PRECAST CONCRETE SEISMIC RESISTANT BRIDGES IN WASHINGTON STATE

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ABSTRACT

Precasting bridge components are in increasing demand for accelerated bridge construction. Precasting eliminates the need for forming, casting and curing concrete in the work zones, makes bridge construction safer while improving quality and durability.

The majority of all concrete bridges in Washington State are precast, prestressed girder bridges with monolithic connections to meet the code prescribed ductility requirements for high seismicity. Design, constructability, and structural performance of connections are of great concern in seismic resistant bridges. This paper delineates design methodology, and discusses constructible details for seismic resistant precast bridges. Numerical applications are provided and the applicability of the AASHTO LRFD[1] Specifications is studied.

Keywords: Bridge, Precast Concrete, LRFD, Seismic, Ductility, Connections, Monolithic

INTRODUCTION

The relatively recent earthquakes in western states resulted in a major research project headed by the Applied Technology Council[2] (ATC). The outcome of this effort was the development of seismic design guidelines for bridges in 1982 (Report No. ATC-6) titled "Seismic Design Guidelines for Highway Bridges". These guidelines incorporate an elastic Response Spectrum Analysis (RSA), with factors to account for redundancy in the structure, and ductility of the structural components. These guidelines emphasize detailing for ductile behavior and prevention of collapse even after significant structural damage occurs.

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.

Development of a precast bridge construction system provides an effective and economical design concept, and can be implemented for new bridge construction as well as for the rehabilitation of existing bridges. Using precast components in construction shortens the time of bridge closures, and minimizes interference with traffic flow. The benefits of precast components in bridge construction enhance the philosophy of "get in, get out, stay out".

SEISMIC ANALYSIS AND DESIGN

There are two general approaches to evaluate the seismic response of a bridge. The first approach is the conventional force-based analysis and the second involves the use of a displacement criterion. In recent years, more emphasis has been placed on the displacement method. The current Washington State Department of Transportation (WSDOT) Bridge Design Manual[3] (BDM) requires force-based analysis for all ordinary structures. Displacement-based method may be used for major bridge projects in regions of high seismicity.

With the conventional force method[4], bridge analysis is performed and the forces on its various components are determined. The capacities of the components are evaluated and the demand to capacity ratios (D/C) is 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 components and connections.

With the displacement method[4], a more rational form of ductility assessment is investigated by taking the effect of sequential yielding into account when evaluating capacity. Capacity thus takes on a more global meaning since it refers to the entire structure rather than a given component, as in the force analysis. Displacement is taken as the measure of the capacity of the structure. Failure occurs when enough plastic hinges have formed to render the structure unstable or when a plastic hinge cannot sustain any further increase in rotation.

BRIDGE DESIGN SPECIFICATIONS

The provisions contained in 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.

Plastic Hinging is the basis of the ductile design for bridge structures. Plastic Hinges may be formed at one or both ends of a 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.

In a seismic event, it is essential to have plastic hinging occur in the column rather than the superstructure or footing. This is because plastic hinging is accompanied by a certain degree of damage in the form of inelastic displacements, cracked and spalled concrete and yielded reinforcement. Allowing such damage to occur in the superstructure near the ends of a span could reduce the load-carrying capacity of the superstructure, thereby increasing the likelihood of collapse. Damage to a footing or pile system is not easily detected and is extremely difficult to repair. Plastic hinging in the column can be quickly identified by inspection and sometimes repaired. More importantly, a properly confined column will continue to carry axial load and therefore structural collapse may be avoided.

The AASHTO LRFD Specifications[1] incorporate many of the seismic provisions of the 1992 Standard Specifications, but have updated them in light of new research developments. The principal areas where provisions were updated are:

1. The introduction of separate soil profile site coefficients and seismic response coefficients (response spectra) for soft soil conditions.
2. Definition of three levels of importance, namely critical, essential, and others. 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 response modification factors for bridge substructure and connections are specified in Table 1. These factors could safely be used for bridges made with precast components. The importance category for all typical WSDOT bridges is considered "others" unless otherwise instructed by the Bridge Engineer.

Table 1. Response Modification Factors for Concrete Bridges

Substructure	**Importance Category**		
	Critical	Essential	Other
Wall Type Piers	1.5	1.5	2.0
Single Column Bents	1.5	2.0	3.0
Multiple column Bents	1.5	3.5	5.0
Columns, Piers, or pile bents to cap beam or superstructure	1.0		
Columns or piers to foundation	1.0		

The AASHTO LRFD Extreme Event-I limit state includes the effect of seismic loading. A load factor of γ_{EQ} is specified for transient loads. A possibility of partial live load may be considered for bridges in urban area. The commentary in the AASHTO LRFD Specifications indicates that $\gamma_{EQ} = 0.5$ is reasonable for a wide range of bridges with high average daily truck traffic (ADTT). WSDOT uses $\gamma_{EQ} = 0.5$ for bridges located in urban areas.

SEISMIC RESISTANT PRECAST CONCRETE BRIDGES

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 demand at the base of the column.

The essence of a seismic resistant connection is to transfer the plastic moment demands at the top of the column into the superstructure without yielding either the connection itself or the girder ends. To achieve this, both the connection and the girder ends must be designed to provide a design strength exceeding the required strength from the forces transferred. The connection should also be detailed to ensure adequate distribution of the longitudinal moment from the top of the column to girders. Fig. 1 shows a typical monolithic moment resistant connection used for WSDOT precast girder bridges.

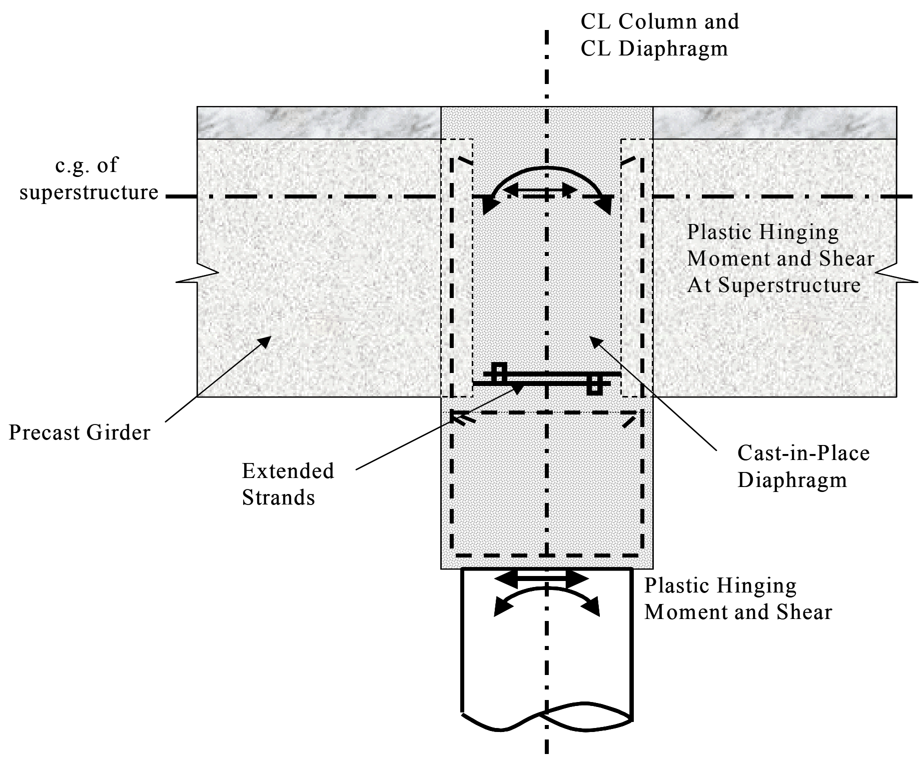

Fig. 1. Superstructure to Substructure Connection at Intermediate Pier

CURRENT WSDOT PRACTICE FOR SEISMIC RESISTANT PRECAST BRIDGES

Seismic design and detailing requirements vary from region to region, and also depends on the level of anticipated seismic activity. Integral monolithic moment resistant connections at intermediate piers are the key to seismic resistant bridges. However, integral superstructure-to-substructure connections at end piers may not be necessary to resist earthquake forces.

END PIER CONNECTION FOR PRECAST PRESTRESSED GIRDER BRIDGES

Precast end piers are occasionally used for smaller single span bridges. These types of bridges are often completed with precast slab or precast decked superstructures. The typical WSDOT practice for end piers in seismic zones is cast-in-place concrete supported on spread footing, pile, or shaft foundation. 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 are typically used for precast bridges.

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. Fig. 2 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.

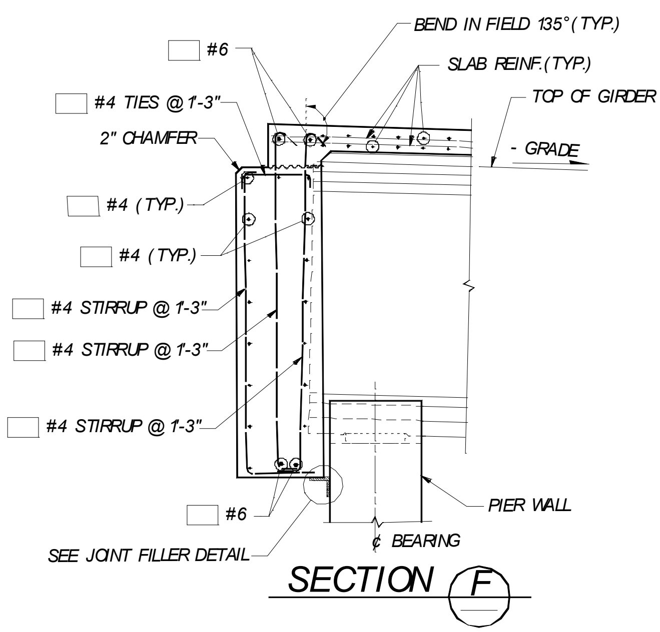

Fig. 2. Semi Integral End Pier Connection

Fig. 3 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.

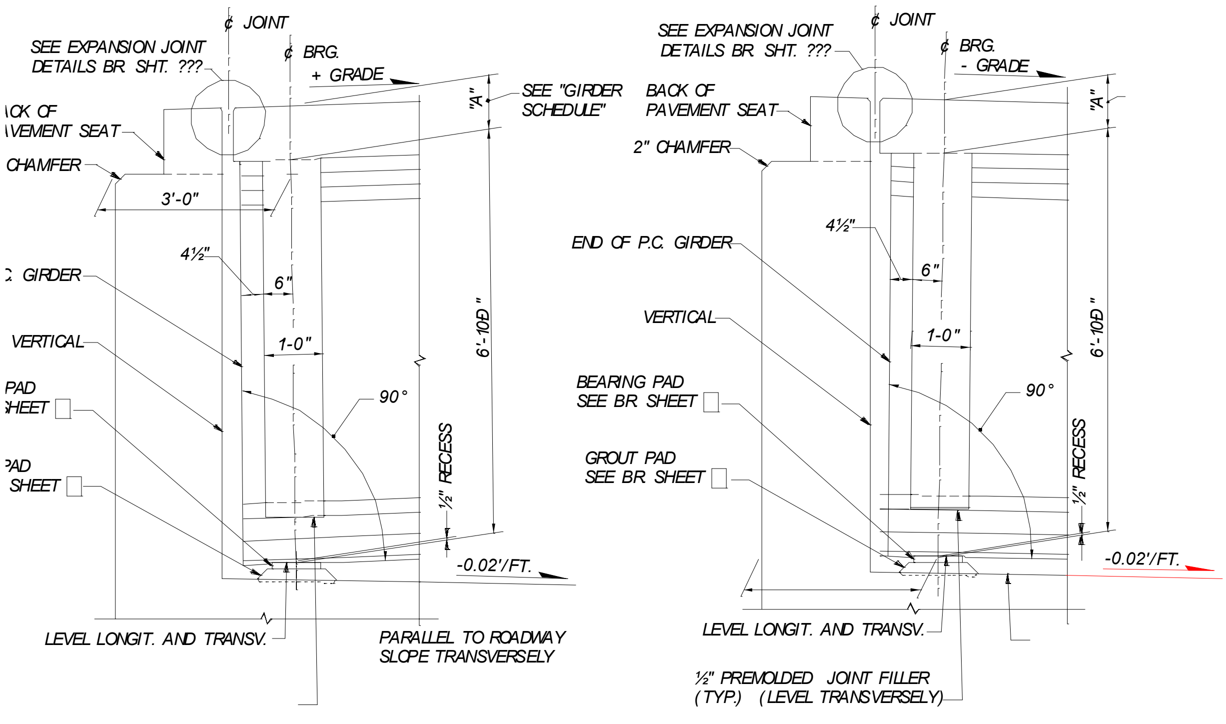

Fig. 3. L-Shape End Pier Connection

The girder stop detail is shown in Fig. 4. The elastomeric pads provided at the sides of girder stops prevent concrete-to-concrete impact during a seismic event while allowing bridge longitudinal movement under service conditions

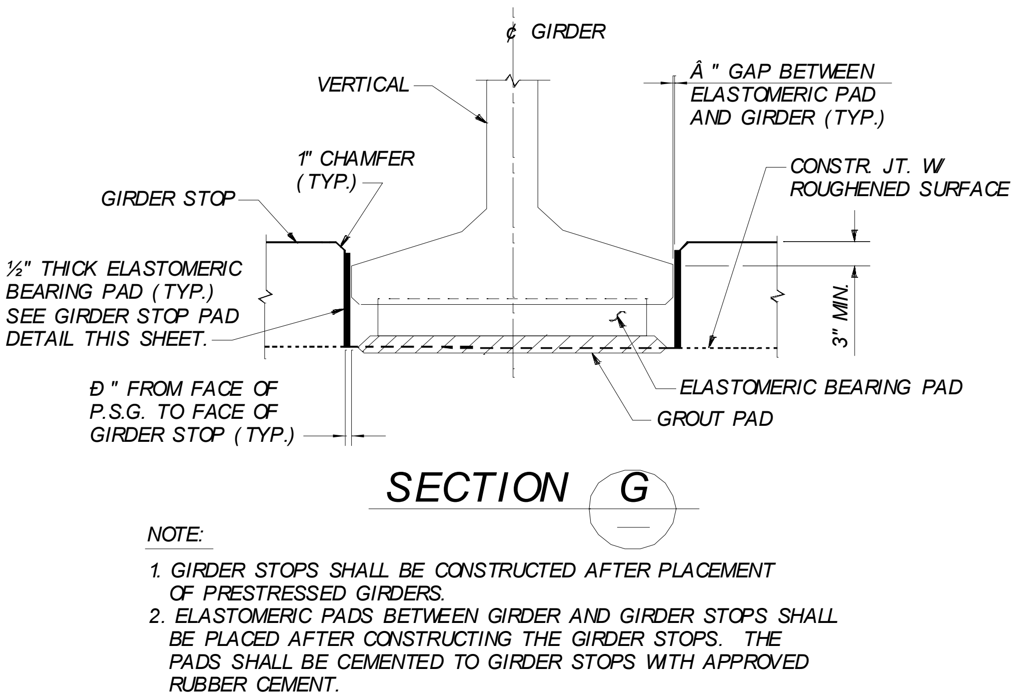

Fig. 4. Girder Stop at End Piers

In L-shape end pier, the minimum displacement requirements at the expansion bearing shall accommodate the greater of the maximum displacement calculated from the seismic analysis or a percentage of the empirical seat width, N, specified in Equation 1.

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

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

The empirical seat width will be increased by factors accounting for seismic zones as specified in table 2.

Table 2. Seat width increasing factors

Seismic Zone	Seat Width Increasing Factors
1	1.0
2	1.0
3	1.5
4	1.5

Most older bridges made with precast girders fail to meet the minimum seat width required in equation 1. WSDOT, as part of the seismic retrofit program, requires longitudinal restrainers to ensure superstructure survival in case of major seismic events. Restrainers are designed for a force calculated with the acceleration coefficient times the permanent load of the lighter of the two adjoining spans, or part of the structure.

INTERMEDIATE PIER CONNECTION FOR PRECAST PRESTRESSED GIRDER BRIDGES

The most common types of connections for precast prestressed girder bridges are fix connection for high seismic zones (western Washington), and hinge connection for low seismic zones (eastern Washington). In both cases the superstructure consists of a cast-in-place concrete deck on precast prestressed concrete girders made continuous at intermediate piers. Precast girders are temporarily supported on oak 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 cross beam for loading from the oak block including dead loads from girder, slab, and construction loads. Precast column could be used if monolithic moment resistant connections meeting seismic design and detailing requirements are provided.

The hinge connection shown in Fig. 5 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 type.
2. Design girders as simple spans 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. 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.

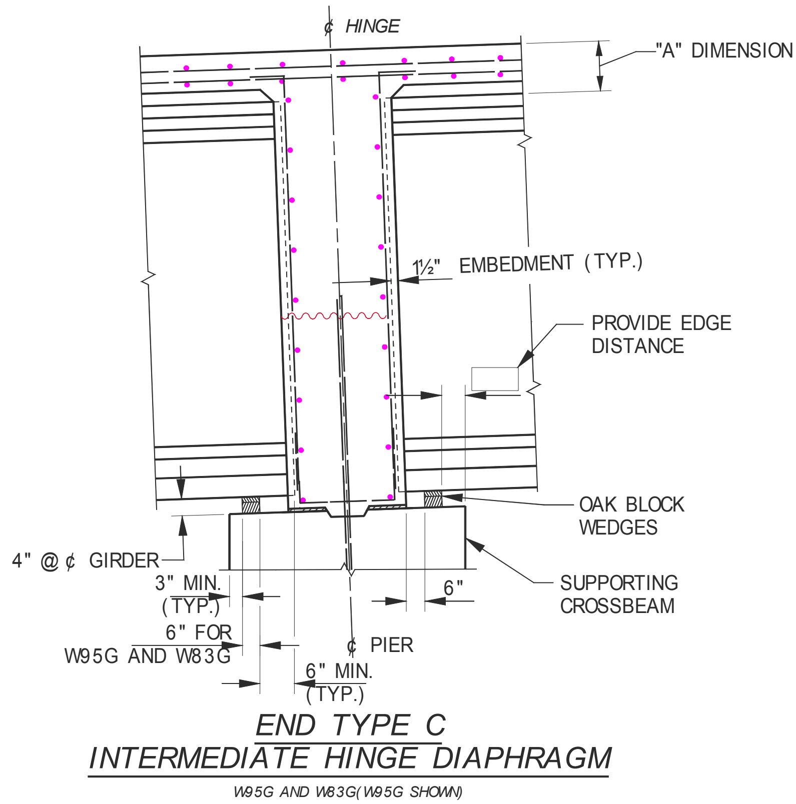

Fig. 5. Intermediate Hinge Diaphragm

The connection shown in Fig. 6 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 oak blocks on lower crossbeam. 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 shear friction.

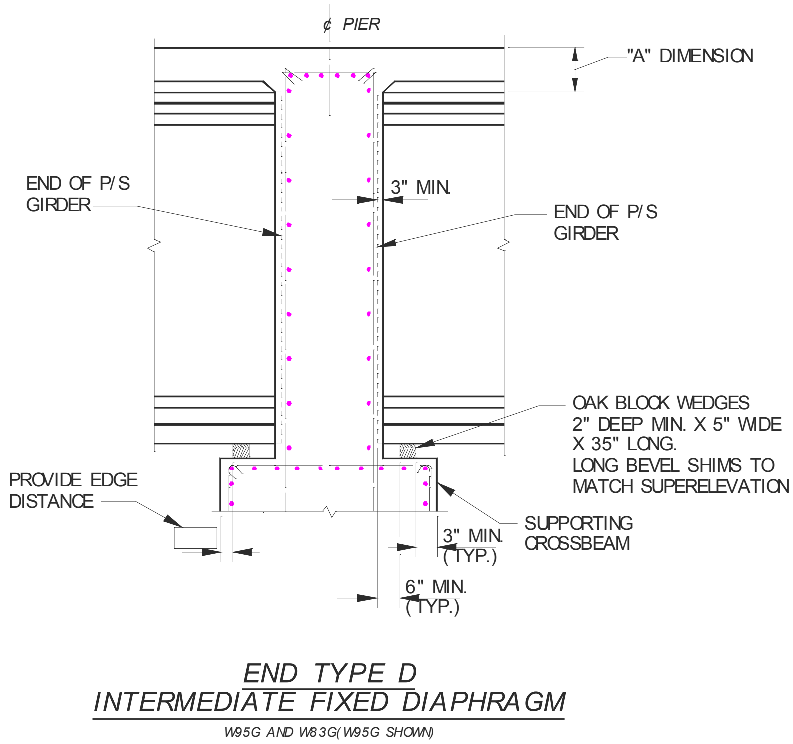

Fig. 6. Intermediate Fix Diaphragm

POSITIVE MOMENT CAPACITY AT INTERMEDIATE DIAPHRAGMS

Strand extension details shown in Fig. 7 for alternate1, Fig. 8 for alternate 2, and Fig. 9 for alternate 3. These details will be used for continuous spans at diaphragms, and are not applicable for continuous spans using hinge diaphragms. Alternates 1 and 2 are suitable for most common prestressed girder bridges. A minimum of 4 extended strands shall be provided regardless of design requirements.

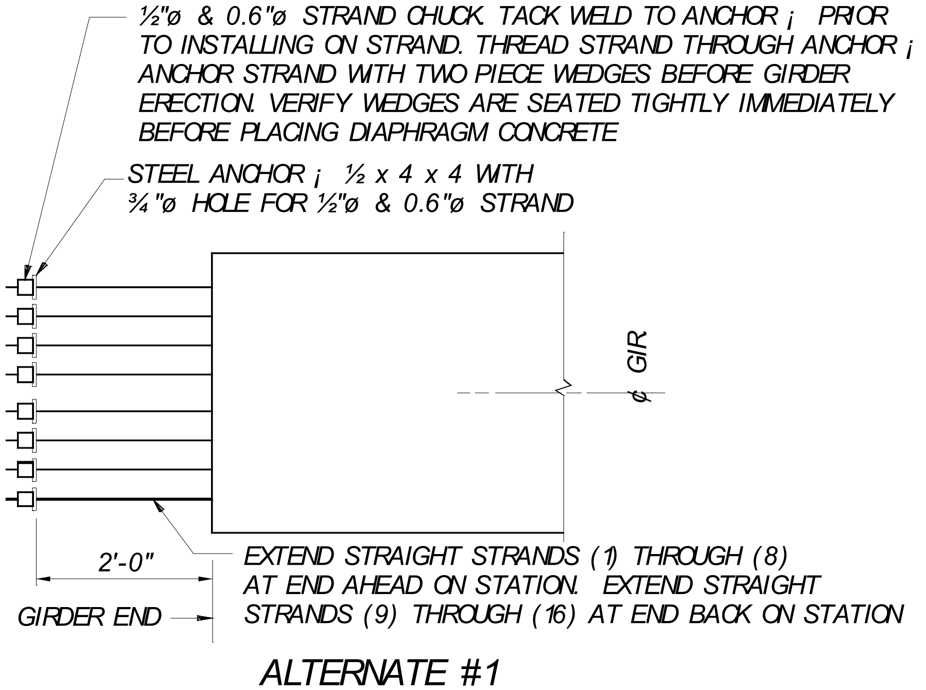

Fig. 7. Strand Extension Detail, Alternate 1

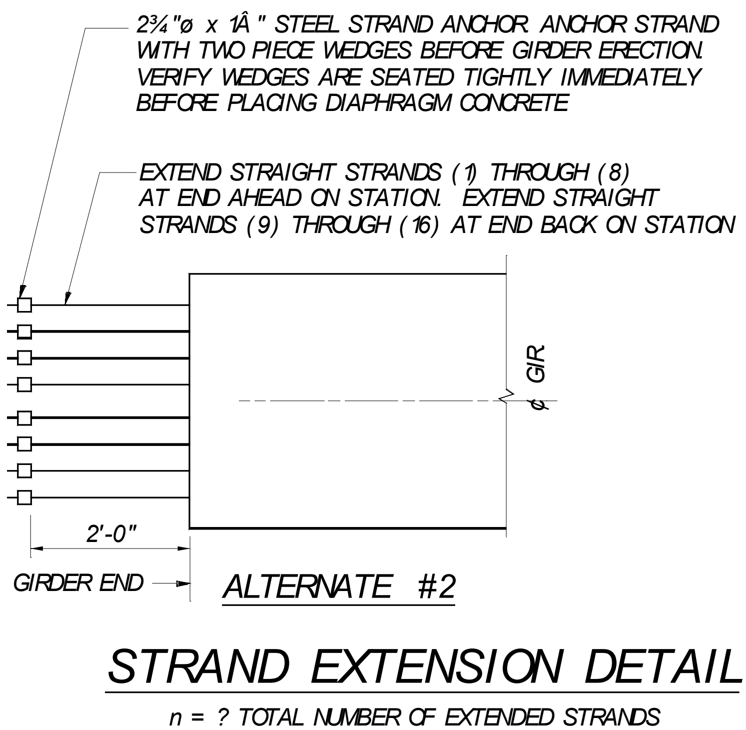

Fig. 8. Strand Extension Detail, Alternate 2

Alternate 3 is only used if additional strands beyond those shown in alternates 1 and 2 are required. This may happen when a large diameter column is supporting a shallow superstructure. Due to a shallow structure, resisting plastic hinging forces in this case requires more extended strands than those provided in alternates 1 and 2.

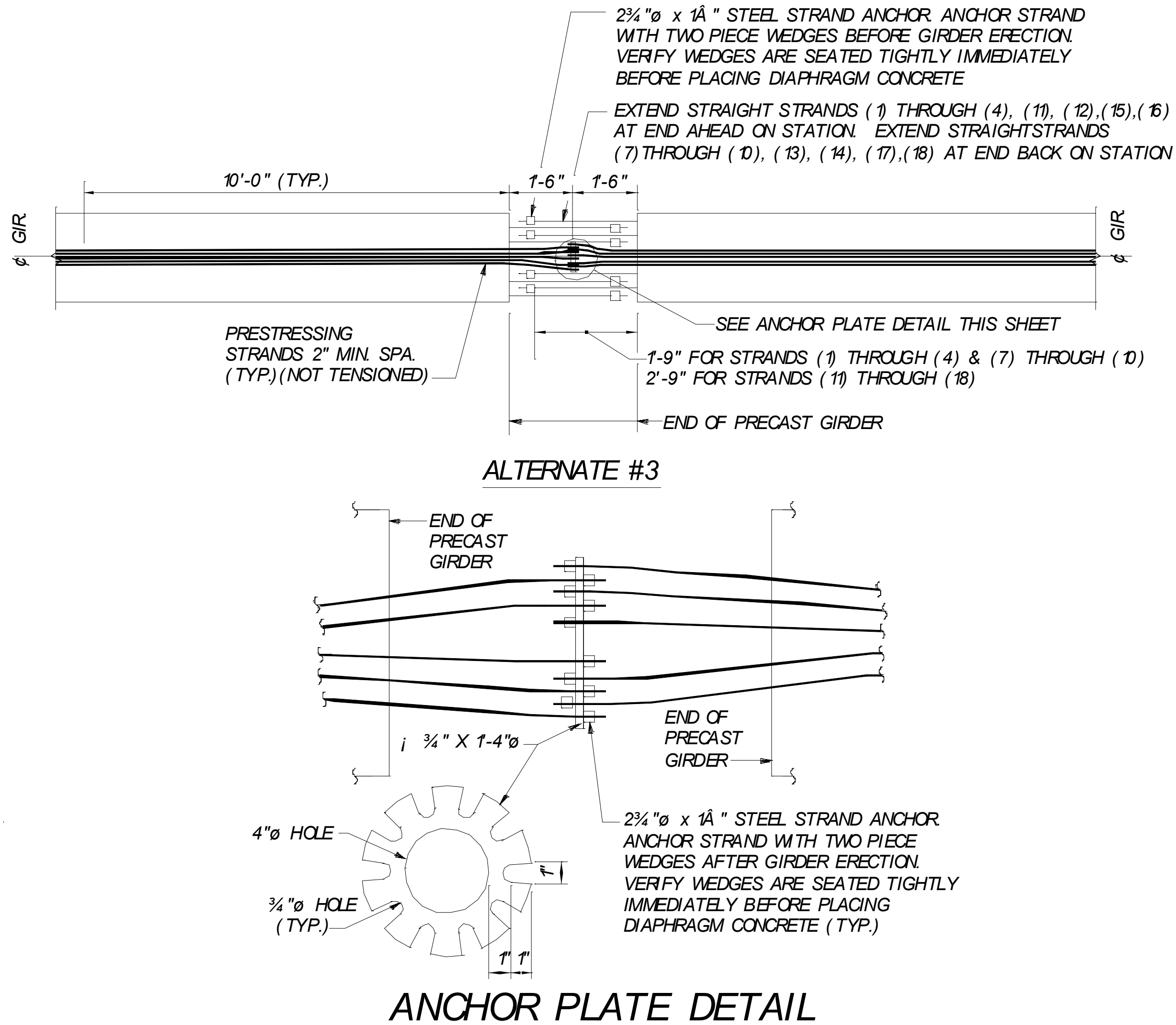

Fig. 9. Strand Extension Detail, Alternate 3

The design procedure to calculate the required number of extended strands to develop the seismic moment capacity is described herein. This calculation is based on developing tensile strength of the strands at ultimate loads. 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 total number of extended strands, N_{PS}, from the end of prestressed girder at fix intermediate diaphragm shall not be less than:

$$N_{PS} = 12(M_C + V_C x h - M_{SIDL}) \frac{N_C}{N_g} x \frac{k}{0.9 A_{PS} x f_{PS} x d} \qquad (2)$$

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 $L_1 = 2L_2$)

Table 2 shows the extended strand calculations for several bridge projects. Seismic forces are obtained by the response spectrum analysis with an average seismic acceleration coefficient of 0.3g.

Table 2. Extended Strands for Positive Moment Connection at Fixed Diaphragms

	Bridge Projects				
	SE 8th I/C	Methow River	HPC Showcase	Bone River	Jenkins Creek
Type of Girder	W58G	W83G	W74G	W74G	W74G
Number of girders, N_g	4	7	5	5	9
Column diameter (ft)	6	5	4	4	3.5
Number of columns, N_c	1	2	2	2	3
Column plastic hinging moment, M_p (ft-kips)	9630	6250	4840	4920	3660
Column plastic shear, V_p (kips)	1067	887	494	465	343
Column Elastic moment, ME (ft-kips)	12420	16110	8240	8060	6280
Column EQ Elastic shear, V_E (kips)	1333	2270	567	762	571
Top of column to c.g. of super. h (ft)	8	11.8	8.167	9.167	9.167
SIDL Moment per girder, (ft kips)	178	210	194	204	188
Top of slab to c.g. of strands, (in)	62	90	80.75	75	75
Area of each strand, A_{ps} (in^2)	0.153	0.217	0.217	0.217	0.217
Number of extended strands, N_{ps}	12	6	5	6	4
Strand Extension Alternative	3	1 or 2	1 or 2	1 or 2	1 or 2

PRECAST SUBSTRUCTURE COMPONENTS

Precast columns meeting seismic requirements have successfully 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. 10 shows a recent application of precast columns in a precast prestressed girder bridge. In this case the precast columns were kept in place on a temporary support for casting of foundation concrete. A cast-in-place bent cap was then provided to support precast prestressed girders. 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.

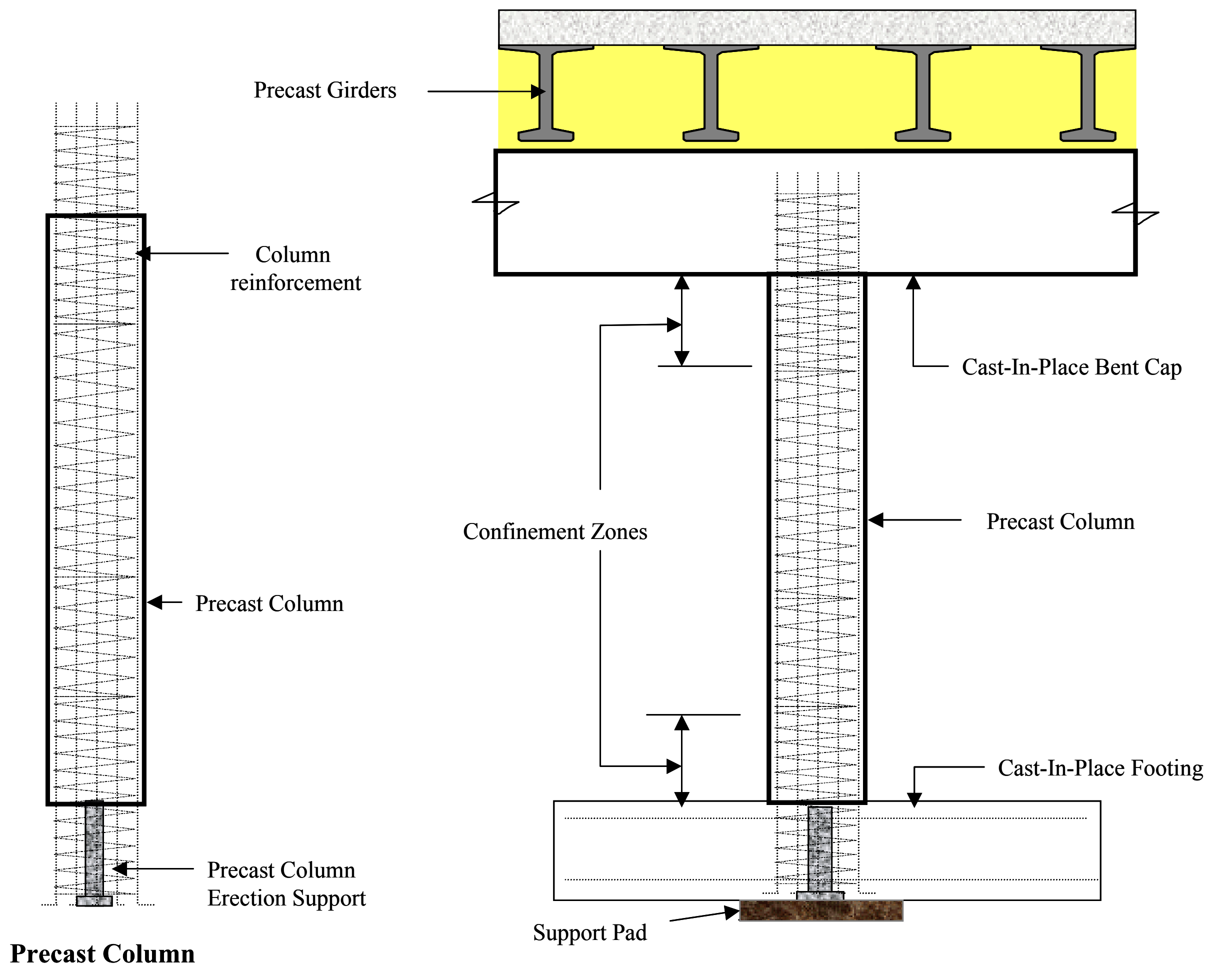

Fig. 10. Precast Columns on Spread Footing for a Prestressed Girder Bridge

The same concept could be employed where drilled shafts are used instead of spread footings. In this case precast columns are kept in place on temporary supports prior to placement of top of the shaft concrete. WSDOT requires permanent casing on top portion of drilled shafts. Shaft diameter shall at least be 3 ft larger than column diameter for ease of construction, and construction tolerances.

Slanted columns are, in general, difficult for forming, casting, and curing of concrete. Precasting of slanted columns is desirable, and has recently been used in a WSDOT bridge project as shown in Fig. 11. In this case precast columns were kept in place on temporary supports prior to casting of footing concrete. The temporary supports for columns were then kept in place until the cast-in-place superstructure was completed. The monolithic connections at the top and bottom of the column were designed and detailed to meet the plastic hinging requirements.

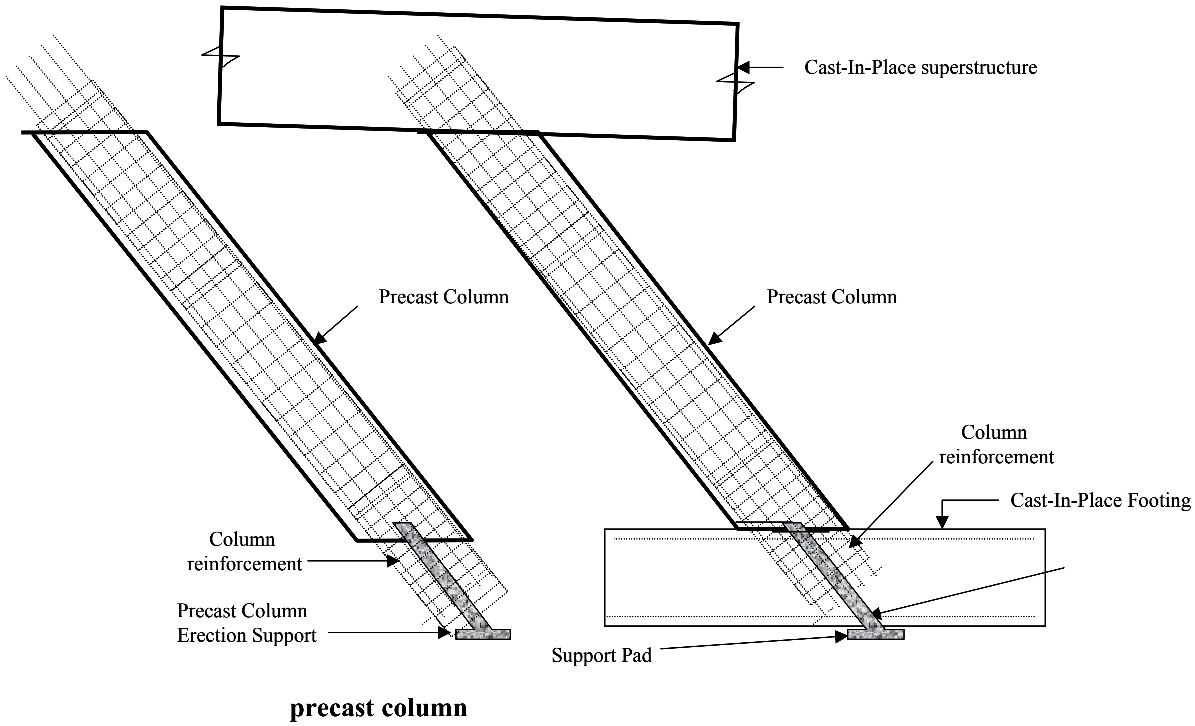

Fig. 11. Slanted Precast Column on Spread Footing for a T-Girder Bridge

PRECAST SEISMIC RESISTANCE BRIDGE

A conceptual design and detailing for a precast bridge is shown in Fig. 12. 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 will 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 higher percentage of longitudinal reinforcement to meet seismic loading requirement (4% A_g Max.). The plastic hinging moment at top of column with reduced rebar cage will be approximately the same as the bottom of the column. This may be achieved by designing the column for minimum reinforcement (1%A_g Min.).

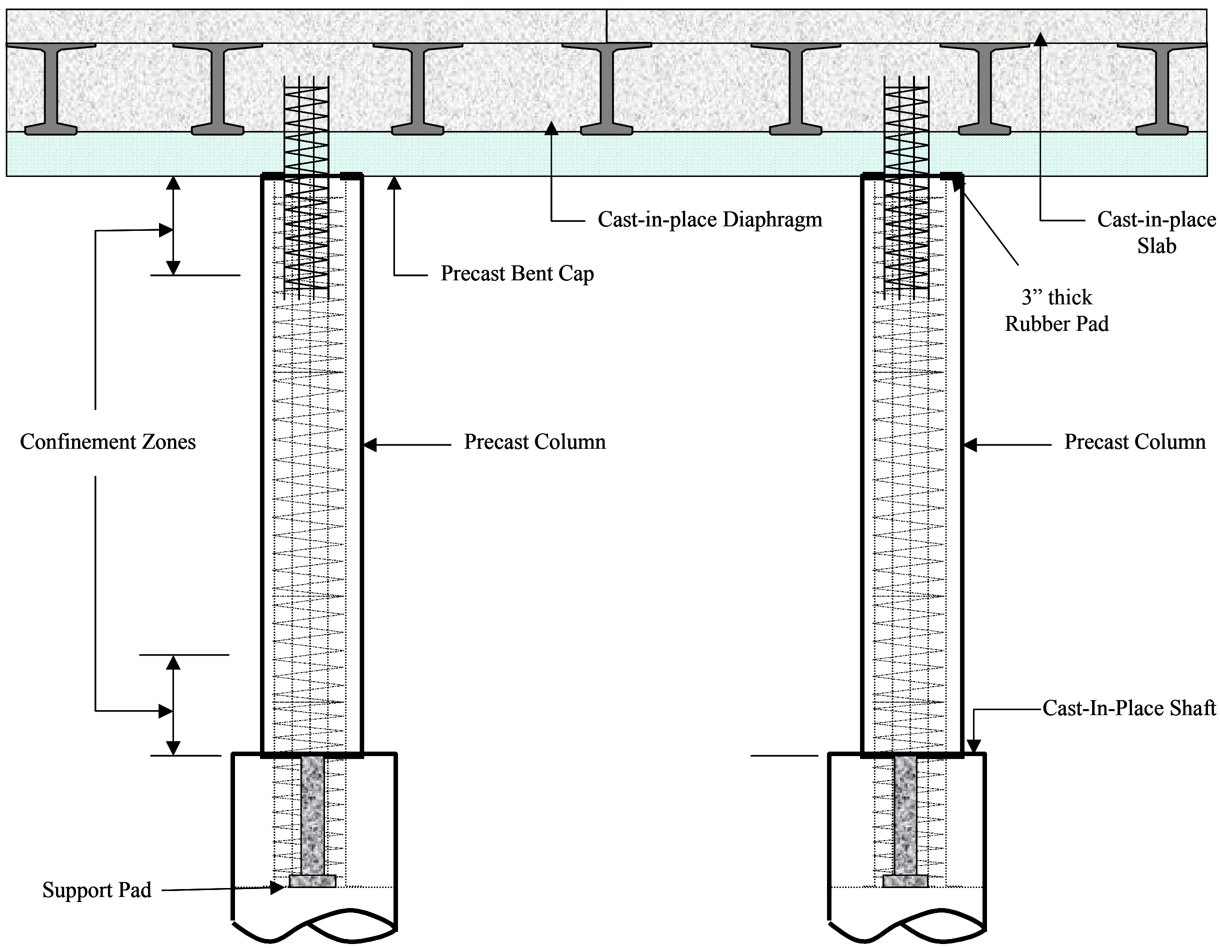

Fig. 12. Precast Seismic Resistant Bridge

The top of the column reinforcement may be increased to produce the same plastic moment capacity as the bottom of the column. This matches the original column design if the column section on the top was not reduced. The designer may choose to specify less rebar for the upper column cage. In this case the top of the column will tend to act as a hinge 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. A numerical application of the above concept is shown in Table 3.

Larger diameter column with the same size of gap on top and bottom of the column could be used for ease of analysis. In this case adjustment for the longitudinal reinforcement in the column is not necessary.

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.

Table 3. Numerical Application of Precast column to Precast Bent cap Design

	Bottom of Column	Top of Column		Larger Diameter Column
Diameter, ft	5	4		**5** (4 at Recess)
Axial Load, kips	1945	1869		1945
Seismic Elastic Moment, ft-kips	22530	21360		22530
Reduced Moment, ft-kips	4506	4272		4506
Reinforcement Ratio	1%	3.75%		3.25%
Resistance Factor	0.75	0.68		0.68
Plastic Hinging Moment, ft-kips	8406	8390		860

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.

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

1. 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 or dilled shaft.
3. Place precast bent cap on the top of the column. A minimum of 3 IN rubber pad or similar material will be provided on the top of the column prior to placement of precast bent cap.
4. Cast concrete to achieve monolithic column to bent cap connection.
5. Place precast girders with the adequate number of extended strands and strand anchors to develop seismic positive moment.
6. Cast lower pier diaphragm and intermediate diaphragms to ensure precast girder stability for slab casting.
7. Cast and cure deck slab concrete.
8. Complete casting concrete for intermediate diaphragm
9. Cast traffic barriers and sidewalk if applicable.

The recommended design procedure for the above precast system is as follows:

1. Perform seismic analysis. The top of the column diameter should be at least 12" smaller than the bottom of column. The column stiffness for seismic analysis should be kept constant based on the bottom of column sectional properties. Cracked section properties based on actual column axial load and reinforcement ratio shall be used for seismic analysis.
2. Use applicable response modification factors, design column reinforcement and calculate plastic moment capacity at the top and bottom of the column.
3. Increase top of column reinforcement to get approximately equal plastic moment capacity on top and bottom of column.
4. Complete redistribution of forces for multiple column bents. Calculate plastic shear.
5. Design precast bent cap for flexural and shear capacity.
6. Design interface shear between cast-in-place concrete and precast bent cap. Interface shear capacity based on shear keys shall be checked. Reinforcing bars with mechanical couplers may be used in addition to shear keys to satisfy interface shear demand.
7. Design foundation and bent cap connections for the lesser of full elastic or plastic hinging moment and associated shear.

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 prestressed I-girder superstructure as are mentioned above.

Using raised crossbeam instead of lower bent cap is more complicated and challenging, but not impossible. A temporary shoring to support precast trapezoidal tubs until the completion of cast-in-place diaphragm is necessary. Inverted precast T-beam with dapped trapezoidal tubs may be used to eliminate the need for temporary shoring. In this case, satisfying positive seismic moment capacity with extended strands may be extremely difficult, unless if post-tensioning is employed.

Precast decked girders, such as deck bulb tees, tri beams, double tees, and slabs may be used in conjunction with precast bent cap and precast columns to achieve a complete precast bridge. However, the use of precast decked members, because of possibility of longitudinal reflective cracking, is not recommended for bridges with high ADTT. WSDOT uses a 5 IN cast-in-place deck with one layer of reinforcement on top of precast decked members to eliminate the possibility of reflective cracking. The details for this type of structures are shown in Fig. 13.

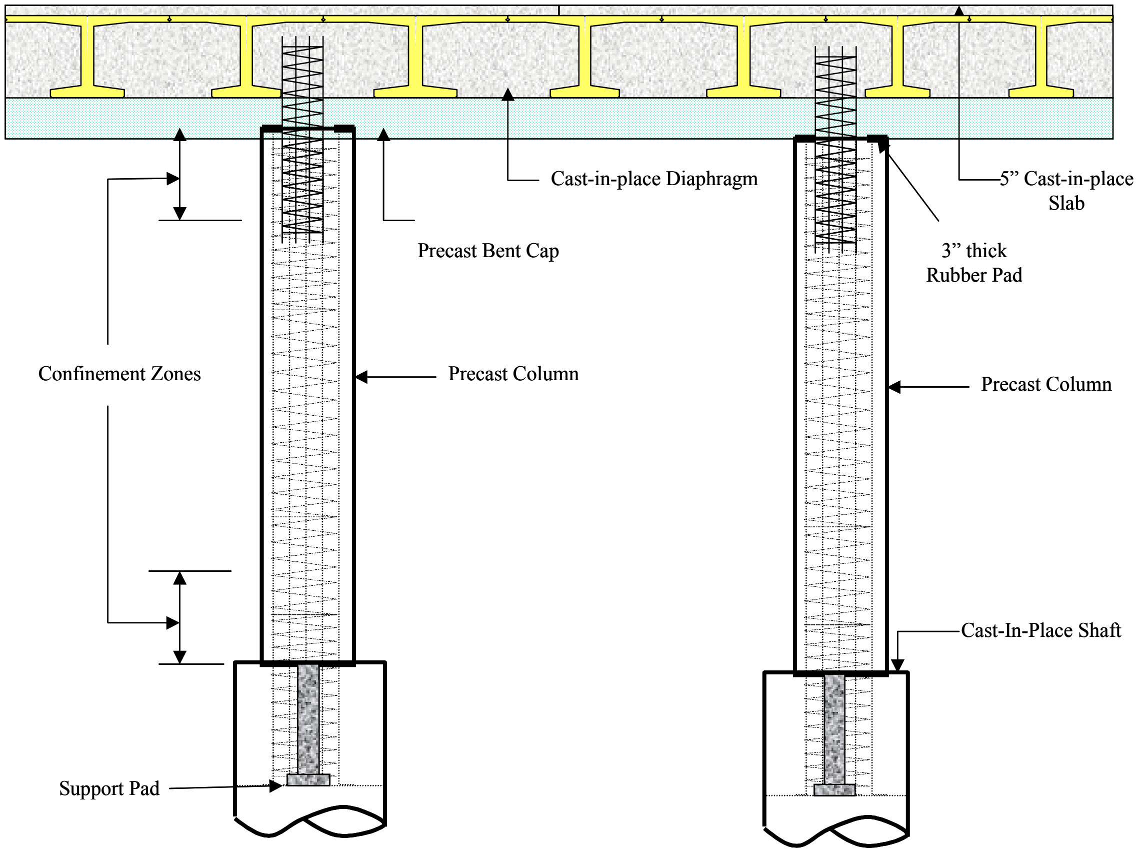

Fig. 13. Precast Seismic Resistant Bridge with Decked Girders

CONCLUSIONS

1. Precast prestressed concrete bridge system is economic and effective system for rapid bridge construction. This system can eliminate traffic disruptions during bridge construction while maintaining quality and long-term performance.

2. Precast bridges with monolithic connections meeting the AASHTO LRFD seismic design and detailing requirements could safely be used in seismic zones.

3. Extended strands from the bottom flange of precast girder provide adequate positive flexural capacity to resist the resulting plastic hinging forces at centroid of superstructure.

4. Longitudinal reinforcement at girder ends shall be provided to ensure the load transfer from girders to cast-in-place diaphragm through interface shear. A combination of saw teeth and longitudinal reinforcement is often used.

5. Force transfer between cast-in-place concrete and the recess in the precast bent cap shall be checked for interface shear. A combination of shear key and reinforcement with mechanical coupler may be necessary to satisfy interface shear resistance.

6. The reinforcing bar in the precast bent cap shall provide adequate flexural and shear capacity for positive and negative moment, and shear due to the weight of diaphragm and precast girders.

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