

HIGHWAY STRUCTURES
DESIGN HANDBOOK
Volume I, Chapter 13

SUBSTRUCTURE
DESIGN
FOR
STEEL
BRIDGES

AISC MARKETING, INC.
437 Grant Street/Suite 1615
Pittsburgh, PA 15219
Telephone: (412) 394-3700

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AISC MARKETING, INC.

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I. Introduction

A continuing need for new bridges, while construction costs rise steadily and revenues fall, requires optimum utilization of available highway monies.

To accomplish this, efficiency and economy in all aspects of bridge design must be sought and carried out. In recent years, considerable effort has been made to develop useful publications that provide up-to-date information for producing safe, cost-efficient designs. Some such publications include: the *Highway Structures Design Handbook*, Volumes I and II, and the *Composite Steel Plate Girder Bridge Superstructures Handbook*. These design aids, along with promotion of load factor design, composite construction and simplification of details are intended to help achieve greater bridge economy principally with respect to steel superstructures.

But, to achieve maximum overall efficiency, it is necessary to emphasize a consistent and economical approach to *both* superstructure and substructure. Expenditures for substructures are a large portion of the total cost of steel bridges, particularly long span, high clearance types where significant transverse and longitudinal loads must be transmitted to the foundations.

This publication reviews and summarizes various aspects of substructure design, especially as it pertains to steel plate-girder bridge systems. Also included are design concepts that can minimize substructure costs.

The integrity of an entire bridge system depends on at least one important feature of substructure design: proper selection and design of the type of foundation, whether it be spread footings, caissons or piles. This selection must only be made *after* a comprehensive subsurface investigation by a geotechnical specialist working hand-in-hand with the bridge engineer. Criteria for design and use of steel piling is given in Chapter 10, Volume I, and Chapter 11, Volume II, of the *Highway Structures Design Handbook*. General criteria for selection and design of foundation types may be found in many geotechnical references; extensive discussion is beyond the scope of this paper.

The discussion that follows is based on the *Standard Specifications for Highway Bridges*, Thirteenth Edition, 1983, adopted by the American Association of State Highway and Transportation Officials (AASHTO); a specification that governs all aspects of bridge design.

II. Loading Concepts

A. General

Proper substructure design requires a thorough understanding of the interaction between superstructure and substructure. Because of the many possible loads and load combinations, analysis can be very complex. AASHTO Section 3, TYPES OF LOADS, specifies the type and magnitude of loads that should be considered in substructure design. While some of these loads are applied directly to the substructure, others are applied to the superstructure and then transmitted to the substructure through the bridge bearings.

Some of the loads to be resisted by the substructure include:

- 1) Dead Load from the Superstructure
- 2) Dead Load of the Substructure
- 3) Live Load and Impact from the Superstructure
- 4) Wind Loads on the Superstructure and Substructure
- 5) Wind Load on the Live Load
- 6) Centrifugal Force from the Superstructure
- 7) Longitudinal Force from Live Load
- 8) Dead Load Friction from Bearings
- 9) Earth Pressure
- 10) Stream Flow Pressure
- 11) Ice Pressure
- 12) Earthquake Forces
- 13) Thermal and Shrinkage Forces
- 14) Ship Impact Forces

What follows is an interpretation, based on AASHTO Specifications, of a) the calculation and application of loads normally considered in substructure design, and b) how these loads are transmitted to the foundations. Many State Transportation Departments also using AASHTO Specifications, have developed similar criteria for the application of longitudinal and transverse forces to piers, and for combining loads into various design groups. However, there are certain variations among these procedures; hence, methods discussed herein are proposed for consideration by all agencies as an attempt to develop a more consistent design approach.

B. Live Load Distribution

Highway live loading is that loading specified or approved by the contracting agency, i.e., Standard AASHTO Highway Loadings, Section 3.7, or any other special loading condition peculiar to the type or location of the bridge structure.

Live load reactions obtained from design of individual members of the superstructure should not be used for substructure design. These reactions are based upon maximum conditions for one beam and make no allowance for distribution of live loading across the roadway. Use of these maximum loadings would result in a pier design with an unrealistically severe loading condition and uneconomical sections.

For substructure design, a maximum design traffic lane reaction using either the standard truck load or standard lane load should be used. Design traffic lanes are determined according to AASHTO

Section 3.6. For the calculation of the actual beam reactions on the pier, the maximum lane reaction can be applied within the design traffic lanes as wheel loads, and then distributed to the beams assuming the slab between beams to be simply supported (Fig. 1). Wheel loads can be positioned anywhere within the design traffic lane with a minimum distance between lane boundary and wheel load of 2 ft.

Arrange the design traffic lanes and the live load within the lanes to produce beam reactions that result in maximum stresses in the pier. AASHTO Section 3.12 provides for load reductions due to multiple lanes being loaded simultaneously.

Live load stresses are increased due to impact effect. The amount of increase and the structures subjected to this loading are specified in AASHTO Section 3.8.

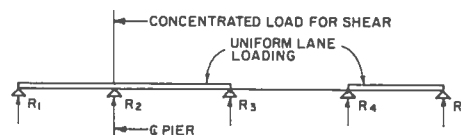
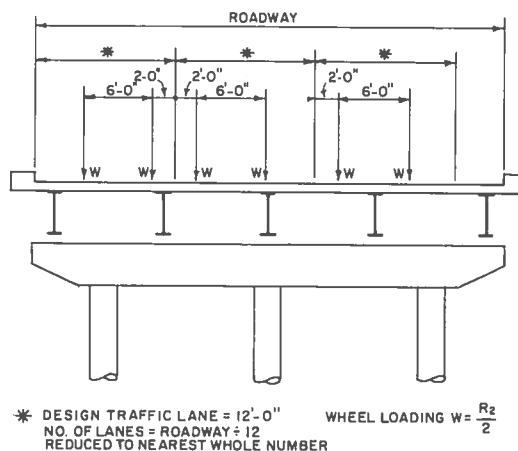


Figure 1. Wheel loading.

D. Longitudinal Loading

The longitudinal forces acting on the substructure or transferred to substructure from the superstructure include:

1) Longitudinal Force from Live Load (LF)

The magnitude of LF for H20 and HS20 loading in kips = $(0.64 \times \text{bridge length} + 18) \times (\text{Number of design traffic lanes loaded with one direction traffic}) \times 5\% \times (\text{Reduction factor for multiple lane loadings, AASHTO Section 3.12}) \times (\text{Section 3.9.1})$.

2) Wind Loads

a. Wind Load of Superstructure (W_L).

The magnitude of W_L = (Area of the exterior beam, roadway slab and parapet per lineal foot as seen in the transverse elevation) \times (Bridge length) \times (Unit longitudinal wind load as specified in AASHTO Section 3.15.2.1).

b. Wind Load on the Substructure (W_{SL}).

The magnitude of W_{SL} = (Area of the exposed substructure as seen in the longitudinal elevation) \times (Unit longitudinal wind load as specified in AASHTO Section 3.15.2.2).

Apply the longitudinal wind W_{SL} at the center of gravity of the exposed area.

c. Wind Load on a Moving Live Load (WL_L)

The magnitude of WL_L = (Bridge length) \times (Unit wind load on a moving live load as specified in AASHTO Section 3.15.2.1.2).

3) Friction Forces

Friction forces are developed at bridge expansion bearings as a resistance to movement. Temperature variations causing expansion and contraction of the superstructure as well as longitudinal loads from live load and wind are all resisted to a certain extent by friction at expansion shoes. The maximum friction force that can be sustained, however, is that force necessary to initiate movement.

The maximum friction force that can be developed at typical bridge bearings is as follows (Fig. 5):

- a. At sliding expansion bearings the maximum friction force (T) is equal to:

$$T = DL\mu$$

DL = Total dead load reaction at the support

μ = Coefficient of friction

μ is determined by the type of material in sliding contact. The value may be recommended by the manufacturer or specified by the contracting agency. Typical values are 0.25 for steel bearing on steel and 0.10 for self lubricating bronze plates.

- b. For steel bearing rocker bearings the mechanical properties of the rocker determine the friction force (T).

$$T = DL\mu' \text{ where } \mu' = .25 \frac{r}{R}$$

DL = Total dead load reaction at the support

r = Radius of pin

R = Radius of rocker

The coefficient μ' for rocker bearings is analogous to μ for sliding bearings. A typical value of μ' for rocker bearings is .05.

- c. At fixed supports the coefficient of friction is assumed to be infinite, therefore the longitudinal capacity of a fixed support is infinite. The force developed at the fixed pier will be the balancing force that exists when all the expansion bearings are loaded to their longitudinal capacity.

- d. Elastomeric bearing design should follow the specifications of AASHTO Section 14 and the DuPont publication "Design of Neoprene Bridge Bearing Pads," except as modified by the contracting agency standards or design criteria.

The shearing resistance of the designed bearing is equivalent to the friction force that is developed at a conventional sliding plate or rocker type bearing.

$$R = \frac{M L T \Delta}{E.R.T.}$$

R = Shearing resistance of bearing

M = Shearing modulus of neoprene, dependent upon hardness and temperature

L = Bearing length

T = Bearing width

Δ = Maximum anticipated deflection

E.R.T. = Effective Rubber Thickness

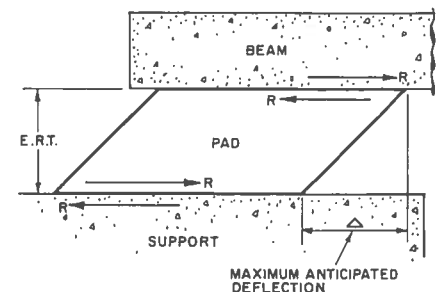
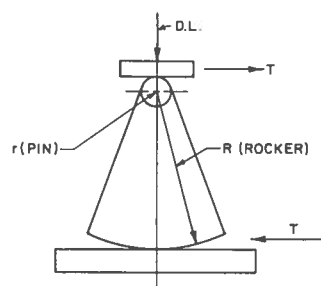
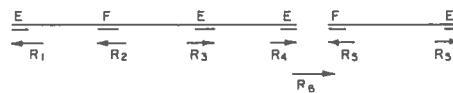


Figure 5.

The force developed at a support is the sum of the forces developed at each bearing. Unlike conventional bearings the elastomeric bearing does not relieve itself by sliding or rotation, therefore, the developed force (R) is dependent upon the deflection (Δ).

It is assumed that fixed elastomeric bearings are prevented from distorting, therefore, the friction force developed at the fixed pier will be that balancing force which exists when all the expansion bearings have attained their maximum anticipated deflection.



Distribution of Friction Forces

$$R_2 = R_3 + R_4 - R_1 \quad R_5 = R_6 - R_4$$

4) Other Forces

All other forces which are applied directly to the substructure in the longitudinal direction, must be considered in the appropriate AASHTO group loadings of Section 3.22. Such loads may include the influence of stream current, floating or freezing ice, drift, earth pressure or earthquake forces. Calculation and application of these loads is specified in AASHTO Section 3, LOADS.

E. Longitudinal Load Application

The loadings from the superstructure that must be resisted by the supports are wind load on the superstructure, wind load on a moving live load and longitudinal force from live loads and from friction due to temperature movement. For the longitudinal force distribution, with conventional bearings, it will be assumed that each load is resisted equally by the fixed support and all expansion supports that have not reached their longitudinal capacity in the direction of loading. (Sometimes expansion bearings are omitted when the piers are flexible enough to permit some longitudinal movement. The relative stiffness of the piers should be considered when distributing longitudinal forces to several piers within such a continuous system. Taller piers will tend to deflect more easily, causing longitudinal load to redistribute to shorter, stiffer piers.)

When an expansion support becomes loaded to its longitudinal capacity the bearing will give. At this point, the remaining longitudinal force must be resisted by all the other bearings that have not attained their capacity. If all the expansion bearings become loaded to their capacity, the remaining load and all additional loadings must be resisted by the fixed support.

The distribution of longitudinal forces makes it necessary to investigate the total structure. This is necessary because any number of load combinations can cause an expansion bearing to reach its load capacity. Load combinations may be investigated according to the following AASHTO group loads:

- 1) Group II
 - a. Full Wind on the superstructure (W_L)
- 2) Group III
 - a. 0.3 Full Wind on superstructure
 - b. Wind on Live Load
 - c. Live Load Traction Force

C. Transverse Loading

The transverse forces acting on the substructure or transferred to the substructure from the superstructure include:

1) Wind Loads

a. Wind Load on the superstructure (W_t)

The magnitude of W_t = (area of exterior beam, roadway slab and parapet per lineal ft as seen in the transverse elevation) x (Average span length of two spans adjacent to the pier under consideration) x (Unit transverse wind load as specified in AASHTO Section 3.15.2.1.) Apply the transverse wind (W_t) at the C.G. of the exposed areas, except as modified for frame analysis (Fig. 2).

b. Wind Load on Substructure (WOS_t)

The magnitude of WOS_t = (area of exposed substructure as seen in the transverse elevation) x (Unit transverse wind load as specified in AASHTO Section 3.15.2.2.) Apply the transverse wind (WOS_t) at the C.G. of the exposed area, except as modified for frame analysis (Fig. 3).

c. Wind Overturning Forces (WOF)

The magnitude and application of WOF is specified in AASHTO Section 3.15.2. Apply this force in the design of hammerhead or T piers, but it is not necessary to investigate this loading in the design of rigid frame bent piers.

d. Wind Load on Moving Live Load (WL_t)

The magnitude of WL_t = (Average span length of two spans adjacent to the pier under consideration) x (Unit wind on a moving live load as specified in AASHTO Section 3.15.2.1.2.) Apply WL_t 6 ft above the roadway, except as modified for frame analysis.

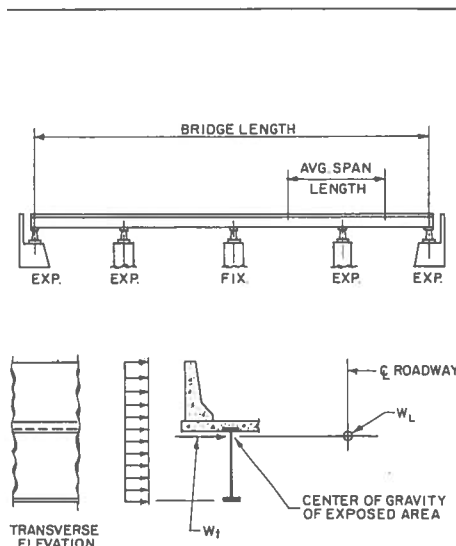


Figure 2.

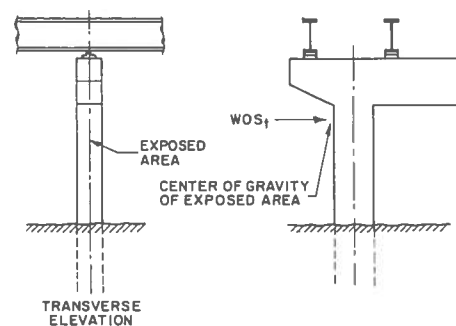


Figure 3.

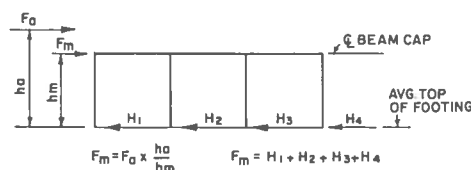


Figure 4.

2) Centrifugal Force

Calculate and apply the horizontal radial force (CF) due to structure curvature as specified in AASHTO Section 3.10, except as modified for frame analysis.

3) Other Forces

All other conditions which may subject the structure to transverse load should be considered in the appropriate group of loadings of AASHTO Section 3.22. Such loads may include the influence of stream current, floating or freezing ice, drift, earth pressure and earthquake forces, which will be discussed in a later section of this report. In the case of rigid frame bent piers, internal stresses due to temperature change and shrinkage forces are included in the appropriate AASHTO group loadings.

Piers constructed in navigable waterways may be subject to ship impact forces.

Transverse forces are applied at the elevations specified in AASHTO for the design of hammerhead or T piers.

For the analysis of rigid frame bents, some agencies modify the horizontal forces (Fig. 4), in order to approximate the effect the forces would have on the frame had they been applied at the specified elevations.

The total F_m applied at the centerline of the beam cap is equal to the summation of all modified individual transverse forces. It should be recognized that $F_m > F_a$, therefore, resultant transverse H is greater than actual applied loads and is a conservative method of design. For footing design, the original forces, F_a , should be used to determine column shears at the top of footing.

- 3) Group VI_a
- Friction Force due to temperature drop
 - 0.3 Full Wind on the superstructure
 - Wind on Live Load
 - Live Load Traction Force
- 4) Group VI_b
- Friction Force due to temperature rise
 - 0.3 Full Wind on the superstructure
 - Wind on Live Load
 - Live Load Traction Force
- 5) Group VI_c
- 0.3 Full Wind on superstructure
 - Wind on Live Load
 - Live Load Traction Force
 - Friction Force needed to reach longitudinal capacity of the support.

In all cases the loads are applied in the order listed until the capacity of the support is attained.

The magnitude of longitudinal forces applied to the piers is governed by the type of bearing, and the following conditions usually govern. The group loading resulting in the largest applied forces should be used for design.

Type of Bearing on Pier	Investigate
Fixed Bearings	Group III, VI _a and VI _b *
Expansion Bearings	Group III and VI _c

If a structure is discontinuous at a pier the effect of loading from both spans should be considered when accumulating loadings for the pier design.

The following design example for a four-span structure illustrates the division of longitudinal forces between piers (Fig. 6).

*Loads on fixed bearings may be governed by Group V_a (Full Wind on the Superstructure + Friction Force due to temperature drop) or Group V_b (Full Wind on the Superstructure + Friction Force due to Temperature rise) if the girders are deep.

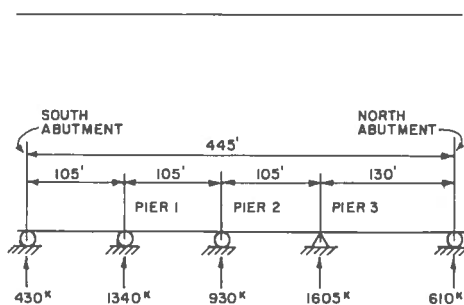


Figure 6.

Given 4-span continuous bridge

Dead Load reactions as shown
Expansion Bearings—rockers pin $r = 2\frac{1}{2}$ "
rocker $R = 1'-2"$

$$T = 0.25 \text{ DL } \frac{r}{R} = 0.044 \text{ DL}$$

Effective bridge depth for wind =
8'-0" all spans,
4 lanes of HS20-44 loading,
3 lanes one-way

Friction Forces

$$\begin{aligned} \text{S. Abut } 430 \times 0.044 &= 19 \text{ k} \\ \text{Pier 1 } 1340 \times 0.044 &= 59 \text{ k} \\ 2 \quad 930 \times 0.044 &= 41 \text{ k} \\ 3 \text{ Unlimited} &= \text{Unlimited} \\ \text{N. Abut } 610 \times 0.044 &= 27 \text{ k} \end{aligned}$$

Full Longitudinal Wind on Superstructure W_L

$$\begin{aligned} \text{Assuming } 60^\circ \text{ wind skew} \\ W_L = 0.019 \times 445 \times 8.0 &= 68 \text{ k—AASHTO} \\ 3.15.2.1 \end{aligned}$$

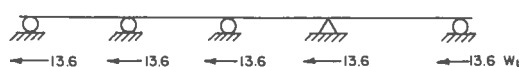
Full Longitudinal Wind on the Live Load WL_L

$$\begin{aligned} WL_L 60^\circ &= 0.038 \times 445 = 17 \text{ k—AASHTO} \\ 3.15.2.1 \end{aligned}$$

Longitudinal Force LF

$$\begin{aligned} LF &= 0.05 \times 3 (0.640 \times 445 + 18) 0.9 = 41 \text{ k} \\ \text{—AASHTO 3.9.1} \end{aligned}$$

Distribution of Full $WL = 68 \text{ k—Group II}$



Distribution of Forces—Group III

← 4.1	← 4.1	← 4.1	← 4.1	← 4.1	$0.3W_L = 20.4^k$
← 3.4	← 3.4	← 3.4	← 3.4	← 3.4	$W_{LL} = 17.0^k$
← 8.2	← 8.2	← 8.2	← 8.2	← 8.2	$LF = 41.0^k$
← 15.7	← 15.7	← 15.7	← 15.7	← 15.7	

Distribution of Forces, Pier 1—expansion pier use Group VI_c

$R = 19$ 	$R = 59$ 	$R = 41$ 	$R = \text{UNL'D}$ 	$R = 27$ 	
← 4.1	← 4.1	← 4.1	← 4.1	← 4.1	$0.3W_L = 20.4^k$
← 3.4	← 3.4	← 3.4	← 3.4	← 3.4	$W_{LL} = 17.0^k$
← 8.2	← 8.2	← 8.2	← 8.2	← 8.2	$LF = 41.0^k$
← 15.7	← 15.7	← 15.7	← 15.7	← 15.7	
	← 43.3 (= 59.0 - 15.7)				
	← 59.0				ADDITIONAL FRICTION FORCE

Distribution of Forces, Pier 3—fixed pier use Group VI_a or Group VI_b

Group VI_a

→ 19	→ 59	→ 41	← 92	← 27	= 0
← 5.1	← 5.1	← 5.1	← 5.1		$0.3W_L = 20.4^k$
← 4.3	← 4.3	← 4.3	← 4.3		$W_{LL} = 17.0^k$
← 10.3	← 10.3	← 10.3	← 10.3		$LF = 41.0^k$
← 0.7	→ 39.3	→ 21.3	← 111.7	← 27	

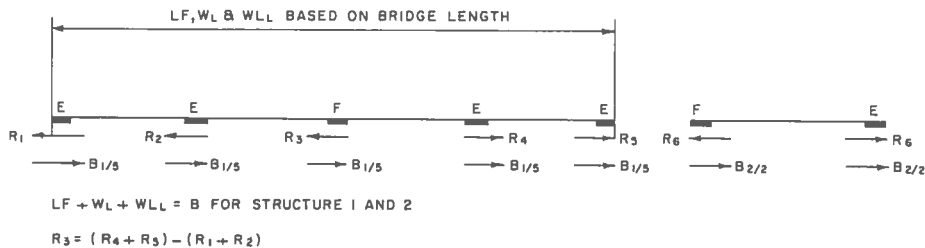
Group VI_b

← 19	← 59	← 41	→ 92	→ 27	= 0
			← 10.2	← 10.2	$0.3W_L = 20.4^k$
			← 8.5	← 8.5	$W_{LL} = 17.0^k$
			← 20.5	← 20.5	$LF = 41.0$
← 19	← 59	← 41	→ 52.8	← 12.2	

Group VI is the controlling loading at the fixed pier. Note that this pier was subjected to an additional 92 kips that balanced adjacent friction forces. It is possible in other configurations that this balancing force could be zero, due to symmetrical spans and symmetrical dead load reactions. Some agencies specify that the fixed pier be designed with a force not less than the friction force

applied to an adjacent expansion pier. Elastomeric bearings have a high resistance to deformation when subjected to longitudinal forces and behave more like fixed bearings. Therefore, it is assumed that all supports contribute equally in resisting the applied longitudinal loads and that these loads act coincidentally with and in the same direction as the equivalent friction force (R).

Elastomeric Bearings



Common assumptions for the height of application of longitudinal forces are as follows:

- 1) For all conventional expansion and all elastomeric bearing, the forces shall be applied at the beam seat elevation.
- 2) For all conventional fixed bearings the load shall be applied 1 ft (1'-0") above the top of the cap beam.

F. Skewed Piers

The previous discussion of load determination and application has been for a pier with 0° skew. The skew angle is measured from the perpendicular to the longitudinal axis (Fig. 7).

When the skew angle is not equal to zero, the longitudinal and transverse forces are resolved into component forces which are normal and parallel to the centerline of the pier. The resolved components should be combined, disregarding the point of application, to produce the maximum forces to be applied to the pier. Because of the skew,

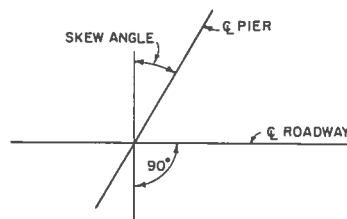


Figure 7.

investigation of several loading conditions may be required to determine the maximum normal force, the maximum parallel forces or the loading combination that produces the most critical condition for design (Fig. 8).

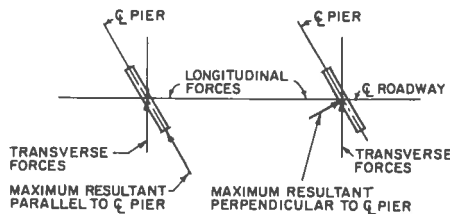


Figure 8.

Note that if components parallel to the pier add, components normal to the pier subtract. If either the applied longitudinal or transverse forces were in the opposite direction, components parallel to the pier would subtract and components normal to the pier would add.

These resultant forces are then applied to the pier as specified for transverse and longitudinal forces, dependent upon the type of pier and type of bearing.

G. Load Combinations

It is possible for any part or all of the structure to be subjected to several of the previously mentioned loads simultaneously. Any reasonable combination of loadings can occur and AASHTO

Section 3.22 has set up ten different loading groups to be used in design. These loads and combinations of loads may be modified to fit particular requirements as specified by the contracting agency.

The concept of group loadings implies that some loads can probably act simultaneously. In service load design, an increase in allowable stresses is specified for certain group loadings. In load factor design, lower load factor coefficients are specified for certain load groups. This is allowed because it is recognized that the maximum values of all forces acting simultaneously will occur very few times in the life of the structure. Therefore, it is reasonable for the factors of safety to be reduced.

In the design of substructures, several group loadings may require investigation to determine the most critical case. One group load may govern stability, another may produce maximum foundation pressures and another may produce maximum stresses in the column design. Usually, pier design is governed by Group I, Group III or Group VI. Rigid frame bents are often governed by Group VI. Abutment design is usually governed by Group I loading.

Each structure must be analyzed for the loadings which most probably will occur and the design based on the group loadings producing the maximum stresses.

H. Earthquake Considerations

AASHTO Figures 3.21.1E, F and G show the Seismic Risk Maps of the United States and outline the geographical areas with their associated Zones of seismic activity. United States Coast and Geodetic Survey (USCGS) publishes a similar map and defines the zones as:

- Zone 0—no damage
- Zone 1—minor damage
- Zone 2—moderate damage
- Zone 3—major damage

AASHTO and USCGS specify A, the maximum expected rock acceleration,

for each zone. The comparison of A of the two agencies shows that AASHTO uses a slightly higher value in Zones 1 and 2, but makes no allowance, as USCGS does in Zone 3, for the location of the area in relation to any major active fault.

The Bulletin of the International Seismological Center (BISC) lists earthquake data recorded globally since 1918.

When bridges have been destroyed by earthquakes, it is normally not the result of superstructure failure. The failures are caused by 1) Overturning of piers, 2) Loss of soil strength or settlement under the piers, 3) Superstructure being shaken off its bearings, and 4) Structural failure of the supporting piers. All failures are initiated by ground motion. The first two causes could be called static, although initiated by earthquake ground vibrations. The resulting large scale earth motion occurs relatively slowly and does not set up appreciable inertia forces in the piers. The last two causes are dynamic responses of the piers to earthquake ground accelerations and are affected by the elastic characteristics and distribution of weight of the structure. The pier analysis for this condition involves structural dynamics and becomes quite involved.

AASHTO Section 3.21.1.1 specifies the lateral load (EQ) that is to be applied due to earthquake. This greatly simplified loading is an equivalent static force equal to a percentage of the weight of the structure under investigation. The force is applied as a horizontal force at the center of the gravity of the structure and acting in any direction. Earthquake loading is considered in AASHTO Group Loading VII which combines Dead Load and Earthquake. When applied longitudinally, EQ load from the superstructure should be applied to the substructure at the bearing elevation, in a manner similar to other longitudinal loads.

The Department of Transportation in California, because of the high risk condition that exists, has modified AASHTO Section 3.21.1.1 based on

Figure 9. Collision forces recommended or used in various bridge projects:

- 1) Storebaelt Bridge, Denmark, from Rasmussen (1982).
- 2) Oresund Bridge, Sweden/Denmark, from Von Olhausen (1983).
- 3) Luling Bridge, USA.
- 4) Great Belt Bridge, Denmark, from Fjeld (1982).
- 5) Faro Bridges, Denmark, from Jensen and Sorensen (1983).
- 6) Bahrein, Saudi Arabia, from Fjeld (1982).

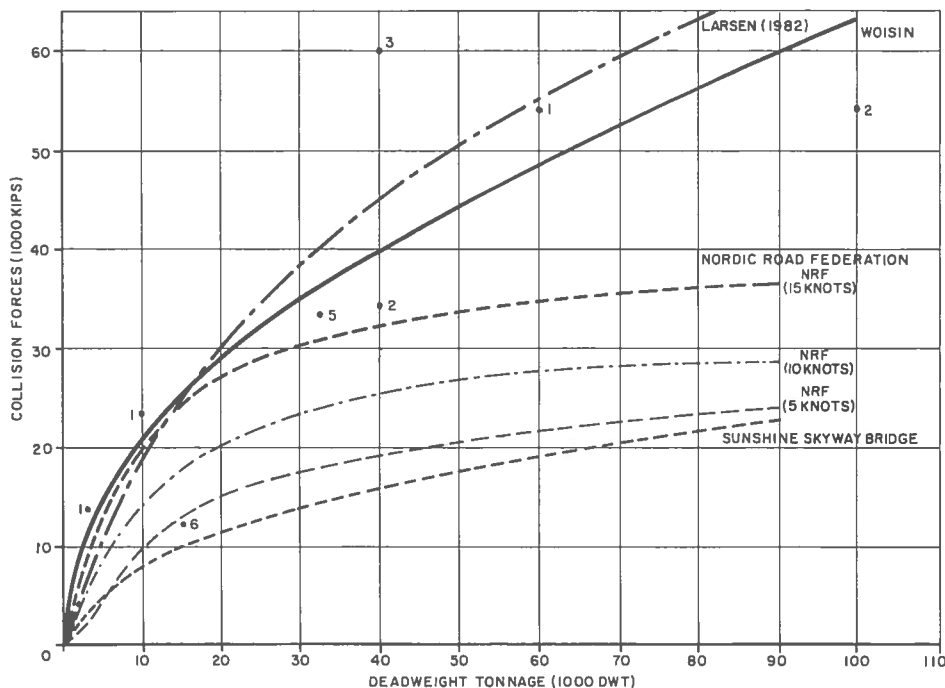
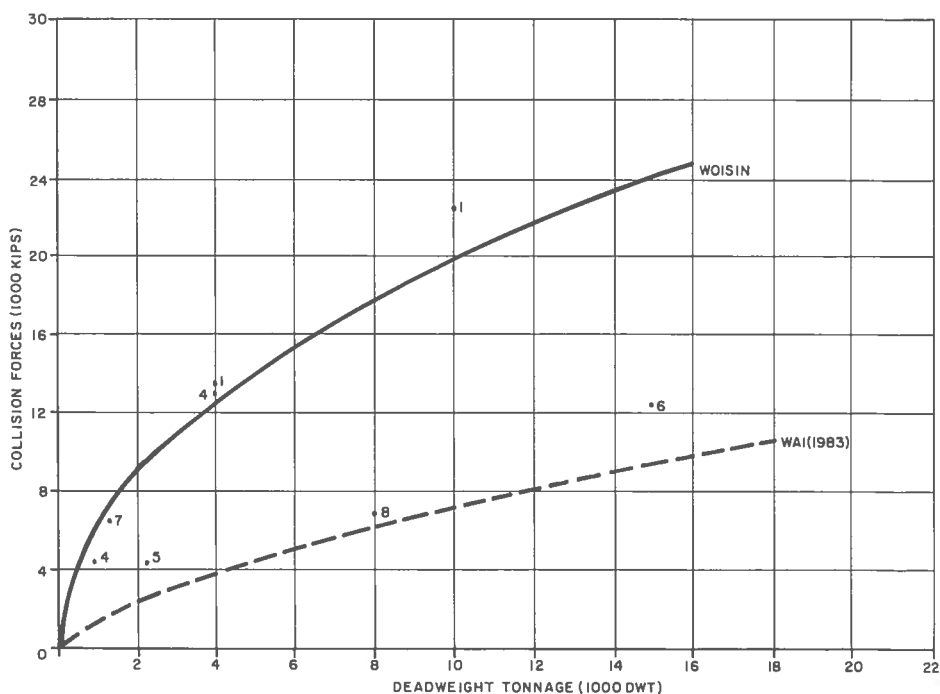


Figure 10. Forces in the lower energy range recommended or used in various bridge projects:

- 1) Storebaelt Bridge, Denmark, from Rasmussen (1982).
- 4) Great Belt Bridge, Denmark, from Fjeld (1982).
- 5) Faro Bridges, Denmark, from Jensen and Sorensen (1983).
- 6) Bahrein, Saudi Arabia, from Fjeld (1982).
- 7) New German Railway Code (Bridge Piers Across the Rhine River), from Saul and Sorensen (1982).
- 8) Norwegian Bridge Piers, from Per Tambs-Lyche (1983).



earthquake research and documented investigation of past seismic activity. This modified section relates specifically to the earthquake problem in California.

Many state transportation departments in low risk seismic areas do not consider earthquake loading. When considered, however, even in Zone 1 areas, AASHTO Group VII (DL + EQ) can be the critical load combination for pier column and footing design.

1. Ship Impact Forces

The design of piers in navigable waterways should take into account the possibility of ship impact. Piers should be located and protected to minimize the possibility of vessel collision with the piers. If an unprotected pier can be reached by an errant vessel, the pier should be designed for ship impact forces.

The forces developed during ship impact depend upon several factors such as:

- 1) Vessel Characteristics
 - a) Size and displacement
 - b) Construction, strength and crushing behavior
 - c) Velocity
- 2) Pier Characteristics
 - a) Geometry
 - b) Mass
 - c) Strength and deformation characteristics
 - d) Type of foundation and soil properties

3) Location and Direction of Impact
A dynamic analysis is needed to calculate properly the forces developed during a vessel-pier collision. Such an analysis is usually not practical, and equivalent static ship impact forces are calculated by several empirical formulas, some of which are based upon model tests and/or studies of actual collisions.

The report, "Criteria for the Design of Bridge Piers with Respect to Vessel Collision in Louisiana Waterways," as prepared for the Louisiana Department of Transportation and Development and the Federal Highway Administration by Modjeski and Masters, Consulting Engineers, November, 1984, reviews the literature and presents methods for estimating ship impact forces in Louisiana waterways. Figures II-5 and II-6 from that report show plots of some of the proposed formulas and collision forces that have been used for various bridges (Figs. 9 & 10).

The formulas and collision forces given in that report vary greatly. The forces to be used at a specific bridge site should be established by the owner or jointly by the owner and engineer.

The maximum collision force is assumed to occur parallel to the shipping channel. Some criteria approximate an angle collision by applying half the maximum collision force at right angles to the channel. The elevation of the applied ship impact force is based upon ship characteristics and variations in water level.

Transfer of a portion of ship impact force from the impacted pier through the superstructure to the adjacent piers should be considered.

The superstructure can be especially effective in transferring the component of impact force that acts parallel to the longitudinal axis of the bridge. Fixed bearings or expansion bearings with shear keys to limit movement will transfer load from the impacted pier to the superstructure, and the superstructure acting in tension or compression will transfer load to the adjacent piers.

The superstructure can also transfer a portion of the impact force that acts normal to the bridge. The portion that can be transferred to adjacent piers depends upon many factors such as:

- 1) The stiffness of the impacted pier, the stiffness of the superstructure in lateral bending, and the stiffness of the adjacent piers.
- 2) The elevation at which the impact load is applied.
- 3) The bridge bearing details.
- 4) The stiffness of the bridge cross frame above the pier.

Greater load can be transferred to adjacent piers when the superstructure is laterally stiff, and the bearing details and the cross frames above the bearings are adequate to transfer load from the piers to the superstructure. An analysis of a two-dimensional model will determine the percentage of load transferred to adjacent piers.

III. Pier Configuration

A. General Considerations

To achieve maximum economy on large structures, pier type and configuration should be determined on a project-by-project basis. Various parameters that must be considered include the type and width of superstructure, skew, overall pier height, vertical and horizontal clearance requirements, physical location, foundation requirements, and aesthetics. Of principal concern is the lifetime durability and integrity of the pier with respect to the magnitude and distribution of applied load from the superstructure.

The most commonly used pier types, are the Solid Shaft Pier, Hammerhead or T-Pier, and the Open Bent or Rigid Frame Pier (Fig. 11).

Solid shaft piers are used primarily for river or stream crossings, low clearance bridges, and in some instances for grade crossings over divided highways where raised medians can create low clearances at pier locations. Solid piers are also used for crash protection adjacent to railroads.

With increasing pier height and width, the T-Pier and Rigid Frame Pier become more economical than Solid Shaft piers with reductions in material and forming. For high-level river crossings, T-Piers and Rigid Frame Piers are often combined with solid shaft bases with rounded or protected ends that extend above maximum flood elevation (Fig. 12).

In selecting the optimum pier type for a large bridge, all of the above factors should be considered. Preliminary designs should be made for various configurations for cost comparison. Scale drawings of each configuration with the superstructure should be made to evaluate the final proportions and aesthetics.

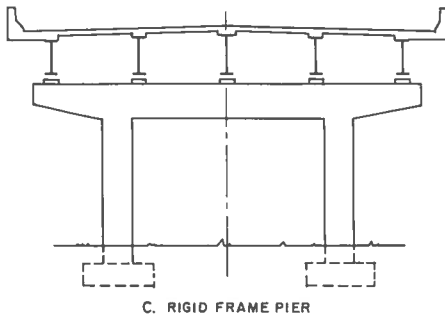
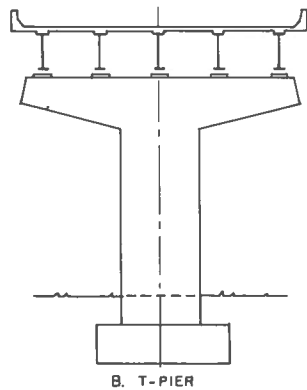
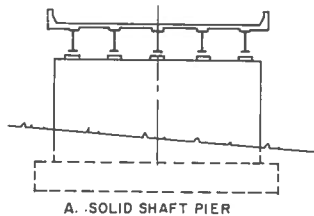


Figure 11. Typical pier types

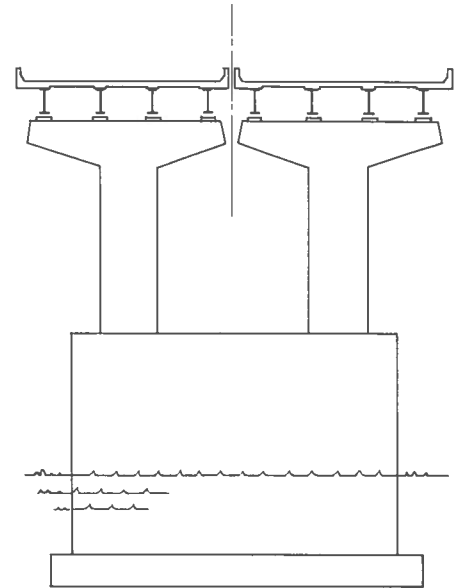


Figure 12. Combination T-Pier with solid shaft base

B. T-Piers vs Rigid Frame Piers

For long span, high-clearance bridges, T-Piers and Rigid Frame Piers are the most common types used. Each type has certain advantages and disadvantages to consider.

For narrower superstructures, T-Pier construction minimizes forming by using only a single column. Foundation requirements are generally simpler. With a single column there are no internal shrinkage and temperature forces built up that could affect the design.

As superstructure widths increase, the cantilevered cap beam of the T-Pier type becomes the critical element of design. For an optimum column size, a long cantilever supporting heavy reactions may require an excessive depth for the cantilever beam. To reduce the length of the cantilever, the column must be widened at the top, resulting in loss of economy.

Lateral loads in the T-Pier are transmitted to the foundations by the column acting as a vertical cantilever. In high-level bridges, beam-column action, that results in moment amplification, can have a significant effect on the column proportions. In such instances, rigid frame piers, that have greater stiffness in the transverse direction, will allow more efficient column design and may result in greater economy. Multiple columns in the rigid frame pier will also allow the use of a shallower cap beam.

Several states have criteria giving general proportions and limiting dimensions for T-Piers and rigid frame piers when used for grade separation bridges. These criteria and dimensions have been developed primarily as guides for desired general appearance. Member proportions should always be adjusted to minimize stresses and produce economical design, consistent with good aesthetics. For large, high clearance bridges, these criteria, especially the limiting length-to-height ratios and maximum pier lengths, do not necessarily apply. Ultimate choice of a pier type should be made only after careful study.

C. Forming Considerations

In heavy reinforced concrete structures, the labor and material costs for formwork often average 30 to 50 percent of the total in-place cost of the structure. Of the total formwork cost, the formwork labor cost is generally 2 to 3 times the formwork material cost. Thus, in order to achieve true economy with a given design, this question must be considered: "How will it be formed?" Design configurations that tend to conserve concrete but at the same time increase the amount and complexity of the forming will generally not prove to be cost effective.

In all pier configurations, simplicity and repetition are the key elements of economy. Configurations that lend themselves to commercially built forming systems will generally be cheaper than designs that require special job-built forms. If special configurations are required, the high initial cost of such forms can be minimized if forms can be used several times.

Generally, in normal column construction the circular shape is the simplest and most economical to form because commercially built forms are readily available in many standard diameters and they are easy to set up, simple to strip, and require no form ties.

Forming costs for square and rectangular shapes are minimized if sizes are kept in whole foot increments. Commercially available forms for these shapes do not have the variation in standard sizes that circular forms have. Non-standard dimensions in square or rectangular columns should be avoided wherever possible.

In wide columns with circular ends, the middle straight segment should also be kept in whole foot increments to match standard form sizes. Odd sizes will require a job-built form that can significantly increase cost.

Battering the columns tends to increase form costs but the effect is not significant for small batters up to 1½ inch/ft. Battering only one side, as is commonly done on the outside faces of 2-column rigid frame piers, is the cheapest battering system. Battering two opposite sides of a column is not significantly greater in cost for small batters. On the other hand, battering four sides always requires special forming and should be avoided; for example, columns can be economically increased in section by battering two opposite faces and stepping the other two faces at construction joints (Fig. 13).

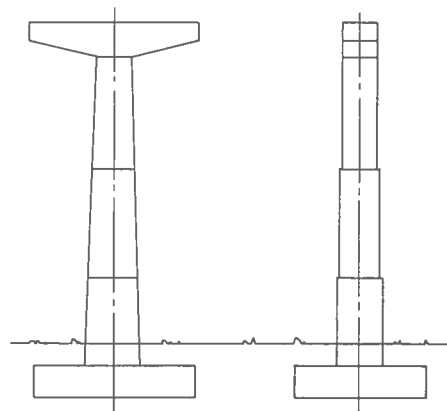


Figure 13. Battering and stepping column on a high level bridge pier to reduce the column section.

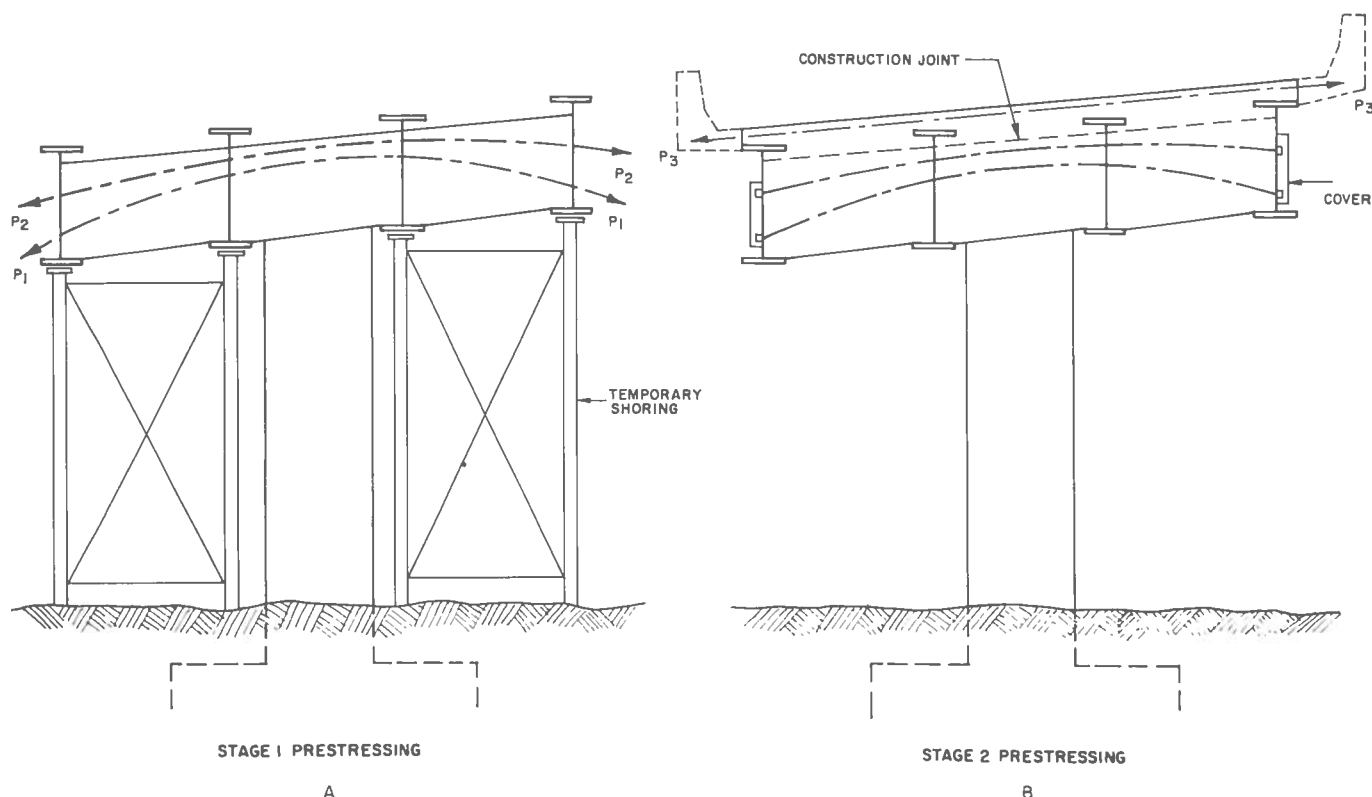


Figure 14. Integral, steel girder pre-stressed concrete cap designed for the Interstate Highway Interchanges, Knoxville, Tennessee.

D. Pier Caps

For smaller grade-crossing bridges, pier cap widths for T-Piers and Rigid Frame Piers are normally set at a minimum of 2'-6" to 3'-0", and 2'-0" minimum for solid shaft piers, unless wider caps are required for superstructure bearings. In large bridges, cap widths should also be kept as narrow as possible and still accommodate the bearings. Cap beam depths can be increased as required for stress. Wide, shallow beams are less

efficient structurally and when using load factor design, they are more influenced by cracking control which may require additional reinforcement.

Cap beam widths and lengths on large piers should be set in whole-foot increments to facilitate forming.

The spacing of columns and the length of the end cantilever on rigid frame piers should be set to balance positive and negative moments in the cap beam as much as practicable.

Special care should be taken in spacing top reinforcement in cap beams to facilitate placement of concrete and setting of anchor bolts. Anchor bolts should preferably be set in cap beams prior to placing concrete rather than drilling and grouting them in later, when

it becomes significantly greater in cost. Anchor bolts can be set in sleeves which allow some adjustment when setting shoes.

Certain conditions can tax the ingenuity of the designer, and will often result in unusual designs. As a specific example, alignment conditions on highway interchanges near Knoxville, Tennessee, posed severe horizontal and vertical clearance problems that prevented use of a conventional pier cap. In addition, girders crossing the pier were designed as continuous. The designers developed an integral cap, in which the girders pass directly *through* the pier's concrete cap rather than over the top of the cap as in the familiar and traditional manner. Figure 14 shows the arrangement used with a prestressed cap, but the cap may also be conventionally reinforced.

E. Pier Columns

Efficient column design is necessary for overall substructure economy to reduce weight, concrete volume and forming area. Structural strength requirements for columns are greatest at the top of footing where moments due to lateral loads are greatest. In rigid frame piers with heavy cap beams, lateral loads in the transverse direction may produce maximum column bending at the underside of the cap beam and at the top of footing due to frame action, but longitudinal loads always produce maximum bending at the top of footing because the pier acts as a vertical cantilever for loads in this direction. Thus, the combination of vertical, transverse and longitudinal loads will produce the critical condition for column design at the top of the footing.

Column requirements for bending will generally decrease above the top of footing. Battering or stepping columns in high level piers should be considered as a way to reduce column sizes effectively and economically.

Reduction in column size can also lower the required amount of reinforcement since a minimum area of reinforcement equal to one percent of the gross concrete area must be provided. This minimum requirement often controls the amount of reinforcement in the upper portions of pier columns.

When columns have larger cross sections than required by loading considerations, AASHTO allows a reduced effective area to be used for determining the minimum longitudinal reinforcement. In no case, however, should less reinforcement be used than that required by the minimum section designed with one percent of longitudinal reinforcement.

In high level rigid frame piers, rectangular column sections with the largest dimension in the longitudinal direction are generally more efficient than square or circular columns. This follows from

the fact that the rigid frame pier has an inherent stiffness in the transverse direction that minimizes the effects of column slenderness. In the longitudinal direction where the pier acts as a vertical cantilever, column design is much more affected by slenderness. Thus, a column section with greater section properties in the longitudinal direction will be more effective in reducing the impact of column slenderness and will be more efficient.

Intermediate horizontal struts should also be considered as a means of reducing transverse moments in pier columns and of further improving the transverse slenderness conditions in high-level rigid frame piers.

In shorter, grade separation bridge piers, where longitudinal column slenderness is not as critical as in tall piers, circular column sections have economic advantages. Formwork is simpler and placing concrete is generally easier because of the elimination of corners where it is normally difficult to get good concrete placement. Reinforcement is simplified in both tied and spiral circular columns, because extensive intermediate cross ties are eliminated. Use of spirals further allows greater column capacity over normal tied circular and rectangular columns. Certain high level bridges in Europe and in the United States have used large rectangular columns with hollow centers. These special column types achieve maximum stability for horizontal loads with a minimum of material, but the amount of forming required per cubic yard of concrete is significantly greater than that for normal column construction, and thus the in-place unit cost of concrete is also significantly higher. To be economical, the overall savings in material has to offset the increased cost of forming. It is possible that special techniques such as slip-forming might be used to cut the increased forming cost to a minimum and thus make this type of column construction more competitive.

IV. Design Criteria

A. Materials

1) Concrete

Some State Transportation Departments have set standard criteria for the concrete strength to be used for design of various bridge components. Concrete for bridge substructure generally has specified design compressive strengths ranging from 3000 psi to 3500 psi. Concrete in bridge decks generally has a higher specified strength. Some Departments use lower allowable stresses for design than the stresses AASHTO Specifications permit for the concrete strength being used. This is done especially in bridge decks presumably to achieve a longer deck life, but research and experience have indicated that using epoxy coated rebars and increasing the concrete cover of the top reinforcement are more effective in increasing the durability of the deck. Prestressing the concrete deck is also beneficial in extending the life of the deck as well as increasing its strength.

AASHTO Specifications for Construction, Section 4—Concrete Structures, states that concrete with a design compressive strength of 4,000 psi is generally used in heavily reinforced substructures. In looking at the overall economy of substructure design, the possibility of using higher concrete strengths than currently used by some States should be considered in certain circumstances.

In 1971, a study was reported in American Concrete Institute Publication SP-26 by Li and Kuo comparing the relative economy of various strength concrete mixes when used in typical structural concrete elements. The study concluded that higher strength concrete could be more economical in certain substructure members. In frame members subject to direct stress and bending with compression predominant and not controlled by slenderness criteria, the unit costs of members reach minimum values at concrete strengths between 5300 and 5700 psi. In typical reinforced concrete flexural members

the unit cost reaches a practical minimum at about 5,000 psi.

When considering higher strength concrete for a particular structure, it must be determined if concrete of the desired strength can be economically and reliably produced in the respective locality.

Consideration must also be given to the type of construction and the configuration of the structure. Higher strength concrete can generally have a greater economic effect in heavily stressed frame structures than in massive type structures where the additional concrete strength cannot be utilized. In pier cap beams, the depth required for bending is more often controlled by the amount of reinforcement that can be effectively utilized than by compressive stress. Where heavy shear is a problem, however, higher strength concrete can be useful. Where pier columns are heavily affected by bending and slenderness, attempts to reduce column sizes by increased concrete strength may not be beneficial since moment amplification factors increase as the columns become more slender.

2) Reinforcement

The following two grades of reinforcement are used in reinforced concrete construction:

Grade 40: $f_y = 40,000$ psi

Grade 60: $f_y = 60,000$ psi

In some regions of the country, deformed bars in sizes #7 and higher are available in Grade 60 only. Grade 60 bars are generally used for beam stirrups. Grade 40 bars are easier to bend and may be used for column ties or for beam stirrups where loads are small.

B. Design Methods

AASHTO Specifications for Reinforced Concrete Design, Section 8, permit the use of two design methods for proportioning reinforced concrete bridge members. The design can be made

either with reference to service loads and allowable stresses, as provided in SERVICE LOAD DESIGN, or alternatively, with reference to load factors and strengths, as provided in LOAD FACTOR DESIGN. Service Load Design had been used for many years, except for column design, and was referred to previously as Working Stress Design. Load Factor Design or Strength Design is based upon providing a member strength sufficient to carry loads which are specified multiples of the service loads.

In each design method, an elastic analysis is used to determine the member moments and forces resulting from loads on a given structure. In Load Factor Design, the applied loads can be factored before computing member forces, moments and shears, or the member forces, etc. can be factored after the analysis is made using the actual loads. Foundation characteristics, however, such as bearing pressures and pile loads, and also foundation stability checks, such as safety factors against overturning, sliding, etc., should always be calculated for the actual loads applied to the structure.

When using Load Factor Design for substructure flexural members, special serviceability provisions, such as concrete cracking and deflection control, must also be considered.

Concrete column design in the current AASHTO Specification for Service Load Design has been completely revised from the modified elastic theory approach for column design under the old working stress provisions in use prior to 1974. A strength relationship is now used for both Service Load Design and Load Factor Design. The strength approach has been recognized for years as the only realistic approach to concrete column design.

More economical structures may result from using Load Factor Design rather than Service Load Design for bridge piers, especially for long span bridges where dead loads are a higher portion of the total load. This results from the fact

that, in the Load Factor Design method, load factors used for dead load are less than those used for live load, whereas, in Service Load Design, the stresses due to Dead and Live Load use the same safety factor.

As in the design of the superstructure, use of Load Factor Design in substructure design tends to give a more uniform and consistent Factor of Safety throughout.

C. Column Slenderness

AASHTO Specifications in 1974 adopted the 1971 American Concrete Institute (ACI) Building Code Requirement for slenderness effects in design of reinforced concrete compression members. The code requires that column capacity be determined considering both the strength of the cross section and column slenderness. The degree of slenderness determines to what extent the lateral deflection of the column affects the forces and moments acting at various sections. Column slenderness effects in column design can be considered by one of two approaches as outlined in AASHTO Sections 8.16.5.2.1 and 8.16.5.2.2.

Section 8.16.5.2.1 calls for an improved structural analysis that takes into account the influence of axial loads and variable moment of inertia on member stiffness and fixed-end moments, the effect of deflection on the moments and forces, and the effects of the duration of the loads.

If such an improved analysis is not possible or practical, Section 8.16.5.2.2 provides for an approximate design method based on a moment magnifier principle. In this method, column moments are computed in an ordinary first order, elastic analysis and are then increased by a "moment magnifier" which is a function of the axial load and the column's critical buckling load. The column cross section is then designed for the combination of the axial load and the magnified moment.

The ACI Building Code encourages the use of an improved structural analysis for considering slenderness effects. The *Commentary On Building Code Requirements for Reinforced Concrete* (ACI 318-83) indicates that, generally, the moments from a second order analysis are a better approximation to the real moments than those from the alternate method. For sway frames in particular, economy can be achieved by the use of second-order analyses.

In the design of concrete columns for tall bridge piers, the effects of column slenderness can become significant. It may be beneficial in some cases to consider an improved, second-order analysis in lieu of the approximate method to reduce design moments and thereby achieve better economy.

In using the moment amplification method, approximate formulas are given in AASHTO, Section 8.16.5.2.7 for the value of EI , the flexural stiffness of a

compression member. These formulas are as follows:

$$EI = \frac{E_c I_g + E_s I_s}{1 + \beta_d} \quad \begin{array}{l} (8-43) \text{ AASHTO} \\ (10-10) \text{ ACI} \end{array}$$

or conservatively,

$$EI = \frac{E_c I_g}{1 + \beta_d} \quad \begin{array}{l} (8-44) \text{ AASHTO} \\ (10-11) \text{ ACI} \end{array}$$

In columns with steel percentages close to 1 percent, which is the case in many piers, equations 8-44 is more accurate. This is shown in Fig. 10.11.5, of the *ACI Commentary on Building Code Requirements for Reinforced Concrete*. (ACI 318-83). In columns with large bending moments resulting in low ratios of P_n/P_o , Fig. 10.11.5(b) from the ACI Commentary also shows that the theoretical EI is greater than EI as determined by approximate Equation 8-44 by a factor of at least 1.4.

P_n = nominal axial load strength at given eccentricity.

P_o = nominal axial load strength at zero eccentricity.

When steel percentages and P_n/P_o ratios are low, some designers have modified the value of EI as determined by equation 8-44 (ACI 10-11) by using the lower bound curve for $\rho = 1\%$ of Fig. 10.11.5(b) from the ACI Commentary. Using a higher, more realistic value of EI in this case results in a higher theoretical column buckling load and hence, a lower moment magnifier.

V. Reinforcement Details

A. General

Reinforcement details and spacing must effectively accommodate stress requirements, but they should be selected to facilitate placement and vibration of concrete. In beams and columns, main reinforcement should be located as far from the neutral axis as practicable to gain maximum efficiency. Proper consideration should always be given to minimum spacing requirements so that concrete can completely embed the reinforcement and fill the forms without honeycombing. Special attention should be paid to spacing of top reinforcement in pier caps. Top longitudinal bars should have 6 inches minimum clear spacing or more if practicable. Bar spacings should be carefully set to avoid interference with anchor bolts and other intersecting reinforcement such as at locations where horizontal cap beam bars pass vertical column bars.

Bar bending and detailing should be in accordance with the current *Manual of Standard Practice for Detailing Reinforced Concrete Structures*, published by the American Concrete Institute.

Reinforcement should be designed without hooked ends where sufficient straight development length can be provided.

B. Main Column Reinforcement

No. 10 or 11 bars are commonly used in bridge pier columns because they are the largest sizes that can be lap spliced.

In large columns with reinforcement percentages close to the minimum, single No. 11 bars spaced around the column periphery will generally be sufficient for capacity. The required tension embedment of No. 11 bars using standard hooks can generally be accommodated within the thicknesses required for heavy pier footings.

When columns require greater reinforcement than No. 10 or 11 bars placed singly or in 2-bar bundles, the designer should consider an interior row of reinforcement, No. 11 bars in 3-bar bundles, No. 14, or No. 18 bars. Each

of these alternatives presents certain disadvantages:

- 1) Reinforcement placed in an interior row is less efficient than reinforcement placed on the outside periphery, but in large columns, the efficiency loss may be minimal. This arrangement should be considered when small amounts of additional reinforcement are needed.
- 2) No. 11 bars in 2-bar or 3-bar bundles require a reduced column tie spacing which increases the labor required for bar placement. The additional ties also make it more difficult to get good concrete placement. Since lap splices of bundled bars must be staggered, the dowel extension above the top of footing and above construction joints required for lap splicing a 3-bar bundle become very lengthy.
- 3) Using bundles of 2-No. 14 bars in lieu of 3-No. 11 bars provides 96% of the reinforcement area with only 66% of the number of bars. Reducing the required number of bars can effect a savings in the cost of rebar placement. No. 14 bars must be butt spliced which is more expensive than lap splicing No. 11 bars of equal total area. The No. 14 bar tension development length is about 36% greater than that for a No. 11 bar and may cause placement problems in the footing.
- 4) The tension development length of a No. 18 bar is about 76% greater than that of a No. 11 bar and is generally excessive for common pier footing thicknesses.

C. Column Bar Splices

In the 1974 AASHTO Interim Specifications, prior bond stress requirements for determining anchorage and splice lengths were replaced with the "Development Length" concept. Development and splice requirements as now presented in AASHTO, Sections 8.25 to 8.33 inclusive, are the result of a more realistic evaluation of how reinforcement performs in concrete. The provi-

sions are applicable to either the Service Load or Load Factor Design method.

In bridge piers where columns must support significant lateral loads resulting in large bending moments, splices in column reinforcement are generally controlled by the requirements for tension. The minimum length of lap required is determined as a function of the tensile development length, L_d , for the specified yield strength of the reinforcement. The following three classes of tension splice are allowed in the Specifications:

Class A splice: Length = $1.0 L_d$

Class B splice: Length = $1.3 L_d$

Class C splice: Length = $1.7 L_d$

Class B or C splices are normally used in bridge pier columns.

In regions of high tensile stress, i.e., where the tensile reinforcement provided is equal to or less than twice that required for strength, lap splices must meet the requirements of Class B or Class C depending on what proportion of the reinforcement is spliced. Class B splices can be used if no more than one-half the bars are lap spliced within a required lap length. If more than one-half of the bars are lap spliced within a required lap length, Class C splices must be used.

Class A splices are used in regions where the tensile reinforcement provided is more than twice that required for strength, and if no more than three quarters of the bars are lap spliced within a required lap length.

When three bars or more are bundled together, the splice length of an individual bar within the bundle must be increased because of the reduction in bar perimeter that is effective for bond. The length of lap must be increased for a four-bar bundle. These increases are added to the splice lengths as determined for an individual, non-bundled bar.

When a two-bar bundle is lap spliced for tension, there are actually 3 bars bun-

dled together within the limits of the splice. Hence, the 20 percent increase in splice length should be used. Correspondingly, when a 3-bar bundle is lap spliced in tension, there are 4 bars bundled together within the limit of the splice, and the 33 percent increase in splice length should be used. When lap splices are used, bundles are limited to a maximum of three effective bars.

Laps of bars within a bundle must be staggered so that individual bar splices do not overlap each other. In regions of high tensile stress, Class B splice requirements can normally be considered for an individual splice, since not more than half the number of bars in the bundle would be spliced within a required lap length.

Lap splices are not permitted for No. 14 and No. 18 bars. These bars must have welded splices or other mechanical connections which develop at least 125 percent of the specified yield strength of the bar. It is recommended that welded

or mechanical splices be staggered as a safe design practice. In bundled bar construction, staggering is also necessary for erection convenience. Some types of mechanical couplers cannot be used for splices in contact bundled bars because of clearance requirements.

D. Column Tie Criteria

In the 1983 AASHTO Specification, column intermediate cross ties must be arranged so that no longitudinal bar is farther than 2 ft (measured on each side along the tie) from a longitudinal bar with lateral support. This results in intermediate cross ties spaced at a maximum of 2 ft plus a bar space. The previous code requirements, adopted in 1974, required that every corner and alternate longitudinal bar have lateral support, and that no unsupported bar could be farther than 6 inches clear on either side from a laterally supported bar. These requirements were later assessed to be unnecessarily conservative for bridge pier columns and were revised in 1980.

Where column bars are located around the periphery of a circle, a complete circular tie may be used.

Vertical spacing of ties must neither exceed the least dimension of the compression member nor 12 inches. The 12-inch criterion always controls for pier columns. When two or more bars larger than No.10 are bundled together, the vertical tie spacing must be reduced by one-half, which results in 6-inch spacing.

Column reinforcement is generally erected in preassembled cages, but some contractors may choose to place ties in the field after the vertical bars are positioned. Ties consisting of one-piece closed hoops provide maximum rigidity for preassembled cages, but are difficult to place on free standing vertical bars in the field (Fig. 15). Two-piece column ties and straight cross ties with hooked ends are suited for both types of assembly and are generally preferred for large heavily reinforced columns (Fig. 16).

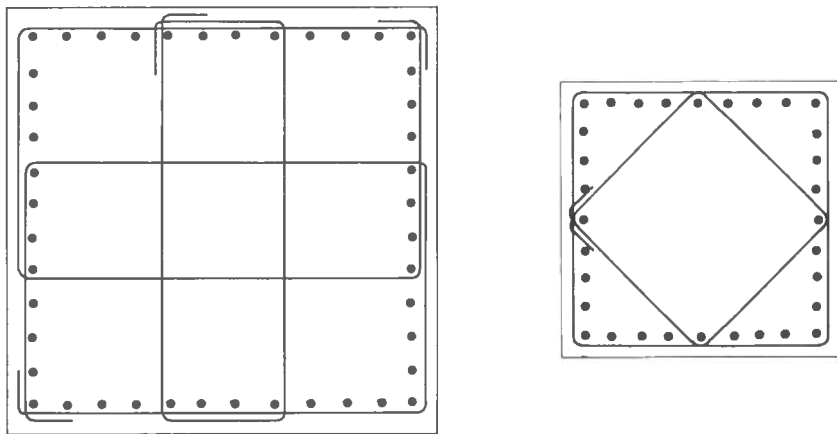


Figure 15. Typical column ties applicable for preassembled cages.

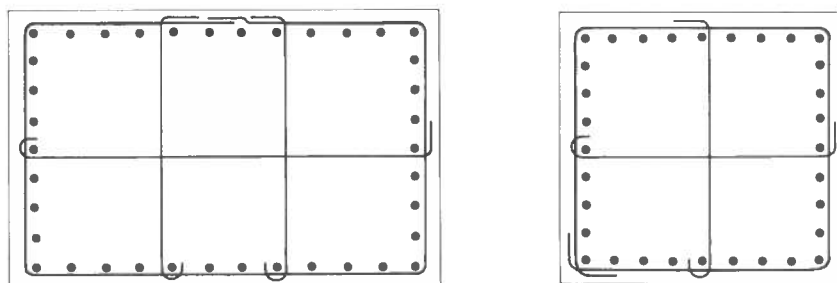


Figure 16. Typical column ties applicable for either preassembled cages or field erection.