SEISMIC REPAIR ASSESSMENT OF HYBRID SLIDING-ROCKING BRIDGE COLUMNS THROUGH INTEGRATED EXPERIMENTATION AND EXPERT PANEL SOLICITATION

Jakub Valigura[1], Mohammad Salehi[2], Abbie B. Liel[3] and Petros Sideris[4]

ABSTRACT

Due to large number of bridges that will need upgrade, retrofit, or replacement in coming years, there is an increasing need for seismic bridge design techniques that are compatible with accelerated bridge construction (ABC). This study examines one promising column design strategy, the hybrid sliding-rocking (HSR) system, which incorporates precast segmental columns with unbonded posttensioning, and both rocking and sliding joints. The goal of the study is to evaluate damage states and identify repair strategies for these columns through integrated experimental testing and expert panel solicitation. The expert panel methods use two different established "group solicitation techniques" to identify seismic repair objectives for bridges, and to propose repair strategies for the HSR columns that are consistent with these objectives. In parallel, a series of large-scale pseudo-static cyclic tests at the Texas A&M Center for Infrastructure Renewal are carried out on an HSR column. The column is then repaired, based on the guidance of the expert panel, and tested again. The results show that the column experiences limited damage, consisting of spalling of concrete near the rocking joints, up to 4% drift (consistent with hazard levels with return periods greater than 4500 years). This damage can be repaired with grout and a carbon fiber reinforced polymer jacket. Most of the residual drift can be recovered by recentering sliding joints. The panel found that the HSR columns are less damageable

[1] Design Engineer, KPFF Consulting Engineers, San Francisco, formerly Graduate Research Assistant and Ph.D. Candidate, Department of Civil, Environmental and Architectural Engineering, University of Colorado at Boulder, Boulder, CO, 80309, USA. E-mail: Jakub.Valigura@colorado.edu
[2] Graduate Research Assistant and Ph.D. Candidate, Zachry Department of Civil Engineering, Texas A&M University, College Station, TX, 77843, U.S.A. E-mail: Mohammad.Salehi@tamu.edu
[3] Associate Professor, Department of Civil, Environmental and Architectural Engineering, University of Colorado at Boulder, Boulder, CO, 80309, U.S.A. (corresponding author). E-mail: Abbie.Liel@colorado.edu
[4] Assistant Professor, Zachry Department of Civil Engineering, Texas A&M University, College Station, TX, 77843, U.S.A. E-mail: Petros.Sideris@tamu.edu

than the conventional columns, and promising for application in high seismic areas. The damage states and repair strategies identified will facilitate future performance-based engineering assessments of the new HSR columns.

INTRODUCTION

Under modern bridge seismic design provisions, the most common earthquake-resisting system is a ductile substructure and an essentially elastic superstructure (AASHTO 2011; Caltrans 2013). The substructure, generally single or multi-column bents, is critical because it is where nonlinear response and damage is expected to concentrate, and it therefore dictates the dynamic behavior of the bridge. In seismic areas, currently, reinforced concrete (RC) bridges most commonly have monolithic RC column substructures (Caltrans 2013). After a moderate or strong earthquake, these columns can suffer extensive damage due to flexure, as well as large residual displacements (He et al. 2015; Kawashima and Unjoh 1997). The cost and time of the necessary repairs can have significant economic impacts (Moore et al. 2006).

To reduce these impacts, researchers have proposed and tested different column systems, for example, incorporating post-tensioning (PT) and/or rocking to promote self-centering and reduce damage, *e.g.,* Billington and Yoon (2004), Guerrini et al. (2015), Motaref et al. (2011), Ou et al. (2010), Sideris (2012), and Mohebbi et al. (2018). Among these is the hybrid sliding-rocking (HSR) bridge column (Sideris 2012; Sideris et al. 2014b, 2014c, 2015), which possesses internal unbonded post-tensioning and end rocking joints, as well as intermediate sliding joints to dissipate energy. HSR bridge columns are a precast system, a key characteristic of accelerated bridge construction (ABC). ABC has also been shown to reduce project delivery time, facilitate on-site constructability, improve work-zone safety, and raise material and construction quality (Culmo 2011; Restrepo et al. 2011; Tazarv and Saiidi 2016). While ABC has been increasingly adopted, even in high seismic areas, the use of ABC to date has mostly involved precast *superstructure* elements, while the *substructure* remains cast-in-place (Caltrans 2008a). However, Caltrans recently finished a pilot multi-span precast bridge in Vallejo, CA, which included precast RC columns and cap beam (Caltrans 2018). In 2018, AASHTO released the first edition of a design guide for ABC, which includes sections addressing seismic design of substructure elements (AASHTO 2018), Washington State Department of Transportation (WSDOT) has a chapter on ABC design in their design code (WSDOT 2018), and Caltrans (2018) is currently gathering lessons learned from their pilot projects to prepare standardized guidance for ABC adoption.

To date, research on HSR and other ABC column systems has mainly focused on physical proof-of-concept of the systems or their performance advantages with respect to conventional construction, *e.g.,* Billington and Yoon (2004), Guerrini et al. (2015), Motaref et al. (2011), Ou et al. (2010), or Sideris (2012) and others. For example, Sakai and Mahin (2004) investigated the effect of unbonded PT strands on residual drifts of bridge columns, showing that a single bundle of unbonded strands at the center of the column can limit residual drifts to only 14% those of conventional columns. However, even if new bridge column systems have been shown to have good seismic response, they are unlikely to be a viable substitute for the conventional system unless they can be shown to lower costs over the entire life-cycle of the bridge (WSDOT 2009). A critical component of a life-cycle assessment is a probabilistic seismic performance assessment that quantifies the economic impacts of earthquake damage and repairs, *e.g.,* Mackie and Stojadinovic (2005), Mackie et al. (2008), Yang et al. (2009), or Valigura et al. (2019). Performance-based earthquake engineering assessments (Deierlein et al. 2003; Porter 2003) of innovative bridge column systems have been somewhat limited. However, Lee and Billington (2011) conducted a seismic loss assessment to compare a bridge with conventional RC columns to a bridge with RC columns with unbonded PT strands. Their assessment showed that, for a given shaking intensity, the repair costs for the unbonded system were slightly higher, but the repair times were significantly lower. This assessment suggested that there is an advantage to using PT systems if repair costs and time are considered together, while simultaneously demonstrating the need for well-defined damage states to compare loss assessments for competing systems.

Making matters more complicated, there is a lack of systematic guidance on performance objectives of the repaired bridges, design of seismic repairs, or selection of repair strategies, even for conventional bridges. Although Caltrans offers a damage assessment guide (Veletzos et al. 2006), and has sponsored research on design and assessment of repair strategies, such as Saini and Saiidi (2013), these documents stop short of explicit guidance on repair design or prescribed statements of repair objectives.

This study uses expert solicitation methods to develop damage states and repair strategies for a new column system for which no field performance data is available, linking large-scale experimental studies with the expert panel's input, illustrated in Figure 1. In the framework of performance-based engineering, determination of damage states and repair methods has been a major challenge, even for existing structural designs. New systems pose an even larger challenges, because "damage" does not always appear in a conventional form. This study addresses these challenges by combining, in a novel way, expert solicitation and large-scale testing to determine damage states and repair methods.

The method is exercised to investigate seismic damage and repair actions for an ABC-column system, the HSR system, which has low damage behavior, making it promising for application in high seismic areas. The authors invited an expert panel of bridge engineers from industry and academia to assess the lateral behavior of HSR columns, and to design repair strategies. This effort asks the panel to identify desirable objectives of post-earthquake repair actions for bridges, and considerations in selection of repair strategies, due to the lack of standardized guidance on this topic. The panel then proposed repair strategies for HSR columns that could be easily implemented in the field for the types of damage observed during testing. In parallel, large-scale cyclic testing is conducted to assess the behavior of the latest generation of HSR columns under specific, meaningful hazard levels (ranging from 5% to less than 1% probability of exceedance in 50 years, or return periods of 975 to more than 4750 years). The tested HSR column is then repaired using a repair strategy informed by the recommendations of the expert panel, and the performance evaluated by the expert panel. These efforts are intended to set the stage for comparative seismic performance and life-cycle assessment of HSR bridge columns. These tests do not investigate the ultimate displacement capacity of the HSR columns. The experimental testing and the in-person portion of the expert panel solicitation were conducted at the Texas A& M Center for Infrastructure Renewal (CIR).

HSR COLUMN SYSTEM

HSR columns are precast concrete segmental RC columns with end rocking joints, intermediate sliding joints distributed over the column height, and internal unbonded post-tensioning (Sideris 2012; Sideris et al. 2014b, 2014c, 2015). Rocking joints aim to eliminate concrete tensile damage from flexure and provide self-centering capabilities, while sliding joints provide energy dissipation. Joint sliding is essentially a non-damaging response mechanism, because the residual joint sliding is restorable using a hydraulic jack system. Sideris et al. (2014c) proposed that rocking joints are located at member ends, and sliding joints at intermediate locations. Sliding is not allowed at the member ends, because the concrete compressive damage would make the sliding response unpredictable. The study further provides design equations for column geometry to provide the intended behavior. More recently, Salehi (2019) showed that one or two sliding joints are sufficient to achieve good seismic performance.

Experimental research on large-scale HSR columns and a section of a bridge has demonstrated the superior seismic performance of HSR columns relative to conventional RC columns in terms of peak and residual drift demands, and extent of damage (Sideris 2012; Sideris et al. 2014b, 2014c, 2015). In particular, Sideris et al. (2012, 2014b, 2014c, 2015) performed several shake table and quasi-static lateral cyclic tests of a large-scale one-span HSR bridge specimen and two HSR columns (both with a length scale factor of 1:2.4). The sliding joints were concrete-on-concrete interfaces with a thin layer of a silicone material to reduce friction. Rocking joints were located at the ends with, again, dry concrete-on-concrete interfaces. (These HSR columns are referred to here as "Generation 1", to differentiate them from the columns tested in this study, which are described below, and referred to as "Generation 2".)

For the purposes of the study here, the damage documented in Sideris (2012) and Sideris et al. (2014c) during quasi-static tests was of particular interest due to the completeness of damage descriptions available at a range of displacement levels, examples of which are provided in Figure 2. Limited damage occurred at drift ratio levels below 7.8%, consisting of cracks and some spalling at the rocking joint, as well as minor spalling and cracking near sliding joints. The damage for drift ratios between 7.8% and 14.9% consisted of more severe spalling and crushing of the core concrete at the rocking joint. Extensive spalling of the concrete cover was also observed around the sliding joints. Furthermore, after disassembling the column, permanent bearing deformations in the ducts and localized permanent deformations on the PT strands at the sliding joints were also observed.

The Generation 2 HSR columns considered in this study have a smaller number of sliding joints and PTFE-on-PTFE (*i.e.,* "Teflon") sliding interfaces encased within thin steel plates. The PTFE-on-PTFE interface achieves lower friction, while the steel plates alleviate segment surface unevenness and reduce concrete spalling at joint vicinity, an issue identified by Sideris (2012). The Generation 2 columns were designed so that the onset of sliding precedes rocking. This differs from Generation 1 HSR columns, where the rocking preceded the sliding. This change was intended to essentially eliminate damage during small-to-medium displacement demands. At higher displacement demands, the design promotes rocking response, which (unlike conventional columns' plastic hinge formation) guarantees self-centering and limits residual drifts. The new design also has circular rather than square cross-sections, which may reduce stress concentration and spalling propagation observed around corners of the Generation 1 columns (Figure 2).

Salehi (2019) tested several identical specimens. The experiments included lateral cyclic tests up to 8% of drift ratio (later in the text referred to as Tests PR 1-12), biaxial bending tests, and torsional tests. That study showed that HSR columns have effective damping ratios of 10-50% of the critical damping value for drift ratio demands between 1-4%. As a result of this large damping, displacement demands are lower than those of a conventional column at the same ground motion intensity or hazard level (Sideris et al. 2014b; Valigura 2019).

EXPERT SOLICITATION METHODS

The invited panel of bridge experts participated in several questionnaires. The main goals of this part of the study were to: 1) identify desirable objectives of bridge repairs, 2) propose repair strategies consistent with these objectives for HSR columns, and 3) assess the behavior of Generation 2 HSR columns and the recommended repaired strategies. The panel also identified barriers to adoption and implementation of the HSR column system in practice.

Selection of methods of expert solicitation

This study uses two well-established methods of expert solicitation: the *staticized group technique* (SGT) and the *nominal group technique* (NGT) (Delbecq et al. 1975; Dillman 2000).

The SGT involves one round of gathering information from qualified experts through structured interviews or surveys, and can be performed relatively quickly. The NGT method is an extension of SGT that involves performing multiple rounds of questionnaires, during which information is shared in-person between panel members between the rounds of questionnaires. Before each subsequent round, each panel member can review other panelists' responses and participate in a discussion. This controlled feedback, delivered via summary of answers and discussion, allows the panelists to review different points of view that they may have not otherwise considered. Panelists are then able to adjust their answer to converge to a "consensus" answer if one exists (Hallowell and Gambatese 2010).

The study adopted the NGT for questionnaires administered during the meeting of expert panel on site. NGT was chosen because the panel was present in a single location, and when correctly administered, NGT can reduce strong individual bias and quickly converge to a single answer as compared to less structured group techniques (Sillars and Hallowell 2009). SGT was employed for the questionnaires that followed the site visit. The use of SGT reduced the time that panelists needed to spend on the surveys, and, because many of the issues regarding the questions asked in the surveys had already been discussed during the site visit, further discussion was not needed. Steps taken to reduce bias from dominant individuals are described in Valigura (2019). For each questionnaire, the questions and potential answer choices were devised and written so as they did not lead the respondents to certain answer(s) (Dillman 2000).

Panel selection

The authors selected panelists for this study based on their expertise in seismic design and repair of RC bridges and other structures. The number of panelists, eight, was determined prior to issuing invitations; although this number is on the lower end of recommendations in the literature, it was the maximum that could be accommodated given other constraints. The study sought an equal number of experts from academia and industry.

Prior to selecting possible candidates, a point scale was devised, as shown in Table 1, to represent candidates' experience relevant to the objectives of this study. Each candidate could receive a maximum of 18 points. To be considered as a member of the expert panel, a candidate needed to receive at least 10 points and receive points in at least four of the rows, indicating breadth of experience. The points in Table 1 were allocated among categories such that participants would need both breadth and depth of experience to reach 10 points. The point average of the participating panelists was 11.4, with a range between 10 and 17 points.

Structure of expert panel solicitation

The expert solicitation consisted of four questionnaires; complete text and responses are provided in Valigura (2019). The experts were first gathered in the CIR at TAMU and participated in three different tasks. The goal of the first questionnaire, titled *Objectives of Bridge Repairs* and administered as an NGT with two rounds, was to characterize the seismic performance objectives for a repaired bridge, along with the factors that influence these objectives. The questionnaire included questions directly related to the post-repair performance, such as: "*I believe the post-repair stiffness of a modern bridge (expressed as a percentage of the stiffness of the original bridge) should be at least…*"; options ranged from "*less than 80%*" to "*more than 110%*". This questionnaire was administered first in order to initiate a discussion about goals of post-repair performance of bridges to set the stage for later discussions.

Subsequently, the authors made a presentation to the expert panel participants, summarizing damage to the Generation 1 HSR columns tested in previous studies at several different displacement levels (Sideris, 2012; Sideris et al., 2014b, 2014c, 2015). After this presentation, the panel discussed the findings and observed the Generation 2 HSR specimen in the CIR. The intent of this presentation and discussion was to introduce the panel to HSR column lateral behavior and damage, and to ask the panel to identify damage that can or cannot be repaired in field and laboratory environments.

The second on-site questionnaire, termed *Repair Assessment of HSR Columns*, aimed to develop suitable repair strategies for HSR columns for two different damage levels (or damage states). In this questionnaire, the damage to the column was presented using illustrations, pictures (like that shown in Figure 2), and short text descriptions. The panelists were then asked to suggest repair strategies in open text boxes. Although two rounds were planned, the panelists' responses converged after the first round and only one round, followed by an open discussion, was conducted.

The next questionnaire, *Field Use of HSR Columns*, was completed within two weeks of the on-site expert panel gathering. The objective of this questionnaire was to help the authors understand what the panelists believed were the main benefits and drawbacks of the proposed HSR columns. The final questionnaire, *Assessment of New Generation of HSR Columns*, provided documentation of the tests performed on the Generation 2 HSR column, including experiments done on the damaged HSR column repaired with strategies recommended in the *Repair Assessment of HSR Columns* questionnaire (*i.e.,* the experimental work described below). The main objective of the questionnaire was to investigate whether the Generation 2 HSR column behavior and post-repair performance were acceptable for high seismic areas.

EXPERIMENTAL METHODS

Experimental design and test set-up

In parallel with the expert solicitation, a Generation 2 HSR column was tested under quasi-static lateral loading, repaired, and re-tested under the same conditions to compare the repaired column behavior to the original column behavior. This experimental work involved a total of six tests conducted in the Structural and Materials Testing Laboratory at TAMU's CIR: three quasi-static cyclic tests of a Generation 2 HSR column (Tests O1, O2, & O3), and three tests of the same column after repairs (Tests R1, R2, & R3). The column was of 1:2.1 scale and consisted of three segments with hollow circular cross sections, as shown in Figure 3.

The column was designed for a location in downtown Los Angeles, CA (34.039, -118.274) according to AASHTO (2011) design spectra. The period of the prototype column was 0.7 sec, translating into 0.35 sec in model (testing) dimensions. The two end joints, *i.e.,* the column-to-foundation joint and the column-to-cap beam joint, were designed as rocking joints. The two intermediate joints were designed as sliding joints with PTFE-on-PTFE interface. The sliding capacity of each joint was 3.8 cm (1.5 in), representing 1% of drift ratio of sliding capacity at each joint. PTFE layers were the 25% glass-filled type for their higher compressive strength and were bonded to 3 mm (0.125 in) steel plates, which were then bonded to the concrete segment end surfaces. The PTFE surfaces were lubricated with grease to obtain the desired coefficient of friction of 0.05. The location of the rocking and sliding joints were determined based on Sideris et al. (2014c) and Salehi (2019).

The segments consisted of normal weight concrete with 28-day nominal strength of 34.5 MPa (5 ksi). Each segment was reinforced with 32 #4 longitudinal reinforcing bars. Shear resistance was provided by spiral reinforcement (#3 at 5.7 cm (2.25 in) pitch) on the inside and outside perimeter of each segment. Additional confinement was provided by #3 cross ties fastening the outside and inside reinforcement layers together. All reinforcement conformed to ASTM A615 Gr. 60, with yield strength of 420 MPa (60 ksi). The column was post-tensioned using internal unbonded PT strands. The strands were seven-wire 1.52 cm (0.6 in) diameter monostrands of

ASTM A416 Gr. 270, with ultimate tensile strength of 1862 MPa (270 ksi). Further details on the design of this column can be found in Salehi (2019).

All six tests were performed with a cantilever setup, which is representative of the column behavior in the transverse direction. In this setup, the moment at the top rocking joint is small, so no rocking is expected there. Each tendon in the specimen was post-tensioned to 80 kN (18 kips). The total gravity load applied to the column through two 2600-kN (590-kip) actuators and the cap beam self-weight was equal to 9.0% of the column's nominal axial capacity, representing dead load plus 50% of the live load. Tests were performed sequentially without any repairs between Tests O1 and O3, or between Tests R1 and R3.

The lateral load on the specimen was applied to the cap beam at a height of 76 cm (2.5 ft) above the top of the column using two 980-kN (220-kip) actuators and following a displacement-controlled loading protocol. The loading rate for all tests was 0.125 cm/s (0.05 in/s). The loading protocol involved six full ramp cycles, applied in groups of two, with each group having an amplitude at 1/3, 2/3 and 3/3 of the target peak drift ratio. Three target peak drift ratio demands were considered (Table 2), namely, 1.3%, 2% and 4%, representing hazards levels with 5%, 2% and less than 1% in 50 years probability of exceedance, respectively, at the site the columns were designed for (downtown Los Angeles); these hazard levels have return periods of 975, 2475 and more than 4750 years. The drifts were based on median peak response during nonlinear dynamic simulation (following the modeling strategy by Salehi et al. (2017)) of the tested column when subjected to motions at each hazard level. For each hazard level, 20 ground motions were selected and scaled using Conditional Mean Spectrum method; for more details see Salehi (2019). Although these numbers may seem surprising, the drift ratio demands for a given ground motion intensity or hazard level are typically lower for HSR columns than for conventional columns due to high effective damping ratios (Salehi 2019; Valigura 2019).

Repair strategy

After Tests O1 through O3, repairs were designed following the expert panel recommendations, as shown in Figure 4. The damage consisted of some hairline cracking, mostly in the bottom segment, and spalling at the bottom rocking joint, as described in more detail below. The hairline cracks were deemed to be nonstructural and were not repaired because they were expected to have a negligible contribution to the post-repair behavior. The spalling at the bottom rocking joint was repaired in three steps. First, the spalled concrete was removed, and the spalled area was cleaned using a vacuum and air needle. Then, the cross-section was restored with cementitious grout, with grease applied to the foundation surface to prevent bonding between the restored cross-section and the foundation. Lastly, the section was externally confined with a CFRP jacket. Four of the PT tendons were re-tensioned such that the total PT force was 93% of the original force. The column was then re-tested in Test R1-R3 under the same loading protocol.

Between Test O3 and repair, 13 additional tests were conducted with low drift demands, and varying gravity loads for a related study (Salehi 2019). These tests are denoted here as *post original* (PO) 1-13. These tests were designed such that they did not increase the observed damage to the column, which was confirmed during testing. In addition, after Test R3, the column was pushed to higher drifts (up to 8% drift) through a series of 12 tests (PR 1-12).

RESULTS

Objectives of bridge repairs

The *Objectives of Bridge Repairs* questionnaire began by identifying those factors that are most important in selecting a bridge repair strategy after an earthquake, with experts choosing among repair time, repair costs and post-repair seismic performance. The panel concluded that the choice depends on the role and significance of the bridge. One panelist wrote, "*Typically, a DOT needs to bring structures on-line after an event based on the importance of the route, which will dictate the repair approach.*" Another said, "*If it is a crucial link, … the repair time is the most important one.*" The panel's focus on repair time stemmed from their concern that overall seismic losses and the economic impact of the bridge damage/repair would be driven by the losses due to bridge closure. In particular, one panelist pointed out that some states now prefer ABC construction and/or faster repairs, even if they cost more upfront, because they realize that time has an important effect on the economy. Another important factor identified was the damage level. In the case of minor damage (*e.g.,* minor cracking in RC elements), one panelist observed that "*the repairs are for long-term durability, they are not concerned with the safety of the structure. For these repairs, repair time is the most important … As you go further on the damage scale… it might be post-repair performance as a primary factor.*"

Questions about objectives of bridge repairs examined how much repairs should restore column capacity in terms of lateral strength, stiffness, and deformation, with responses plotted in Figure 5. The majority of the panelists stated that the deformation capacity of the repaired column should be either fully restored, or at least restored to the original design requirements. One panelist wrote that "*the performance [could still be] acceptable if it meets minimum [design] ductility requirements,*" while another panelist suggested that "*older bridges should be restored to levels exceeding original performance criteria, meeting current requirements/standards.*" The panel agreed that the repair strategy should also target full restoration of the lateral strength. In addition, the discussion emphasized that "*additional strength is not the objective since that would be detrimental to capacity protection*". The panel was comfortable with lower restoration of stiffness. They justified this choice because, "*Restoring elastic stiffness properties … is likely not feasible*

...,"; another panelist wrote, "*Lab tests show it [stiffness] is the hardest one to recover.*" However, the lower stiffness may change the post-repair dynamic behavior of the bridge, and one panelist suggested that "*the repair strategies [should be designed] based on the modified stiffness*".

Experimental column response

Figure 6 shows the hysteretic response of the original HSR column and Figure 7 shows the damage after each test. In Test O1, contrary to design expectations, rocking at the bottom was observed prior to sliding, due to breakaway friction. Following sliding initiation, the response of the column exhibited the expected behavior with most (92%) of the displacement demand being taken by sliding in the lower sliding joint. During Test O2, the lateral behavior followed the expected sequence, *i.e.,* sliding initiated in the lower sliding joint and was followed by rocking at the bottom rocking joint. Rocking becomes evident at drift ratios greater than 1.3% in both Tests O2 and O3, and results in a narrowing in the hysteresis loop. For the last set of cycles (targeting 4.0% drift ratio) in Test O3, sliding accounts for about one-third of the drift demand (all concentrated at the lower sliding joint), while rocking accounts for about two-thirds. The response in Test O3 also shows a clear stiffness deterioration during the last set of cycles. The breakaway friction of the upper of the two sliding joints was not overcome in any of the tests. This friction lowered the peak sliding amplitude from the designed 2% to the observed 1.25%. As shown in Figure 6, the repaired column performed similarly to the original column, as described in more detail below. There are, however, differences in the first few cycles due to breakaway friction.

The breakaway friction is larger than the static and kinetic coefficients of friction and is due to cohesion effects at the sliding interface. Goli (2019) found that the breakaway friction coefficient for PTFE depends on past sliding history and amplitudes, and whether the surface is dry (resulting in a lower breakaway friction coefficient, but higher kinetic/static friction coefficient), or lubricated with grease (resulting in a higher breakaway friction coefficient, but lower kinetic/static friction coefficient). As a result, the contribution of the breakaway friction differs among the tests, with lower impacts in later tests due to the prior recent sliding of the joint.

Residual drift ratios were obtained from the last cycle of hysteresis loops for each test, and calculated as a one-half of the distance between points where the curve's base shear reached zero in the positive and negative direction, as reported in Figure 8. The values of the residual drift ratios were further separated into residual drift ratios due to sliding (which can be recovered by hydraulic means), and residual drift ratios due to damage at the rocking joint (which can mostly not be recovered). Most of the residual drift was associated with joint sliding.

Damage states of HSR columns

Definition of damage states is crucial for performance-based assessments, as damage states tie the bridge seismic response to repair strategies. Following FEMA (2012), this study defines a damage state (DS) as a level of damage to a structural element that is associated with unique set of repairs. The geometry and elements of the HSR column does not allow for easy access in the field to all the parts of the column (especially tendons, joint interfaces, and the inside of the column). As a result, the expert panel stressed that, even in the lab, the damage should be assessed only to the parts of the column that are easily accessible. Our goal is to establish damage states that are linked to repair actions and to associate these damage states with both qualitative (to be used in field assessment) and quantitative (to be used in analytical models) descriptors.

Using the experimental test results (from this study and Sideris et al. 2014b, 2014c, 2015), and the panel discussion and questionnaires, five major types of damage and their consequences were identified, as described in Table 3 and illustrated in Figure 9. Out of these five damage states, the first three were observed in our tests of the Generation 2 HSR column. The study groups these five damage states into two categories, as listed in Table 3: segment damage and residual displacement damage. The damage states within each category are sequential as defined in FEMA (2012), but damage states from the two categories can occur simultaneously. Table 3 includes qualitative and quantitative descriptions for each DS. The qualitative description can be used to visually identify the DS on site or during an experiment. The quantitative description is then used for analytical work to tie the engineering demand parameters (EDP) that are recorded during the computational analysis to damage level and repairs. These values were obtained from past experimental data and research. For example, data from research on compressive concrete strain at spalling and crushing (Mander et al. 1988; Mattock et al. 1961; Saatcioglu and Razvi 1992) can be used here because these data are general to concrete behavior.

The panelists in our study emphasized the importance of residual drifts with statements like, "*Bridges performed adequately [in the Kaikoura, NZ earthquake] in terms of life safety; they didn't collapse. Residual deformation was a killer.*" The panel identified two types of residual drifts in HSR columns: residual drifts due to sliding, which can be restored by hydraulic jacks, and residual drifts due to damage to the rocking joint, which can mostly not be recovered. In the responses to the questionnaires, the panelists indicated that the residual drifts *after* restoration should not exceed a threshold value to avoid the need for column replacement; three of the panelists identified a threshold around 1%.

Based on the panel suggestions, the authors defined three residual drift damage states. In DS R0, the sliding residual displacements at each joint are smaller than the residual sliding displacement damage threshold and no repairs are needed. The authors recommend a minimum threshold of 15% of the design sliding amplitude of the joint and of 0.5% of the column's outside diameter dimension. These thresholds are devised so that if the residual sliding is under the limit, and thus not recovered, the joint can still function properly, and misalignment will not have long-term effects on the PT strands. DS R1 involves residual drifts due to sliding that exceed the residual sliding displacement threshold, while residual drifts due to rocking (*i.e.,* permanent residual drifts) are less than a residual rocking drift threshold value; these drifts are recoverable. In DS R2, the residual drifts due to rocking would exceed its threshold value, and the column would need to be replaced. The 1% proposed by the panelists is on a lower end of the residual drift thresholds for triggering column replacement suggested by the literature, which range from 1.0 to 1.5% (Kawashima and Unjoy 1997; Lee and Billington 2011).

Repair strategies for HSR columns

Table 4 summarizes the recommended repair actions. These recommendations were obtained from the expert panel during the *Repair Assessment of HSR Columns* questionnaire and subsequent follow-up discussion. During the panel discussion, several of the panelists mentioned the importance of only repairing the damage in the tests that would be accessible in the field. Thus, for example, recommended repairs would not address bearing damage to tendon ducts, because this damage cannot be identified without column disassembly.

The hairline cracking involved in DS 1 mostly affects durability and, slightly, initial stiffness (a drop of 3% in initial stiffness was measured between Tests O1 and O2). The panel recommendations on the repair follow the common practice for conventional RC components: injection of epoxy to close the cracks. For DS 2, the panel recommended to carefully remove the spalled concrete and to clean the surface with high pressure air needle "*to avoid damaging [the] bond between the remaining concrete and reinforcement*", to restore the cross-section with structural grout (with similar strength and elastic modulus to the original concrete), and to provide a CFRP or light-gage steel jacket to help confine the repair grout. The panel also recommended that the PT tendons should be re-tensioned. This re-tensioning can be possible in the field if the top anchorage of the tendons is detailed properly. This detail would require leaving about 45 cm (1.5 ft) of tendon above the anchorage, so the prestressing jack can be attached to the tendon. The anchorage and the end of the tendons would then need to be enclosed in a box below the roadway.

Repairs for DS 3 are similar to those for DS 2. However, due to damage to the core, the panel recommended unloading the column using hydraulic jacks prior to carrying out the repairs. Moreover, our analytical models (described in Salehi, 2019) showed that some of the PT tendons would yield prior to or coincident with DS 3. The panel suggested developing an anchorage detail in the foundation that would allow access to the tendons and facilitate replacing them if needed.

The jacket designs were based on existing guidelines where possible (e.g., Caltrans 2008b). The CFRP jacket provides additional external confinement to the rocking joint to restore moment capacity and confine the structural grout, preventing spalling. Vosooghi and Saiidi (2013) previously demonstrated the suitability of such an approach. More details are described in Valigura (2019). The jackets were designed to extend over the spalled region and deep cracks propagating from the spalled region, with an extra 10% to 25% jacket height to prevent formation of a weak plane between the original concrete and grout in the spalled area.

For the most severe segment damage state, *i.e.,* DS 4, the segment needs to be replaced because a large part of the contact surface of the rocking joint is damaged. To restore the cross-section, the segment would need to be removed (and hence the column would need to be disassembled), and, in the words of one of the panelists, "*At that point it is easier and safer to just replace it [the column].*" Therefore, this study recommends replacing the entire column.

No repair is required for DS R0. For higher residual sliding (DS R1), the panel felt it would be necessary to push the segments back using hydraulic jacks. The friction forces depend on the gravity load from the superstructure, but they may be low enough that the superstructure would not need to be jacked before sliding the column back. Under DS R2, residual drifts due to rocking are too large to realign the column, and the column needs to be replaced.

Effectiveness of repair methods

The overall behavior of the repaired column was very similar to the original column, as shown in Figure 6. The most observable difference was breakaway friction; the breakaway friction was high for Test O1 with no prior sliding, while it was much lower for Test R1, where the joint experienced sliding due to Tests O1-O3 and PO1-PO13, even though those tests were performed several weeks apart. Valigura (2019) provides photos of the minimal damage observed to the repaired column.

During the tests, the lateral (strength) capacity of the column was not reached. Instead, the study adopts a proxy of the lateral strength demand on the column to assess the influence of the repair on strength; this lateral strength demand was measured as the maximum base shear in the last cycle of each test. The repaired column experienced demands between 88% and 103% of the strength demand of the original column, as shown in Figure 10, and consistent with the objectives in Figure 5. The lower strength demands in Tests R1 and R2 are attributed to the lower forces in the PT tendons of the repaired column (as shown in Figure 11). Had the PT forces been fully restored, the reduction in lateral strength demand would be decreased and perhaps eliminated for the first two tests. For Test R3, the strength demand of the repaired column exceeded the strength demand of the original column in the last two cycles (at 4% drift ratio). During these two cycles, the original column experienced spalling at the bottom rocking joint, resulting in large PT losses; the CFRP jacket limited the spalling of the repaired column, reducing PT losses and increasing column base shear at the same displacement. The test data showed that, at these cycles, the repaired column PT forces were higher than the original column, as shown in Figure 11for Post O3 and Post R3 (showing PT forces at end of Tests O3 and R3, respectively), but still far below yielding strength (~66% of tendon yield stress).

The panel agreed that the change in strength demands is acceptable for use in high seismic areas, as shown in Figure 12. However, one panelist was concerned about the breakaway friction and resulting high lateral strength demand in first cycles of Test O1. The implication of breakaway friction is that, in the field, the sliding joints could resist higher lateral forces for small-to-medium seismic events before sliding initiates. This would lower displacements of the bridge, but could also initiate earlier potentially damaging rocking response.

The initial stiffness of the column during each test was measured as the slope of the elastic part of the unloading force-displacement path after the first displacement reversal (see Figure 6d); the stiffness at the end of Test O3 was also calculated using the same approach, but after the last load reversal. Figure 13 shows that, by the beginning of Tests O2 and O3, the stiffness has slightly decreased due to hairline cracks in the segments. After Test O3, however, the stiffness was calculated as 67% of the original stiffness. This large decrease is due to more serious damage at the rocking joint, which also resulted in large PT losses in the tendons (Figure 11). The results show that the initial stiffness of Test R1 was 88% of the original stiffness; in other words, 64% of the lost stiffness was restored. The restored stiffness (88%) is within a range that the panel agreed for stiffness repair objective (between 80%-100% of original value). The authors expect that this number would be larger if the PT tendons had been fully re-tensioned. The majority of the panel agreed that the stiffness restoration is acceptable for use in high seismic areas (see Figure 12). However, one panelist noted that the effects of stiffness change would need to be evaluated for particular bridge. Further, the stiffness restoration is dependent on the re-tensioning of the PT tendons, and, as another panelist wrote: "*I am not sure how easy it [re-tensioning] would be on the damaged bridge. This needs to be demonstrated in the field.*

The tests showed that neither the original nor the repaired column experienced serious structural damage nor were close to collapse even during Tests O3 and R3 (displacement demand from ground shaking with probability of exceedance of less than 1% in 50 years). During all of the tests, the column displayed stable hysteretic behavior with distinct regions of sliding and rocking. The panel agreed that the deformation capacity and its restoration are acceptable for high seismic areas (as shown in Figure 12). However, one panelist wrote that "*the impacts of the repair on ultimate displacement capacity cannot be determined from the test …*" and that "*understanding the strength degradation of … HSR columns is important.*"

To identify the collapse mechanism of the Generation 2 HSR columns and address concerns from the panel regarding damage that cannot be readily observed but could affect subsequent response, a related study tested the column with Tests PR1-12 (Salehi 2019), after which the column was disassembled. During the most demanding cycle with 8% peak drift ratio in Test PR12, one wire (out of seven) fractured in two different tendons at the wedge anchor. This type of failure has been observed in past studies for unbonded monostrands *(e.g.,* Sideris et al., 2014a). Note that this drift demand is very large, occurring during very rare ground motions. The tendon damage would suggest DS4, however, no significant damage was observed at the rocking joint, due to the external CFRP confinement in the repaired column. After disassembling the column, the concrete inside of the bottom segment near the sliding joint (located at the top of the segment) was observed to have experienced minor cracks propagating parallel to loading direction, which the authors expect to have resulted from tendons bearing against the ducts. Other damage included wearing of the sliding interface, and indentation of the ducts at the sliding interface from contact with PT tendons. However, the column experienced a total of 31 successful tests, and several "preparation" tests; thus, the wearing was the result of over 200 sliding cycles.

Application of HSR columns in field

The final objective of the questionnaires was to investigate the attitude of the panel towards the new system, and to identify improvements of HSR columns so that they could be implemented in the field. The panel agreed that the damage observed in all tests is satisfactory for typical bridges in high seismic areas (see Figure 14). During Test O3, which represented the displacement demand associated with less than 1% chance of occurring in 50 years at a high seismic site, the damage included only hairline cracking and cover concrete spalling at the rocking. As one of the panelists wrote, "*this is a rare earthquake and this modest damage is considered quite good.*" The only negative of the system were large residual drifts (Figure 8), which were observed in all of the tests. In the context of residual drifts, the same panelist wrote, "*A residual drift of 1.3% [observed in the tests] … could result in traffic safety as well as drainage issues.*" However, most of the residual drifts measured in the tests were from residual sliding and the panelists agreed that "*hydraulic jacks [can be used] to reposition the column segments.*" This is of particular importance, because the procedure to restore the residual sliding would not require temporary shoring (and hence rigorous design and permitting) and could thus be accomplished during a short closure of the bridge (*e.g.,* hours to one day). The panel also agreed that the HSR column experienced less damage than a typical, well-designed, cast-in-place column would for the same level of shaking, as the histogram in Figure 14 shows. One panelist observed that "*in the typical RC column, you would expect to start [to observe] some longitudinal bar buckling and severe spalling …*" and "*the HSR system appears to be effective in limiting damage.*"

To ensure confidence in the system and to direct future research in ABC systems for columns in high seismic areas, the panel was asked if the system is a viable replacement of current cast-in-place columns. The panel mostly agreed on this question, although 3 out of 8 panelists were neutral (Figure 14c). The neutral answers occurred because the research has, so far, focused only on seismic performance; however, as one panelist wrote: "*It will require more data on impact [i.e., loading from vehicle crashing into the column], durability, etc. ...,*" while another added: "*there are still questions related to costs, manufacturing, constructability, impact resistance, etc. ....*"

CONCLUSIONS

This chapter applies an expert solicitation method for systematically developing damage states and repair strategies for an innovative ABC bridge column system through integrated expert panel solicitation and large-scale testing. These damage states and repair strategies can be used in life-cycle seismic performance assessments to evaluate the benefits of the new HSR column system. To do so, the study conducted six tests on a half-scale HSR column under lateral cyclic loading up to displacement demands of 4% drift ratio, before and after it was repaired. Both the damage states and repair strategies were determined by the same panel of experts, who were presented with the results of the tests. The expert panel first assessed the column damage at varying displacement demands, and then proposed repair strategies. These strategies were applied to the tested column, and the repaired column performance was evaluated under the same loading.

For the HSR system, two sets of damage states were defined. The first set captures damage to the column segments, which happens primarily in the vicinity of rocking joints. These damage states include hairline cracking, minor spalling at the rocking joint, extensive spalling at the rocking joint with the onset of crushing of core concrete, and core crushing and tendon fracture. The second set describes the damage states due to residual drifts; these are predominantly due to residual sliding. In the tests, damage was limited to spalling at the rocking joint and very small unrecoverable drift ratios (less than 0.2%) during peak displacement demands of 4%.

The expert panel recommended repair strategies for each of the defined damage states. These repair strategies are intended to achieve the repair objectives identified by the panel of restoring 100% of the original strength of the column, 100% of the deformation capacity and 80 to 100% of the stiffness. The repair strategies included epoxy injections, grouting and CFRP or light-gage steel jackets. Furthermore, they recommended re-tensioning and replacement of tendons following yield or fracture. The panel deemed that residual sliding would require re-centering of the segments using hydraulic jacks. Tests of the HSR column repaired per expert panel recommendations showed that repairs achieved sufficient recovery of strength and stiffness.

Beyond the HSR system, the panel-defined objectives for bridge column repair in terms of strength, stiffness and deformation capacity restoration are applicable to other systems, and useful for designing repair strategies and evaluating those strategies. The expert panel also emphasized the importance of both repair time and repair costs in selection of repair strategies, supporting the development and improvement of performance-based procedures that quantify these for improved engineering decision making about repairs.

DATA AVAILABILITY

Some or all data, models, or code generated or used during the study will be made available in a repository online in accordance with funder data retention policies. In the meantime, they are available from the corresponding author.

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FIGURE CAPTION LIST

Figure 1. Metbods employed in this study, showing relationship between experiments and expert panel solicitation.

Figure 2. Damage to Generation 1 HSR column during quasi-static test up to 13.2% drift ratio demands, (43.5 cm or 17 in), as presented to the expert panel (green lines are spaced every 12.7 cm or 5 in). The loading is in east-west direction. Figures from Sideris (2012).

Figure 3. a) Geometry of test specimen, and b) test setup of Generation 2 of HSR column used in this study.

Figure 4. Repair procedure, showing: a) damage to the rocking joint after Test O3, b) restoration of the cross-section with grout, and c) application of the CFRP jacket.

Figure 5. Expert panel responses to the questions about objectives of seismic repair in terms of restoration of column's original deformation capacity, stiffness, and lateral strength. These responses are from the second round of the questionnaire; responses were closer together than in the first round.

Figure 6. Hysteresis response of the original and repaired columns for peak drift ratios of: a) 1.3% (Test O1 and R1), b) 2% (Test O2 and R2), c) 4% (Test O3 and R3), and d) illustration of the measurement of initial stiffness for Test O3 (explained below).

Figure 7. Observed damage at the bottom segment of the original column after each test.

Figure 8. Residual drift ratios calculated for each test.

Figure 9. Illustration of proposed damage states for HSR column segments showing: a) minor cracking at the bottom segment (DS 1), b) spalling at the rocking joint of the bottom segment (DS 2), and c) extensive damage to the rocking joint (DS 4). Photos in a) and b) are from tests of the Generation 2 HSR columns in this study; photo in c) is from Sideris (2012).

Figure 10. Lateral strength demand on repaired column as percentage of demand that was experienced in the corresponding original test.

Figure 11. Total PT force at the beginning of each test as a percentage of force prior to Test O1.

Figure 12. Histogram of panel responses to questions if the restored properties after repair are acceptable for application in high seismicity areas.

Figure 13. Stiffness at beginning of each test, as a percentage of initial stiffness at Test O1. Results for Post O3 are from the last cycle of Test O3.

Figure 14. Histogram of answers to questions if: a) the damage is satisfactory for given hazard levels, b) the observed damage is less than for cast-in-place columns, and c) the HSR columns are viable replacement of cast-in-place columns.

TABLES

Table 1. Point allocation to determine expert experience.

Experience	Points	Notes
PE in structural engineering	1-2	2 points for CA, OR, WA; 1 for others
Experience in seismic RC bridge design (years)	1-3 yrs – 1 4-6 yrs – 2 6-10 yrs – 3 >10 yrs – 4	For experience primarily in seismic design of RC structures other than bridges, subtract 1 point
Experience in RC bridge column repairs (number of projects)	1-3 projects – 1 4-6 projects – 2 6-10 projects – 3 >10 projects – 4	For experience primarily in repair of other RC bridge elements or building columns, subtract 1 point
Experience in post-seismic assessment of RC bridges	"in field" assessment – 3 "on paper" assessment – 2	For experience primarily in assessment of other RC structures, subtract 1 point
PhD in structural engineering	1-2	2 if the topic is closely related to damage/repair of RC elements; 1 otherwise
Committee membership	1 committee – 1 2-3 committees – 2 >3 committees – 3	Committees were counted if related to RC member damage or repair

Table 2. Loading protocol information for each test

Test	Hazard Level	Return period	Sa(T_{design})* (g)	Target Displacement (cm)	Target Drift Ratio (%)
O1, R1	5% in 50 years	975 years	1.2	5.0	1.3
O2, R2	2% in 50 years	2475 years	1.6	7.6	2.0
O3, R3	<1% in 50 years	>4750 years	>2.0	15.2	4.0

* Values are in prototype domain

Table 3. Damage states proposed for HSR columns

DS	Drift ratio (%) at which DS observed during the test of Generation 2 HSR columns	Qualitative description	Quantitative description			
			Engineering demand parameter	Median threshold	Dispersion[a]	Reference for quantitative description
Segment damage states						
DS 1	0.4	Open cracks	Cover concrete strain	0.002	0.40	Moehle (2015)
DS 2	2.7	Spalling at the rocking joint	Cover concrete compressive strain at the ends of the column	0.0038	0.25	Mattock et al. (1961)
DS 3	Not observed	Extensive spalling at the rocking joint, visible reinforcement and onset of crushing of core concrete	Core concrete strain at extreme fibers at the ends of the column	[b] Calculated	0.34	Mander et al. (1988); Saatcioglu and Razvi (1992)
DS 4	Not observed	Extensive spalling at the rocking joint, visible reinforcement and crushed concrete deep in the core; tendon fracture	Core concrete strain in the middle of the column wall thickness at the ends of the column	[b] Calculated	0.34	Mander et al. (1988); Saatcioglu and Razvi (1992)
Residual drift damage states						
DS R0	N/A	Small sliding residual drifts	Sliding residual displacement at each joint	Less than min. (0.5% of D[c], 15% of sliding amplitude)	0.40	This study
DS R1	0.4	Sliding residual drifts	Sliding residual displacement at each joint	Min. (0.5% of D[c], 15% of sliding amplitude)	0.40	This study
DS R2	Not observed	Large rocking residual drifts	Rocking drift ratio	0.01 – 0.015[d]		Lee and Billington (2011)

[a] Dispersion is in the form of logarithmic (ln) standard deviation
[b] Calculated based on the cross-section, concrete and reinforcement properties
[c] Outside diameter of the column
[d] Uniform distribution with lower and upper bound

Table 4. Repair strategies for HSR columns

DS	Qualitative description	Repair strategy
Segment damage states		
DS 1	Open cracks	• Epoxy injections
DS 2	Spalling at the rocking joint	• Clean the spalled concrete with air needle • Patch the spalled area with grout • Apply CFRP or light-gage steel jacket (1 MPa [150 psi]) • Re-tension the tendons
DS 3	Extensive spalling at the rocking joint, visible reinforcement and crushed concrete between reinforcement	• Unload the column • Clean the spalled concrete with air needle • Patch the spalled area with grout • Apply CFRP or light-gage steel jacket (2 MPa [300 psi]) • Replace the tendons
DS 4	Extensive spalling at the rocking joint, visible reinforcement and crushed concrete in the core	• Replace the column
Residual drift damage states		
DS R1	Small sliding residual drifts	• No repair
DS R1	Sliding residual drifts	• Unload the column (if necessary) • Re-center the sliding joint using hydraulic means
DS R2	Large rocking residual drifts	• Replace the column