Use of Precast Concrete Members for Accelerated Bridge Construction in Washington State

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Innovative methods are presented for using precast concrete members in bridge construction; summaries include the essentials of design, detailing, and construction of precast, pretensioned girders; posttensioned spliced girders; and precast deck panels used for bridge structures. The key aspects of using precast members in seismic regions are discussed, with an emphasis on suitable connections that meet ductility requirements. Precast bridges consisting of pretensioned girders, posttensioned spliced girders, trapezoidal open-box girders, and other types of superstructure members have been gaining popularity in recent years. These types of bridges have the advantages of minimizing traffic disruption, accelerating construction, and solving constructability issues in traffic-congested areas and in other specific cases. Handling and shipping limitations often control the span capability of pretensioned girders. Spliced girder design has provided a solution where a one-piece pretensioned girder could not otherwise have been used. Precast deck systems consisting of precast, prestressed concrete deck panels have advantages in accelerated bridge construction and rapid deck replacement. A parametric study has been performed to demonstrate the effectiveness of using precast members in bridge structures. Design examples for each application that use load and resistance factor design specifications are provided. Criteria used in optimizing and developing standard drawings for precast bridge members are discussed. Recommendations for effective use of precast members in seismic regions are provided, and areas for further research are identified.

The relatively recent earthquakes in western states resulted in a major research project headed by the Applied Technology Council. The outcome of this effort was the development of seismic design guidelines for highway bridges (1). These guidelines incorporate an elastic response spectrum analysis 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 and the connection between the 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 longitudinal and transverse directions.

Development of a precast bridge construction system provides an effective and economical design concept that can be implemented for new bridge construction and 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

Two general approaches are used to evaluate the seismic response of a bridge. The first approach is the conventional force-based analysis, and the second approach involves a displacement criterion. In recent years, the displacement method has been emphasized more. The current Washington State Department of Transportation (DOT) Bridge Design Manual requires force-based analysis for all ordinary structures (2). The displacement-based method may be used for major bridge projects in regions of high seismicity.

With the conventional force method, bridge analysis is performed and the forces on its various components are determined (3). The capacities of the components are evaluated and the demand-to-capacity ratios (D:C) are 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, a more rational form of ductility assessment is investigated by considering sequential yielding when evaluating capacity (3). Capacity thus takes on a more global meaning, because 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 load and resistance factor design provisions of the AASHTO LRFD Bridge Design Specifications are based primarily on the conventional force method, by which bridge analysis is performed and the forces on a bridge’s components are determined (4).

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. The reason is that plastic hinging is accompanied by a 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 important, a properly confined column will continue to carry axial load and therefore structural collapse may be avoided.

The AASHTO LRFD specifications incorporate many of the seismic provisions of the 1992 standard specifications but have been updated in light of new research. The specifications were updated mainly as follows:

- Introduction of separate soil profile site coefficients and seismic response coefficients (response spectra) for soft soil conditions and
- Definition of three levels of importance: critical, essential, and others. The importance level is used to specify the degree of damage permitted by using 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 Washington State DOT bridges is considered “Other” unless otherwise instructed by the bridge engineer.

The AASHTO LRFD Extreme Event I limit state includes the effect of seismic loading. A load factor of $\gamma_{EQ}$ is specified for transient loads. The possibility of a partial live load may be considered for bridges in urban areas. 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). Washington State DOT uses $\gamma_{EQ} = 0.5$ for bridges in urban areas.

**SEISMIC-RESISTANT PRECAST CONCRETE BRIDGES**

Monolithic action between superstructure and substructure components is 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 multicolumn bents, particularly in areas of moderate to high seismicity. 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 the girders. Figure 1 shows a typical monolithic moment-resistant connection used for Washington State DOT precast girder bridges.

**Current Washington State DOT Practice for Seismic-Resistant Precast Bridges**

Seismic design and detailing requirements vary from region to region and also depend 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 Washington State DOT 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-shaped diaphragms 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 the 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. Figure 2 shows the connection for semi-integral end piers. This type of end diaphragm eliminates expansion joints at end piers. The gap between the end-pier wall and the end diaphragm should satisfy longitudinal seismic movement requirements.

Figure 3 shows the connection for L-shaped end piers. This type of diaphragm is suitable for longer bridges. The seat width provided at the end pier wall should satisfy the longitudinal seismic movement requirements.

The girder stop detail used at end piers is shown in Figure 4. The elastomeric pads provided at the sides of girder stops prevent concrete-to-concrete impact during a seismic event and allow bridge longitudinal movement under service conditions.

In L-shaped end piers, the minimum displacement requirements at the expansion bearing should 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 (1).

$$ N = (8 + 0.02L + 0.08H)(1 + 0.000125S^2) $$  \hspace{1cm} (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), and
- $S$ = skew angle of the support measured normal to span (degrees).

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**TABLE 1  Response Modification Factors for Concrete Bridges**

<table>
<thead>
<tr>
<th>Substructure</th>
<th>Importance Category</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Critical</td>
</tr>
<tr>
<td>Wall type piers</td>
<td>1.5</td>
</tr>
<tr>
<td>Single column bents</td>
<td>1.5</td>
</tr>
<tr>
<td>Multiple column bents</td>
<td>1.5</td>
</tr>
<tr>
<td>Columns, piers, or pile bents to cap beam or superstructure</td>
<td>1.0</td>
</tr>
<tr>
<td>Columns or piers to foundation</td>
<td>1.0</td>
</tr>
</tbody>
</table>
The empirical seat width should be increased by factors accounting for seismic zones as specified in the following table.

<table>
<thead>
<tr>
<th>Seismic Zone</th>
<th>Seat Width Increasing Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>1.0</td>
</tr>
<tr>
<td>3</td>
<td>1.5</td>
</tr>
<tr>
<td>4</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Most older bridges made with precast girders fail to meet the minimum seat width as required in Equation 1. As part of the seismic retrofit program, Washington State DOT requires longitudinal restrainers to ensure superstructure survival in 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 fixed connection for high-seismicity zones (western Washington) and hinge connection for low-seismicity zones (eastern Washington). In both cases, the superstructure consists of a cast-in-place concrete deck on precast, prestressed concrete girders.

**FIGURE 1** Superstructure-to-substructure connection at an intermediate pier (c.g. = center of gravity)

**FIGURE 2** Semi-integral end-pier connection (CIP = cast in place).
made continuous for live load at intermediate piers. Precast girders are temporarily supported on oak blocks until the cast-in-place diaphragm is completed. The designer should check the edge distance and provide a dimension that prevents edge failure, or spalling, at the top corner of the supporting crossbeam 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 Figure 5 is for continuous spans at intermediate-pier diaphragms. Washington State DOT design assumptions for hinge diaphragms are as follows:

1. All girders of adjoining spans should be of the same depth, spacing, and preferably the same type.
2. Girders should be designed as simple spans for both dead and live loads.
3. Reinforcement for negative moments is provided at intermediate piers in the deck, because of the live loads and superimposed dead loads from traffic barrier, pedestrian walkway, utilities, and so forth.
4. Hinge bars size and spacing are designed for anticipated lateral loads from seismic and other load combinations. Adequate embedment for hinge bars is provided into the crossbeam.

The connection shown in Figure 6 is for continuous spans with fixed moment-resistant connection between superstructure and substructure at intermediate piers. Pier caps are wider for fixed connections, and precast girders are supported on oak blocks on lower crossbeam. The cast-in-place diaphragm is completed in two stages to ensure precast girder stability after erection and completion of the diaphragm following slab casting and initial creep. Adequate extended strands and reinforcing bars are provided to ensure the performance of the connection during a major seismic event. The design assumptions for fixed diaphragms are as follows:

1. Ensure that 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, because of live loads and superimposed dead loads from traffic barrier, pedestrian walkway, utilities, and so forth.
4. Determine resultant plastic hinging forces at the centroid of the superstructure.
5. Determine the number of extended strands to resist seismic positive moment.
6. Design diaphragm reinforcement to resist the resultant seismic forces at the centroid of the diaphragm.
7. Design longitudinal reinforcement at girder ends for shear friction.

**Positive Moment Capacity at Intermediate Diaphragms**

Strand extension details are shown in Figure 7 for strand anchors and strands chucks and in Figure 8 for anchor plates. These details should be used for continuous spans at diaphragms and are not applicable for continuous spans using hinge diaphragms. The effect of time-dependent, positive moments from creep and shrinkage should be
FIGURE 5 Intermediate hinge diaphragm.

FIGURE 6 Intermediate fixed diaphragm.

FIGURE 7 Strand extension detail, Alternative 1.
considered to more accurately determine the positive moment capacity. The strand extension shown in Figure 7 is suitable for most common prestressed girder bridges. A minimum of four extended strands should be provided regardless of design requirements.

The strand extension shown in Figure 8 is used only if additional strands beyond those shown in Figure 7 are required. This situation may occur when a large-diameter column is supporting a shallow superstructure. Because of a shallow structure, resisting plastic hinging forces in this case requires more extended strands than those provided in Figure 7.

The design procedure to calculate the required number of extended strands for developing the seismic moment capacity is described herein. This calculation is based on developing the 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 girders at fixed intermediate diaphragms, in the absence of the time-dependent, positive moment from creep and shrinkage, should not be less than as indicated in Equation 2 (3):

\[
N_{PS} = 12(M_c + V_c \times h - M_{SIDL}) \times \frac{N_c}{N_t} \times \frac{k}{0.9A_{ps} \times f_{ps} \times d}
\]

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 the centroid of the superstructure (ft),
- \( d \) = distance from top of slab to the centroid of the extended strands (in.),
- \( N_c \) = number of columns in the pier,
- \( N_t \) = number of prestressed girders in the pier,
- \( A_{ps} \) = area of each extended prestressing strand (in.\(^2\)),
- \( f_{ps} \) = ultimate strength of prestressing strands (ksi), and
- \( k \) = span coefficient (\( k = 0.5 \) for \( L_1 = L_2 \), \( k = 0.67 \) for \( L_1 \approx 2L_2 \)).

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 0.3 g.

**Precast Substructure Components**

Precast columns meeting seismic requirements have successfully been used in Washington State DOT bridges for accelerated construction. Washington State DOT requires monolithic connections at the top and bottom of the column. Reinforcing bars from the top and bottom of the column should extend into the cast-in-place concrete of the crossbeam and footing. Figure 9 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 the casting of foundation concrete. A cast-in-place bent cap was provided to support precast, prestressed girders. The monolithic connection between the precast column and precast girder was designed and detailed to meet the top of the column plastic hinging forces at the centroid of the superstructure.

The same concept could be used when drilled shafts are used instead of spread footings. In this case, precast columns are kept in place on temporary supports before placement on top of the shaft concrete. Washington State DOT requires permanent casing on the top portion of drilled shafts. The shaft diameter should be at least 3 ft larger than the column diameter for ease of construction and to meet construction tolerances.

Slanted columns are generally difficult for the forming, casting, and curing of concrete. Precasting of slanted columns is desirable and has recently been used in a Washington State DOT bridge proj-

![FIGURE 8 Strand extension detail, Alternative 3.](image-url)
ect, as shown in Figure 10. In this case, precast columns were kept in place on temporary supports before casting of footing concrete. The temporary supports for columns were 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.

**TABLE 2 Extended Strands for Positive Moment Connection at Fixed Diaphragms**

<table>
<thead>
<tr>
<th>Bridge Projects</th>
<th>SE 8th</th>
<th>Methow River</th>
<th>HPC Showcase</th>
<th>Bone River</th>
<th>Jenkins Creek</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of girder</td>
<td>W58G</td>
<td>W83G</td>
<td>W74G</td>
<td>W74G</td>
<td>W74G</td>
</tr>
<tr>
<td>Number of girders, N_g</td>
<td>4</td>
<td>7</td>
<td>5</td>
<td>5</td>
<td>9</td>
</tr>
<tr>
<td>Column diameter (ft)</td>
<td>6</td>
<td>5</td>
<td>4</td>
<td>4</td>
<td>3.5</td>
</tr>
<tr>
<td>Number of columns, N_c</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Column plastic hinging moment, M_p (ft-kips)</td>
<td>9630</td>
<td>6250</td>
<td>4840</td>
<td>4920</td>
<td>3660</td>
</tr>
<tr>
<td>Column plastic shear, V_p (kips)</td>
<td>1067</td>
<td>887</td>
<td>494</td>
<td>465</td>
<td>343</td>
</tr>
<tr>
<td>Column elastic moment, M_E (ft-kips)</td>
<td>12420</td>
<td>16110</td>
<td>8240</td>
<td>8060</td>
<td>6280</td>
</tr>
<tr>
<td>Column EQ elastic shear, V_E (kips)</td>
<td>1333</td>
<td>2270</td>
<td>567</td>
<td>762</td>
<td>571</td>
</tr>
<tr>
<td>Top of column to c.g. of super. h (ft)</td>
<td>8</td>
<td>11.8</td>
<td>8.167</td>
<td>9.167</td>
<td>9.167</td>
</tr>
<tr>
<td>SIDL moment per girder, (ft-kips)</td>
<td>178</td>
<td>210</td>
<td>194</td>
<td>204</td>
<td>188</td>
</tr>
<tr>
<td>Top of slab to c.g. of strands, (in.)</td>
<td>62</td>
<td>90</td>
<td>80.75</td>
<td>75</td>
<td>75</td>
</tr>
<tr>
<td>Area of each strand, A_p (in.²)</td>
<td>0.153</td>
<td>0.217</td>
<td>0.217</td>
<td>0.217</td>
<td>0.217</td>
</tr>
<tr>
<td>Number of extended strands, N_p</td>
<td>12</td>
<td>6</td>
<td>5</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>Strand extension alternative</td>
<td>3</td>
<td>1 or 2</td>
<td>1 or 2</td>
<td>1 or 2</td>
<td>1 or 2</td>
</tr>
</tbody>
</table>

**Precast Seismic-Resistant Bridge**

A conceptual design and detailing for a precast bridge are shown in Figure 11. The monolithic connections between precast components at intermediate pier diaphragms and at foundations are designed to meet the seismic requirements. The reduced top of the column

![Figure 9](image-url)  
**FIGURE 9** Precast columns on spread footing for a prestressed girder.
FIGURE 10  Slanted precast column on spread footing for a T-girder.

FIGURE 11  Precast seismic-resistant bridge.
diameter provides a seat for placement of the precast bent cap. The difference in rebar cage diameter should be at least 12 in., and the column support width for precast bent should 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 requirements [4% gross area of section ($A_g$), maximum]. The plastic hinging moment at the top of the column with reduced rebar cage should be approximately the same as at the bottom of the column. This condition may be achieved by designing the column for minimum reinforcement (1% $A_g$ minimum).

The top of the column reinforcement may be increased to produce the same plastic moment capacity as at 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 the column. Although this condition may be desirable to reduce rebar congestion in the top of the column, it may require a larger foundation. The outcome of the parametric study of this concept is shown in Table 3.

Larger-diameter column with the same size gap on its top and bottom 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 should be carefully dimensioned to eliminate the adverse effect of flares on column stiffness and to ensure that plastic hinges form on top of the column.

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

Interface shear resistance should 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 should be provided. The top of the precast bent cap is to resist the negative moment from the weight of the precast girders and the lower portion of the cast-in-place diaphragm. The deck slab may be cast after completion of the lower diaphragm.

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

1. Precast the columns with adequate longitudinal and transverse reinforcement on their top and bottom.
2. Position the precast columns in place and cast the concrete for spread footing or drilled shaft.
3. Place the precast bent cap on the top of the column. At a minimum, a 3-in.-thick rubber pad or similar material should be provided on the top of the column before placement of the precast bent cap.
4. Cast the concrete to achieve a monolithic column to the bent cap connection.
5. Place the precast girders with an adequate number of extended strands and strand anchors to develop a seismic positive moment.
6. Cast the lower pier diaphragm and the intermediate diaphragms to ensure precast girder stability for slab casting.
7. Cast and cure the deck slab concrete.
8. Complete casting the concrete for the intermediate diaphragm.
9. Cast the traffic barriers and sidewalk if applicable.

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

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

This 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 the prestressed I-girder superstructure as mentioned previously. Using raised crossbeams instead of lower bent caps is more complicated and challenging but not impossible. A temporary shoring to support precast trapezoidal tubs until completing the cast-in-place diaphragm is necessary. Inverted precast T-beam with dapped trapezoidal tubs may be used to eliminate the need for temporary shoring.

<table>
<thead>
<tr>
<th>TABLE 3 Numerical Application of Precast Column to Precast Bent Cap Design</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Diameter, ft</strong></td>
</tr>
<tr>
<td>Diameter, ft</td>
</tr>
<tr>
<td>Axial load, kips</td>
</tr>
<tr>
<td>Seismic elastic moment, ft-kips</td>
</tr>
<tr>
<td>Reinforcement ratio</td>
</tr>
<tr>
<td>Resistance factor</td>
</tr>
<tr>
<td>Plastic hinging moment, ft-kips</td>
</tr>
</tbody>
</table>
In this case, satisfying positive seismic moment capacity with extended strands may be extremely difficult unless posttensioning is used.

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, because of the possibility of longitudinal reflective cracking, the use of precast decked members is not recommended for bridges with high ADTT. Washington State DOT uses a 5-in. cast-in-place deck with one layer of reinforcement on top of the precast decked members to eliminate the possibility of reflective cracking. The details for this type of structure are shown in Figure 12.

CONCLUSIONS

1. A precast, prestressed concrete bridge system is economical and effective 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 AASHTO LRFD seismic design and detailing requirements could safely be used in seismic zones.

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

4. Longitudinal reinforcement at the girder ends should be provided to ensure the load transfer from the girders to the cast-in-place diaphragm. A combination of sawteeth and longitudinal reinforcement is often used.

5. Force transfer between the cast-in-place concrete and the recess in the precast bent cap should 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 should provide adequate flexural and shear capacity for positive and negative moments and shear from the weight of the diaphragm and precast girders.

REFERENCES


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