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SEISMIC DESIGN RECOMMENDATIONS FOR PRECAST SPLICED-GIRDER BRIDGES

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ABSTRACT

A design methodology for precast spliced-girder bridges, which feature superstructure-column continuity, has been developed by the California Department of Transportation in conjunction with the Precast / Prestressed Concrete Manufacturers Association of California and the University of California, San Diego (UCSD). Large scale structural testing at UCSD verified the adequacy of newly developed integral bentcap details under fully reversed simulated seismic loads in the longitudinal direction. Design recommendations were developed based on the test results combined with analytical modeling. Development of these design recommendations, with particular emphasis on integral bentcap design and superstructure performance, is presented.

INTRODUCTION

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, which are referred to as the Bulb-Tee and Bathtub models respectively, in the following text.

As part of the research project, a detailed design study was carried out on the prototype bridges shown in Figure 1. Due to the repetitive nature of the structures, only one bent from each bridge was considered necessary to capture the overall behavior. Hence, the laboratory test setups shown in Figure 2 are 0.4 scale representations of the regions outlined in Figure 1.

Testing of the models verified the adequacy of the newly developed integral superstructure column connection details, with the test units reaching $\mu = 8$ and 6 for the Bulb-Tee and Bathtub models respectively. While further information regarding model design, construction and testing can be found in references 1 and 2, this paper discusses design recommendations developed as part of this research project.

DESIGN RECOMMENDATIONS

The following design recommendations were developed based on the test results of the models, the design study of the prototypes and analytical models developed for this research project. A total of 5 specific design recommendations are presented in this paper, which relate to superstructure design, seismic performance of prestressed bentcaps and bentcap column joint design under longitudinal seismic response.

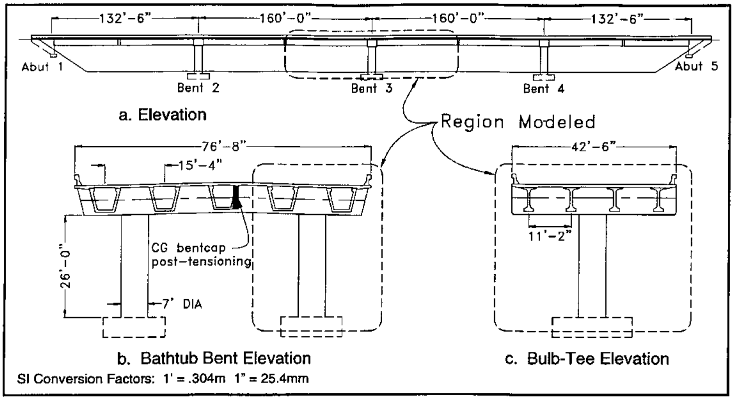


Figure 1. Prototype structures

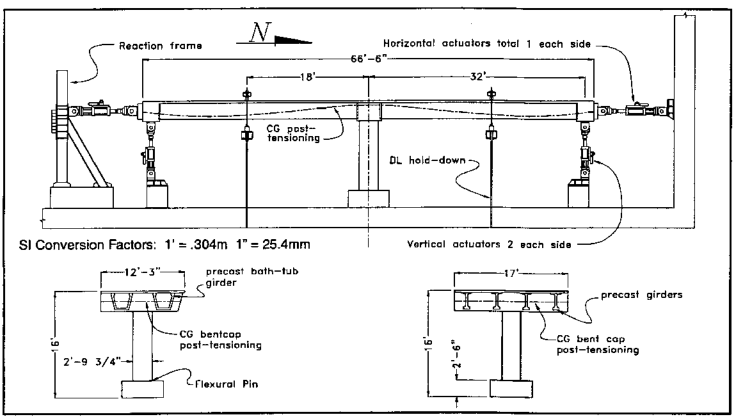


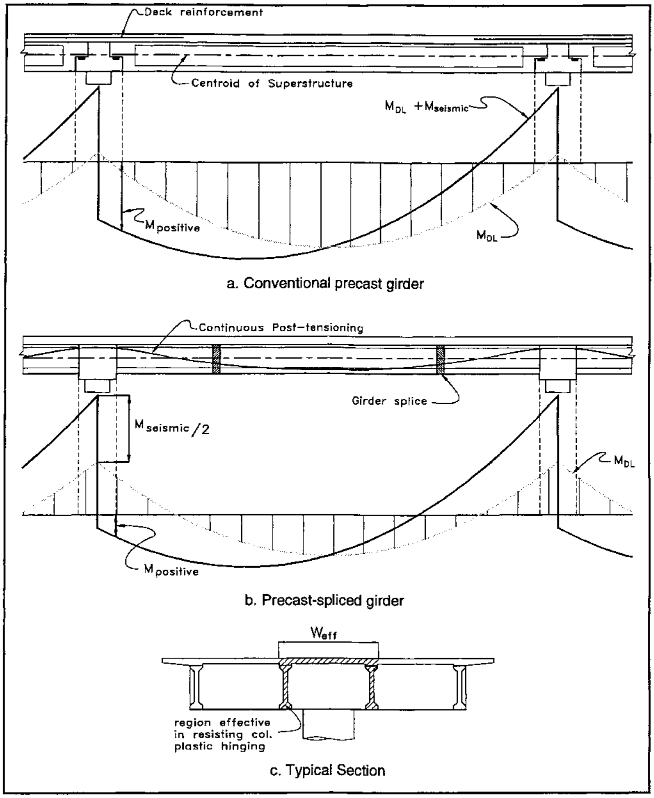
Figure 2. Model test setups

1. Make superstructures continuous for dead load.

Conventional precast girder structures consist of simply supported precast girders made continuous for live loads with a cast-in-place deck. Attempts have been made to provide continuity of the conventional precast-girder superstructure for seismic loading by connecting the bottom of the precast girder to the bentcap. This detail has been shown to be heavily congested, because the column imparts a non-uniform perstructure under longitudinal response. An effective width for "I" or "T" beam girders (superstructure width considered effective in resisting column plastic hinging) has been conservatively assumed as:

$$W_{eff} = D + H_s \tag{1}$$

where D is the gross column diameter and H_s is the height of the superstructure.[3] The effective width shown in Figure 3c limits the positive reinforcement to the bottom flange of the two girder adjacent to the column. Also, the design positive moment at the face of cap is approximately 1/2 of the column plastic hinge overstrength moment since the structure is non-continuous for dead load moment, as shown in Figure 3a. Therefore, the amount of reinforcement required in the bottom flange of the two girders adjacent to the column (A_{sf}) is essentially 33% of the column longitudinal reinforcement.



a. Conventional precast girder

b. Precast-spliced girder

c. Typical Section

Figure 3. Superstructure Design Moments

$$A_{sf} = \frac{1}{4} A_{sc} \frac{f_{ult}}{f_y} \tag{2}$$

A_{sc} is the longitudinal column reinforcement, f_y is the girder yield strength and f_{ult} is the ultimate stress of the column rebar. If the ultimate stress of the column reinforcement is 30% higher than the yield stress of the girder, then:

$$A_{sf} = 0.25 \times 1.3 A_{sc} = 0.33 A_{sc}$$

As shown in Figure 3c, the gross area of the bottom flange of the precast girders is a small fraction of the column area. Hence, the resulting connection detail is heavily congested.

One particular solution to this problem is to use precast concrete spliced-girder bridges, where precast girder elements are spliced together with mild reinforcement or prestressing, with an integral column-bentcap-superstructure connection. As shown in Figure 3b, by splicing the girder segments together, the superstructure can be made continuous for dead load. The positive design moment at the face of cap now is the difference between the seismic and dead load moments, which is only a fraction of the positive design moment required for the conventional precast girder system. Hence, the additional superstructure reinforcement required for the integral bentcap detail can now be located in the top flange of the superstructure, where there is substantially more room to place the reinforcement

2. Non-midspan intermediate diaphragms reduce superstructure effective width.

A three dimensional finite-element model composed of frame elements was developed to evaluate the effects of the intermediate diaphragm at the girder splice location shown in Figure 3b. To build a three dimensional model of the Bulb-Tee test setup, the superstructure was idealized as a grillage of longitudinal (girder) and transverse (deck) flexural beam elements. Since the models were symmetric about the centerline of the bridge, half of the structure was modeled, as shown in Figure 4.

Both test units were discretized as shown in Figure 4, where the cracked stiffness was used for the column, and the girder stiffness was 75% of the gross stiffness.

Since most of the torsion is resisted by high stress in the outer fibers of the solid square or rectangular section, the equations used for a thin walled tube were used to approximate the torsional stiffness.[4] The wall thickness of the equivalent tube was calculated as:

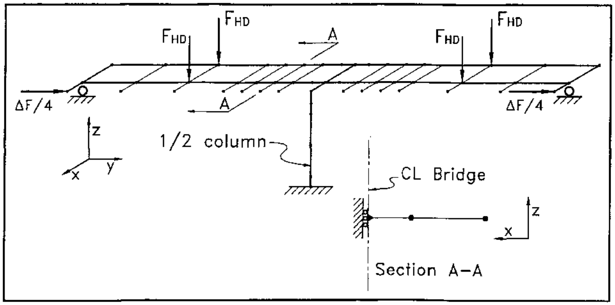


Figure 4. Test-unit grillage model

$$t_c = \frac{3}{4}\frac{A_c}{p_c} \tag{3}$$

Where A_c and p_c are the gross concrete area and perimeter respectively. The torsion stiffness constant was then calculated as:

$$K = \frac{4A_o{}^2 t_c}{p_o} \tag{4}$$

where A_o and p_o are the area and perimeter of the concrete region enclosed by the centerline of the equivalent tube, respectively.[4]

In order to capture the joint flexibility, including the effects of yield penetration, the prismatic column member extended to the intersection of the joint centerlines (no rigid end blocks were used).[3] Assuming the column imparts a uniform torque along the bentcap in the column region, the bentcap torsional stiffness was twice that calculated using Equations (3) and (4), in the joint region.

Under longitudinal seismic attack, girders adjacent to the column deflect in a similar manner to the exaggerated profile shown in Figure 5. Since the girder adjacent to the column experiences a higher demand than the exterior girders, a larger vertical deflection should be observed. However, the intermediate diaphragm prevents the girders from deflecting relative to each other. As a consequence, the interior girder resist the diaphragm shear, which has the same direction as the seismic shear.

On the midspan side of the diaphragm, a slight reduction of moment demand is observed, but at the capface, the moments have increased.

In the analytical model studied, the moment increased by only 12%. However, the interior girder seismic shear, which was also the slope of the moment diagram, increased by 50%.

The overall effect of the intermediate diaphragms in the non-midspan locations is to stiffen the interior girders in the critical region relative to the exterior girders. Hence, moment and shear demands increase for the interior girder in the critical region.

3. Bentcap prestressing improves longitudinal seismic performance.

Lacking the presence of the soffit slab across the width of the superstructure, as is the case for cast-in-place box girder bridges, longitudinal column plastic hinge moments are resisted entirely through torsional mechanisms. The grillage model in Figure 4 shows that the moments distributed between the girders adjacent to the column and the exterior girders are shared based on relative stiffness. Once cracking torque is reached, the stiffness drops to a

Table 1. Ratio of interior vs. exterior girder moments

model	Theory	Experiment
Bulb-Tee	1.38	2.00
Bathtub	1.32	1.08

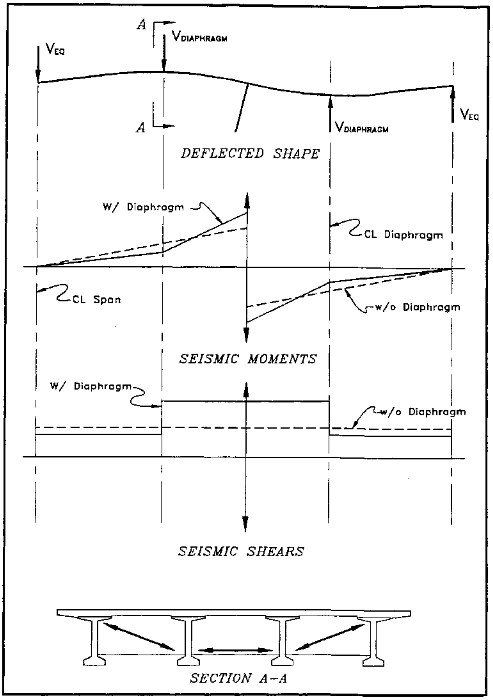


Figure 5. Effects of non-midspan intermediate diaphragms on the girders adjacent to the column

small fraction of the precracked state.

Based on this observation, it has been suggested that the superstructure capacity can be calculated as the moment capacity of the interior girders plus the cracking torque of the bentcap.[5] Using the equivalent tube analogy presented previously, the cracking torque can be calculated as:

$$T_{cr} = \frac{A^2}{p} k \sqrt{f'_c} \sqrt{1 + \frac{f_p}{k\sqrt{f'_c}}} \tag{5}$$

where A and p are the gross cross section area and perimeter of the bentcap respectively, k = 4 (psi) [0.33 (MPa)], f_p is the prestressing stress and f'_c is the compressive strength in psi [MPa].[4]

As compared to an equivalent conventionally reinforced bentcap, prestressing improved the cracking torque by 55% for both models. For the Bulb-Tee model, the computed cracking torque was 27% of the overstrength moment resisted by the superstructure. Hence, if the interior and exterior girders resisted the full overstrength moment equally, the capbeam would have remained uncracked.

4. 2-D joint shear model needs modification for 3-D effects.

The bentcap column joint was designed using the two dimensional force transfer mechanism shown in Figure 6a, which was based on the method developed at UCSD. In this mechanism, it was assumed that 75% of the longitudinal reinforcement was either clamped by the main diagonal strut D_1 or in compression. The remaining longitudinal reinforcement was clamped by compression struts D_2 and D_3, as shown in the strut and tie model of Figure 6b. It was assumed that the vertical components of D_2 and D_3 shared the vertical force of T_c equally. The vertical component of D_2 was resisted by the joint stirrup labeled T_{sv}. The horizontal component of D_2 was resisted by the horizontal leg of the joint stirrups shown as T_{sh}.

The horizontal components, of D_2 and D_3 in Figure 6b, are clearly unequal. This unbalanced horizontal force was resisted by the column core hoops and is designated as F_h.

The portion of the column force clamped by the external strut T_c was computed from the parameters of Table 2. The bars making up T_c are indicated by the shaded bars in Figure 6c. The neutral axis depth y_{na} and the curvature at the top of the column ϕ_{top} were obtained from moment curvature analysis of the column cross section, using the program "xSECTION".[6] The distance from the neutral axis to T_c is labeled as y and was used in conjunction with the curvature to compute the strain and, with the following equation, the average stress.[7]

$$f_s = f_{su} - (f_{su} - f_y)\left[\frac{\varepsilon_{su} - \varepsilon_s}{\varepsilon_{su} - \varepsilon_{sh}}\right]^p \tag{6}$$

Where f_{su} and ε_{su} are the ultimate stress and strain, respectively. f_y is the yield stress and ε_{sh} is the hardening strain.

With the parameters of Table 2 the tie forces for the Bulb-Tee model were calculated as:

$$T_c = f_s A_{s/bar} \# = (101)(0.44)(10) = 443 \text{ kip [1970 kN]}$$

$$T_{sv} = \frac{443}{2} = 221 \text{ kip [983 kN]}$$

$$T_{sh} = \frac{443}{2}\left(\frac{6.25}{23.3-1.1}\right) = 62.6 \text{ kip [278 kN]}$$

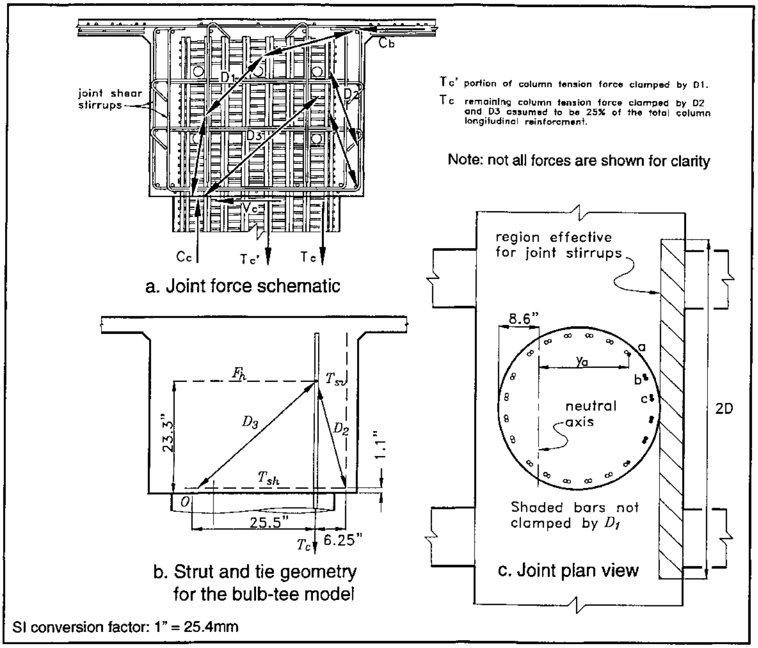


Figure 6. Assumed joint force-transfer mechanism

$$F_h = \frac{443}{2}\left[\frac{25.5}{23.3} - \frac{6.25}{(23.3-1.10)}\right] = 180 \text{ kip [801 kN]}$$

Forces from both models are summarized in Table 3, along with the experimentally determined values. Since the forces could not be measured directly, rebar strain data was used to determine the forces. Strain values were converted to stress using: an assumed modulus of 29,000 ksi [200 GPa] if the strain was less than yield, the yield stress in the plateau region and Equation (6) for strains in the strain hardening region. Once the stress was determined, an average stress of the reinforcement was then taken in the region determined effective for joint shear resistance (for example, vertical stirrup legs located in the shaded region of Figure 6c was effective for T_{sv}).[3] The force was then computed as the product of the average stress and the total area of reinforcement.

As shown in Table 3, the theoretical force exceeded the experimental values for the vertical joint reinforcement by a significant margin. This was likely due to the additional clamping of the column longitudinal reinforcement provided by the bentcap prestressing not considered in the joint design model.

The soffit reinforcement stresses were under predicted significantly. The reason for this was that the column shear has to be transferred from the column compression zone to the girder bottom flanges, which was not considered in the 2-D model. Figure 7 shows the bentcap soffit with a strut and tie schematic. As shown, the force from horizontal legs of the joint stirrups T_{si} transfer the column shear force from the column compression zone at the left side of the bentcap to the right side, where the force was then transferred to the inclined diagonal struts.

It was not necessary to have all of the strut dimensions to determine the demand on the soffit joint stirrups. The stirrup force in between the two interior girders along with the column hoop force of Figure 7 was the column shear. Hence:

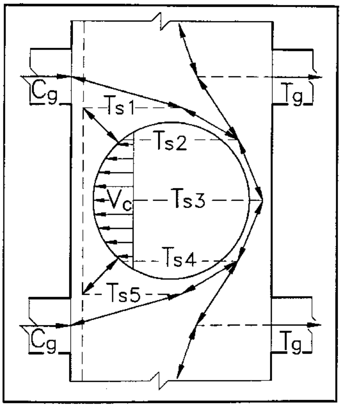


Figure 7 Bentcap soffit force transfer mechanism

$$\Sigma T_{si} + F_{hs} = V_c \qquad (7)$$

Where T_{si} was the soffit stirrup force, F_{hs} was the contribution of the column hoops at the bottom of the joint assumed to be the lowest 40%, and V_c was the column shear force.

The revised computed force of the soffit steel is shown in Table 4 for both models along with the column shear forces. The revised forces are certainly closer to the experimental values than what was determined by the 2-D model. The forces for the Bathtub model are lower than the experimental due to the assumption that the hoops at the bottom of the joint had reached the yield stress of 66 ksi [455 MPa]. The computed stresses include the forces required to complete the mechanism shown in Figure 6b.

5. Allow 30% redistribution of seismic superstructure moments.

In an effort to reduce reinforcement congestion, specifically the amount of reinforcement required for positive connection between the girder and the bentcap, moment redistribution is a means of further reducing the reinforcement. Moment redistribution implies at least limited ductile behavior in the beam elements.

Superstructure testing of both models has shown that limited ductile response can be obtained for both positive and negative bending without specific seismic detailing in the critical regions.[1,2]

CONCLUSIONS

Testing has shown that precast girder superstructures can perform equivalent to cast-in-place under seismic loading. However, there are several items which should be added to the current design procedures used for the design of cast-in-place superstructures, which have been discussed in this paper.

Table 2 Input parameters for column tension force T_c

Item	units	Bulb-Tee model		Bathtub model	
	US [SI]	US	SI	US	SI
y_{na}	in [mm]	8.60	210	8.30	211
y	in [mm]	21.2	538	22.1	561
ϕ_{top}	r/in [r/m]	2.10	53.3	2.35	59.7
f_y	ksi [MPa]	68.4	472	64.0	441
f_{su}	ksi [MPa]	109	752	109	752
ε_{sh}	%	0.80		0.60	
ε_{su}	%	10.0		10.0	
p		3.1		3.1	
ε_s	%	4.60		5.20	
f_s	ksi [MPa]	101	696	104	717

Three dimensional effects must be evaluated in the design of precast girder bridges with an integral superstructure column connection. This is taken into account typically with the effective width requirements suggested in references 1,2 and 3. Three dimensional elastic grillage models should be used in the design if non-midspan intermediate diaphragms are required, to determine the effects on the moment and shear profiles of the girder. It was also demonstrated that joint behavior described by the 2-D model must be modified to take into account the shear transfer between the bottom of the girders and the top of the column.

This paper has demonstrated improved seismic performance can be obtained with bentcap prestressing. Joint shear

Table 3 Model joint tie forces

Item	Bulb-Tee			Bathtub		
	theory	exp.	theo/exp	theory	exp.	theo/exp
T_c	443 [1970]			455 [2020]		
T_{sv}	221 [983]	125 [556]	1.77	228 [1010]	88 [391]	2.58
T_{sh}	62.6 [278]	250 [1110]	0.25	62.4 [278]	133 [592]	0.47
F_h	180 [801]	175 [778]	1.03	192 [854]	97 [431]	1.98

Forces in kips [kN]

Table 4 Revised soffit stirrup forces

Item	Bulb-Tee model		Bathtub model	
	kip	[kN]	kip	[kN]
V_c	351	1560	203	903
F_{hs}	153	681	153	681
ΣT_{si}	198	881	50	222
T_{sh}	62.6	278	62.4	278
$T_{sh\ (revised)}$	261	1160	112	498
$T_{sh\ (exp.)}$	250	1110	133	592

performance is enhanced with prestressing. This improvement is a result of reduced cracking, better bond and enhanced confinement of the column longitudinal rebar extension into the bentcap. Also, the superstructure width effective in resisting column plastic hinging is improved since the contribution of nonadjacent girders is dependent on the bentcap cracking torque.

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