Connections for Accelerated Bridge Construction of Jointless Precast Concrete Bridges in Seismic Regions

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ABSTRACT

Prefabricated jointless 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. Connections in precast concrete substructures are typically made at the beam-column and column-foundation interfaces to facilitate fabrication and transportation. However, for structures in seismic regions, those interfaces represent locations of high moments and shears and large inelastic cyclic strain reversals.

Jointless bridge superstructures are constructed to work integrally with the abutments. Movements due to creep, shrinkage and temperature changes are accommodated by using flexible bearings or foundation and through incorporating relief joints at the ends of the approach slabs. In addition to reduced maintenance costs, other advantages of jointless bridges include improved structural integrity, reliability and redundancy, improved longterm serviceability, improved riding surface, reduced initial cost, and improved aesthetics. In recent times, jointless bridges have been built in seismically sensitive areas.

Developing connections that can accommodate inelastic cyclic deformations and are readily constructible is the primary challenge for ABC in seismic regions. The AASHTO LRFD Specifications do not explicitly address the jointless precast, pretensioned or post-tensioned elements. The seismic design and detailing, accomplished research, construction practices of jointless bridges, and implementation of a precast concrete bridge bent system that is intended to meet those challenges are presented. This paper will attempt to capture the state-of-practice of jointless continuous bridges in seismic regions.

Key words: Accelerated Bridge Construction, Connections, Seismic, Jointless, Precast Concrete

INTRODUCTION

Jointless bridges are defined as bridges with no expansion joints between the superstructure and the supporting abutments. Because of several problems resulting from the traditional practice, the jointless bridge has been widely adopted. The focus of this paper is on the seismic design of jointless bridges.

Concrete superstructures are less sensitive to temperature changes due to the lag between the air temperature and the interior temperature of a concrete member with its relatively large mass. This phenomenon is reflected in AASHTO LRFD Bridge Design Specifications (AASHTO BDS), which provides lower design temperature variations for concrete superstructures than for steel. In a moderate climate, a concrete superstructure will expand and contract a total of approximately 12 mm per 30 m of bridge length with seasonal temperature variation. However, a steel superstructure will typically expand and contract approximately 25 mm per 30 m of length.

Bridge piers and abutments restrain thermal movements and induce tensile or compressive forces in the superstructure. With properly proportioned piers and abutments these restraint forces are routinely and safely ignored in the design of the superstructure.

Thermal movements of a cast-in-place concrete superstructure are similar to those of a precast, prestressed concrete superstructure. However, creep and shrinkage movements are considerably greater for cast-in-place than for precast superstructures. For these reasons, shrinkage and creep movements of precast, prestressed concrete superstructures are frequently ignored for structures of moderate length. However for longer spans the differential shrinkage between the cast-in-place slab and the precast girder in addition to creep and thermal effects should be considered.

INTEGRAL JOINTLESS BRIDGES

Jointless bridges consist of superstructures, abutments, intermediate piers, and foundations. The design of jointless bridges is generally similar to that of conventional bridge design. Special analysis and design considerations required for jointless bridges are primarily associated with the need to accommodate volumetric changes in the structure, such as thermal movements.

Jointless bridges accommodate superstructure movements without conventional expansion joints. The superstructure is rigidly or semi-rigidly connected to the abutments. Approach slabs, connected to the abutment and/or deck slab with reinforcement, move with the superstructure. Generally, at its junction with the approach pavement, the approach slab is supported by a sleeper slab or grade beam. The superstructure movement here is accommodated using flexible pavement joints.

Jointless construction is well-suited to both single- and multiple-span bridges. For single-span bridges, stability is provided by passive pressure behind the backwall and for multiple-span bridges, intermediate piers contribute to the bridge's stability. Jointless bridges could be founded on piles elements or shafts or spread footings on soil if the soil is well compacted and the possibility of settlement of the foundation is considered in the design as shown in Figure 1.

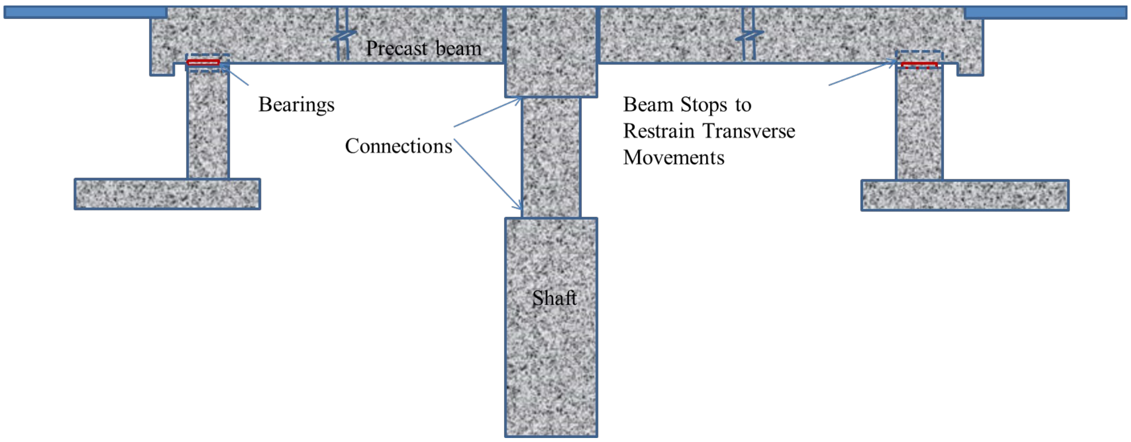

Figure 1. Continuous Jointless Bridge

BENEFITS OF JOINTLESS BRIDGES

Jointless bridges provide substantial reserve capacity to resist potentially damaging overloads by distributing loads along the continuous and full-depth diaphragm at bridge ends. The close tolerances required when utilizing expansion bearings and bridge seats need to conform to girder flange slope and camber corrections, since the girder loads are ultimately carried by the concrete comprising the end diaphragm.

Continuity in bridge superstructure provides added redundancy and capacity for all types of catastrophic events. In designing for seismic events, considerable material reductions can be achieved through the use of continuity by negating the need for enlarged seat widths and restrainers. Further, the use of jointless abutments eliminates loss of girder support; the most common cause of damage to bridges in seismic events. Joints introduce a potential collapse mechanism into the overall bridge structure. Jointless abutments have consistently performed well in actual seismic events and have significantly reduced or avoided problems of backwall and bearing damage that are associated with seat-type jointed abutments. The dampening arising from soil-abutment interaction has been proven to significantly reduce the lateral loads taken by intermediate substructure columns and footings. The following limitations may be considered in use of jointless bridges:

1. Limitations on length are concerned with passive pressure effects, stresses in the deep foundation elements, and the movement capacity of the joint between the approach slab and the approach pavement. Many state departments of transportation limit lengths to 100 m for steel superstructures and 200 m for prestressed concrete superstructures. A few states, like Washington and Tennessee, have successfully used longer lengths.
2. Skew angles have generally been below 45 deg. However some states have used this method of construction extensively and effectively for curved bridges as well as bridges with skew angles up to 75 deg.
3. Jointless bridges require end diaphragms to be supported on flexible foundation types and bearings

DESIGN REQUIREMENT FOR JOINTLESS BRIDGES IN SEISMIC REGIONS

The AASHTO Guide Specifications for LRFD Bridge Seismic Design (LRFD SGS) is a displacement-based requiring bridge to be design with adequate displacement capacity to accommodate earthquake demands. The displacement capacity of bridges is checked using a displacement-based procedure, especially for those bridges located in regions of high seismic risk. The force-based methodology of the LRFD Specifications has also been used in some states with lower seismic demand. The authors recommend the displacement based design of AASHTO SGS for jointless bridges.

The overall objective of the performance criteria is life safety during a 1,000-year seismic event. Bridges have a low probability of collapse but may suffer significant damage and significant disruption to service. Partial or complete replacement may be required." In a major event, offsets, cracking, reinforcement yielding, and major spalling of concrete are expected. While the 1,000-year return period is judged as applicable to most bridges, higher levels of performance may be required by the bridge owner, as in the case of "critical" or "essential" bridges that provide life safety transportation, bridges that are essential to the economy, or bridges required for local emergency plans. Site- or project-specific design criteria are generally developed for such projects.

The LRFD SGS does not explicitly address the jointless precast, pretensioned or post-tensioned elements. The precast beams made continuous for live loads must have beam–to–beam or beam–to–cap connections that can be expected to remain undamaged during the 1,000-year seismic event. Opening and closing of the bottom flange-to-flange or flange-to-cap joint connection is not permitted.

In the force-based analysis method, a linear elastic multi-modal response spectrum analysis is performed and the force effects in various bridge or structure components are determined. Equivalent static analysis of lateral loads based on a percentage of the dead load is also permitted by some agencies. The capacities of the components are evaluated and the component demand/capacity (D/C) ratios are then calculated. A particular component is said to have adequate capacity if its D/C ratio is less than the permissible force reduction factor, R, for that component.

Pushover analysis addresses typical sources of material nonlinearity as well as geometric nonlinearity. Material nonlinearity includes soil, concrete, soil-structure interaction, and yielding of the reinforcement. Geometric nonlinearity refers to the P-Δ effect. The bridge frame is pushed laterally along both its longitudinal and transverse directions until the target displacement is obtained.

Designing for life safety means that significant damage can result. Significant damage includes permanent offsets, damage between approach structures and the bridge superstructure, between spans at expansion joints, permanent changes in bridge span lengths, and permanent displacements at the top of bridge columns. Damage also consists of severe concrete cracking, yielding and buckling of reinforcement, major spalling of concrete and severe cracking of the bridge deck slab. These conditions may require closure of the bridge to repair the damages. Partial or complete replacement of columns may be required in some cases. For sites with lateral flow due to liquefaction, deep foundation elements may suffer significant inelastic deformation and partial or complete replacement of the columns and deep foundation elements may be necessary.

ABUTMENT TYPES FOR JOINTLESS BRIDGES

The beginning or ending substructure element of a bridge is commonly referred to as an abutment or end bent. There are numerous variations that are used in further describing these units, such as bench-type, spill-through, stub, deep, etc. Figure 2 shows different types of jointless abutments. For consistency within this report, these units will be collectively referred to as abutments, with only minimal added description of their variation in type.

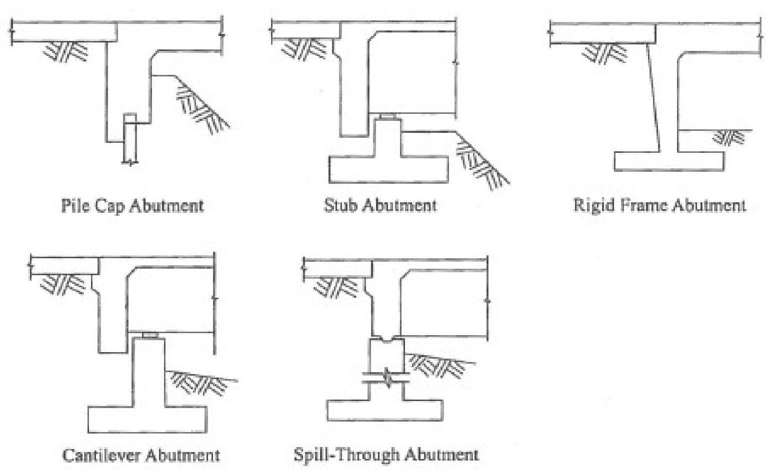

Figure 2. Integral and Semi-Integral Jointless Abutment Types

In jointless bridges, the ends of the girders are fixed to the abutments and expansion joints are eliminated at these supports. With the expansion joints eliminated, forces are induced in the substructure due to resistance to thermal movement and to creep and shrinkage that have to be considered in the design of jointless abutments.

While jointless abutments have been used successfully for 50 years, their implementation has not been an exact science, but rather a matter of intuition, experimentation and observation. Inspection of many bridges with failed expansion bearings has revealed that anticipated catastrophic damage has not always occurred.

The most common technique used in foundation design is the utilization of nonlinear soils-spring method also known as p-y method. Using this procedure, deep foundation response is obtained by an interactive solution of differential equations using finite-difference techniques. The soil response is described by a family of non-linear curves (p-y curves) that compute soil resistance “p” as a function of deep foundation deflection “y”.

APPROACH SLABS AND BACKFILL

Due to the difficulties in obtaining proper embankment and backfill compaction around abutments, approach slabs are recommended; especially for new construction. Approach slabs offer many benefits other than acting as a bridge between the abutment and more densely compacted embankments. Approach slabs provide a transition from the pavement to the bridge if embankment settlement occurs. Such transitions provide a smooth ride while reducing impact loads to the bridge. Approach slabs also provide greater load distribution at bridge ends, which aids in reducing damage to the abutments; especially from overweight vehicles. Finally, properly detailed approach slabs help control roadway drainage, thus preventing erosion of the abutment backfill or freeze/thaw damage resulting from saturated backfill.

The approach slab could be anchored into the abutment backwall so that it moves in concert with the bridge. Otherwise, cyclic expansions will force the slab to move with the bridge without a mechanism to pull it back when the bridge contracts. As debris fills the resulting opening, repeated cycles will ratchet the slab off its support. The anchorage used to fasten the approach slab should be detailed to act as hinge so that the slab can rotate downward without distress as the embankment settles.

Where the anticipated total movement at an abutment exceeds 12 mm and the approach roadway is asphalt, an expansion joint at the pavement should be considered. The reason for the latter is that larger movements can damage asphalt adjacent to the end of the approach pavement in the expansion cycle. During the contraction phase, a significant gap is created through which water can infiltrate the subgrade. If regular maintenance can be arranged to fill this gap with a suitable joint sealer in cold weather, no joint will be needed.

Approach slabs have been found to be one of the most critical components of a jointless bridge. The approach slabs serve two primary purposes:

1. Approach slabs reduce the compaction of the backfill material behind the backwall due to traffic. Control of excessive passive soil resistance to thermal expansion is also achieved.
2. The thermal movements of the system are transferred from the end of the bridge to the point where the approach slab joins the roadway pavement. A flexible pavement joint is provided at this point. In addition, some agencies use plastic sheets or expanded polystyrene boards below the approach slab to provide a positive separation from the subgrade to enhance movement.

Approach slabs are generally about 6 to10 m long and are standardized in most states. The flexible pavement joint should match that of the particular joint material used to accommodate the movement rating desired. Theoretically, the reinforcement needed for connection to the abutment should exceed the weight of the slab multiplied by the coefficient of friction between poured concrete and sub-base material used. Another method, which has been used in some states is to design the approach slab bottom reinforcement based on a span equal to 50% of the slab length, usually 6 to 10 m. Assuming that the approach slab is dragged on the approach fill, the reinforcement to tie the slab to the abutment backwall is nominal. The width of the joint at the free end of the approach slab should be kept small.

SEISMIC DESIGN OF INTEGRAL ABUTMENTS

The participation of abutment walls in providing resistance to seismically induced inertial loads may be considered in the seismic design of bridges either to reduce column sizes or reduce the ductility demand on the columns. Damage to backwall and wingwalls during earthquakes may be considered acceptable when considering no collapse criteria, provided that unseating or other damage to the superstructure does not occur. Abutment participation in the overall dynamic response of the bridge system shall reflect the structural configuration, the load transfer mechanism from the bridge to the abutment system, the effective stiffness and force capacity of the wall-soil system, and the level of acceptable abutment damage. The capacity of the abutments to resist the bridge inertial loads shall be compatible with the soil resistance that can be reliably mobilized, the structural design of the abutment wall, and whether the wall is permitted to be damaged by the design earthquake. The lateral load capacity of walls shall be evaluated on the basis of a rational passive earth-pressure theory.

Under earthquake loading, the earth pressure action on abutment walls changes from a static condition to one of two possible conditions:

- The dynamic active pressure condition as the wall moves away from the backfill, or
- The passive pressure condition as the inertial load of the bridge pushes the wall into the backfill.

The governing earth pressure condition depends on the magnitude of seismically induced movement of the abutment walls, the bridge superstructure, and the bridge abutment configuration. The semi integral abutment and corresponding loading diagram is shown in Figure 3.

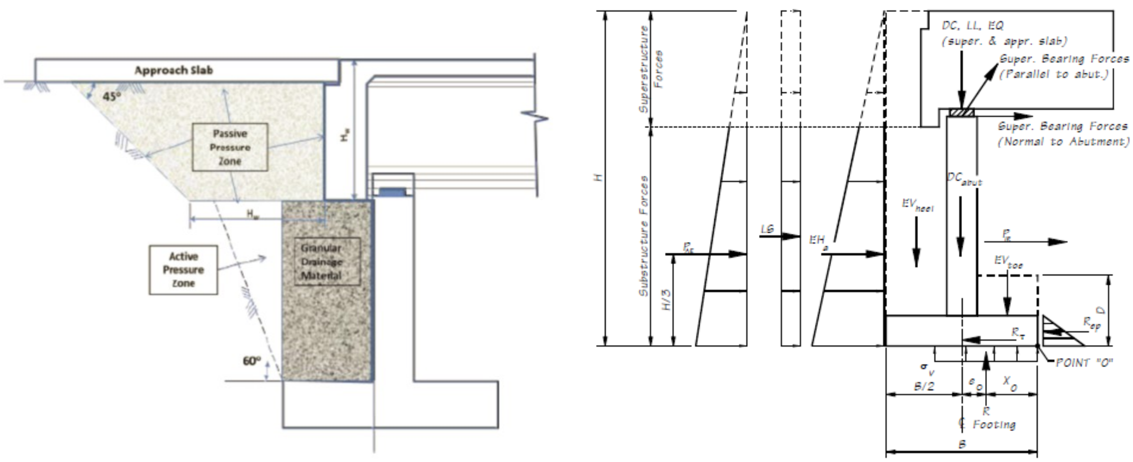

Figure 3. Semi Integral Abutment and Corresponding Loading Diagram

Abutment resistance shall be limited to 70% of the value obtained using the procedure given in the AASHTO SGS. Abutment stiffness, K_{eff}, and passive capacity, P_p, should be characterized by a bilinear or other higher order nonlinear relationship. When the motion of the back wall is primarily translation, passive pressures may be assumed uniformly distributed over the height.

Where the passive pressure resistance of soils behind semi-jointless or L-shape abutments will be mobilized through large longitudinal superstructure displacements, the bridge may be designed with the abutments as key elements of the longitudinal Earthquake Resisting System (ERS). Abutments are designed to sustain the design earthquake displacements. When abutment stiffness and capacity are included in the design, it should be recognized that the passive pressure zone mobilized by abutment displacement extends beyond the active pressure zone normally used for static service load design. Dynamic active earth pressure acting on the abutment need not be considered in the dynamic analysis of the bridge.

Jointless short span bridges could also be supported by a Geosynthetic wall and MSE wall as shown in Figure 4. This type of bridges are suitable for simple span bridges since its seismic performance has not been studied, These bridges shall conform to the following requirements:

1. Walls shall be 10 m or less in total height, which includes the retained soil height up to the bottom of the embedded spread footing.
2. For structural earth walls, the front edge of the bridge footing shall be placed 1.2 m. minimum from the back face of the fascia panel. For geosynthetic retaining walls with a wrapped face, the front edge of the bridge footing shall be placed 610 mm minimum from the back face of the fascia panel.
3. The abutment footing shall be covered by at least 150 mm of soil for frost protection.
4. The superstructure of continuous span bridges shall be designed for differential settlement between piers.

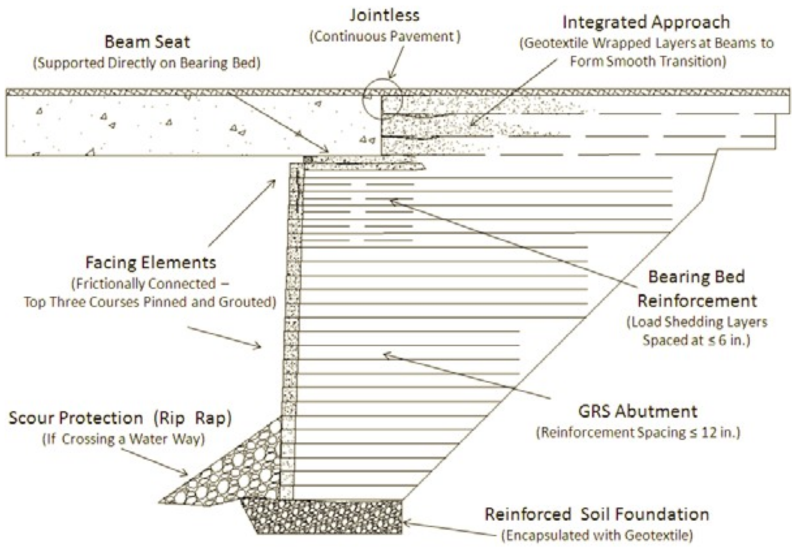

Figure 4: Jointless Bridge supported by a geosynthetic wall or SE wall

ABUTMENT CONNECTION FOR PRECAST JOINTLESS BRIDGES

The typical abutment in regions of moderate and high seismic hazard is a cast-in-place concrete pier wall supported on spread footings, deep foundation elements, or shaft foundations. Precast beams are often supported on elastomeric bearing pads at end piers. Semi-jointless end diaphragms may be used for shorter bridges. The bearing system is designed for the service load condition but may not be adequate to resist seismic loading. The bearings are designed to be accessible so that the superstructure can be lifted and the bearings replaced after a major seismic event. Approach slabs rest on a notch provided at the superstructure end, thereby providing a ramp up to and on to the bridge, should soil behind the abutment settle during a seismic event.

Figure 5 shows a semi-jointless end pier detail. This type of end diaphragm eliminates the need for expansion joints at end piers. The gap between the end pier wall and the end diaphragm is designed to be greater than the longitudinal seismic movement requirement for the extreme event limit state, and thermal expansions at the service limit state for bridge lengths less than 150 m.

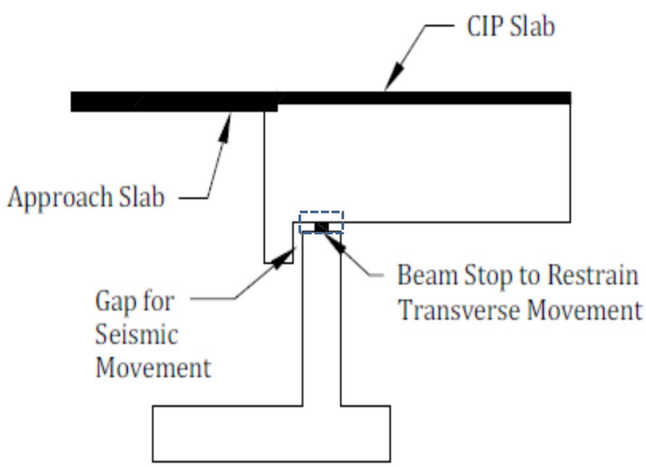

Figure 5. Semi-jointless End Pier Connection

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 1.

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

where

N = minimum support length, m

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

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

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

The empirical seat width is modified as shown in Table 1 for different Seismic Design Category (SDC).

Table 1. Percentage N by SDC and Effective Peak Ground Acceleration, As

Seismic Zone	Effective peak ground acceleration, As	Percentage, N
A	< 0.05	≥ 75
A	≥ 0.05	100
B	All Applicable	150
C	All Applicable	150

The hinge seat length in "well-balanced frames" (adjacent frames for which the ratio of the natural periods is equal to or greater than 0.7) to be evaluated as follows:

$$N = \Delta_{p/s} + \Delta_{cr+sh} + \Delta temp + \Delta_{eq} + 100 \text{ mm} \quad (2)$$

Where:

Δ_{eq} = relative earthquake loading longitudinal displacement demand, mm

$\Delta_{p/s}$ = displacement due to prestressing

Δ_{cr+sh} = displacement due to creep and shrinkage

Δ_{temp} = displacement due to temperature effects

Precast abutments can be a very efficient solution for standard pier shapes or when accelerated bridge construction is necessary. Precasting can also be the best solution for unique sections that require high-quality concrete or geometry control, when there is a long lead time that allows the contractor to fabricate abutment sections concurrently with precast superstructure members, and when a precasting yard is located in the region. The cast-in- place backwall and the shear key are designed to resist the lateral seismic forces from the retained soil. If the weight of the abutment members is too high for conventional bridges, they may be prefabricated in segments and assembled on the jobsite.

ABUTMENT LONGITUDINAL RESPONSE FOR SDCS

The AASHTO SGS suggests that abutments designed for bridges in SDC B or C will likely resist earthquake loads with minimal damage. For seat-type abutments, minimal abutment movement could be expected under dynamic passive pressure conditions. However, bridge superstructure displacement demands may be 100 mm. or more and could potentially increase the soil mobilization.

For SDC D, passive pressure resistance in soils behind jointless abutment walls and backwalls for seat abutments will usually be mobilized because of the large longitudinal superstructure displacements associated with the inertial loads. The following two alternatives based on the AASHTO SGS may be considered:

> Case 1: Earthquake-Resisting System (ERS) without Abutment Contribution. The bridge ERS shall be designed to resist all seismic loads without any contribution from abutments. Abutments may contribute to limiting displacement, providing additional capacity and better performance that is not directly accounted for in the analytical model. To ensure that the columns will be able to resist the lateral loads, zero stiffness and capacity at the abutments should be assumed. In this case, an evaluation of the abutment that considers the implications of significant displacements from seismic accelerations shall be considered. As appropriate, this evaluation should include overturning for abutments.
>
> Case 2: Earthquake-Resisting System (ERS) with Abutment Contribution. In this case, the bridge could be designed with the abutments as a key element of the ERS. Abutments are designed and analyzed to sustain the design earthquake displacements. When abutment stiffness and capacity are included in the design, it should be recognized that the passive pressure zone mobilized by abutment displacement extends beyond the active pressure zone normally used for static service load design. Figure 6 shows abutment stiffness and passive pressure diagram.

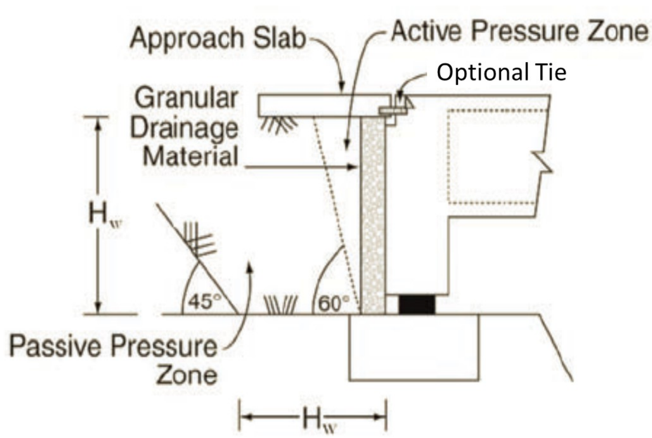

Figure 6. Abutment Stiffness and Passive Pressure

For transverse stiffness concrete shear keys shall be considered sacrificial where they are designed for lateral loads lower than the design earthquake loads as stated in the WSDOT Bridge Design Manual. A minimum level of design for shear keys corresponds to lateral loads not including earthquake loads. If sacrificial concrete shear keys are used to protect the deep foundation elements, the bridge shall be analyzed and designed as applicable. If a fuse is used, then the effects of internal force redistribution resulting from fusing shall be taken into account in the design of the bridge. The elastic resistance shall be taken to include the use of:

- Elastomeric bearings,
- Sliding, or isolation bearings designed to accommodate the design displacements,
- Breakaway elements, such as isolation bearings with a relatively high yield force;
- Shear keys;

- Yielding elements, such as wingwalls yielding at their junction with the abutment backwall;
- Elastomeric bearings whose connections have failed and upon which the superstructure is sliding;
- Spread footings that are proportioned to slide; or
- Deep foundation elements that develop a complete plastic mechanism.

ABUTMENT TRANSVERSE RESPONSE FOR SDCS

Transverse stops and shear keys are provided to resist the horizontal seismic force not less than the acceleration coefficient, A_s, times the tributary permanent load. Fusing is not expected for SDC B or C; however, if deemed necessary, fusing shall be checked using the procedure applicable to SDC D, taking into account the overstrength effects of shear keys. For structures in this category, either elastic resistance or fusing shall be used to accommodate transverse abutment loading. The elastic forces used for transverse abutment design shall be determined from an elastic demand analysis of the structure.

Where a shear key fusing mechanism is used for deep foundation supported abutments, the combined overstrength capacity of the shear keys shall be less than the combined plastic shear capacity of the deep foundation elements. Soil friction and passive earth pressure shall not be included in the transverse abutment resistance of deep foundation-supported abutments. For concrete shear keys that are not intended to fuse, the design should consider the unequal forces that may develop in each shear key.

For deep foundation-supported abutments, the stiffness contribution of deep foundation elements less than or equal to 450 mm in diameter or width shall be ignored if the abutment displacement is greater than 100 mm unless a displacement capacity analysis of the deep foundation elements is performed and the deep foundation elements are shown to be capable of accommodating the demands.

SUPERSTRUCTURE CONTINUITY AT PIERS

Piers for jointless bridges have similar design requirements and share common design procedures with those of piers of more traditional bridge types. The primary distinguishing features of the piers of a jointless bridge involve accommodation of potentially large superstructure movements and the sharing of transverse (perpendicular to the longitudinal centerline of the bridge) and longitudinal (parallel to the centerline of the bridge) forces among substructure units.

To successfully design the piers to accommodate potentially large superstructure movements, the designer has several options:

1. Flexible bents - rigidly connected to the superstructure;
2. Isolated rigid piers - connected to the superstructure by means of flexible bearings;
3. Semi-rigid piers- connected to the superstructure with dowels and elastomeric bearing bearing pads; or
4. Hinged-base piers - connected to the superstructure with dowels and elastomeric bearing bearing pads.

The most basic precast bridge consists of precast, prestressed concrete beams made continuous for live load by forming and placing a continuous deck. Precast beams are erected onto the cap and temporarily supported on elastomeric bearings or wood blocks until the cast-in-place concrete diaphragm is complete. The strands from the beam ends are sometimes extended for additional continuity.

Piers supporting long, multiple-span jointless superstructures frequently require specialized analytical models to predict transverse load distributions, forces induced as a result of superstructure movements, pier stiffness, and slenderness effects.

FIXED CONNECTIONS AT INTERMEDIATE PIERS

Fixed piers are defined as piers whose base is considered fixed against rotation and translation. The connection to the superstructure is usually detailed in a way restrain free longitudinal transverse movements. This type of detailing permits the superstructure to undergo thermal movements freely, yet allows the pier to participate in carrying transverse forces.

In modern precast concrete bridges with this type of pier, the superstructure is supported on relatively tall laminated elastomeric bearing bearing pads. A shear block, isolated from the pier diaphragm with a compressible material such as cork, is cast on the top of the pier cap to guide the movement longitudinally, while restraining transverse movements as shown in Figure 7.

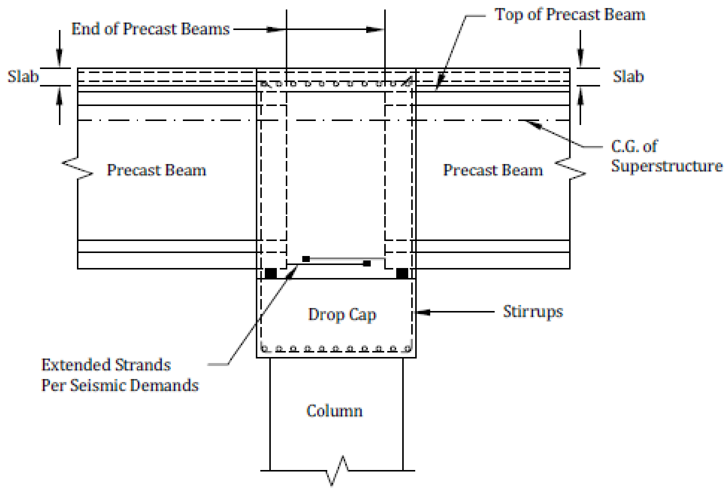

Figure 7. Fixed Pier Detail

HINGED CONNECTION AT INTERMEDIATE PIERS

A typical semi-rigid pier superstructure connection is shown in Figure 8. The precast girders bear on elastomeric pads 10 to 40 mm thick. A diaphragm is placed between the ends of the girders, and dowels, perhaps combined with a shear key between girders, connect the diaphragm to the pier cap. Compressible materials are frequently introduced along the edges of the diaphragm and, along with the elastomeric bearing pads, allow the girders to rotate freely under live load.

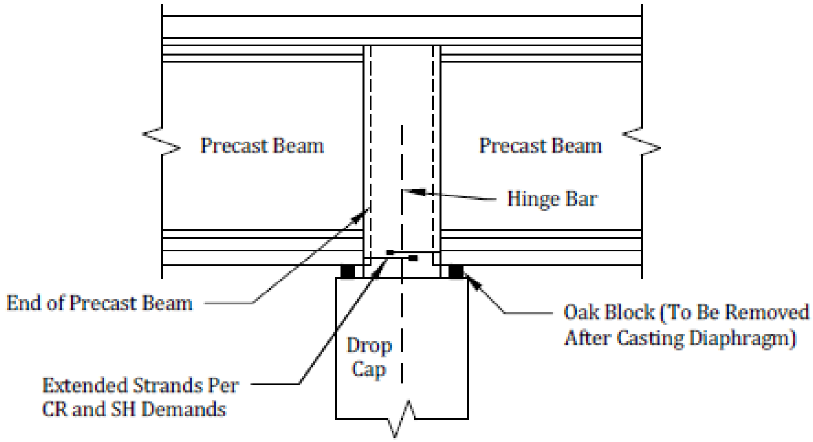

Figure 8. Hinge Connections at Intermediate Piers

The requirements for using this detail are:

- All beams of adjoining spans should be of equal depth, spacing, and type
- Reinforcement for negative moments due to live loads and superimposed dead loads from traffic barriers, pedestrian walkways, utilities, etc. is provided in the deck at intermediate piers

The hinge bar size and spacing is designed for anticipated lateral loads due to seismic and other load combinations. Distinction must be made between slab continuity and girder continuity at the piers. For a bridge to be classified as a jointless bridge, it is obvious that the slab must be physically continuous. Girder continuity at the piers, however, is not a necessity unless the superstructure is designed for continuity. Lack of girder continuity decreases the redundancy of the structure and increases its vulnerability to catastrophic events such as the loss of girder end pier supports observed in Chili Earthquake. Deck continuity at piers not only eliminates the potential leakage of water through expansion joints.

CONCLUSIONS

The use of jointless bridges with jointless abutments is growing in the United States, because of the benefits achieved in lowering first cost in construction and minimizing future maintenance. Further benefits of this type construction are design efficiency, added system redundancy, ease of construction and greater flexibility in span arrangement particularly with fully continuous beam systems.

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