A cost-effective integral bridge system with precast concrete I-girders for seismic application

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- To promote accelerated bridge construction in seismic regions, a large-scale experimental investigation was conducted to examine the seismic sufficiency of precast concrete I-girders in integral bridge superstructures.
- A half-scale, 17.8 m (58.5 ft) long test unit modeling a portion of a prototype bridge was used to experimentally verify that precast concrete members employing accelerated construction techniques can be used in integral superstructures and provide excellent seismic performance.
- Comparison of the as-built girder-to-cap connection detail with an improved detail shows that the as-built detail will satisfactorily resist positive and negative seismic moments and allow plastic hinges to develop at the column tops in existing bridges; however, the improved detail is recommended for new bridges to avoid potential deterioration of the girder-to-cap connection.

Accelerated bridge construction is increasingly being pursued and promoted across the United States. Many state transportation departments are dealing with aging infrastructure along with increased demand due to continuing economic and population growth.[1] The rapid construction of bridge projects to meet these needs is beneficial.[2] Similarly to the rest of the country, the California Department of Transportation (Caltrans) is interested in the benefits of accelerated bridge construction techniques, provided that seismic concerns can be addressed. The desire to improve and increase the possibilities of accelerated bridge construction methods is highlighted in its lessons learned report[3] and the related strategic plan.[4]

The obvious primary benefit to the incorporation of accelerated bridge construction methods is the reduction of on-site construction time, along with the associated mitigation of traffic delays. A common way to decrease time in the field is to employ prefabricated components as much as possible. The use of precast concrete members instead of cast-in-place concrete sections also results in the elimination of the need for falsework and an overall improvement in quality control by relocating production from unpredictable field conditions into a controlled shop environment.

Though accelerated bridge construction methods have notable advantages, the incorporation of such techniques in moderate-to-high seismic regions has been slowed because of the poor performance of precast concrete structures in previous earthquake events. The vulnerability of precast concrete structures has been due to the inadequate performance of connections and failure to ensure satisfactory load paths. Precast concrete structures were observed to experience connection failures (especially in buildings) in past seismic events, including the Loma Prieta earthquake in 1989[5] and the Northridge earthquake in 1994.[6] Bridge data show that cast-in-place concrete accounts for over 70% of current California bridge superstructures, while precast concrete accounts for about 5%.[7]

Increased opportunities to incorporate accelerated bridge construction techniques and the associated benefits will be realized if precast concrete connections can be developed that are viable for quick implementation in the field, do not significantly increase cost, and are able to sustain high demands resulting from seismic loading. Capacity design is the most common approach in designing for earthquake loads. Using this approach, bridges are designed to exhibit ductile behavior at the column ends, which are specifically detailed to accommodate sufficient inelastic action while maintaining strength. These specially detailed regions are referred to as plastic hinges. When a large seismic event occurs, the plastic hinge regions undergo inelastic deformation, thereby dissipating seismic energy, while the remainder of the structure continues to experience elastic behavior even when subjected to high seismic demand. By incorporating this design philosophy, structures can be economically designed to accommodate large lateral seismic displacements.

The bridge superstructure, including the deck, is protected from any inelastic action while allowing plastic hinge formation in the columns. The girder-to-cap connections, in particular, require careful attention for integral superstructure concepts, because the girders must have sufficient moment capacity across the cap beam. Integral designs are advantageous in seismic regions because the moment continuity in the superstructure above the column bents provides a possible plastic hinge location in the column just below the cap beam. The development of girder-to-cap connections that facilitate rapid construction techniques in the field and provide sufficient shear and moment continuity for integral connections in high-seismic regions will provide greater opportunity to employ precast concrete members and their associated benefits without increasing the cost.

Research significance

The work detailed here was undertaken to accomplish two primary objectives. First, an existing inverted-tee cap-beam concept had been previously used to facilitate precast concrete dapped-end girders, but it was not considered to have sufficient moment capacity to be an integral connection

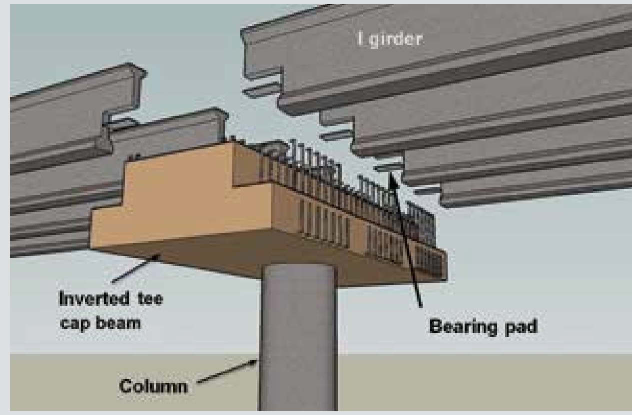

Figure 1. Inverted-tee bent cap concept.

under high seismic loading. Second, a portion of this work focused on developing an improved girder-to-cap connection detail. This detail was shown to be able to reliably establish an integral superstructure connection between precast concrete cap beam and girder components with minimal on-site construction.

The inverted-tee concept is well suited for accelerated methods because of its incorporation of precast concrete girders and easily constructible girder-to-cap connections without needing temporary falsework. Also, the concept provides the potential to use precast concrete cap beams and make the completed structure aesthetically comparable to cast-in-place concrete box girder bridges.

A precast concrete bridge system for seismic regions

A frequently used precast concrete section is the California I-girder.[8] A detail that has been employed to facilitate the use of such I-girders and accelerated bridge construction is the inverted-tee bent cap concept (**Fig. 1**). It has typically been implemented with cast-in-place concrete columns and cast-in-place concrete inverted-tee cap beams. Once the cap beam is built, the ledge on each side of the cap-beam stem works well to support the dapped end of precast concrete girders. The girder dapped ends can subsequently be integrated with the cap beam by the use of a cast-in-place concrete diaphragm and by appropriate connection reinforcement. Finally, the cast-in-place concrete bridge deck can be placed over the completed superstructure.

Seismic performance and limitations

Whereas the inverted-tee bent cap concept has been employed in California, the superstructure has been designed according to current design recommendations.[8,9] Accordingly, the degradation of the positive-moment connection due to large seismic displacements and the loss of tension continuity in the girder lower flange connection are expected. **Figure 2** illustrates the moment reversal that

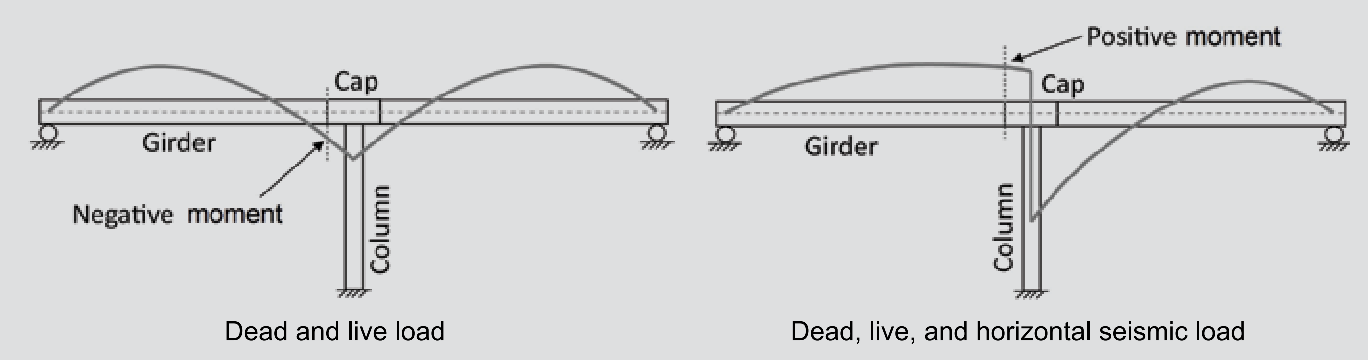

Figure 2. Moment diagrams due to different loading conditions.

occurs due to large horizontal seismic loads. Under dead and live loads, the girder-to-cap connections are subjected to negative moment only (Fig. 2). Under this condition, the deck reinforcement, which runs continuously over the top of the connection, provides tension continuity, and therefore robust negative-moment capacity. However, when the horizontal seismic load is added, the moment diagram shifts (Fig. 2). Large seismic loads will produce a reversal of moment in the connection region, resulting in tension in the bottom of the connection region where there is no reinforcement continuity. Therefore, the recommendations stipulate that the cap-to-girder connection should be considered to have zero moment resistance under combined gravity and seismic loading.

Furthermore, the Caltrans *Seismic Design Criteria*[9] require that a static vertical load, both upward and downward, equal to 25% of the dead load needs to be incorporated for ordinary standard bridges where the site peak ground acceleration is 0.6g or greater (where g is acceleration due to gravity). When this acceleration must be considered, longitudinal-side mild reinforcement in the girders should be provided that is capable with shear friction to resist 125% of the dead-load shear at the cap-beam interface. This requirement, which exists to protect against potential shear failures when the bottom of the connection opens up, has been disadvantageous for two reasons:

- There may not be sufficient room to place such reinforcement.
- This detail would increase the girder and assembly costs while introducing constructability challenges.

Eliminating this requirement, which was not based on a comprehensive investigation, would make the inverted-tee detail more attractive in seismic regions.

Development of girder-to-cap connections that provide full moment resistance will offer the possibility of incorporating design approaches that take advantage of fully integral superstructure behavior while using accelerated bridge construction. The result is a precast concrete superstructure design that is a competitive alternative to cast-in-place concrete and provides the opportunity to incorporate accelerated bridge construction in high-seismic regions. In addition, total cost benefits for precast concrete structures, such as increasing quality, reducing effects on traffic, and improving worker safety, are not integrated into the construction cost of the bridge but are important advantages of such approaches.

Prototype bridge design

To formulate the experimental plan, a prototype bridge using the inverted-tee concept was developed (**Fig. 3**). The four-span bridge incorporated reinforced concrete columns in single-column bents, concrete inverted-tee cap beams, and five precast, prestressed, I-shaped concrete girders per span. The design was based on the American Association of State Highway and Transportation Officials' third edition of the *AASHTO LRFD Bridge Design Specifications*[10] with interims and California amendments[11] following the guidelines from the Caltrans *Bridge Design Aids*,[8] Caltrans *Bridge Design Specifications*,[12] and Caltrans *Seismic Design Criteria* version 1.5.[13] The design used Caltrans's deepest standard I-girder section (5.5 ft [1.7 m] depth), with a five-girder superstructure and single-column bents, to develop maximum demand in the cap and connection region.

The overall concept behind the design of the prototype bridge was to use a configuration that would generate maximum demand in the girder-to-cap connection region with a single-column bent and California I-girders. Detailed information and design calculations for the prototype bridge can be found in Theimann.[14]

Laboratory testing plan and procedure

A large-scale experimental investigation was conducted to determine the seismic behavior of the inverted-tee bridge system and to carefully investigate and quantify the girder-to-cap connection performance. The experimental work was divided into two phases. The primary purposes of phase 1 were the following:

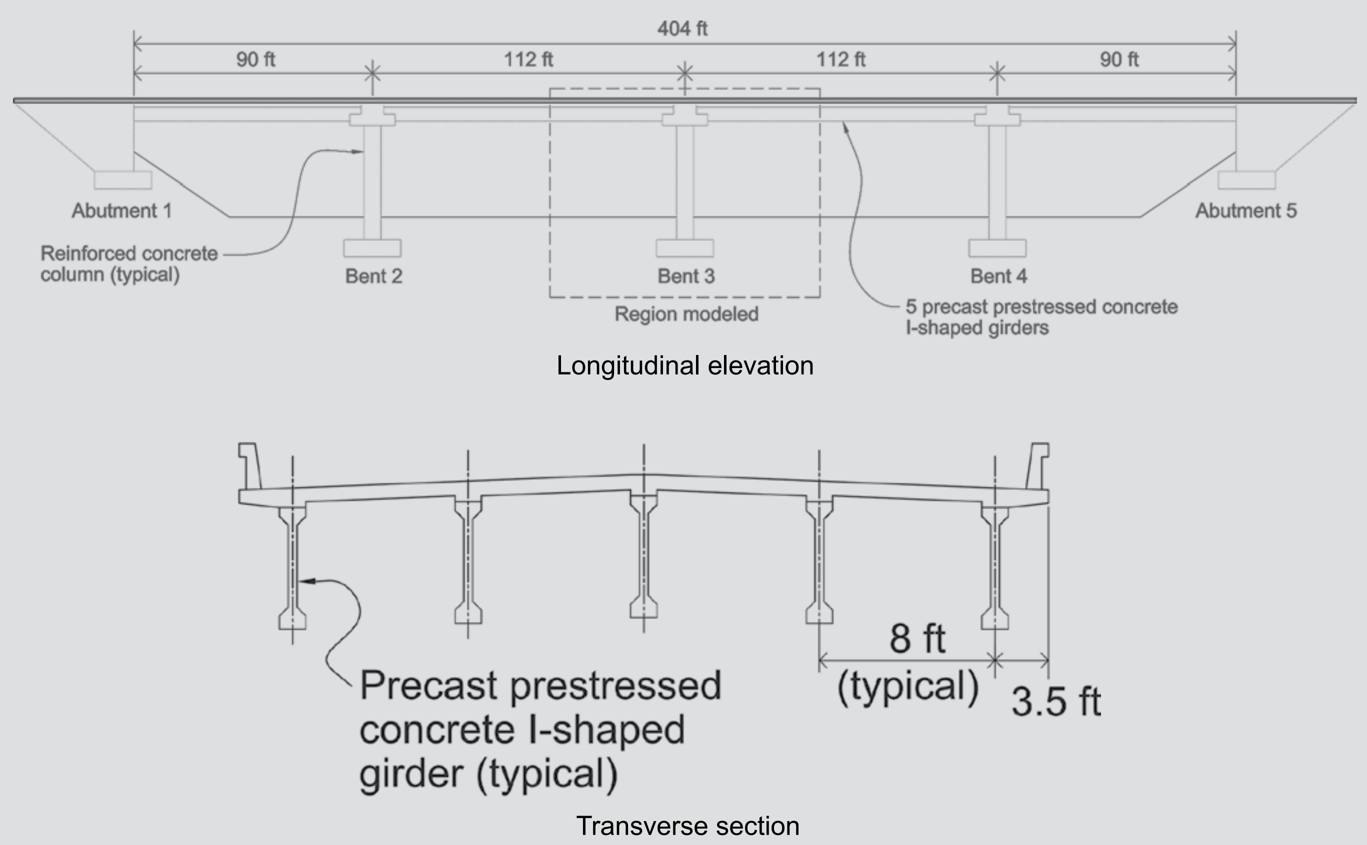

Figure 3. Prototype integral bridge structure. Note: 1 ft = 0.305 m.

- validate the overall system for high-seismic regions
- determine the ability of the girder-to-cap connections to maintain elastic superstructure action while allowing plastic hinges to fully develop at column ends
- compare and contrast the existing Caltrans girder-to-cap connection detail with an improved detail

The primary purpose of phase 2 was to exercise the girder-to-cap connections to realize their full potential by applying connection demands beyond what would be permitted by the typical overstrength capacity of the column plastic hinge region.

Experimental configuration

The experimental configuration was developed to allow the investigation of the existing connection detail for inverted-tee cap beams and dapped-end I-girders (referred to as the as-built connection) and a modified connection detail concept (referred to as the improved connection) in a single test unit. The test unit was designed at a 50% dimensional scale of the prototype structure. It modeled the full five-girder width of the prototype on both sides of bent 3, with the girder length extending approximately to the midspan on either side of the column (dashed region in Fig. 3).

Figure 4 provides a schematic of the test unit. The as-built connection was incorporated for the five girders on one side of the cap beam, while the improved connection was incorporated for the five girders on the other side of the cap. Detailed information on both connection concepts is provided later in this paper.

The termination of the girders at the location representing the prototype midspan resulted in support locations at the approximate girder inflection points under horizontal seismic loading. Hold-downs were used to properly simulate the effects of gravity load in the girder-to-cap connection region; these hold-downs were located at the approximate girder inflection points during the service-load-only condition, with load application occurring in two stages as detailed in the next section. Two pairs of horizontal actuators, one at each end of the test unit, were used to apply quasistatic horizontal seismic loads, and pairs of vertical actuators at each end were used to provide vertical support and to accommodate the column growth expected, due to the formation of plastic hinges, without introducing additional load to the superstructure. This support condition was accomplished by programming the vertical actuator control based on the predicted column growth at various horizontal displacement levels, following the procedure outlined by Holombo et al.[15]

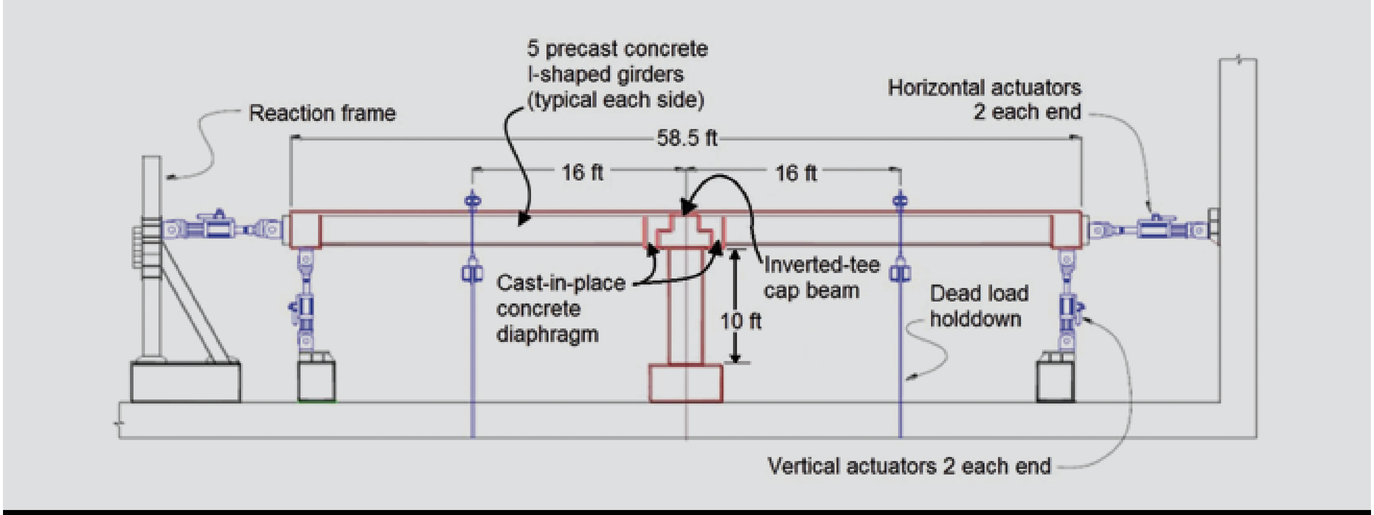

Figure 4. Schematic of test unit configuration for phase 1. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

Staged loading to simulate prototype gravity effects

The test configuration was designed to provide stress simulation of the prototype girder-to-cap connection region. To accomplish this simulation, the progression of the prototype connection load transfer capabilities during construction needed to be replicated as closely as possible. In the field construction of existing structures using the inverted-tee system, the girders are set in place without moment restraint prior to the diaphragm placement. The casting and subsequent curing of the diaphragm concrete in the girder-to-cap connection then creates a moment connection. The initial loads between the girders and cap beam prior to diaphragm casting are transferred as if the girders were simply supported. However, after placement of the diaphragm, the loads between the girders and cap are transferred through a moment connection. The experimental test unit was not designed to model the full length of the girders. Therefore, the hold-downs were used to simulate additional girder dead loads, barrier loads, and wearing surface loads. Because the girder loads would be present prior to diaphragm placement and the barrier and wearing surface loads would be added after deck and diaphragm placement, the vertical load simulations were introduced in a staged process to properly simulate the connection fixity during each stage of the loading process.

Figure 5 compares the moment profile of the prototype and the test unit at stage 1 (dead loads prior to connection moment capacity) and stage 2 (additional dead loads, such as barrier and wearing surface, after the connection moment capacity is developed). Figure 5 shows how the hold-down forces were used to accurately simulate the moments in the connection region. Similar comparisons were conducted to ensure proper simulation of the shear in the connection region but, for brevity, are not included here. The stage 1 loads (33.4 kip per girder) were applied using a spreader beam and spacer plates on the girders to simulate the additional girder self-weight load that would be present prior to deck placement. The stage 2 loads (45.2 kip per girder) were applied to the spreader beam

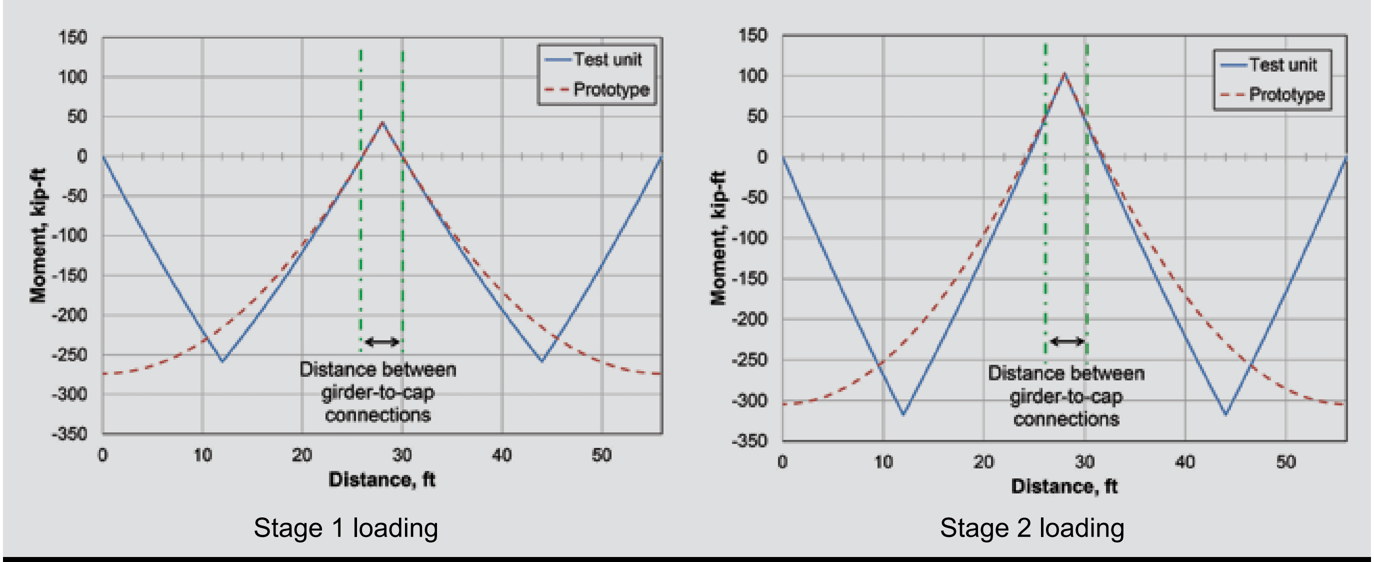

Figure 5. Comparison of prototype and test-unit moment profiles during stage loading (test-unit scale). Note: 1 ft = 0.305 m; 1 kip = 4.448 kN.

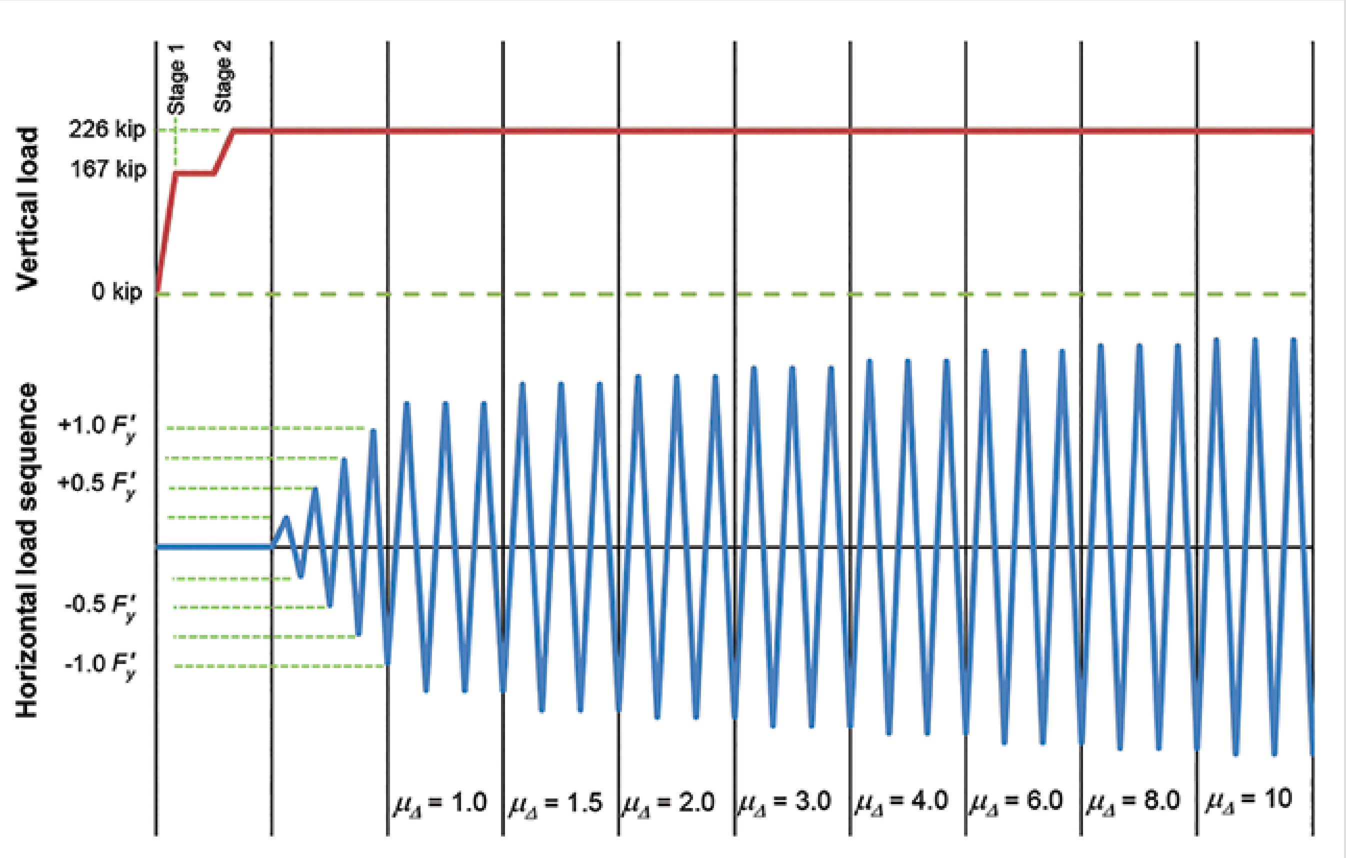

Figure 6. Phase 1 load sequence. Note: F_y' = estimated first yield strength of bridge system; μ_Δ = horizontal displacement ductility. 1 kip = 4.448 kN.

after deck and diaphragm placement to properly simulate the connection moment transfer.

Seismic load protocol

A cyclic quasistatic load protocol was planned to simulate the effects of horizontal earthquake loads in phase 1 testing. The horizontal actuators (Fig. 4) applied the cyclic horizontal loads. The dead load hold-downs remained engaged to simulate gravity loads, and the vertical actuators served primarily as adjustable supports. **Figure 6** shows the horizontal load sequence. Single load cycles were used to apply loads using the horizontal actuators under force control at peaks of ±0.25 F_y', ±0.5 F_y', and ±0.75 F_y', where F_y' was the estimated first yield strength of the system. The remainder of the phase 1 test was conducted using the horizontal actuators under displacement control, using three fully reversed quasistatic displacement cycles at system displacement ductility μ_Δ levels varying from to 1.0 to 10.0. The moment demand in the connections due to time-dependent effects was beyond the scope of the study.

Figure 7 provides the moment profile for the test-unit superstructure when subjected to large horizontal loads

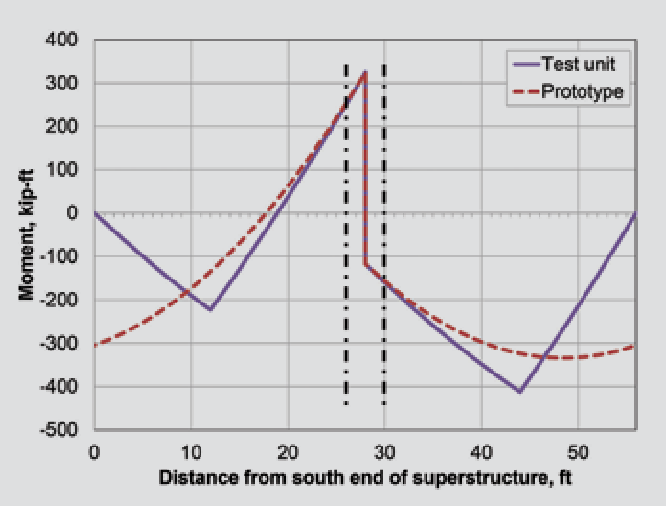

Figure 7. Comparison of test-unit and prototype moment profiles. Note: 1 ft = 0.305 m; 1 kip = 4.448 kN.

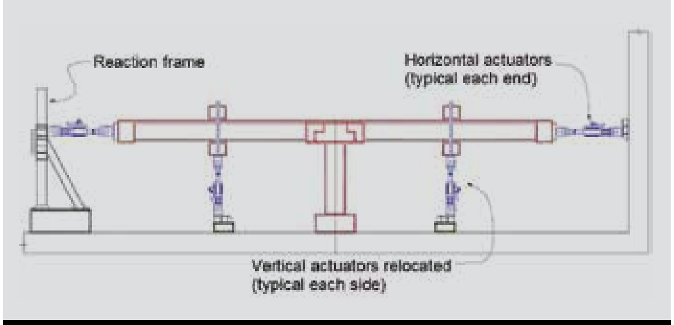

Figure 8. Configuration of test unit for phase 2.

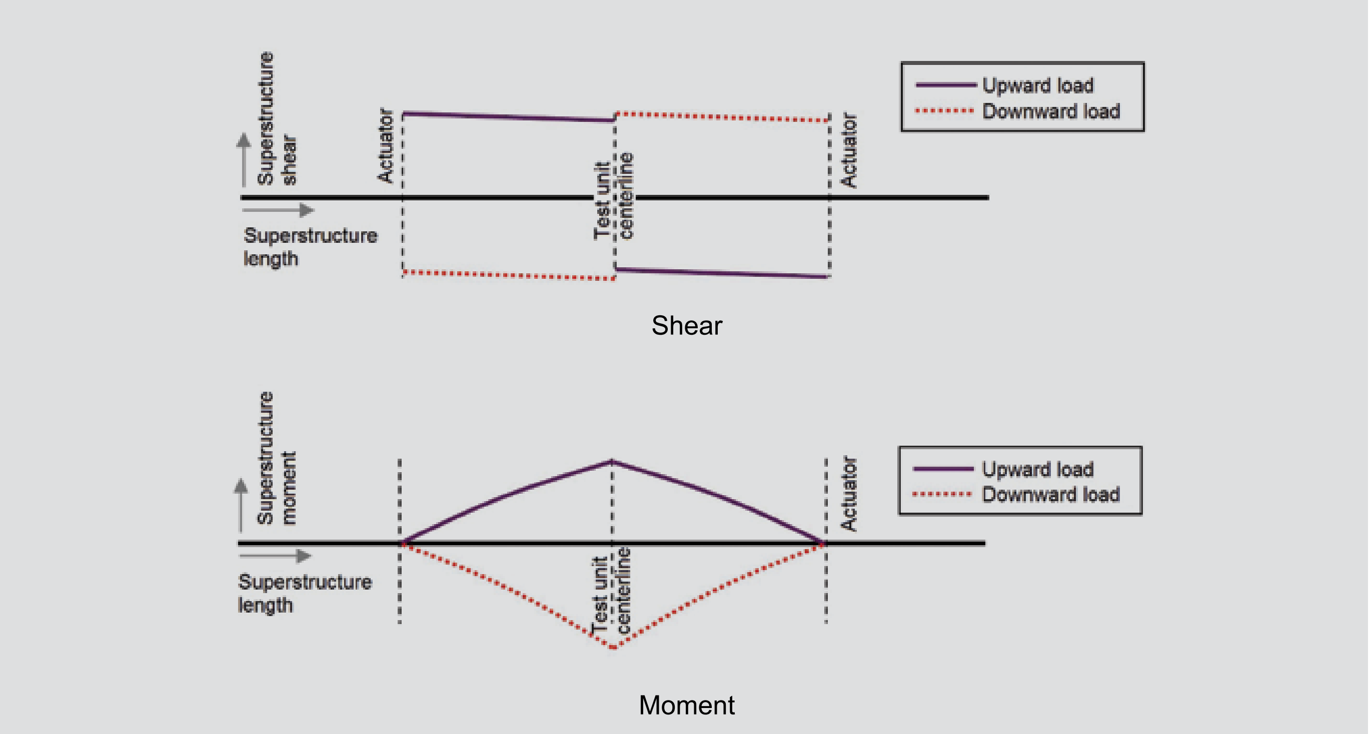

Figure 9. Phase 2 test-unit shear and moment envelopes.

that simulate seismic effects, assuming integral connection behavior in both the prototype and test unit. The sign convention used in this figure is based on positive moment, producing compression on the bottom surface of the superstructure. The test-unit profile also shows the negative moment peaks that result from the hold-down forces to simulate gravity effects. Comparing the test-unit moment profile with the prototype moment profile, also included in this figure, confirms that the test-unit and prototype profiles match well between the tie-down locations and match almost perfectly in the connection region near the column.

Phase 2 of the load protocol was planned to fully exercise the girder-to-cap connections and provide a comparison of the as-built and improved connection details. Phase 2 used the same test unit but reconfigured the actuators to allow maximum load and displacement to be applied to the girder-to-cap connection regions. The dead load hold-downs were removed, and the vertical actuators were relocated to these locations (**Fig. 8**). In this configuration, the vertical actuators were used to apply the primary load sequence, using displacement control to cyclically exercise the girders up and down, producing large shear and moment conditions in the girder-to-cap connection regions. The horizontal actuators were used to maintain system stability.

Figure 9 provides sketches of the test-unit shear and moment envelopes intended during the phase 2 loading, again assuming moment continuity in the superstructure over the column. These shear and moment profiles do not match

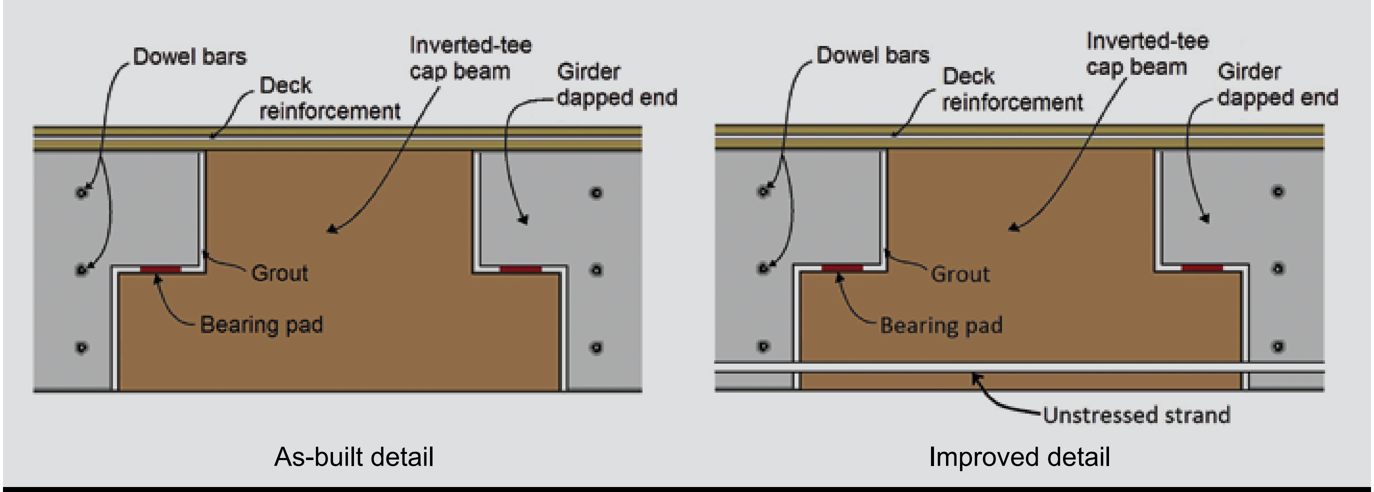

Figure 10. Girder-to-cap connection concepts.

the prototype profiles along the length of the superstructure, but the load configuration provides an excellent way to generate maximum shear and moment demand in the connection region. Because the primary focus of the testing related to the superstructure is the performance of the connection region, this configuration is well suited for the desired objective. The configuration is also suitable for simulating vertical acceleration effects in the connection region because the load generation in the test unit is coming from the vertically oriented actuators.

Test-unit details

Figure 10 shows the as-built girder-to-cap connection that has been used for the inverted-tee system. This detail incorporates three dowel bars that pass through ducts in the webs of the precast concrete girders near their dapped ends. After the girders are placed on the corbel of the inverted tee, the dowel bars are grouted into place in the girder webs, and a cast-in-place concrete diaphragm is used to encase the dapped end and dowel bars, thus achieving connection continuity.

For the detail to maintain its integral performance during seismic loading, it needs to successfully transfer vertical shear as well as positive and negative moments. Downward vertical shear in the as-built detail is easily transferred from the girder dapped end to the cap-beam corbel, due to the direct support configuration of the dapped end on the corbel. The as-built detail also has significant negative-moment capacity because the deck reinforcement provides tension continuity across the girder-to-cap joint. The dowel bars provide some resistance to upward shear and positive-moment loading that could occur during a large seismic event. However, because the detail includes no tension continuity near the girder bottom flange, rapid degradation of the girder-to-cap connection region is anticipated and will commence under high positive-moment action. Therefore, the Caltrans *Seismic Design Criteria* recommendations currently require that it be treated as a pin connection when subjected to seismic loading.

One of the objectives in the experimental investigation was to determine whether Caltrans's treatment of the existing detail is overly simplistic. While the connection detail may deteriorate when subjected to large seismic displacements, the girder subjected to positive moment demand should first overcome the adhesion and shear friction between the girder and diaphragm. The deterioration of the corresponding mechanisms is difficult to quantify; thus, experimental work to enhance understanding of these mechanisms and fully quantify their behavior would be beneficial to the design and implementation of this detail.

An additional objective of this research was to develop an improved detail that would provide a dependable load path for the girder-to-connection interface to sustain the positive moment and the corresponding shear. The as-built detail's main limitation is the lack of positive-moment tension continuity. To address this deficiency, the improved detail incorporated unstressed strands to provide tension continuity between the girder bottom flange and the cap-beam corbel (Fig. 10). Grade 270 (1860 MPa), seven-wire, uncoated prestressing strands were used. The strands were sized assuming that they needed to be capable of providing the needed tensile resistance of the positive moment in the girder-to-cap connection corresponding to the ultimate limit state of the plastic hinge region of the column. The strands were threaded through ducts in the bottom flange of the precast concrete girder and continued across the girder-to-cap interface into aligning ducts in the cap beam. After the strands were positioned, they were grouted in place to provide anchorage in the girder and cap. This connection detail retains the same negative moment and vertical shear capabilities as the as-built detail.

Figure 11 shows details of the test unit. Because the length scale factor was 0.5, the reinforcement area was determined by scaling the prototype design by 0.25 and using bar sizes and spacing that met constructibility requirements. Figure 11 provides the column-to-cap connection detail, with the column longitudinal reinforcement anchored into the cap beam. Figure 11 also shows the cap beam and joint reinforcement details, deck reinforcement, beam ledge reinforcement, and dowel bar details, all designed to replicate the Caltrans detail at the test unit scale. The one exception to the replication of the detail is the unstressed posttensioning tendon identified on the bottom-right side of the figure. On the side of the improved connection detail, these tendons were continuous from the girder bottom flange and through the bottom of the cap-beam corbel and terminated at the face of the cap beam on the side of the as-built detail. This configuration resulted in a test unit that modeled the improved girder-to-cap connection on the left side (in the figure orientation) of the cap and the as-built connection on the right side.

Figure 11 provides an elevation of a single girder. The girder strand layout, designed as a scale model of a typical Caltrans I-girder CA I66 strand layout while using constructible size and spacing details (Fig. 11). This figure also shows the layout of the ducts for the unstressed strand for the girders on the improved connection side. These ducts were positioned not to interfere with the strand layout (Fig. 11); because they were only required for the improved connection detail, they were not included in the girders on the as-built side of the test unit. Finally, Fig. 11 shows the reinforcement details for the full-depth blocked-end portion of the girders, and the details in the dapped-end portion of the girders immediately adjacent to the blocked end. The blocked- and dapped-end details were developed as scaled models of the prototype details, except for modifications based on a strut-and-tie concept. A strut-and-tie analysis was used to quantify the continu-

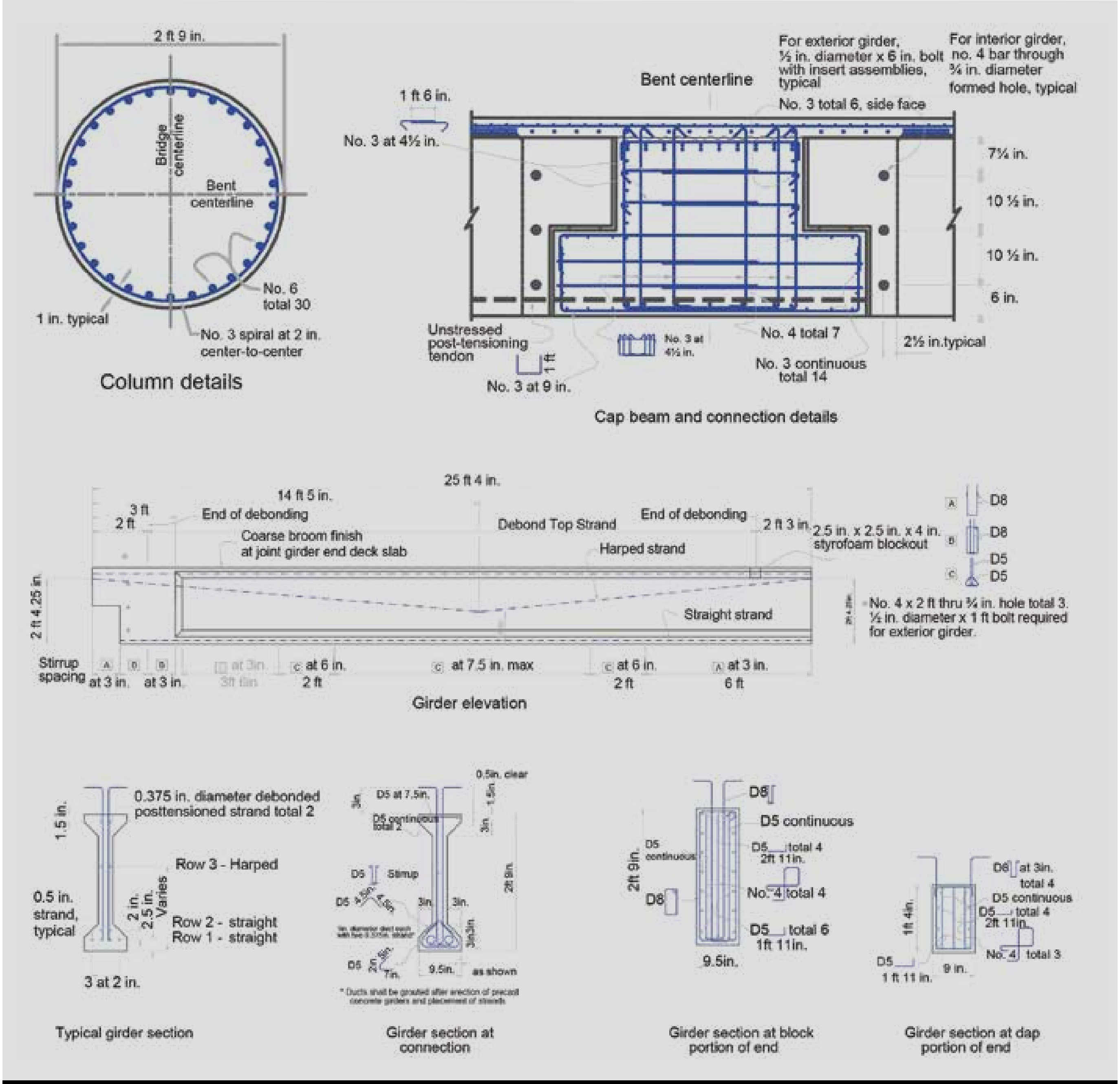

Figure 11. Test unit details. Note: no. 3 = 10M; no. 4 = 13M; no. 6 = 19M; 1 in. = 25.4 mm; 1 ft = 0.305 m.

ous horizontal-and-vertical tie (Fig. 11) and to eliminate some of the reinforcement congestion in the dapped-end region that often occurs when using conventional analysis approaches. Engineered wire mesh was used to provide the transverse reinforcement in the girders. The wire mesh was incorporated to validate its use in place of traditional transverse reinforcement in precast concrete girders. All of the prototype design details can be found in Theimann.[14]

Construction

To make the test unit as close to an actual inverted-tee bridge as possible, typical field construction practices and techniques were incorporated into the test-unit construction. The footing and column were constructed first, and temporary shoring was erected around the column to support the construction of the inverted-tee cap beam. **Figure 12** provides a photograph of the cap connection region for the center girder. The five-girder configuration results in the center girders attaching to either side of the cap beam adjacent to the cap beam–to–column connection. Because of this connection proximity, the strand ducts in the cap beam for the center-girder improved connection were curved around the column longitudinal reinforcement cage. While the introduction of these curves was a concern in terms of feeding the strand through the ducts and successfully grouting after placement, it did not pose any challenges during construction.

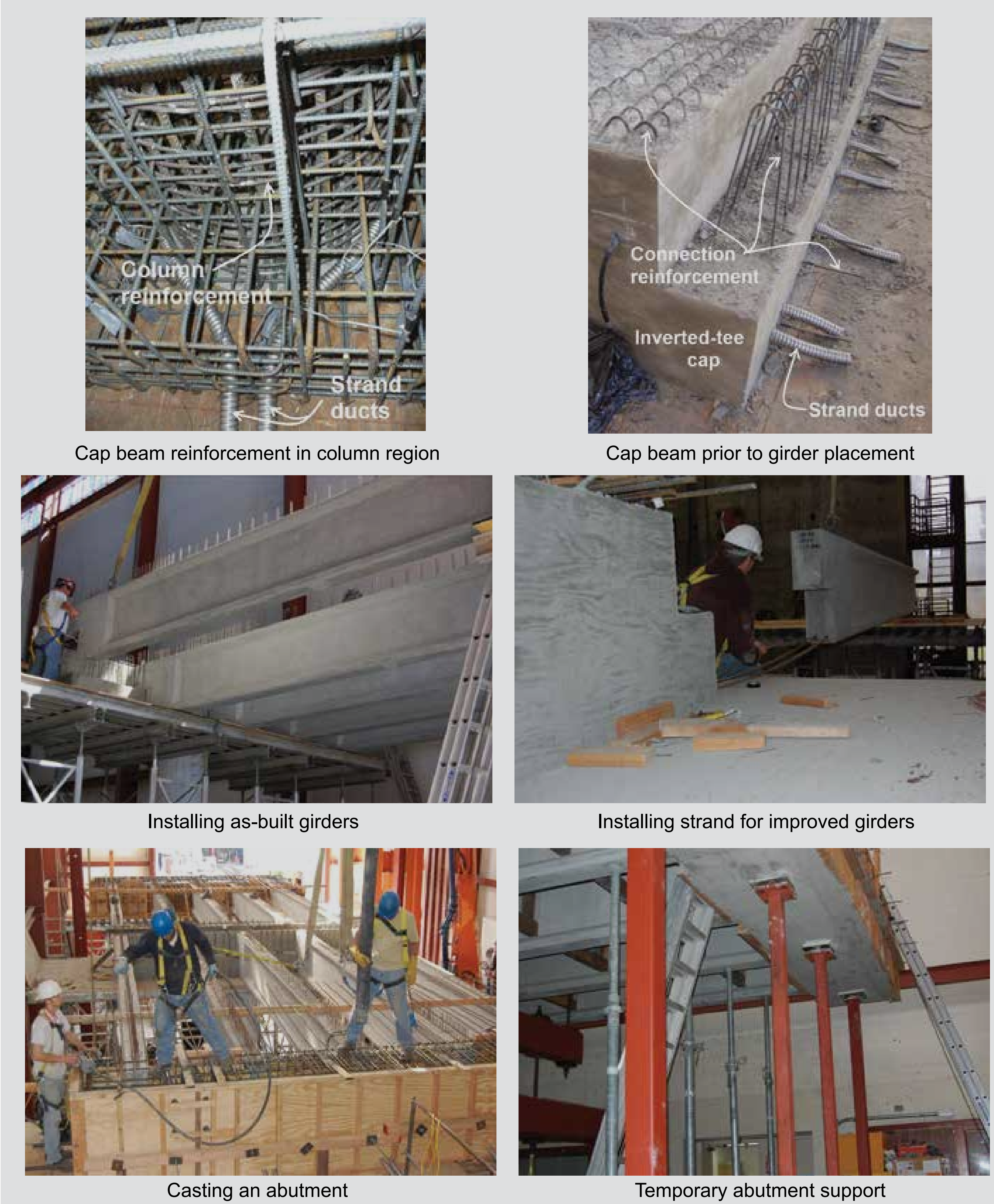

Figure 12. Photographs of construction.

Figure 12 shows the cap beam atop the column, prior to girder placement. The ducts in the figure were trimmed adjacent to the cap beam's vertical face and then mated with the ducts in the girders to allow the unstressed strands to be positioned continuously between the girders and the cap beam. The girders were fabricated off-site at a precast concrete production facility using typical methods. After the girders were delivered to the labora-

Column at 7 in. lateral displacement (displacement ductility +10.0)

Buckled bars in the column top plastic hinge region

Flexural cracking across the entire deck width

Figure 13. Test-unit photographs during and after phase 1 testing. Note: 1 in. = 25.4 mm.

tory, temporary shoring was used to support them in position on the cap beam. The strands for the improved connection were then properly positioned through the cap-beam ducts and grouted in place. Temporary shoring was also used to aid in the construction of the diaphragm in the connection region. Figure 12 also shows the abutment formwork and temporary shoring that was used prior to final configuration of the loading actuators and support system.

To provide temporary stability to the system, the concrete in the lower third of the diaphragm was placed without fully constraining the girder ends and prior to applying the stage 1 hold-down forces. Following the stage 1 load application, the diaphragm concrete placement was completed, and the abutment and deck concrete was placed. After the hardening of the deck concrete, the stage 2 hold-down load was applied to each span to simulate the additional weight of parapets and wearing surface that would be added to the prototype structure following deck concrete placement.

Phase 1 test results

Primary purposes for phase 1 included the following:

- validating the overall system for high-seismic regions
- verifying the capability of the girder-to-cap connections to maintain elastic superstructure action up to high seismic displacements (that is, the sufficiency of the girder connections to provide adequate resistance to develop plastic hinges in the column)
- comparing and contrasting the existing Caltrans girder-to-cap connection detail with an improved detail

General observations of test-unit performance

General observation of the displacement-control portion of the testing in phase 1 indicated excellent seismic behavior. **Figure 13** shows the column during the phase 1 test-

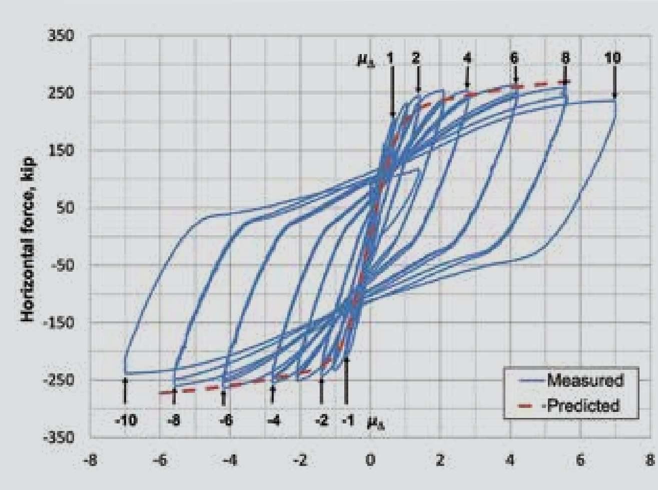

Figure 14. Horizontal force-displacement response for system test unit. Note: μ_Δ = horizontal displacement ductility. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

ing. Plastic hinges were developed at both the base of the column above the footing and at the top of the column just below the cap beam, indicating successful performance of the superstructure. The successful superstructure performance was notable, as it contradicted Caltrans's current conservative treatment of the as-built connection as having limited moment resistance at high seismic displacements. Further, no signs of distress were observed in the girders beyond the connection region, validating the use of wire mesh reinforcement as a substitute for traditional transverse reinforcement in precast concrete girders.

To quantify the test unit's performance, ductility μ_Δ of 1.0 was established as corresponding to idealized yield. This was determined based on measured experimental yield displacement and theoretical moment resistance for the first yield and idealized yield limit state, following the approach of Priestley et al.[16] Overall, the structure achieved a displacement ductility μ_Δ of 10.0, corresponding to 7 in. (180 mm) of total horizontal displacement in each direction. At this displacement, several column longitudinal bars had buckled (Fig. 13). Both the improved and as-built connections between the cap beam and girders behaved as fixed connections and did not show signs of significant damage or degradation. Flexural cracking was observed across the width of the deck, indicating that the diaphragm action of the deck had engaged all of the girders in resisting the column seismic moment. The seismic load distribution among the center, intermediate, and exterior girders was estimated to be 0.205:0.239:0.158, and this load distribution occurred in the girder-to-cap connection region immediately adjacent to the cap starting at early load levels. Detailed information on seismic load distribution in the superstructure is reported in Vander Werff and Sritharan.[17]

Phase 1 testing was concluded when, during the displacement cycle corresponding to ductility 10, a horizontal strength loss of approximately 10% was observed due to buckling of the longitudinal column reinforcement in the plastic hinge region just below the cap beam (**Fig. 14**). This failure mechanism was an excellent indicator that the superstructure behaved as intended, fulfilling its purpose according to the capacity design approach of maintaining sufficient elastic strength to produce plastic deformation and failure in the column hinge region prior to superstructure failure. In addition, a displacement ductility level of 10.0 far surpassed the minimum recommended ductility of 4.0 required in the Caltrans *Seismic Design Criteria*[9] and AASHTO LRFD specifications[10] for structures such as the prototype with single column bents supported on fixed foundations. This performance indicated the seismic capability of both the as-built and improved connection details because had the as-built connection deteriorated to a pinlike condition, the excellent strength retention observed in Fig. 14 would not have been possible.

Force-displacement response

The horizontal force-displacement response of the test unit (Fig. 14) indicates excellent seismic performance,

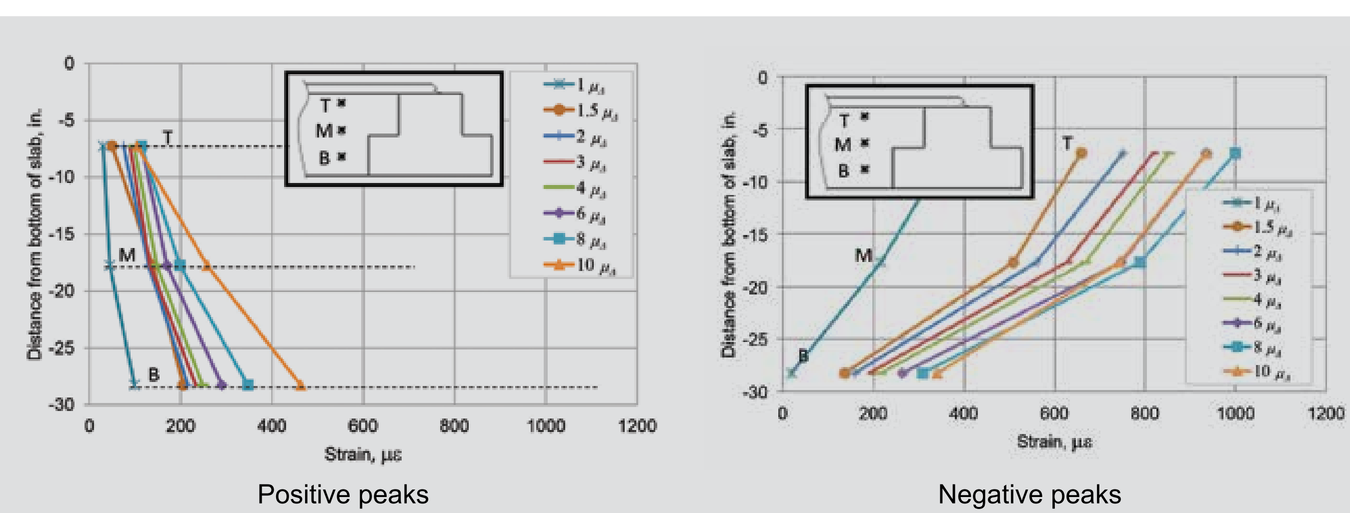

Figure 15. Dowel-bar strains at peak displacements. Note: μ_Δ = horizontal displacement ductility. 1 in. = 25.4 mm.

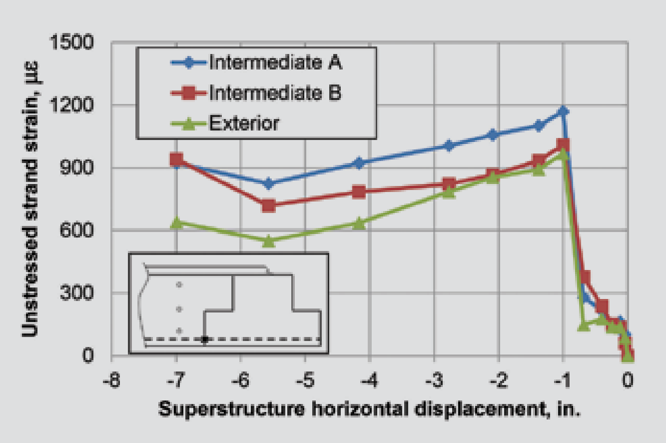

Figure 16. Unstressed strand strains in exterior and intermediate girders at peak displacements producing positive moment in improved connection region. Note: 1 in. = 25.4 mm.

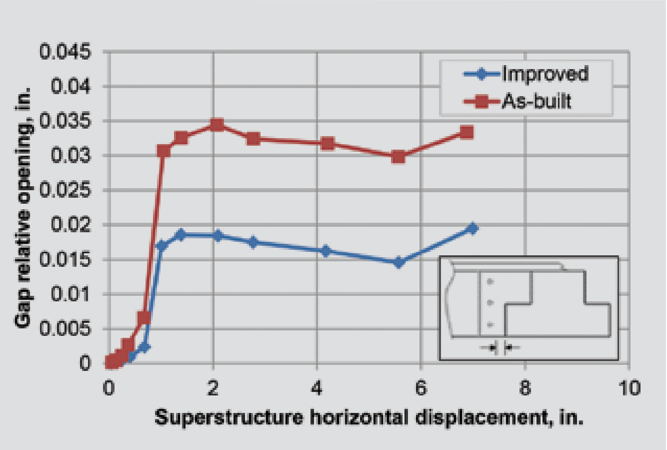

Figure 17. Gap opening at bottom girder-to-diaphragm interface (phase 1). Note: 1 in. = 25.4 mm.

as strength retention was maintained all the way to μ_Δ of ±8.0. Also, while longitudinal bar buckling and beginning of confinement loss occurred at μ_Δ equal to ±10.0, significant strength still remained in the system. This strength exhibits the ability of the system to continue carrying gravity load even at large seismic displacements.

Figure 14 also provides the predicted force-displacement response. This prediction was the result of an analytical investigation that incorporated a finite element model analysis, detailed in Theimann,[14] and an associated grillage analysis, detailed in Snyder et al.[18] The predicted force-displacement response compared favorably with the experimental horizontal force-displacement response of the superstructure. While there is a slight variation in the elastic region, the results converged more closely at higher levels of displacement as more of the test unit began to soften due to the development of cracks and yielding of longitudinal reinforcement.

Connection response

The behavior of the girder–to–cap beam connections was a primary area of interest, particularly verifying whether the superstructure remained elastic while allowing the column plastic hinges to fully develop. Visual observations during phase 1 indicated that the superstructure did indeed remain elastic, as plastic hinges were developed in the column and no significant spalling, bar buckling, or other failure indicators were observed in the superstructure. Data gathered during phase 1 testing was used to validate these observations. **Figure 15** shows dowel-bar strains measured in the improved connections for the center girder; it also shows dowel-bar strains at the peak displacements that produced positive moment in the connection region, and shows how strains are provided for the negative-moment peaks ($1.5\mu_\Delta$ corresponds to 1.0 in. [25 mm] horizontal displacement, $3\mu_\Delta$ corresponds to 2.0 in. [50 mm] horizontal displacement, and so on). The maximum measured strain was approximately 1000 $\mu\varepsilon$, well below the yield strain (approximately 2000 $\mu\varepsilon$). The dowel-bar strains in the as-built connections (not shown) were similar to those in the improved connections. The relatively low strain observed in all of the dowel bars indicates that the dowel bars remained elastic throughout the phase 1 test for both the as-built and improved connection details. In addition, the systematic behavior of the dowel bars, with larger strains at the top under negative moment and larger strains at the bottom under negative moment, shows that the dowel bars were engaged and providing moment resistance in both directions of loading.

The improved connection added unstressed strands for positive-moment continuity that were not included in the as-built detail. An interesting observation related to the dowel-bar data is that the dowel-bar strains in the improved connection are not lower than the dowel-bar strains in the as-built connection, as might be expected if some of the positive-moment tension load is diverted from the dowel bars to the strands. **Figure 16** shows strain data from the strands in the improved connections of one exterior and two intermediate girders. All of the strands seem to have been engaged already at low displacements, but all exhibited a noticeable increase in engagement when the superstructure was displaced to $1.5\mu_\Delta$ (1.0 in. [25 mm]). This sudden increase corresponds closely to the time when a noticeable opening of the girder-cap interface of the improved connection was first observed. The opening at this point was an indication that the concrete tensile capacity for resisting positive moment in the girder-cap interface was fully lost, producing significant load transfer to the strands. The dowel-bar strain profiles from the connection region (Fig. 15) also show the largest incremental increase between displacement steps $1.0\mu_\Delta$ and $1.5\mu_\Delta$. This behavior indicates that the dowel bars and unstressed strand were acting together, as both increased in strain proportionally and simultaneously.

Figure 16 also shows that after the sudden engagement of the strands between displacement steps $1.0\mu_\Delta$ and $1.5\mu_\Delta$,

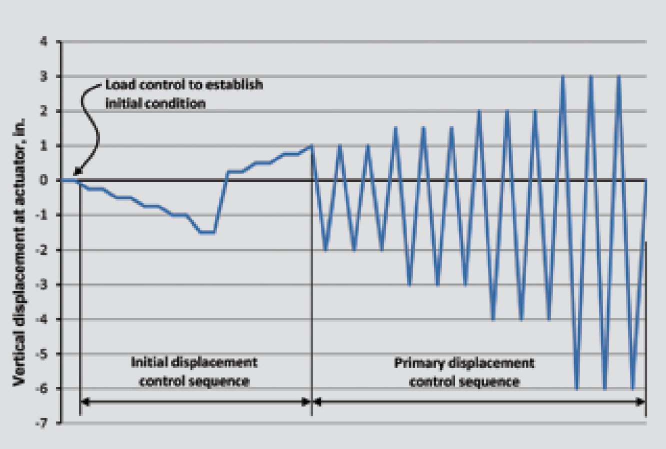

Figure 18. Test unit phase 2 load sequence. Note: 1 in. = 25.4 mm.

the strand strains at subsequent larger peak displacements decreased slightly, while still exhibiting noticeable engagement. Because the strand strains are below the yield strain, this behavior is not indicative of the strands relaxing after plastic behavior. Rather, it is more likely indicative of the load path and combined mechanism between the strands and dowel bars gradually changing due to concrete cracking and softening once the mechanism was fully engaged.

The relative behavior of the gap between the girder bottom flange and the adjoining edge of the diaphragm provides further insight into the difference between the improved and as-built connection behaviors. **Figure 17** compares the relative gap-displacement data for both the improved and as-built connections at positive-moment-direction peak-load conditions. The comparison reveals similar trends for both connections, but larger gap-displacement magnitudes in the as-built connection, showing that the unstressed strand in the improved connection was effective in reducing the gap opening under positive-moment loading.

Strains measured in the deck reinforcement, which acted as the primary tension reinforcement for the connections under negative-moment loading, were also used to investigate the superstructure behavior. A primary finding was that the deck strains exhibited the engagement of all five girders in resisting the column moment, from the early load stages all the way through to the overstrength moment. More details on this topic can be found in Vander Werff and Sritharan.[17]

Results from the phase 2 test

Because the superstructure, including the connection, maintained elastic response in the phase 1 test as expected, the phase 2 test commenced as planned to fully exercise the girder-to-cap connections, further quantify their behavior, and give an indication of their capability when subjected to vertical seismic effects.

Load protocol

For phase 2, the relocated vertical actuators were used as the primary control mechanism, while the horizontal actuators were configured to remain at zero load to retain horizontal stability in the test unit without affecting the load condition. **Figure 18** provides the load protocol used with the vertical actuators. The actuators were initially adjusted under load control to establish the initial condition, matching the endpoint of the phase 1 test and corresponding with the left edge of the sequence. Displacement control was then used to apply small, incremental vertical displacements at the actuator locations down to 1.5 in. (38 mm) below the initial girder positions (producing negative moment in the connection regions) and then up to 1.0 in. (25.4 m) above the initial girder positions (producing positive moment in the connection regions). The vertical displacements were applied to both sides of the test unit simultaneously to limit the force demand in the cap beam. The initial displacement control sequence was used to establish the test procedure and ensure specimen performance without going beyond load levels produced during phase 1. Following the initial sequence, the primary displacement sequence used three cycles per displacement level up to a maximum displacement of 6.0 in. (150 mm) downward and 3.0 in. (75 mm) upward on each side of the test unit.

General observations

The primary observation during phase 2 was the contrast in performance between the as-built connections and the improved connections. Throughout the test, the improved connections exhibited no signs of damage, whereas the as-built connections in all five girders experienced progressive deterioration near the interfaces, with all girders eventually pulling out of the diaphragm.

At a displacement of +0.5 in. (13 mm), the as-built connection was already subjected to a moment approximately 27% greater than the maximum positive moment achieved during the phase 1 test. At a displacement of +0.75 in. (19 mm), the improved connection side remained unchanged, but the as-built side began to exhibit significant degradation, reducing the positive-moment resistance. Significant spalling in the diaphragm on the as-built side was indicative of the distress in the connection dowel bars (**Fig. 19**). The gap between the girder bottom flanges and the cap beam widened to approximately 0.2 in. (5 mm), and the 1 in. (25 mm) thick grout along the bottom interface between the girders and cap had begun to separate and fall off. Cracks in the diaphragm concrete, indicative of the girder bottom dowel bar trying to pull out, were observed on the as-built connection side. At +1.0 in. (25 mm) displacement, the as-built connection continued to exhibit interface grout spalling, significant crack opening, and bottom flange girder pullout. A significant crack between

Spalling around dowel bars in girder connection region

Deterioration around dowel bars near exterior girder

Partially spalled grout pad at as-built girder-to-cap interface at +0.75 in. (19 mm) displacement

Opening of as-built connection under large positive-moment load

Figure 19. As-built connection region during latter stages of phase 2 testing. Note: 1 in. = 25.4 mm.

the underside of the deck and the top of the diaphragm on the as-built side indicated a connection separation. The improved connection remained unchanged.

The higher displacement cycles continued to show the trend of increased deterioration on the as-built side, with little change on the improved side. Figure 19 shows the exposed dowel bars on the as-built side. (Headed bars were used only for the exterior girders, given the limited embedment length outside the exterior girders. However, the deterioration was similar around the interior dowel bars as well.) Eventually, the as-built connection deteriorated to the point of behaving as a pin connection under positive-moment loading. Figure 19 shows the deterioration of the as-built connections at large positive displacements. The pin behavior of the as-built connection and the plastic hinge formed at the top of the column during phase 1 prevented any further testing to increase the positive moment in the improved connection to the point of ultimate failure. Based on the force-displacement plots for the structure at a downward displacement of 6.0 in. (150 mm), both connection details seemed to have additional negative-moment capacity. However, when the structure was cycled to an upward displacement of 3.0 in. (75 mm), a 42% drop in strength was observed, indicating that the as-built connection had already reached its ultimate capacity and therefore, the test was terminated at this point.

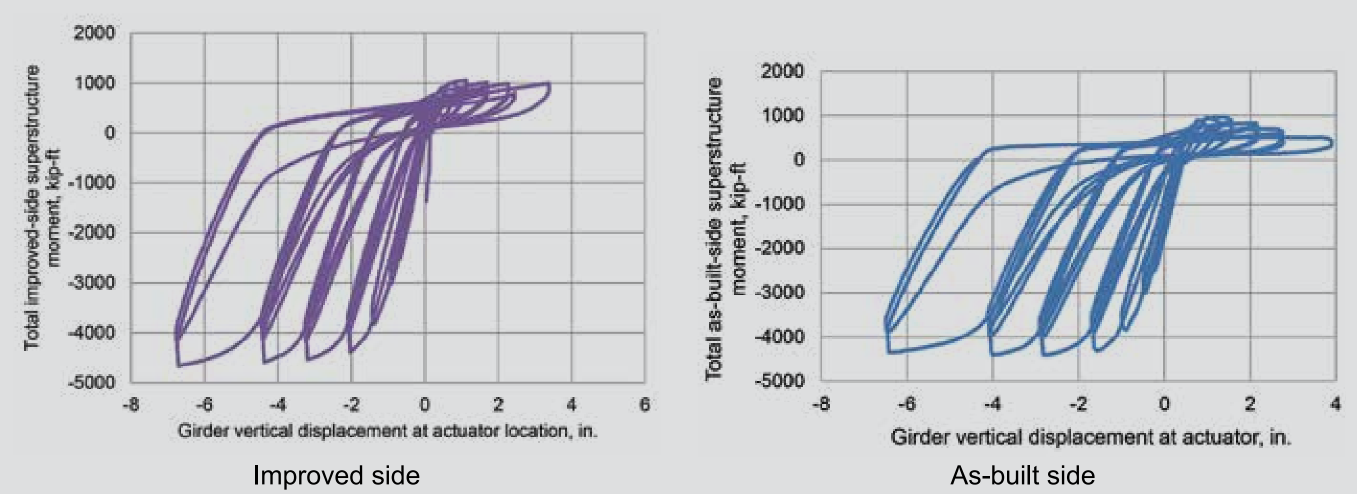

Figure 20. Vertical load-displacement response from phase 2 test. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

Force-displacement response

Figure 20 depicts the vertical load-displacement response for the center girder on both the improved and as-built sides during the phase 2 test. One notable difference between the two sides was the positive-moment peak resistance at high displacements; the improved side maintained its strength, whereas the as-built side exhibited a decrease in resistance as the displacement increased. Besides the positive-moment peak behavior, initial inspection of the remainder of the data in this figure seem to indicate similar inelastic responses on both the improved and as-built sides of the superstructure. However, a systematic analysis of the data was subsequently conducted, taking into account the experimental rotations, column load, horizontal actuator load, and vertical loads from both sides. This analysis was used to determine the total moment on the two sides, independent of each other. These separated data for each girder (**Fig. 21**) provide a much clearer picture of the test-unit behavior.

The response envelopes of the as-built and improved connections when subjected to positive moment are different (Fig. 21). While the as-built connection showed softened behavior up to 0.75 in. (19 mm) and significant strength deterioration thereafter, the improved connection was not subjected to relative displacements (adjusted to account for girder rotation) higher than approximately 0.35 in. (9 mm), and the response was linear and elastic. Both connections demonstrated reserve positive-moment strength well beyond the maximum demand experienced under horizontal seismic loading (dashed line on Fig. 21). However, the as-built connection experienced a loss in stiffness at loading above the maximum horizontal demand, whereas the improved connection performance was elastic.

Figure 21 compares the negative-moment behavior of the two connections. The improved connection exhibited a slight increase in performance over the as-built connection. The relative displacement difference between the improved and as-built connections at the large displacements is likely due to the loss of the grout pad in the as-built connection. The difference in negative-moment performance between the improved and as-built connections is less pronounced than for the positive-moment behavior. This similarity is not surprising, because the deck reinforcement provided the primary tension-transfer mechanism for the two connections,

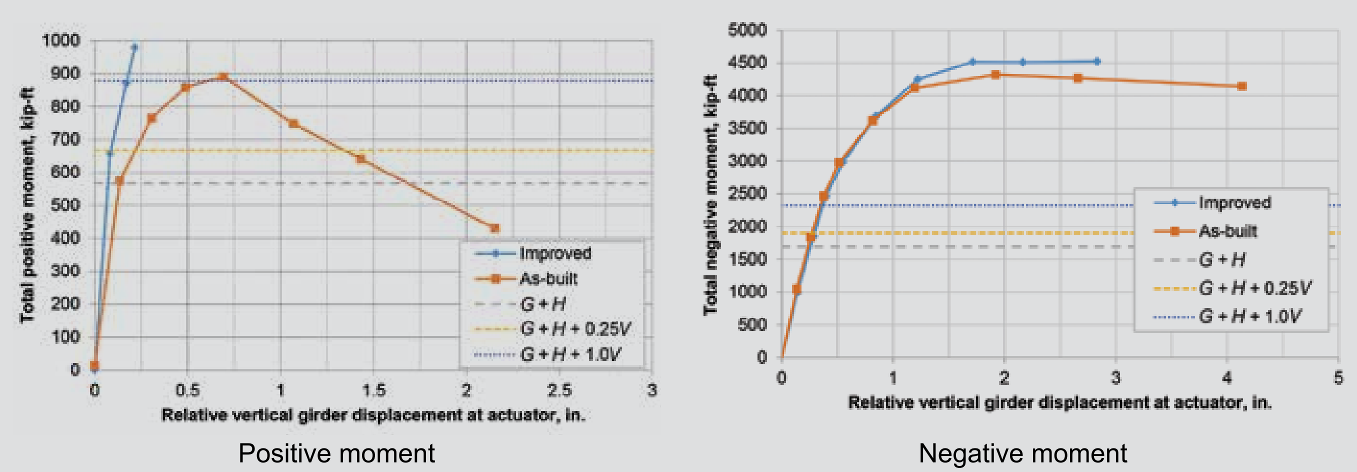

Figure 21. Comparison of girder-to-cap connection moment behavior. Note: *G* = gravity load condition; *H* = full horizontal seismic load condition; *V* = seismic vertical acceleration load condition. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

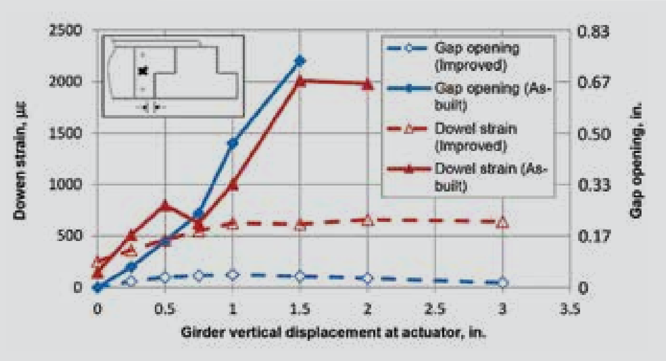

Figure 22. Gap opening and dowel strain at girder-cap interface (phase 2). Note: 1 in. = 25.4 mm.

and both connections used the same deck-reinforcement detail. Both connections exhibited excellent negative-moment performance, resisting moments that were 2.5 to 3.0 times higher than the maximum demand realized under horizontal seismic loading.

Comparison of phase 2 load with vertical acceleration effects

As previously mentioned, Caltrans recommendations call for the incorporation of a static vertical load, both upward and downward, equal to 25% of the dead load for ordinary standard bridges where the site peak ground acceleration is 0.6*g* or greater. In addition, these bridges need to include longitudinal-side mild reinforcement in the girders that is capable by means of shear friction to resist 125% of the dead-load shear at the cap beam interface. Neither connection detail in this test incorporated the additional longitudinal steel, yet both details showed successful performance when subjected to vertical load.

To correlate the phase 2 performance to the current recommendations, the loads reached during phase 2 were used to determine the equivalent vertical acceleration magnitudes that were reached during the test. Figure 21 includes lines that indicate the connection moment that would be expected during the gravity G, full horizontal seismic H, and no vertical acceleration V condition ($G + H$); the gravity, full horizontal seismic, and 0.25*g* constant vertical acceleration condition ($G + H + 0.25V$); and the gravity, full horizontal seismic, and 1.0*g* constant vertical acceleration condition ($G + H + 1.0V$). The equivalent vertical acceleration action included for the positive and negative moments has been added in the direction that increases moment magnitude for each direction (that is, upward vertical acceleration for positive moment, downward vertical acceleration for negative moment).

Figure 21 shows that both the as-built and improved connections exhibited moment capacity in excess of the 0.25*g* vertical acceleration stipulated by Caltrans.[8] In fact, the as-built connection did not begin to lose strength until the equivalent vertical acceleration reached 1.0*g*, although it began to show reduction in stiffness for loading beyond phase 1 testing. The improved connection exhibited elastic positive-moment behavior throughout the test, indicating that it had capacity in excess of 1.0*g*. Figure 21 shows that both the as-built and improved connections performed successfully under moment demands larger than what would be expected if the prototype structure experienced the gravity, full horizontal seismic, and 1.0*g* vertical acceleration condition. These results show the suitability for both connection details in resisting vertical acceleration effects in both the upward and downward directions that are greater than the requirement of 0.25*g*.[8]

Behavior of connection details

Figure 22 shows the dowel strain demand and the gap opening at the lower girder-cap interface for the improved and as-built connections in phase 2. The superior performance of the improved connection's data (shown in dashed lines for both gap opening and dowel strain) is even more pronounced in this figure. The gap opening is larger for the as-built connection. In addition, the magnitudes of dowel-bar strain are lower in the improved connection. Both of these observations show the benefit of the grouted, unstressed strand.

Figure 23 shows the strain envelope at peak displacements for the strains measured on the dowel bars for the duration of the phase 2 test. The behavior under displacements producing negative connection moment was similar for both the as-built and improved connections. For displacements producing positive connection moment, there was a notable difference between the as-built and improved dowel bars. The as-built connection exhibited quickly increasing strains at positive displacements of 1.0 in. (25 mm) and continued to experience an increase in strain until it yielded at a girder displacement of 1.5 to 2.0 in. (38 to 50 mm). However, the improved connection strain plateaued at a positive displacement of 1.0 in. (25 mm) and exhibited no increase during the remainder of the test. This plateau behavior was attributed to the deterioration in the as-built connection preventing significant increase in moment in the improved connection at the high displacements. Because the dowel strain continued to increase until

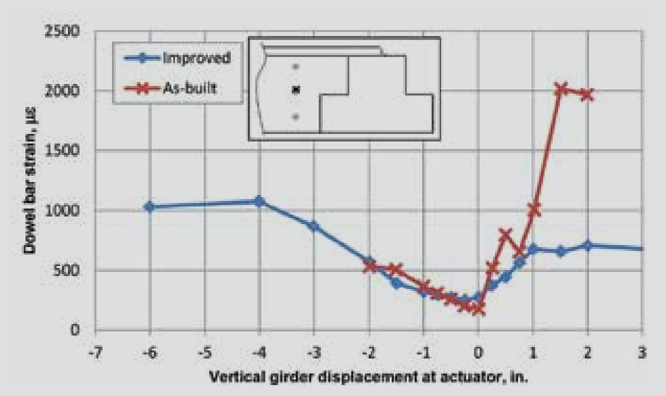

Figure 23. Dowel-bar strains in center girder midlevel bars at peak displacements. Note: 1 in. = 25.4 mm.

the moment plateaued, the dowel bars were engaged in the improved connection, though the addition of the strand provided significant positive-moment tension continuity. Although the unstressed strand improved the connection performance, the combined mechanism of the dowel bars and the unstressed strand was responsible for the positive-moment tension resistance.

The strain in the improved connection's unstressed strand during the phase 2 test (data not included for brevity) exhibited similar behavior in the exterior, intermediate, and center girders up to a girder vertical displacement of 0.75 in. (19 mm). In each of the girders, the strain gradually increased as the girder displacement increased. At higher displacements, the strand strain behaviors among the three girders were less consistent, likely due to shifting loading mechanisms as the as-built side of the superstructure began experiencing distress. However, even at the high displacements, the maximum measured strain values were around 2000 $\mu\varepsilon$. This magnitude is less than 40% of the strand yield strain, showing that the strand remained intact throughout the duration of testing.

Conclusion

Details that facilitate the use of accelerated bridge construction methods in high-seismic regions are desirable. The inverted-tee cap beam offers an excellent possibility for using precast concrete elements in bridge superstructures susceptible to high seismic loads, provided that the girder-to-cap connection region is properly addressed so that the bridges can be designed cost effectively. This study has shown that connections that are sufficient for high seismic load are feasible. In particular, the combination of inverted-tee cap beams and I-girders was demonstrated to be a viable accelerated bridge construction system. Constructibility of the detail was straightforward, and sufficient seismic performance was demonstrated. Based on this study, the following conclusions can be drawn:

- The as-built girder-to-cap connection as prescribed in the Caltrans *Seismic Design Criteria* was successful in behaving as a rigid connection during the phase 1 testing, simulating gravity and full horizontal seismic conditions. It remained elastic throughout the phase 1 test while allowing plastic hinges to fully develop at the column ends, though its relative displacements at the girder-to-cap interface were larger than for the improved connection. The as-built detail also successfully transferred shear forces and did not allow vertical unseating or collapse of the superstructure. The as-built connection's successful performance indicates that existing inverted-tee dapped-end bridge systems are sufficient without retrofitting to meet current seismic recommendations. Because the column top end was not expected to form a plastic hinge in past design, a retrofit of this region may be necessary.

- The as-built cap-to-girder connection successfully resisted positive moment demand equivalent to the gravity, full horizontal seismic, and 1.0*g* constant vertical acceleration condition but exhibited nonlinear response under positive and negative moments. This capacity surpasses current recommendations related to vertical acceleration and suggests that the longitudinal-side mild-reinforcement requirement in the Caltrans *Seismic Design Criteria* can be eliminated for the I-girders.

- The improved connection detail, which incorporated grouted, unstressed strands for positive moment continuity, provided excellent performance. The improved connection remained elastic under positive moments throughout phase 1 testing and exhibited lower relative displacement at the girder–to–cap beam interface than an as-built connection. The improved detail also successfully transferred shear forces and did not allow vertical unseating or collapse of the superstructure. However, the test setup did not allow full quantification of the improved detail. Further investigation into this detail is under way and can be found in Sritharan et al.[19,20]

- The superior performance of the improved connection was more apparent during phase 2 loading, which produced maximum shear and moment conditions in the connection region that were approximately double the expected maximum demand from the gravity and full horizontal seismic condition. The improved connection remained elastic and produced a gap opening at the lower girder-cap interface under positive-moment loading that was only 6% of the corresponding opening in the as-built connection. The elastic performance of the improved connection in the positive-moment connection was verified at moment demands well in excess of the gravity, full horizontal seismic, and 1.0*g* constant vertical acceleration simulated load condition.

- The as-built connection deteriorated when subjected to the phase 2 load sequence that exercised the connection beyond the gravity and full horizontal seismic condition. While the as-built connection produced a positive moment resistance that was over 50% higher than the expected gravity and full seismic condition, it did not maintain this resistance at large vertical girder displacements, and the girders eventually pulled out of the cap. While the as-built connection is sufficient for existing structures, the improved connection is recommended for new structures because of its superior performance.

- The unstressed strand in the improved connection reduced the girder–to–cap beam gap opening and maintained elastic behavior of the connection. However, it did not drastically reduce the dowel-bar strains except at high displacements. The results demonstrated that the unstressed strand and dowel bars worked together to form a viable positive-moment-tension load-transfer mechanism.

- Wire-mesh reinforcement was demonstrated to be an acceptable detail in lieu of traditional transverse reinforcement in precast concrete girders.

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Notation

F_y' = estimated first yield strength of bridge system

g = acceleration due to gravity

G = gravity load condition

H = full horizontal seismic load condition

V = seismic vertical acceleration load condition

μ_Δ = horizontal displacement ductility

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Abstract

To promote accelerated bridge construction in seismic regions, a large-scale experimental investigation was conducted to examine the seismic sufficiency of precast concrete I-girders in integral bridge superstructures. Such structures are not frequently used in high seismic regions due to lack of design guides and overly conservative design approaches. A half-scale, 17.8 m (58.5 ft) long test unit modeling a portion of a prototype bridge incorporating a concrete column, ten I-shaped precast concrete girders, and an inverted-tee concrete cap beam was used to experimentally verify that precast concrete members employing accelerated construction techniques can be used in integral superstructures and provide excellent seismic performance. Comparison of an as-built girder-to-cap connection detail with an improved detail shows that the as-built detail in existing bridges will satisfactorily resist positive and negative seismic moments and allow plastic hinges to develop at the column tops, though this was not the original design intent. However, the improved detail, which exhibited excellent seismic moment resistance, is recommended for new bridges to avoid potential deterioration of the girder-to-cap connection.

Keywords

Accelerated bridge construction, connection, experimental, girder, large scale, seismic.

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