

SUITABILITY OF BRIDGE MADE WITH PRECAST COMPONENTS IN AREAS OF HIGH OR MODERATE SEISMISITY

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Abstract

The need for rapid construction arises from the inevitable increases in traffic congestion that have occurred in the past few decades, and the corresponding costs and safety manifested in many forms, including exposure of workers to traffic hazards, and most importantly, the waste of time due to delays. Prefabricated bridge components are in increasing demand for accelerated bridge construction. Precasting eliminates the need for forming, casting, and curing of concrete in the work zones, making bridge construction safer while improving quality and durability. This paper provides an overview of WSDOT precast bridges, relevant research and bridge projects using precast members.

Introduction

Precast concrete bridge systems provide effective and economical design solutions for new bridge construction as well as for the rehabilitation of existing bridges. Proper seismic design begins with a global analysis of the structure and a detailed evaluation of the connections between precast components as well as the connections between superstructure and the supporting substructure system. The system must be made to protect the superstructure from force effects due to ground motions through fusing or plastic hinging. In seismic regions, provisions must be made to ensure ductile behavior in both longitudinal and transverse directions.

Studies have already been carried out to investigate seismic resistance and rapid construction. Requiring both characteristics together poses some special problems. While precast concrete is used to speed site assembly of components, the locations of the structural connections are critical. Transportation constraints point towards the use of line members, such as beams and columns, yet these lead to the need for making connections where bending moments are largest and inelastic behavior is virtually inevitable.

Seismic Response of Bridges with Precast components

The provisions contained in the AASHTO LRFD Bridge Design Specifications¹, are largely based on the Conventional Force method, where bridge analysis is performed and the forces on its various components are determined. Plastic Hinging is the basis of the ductile design for bridge structures. Plastic Hinges may be formed at one or both ends of a

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reinforced concrete column. After a plastic hinge is formed, the load path will change until the second plastic hinge is formed. The philosophy of ductility and the concept of plastic hinging are applicable to precast bridges if connections are monolithic.

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 seismic zones. 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 sizes and overall cost of the adjoining foundations are also reduced.

Potential causes of failure in Precast bridges

Precast Bridge failures during an earthquake have been attributed to one or more of the following causes described below:

1. Unseating of the superstructure at abutments, hinges, intermediate supports or expansion joints due to insufficient support length.
2. Joint shear failure at critical superstructure-substructure connections.
3. Columns punching through the superstructure due to large vertical acceleration or inadequate connection details.
4. Inadequate transverse support or transverse stop mechanism at supports.
5. Pile to pile cap connection failure due to inadequate connection details.
6. Joint failure at intermediate piers due to inadequate positive moment resistance.

Precast Superstructure

The majority of bridges in Washington State are prestressed girder bridges. In Washington State, the use of prestressed I-girders started in the 1950's. The complete description of standard prestressed girders and their span capability is presented in WSDOT Bridge Design Manual (BDM)³, and can be downloaded from the WSDOT website at: <http://www.wsdot.wa.gov/eesc/bridge/index.cfm>. Both AASHTO LRFD Bridge Design Specifications² and BDM are used for the design of prestressed girders.

In 1997, long span deep prestressed girders⁴ in both pretensioned and post-tensioned spliced-girders were added to the WSDOT inventory. In 2001, pretensioned trapezoidal tub girders, commonly called "bath-tubs", were adopted. In 2004 wide flange pretensioned I-

girders^{5,6} were added, and in 2008, wide flange WF100G girders with span capability of over 200 ft (61 m) were added. The cross sections of deep prestressed girders and companion post-tensioned spliced girders used for composite superstructures are shown in Figs. 1 and 2 respectively. The span capabilities for all pretensioned and post-tensioned spliced girders are presented in reference 1, based on the strain compatibility and the recent modifications to the flexural design of prestressed concrete girders⁷.

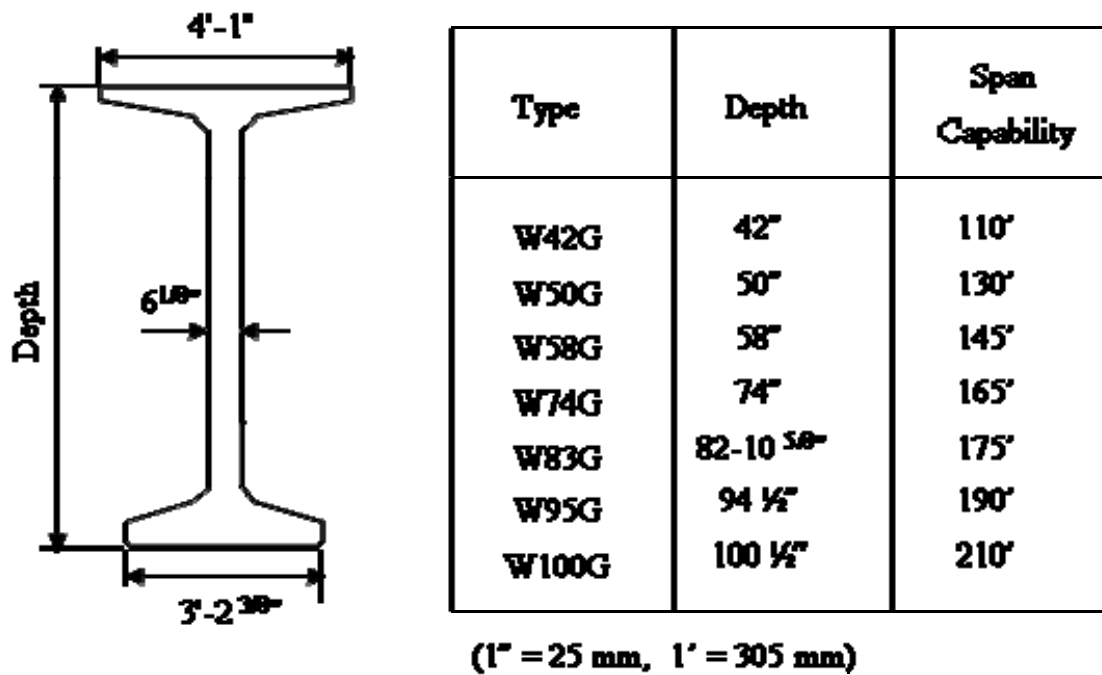


FIG. 1 WSDOT WIDE FLANGE PRESTRESSED GIRDERS

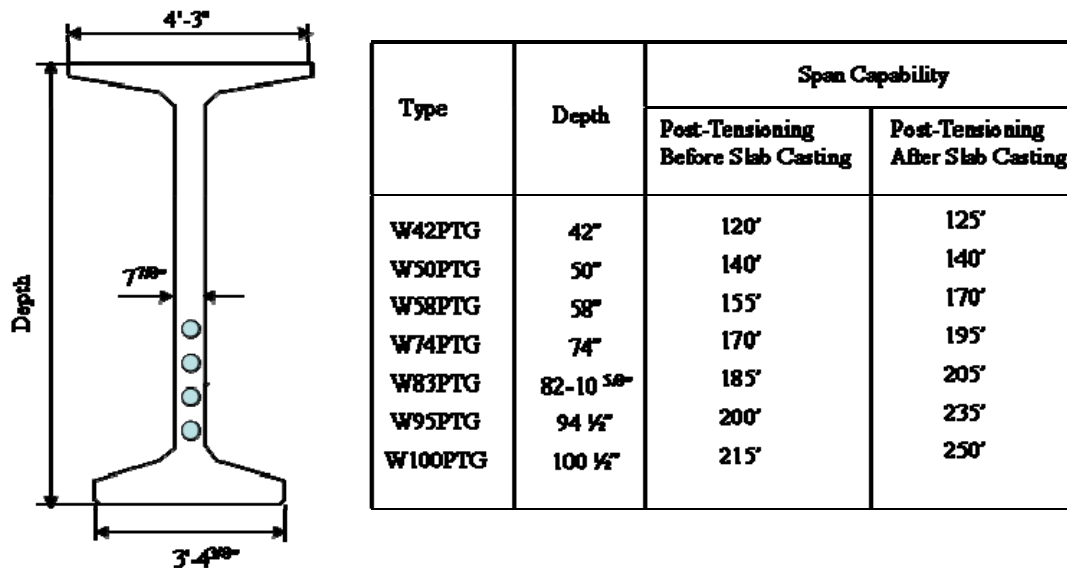


FIG. 2 WSDOT PRECAST PRESTRESSED SPLICED GIRDERS

Connection of precast girders at intermediate piers

Monolithic action between the superstructure and substructure components is the key to seismic resistant precast concrete bridge systems. Lack of monolithic action causes the column tops to behave as pin connections resulting in substantial force demands on the foundations of multi-column bents, particularly in areas of moderate to high seismicity. Fig. 3 shows a typical monolithic moment resistant connection used for WSDOT precast girder bridges.

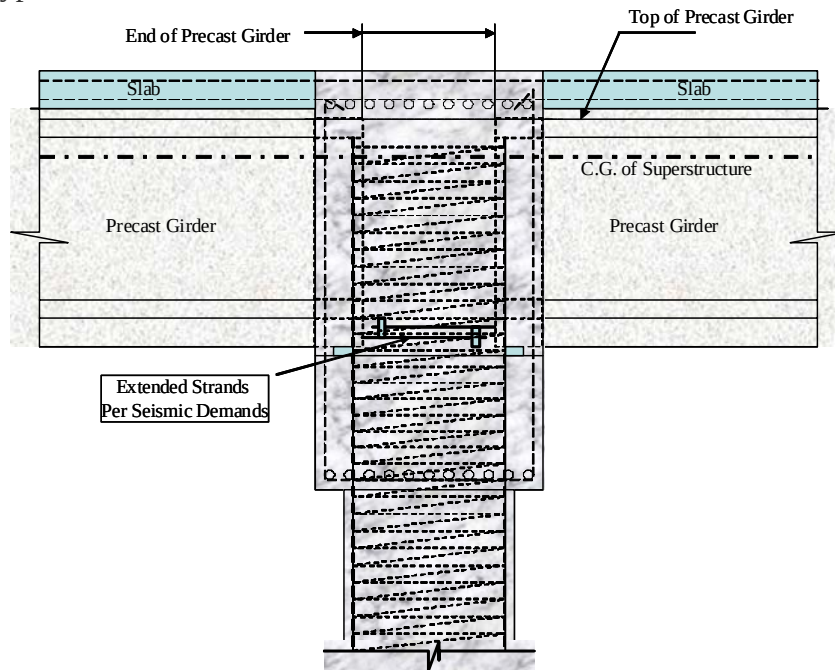


FIG. 3 TYPICAL MONOLITHIC MOMENT RESISTANT

The connection shown in Fig. 3 is for continuous spans with fixed moment resistant connection between super and substructure at intermediate piers. Cast-in-place diaphragm is completed 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. 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. Determine resultant plastic hinging forces at centroid of superstructure.
4. Determine the number of extended strands to resist seismic positive moment.
5. Design diaphragm reinforcement to resist the resultant seismic forces at centroid of diaphragm.

6. Design longitudinal reinforcement at girder ends for interface shear friction.

For girders made continuous for live load, extended bottom prestress strands are used to carry positive EQ load, creep, and other restrained moments from one span to another. Strands used for this purpose must be developed in the short distance between the two girder ends. The strand end anchorage devices as shown in Fig. 4 are used, per WSDOT Standard Plan, is a 2'-0" (610 mm) strand extension with strand chuck and steel anchor plate. The number of strands to be extended cannot exceed the number of straight strands available in the girder and shall not be less than four.



FIG. 4: STRAND ANCHORS AT MOMENT RESISTANT CONNECTIONS

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.

The design moment at the center of gravity of superstructure is calculated using the following:

$$M_{po}^{CG} = M_{po}^{top} + \frac{(M_{po}^{top} + M_{po}^{Base})}{L_c} h \quad (1)$$

where:

- M_{po}^{top} = plastic overstrength moment at top of column, kip-ft.
- M_{po}^{Base} = plastic overstrength moment at base of column, kip-ft.
- h = distance from top of column to c.g. of superstructure, ft.
- L_c = column clear height used to determine overstrength shear associated with the overstrength moments, ft.

The superstructure does not resist column overstrength moments uniformly across the width. To account for this, an effective width approximation is used, where the maximum resistance per unit of superstructure width of the actual structure is distributed over an equivalent effective width to provide an equivalent resistance. Per LRFD Specifications, the effective width for girder slab bridges should be calculated as follows:

$$B_{eff} = D_c + D_s \quad (2)$$

where:

D_c = diameter of column

D_s = depth of superstructure including cap beam

Based on the structural testing conducted at the University of California at San Diego La Jolla⁸, California in the late 1990's (Holombo 2000), roughly two-thirds of the column plastic moment to be resisted by the two girders adjacent to the column (within the effective width) and the other one-third to be resisted by the non-adjacent girders. Therefore the moment per girder line is calculated as follows:

Adjacent girders (encompassed by the effective width): $M_{sei}^{Int} = \frac{2M_{po}^{CG}}{3N_g^{int}}$

Non-adjacent girders: $M_{sei}^{Ext} = \frac{M_{po}^{CG}}{3N_g^{ext}}$

Seismic Moment:

$$\begin{aligned} \text{if } M_{sei}^{Int} \geq M_{sei}^{Ext} & \quad \text{then } M_{sei} = M_{sei}^{Int} \\ \text{if } M_{sei}^{Int} < M_{sei}^{Ext} & \quad \text{then } M_{sei} = \frac{M_{po}^{CG}}{N_g^{int} + N_g^{ext}} \end{aligned}$$

where:

N_g^{int} = Number of girder within the effective width.

N_g^{ext} = Number of girder outside the effective width.

Number of extended straight strands needed to develop the required moment capacity at the end of girder is based on the yield strength of the strands.

$$N_{ps} = 12 \left[M_{sei} \cdot K - M_{SIDL} \right] \cdot \frac{1}{0.9\phi A_{ps} f_{py} d} \quad (3)$$

where:

A_{ps} = area of each extended strand, in²

f_{py} = yield strength of prestressing steel specified in LRFD Table 5.4.4.1-1

d	=	distance from top of slab to c.g. of extended strands, in.
M_{SIDL}	=	moment due to SIDL (traffic barrier, sidewalk, etc.) per girder
k	=	span moment distribution factor
ϕ	=	flexural resistance factor

PRECAST GIRDER CONNECTION AT END PIERS

Precast girders are often supported on elastomeric bearing pads at end piers. Semi integral cantilever abutments are used for precast bridges less than 450 ft (137 m), and L abutments for longer bridges are typically used for precast girder bridges. Bridge ends are free for longitudinal movement, but restrained for transverse seismic movement by girder stops. The bearings are designed to be accessible so that the superstructure can be jacked up to replace the bearings after a major seismic event. Fig. 5 shows the semi integral abutment and L-abutment. Fig. 6 shows the girder stop at end piers to resist transverse seismic loads.

In L-shape end piers, 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 4:

$$N = (8 + 0.02L + 0.08H) (1 + 0.000125 S^2) \quad (4)$$

where:

N	=	minimum support length, in
L	=	bridge length to the adjacent expansion joint, or to the end of the bridge, ft
H	=	average height of abutment wall supporting the superstructure, ft
S	=	skew angle of the support measured normal to span, deg

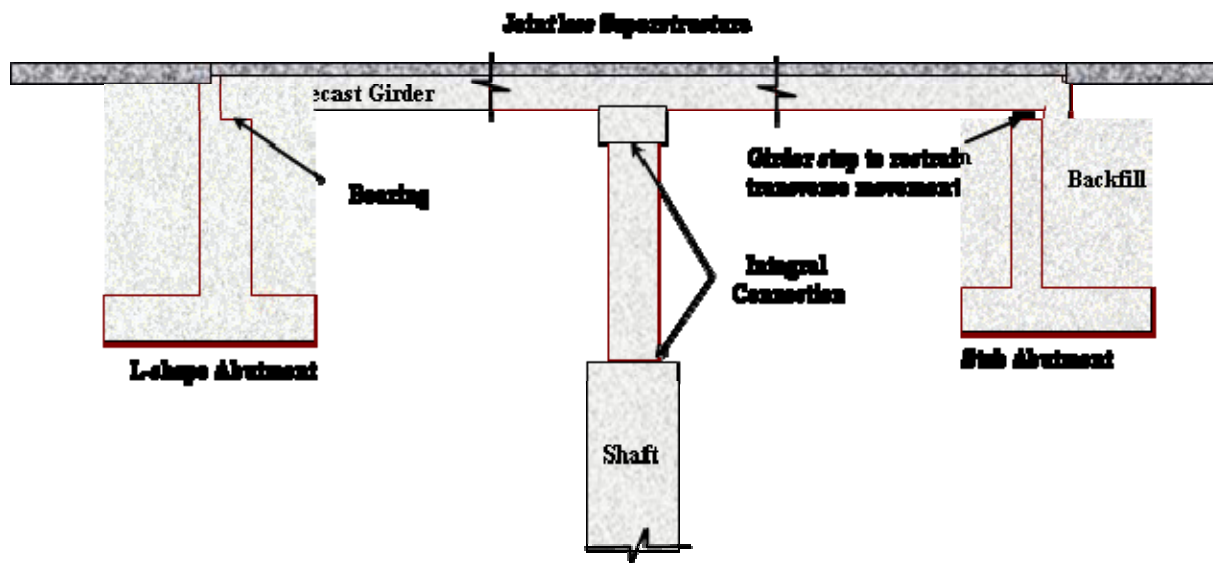
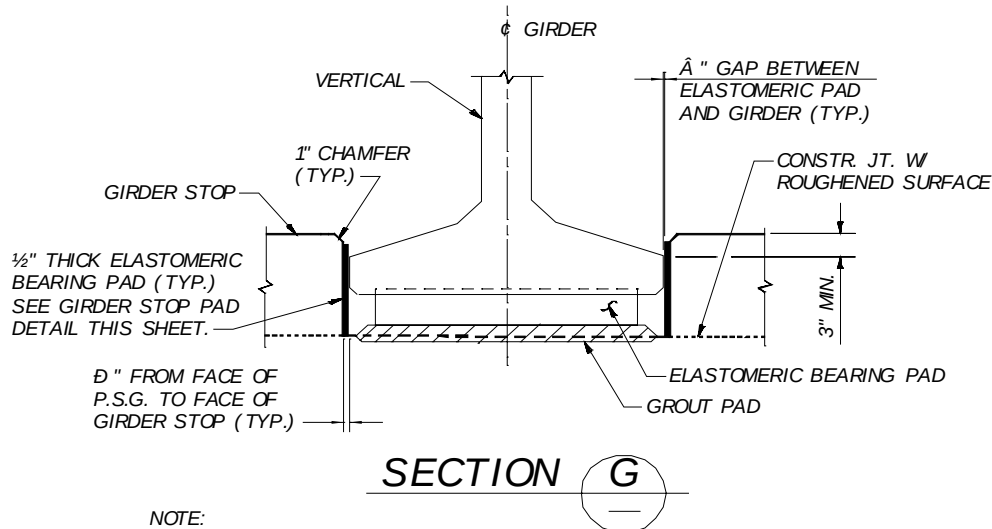


FIG. 5 SHOWS THE SEMI INTEGRAL ABUTMENT AND L-ABUTMENT



NOTE:

1. GIRDER STOPS SHALL BE CONSTRUCTED AFTER PLACEMENT OF PRESTRESSED GIRDERS.
2. ELASTOMERIC PADS BETWEEN GIRDER AND GIRDER STOPS SHALL BE PLACED AFTER CONSTRUCTING THE GIRDER STOPS. THE PADS SHALL BE CEMENTED TO GIRDER STOPS WITH APPROVED RUBBER CEMENT.

FIG. 6. GIRDER STOP AT END PIERS CONNECTION

Seismic Design Criteria

Acceptable seismic performance criteria for precast concrete bridge structures must satisfy both safety and economic criteria. Requiring all bridges to be serviceable immediately after an earthquake has not been economically feasible for many agencies. Still, preventing bridge collapse and possible loss of life can and must be achieved. According to the AASHTO Guide Specifications for LRFD Seismic Bridge Design², bridges should be designed for life safety performance objective considering a seismic hazard corresponding to a 7% probability of exceedance in 75 years. Higher levels of performance, such as the operational objective, may be deemed necessary by the bridge owner.

The current AASHTO LRFD Bridge Design Specifications¹ is a probability-based limit state code. Earthquake is categorized under the load combination referred to as “Extreme Event I”. Live load factor is adjustable according to the owner’s prerogative (NCHRP Report No. 489 recommends a value of 0.25; and WSDOT uses 0.5 for all bridges³).

The AASTO Guide Specifications for LRFD Seismic Bridge Design² require the design event to that having a return period of 1000 years. The site class definitions, site factors, and response spectra are revised accordingly. The analysis and design procedure remains unchanged except for the following:

- Eccentric axial load (“P-Δ”) effects on columns must now be kept less than 25% of the factored resistance.

- Design of longitudinal column steel must be between 1 and 6% of the gross cross-section area in Zone 2, and between 1 and 4% in Zones 3 and 4.
- The resistance factor for column flexural design has been revised to a constant value of 0.9.

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. Both forced based and displacement analysis methods are applicable to precast bridges with monolithic connections.

Force-based analysis 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 ratios are then calculated. A member has adequate capacity if its ratio is less than a prescribed force reduction factor, R .

Displacement-based analysis is an inelastic static analysis using expected material properties of modeled members. 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. These criteria are intended to achieve a “No Collapse” condition for standard ordinary bridges using one level of Seismic Safety Evaluation. 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.

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.
3. Calculate the moment-curvature diagram for each column and from that, the elastic and 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. 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.

Precast Column

Precast columns meeting the code prescribed seismic requirements have occasionally been used in WSDOT bridges for accelerated construction. WSDOT requires monolithic connections at the top and bottom of the column. Reinforcing bars from the top and bottom of the column shall extend into the cast-in-place concrete of the crossbeam and footing. Fig. 7 shows the recent application of precast columns in bridge construction. In both cases, the precast columns were kept in place on a temporary support until casting of foundation concrete. A cast-in-place crossbeam was then provided to support precast prestressed girder superstructure. The monolithic connection between precast column and precast girder was designed and detailed to meet the top of the column plastic hinging forces at centroid of superstructure.



FIG. 6: PRECAST COLUMNS ON SPREAD FOOTING

Seismic Performance of Precast Concrete Bent Caps

The overall objective of the work is to develop bridge structural systems that will lead to both rapid construction and good seismic resistance. Early in the program, the bridge bents, rather than deck systems, were identified as key subsystems on which effort was to be concentrated. An experimental research program at the University of Washington has developed and evaluated details for a precast concrete bridge bent substructure system having satisfactory seismic performance and suitability for rapid construction. The objective of this research is to examine two design procedures for precast concrete piers:

- 1- An equivalent lateral force design procedure.
- 2- Direct displacement-based design procedure.

Details of the cap beam-column connection consist of six #18 vertical column steel bars grouted into eight inch diameter corrugated metal ducts embedded in the cap beam as shown in Fig. 8. Precast concrete columns with six bars protruding are brought onto site,

braced, and then cast integrally with their footing. Later, the precast cap beam is fitted over the column bars through the corrugated ducts and grouted in place, completing the bent substructure. The small number of bars and the generous tolerances in the connection lead to good constructability, but the structural integrity of the connection depends on the anchorage of the bars in the ducts.

Full scale monotonic pull-out tests, with different embedment lengths, were first conducted to investigate the bond characteristics of large bars grouted into corrugated ducts. These tests confirmed that the #18 bars could be developed in the depth of the cap beam.



FIG. 8. PRECAST PIER RESEARCH PROJECT

Two one-third scaled connections, one with fully bonded vertical bars in ducts and another debonded eight bar diameters in the cap beam, were tested under 10% axial load and were subject to cyclic lateral displacements to study their performance. Both specimens performed well to 7% drift, failing as a result of bar buckling and fracture in the hinge region. Less damage to the cap beam was observed in the debonded specimen than the bonded, which saw moderate spalling around the underside of the beam as a result of duct slip. However, both demonstrated satisfactory strength and ductility, while allowing easy and rapid erection and generous construction tolerances.

Precast Bent cap Project

The above laboratory experiment was applied to a three span prestressed precast concrete bridge in high seismic zone of western Washington. The project increases mobility

and safety within the growing metropolis. This project is the first application by the highway owner that uses precast concrete for bridge girder support crossbeams. Based on the project success, the owner anticipates incorporating this method as an available practice for future designs. The bridge site is an extremely congested urban area with high visibility from the traveling public and high scrutiny from associated municipalities. To open the bridge as quickly as possible the contractor proposed precasting intermediate pier bent caps crossbeams in lieu of the cast-in-place requirements in the contract plans. This change would save the owner and the contractor several weeks on the contract duration.



FIG. 9: PRECAST BENT CAP UNDER CONSTRUCTION IN WASHINGTON STATE

The bridge uses wide flange WF74G girders to span a wetland a railroad right of way and an urban arterial. Precast concrete girders were the best choice for the superstructure. They are durable and have low maintenance and lifecycle costs. Precasting the girders increases the public's safety and convenience during construction by minimizing road closures and eliminating falsework over traveled lanes. The substructure cross beam was precast in order to save construction time. The use of precast concrete made duplicating the cast-in-place design feasible. As shown in Fig. 9, the 14 #14 column bars went through the 4" (100 mm) galvanized steel ducts placed in the precast bent cap using a template.

Precast Seismic Resistance Bridge

A conceptual design and detailing for a precast bridge is shown in Fig. 10. 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. The difference in rebar cage diameter shall be at least 12 in (305 mm), and the column support width for precast bent shall be at least 6 in (150 mm). The reduced rebar cage diameter on top of the column requires a higher percentage of longitudinal reinforcement to meet seismic loading requirement. The proposed sequence of construction for completion of precast bridge system is as follows:

1. Cast foundation to the construction joint.
2. Position precast column in place on a temporary support and provide bracing.
3. Cast concrete at column to shaft connection.
4. Place elastomeric bearings on top of columns.
5. Place precast bent cap or precast shell on the top of the column.
6. Cast concrete to achieve monolithic column to bent cap connection.
7. Place precast girders with the adequate number of extended strands.
8. Cast lower pier diaphragm and intermediate diaphragms to ensure girder stability.
9. Place precast deck panels if applicable
10. Cast and cure deck slab concrete.
11. Complete casting pier diaphragm concrete.
12. Cast traffic barriers and sidewalk if applicable.

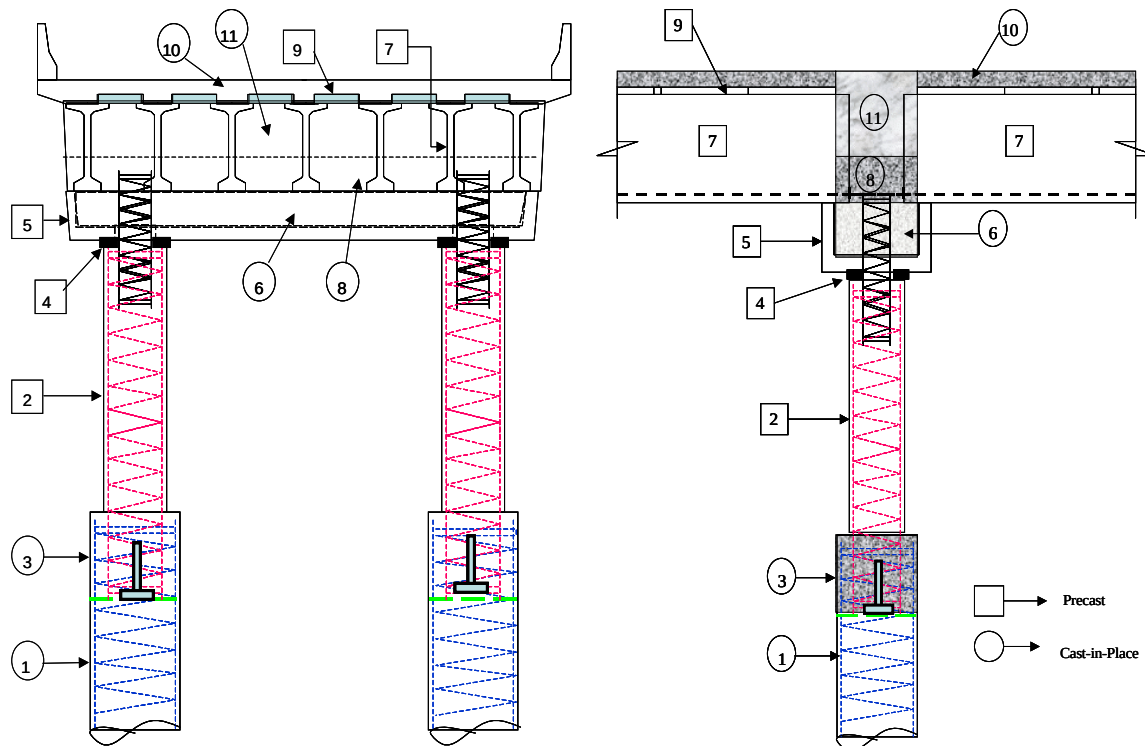


FIG. 10. PRECAST SEISMIC RESISTANT BRIDGE

Conclusions

Precast prestressed concrete bridge systems are an economical and effective for rapid bridge construction. Precasting eliminates traffic disruptions during bridge construction while maintaining quality and long-term performance.

Precast bridges with monolithic connections meeting the AASHTO LRFD seismic design requirements could safely be used in seismic zones. 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 use of precast bent caps results in cost savings by eliminating the need for elevated falsework and its foundation. It also improves on workers safety as rebars and concrete can be placed at the ground level.

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