PRECAST BRIDGE SUBSTRUCTURES IN AREAS OF HIGH OR MODERATE SEISMISITY

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ABSTRACT

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. Precast bridges consisting of pretensioned girders, post-tensioned spliced girders, trapezoidal open box girders, and other types of superstructure members are often used for accelerated bridge construction; however, bridge engineers are concerned with the durability and performance of bridges made of precast members in areas of high or moderate seismisity.

This paper examines the applicability of the AASHTO LRFD Specifications to precast prefabricated bridges in areas of high or moderate seismisity, discusses the different seismic design methodologies, and provides guidance in their application to precast bridges. It provides an overview of WSDOT design criteria and recent research and bridge projects using the accelerated bridge construction technique in Washington State.

Keywords: Bridge, Concrete, Precast, Seismic, LRFD, Accelerated Construction, Design

INTRODUCTION

Precast concrete bridge systems provide effective and economical design solutions for new bridge construction as well as for the rehabilitation of existing bridges. The 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. In seismic regions, provisions must be made to transfer greater forces through connections and to ensure ductile behavior in both longitudinal and transverse directions.

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.

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. Since then the Washington State Department of Transportation (WSDOT) has developed standard girders for composite and non-composite sections to facilitate economical design and construction. The complete description of standard prestressed girders and their span capability is presented in WSDOT Bridge Design Manual (BDM)[1], and can be downloaded from the WSDOT website at: http://www.wsdot.wa.gov/eesc/bridge/index.cfm. Both AASHTO LRFD Bridge Design Specifications[2] and BDM are used for the design of prestressed girders.

In 1997, long span deep prestressed girders[3] in both pretensioned and post-tensioned spliced-girders were added to the WSDOT inventory. In 2001, a newly developed pretensioned trapezoidal tub girder, commonly called "bath-tubs", was adopted. In 2004 wide flange pretensioned I-girders[4] were added.

The cross sections of deep prestressed girders and pretensioned trapezoidal tub girders used for composite superstructures are shown in Fig. 1. The span capabilities for these types of girders are presented in reference 1.

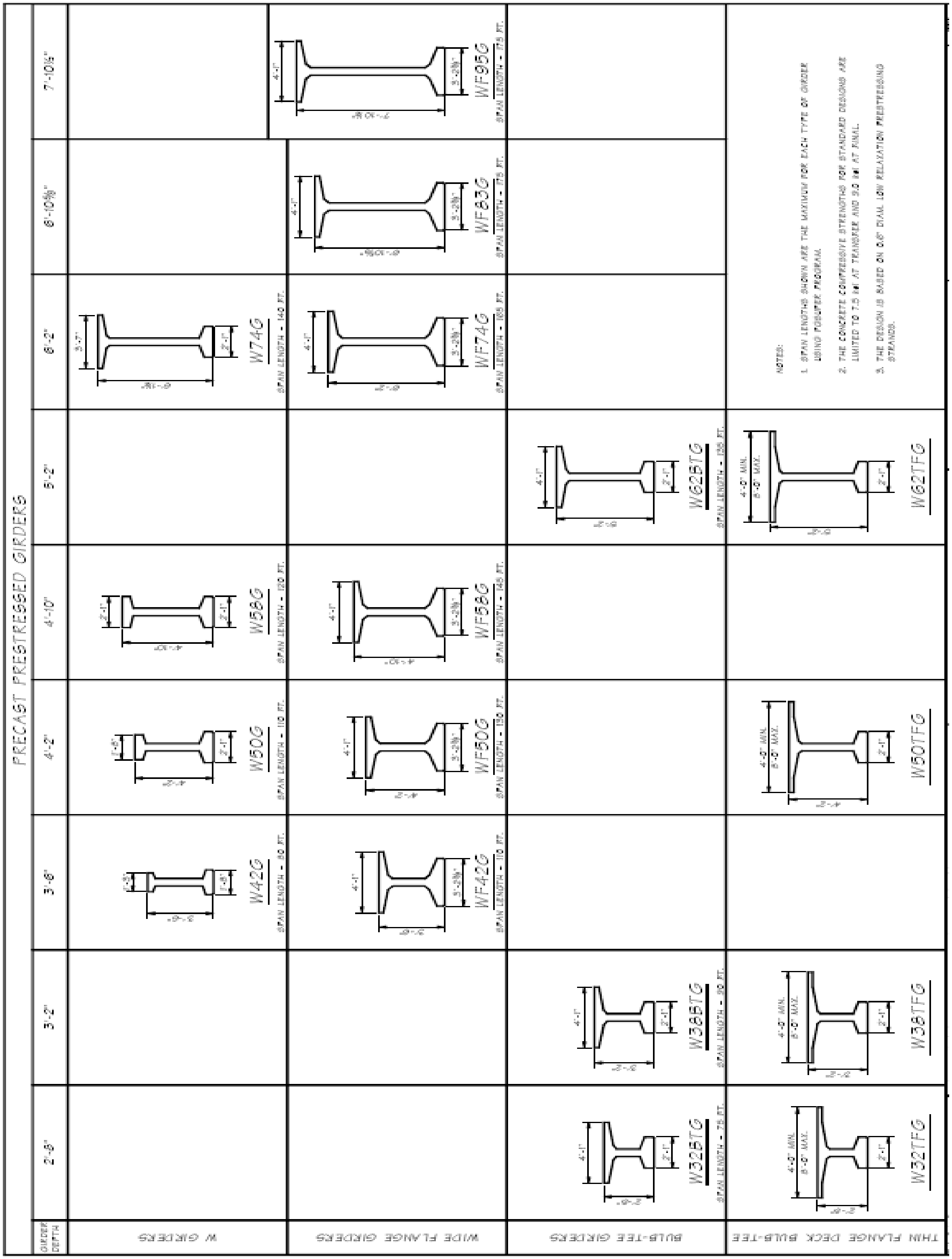

Fig. 1 WSDOT Precast Pretensioned Girders

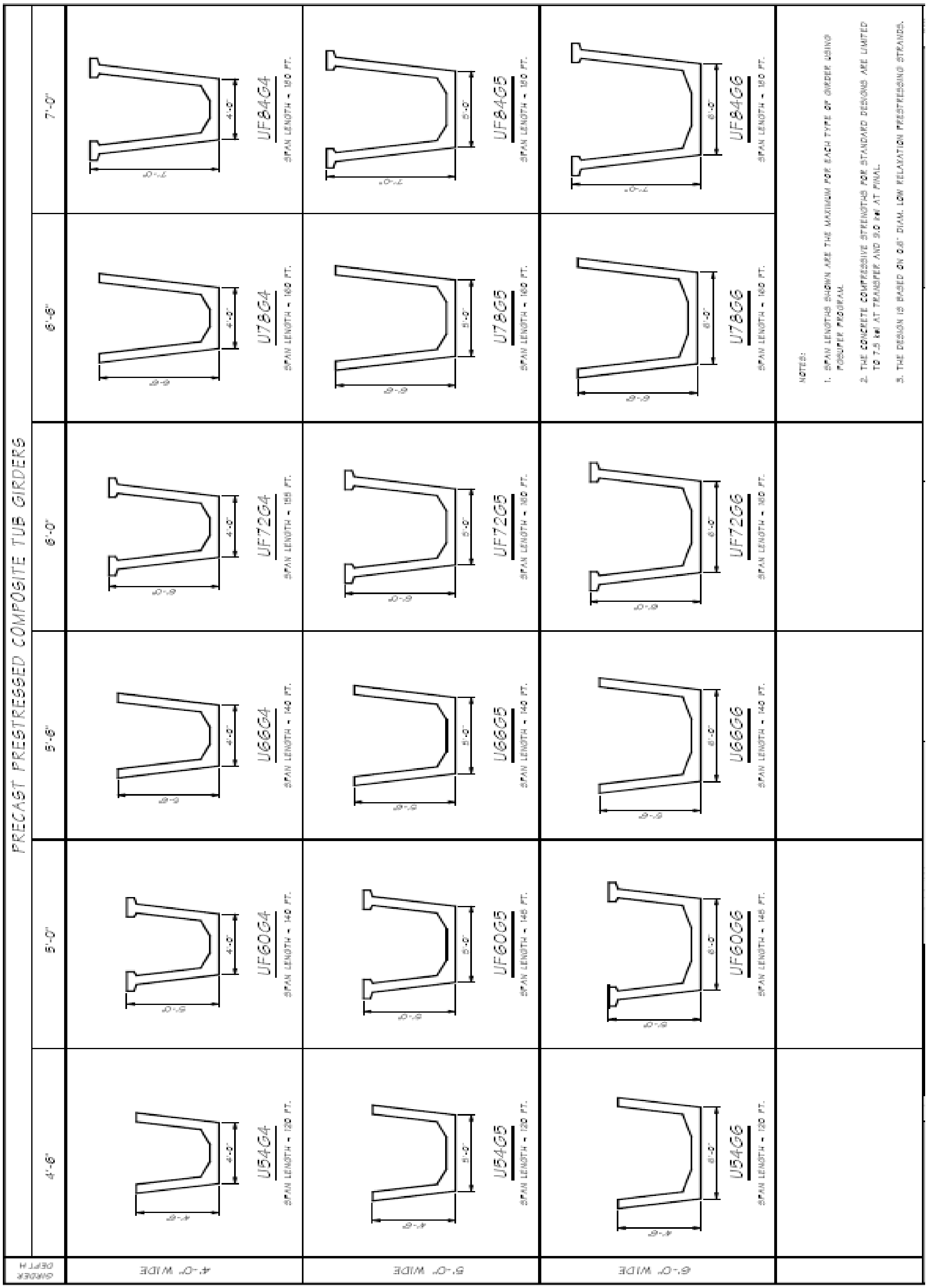

Fig. 2 WSDOT Precast Prestressed Trapezoidal Tub Girders

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 bottom connection details.
4. Inadequate transverse support or transverse stop mechanism at supports.
5. Pile to pile cap connection failure.

SEISMIC RESPONSE OF BRIDGES WITH PRECAST COMPONENTS

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.

CONNECTION OF PRECAST GIRDERS AT INTERMEDIATE PIERS

The most common types of connections for precast prestressed girder bridges are fixed connection for high seismic zones (western Washington), and hinge connection for low seismic zones (eastern Washington). Precast column could be used if monolithic moment resistant connections meeting seismic design and detailing requirements are provided.

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 seismisity. Developing a moment connection between the superstructure and substructure reduces the moment. Fig. 3 shows a typical monolithic moment resistant connection used for WSDOT precast girder bridges.

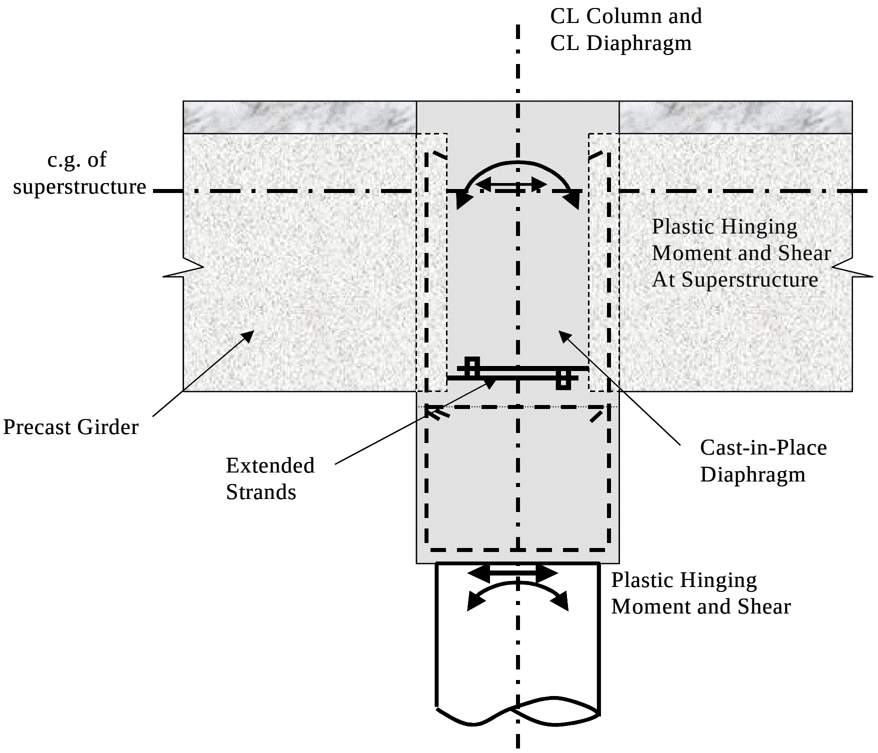

Fig. 3 Typical monolithic moment resistant

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. Design girders as simple span for both 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.
6. Design diaphragm reinforcement to resist the resultant seismic forces at centroid of diaphragm.
7. Design longitudinal reinforcement at girder ends for interface shear friction.

STRAND FOR POSITIVE EQ MOMENT

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 device used, per WSDOT Standard Plan, is a 2'-0" 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.

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:

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

where:

M^{top}_{po} = plastic overstrength moment at top of column, kip-ft.
M^{Base}_{po} = 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.

This moment is resisted by the bent cap through torsion forces. The torsion in the bent cap is distributed into the superstructure based on the relative flexibility of the superstructure and the bent cap.

Hence, 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. It has been suggested that for concrete bridges, with the exception of box girders and solid superstructure, this effective width should be calculated as follows:

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

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 Jolla[6], 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 (encompassed by the effective width) and the other one-third to be resisted by the non-adjacent girders. The effective width, the moment per girder line is calculated as follows:

Adjacent girders (encompassed by the effective width):

$$M_{sei}^{Int} = \frac{2M_{po}^{CG}}{3N_g^{\text{int}}}$$

Non-adjacent girders:

$$M_{sei}^{Ext} = \frac{M_{po}^{CG}}{3N_g^{ext}}$$

Seismic Moment:

If $M_{sei}^{Int} \geq M_{sei}^{Ext}$ then $M_{sei} = M_{sei}^{Int}$

if $M_{sei}^{Int} < M_{sei}^{Ext}$ then $M_{sei} = \frac{M_{po}^{CG}}{N_g^{\text{int}} + N_g^{ext}}$

where:

N^{int}_g = Number of girder encompassed by the effective width.
N^{ent}_g = 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.

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

where:

A_{ps} = area of each extended strand, in^2
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 use maximum of (K1 and K2)
ϕ = flexural resistance factor

Assuming EI is constant and Girders have fixed-fixed supports for both spans.

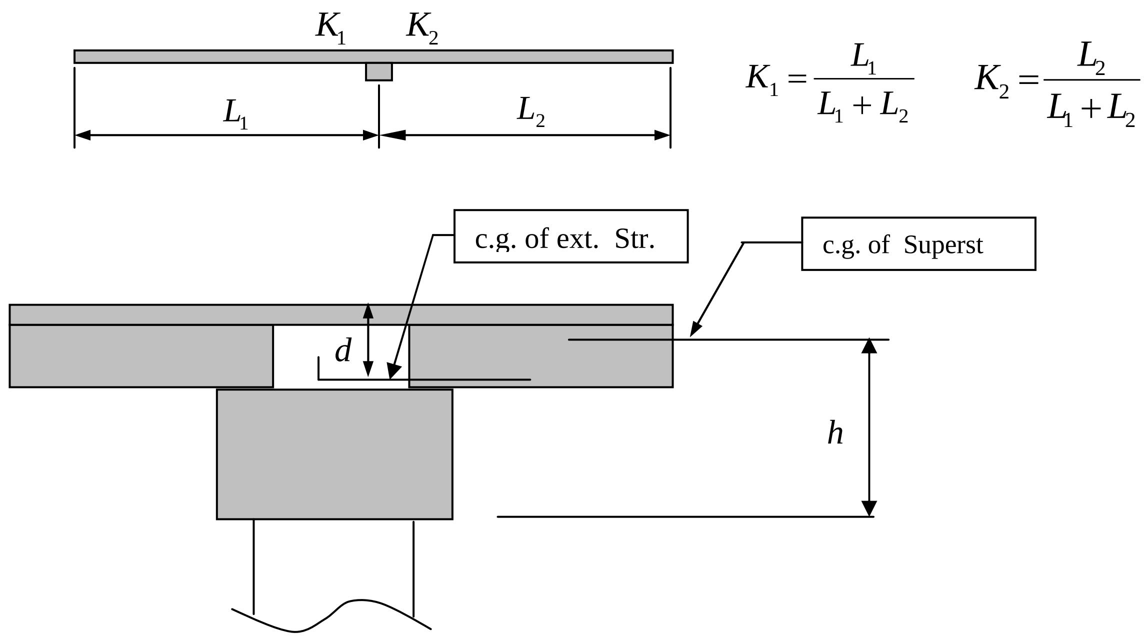

Fig. 4 Extended Strand Design parameters

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 shorter bridges, 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.

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:

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

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 girder stop at end piers to resist transverse seismic loads.

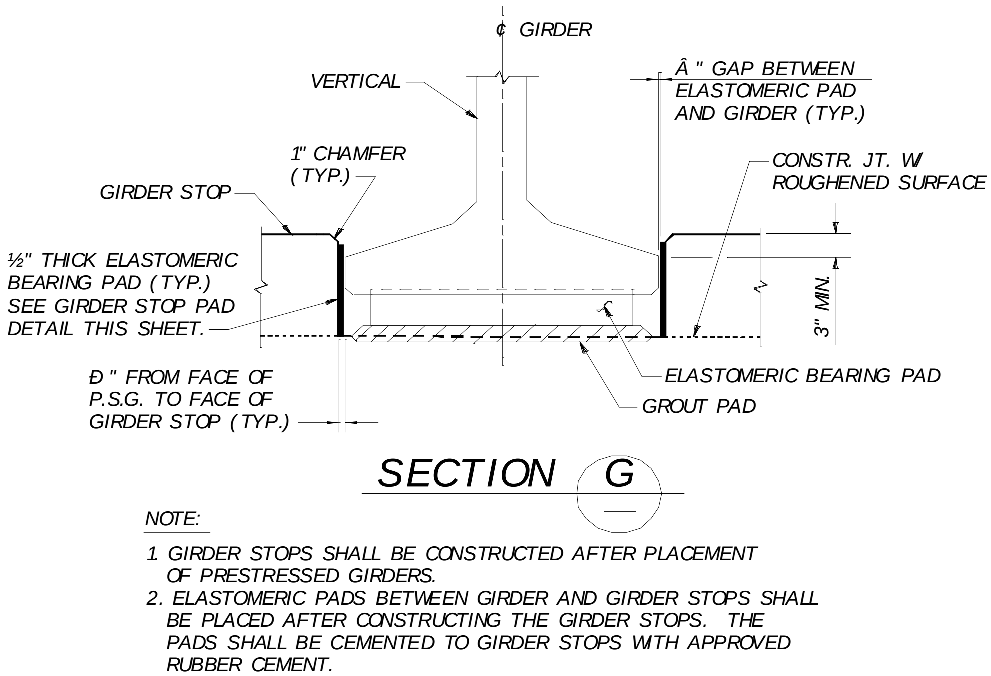

Fig. 5. Girder Stop at End Piers connection

SEISMIC DESIGN CRITERIA

The current AASHTO LRFD Bridge Design Specifications 4th edition 2007 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 bridges (Ref. 1). The seismic load is factored by 1.0. The 4th edition of the AASHTO LRFD Bridge Design Specifications includes:

- Separate soil profile site coefficients and seismic response coefficients (response spectra) for soft soil conditions.
- Three levels of importance “critical”, “essential” and “other” as opposed to two defined in previous AASHTO provisions. The R factors are adjusted accordingly.

The new AASHTO Guide Specification for LRFD Seismic Bridge Design is intended to be an improvement to seismic design of bridges. The document is applicable to conventional slab, beam, girder and box girder superstructure construction with spans not exceeding 500 ft.

The LRFD Guidelines revise 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. The forced based analysis method is 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.

RESEARCH PROJECT ON PRECAST CONCRETE PIERS IN SEISMIC REGIONS

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. 9. 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.

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.

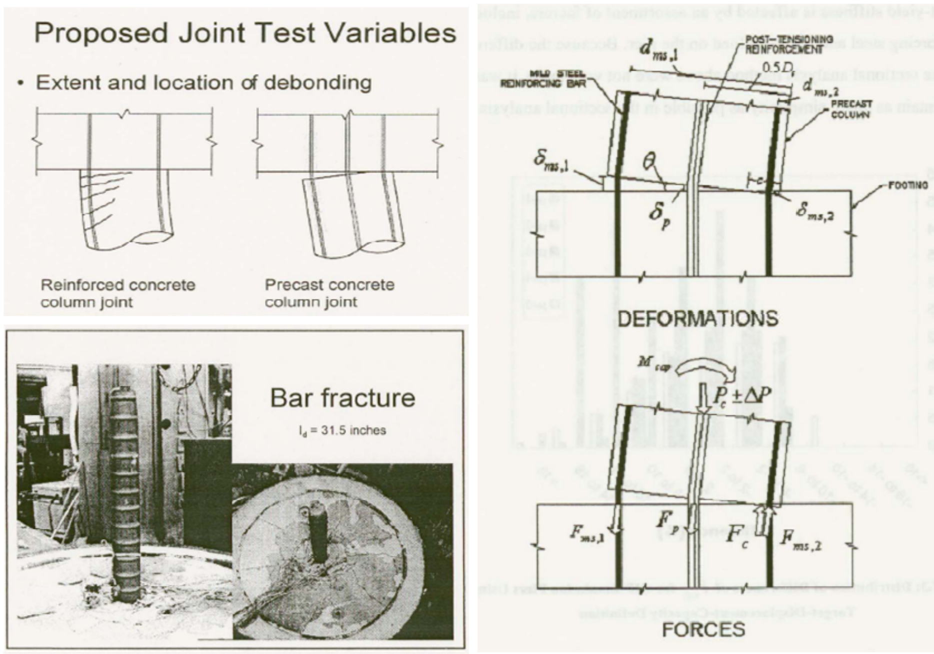

Fig. 6. Precast Pier Research Project

PRECAST BENT CAP PROJECT

The SR 202 Bridge flyover structure, located in high seismic zone of western Washington, is a three span prestressed precast concrete bridge in a major urban area. The project increases mobility and safety within the growing metropolis. Bridge description is given in Appendix A. 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 uses wide flange 74 inch deep 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 construction time savings were significant. The use of precast concrete made duplicating the cast-in-place design feasible.

Rapid Construction with Precast Crossbeam

The bridge site is an extremely congested urban area with high visibility from the traveling public and high scrutiny from associated municipalities. Involvement of these same entities created a desire to open the flyover bridge as quickly as possible. With this in mind, the project contractor proposed a change to the contract that consisted of precasting both intermediate pier 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.

Using ongoing research at a nearby university, the contractor developed a construction method to meet stringent owner requirements for erection and seismic loadings. Erection was made challenging due to the crossbeam size of 45ft length, 7ft width and 2ft to 5ft height with total weight of 180,000lbs. Seismic detailing led to increased emphasis on quality of the connection between the precast (crossbeam) and cast-in-place (column) elements. This connection used steel ducts that were cast into the crossbeam to provide openings into which the column longitudinal reinforcement could be threaded during erection. The annular space around the reinforcement in the ducts was filled with grout to provide a structural connection.

Bridge Architecture

The bridge substructure is designed to compliment the adjacent existing bridges. The new bridge has round columns, although the existing bridges have pier walls with rounded ends. This cost savings of the round columns on single shafts in lieu of pier walls outweighed the visual requirements to exactly match the new with the existing. The pier cap has classic pointed ‘darts’ at the columns and a raised section mid span. Bridge engineers detailed the column construction joint to be inside the crossbeam, which simplified construction. The adjacent urban fabric requires architectural detailing of the bridge including local artist designed wall finishes. Computer rendering of precast crossbeam is shown in Fig. 7.

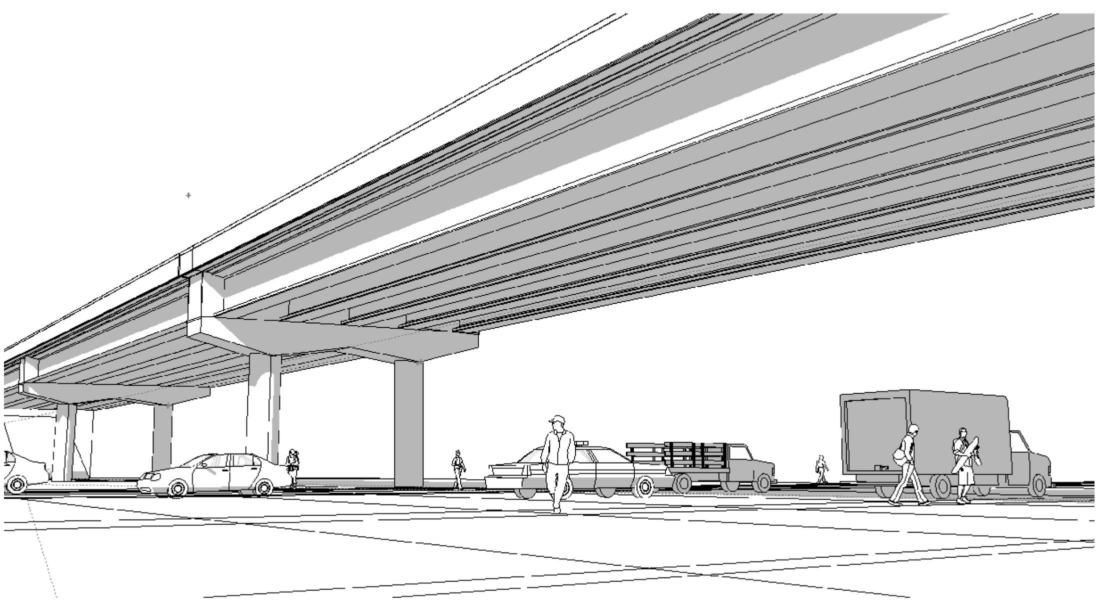

Fig. 7. Computer Rendering of Precast Crossbeam

Precast bent cap systems eliminate the need for forming, reinforcement, casting, and curing of concrete on the jobsite removing the bent cap construction from the critical path. Fig. 8 shows a precast bent cap under construction in Washington State. The #14 column vertical reinforcement will be placed through sleeves installed in the precast bent cap. Sleeves are made of 4 in. diameter corrugated galvanized metal ducts allowing adequate construction tolerance and room for grouting.

Fig. 8: Precast Bent Cap under Construction in Washington State

PRECAST SEISMIC RESISTANCE BRIDGE

A conceptual design and detailing for a precast bridge is shown in Fig. 9. 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. and the column support width for precast bent shall be at least 6 in. The reduced rebar cage diameter on top of the column requires a higher percentage of longitudinal reinforcement to meet seismic loading requirement. The plastic hinging moment at top of column with reduced rebar cage diameter will be approximately the same as the bottom of the column. This may be achieved by designing the column for minimum reinforcement.

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 and provide bracing.
3. Cast concrete at column to shaft connection.
4. Place elastomeric bearings on top of columns.
5. Place precast bent cap 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 deck panels, cast and cure deck slab concrete.
10. Cast deck slab concrete.
11. Complete casting pier diaphragm concrete.
12. Cast traffic barriers and sidewalk if applicable.

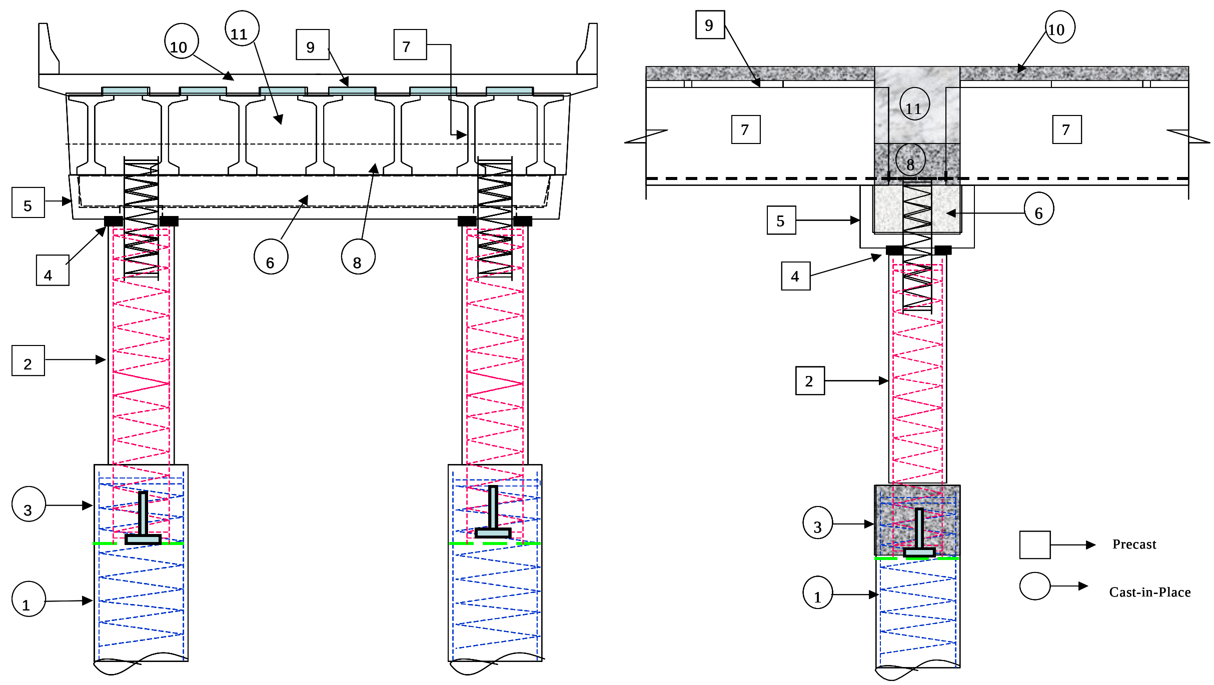

Fig. 9. Precast Seismic Resistant Bridge – Elevation (Left) – Section (Right)

The top of column reinforcement should be sufficient to produce the same plastic moment capacity as the bottom of column. This matches the original column design if the column section on the top was not reduced; however, the designer may choose to specify less rebar for the upper column cage and design it accordingly. In this case the top of column will tend to act with less fixity resulting in more forces transferred to the bottom of column. Although, this may be desirable to reduce rebar congestion in top of the column, it may however, require larger foundation.

CONCLUSION

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 and detailing 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.

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Appendix A:

Precast Bridge Description

1. Type of bridge or structure: Flyover Ramp over State Highway

2. Geometry of the bridge or structure as applicable:
   a. Overall length: 435 ft
   b. Overall width: 48.0 ft (roadway width varies from 46'-0" to 45'-0")
   c. Total area(s) of deck :
      - Bridge Deck = 20518 ft^2
   d. Number of spans and their lengths:
      - Span 1 = 145'
      - Span 2 = 145'
      - Span 3 = 145'
   e. Skew(s) of the support(s) measured from a line normal to the bridge centerline:
      - All piers are normal to bridge
   f. Radius of horizontal curve: Not Applicable
   g. Predominant grade in percent: (grade varies along length of bridge due to vertical curve profile)
      - +3.3% at Pier 1 abutment
      - -6.9% at Pier 4 abutment
   h. Number of lanes: 2

3. A list of all precast components used with quantities and dimensions if practical:
   There are five precast, prestressed WA state standard WF74G girders for each span. The girders were each constructed and delivered in one piece with no field splices. The girders are 6'-2" tall. Girder weights are computed using a concrete unit weight of 0.160 $kips/ft^3$.
      - Span 1: Girder Length = 141'-8.0" / Girder, Weight = 144.5 kips each
      - Span 2: Girder Length = 140'-0.0" / Girder, Weight = 142.8 kips each
      - Span 3: Girder Length = 141'-8.0" / Girder, Weight = 144.5 kips each

   There are 2 precast crossbeams, one at each intermediate pier. Each precast crossbeam utilizes 42 CY of concrete.

4. Cost information:
   a. Total cost of the project:
      March 2007 low bid by Tri-State Construction = $9,988,000
   b. Total cost of the bridge including costs per square foot:
   - Superstructure (including barrier, fence, and roof) = $1,300,000 or $64/SF
   - Substructure = $2,414,000 or $117/SF
   - Total Bridge = $3,714,000 or $181/SF

   Costs per square foot above are based on the areas listed below:

- Bridge = 20518 ft^2

5. Schedule information
   a. Time in design: Need info from Ann Marie.
   b. Total time of construction:
      - Project awarded March 16, 2007
      - Construction started May 2007
      - Construction completed April 2008 (approximately 12 months)
   c. Total time of road closure: X nights for setting girders
   d. Date of completion: Opened on April 2008

6. Summary list of key design challenges (geography, geometry, aesthetics, cost, time, etc.):

- The bridge crossbeam shape is designed to compliment the adjacent existing bridge. The crossbeam has a classic "V" shape at the columns and a raised section mid span between the columns.
- Construction challenge of minimizing highway closures and eliminating falsework over the traveled lanes.

7. Summary of innovations or accomplishments using precast concrete and all other design accomplishments of note:
- The use of precast girders eliminated falsework over the traveled lanes. This is an important consideration as our state has had several high load hits on falsework in the past.
- The use of precast crossbeam have 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.