PERFORMANCE-BASED SEISMIC DESIGN OF REINFORCED CONCRETE BRIDGE COLUMNS

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SUMMARY

There is a new focus in seismic design on the performance of reinforced concrete bridges. In a performance-oriented environment, a bridge is designed to meet specified performance levels. Recent earthquakes have provided preliminary data demonstrating that large losses can result from inadequate performance of bridge structures. Further research is required to better define performance indices and develop overall design methodologies to improve the understanding of bridge performance.

A research program was undertaken to improve methods to evaluate the performance of reinforced concrete bridge columns using experimental and analytical methods. In the experimental investigation, columns with various longitudinal reinforcement ratios and aspect ratios were tested to characterize the response of modern bridge columns subjected to lateral loading. In the analytical investigation, methods to numerically assess engineering parameters to evaluate element damage were developed using the experimental results. The research results were used to develop a performance-based seismic design framework for reinforced concrete bridge columns.

INTRODUCTION

Performance-based design may be defined as design to reliably achieve performance objectives. Each performance objective is defined by a single pairing of a structural performance level and a seismic demand level. The structural capacity for each performance level is related to a specific state of damage or required repair and is quantified using one or more engineering limit states. For reinforced concrete bridges supported by columns, key aspects of structural performance include cracking, spalling, and cross section fatigue. Development of performance-based seismic design provisions for reinforced concrete bridges requires engineering approaches that consider these aspects to define the state of structural performance.

Recent earthquakes, including the Loma Prieta and Northridge earthquakes, have provided preliminary data to study the seismic performance of bridges. In these earthquakes, damage was primarily focused in older bridge construction; damage to modern reinforced concrete bridges was predominantly in the form of cracking and minor spalling. As demonstrated by damage to older construction, damage to important bridges that results in delayed operation may be of significant economic cost.

Although past performance has indicated that modern bridges performed well, the future performance of modern bridges is not known. Current seismic design standards for reinforced concrete bridges do not provide adequate performance design requirements. Most current codes specify seismic design force levels and standard details for key structural components. Likewise, previous seismic research has focused primarily on strength and detailing rather than structural performance. Although future earthquakes may result in damage to reinforced concrete bridges, it is possible that the damage may be limited; this result may indicate that current detailing requirements have resulted in overly conservative column design for the selected design earthquake.

Although previous research on the response of reinforced concrete bridge columns is extensive, these studies are not adequate to develop all aspects of performance-oriented design. Previous experimental research primarily

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has emphasized improving the design and understanding of reinforced concrete bridge columns subjected to significant plastic cyclic displacement demands. Development of performance-based design methods requires further experimental and analytical investigations to evaluate intermediate damage levels and to develop analytical models and appropriate design methodologies.

Recognizing the shortcomings of current information, a research program was designed to develop improved methods to evaluate the performance of modern bridge columns over the range of typical geometries and range of performance levels. The objective of the research program was to characterize and quantify the seismic performance of reinforced concrete bridge columns. The research objective was achieved by designing an experimental and analytical investigation that would characterize the seismic performance of modern bridge columns at various damage states. Results from the experimental and analytical investigation were used to develop and evaluate the important aspects of structural damage. Engineering design criteria for use within the performance-based seismic design framework for reinforced concrete bridges were developed using the analytical and experimental results.

PERFORMANCE-BASED SEISMIC DESIGN FRAMEWORK

Performance-based seismic design may be defined as design to reliably achieve targeted performance objectives. Figure 1 illustrates the differences between performance-based seismic design and current seismic design provisions (e.g., AASHTO for bridges and UBC for buildings). Those codes define a single level of seismic hazard and a single level of performance that is generally understood to be life-safety. Furthermore, those codes use indirect methods such as base shear strength and linear-elastic analysis to define the performance state, which can be expected to be relatively inaccurate. Performance objectives other than the life-safety are not evaluated explicitly; whether these performance objectives are achieved depends somewhat randomly on the seismic environment characteristics of the structural and nonstructural components.

Where current seismic design provisions specify design requirements for a single hazard and performance level, performance-based seismic design can specify performance for a range of hazard levels. Performance can be defined in terms of structural component parameters (e.g., spalling), structural parameters (e.g., stability), or functionality. Categorizing structures as ordinary or important, the example performance based-seismic design framework has multiple performance objectives for ordinary structures important structures.

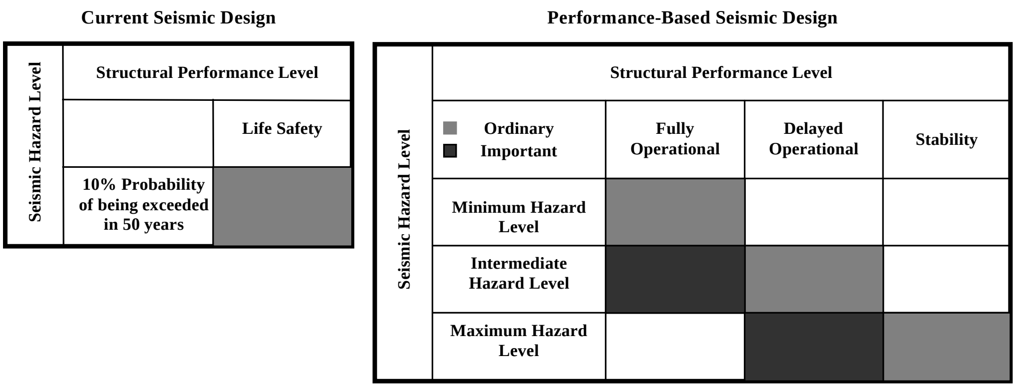

Figure 1 Performance Objectives for Current and Performance-Based Seismic Design

Performance-based seismic design of bridges requires that the engineer complete the following tasks: select performance objective(s), define performance level using engineering limit state, define site hazard level at site, perform structural design and evaluation using engineering approaches and quality assurance. The following sections summarize recommendations for the tasks of defining and quantifying the performance objectives within the context of a performance-based seismic design framework for the design of reinforced concrete bridges. Details of appropriate engineering approaches may be found elsewhere [Lehman 1998].

Performance Levels

Structural performance may be defined by the required repair effort. To date most codes and documents advocating performance-based design of bridges have adopted a two-level design framework. Logically, a the two levels correspond to the seismic hazard level for which structural repair is not required and the seismic hazard level corresponding to severe damage without collapse (structural stability), however this is not always the case (e.g. ATC 32). The performance-based seismic design framework recommended herein adopts three performance levels. The three performance levels outlined in Table 1 are designated as Fully Operational

Performance Level, Delayed Operational Performance Level, and Stability Performance Level. Each performance levels is defined by the expected bridge serviceability, required repair effort, and future performance. This framework can be easily condensed to a two-level framework. However, the distinct disadvantage of the two-level approach is that the level for which repair is required is not specified.

For a bridge meeting the Fully Operational Performance Level, repair is not required and the bridge is expected to be fully serviceable immediately following the earthquake. The future seismic performance will be essentially unaffected which requires negligible damage accumulation. A bridge meeting the Delayed Operational Performance Level requirements is expected to have sustained some damage during the earthquake. The bridge should provide limited service to emergency vehicles. Closure of the bridge should be limited to several days provided sufficient resources are available. In future, more significant events, the bridge performance is expected to be close to the original performance. A bridge meeting the Stability performance level is expected to have sustained significant damage. As a result, partial or full replacement of bridge elements (including columns and restrainers) may be required and the bridge may remain out of service for several weeks or months. The future performance of the structure is limited; however, in its damaged state, the bridge is expected to survive an aftershock of lesser intensity.

Table 1 Performance Levels

Performance Level	Required Repair Effort Serviceability	Future Performance
Fully Operational	Minimal Damage Fully Serviceable	Original Level
Delayed Operational	Repairable Damage Delayed Service	Slightly Reduced Relative to Original
Stability	Unrepairable damage Unserviceable	Minimal Level (Aftershock)

Seismic Hazard Levels

Ideally, seismic hazards should be considered in terms of ground motion, including temporal and spatial variation, pounding, liquefaction, lateral spreading, and landslides. Although all types of hazard levels, especially ground shaking and lateral spreading, may be critical for bridge design, seismic hazards are generally defined only in terms of ground shaking. This approach will be adopted herein.

For the framework under discussion, three performance levels have been defined; each performance objective requires a pairing of a performance level and a minimum seismic hazard level. Therefore, three seismic hazard levels are defined. Ideally, the return period for the seismic hazard level will depend on the seismicity of the region and the site and is defined to match an acceptable level of uniform risk. Although defining a single return period for each hazard level may expedite the design process, it may be more realistic to consider the risk for each performance level. However, defining a uniform level of risk for each performance level depends on the seismicity of the region, economic factors, and structural importance of the bridge. In the context of the performance-based design framework proposed (Figure 1), the expected performance for ordinary structures is expected to be fully operation for the minimum level event, delayed operational for the intermediate level event and life safe for the significant level event. Therefore, the earthquake levels are defined in a broad sense, without specific reference to uniform risk or hazard levels.

Performance Objectives

A performance objective is the pairing of a performance level and a seismic hazard level. Discrete performance objectives are defined for each seismic hazard level. Five performance objectives are defined within the performance-based design framework (Figure 1). Three are defined for Ordinary bridge structures; two are defined for Important bridge structures. Ordinary bridges are structure is expected to meet the objectives of the Fully Operational performance level for the Minimum hazard level, the objectives of the Delayed Operational performance level at the Intermediate hazard level, and the objectives of the Stability performance level at the Significant hazard level. Important bridges are expected to meet the objectives of the Fully Operational performance level at the Intermediate hazard level and the objectives of the Fully Operational performance level at the Significant hazard level. Closure of Important bridges is not permitted for the hazard levels specified.

EXPERIMENTAL PROGRAM

Seismic design of reinforced concrete bridges requires that yielding elements withstand the expected cyclic deformation demand. Sophisticated numerical modeling may be required to fully characterize the cyclic response of the structure. The inelastic response of reinforced concrete elements, such as columns, joints, and beams is complex, and even the most sophisticated modeling of an element can require simplification. Therefore, analysis and design methods must be evaluated using experimental results.

Many parameters can influence the inelastic cyclic behavior of a cantilever bridge column. Previous research has demonstrated that for modern columns, the parameters that have the most influence on the response include spiral reinforcement ratio, the column shear demand, the axial load ratio, the column aspect ratio, and the quantity of longitudinal reinforcement are the most significant. However, current code requirements restrict the range of these five parameters and as a result there are typical ranges of each parameter found in modern construction. For example, codes specify a minimum spiral reinforcement ratio. In capacity design, column shear demands are influenced primarily by aspect ratio and longitudinal reinforcement ratio; most seismic codes limit shear stress ratios to $12\sqrt{f'_c}$. Axial load levels due to dead load are typically less than 0.1 $f'_c A_g$, where f'_c = concrete compressive strength and A_g = the gross area. Column aspect ratios depend on bridge geometry and column diameter (typically sized to satisfy maximum axial load ratios) and typically vary between 1 and 10. On average, longitudinal reinforcement quantities fall between 2% and 4% of the gross cross-sectional area.

A study of previous research results [Taylor 1993] indicates that although a wide range of the parameters have been studied, columns with aspect ratios of 4 or greater and longitudinal reinforcement ratios between 2% and 2.5% have received limited attention. These results indicate that longitudinal reinforcement ratio and aspect ratio, which may vary significantly in the field, had not been thoroughly investigated in the laboratory. Therefore, an experimental research program was designed to investigate the influence of these parameters on response and failure of modern bridge columns in the context of performance-based seismic design.

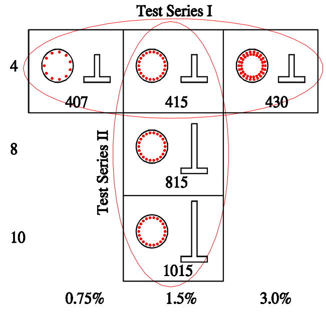

Figure 2 Test Matrix

The test program was designed to model the behavior of a full-scale reinforced concrete bridge column assembly, measure local and global response quantities, and facilitate comparison with previous research studies. The experimental research study was developed to establish the effects of column aspect ratio and longitudinal reinforcement ratio on seismic behavior. Two test series were developed to individually study each focus parameter and are shown in Figure 2; the two study parameters are identified for each column. The column designations are indicated; each designation has three or four numerals. The latter two numerals denote the percentage of longitudinal steel; the first one or two numerals indicate the column aspect ratio, e.g., for column designation 815, 8 indicates an aspect ratio of 8 and 15 indicates a reinforcing ratio of 1.5%).

Test series I consisted of three columns that varied in longitudinal reinforcement ratio; the aspect ratio of each column was 4 to 1. The center column, representing an "average" bridge column, was reinforced with 1.5% steel longitudinally and was denoted Column 415. Column 407, shown to the left of Column 415, had half the amount of longitudinal steel (0.75%); Column 430, shown to the right of Column 415, had twice the amount of longitudinal steel (3.0%) and was detailed with bundled bars. The three specimens of second test series, which will be denoted Test Series II for the remainder of the report, are depicted in the center column of the matrix. The aspect ratios of the three specimens of Test Series II varied between 4 and 10; the columns were reinforced with 1.5% longitudinal steel. Therefore, Columns 815 and 1015 had an aspect ratio of 8 and 10, respectfully.

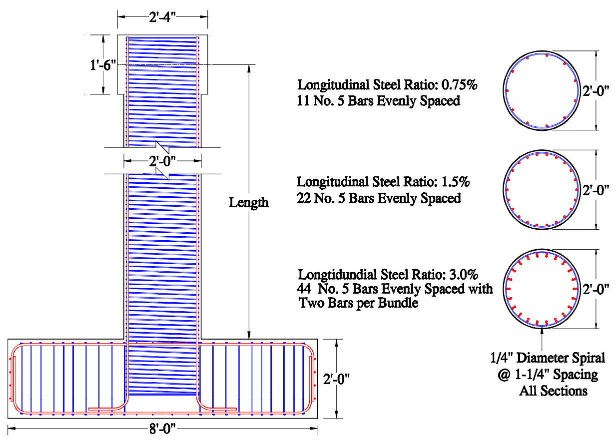

Figure 3 Specimen Details

Details of the geometry and reinforcement are shown in Figure 3. The column diameter was 2 feet to model a 6-foot diameter prototype column. The three columns of Test Series I had lengths of 8 feet. Columns 815 and 1015, had column lengths of 16 feet and 20 feet, respectively. The columns were reinforced longitudinally with No. 5 bars. The longitudinal reinforcement was embedded into the joint to a depth of 21.5 inches, approximately 34-bar diameters. The column spiral reinforcement ratio was 0.7%. The spiral was 1/4 inches in diameter smooth wire and spaced at 1-1/4-inches. The spiral reinforcement was continuous throughout the column height and joint depth. Footing ties were 1/4 inches in diameter spaced at 4 inches on center.

Material Properties

Table 2 shows the specified, expected and actual strengths of the longitudinal steel, spiral steel, and the concrete. The specified strength is the minimum permissible strength. The expected strength is used in capacity design to predict the upper-bound demand from inelastic action of adjacent elements. The actual strength is the strength measured from the actual materials used in the test specimens. Details of the testing procedures and the measured stress-strain responses for each material can be found elsewhere [Lehman 1998].

Table 2 Material Properties

Material	Specified (ksi)		Expected (ksi)		Actual (ksi)	
	Yield	Ultimate	Yield	Ultimate	Yield	Ultimate
Longitudinal Steel	60	80	66	92.4	68.4	93.3
Spiral	80		88		96.9	98.9
	Peak	Confined	Peak	Confined	Peak	Confined
Concrete	3.3		4.2	6.3	Varies	N/M

Loading

Axial and lateral loads were applied to the top of the column. Figure 4 depicts the experimental configuration. The applied axial load of 147 kips was approximately $0.07f'_{ca}A_g$, where f'_{ca}= the actual concrete compressive strength. The axial load ratio chosen corresponded to average axial load ratios found in single column bent bridge construction. The axial load was applied through a spreader beam using a post-tensioning rods placed on either side of the column.

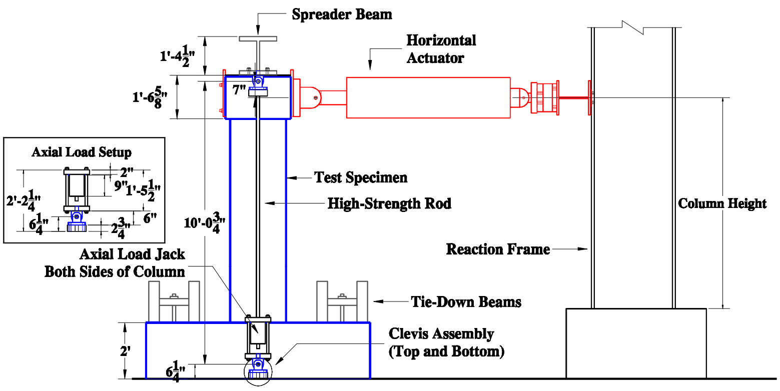

Figure 4 Experimental Set-up

The lateral displacement was applied using a servo-controlled hydraulic actuator that was attached to the top of the column. The imposed displacement history included three cycles at each displacement level. The primary displacement levels were monotonically increasing to provide an indication of damage accumulation. Both pre-yield and post-yield displacement levels were imposed. The pre-yield displacement levels are defined to include a displacement level prior to cracking, two levels between cracking and yielding, and a level approximately corresponding to the first yield of the longitudinal reinforcement. The post-yield displacement levels are defined to include all subsequent cycles. For the post-yield displacement levels, a small displacement cycle was imposed following the three main cycles. Imposed displacement histories were determined for each column according to the column aspect ratio from nominally identical displacement ductility histories. As a result, the three columns of Test Series I were subjected to the same displacement history. Details of the loading regime may be found elsewhere [Lehman 1998]

EXPERIMENTAL IMPLICATIONS FOR PERFORMANCE CRITERIA

To quantify the structural capacity (or performance) for each performance level, the experimental observations must be quantified to define the performance levels. As described previously, the performance levels are defined in terms of the required repair effort. Therefore, the damage states must relate to the repair levels. The repair levels of interest are the no repair, repairable, and beyond repair without collapse.

Observations during testing of the five columns suggest that the sequence of damage was similar of the five columns. This section provides a general description of the progression of damage listing each category of damage chronologically. A brief description of the visual indications is provided. Specific occurrences of each stage of damage are provided for each column may be found elsewhere [Lehman 1998].

Cracking Typically, cracking was not detected during the initial displacement level but was initiated during the subsequent cycle. The crack spacing in the lower portion of the column stabilized following the yield.

Initial yielding of longitudinal steel Yielding of the extreme longitudinal reinforcing bar was noticeable in the force-displacement response or in the physical response. Yielding was detected using strain gauges that had been placed on the longitudinal steel prior to construction and were monitored during testing.

Spalling Initial spalling occurred above the column-footing interface. With continued loading, the spalling region increased in elevation, around the circumference, and into the column core.

Exposure of spirals and longitudinal steel Complete loss of the concrete cover exposed the spirals and longitudinal steel.

Visual extension/fracture of spiral and longitudinal bar buckling Subsequent loading resulted in a permanent displacement of the lower column spirals. Longitudinal bar buckling was visually evident.

Spiral fracture The spirals located within the buckled length continued to extend as the bar continued to buckle until the spiral fractured. The lateral stiffness decrease as a result of spiral fracture which permitted the other longitudinal bars to buckle over a longer length.

Longitudinal bar fracture Fracture of the longitudinal bars occurred after bar buckling. Typically, fracture of one or more longitudinal bars resulted in strength loss significant enough to cause column failure.

Previous research suggests that requirement for a limited repair depends on the crack widths. Recent studies [Elkin 1998] indicate that the extent of spalling, core crushing and damage to the longitudinal reinforcement dictates the ability to repair a bridge column primarily responding in a flexural mode. Similar work suggest difficulty in repairing reinforced concrete cross-sections damaged due to fatigue of the core concrete and longitudinal reinforcement without replacement effort. Engineering parameters including tensile strain in the longitudinal reinforcement, compressive strain in the cover concrete, and strain-based low-cycle fatigue damage index are used to evaluate the damage states and thereby required repair effort. Performance levels are then defined using engineering limit states expressed using the applicable engineering parameters.

Cracking

Crack widths and crack patterns may be used to indicate if repair is required. Large residual crack widths (from 0.01-0.02 in.) may be required to be filled with epoxy or other material to restore the tensile strength. The residual crack widths measured during testing were used to postulate maximum permissible displacement ductility demand to ensure minimum crack widths. Although strain demands should provide a more uniform assessment of crack widths, observations of the post-yield response of the longitudinal strain gauges indicated the measurements were not reliable when the yield plateau was reached. In addition, since yielding of the cross section is progressive, local strain readings do not indicate cross section crack widths.

In general, the measured response indicates that the residual crack widths were 0.01 inches or less for displacement ductility demands less than 1.5 and are 0.02 inches or less for displacement ductilities less than 2. To limit residual crack width, the displacement demand should be less than the twice the effective yield displacement. Limiting the displacement demand to the effective yield displacement will ensure crack widths that do not require column repair and essentially linear response.

Spalling

Cover spalling may result in a reduction in the lateral stiffness of the cross section, durability, and lateral restraint on the longitudinal bar. Post-earthquake damage to the concrete cover can require concrete patching; core damage can require partial or complete replacement of the structural element.

The experimental observations made during this study, which are relevant to the behavior of modern columns, are used to correlate physical damage and predicted response. Additional observations from experimental research [Elkin 1998], which focused on the repair of modern bridge columns, and experimental research by [Kunnath 1997] and [Calderone 1998], both of which focused on the performance of modern bridge columns, are included to substantiate the findings.

An analysis was performed for each test specimen using a discrete modeling technique developed by the authors using the measured material properties. Details of the model and analysis may be found elsewhere [Lehman 1998]. Table 3 summarizes the results. The measured initial spalling displacement is provided in the second column. The predicted compressive strain in the extreme fiber corresponding to the initial spalling displacement is recorded in the last column of the table. The results indicate that the strain corresponding to initial spalling of the cover is in the range of -0.008 to -0.01. For the provided data, the mean spalling strain is -0.009 with a standard deviation of 0.001. Although spalling is not uniquely related to strain demand, the results indicate that compressive strain may provide a reasonable estimate of initial concrete spalling. For design, a compressive strain of -0.007, is suggested. Due to the size of the sample set however, further analysis is warranted.

Table 3 Spalling Strains

Research Team	Column	Measured Spalling Displacement	Predicted Spalling Strain
Lehman and Moehle	407	1.5 in.	-0.008
	415	1.5 in.	-0.008
	430	1.5 in.	-0.01
	815	5.25 in.	-0.009
	1015	7.5 in.	-0.008
Kunnath et al.	A1	1.1 in.	-0.01
Calderone et al.	328	0.8 in.	-0.01
	828	5.25 in.	-0.01
	1028	7.5 in.	-0.01
		Mean	-0.009
		Standard Deviation	0.001

Fatigue and Cross Section Failure

The response and failure of reinforced concrete elements subjected to seismic loading can be influenced by the imposed displacement history. Fatigue of the concrete cover may require removal and replacement of the damaged concrete. Cross section failure, which may include longitudinal bar fatigue and fatigue of the core concrete, may require partial or full replacement of a structural element. Experimental observations suggest that response of a longitudinal bar subjected to cycle loading depends on the lateral restraint provided. The lateral bar restraint, in turn, depends on the condition of the surrounding concrete, and the stiffness and spacing of the spiral. Experimental evaluation of the use of such replacement techniques on modern bridge columns may be found in the literature [Elkin 1998].

Herein, a preliminary study was undertaken to develop a new damage index to model the observed experimental response. Damage resulting from cyclic loading is modeled using a two-phase model. The first phase models damage to the lateral restraint on the longitudinal bar as damage to the concrete cover. The second phase models damage of the longitudinal steel. Both phases use a modified format of the Coffin-Manson equation as shown in Equation 1; the expression relates the number of complete cycles to failure, N_f, to a normalize strain, $\varepsilon_r = \varepsilon/\varepsilon_o$. Miner's rule is employed to determine the damage index, DI (Equation 2). Failure is predicted when $DI = 1$. Equations 1 and 2 were used to develop the expressions for the concrete damage phase and the steel damage phase. The model was developed using the experimental results from Kunnath et al. Details of the development of the model may be found elsewhere [Lehman 1998].

$$N_f = a(\varepsilon/\varepsilon_o) + c \quad DI = \sum_i \left(1/N_f\right) \qquad \text{Eqs. 1 and 2}$$

Using Equation 1, two expressions were developed to estimate the number of cycles required to fully damage the concrete cover and the longitudinal steel at a particular strain demand. Equation 3 predicts the number of cycles required to completely remove the concrete cover, $(N_f)_c$, at a strain ratio of $\varepsilon_c/\varepsilon_{csp}$ where ε_c is the compressive strain and ε_{csp} is the strain corresponding to spalling in the extreme fiber of the confined core (see previous section). Equation 4 predicts the number of subsequent cycles to failure, $(N_f)_s$, following fatigue-induced failure of the cover (i.e., $DI_c = 1$). Note that the expression correctly predicts that failure will result if the column is subjected to one cycle for which $\varepsilon_s = \varepsilon_{su}$.

$$\left(N_f\right)_c = 33\left(\varepsilon_c/\varepsilon_{csp}\right)^{-5} \qquad \left(N_f\right)_s = 0.08\left(\varepsilon_s/\varepsilon_{su}\right)^{-5.5} + 0.92 \qquad \text{Eqs. 3 and 4}$$

$$DI_c = \sum_i \frac{1}{\left(N_f\right)_{ci}}; DI_c \le 1 \qquad DI_s = \sum_i \frac{1}{\left(N_f\right)_{si}} \; if \; DI_c = 1; DI_s = 1 \qquad \text{Eqs. 5 and 6}$$

The concrete and steel damage indices, DI_c and DI_s respectively, are calculated using Miner's rule, as shown by Equations 5 and 6. The experimental results indicate that low-cycle fatigue of the longitudinal reinforcement follows complete spalling of the concrete cover. Mathematically, this is indicated when $DI_c = 1$. Damage to the longitudinal reinforcement is modeled using Eq. 6. Cross-section failure resulting from failure of the longitudinal steel corresponds to approximately $DI_s = 1$. The dual-phase damage index is employed in two stages. The steel fatigue index, $(DI)_s$, is set equal to zero until the concrete damage index, $(DI)_c$, is equal to one. Element failure corresponds to a steel fatigue index value of one, i.e., $(DI)_s=1$.

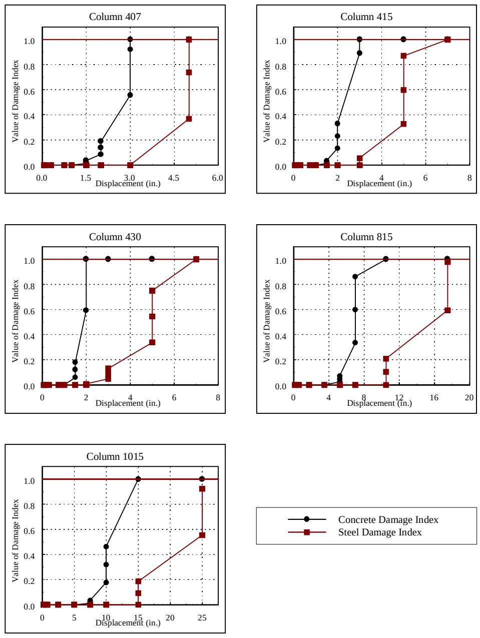

Figure 5 Results using Dual-Phase Damage Index

The dual-phase damage index was evaluated using the experimental results from the present study. The analytical results are shown in Figure 5. The progression of damage predicted by the concrete fatigue model is shown by a line marked with circles. The progression of damage predicted by the longitudinal steel fatigue model is shown by a line marked with squares. The dual-phase index correctly predicts failure for Columns 407 415, and 815. Failure of Column 430 is predicted a bit early. A damage index value of 0.92 corresponds to failure of Column 1015.

The Fatigue Engineering Limit States are specified for the Delayed Operational performance level and the Stability performance level; each corresponds to the required repair effort specified in 4. Fatigue failure of the concrete is not permitted for a bridge designed to meet the Delayed Operational performance state. For the Life Safe Performance Level, failure of the concrete is permissible (i.e., $DI_c = 0$); however, failure of the longitudinal steel should be avoided (i.e., $DI_s < 0.9$). Repair (i.e. concrete patching) will be required if $DI_c > 0$.

Table 4 Engineering Limit States for Different Levels of Repair

Numerical Expression		Physical Damage	Repair
Tensile Strain in Steel		Cracking	Epoxy Injection
Compressive Strain in Cover		Initial Spalling	Patching
Residual Drift		Residual Drift	Plumb Structure
Dual-Phase Damage Index	$(DI)_c = 1$ $(DI)_s = 1$	Complete Spalling	Concrete Replacement
	$(DI)_s = 0.9$	Bar/Spiral Failure	Replacement Only

CONCLUSIONS

In modern construction, the seismic performance of reinforced concrete bridge structures depends on the response of the ductile hinge regions. A research program was undertaken to characterize the response of well-confined, circular cross section, concrete bridge columns. The research objectives included evaluating current design procedures and recommended performance-based design procedures for reinforced concrete bridges in seismic zones. The research was executed in three stages. The existing literature was reviewed and used to guide the design of the experimental and analytical components of the investigation. The results of the research study were used to successfully develop a performance-based design framework for reinforced concrete bridges.

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