditional reinforced concrete systems (e.g. spalling) are eliminated altogether and the new DMs for this enhanced-performance system are less costly and less likely to be reached.

Calibration of Finite Element Models

The fragility curve development for the proposed bridge pier system is dependent on accurate simulation of system performance. Previous research by Kwan and Billington [5] and Yoon and Billington [14] has shown the ability of finite element simulations to capture the damage response of unbonded post-tensioned concrete structural systems. Previous approaches to simulation are being further validated by the authors through simulations of the large-scale tests performed and discussed above.

The specimens are modeled using 2D nonlinear analyses with a commercial finite element program and are subjected to a monotonic "pushover" loading. The concrete and HPFRCC segments are modeled using nine-noded quadrilateral isoparametric plane stress elements with a 3 by 3 integration scheme. The elements include material nonlinearity; a smeared rotating crack model is used in tension and elastic-perfectly plastic response is considered in compression. The mild steel reinforcement is modeled as elastic-perfectly plastic and is embedded such that it is assumed to have perfect bond with the concrete elements. The unbonded steel post-tensioning tendons are modeled with two-noded truss elements that are tied at their nodes to the concrete elements, allowing the strain in the concrete to be distributed along the entire length of the tendons. The interface behavior between the precast segments is modeled using six-noded interface elements with a discrete cracking model that uses fracture energy-based linear tension softening. A more detailed description of the procedures is not discussed here for brevity. A similar and more detailed description of the modeling procedures can be found in Kwan and Billington [5] and Yoon and Billington [14].

A contour plot of the principal compressive stresses (Figure 7a) and cracking pattern (Figure 7b) for Specimen 1 at a drift ratio of 2% are shown below. Both are superimposed on the deformed shape of the specimen, which has been magnified by a factor of four. The opening of the construction joints closest to the foundation and cap beam stub can be seen in both figures. In Figure 7a, the highest compressive stresses can be seen in the compressive region of the column section where the gap has opened between segments. The cracking pattern, shown in Figure 7b, is comparable to what was observed in the large scale testing (see Figure 6b).



(a) Principal Compressive Stresses (b) Cracking Pattern

Figure 7 Finite element simulation of Specimen 1 at 2% drift.

Development of Analytical Fragility Curves

In order to evaluate the benefits of the proposed enhanced performance system, a prototype UBPT bridge pier was designed for comparison with a traditional reinforced concrete bridge pier. A "base" design was first generated, from which a number of the design parameters are varied as a means of incorporating the design uncertainty into the fragility curves. The bridge pier was designed in general accordance with the California Department of Transportation's (Caltrans) Bridge Design Specifications [15] and Seismic Design Criteria (SDC) [16]. The pier was designed as part of a single-column-bent, single-bent overpass highway bridge with two equal spans, with dimensions chosen such that they would be representative of the majority of bridges of this type in California. A similar study was performed for ordinary reinforced concrete bridge piers by Mackie and Stojadinovic [17].

The superstructure for the proposed bridge was assumed to be a 4 cell reinforced concrete box girder with a capacity for three lanes each 3.6 m in width. The bridge has two equal spans of 60 m, and the column height is 10.4 m from the top of the footing to the center of gravity of the superstructure. The column was designed to be constructed as a precast segmental system; therefore the section was designed to be hollow to reduce the weight of the segments. A cross-section of the design is shown below in Figure 8.

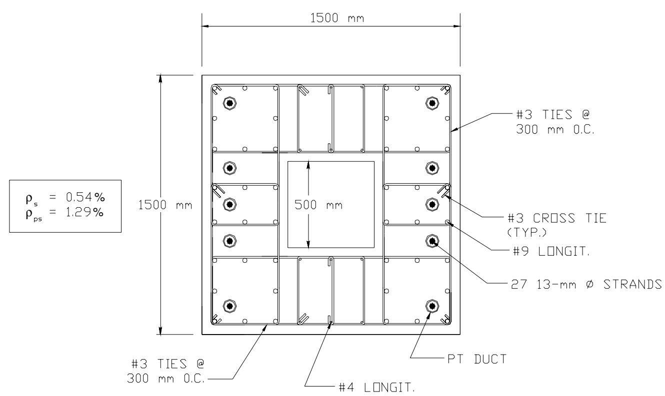


Figure 8 Prototype UBPT column design (cross-section).

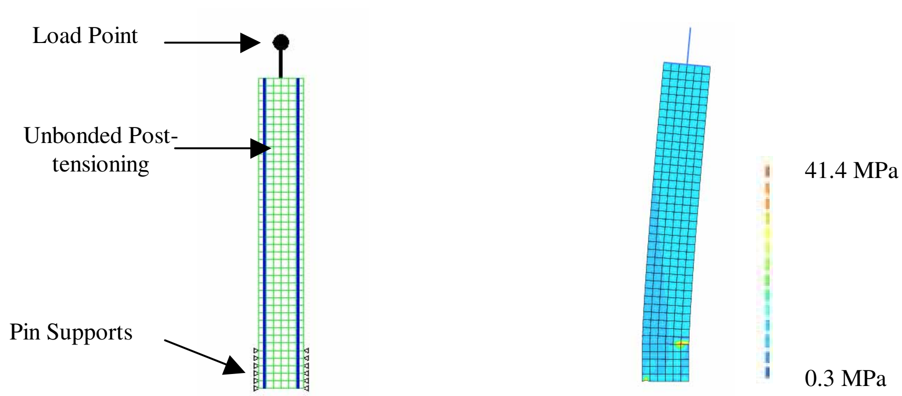
The design loads for the prototype bridge pier were determined according to the procedures in the Caltrans SDC. The determination of design loads is deterministic and is based on the occurrence of the Maximum Credible Earthquake. The bridge was designed for a magnitude 6.5 earthquake with a peak ground acceleration of 0.5 g. The design soil profile for the bridge was a dense soil or soft rock, corresponding to Caltrans SDC soil profile C (NEHRP Class C/USGS Class B). The spectral acceleration at the fundamental period for 5 percent elastic damping was determined to be 0.45 g and was obtained from Caltrans SDC's Acceleration Response Spectrum curves.

Once the base design was generated, a finite element model was created for the prototype bridge pier. A systematic approach to incorporating variations in the properties and parameters is under development. However to demonstrate the general method, we present here an initial, simplistic set of analyses for developing the analytical fragility curves. Using mean values for selected material and design parameters (compressive strength and elastic modulus of concrete, yield strength of steel, and effective prestress in post-tensioning tendons), the model was subjected to 16 non-linear static pushover analyses (four parameters at four different values each) while varying these parameters. Table 2 shows the parameters used and their respective ranges.

The damage state for the fragility curve is the onset of compressive crushing of the concrete, as defined by the point at which the principal compressive strain reaches 0.003 mm/mm at all integration points within the element experiencing the highest compressive stresses. A contour plot of principal compressive stresses at the onset of crushing for one of the analyses is shown below in Figure 9. Figure 10 shows the lateral load versus drift ratio for the same column, and the point at which crushing began to occur is labeled. The results of these analyses were used to generate an example fragility curve for the prototype system (Figure 11). A standard cumulative log-normal distribution is fit to the data. The slope of this fragility curve is relatively steep; this simply shows that the damage state in question is not especially sensitive to the chosen varied parameters.

Table 2 Material and Design Parameters for Fragility Curve Analysis

Parameter	Range	Increment
Concrete Compressive Strength	40 to 50.5 MPa (6 to 7.5 ksi)	3.5 MPa (500 psi)
Concrete Elastic Modulus	28 to 36 GPa (4000 to 5200 ksi)	2.8 GPa (400 ksi)
Mild Steel Yield Strength	414 to 496 MPa (60 to 72ksi)	27 MPa (4 ksi)
Post-tensioning Stress	895 to 1105 MPa (130 to 160 ksi)	70 MPa (10 ksi)



(a) Schematic of FE Model (b) Principal Compressive Stresses

Figure 9 Finite element simulation of prototype UBPT column.

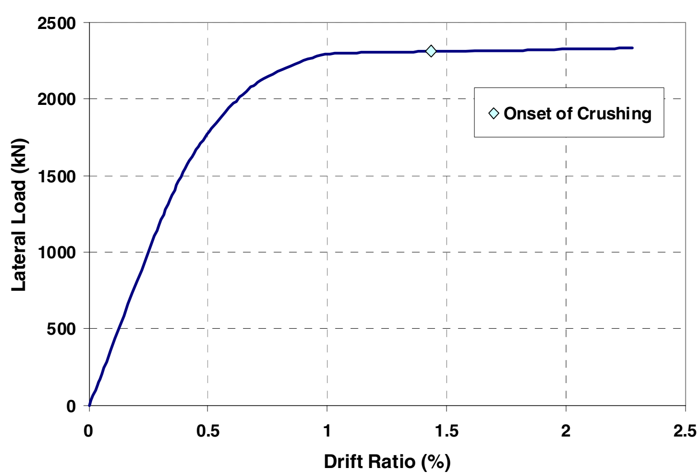


Figure 10 Lateral load vs. drift ratio for prototype UBPT column.

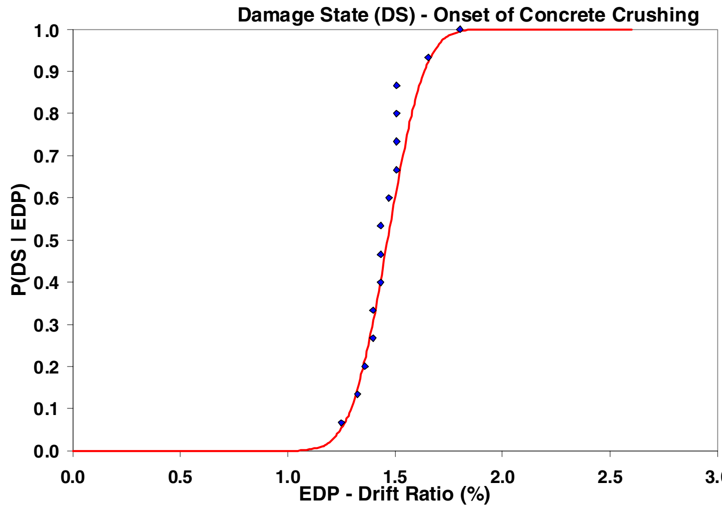


Figure 11 Example fragility curve for prototype UBPT column.

FUTURE WORK

Once the IM-EDP and EDP-DM relationships have been developed for the proposed UBPT system, the next step in the PEER-PBEE assessment is the development of loss functions to relate the damage measures to the decision variables. As discussed previously, the decision variables are typically expressed in terms of monetary losses, casualties, and structure or facility downtime. While quantifying direct monetary losses due to repair costs is a fairly straightforward procedure, the calculation of indirect losses from closure of portions of a transportation network is a much more complex procedure, and is currently a topic of ongoing investigation. This research will focus on the development of loss functions related to direct losses from repair costs. Repair costs can be estimated from various construction cost databases. However, as with all of the assessment portions in the PEER-PBEE framework, the relationship between cost and damage must be developed in a probabilistic fashion. The loss functions will be developed in a similar fashion to the fragility functions, where the relationship is displayed as the probability of the cost exceeding a certain level given a certain damage state, and where sources of uncertainty can include differences in cost between contractors for various services and differences in repair techniques (Aslani and Miranda [11]).

SUMMARY & CONCLUSIONS

An enhanced performance, precast segmental bridge pier system has proposed for use in seismic regions for improved post-earthquake serviceability. The proposed system provides reduced residual displacements through the use of UBPT, and improved hysteretic energy dissipation and post-peak ductility and capacity through the use of HPFRCC. An approach for assessing the performance of this new system was described, following the PEER-PBEE methodology. In particular, an example development of an EDP-DM relationship was demonstrated using numerical simulations. When using numerical simulations in the absence of a database of experimental results, the accuracy of the simulations must be verified and uncertainty must be incorporate in the analyses.

ACKNOWLEDGEMENTS

The research on performance-based assessment of the bridge pier system is sponsored by the Pacific Earthquake Engineering Research Center through the Earthquake Engineering Research Centers Program of the National Science Foundation. The experimental research reported was sponsored by the National Science Foundation under grant CMS-9984127. Additional funding from the Post-Tensioning Institute is gratefully acknowledged. Special thanks are due to MidState Precast and Charles Pankow Builders, Inc. for their painstaking care and generous donation of time and materials to construct the precast specimens. Donations of PVA fibers from Kuraray Company, Ltd. and epoxy from Sika USA are also greatly appreciated. The results presented here are the opinions of the authors and not necessarily those of the sponsors or donors to this research.

REFERENCES

1. Deierlein GG, and Moehle J. "A Framework Methodology for Performance-Based Earthquake Engineering." Proceedings of the 13th World Conference on Earthquake Engineering (13WCEE*),* Vancouver, August 2004.
2. Billington SL, Barnes RW, Breen JE. "Alternate Substructure Systems for Standard Highway Bridges," ASCE J. Bridge Engineering 2001; 6(2): 87-94.
3. Priestley MJN, MacRae GA. "Seismic Tests of Precast Beam-to-Column Joint Subassemblages with Unbonded Tendons," PCI Journal 1996; 41(1): 64-80.
4. Ikeda, S. "Seismic behavior of reinforced concrete columns and improvement by vertical prestressing," Proc. XIIIth FIP Congress on Challenges for Concrete in the Next Millennium, V.1, Balkema, 1998.
5. Kwan WP, Billington SL. "Influence of Hysteretic Behavior on Equivalent Period and Damping of Structural Systems," ASCE J. Structural Engineering 2003; 129(5): 576-585.
6. Kwan WP, Billington SL. "Unbonded Post-tensioned Bridge Piers: Part I - Monotonic and Cyclic Analyses," ASCE J. Bridge Engineering 2003; 8(2): 92-101.
7. Li VC, Leung CKY. "Steady-State and Multiple Cracking of Short Random Fiber Composites," ASCE J. Engrg. Mech 1992;. 118(11): 2246-2264.
8. Li VC. "Reflections on the Research and Development of Engineered Cementitious Composites (ECC)," Proceedings of the JCI International Workshop on Ductile Fiber Reinforced Cementitious Composites (DFRCC) - Application and Evaluation (DRFCC-2002), Takayama, Japan, 2002, pp. 1-21.
9. Rouse, JM. "Cyclic Response of Unbonded Post-tensioned Columns made with Ductile Cement-based Composites." Doctoral Thesis 2004, Cornell University, Ithaca, New York. (In preparation).
10. Berry MP, Eberhard MO. "A Practical Performance Model for Bar Buckling." ASCE Journal of Structural Engineering 2003, *in review*
11. Aslani H, Miranda E. "Fragility of Slab-Column Connections in Reinforced Concrete Buildings." ASCE Journal of Structural Engineering 2004, *in preparation*
12. Baker JW, Cornell CA. "Uncertainty Specification and Propagation for Loss Estimation Using FOSM Methods." Proceedings of the 9th International Conference on Applications of Statistics of Probability in Civil Engineering ICASP9, San Francisco, California, 2003.
13. Hose Y, Silva P, Seible F. "Development of a Performance Evaluation Database for Concrete Bridge Components and Systems Under Seismic Loads." Earthquake Spectra 2000; 16(2): 412-442.
14. Yoon JK, Billington SL. "Experimental and Numerical Studies of Precast Unbonded Post-tensioned Bridge Columns with Engineered Cementitious Composites." Research Report 02-03. Cornell University, Ithaca, New York, 2002.
15. California Department of Transportation. Bridge Design Specifications 2000.
16. California Department of Transportation. Seismic Design Criteria Version 1.2 2001.
17. Mackie K, Stojadinovic B. "Seismic Demands for Performance-Based Design of Bridges." Pacific Earthquake Engineering Research Center Report 2003.