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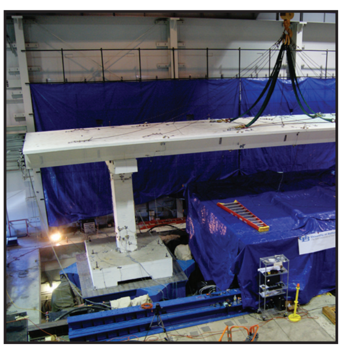
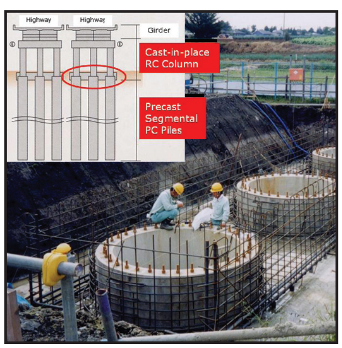
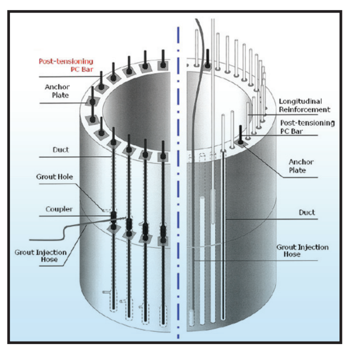
Proceedings of the Special International Workshop on Seismic Connection Details for Segmental Bridge Construction

Edited by
W. Phillip Yen and George C. Lee







Technical Report MCEER-09-0012

December 21, 2009

This workshop was held in Seattle, Washington on July 22-24, 2009. It was sponsored by the Federal Highway Administration under contract number DTFH61-07-C-00020.

NOTICE

Proceedings of the Special International Workshop on Seismic Connection Details for Segmental Bridge Construction

Held in
Seattle, Washington
July 22-24, 2009

Editors and Workshop Co-Chairmen:

W. Phillip Yen[1] and George C. Lee[2]

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Participants of the Special International Workshop on Seismic Connection Details for Segmental Bridge Construction

TABLE OF CONTENTS

TABLE OF CONTENTS (CONT'D)

Section 1

Introduction

1.1 Overview

Using precast/prestressed concrete components to accelerate the construction of highway bridges has been a major professional thrust of the bridge engineering community. Rapid construction leads to many positive effects on costs, quality and safety. Thus, accelerated bridge construction (ABC) has become a popular option for both new installations and in bridge rehabilitation projects. ABC covers a broad spectrum of activities, including the development of high performance materials; fabrication processes; design, construction and erection methods; and techniques and equipment for transportation, installation, removal and reuse of existing bridges. Additionally, ABC can reduce negative environmental impacts and detour times due to new bridge construction and/or retrofit projects.

The behavior of segmental bridges in regions of moderate to high seismicity has not been carefully studied, and therefore, there are no design provisions for seismic ABC (SABC). In recent years, SABC has been a new emphasis of AASHTO. For this reason, the Federal Highway Administration (FHWA) has sponsored a research project at the Multidisciplinary Center for Earthquake Engineering Research (MCEER) since 2006 on SABC, in parallel with NCHRP 12-74, "Development of Precast Bent Cap Systems for Seismic Regions." The MCEER project emphasizes two specific aspects of SABC: (1) seismic performance and design of segmental piers, and (2) seismic performance of a segmental bridge as a system.

Earthquake resistant design typically follows one of two basic philosophies: (1) provide adequate seismic strength and ductility, and (2) reduce the seismic demand by using isolation systems. The MCEER project gives more emphasis to innovative systems and at the same time stays within these basic design philosophies to ensure that new research results can be more readily accepted and used by engineers in practice. One example is the well accepted approach for segmental columns to perform equal to or better than cast-in-place (CIP) columns under earthquake ground motions (Culmo, 2009; PCI, 2008). On the other hand, the MCEER project also emphasizes more innovative development by permitting gap opening/closing between bridge segments, which requires the use of seismic response modification systems, and high performance steel rebars as well as high performance concrete to achieve a higher level of seismic performance with reduced cost.

The MCEER project on SABC has several subtasks including substructure, superstructure, connections and system performance. The first major effort is given to substructures, which is an international cooperative project between MCEER and

NCREE (National Center for Research on Earthquake Engineering in Taiwan). Funding is provided by both FHWA (to MCEER) and by the National Science Council of Taiwan (to NCREE) working on the same SABC project on segmental piers (see papers by Chang et al. and Ou et al. in Section 2 of this report).

The second major research effort is on large scale laboratory experiments to observe the seismic performance (connection behavior and system behavior) of a segmental bridge of this type. The MCEER project is using the pair of synchronized shaking tables at the University at Buffalo (UB) to carry out the experiments.

The specific objectives of this workshop are to gather information on current research and/or practice on the connection details of segmental bridges for SABC from Japan, Taiwan and the State of Washington in the U.S., so that they can be documented and integrated with the MCEER studies to formulate a SABC monograph on precast segmental concrete bridges. (Note: SABC practice in some U.S. states such as California and Utah are familiar to the workshop organizers.) The workshop is organized by a committee consisting of the following individuals:

Co-chairs: George C. Lee and Phillip W. H. Yen
Members: Amjad Aref, Stuart Chen, Il-Sang Ahn and Jane Stoyle Welch

The workshop participants are listed in Appendix A and the workshop agenda is provided in Appendix B.

1.2 MCEER SABC Project

The SABC research project at MCEER is a five-year program of study funded by FHWA. This is a SAFETEA-LU project which began in 2006. The major program thrust is to develop design recommendations for segmental bridges of short-to-medium spans in strong seismic regions. Specific emphases are given to the seismic behavior and design of segmental piers and the integrity of system performance of the segmental bridges considered in this research project. A number of coordinated tasks are being carried out, including:

- Segmental substructures (a US-Taiwan joint project)
- Segmental superstructures
- System performance of segmental bridges
- IT systems for Accelerated Bridge Construction in seismic regions
- Scaling issues of down-scaled segmental column testing
- Seismic isolation system/devices
- Structural fuse and energy dissipation systems/devices
- Other innovative technologies

The study of segmental substructures is an international cooperative project between MCEER and NCREE (the National Center for Research on Earthquake Engineering) of

Taiwan. This cooperative project addresses the seismic performance and design of segmental columns by considering the column ductility using regular rebars, high performance rebars, high performance concrete, and the use of isolation devices.

The MCEER project deliverables are described in the figure below.

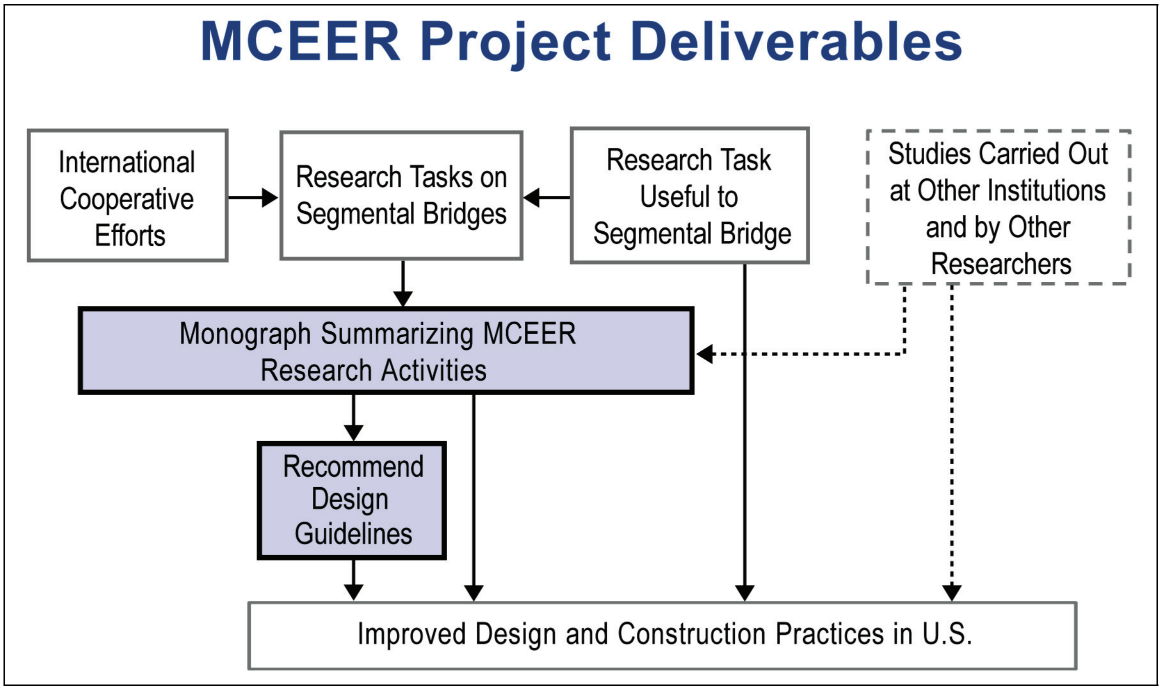

Figure 1: MCEER Project Deliverables

There are many issues that must be considered in developing a set of design recommendations. Therefore, it was decided that this project will only address standard precast girder type bridges with short-to-medium spans (less than 500 feet). Efforts will be devoted to the seismic behavior of segmental column connections and their design details. Furthermore, the integrity of the system and behavior of the segmental bridges will be studied using the shaking tables at UB. These studies will be summarized with additional relevant research results carried out by others, in a technical monograph. Design recommendations for the type of segmental bridges chosen for study will be developed as the final deliverables for FHWA.

1.3 References

Culmo, M. P. *Connection Details for Prefabricated Bridge Elements and Systems* (FHWA-IF-09-010). FHWA 2009.
PCI. *PCI Connections Manual for Precast and Prestressed Concrete Construction* (1st Ed.). Precast/Prestressed Concrete Institute. 2008.

Section 2

Short Papers by Workshop Participants

State-of-Practice and Research of Precast Segmental Concrete Bridge Columns in Japan

Jun-ichi Hoshikuma

INTRODUCTION

With a background of the generalization of the performance-based design concept into practices, the applications of new materials, new designs, and new structures have initiated to be actively employed with necessary performance verifications. The precast segmental bridge columns are one of such new applications, and effectively use the combination of high strength materials including steels and concrete. The precast segments are fabricated at factory, so that the precast segmental bridge columns can be easily achieved to have better-quality. Therefore, the precast segmental bridge columns are expected to improve the constructionability at sites and shorten the construction period. This paper briefly introduces the state-of-practice of the segmental concrete bridge columns in Japan. Furthermore, recent research activities for the precast segmental concrete bridge columns are summarized.

STRUCTURAL CONCEPT OF PRECAST SEGMENTAL BRIDGE COLUMNS

Figure 1 shows the outline of the precast segmental bridge columns which have been designed and constructed in the past in Japan. The precast segments are produced at factory and transported to the construction site. These segments are piled up at the site and connected each other through the steel bars, to be a column. It would be an important advantage to shorten the construction period at the site because of no need of formwork, placement and curing of concrete. Therefore, the precast segmental bridge system would be expected to be applied for overpass crossings in the urban areas in order to decrease the traffic jamming and then to minimize the effect on existing traffic.

Structural details of the conventional precast segmental bridge columns are shown in **Figure 2**. There are two types of the precast segmental columns. The precast PC columns are built with the segments connected through the high strength steel bars columns, as illustrated on the left side section in **Figure 2**. Each segment is post-tensioned by all high strength steel bars, to integrate with column structure. After post-tensioning, the following segment is piled up on the lower segment and the high strength steel bars are installed into the section through the ducts. These bars are coupled with the lower high strength steel bars and the grout is injected to the duct to be bonded. Details of the segment connection are shown in **Figure 3**. These processes are repeated up to the column height.

On the other hand, the precast RC columns are built with the segments connected basically through the nominal strength steel bar, as illustrated on the right side section in **Figure 2**. A few high-strength steel bars are installed and minimum post-tension required for setup of the segment is applied to these bars. The steel bars are coupled and grouted with the same procedure as the precast PC column.

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Revised July 31, 2009

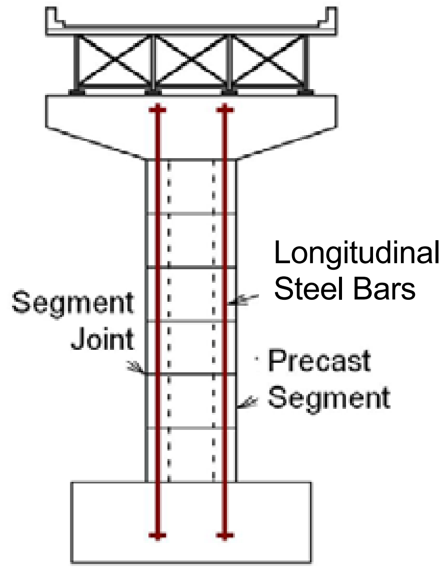

Figure 1 Illustration of Precast Segmental Concrete Columns

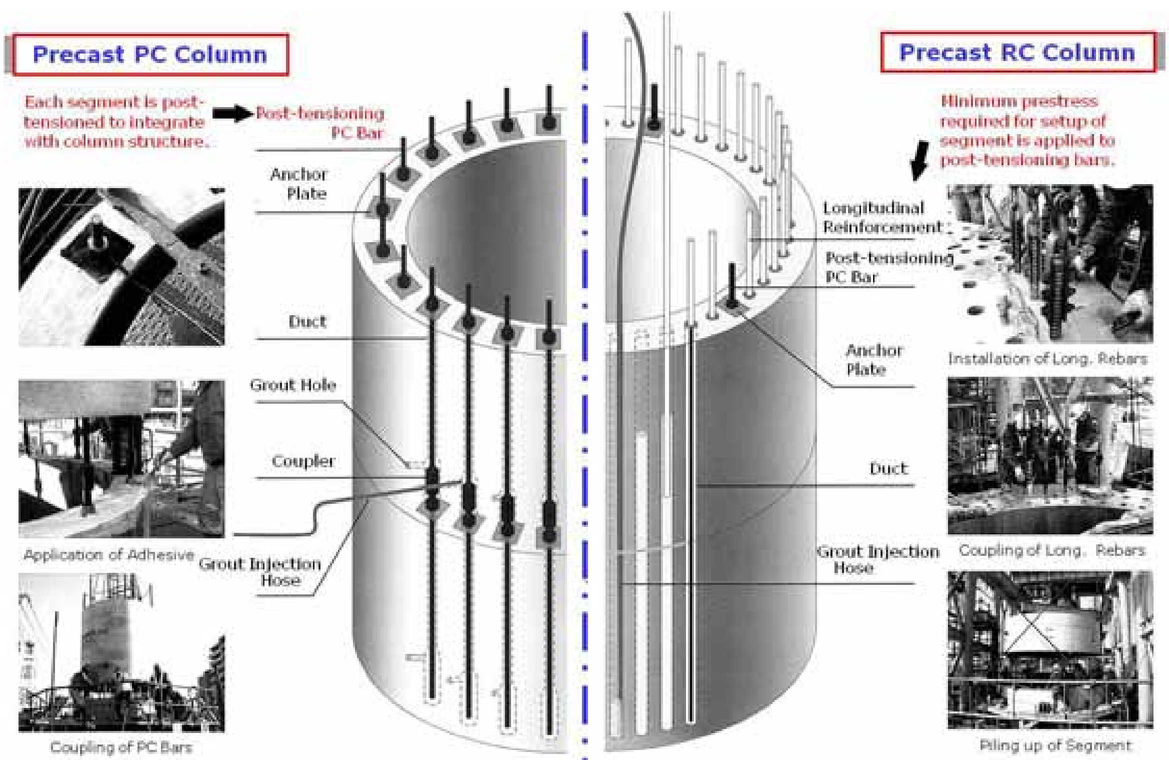

Figure 2 Concepts of Connection Details in Conventional Precast Segmental Concrete Columns

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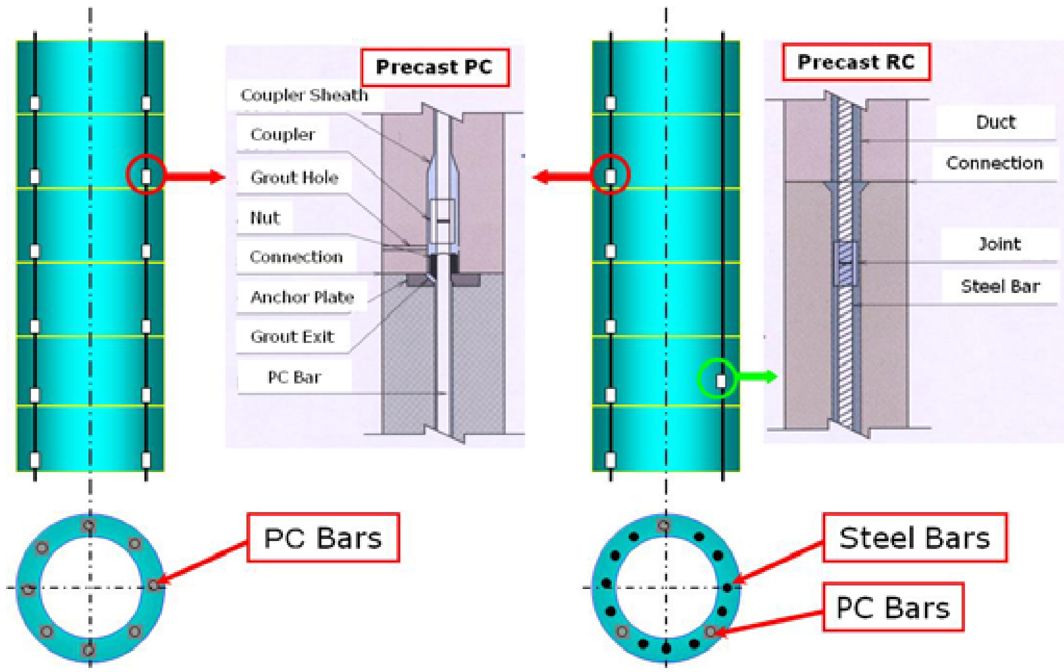

Figure 3 Connection Details of Steel Bars in Segment

Photo 1 Connection of Precast Segmental PC Piles with Cast-in-place Footing

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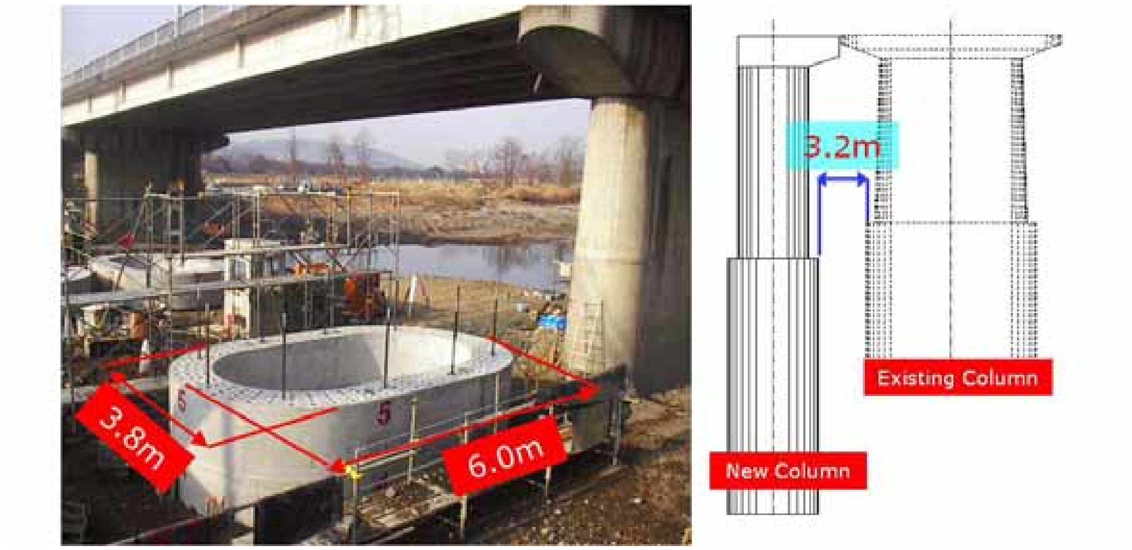

Photo 2 Precast RC Oval Column Constructed Very Close to Existing Column

Construction samples of the precast segmental concrete columns or piles are shown in **Photos 1** and **2**. **Photo 1** shows connection of the precast segmental PC piles with the cast-in-place footing. **Photos 2** shows the precast RC oval column constructed very close to existing bridge column.

RECENT RESEARCH ACTIVITIES FOR PRECAST SEGMENTAL BRIDGE COLUMNS IN PWRI

PWRI has conducted the joint research program on the new precast segmental concrete columns with 3 private companies including Kajima Co., Sumitomo Mitsui Construction Co., Ltd., and P.S. Mitsubishi Construction Co., Ltd. Three types of the precast segmental concrete column details were proposed. Research issues were to obtain the data on the failure mechanism, the strength and ductility performance, and the dynamic behavior of proposed precast segmental bridge columns, and to develop the design method including the limit states to achieve necessary seismic performance, detailed design methods for segments, joints, PC cables, bending–shear resistance evaluation, and construction methods. In the joint research program, a series of cyclic loading tests, shaking table tests, and analytical studies were made to develop the seismic design guidelines for the proposed precast segmental concrete columns.

Structural concepts of the proposed precast segmental concrete columns are shown in **Figures 4** to **6**. The structural details and properties are described in the followings.

Precast Segmental PC Columns 1 (Proposed by Kajima Co., Ltd.)

Figure 4 shows the outline of the precast PC column proposed by Kajima Co. The segments are piled up at the site, and each segment has the outer and inner steel pipes. Outer steel piles are embedded in the segment when it is produced at factory, and inner steel pipe are installed at the site between the segments. The inner steel pipe is to resist against the shear force acted the joints of segments as a shear key. After the piling up of all segments for columns, the vertical tensioning force is applied for segments by PC cables through the inner pipes, in which the PC bars work as longitudinal steel.

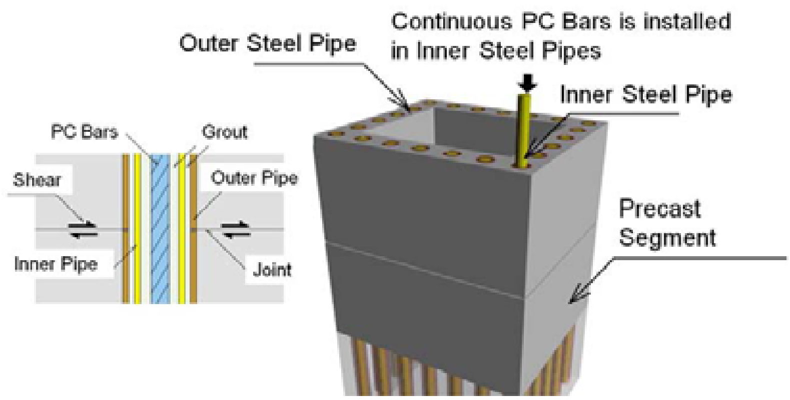

Figure 4 Precast Segmental Columns Proposed by Kajima Co., Ltd.

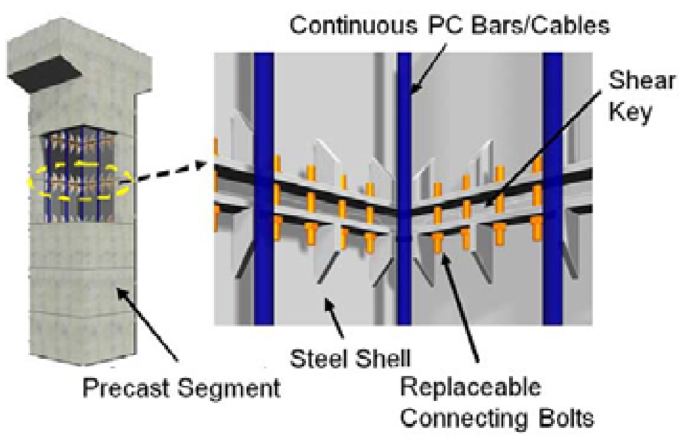

Figure 5 Precast Segmental Columns Proposed by Sumitomo Mitsui Co., Ltd

Precast Segmental PC Columns 2 (Proposed by Sumitomo-Mitsui Co.)

Sumitomo Mitsui Construction Co., Ltd proposed the precast segmental PC columns as shown in **Figure 5**. The segments are piled up at the site. Each segment is made of combination of inside steel shell and outside concrete. Inside steel shells of the segments are connected by steel bolts. After the piling up of all segments for column, vertical tensioning is applied for segments by external PC bars/cables. At the joints between segments, shear keys are provided at the edge of steel shell and concrete mortar is placed between the segments outside concrete. Therefore, vertical axial force by dead load and live load is carried by inside steel shell but the earthquake force is carried by steel shells, bolts, and outside concrete. Shear force acted at the joints is carried by the shear keys of steel shell.

Steel shells and bolts, and PC cables works as longitudinal steel. The bolts are designed to be firstly yielded when the deformation is exceeding the elastic limit and then the steel shells are not expected to be damaged. It is an important concept for this column that the yielded bolts can be replaceable after the earthquake and then the columns can be easily recovered to the original performance.

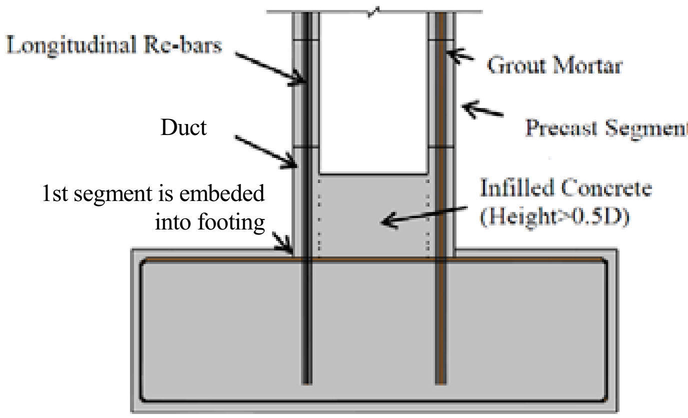

Figure 6 Precast Segmental Columns Proposed by P.S. Mitsubishi Co., Ltd

Precast Segmental RC Columns (Proposed by P.S. Mitsubishi Construction Co., Ltd.)

Figure 6 illustrates the precast RC column proposed by P.S. Mitsubishi Construction Co., Ltd. The concrete segments are piled up at the site with a few temporally PC bars. Those bars are provided not for tensioning as longitudinal steel but for just construction to assure the quality of the joint connection between segments by resin. After piling up of all segments, mild longitudinal steel bars are inserted into the ducts, which are pre-grouted with the mortar from the top to the bottom of the column. The columns are made of segments but the design concept is the same as nominal reinforced concrete column. Since the longitudinal steel bars are placed inside the sheathe of segments, so the confinement effect to prevent the buckling of longitudinal bars is much higher than the nominal reinforced concrete columns which is confined by the cover concrete and lateral steel bars.

Seismic Testing for Proposed Precast Segmental Concrete Columns

In order to obtain the data on the failure mechanism, the strength and ductility performance, and the dynamic behavior of proposed precast segmental concrete columns, as well as in order to develop the design method, a series of dynamic loading tests and shaking table tests were performed. Test results are reported in the company papers (written by Dr. Sakai and Mr. Taira)

CONCLUSIONS

This paper introduces the state-of-practice of the conventional segmental concrete bridge columns in Japan. Also, recent research works for the development of the new precast segmental concrete columns were introduced in this paper. Design Guidelines for the seismic effect of the precast segmental concrete columns is scheduled to be published in 2010. The seismic performance and the seismic limit states of the precast segmental columns will be described in the guidelines. Furthermore, connection design details of those columns will be specified.

ACKNOWLEDGMENT

Some illustrations and photos of the structural details of the precast segmental concrete bridge columns in this paper were provided by PC-WELL Method Association, Kajima Co., Sumitomo Mitsui Construction Co., Ltd., and P.S. Mitsubishi Construction Co., Ltd. Author would like to appreciate their contribution for the paper.

Cyclic Behavior of Precast Segmental Concrete Bridge Columns with High Performance or Conventional Steel Reinforcing Bars as Energy Dissipation Bars

Yu-Chen Ou[1], Mu-Sen Tsai[2], Kuo-Chun Chang[3], and George C. Lee[4]

ABSTRACT

The cyclic behavior of precast segmental concrete bridge columns with high performance (HP) steel reinforcing bars and that with conventional steel reinforcing bars as energy dissipation (ED) bars were investigated. The HP steel reinforcing bars are characterized by higher strength, greater ductility and superior corrosion resistance compared to the conventional steel reinforcing bars. Three large-scale columns were tested. One was designed with the HP ED bars and two with the conventional ED bars. The HP ED bars were fully bonded to the concrete. The conventional ED bars were fully bonded to the concrete for one column while unbonded for a length to delay fracture of the bars and to increase energy dissipation for the other column. Test results showed that the column with the HP ED bars had greater drift capacity, higher lateral strength and larger energy dissipation than that with fully-bonded conventional ED bars. The column with unbonded conventional ED bars achieved the same drift capacity and similar energy dissipation capacity as that with the HP ED bars. All the three columns showed good self-centering capability with residual drifts not greater than 0.4% drift.

INTRODUCTION

In a typical design of a precast segmental concrete bridge column, prestressing tendons throughout the column are stressed to apply compression forces across precast connections, which with the compression forces from gravity loads provide required flexural and shear strengths at the connections. Mild steel reinforcement is normally not continuous across the connections and thus contributes little to the strengths. Under lateral loads, the column shows a behavior same as a monolithic column prior to opening of the connections. Once the connections open, the column exhibits a nonlinear behavior with little energy dissipation but a small residual drift upon unloading (Wang et al 2008). To increase energy dissipation, Wang et al. (2008) and Ou et al. (2009) propose to add mild steel reinforcing bars across the connections. The bars are referred to as energy dissipation (ED) bars to emphasize their function and to distinguish them from other mild steel bars that are not continuous across the connections. It has been shown that the use of the ED bars can significantly increase energy dissipation. However, when the columns are subjected to a large lateral displacement,

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significant connection opening will occur and likely cause premature fracture of the ED bars. Unbonding the bars for a length can decrease the strains and delay fracture. Alternatively, fracture can be delayed by using more ductile steel capable of absorbing greater energy before fracture. In this research, high performance (HP) steel reinforcing bars commercially known as Enduramet 32, designated as S24100 in ASTM A955, are investigated for use as ED bars. They have superior ductility capacity than conventional carbon steel reinforcing bars. Additionally, it has excellent corrosion resistance. This can address potential corrosion problems resulting from opening of precast connections.

The energy dissipation and residual drift of a segmental column with ED bars increase as the strength contribution of the ED bars to the lateral strength of the column increases (Ou et al. 2009). In Japan, it has been found difficult to re-center the superstructure of bridge columns with a residual drift exceeding 1/60 or with a residual displacement more than 15 cm, whichever is smaller (Zatar and Mutsuyoshi 2002). As a result, the 1996 Japanese seismic design specifications of highway bridges requires that the residual drift developed at a bridge column after an earthquake not be greater than 1% (Kawashima 2000). Test results show that the segmental column can maintain a residual drift not greater than 1% provided the strength contribution of the ED bars is below approximately 35% of the lateral strength of the column (Ou et al. 2009).

Three large-scale precast segmental concrete columns were tested in this research under lateral cyclic loading to demonstrate the cyclic performance of segmental columns with HP reinforcing bars as ED bars in comparison to conventional reinforcing bars.

SPECIMEN DESIGN AND TEST SETUP

The design and test setup of the segmental columns tested are shown in Figs. 1(a) and 1(b), respectively. Table 1 lists major design parameters. Each column consisted of four precast segments with a hollow cross section and one precast cap beam. Prestressing tendons were anchored at the underside of the foundation at one end and anchored on the top of the cap beam at the other end. The tendons were unbonded to the concrete and passed through ducts in the foundation, through hollow core of the segments and through ducts in the cap beam. Note that “unbonded” does not mean the tendons are “ungrouted” against corrosion. The tendons can be placed in smooth polyethylene pipes that are not bonded to concrete and fully grouted for corrosion protection. The tendons can also be epoxy coated for enhanced corrosion resistance. The total prestressing force was 1042 kN, which was carried by four tendons, each consisting of two D15 seven-wire strands. The prestressing force was determined to ensure no opening of the precast connections under a lateral force corresponding to an assumed moderate earthquake. The prestress corresponded to 55% tendon yield stress. This is lower than typically used for prestressed concrete. Lower initial prestress and unbonding the tendons were intended to minimize yielding of the tendons and hence preserve an axial force necessary for self-centering capability. The specified gravity load was 1456 kN or 0.1 $f'_{co} A_g$, typical for a bridge column, where f'_{co} is the specified concrete strength, 28 MPa, and A_g is the gross cross-sectional area of the column. The gravity load was applied to the cap beam by two hydraulic actuators and remained constant throughout the testing. The displacement-controlled lateral cyclic loading was applied by a hydraulic actuator at one end anchored to the reaction wall and at the other end to the cap beam. The drift levels included 0.25%, 0.375%, 0.5%, 0.75%, 1.0%, 1.5%, 2.0%, 3.0%, 4.0%, 5%, and 6% with each drift level repeated twice.

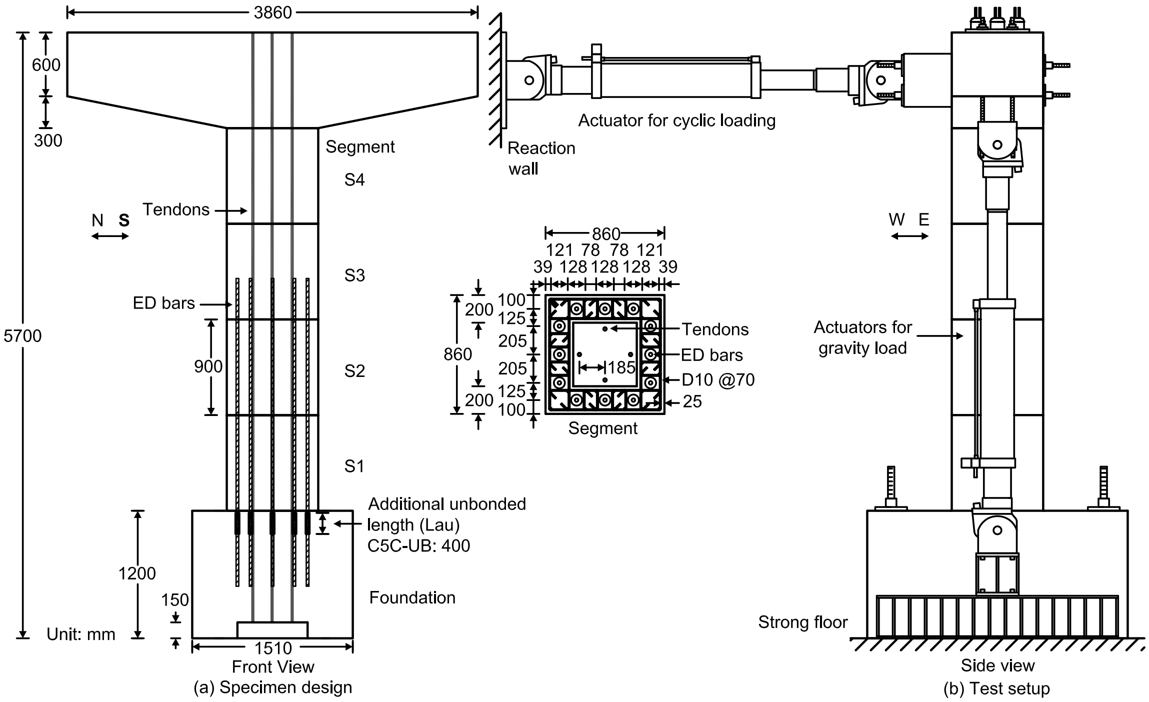

Figure 1. Specimen design and test setup

Table 1. Design parameters

Column	Gravity load (kN)	Prestressing force (kN)	ED bar ratio (%)	L_{au} (mm)
C5C-FB	1456	1042	0.5	0
C5C-E32	1456	1042	0.5	0
C5C-UB	1456	1042	0.5	400

Conventional reinforcing bars used in this research were low-alloy steel deformed bars conforming to ASTM A706, typical for seismic design. Fig. 2 shows the monotonic tension responses of a conventional reinforcing bar and a HP reinforcing bar. The uniform elongation before necking of the HP reinforcing bar was 48%, more than three times that of the conventional reinforcing bar, 13%. In addition, the HP reinforcing bar had higher yield and ultimate strengths than the conventional reinforcing bar as listed. The ED bar contribution to the column lateral strength, denoted as λ_{ED} and defined by Eq. (1), was calculated to be lower than 35% for the three columns. The 35% limitation was to ensure that the residual drift would not exceed 1% before failure of the column as previously mentioned.

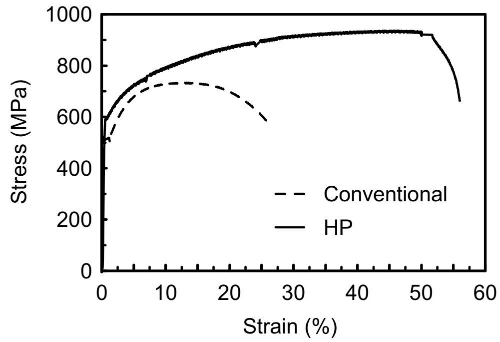

Figure 2. Monotonic tension behavior of conventional and HP ED bars

$$\lambda_{ED} = \frac{V - V_0}{V} \tag{1}$$

where V =lateral strength of a column with ED bars; and V_0 =lateral strength of that column without considering the ED bars.

The ED bars were inserted through corrugated steel ducts during assembling of the columns. The ducts were grouted using a cement-based grout with an actual compression strength of 49 MPa. The ED bars were terminated in segment S3. This was because the peak moment demand at the connection between segments S3 and S4, denoted as connection S3-S4, was found to be lower than the moment that would result in a compression depth lower than half the sectional diameter. The embedded lengths of the bars into segment S3 were determined by multiplying the development length calculated from AASHTO Section 5.11.2.1 (AASHTO 2007) by a ratio of the predicted bar stress at that connection to the ultimate stress of the bar. For the HP ED bars, the computed development length was further multiplied by 1.1 to take into account the higher ratio of the ultimate to yield strengths of the bars. Columns C5C-FB and C5C-UB used conventional reinforcing bars while column C5C-E32 used HP reinforcing bars as ED bars. The ED bars of columns C5C-FB and C5C-E32 were fully-bonded, i.e., bonded to the concrete along their entire length while those of column C5C-UB were unbonded starting from connection foundation-S1 into the foundation for a length of 400 mm. The length is denoted as additionally unbonded length or L_{au}. The unbonding was done by wrapping the bars with duct tape. The 400-mm unbonded length was to ensure no fracture of the ED bars up to 5% drift. The method to determine L_{au} is discussed in detail in Ou et al (2009).

EXPERIMENTAL RESULTS

The lateral force-displacement relationships are shown in Figs. 3(a), 3(b), and 3(c) for columns C5C-FB, C5C-E32, and C5C-UB, respectively. All the three columns failed due to fracture of the ED bars, which occurred at 3%, 6%, and 6% drifts, respectively. Test results in terms of peak values are summarized in Table 3. The ultimate drifts of the three columns were defined as the drift level at which two cycles of loading were completed without fracture of the ED bars. Thus, the drift capacities were set as 2%, 5%, and 5%, respectively. All the three columns showed good self-centering capability with residual drifts not greater than 0.4%. The values of λ_{ED} of the three columns calculated from measured lateral strengths ranged from 23% to 28% (Table 3).

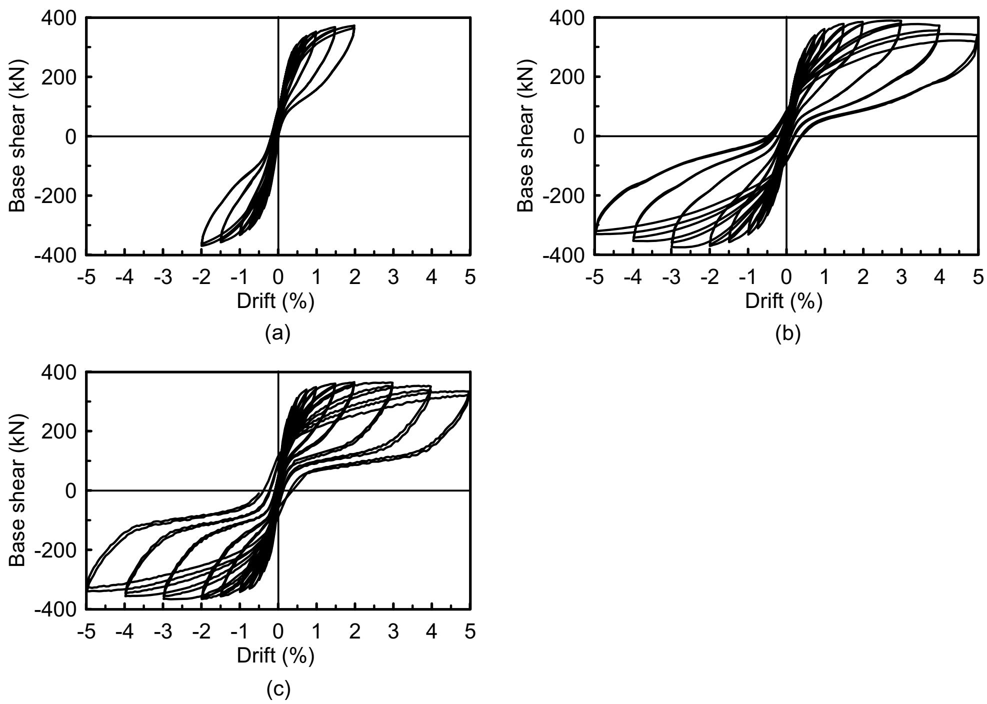

Figure 3. Base-shear versus drift relationships: (a) C5C-FB, (b) C5C-E32, and (c) C5C-UB.

Table 3. Test results

Column	Lateral strength (kN)	Ultimate drift (%)	λ_{ED} (%)	Max ζ_{eq} (%)	Max residual (%)
C5C-FB	370	2.0	25	10	0.1
C5C-E32	386	5.0	28	15	0.4
C5C-UB	363	5.0	23	16	0.4

For column C5C-FB, six of the critical ED bars, which were located at the east and west walls at connection foundation-S1 of the column fractured at 3% drift. For column C5C-E32, the HP ED bars were able to sustain cyclic loading up to 5% drift of the column without any fracture. Compared to column C5C-FB, the drift capacity of column C5C-E32 was greatly increased from 2% to 5% and hence the maximum energy dissipation in terms of equivalent viscous damping ratio ζ_{eq} was improved from 10% to 15% (Table 3). Additionally, column C5C-E32 showed a higher lateral strength. These demonstrated the capability of the HP ED bar to increase the drift capacity, energy dissipation and lateral strength of a segmental column. Instead of using the HP ED bars, fracture of the conventional ED bars can be delayed by unbonding the bars for a length. Unbonding slightly decreased energy dissipation for a given drift but increased maximum energy dissipation by delaying fracture of the bars. The column with unbonding achieved similar performance to that with the HP ED bars in terms of drift capacity and energy dissipation (Table 3). However, unbonding requires additional labor work. In addition, unlike the prestressing tendons, which are fully unbonded and hence can be replaced if corrosion occurs, it is difficult to replace the ED bars. Better corrosion resistance is expected for the column with the HP ED bars than with unbonded conventional ED bars, because not only the HP ED bars have superior corrosion resistance but they are also fully grouted, which further enhances corrosion protection.

CONCLUSIONS

The cyclic behaviors were investigated for three precast segmental concrete bridge columns designed with fully-bonded conventional ED bars, with fully-bonded HP ED bars, and with conventional ED bars that were unbonded for a length. Important conclusions are summarized as follows.

(1)The column with the HP ED bars showed higher drift capacity, greater energy dissipation, and higher lateral strength than that with fully-bonded conventional ED bars. Uubonding the conventional ED bars for a length of 400 mm effectively delayed fracture of the bars while slightly decreased the lateral strength. The column with such unbonded bars achieved the same drift capacity and similar energy dissipation capability as that with the HP ED bars. However, unbonding requires considerable labor work and weakens corrosion protection of the bars.

(3)All the three columns showed good self-centering capability with residual drifts not greater than 0.4%. The measured strength contributions of the ED bars to the lateral strengths of the three columns ranged from 23% to 28%.

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A Study on Restorable Precast and Prestressed Hybrid Piers

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Junichi Hosikuma[2]

ABSTRACT

For the urban viaduct projects, rapid constructions are generally required in order to minimize the temporary traffic control and it is an effective solution to apply the precast and prestressed column structures for the substructures. Precast and prestressed hybrid (P&PH) pier is a new structural type that consists of precast segments and external tendons inside the hollowed pier. The prestressed piers can also reduce the residual displacement after the earthquakes in the seismic regions.

Important structures such as bridges are required to be restorable and reusable quickly as possible even after the large earthquake. Therefore, another advantage of the P&PH pier is the restorable structure. External prestressing cables can minimize the residual displacement of the pier and the hybrid segments of steel and concrete are linked with vertical replaceable steel reinforcement inside the hollow section. As results, the behavior of the pier can be restored quickly after the earthquakes and it is possible to reuse the pier immediately by replacing damaged joint bars.

This paper describes the structural characteristics and some test results of P&PH pier.

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INTRODUCTION

Column members such as bridge piers that resist seismic forces are generally designed as ductile reinforced concrete members with enhanced energy absorption capacities. The fracture mode of members of this type is spalling of the cover concrete due to seismic forces, following which the vertical reinforcements buckle and strength is lost. If a member that has been damaged is to be reused, it needs to undergo a restoration process that typically includes strengthening or repair, such as retrofitting by wrapping in steel plates or adding concrete to increase the thickness. In addition, if there is a large amount of residual displacement, the need to restore the member to its original position in order to reuse it can be particularly problematic.

There have recently been suggestions of using a prestressed concrete column or pier to keep residual displacement to a minimum at the expense of energy absorption capacity.[1][2] This sort of approach is predicated on reuse of the member after an earthquake, and enables a design which will require as little strengthening or repair as possible. Nevertheless, since the prestressing steel still yields in such members, there may be a need to add additional steel or reinforcement the member in some other way to take account of low-cycle fatigue.

Given this background, there has not been sufficient debate concerning the incorporation of seismic performance in LCC (life cycle cost) calculations for bridges. For asset management purposes, engineers assess the risk of damage to a building using the concept of Probable Maximum Loss (PML), calculated as the maximum probable loss (repair cost)/cost of reconstruction x 100 (%). Since adoption of Japan's new anti-seismic design code in 1981, the PML for buildings has generally been considered to be in the range 10-20%.[3] Consequently, a similar approach has been taken to LCC assessment for bridges, in the belief that cost can be minimized. The corollary of this is that seismic design should not only be determined by initial cost, but should employ an approach to design that aims to minimize the cost of restoration after an earthquake while ensuring that the structure retains at least minimal functionality as a transportation route.

This paper presents a structure for minimizing residual displacement after an earthquake while also keeping restoration costs to a minimum when the structure is reused.

CONCEPT OF A NEW STRUCTURE

Figure 1 shows a proposal for a precast prestressed hybrid pier (P&PH pier) structure that can be reused, but which also reduces residual displacement after an earthquake. The proposed structure has the following characteristics.

- Composite structure comprising steel plates on the inside, and concrete on the outside
- Prestressing force is introduced only to the steep plates using external cables
- Segments are precast, and fitted together with un-welded metal-to-metal joints between the steel plates
- Concrete parts are jointed with non-shrink mortar, forming composite members with respect to compressive forces
- Joints between steel plates incorporate joint bars
- After an earthquake, pier performance can be restored by replacing damaged joint bars
- Shear forces are resisted by shear keys in the steel plates

In general, since precast prestressed concrete piers have prestressing applied in the concrete section, a large amount of prestressing is required. In contrast, the P&PH pier only has prestressing applied to the steel plates, so the amount of prestressing required is only small. Also, since compressive forces are resisted as a composite structure with the concrete, the structure ensures that the steel plates are not subjected to excessive load. Furthermore, short steel joint bars are employed to act as yield members for the joints between segments, with the aim of absorbing a certain amount of energy through yielding of the joint bars without the steel plates yielding. As a result of this configuration, the proposed P&PH pier is a rational structure that maximizes the advantages of each of the individual materials and does not result in concrete spalling or yielding of the steel plates, and, because the steel joint bars employed as yield members only with short buckling length, the steel joint bars do not buckle. In addition, the structure also has the particular characteristic of being able to replace the members that are subject to damage, which enables the pier to be reused with only a small cost.

The aim of this research was to ascertain the seismic performance of the P&PH pier, so experiments were performed on a scale model to examine behavior under cyclic loading and to examine behavior after replacing damaged members.

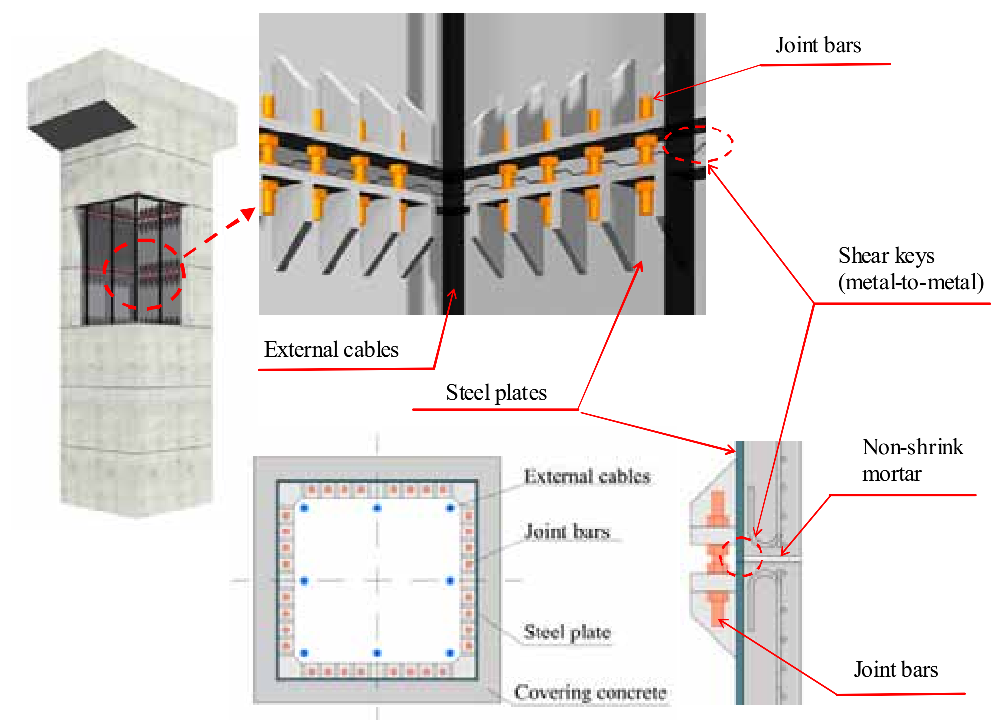
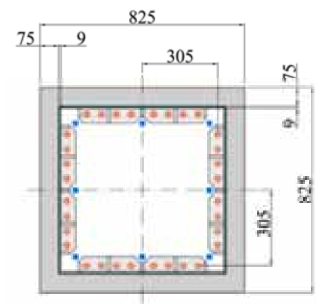


Figure 1 Detail of P&PH pier

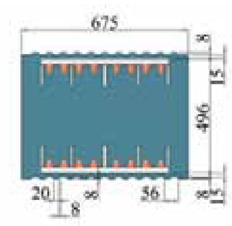
TEST OUTLINE

(1) Specimens

Envisaging an urban viaduct, the specimens were 1/4-scale models of a P&PH pier (full size: width 3.3 m x 3.3 m, pier height: 14.0 m, superstructure weight 8076 kN, external cables: 12S15.2 x 8). The model used as the specimen is shown in Figure 2.



Joint cross-section



Shape of steel plates

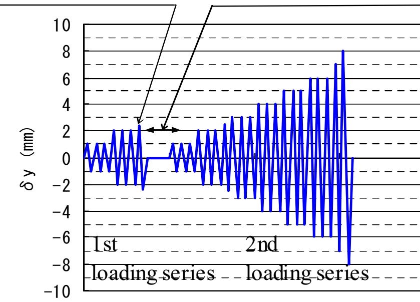
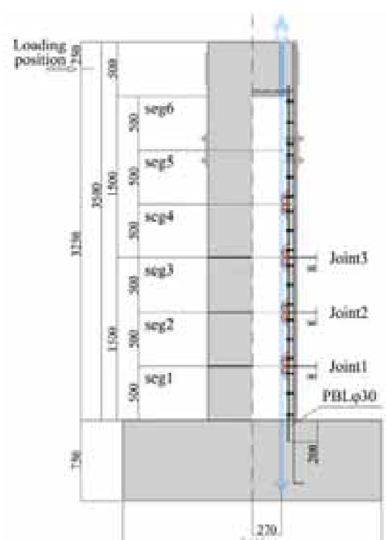


Figure 3 Loading steps



Side view

Figure 2 Specimen

Figure 4 Test equipment and model

The pier itself consists of 6 segments (width: 0.825m x 0.825m, height: 0.500m, concrete thickness: 75 mm), with a total height of 3.250 m from the top of the footing to the loading point. Concrete strength is 60 N/mm^2 for the segments and 40 N/mm^2 for the footings. The steel plates have a thickness of 9 mm (SM490), and between the segments they incorporate shear keys (width: 20 mm-36 mm, height: 8 mm). The steel plates have metal-to-metal joints, and the bottom segment. 1, which forms the base of the pier, is joined by means of Perfobond shear connectors (PBL) embedded in the footing. Eight external cables are distributed around the perimeter of the central void, and the joint bars are M16 bolts (SS400), distributed around the whole of the perimeter. The cross-sectional area of the joint bars is calculated so as to be below the allowable stress for a level 1 earthquake, in contrast to the steel plates, which have thickness calculated so that the plates do not yield when subjected to a level 2 earthquake. In the specimen, the cross-sectional area of steel for one flange was 6075 mm^2 for the steel plate and 1256 mm^2 for the joint bars.

The initial axial compressive stress for the model of the pier was a total of 4.4 N/mm^2, including initial prestressing, the weight of the superstructure, and the weight of the pier.

(2) Loading steps

Since the structure is designed so that the joint section an ultimate state before the base of the pier, cyclic loading test is based on a displacement δy, at which the Joint 1 joint bars on the tension side yield. It was determined that the test would be performed by applying three cycles of cyclic loading that produce displacements that are an integral multiple of δy.

Figure 3 shows the loading steps used in the test, and Figure 4 shows the test rig at the time of the test.

In order to verify recoverability after earthquake damage, a first cyclic loading test series was performed first of all to produce earthquake damage in the specimen. Then, only the joint bars were replaced before performing a second cyclic loading test series. Reproduction of the earthquake damage focused on the joint bars in Joint 1, and the strain producing the damage was assumed to occur in the situation where maximum tensile strain was attained in dynamic analysis. The dynamic analysis, performed in advance, utilized E-W waves recorded on Kobe Port Island in the Hyogo-Nanbu Earthquake for ground type III.

TEST RESULTS

In the 1st loading series, the Joint 1 joint bars attained the standard strain for earthquake damage (the level set for this test) at a displacement of 2.4δy. At that point, the first loading series was terminated and the joint bars were replaced before performing the second series up to a displacement of 8δy. Figure 5 shows the load-displacement curves for both the first and the second series.

In the 2nd loading series, first of all a horizontal crack occurred in the base of the pier at 1δy and then at 2δy Joint 1 began to open. Maximum load was reached at 4δy, then when loading to 5δy one of the joint bars in Joint 1 began to fracture, resulting in a substantial decline in load. At that point, however, the segment showed no significant spalling of the covering concrete. At 7δy, all 16 joint bars in the plane subjected to loading in Joint 1 fractured, after which the decline in load became much smaller. Loading was continued but no significant opening was observed at any of the joints other than Joint 1. Furthermore, no significant damage was observed at the base of the pier, demonstrating that with this structure, damage is concentrated at Joint 1. The amounts by which Joint 1 and Joint 2 opened are shown in Figure 6.

Examining the load-displacement curve reveals a strong tendency to return to the original position when the load is removed, demonstrating that this structure can control residual displacement in a similar manner to other prestressed concrete structures. Furthermore, although the specimen was configured so that the joint bars are not subject to compression when the load is removed, it would also be possible to modify this configuration to produce a structure likely to absorb energy due to the hysteresis loop of the joint bars.

Regarding recoverability, the fact that no significant differences between the load-displacement curves for the 1st loading series and the 2nd loading series can be observed demonstrates that when the structure is subject to earthquake forces that damage the joint bars, the load resistance performance of the pier can be restored by replacing the damaged joint bars.

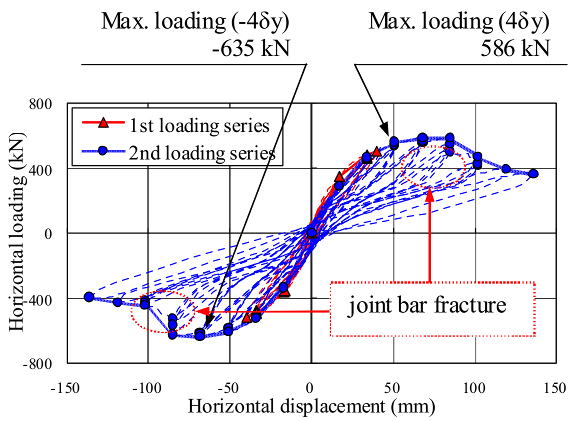


Figure 5 Load-displacement curves

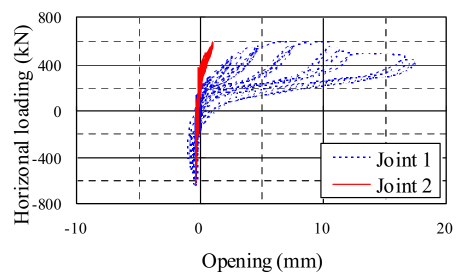


Figure 6 Joint opening

CONCLUSIONS

In summary, the research drew the following conclusions about the proposed P&PH pier.

(1) The P&PH pier can control residual displacement after earthquakes in a similar manner to other prestressed concrete structures. Also, although cracking occurred in the base of the pier damage was successfully restricted to the segment joints.

(2) Even after being damaged by an earthquake, it was possible to restore load-bearing performance by replacing joint bars.

Future research includes examination of ways to enhance performance, including increasing the energy absorption capacity by making the joint bars subject to compression.

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Development of Seismic Design Method for Precast Segmental Concrete Bridge Column

Junichi Sakai, Shigeki Unjoh and Jun-ichi Hoshikuma

ABSTRACT

This paper presents research on the development of seismic design method for precast segmental concrete bridge columns. The precast segmental concrete bridge column would be a suitable structure for accelerated bridge construction because the construction period at the site can be shorten due to no need of formwork, placement and curing of concrete. Thus, they are expected to be applied for the construction of overpass crossings in urban areas to minimize the effect on existing traffic. Additionally, high quality of the concrete members would be ensured because the concrete segments are fabricated at factories.

Public Works Research Institute had conducted 2-years joint research program with three private companies including Kajima Co., Sumitomo Mitsui Construction Co., Ltd., and P.S. Mitsubishi Construction Co., Ltd. in 2007-2008. In the research program, three types of structural details of precast segmental concrete bridge columns were proposed and the failure mechanism of the proposed columns was investigated through a series of shake table test. Based on the experimental studies, seismic design methods including limit states to achieve the required seismic performance, detailed design methods for segments, joints, prestressing steel, and bending–shear resistance were determined.

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INTRODUCTION

The precast segmental concrete bridge columns would be a suitable structure for accelerated bridge construction because the construction period at the site can be shorten due to no need of formwork, placement and curing of concrete to construct bridge substructures. Thus, they are expected to be applied for the construction of overpass crossings in urban areas to minimize the effect on existing traffic. Additionally, high quality of the concrete members would be ensured because the concrete segments are fabricated at factories. However, the seismic design method of such precast segmental concrete bridge columns has not yet been developed because the seismic performance of the structure has not yet been investigated comprehensively. To accelerate the implementation of such structures, it is needed to develop the seismic design method.

Public Works Research Institute had conducted 2-years joint research program with three private companies including Kajima Co., Sumitomo Mitsui Construction Co., Ltd., and P.S. Mitsubishi Construction Co., Ltd. in 2007-2008. In this research program, three types of structural details of precast segmental concrete bridge columns were proposed and the failure mechanism of the proposed columns was investigated through a series of shake table test. Based on the experimental studies, design methods including limit states to achieve the required seismic performance, detailed design methods for segments, joints, prestressing steel, and bending–shear resistance were determined.

This paper introduces the seismic design method proposed based on the results from the joint research program, and especially focuses on the limit states of the precast segmental concrete columns.

STRUCTURAL DETAILS OF PRECAST SEGMENTAL BRIDGE COLUMNS

In the joint research program, the seismic performance of three types of precast segmental concrete bridge columns was investigated. Since structural details can be found in the company papers (written by Dr. Hosikuma and Mr. Taira), main features are briefly introduced in this paper.

Figure 1 shows the precast segmental prestressed concrete bridge column proposed by Kajima Co. Only prestressing steel but no mild reinforcement is provided as longitudinal steel bars. Inner pipes are provided for shear resistance.

Figure 2 shows the precast segmental hybrid bridge column proposed by Sumitomo Mitsui Construction Co., Ltd. The segment of this column is a hybrid structure made of steel shell and concrete. Connecting bolts are the main longitudinal steel bars, and these bolts are designed to be replaceable after a design earthquake to ensure the repairability. Shear keys of the steel shells are provided for shear resistance.

Figure 3 shows the precast segmental reinforced concrete bridge column proposed by P.S. Mitsubishi Construction Co., Ltd. The column is longitudinally reinforced by mild reinforcement going through ducts of the precast segments. The seismic performance of this column is expected to be similar or even better than the conventional reinforced concrete bridge column because the longitudinal reinforcing bars that are provided in the mortal grouted ducts have better performance on anti-buckling.

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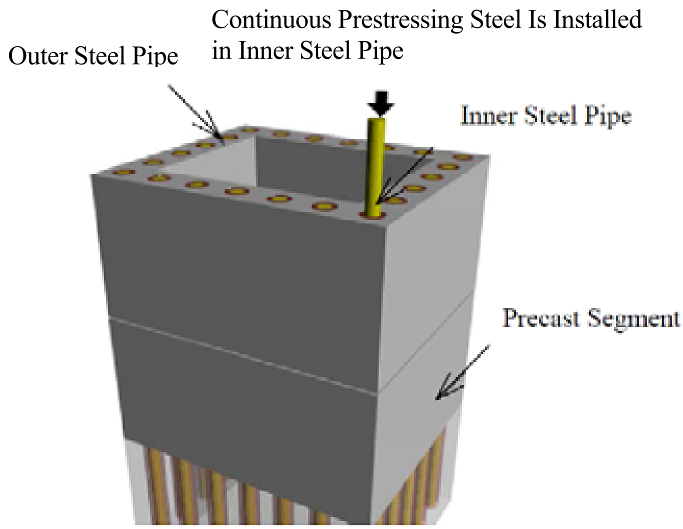

Figure 1 Precast segmental prestressed concrete bridge column

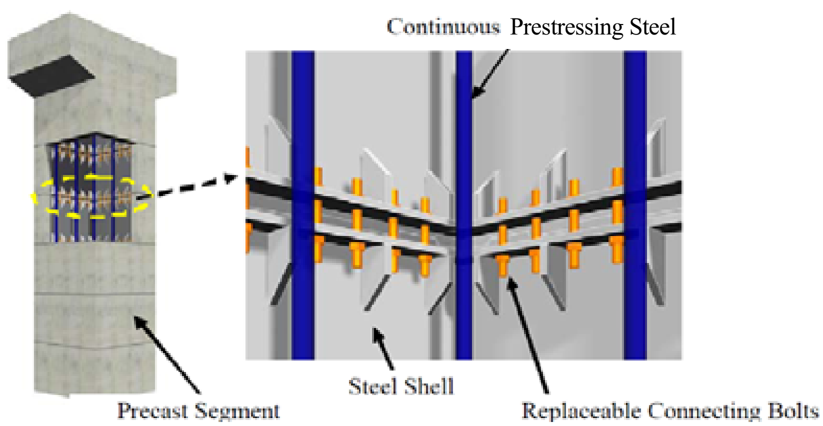

Figure 2 Precast segmental hybrid bridge column

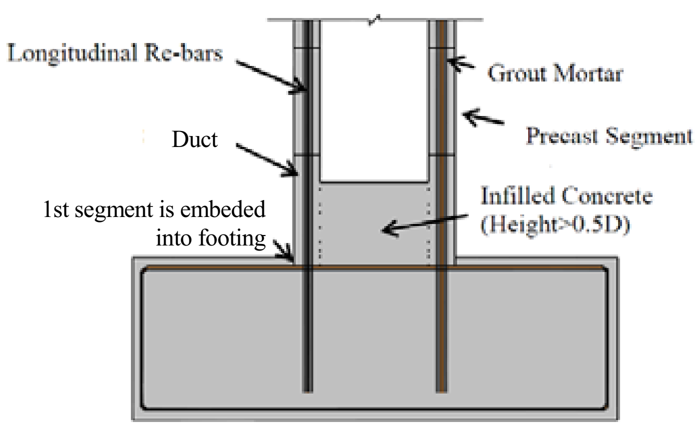

Figure 3 Precast segmental reinforced concrete bridge column

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LIMIT STATES OF PRECAST SEGMENTAL BRIDGE COLUMNS

General Concepts of Limit States

To develop the seismic design method, it is essential to determine the limit state for each seismic performance level. In Specifications for Highway Bridges issued by Japan Road Association, three seismic performance levels are considered for intensities of design ground motions and importance of the bridges. The limit states are determined based on the seismic performance, which are described as the safety, serviceability and repairability. The schematic image of the limit state of the conventional reinforced concrete bridge column is shown in **Figure 4**.

For the seismic performance level 1, it is required to ensure the normal functions of bridges after an earthquake. As limit states, the mechanical properties of the bridges are maintained within the elastic ranges. For each structural member, the stress induced by an earthquake shall not exceed its allowable stress.

For the seismic performance level 2, it is required to ensure the serviceability after an earthquake, and the repairability is also ensured. As the limit state, the structural members in which the nonlinear behavior is allowed deform beyond elastic range but within a range of easy functional recovery.

For the seismic performance level 3, it is required to ensure the structural safety after an earthquake. Neither serviceability nor repairability is required. As the limit state, structural members in which the nonlinear behavior is allowed deform within the ultimate ductility capacity.

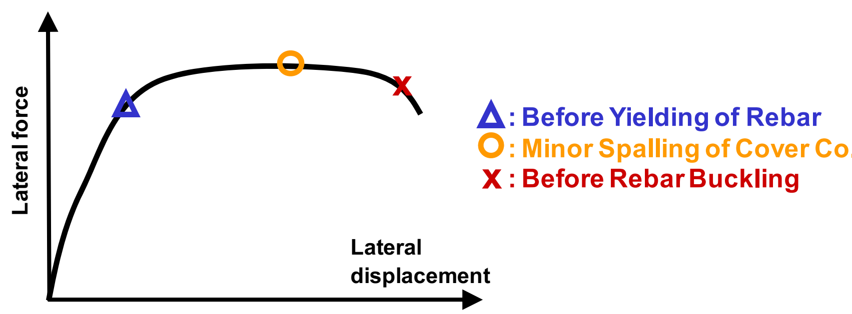


Figure 4 Limit states of conventional reinforced concrete column

Limit States of Precast Segmental Concrete Bridge Column

General

The limit states of the precast segmental concrete bridge columns shown in **Figures 1**, **2** and **3** are determined considering the structural properties and nonlinear behavior of each structure.

For the seismic performance level 1, the limit states of the precast segmental structures are determined to be same to a conventional reinforced concrete column. For each structural member, the stress induced by an earthquake shall not exceed its allowable stress.

The limit states for the seismic performance levels 2 and 3 are determined based on the nonlinear behavior of each structure, which are introduced in details below.

Limit States of Precast Prestressed Concrete Bridge Column

Because no mild longitudinal reinforcement is provided in the precast prestressed concrete bridge column, the yielding of the prestressing steel is the important limit state. Once the yielding of the prestressing steel occur, it is difficult to recover the required functions. Based on these conditions, the limit states are determined as shown in **Figure 5**.

For the seismic performance level 2, the yielding of the prestressing steel is determined as the limit state to ensure the serviceability and repairability. **Figure 6** shows the force-displacement hysteresis and the failure mode obtained from the shake table tests during the design level earthquake ground motion. The prestressing steel remained in the elastic range and minor spalling of cover concrete was observed.

Although the column model performed well beyond the yielding of the prestressing steel in the shake table test, the range beyond this point is not considered in the seismic design of the precast prestressed concrete bridge for safety consideration. Thus, the same limit state is used for the seismic performance levels 2 and 3. Further research is needed for consideration of the behavior after yielding of the prestressing steel.

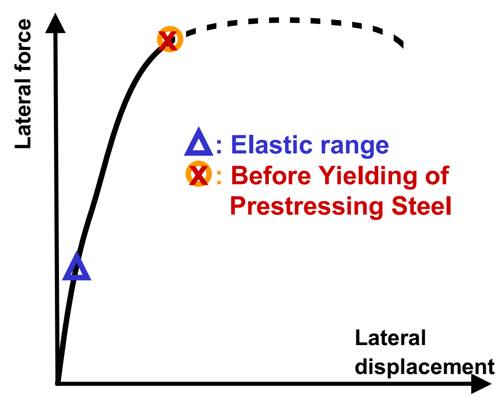


Figure 5 Limit states of precast prestressed concrete column

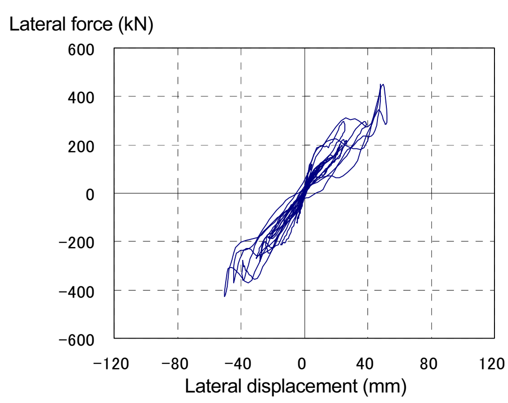


Figure 6 Response hysteresis and failure mode after design level test for precast prestressed concrete column

Limit States of Precast Hybrid Bridge Column

The schematic image of the limit states of the precast hybrid bridge column is shown in **Figure 7**. Because the key feature of this column is the repairability by replacement of the connecting bolts, the limit states of the seismic performance level 3 is determined to be same as those of the seismic performance level 2.

The replaceable limit of the connecting bolts is determined as the limit state for the seismic performance level 2. The allowable strain of the bolts is set to be 2% based on the low-cycle fatigue tests and the shake table tests. The other structural members should remain in the elastic range. **Figure 8** shows the force-displacement hysteresis and the failure mode obtained from the shake table tests during the design level earthquake ground motion. The results from first series (case 1) and post-repair series (case 2) are compared. After the design level tests in the post-repair series, the response displacement was still smaller than the design displacement while about 2% strain was induced in the connecting bolts and minor spalling of cover concrete was observed.

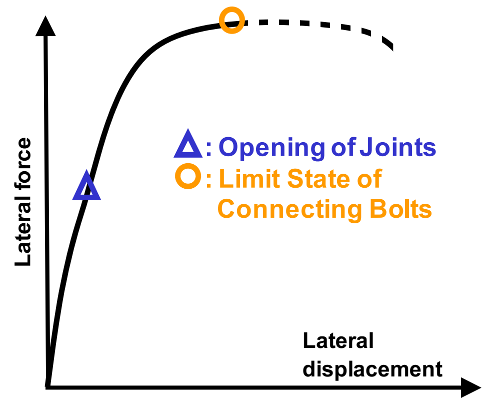
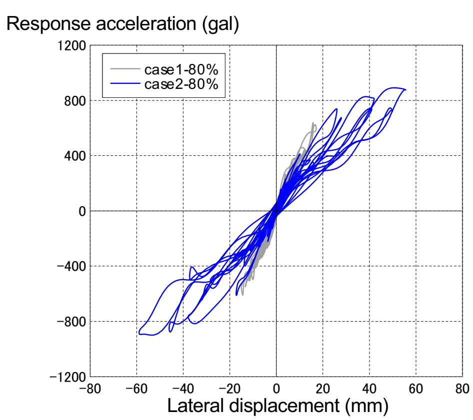


Figure 7 Limit states of precast hybrid column



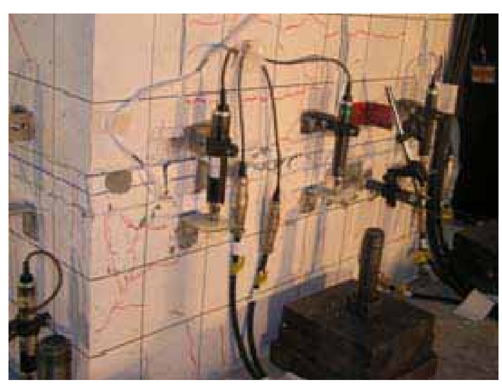


Figure 8 Response hysteresis and failure mode after design level test in post-repair series for precast hybrid column

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Limit States of Precast Reinforced Concrete Bridge Column

It was confirmed that the nonlinear behavior of the precast reinforced concrete bridge column is similar or even better than the conventional reinforced concrete bridge column because the longitudinal reinforcing bars that are provided in the mortal grouted ducts have better performance on anti-buckling. The limit states of this column are determined to be the same as those of the conventional ones, which is shown in **Figure 4**.

Figure 9 shows the force-displacement hysteresis and the failure mode obtained from the shake table tests during the design level earthquake ground motion. Only flexural cracks were observed, and the stable hysteresis loop is obtained.

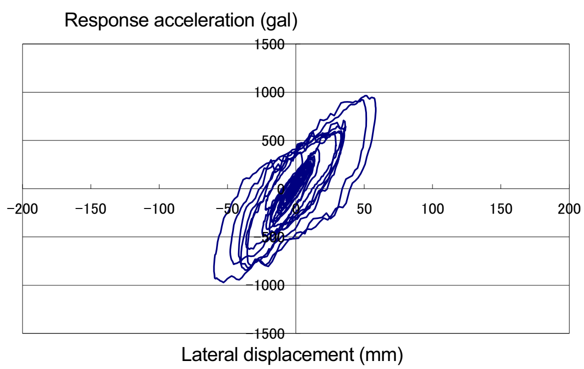

Figure 9 Response hysteresis and failure mode after design level test for precast reinforced concrete column

CONCLUSIONS

To develop the seismic design method for precast segmental concrete bridge columns, a joint research program was conducted. Three types of structural details of precast segmental concrete bridge columns were proposed and the failure mechanism of the proposed columns was investigated through a series of shake table test. Based on the experimental studies, seismic design methods including limit states to achieve the required seismic performance, detailed design methods were determined, and the design limit states of each proposed structure are introduced in this paper.

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Analytical and Experimental Investigation of Precast Segmental Bridge Systems

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ABSTRACT

Precast segmental bridge construction is currently receiving increasing attention in North America and Europe mainly due to the advantages of accelerated construction and high quality control that offers over the traditional cast-in-place techniques. Despite these advantages, concerns have arisen amongst the engineering design community regarding the response of such systems under intense earthquake shaking, which may include joint opening and/or relative sliding between adjacent segments. In this study, a novel segmental concrete bridge system, which is going to be used for shake table and quasi-static testing, is presented as well as an approach to efficiently model segmental systems using 2-node elements widely available is most structural analysis software. The results of the analyses conducted for the proposed bridge system using this modeling approach indicate that segmental systems may exhibit high ductility and enhanced self-centering capabilities under severe ground excitation.

1. INTRODUCTION

Precast segmental concrete bridge construction was first introduced in Europe in the 1950s, whereas the first application of this type of construction in the United States was the John F. Kennedy Memorial Causeway in Corpus Christi, Texas in 1972. Since then, the number of applications of precast segmental bridge systems has increased substantially both in the United States and around the world mainly due to the advantages that precast segmental construction offers over the traditional cast-in-place techniques. These advantages include: (i) higher construction quality, since the segments are constructed in a shop under well-controlled quality conditions, and (ii) rapid construction, since, as soon as the segments have been carried to the construction site, only assembly and preparation of the joint connections is required. Assembly is usually achieved by internal grouting or external tendons, while, at the joints, epoxy adhesive high-strength materials or male-female indentation connections are utilized.

Despite the apparent advantages of the precast segmental construction, concerns have arisen

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amongst the engineering design community regarding the performance of such structural systems in regions of moderate to high seismicity. These concerns, which have limited the applications of this construction technique only to low seismicity areas, primarily refer to: (i) the effects of significant joint opening and/or sliding between adjacent segments during strong earthquake shaking on the global stability of the structural system, and (ii) the reliability of existing analysis tools in predicting the dynamic response of segmental bridge systems under tri-axial earthquake excitation, which is necessary in the context of performance-based design.

This paper presents the partial results of a study investigating the seismic performance of precast segmental concrete bridges both experimentally and analytically/numerically. As far as the experimental part is concerned, a large-scale bridge specimen intended to be used for quasi-static and shake table testing is described, whereas, in the context of the analytical investigation, an approach to model efficiently segmental systems using 2-node elements is illustrated.

2. TEST SPECIMEN: PRECAST SEGMENTAL CONCRETE BRIDGE

2.1. General Description

The prototype bridge structure on which the design of the test specimen is based is a five span single cell box girder bridge considered by Megally et al. (2002). Each span of the prototype bridge is post-tensioned with a harped shaped tendon. The test specimen considered in this study is a large scale (~1:2.4) single-span (referring to the mid-span of the prototype system) single cell box girder segmental concrete bridge with both of its supports overhanging at equal lengths, as shown in Figure 1. Following the principles of the "Accelerated Bridge Construction" (ABC) technique, the deck of the bridge specimen consists of 8 hollow segments of trapezoidal cross-section which are post-tensioned together using 10 to 14 internal unbonded tendons, whereas each pier consists of 5 segments of hollow square cross-section that are post-tensioned together by 8 internal unbonded tendons. Further information on the preliminary design of the specimen superstructure may be found in Anagnostopoulou (2009). The geometric properties of the superstructure segments and the pier segments are shown in Figure 2 and Figure 3, respectively.

Although use of internal unbonded tendons has never been considered in any of the existing segmental bridges and has never been reported in the literature for segmental bridge superstructures, preliminary studies show that they may result in better system performance, since local tendon yielding and rupture, which could result in global system instability, is avoided because deformation is distributed over larger tendon lengths. Consequently, tendon yielding occurs at larger system deformations providing the bridge system with greater ductility capacity and enhanced self-centering capabilities. Similar findings have been reported for bridge segmental piers by Ou (2007). The segment-to-segment joints for both the deck and the piers are simple friction-type connections defined by direct contact of adjacent segments. Thus, shear resistance at the joints is provided only by the friction generated between concrete segment end surfaces in contact, and the "dowel effect" of the tendons. Similarly to the unbonded tendons, use of friction-type joints has never been reported in the literature for bridge superstructures and has never been used in any of the existing bridges; however, this connection type performed adequately in segmental bridge piers (Ou 2007).

Another novelty incorporated in this study is the use of gap restrainers in order to distribute the gap opening over the length of segmental members. Theoretical investigation has shown that as soon as gap opening occurs at a joint, and as long as the applied member force distribution does not change, no gap opening (or no significant gap) opening is highly likely to develop in the adjacent sections. The same studies suggest that the gap opening of a joint has to be restrained in order to allow gap opening at the adjacent joints. For this reason, gap restrainers are currently under design for the piers, which are the most critical components for the stability of the bridge system. Distribution of gap opening at segmental piers has already been attempted by energy dissipation bars (Ou 2007). However, although the energy dissipation provided by the member increased, insignificant gap opening distribution was achieved.

To facilitate support of the deck on the piers, a cap beam of trapezoidal solid shape presented in Figure 4 is placed on top of each pier. At the two ends of each cap beam, stoppers are installed to prevent lateral sliding of the deck, while sliding of the deck in the longitudinal direction is prevented by stoppers attached to the bottom of the deck segment located on top of the cap beam. Anchorage of each pier on one of the two adjacent relocatable shake tables of the Structural Engineering and Earthquake Simulation Laboratory (SEESL) of the University at Buffalo, where shake table and quasi-testing will be conducted, is achieved by a foundation concrete block mounted on each shake table, as shown in Figure 1. The foundation block, the cap beam and the 5 pier segments are all post-tensioned together by the same tendons, as illustrated in Figure 5. In the same figure, the stoppers against lateral sliding are presented, whereas the stoppers for the longitudinal sliding are shown in Figure 6, which presents a lateral elevation view of the bridge specimen.

The bridge specimen was designed as if it was a monolithic system according to AASHTO LRFD Bridge Design Specifications (2007), while its design was also assisted by the PCI Bridge Design Manual (2003). The performance objectives stated in these codes of practice were properly adjusted to abide by the concepts and principles of the ABC system. For example, the requirement for crack opening control under service loads was extended to include joint opening control under the same loading scenario. To determine the seismic loads, the prototype bridge on which the bridge specimen is based was assumed to be located at a site in the Western United States. The seismic loads, along with all other loads, were properly scaled in order to be consistent with the assumptions of the similitude analysis.

Figure 1. Precast segmental concrete bridge specimen on the two adjacent relocatable shake tables of Structural Engineering and Earthquake Simulation Laboratory (SEESL) at University at Buffalo (UB)

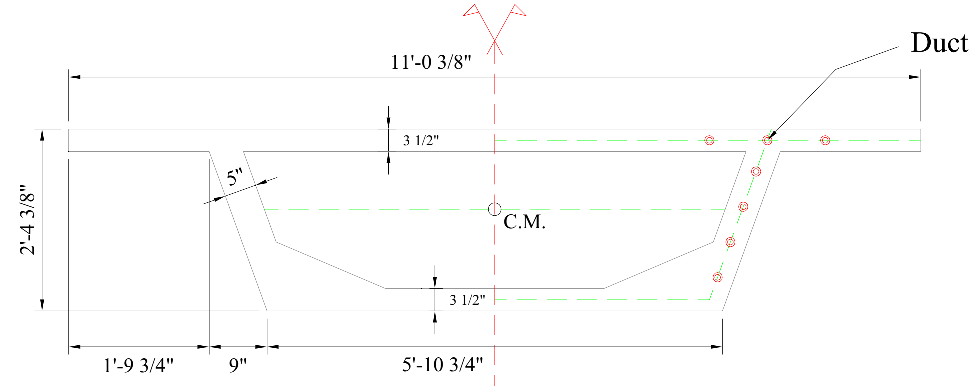

Figure 2. Superstructure cross-section

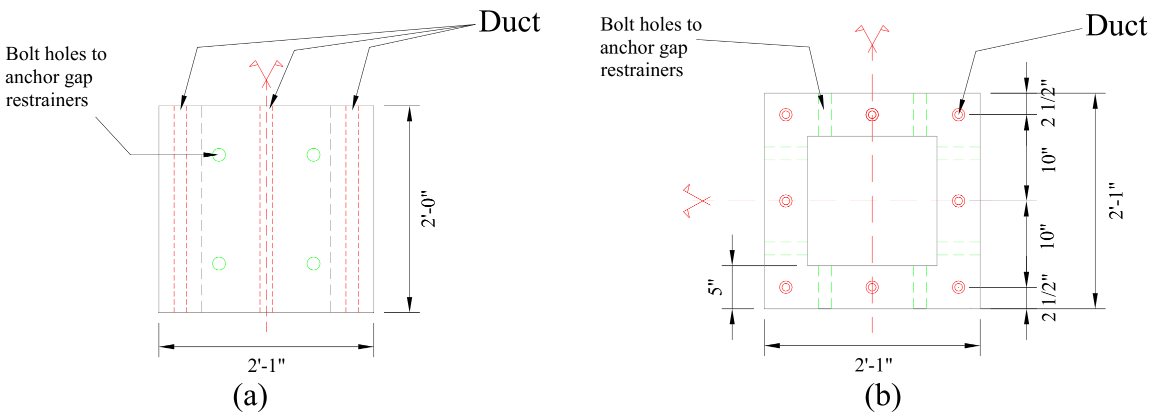

Figure 3. Pier segment: (a) Elevation view, (b) Cross-section plan view

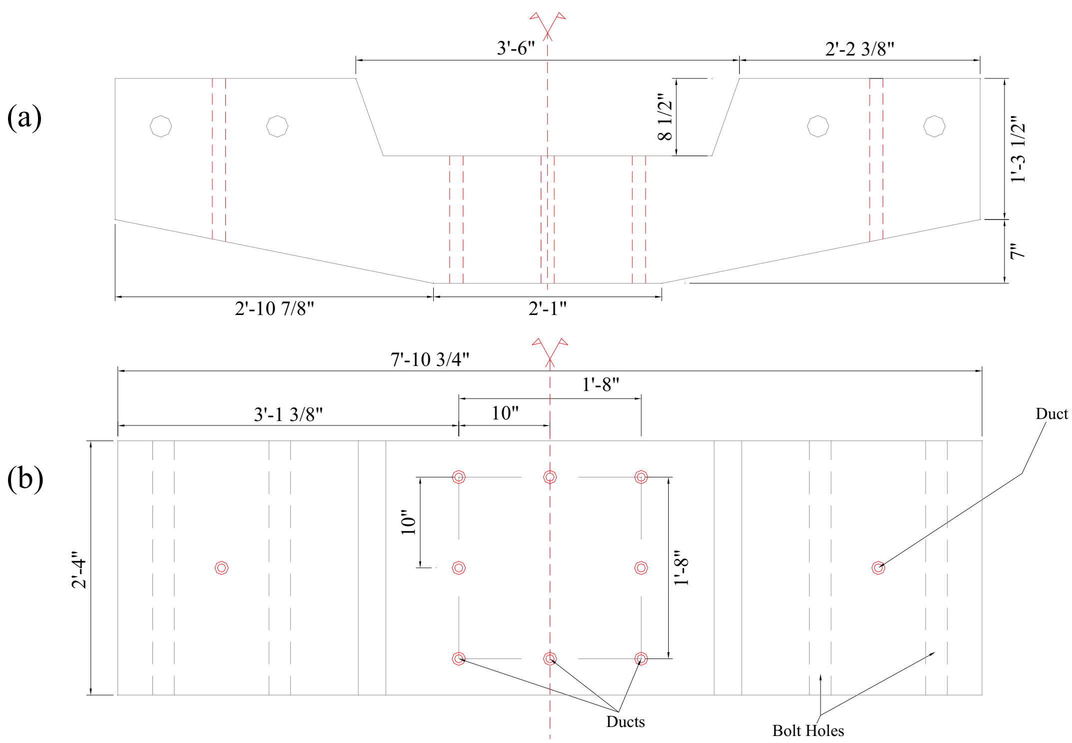

Figure 4. Cap beam: (a) Elevation view, (b) Plan view

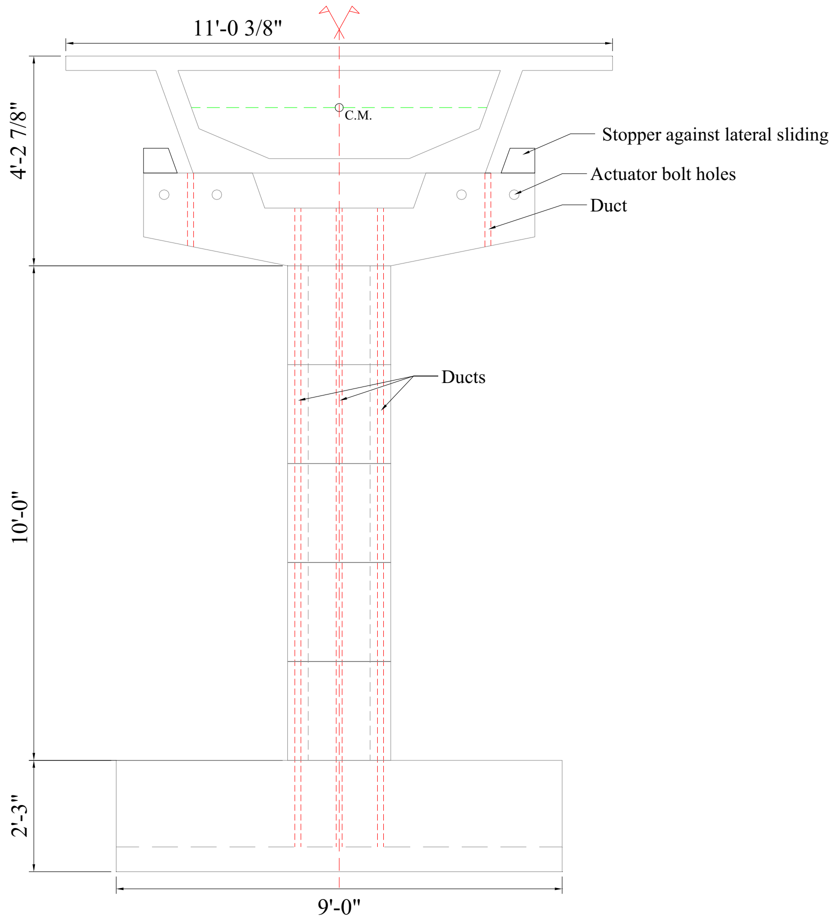

Figure 5. Bridge specimen longitudinal elevation view

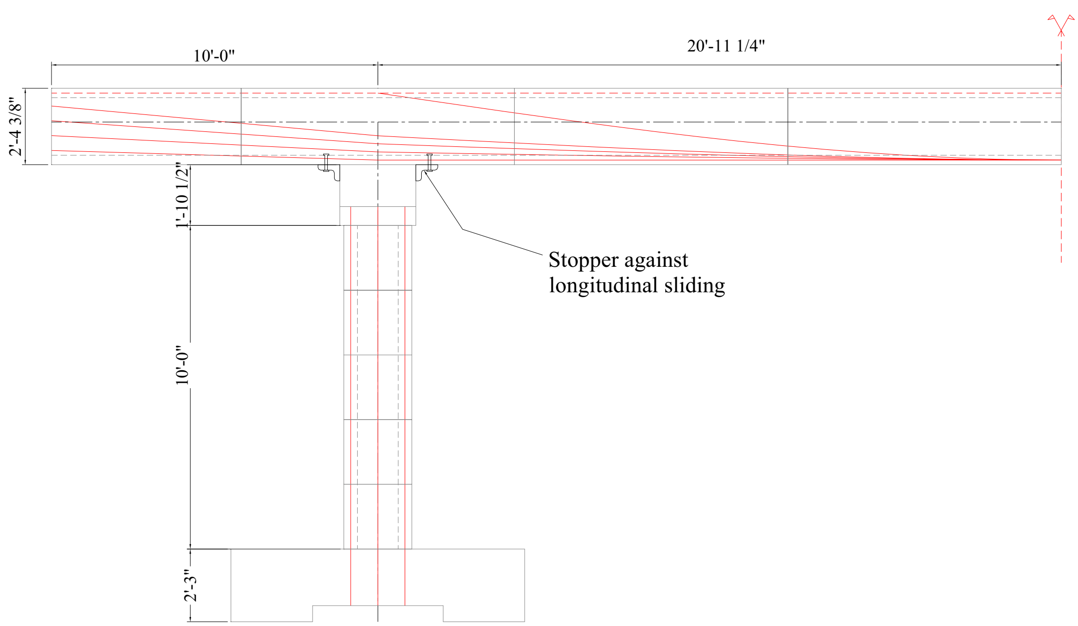

Figure 6. Bridge specimen lateral elevation view

3. ANALYTICAL/NUMERICAL INVESTIGATION USING 2-NODE ELEMENTS

3.1. Proposed Modeling Approach

The response of precast segmental systems under static and/or dynamic loading is significantly different from the response of conventional monolithic systems, since they potentially experience joint opening and sliding at the segment-to-segment interface. Although finite element analysis may efficiently predict the response of such non-conventional systems (Ou 2007, Aref et al. 2008), it requires large computational resources, extensive time, and use of complicated analysis software, which discourage practicing engineers from using it. On the other hand, widely available commercial structural analysis software, which mainly incorporate two-node elements or two-node macro-elements to predict the response of conventional structural members (i.e. beams and columns), seem incapable of capturing the main characteristics of the response of segmental systems.

In the present study, an approach to consistently model the response of segmental systems using typical nonlinear two-node beam-column elements and special connection elements available in most structural analysis programs is presented, and utilized to model the bridge specimen described earlier. In the proposed approach, segments are modeled as beam-column elements except for a region at the ends of each segment which is discretized into several 2-node axial (or axially-dominated) "fiber" springs. The "fiber" springs, which are independent from each other, are connected to the ends of the corresponding beam-column element through rigid body constraints/links. In order to ensure consistent behavior of the segment super-element (beam-column element and fiber springs), the number (minimum) and appropriate distribution of the "fiber" springs over a cross-section is recommended to be selected such that the exact second moments of inertia of the cross-section, I_y and I_z, and the approximate values of the associated integrals based on the selected discretization deviate from each other by less than 1%. The approximation of these integrals is shown below:

$$I_y = \int_A y^2 dA \approx \sum_{i=1}^{N} y_i^2 A_i \quad and \qquad I_z = \int_A z^2 dA \approx \sum_{i=1}^{N} z_i^2 A_i \tag{1}$$

where A_i is the area of the i-th fiber spring, and (y_i, z_i) is its location on the cross-section. If non-uniform material is considered, this concept should be extended to the cross-section moduli EA, EI_y and EI_z. The approximation of the corresponding integrals is:

$$EI_y = \int_A Ey^2 dA \approx \sum_{i=1}^{N} y_i^2 E_i A_i \qquad and \qquad EI_z = \int_A Ez^2 dA \approx \sum_{i=1}^{N} z_i^2 E_i A_i \tag{2}$$

$$EA = \int_A E_i dA \approx \sum_{i=1}^{N} E_i A_i \tag{3}$$

where E_i is the modulus of elasticity of the i-th fiber spring.

The nonlinear properties of the "fiber" springs are selected so that the segment super-elements demonstrate consistent axial and bending behavior. Thus, assuming bilinear hysteretic response, the total axial yield force provided by the "fiber" springs should be the same as that provided by the beam-column element, while the yield axial strain should be the same for all "fiber" springs and the beam-column element.

$$F_y^{Beam-Column} = \sum_{i=1}^{N} F_{y,i} \quad and \quad \varepsilon_y^{Beam-Column} = \varepsilon_{y,i}\ ,\ \ for\ i = 1,...,N \tag{4}$$

Elastic and post-yield stiffnesses should also satisfy similar concepts.

Considering that moment – axial force interaction is directly taken into account at the end regions of a segment by the distributed "fiber" springs, it is necessary to consider inelastic behavior at the beam-column elements with moment-axial force interaction envelopes in order to maintain consistent nonlinear behavior over the super-element. Furthermore, to avoid penetration of a segment into its adjacent ones, contact springs (e.g. hertzian contact law) should be considered at the perimeter of the end cross-sections.

As far as the selection of the portion of a segment which is modeled by "fiber" springs is concerned, it may be based on the geometric characteristics (cross-section depth, wall thickness, segment length) of the segment and the level of nonlinear behavior considered and/or expected at the segmental joints of interest. Thus, for longer segments, the length of the "fiber" springs may equal the depth of the cross-section, whereas, for shorter segments, the "fiber" element length may be taken such that the contact elements at the perimeter of the cross-section are not activated (or not significantly activated) during the analysis. For short segments, it is also suggested that shear deformations be considered for the beam-column element.

Due to the nature of the proposed modeling approach, three generalized section forces can be generated at the ends of the super-element; an axial force N_S and two moments, $M_{S,y}$ and $M_{S,z}$, defined as:

$$F_S = \sum_{i=1}^{N} F_i \ \ , \quad M_{S,y} = \sum_{i=1}^{N} z_i F_i \quad and \quad M_{S,z} = -\sum_{i=1}^{N} y_i F_i \tag{5}$$

where F_i is the force of the i-th "fiber" spring located at $\left(y_i, z_i\right)$ on the cross-section. Shear and/or torsional resistance may be introduced either by considering a shear springs between every pair of nodes of a "fiber" spring, or by considering one shear/tortional spring between the two end nodes of two adjacent beam-column elements. The properties of these springs are determined by the properties of the segments in contact and the segment-to-segment interface.

Regarding the tendons of the post-tensioning system, each of them is modeled by several axial "tension-only" bilinear springs in series. Each node of this spring assembly is laterally constrained to the adjacent segmental joint. Post-tensioning is achieved by considering initial element stressing, either as a thermal effect, or as pre-stressing.

3.2. Model Definition

A 2-D numerical model for the bridge specimen in the lateral direction is developed using the structural analysis software Ruaumoko (Carr 2004) to illustrate the proposed multi-element modeling approach. The model, which is shown in Figure 7, utilizes “compression-only” bilinear hysteretic springs with slackness as “fiber” springs, beam-column elements incorporating moment-axial force interaction diagrams, hertzian contact elements, and “tension-only” axial bilinear springs with slackness to model the post-tensioning system The pier segment cross-section is divided into 9 “fiber” spring areas along its depth, 5 of which are located in the webs, whereas the remaining 4 are located in the two flanges (2 in each flange). The portion of the segments (longitudinally) modeled by “fiber” springs is 6 inches at each end, resulting in “fiber” elements of total length of 12 inches. Although the cap beam and the foundation block are assumed to be rigid compared to the pier segments, short “fiber” springs (3 inches in length) are considered in the area of contact in both the cap beam and the foundation block to account for the existing local flexibility. The properties of the “fiber” elements are derived by taking into account the effect of both the concrete and the steel reinforcement, while the moment-axial force interaction diagrams are determined using appropriate computer codes. To avoid penetration of a segment into the adjacent ones, contact springs are placed at the edges of the cross-section and in the middle of the cross-section depth.

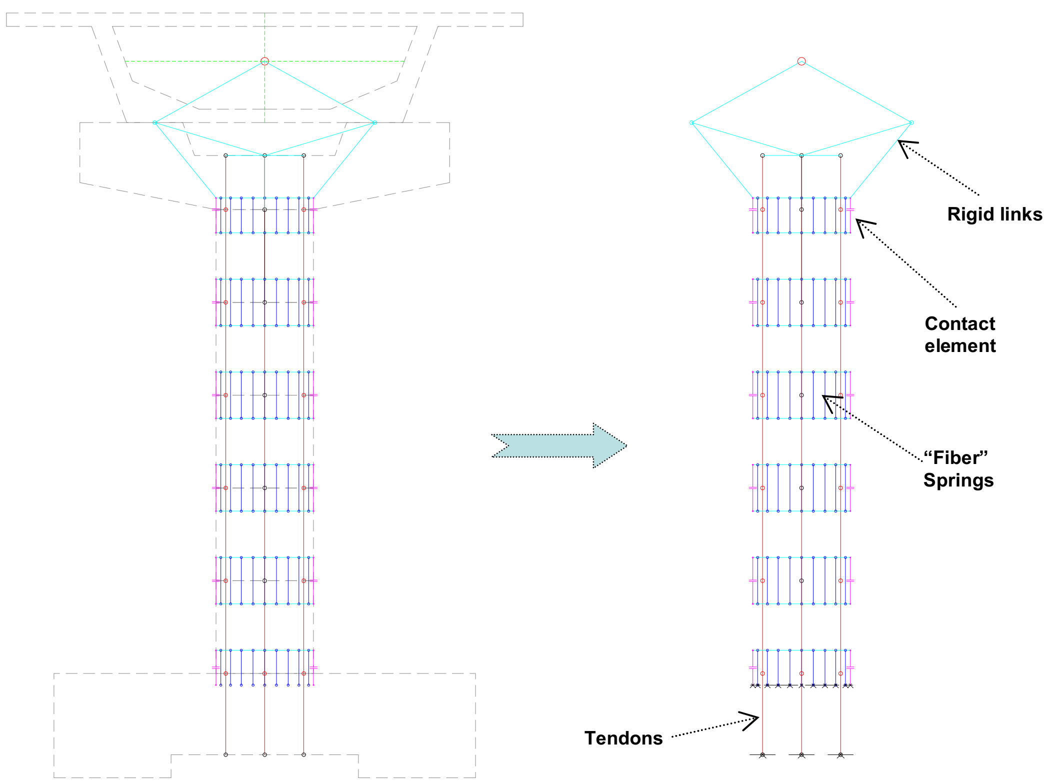

Figure 7. Numerical model for bridge specimen in the lateral direction

The post-tensioning system, which consists of 8 tendons, is modeled by three series of “tension-only” axial bilinear springs. The two series closer to the exterior surfaces of the segments model the tendons located at the flanges (3 at each flange), whereas the one series in the middle of the segment depth models the two tendons located at the webs. No sliding is considered at the segment-to-segment joints and at the interface between the cap beam and the deck.

3.3. Modal and Dynamic Analyses

By performing a modal analysis for the bridge specimen using the model presented in Figure 7, the first few modes of the response are computed. Thus, the first natural period of the system is found to be 0.224 seconds, while the 2nd and the 3rd natural periods are found to be 0.018 seconds and 0.009 seconds, respectively. From these values, in the context of the similitude analysis performed for the design of the test specimen, the natural periods of the prototype may be approximated as 0.535 seconds, 0.043 seconds and 0.021 seconds for the 1st, 2nd and 3rd mode, respectively.

Dynamic analyses are performed with 200% amplitude of the North-South component of the 1940 El Centro record. To abide by the assumption of the similitude analysis, the earthquake accelerogram is also scaled both in time ($\times 1/2.4$) and amplitude ($\times 2.4$), resulting in a peak ground acceleration of 1.62g (compared to the original 0.34g). Rayleigh damping is considered for the numerical model with a 3% damping ratio assigned to its two first modes. From the data obtained from the time history analysis of the system, observations may be made regarding the dynamic behavior of the bridge specimen in the lateral direction. Thus, the ground acceleration applied to the system and the total acceleration computed at the deck are shown in Figure 8. From these time histories, it can be seen that the absolute deck acceleration does not exceed an upper bound of 0.5g to 0.6g. Such behavior may be attributed to the joint opening at the base of the system (pier segment-to-foundation block joint) which controls/limits the maximum base moment that can be developed at the joint, and, consequently, controls the maximum seismic force applied to the system as well. An illustration of the self-centering capabilities of the bridge specimen considered in this study is presented in Figure 9 which shows the relative displacement response of the deck. According to this figure, the residual deformation of the system is negligible, which may be attributed to the fact that the tendons did not yield during the analysis, while the concrete segments experienced negligible inelastic deformations.

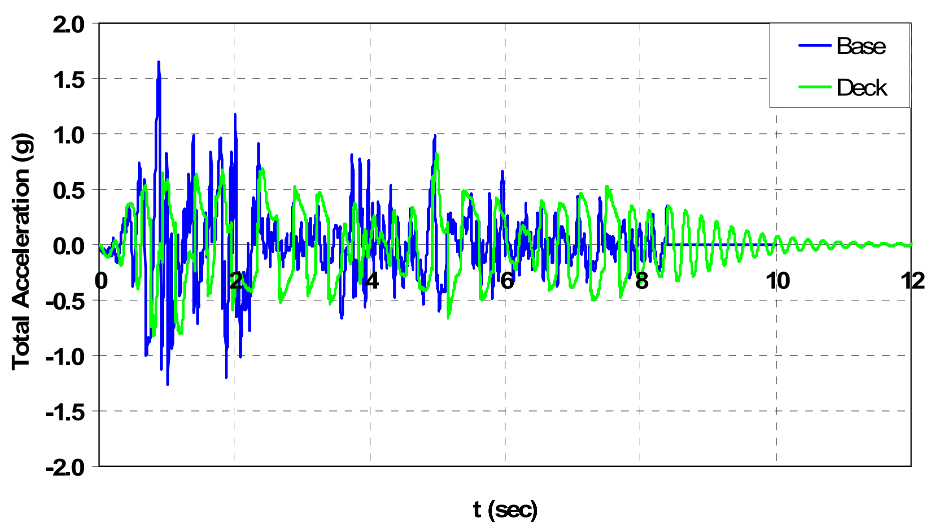

Figure 8. Total acceleration time histories

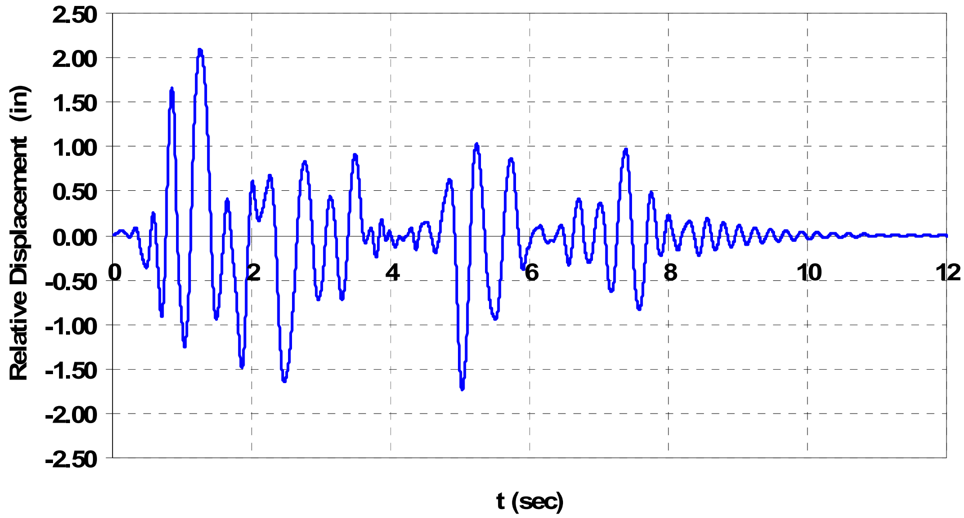

Figure 9. Relative displacement time history of deck (measured at its center of mass)

4. CONCLUSIONS

The proposed precast segmental bridge system with internal unbonded tendons and shear resistance at the joints provided only by friction between adjacent segments in contact seems to perform satisfactorily under earthquake excitation. Preliminary numerical analyses show that the system can be designed to exhibit high ductility and enhanced self-centering capabilities. These analyses followed the concepts presented above for modeling of segmental systems with 2-node elements, and they seem to have provided reasonable results.

Significant information on the seismic behavior of precast segmental systems following the design concepts described earlier is expected to be obtained from quasi-static and shake table testing of the bridge specimen described earlier. The tests will be conducted in the SEESL of the State University of New York at Buffalo, U.S.A.

5. ACKNOWLEDGEMENTS

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Research and Application of Precast Segmental Concrete Bridge Columns in Taiwan

Kuo-Chun Chang[1], Yu-Chen Ou[2], Mu-Sen Tsai[3], and George C. Lee[4]

ABSTRACT

This paper presents the design concepts and connection details of precast segmental concrete bridge columns investigated in two experimental studies and implemented in a highway project in Taiwan. Post-tensioning was used to assemble the columns. The two experimental studies showed that precast segmental columns with or without mild steel reinforcing bars across the precast joints (also referred to as energy dissipation bars, ED bars) and allowing nonlinear behavior of precast connections can be designed for ductile and self-centering behavior under earthquake loading. Current application of precast segmental columns in Taiwan uses CIP hinge region and seismic isolation. The limit state of the columns is not controlled by nonlinear behavior of precast connections.

INTRODUCTION

In recent years, growing attention has been given to the investigation, development and application of precast concrete bridge elements and systems for highway bridges. Cast-in-place concrete bridge construction normally causes traffic disruption for long periods of time. Precast concrete bridge construction offers a viable solution to this problem, by shifting most of the construction activities into precast factories where quality control is more reliable, thus minimizing traffic disruption and improving construction quality. Reducing on-site construction activities also means that work zone safety can be improved and environmental impact reduced (TRB 2003). Another advantage is ease of construction, particularly for bridges over waterways and across mountains, because on-site temporary support work, formwork and reinforcing work are reduced. Precast segmental bridge columns have been used in many bridge construction projects in regions of low seismicity in the U.S. (e.g. Billington et al. 1999 and Figg and Pate 2004). Recent applications were found in the Victory Bridge in New Jersey and the Colorado River Bridge of Hoover Dam Bypass in Nevada. This paper presents the design concepts and connection details of precast segmental concrete bridge columns that were examined in two experimental studies and those implemented in a highway project in Taiwan.

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TALL PRECAST SEGMENTAL CONCRETE BRIDGE COLUMNS FOR HIGHWAY BRIDGES IN TAIWAN

Background

This research was funded jointly by the Taiwan Area National Expressway Engineering Bureau and the National Center for Research on Earthquake Engineering (NCREE) in Taiwan. The objective of this research was to investigate the use of precast segmental construction in tall concrete bridge columns in a highway to be built in areas with beautiful mountains and coastline to reduce disturbance to the environment. Emphasis was given to the seismic performance of the columns since Taiwan is a region of high seismicity. Details of this research can be found in Wang et al. (2008).

Connection Design Details

Four large-scale specimens were tested. Figs. 1 and 2 show the specimen design and section design, respectively. Post-tensioning was used to connect precast segments. Specimen P1 was post-tensioned with unbonded tendons. The tendon ducts formed a U-loop in the foundation. Plain concrete shear keys were introduced at the segment connections from segments S1 to S9. The purpose of the shear keys was mainly to provide an alignment guide during assembling. No shear key was used at connection foundation-S1 (the connection between the foundation and segment S1) to examine shear transfer at the critical connection only by shear friction. The mild steel longitudinal reinforcement was discontinued at the segment connections.

Test results of specimen P1 showed that it had much lower hysteretic energy dissipation capacity as opposed to conventional monolithic columns as expected. To improve this, specimen P2 was designed with 8 D36 high strength reinforcing bars at the critical section, which was located at connection foundation-S1. These bars are also referred to as energy dissipation (ED) bars herein to distinguish them from other steel reinforcing bars. These bars were anchored with T-headed threaded couplers embedded in the foundation. 4 of the 8 ED bars were extended through grouted steel corrugated ducts to the top of segment S2 and anchored with steel plates and nuts. The other 4 were continued to the top of segment S5 and anchored using the same method.

Test results of specimen P2 showed its hysteretic energy dissipation was increased marginally compared to specimen P1. One reason to this was attributed to the large opening of connection foundation-S1, which was considered detrimental because it limited the inelastic deformations of segment S1 (plastic hinge mechanism). Therefore, connection foundation-S1 of specimen P3 was strengthened to avoid opening. The base of segment S1 was enlarged and tied down to the foundation with D32 high strength steel bolts. In addition, the height of segment S1 was increased to 2 m to reduce the influence of expected opening of connection S1-S2 on the plastic hinge mechanism. In addition, mild steel bars with a lower yield strength (490 MPa) were used for ED bars in specimen P3. 20 ED bars were precast with segment S1 with one end protruding out of the bottom surface of the segment with a length of 200 mm. The protruding portion of the bar was later inserted into grouted coupler embedded in the foundation. In this specimen, the critical section was located at connection S1-S2 (the connection between segments S1 and S2). 12 of the 20 bars were extended across this connection. 4 of the 12 bars were extended through grouted steel corrugated ducts to the top of segment S3 and the other 8 bars continued to the top of segment S5 based on the moment capacity-demand diagram. This specimen did not have shear keys because it was found that relative shear slip between the segments and the alignment of the segments during assembling without shear keys did not cause any problem.

Specimen P3 demonstrated a good hysteretic energy dissipation capability during testing. Thus, specimen P4 was designed using the same design concepts as specimen P3 but with a different method of constructing segment S1. In specimen P4, 12 ED bars were precast with segment S1 with one end protruding out of the bottom surface of segment S1 with 90 degree bents plus 612 mm extension, which together with the bottom 300 mm portion of segment S1 were later cast-in-place with the foundation. This is a partially-precast method, as compared to fully-precast methods used in the previous specimens, since fresh concrete was still needed during assembling.

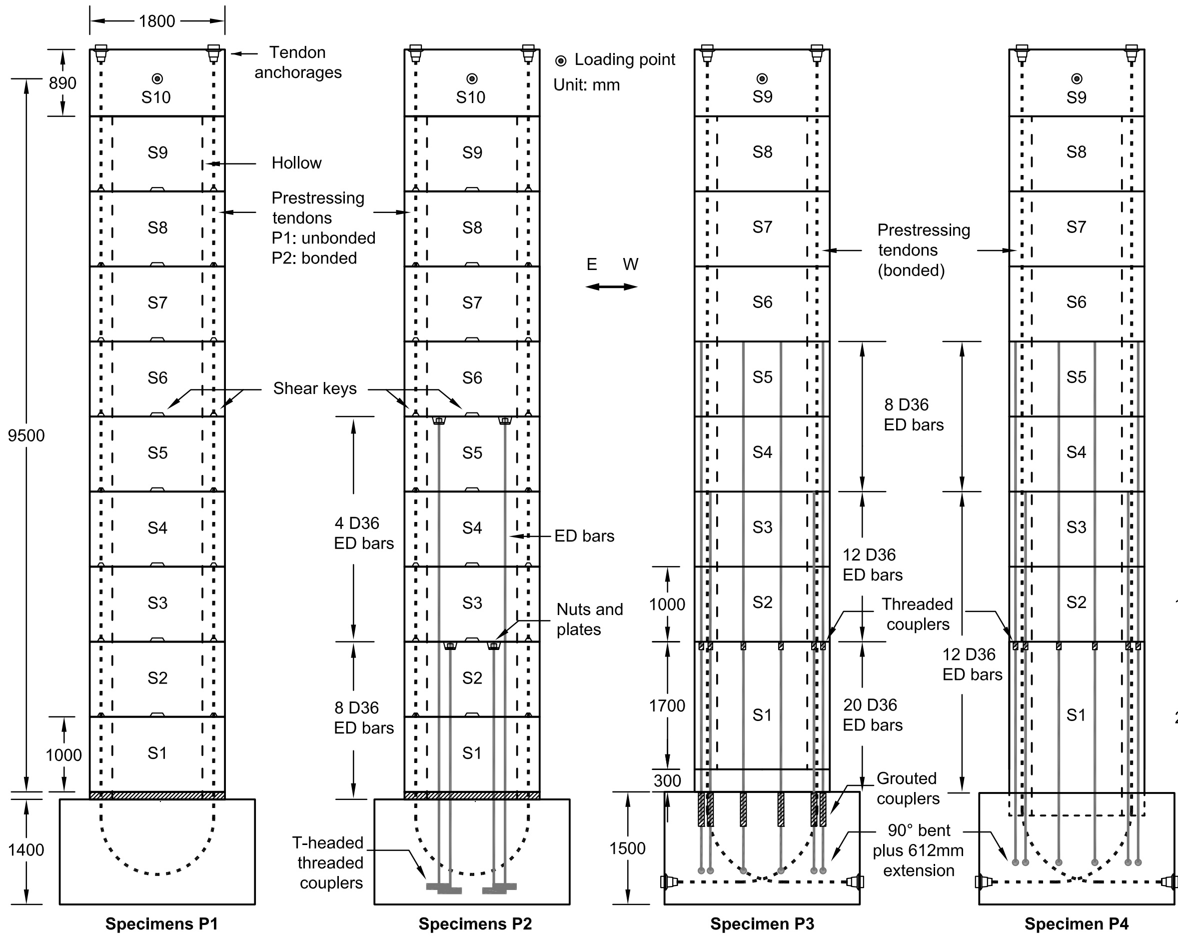

Figure 1. Column design (Front elevation view).

Test Results

All the specimens exhibited satisfactory ductile behavior as shown in Fig. 3. No shear-slip failure occurred at the segment connections despite the fact that a significant amount of connection opening was observed in all the specimens during testing. Specimen P1 had an energy dissipation capacity that was much lower than the other specimens. However, this specimen could self-center to its original position at the end of testing with minor damage. Specimens P3 and P4 demonstrated ductile behavior and significantly higher hysteretic energy dissipation capacity than specimen P1.

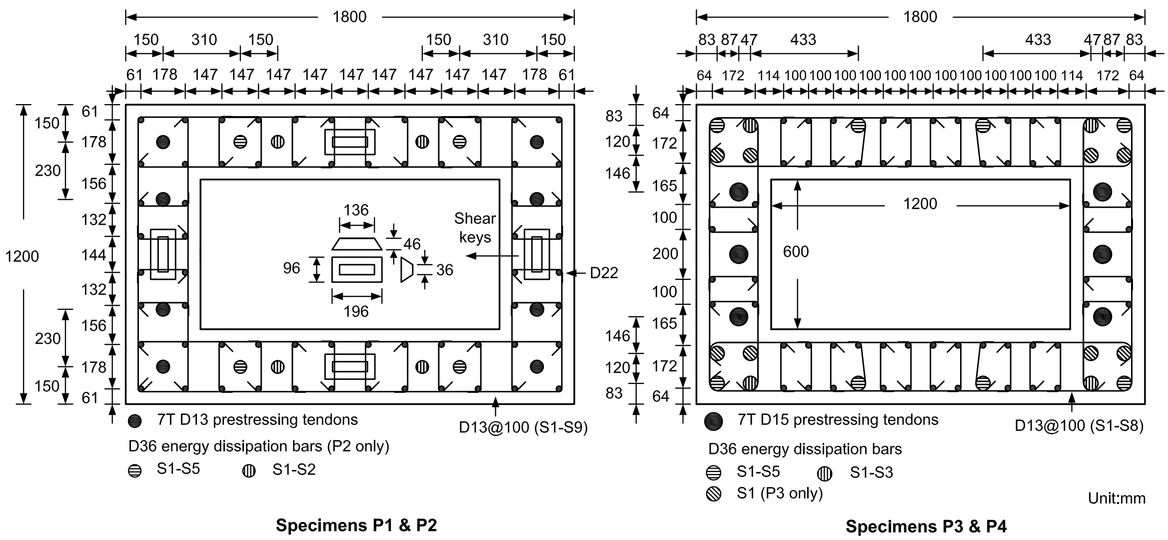

Figure 2. Section design.

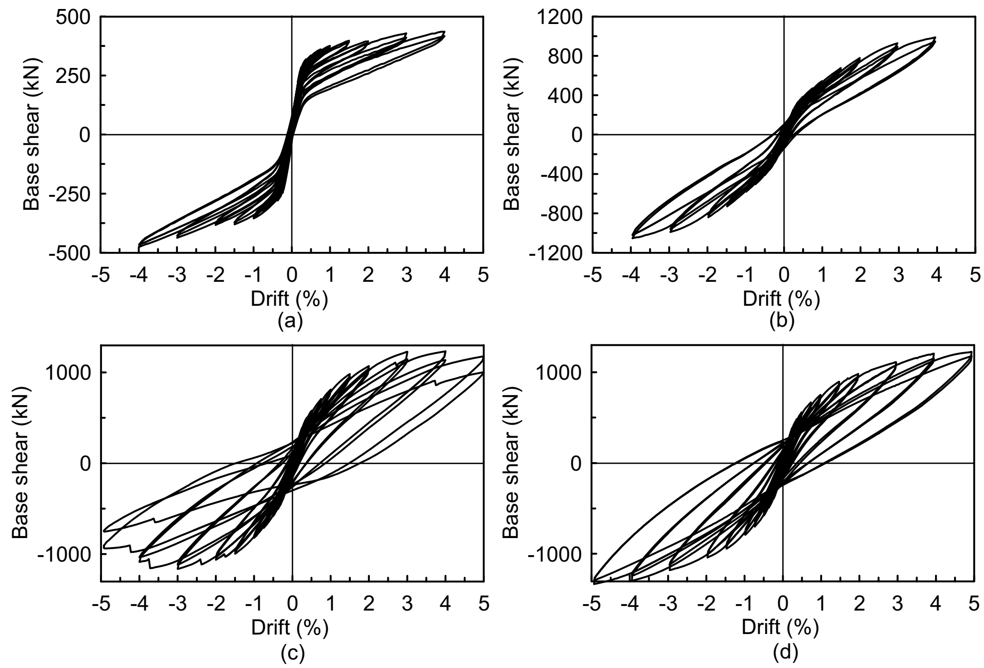

Figure 3. Base-shear versus drift behavior: (a)P1, (b) P2, (c)P3, and (d)P4.

SELF-CENTERING PRECAST SEGMENTAL CONCRETE BRIDGE COLUMNS

Background

This research was part of the Accelerated Bridge Construction (ABC) project funded by the Federal Highway Administration (FHWA) to the Multidisciplinary Center for Earthquake Engineering Research (MCEER) on the U.S. side and funded by Taiwan's National Science Foundation to NCREE on the Taiwan side. It aimed at developing precast segmental concrete bridge columns that have enhanced energy dissipation capacity while maintaining self-centering capability. More information on this research can be found in Ou et al. (2007) and Ou et al. (2009).

Specimen Design

Four specimens were tested. Major design details of the specimens are illustrated in Fig. 4. Post-tensioning was used to connect precast segments. The post-tensioning tendons were placed inside the hollow core of the column same as external unbonded tendons. The use of unbonded tendons reduces the loss of prestress forces as compared to bonded tendons. No shear keys or epoxy was provided at the interface of the adjoined segments for the purpose of shear transfer.

Specimen C0C was designed without any ED bars. Specimens C5C and C5C-1 were designed with 0.5% ED bar ratio (12 D16 bars), which was intended to produce a flag-shape hysteretic behavior with a significant amount of energy dissipation and a small residual displacement. Specimen C5C-1 had a lower post-tensioning force than specimen C5C. The ED bar ratio in specimen C8C was further increased to 1% (12 D25 bars). Eq. (1) defines a factor, λ_{ED}, which is related to the ED bar contribution to the expected column strength. λ_{ED} for specimens C5C, C5C-1 and C8C was calculated to be approximately 28%, 35% and 50%, respectively.

$$\lambda_{ED} = \frac{V_{exp} - V_{exp0}}{V_{exp}} \tag{1}$$

where V_{exp} =maximum expected shear demand of the specimens; and V_{exp0} =maximum expected shear demand not considering the contribution from the ED bars. To avoid premature low cycle fatigue failure of the ED bars at the critical connection, which was located at the base of the column (connection Found-S1), due to the repeated opening and closing of the connection, an additional unbonded length, L_{au}, was introduced to the bars from the face of the foundation extending into it (see Fig. 4). This length was created by wrapping the bars with duct tape. The performance objective of L_{au} was set to prevent low cycle fatigue failure of the ED bars under cyclic loading up to 5% drift. The procedure to determine the unbonded length L_{au} can be found in Ou et al. (2009).

Test Results

Test results (see Fig. 5) showed that the proposed columns possessed excellent drift capacities that are adequate for use in regions of high seismicity. The specimens without and with the ED bars exhibited drift capacities of 4.6% and 5%, respectively. The former failed mainly due to the P-delta effect and the latter due to fracture of the ED bars. The hysteretic energy dissipation capacity and residual drift of the columns increased as λ_{ED} increased. The specimens with λ_{ED} of 28% and 35% exhibited flag-shape hysteretic behaviors with significant amounts of hysteretic energy dissipation while maintaining small residual drifts. The specimen with λ_{ED} of 50% showed a further improved energy dissipation capacity but had a maximum residual drift that was much higher than the other specimens. Based on the test results, λ_{ED} of more than 35% is not recommended for maintaining self-centering capability.

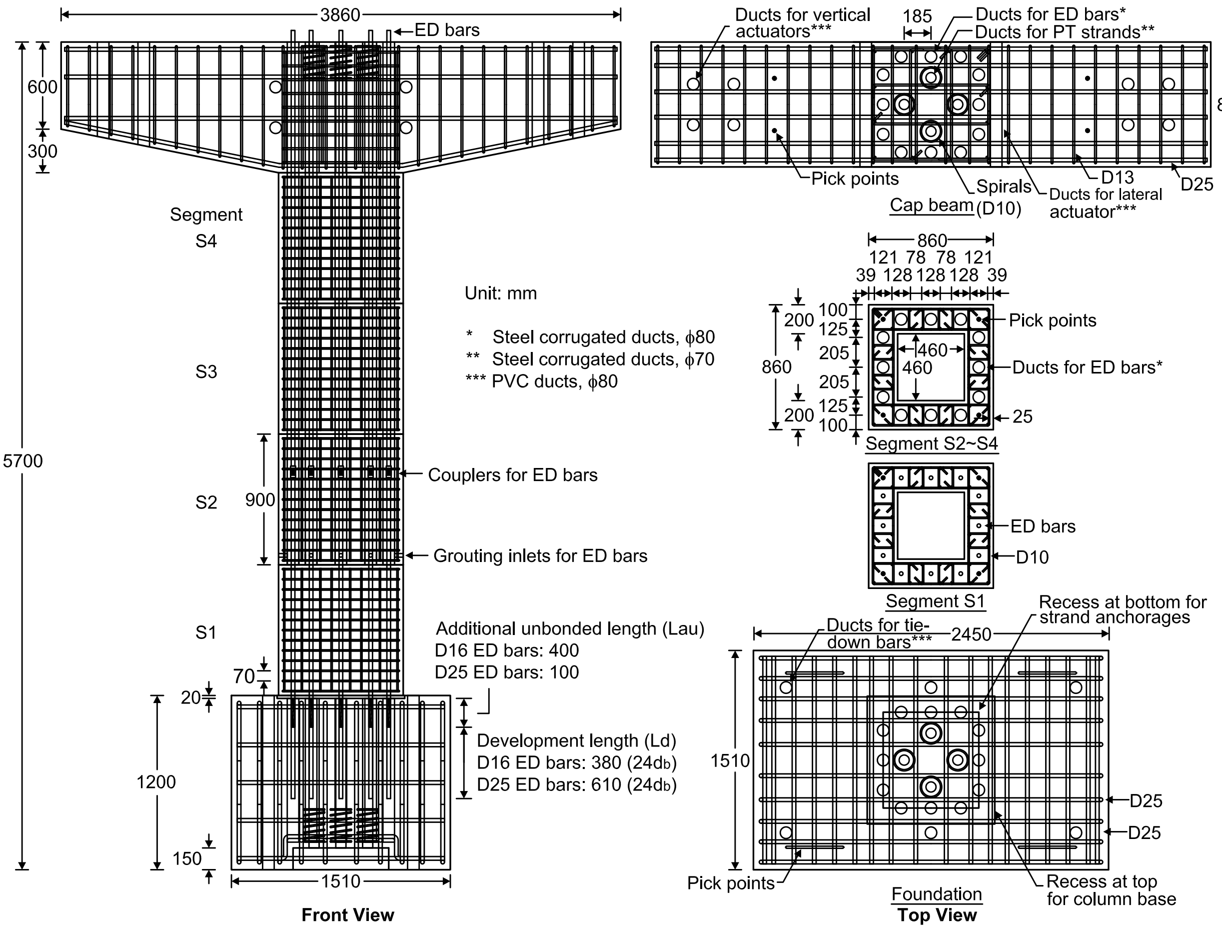

Figure 4. Specimen design

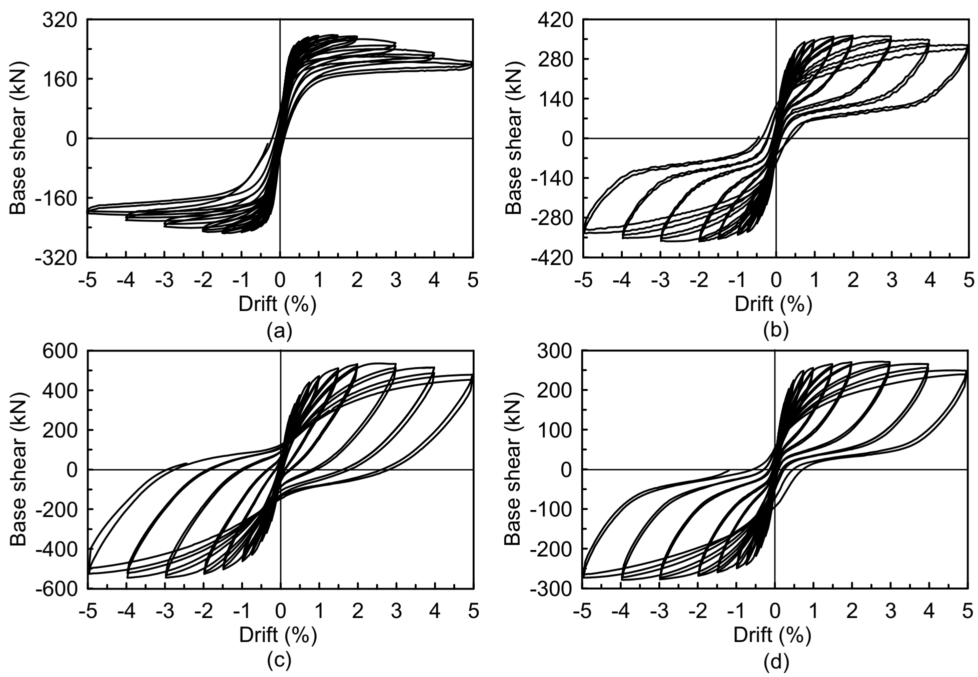

Figure 5. Base-shear versus drift behavior: (a)C0C, (b)C5C, (c)C8C, and (d)C5C-1.

APPLICATION OF PRECAST SEGMENTAL BRIDGE COLUMNS IN TAIWAN

Two of the elevated portions of a highway in central Taiwan were designed with precast concrete segmental columns. One elevated portion was designed with tall segmental columns with heights ranging from 17 to 31 m (see Fig. 6) and the other portion with relatively short segmental columns with heights ranging from 8 to 14 m (see Fig. 7). Post-tensioning was used for both types of columns. For the tall columns, the potential plastic hinge region was cast-in-place (CIP). The level of the post-tensioning force was chosen so that the precast connections would not open during a design earthquake (475 return period). The post-tensioning tendons formed a U-loop in the CIP region a distance above the base of the column. The ultimate state of the column was designed at the section at the base of the CIP regions. Thus, the column is expected to behave similarly to a conventional monolithic column at the ultimate state. For the short columns, lead rubber seismic isolation bearings were used to reduce the seismic demand. The post-tensioning force was selected so that under the forces of the bearings the precast connections would not open. The purpose of the shear keys at precast connections in both types of columns was to facilitate assembling process.

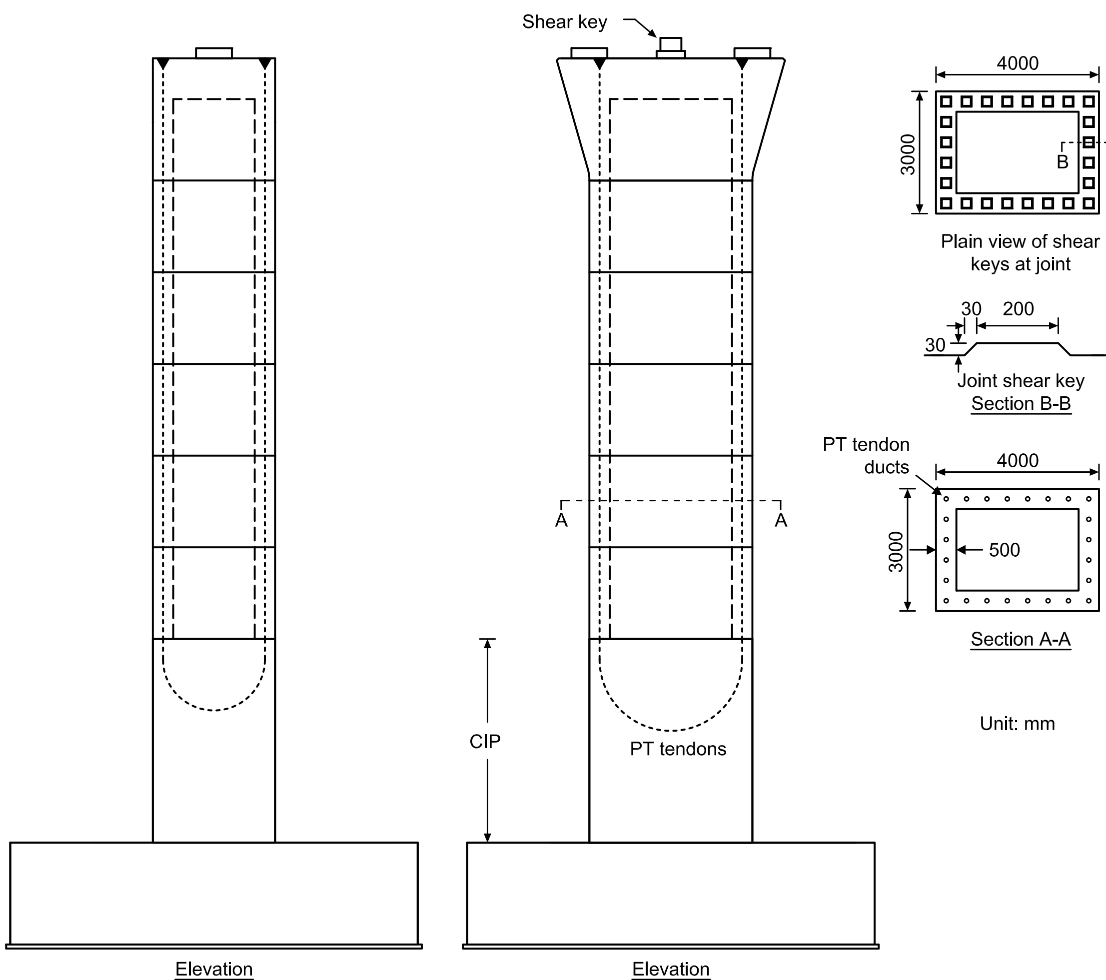

Figure 6. Tall column design

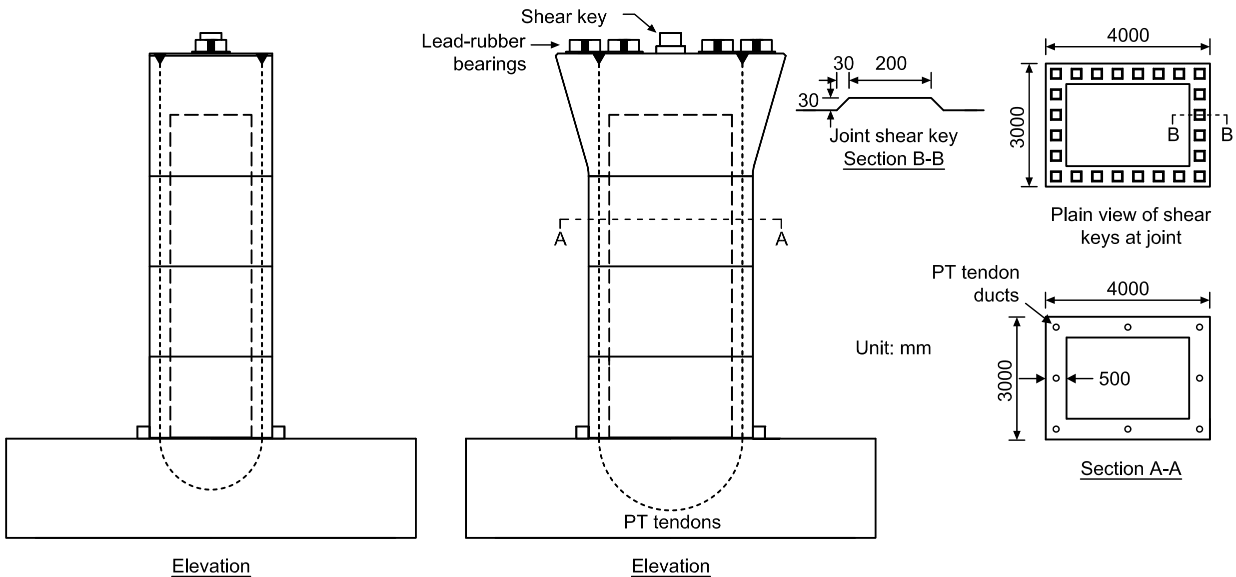

Figure 7. Short column design

CONCLUSIONS

The following conclusions are drawn from the study presented herein.

(1) Experimental studies carried out by the authors have shown that precast segmental columns assembled by post-tensioning and allowing nonlinear behavior of precast connections (with or without ED bars) can be designed for ductile and self-centering behavior under earthquake loading.

(2) Current application of precast segmental columns in Taiwan uses the concept where limit state of the column is not controlled by nonlinear behavior of precast connections (CIP hinge region and seismic isolation). The behavior of the columns is expected to be similar to conventional monolithic columns during a design earthquake.

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A Precast Concrete Bridge Bent for Seismic Regions

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ABSTRACT

This paper describes a precast concrete bridge bent connection that is suitable for rapid construction in seismic zones. Its features include speed and simplicity of erection, generous tolerances and good seismic performance. It was been developed with the help of structural designers, a precast concrete fabricator a general contractor and the state Department of Transportation. It uses a different connection system at the top and bottom of the column. Lateral load tests on the top connection system have shown that it has strength and ductility similar to those of a comparable cast-in-place connection. Tests on the bottom connection system are ongoing.

INTRODUCTION

Bridge construction frequently leads to traffic delays, which incur costs that can be measured in time, money and wasted fuel. Agencies are therefore seeking method for accelerating bridge construction, referred to as ABC. Such methods offer not only for relief to traffic delays, but also reduced environmental impacts, better worker safety, higher quality construction and lower lifecycle costs (Wacker et al. 2005). Use of precast concrete represents promising technology for ABC, and has been successfully used for bridge substructures in non-seismic regions (Matsumoto et al. 2001). Connections are typically made at the beam-column and column-foundation interfaces to permit the use of straight elements. This facilitates fabrication and transportation. However, for structures in seismic regions, those interfaces represent the locations of high moments and large inelastic cyclic strain reversals. Devising connections that are not only sufficiently robust to accommodate those cyclic loads, but are also readily constructible, represents a challenge.

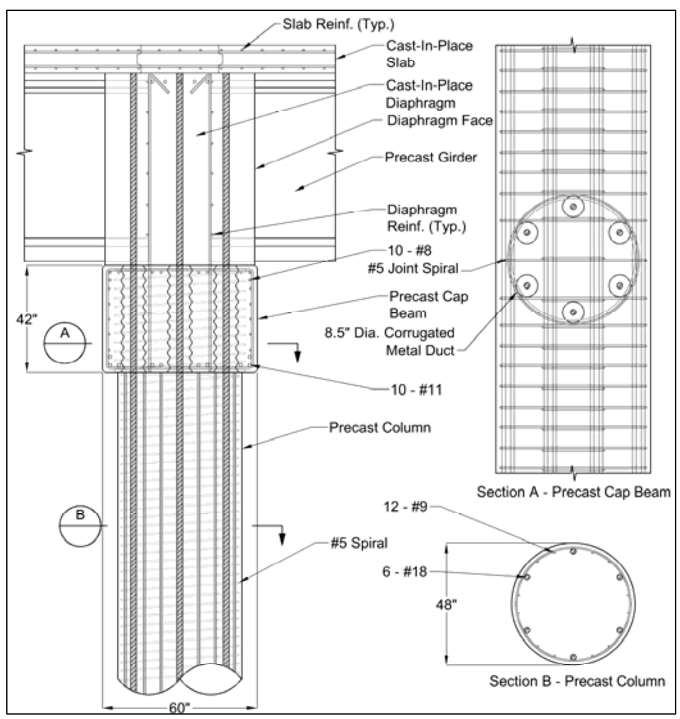


Figure 1 – Details of Full Scale Connection

The precast concrete bridge bent system described in this paper is intended to satisfy the combined needs of seismic performance and constructability. Different connection systems are used at the column-to-foundation and the cap beam-to-column interfaces, because the conditions at each location offer unique opportunities. The cap beam connection has already been tested, and has proved to be very satisfactory. Results from those tests are presented first. The foundation connection has been designed and, at the time of writing, the test specimens are being cast. Testing is due to be complete by the spring of 2010.

While the concepts were developed and tested at the University of Washington, significant assistance was provided by the Washington State Department of Transportation, Tri-State Construction, Concrete Technology Corporation and Berger/ABAM Engineers.

BEAM-TO-COLUMN CONNECTION

CONCEPT

The connection concept consists of bars that project from the column and are grouted into ducts in the cap beam. This concept has been used before by others (e.g. Getty, *website*). Such designs have used a conventional arrangement of longitudinal reinforcement, consisting of, say, 18 D35 bars in a 1.5m diameter column (18 No 11 bars in a 5'-0" diameter column). In such a configuration, the vertical ducts need to be quite small in order to fit between the horizontal beam bars. Then, fitting the 18 column bars into 18 relatively small ducts in the cap beam requires very accurate fabrication and field assembly. It does not constitute a readily constructible solution.

The system developed here is shown in Figure 1. It uses a small number of large bars in order to permit larger ducts and thereby facilitate the fit-up on site. In a typical 1.5m (5'-0") circular column, 8 D57 (No. 18) bars in 200 mm (8") diameter ducts will often suffice. The major obstacle then lies in anchoring the bars in the available space, because the AASHTO LRFD Specifications require a development length in 28 MPa (4000 psi) concrete of 2.75m. (9ft), whereas the cap beam is typically only about 1m (3'-6") deep.

Two approaches may be taken to solve this problem. The first is to demonstrate that the anchorage available from the grouted ducts is sufficient to develop the bar. The second is to note that, in a typical dropped cap beam design, such as the one used here, a diaphragm is cast in place over the cap beam once the girders have been placed and the deck has been cast. That diaphragm, shown in Figure 1, provides additional anchorage length that can be used together with the anchorage in the ducts to resist the cyclic forces on the bar caused by seismic loading. The ducts alone then need to provide only enough development to ensure strength and stability during erection. It is likely that development to resist the yield stress under static loading, rather than the ultimate strength under cyclic loading, would be enough for that purpose.

TESTING

Bar Anchorage

The anchorage characteristics of large bars were first investigated by conducting full scale pull-out tests on D32, D45 and D57 (No.10, 14 and 18) bars grouted into corrugated ducts. These tests confirmed that the D57 bars could be fully anchored in a length significantly shorter than the depth of the cap beam (Steuck et al. 2007). Those tests combined with earlier work (Raynor et al., 2002; Precast, 2004; Moustafa et al. 1974) showed that, under static loading, the lengths needed to provide anchorage for yield and fracture were approximately $6d_b$ and $10d_b$ respectively. An additional allowance of 50% was suggested to account for cyclic loading. The 50% is believed to be conservative, and is supported by a small number of Raynor's tests that were conducted cyclically, and by reference to Chapter 21 of ACI 318-08, which requires an increase of approximately 30% for the development length of bars cast directly in concrete. Even with that 50% increase in l_d, the bars can be anchored within the depth of the cap beam so that, when they eventually fail under cyclic loading, it is by fracture rather than pullout.

Connection tests

Table 1 - Test Matrix

Specimen	**Description**	**ρ (%)**	**Longitudinal Reinforcement**	**f'_{co} (ksi)**	**P_{axial} (kips)**	**α_{axial} (%)**
REF	Reference cast-in-place reinforced column	1.58	16- #5	6.83	240	11.2
LB-FB	Precast column with bars fully grouted in corrugated ducts in beam	1.51	6- #8 (12- #3 for spacing)	8.34	212	8.1
LB-D1	LB-FB with bars debonded 8 d_b in the grouted ducts using Method 1	1.51	6- #8 (12- #3 for spacing)	7.69	260	10.8
LB-D2	LB-FB with bars debonded 8 d_b in the grouted ducts using Method 2	1.51	6- #8 (12- #3 for spacing)	6.20	240	12.3

Four scaled sub-assemblage tests were also conducted to evaluate the seismic performance of the proposed connection. The primary study variable was the anchorage of the longitudinal bars. In two specimens, those bars were debonded over a short length near the beam-column interface, to reduce the strain concentration that might otherwise occur there because the bond resistance provided by the grouted duct is very high.

Each specimen was a 40% scale sub-assemblage of the proposed beam-to-column connection. The full-scale prototype was assumed to be a 1.2m (4'-0") diameter circular column. Table 1 and Figure 2 show the details of the scaled test specimens, which included a 445mm (17 ½-in) deep cap beam, a 470 mm (18 ½-in) deep portion of the diaphragm, and a 1.5m (60-in) tall segment of the 500 mm (20-in) in diameter column. Specimen REF is a scaled model of a typical Washington State cast-in-place bridge column, with 16 D16 (16 No. 5) bars evenly distributed round the perimeter, giving a longitudinal reinforcement ratio, ρ, of 1.58 percent. The column bars were cast in place into the cap beam and diaphragm. The transverse reinforcement consists of 6 mm (1/4-in) diameter spirals spaced at 30 mm (1¼-in). This specimen provides a baseline for evaluating the performance of the proposed system.

The other three specimens, LB-FB, LB-D1, and LB-D2, represented possible variations of the proposed precast connection. The columns were reinforced longitudinally with 6 D25 (#8) bars that were grouted into 100 mm (4-in) diameter corrugated metal ducts in the cap beam and further anchored in concrete within the diaphragm, providing a longitudinal reinforcement ratio, ρ, of 1.51 percent. In Specimen LB-FB the bars were fully bonded into grouted ducts, whereas in specimens LB-D1 and LB-D2, two methods of local debonding were studied. The bars in LB-D1 and LB-D2 were debonded over a length of 8 bar diameters, d_b, into the cap beam using two different methods. LB-D1 was debonded using a 25 mm (1-in) SCH-40 PVC pipe slit longitudinally, fitted tightly around the #8 bar, taped together, and sealed with caulk at the ends. LB-D2 was debonded using a 25 mm (1-in) SCH-30 PVC pipe. That pipe fitted more loosely round the bar, and provided no restraint against buckling. The detail was constructed by sliding the pipe over the bar and sealing it with caulk at the ends.

For all precast specimens, 12 D10 (#3) bars were added to the column. They stopped at the interface and so provided no additional flexural capacity; they were included purely to meet AASHTO bar spacing requirements. The spiral reinforcement in the columns was the same as in the reference specimen, and it continued at the same spacing into the cap beam to confine the joint region. A thin grout pad was also provided at the beam-column interface to simulate field erection of the precast pieces. Fluid, high-strength, non-shrink grout with an average compressive strength of 63 MPa (9 ksi) at 5 days was used. 420 MPa (gr. 60) bars were used for the mild steel reinforcement, while 630 MPa (gr. 90) wire was used for the spirals. The design concrete strength was 42 MPa (6 ksi).

Experimental Setup

Each specimen was tested under constant axial load and a cyclic lateral displacement history. The test set up is shown in Figure 2. Axial load was applied via a 10 MN (2400 kip) Baldwin Universal Test Machine, equipped with a swivel head and a low-friction sliding PTFE track. Lateral displacements were applied using a 1 MN (220-kip) MTS displacement-controlled actuator, located 1.5 m (5 ft) above the interface. The lateral loading protocol was a slightly modified version of that recommended in the NEHRP Recommended Provisions (Building, 2004), and consisted of 3 cycles at each of a series of increasing displacements.

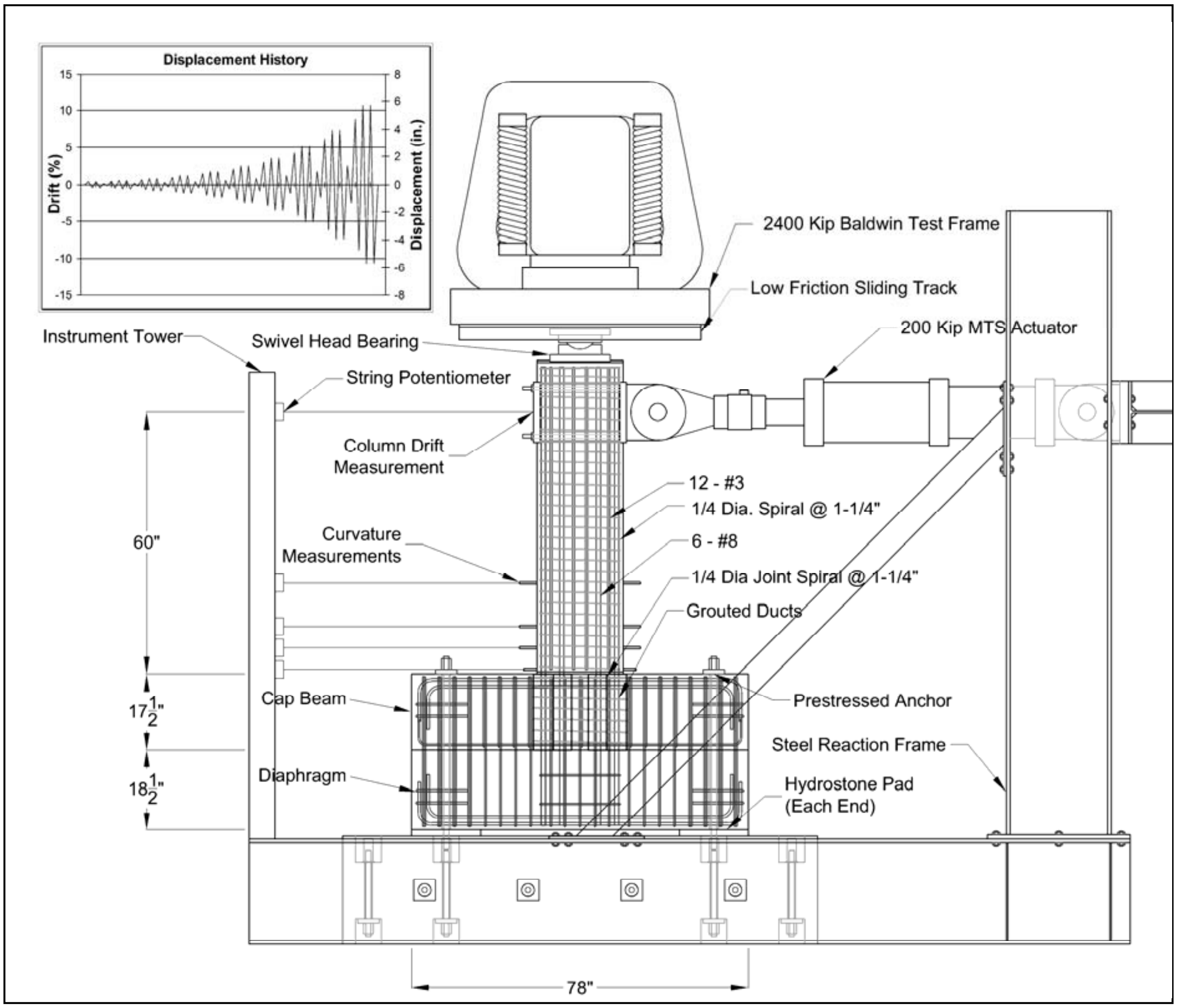


Figure 2 - Test Setup, Showing Precast Connection

Experimental results

All four specimens demonstrated nearly identical force-displacement responses and levels of physical damage. Specimens REF, LB-FB, LB-D1, and LB-D2 maintained 80 percent of their peak lateral resistance out to drifts of 5.5%, 5.2%, 5.7%, and 5.8%. All of these values greatly exceed the drift demands expected in even a severe earthquake.

Figure 3 shows the load-deflection curves for each specimen. They are expressed as Equivalent Moment vs. drift ratio, where the equivalent moment consists of the overturning effects of both the lateral load acting at the specified height and the vertical load multiplied by the lateral displacement. The drift ratio is the horizontal displacement divided by the height to the loading point. The curves are remarkably similar, despite the different construction methods. The slightly different peak loads are mainly attributable to minor differences in the applied axial load. There is a small amount of pinching as the system crosses zero displacement, caused by the fact that the compressive force is resisted by the bars alone prior to closing of the cracks in the concrete. There is also little loss in strength until the drift reaches at least 4 percent, despite the increasing damage accumulating in the plastic hinge region.

The majority of deformation for specimens LB-FB, LB-D1 and LB-D2 resulted from a large localized crack opening at the interface. Rotations measured over the bottom 38 mm (1.5 in) of the column accounted for more than 90 percent of the total column displacement. The additional 12 D10 (#3) bars in the column, which did not cross the interface, made the body of the column stronger and stiffer than the interface, so most of the deformation was forced to occur at the interface and the precast members essentially rotated as rigid bodies. This approach, in which the connection is designed to behave in a ductile manner during an earthquake, has been successfully used in precast building design and tested in the PRESSS Program (5). In contrast, in Specimen REF, the curvature was more evenly distributed over the bottom 1 m (40 in) of the column, as is common in cast-in-place systems. The width of the interface crack was about the same as that of some of the flexural cracks above.

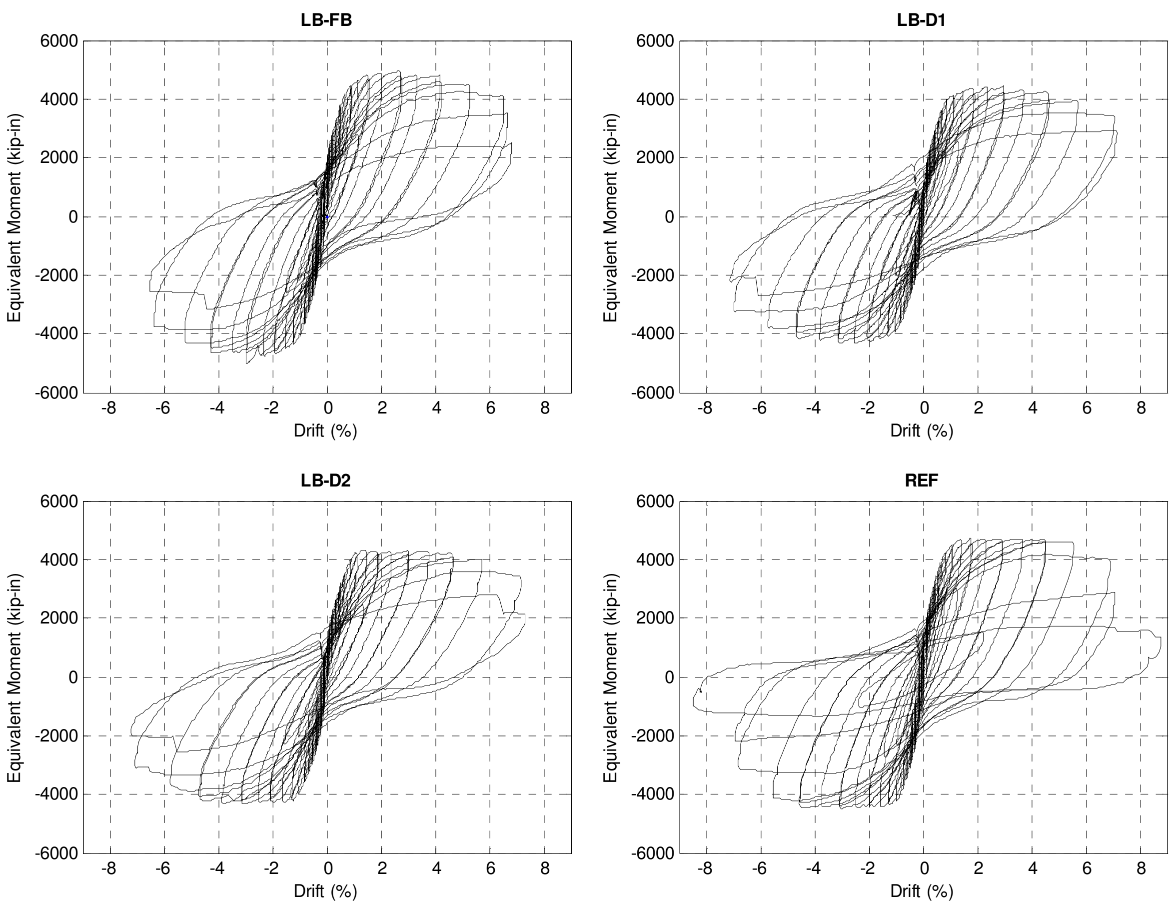
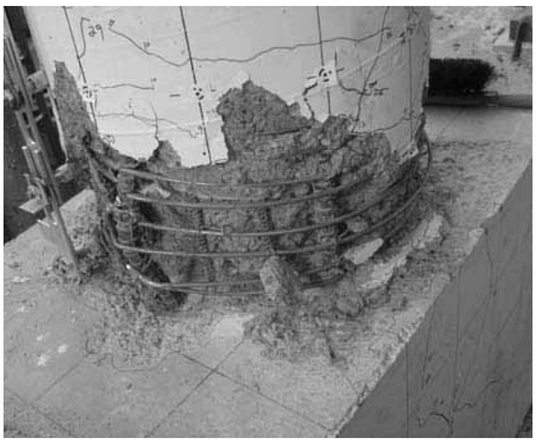


Figure 3 - Equivalent Moment vs. Drift Ratio. (*Note: 1 kip-in = 0.113 kN-m*).

Observed Damage

The types and amount of physical damage were nearly identical for all specimens, including Specimen REF, which had a different configuration of reinforcement and a slightly larger reinforcement ratio. Figure 4 shows the damage, including spalling of the concrete cover, damage to the cap beam, bar buckling, spiral fracture, and longitudinal bar fracture. Damage consisted of moderate spalling of the concrete cover and crushing of the core concrete over a plastic hinge region about 12 in. long. Spalling initiated at drift levels of 2.0%, 2.0%, 2.4%, and 2.1% for specimens REF, LB-FB, LB-D1 and LB-D2, respectively.

At drift levels of 5.6%, 5.3%, 5.7%, and 5.8%, respectively, the two bars at the extreme north and south locations began to buckle, pushing out on the spiral. The spirals fractured when buckling was first observed, or



(a) LB-FB cumulative spalling damage to beam.

(b) LB-D1 level of spalling at initiation of bar buckling with no damage to beam.

(c) LB-D2 bar buckling and spiral fracture.

(d) LB-D2 spiral and bar fracture, and core crushing.

Figure 4. Observed Damage

shortly thereafter. Bar buckling is shown in Figure 4c. Severe necking occurred before the spiral fractured. When the bars were next loaded in tension, they straightened and fractured. The finding that buckling occurred at almost the same drift in each specimen was surprising, given that in two specimens the bars were debonded over a length of 200 mm (8 in), while in the other two specimens they were fully bonded. The buckling always occurred in the column, whereas the debonded region was in the cap beam.

Bars in specimens LB-FB, LB-D1 and LB-D2 fractured at drift levels of 6.5%, 7.1%, and 7.4%, respectively. Bar fracture was brittle with no visible necking, and occurred approximately 6" above the interface, at the peak of the buckled shape. Figure 4d shows bar fracture and damage in the hinge region. Specimen REF underwent slightly larger drifts before fracture occurred. Three bars on the north and two bars on the south fractured at a drift of 8.8%. It was observed that a partially fractured bar showed cracks propagating from the inner side of the buckle. This, along with the lack of necking, indicates that bar fracture occurs as a consequence of bar buckling instead of strain concentrations at the interface.

Effects of Debonding

In specimens LB-D1 and LB-D2 the bars were intentionally debonded to reduce the maximum strain at the interface. Deformation in the bar was thus distributed over the debonded length rather than being concentrated at the interface crack. The debonded region was placed in the cap beam for several reasons. First, the joint region constitutes a large, relatively rigid, block of concrete that provides restraint to local bar buckling, which is otherwise a possibility in a debonded system. Second, the bond stresses needed for anchorage will develop deep inside the cap beam, rather than at the surface where they are more likely to lead to surface damage. Last, it is a more constructible alternative, as bars could be easily sleeved for debonding after casting and prior to erection.

Despite their different capacities for inhibiting buckling, the two details performed almost identically. This is confirmed by the similarity in their force-displacement curves in Figure 3. The primary reason was that the debonding was located in the cap beam, whereas the buckling deformations occurred in the body of the column, where the configurations of both specimens were identical.

The need for any debonding was also brought into question, because the attack end of the grouted bar pulls out a conical wedge of concrete from the duct. This region then behaves as an intentionally debonded region and relieves the majority of any strain concentration.

COLUMN-TO-FOUNDATION CONNECTION

CONCEPT

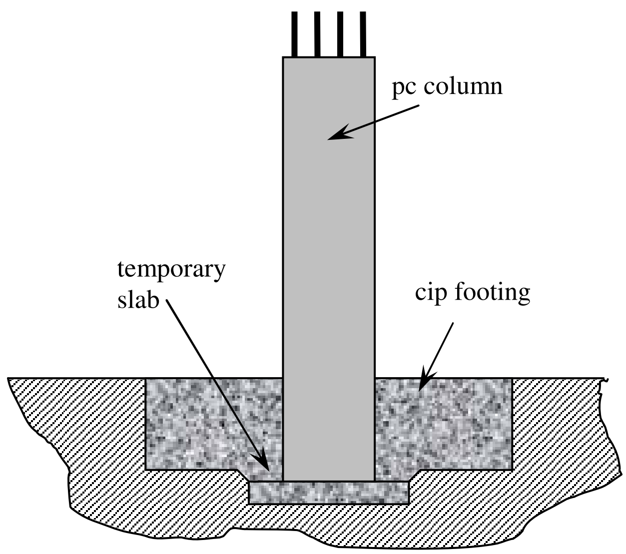


Figure 4. Socket Connection: Concept.

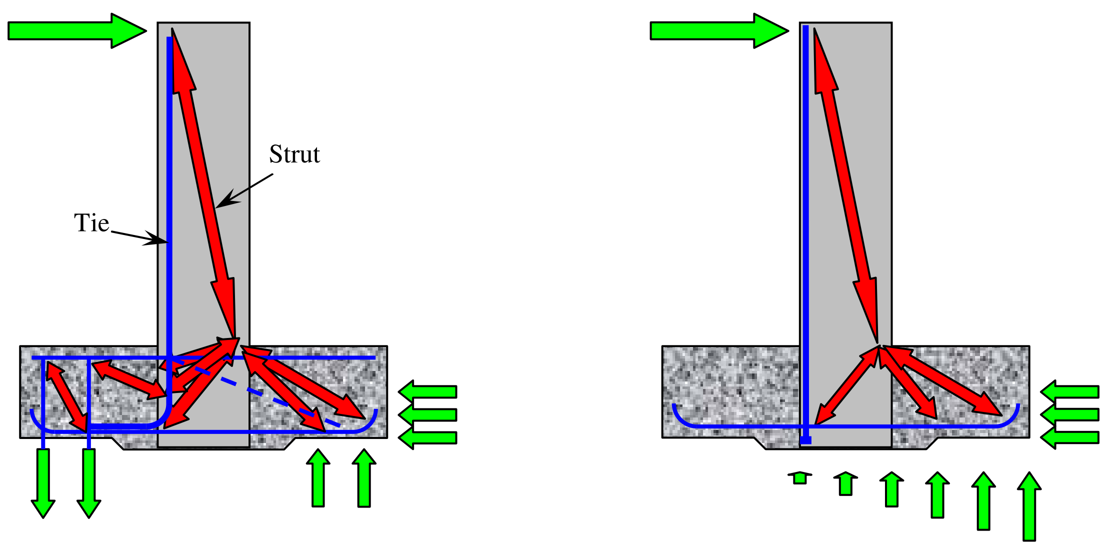
The concept adopted for joining the column to the footing is referred to as a socket connection, and is shown in Figure 4. It is suitable for use with spread footings. The construction sequence is as follows. First the column is precast, with a roughened surface in the region that will eventually be embedded in the footing. Then the foundation is excavated and, in the bottom, a small temporary slab is cast on which to set the column. The column is set, plumbed, leveled and braced, the footing steel is placed, and then the footing concrete is cast.

The constructability advantages of the system are that it is quick and simple to build, it avoids any potential problems of fit-up of bars in ducts, the column detailing can be almost identical to that of a cast-in-place column, and no grouting is needed.

A possible variant on the system is to cast the footing before setting the column, leaving a void in the footing into which the column can subsequently be set and grouted. While this alternative adds an extra step (grouting), it has the advantage that the top of the footing provides a solid surface to which the column braces can be attached.

The simplicity of the socket connection suggests that it could also be used at the beam-to-column interface, by creating a cap-beam with a large hole into which the precast column would fit. While this configuration is possible, the cap beam would have to be significantly wider than the column to provide sufficient beam strength at the connection. The weight of the cap beam, and the crane needed for lifting it, would have to be considered in evaluating the overall economy of the system. If the bridge girders are long and approximately the same weight as the cap beam, the crane required capacity may be controlled by the girders and the large cap beam may impose no real penalty.

The longitudinal reinforcement in the column is developed at the base by mechanical anchors, rather than the traditional method of bending the bars outwards. Doing so offers the construction advantage that the precast column becomes a large concrete cylinder with no protruding reinforcement, and is therefore simpler and safer to cast and transport. The headed bars also offer a much more direct transfer of forces between the column and the footing, as demonstrated by the strut-and tie models shown in Figure 5. Figure 5a (copied from Xiao et al. 1996) shows the model typically applied for bent-out bars. It requires a top mat and extensive stirrup steel in the footing. Furthermore the hook on the main bar is in effective for anchorage because it is facing the wrong way; the diagonal strut that joins it to the compression side of the column must transfer its load to the bar by bond rather than bearing, because its orientation is tangential, rather than radial, to the hook. The model proposed for use with the headed bars (Figure 5b) is, by contrast, very simple, and needs no top mat or tie steel, unless the "uplift" side of the footing does indeed lift off, in which case it should be reinforced to carry the cantilever moments due to the footing self-weight on that side. The headed bars thus offer advantages for both constructability and seismic performance viewpoints. This is quite unusual: more commonly a change that benefits one tends to constitute a disadvantage for the other.



(a) With bent out bars

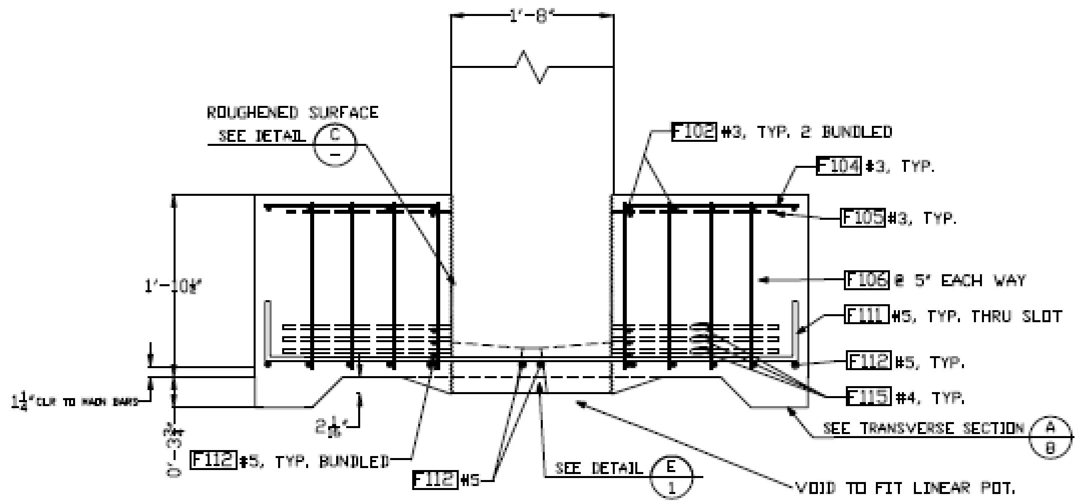
(a) With headed bars

Figure 5. Strut and tie models for (a) bent out bars and (b) headed bars.

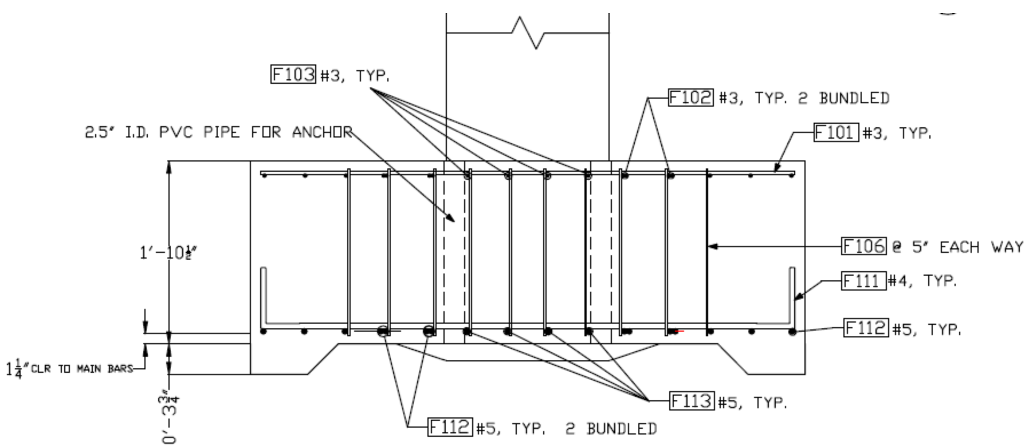
TESTING

The major questions associated with the connection's seismic performance are expected to be in the transfer of forces from the column to the footing, so the planned testing focuses on them. The connection must be able to resist the cyclic column moments without significant damage to the footing, and without the column punching through the footing under gravity load. Osanai et al. (1996) tested socket connections for building columns and concluded that, unless special conditions were satisfied, the footing depth should be at least 1.5 times the column diameter. However, the sockets they used were much smaller in plan than a typical spread footing for a bridge, so the present tests, which are based on a footing depth/column diameter ratio of 1.0, are expected to demonstrate that the connection is strong enough to induce a plastic hinge in the column itself, just above the footing, despite the relatively small footing depth. Ensuring that the yielding occurs in the columns is important, not only because it is advocated by the AASHTO Specifications, but also because post-earthquake inspection and repair are more expensive for footings than for columns.

The first two planned test specimens are shown in Figure 6. In both, the exterior of the column is roughened near the bottom to improve the transfer of shear stress. The column also extends just below the footing, to be sure that the force transfer at the node at the bottom of the column bars can take place satisfactorily.



(a) Specimen A (Conservative)



(a) Specimen B (Stripped down)

Figure 6. Socket Connection Test Specimens

Specimen A is the more conservative option. It is needed because the WSDOT has agreed to build a bridge over I-5 using the technology advocated here. The bridge is due to be bid before all the research test results are available, so a system with a very high probability of success had to be included. Two footing bars run through a slot under the column in each direction to ensure their direct engagement with the tension steel in the column. Sets of diagonal bars in the horizontal plane are placed in the top and bottom of the footing round the column. Their purpose is to act as "shear friction" steel that will provide the friction forces needed to prevent punching failure along the pc/cip interface. One set is placed in the top and three, stacked, in the bottom. Ties are included in accordance with the Caltrans recommendations (Caltrans, 2006). The AASHTO Guide Specification for Seismic Design (AASHTO Guide, 2009) is based on the Caltrans recommendations, but contains no tie requirements. The omission appears (Marsh, 2009) to be an oversight that is likely to be corrected, so the ties were included here in anticipation of their being required in future designs.

Specimen B was designed to determine whether the system could be simplified further. First, in each direction, the two center bars in the bottom mat of the footing steel were moved so they no longer passed under the column, but are placed just outside it. There they are bundled with the existing bar in that location. That placement frees the bottom mat to be placed at any time, thereby improving constructability. Second, the shear friction steel was reduced so that only one set of four bars was used at the top and bottom of the footing. The reasoning was that the bottom mat alone provides sufficient steel, provided that that steel can be used to provide both flexural strength and shear friction. (It should be noted that AASHTO allows some shear force to be resisted by a cohesive component of the concrete shear strength, but ACI 318 does not. In this case the cohesion component alone is theoretically sufficient to resist the axial force, which was taken as $0.1f'_cA_g$. The shear friction steel in Specimen A was designed conservatively using the ACI requirements, ignoring the cohesion component, to minimize any chance of vertical slip). One set of diagonal bars was retained at the top and bottom to act as "trimming" reinforcement at the corners of the square opening in the footing mat. Last, the tie steel was reduced by 50%, on the basis that the Caltrans recommendations appeared to be developed for a system in which the column steel was bent out, rather than being anchored by heads. In that case the strut and tie model suggests that tie steel is largely unnecessary.

CONCLUSIONS

A precast concrete bridge bent system is presented that is simple and rapid to construct and offers excellent seismic performance. The following conclusions are drawn:

1. The details have been developed with extensive input from a structural design engineer, the Washington State DOT, a precaster and a general contractor. The assistance from a range of disciplines was critical to achieving the constructability goals.
2. The column to cap beam connection is made with a small number of large (D57 or No. 18) bars column grouted into ducts in the cap beam. Their small number, and the correspondingly large ducts sizes that are possible, lead to a connection that can be assembled easily on site.
3. The column-to-cap-beam connection has been tested under cyclic loading in three different variations. All three displayed behavior that was essentially identical to that of a cast-in-place column with similar properties. All reached a drift of approximately 6% before bar buckling in the column precipitated failure.
4. The footing-to-column connection is to be tested in the near future. One conservative option and one stripped-down option are to be tested. Strut and tie analyses suggest that the conservative connection detail will be stronger than the column, and that failure will occur by plastic hinging in the column, as desired. The stripped-down detail uses less steel and is simpler to construct. The constructability benefits are clear, and it is hoped that its seismic performance will also prove adequate.

ACKNOWLEDGEMENTS

Support for this work was provided by the Federal Highway Administration, through their Highways for Life program, by the Washington State Department of Transportation, and by the Valle Foundation at the University of Washington. The findings and conclusions contained herein are those of the authors alone. Thanks are extended to graduate students Kyle Steuck and Laila Cohagen, Laboratory Manager Vince Chaijaroen, the staff of Tri-state Construction, Mr. Steve Seguirant of Concrete Technology Corporation, Dr. Bijan Khaleghi and Mr. Jugesh Kapur of the WSDOT, and Dr. Lee Marsh of Berger/ABAM Engineers for their valuable assistance, without which the project would not have been possible.

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Precast Bridge Members in Areas of High or Moderate Seismicity

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SYNOPSIS

Bridge construction with prefabrication of modular components offers an attractive alternative to conventional bridges. Prefabricated bridge components are in increasing demand for accelerated bridge construction. Precasting eliminates the need for forming, casting, and curing of concrete in the work zones, making bridge construction safer while improving quality and durability. Precast bridges consisting of pretensioned girders, post-tensioned spliced girders, trapezoidal open box girders, and other types of superstructure members are often used for accelerated bridge construction; however, bridge engineers are concerned with the durability and performance of bridges made of precast members in areas of high or moderate seismicity.

This paper examines the applicability of the AASHTO LRFD Specifications to precast prefabricated bridges in areas of high or moderate seismisity, discusses the different seismic design methodologies, and provides guidance in their application to precast bridges. It provides an overview of WSDOT design criteria and recent research and bridge projects using the accelerated bridge construction technique in Washington State.

The outline of WSDOT strategic plan for implementation of accelerated bridge construction is discussed herein. It introduces the innovation through Highways for Life project of a totally precast concrete bridge bent system that can be used in seismic regions.

INTRODUCTION

Precast concrete bridge systems provide effective and economical design solutions for new bridge construction as well as for the rehabilitation of existing bridges. The proper seismic design entails a detailed evaluation of the connections between precast components as well as the connection between superstructure and the supporting substructure system. In seismic regions, provisions must be made to transfer greater forces through connections and to ensure ductile behavior in both longitudinal and transverse directions. The system must be made to protect the superstructure from force effects due to ground motions through fusing or plastic hinging. In seismic regions, provisions must be made to ensure ductile behavior in both longitudinal and transverse directions.

The Federal Highway Administration (FHWA) has been actively promoting the advantages of ABC. Proven benefits include minimized traffic disruption, improved work zone safety, and reduced on-site environmental impacts. Related traffic impacts derive from both expedited congestion relief projects and minimized traffic disruption due to reduced on-site highway construction activities.

SEISMIC RESPONSE OF BRIDGES WITH PRECAST COMPONENTS

Plastic hinging is the basis of the ductile design for bridge structures. Plastic hinges may be formed at one or both ends of a reinforced concrete column. After a plastic hinge is formed, the load path will change until the second plastic hinge is formed. The philosophy of ductility and the concept of plastic hinging are applicable to precast bridges if connections are cast-in-place emulative.

The lack of monolithic action between the superstructure and bent cap in precast, prestressed concrete beam systems causes either the girder seats or the column tops to act as pinned connections. Consequently, while the transverse stability of multi-column bents is ensured by frame action in that direction, stability in the longitudinal direction requires the column bases to be fixed to the foundation supports. This requirement places substantial force demands on the foundations of multi-column bents, particularly in areas of moderate to high seismic zones. Developing a moment connection between the superstructure and substructure makes it possible to introduce a reduced fixity connection at the base of column

PRECAST SUPERSTRUCTURE

In Washington State, the use of prestressed I-girders started in the 1950's. Since then the Washington State Department of Transportation (WSDOT) has developed standard girders for composite and non-composite sections to facilitate economical design and construction. The complete description of standard prestressed girders and their span capability is presented in WSDOT Bridge Design Manual (BDM)[1], and PCI Journals[3,4] can be downloaded from the WSDOT website at: http://www.wsdot.wa.gov/eesc/bridge/index.cfm.

CONNECTION OF PRECAST GIRDERS AT INTERMEDIATE PIERS

The most common types of connections for precast prestressed girder bridges are fixed for high seismic zones (western Washington), and hinge connection for low seismic zones (eastern Washington). Precast column could be used if monolithic moment resistant connections meeting seismic design and detailing requirements are provided.

Monolithic action between the superstructure and substructure components is the key to seismic resistant precast concrete bridge systems. Lack of monolithic action causes the column tops to behave as pin connections resulting in substantial force demands on the foundations of multi-column bents, particularly in areas of moderate to high seismisity. Developing a moment connection between the superstructure and substructure reduces the moment in the footing. Fig.1 shows a typical monolithic moment resistant connection used for WSDOT precast girder bridges.

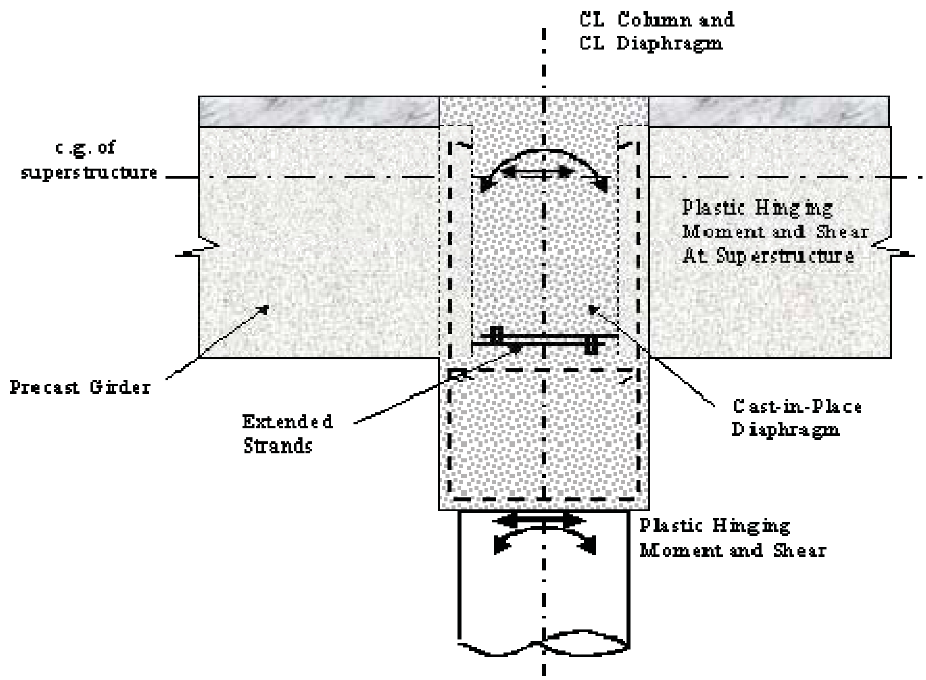

Fig. 1 Typical Monolithic Moment Resistant

The cast-in-place concrete for intermediate diaphragm of continuous bridges is completed in two stages to ensure the stability of precast girders after erection, and occurrence of initial creep. Extended strands and reinforcing bars are provided to ensure adequate performance of the connection during a seismic event. The design assumptions for fixed diaphragms are:

1. All girders of adjoining spans are the same depth, spacing, and preferably the same type.
2. Design girders as simple span for both dead and live loads.
3. Provide reinforcement for negative moments at intermediate piers in the deck due to live loads and superimposed dead loads from traffic barrier, pedestrian walkway, utilities, etc.
4. Determine resultant plastic hinging forces at centroid of superstructure.
5. Determine the number of extended strands to resist seismic positive moment.
6. Design diaphragm reinforcement to resist the resultant seismic forces at centroid of diaphragm.
7. Design longitudinal reinforcement at girder ends for interface shear friction.

STRAND FOR POSITIVE EQ MOMENT

The design procedure to calculate the required number of extended strands is described herein. This calculation is based on developing tensile strength of the strands at ultimate loads. Since the distance across the connection is too short to develop the strands by concrete bond alone, mechanical anchors are provided to develop the yield strength of the strands. Strand extension details with strand anchors and strand chucks are used for continuous spans at diaphragms is shown in Fig. 2.

Extended bottom prestress strands are used to carry positive EQ load, creep, and other restrained moments from one span to another. Strands used for this purpose must be developed in the short distance between the two girder ends. The strand end anchorage device used, per WSDOT Standard Plan, is a 2'-0" (610 mm) strand extension with strand chuck and steel anchor plate. The number of strands to be extended cannot be less than four.

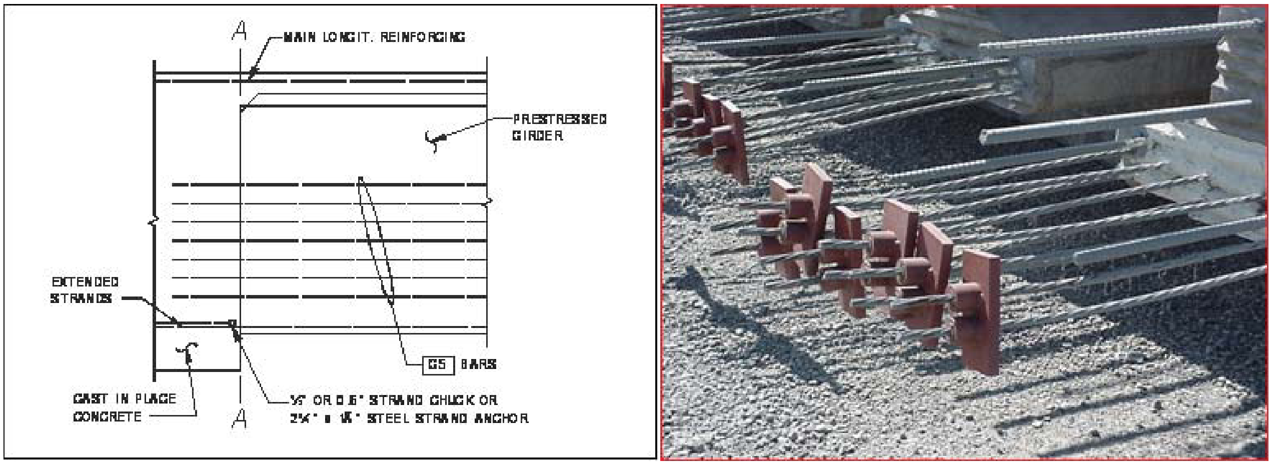

Fig. 2 Strand Extension Detail

The torsion in the bent cap is distributed into the superstructure based on the relative flexibility of the superstructure and the bent cap. Hence, the superstructure does not resist column overstrength moments uniformly across the width. To account for this, an effective width approximation is used, where the maximum resistance per unit of superstructure width of the actual structure is distributed over an equivalent effective width to provide an equivalent resistance.

Based on the structural testing conducted at the University of California at San Diego La Jolla[5], California in the late 1990's (Holombo 2000), roughly two-thirds of the column plastic moment to be resisted by the two girders adjacent to the column (encompassed by the effective width) and the other one-third to be resisted by the non-adjacent girders.

Number of extended straight strands needed to develop the required moment capacity at the end of girder is based on the yield strength of the strands.

$$N_{ps} = 12\,[M_{sei} \bullet K - M_{SIDL}] \bullet \frac{1}{0.9\phi A_{ps} f_{py} d} \qquad (1)$$

where:

A_{ps}	=	area of each extended strand, in^2
f_{py}	=	yield strength of prestressing steel, ksi
d	=	distance from top of slab to centroid of extended strands, in.
M_{sei}	=	Seismic moment demand at c.g. of superstructure
M_{SIDL}	=	moment due to SIDL (traffic barrier, sidewalk, etc.) per girder, kip-ft
k	=	span moment distribution factor
ϕ	=	flexural resistance strength reduction factor

PRECAST GIRDER CONNECTION AT END PIERS

Precast girders are often supported on elastomeric bearing pads at end piers. Semi-integral cantilever abutments are used for shorter bridges, and L abutments for longer bridges are typically used for precast girder bridges. Bridge ends are free for longitudinal movement, but restrained for transverse seismic movement by girder stops. The bearings are designed to be accessible so that the superstructure can be jacked up to replace the bearings after a major seismic event.

In L-shaped end piers, the minimum displacement requirements at the expansion bearing should accommodate the greater of the maximum displacement calculated from a displacement analysis or a percentage of the empirical support length, N, as specified in the LRFD Guide Specifications[2]:

for Seismic Design Category A, B and C: $N = (8 + 0.02L + 0.08H)(1 + 0.000125\ S^2)$ (2)

for Seismic Design Category D: $N = (4+1.65\Delta_{eq})((1+0.00025\ S^2) > 24$ in. (3)

Where:

- $L =$ bridge length to the adjacent expansion joint, or to the end of the bridge, ft
- $H =$ average height of abutment wall supporting the superstructure, ft
- $S =$ skew angle of the support measured normal to span, deg
- $\Delta_{eq} =$ seismic displacement demand of the long period frame, in.

RESEARCH PROJECT ON PRECAST CONCRETE PIERS IN SEISMIC REGIONS

An experimental research program at the University of Washington[7] has developed and evaluated details for a precast concrete bridge bent substructure system having satisfactory seismic performance and suitability for rapid construction.

Details of the cap beam-column connection consist of 6 #18 vertical column steel bars grouted into 8 in. (200 mm) diameter corrugated metal ducts embedded in the cap beam as shown in Fig. 3. Precast concrete columns with six bars protruding are brought onto site, braced, and then cast integrally with their footing. Later, the precast cap beam is fitted over the column bars through the corrugated ducts and grouted in place, completing the bent substructure.

Full scale monotonic pull-out tests, with different embedment lengths, were first conducted to investigate the bond characteristics of large bars grouted into corrugated ducts[7]. These tests confirmed that the #18 bars could be developed in the depth of the cap beam.

Fig. 3 Precast Pier Research Project

Two one-third scaled connections, one with fully bonded vertical bars in ducts and another debonded eight bar diameters in the cap beam, were tested under 10% axial load and were subject to cyclic lateral displacements to study their performance. Both specimens performed well to 7% drift, failing as a result of bar buckling and fracture in the hinge region. Less damage to the cap beam was observed in the debonded specimen than the bonded, which saw moderate spalling around the underside of the beam as a result of duct slip. However, both demonstrated satisfactory strength and ductility, while allowing easy and rapid erection and generous construction tolerances.

PRECAST BENT CAP

Precast bent cap systems eliminate the need for forming, reinforcement, casting, and curing of concrete on the jobsite removing the bent cap construction from the critical path. Fig. 4 shows a precast bent cap under construction in Washington State. The #14 column vertical reinforcement will be placed through sleeves installed in the precast bent cap. Sleeves are made of 4 in. (100 mm) diameter corrugated galvanized metal ducts allowing adequate construction tolerance and room for grouting. The completed bent cap will be performing as conventional cast-in-place concrete pier with column bars extended through the precast bent cap to the top of diaphragm.

Fig..4. Precast Bent Cap under Construction in Washington State

Fully Precast Bridge Bents for Use in Seismic Region

Bridge construction with prefabrication of modular components offers an attractive alternative to conventional bridges. This concept has been used for many years for bridge girders, which are often prefabricated in steel or prestressed concrete and are lifted into place once the bents have been constructed. The product innovation through Highways For Life (HFL) project consists of a totally precast concrete bridge bent system that can be used in seismic regions. The proposed system uses a small number of large bars grouted in ducts to achieve the connection between components so that it can be constructed rapidly and safely, and in contrast with systems developed previously, it has the structural resilience to resist earthquake shaking. To apply the system in a wide range of girder bridges, the product innovation will be accompanied by a design methodology, laboratory specimen testing, as well as guidelines for fabricators, contractors, and practicing bridge engineers.

WSDOT HFL project consists of a totally precast bridge bent system, including precast segmental columns, precast bent cap, and precast superstructure as shown in Fig. 5. To accelerate construction without sacrificing seismic resistance, the beam-to-column connections are made with a small number of large-diameter reinforcing bars that are grouted into much larger-diameter ducts. The HFL project will ensure that the product can be deployed in a wide range of applications. HFL project includes four phases:

1. Proof testing of project-specific and alternative-design variations of the system,
2. Development of project-specific and general design provisions and specifications,
3. Development of design examples, and
4. Deployment of the basic system in the field.

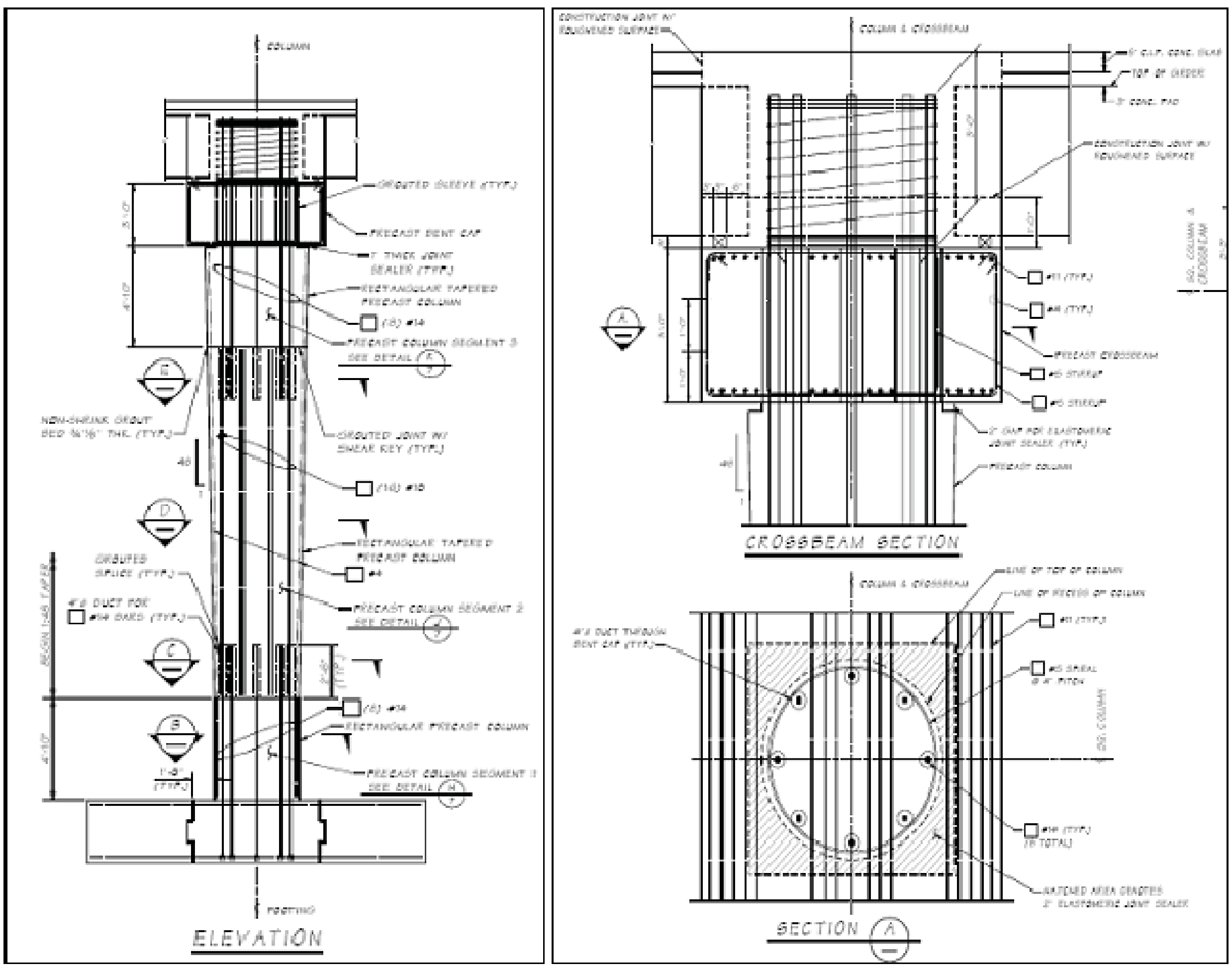

Fig. 5. WSDOT Fully Precast Bridge Bents for Use in Seismic Region

WSDOT Strategic Plan for Accelerated Bridge Construction

Starting in 2008, WSDOT initiated a practice development and implementation for accelerated bridge construction. WSDOT has established a task force that is headed by an ABC Advisory Committee to develop standards, guidelines, and key policies for implementing structural design for accelerated bridge construction.

Consisting of subject matter experts from the Bridge Design Office, Bridge Construction, Regions, FHWA, Consultants, Research, Precast producers, Maintenance, Materials, and other relevant fields, the task force outlined a strategic plan to develop, implement, and promote ABC practice in Washington. The WSDOT ABC team has formulated strategy and work plans with the specific tasks outlined below.

The goal of ABC is to deliver projects earlier to the traveling public; to effectively reduce the impacts of on-site construction to motorists. The Department's larger goal, as stated in its Mission/Vision statement, is to enhance mobility. Therefore, ABC should be viewed as a subset of a larger "accelerated project delivery" effort encompassing all aspects of project development through construction contract acceptance. This latter requirement stems from the fact that quite often new techniques involve unassigned risk that must be borne by the Contractor at a premium until the comfort level garnered from successes has been realized.

CONCLUSION

Precast prestressed concrete bridge systems are economical and effective for rapid bridge construction. Precasting eliminates traffic disruptions during bridge construction while maintaining quality and long-term performance.

Precast bridges with monolithic connections meeting the AASHTO LRFD seismic design and detailing requirements could safely be used in seismic zones. Proper seismic design entails a detailed evaluation of the connections between precast components as well as the connection between superstructure and the supporting substructure system. Monolithic connections are the key to proper seismic performance of precast bridges.

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Seismic Performance of Precast Bridge Columns with Built-in Elastomeric Pad

S. Motaref, M. Saiidi, and D. Sanders

ABSTRACT

A one-third scale precast concrete segmental column with an energy dissipating plastic hinge was designed and tested on a shake table at the University of Nevada, Reno. A built-in elastomeric pad integrated with the footing and a concrete segment constituted the plastic hinge. The purpose of using the pad was to minimize damage while dissipating energy through yielding of the longitudinal bars and deformation of the pad. The column was subjected to the Sylmar earthquake (Northridge 1994) record with increasing amplitudes until failure happened. Compared to conventional reinforced concrete construction, the column showed superior performance with considerable energy dissipation, minimal residual displacement, and a damage-free plastic hinge. These advantages make the proposed detail a better alternative for accelerated bridge construction in high seismic zones.

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INTRODUCTION

Conventional bridge construction involves a time consuming process associated with traffic delays and risk to public safety. In contrast, prefabricated bridge systems can expedite construction, thus minimizing traffic delays and construction site safety risk. Existing details for precast segmental columns offer minimal energy dissipation as a result of the discontinuation of longitudinal reinforcement; therefore, their usage is very limited in high seismic zones.

Many studies have been conducted since the mid 90's to investigate the use of precast building and bridge components reinforced with unbonded prestressing steel in regions with seismic activities. Tests by Priestley and MacRae (1996), Mander and Cheng (1997), Billington et al. (1999), Chou and Chen (2005), Hewes and Priestley (2002), Yamashita and Sanders (2006) revealed that precast concrete elements with unbonded tendons can undergo very larger lateral drifts while exhibit less damage and residual displacement than conventional CIP (cast-in-place) RC (reinforced concrete) (Hewes, 2007).

Yamagishi and Kawashima (2006) used the high damping rubber in plastic hinge area to minimize the damage and provide larger deformation and energy dissipation capacity.

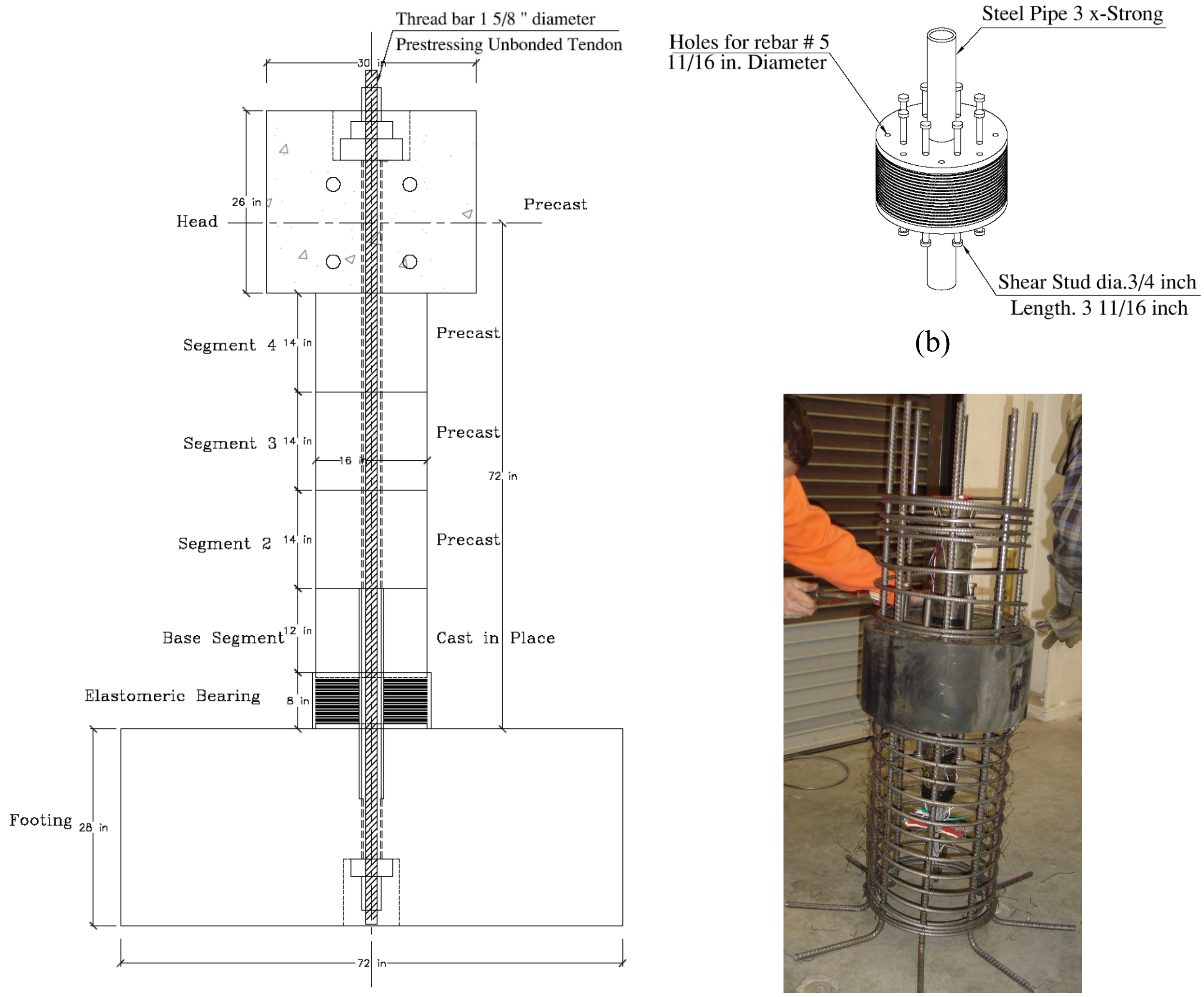
A series of innovative precast concrete segmental column details are being proposed and studied at the University of Nevada, Reno through a research project funded by the California Department of Transportation.

The first phase of this project involved analytical and experimental study of a segmental concrete column incorporating an elastomeric bearing pad in the plastic hinge. This article presents a summary of the experimental part of the first phase of the study.

SPECIMEN DETAILS

A one-third scaled cantilever column model referred to as SBR-1 (segmental column with built-in rubber pad) was constructed and tested on a shake table under simulated earthquakes. The elastomeric pad was connected to the footing via reinforcing bars and was intended to increase the energy dissipation and to minimize the damage. An unbonded post-tensioning rod was used to connect the segments and to minimize the residual displacements. The model consisted of five segments. The base segment, including the elastomeric bearing pad in the lower part and the upper concrete part, was connected to the footing by steel longitudinal bars. The second, third, and fourth segments were identical RC segments. The fifth segment was a concrete head to connect the specimen to the mass rig system via a steel link (Figure 1. (a)).

The elastomeric bearing pad was designed by controlling the failure of the rubber when subjected to axial compression and bending moment (Aiken et al., 1989). Total height of the bearing was 8 in. composed of 21 layers of rubber, and 20 layers of steel shim. Each layer of rubber and steel was 3/16 in. and 1/8 in. thick, respectively. The shear deformation in the elastomeric bearing pad was restrained using a 3-1/2 in. diameter x-strong steel pipe used at the center of bearing. Eight shear studs were welded to the top and bottom steel plates to provide anchorage to concrete. Eight holes with 11/16 in. diameter were drilled through the bearing pad to allow for the passage of the longitudinal bars (Figure 1.(b) and (c)).



(a) (c)

Figure 1. a) Specimen Detail b) Elastomeric Bearing c) Base Segment Detail

Column diameter was 16 in. and the column height was 72 in. leading to an aspect ratio of 4.5 for the column. The RC portion of the base segment was 12 in. high. The depth of each of the other segments was 14 in. 8-#5 bars spaced evenly in a circular pattern were used to reinforce the base segment, leading to a longitudinal steel ratio of 1.2%. This amount of steel at base segment guaranteed that the yielding of the bars precedes the gap opening. Other segments were minimally reinforced with 8-#4(a steel ratio of 0.8%). Due to the short length of these segments, the cross sections were not expected to yield within the segments, and hence only a small amount of steel was used.

The loading head and footing were designed to be sufficiently rigid and strong to remain free from damage. The Footing was 72x72x28 in. and head block was 30x30x24 in. Axial load included the gravity load of 80 kips and the post-tensioning force of 100 kips. It was corresponding to axial load index of 0.20 (Axial load index is defined as the ratio of the axial load to the product of the gross section area and the concrete compressive strength). There are no established guidelines for the level of post-tensioning force, and hence the PT force was designed to be close to the gravity load. This force was not sufficient to prevent opening of the gaps in between the segments under moderate and higher lateral drifts.

A 1-5/8 in. diameter post-tensioning unbonded high strength Dwyidag rod was used in the column central core. Other researchers have reported a significant increase in the post-tensioning force under large drifts (Hewes and Priestley, 2001). Computer program OpenSees was used to determine the design post-tensioning force. The diameter of the rod was selected such that the maximum force in the rod does not reach the yielding point for the expected maximum 14% drift at failure.

COLUMN CONSTRUCTION AND TEST SET UP

Construction stages of SBR-1 included building the steel cages, pouring the footing, pouring the segments, and finally, assembling the column (Figure 2). The second and fourth segments were cast on top of the base and the third segments with match cast method. Column segments were assembled on the shake table inside the laboratory in approximately three hours. A small amount of epoxy was applied on top of each surface before placing the next segment to stabilize the segments during construction.

Figure 2. Assembling the Column at UNR Large Structural Laboratory

Figure 3. (a) shows the shake table setup of the specimen. The column was attached rigidly to the shake table and the mass rig link. The steel spreader beam was bolted to the top of the column head and two hydraulic jacks applied the axial force. Large number of instruments including, sixty four strain gauges, displacement transducers, load cells, and accelerometers were installed on the column to measure the longitudinal and transverse bar strains, axial load, lateral load, lateral displacement, curvatures, and accelerations. Details of instrumentation are provided in S. Motaref et al.

SBR-1 was subjected to a series of simulated Sylmar ground motions (Figure 3. (b)) with the acceleration amplitude scaled by an increasing factor in subsequent runs. The testing was continued until failure. To determine the dynamic characteristics of the column as the level of motions increased, a white noise motion was applied to the specimen after each earthquake motion. Table 1 displays the testing program.

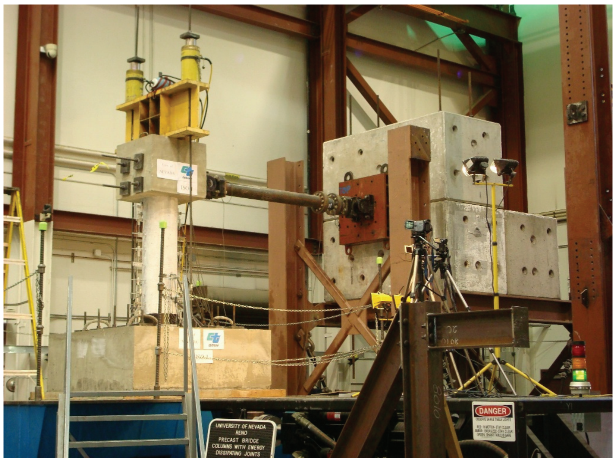


Figure 3. (a) Shake Table Test Setup

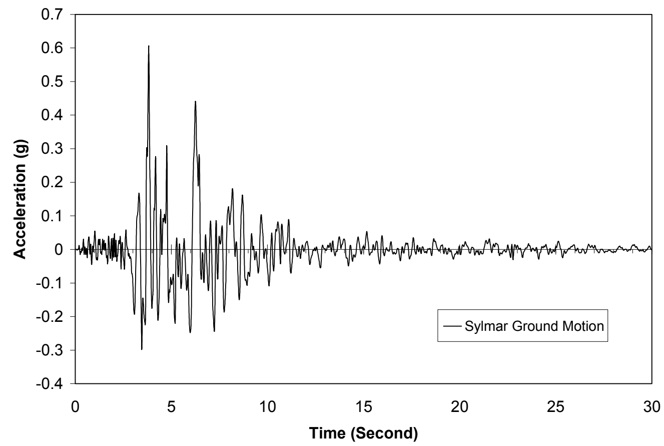


Figure 3. (b) Sylmar Earthquake Time History

Table 1. Input Earthquake Amplitudes

Run	Input Ground Motion
A	White noise
1	Sylmar X 0.1
B	White noise
2	Sylmar X 0.25
C	White noise
3	Sylmar X 0.5
D	White noise
4	Sylmar X 0.75
E	White noise
5	Sylmar X 1.00
F	White noise
6	Sylmar X 1.25
G	White noise
7	Sylmar X 1.50

OBSERVATION AND TEST RESULTS

The dominating failure mode in SBR-1 was concrete crushing at the interface of base and second segments due to the gap opening and excessive rotation between base and first segment. Concrete spalled on one side of column at Run 5 (1× Sylmar with PGA of 0.6 g), which corresponded to the first gap opening between the base and the second segments (Figure 4. (a)). Cover crushed on both sides of base segment, and some core crushing occurred during run 7

(1.5× Sylmar with PGA of 0.9 g) which was considered as the failure of column (Figure 4. (b)). Column displacement at early runs was due to the rotation of elastomeric bearing and yielding of steel bars. On the other hand, at later runs, gap opening was the main source of displacement of column.

No damage was observed in the elastomeric bearing pad. The force in the unbonded rod remained under the yield force (it reached 67% of the ultimate strength during run 7). During run 6, the maximum drift ratio was 6.9% and the residual drift ratio after this run was 0.19%, indicating a very small residual drift under what could be considered as the design motion. During run 7 the maximum drift ratio was 14% and the residual drift ratio was 2.9%.

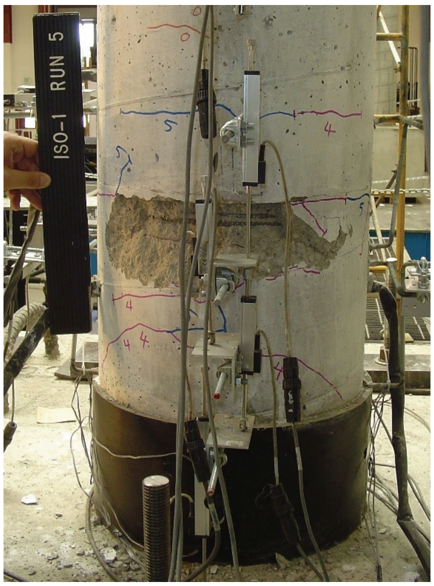


Figure 4.(a) Specimen after Run 5 (PGA 0.6g)

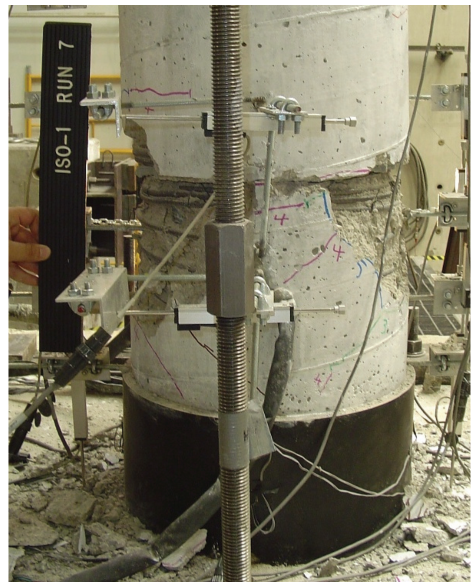


Figure 4.(b) Specimen after Run 7 (PGA 0.9g)

Figure 5 shows the cumulative measured force-displacement curves. The maximum force of 26.5 kips and maximum displacement of 10.12 in. (14% Drift) was measured during run 7.

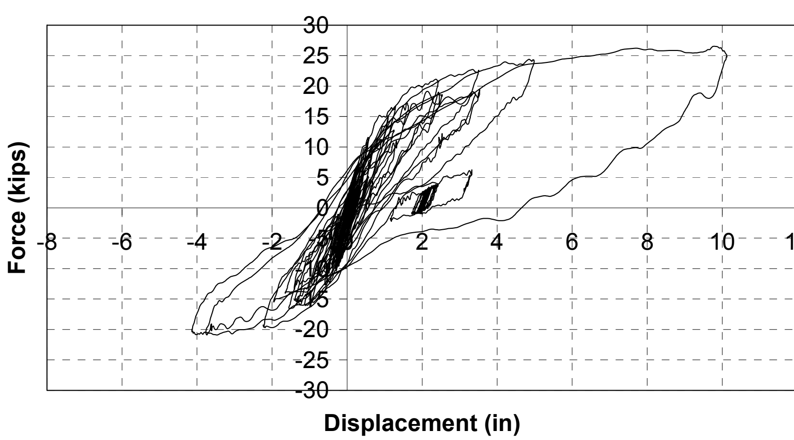


Figure 5. Cumulative Measured Force-Displacement Curves

A detailed OpenSees model of the column was developed and very good correlation was obtained between the measured and calculated data. The model consisted of different sections and elements, nonlinear beam-column element, elastic beam column element, corotational truss element and zero length elements to model different parts of SBR-1 (Mazzoni et al., 2006). This model was capable to simulate force, displacement, post-tensioning force, gap opening and material strain. The description of the OpenSees model is provided in S. Motaref et al. A comparison of the analytical and experimental results is also presented in S. Motaref et al.

ADVANTAGES OF SBR-1 OVER CONVENTIONAL PRECAST SEGMENTAL CONCRETE COLUMNS

The test result revealed the potential use of SBR-1 in high seismic zones because of its advantages over conventional precast concrete column. Superiority of this detail includes construction speed, large energy dissipation, minimal damage in the plastic hinge zone and minimal residual displacement. Energy dissipation took place mostly through the rotation of elastomeric bearing and yielding of the longitudinal bars at base segment. The measured data showed that 56% of dissipated energy was due to rotation of elastomeric bearing and the rest was through the yielding of the bars. Unlike the bearing pads used in Yamagishi and Kawashima, 2006, the pad in the present study was shimmed and hence buckling of longitudinal bars was prevented.

To verify the superiority of SBR-1 over conventional precast segmental column, a conventional precast concrete segmental column with no extended bars and elastomeric bearing was modeled in OpenSees. The results showed that energy dissipation in SBR-1 was 244 % larger than that of a conventional segmental column in which the base segment is not connected to the footing by dowels.

CONCLUSIONS

The shake table and analytical studies of a one-third scaled cantilever precast concrete segmental column incorporating a steel shimmed elastomeric bearing pad showed that this detail can substantially reduce damage in the plastic hinge while dissipating the earthquake energy. Post-tensioning of the column minimized the residual displacement of the column. Damage to confined concrete was observed in between segments. External confinement of the segments appears to potentially delay the damage. Residual displacement and concrete damage were small in this column. The energy dissipation in SBR-1 was 80% larger than a similar conventional precast segmental column. A comprehensive OpenSees computer model of the columns was developed and very good correlation was observed between the measured and calculated results (S. Motaref et al.). The detail incorporated in SBR-1 can be a potential alternative for accelerated bridge construction in high seismic zone.

ACKNOWLEDGMENT

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IT Roles in Seismic ABC

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ABSTRACT

BrIM has burst onto the radar screen of bridge enterprise stakeholders not only in design and construction but also in operations and management. These stakeholders are increasingly realizing that a well thought out leveraging of bridge data for multiple purposes through the entire bridge lifecycle is increasingly important. Principal questions, issues, and challenges that have been raised by various stakeholder audiences hearing about BrIM will be summarized to help clarify the way forward to increased industry acceptance and deployment of BrIM-enabled workflows. In particular, aspects of using it in Accelerated Bridge Construction Scheme with particular focus on seismic considerations are discussed herein. A streamlined design approach which reduces manual data re-entry and accelerates integrated project delivery has been investigated and assessed.

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1 BACKGROUND AND INTRODUCTION

For centuries engineering and construction activities have relied on paper-drawings as the primary representation of construction documentation. Other closely related industries have documented and/or projected reduced costs, faster delivery, and improved quality as a result of implementing 3-D CAD based integrated design and manufacturing processes along with accompanying interoperability standards. The bridge enterprise is nearing the end of an era and we in the bridge enterprise are overdue to do the same. Failure to do so has been documented as a major cost center in the closely related capital facilities industry (Gallaher et al. 2004). Advances in automation and communication technologies in recent years have been significant (Chen and Shirole' 2006), but they have not yet been fully adapted and integrated with each other and then deployed to accommodate the unique requirements of the bridge construction engineering and management enterprise. At the same time, the recent explosion of interest in integrated project delivery, e.g., (Post 2008) and (Tulacz 2008), BIM (Building Information Modeling), e.g., (Rubin 2008) and (Jordani 2008) and its closely related cousin BrIM (Bridge Information Modeling) (Chen and Shirole' 2007), (Puckett et al 2008) are raising awareness in the industry that there are significant potential efficiencies and competitive advantage to be gained by developing these integrative approaches to greater maturity. Various aspects of BrIM are increasingly being used in bridge infrastructure delivery, albeit in piecemeal fashion. Aspects of BrIM methodology are currently being used in the design and construction of large and complex bridges, such as for visualization and detailing as well as in bridge operations and management through the use of AASHTOWare. Other prior work includes that described in (Chen, 2002) and (Chen et al., 2003).

BIM is changing the product delivery model in the building industry and is certainly leading the bridge community in the area of architecture, structural and mechanical engineering integration. Fabrication and construction processes are being effectively linked as well. BIM technology is paving the way for different delivery models specifically more toward design/build where the design and construction professionals team for the entire project duration. Furthermore, all design professionals are coming on board earlier in the project lifecycle, at a time when critical decisions are made (Hall 2007). Also of note, GSA now requires BIM for their major projects (GSA 2009, Hardy 2006).

The intent is to demonstrate the viability of integrated bridge project delivery and life cycle management via a prototype integrated system that illustrates representative data exchanges and applications throughout the bridge life cycle, with focus in this paper on project delivery with particular interest in seismic aspects. This effort is motivated by the recognition that the current U.S. practice of information transfer during the bridge planning/ design/ fabrication/ construction processes involves repeated manual transcription of data that is error-prone, time – consuming approvals (e.g. of shop drawings), and a lack of standardized formats that hinder electronic information transfer. It is also being recognized that without such standards, electronic information exchange is cumbersome at best, and often not possible.

Authors are developing BrIM for a recently built 3-span bridge (Quincy Avenue Bridge over I-25 in Denver, Colorado) and modeling it for its lifecycle. The processes outlined and associated figures were derived from demonstrating BrIM for two alternative bridge designs: steel and prestressed concrete.

2 THE BRIDGE ENTERPRISE

Although at the present time there are no non-proprietary standards for electronic exchange of lifecycle bridge data, it is the vision of this project to facilitate the development of an integrated system for the entire bridge life cycle (i.e., "from cradle to grave"). In the future, a complete modeling of bridge information in standardized format can be anticipated to facilitate integration of computer-aided design (CAD), computer-aided engineering (CAE), computer-integrated manufacturing (CIM), construction engineering and management (CEM) and bridge management (BM) that will enable not only rapid and better quality project delivery but also subsequent cost-effective life-cycle management. All three fundamental objectives of bridge delivery, namely higher quality, faster delivery, and more economical cost over the bridge lifecycle, would then be attained.

Historically, in the development of various computational tools for supporting these various aspects (e.g., planning, design, detailing, estimating, fabrication, construction project management, bridge operations, and bridge management), individual aspects were typically addressed in standalone fashion without sufficient regard for complications arising from multiple data sources. Some of these complications involve the need for tedious, manual, error-prone re-entry of duplicate data into several software "stove piped" applications. If a coordinated shepherding of data supporting these individual applications were developed, bridge data integrity would be more easily maintained, and "handoff" processes from one application to another would be streamlined if not made altogether seamless.

Just such an attempted coordinated shepherding of bridge data is shown in Figure 1, which shows a conceptual view of the organization of representative aspects of the "cradle-to-grave" bridge design and construction enterprise and how they each depend on bridge data represented in the center of the diagram. A coordinated handling and leveraging of that data could prevent the proliferation of problems resulting from multiple (potentially inconsistent) sources of bridge data.

How information flows among particular applications is shown in Fig. 2 for a steel bridge superstructure and in Fig. 3 for a concrete bridge superstructure, respectively. It should be noted that the particular commercial software applications appearing in these figures are just examples based on particular applications investigated by the project team. Advantages of combining BrIM with seismic ABC include the following. First, BrIM can accelerate project delivery from bridge seismic design to bridge construction. For example, after designing the rebar size and number from SAP2000, the data can be passed to detailing software like Tekla to form the rebar lists automatically. Second, the virtual assembly and visualization function can help contractors avoid conflicts before construction, which minimize the traffic interruption due to rework. There are significant potential benefits to be obtained by investigating this approach.

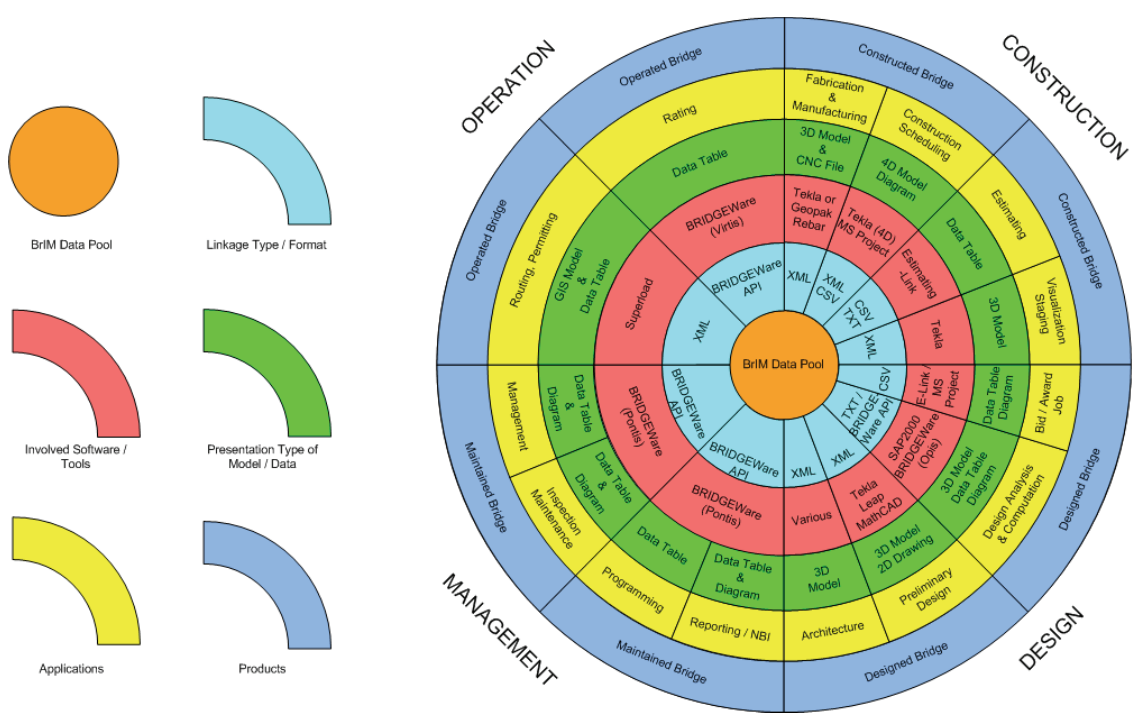


Figure 1. Data Model Centric View of the Bridge Enterprise

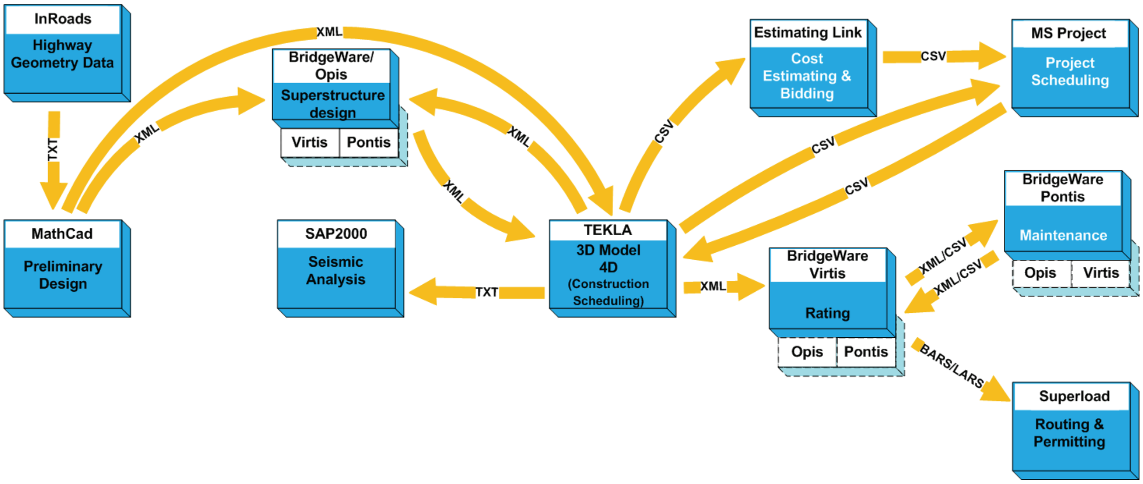


Figure 2. Workflow and Software Interoperabilities: Steel Alternate

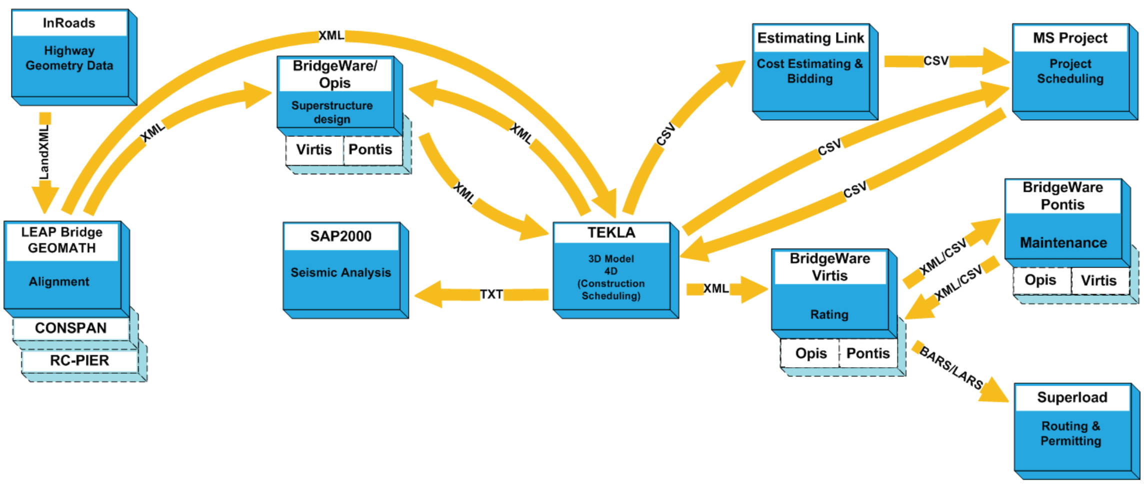


Figure 3. Workflow and Software Interoperabilities: Concrete Alternate

3 CASE STUDY BRIDGE

Figure 4 shows the case study bridge configuration used in this study. The Quincy Avenue Bridge over I-25 in Denver, Colorado is a 3-span continuous bridge with a cast-in-place reinforced concrete deck. The bridge is 76 ft wide, and its span configuration is 45.5 ft - 122 ft -109.5 ft as shown in Figure 1. The bridge has two skew angles, 21.25 ° and 26.19 °. Seven BT72 prestressed concrete girders are spaced at 11ft-3in center-to-center. The deck consists of an 8 in structural thickness with a 2 in integral wearing surface (IWS). Each pier bent has five 3 ft-diameter columns. The bent cap beam is 3 ft-6 in wide and 4 ft deep.

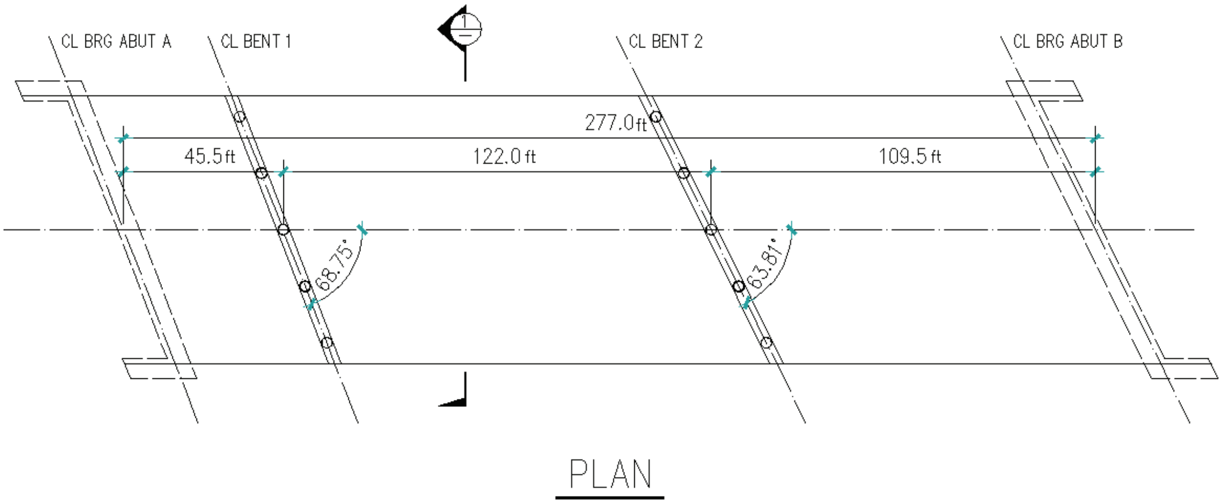


Figure 4. Superstructure Configuration

4 DESIGN STAGE

Figure 5 shows a portion of the quantitative data input/verification for the concrete girder alternative of the bridge case study considered in this investigation. As indicated in Fig. 3, this data was electronically passed into BridgeWare (Opis) and further "downstream," e.g., using XML in order to avoid redundant data entry.

Specification checks are performed on the trial section and can be inspected either for the concrete alternate or for the steel alternate. Figure 6 shows a portion of the specification checks performed by Opis on the steel alternate of the Quincy Avenue case study bridge. The software linkage that takes data from MathCad (see Fig. 2) into Opis for this check is illustrated in Fig. 7, which shows quantitative data in the MathCad worksheet that is passed using XML into Opis, for which the Opis Explorer is shown in Fig. 7.

In turn, the bridge model is transferred from BridgeWare (Opis) to SAP2000. In SAP2000/Bridge, users can construct the design response spectrum by defining the site classes and the bridge location using the latitude and longitude or the postal zone. From the response spectrum form, the values for S_{DS} and S_{D1} are determined by SAP2000. Based on the resulting case study bridge's Seismic Design Category (SDC), SAP2000 calculates the displacement demand based on the response spectrum defined by users and calculates the displacement capacity using the equations provided by AASHTO Seismic Guide Specification.

After the displacement capacity analyses have been completed, SAP2000/Bridge reports the displacement demands, displacement capacities, and the ratio of the Demand/Capacity displacements.

5 DESIGN INTO CONSTRUCTION

Figure 8 shows a 3D view of a portion of the bridge computer model that would have been used to generate design and construction information, including contract plans and shop drawings. That model has its geometry generated such that its geometry is entirely consistent with the roadway stationing, plan, and profile on which the bridge lies. Drawings in turn are (merely) extracted sections from such a model, an example of which is shown in Figure 9.

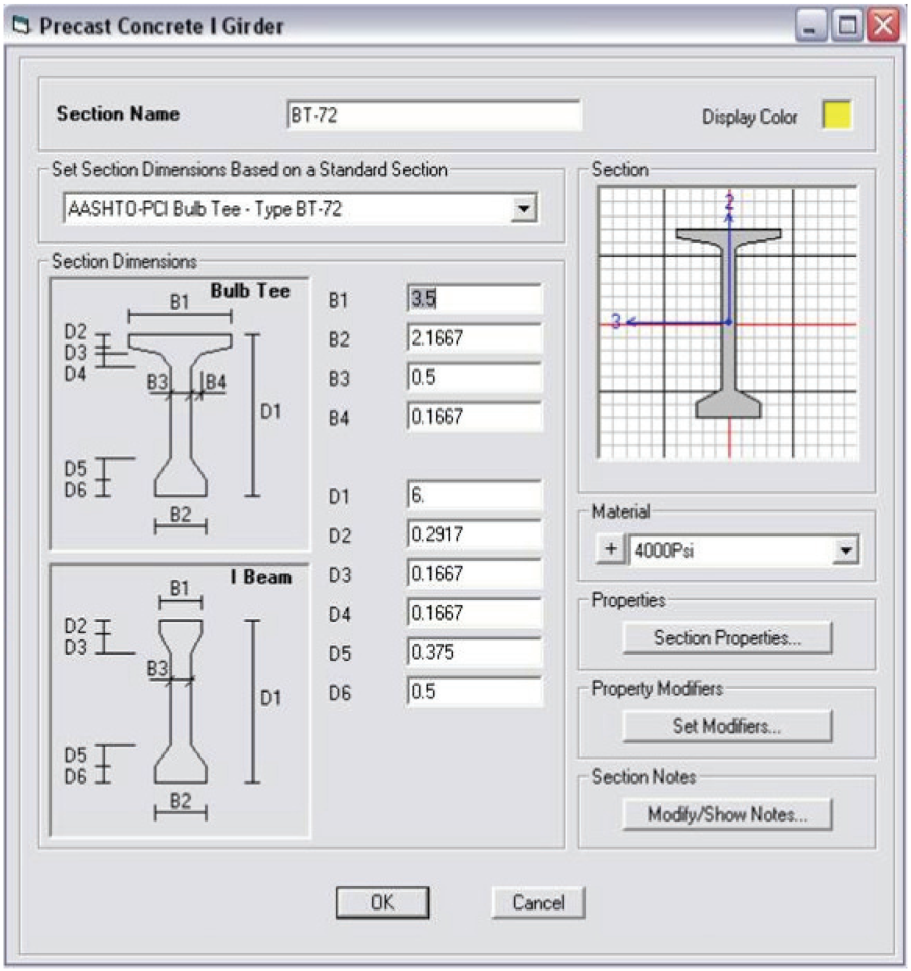


Figure 5. Concrete Girder Data Input/Verification

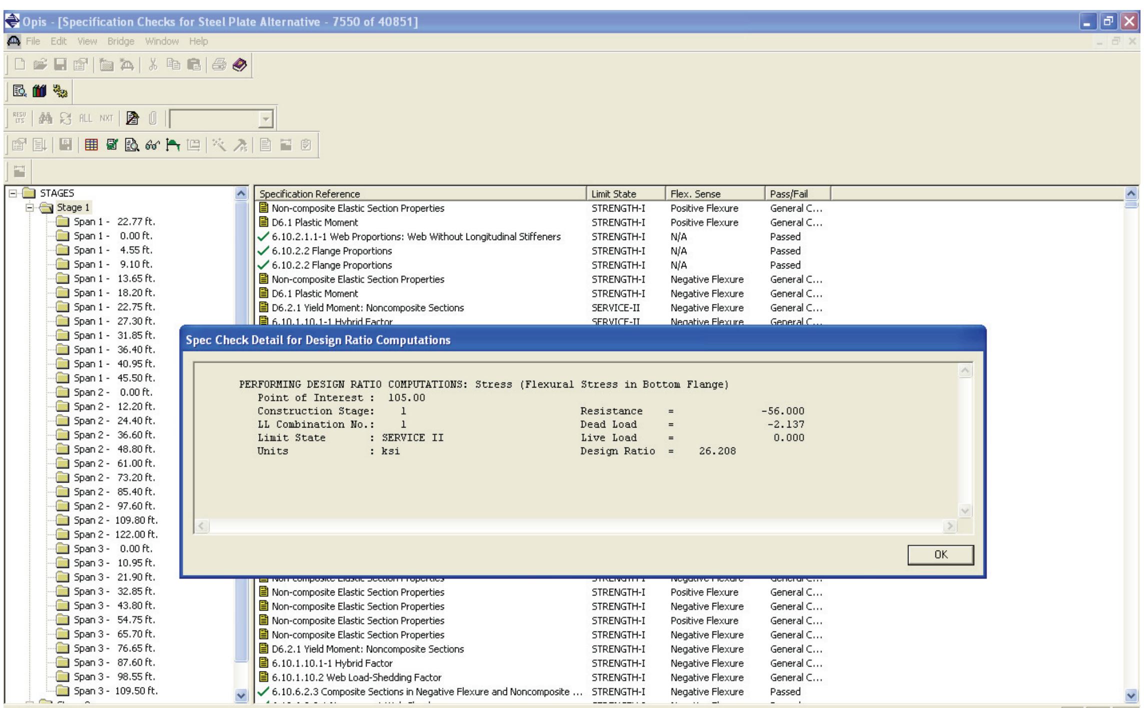


Figure 6. Specification Checking in Opis (Steel Alternate)

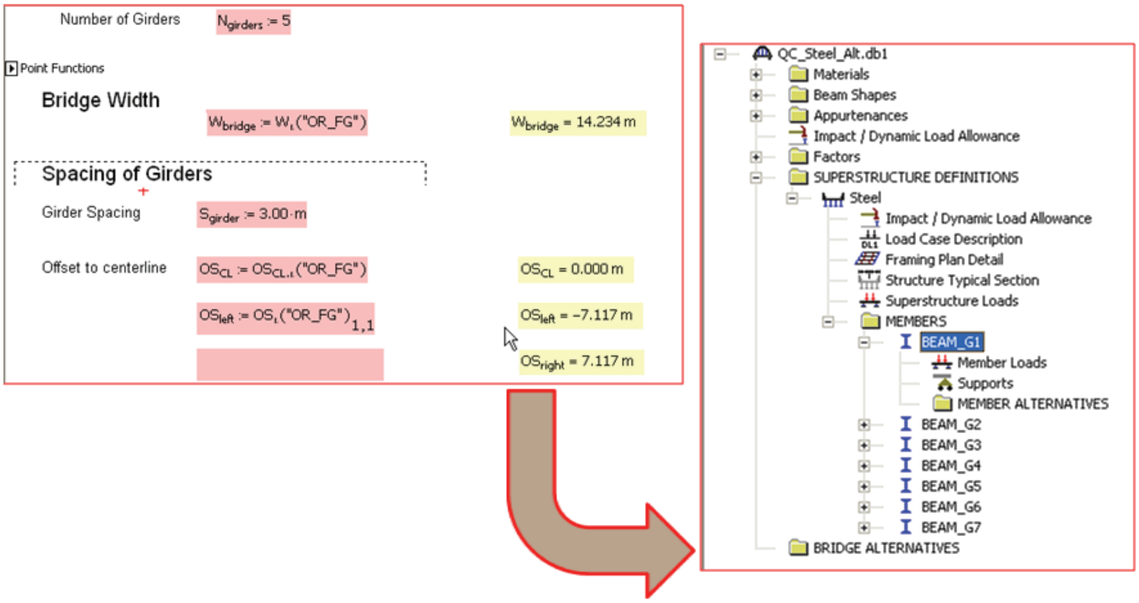


Figure 7. Linking from MathCad to Opis via XML

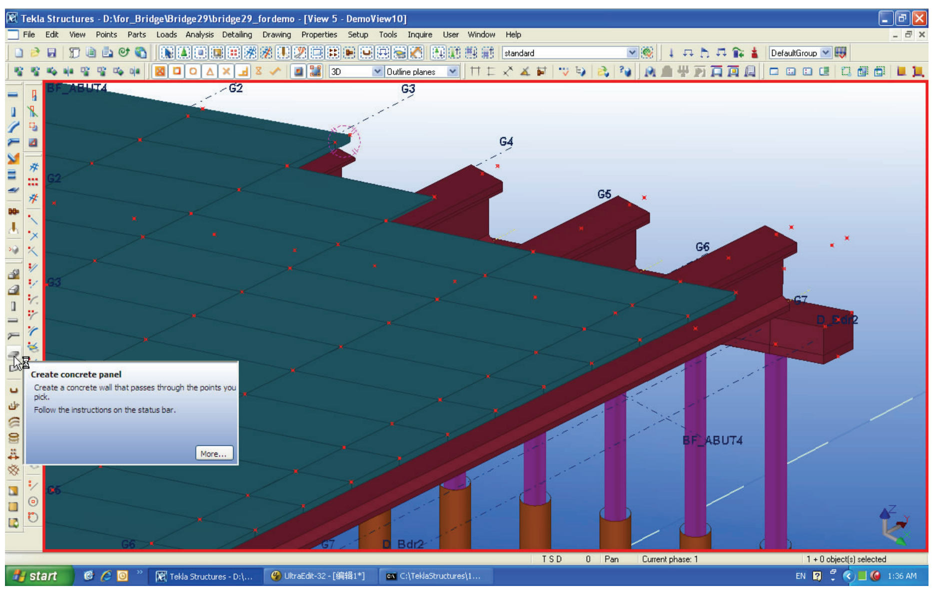


Figure 8. 2D View of a Portion of the Bridge Computer Model

Detailing for fabrication follows on the heels of the final design. That final design culminated in a set of construction documents for which bids could be prepared, but those documents typically do not contain all information needed for fabrication and construction. Hence the usual detailing phase, in which all relevant design data traditionally is manually re-entered by the contractor/fabricator in order to generate shop drawings containing all information needed for fabrication.

In an integrated workflow proposed and envisioned herein, the bridge model data used for final design documentation itself becomes the electronic starting point for the addition of detailing information consistent with both the contractors' means and methods and the design intent (the latter presumably verified as part of the shop drawing review process). Thus, time-consuming error-prone manual re-entry of data is avoided. Having the same model shared with "downstream" CEM operations is a significant advantage. Multiple versions of bridge data do not proliferate and require disambiguation. Updates to the data have clearly defined access mechanisms to prevent this proliferation.

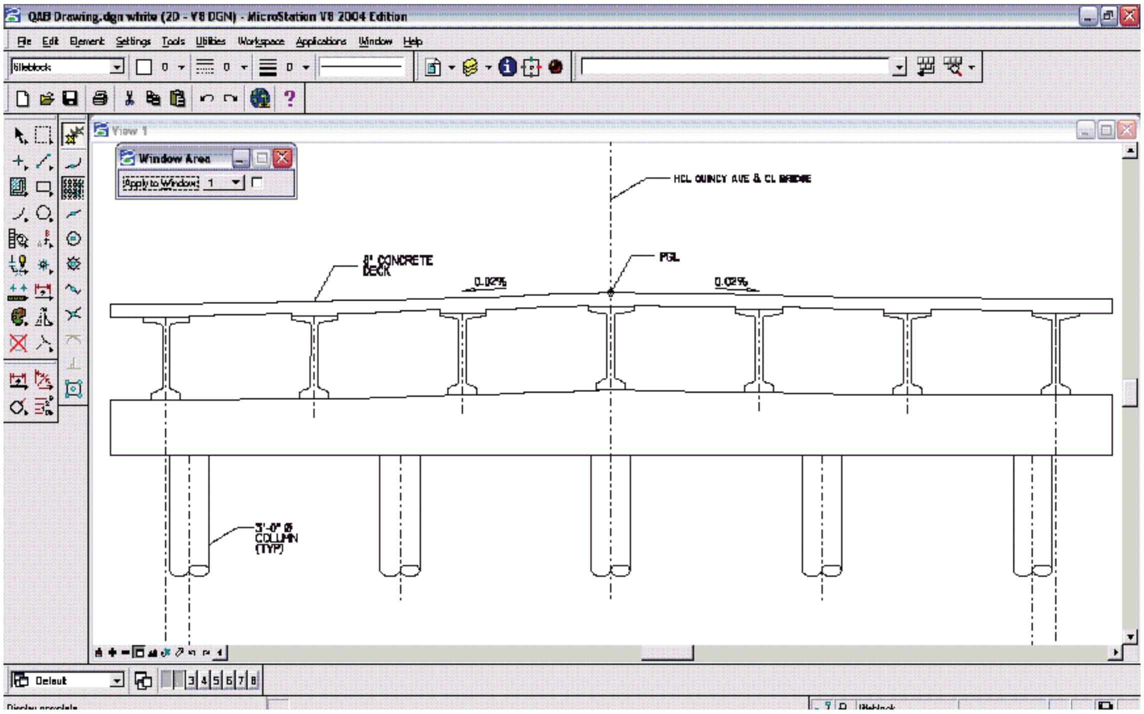


Figure 9. 2D Bridge Cross Section Drawing Extracted Directly from 3D Model

For electronic data exchange purposes, even in lieu of industry-wide standards there are several approaches that may be taken. Among these, we have test-driven application-specific API's (Application Programming Interfaces), XML (eXtensible Markup Language), and direct export of CNC (Computer Numerical Control) files when such export is supported by the detailing software. These are used for transferring design model data into the detailing software in order to avoid duplicate data entry and for export to CNC-driven shop equipment. Such equipment exists for such tasks as hole-drilling, automated welding, and rebar bending, for example. Thus, with a suitably equipped fabrication shop and raw material lined up, it is technically possible to start fabricating as soon as the detailing is approved – and the shop itself has no need for "shop drawings"!

In detailing software application, Figure10 shows a portion of a bridge superstructure model, the girders (data) for which were imported using the API or XML.

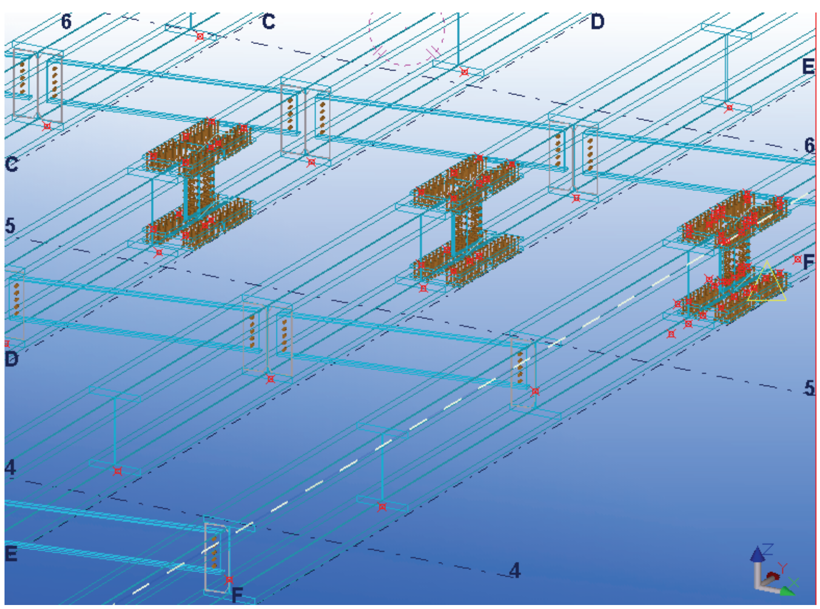


Figure 10. Portion of Steel Superstructure Model in Steel Detailing Application

This detailing model has associated with the bolted connections shown all data needed for material procurement (e.g., bolt diameters, grades, lengths, washers, etc). Similarly, for welded connections this detailing model has electrode information, total lengths of welds of various sizes, in short all quantity information that would be needed by a fabricator to conduct a detailed estimate of fabrication costs. Figure 11 shows the export of CNC files directly from the detailing software as described above, for direct use by suitable shop equipment.

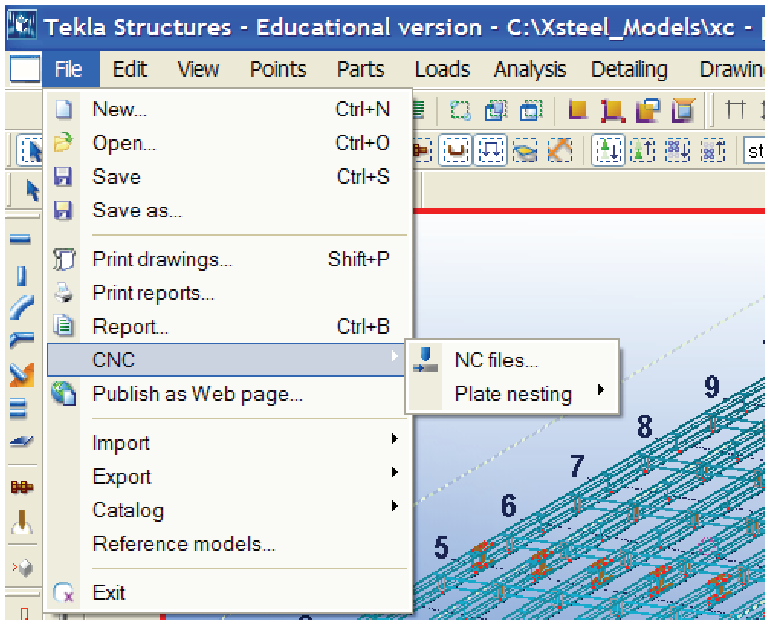
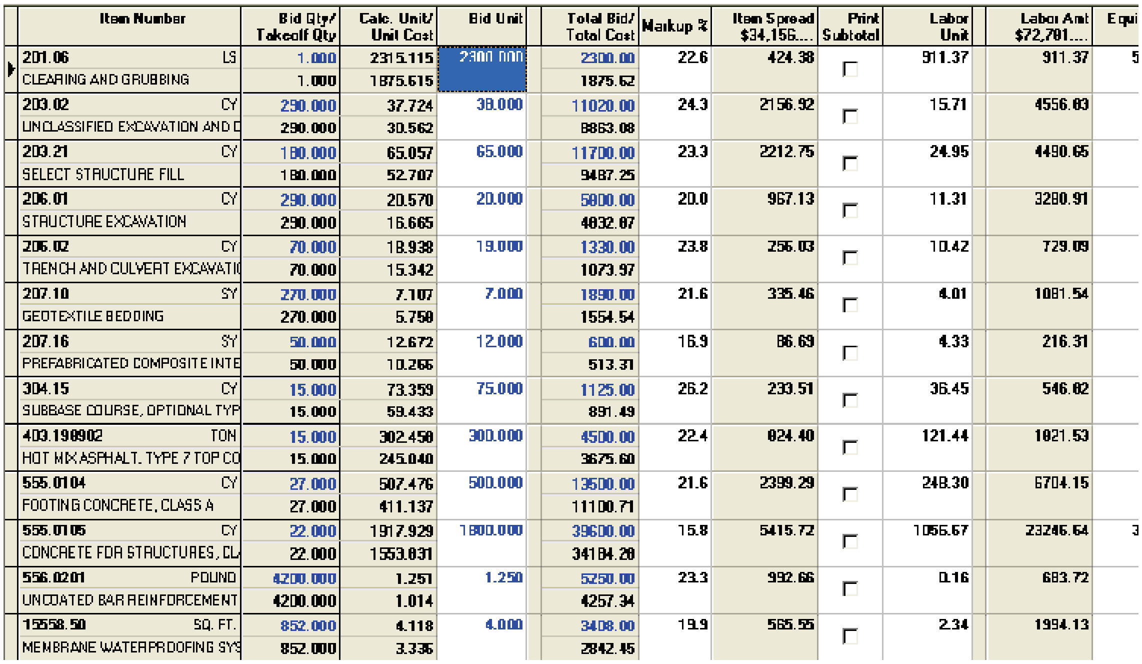


Figure 11. CNC File Export

As shown in Figures 2 and 3, quantity information from the model can be exported to an estimating application. Just as 2D drawings are merely reports generated from the model, bill of material lists are similarly reports generated from the model. These reports, furthermore, are guaranteed to be consistent with the information contained in the model. These quantity reports can be used for cost estimating or for actual material procurement. Interfacing such a list (e.g., in a spreadsheet software application) to a unit price database then provides the means to carry out cost estimating based on the extracted model quantities. Figure 12 shows a sample screen from the estimating application being used in the present study.



Item Number		Bid Qty/ Takeoff Qty	Calc. Unit/ Unit Cost	Bid Unit	Total Bid/ Total Cost	Markup %	Item Spread $34,156....	Print Subtotal	Labor Unit	Labor Amt $72,701....	Equi
201.06 CLEARING AND GRUBBING	LS	1.000 1.000	2315.115 1875.615	2300.000	2300.00 1875.62	22.6	424.38	☐	911.37	911.37	5
203.02 UNCLASSIFIED EXCAVATION AND D	CY	290.000 290.000	37.724 30.562	38.000	11020.00 8863.08	24.3	2156.92	☐	15.71	4556.03	
203.21 SELECT STRUCTURE FILL	CY	180.000 180.000	65.057 52.707	65.000	11700.00 9487.25	23.3	2212.75	☐	24.95	4490.65	
206.01 STRUCTURE EXCAVATION	CY	290.000 290.000	20.570 16.665	20.000	5800.00 4832.87	20.0	967.13	☐	11.31	3280.91	
206.02 TRENCH AND CULVERT EXCAVATIO	CY	70.000 70.000	18.938 15.342	19.000	1330.00 1073.97	23.8	256.03	☐	10.42	729.09	
207.10 GEOTEXTILE BEDDING	SY	270.000 270.000	7.107 5.758	7.000	1890.00 1554.54	21.6	335.46	☐	4.01	1081.54	
207.16 PREFABRICATED COMPOSITE INTE	SY	50.000 50.000	12.672 10.266	12.000	600.00 513.31	16.9	86.69	☐	4.33	216.31	
304.15 SUBBASE COURSE, OPTIONAL TYP	CY	15.000 15.000	73.359 59.433	75.000	1125.00 891.49	26.2	233.51	☐	36.45	546.82	
403.198902 HOT MIX ASPHALT, TYPE 7 TOP CO	TON	15.000 15.000	302.458 245.040	300.000	4500.00 3675.60	22.4	824.40	☐	121.44	1821.53	
555.0104 FOOTING CONCRETE, CLASS A	CY	27.000 27.000	507.476 411.137	500.000	13500.00 11100.71	21.6	2399.29	☐	248.30	6704.15	
555.0105 CONCRETE FOR STRUCTURES, CL	CY	22.000 22.000	1917.929 1553.831	1800.000	39600.00 34184.28	15.8	5415.72	☐	1056.67	23246.64	3
556.0201 UNCOATED BAR REINFORCEMENT	POUND	4200.000 4200.000	1.251 1.014	1.250	5250.00 4257.34	23.3	992.66	☐	0.16	683.72	
15558.50 MEMBRANE WATERPROOFING SYS	SQ. FT.	852.000 852.000	4.118 3.336	4.000	3408.00 2842.45	19.9	565.55	☐	2.34	1994.13	

Figure 12. Bridge Project Cost Estimating Screen

The model detailing software also has capabilities for incorporating "4D" (i.e., time or sequence based) information into the model for use in construction planning. This kind of capability is increasingly being provided in 3D CAD software applications (de Vries and Harink 2007). "User attribute" data fields associated with the various bridge component objects in the CAD model can be used for such things as erection phasing and material management (both for shipping bills of lading and for on-site material management) as well as for interfacing with Scheduling software applications such as MS Project. Figure 13 shows a glimpse of 4D modeling of a case study bridge for erection sequence planning purposes.

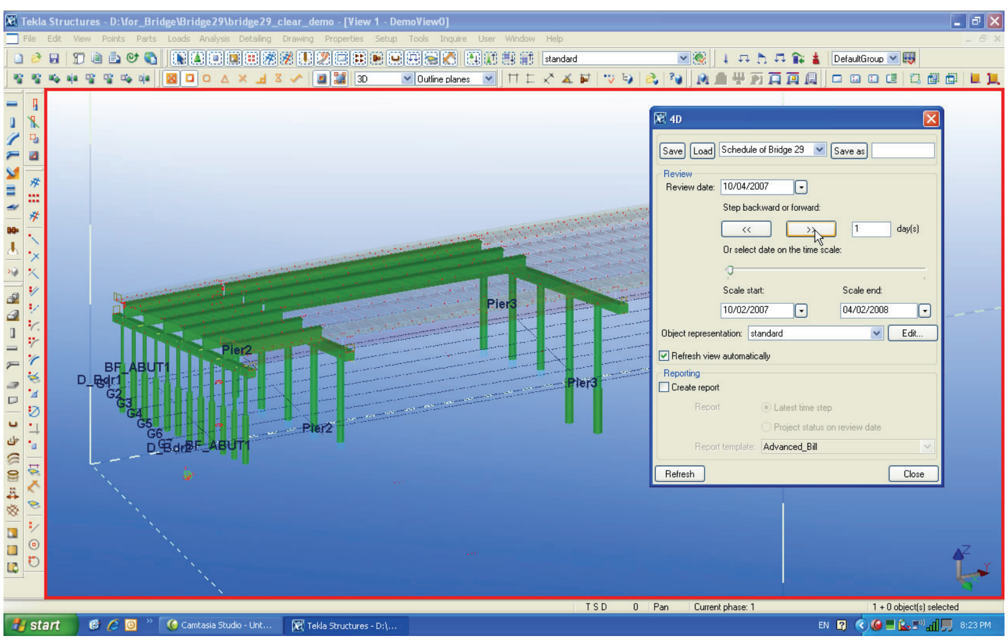


Figure 13. 4 Modeling for Erection Sequence Planning

Companion structural analysis software in some cases has the capability to selectively turn "on" or "off" not-yet-erected bridge components in order to evaluate structural safety of partially braced partially constructed conditions, or of differing deck concrete pouring sequence scenarios (e.g., in order to evaluate bearing uplift potential).

6 SUMMARY AND CONCLUSIONS

This paper has described an overview of current ongoing work to conceptualize and demonstrate key aspects of bridge information modeling (BrIM) for the lifecycle with particular incorporation of seismic concerns. This approach involves a comprehensive shepherding of bridge design, and construction data that can in turn be "handed off" to subsequent operation and maintenance data management to span the entire lifecycle. Highlights of this approach and accompanying software demonstration include the following:

- Use of a comprehensive "cradle to grave" view of the data needed to support bridge lifecycle activities, and
- Tools and technologies (e.g., XML, API's) are proliferating which make possible reliable electronic exchange of bridge data in support of lifecycle applications, although resulting solutions are then necessarily ad-hoc, and demonstrations of implemented software linkages is in the context of two 3-span straight grade separation bridges.

Advances in automation and communication technologies in recent years have been significant, but they have not yet been fully adapted and integrated with each other and then deployed to accommodate the unique requirements of the bridge construction engineering and management enterprise. Other industries have experienced significant time and cost savings when an integrated approach was utilized for project delivery (Post 2009a). Examples of other industries that have experienced significant time and cost savings are; building (GM plant, Denver Museum), ship-building (Queen Mary II completely built and on the ocean in two years). The ENR citations in the paper are intended to provide a brief pointer to other industries that have benefited from deployment of related technologies. However, the AE industry is learning and continuing to report successes, shortfalls, suggestions for implementation (Post 2009b) including discussions of adjustments to recently developed model contract language to address liability concerns (Post 2009c).

The lack of industry-wide standards for interoperability and compatibility continue to contribute to the bridge industry's lag behind the building industry in BIM/BrIM related areas. The bridge industry can expect to have experience similar to the building industry. The integrated project delivery approach (BrIM) encourages a more holistic life-cycle cost perspective when considering issues of cost. There currently exists no data upon which to make an informed statement as to the effect of BrIM on design costs. Under BrIM the actual design tasks will be no different than currently required of a bridge designer and as such should not affect the design costs. However, BrIM will enable the designer to explore and evaluate more alternative designs at much less additional cost than currently possible. Additional tasks in the application of BrIM methodology will be model creation and management through the bridge life cycle, not just the design phase, cost of which, based upon experiences in other industries, will be more than compensated through the time and cost savings during construction phase alone

7 ACKNOWLEDGEMENTS

Funding support Federal Highway Administration and assistance from its Contracting Officer's Technical Representative, Mr. Krishna Verma, is gratefully acknowledged, as is earlier support from the National Cooperative Highway Research Program and technical advice from their oversight panels. The opinions and conclusions expressed or implied in the report are those of the authors. They are not necessarily those of the Transportation Research Board, the National Research Council, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, or the individual states participating in the National Cooperative Highway Research Program. The contributions of collaborators A. Shirole', T. Riordan, J. Puckett, and current and former graduate students J. Li, R. Srikonda, V. Tangirala, R. Patil, N. Kannan, and K. Potturi are gratefully acknowledged.

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Section 3

Summary and Future Research Needs

The workshop participants discussed the specific deliverables of the MCEER research project and the importance of focusing the outcomes of this workshop to ensure that discoveries from both are included in the project deliverables. A technical monograph is planned as one of these deliverables. The monograph will include practical analysis and design guidelines that encompass rocking and CIP-emulative connection philosophies and approaches for the design of segmental columns of typical bridges with precast configurations, and their interconnections for accelerated bridge construction in seismic regions. These guidelines will be supplemented by the results from shake table experiments of the bridge system being conducted by MCEER and illustrated in the form of complementary worked step-by-step design examples.

The workshop assembled a diverse international group of researchers, who presented work-in-progress research and joined together in discussions that addressed various issues at the intersection of modular precast concrete accelerated bridge construction and seismic analysis and design. Several of the research advances presented have been deployed in actual bridge construction projects on at least a trial basis. Recommendations for further research are provided below based on the group discussions.

Key challenges and issues that must be addressed in order to eventually achieve widespread deployment of the types of technologies presented at this workshop include the following:

- The prevalence of existing code provisions that make "CIP-emulative" approaches easier to implement than alternative (e.g., "rocking-column" "ED-bars") approaches.
- Reliable characterization and performance of cyclic behavior of gap-opening-and-closing behavior.
- Combining the multiple approaches at the designer's disposal into a comprehensive and integrated framework for future revisions of the *AASHTO Guide Specifications* for *LRFD Seismic Bridge Design* and the *AASHTO Guide Specifications for Seismic Isolation Design* to incorporate rocking-column approaches with and without supplemental damping (e.g., as provided by ED-bars).
- Keep the "door open" in any newly proposed guidelines for engineering practice to incorporate multiple hazards considerations in the future.

Recommendations for future research and development include the following:

- Solicit greater input from the design and contracting communities in the assessment of design and detailing concepts (e.g., for constructability and economic assessment), to build upon their involvement in the monograph's companion design examples document.

- Include local connection-oriented studies to sufficiently populate the database(s) of low-cycle fatigue behavior of each key component of capacity-designed elements in the seismic load path of each type of seismic force resisting system for typical bridges (there are currently three types).

- Incorporate studies oriented toward global/system performance, understanding system behavior and simplified methods that are suitable for design offices to allow designers to efficiently explore the principal options available for using seismically-resilient ABC for a given crossing.

Appendix A
Workshop Participants

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INTERNATIONAL SPECIALTY WORKSHOP ON SEISMIC CONNECTION DETAILS FOR SEGMENTAL BRIDGE CONSTRUCTION SEATTLE, WASHINGTON, JULY 22 – 24, 2009

Workshop co-chairs: W. Philip Yen and George C. Lee
Workshop recorders: Stuart Chen and Il-Sang Ahn

Workshop Objectives

- Share views and knowledge among experts on state-of-research and state-of-practice on seismic connection details of segmental bridges.
- Seek advice from experts on the focus and contents of the proposed MCEER monograph on seismic ABC.
- Develop a workshop report to consist of short papers (4-8 pages each) on analysis, design, construction, case studies, examples, references on connection details of segmental bridges in seismic regions.

Tentative Agenda

Tuesday, July 21 - Arrival

Wednesday, July 22 (meeting room TBD)

7:00 – 8:00 am		Continental breakfast and registration* **** All presenters provide presentation material to be loaded in the presentation laptop.***	
8:00 – 9:30 am		**Session I:**	
	8:00 – 8:15	Introduction, Welcome and Agenda	Yen/Lee
	8:15 – 8:30	Opening Remarks	Myint Lwin
	8:30 – 9:00	Overview of MCEER ABC Project and Objectives of Workshop	Yen/Lee
	9:00 – 9:30	MCEER Monograph on Seismic ABC	Aref
	9:30 – 10:00	General Discussion, revision of agenda	Yen/Lee
10:00 – 10:20 am		Break	

10:20 – 12:00 pm	**Session II:**	
10:20 – 10:35	Design of Precast Concrete Bridge Structure	Jun-ichi Hoshikuma
10:35 – 10:45	Q & A	
10:45 – 11:00	Precast Segmental Concrete Bridge Columns with High Performance Steel Rebar as ED Bars	Yu-Chen Ou
11:00 – 11:10	Q & A	
11:10 – 11:25	A Study on Restorable Precast Prestressed Hybrid Piers	Yoshihiko Taira
11:25 – 11:35	Q & A	
11:35 – 11:50	Development of Design Method of Precast Segmental Concrete Bridge Column	Junichi Sakai
11:50 – 12:00	Q & A	
12:00 – 1:20 pm	Lunch Break (Group Photo)	
1:20 – 3:00 pm	**Session III:**	
1:20 – 1:35	Analytical and Experimental Investigation of Precast Segmental Bridge System	Amjad Aref
1:35 – 1:45	Q & A	
1:45 – 2:00	Research and Application of Precast Segmental Concrete Bridge Columns in Taiwan	Kuo-Chun Chang
2:00 – 2:10	Q & A	
2:10 – 2:25	Univ. of Wash. Research Activities on Seismic ABC	John Stanton/Marc Eberhard
2:25 – 2:35	Q & A	
2:35 – 2:50	Presentation by Washington DOT	Jugesh Kapur/ Bijan Khaleghi
2:50 – 3:00	Q & A	
3:00 – 3:20 pm	Break	
3:20 – 5:00 pm	**Session IV:**	
3:20 – 3:35	Segmental Bridge Columns with Damage-Free Plastic Hinges	Saiid Saiidi
3:35 – 3:45	Q & A	
3:45 – 4:00	Design of Integral Abutment in Japan	Masahiro Shirato
4:00 – 4:10	Q & A	
4:10 – 4:40	IT Roles in Seismic ABC	Stuart Chen
4:40 – 4:50	Q & A	
4:50 – 5:30 pm	**Summary of Day One and Agenda of Day Two**	Aref and Chen
6:30 pm	Dinner (hosted by MCEER)	

Thursday, July 23

8:00 – 8:30 am	Continental breakfast	
8:30 – 10:00 am	**Discussion Session I**	Aref and Chen
10:00 – 10:30 am	Break	
10:30 – 12:00 pm	**Discussion Session II**	Aref and Chen
12:00 – 1:30 pm	Lunch	
1:30 – 3:00 pm	**Wrap-up, Summary and Conclusion**	Lee and Yen
3:00 pm	Adjournment	

Friday, July 24

Optional Post-Workshop Technical Tour – WADOT
(Details will be available.)

Appendix C
Presentations

Welcome/Overview
George Lee

Development of Monograph on Seismic Accelerated Bridge Construction
Amjad Aref

Recent Research Activities for Seismic Design of Segmental Concrete Bridge Columns
Jun-ichi Hoshikuma

Precast Segmental Concrete Bridge Columns with High Performance Steel Rebar as ED Bars
Yu-Chen Ou

A Study on Restorable Precast Prestressed Hybrid Piers
Yoshihiko Taira

Development of Design Method of Precast Segmental Concrete Bridge Columns
Junichi Sakai

Analytical and Experimental Investigation of Precast Segmental Bridge System
Amjad Aref

Research and Application of Precast segmental Concrete Bridge Columns in Taiwan
Kuo-Chun Chang

Accelerating Bridge Construction in Regions of High Seismicity
John Stanton and Marc Eberhard

Seismic Connection of Precast Concrete Bridges in Washington State
Jugesh Kapur and Bijan Khaleghi

Segmental Bridge Columns with Damage-Free Plastic Hinges
S. Motaref, M. Saiidi and D. Sanders

IT Roles in Seismic ABC
Stuart Chen

International Specialty Workshop on Seismic Connection Details for Segmental Bridge Construction
Seattle, Washington, July 22 – 24, 2009

Workshop Agenda

Tuesday, July 21 - Arrival

Wednesday, July 22

7:00 – 8:00 am		Continental breakfast and registration* * All presenters provide presentation material to be loaded in the presentation computer.	
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	9:30 – 10:00	General Discussion, revision of agenda	Yen/Lee
10:00 – 10:20 am		Break	

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop on Seismic Connection Details for Segmental Bridge Construction

Workshop Agenda (Cont'd)

Wednesday, July 22 (cont'd)

10:20 – 12:00 pm		**Session II:**	
	10:20 – 10:35	Design of Precast Concrete Bridge Structure	Jun-ichi Hoshikuma
	10:35 – 10:45	Q & A	
	10:45 – 11:00	Precast Segmental Concrete Bridge Columns with High Performance Steel Rebar as ED Bars	Yu-Chen Ou
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	11:50 – 12:00	Q & A	
12:00 – 1:20 pm		Lunch Break (Group Photo)	

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop on Seismic Connection Details for Segmental Bridge Construction

Workshop Agenda (Cont'd)

Wednesday, July 22 (cont'd)

1:20 – 3:00 pm		**Session III:**	
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	2:00 – 2:10	Q & A	
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	2:25 – 2:35	Q & A	Marc Eberhard
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4:35 - 5:15 pm		**Summary of Day One and Plans for Day Two**	Aref and Chen
		6:30 pm Dinner (hosted by MCEER)	

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop on Seismic Connection Details for Segmental Bridge Construction

Workshop Agenda (Cont'd)

Thursday, July 23

8:00 – 8:30 am	Continental breakfast	
8:30 – 10:00 am	**Discussion Session I**	Aref and Chen
10:00 – 10:30 am	Break	
10:30 – 12:00 pm	**Discussion Session II**	Aref and Chen
12:00 – 1:30 pm	Lunch	
1:30 – 3:00 pm	**Wrap-up, Summary and Conclusion**	Yen and Lee
3:00 pm	Adjournment	

Friday, July 24

Optional Post-Workshop Technical Tour – WADOT
(Details will be available.)

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

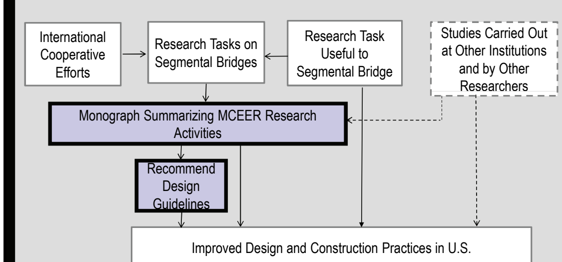
Overview of MCEER Research Project on Accelerated Bridge Construction

- A SAFETEA LU Project Funded by FHWA (2006 – 2011)
- Topic: Behavior, Design and Construction of Concrete Segmental Bridges in Seismic Regions.
- Focus: Repetitive Bridge Construction Projects that Lend Themselves to Standardization of Structural Components and Installation

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

MCEER Project Deliverables



MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

MCEER Research Tasks

- Substructures (US-Taiwan Cooperative Project)
- Superstructures
- Systems Performance (Shaking Table)
- IT Systems for Accelerated Design/Fabrication/Construction
- Scaling Issues
- Seismic Isolation Systems
- Energy Dissipation Systems
- International Cooperation and Knowledge Sharing
- Development of Monograph/Design Guidelines

Note: (1) Superstructure – Substructure connections (Bent-Cap) considered by NCHRP 12-74.
(2) Connection details between segments are considered within each task.

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

MCEER Research Tasks Described in this Workshop

- Presentations by MCEER Researchers
  - Substructures (A US-Taiwan Cooperative Research Project) (Lee – US side PI)
  - Tentative Outline of Monograph (Aref)
  - Superstructures and Systems Performance (Aref)
  - IT System for ABC (Chen)
- Presentation on substructures by NCREE Researchers
  - Use of Seismic Isolation Bearings (Chang–Taiwan side PI)
  - Connection Details for Segmental Piers (Ou–Taiwan side Co-PI)

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

Targeted Publication Schedule of Monograph

- October 2009: Firm Up Detailed Table of Contents and introductory chapters.
- April 2010: Complete 1st (80%) Draft.
- August-Sept 2010: Complete second (95%) draft and distribute it to the Professional Community (or through an international workshop) for comments.
- June 2011: Submit Final (100%) Manuscript and draft design guidelines to FHWA.

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

Workshop Objectives

- Share views and knowledge among experts on state-of-research and state-of-practice on seismic connection details of segmental bridges.
- Seek advice from experts on the focus and contents of the proposed MCEER monograph on seismic ABC (to be described in the next presentation).
- Develop a workshop proceedings (MCEER Technical Report) to include ppts and 4-8 page papers of workshop participants.

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

The Workshop Proceedings

- The short papers will have a title, authors/affiliation and a brief description of issues/problems/solutions/case studies and relevant references. Each participant may prepare more than one abstract.
- Additional short papers by other knowledge/ experienced individuals recommended by the participants may also be recruited.
- All short papers are due by September 15 for publication of the workshop proceedings in October 2009.

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

Workshop Format (strawman)

Day One

Session I
- Overview of MCEER ABC Project and deliverables
- Discussion of Workshop objectives and format

Sessions II, III, IV
- Contributions from participants
- Summary of important information presented

Day Two

Discussion Session I: Brain Storming
- Discuss/prioritize important performance/design parameters.
- Establish what we know and what we need to know.

Discussion Session II: Organization and Contents of workshop proceedings
- Additional (invited) short papers?

Summary and Conclusions

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

US-Taiwan Cooperative Research on Pre-Cast Segmental Concrete Bridge Piers in Seismic Regions (MCEER Task On Substructures)

- Funding: US-FHWA/ MCEER/ NYS; Taiwan-NSC/ NCREE/ Industry
- Lead Institution: Theory – Both Sides
  - Experiments – Small Scale – MCEER
  - Large Scale – NCREE
  - Two Shaking Tables – MCEER
  - Field Monitoring – NCREE
- Publication: Joint Authorship
- Design Guidelines: MCEER ⟶ US
  - NCREE ⟶ Taiwan

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

US-Taiwan Cooperative Research on Pre-Cast Segmental Concrete Bridge Piers in Seismic Regions

Planning of Cooperative Research

- **Identify important performance/design parameters based on:**
  - Constructability
  - Seismic capacity (energy dissipation) and ductility
  - Durability (impact strength, corrosion resistance)
  - Maintainability (inspection, repair, replacement, etc.)
- **Selected Foci:**
  - Ductility
  - Isolation
  - High performance rebars
  - High performance concrete

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

US-Taiwan Cooperative Research on Pre-Cast Segmental Concrete Bridge Piers in Seismic Regions

Four Phases

- Phase I 2005 – 2008
- Phase II 2008 – present
- Phase III 2008 – present
- Phase IV 2009 -

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

US-Taiwan Cooperative Research on Pre-Cast Segmental Concrete Bridge Piers in Seismic Regions (contd)

- Phase I: Developed hollow rectangular column segments with conventional rebars across joints (MCEER – NCREE).
- Phase II: Study the performance of segmental columns without rebars across the joints by using isolation bearings (MCEER – NCREE).
- Phase III: Use high performance (high strength stainless steel, etc.) rebars across the joints (MCEER – NCREE).
- Phase IV: Use high performance concrete (high strength concrete, steel fiber concrete, etc.) to dissipate impact energy and increase shear strength, with and without rebars across the joints (MCEER – NCREE – Houston).

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

US-Taiwan Cooperative Research on Pre-Cast Segmental Concrete Bridge Piers in Seismic Regions (contd)

Current Status

- Phase I: Completed
- Phase II: Started with NCREE taking the lead (Professor Chang presentation).
- Phase III: Currently active both at MCEER and at NCREE (Professor Ou presentation).
- Phase IV: Work started at MCEER, NCREE and U. Houston (Professor Y. L. Mo).

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Specialty Workshop

Next Presentation

MCEER Monograph on Seismic ABC: Abstract and Content

by
Dr. Amjad Aref
University at Buffalo

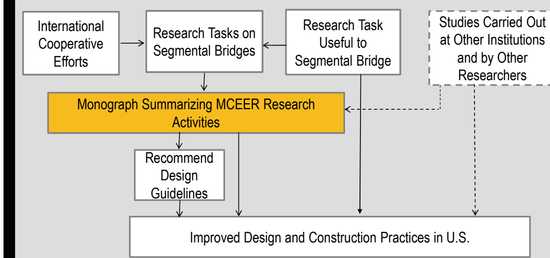
MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

Development of Monograph on Seismic Accelerated Bridge Construction

Amjad J. Aref
Associate Professor
Department of Civil, Structural and Environmental Engineering
University at Buffalo – The State University of New York at Buffalo

International Specialty Workshop on Seismic Connection
Details for Segmental Bridge Construction
Seattle, Washington, July 22 – 24, 2009

Project Deliverables



MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

Overview of Monograph Task Objectives and Scope

- The monograph is the deliverable associated with the SAFETEA LU Project funded by FHWA (2006 – 2011)
  - Objectives: To study the behavior, design and Construction of concrete segmental bridges in seismic regions
  - Scope: Repetitive and accelerated bridge construction (ABC) projects with emphasis on segmental precast bridge components and systems

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

Topical Outline of the Monograph

- Introduction, Scope and Objectives
- Precast Concrete Components and Systems in ABC
- Bridge Information Systems for Accelerated Bridge Construction
- Superstructure
- Substructure
- Connections
- Seismic Design Principles
- Seismic Analysis and Design Procedures
- Summary and Future Research
- Appendices

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

Precast Concrete Components and Systems in ABC

- This section focuses on scanning available and typical precast concrete systems that fit in with the accelerated bridge construction concepts.

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

Bridge Information Systems for Accelerated Bridge Construction

- Envisioned integrated design and construction process and benefits
- Bridge information modeling for accelerated project delivery
- Discussion and evaluation of the methodology
- Implications of the envisioned methodology on practice

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

Superstructure

- Precast concrete deck panels
- Precast concrete girders
- Precast segmental box girders
  - Analytical studies
  - Segmental bridge system testing

Substructure

- Precast Segmental Columns
  - With Rebar across joints
  - Without Rebar across joints
- Precast bent caps
- Integral caps
- Non-integral caps
- Other types of precast substructures
- Connection details

Connections

- Segments within Superstructure
- Superstructure to Substructure
- Segments within Substructure
- Substructure to Foundation
- Other

Seismic Design Principles

- Design Principles
- Force-Based Approach
- Displacement-Based Approach
- Base Isolation
- Other

Seismic Analysis and Design

- Seismic Analysis Methods
- Design Procedures
- Design Examples: Overview and Highlights

Summary and Future Research

Appendices

- Literature Review
- Framework for Design Guidelines and Commentary
- Illustrative Design Examples
- Segmental Bridge System Experiment
- BrIM Examples

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

Thank you

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

International Workshop on Seismic Connection Details for Segmental Bridges Construction

RECENT RESEARCH ACTIVITIES FOR SEISMIC DESIGN OF SEGMENTAL CONCRETE BRIDGE COLUMNS

Jun-ichi HOSHIKUMA

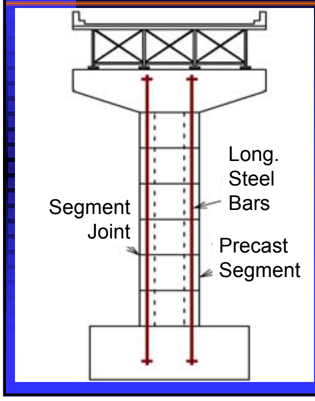
Chief Researcher

Center of Advanced Engineering Structural Assessment and Research,
Public Works Research Institute

SEGMENTAL CONCRETE COLUMNS (Precast Concrete Columns)

- Background
  - ・Performance-based Design Concept
  - ・Increase of Applications of New Materials, New Designs, and New Structures with Necessary Performance Verifications
- Precast Columns with Combination of High Strength Materials
  - ・Better-Quality Precast Members produced at Factory
  - ・Improvement of Constructionability at Sites and Shortening of Construction Period

STRUCTURE OF PRECAST COLUMNS

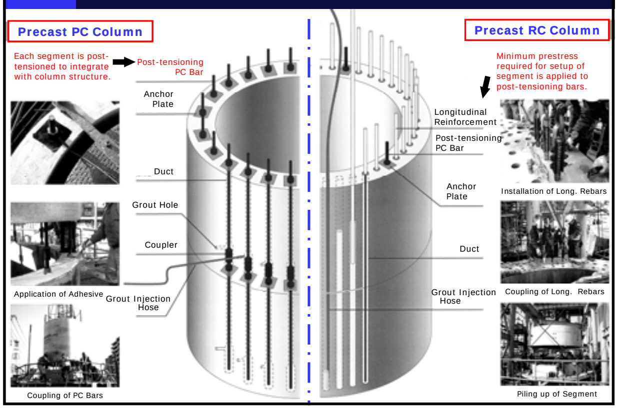


- Transport of Segments from Factory or on-site
- Pile-up of Segments
- Connect Segments by Longitudinal Steel Bars
- Advantages

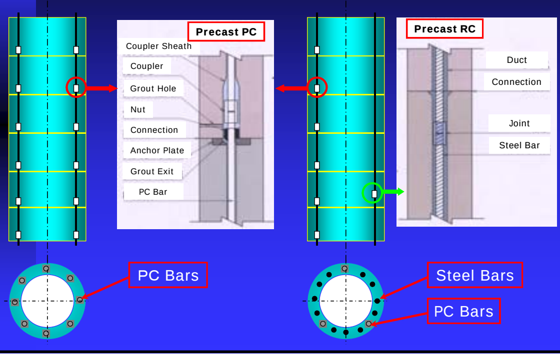
→ Shorten the Construction Period at Sites without formwork, placement and curing of concrete

→ So, expected to be applied for Overpass Crossings in the urban areas to minimize the effect on existing traffic

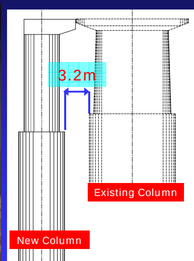
Concept of Connection Details in Precast PC Columns and Precast RC Columns



Basic Connection Details of Precast PC Column / Precast RC Column



Precast RC Oval Column Construction at Close Existing Column



Connection of Precast Segmental PC Piles with Cast-in-place Footing

Precast Segmental RC Shaft Constructed on Coastal Site

2-Years Joint Research on Precast Segmental Columns

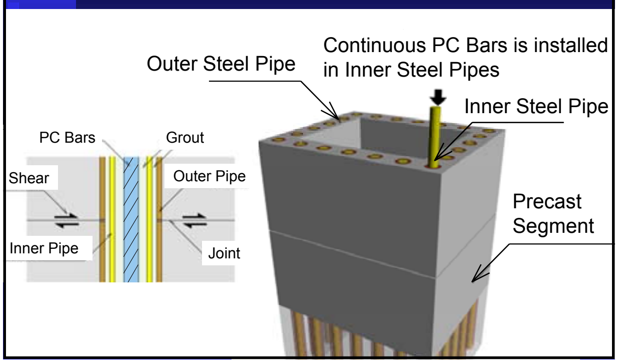
- **Objectives**
  - Though there are many existing segmental PC/RC shaft for bridge substructures, inelastic seismic performance of those structures is controversial due to lack of test data and technical information.
  - Needs of Study on Failure Mechanism, Strength and Ductility Performance
- **2-Years Joint Research on Seismic Design Methods for Precast Segmental Columns**
  - PWRI, Kajima Co., Mitsui-Sumitomo Co., PS Mitsubishi Co.
  - Proposed Precast Segmental Columns Details

Research Topics for Precast Segmental PC/RC Columns

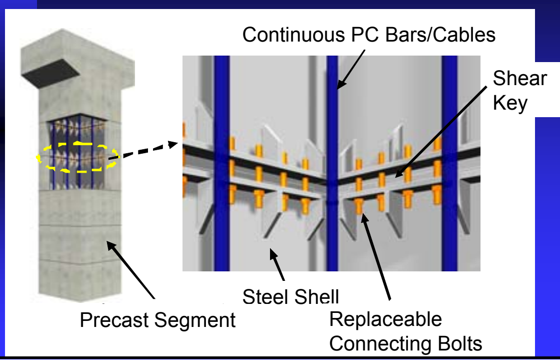
- **Research Issues**
  - Failure Mechanism, Strength and Ductility Performance, Dynamic Behavior
  - Design Methods:
    1) Limit States to Achieve Necessary Seismic Performance
    2) Design Methods including Segments, Joints, PC Bars, Bending–Shear Resistance Evaluation
    3) Design Details
    4) Construction Methods

  → Material Tests, Cyclic Loading Tests, Shaking Table Tests, Joint Shear Test, Analytical Study

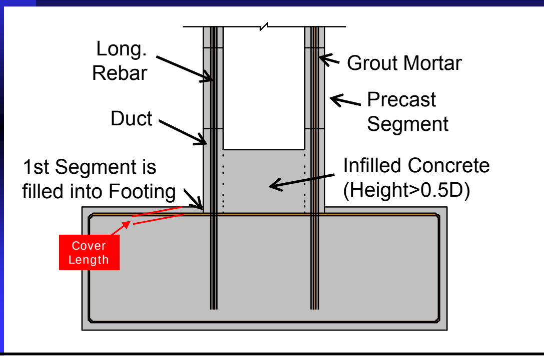
Example#1 of Precast PC Columns <Kajima Co.>



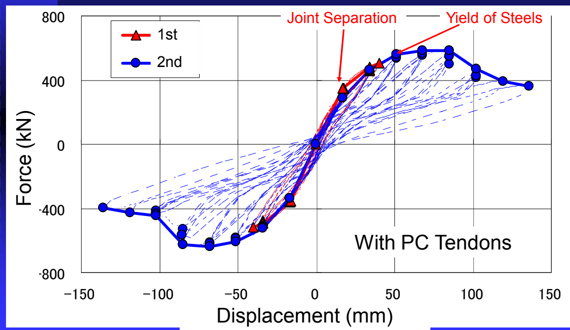
Example#2 of Precast PC Columns <Mitsui-Sumitomo Co.>



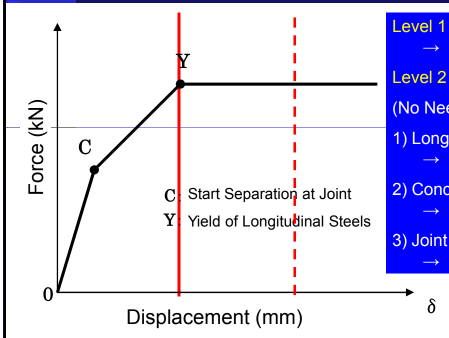
Example#3 of Precast RC Columns <PS-Mitsubishi Co.>



Cyclic Loading Test Results for Precast PC Columns



CONCEPTS OF LIMIT STATES



Level 1 EQ : Limit State
→ Basically Elastic

Level 2 EQ : Limit State

(No Need of PC Repair)

1) Long. PC Steels
→ Elastic Limit

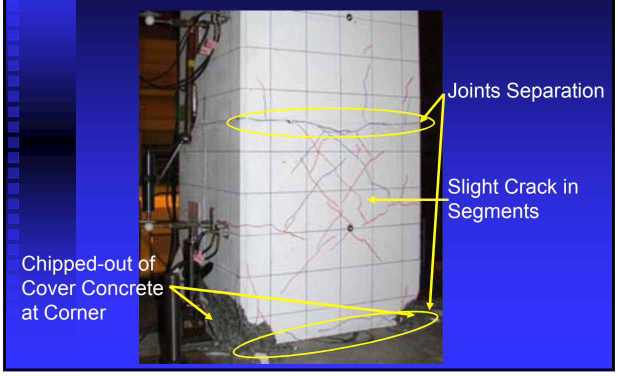
2) Concrete
→ εcu at section end

3) Joint Bolts
→ Replaceable

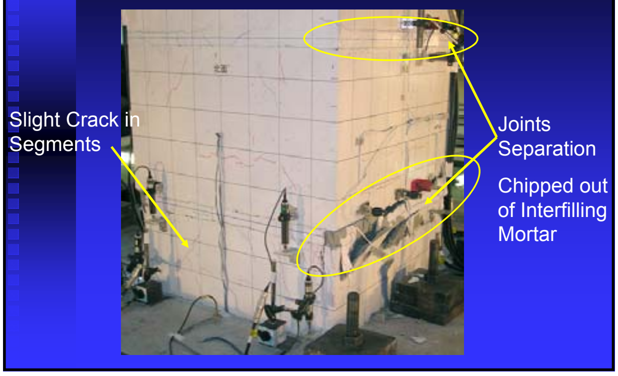
PERFORMANCE VERIFICATIONS -SHAKE TABLE TESTS-

- Objectives
  - ･Earthquake Response Characteristics and Failure Mechanism
  - ･Performance Evaluation subjected to the Design Level Earthquake and the Exceeding Level
    → Repairablity (Replace of Members and Performance of Segments and PC Bars/Cables)
  - ･Verification of Design Model

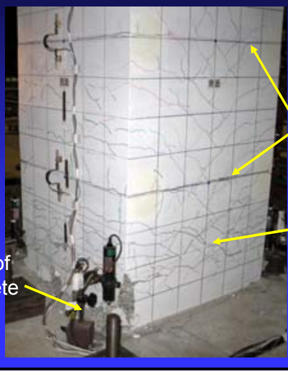
FAILURE MECHANISM -Precast PC Column #1-



FAILURE MECHANISM -Precast PC Column #2-



FAILURE MECHANISM
-Precast RC Column #3-



Joints Separation

Slight Crack in Segments

Chipped-out of Cover Concrete at Corner

SHAKE TABLE TESTS

- Tests Results
  - ・Expected Behavior and Performance
  - ・Even Against Ground Motion Exceeded Design Earthquake Level, the Columns behaved very well.
    → Deformation was developed at joints and segments.
  - ・Design Model was compared through the Simulation Analyses of Shake Table Tests Data.

DEVELOPMENT OF SESIMIC DESIGN GUIDELINES (1)

- Outcome of Joint Research
  - ・Seismic Design Guidelines is now being developed.
  - ・Expected to be published in this year
- Table of Contents

DEVELOPMENT OF SESIMIC DESIGN GUIDELINES (2)

- Table of Contents

RECENT RESEARCH ACTIVITIES FOR SEISMIC DESIGN OF SEGMENTAL CONCRETE BRIDGE COLUMNS

Thank you for kind attention.

Precast Segmental Concrete Bridge Columns with High Performance Steel Rebar as ED Bars

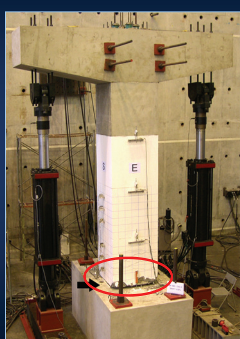
Yu-Chen Ou
Assistant Professor
Department of Construction Engineering
National Taiwan University of Science and Technology

International Specialty Workshop on Seismic Connection Details for Segmental Bridge Construction
Seattle, Washington, July 22 – 24, 2009

US-Taiwan Cooperative Research on Pre-Cast Segmental Concrete Bridge Piers in Seismic Regions

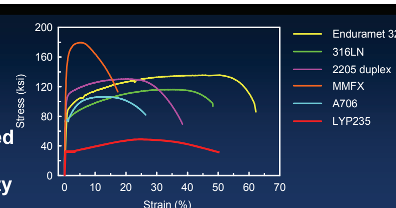
- Phase I : Developed hollow rectangular column segments with conventional rebars across joints (MCEER – NCREE).
- Phase II: Study the performance of segmental columns without rebars across the joints by using isolation bearings (MCEER – NCREE).
- Phase III: Use high performance (high strength stainless steel, etc.) rebars across the joints (MCEER – NCREE).
- Phase IV: Use high performance concrete (high strength concrete, steel fiber concrete, etc.) to dissipate impact energy and increase shear strength, with and without rebars across the joints (MCEER – NCREE – Houston).

Motivations of this research

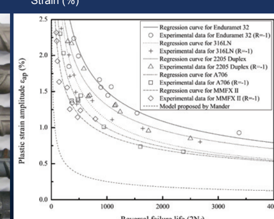


- Opening of precast joints under lateral loads can cause premature fracture of mild steel rebar crossing those joints. (for supplemental energy dissipation, ED bars)
  => unbonding or HP rebar
- Potential corrosion problems for ED bars after joints open.
  => HP rebar

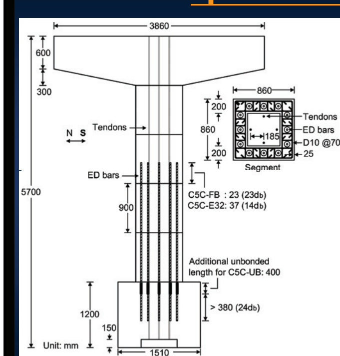
High performance rebar



- Higher expected service life
- Greater ductility



Specimen design



Cap beam

Segment

Major design parameters

Specimen	ED bar material	ED bar (Ag)	Total axial force (fc'Ag)	λED (%)	Lau (mm)
C5C-FB	A706	0.5%	0.17	25	0
C5C-E32	HP	0.5%	0.17	28	0
C5C-UB	A706	0.5%	0.17	23	400

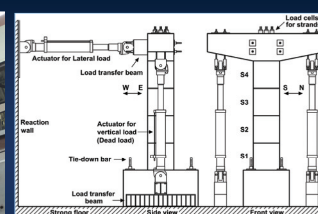
C5C-FB: A706 rebar with full bond
C5C-E32: HP rebar with full bond
C5C-UB: A706 rebar with intentionally unbonding for a length (Lau)

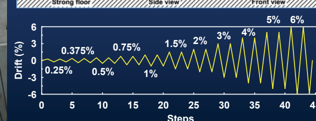
Unbonding A706 ED bars by duct tape

Assembling process

Assembling process (cont'd)

Test setup and loading protocol



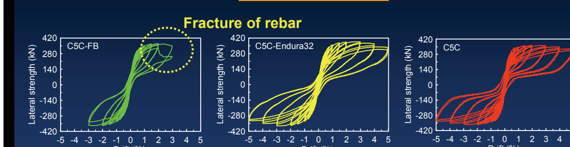


Loading protocol for cyclic loading

Column C5C-E32 at 6% drift

Opening of joint foundation-S1

Test results



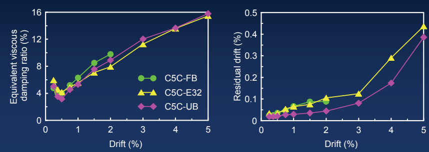
A706 ED bars without unbonding

Endura32 ED bars without unbonding

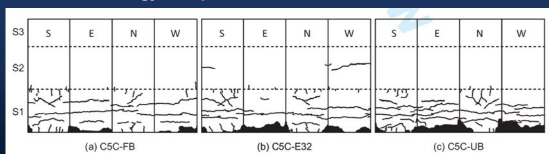
A706 ED bars unbonded for 400 mm

Specimen	ED bar ratio (%)	Total axial force (fc'Ag)	λED (%)	Drift Capacity (%)	Strength (kN)	Max ζeq (%)	Max residual (%)
C5C-FB	0.5	0.17	25	2	371	8	0.1
C5C-Endura32	0.5	0.17	28	5	386	14	0.5
C5C	0.5	0.17	23	5	363	16	0.4

Test results (cont'd)

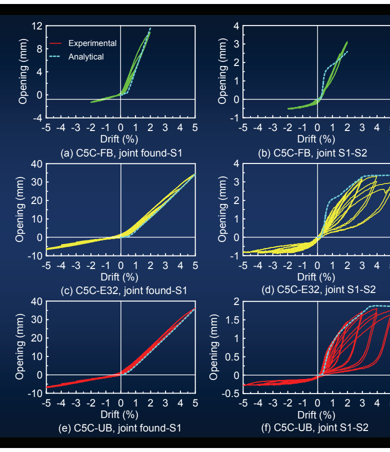


Energy dissipation **Residual drift**



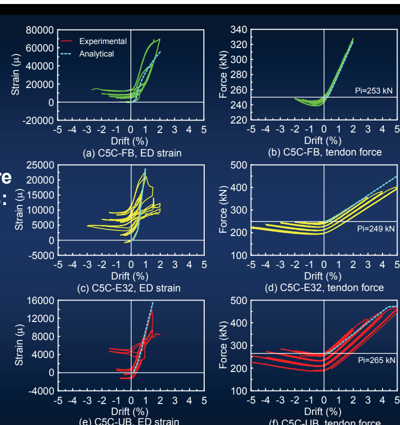
Damage distribution

Test results (cont'd)



- Joint found-S1: large opening.
- Joint S1-S2: much smaller.
- Other joints: negligible.

Test results (cont'd)



- Predicted fracture strain of ED bars:
  A706: 7%
  HP : 14%
- Prestress loss was minor.

Summary

- For the column with fully bonded A706 ED bars, five out of the six critical ED bars fractured at 3% drift. No fracture of the ED bars was observed up to 5% drift for the column with unbonded A706 steel ED bars.
- The column with HP steel ED bars (C5C-E32) was capable of sustaining cyclic loading up to 5% drift without fracture of the bars. The predicted fracture strain of HP ED bars was 14% as compared to 7% of A706 ED bars.

A STUDY ON RESTORABLE PRECAST PRESTRESSED HYBRID PIERS

Sumitomo Mitsui Construction

Yoshihiko Taira

An structural concept
for such as urban viaducts projects

- Accelerated construction to minimize the traffic restriction
  - -Precast Concrete Piers
- Restorable structure to shorten the re-opening of the traffic after large earthquake
  - - Prestressing technique would be useful

⇩

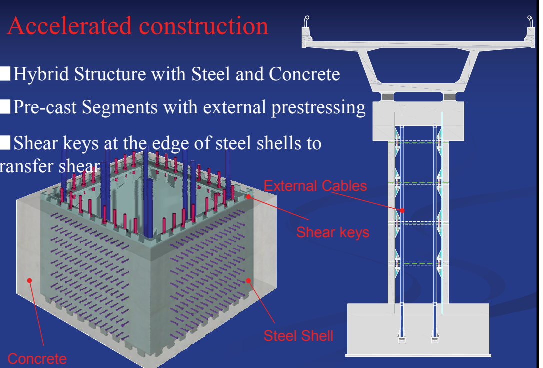
Precast & Prestressed Hybrid Pier : P&PH Pier

- Accelerated construction

■Hybrid Structure with Steel and Concrete

■Pre-cast Segments with external prestressing

■Shear keys at the edge of steel shells to transfer shear



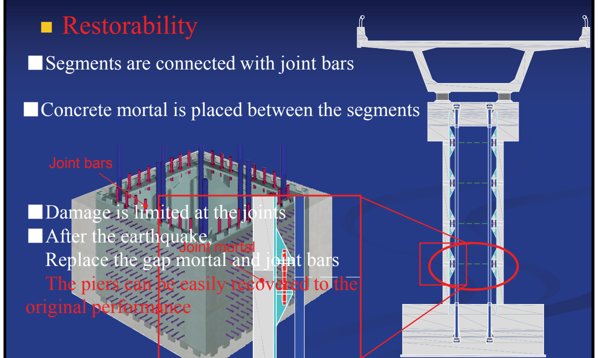
- Restorability

■Segments are connected with joint bars

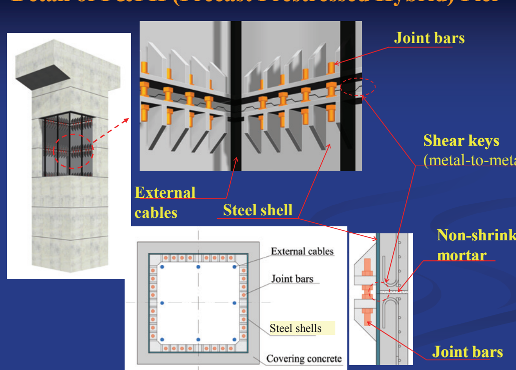
■Concrete mortal is placed between the segments

■Damage is limited at the joints

■After the earthquake
Replace the gap mortal and joint bars
The piers can be easily recovered to the original performance



Detail of P&PH (Precast Prestressed Hybrid) Pier



Construction Sequences

(1) Cast-in-situ footing with precast base segment

(2) Precast segments are stacked

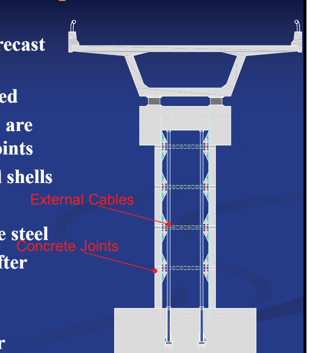
(3) Steel shells with shear keys are un-welded metal to metal joints

(4) Joint bars connect the steel shells

(5) External prestressing to the steel

(6) Concrete joints are filled after tensioning

(7) Superstructures on the pier



Limit State of P&PH Pier

Structural members	Service limit	Level-1 E.Q.	Level-2 E.Q.
Pier Base	Elastic behavior		
Pre-cast Segment	Elastic behavior		
Segment Joints, Joint bars	No opening	Less than yield	-Exceed yield -Replace the Joint bars after E.Q.
External Cables	Elastic behavior		

Cyclic Loading Test

Objectives of Cyclic Loading Test

1. Performance up to Ultimate Loading

・Confirmation of damage position, order and fracture mode

Damage is restricted to the segment joints under L2 level

→ Damage of joint bars and gap mortal
No damage in concrete and steel box

2. Confirmation of restorability

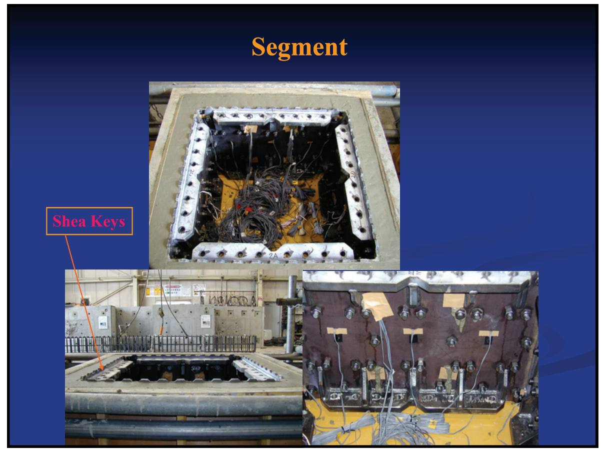
1st step : L2 level cyclic lading
2nd step : Replacement of joint bars which reached yield
3rd step : Re-loading of L2 level and confirmation of the same performance

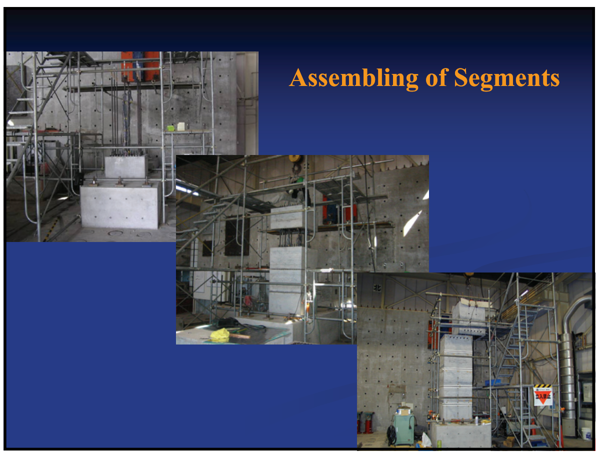
Steel Shells

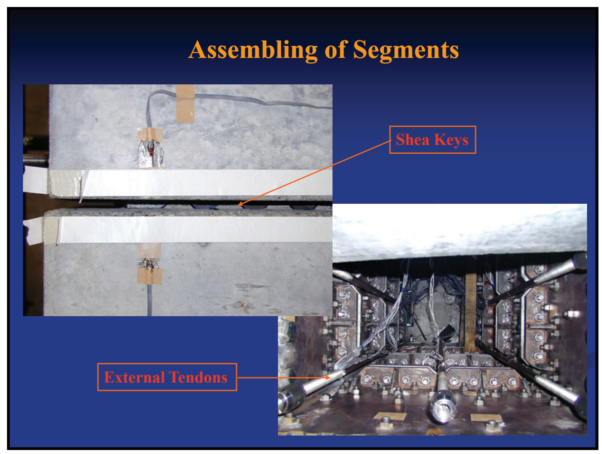
Footing and Base Segment

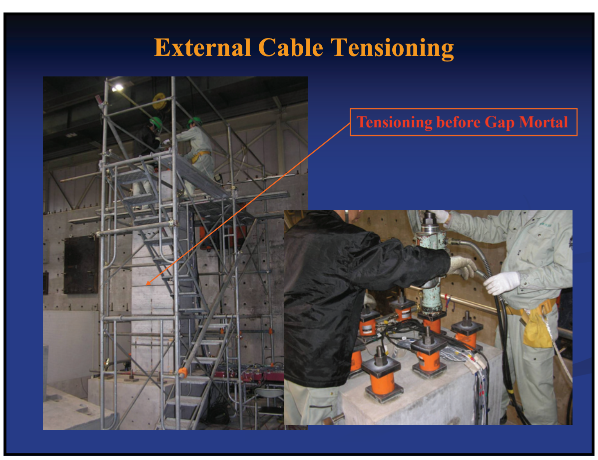
Steel Shell and Re-bar Cage


Casting Concrete


Segment
Shea Keys


Assembling of Segments

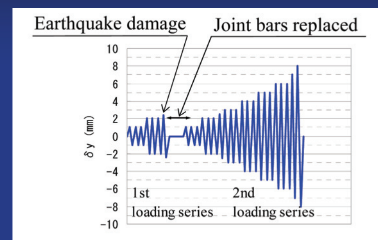

Assembling of Segments
Shea Keys
External Tendons


External Cable Tensioning
Tensioning before Gap Mortal

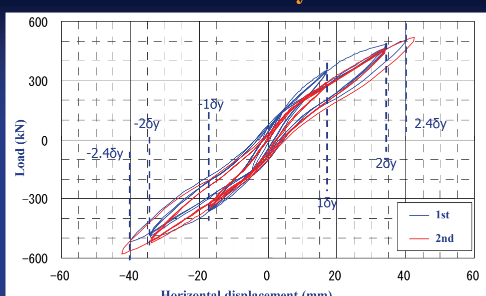

Gap Mortal

Test Equipment and Model

Cyclic Loading steps

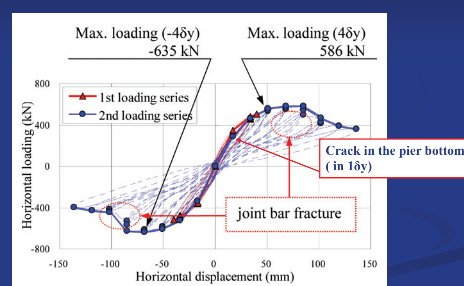


Restorability

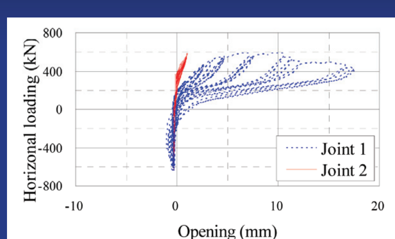


P-δ curves are almost the same

Load-Displacement Curves



Joint Opening



P&PH Pier under Ultimate Loading

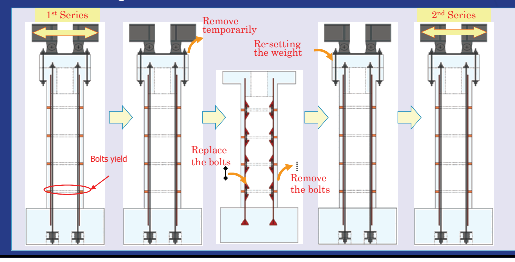
P&PH Pier after 8δy

Damage is limited at the Bottom Joint

Shake Table Test (at PWRI)

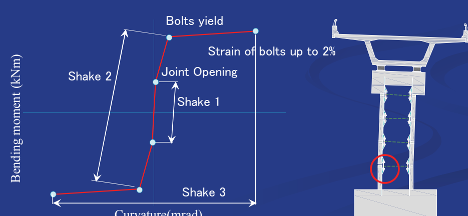
Test Outline

- 1st series; 3 levels of shaking
- 2nd series; Replace the bolts after 1st series and re-shaking with the same levels

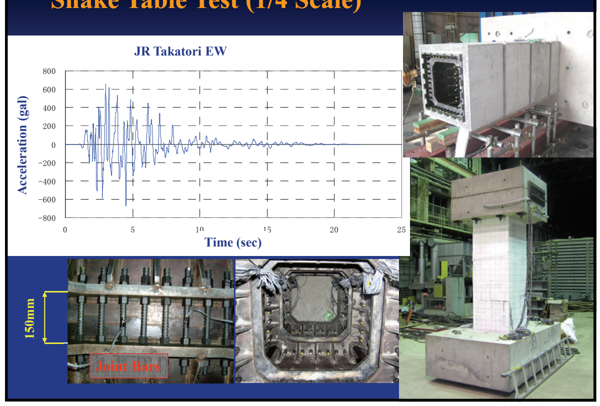


Test Outline

- 1st series; 3 levels of shaking
- 2nd series; Replace the bolts after 1st series and re-shaking with the same levels

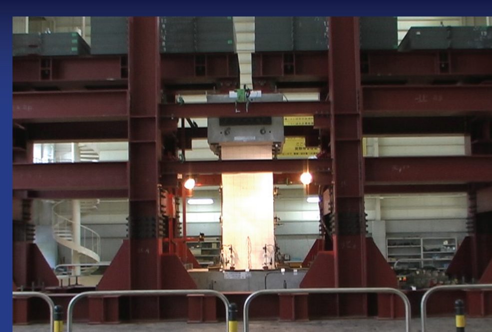


Shake Table Test (1/4 Scale)



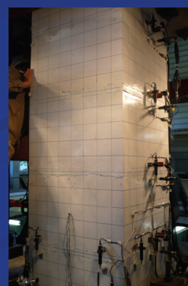
Test Specimen and Frame (at PWRI)

Shake Table Test in the 1st Step



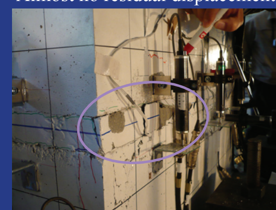
Test Results at the bottom joints in the 1st series

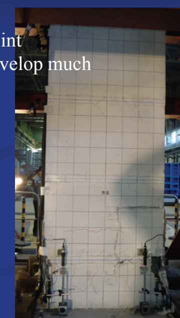
-Damage was limited at the bottom joint.
- Max. strain of joint bar is 1.5%
- Small crack at the bottom of pier
- Almost no residual displacement



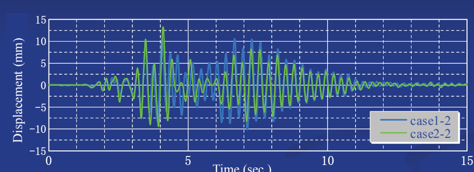
Test Results at the bottom joints in the 2nd series

-Damage was limited at the bottom joint
- Local spalling started at the bottom joint
- Crack at the bottom of pier did not develop much
- Almost no residual displacement



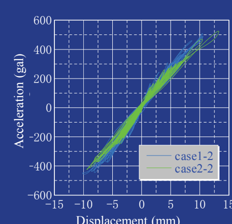


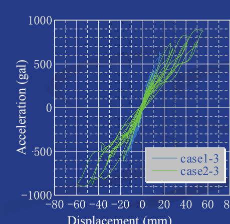
Horizontal Displacement at the top of the pier



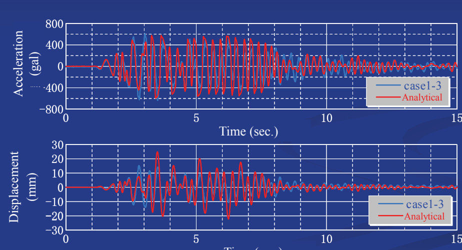
- Small residual displacement
- Similar behaviors between before and after replacement

Displacement v.s. Acceleration at the top of the pier





Displacement v.s. Acceleration at the top of the pier



Conclusions

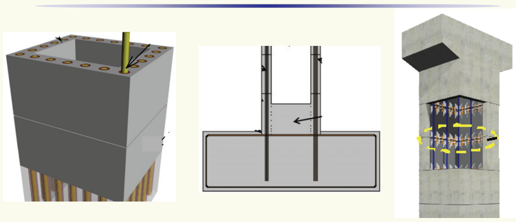
(1) **Residual displacement of P&PH pier could be limited after earthquakes.**

(2) **Although cracking occurred in the base of the pier, damage was successfully limited to the segment joint.**

(3) **Even after damage, it was possible to restore the load bearing performance by replacing joint bars.**

(4) **Performance on the shake table is needed to confirm dynamic behavior of this new structures.**

DEVELOPMENT OF DESIGN METHOD OF PRECAST SEGMENTAL CONCRETE BRIDGE COLUMNS



Junichi Sakai
Public Works Research Institute

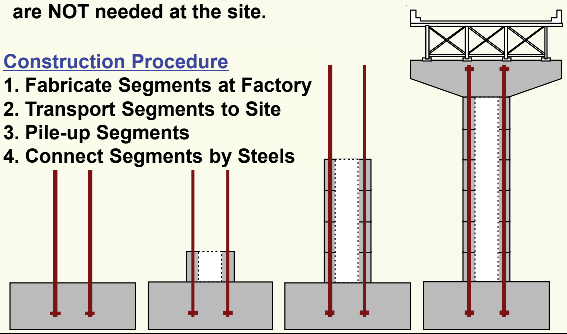
INTERNATIONAL SPECIALITY WORKSHOP ON SEISMIC CONNECTION DETAILS FOR SEGMENTAL BRIDGE CONSTRUCTION
July 22nd, 2009, Seattle, WA, USA

Application of Precast Segmental Structure to Substructure

Shorten construction period at sites because formwork, placement and curing of concrete are NOT needed at the site.

Construction Procedure

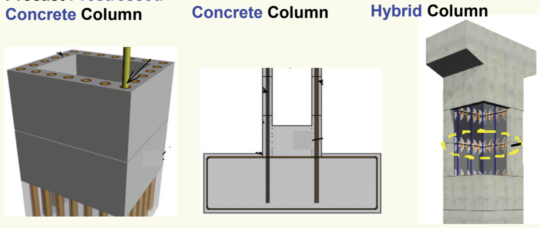
1. Fabricate Segments at Factory
2. Transport Segments to Site
3. Pile-up Segments
4. Connect Segments by Steels



2-Years Joint Research Program

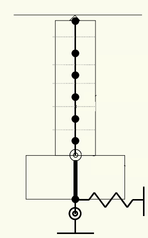
Investigate Seismic Performance of Precast Columns
- Seismic Design Method
- Seismic Details

Kajima Co.	PS Mitsubishi Co.	Sumitomo Mitsui Co.
Precast Prestressed Concrete Column	Precast Reinforced Concrete Column	Precast Prestressed Hybrid Column



Development of Seismic Design Method

Limit States
Analytical Models to Evaluate Demand & Capacity
- ✓ Cyclic Loading Test
- ✓ Shake Table Test
- ✓ Nonlinear Dynamic Response Analysis



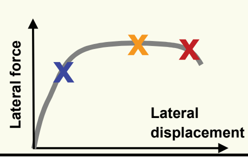
Limit States for Seismic Performance Levels

Limit State 1
Structure shall be serviceable without repair.
Members shall remain in elastic range.

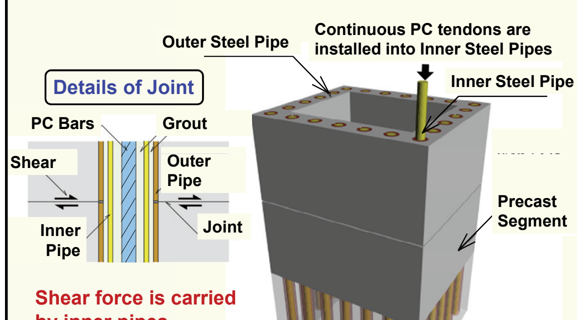
Limit State 2
Structure shall be serviceable after minor repair.
Some nonlinearity is allowed.

Limit State 3
Structure shall be safe. Repairability is NOT required.
Response shall not exceed ultimate status.

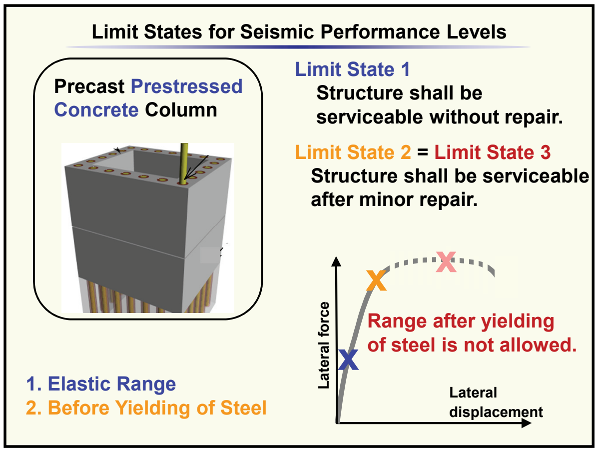
For Reinforced Concrete Members
1. Before Yielding of Rebar
2. Minor Spalling of Cover Co.
3. Before Rebar Buckling

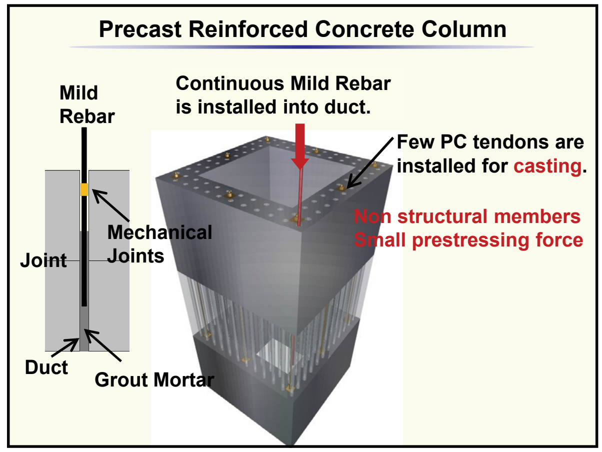


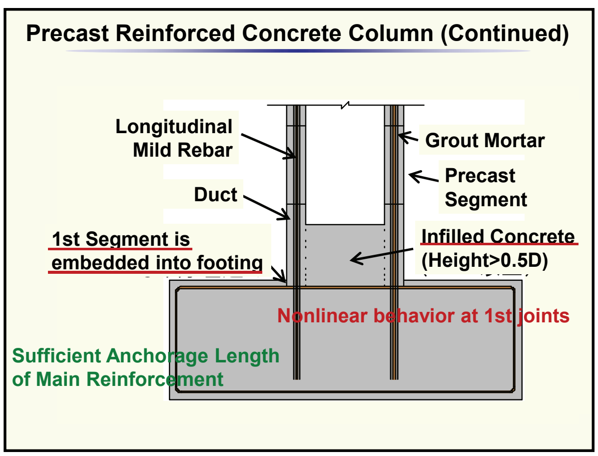
Precast Prestressed Concrete Column

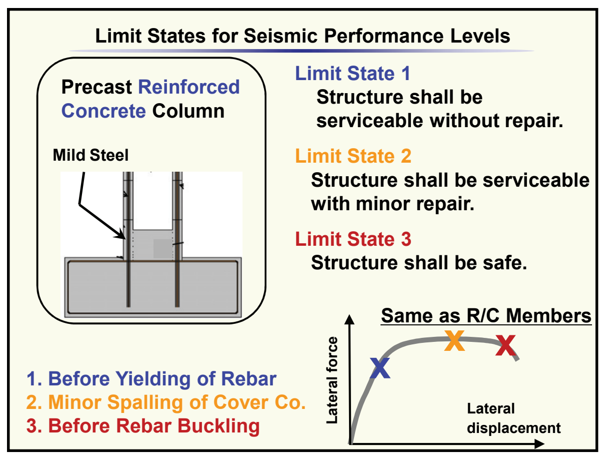


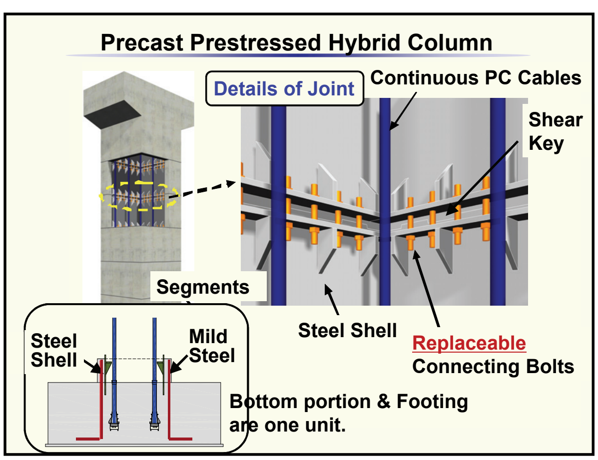
Shear force is carried by inner pipes.

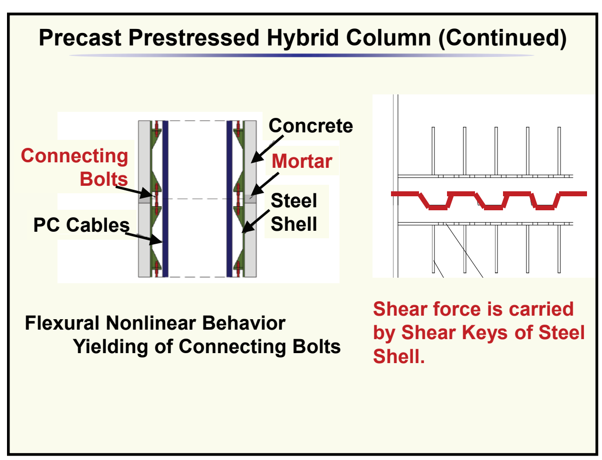

Limit States for Seismic Performance Levels
Precast Prestressed Concrete Column
Limit State 1
Structure shall be serviceable without repair.
Limit State 2 = Limit State 3
Structure shall be serviceable after minor repair.
Range after yielding of steel is not allowed.
Lateral force
Lateral displacement
1. Elastic Range
2. Before Yielding of Steel


Precast Reinforced Concrete Column
Mild Rebar
Continuous Mild Rebar is installed into duct.
Few PC tendons are installed for casting.
Non structural members
Small prestressing force
Mechanical Joints
Joint
Duct
Grout Mortar


Precast Reinforced Concrete Column (Continued)
Longitudinal Mild Rebar
Grout Mortar
Precast Segment
Duct
1st Segment is embedded into footing
Infilled Concrete (Height>0.5D)
Nonlinear behavior at 1st joints
Sufficient Anchorage Length of Main Reinforcement

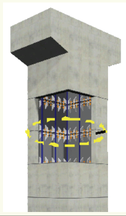

Limit States for Seismic Performance Levels
Precast Reinforced Concrete Column
Mild Steel
Limit State 1
Structure shall be serviceable without repair.
Limit State 2
Structure shall be serviceable with minor repair.
Limit State 3
Structure shall be safe.
Same as R/C Members
Lateral force
Lateral displacement
1. Before Yielding of Rebar
2. Minor Spalling of Cover Co.
3. Before Rebar Buckling


Precast Prestressed Hybrid Column
Details of Joint
Continuous PC Cables
Shear Key
Segments
Steel Shell
Replaceable Connecting Bolts
Steel Shell
Mild Steel
Bottom portion & Footing are one unit.


Precast Prestressed Hybrid Column (Continued)
Connecting Bolts
Concrete
Mortar
Steel Shell
PC Cables
Flexural Nonlinear Behavior
Yielding of Connecting Bolts
Shear force is carried by Shear Keys of Steel Shell.

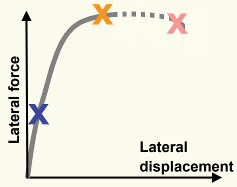
Limit States for Seismic Performance Levels

Precast Prestressed Hybrid Column



Limit State 1
Structure shall be serviceable without repair.

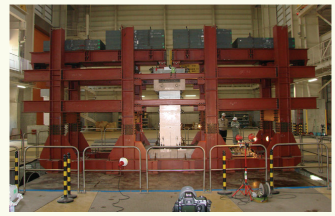
Limit State 2 = Limit State 3
Structure shall be serviceable with minor repair.
Bolts shall be replaceable.



1. Opening of Joints
2. Limit State of Bolts
   Strain of Bolts < 2%

Shake Table Tests

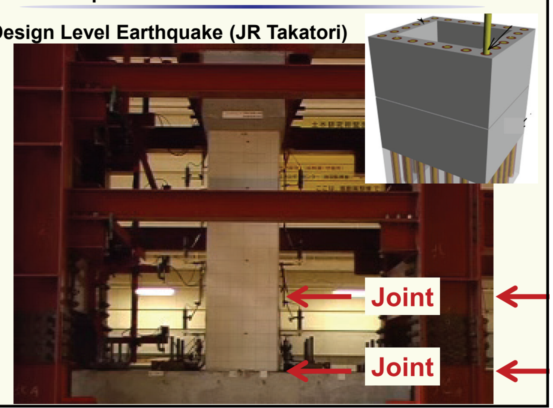
Objectives
Dynamic Response Characteristics
Failure Mechanism
Data for Analytical Model Calibration



- Elastic Level Test
- Design Level Test
- Maximum Level Test

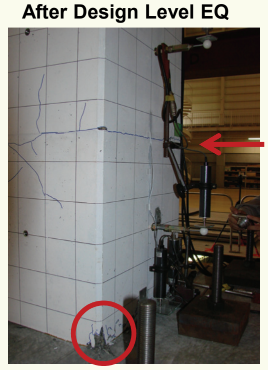
Dynamic Response of Precast Prestressed Concrete Column

Design Level Earthquake (JR Takatori)

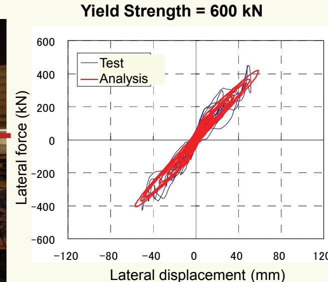


Response of Precast Prestressed Concrete Column

After Design Level EQ



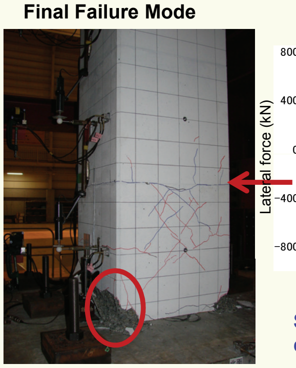
Yield Strength = 600 kN



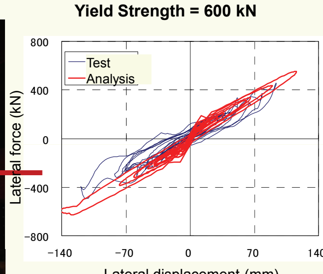
PC tendons remained in elastic range.

Response of Precast Prestressed Concrete Column

Final Failure Mode



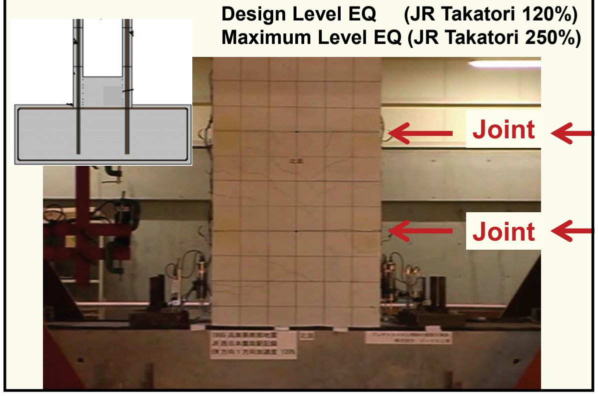
Yield Strength = 600 kN



Stiffness decreased due to unbonding of tendon.

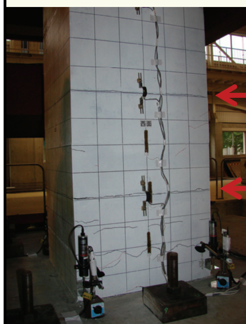
Dynamic Response of Precast Reinforced Concrete Column

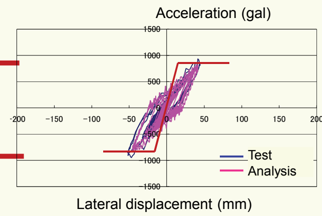
Design Level EQ (JR Takatori 120%)
Maximum Level EQ (JR Takatori 250%)



Response of Precast Reinforced Concrete Column

After Design Level EQ

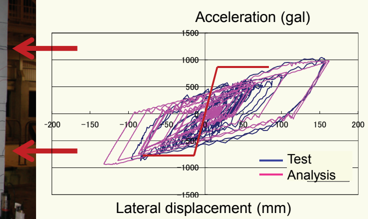




Minor Cracks

Failure Mode of Precast Reinforced Concrete Column

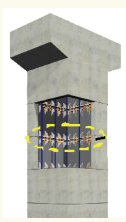
Final Failure Mode

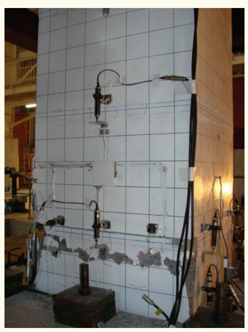


Minor Cracks
Minor Spalling of Cover Co.
No Buckling Occurred

Dynamic Response of Precast Prestressed Hybrid Column

1st Series → Repair → 2nd Series



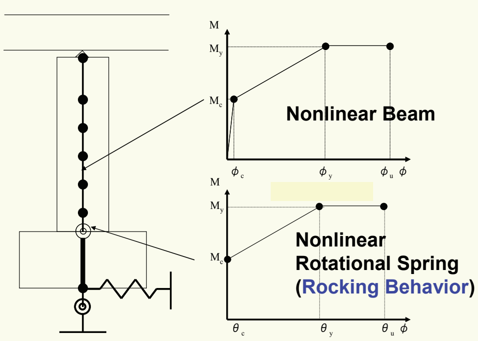


Conclusions

- To apply precast segmental structure to bridge substructures, seismic design method was developed through cyclic loading tests, shake table tests and nonlinear dynamic analysis.
- Limit states for seismic performance levels have been determined.
- Analytical models were calibrated with shake table test data.
- Seismic design guidelines is to be published soon.

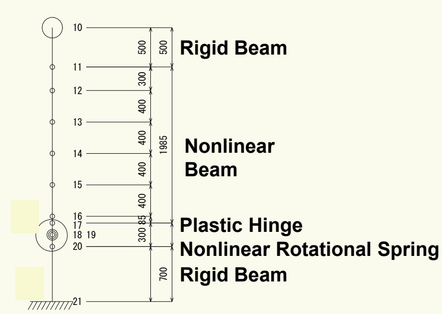
Analytical Models

Precast Prestressed Concrete Column



Analytical Models

Precast Reinforced Concrete Column



Analytical and Experimental Investigation of Precast Segmental Bridge System

Amjad J. Aref, Ph.D.
Associate Professor
Department of Civil, Structural, and Environmental Engineering
University at Buffalo – The State University of New York

International Specialty Workshop on Seismic Connection Details for Segmental Bridge Construction
Seattle, Washington, July 22 – 24, 2009

TEAM MEMBERS

- **Petros Sideris**
  Ph.D. Candidate
- **Myrto Anagnostopoulou**
  Senior Structural Engineer, SEESL
- **Amjad J. Aref**
  Associate Professor
- **Andre Filiatrault**
  Professor

OVERVIEW

- Introduction
- Analytical Study
  - Theory
  - Application
- Experimental Study
  - Accelerated Bridge Construction
  - Prototype Bridge
  - Shake table testing of bridge model

INTRODUCTION

- Analytical study
  - Development of a simplified flexibility-based macro-element capable of capturing:
    - Gap opening between segments in segmental systems
    - Rocking of rocking systems, i.e. rocking columns
  - Obtain similar analysis results with FE models with less computational effort and time
- Experimental study
  - Design of a single span bridge specimen for testing in the SEESL at UB
  - Investigate the response of a complete precast segmental bridge system designed using Accelerated Bridge Construction (ABC) concepts under seismic excitations

ANALYTICAL STUDY

- Theory
  - Derivation of Section vs. Deformation relationships in an incremental form:

$$\begin{Bmatrix} \Delta N(x) \\ \Delta M_z(x) \\ \Delta V(x) \end{Bmatrix} = [K_s(x)] \begin{Bmatrix} \Delta \varepsilon_{xx,0}(x) \\ \Delta \phi_z(x) \\ \Delta \gamma_{xy}(x) \end{Bmatrix} + \{r_s(x)\} \Leftrightarrow \{\Delta D(x)\} = [K_s(x)]\{\Delta d(x)\} + \{r_s(x)\}$$

  - Derivation of Nodal Element Forces vs. Nodal Displacements in an incremental form:

$$\{\Delta q_b\}_i = [F_{el}]_i \{\Delta Q_b\}_i + \{q_{b,res}\}_i$$

  - Nodal Forces vs. Section Forces: $\{D(x)\} = [b(x)]\{Q_b\}$
  - Section deformations vs. Nodal Deformations:

$$\{q_b\} = \int_0^L [b(x)]^T \{d(x)\} dx$$

ANALYTICAL STUDY

- Theory
  - Element was considered in a Co-rotational Framework to account for large displacements
  - Element state determination
    - Compute element nodal forces for prescribed nodal displacement (at each N-R iteration for the structure)
    - For given vector $\{q_b\}$ find $\{d(x)\}$ that satisfies compatibility
    - This is equal to "*Seek the solution of an a additional system of nonlinear algebraic equations*"

ANALYTICAL STUDY

- Theory
  - Element state determination

Require Equilibrium of Structure

Structure – 1st Newton-Raphson (index *i*)

$$\begin{cases} \vec{H}_s(\vec{u}_s(t)) = \vec{P}_{ext}(t) - \vec{F}_{int}(\vec{u}_s(t)) - [C]\dot{\vec{u}}_s(t) - [M]\ddot{\vec{u}}_s(t) \\ \vec{u}_{s,i+1} = \vec{u}_{s,i} + \left[K_s|_{\vec{u}_s=\vec{u}_{s,i}}\right]^{-1} \vec{H}_s(\vec{u}_{s,i}(t)) \end{cases}$$

Require Displacement Compatibility at Element Level

Element

- Find element displacements at LCS: $\vec{u}_{s,i+1} \rightarrow \vec{u}_{el}$
- Remove rigid body modes: $\vec{u}_{el} \rightarrow \vec{q}$

2nd Newton-Raphson (index *j*)

$$\begin{cases} \vec{H}_{el}(\vec{Q}) = \vec{q}(t) - \vec{q}_{int}(\vec{Q}) \\ \vec{Q}_{j+1} = \vec{Q}_j + \left[K_{el}|_{\vec{Q}=\vec{Q}_j}\right] \vec{H}_{el}(\vec{Q}_j) \end{cases}$$

Require Section Equilibrium

Section (at $x=x_n$)

- Compute section forces: $\vec{Q}_{j+1} \rightarrow \vec{D}(x_n)$

3rd Newton-Raphson (index *k*)

$$\begin{cases} \vec{H}_{sec}(\vec{d}(x_n)) = \vec{D}(x_n) - \vec{D}_{int}(\vec{d}(x_n)) \\ \vec{d}_{k+1} = \vec{d}_k + \left[K_{sec}|_{\vec{d}=\vec{d}_k}\right]^{-1} \vec{H}_{sec}(\vec{d}_k) \end{cases}$$

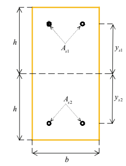
- Compute internal element displacements:

$$\{q_{int,j+1}\} = \int_0^L [b(x)]^T \{d_{k+1}\} dx$$

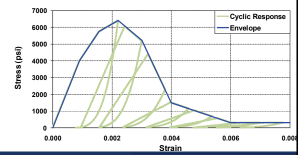
ANALYTICAL STUDY

- Application: Precast Concrete Rocking Column
  - Member geometric properties

Column Length	L (in)	120.0
Half of Section Height	h (in)	5.0
Section Width	b (in)	5.0
Area of Steel Reinforcement	$A_{s1}=A_{s2}$ (in^2)	0.5 (ρ=0.02)
Position of Steel Rebars	$y_{s1}=y_{s2}$ (in)	4 (0.8h)

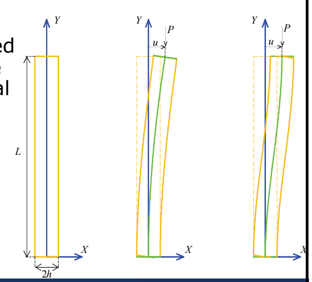


  - Material properties
    - Steel: Linear elastic with E=29000 ksi
    - Concrete:
      - Cyclic behavior according to Otter and Naaman (1989)
      - Skeleton Curve from FIP-CEB (1990) and f_{cm}=6400 psi



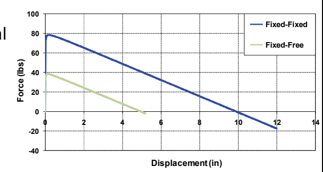
ANALYTICAL STUDY

- Application: Precast Concrete Rocking Column
  - Boundary conditions:
    - Fixed-Fixed → Gap opening at both ends
    - Fixed-Free → Gap opening at the lower end
  - Gap opening is captured by assuming no tensile strength of the material at the end sections
  - Quasi-Static loading:
    - Step 1: Compressive load P = 1000 Lbs
    - Step 2: Lateral displacement u

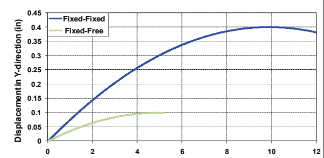


ANALYTICAL STUDY

- Application: Precast Concrete Rocking Column
  - Results:
    - Lateral force vs. lateral displacement Curves

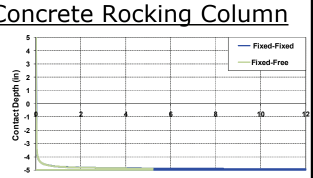


    - Orbit of the top end



ANALYTICAL STUDY

- Application: Precast Concrete Rocking Column
  - Results:
    - Contact depth at bottom end vs. lateral displacement



  - Brief Conclusions
    - The proposed model appears to provide reasonable estimations of the response of a rocking segment
    - The contact depth appears to become minimum (depending on the material strength) at very small lateral displacements

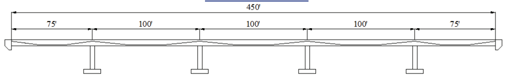
EXPERIMENTAL STUDY

- Accelerated Bridge Construction (ABC)
  - Reduces on-site construction time
  - Minimizes traffic impacts and community disruption
  - Improves work zone safety
  - Decreases environmental disruption
  - Increases product quality
  - Reduces life-cycle costs
- Prototype Bridge
  - Single-cell box girder bridge consisting of 5 spans
  - Each span is post-tensioned with a harped shape tendon
  - The piers are square hollow sections of ~30 feet height
  - The 'Span-by-Span' construction method is assumed

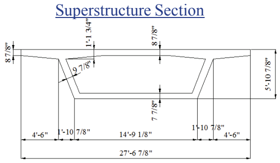
EXPERIMENTAL STUDY

- Prototype Bridge
  - Due to geometric symmetry <u>only one span</u> will be considered for the test specimen and a cantilever on each side of the supports

<u>Front Elevation</u>



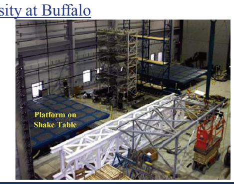
<u>Superstructure Section</u>



EXPERIMENTAL STUDY

- Test Specimen
  - Single-span bridge model with both of its supports overhanging at equal lengths
  - Tests are to be conducted in the adjacent re-locatable shake tables of the SEESL at UB
  - Prototype bridge was scaled down to abide by the shake table requirements and lab space limitations
  - Scaling factor: **2.5** (for length)

<u>SEESL at University at Buffalo</u>



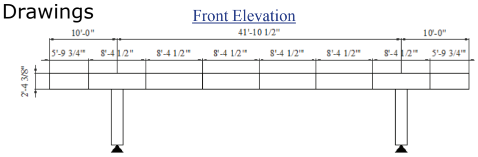
EXPERIMENTAL STUDY

- Test Specimen
  - Superstructure and substructure are divided into segments in order to
    - Comply with the ABC techniques
    - Examine the segment-to-segment joint behavior under seismic loading
  - The superstructure is divided into 8 segments
  - The substructure into 5 segments
  - The segments are post-tensioned together using unbonded tendons
  - The test specimen is designed using:
    - <u>*AASHTO LRFD Bridge Design Specifications (2007),*</u> and
    - <u>*Precast / Prestressed Concrete Bridge Design Manual (2003)*</u>

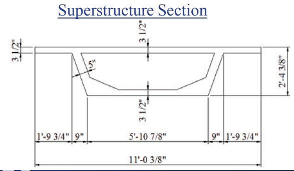
EXPERIMENTAL STUDY

- Test Specimen
  - Drawings

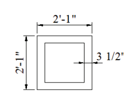
<u>Front Elevation</u>



<u>Superstructure Section</u>

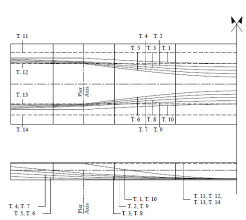


<u>Pier Section</u>

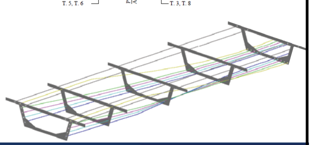


EXPERIMENTAL STUDY

- Test Specimen
  - Post-tensioning system: Monostrand of 0.5"
    - Tendons in plan view



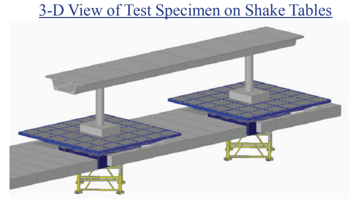
- Tendons in 3-D view



EXPERIMENTAL STUDY

- Test Specimen
  - Superstructure and pier segments will be manufactured at a local precast plant and then delivered to the Laboratory.
  - Assembly and post-tensioning of the superstructure segments on the Laboratory Floor
  - Assembly and post-tensioning of the piers and cap beams on the Shake Tables

<u>3-D View of Test Specimen on Shake Tables</u>



Thank you

Questions?

University at Buffalo The State University of New York

Research and Application of Precast Segmental Concrete Bridge Columns in Taiwan

Kuo-Chun Chang

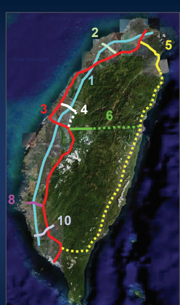
Department of Civil Engineering, National Taiwan University
National Center for Research on Earthquake Engineering, Taiwan

International Specialty Workshop on Seismic Connection Details for Segmental Bridge Construction
Seattle, Washington, July 22 – 24, 2009

Presentation outline

- Research:
  - 1999-2000: NCKU (Y.-L. Mo)
    - Cast-in-place (CIP) hinge region
  - 1999~2003: NCREE and TANEEB
    - Fully-precast tall columns
  - 2005~present: NCREE/TANEEB/CECI and MCEER
    - Self-centering
    - Seismic isolation
    - High performance rebar (presented by Ou)
- Application
  - National Freeway Taichung Metro-area No. 4 (CECI)
    - CIP hinge region
    - Seismic isolation
  - Neihu MRT: precast cap beams and girders

Decrease environmental impact



Major highway systems
— Completed
····· Under planning or construction

Conventional CIP construction

Decrease Traffic disruption

Neihu MRT, Taipei City
- Precast cap beams: 8.5 m~9 m (width), 222 pieces
- Precast U girders: 9 m~25 m (span), 546 pieces
- Precast girder segments: 2.5 m (length), 407 pieces

Precast U girders (2007)
(Erection at night)

Precast segmental girders (2007)

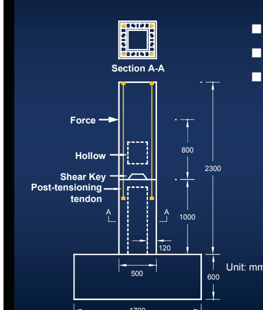
Precast cap beam at Neihu MRT

Assembling
(welded connection)

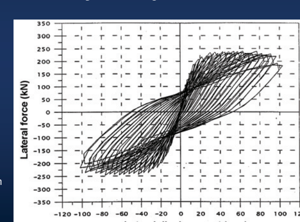
Verification testing at NCREE (2006)
- 200 million Dead(150tf)+Live(60tf) fatigue test: no damage at welds
- D+4L vertical load test: no damage

Y.-L. Mo (1999-2000)

- CIP hinge region.
- Post-tensioning
- Shear key at the joint

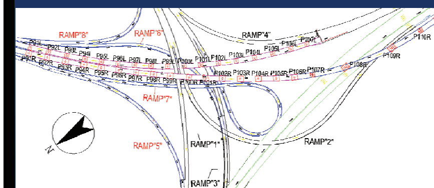


Column design



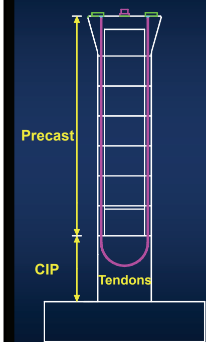
Test results

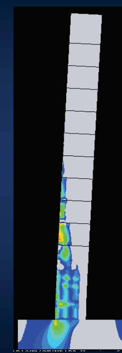
Taichung Metro-area No. 4 - Application of precast segmental columns -



Precast segmental columns:
- P91R～P106R
  Tall columns: CIP hinge region
- P91L～P107L
  Short columns: Lead-rubber seismic isolation

Taichung Metro-area No. 4 tall columns - CIP hinge region -

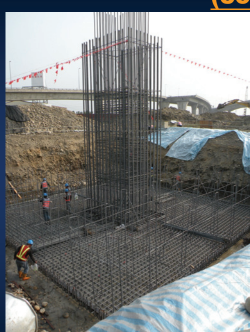




FEM Analysis results (failure at CIP region)

16 columns
- CIP hinge region, post-tensioning, shear keys at joints
- Column height: 17m~31m
- CIP region height: 7m~9m
- Column segment: 2.3m (height), 116 pieces
- Cap: 3m (height)

Taichung Metro-area No. 4 tall columns (cont'd)

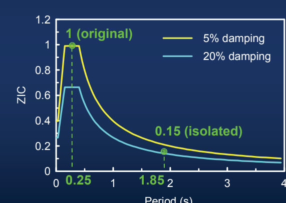


Foundation and CIP region

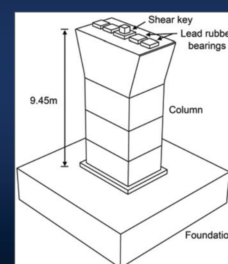
Top CIP region: tendon ducts (Photo taken on 7/16/2009)

Taichung Metro-area No. 4 short columns - Lead rubber seismic isolation -

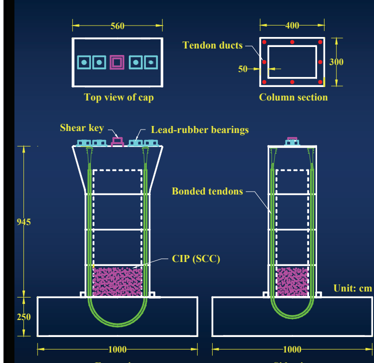
Performance objective: no damage of the column after a design earthquake: no tensile stress at the extreme fiber at the base of the column



ZIC spectra for one of the continuous units



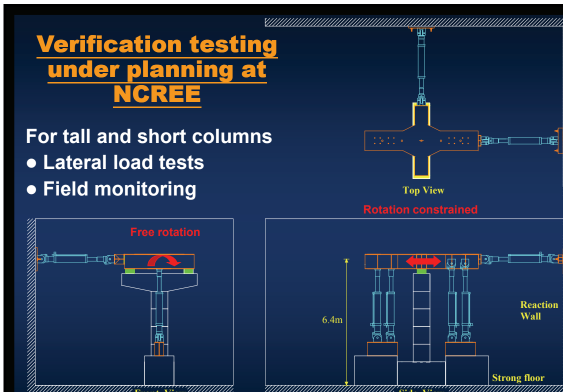
Column design



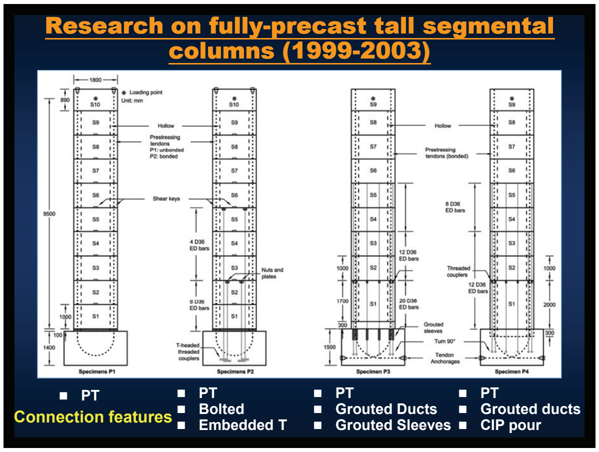
17 columns
- LRB, post-tensioning, shear keys at the joints
- Column height: 8m~14m
- Column segment: 1m~2.3m (height), 62 pieces
- Cap: 3m (height)

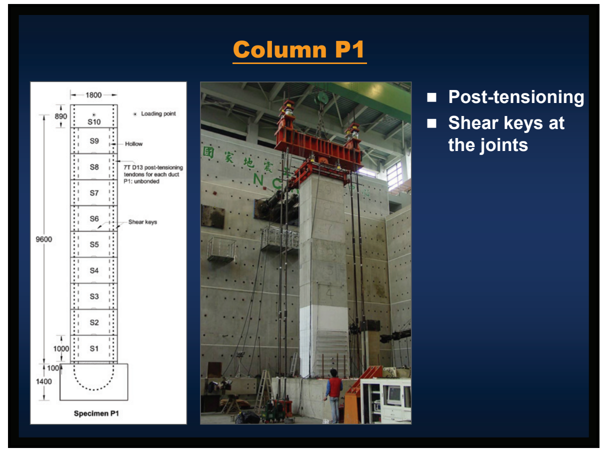
Verification testing under planning at NCREE

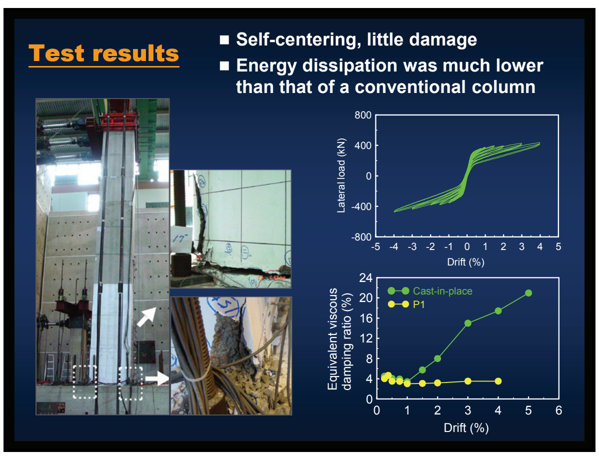
For tall and short columns
- Lateral load tests
- Field monitoring

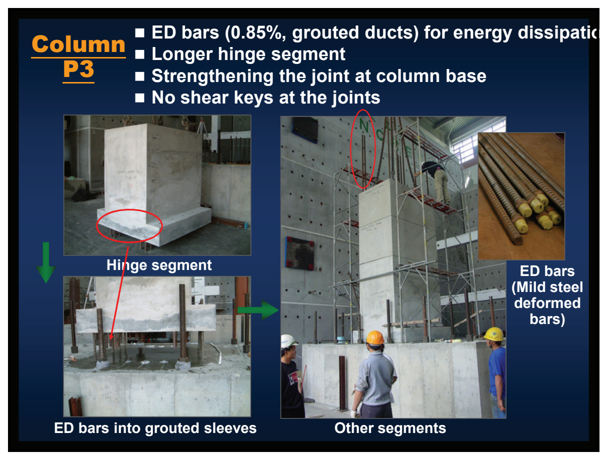


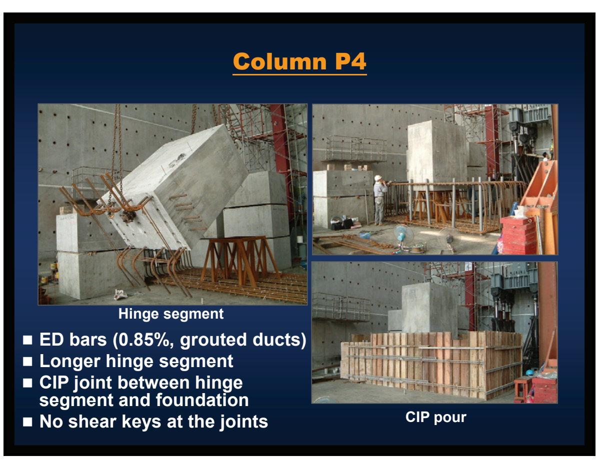
Lateral load testing for segmental columns with LRB

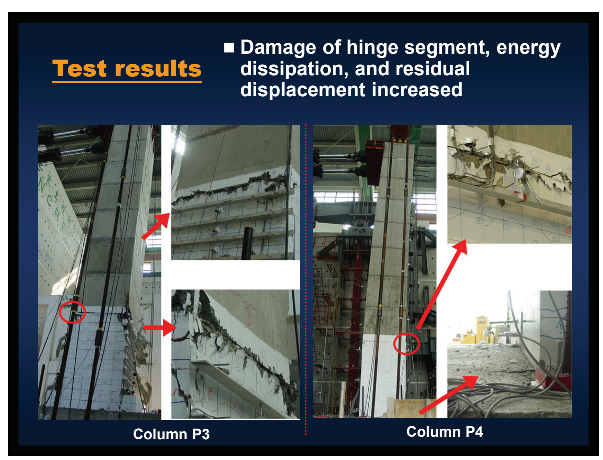

Research on fully-precast tall segmental columns (1999-2003)
Specimens P1
Specimens P2
Specimen P3
Specimen P4
Connection features
PT
PT
Bolted
Embedded T
PT
Grouted Ducts
Grouted Sleeves
PT
Grouted ducts
CIP pour


Column P1
Specimen P1
Post-tensioning
Shear keys at the joints

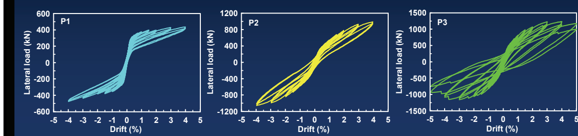

Test results
Self-centering, little damage
Energy dissipation was much lower than that of a conventional column
Lateral load (kN)
Drift (%)
Equivalent viscous damping ratio (%)
Cast-in-place
P1
Drift (%)


Column P3
ED bars (0.85%, grouted ducts) for energy dissipatic
Longer hinge segment
Strengthening the joint at column base
No shear keys at the joints
Hinge segment
ED bars into grouted sleeves
Other segments
ED bars (Mild steel deformed bars)

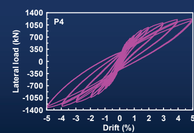

Column P4
Hinge segment
ED bars (0.85%, grouted ducts)
Longer hinge segment
CIP joint between hinge segment and foundation
No shear keys at the joints
CIP pour


Test results
Damage of hinge segment, energy dissipation, and residual displacement increased
Column P3
Column P4

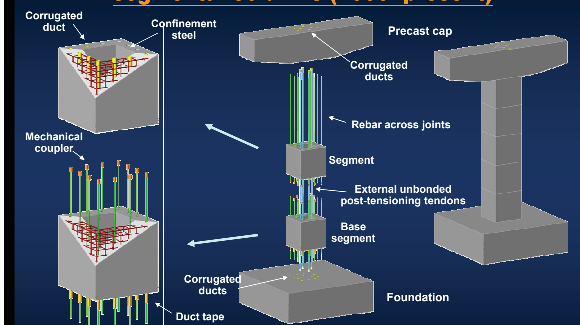
Hysteretic behavior



Column	P1	P2	P3	P4
Max ζeq	4%	6%	13%	11%
Strength (kN)	436	988	1232	1224
Max residual (%)	<0.1	0.3	1.8	1.2



Research on self-centering segmental columns (2005~present)



- Fully-precast column, unbonded post-tensioning
- ED bars (grouted ducts, partially unbonded), control λ_{ED} ratio
- No shear keys at the joints

Large-scale column testing

- Major design parameters

Specimen	ED bars (%Ag)	Axial force (fc'Ag)	λ_{ED} (%)	Lau (mm)
C0C, C0P	0	0.17	0	N.A.
C5C, C5P	0.5	0.17	28	400
C8C, C8P	1	0.17	50	100
C5C-1	0.5	0.12	35	400

λ_{ED} : ED bar contribution to column strength

$$\lambda_{ED} = \frac{V_{exp} - V_{exp0}}{V_{exp}} \qquad V_{exp} : \text{Lateral strength of column} \qquad V_{exp0} : \text{Lateral strength of column without ED bars}$$

Lau: Additional unbonded length

Elements of a column

Cap beam

Foundation

Base segment

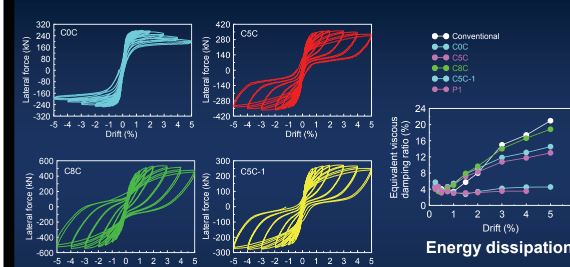
Segment S2

Segments S3,S4

Assembling process

1st segment
↓
2nd segment
↓
3rd segment
↓
4th segment
↓
Cap beam
↓
Post-tensioning
↓
Grouting

Hysteretic behavior



Energy dissipation

- Ductile behavior for all the columns (with or without ED bars)
- ED bars + unbonded length => significant increase in energy dissipation.
- λ_{ED} < 35%, residual drift <1%

Conclusions

- Current application of precast segmental columns in Taiwan use the concept where limit state of the column is "not" controlled by nonlinear behavior of precast joints (CIP hinge region and seismic isolation).
- Experimental results have shown that precast segmental columns allowing nonlinear behavior of precast joints (with or without ED bars) can be designed for ductile and self-centering behavior under earthquake loading. Research is underway to develop simplified design methods for engineers to use this type of columns.

Accelerating Bridge Construction in in Regions of High Seismicity

John Stanton
Marc Eberhard

University of Washington

International Specialty Workshop on Seismic Connection Details for Segmental Bridge Construction.
22 July 2009, Seattle, Washington

Acknowledgments

- FHWA
- WSDOT
- PEER / State of California
- TransNOW

Precast Concrete Connections using Bars Grouted into Sleeves.

"Many Ducts" Connection

Emulates typical c.i.p. connection.

Tight tolerances.

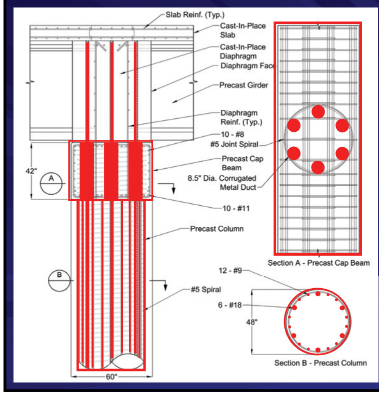
"Large-Bar" Connection

- Concept
  - Larger bars (e.g., #18)
  - Fewer bars (e.g., 6-8)
  - Much larger ducts (e.g., 8-in. dia.)
- Constructability
  - → More generous tolerances
  - → Easier fabrication
  - → Faster alignment

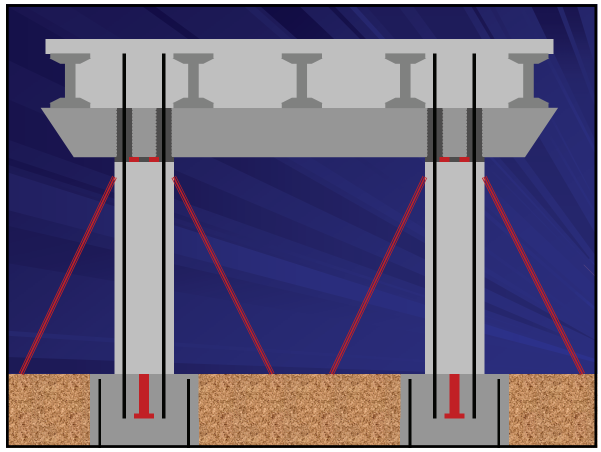
"Large-Bar" Connection

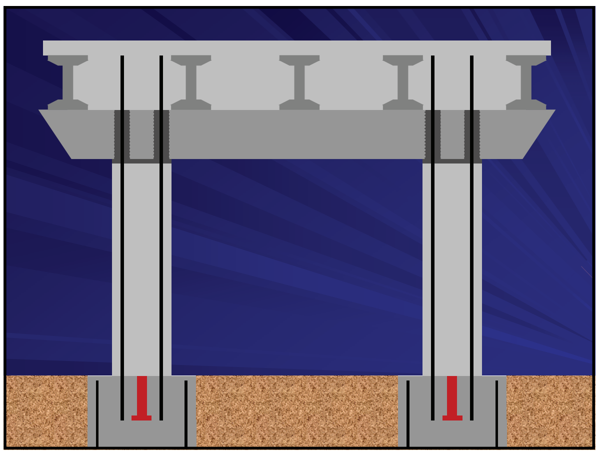
- Suitable for beam-column connection.
- Can be used with single-piece or segmental columns.
- Column configuration depends on circumstances:
  - Column weight (crane size).
  - Column height (stability during erection).
  - etc.

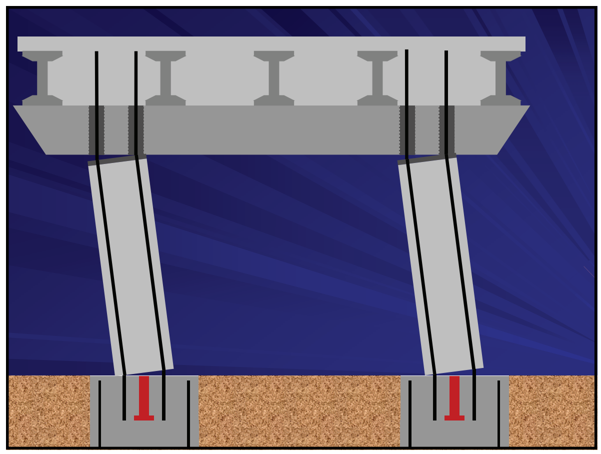
Large-Bar Connection

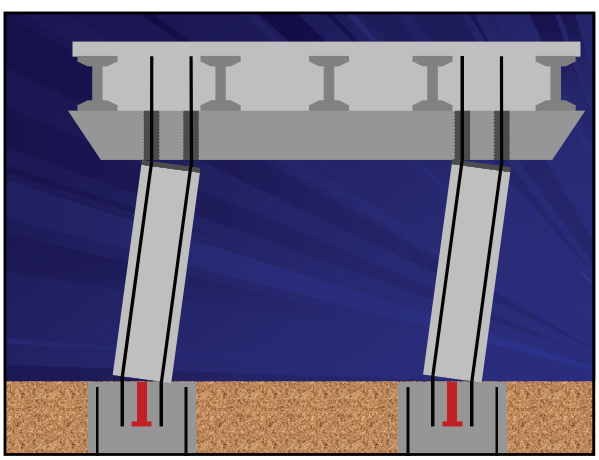


- 4ft Diameter Column
- 5ft x 3.5ft Cap Beam
- 6 # 18 rebar
- 8.5” Corrugated Metal Ducts
- 12 # 9 rebar
- High Strength Grout
- Debond Intentionally?

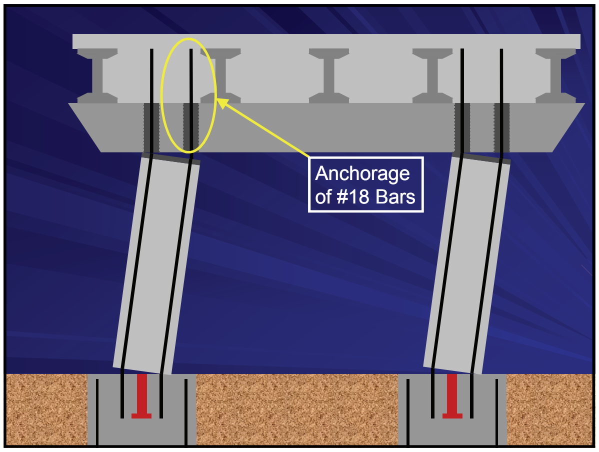








Seismic Performance

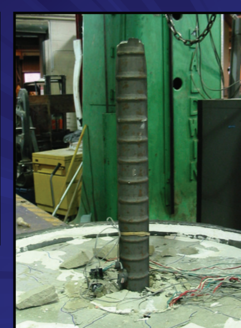


Full-Scale Anchorage Tests

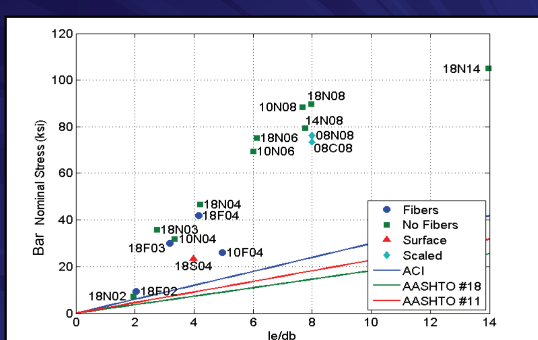
Anchorage Test Results.

- #8, #10, #14, #18 bars.
- Behavior determined by l_e/d_b.
- Low L_e/d_b: bond failure.
- High L_e/d_b: bar yield and fracture.

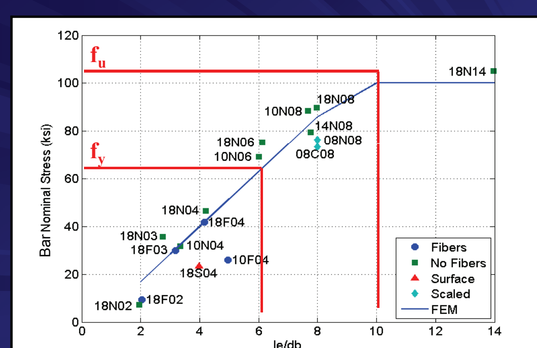
Full-Scale Anchorage Tests



Full-Scale Anchorage Tests

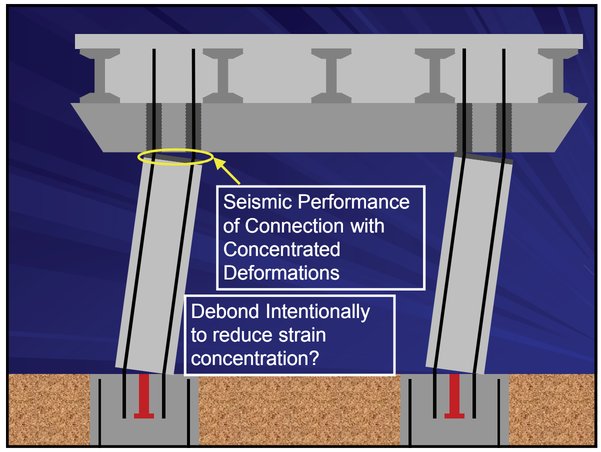


Full-Scale Anchorage Tests

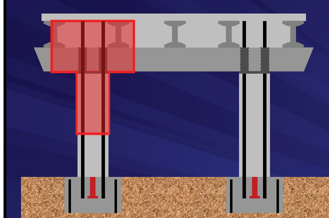


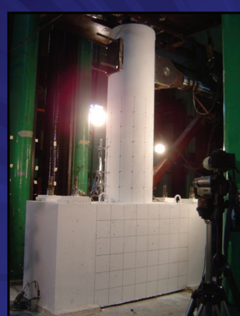
Anchorage Test Results.

- For f_y, need $L_e/d_b \geq 6$
- Bond failure at the bar surface: “confined bond failure”.
- Bond stress = $0.25f_s/(L_e/d_b)$ = 2500 psi = $27\sqrt{f'_g}$ = $0.31f'_g$.
- Consistent with previous research on smaller bars. (e.g. Raynor, Moustaafa).



Seismic Performance



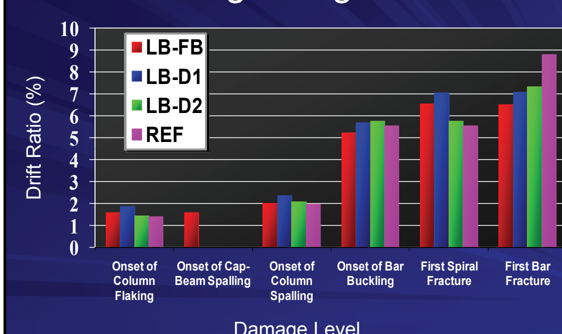


- Lab tests at 42% scale.

Test Matrix				
	Longitudinal Reinforcement	Reinforcement Ratio	Grouted Ducts	Debonding ?
REF.	16 - #5	1.58%	No	None
LB-FB	6 - #8	1.51%	Yes	None
LB-D1	6 - #8	1.51%	Yes	Method 1
LB-D2	6 - #8	1.51%	Yes	Method 2

Equivalent Moment vs. Drift

Damage Progression



Failure Mechanisms

LB-FB: Bar buckling and spiral fracture at 6.5% drift

Implementation

Highways for Life Program: Team Membership.

- FHWA
- BERGER-ABAM
- University of Washington
- WSDOT
- Tri-State Construction
- Concrete Technology Corporation

Highways for Life Program: Tasks.

- Develop suitable connections (Column to cap-beam and footing)
- Lab tests for seismic performance.
- Build the bridge, monitor constructability:
  - Fabricate columns.
  - Erect bents (note skew).
- Develop specification language.
- Prepare design examples.

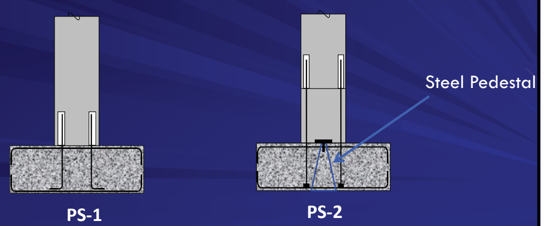
The Bridge
(SR12 Over I-5)

Connections to be used

- Top: 8#18 in 48” square column.
- Bot: Still under development. Watch this space!
- Possible footing connections:

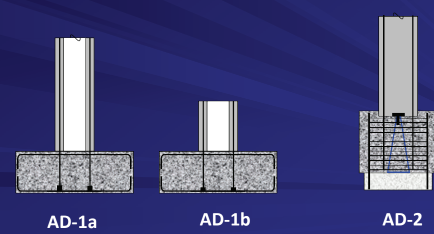
Project-Specific Tests

- **PS-1**: PC column grouted over bars in CIP spread footing
- **PS-2:** Spread footing cast around bars projecting from segmental column.



Increased Versatility

- AD-1a &1b: Hollow Columns
- AD-2: Connection to Drilled Shaft



Conclusions

- Large-Bar precast systems can be constructed rapidly.
- Many possible variants for footing connection.
- Seismic performance similar to c.i.p.

INTERNATIONAL SPECIALTY WORKSHOP ON SEISMIC CONNECTION DETAILS FOR SEGMENTAL BRIDGE CONSTRUCTION

Seismic Connection of Precast concrete Bridges in Washington State

Jugesh Kapur
State Bridge and Structures Engineer

Bijan Khaleghi
State Bridge Design Engineer

Washington State Department of Transportation
Bridge and Structures Office

Washington State Department of Transportation

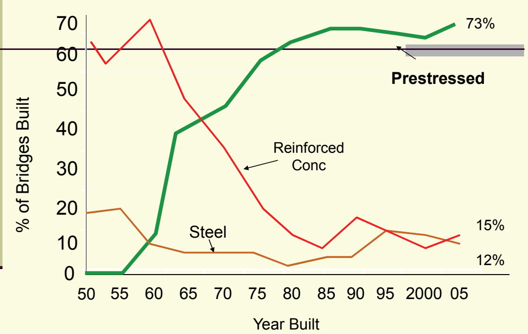
SEATTLE, WASHINGTON, JULY 22 – 24, 2009

PRESENTATION OUTLINE

- Precast Concrete Bridges
- Precast Substructure
- Seismic Connection Design/Detail
- Accelerated Bridge Construction

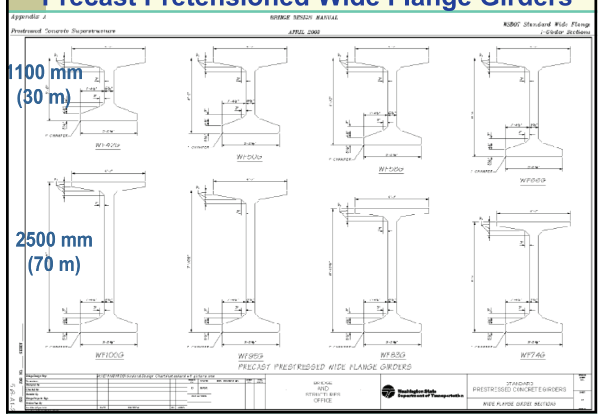
Conventional Bridges (CIP Emulative)

Ordinary Bridges



7 out of 10 WSDOT bridges built in the past 10 years are precast prestressed or post-tensioned concrete

Precast Pretensioned Wide Flange Girders



BRIDGE DESIGN SPECIFICATIONS

Effective 2008 - LRFD SEISMIC GUIDE

"Displacement-based Seismic Provisions"

Life Safety Performance Criteria During an EQ with 7% Probability of Exceedance in 75 years =1000 Years Return Period

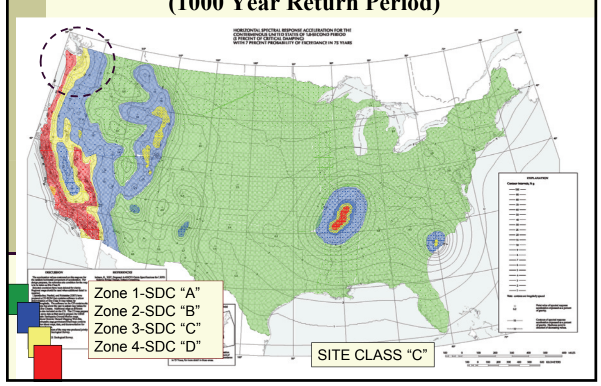
AND

WSDOT Design Policy

Limitations on: ERS, ERE, Design and Details

Seismic Design Category (zone)

(1000 Year Return Period)

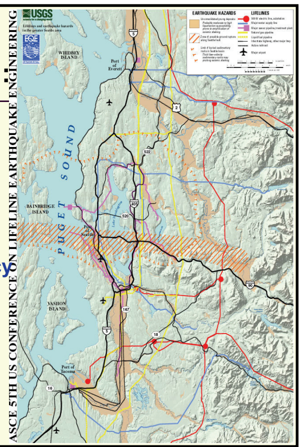


Western Washington:

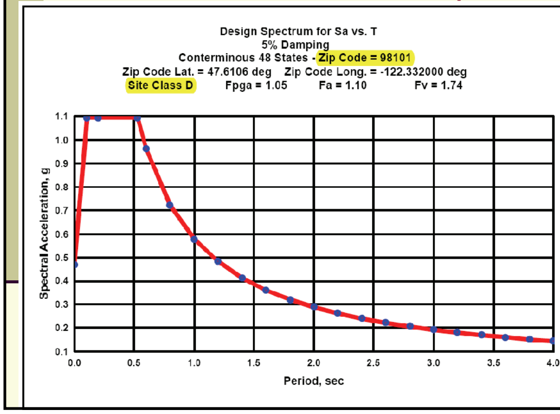
- Liquefaction

New Bridges

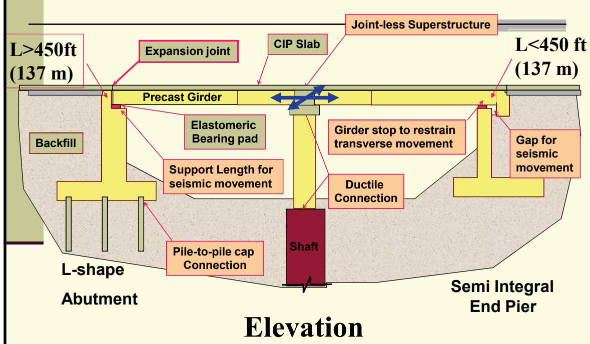
Bridge Widenings
(WSDOT Executive Policy for Bridge Widenings in Liquefiable Sites



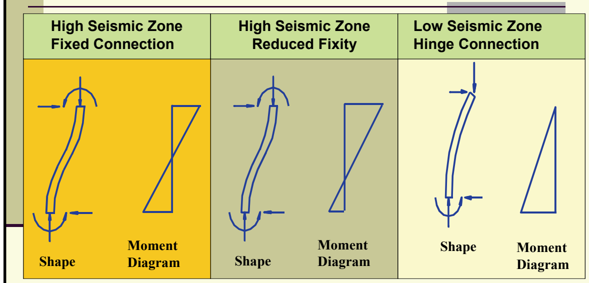
SDC – D, Site Class D – Potential Liquefaction



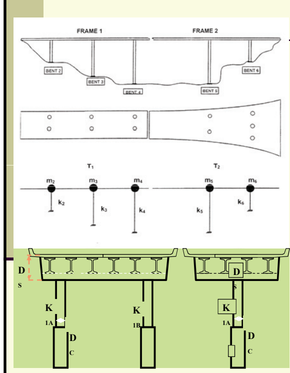
Seismic Design of Concrete Bridges



Seismic Design of Concrete Bridges



BALANCING STIFFNESS



Constant Width Frames | Variable Width Frames

$\frac{k_i^e}{k_j^e} \geq 0.5$ $\frac{k_i^e m_j}{k_j^e m_i} \geq 0.5$

Between Two Column Within A Bents

$\frac{k_i^e}{k_j^e} \geq 0.75$ $\frac{k_i^e m_j}{k_j^e m_i} \geq 0.75$

Between Adjacent Bents Within A Frame

$\frac{Ti}{Tj} \geq 0.7$

Ratio of Fundamental Period of Vibration Between Adjacent Frames

Positive Seismic Moment Capacity

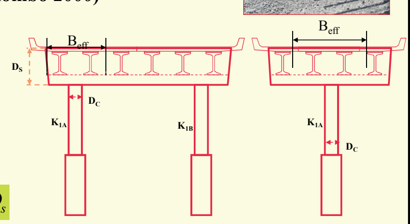
$$M_{po}^{CG} = M_{po}^{top} + \frac{(M_{po}^{top} + M_{po}^{Base})}{L_c} h$$

UC San Diego (Holombo 2000)

$$M_{sei}^{Int} = \frac{2M_{po}^{CG}}{3N_g^{int}}$$

$$M_{sei}^{Ext} = \frac{M_{po}^{CG}}{3N_g^{ext}}$$

$$B_{eff} = D_c + D_s$$

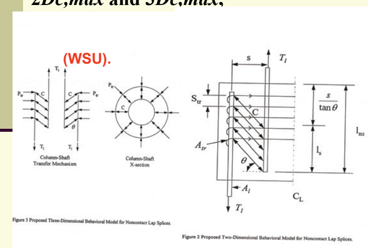


Extended Strands: $N_{ps} = 12\left[M_{sei} \bullet K - M_{SIDL}\right] \bullet \frac{1}{0.9\phi A_{ps} f_{py} d}$

Extended Strands for Positive Seismic Moment Capacity At Intermediate Piers Of Prestressed Girder Bridges

Column – Shaft Connection

- Connection Design of Column-Oversized Shafts Based On Expected Nominal Flexural Capacity at 1.00 Times The Overstrength Moment, Mpo of Column.
- Column longitudinal reinforcement into enlarged shafts in a staggered manner with the minimum embedment lengths of 2*Dc,max* and 3*Dc,max*,



Column Dia	CIP Joint
4 ft	12 ft
5 ft	15 ft
6 ft	18 ft

$$S_{tr} = \frac{2\pi A_{sp} f_{yr} l_s}{k A_l f_{yt}}$$

Precast Substructure:
Precast Column and Wall on spread Footing

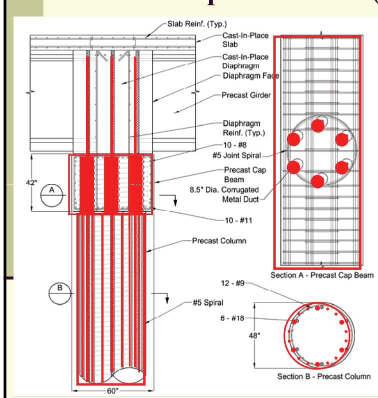
Monolithic Connection

PRECAST COLUMN ON SPREAD FOOTING

Monolithic Connection

Bellevue Direct Access Project

Precast Bent Cap Connection (Emulative Design)

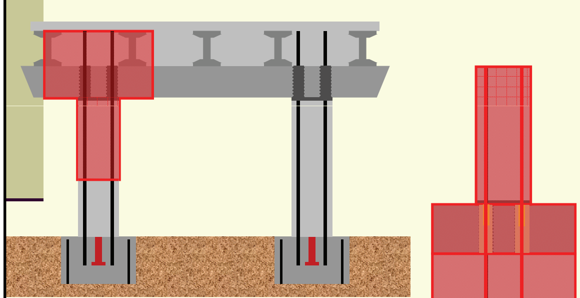


Large Bar Connection

- 4ft Diameter Column
- 5ft x 3.5ft Cap Beam
- 6 # 18 rebar
- 8.5" Corrugated Metal Ducts
- 12 # 9 rebar
- High Strength Grout
- Debond Intentionally?

Seismic Performance – UW Test

WARD 648.2 Rapidly Constructible Large-Bar Precast Bridge-Bent Seismic Connection



Seismic Performance – UW Test

WARD 648.2 Rapidly Constructible Large-Bar Precast Bridge-Bent Seismic Connection

- 20" Diameter Column
- 18" Cap-Beam
- 18" Diaphragm
- 6 # 8 Bars
- 4" Grouted Ducts
- 8 d_b Debonding

Precast Bent cap

SR 202 / SR 520

Duct Template

Tolerances - Tack Weld

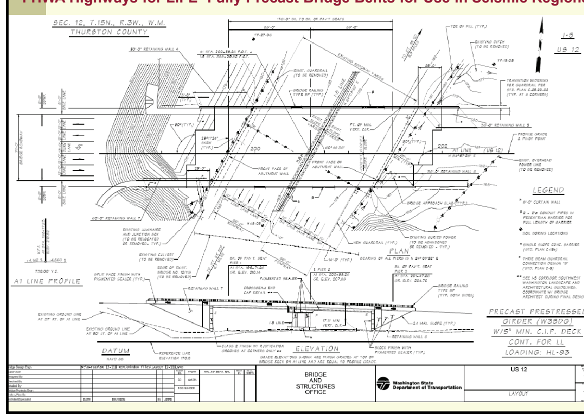
Spirals

Precast Bent cap

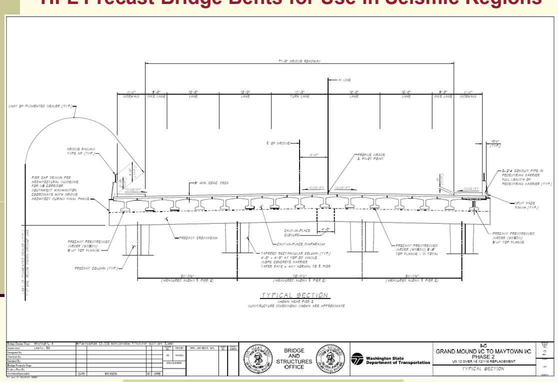
SR 202 / SR 520

1$^{1/2}$ Hours +/- Bent Cap Erection

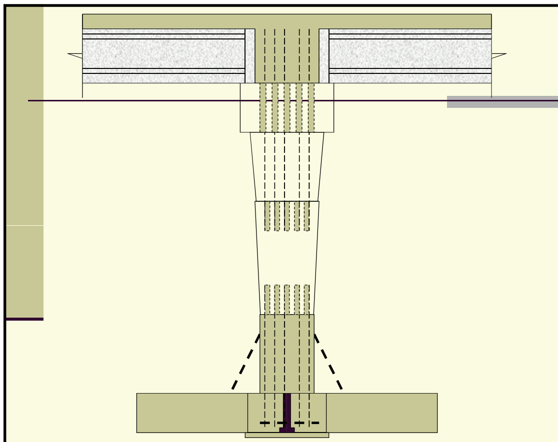
FHWA Highways for LIFE- Fully Precast Bridge Bents for Use in Seismic Regions



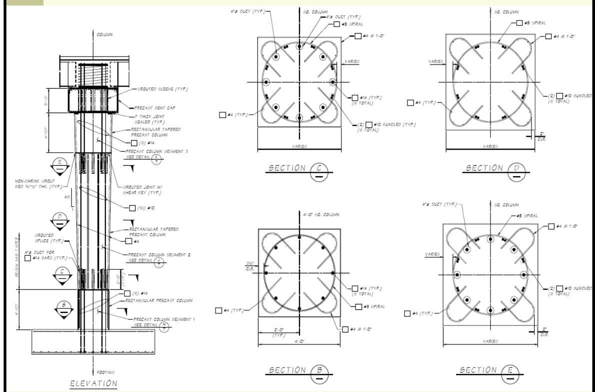
HFL Precast Bridge Bents for Use in Seismic Regions



Typical Section – All Precast

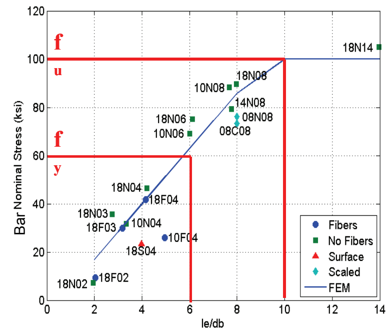


HFL Fully Precast Bridge Bents for Use in Seismic Regions



Seismic Performance – UW Test

WARD 648.2 Rapidly Constructible Large-Bar Precast Bridge-Bent Seismic Connection



Full-Scale Anchorage Tests

- Yield in 6 db
- Fracture in 14 db
- Enough length for debonding and anchorage

Recommended Duct size and embedment length for Grouted Sleeves

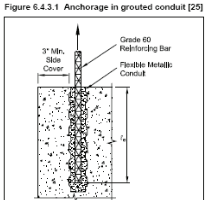
Precast Substructure Elements

UDOT - Specification 03131S

Bar Size	Outside Diameter (inches)	Length of Sleeve (inches)
4	2.625	14.125
5	3.000	14.125
6	3.000	14.125
7	3.000	18.75
8	3.500	18.75
9	3.500	18.75
10	3.500	23.5
11	4.000	23.5
14	4.000	28.375
18	4.500	39.625

PCI Figure 6.4.3.1

Figure 6.4.3.1 Anchorage in grouted conduit [25]



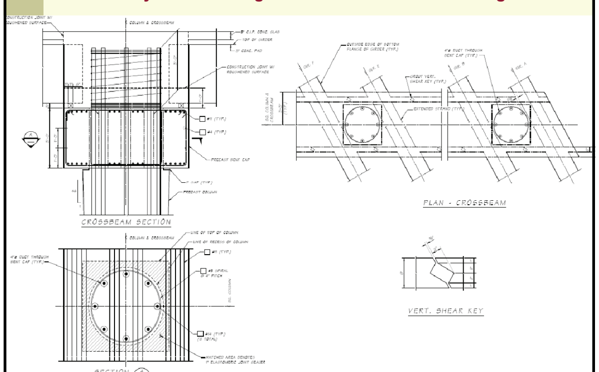
Bar Size	Bar embedment length, ℓ_e in.*
3	12
4	12
5	12
6	15
7	21
8	27

* For grout strengths higher than 5000 psi, multiply table values by $\sqrt{5000/f'_c}$.

WSDOT BDM Recommended Duct Size And Embedment Length For Grouted Sleeves

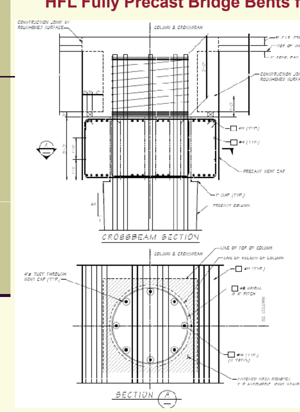
Bar Size	Nominal Duct Size, in.	Embedment Length, in.	Embedment / Bar Diameter
#3	2	12	29
#4	2	15	27
#5	3	15	21
#6	3	15	18
#7	3	20	21
#8	4	20	18
#9	4	20	16
#10	4	25	18
#11	4	25	16
#14	4	30	16
#18	5	40	16

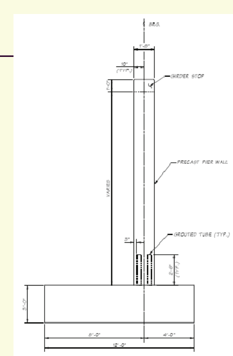
HFL Fully Precast Bridge Bents for Use in Seismic Regions

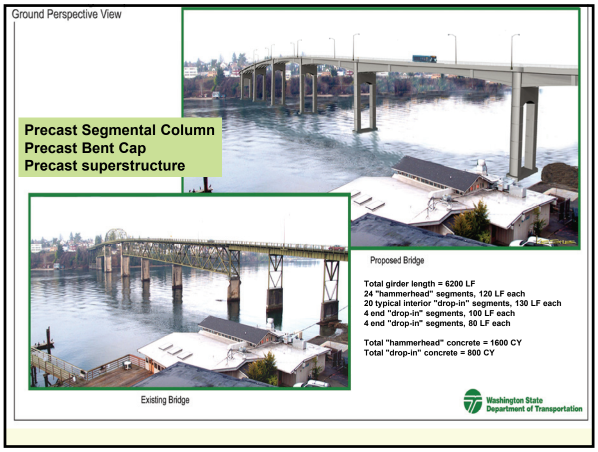


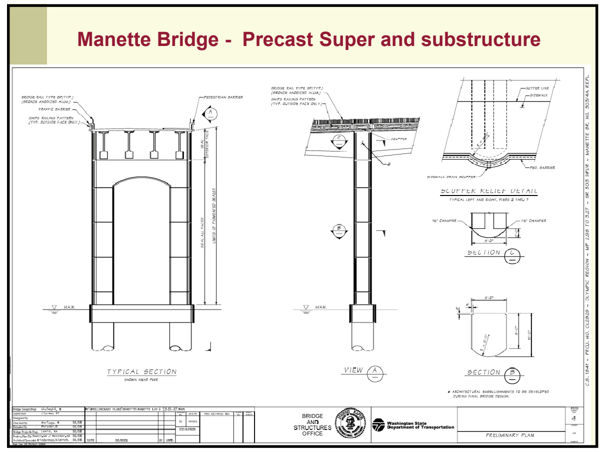
Column-to-Bent Cap Connection

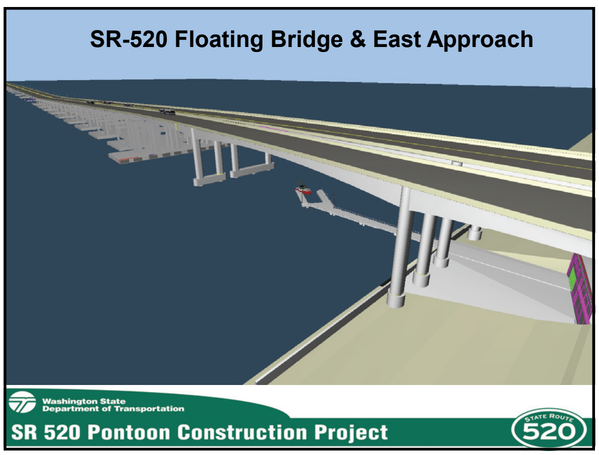
HFL Fully Precast Bridge Bents for Use in Seismic Regions

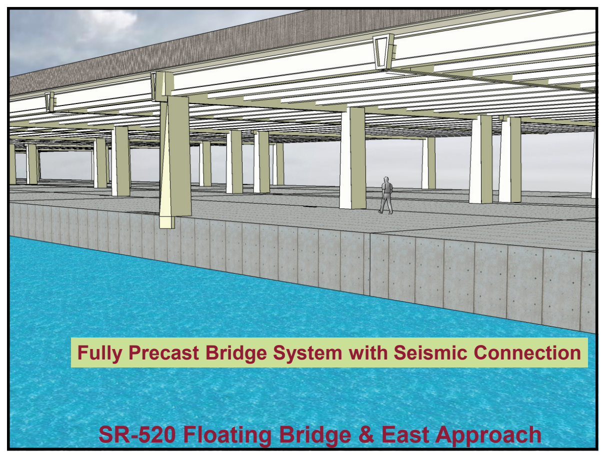




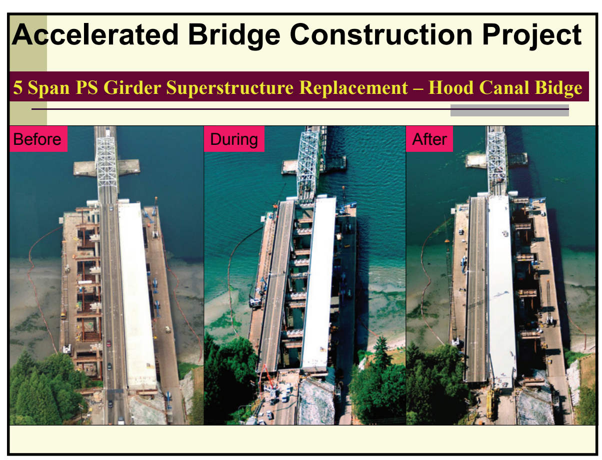

Ground Perspective View
Precast Segmental Column
Precast Bent Cap
Precast superstructure
Proposed Bridge
Total girder length = 6200 LF
24 "hammerhead" segments, 120 LF each
20 typical interior "drop-in" segments, 130 LF each
4 end "drop-in" segments, 100 LF each
4 end "drop-in" segments, 80 LF each
Total "hammerhead" concrete = 1600 CY
Total "drop-in" concrete = 800 CY
Existing Bridge
Washington State Department of Transportation


Manette Bridge - Precast Super and substructure
TYPICAL SECTION
VIEW
SECTION
SCUPPER RELIEF DETAIL
BRIDGE AND STRUCTURES OFFICE
Washington State Department of Transportation
PRELIMINARY PLAN


SR-520 Floating Bridge & East Approach
Washington State Department of Transportation
SR 520 Pontoon Construction Project
STATE ROUTE 520


Fully Precast Bridge System with Seismic Connection
SR-520 Floating Bridge & East Approach

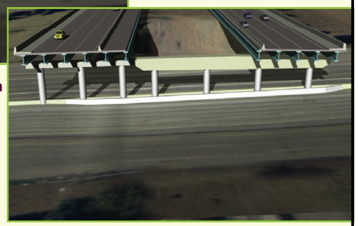

AWV – Bored Tunnel - Interior Roadway Structure
16.5'
16.5'
12'
12'
4'
8'
12'
12'
8'
4'
54' Diameter
Typical Section
Over 7000 Precast Members for Roadway Structure


Accelerated Bridge Construction Project
5 Span PS Girder Superstructure Replacement – Hood Canal Bidge
Before
During
After

Innovative Bridge Construction - WA

SR 405 - Bellevue Access NE 8th Street

Stage -2 Construction

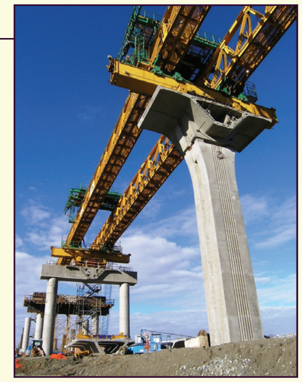


Stage -3 Construction

Sound Transit Light Rail Precast Segmental

- Span-by-Span

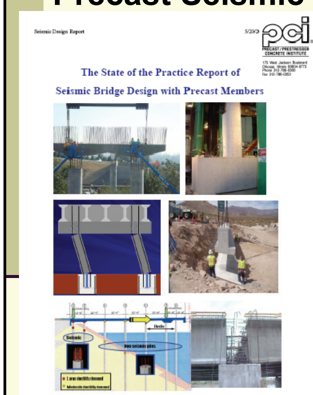
Erection Gantry



ST Light Rail - Precast Segmental

- Balanced Cantilever

Precast Seismic Design Reports



Chapters:

1. Introduction
2. Considerations when Selecting the Structural System
3. Seismic Design Criteria
4. Seismic Analysis
5. Connection Details
6. Design Examples
7. Applicable Research

WSDOT Strategic Plan for Accelerated Bridge Construction (ABC)

WSDOT ABC Website

Next Step

- ABC Chapter for WSDOT BDM
  - Design Criteria
  - Design Examples
  - STD Details - Connections
- STD Specifications for ABC
- ***Design for ABC !***

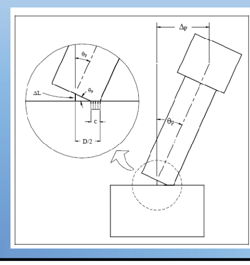
Segmental Bridge Columns w/ Damage-Free Plastic Hinges

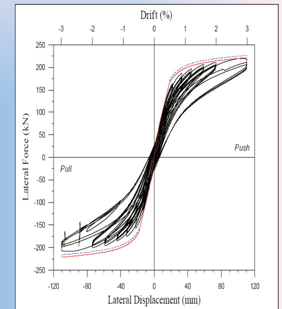
S. Motaref, PhD student
M. Saiidi, Professor
D. Sanders, Professor

University of Nevada, Reno

Main Shortcomings of Post-Tensioned Precast Columns under Seismic Loads

- Small energy dissipation
- Damage at end segments





Measures to Improve Seismic Response of Precast Columns

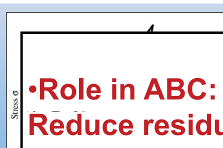
- Reduce damage by placing jackets (steel or FRP) in critical zones
- **Increase hysteretic energy dissipation by extending longitudinal reinforcement of the base segment into the footing**
- **Use advanced materials/details in plastic hinge region to dissipated energy, and minimize damage in plastic hinges**
- **Develop connections w/ dampers**

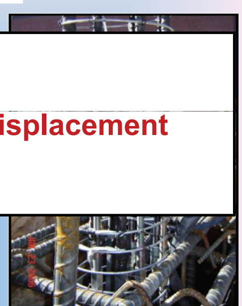
Innovative Materials in Plastic Hinges

- **SMA (Shape Memory Alloys)**
- **ECC (Engineered Cementitious Composites)**
- **FRP or steel jackets**
- **Built-in Elastomeric Pads**
- **Pipe Pin Connection Dampers**

SMA (Shape Memory Alloy)

NiTi **Superelastic shape memory alloy**

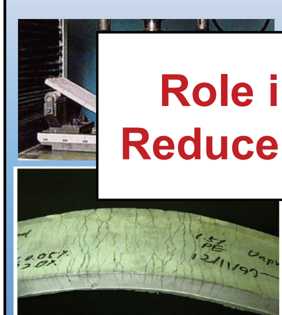


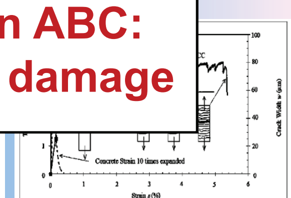


•Role in ABC:
Reduce residual displacement
Dissipate energy

ECC (Engineered Cementitious Concrete)

High tensile strain capacity

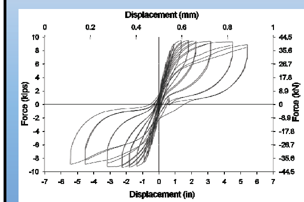




Typical ECC Tensile Stress Strain Curve

Role in ABC:
Reduce damage

Past Research: Cast-in-Place Construction
SMA/ECC



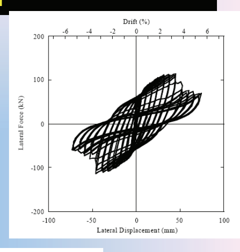
FRP or Steel Jackets

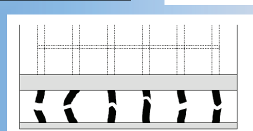
Role in ABC: Reduce damage

UNR- CIP

UCSD- Multi-Seg.; No anchored bars

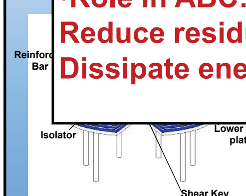
Mode of failure: Buckling and rupture of longitudinal bars in rubber

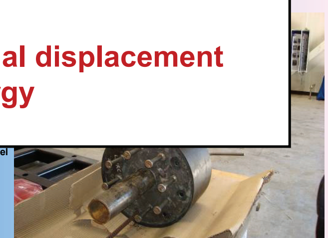




New Elastomeric Pad Developed at UNR: Shimmed pads to prevent buckling and rupture of longitudinal bars

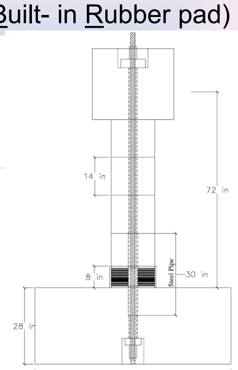
•Role in ABC:
Reduce residual displacement
Dissipate energy





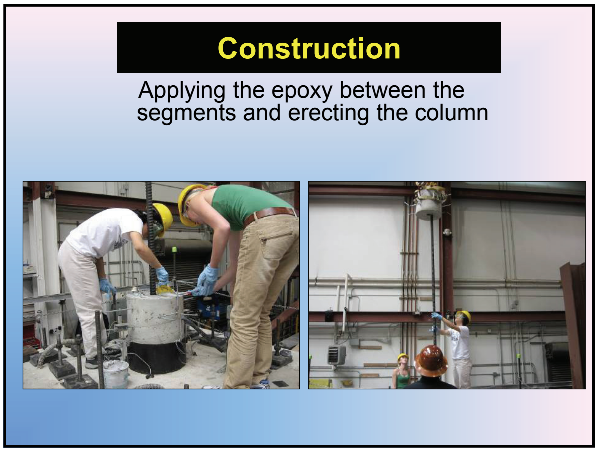
SBR-1
(Segmental Column with Built- in Rubber pad)

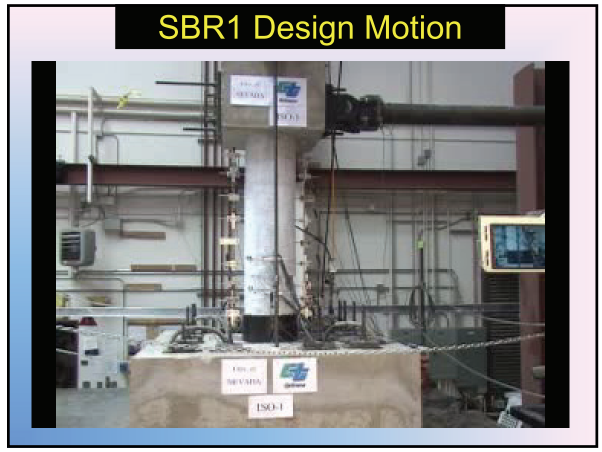
- 5 segments
- Unbonded post-tensioned tendon to connect segments
- Extended longitudinal reinforcement from first segment to the footing
- Steel pipe as a shear key at center of elastomeric bearing
- Tested on shake table under Sylmar earthquake'94

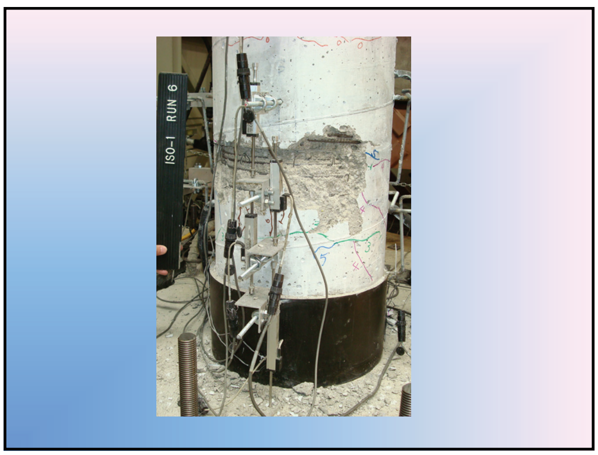


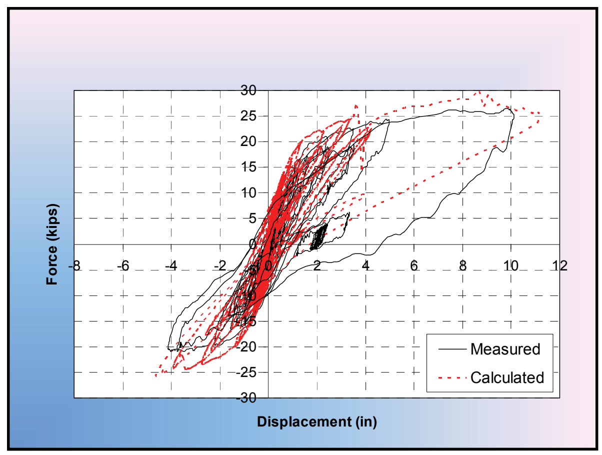
Construction

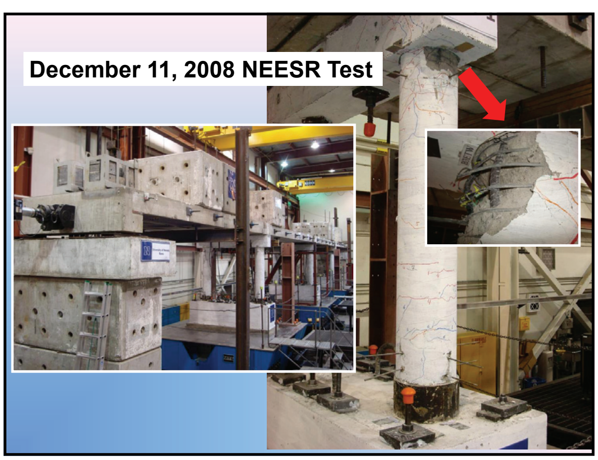
Installing the longitudinal reinforcement in elastomeric bearing

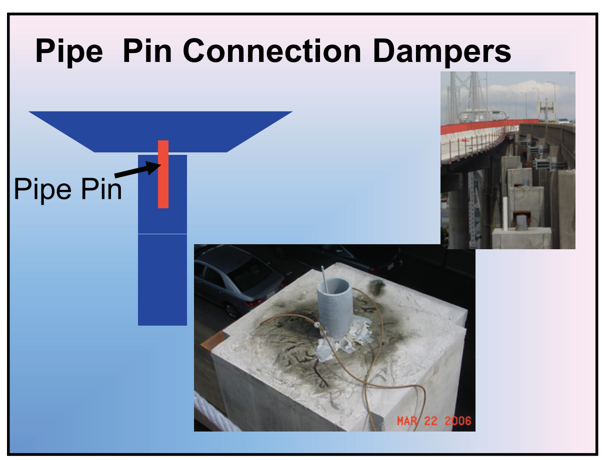

Construction
Applying the epoxy between the segments and erecting the column


SBR1 Design Motion
ISO-1

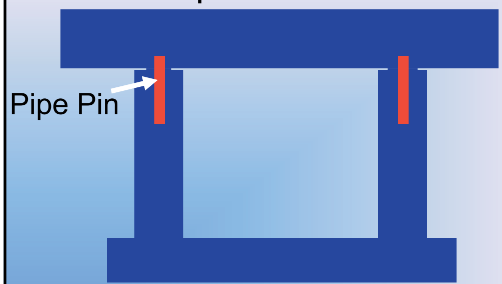

ISO-1 RUN 6


Force (kips)
Displacement (in)
30
25
20
15
10
5
-20
-25
-30
-8
-6
-4
-2
2
4
6
8
10
12
Measured
Calculated

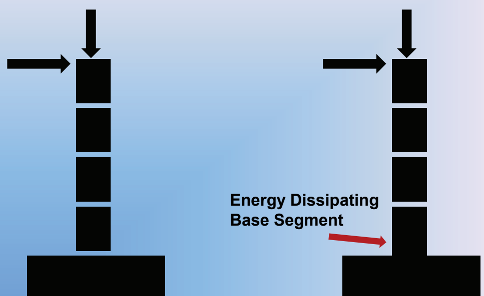

December 11, 2008 NEESR Test


Pipe Pin Connection Dampers
Pipe Pin
MAR 22 2006

Pipe Pin for ABC



Future Test Models



Alternatives at Base

- Built-in elstomer
- Conventional steel RC
- Conventional steel RC wrapped w/ FRP
- ECC
- ECC w/ SMA bars

Summary

- **The plastic hinge was free from damage**
- **The shims were effective in preventing bar buckling and achieving large drifts**
- **Residual displacement was negligible until very large motions.**
- **Energy dissipation was comparable to that of conventional concrete segmental column**
- **Pipe pin connections promising for ABC**

IT Roles in Seismic ABC

S. S. Chen, Ph.D., P.E., Dept. of Civil, Structural, & Environmental Engineering, University at Buffalo

In collaboration with

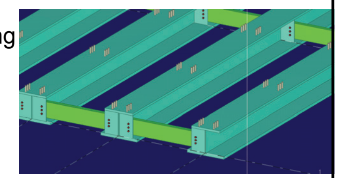
A. M. Shirole', P.E., Arora and Associates, P.C., J. Puckett, Ph.D., P.E., Univ. of Wyoming, and current and former students I.-S. Ahn, Q. Gao, H. Hu, R. Srikonda, J. Li, V. Tangirala, K. Potturi, R. Patil, J. Repp, and N. Kannan

Vision

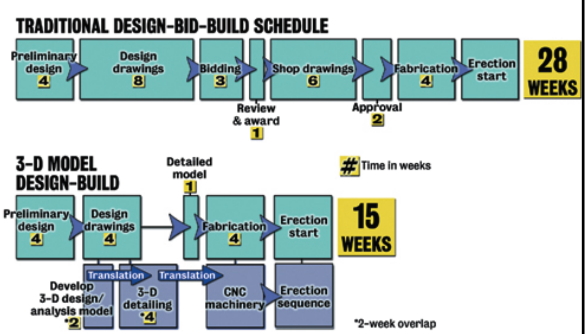
Bridge delivery via integrated 3D parametric Bridge Information Modeling (BIM, BrIM) that is:

- better,
- faster, and
- more economical

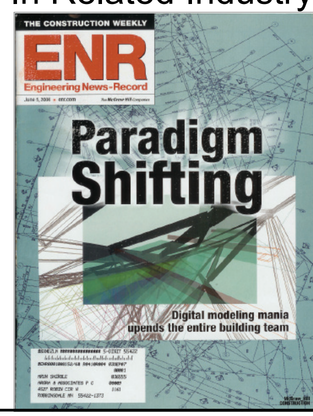
and provides a basis for ongoing lifecycle mgt.



What This Is About



In Related Industry…



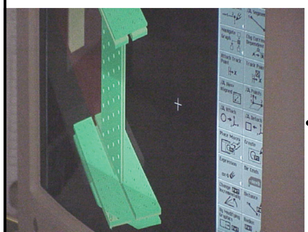
Outline

- Vision
- Selected Background History
- Some Progress
- Towards Changing an Entire Industry
- Some Recent and Current Developments

Automation Topics

- **Computer Aided Detailing**
  - Solid Modeling of Details
  - Link to Fabrication Equipment
- **Automated Welding and Cutting**
  - CAD files to machine control code
  - Alternate welding procedures
- **Automated Inspection**
  - Automated Ultrasonic Inspection
  - Reduced Use of Radiography

Shop Plans vs. Solid Model



- Solid Model Shows Fit-Up of Pieces
  - Dimensioning Errors Found
  - Interferences Located
  - Must Provide Piece Sizes for Designer Review
- Are Drawings Needed for Review?
  - Three Dimensional Solid Model Easier to Review
  - Faster Turn Around

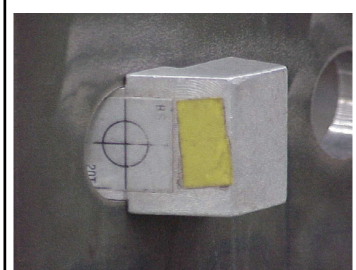
Dimension Checks (Pre-Assembly, a.k.a. Laydowns)

- Slow
- Ties up space
- Is it necessary?
  - Owner: Eliminates field construction delays and user costs
  - Fabricator/Erector: Not required for many jobs

As Applied to Segmental

- Many pieces
- Match cast
- Bridge geometry controlled by adjusting each piece
- Fabricator and erector the same

Why Not Virtual Assembly?



- Survey geometry and compare with 3-Dimensional drawing data
- Corrections?
  - Connection plate geometry
  - Milling of sections

Automated Three Dimensional Fabricated Assembly Geometry Check

- **Accurate**-Laser +/- 0.5 mm.
- Measurements Checked with Structures Geometry File
- **Fast**- Automated Scanning and Data Recording
- Targets Can Be Used for Erection Geometry Check
  - Actual As Built Geometry Recorded
  - Use for Future Structural Health Monitoring

The Future
No Paper?

Computer Aided Engineering and Modeling

Computer Aided Drawing Extraction

Machine Control

Geometry Verification

One File

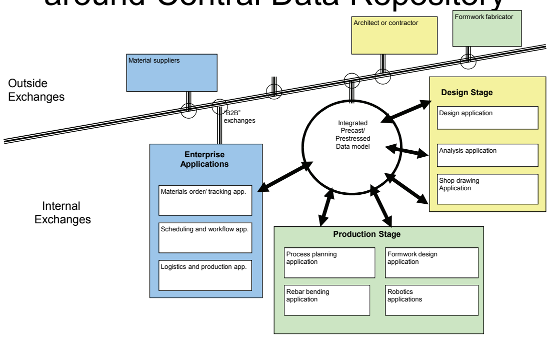
“Theme Areas” Progress 2001-Present

1) 3D Modeling & Electronic Info. Transfer: NCHRP 20-07 Task 149 & IDEA N-912 Projects (Nov. 2003 & Aug. 2006)
2) Standardized Specs and Approval Processes: NSBA/AASHTO Collab’n
3) Standardized Design Details: NSBA/ AASHTO Collab’n
4) Showcase of Benefits of Automation ➔ current FHWA work

2D vs. 3D

2D CAD provides an Electronic “drawing board”	*3D CAD enables a parametric model*
2D Drawings contain the information	*3D model contains the information; drawings are only reports*
2D Drawings intended to be human-readable; separate manual data entry is required for analysis	*3D model is computer-readable, such that direct analyses are possible*
Coordination is difficult; information is scattered among different drawings and specifications clauses	*Coordination is automatic: 3D model is the single source for all product information*
Manual checking	*Automated checking*
No support for production	*Potentially full support for production (via CNC codes etc.)*

Concept: Process Integrated around Central Data Repository



Fundamental Principles

- The 3D – centric approach conceptually boils down to two distinct principles that must be enforced consistently and according to appropriate standards:
  - ***Nobody drafts anything*** (model it in parametrically in 3D instead).
  - ***Enter*** each given ***data*** entry item ***only once***.
- Thus:
  - Drawings, if needed, can be generated from the 3D model whenever possible, and
  - Electronic data exchange is required between current “islands of automation” (a.k.a. “stovepipes”).

Benefits (e.g.)

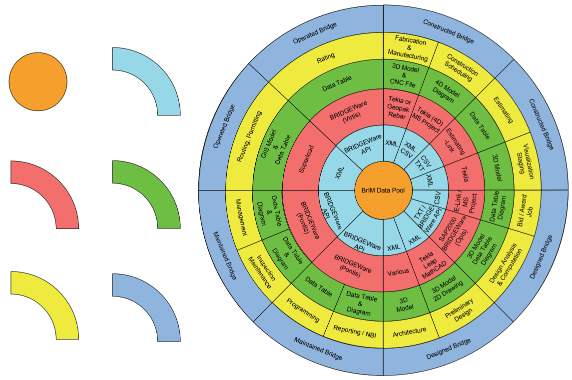
Description	**Better**	**Faster**	**Econo-mical**
Avoid error-prone manual data re-entry	X	X	
Avoid errors due to inconsistent information	X		X
Leverage design data into construction and beyond	X	X	X
Can avoid physical pre-assembly		X	X
Accelerated construction via prefabrication		X	

What is Needed to Make It Happen

- Parametric 3D Product Modeling (BIM/BrIM: Bridge Information Modeling)
- Electronic Data Transfer/Translation among the various “islands of automation” (stovepipes)
  - “If there is an ***industry - wide*** standard for electronic data exchange of (life-cycle) bridge data that is ***non-proprietary***, we will gladly write translators for it” – a software solution provider

Current FHWA project work is addressing these, emphasizing the 2nd (in lieu of a data standard).

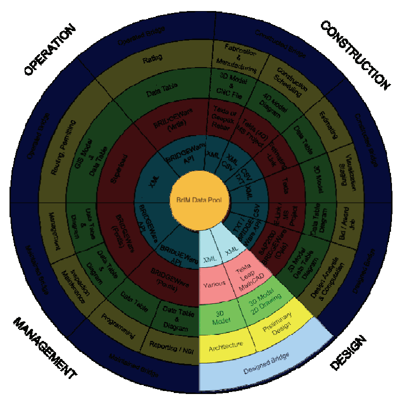
The Bridge Enterprise



Linking These Enables (e.g.)

- Exchange of model data with various project stakeholders
- Modeling of structure design and detailing in 3D (➔ 4D ➔ 5D) and managing changes
- Data transfer with planning and (e.g., CNC) automation systems for fabrication
- Delivery of quality output (construction documents) for an error-free project
- Planning and managing of erection in 3D (➔ 4D)
- Bridge Operations & Maintenance: e.g., routing & permits, update from inspections for rating

Preliminary Design



Preliminary Design: Commercial Software Considered

- LEAP Bridge, RM Bridge
- Geopak Bridge
- Opis (AASHTOWare)
- MathCad for alignment-compatible geometry
- QConBridge/WSFL
- SAP2000/BridgeModule

Preliminary Design: Linkage Software Developed

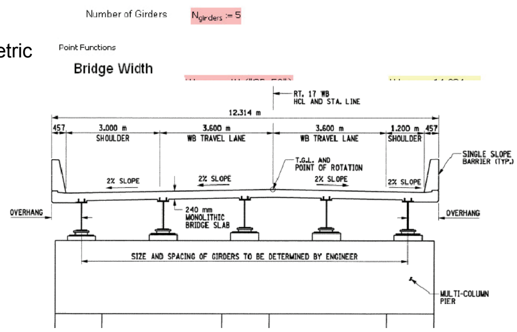
Using:	For:
• VBA	• Substructure geometry
• XML	• Mathcad interoperability
• landXML	• Hwy alignment compatible superstructure
• XML	• AASHTOWare

Preliminary Design: Product(s) of the Linkage Software

- Preliminary 3D parametric model
- 2D drawings that can be extracted/generated from it (e.g. design alternates)
- Textual reports

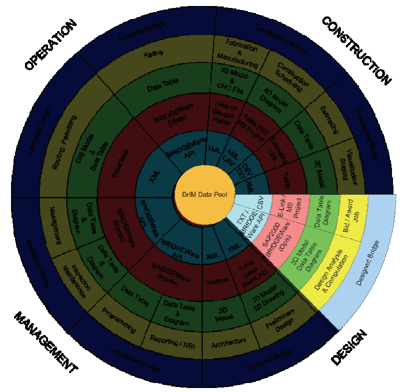
Preliminary Design (cont'd)

e.g., parametric design inputs



e.g., 2D view generated from inputs

Final Design



Final Design: Commercial Software Considered

- LEAP Bridge, RM Bridge
- Geopak Bridge/Rebar
- Opis (AASHTOWare)
- Excel/MathCad
- QConBridge/WSFL
- SAP2000/BridgeModule
- Tekla Structures
- StructureWorks/SolidWorks

Final Design: Linkage Software Developed

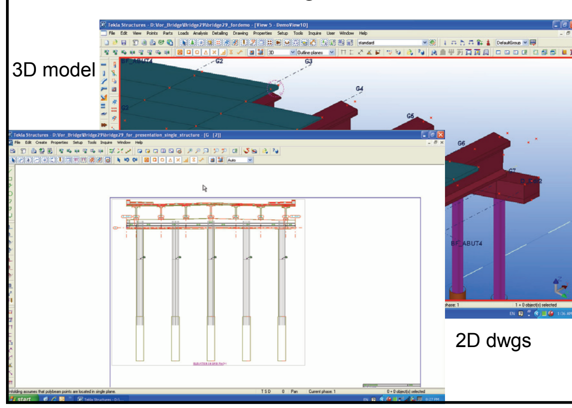
Using:	For:
• XML	• Bridge geometry into detailing software
• Text file generation	• POL – based input
• Direct export	• 2D dwg production

Final Design: Product(s) of the Linkage Software

- Detailed 3D parametric model (e.g., bolts, welds, rebar details, etc.) augmentable for "downstream" operations
- 2D drawings that can be extracted from it
- Textual reports

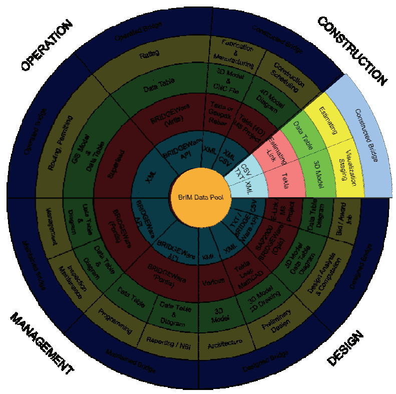
Final Design, cont'd

3D model



2D dwgs

Construction (Estimating/Scheduling)



Construction (Estimating/Scheduling): Commercial Software Considered

- Bentley TriForma
- Timberline estimating
- Estimating link

Construction (Estimating/Scheduling): Linkage Software Developed

Using:

- Direct quantity extraction (Bentley); Tekla
- "4D" in Tekla
- Direct linkage (EL to MS Project)

For:

- Quantity takeoffs for estimate spreadsheet
- Connect shipped objects to schedule (Mat'l Management)
- Connect shipped objects to schedule (Mat'l Management)

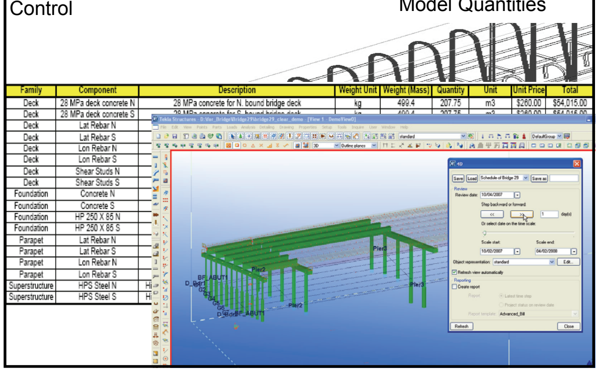
Construction (Estimating/Scheduling): Product(s) of the Linkage Software

- Model quantity takeoffs suitable for estimating and/or material procurement and materials management
- Cost estimates – easily modifiable
- "4D" erection schedule support
- Cash flow management via model-generated piece marking and associated shipping and installation status tracking

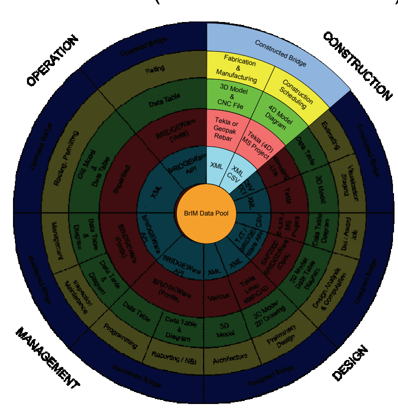
Construction (Estimating/Scheduling) cont'd

"4D" Schedule Control

Estimates
Model Quantities



Construction (Fabrication/Erection)



Construction (Fabrication/Erection): Commercial Software Considered

- Bentley/ Geopak Rebar
- Tekla Structures
- Fabtrol
- Solidworks/Structureworks
- RM Bridge

Construction (Fabrication/Erection): Linkage Software Developed

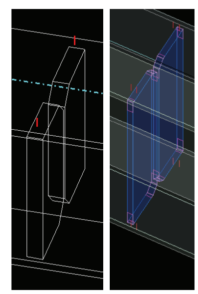
Using:	For:
• Tekla API	• Model info into detailing
• XML	• Model info into detailing
• CNC export	• Driving CNC simulations of factory equipment

Construction (Fabrication/Erection): Product(s) of the Linkage Software

- Fully detailed model
- 2D dwgs (e.g., for shop drawing review), extractable from model, containing all information needed for fabrication
- CNC export (to drive suitable fabrication equipment)

Manufacturing Too (via CNC)

– Automatic Pop-marking
– Stiffener plates etc.,
– Avoid manual layout process

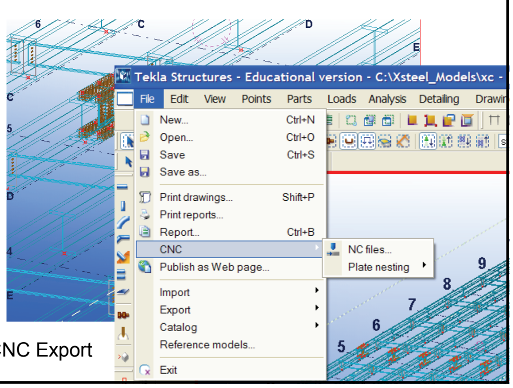


MULTI-USER MODE

- Different people working together using a single model
- Within an organization and discipline
- Between organizations and disciplines

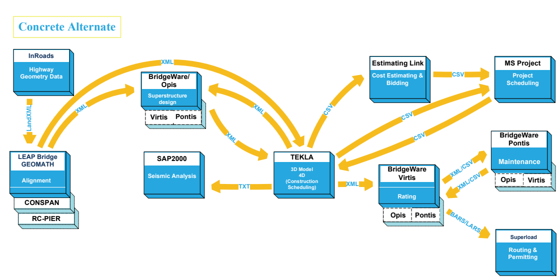
Construction (Fabrication/Erection) cont'd

Fully detailed model



CNC Export

Information Workflow



Resolution by AASHTO SCOBS (June 2005)

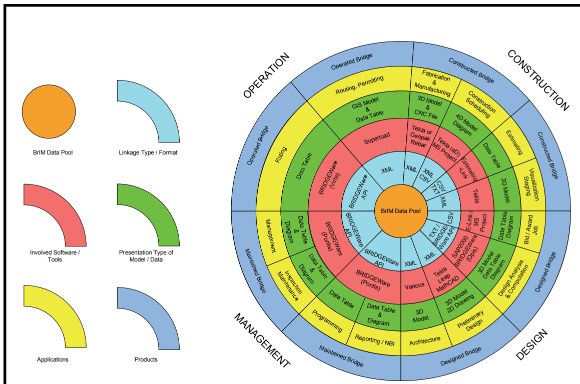
Be it Resolved: That the AASHTO Highway Subcommittee on Bridges and Structures acknowledges the importance of "Comprehensive Integrated Bridge Project Delivery through Automation" in achieving its goals. Further Subcommittee affirms its leadership role by charging one of its existing Technical Committees or a separate Task Force to coordinate further development, refinement and transfer of this technology in partnership with the FHWA.

Extending Linkages

- CAD (Computer-Aided Design)
- CIM (Computer-Integrated Manufacturing)
- Construction Modeling (e.g., Erection)
- Construction Management
- Operations, Maintenance, Lifecycle Management

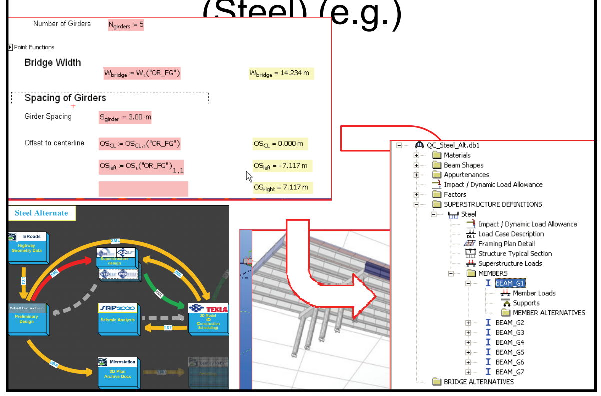
What is Needed

- Complete 3D Parametric Modeling in (Standardized?) (Accessible? - via API?) Digital Format
- D/B Mindset vs. D/B/B Adversarial Fragmentation
- Re – Shaping of Stakeholder Roles

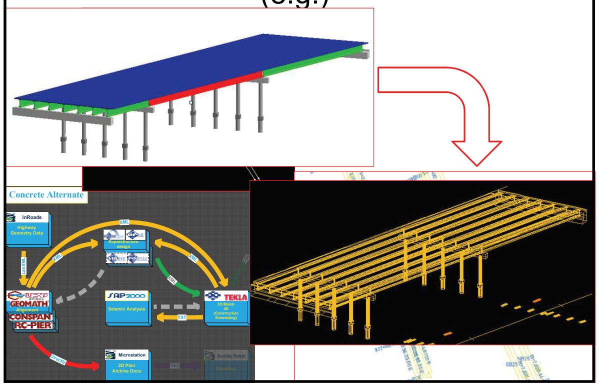


Quincy Avenue Bridge

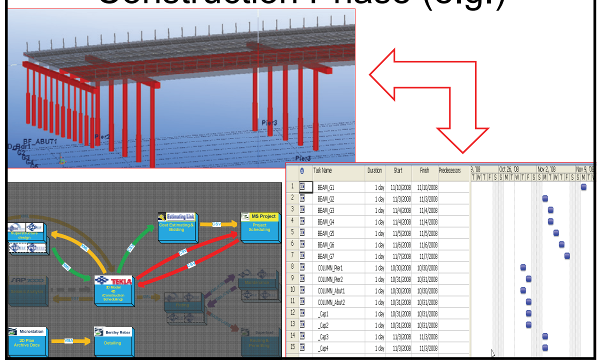
Linkages in Design Phase (Steel) (e.g.)



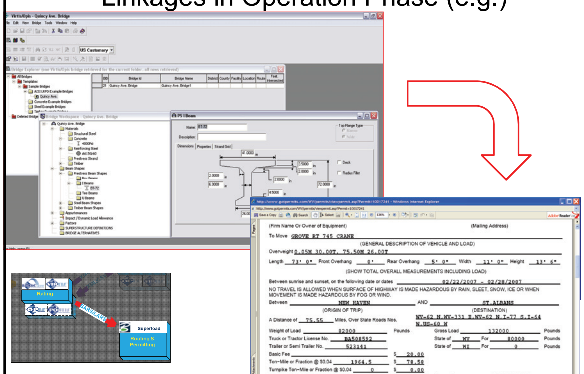
Linkages in Design Phase (Concrete) (e.g.)



Linkages in Fabrication & Construction Phase (e.g.)

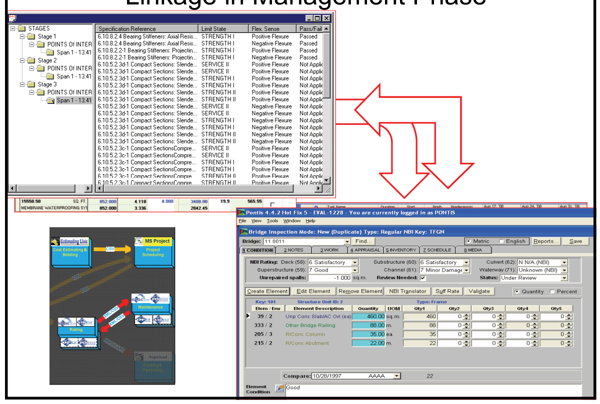


Linkages in Operation Phase (e.g.)



Slide courtesy BSI

Linkage in Management Phase



QuickTime™ and a decompressor are needed to see this picture.

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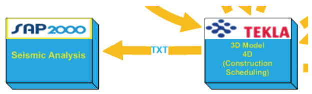
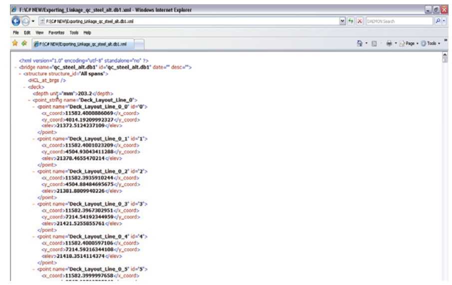
SAP2000 Linkage



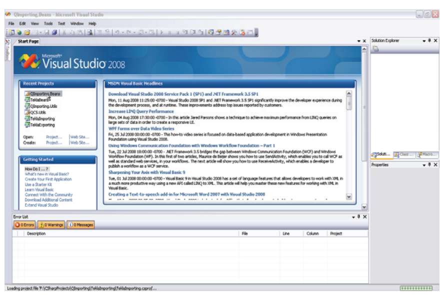
SAP2000 Linkage

The XML file



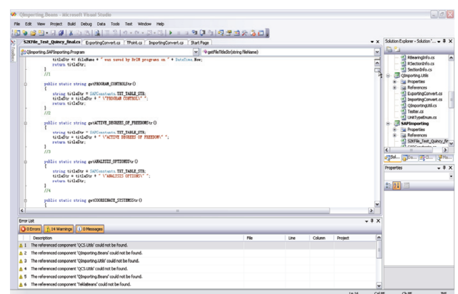
SAP2000 Linkage

Visual Studio software



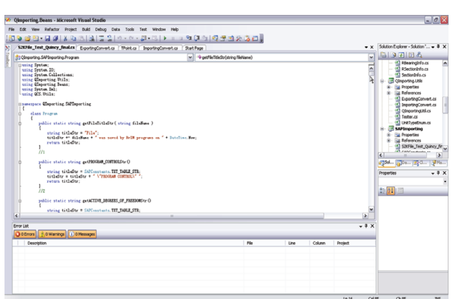
SAP2000 Linkage

C# code



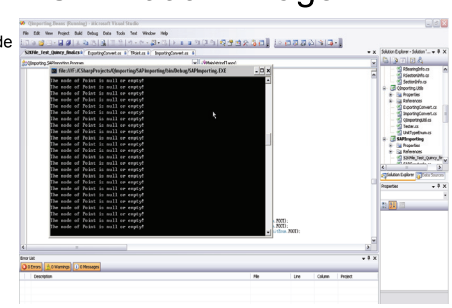
SAP2000 Linkage

C# code



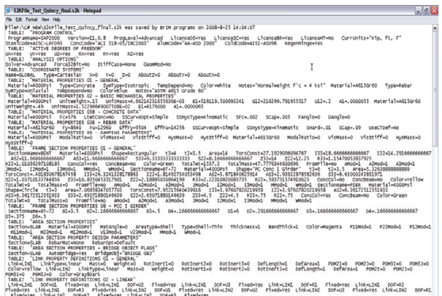
SAP2000 Linkage

Run C# code



SAP2000 Linkage

SAP2000 text input file (*.$2k file)



Summary and Status

- Developed a Prototype Integrated System, Illustrating Data Exchanges and Applications, that Addresses Entire Bridge Life Cycle
- Utilized 3D Bridge Information Modeling (BrIM) as a Technology to Accelerate Bridge Project Delivery and Enhance Life Cycle Management
- Will be Demonstrating the Viability, Efficiencies, and Benefits of the Integrated Bridge Project Delivery and Life Cycle Management Concept Through One-Half-Day and Two-Day Presentations of the Prototype Integrated System to Stakeholders Around the Country

Some (Hard?) Questions

- (Civil/Bridge) Engineering Education: Whither BIM/BrIM (CAD for that matter)?
- State DOT - drivenness: hindrance or opportunity?
  - Planning/Env'l/Design/Const'n/Maint: “You cannot have those silos anymore”; “Connectivity of systems gives you power”; Transparency has advantages
  - You can drive positive change (e.g., NYSDOT 3D substructure requirement)
  - 2004: “…and you want me to go through that again?”
  - 2008: “You like these world hunger problems, don't you?
- Software Development Companies: Acquisition = Integration?; Organizational Stovepipes?

Summary and Conclusions

Envisioning future accelerated bridge delivery based on the following:

- Comprehensive information-centric (BrIM) approach to the planning, design, construction, operation and maintenance of bridges through a single coordinated shepherding of that information as it evolves
- Coordinated leveraging of design information into “downstream” operations, including ongoing operation & maintenance

Acknowledgements

- NCHRP, Mr. D. Beal, P.E., and Dr. I. Jawed, Program Officers; FHWA, Dr. P. Yen and Mr. K. Verma, P.E., COTR's
- Bentley Systems, Inc.
- Various Academic Software Discounts and Tech Support

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NCEER-87-0001 "First-Year Program in Research, Education and Technology Transfer," 3/5/87, (PB88-134275, A04, MF-A01).

NCEER-87-0002 "Experimental Evaluation of Instantaneous Optimal Algorithms for Structural Control," by R.C. Lin, T.T. Soong and A.M. Reinhorn, 4/20/87, (PB88-134341, A04, MF-A01).

NCEER-87-0003 "Experimentation Using the Earthquake Simulation Facilities at University at Buffalo," by A.M. Reinhorn and R.L. Ketter, to be published.

NCEER-87-0004 "The System Characteristics and Performance of a Shaking Table," by J.S. Hwang, K.C. Chang and G.C. Lee, 6/1/87, (PB88-134259, A03, MF-A01). This report is available only through NTIS (see address given above).

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