PRECAST BEAMS WITH CAST-IN-PLACE CONNECTIONS

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Abstract

Widening of existing bridge structures or new bridge construction in heavily congested areas has become a necessity due to the increasing traffic demands on Nevada's highway systems. The purpose of this study is to develop and examine integral connection details of precast superstructures with cast-in-place bent caps subjected to longitudinal seismic loading. Analytical modeling and experimental testing of four, 40 percent precast "U" girder specimens will be used to develop a design methodology. The main parameters of this study are the magnitude of post-tensioning and the type of conventional reinforcement connection.

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Introduction

Bridge structures are an integral part of this nation's highway infrastructure. As the infrastructure continues to age, existing bridges may need to be widened, retrofitted due to increasing traffic demands, or new bridges may need to be added. Often, widening or replacing of existing bridges or new bridge construction occurs in heavily congested areas where traffic delays and public safety are of major concerns. This is especially true in seismic regions where bridges are typically continuous, cast-in-place concrete superstructures that are integral with cast-in-place concrete substructures in order to transfer high seismic moment and shear forces. Monolithic bridge construction provides good continuity for transfer of seismic forces; however, falsework over the traffic lanes is needed while the superstructure is cast. This falsework can potentially create significant traffic delays due to reduced number of lanes provided for the public, or unsafe driving conditions and unsafe working conditions for construction workers due to clearance issues.

Using precast concrete girders for the superstructure eliminates the need for falsework over traffic lanes and also allows for accelerating the construction time needed to place the superstructure, thereby reducing the traffic delay to the public and reducing the danger to the construction workers. This construction process has great advantages and applications when widening and retrofitting existing bridges as well as new bridge construction in highly congested areas. However, the uncertainty in behavior of the precast girder connections to cast-in-place bent caps for transferring seismic forces has led designers and agencies not to use this construction and design method. The purpose of this study is to investigate the seismic behavior of the integral connection between precast concrete girders and cast-in-place concrete and develop design guidelines based on analytical and experimental testing for the Nevada Department of Transportation.

Background

Limited research has been conducted on seismic behavior of integral precast to cast-in-place concrete connections and codes provide very little guidance for design. In order to gain further insight for current design and construction practices, a survey was distributed to Department of Transportation agencies in seismic regions to examine what types of methods are being used. The following sections describe the basic research and the results of the survey.

Previous Research

The only prior experimental research pertaining to the integral precast superstructure connection to a cast-in-place bent cap was conducted at the University of California at San Diego La Jolla, California in the late 1990's (Holombo 2000). This study investigated the continuity of a post-tensioned spliced precast girder system subjected to longitudinal seismic forces. Two 40% scaled bridge models featuring bulb-tee girders and bathtub girders that represented typical bridge construction in California were tested. In both tests, the superstructure was designed to perform elastically while the inelastic behavior was to occur in the column. Negative moment continuity was provided by post-tensioning of the girders over the bent cap and positive moment continuity was provided through splicing the extended bars and strands at the bottom of the girder. The results of the test indicated good ductility performance of the integral connection with only minor strength degradation. The superstructure was able to remain essentially elastic with only minor cracks occurring that closed after the removal of seismic loading. Another important conclusion the researchers reported was the proportion of the column seismic moment to be resisted over the width of the superstructure. They concluded that the moment should be proportioned according to the relative stiffness of the integral system, or roughly two-thirds of column moment to be resisted by the two adjacent girders and the other one-third to be resisted by the remaining girders. Another important detail they recommended was to extend the column longitudinal reinforcement as far as possible into the bent cap for better transfer of the seismic forces.

While the tests conducted at the University of California San Diego are the only tests investigating seismic performance of the connections, tests on simple-span girders made continuous for live load were conducted and reported in NCHRP Report 519 (Miller et al. 2004). The researchers tested several different types of positive moment connections consisting of untensioned bent prestressing strands and bent mild steel bars while the negative moment connection was made with a composite concrete deck. Based on their results, the researchers made recommendations that would be useful for future seismic performance investigations. Probably the most significant recommendation is that bent-strand connections tended to slip under cyclic live loads more so than the bent bar connections, thereby making the strands not very suitable for seismic applications. The researchers also reported an increased connection ductility performance when the girders are embedded into the bent cap and confining stirrups are provided in the bent cap just outside the end of the girder. However, providing these stirrups would significantly increase joint reinforcement congestion and may not be very feasible from a construction aspect. Their final recommendation consisted of conducting additional testing on continuity diaphragms connected to the pier cap where negative reinforcement is provided in the composite deck and positive moment is provided using bent bar connections where limited seismic forces are transferred between the superstructure and substructure.

Survey

A survey inquiring about the use of integral precast/cast-in-place connections and current design practices was sent to DOT’s in high seismic regions. The following sections report on the survey’s significant findings from the agencies that either indicated they used integral connections or provided interesting feedback. Also provided is a summary of bridge site visit in Reno, NV where the integral connection was being utilized.

The use of the integral connection is a relatively new method for Nevada Department of Transportation (NDOT), which is the reason for this study. The typical bridge configuration in Nevada consists of a continuous cast-in-place superstructure with an integral substructure. The few bridges with precast superstructures had similar details to those used in the University of California at San Diego research program, where full length post-tensioning was used in conjunction with mild-steel positive moment connections. NDOT primarily uses precast U-shaped girders, therefore this study primarily focuses on U-shaped girders, however, the concept can be applied to other precast girder shapes.

The only other state that reported utilizing the integral connections was the Washington State Department of Transportation (WSDOT). They indicated that 75% of their bridges in high seismic regions utilize the integral moment resisting connections. In lower seismic areas, a hinged joint connection, similar in concept to those reported in NCHRP Report 519 (Miller et al. 2004), is used. The most common precast girder shapes used by WSDOT are I-girders and trapezoidal tub girders where negative moment continuity is provided by a composite deck and positive moment capacity is provided by untensioned prestressing strands with headed anchor studs welded on the end of the strand and embedded into the bent cap. WSDOT indicated that post-tensioning is not used for their connections.

The Federal Highway Administration (FHWA) indicated that they do not use integral connections, but continuity is provided for the superstructure and a pin connection is used between the superstructure and substructure to transfer shear and axial forces, but no seismic moment transfer. The Utah Department of Transportation (UDOT) indicated they use the same connection in 25% of their bridges. Both UDOT and FHWA indicated that the girders were made continuous using either prestressing strands and/or dowels for the positive moment connection and a composite deck was used for the negative moment connection. Neither agency indicated the use of post-tensioning for their bridge systems in seismic areas. FHWA points out that if a moment connection is desired, AASHTO (1998) requires an overstrength factor of 1.3 times the column plastic moment for the connection design moment.

California Department of Transportation (Caltrans) indicated in the survey that they do not currently utilize integral connections with precast beams because the precast solution is not as cost-effective as their more typical cast-in-place box girder bridges. It is also interesting to note that Caltrans requires a connection design moment of 1.2 times the plastic moment capacity of the column (Caltrans 2004) as compared to the 1.3 factor used by AASHTO (1998). For comparison, ACI 318 (2002) requires a factor of 1.25 times the yield strength of the longitudinal reinforcement for moment resisting joints.

Some construction issues were addressed when a site visit was made to the McCarran Bridge over US395 in Reno, NV. This bridge implemented the precast girder superstructure to cast-in-place bent cap connection. The girders utilized post-tensioning for negative moment continuity and either lap-spliced or mechanically spliced longitudinal reinforcement for positive moment continuity. The construction issue the crew mentioned was the poor construction quality of the girders. They stated that the post-tensioning ducts did not line up where the girders needed to be spliced together. Girders needed to be paired in a certain configuration and some girders were rejected. Also, utilizing mechanical splices created problems when aligning the reinforcement between the two girders to be spliced making lap splices easier connections for construction purposes. Other than these problems, which are heavily dependent on the precast fabrication, there were no other significant construction problems to report. Figure 1 shows the NDOT U-Girder at an intermediate span splice location. Figure 2 shows the U-Girder splice between the bent cap.

Background Summary

From the previous research and survey study, three types of connections are currently used in practice. The most popular detail consists of post-tensioning for negative moment continuity and spliced mild reinforcement for positive moment continuity. Another detail used by WSDOT utilizes the composite deck for the negative moment continuity and untensioned prestress strands with headed anchor studs welded to the ends and embedded in the bent-cap for the positive moment connection. The last detail, used by FHWA and UDOT, utilizes a continuous superstructure and a pinned connection between the superstructure and substructure. This detail is not applicable to our study. The results of the survey and previous research were considered when choosing the parameters for the study.

Integral Connection Parameter Study

To establish the most important parameters for the integral connection, a prototype U-Girder representative of NDOT U-Girders was developed and is shown in Figure 3. This girder section was analyzed using the moment-curvature program XTRACT (2002) to determine the moment capacity and ductility characteristics. Based on this information, a scaled version of the prototype girder was chosen and the most favorable connection details determined from the survey and literature search were applied to develop proper negative and positive bending moment capacity. Ultimately, a 40% scale of the prototype was developed (see experimental program for reasoning) using post-tensioning for negative moment continuity and spliced mild steel for positive moment continuity. The girder is shown in Figure 4.

Post-tensioning over the bent cap is advantageous because this allows the section to have a high negative moment capacity without having to increase the amount of reinforcement in the composite deck. However, when the post-tensioning is part of the system, it introduces positive secondary moments that reduce the negative dead load moment effect. These secondary moments can be minimized and controlled through the tendon configuration. The post-tensioning also contributes significantly to positive moment capacity, which again will reduce the amount reinforcement needed in the bottom flange of the section at the face of the bent-cap. Therefore, adjusting the amount of post-tensioning significantly affects the negative and positive moment capacities as well as the curvature ductility of the section making post-tensioning a very advantageous and important parameter in this study.

Mild reinforcement and untensioned strands are both commonly used for the positive moment connection between the girder and the bent-cap. The decision to use mild reinforcement in the bottom flange of the girder, as shown in Figure 4, over strands is due to two primary reasons listed below:

1. NCHRP Report 519 (Miller et al. 2004) reported tendency of strands to slip under cyclic loads more than the mild steel;
2. The use of head anchors welded to strand ends, as suggested by WSDOT details, would create more congestion in the connection than the use of mild reinforcement.

The objective of testing the girder shown in Figure 4 is to assess the ability to distribute the column moment using the positive and negative continuity connections until failure. This will show the dominate failure mode of the connection and allow for the development of more specific design guidelines.

Experimental Program

The objective of the experimental program is to verify the integral connection's ability to adequately transfer the seismic forces. The 40% scale U-Girder was selected primarily for two reasons, first, scaling smaller than 40% produced webs that were extremely small and problems such as cracking could occur when transporting and placing the girders with the equipment available. Secondly, a smaller scale would require mild reinforcement below #3 rebar. Therefore, if a smaller scale than 40% is to be used, then bars would have to manually deformed and treated until they were representative of actual field conditions.

The specimens will be tested at the University of Nevada, Reno Large Scale Structures Laboratory. The feasibility of two different testing methods was investigated. The first method consisted of testing the system using the UNR shake tables. An inelastic dynamic analysis was performed using SAP2000 (2005) subjected to different ground motions. The results indicated that theoretical failure of the system, using a 2-ft diameter, 9.5-ft high column, would occur at the upper limit of the shake table. In order to have enough factor of safety between system failure and shake table capacity, a 13.5-ft column would be needed, which is not representative of typical columns. Therefore, the set-up shown in Figure 5 was adopted, where actuators supply the seismic force at either end. The span lengths on either side of the bent cap of the scale model are half of the prototype span length in order to get the desired seismic shear across the joint. Since failure of the superstructure connection detail is desired, the column was designed to yield but not experience significant levels of inelastic behavior. The bent cap was designed using Caltrans (2004) specifications since AASHTO does not provide a clear design procedure for joint design where seismic forces are transferred between the substructure and superstructure. The details contained in the Caltrans specifications are similar to those given in the book *Seismic Design and Retrofit of Bridges* (Priestly et al. 1996) and Prestressed Concrete Institute (PCI 2003). This method was used in the experimental study done at the University of California, San Diego (Holombo 2000) and the bent cap performed adequately. Lead was added to the superstructure, see Figure 5, in order to model the scaled dead of the prototype correctly. The column-footing connection was designed as a two-way hinge using a methodology developed from research conducted at UNR (Cheng et al. 2006). The study found that shear friction theory either overestimated or underestimated the hinge shear strength. A new method was developed based on observed shear failure mechanisms. Using a base hinge reduced the column seismic shear force required to develop the superstructure moments as compared to a fixed case. Finally, as shown in Figure 5, the experimental test set-up consisted of one girder on either side of the bent-cap. Since recommendations for contributory superstructure stiffness to resist the column moment were made from the researchers at University of California at San Diego (Holombo 2000), it was felt one girder on either side of the bent cap would be sufficient. The following experimental program is planned for the project.

1. Test connection detail as shown if Figure 4 using 4 strands per girder web.
2. Test connection detail as shown in Figure 4 using 3 strands per girder web.
3. Analyze results of experimental tests 1 and 2.
4. Based on analysis, perform two additional tests from the following:
    a. Test detail similar to WSDOT, no post-tensioning;
    b. Use untensioned prestress strands for positive moment continuity;
    c. Use the results from previous tests to design a connection as it would be built in the field (i.e. elastic behavior in superstructure, inelastic behavior in column)
    d. Use details similar to tests1 and 2 except use mechanical splices for the positive moment connection instead of lap splices; or
    e. Use details similar to tests1 and 2 but with a longer cast-in-place section.

Therefore, a total of four connection details consisting of precast U-Girders and cast-in-place substructure will be tested. From these experimental tests, along with analytical work, a design methodology for integral connections will be developed.

Strut-and-Tie Models

Another important aspect of the project includes using strut-and-tie models to describe the force transfer between the substructure and the superstructure. The most widely used model consists of using joint stirrups outside the column core region to transfer the forces (external joint transfer model) (Priestley et al. 1996) shown in Figure 6 (a). Note the “T” designates tension members where C designates compression members. More recently, research on the transverse loading of bridge tee joints by Sritharan (2005) was conducted. Sritharan’s research suggested that the joint design based on Priestley et al. models was conservative when the joint was prestressed and unconservative when the joint did not include prestressing. Therefore he suggested a modified external strut and tie force transfer model. Figure 6 (c) shows a model that requires joint stirrups on both sides of the joint for one direction of loading when prestressing is not included. Figure 6 (b) and 6(d) show models for when the joint is prestressed and partially prestressed respectively. Notice the prestressed joint doesn’t require joint stirrups for the force transfer while the partially prestressed joint only requires a small amount. Hence only nominal joint reinforcement would be required. These models were verified through experimental testing. Based on these models, similar models will be developed based on the experimental results of the program detailed above.

Summary

Based on the work presented in this paper, the following summary can be made considering seismic performance of precast concrete girder connections to cast-in-place concrete bent cap.

1. Limited research and code guidance is available pertaining to the design of integral connections for precast superstructures. This lack of documented information has led to reluctance to use these connections as shown in the DOT survey results.
2. Using post-tensioning to provide negative moment continuity instead of placing more reinforcement in the deck is an advantageous connection because it relieves the reinforcement congestion in the deck.
3. Mild reinforcement for positive moment continuity was chosen due to common detailing practices and the conclusion that untensioned strands tended to slip under cyclic loads (Miller et al. 2004).
4. Experimental tests capturing the failure modes of the connection will be used to develop design guidelines pertaining to proper detailing of integral connections.
5. Strut-and-tie models will be used as tool to describe the force transfer between the substructure and the superstructure.

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Fig. 1 NDOT Precast U-Girder for McCarran/US395 Bridge

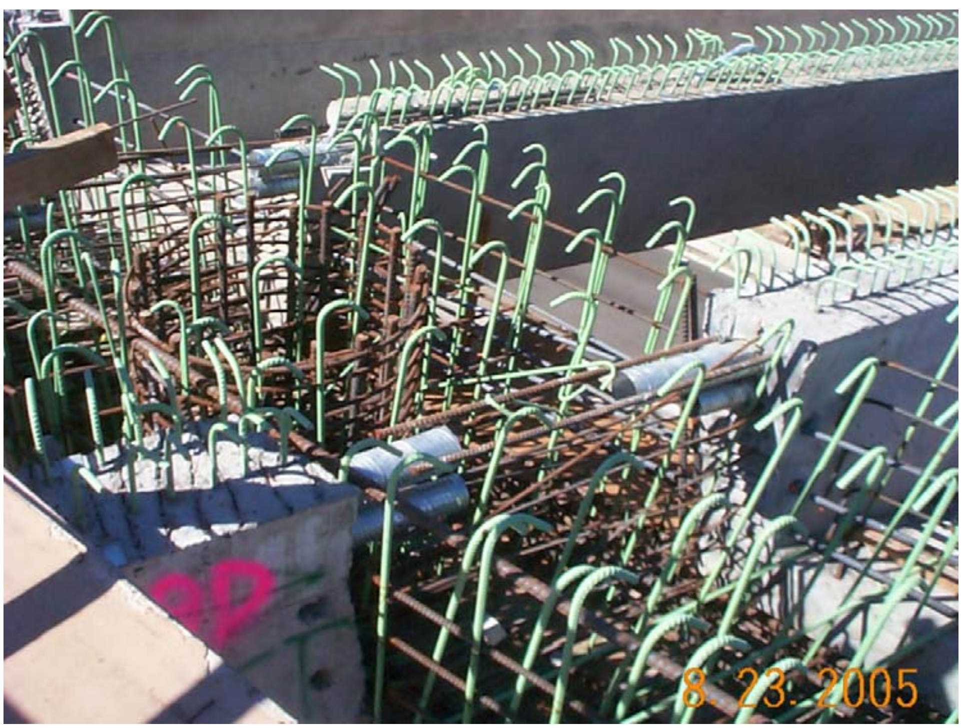

Fig. 2 NDOT Precast U-Girder - Bent Cap Region for McCarran/US395 Bridge

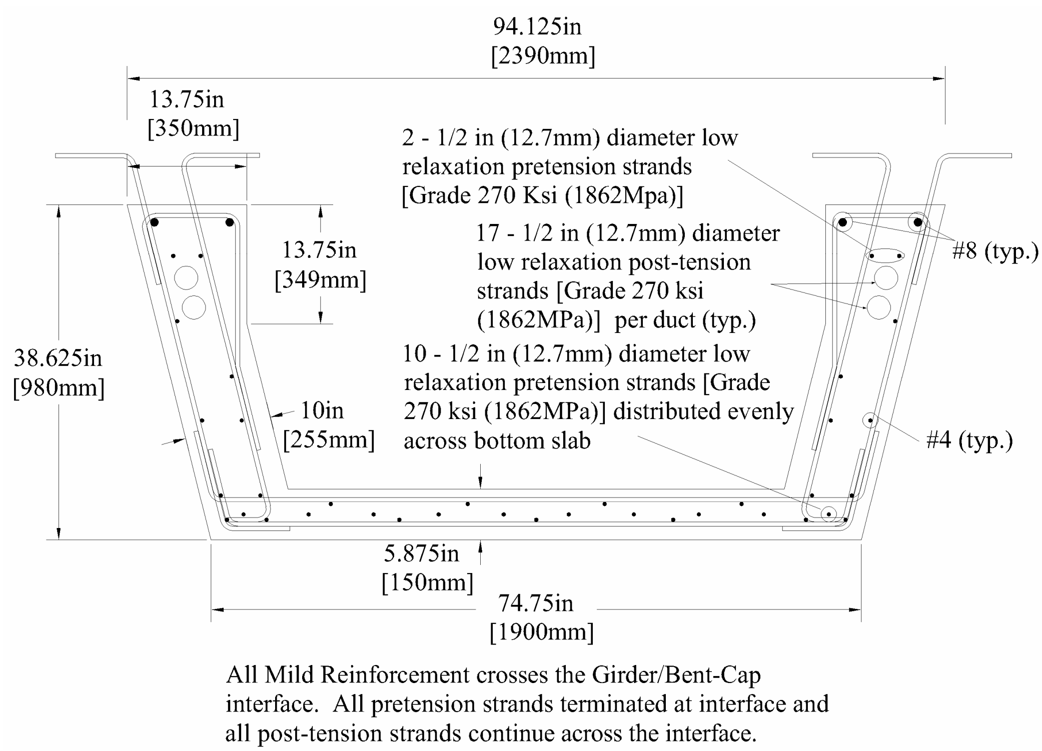

Fig. 3 NDOT Prototype U-Girder Cross-Section at Girder/Bent-cap Interface

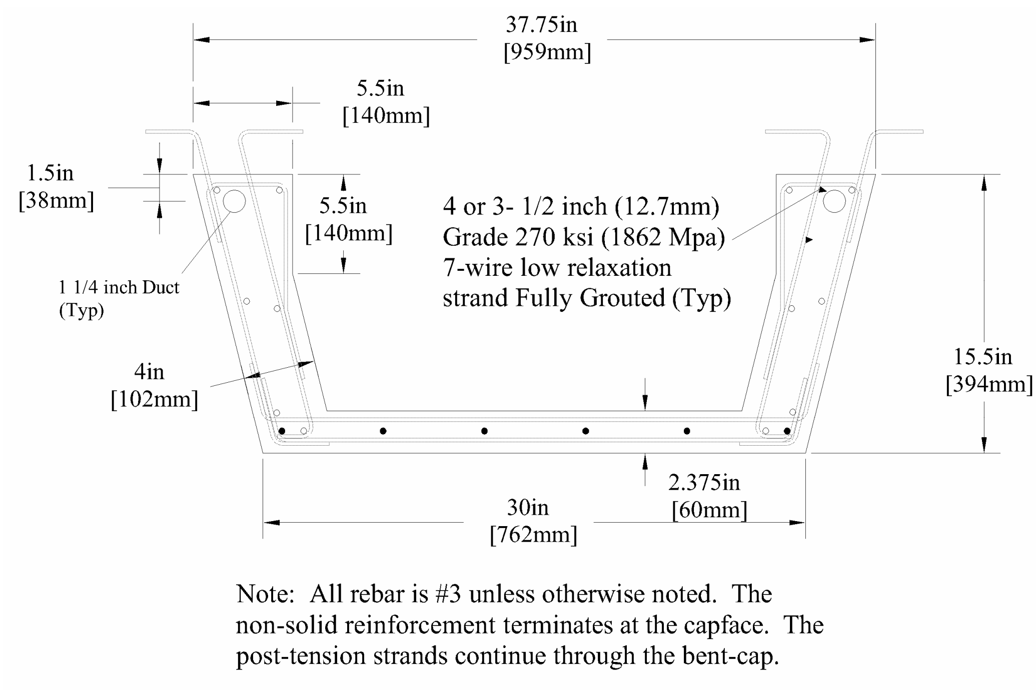

Fig. 4 NDOT 40% Scale U-Girder Cross-Section at Girder/Bent-Cap Interface

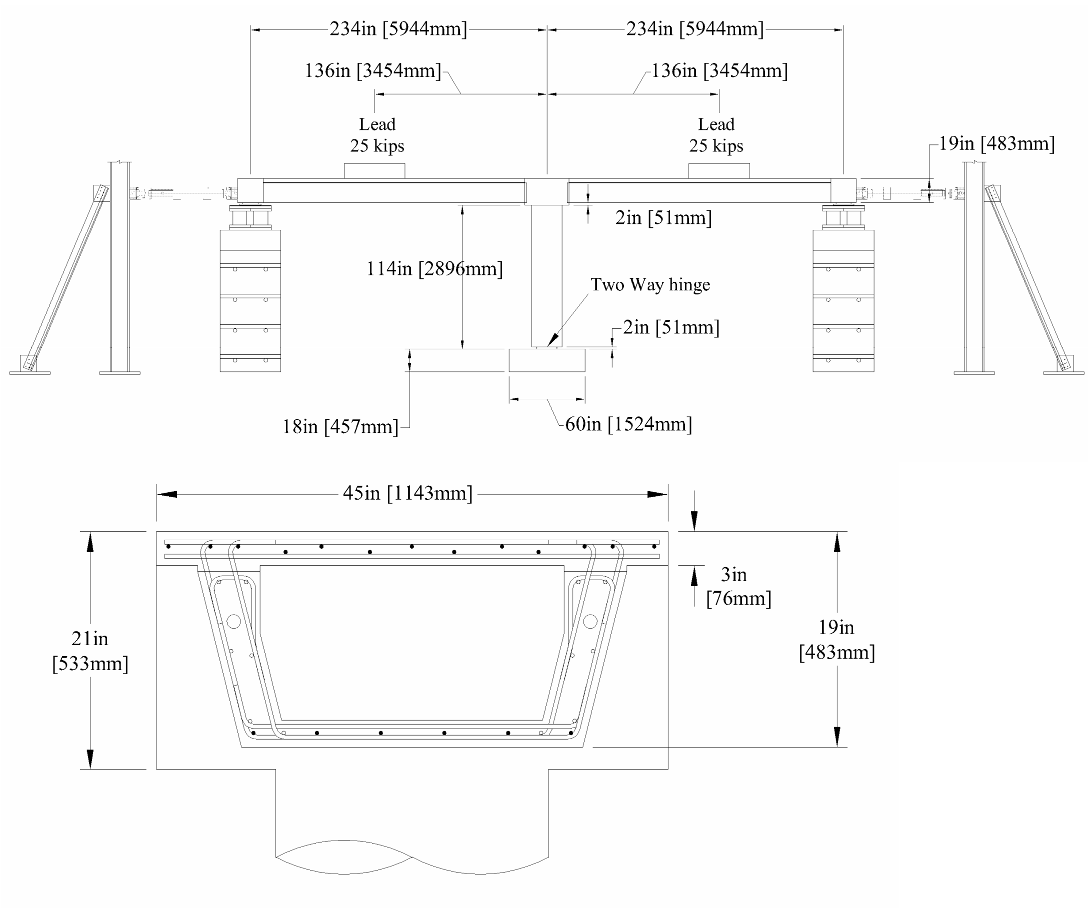

Fig. 5 Test Configuration and Cross-Section on Either Side of Bent-Cap

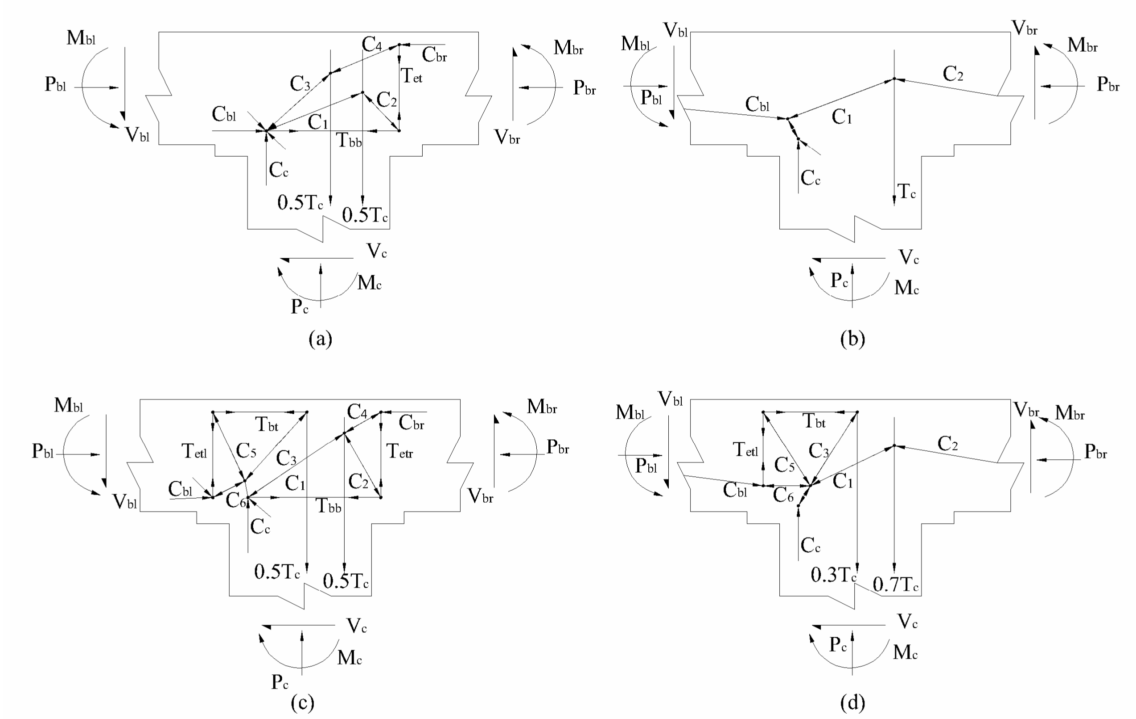

Fig. 6 Previously Developed Strut and Tie Models, (a) Priestley et al. (1996), (b)-(d) Sritharan (2005)