SEISMIC RESPONSE OF RECENTERING LOW-DAMAGE PRECAST CONCRETE DUAL-SHELL STEEL COLUMNS

PEER Transportation Systems Research Program

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Introduction

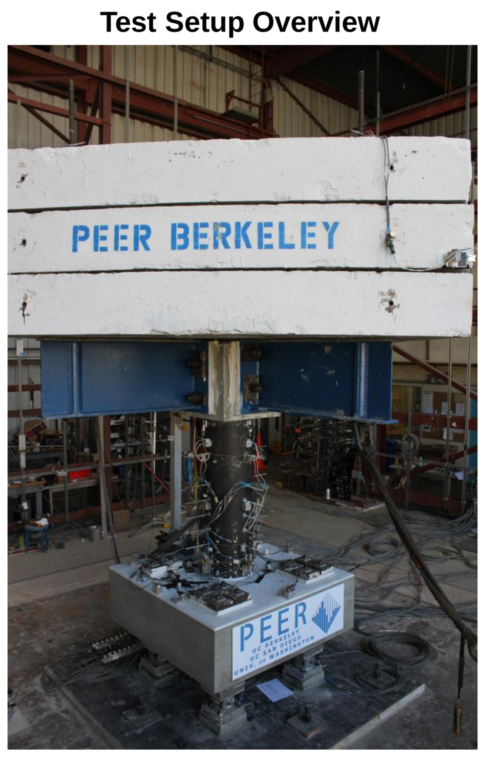
An innovative bridge pier technology for application in seismic regions is being developed at UC San Diego Structural Engineering Department[4, 13], in collaboration with the PEER-TSRP bridge group, and was tested on PEER Center's shake table at UC Berkeley - Richmond Field Station. This element combines a precast concrete hollow-core column with on-site post-tensioning and added energy dissipation. The column in this project is an extension of the steel jacket concept used by Caltrans for retrofitting deficient columns, of Caltrans-funded research work on Cast-In-Steel-Shell piles[3] , of precast post-tensioned rocking systems that stemmed from the PCI-funded PRESSS research program[9], and of research work on self-centering precast walls and columns performed in the US[5, 6, 11, 13] and New Zealand[7, 8, 10, 14].

The column consists of two steel shells running for the full-height of the column, with concrete sandwiched in between[4, 13]. The outer shell acts as permanent formwork and substitutes longitudinal and transverse reinforcement. The inner shell also behaves as permanent formwork, and prevents concrete implosion under large compressive strains. Large plastic rotations can be attained with minimal structural damage: column-cap and column-foundation joints are allowed to open in tension under severe earthquake excitation, and to close subsequently upon load reversal. Self-centering behavior is ensured by post-tensioned longitudinal bars, designed to respond elastically. A special connection between column, bent-cap, foundation, and PT bars[4] allows for eventual bar replacement, and protects bars from yielding. Energy dissipation is provided by internal[4, 10, 13] stainless steel dowels which are allowed to yield along a debonded segment. High-performance mortar and headed reinforcement are used to protect from mortar-joint premature crushing[12].

Constructability is enhanced by the use of a precast element of reduced weight (hollow-core section) without a reinforcing cage, allowing for reduction of on-site construction time, traffic impact, environmental disruption, and life-cycle costs. In fact, this project responds to the Accelerated Bridge Construction (ABC)[1] initiative promoted the Federal Highway Administration (FHWA). In parallel, this solution represents a resilient, economically viable technology, where earthquake-induced damage to the main structural elements is reduced, and self-centering properties allow the structural system to return to its original position after a strong-intensity jolt.

Specimen Features

Test Setup Overview

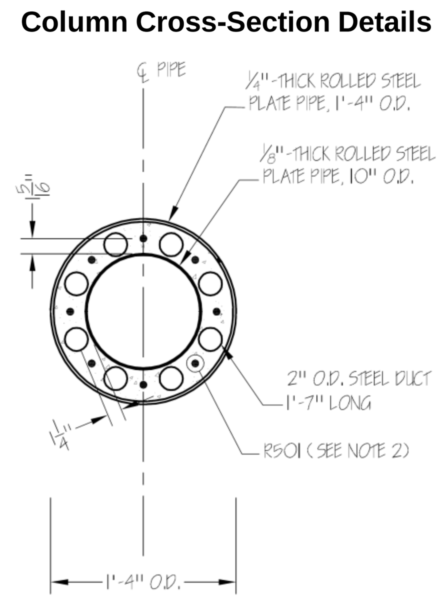


Column outer diameter: 16 in.
Column length: 52 in.
Shear span: 96 in.
Outer steel shell: ¼ in. thick, A572, Grade 50
Inner steel shell: ⅛ in. thick, A572, Grade 50
Concrete: 8 in. slump, 10.2 ksi on day of test (9 ksi specified)
Stainless steel dowels: 8 #3, 316LN, Grade 75, debonded for 6 in.
Headed rebars: 8 #5, A706, Grade 60, with 2 in. diameter head
Mortar bed: ¾ in. thick, 17.5 ksi, with polypropylene fibers
Post-tensioning: one 1-¾ in. DSI threadbar, A722, Grade 150
Bearing: 1-⅞ in. thick, 90-durometer-A adiprene
Inertial mass: 53 kips
Initial post-tensioning force: 98 kips

Concentric Steel Shells

Permanent formwork, replace reinforcing cage, prevent concrete implosion.

Column Cross-Section Details



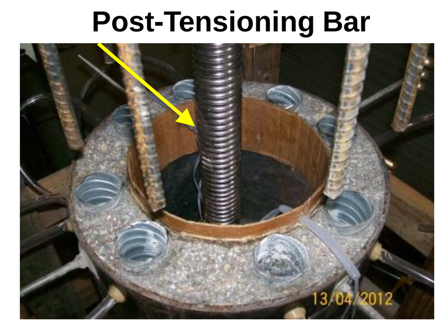
Precast Column

Designed for accelerated bridge construction.

Stainless Steel Dowel Bars

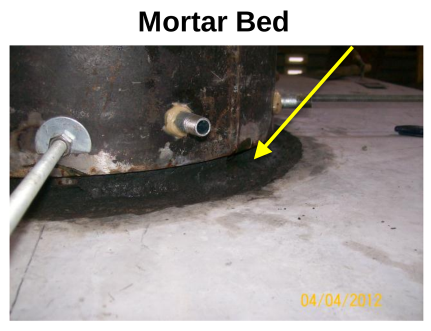
Dissipate energy by yielding along the debonded segment.

Post-Tensioning Bar



Provides self-centering behavior together with gravity. Threadbar can be easily replaced.

Mortar Bed



Accommodates construction tolerances, undergoes large compressive strains upon gap opening: high performance material needed.

Headed Rebars

Reduce the potential for crushing of the mortar bed.

Bearing

Placed in series with post-tensioning bar to prevent premature yielding and loss of post-tensioning.

Test Protocol

The specimen was subjected to 3-dimensional earthquake excitation. Eight historical earthquake records were selected and scaled for this experiment, based on the lateral displacement demand imposed on a reference reinforced-concrete column designed according to Caltrans' Seismic Design Criteria (SDC)[2]. The chosen ground motions are listed in the table below.

EQ1 was intended to impose a demand within the elastic range of response of the reference column.

EQ2 and EQ3 targeted displacement demands beyond the elastic limit, but below the design-base level.

EQ4 and EQ5 were selected to represent design-base level events, which are associated to a return period of 975 years according to Caltrans' SDC.

EQ6, EQ7, and EQ8 targeted near-collapse conditions for the reference column.

EQ9 was added at the end of the protocol, as a repeat of the design-base level event EQ5.

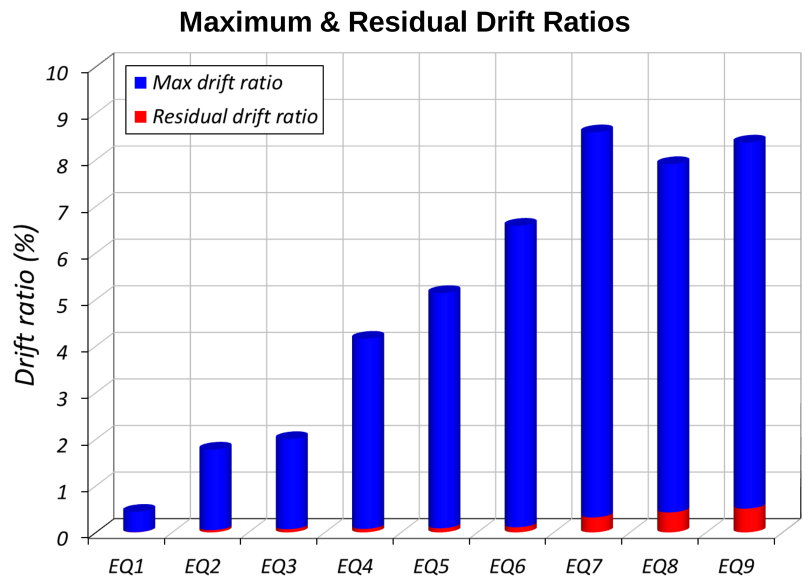
White-noise excitations with root-mean-square acceleration equal to 0.3% and 0.8% of gravity were run between the listed records, in order to assess the evolution of the dynamic properties of the system. Free-vibration (pull-and-release) tests were also performed at the beginning of the sequence, to identify the initial elastic properties.

Test	Target Drift Ratio on Ref. Column (%)	Predicted Max Drift Ratio on Specimen (%)	Event	Date	Unscaled Magnitude	Station	Scale Factor
EQ1	0.4	0.5	Coalinga	1983/05/09	5.0	Harris Ranch – Hdqtrs (temp)	2.50
EQ2	2.2	1.7	Imperial Valley	1979/10/15	6.5	EC Meloland Overpass FF	0.80
EQ3	2.1	2.5	Morgan Hill	1984/04/24	6.2	Coyote Lake Dam (SW abut)	0.70
EQ4	4.0	3.9	Northridge	1994/01/17	6.7	Rinaldi Receiving Station	0.56
EQ5	5.5	4.9	Northridge	1994/01/17	6.7	Sylmar – Olive View Med FF	-0.80
EQ6	7.6	8.2	Northridge	1994/01/17	6.7	Rinaldi Receiving Station	0.90
EQ7	8.2	8.3	Kobe	1995/01/16	6.9	Takatori	0.77
EQ8	7.3	8.8	Kobe	1995/01/16	6.9	Takatori	-0.90
EQ9	N.A.	N.A.	Northridge	1994/01/17	6.7	Sylmar – Olive View Med FF	-0.80

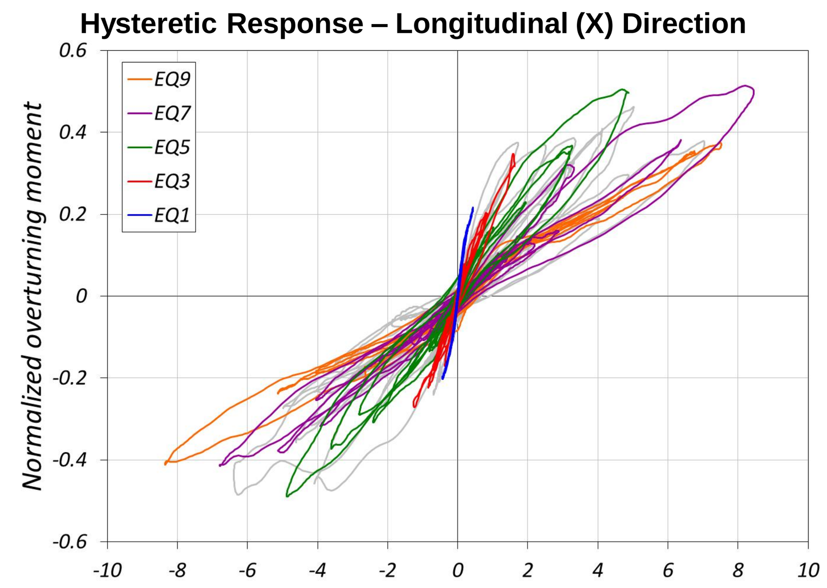
Specimen Response

1) The specimen displayed an excellent self-centering behavior up to the end of the test protocol, when a residual drift ratio of 0.5% was recorded after reaching 8.6% drift ratio.
2) After EQ9 the post-tensioning force had dropped from its initial 98 kips to 70 kips, with a 29% loss. This may have been caused by permanent compressive deformation (not visible) of concrete and mortar bed, or partial mortar crushing (visible by inspection), as the PT-bar strain never exceeded its yield value. Increasing PT losses were recorded after each test.
3) Hysteretic loops show a progressive stiffness degradation, which may be due to permanent compressive deformation (not visible) of concrete and mortar bed, or partial mortar crushing (visible by inspection). Overturning moments are normalized by the product of superstructure weight times shear span.
4) One energy-dissipating dowel bar on the north-east side fractured during EQ6. Three more dowels fractured during EQ7, two on the south-west side and one on the north-west side. Two additional dissipating bars fractured during EQ8, one on the south-east side and one finally on the north-west side. Fractures are visible on the hysteretic loops in the longitudinal direction, where EQ7 and EQ9 cycles show a 20% strength degradation when compared to EQ5.

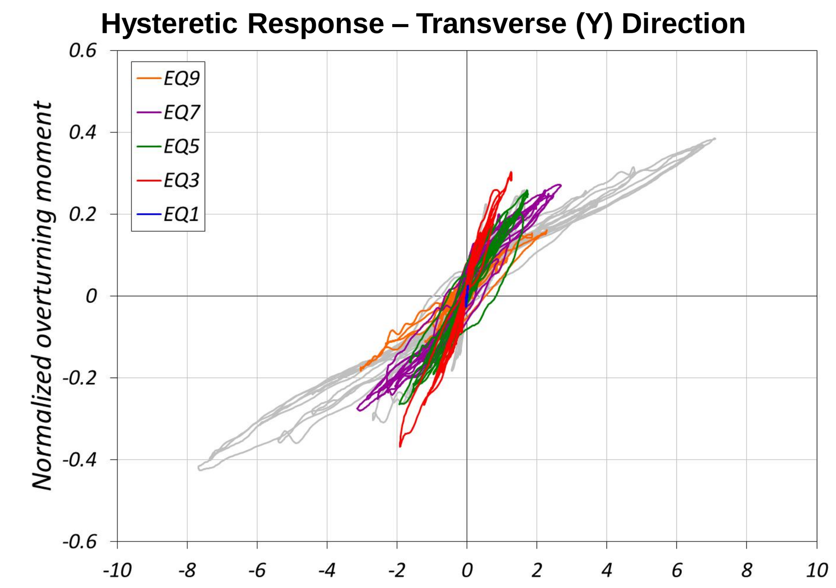
Maximum & Residual Drift Ratios



Hysteretic Response – Longitudinal (X) Direction



Hysteretic Response – Transverse (Y) Direction



Summary

1) Excellent self-centering behavior was maintained even after reaching 8.6% drift ratio under EQ7 and EQ9, with a residual drift ratio of 0.5%.
2) Large drift ratios (6.6%) were achieved when energy-dissipating dowel bars started fracturing.
3) The post-tensioning bar did not yield even after reaching drift ratios of 8.6%. Introducing a deformable adiprene bearing in series with the bar, at its anchor, proved to be effective.
4) The combination of high-performance mortar and headed reinforcement for compression transfer retarded and limited mortar-bed crushing. This, in turn, reduced PT losses due to mortar crushing and preserved self-centering capability up to the end of the test protocol.
5) Shear sliding at the column-footing interface did not represent an issue.
6) The use of a post-tensioning bar allowed for easy bar de-tensioning and removal.
7) The numerical prediction of maximum drift ratio demands was accurate under both low- and strong-intensity excitation. Errors smaller than 20% were achieved, except for EQ6 where an error of 25% was found.

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