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# LONGITUDINAL SEISMIC RESPONSE OF PRECAST SPLICED GIRDER BRIDGES

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## ABSTRACT

A research project is underway at the University of California, San Diego (UCSD) in conjunction with the California Department of Transportation (Caltrans) and the Precast/Prestressed Manufacturers Association of California (PCMAC) to cyclically test precast/prestressed high performance concrete (HPC) spliced girder bridges which incorporates Caltrans' rigorous seismic design and detailing requirements. At UCSD, two 40% of full-scale models were tested under fully reversed seismic loading in the longitudinal direction (parallel to the direction of traffic) to verify the adequacy of the newly developed integral capbeam-girder-column details. The first unit incorporated Modified Florida Bulb-Tee girders, while the second unit featured Bathtub girders. Testing of both models was completed in May 1997. Model design, with particular emphasis on seismic design and detailing, model construction and preliminary test results are the subjects of this paper.

## INTRODUCTION

In California, bridges with precast elements made up only 3% of the total bridges built in the late 1980's and the early 1990's[1]. An important reason for this statistic is due to seismic concerns. Typical precast bridges constructed in California consist of simply supported girder elements made continuous with a cast-in-place deck, while in-situ construction features full continuity between the column and superstructure. Design engineers have been reluctant to consider the connection between the precast girders and the columns as completely fixed under seismic loading and the resulting substructure of the precast girder bridges have been larger than cast-in-place bridges to compensate for this lack of continuity.

As a consequence, a research project was started by the California Department of Transportation (Caltrans), in conjunction with the Precast Manufacturers of California (PCMAC) to evaluate this problem. Their findings were to replace the existing simply supported configuration with a construction system which incorporates precast high performance concrete (HPC) spliced girders connected with continuous post-tensioning[2]. Splicing the girders allows the system to be continuous for dead load as well as live load, making the system more efficient. Along with the use of precast elements featuring HPC,

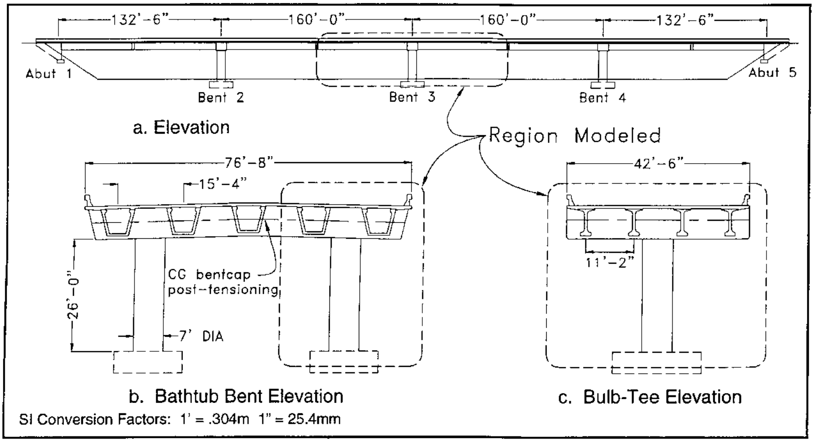



**Figure 1. Prototype**

this construction scheme allows a substantial reduction of superstructure weight and corresponding seismic mass.

Preliminary design studies showed that designing and detailing an integral superstructure-column connection was possible. However, verification through large scale testing was required[2]. As a result, a research project is currently underway at the University of California San Diego (UCSD), sponsored by Caltrans and PCMAC, to cyclically test precast/prestressed concrete-spliced girder bridges which incorporate Caltrans' rigorous seismic design and detailing requirements. At UCSD, two 40% of full-scale models were tested under fully reversed seismic loading in the longitudinal direction (parallel to the direction of traffic) to verify the adequacy of the newly developed integral capbeam-girder-column details. The first test featured Modified Florida Bulb-Tee girders while the second incorporated Bathtub or "U" shaped girders referred to, in the following text, as the Bulb-Tee Model and the Bathtub Model respectively.

## MODEL DESIGN

### Prototype

A detailed design study was carried out on the prototype bridges shown in Figure 1. Dimensions and forces for both models were scaled from the prototype. Due to the repetitive nature of the structural configuration, only one bent was needed to capture the seismic behavior. The region studied extends from midspan to midspan, roughly the location of the seismic moment inflection points in the spans adjacent to Bent 3.

Two design studies were carried out on the four span prototype. The Bathtub girder bridge prototype had two columns per bent which allowed flexural pins at the base of the columns. Since the study was concentrating on the longitudinal response, it was assumed that modeling one of the two columns, along with the adjacent girders and contributory bentcap and deck, was sufficient to capture the behavior. The Bulb-Tee girder prototype represents a typical two lane single column bent structure. Hence, full fixity at the bottom of the column was required.

The design 28 day concrete compressive strength for the girders, listed in Table 1, was calculated based on the maximum service compression stress divided by 0.4, as required by Caltrans Bridge Design Specifications[3]. A full live load analysis was performed using HS 20-44 loading for service loads, and a combination of Permit and HS loads was used to determine the ultimate moments and shears. No mild reinforcing was required over the bentcap for the gravity ultimate loads.

*Table 1. Prototype Design Concrete Strengths*

| Item | Design 28 Day Compressive Strength MPa (psi) |
|---|---|
| Girder Segments | 43.8 (6,350) Bulb-Tee<br>44.8 (6,500) Bathtub |
| Deck | 27.6 (4,000)* |
| Bentcap | 27.6 (4,000)* |
| Column | 22.4 (3,250)* |

* Minimum required compressive strength[3].

## Model Details

**Column** - The amount of longitudinal reinforcing in the columns for both models was determined by both gravity and seismic loading. The area of longitudinal steel required was 1% of the gross cross sectional area. However, 2% was used for both model tests to increase the severity of the seismic effects on the superstructure. Both of the model columns were designed to sustain a structural displacement ductility of 4 ($\mu_\Delta = 4$), which is required by Caltrans Design Specifications[3]. Confinement steel in the plastic hinge region was designed based on moment curvature analysis using the confined concrete model developed by Mander et al[4].

The Bathtub Model incorporated a flexural hinge at the base of the column and the corresponding design shear strength was 60% of the Bulb-Tee Model column. The pin was designed using the following Caltrans criteria:

$$V^o \leq 1.4\left(A_s f_y + P\right) \tag{1}$$

$$V^o \leq A_{core} \times 0.2 f'_c \tag{2}$$

Where $V^o$ is the column overstrength shear demand, $A_s$ is the pin reinforcement required, $A_{core}$ is the concrete area in the pin and $P$ is the axial load due to dead and seismic loads[3].

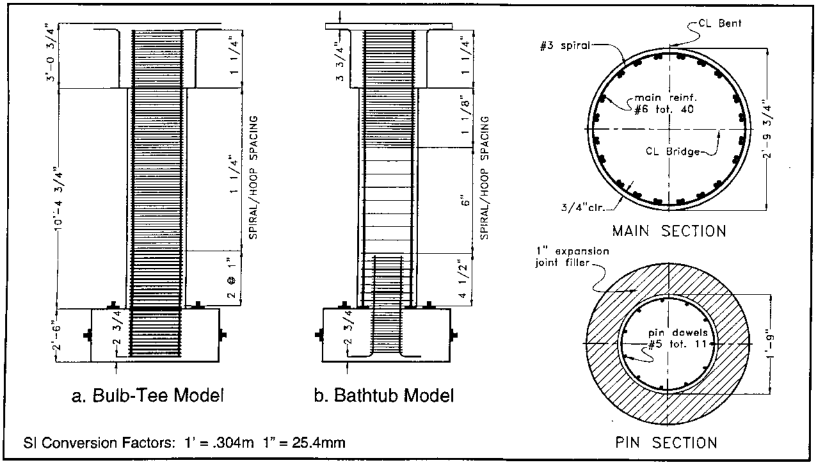



**Figure 2. Model Column Details**

**Bentcap** - The bentcaps for both models, shown in Figure 3, were 92mm (3 5/8") deeper than the superstructure to allow the main reinforcement to pass either above and below the precast girders. As a result, only five post-tensioning tendons were required to pass through the precast Bulb-Tee girders.

Bentcap first stage post-tensioning was designed to carry the girders, the fluid deck weight and miscellaneous construction loads with zero tensile stress at the top fiber of the non-composite bent-cap. The bentcap torsional resistance was calculated based on a plastic shear friction model because the distance from the column face to the side of the girders is relatively small (on the order of several inches) and a full spiral crack, assumed to form in most reinforced/prestressed concrete torsion models, can not form in the cap in this small region. The bentcap prestressing provided most of the normal force on the friction plane perpendicular to the bent centerline, between the column and the first adjacent girder[5].

The joint section, which is the same for both models, is shown in Figure 3c. The bentcap was designed wider than the column so the vertical joint shear reinforcement could be placed outside of the column core region in order to reduce congestion. The purpose of the joint stirrups was to help transfer the column tension force up to the top of the joint. The amount of joint shear reinforcement $A_{jv}$ was calculated using the following equation:

$$A_{jv} = 0.125 A_{sc} f_{yc}^{o} / f_{yv} \tag{3}$$

Where $A_{sc}$ is the area of the column longitudinal steel, $f_{yc}^{o}$ is the ultimate stress of the column reinforcement and $f_{yv}$ is the yield stress of the vertical joint shear reinforcement[5].

The strut and tie mechanism used to develop Equation (3) has an unbalanced horizontal component resisted by the hoops around the column in the joint region. The required volumetric hoop reinforcement ratio can be calculated as:

$$\rho_s = \frac{0.6 A_{sc} f_{yc}^{o}}{l_a^2 f_{yh}} \tag{4}$$

Where $l_a$ is the length of the column bar extension in to the joint region, and $f_{yh}$ is the yield stress of the hoops[5]. The provided ratio was $\rho_{s\ \text{provided}} = 0.011$ and the required ratio was $\rho_{s\ \text{required}} = 0.016$. The difference was made up with the split hairpins shown in Figure 3c. Longitudinal reinforcement was also required to return the horizontal component of the diagonal compression strut to the compression zone of the column. The area of reinforcement required was approximately 1/2 of the vertical area of steel required in Equation (3), and since the joint stirrups continue along the bottom of the joint, the requirement is satisfied.

The Bathtub girders terminate 2 3/8" [60 mm] into the capbeam, as shown in Figure 3d, to allow for easier form-up, and to provide some clamping from the bentcap post-tensioning to improve shear resistance at the bentcap-girder interface. Girder longitudinal soffit reinforcement and the pretensioning strand extended into the cap and were lapped to provide positive moment resistance. Girder post-tensioning ducts were spliced through

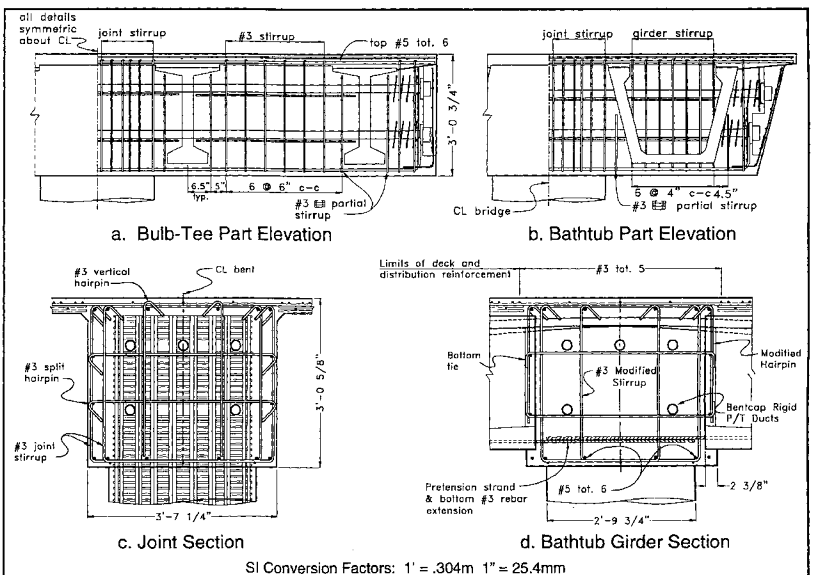



**Figure 3. Model Bentcap Details**

the capbeam.

**Superstructure** - The Bulb-Tee Model part-typical section in Figure 4a shows the different girder segments. The pier segments, which pass continuously through the bentcap, had pretensioning in the top flange of the girder to carry the negative moments due to the girder self weight and the reaction from the span segment. Strands placed in the bottom flange for transportation purposes were blanketed along the girder with an exception of the 787mm (2'-7") bonded region at each end, which were used to anchor the strands. Span segments had an identical cross section to the pier segments with the exception of the two fully bonded strands at the bottom of the girder instead of the top. Pretension strands were left extended from the ends of both the pier and span segments and bent into the girder splice.

The Bathtub Model typical section in Figure 4b shows two different sections of the girder segment. The prismatic (midspan) section runs from the face of the end diaphragm to the start of the flare section, which was 1473mm (4'-10") from the end of the girder. The thickness of the web and the soffit vary linearly in the flare section, from zero to 60mm (2 3/8") at the end of the girder. The girder flare not only increases the concrete shear capacity of the superstructure, it also allows the girder post-tensioning ducts to be placed side by side, thus increasing the prestress eccentricity.

Post-tensioning for both models, listed in Table 2, was applied in stages to model the assumed prototype construction sequence. First stage post-tensioning was applied in the lower tendons after the bentcap (and splices in the Bulb-Tee Model) had reached the code required minimum compressive strength of 20.7MPa (3,000psi) and second stage was applied after the deck was cast[3].

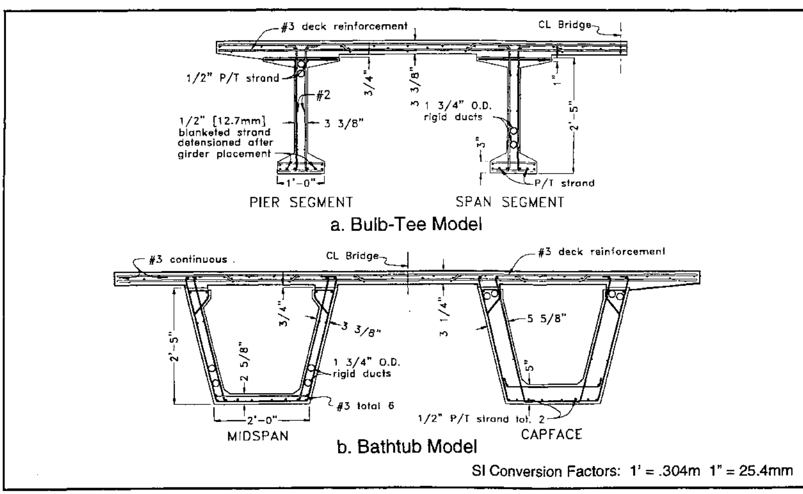



**Figure 4. Model Superstructure Details**

*Table 2. Model Jacking Force (per Tendon)*

| | Bulb-Tee Model | Bathtub Model |
|---|---|---|
| **1st stage** | 413 kN (93 kip) | 556 kN (125 kip) |
| **2nd stage** | 551 kN (124 kip) | 556 kN (125 kip) |

The column plastic moment applied to the superstructure is concentrated at the top of the column, resulting in a non-uniform distribution of moment across the superstructure width. Recognizing this, Caltrans engineers use an "effective width" approach to compute the superstructure capacity to resist plastic hinging. For "T" beam or "I" girder bridge superstructures, an effective width can be calculated as:

$$W_{eff} = D + H_s \tag{5}$$

Where $D$ is the column diameter and $H_s$ is the height of the superstructure[5]. Equation (5) essentially reduces the contributory superstructure width to the two webs (or girders in the case of the Bulb-Tee Model) adjacent to the column.

## TEST SETUP AND LOADING SCHEME

### Horizontal Displacement Cycles

The test setup shown in Figure 5a is a 40% scaled representation of the prototype region shown in Figure 1a. Horizontal actuators were placed on both sides of the unit to model the seismic inertia forces acting along the bridge under longitudinal response, while the four vertical actuators at the corners of the unit applied the seismic shear into the superstructure. By assuming that the seismic inflection points remain at the prototype midspans, the vertical actuators were programmed to hold the ends of the test unit to an elevation essentially constant with respect to the bentcap throughout the loading history.

Since the model self weight and required prototype dead load were different multiples of the model scale, and only 1/2 of the span is modeled on either side of the bent, additional forces were needed to correctly model the dead load and prestress secondary moments. As a result, dead load hold-downs were placed 5.49m (18') from the bent centerline. The dead load hold down forces were applied before the deck was poured in order to correctly model strain differential between the deck and the girders, since the prototype girders initially support the deck as a fluid weight.

For both models, initial horizontal force cycles were performed up to the computed first yield of the main steel reinforcement of the column. The experimental stiffness was calculated based on the measured displacement at this force level. The experimental stiffness based on the displacement measured at this force level was used to calculate $\Delta_y$. Three cycles were then carried out at $0.75\Delta_y$, $\mu_\Delta = 1, 1.5, 2, 3, 4, 6$ and 8.

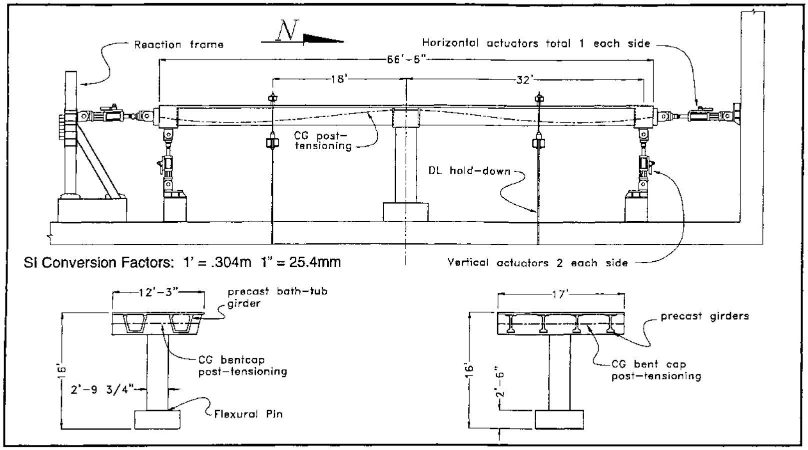



**Figure 5. Model Test Setup**

## Superstructure Capacity Test

Since the structure was designed using capacity design techniques where ductile plastic hinges were to form in the columns, the superstructure/bentcap regions were to remain essentially elastic. Therefore, a second test was performed to evaluate the strength and ductility capacity of the superstructure using the test setup shown in Figure 6a. After the model was returned to approximately zero displacement, the dead load hold-downs and

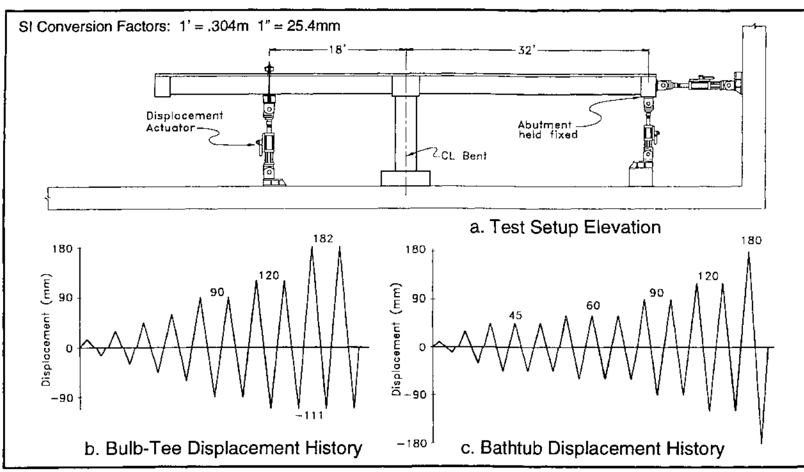



**Figure 6. Superstructure Capacity Test Setup**

the south actuators were removed. A new actuator(s) (two actuators were used for the Bulb-Tee Model while one was used for the Bathtub Model) was placed 5.47m (18') south from the centerline of bent. The north actuators held the north abutment fixed while the south actuator(s) applied increasing cyclic loads to failure.

Vertical displacements were measured at both the east and west edge of deck at the south actuator (5.49 m (18') south of the bentcap centerline). Displacement cycles, first in the push (up) direction followed by the pull (down) direction were carried out using the loading sequence in Figures 6b and 6c.

## PRELIMINARY TEST RESULTS

### Horizontal Displacement Cycles

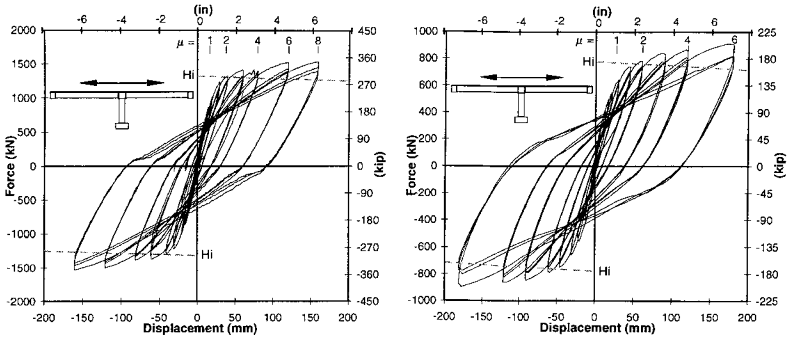
**Bulb-Tee Model** - At first yield the first deck cracks were observed. The first horizontal joint crack was observed at $\mu_\Delta = 1$ and the first diagonal joint cracking occurred at $\mu_\Delta = 1.5$. Cracking of the girder splice was also observed at this ductility level. General cracking of the column, bentcap and superstructure continued up to $\mu_\Delta = 8$. Superstructure and bentcap cracking shown in Figure 9 closed after removal of the horizontal loading, which was largely due to the prestressing in these regions. During the second cycle at $\mu_\Delta = 8$, popping noises were heard coming from the test unit. It is likely that the noise heard were the 9.5mm (#3) column hoops fracturing. After the third cycle at $\mu_\Delta = 8$, several weld fractures of the 9.5mm (#3) column hoops were observed at the base of the column and the longitudinal bars on the south side of the column had displaced laterally, indicating the onset of buckling and failure of the plastic hinge.

The force displacement response, shown in Figure 7a, reflects increasing lateral strength of the model up to $\mu_\Delta = 8$, which far exceeded the design capacity of $\mu_\Delta = 4$. The measured nominal strength $H_i$, with correction for P-Δ effects, was exceeded by approximately 27%. ***Hi*** was calculated using the following equation:

$$H_i = (M_{i(top)} + M_{i(base)})/L \qquad (6)$$

where $M_i$ is the nominal moment capacity of the column based on a moment curvature analysis at a peak compression strain of 0.004 and $L$ is the clear height of the column. However, the measured peak lateral strength of 1544 kN (344 kip) was 6% less than the peak predicted lateral strength. It appears that the lateral strength was less than predicted due to an apparent sleeveing of the column into the joint which increased the effective length of the column.

**Bathtub Model** - At first yield, the first horizontal joint crack appeared at approximately 1/4 of the joint depth from the bentcap soffit, indicating the activation of the joint shear reinforcement. Diagonal joint shear cracks were first observed at $\mu_\Delta = 1$ on the negative bending side of the bentcap. Cracking intensity and measured lateral force continued to increase up to $\mu_\Delta = 6$. During the second cycle, several banging noises were heard coming from the structure, and the base of the column was first observed




a. Bulb-Tee Model b. Bathtub Model

**Figure 7. Horizontal Force Displacement Response**

sliding along the support. The banging noises were likely due to the pin reinforcement fracturing. The sliding displacement at the column base, measured from the peak horizontal push to the peak horizontal pull, was 25 mm (1") during the third cycle. Testing was then stopped at this ductility level due to sliding shear failure of the column base.

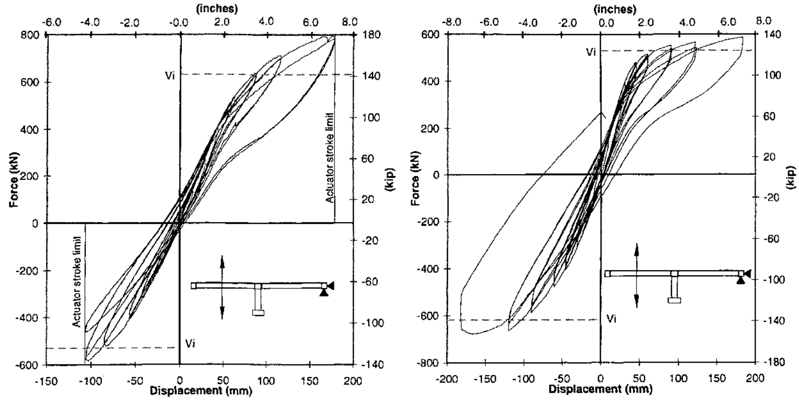
The force displacement response shown in Figure 7b reflects increasing lateral strength of the model up to $\mu_\Delta = 6$, which was greater than the design capacity of $\mu_\Delta = 4$. The nominal strength $V_i$ was exceeded by 31% due to the strain hardening of the longitudinal reinforcement and the lightly loaded main column section ($P_{axial} = 0.04f'_c A_g$, where $A_g$ is the gross concrete area of the column). The peak predicted strength was 4% greater than that measured due to strength degradation of the column pin.

**Superstructure Capacity Test**

**Bulb-Tee Model** - At 90 mm of vertical displacement, web shear cracking extended into flexural cracking in the push direction. In the pull direction flexure cracking developed inclined shear extensions. General flexure shear cracking continued until the actuator stroke limits of 182mm (7.17") and 111mm (4.37") had been reached, in push and pull directions respectively. At this level of displacement, no signs of spalling were observed in the girders, which indicated the likely reserve of significant ductility capacity.

The force displacement response shown in Figure 8a shows only minor degradation up to the actuator stroke limits in both directions which corresponds to $\mu_\Delta = 2.7$ and 1.5 in the push and pull directions respectively. The nominal strength was reached in the pull direction and was exceeded in the push direction by approximately 20%. The idealized nominal vertical force was calculated as:

$$V_i = (M_i \pm M_{DL})/L_{cant} \quad (7)$$




a. Bulb-Tee Model

b. Bathtub Model

**Figure 8. Superstructure Force Displacement Response**

Where $M_i$ is the computed nominal moment capacity at the cap face based on a moment curvature analysis and maximum compression strain of 0.004, $M_{DL}$ is the cantilever dead load moment at the capface and $L_{cant}$ is the distance from the displacement actuator to the face of cap.

**Bathtub Model** - General cracking intensity in the girders and deck increased up to a displacement of 120mm (4.72"). On the push cycle at 120mm (4.72"), the girder soffit pulled out of the bentcap and revealed a wide open crack at the end of the girder. On the reverse cycle, the bottom corners of the girder spalled as the soffit was unable to fit back into the space it previously occupied in the bentcap, as shown in Figure 10. Due to the large inelastic strains in the push direction and to the absence of cover concrete, the 9.5mm (#3) bottom reinforcement in the corners of the girders had buckled during this cycle. At 180mm (7.09") in the push direction, wide open cracking was observed at the girder bentcap interface, accompanied with minor spalling on the top of the deck. Major spalling of the girder soffit occurred in the bottom flange of the girders on the reverse cycle, which extended out approximately 305mm (1'-0") from the face of cap. Spalling along the girder soffit exposed the six 9.5mm (#3) and the two 12.7 mm (1/2") $\phi$ strands which extended into the bentcap. All of the exposed rebar and strand had buckled during the pull cycle.

The force displacement response shows only minor degradation up to a displacement of 180mm (7.09") in both directions, which corresponds to $\mu_\Delta$ = 4.4 and 2.4 in the push and pull directions, respectively, as shown in Figure 8b. The nominal strength was exceeded by approximately 10% in both the push and pull directions.

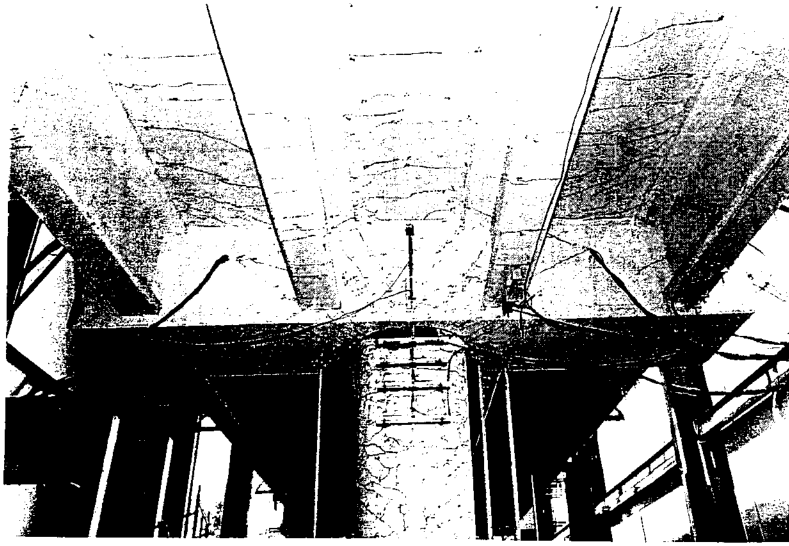


Figure 9. Bulb-Tee Model at $\mu_\Delta = 6$ (Horizontal Loading)


Figure 10. Bathtub Model (Superstructure Capacity Test)

## CONCLUSIONS

Two practical splice-girder designs, which incorporate HPC and an integral superstructure-column connection have been presented. Large scale testing verified the effectiveness of newly developed integral bentcap details under fully reversed simulated seismic loads in the longitudinal direction. Specifically, ductile plastic hinges formed in the columns out to $\mu_\Delta = 8$ and 6 in the Bulb-Tee and Bathtub Models respectively, with only minor strength degradation. Only minor cracking was observed in the bentcap and superstructure, which closed upon removal of the loading. This implies that repair of a prototype superstructure, using the details tested in this project will be essentially cosmetic, after a design level earthquake.

The superstructure capacity tests demonstrated that superstructures which incorporate precast HPC girders can perform in a ductile manner. The Bathtub Model reached $\mu_\Delta =$ 4.4 and 2.4 under positive and negative bending respectively, without any special seismic detailing. While failure in the Bulb-Tee Model was not observed, ductile response was recorded up to $\mu_\Delta = 2.7$ and 1.5 under positive and negative bending respectively.

## ACKNOWLEDGMENTS

This project was funded by Caltrans with generous donations from PCMAC, Dywidag Systems International, VSL International and the Dayton Superior Corporation.

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ACKNOWLEDGMENT

PCMAC wants to thank Caltrans, LoBuono, Armstrong & Associates, the University of California, San Diego and its producer members for their effort and support of this program. The UCSD testing program is a culmination of efforts by Caltrans, LoBuono, Armstrong & Associates, and the Precast Concrete Manufacturers Association of California (PCMAC) to develop concepts and prepare design details for use in seismic-resistant precast girder bridge systems. During this concept development phase which preceded the testing, special emphasis was placed on developing an integral connection between the girders and pier supports.