DESIGN AND ANALYSIS OF PRECAST CONCRETE BRIDGES IN AREAS OF HIGH OR MODERATE SEISMICITY

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ABSTRACT

The seismic design and detailing of bridges made of precast prefabricated members has always been a challenge among bridge engineers and bridge builders. In recent years, there has been an increasing public demand for accelerate bridge construction using precast members or other innovative techniques. However, bridge engineers are concerned with the durability and performance of bridges made of precast members in areas of high or moderate seismicity. This paper examines the applicability of the AASHTO LRFD Specifications and other design specifications in areas of high or moderate seismicity. It discusses the different seismic design methodologies and their application to precast bridges.

INTRODUCTION

Precast bridge construction systems provide an effective and economical design solution, and can safely be used for bridge construction. Using precast components can shorten the time of bridge closures, minimize~~s~~ interference with traffic flow, and fulfills the philosophy of "get in, get out, and stay out." Proper seismic design entails a detailed evaluation of the connections between precast components as well as the connection between superstructure and the supporting substructure system.

This paper focuses on seismic design of ordinary highway bridges containing precast components. Ordinary bridges are well-proportioned structures with span lengths less than 300 feet, without splayed girders or outrigger caps or other unusual geometry, constructed with normal or light weight cast-in-place or precast concrete members. Superstructures are continuous at intermediate piers with either fixed or hinged connections supported on precast or cast-in-place concrete bents and without seismic isolation bearings. Superstructures could be supported on dropped bent caps or integral bent caps at intermediate piers and on conventional bearings at the end piers.

SEISMIC DESIGN CRITERIA

The AASHTO LRFD Specifications incorporates many of the seismic provisions of the 1992 Standard Specifications, but has updated them in light of new research developments. The principal areas where provisions were updated are:

- The introduction of separate soil profile site coefficients and seismic response coefficients (response spectra) for soft soil conditions.
- Definition of three levels of importance, namely "critical", "essential" and "other" as opposed to the two defined in previous AASHTO provisions. The importance level is used to specify the degree of damage permitted by the use of appropriate Response Modification Factors (R factors) in the seismic design procedure. The importance

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category for all typical bridges is considered “others” unless otherwise instructed by the Owner. The response modification factors for bridge substructure and connections could safely be used for bridges made with precast components with monolithic connections.

SEISMIC ANALYSIS METHODS

There are two general approaches to evaluate the seismic response of a bridge. The first approach is the conventional force-based analysis while the second involves the use of a displacement ductility criterion. In recent years, more emphasis has been placed on the displacement method. The force based analysis method is applicable to precast bridges with monolithic connections.

Force-based analysis

The provisions of the AASHTO LRFD Specifications[1] are largely based on the conventional force method, where bridge analysis is performed and the forces on its various components are determined. Structural damage is acceptable as long as it does not result in collapse or loss of life; and, where possible, damage that does occur should be readily detectable and accessible for inspection and repair. Small and moderate earthquakes should be resisted within the elastic range of the structural components without significant damage.

In force-based analysis method, a linear elastic multimodal response spectrum analysis is performed and the forces on its various components are determined. The capacities of the components are evaluated and the component demand/capacity (D/C) ratios are then calculated. A particular component is said to have adequate capacity if its D/C ratio is less than a prescribed force reduction factor, R. This factor allows for limited inelastic behavior and depends on the type of component considered.

Displacement-based analysis

Inelastic static analysis, commonly referred to as “push over” analysis, is used to determine the reliable displacement capacities of a structure or frame as it reaches its limit of structural stability. Inelastic static analysis is performed using expected material properties of modeled members. It is an incremental linear analysis, which captures the overall nonlinear behavior of the elements, including soil effects, by pushing them laterally to initiate plastic action. Each increment pushes the frame laterally, through all the possible stages, until the potential collapse mechanism is achieved. Because the analytical model accounts for the redistribution of internal actions as components respond inelastically, inelastic static analysis provides a more realistic measure of behavior than can be obtained from elastic analysis procedures.

The procedure outlined below is overall outlining for displacement-based analysis. The basic assumption is that the displacement demand obtained from linear-elastic response spectrum analysis is an upper bound of the displacement demand even if there is considerable nonlinear plastic hinging. These criteria are intended to achieve a “No Collapse” condition for standard ordinary bridges using one level of Seismic Safety Evaluation.

1. Perform linear elastic response spectrum analysis of the bridge based on design acceleration spectra specified by national or local specifications.
2. Determine the lateral and longitudinal displacement demands; forces are not of particular concern.
3. Calculate the moment-curvature diagram for each column and from that, the elastic, plastic and ultimate curvatures.
4. Using the above information and pier geometry (single or multi-column configuration), compute the displacement ductility of each column, and ultimate displacement capacity.
5. Perform pushover analysis of each pier in transverse direction. Also perform pushover analysis of the bridge in longitudinal direction. For this purpose, the plastic hinging moment for each column must be computed, and it might be necessary to incorporate foundation flexibility as well.
6. Compare the total displacement capacity of the pier to the displacement demand. If the capacity is insufficient, then higher ductility is required.
7. Perform similar pushover analysis in longitudinal direction for the entire bridge, and check the displacement capacity vs. demand.
8. Design the superstructure and foundation for 20% higher capacity than the plastic capacity of the columns to make sure that plastic hinges occur within the column.

SEISMIC RESPONSE OF PRECAST CONCRETE BRIDGE SYSTEMS

The lack of monolithic action between the superstructure and bent cap in precast, prestressed concrete beam systems causes either the girder seats or the column tops to act as pinned connections. Consequently, while the transverse stability of multi-column bents is ensured by frame action in that direction, stability in the longitudinal direction requires the column bases to be fixed to the foundation supports. This requirement places substantial force demands on the foundations of multi-column bents, particularly in areas of moderate to high seismicity. Developing a moment connection between the superstructure and substructure makes it possible to introduce a pinned connection at the column bases. This results in less expensive foundations. Integral bent caps are beneficial in precast, prestressed concrete beam systems by introducing moment continuity at the connection between the superstructure and the cap, the columns are forced into double-curvature bending, which tends to substantially reduce their moment demands. As a result, the size and overall cost of the adjoining foundations is also reduced.

SUPERSTRUCTURE CONNECTION DETAILS

The most common types of connections for precast prestressed girder bridges are fixed in high seismic zones, and hinged in low seismic zones. In both cases the superstructure consists of a cast-in-place concrete deck on precast prestressed concrete girders made continuous for live load at intermediate piers. Precast girders are temporarily supported on elastomeric or wood blocks until the cast-in-place diaphragm is completed. The designer will check the edge distance and provide a dimension that prevents edge failure, or spalling, at the top corner of the supporting bent cap for loading from the block including dead loads from girder, slab, and construction loads.

Hinge Diaphragm Connection

The hinge connection shown in Fig. 1 is for continuous spans at intermediate pier diaphragms. The design assumptions for hinge diaphragms are:

1. All girders of adjoining spans should be of the same depth, spacing, and preferably the same type.
2. Provide reinforcement for negative moments at intermediate piers in the deck due to live loads and superimposed dead loads from traffic barrier, pedestrian walkway, utilities, etc.
3. Design hinge bars size and spacing for anticipated lateral loads due to seismic and other load combinations. Provide adequate embedment for hinge bars into the crossbeam and hinge diaphragm.
4. Design longitudinal reinforcement at girder ends for shear friction.

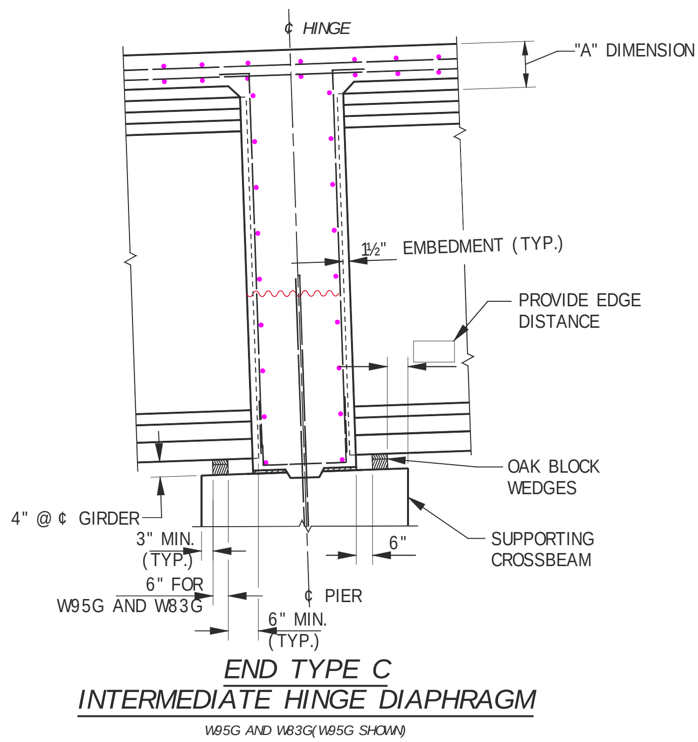

Fig. 1 Intermediate Hinge Diaphragm

1 inch = 2.54 cm

Fix Diaphragm Connection

The connection shown in Fig. 2 is for continuous spans with fixed moment resistant connection between super and substructure at intermediate piers. Pier caps are wider for fixed connections and precast girders are supported on blocks or pads on bent cap. The diaphragm is cast-in-place in two stages to ensure precast girder stability after erection, and completion of diaphragm after slab casting and initial creep occurs. Adequate extended strands and reinforcing bars are provided to ensure performance of the connection during a major seismic event. The design assumptions for fixed diaphragms are:

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

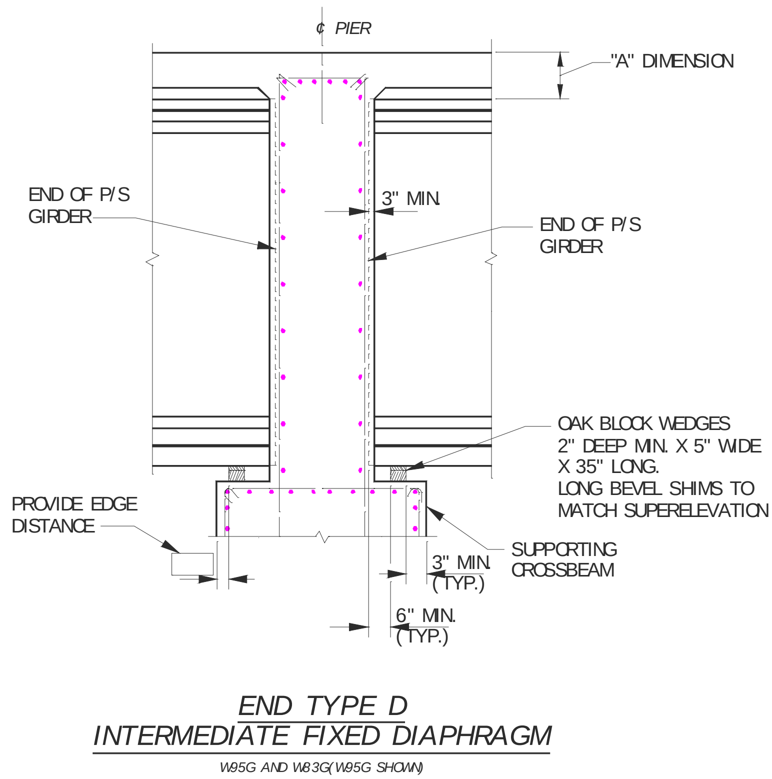

Fig. 2 Intermediate Fix Diaphragm

Positive Moment Connection at Intermediate Diaphragms

Strand extension details with strand anchors and strands chucks are used for continuous spans at diaphragms and are shown in Fig. 3. The effect of time dependent positive moments from creep and shrinkage should be considered in determining the positive moment capacity. A minimum of 4 extended strands shall be provided regardless of design requirements.

The design procedure to calculate the required number of extended strands is described herein. This calculation is based on developing tensile strength of the strands at ultimate loads. Since the distance across the connection is too short to develop the strands by concrete bond alone, mechanical anchors are provided to develop the yield strength of the strands.

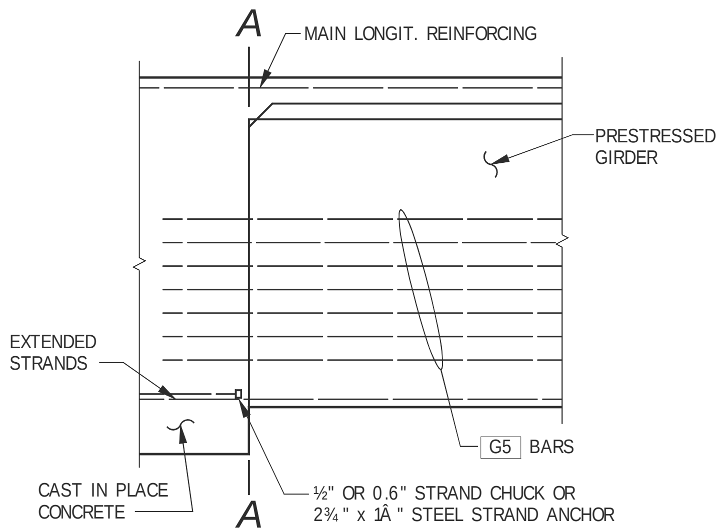

Fig. 3 Strand Extension Detail

The total number of extended strands[2] N_{PS}, from the end of prestressed girder at fixed intermediate diaphragm, in absence of the time dependent positive moment from creep and shrinkage, shall be taken as:

$$N_{PS} = [12(M_C + V_C.h).\frac{N_c}{N_g}.k - M_{SIDL}].\frac{1}{0.9.\phi.A_{PS}.f_{PS}.d} \qquad (1)$$

Where:

M_C = the lesser of seismic elastic or plastic hinging moment of top of column, ft-kips

M_{SIDL}= moment due to superimposed dead loads traffic barriers, sidewalk, ft-kips

V_C = the lesser of elastic seismic shear or plastic hinging shear of column, kips

h = distance from top of column to centroid of superstructure, ft

d = distance from top of slab to centroid of extended strands, in

N_C = number of columns in the pier

N_g = number of prestressed girders in the pier

A_{PS} = area of each extended strands, in^2

f_{PS} = ultimate strength of prestressing strands, ksi

k = span coefficient (k=0.5 for $L_1 = L_2$, k = 0.67 for $L_1 = 2L_2$)

ϕ= flexural resistance factor

Abutment Connection for Precast Prestressed Girder Bridges

The typical practice for abutments in seismic zones is cast-in-place concrete pier walls supported on spread footing, pile, or shaft foundations. Precast girders are often supported on elastomeric bearing pads at end piers. Semi integral end diaphragms are used for shorter bridges, and L-shape diaphragm for longer bridges is typically used for precast bridges. In this type of connection the bridge ends are free for longitudinal movement but restrained for transverse seismic movement by girder stops. The bearing system is designed for service load condition but may not resist seismic loading. The bearings are designed to be accessible so that the superstructure can be jacked up to replace the bearings after a major seismic event.

Semi-Integral End Diaphragm

Fig. 4 shows the connection for semi integral end pier. This type of end diaphragm allows eliminating expansion joints at end piers. The gap between the end pier wall and the end diaphragm shall satisfy longitudinal seismic movement requirements at extreme event limit state, and thermal expansions at service limit state.

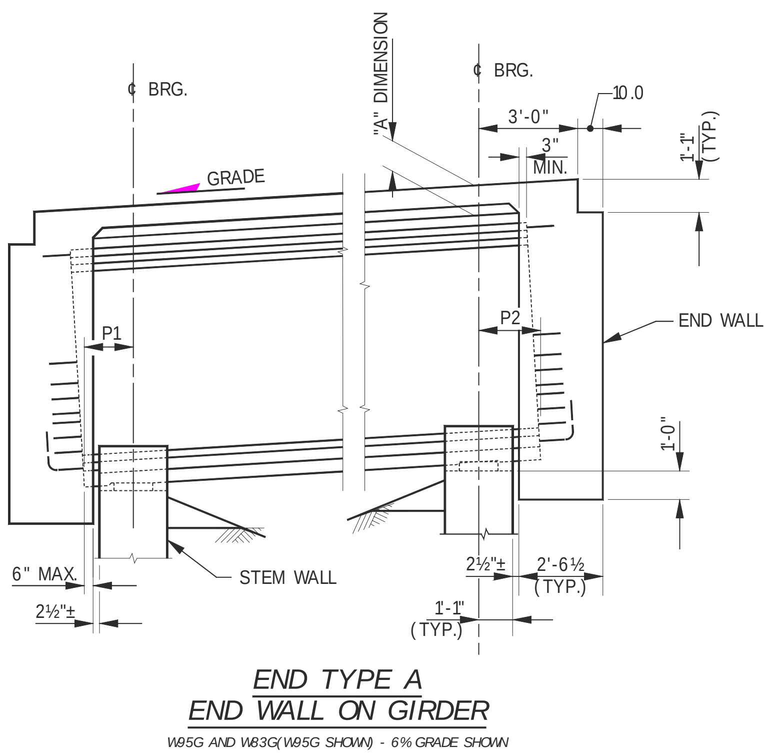

Fig. 4 Semi Integral End Pier Connection

1'-0" = 30.5 cm =305 mm

L-Shape abutment

Fig. 5 shows the connection for L-shape end piers. This type of diaphragm is suitable for longer bridges. The seat width provided at the end pier wall shall satisfy the longitudinal seismic movement requirements. The gap between superstructure and backwall shall satisfy the longitudinal seismic movement at extreme event limit state and thermal expansion at service limit state. For bridge with large longitudinal movement, the backwall could be designed as breakable during seismic event. Provisions shall be made to repair the backwall after earthquake.

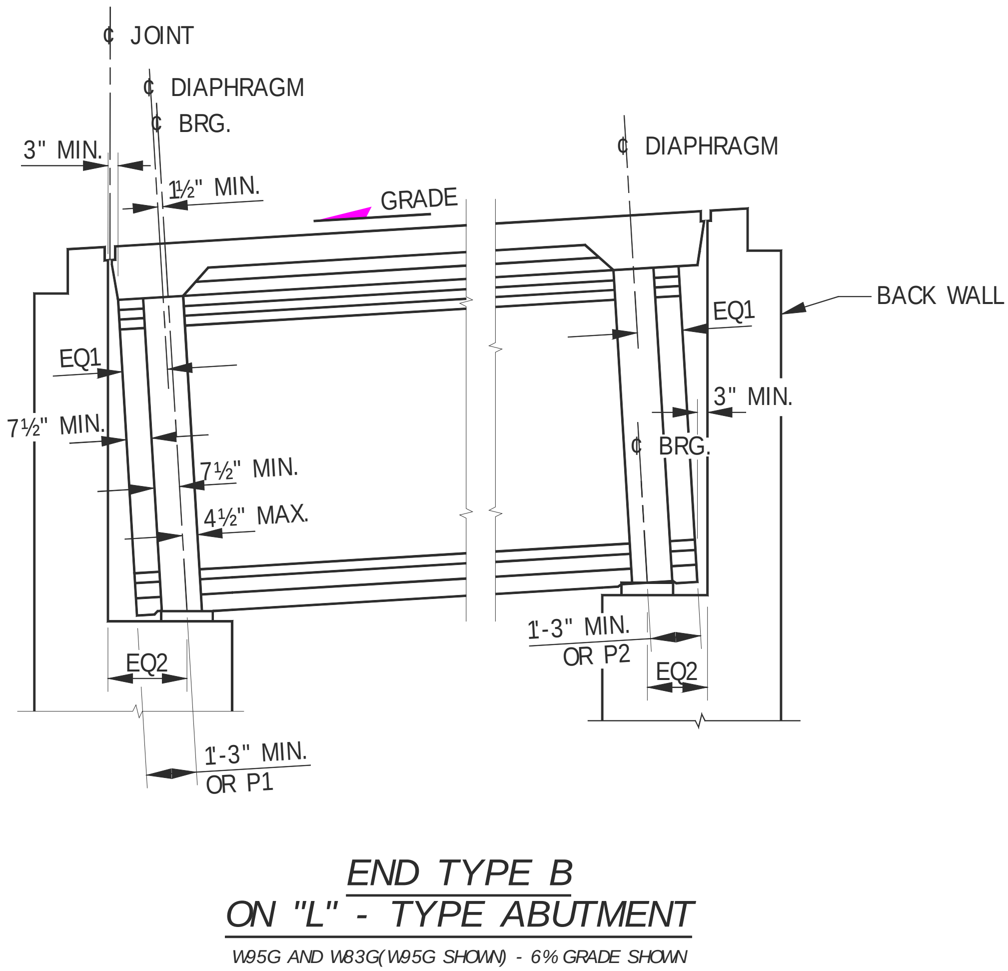

Fig. 5. L-Shape End Pier Connection

In an L abutment, the minimum displacement requirements at the expansion bearing should accommodate the greater of the maximum displacement calculated from a displacement analysis or a percentage of the empirical seat width, N, specified in Equation 2[(1)].

$$N = (8 + 0.02L + 0.08H)\,(1 + 0.000125\,S^2) \qquad (2)$$

Where:

N = minimum support length, inches

L = bridge length to the adjacent expansion joint, or to the end of the bridge, feet

H = average height of abutment wall supporting the superstructure, feet

S = skew angle of the support measured normal to span, degrees

The Seismic Design Criteria permits hinge seat width in “well-balanced frames” to be evaluated as shown below.

$$N = \Delta_{p/s} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_{eq} + 4 \text{ inches.} \qquad (3)$$

Column to Bent Cap/Crossbeam Connections

The goal of a seismic connection at this location is to transfer the plastic moment demands at the top of the column into the superstructure without yielding either the connection itself or the beam-ends. To achieve this, both the connection and the beam-ends must be designed to provide a design strength exceeding the required strength from the forces transferred. Additionally, the connection should be detailed to ensure adequate distribution of the longitudinal moment from the top of the column to the various beams. The design procedure involves the following steps:

1. Determination of the plastic moment capacities at the top and bottom of the column.
2. Calculation of the principal stresses in the bent cap due to joint shear.
3. Design of joint reinforcement.
4. Torsion-shear friction analysis to verify the ability of the bent cap to transfer the column plastic moments to the bridge superstructure.
5. Check the superstructure capacity to ensure that the plastic hinges do rather form in the column than the superstructure.

PRECAST CONCRETE BENT CAP

Precast bent cap systems are of increasing demand for use in highway construction. Precasting moves concrete forming, pouring, and curing operations out of the work zone, making bridge construction safer and more environmentally friendly, and it removes bent cap construction from the critical path. Precasting also improves quality and durability because the work is performed in a more controlled environment.

Precast Column to Precast Bent Cap Connection

A conceptual design and detailing for a precast bridge is shown in Fig. 6. The monolithic connections between precast components at intermediate pier diaphragms and at foundations are designed to meet the seismic requirements. Reduced top of the column diameter provides a seat for placement of the precast bent cap. Architectural flares on precast columns could also be used for precast bent support seat. The gap on top of the column shall be carefully dimensioned to eliminate the adverse effect of flares on column stiffness, and to ensure that plastic hinges form on top of column.

The recess provided in the precast bent cap allows the precast column rebar to extend into the crossbeam to accomplish monolithic connection. Precast girders are then seated on the ledges of precast bent cap with extended strands to provide positive seismic moment capacity. The connection is then completed with a cast-in-place diaphragm to ensure a monolithic connection while maintaining continuity in bridge superstructure.

Interface shear resistance shall be checked at the interface between precast bent cap and cast-in-place concrete at column cage intrusion into the precast bent cap. A combination of shear keys and reinforcing bars may be necessary to provide adequate interface shear resistance.

Top and bottom longitudinal reinforcement in the precast bent cap through the monolithic joint shall be provided. The top of the precast bent cap is to resist negative moment due to the weight of precast girders, and lower portion of cast-in-place diaphragm. Deck slab may be cast the completion of lower diaphragm.

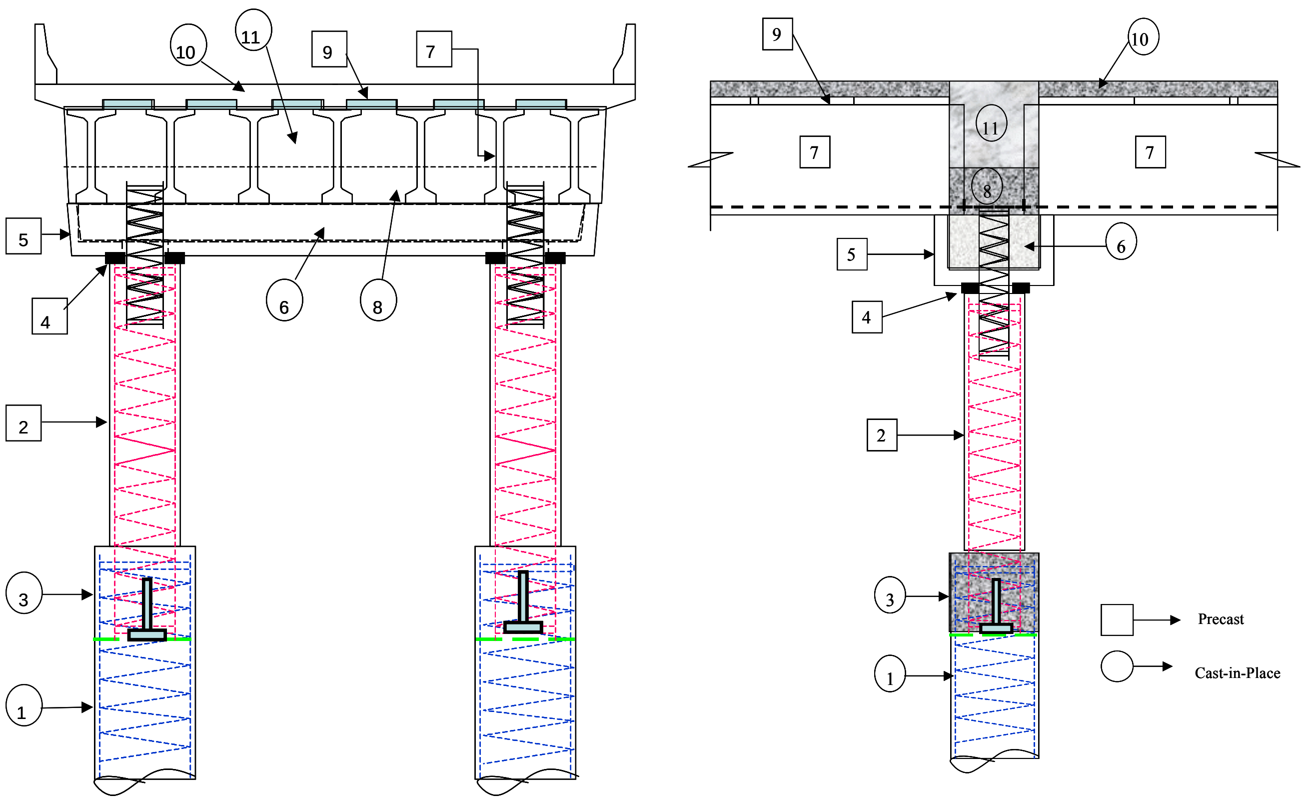

Fig. 6. Conceptual Design and Detailing for a Precast Bridge System

The proposed sequence of construction for completion of precast bridge system is as follows:

1. Design precast columns with adequate longitudinal and transverse reinforcement on the top and the bottom of the column.
2. Position precast column in place and cast concrete for spread footing

3. Place precast bent cap shell on the top of the column. A minimum of 3 inch rubber pad or similar material shall be provided on the top of the column prior to placement of precast bent
4. Cast concrete to achieve monolithic column to bent cap connection.
5. Place precast girders with the adequate number of extended strands with strand anchors to develop seismic positive moment, unless spliced.
6. Cast lower pier diaphragm and intermediate diaphragms to ensure precast girder stability during slab casting or placement of precast deck panels.
7. Place deck panels, cast and cure deck slab concrete.
8. Complete casting concrete for intermediate diaphragm. Use transverse post-tensioning, if necessary.
9. Cast traffic barriers and sidewalk if applicable.

The above type of precast construction is also applicable where precast trapezoidal tubs are used instead of prestressed I-girders. The construction sequences and the design procedures are identical to those of prestressed I-girder superstructure as mentioned above. The above method of construction is suitable for precast members with monolithic connection at intermediate diaphragms.

CONCLUSIONS

Precast concrete bridge construction systems provide an effective and economical design solution, and can safely be used for bridge construction. The following conclusions could be made:

- Proper seismic design entails a detailed evaluation of the connections between precast components as well as the connection between superstructure and the supporting substructure system. Monolithic connections are the key to proper seismic performance of precast bridges.
- The provisions of the AASHTO LRFD Specifications are applicable to precast bridges. However, displacement based analysis shall be considered to determine the displacement capacity of the structure.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, Second Edition, American Association of State Highway and Transportation Officials, Washington, DC, 1998
2. Bridge Design Manual, BDM. Washington State Department of Transportation, PO Box 47340, Olympia, WA 98504-7340