design of all structural elements. Vertical restraint devices shall be provided at all supports where the vertical design earthquake force opposes and is greater than 50% of the static reaction under permanent effects. The vertical restraint device shall be designed to resist an uplift force of not less than 10% of the vertical reaction at the support due to permanent effects. Vertical design earthquake forces (when applicable) shall be considered in the design of horizontal restraints that rely on any component of friction.

Movement bearings are not required to accommodate the horizontal movements due to the design seismic action. The detailing of bearings expected to be damaged due to the design seismic action shall allow for a predictable mode of damage and an anticipated method of repair. The consequent distribution and magnitude of earthquake forces in the bridge shall be fully evaluated and considered in the design of all structural elements.

At expansion ends of the superstructure (including movement joints at an abutment, pier or internal hinge) the superstructure shall overlap the substructure by a sufficient distance to prevent loss of support to the superstructure due to the design seismic action. Sufficient overlap length (as shown in Figure 15.16.2) shall be provided to accommodate the relative longitudinal seismic displacement. The minimum overlap length measured normal to the face of an abutment or pier (L_{bs}) shall satisfy the following:

$$L_{bs} = 1.25\Delta_L + 0.0004L_d + 0.007h_d + 0.005B \geq 0.3 \quad \text{. . . 15.16.2}$$

where

Δ_L = longitudinal seismic displacement at the abutment, in metres

L_d = length of the superstructure to the next expansion joint, in metres

h_d = average height of piers supporting the superstructure length L_d, in metres

B length of the bearing seat transverse to the bridge longitudinal axis, in metres

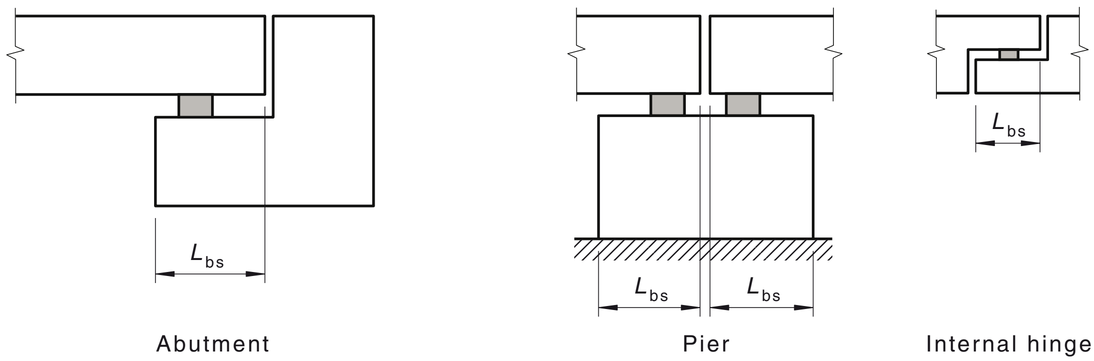


FIGURE 15.16.2 OVERLAP LENGTH L_{bs}

15.16.3 *Pile to pile cap ductile connections*

For bridge structures of BEDC levels 2, 3 or 4, the connection between each pile and its pile cap shall be designed to resist tensile force levels predicted by the analysis, but not less than 10% of the pile ultimate axial compression force (N^*).

16 FORCES RESULTING FROM WATER FLOW

16.1 General

Bridges that cross a river, stream or any other body of water shall be designed to resist the effects of water flow and wave action, as applicable. The design shall include an assessment of how the water forces may vary in an adverse manner under the influence of debris, log impact, scour and buoyancy of the structure.

Tidal and wave actions shall be considered on bridges across large bodies of water, estuaries and open sea.

NOTE: Wave action on bridges is not covered in this Standard. Refer to specialist literature.

16.2 Water flow velocity

Water flow forces for each limit state are dependent on a water flow velocity (V) applicable to the structural element under consideration. For the equations in this Clause (16), the particular choice of V shall depend on hydraulic considerations as follows:

(a) For substructures, V shall be the velocity of flow for the critical average recurrence interval (ARI) through the bridge opening averaged over the depth of flow and over the relevant bridge span.

(b) For superstructures and debris loading, V shall be the approach surface velocity for the critical ARI just upstream of the bridge.

NOTE: For wide flood plains, the watercourse may require guide banks.

(c) For log and vessel impact, the relevant approach velocity shall be at the level of impact being considered; and for surface impact, this shall be taken as 1.4 times the average velocity.

(d) The adverse structural effect of local scour shall be taken into consideration in the design at each limit state.

Where widespread, various amounts of bed scour shall be considered.

NOTE: The beneficial effect of bed scour in reducing velocity should generally be neglected except where widespread mobile alluvium is evident and velocities can be relied upon to occur for the flood event under consideration.

16.3 Limit states

16.3.1 *ULSs*

The ULSs defined in AS 5100.1 Clause 6.3 shall be satisfied for all floods up to and including the 2000 year ARI flood. A load factor of 1.3 shall be used.

16.3.2 *SLSs*

The SLSs defined in AS 5100.1 Clause 6.3 shall be satisfied for all floods up to and including the SLS flood defined in AS 5100.1 Table 11.1. A load factor of 1.0 shall be used.

16.4 Forces on piers due to water flow

16.4.1 *Drag forces on piers*

In bridge structures subjected to water flow effects, the fluid forces on the piers are dependent on the pier shape, pier configuration, the water velocity and the direction of the water flow.

The design drag forces (F_d) parallel to the plane containing the pier shall be calculated as follows:

$$F_d = 0.5\, C_d V^2 A_d \qquad \ldots 16.4.1$$

where

C_d = drag coefficient, depending on pier shape (see below)

A_d = wetted area of the pier normal to the water flow, equal to the thickness of the pier normal to the direction of the water flow multiplied by the height of the water flow.

Consideration shall be given to variations of the direction of the water flow.

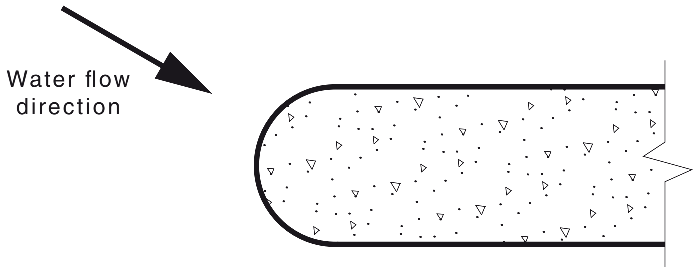
In the absence of more exact estimates, the value of C_d shall be assumed as follows:

C_d = 0.7 (semi-circular pier nosing)

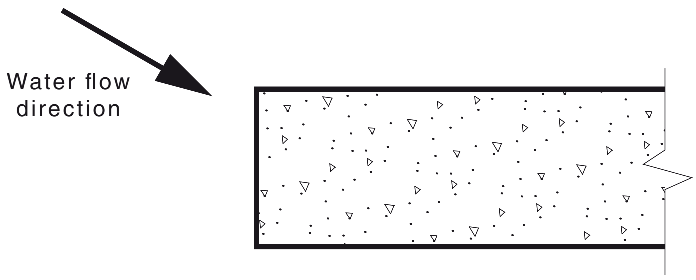
= 1.4 (square end pier nosing)

= 0.8 (90° or sharper wedge, nosing with an angle of 90° or less)

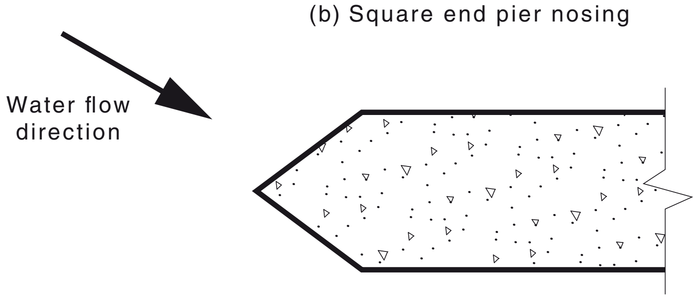
NOTE: For a diagrammatic view of typical pier end configurations, see Figure 16.4.1.



(a) Semi-circular pier nosing



(b) Square end pier nosing



(c) 90° or sharper wedge nosing
(with angle 90° or less)

FIGURE 16.4.1 TYPICAL PIER END CONFIGURATIONS

16.4.2 *Side forces on piers*

The design side forces (F_L), perpendicular to the plane containing the pier, as shown in Figure 16.4.2, shall be calculated as follows:

$$F_L = 0.5\, C_s V^2 A_L \qquad \ldots 16.4.2$$

where

C_s = side force coefficient (which depends on the angle between the water flow direction and the plane containing the pier)

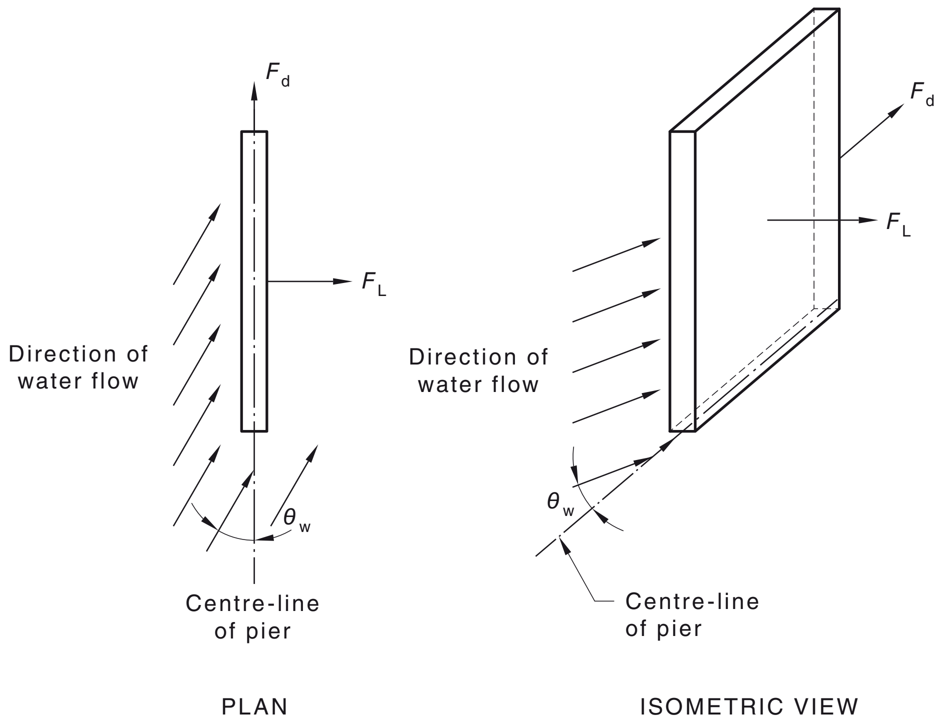
A_L = wetted area of the pier, equal to the width of the pier parallel to the direction of the water flow multiplied by the height of the water flow

In the absence of more exact estimates, the value of C_s shall be assumed as follows:

C_s = 0.9 for $\theta_w \leq 30°$

= 1.0 for $\theta_w > 30°$

where θ_w is the angle between the direction of the water flow and the transverse centre-line of the pier.



LEGEND:
F_d = design drag force
F_L = design side force

FIGURE 16.4.2 DRAG AND SIDE FORCES ON PIERS

16.5 Forces on superstructures due to water flow

16.5.1 *General*

A superstructure that is partially or fully submerged in a flood is subjected to—

(a) a drag force normal to its longitudinal axis;

(b) a vertical lift force (positive upwards); and

(c) a moment about the girder soffit level (clockwise positive with the water flow from left to right).

The loads specified in Items (a), (b) and (c) shall be determined in accordance with Clauses 16.5.2, 16.5.3 and 16.5.4, as appropriate.

16.5.2 *Drag force on superstructures*

The design drag force (F_d) on superstructures shall be calculated as follows:

$$F_d = 0.5\, C_d V^2 A_s \qquad \ldots 16.5.2(1)$$

where

C_d = drag coefficient

A_s = net wetted area of the superstructure, including any railings or barriers, projected on a plane normal to the water flow

The value of C_d for superstructures shall be obtained from Figure 16.5.2(A). The relative submergence (S_r) and the proximity ratio (P_r) shall be calculated as follows:

$$S_r = \frac{d_{wgs}}{d_{sp}} \qquad \ldots 16.5.2(2)$$

$$P_r = \frac{y_{gs}}{d_{ss}} \qquad \ldots 16.5.2(3)$$

where

d_{wgs} = vertical distance from the girder soffit to the flood water surface upstream of the bridge [see Figure 16.5.2(B)]

d_{sp} = wetted depth of the superstructure (including any railings or barriers) projected on a plane normal to the water flow [see Figure 16.5.2(B)]

y_{gs} = average vertical distance from the girder soffit to the bed assuming no scour at the span under consideration [see Figure 16.5.2(B)]

d_{ss} = wetted depth of the solid superstructure (excluding any railings but including solid barriers) projected on a plane normal to the water flow [see Figure 16.5.2(B)]

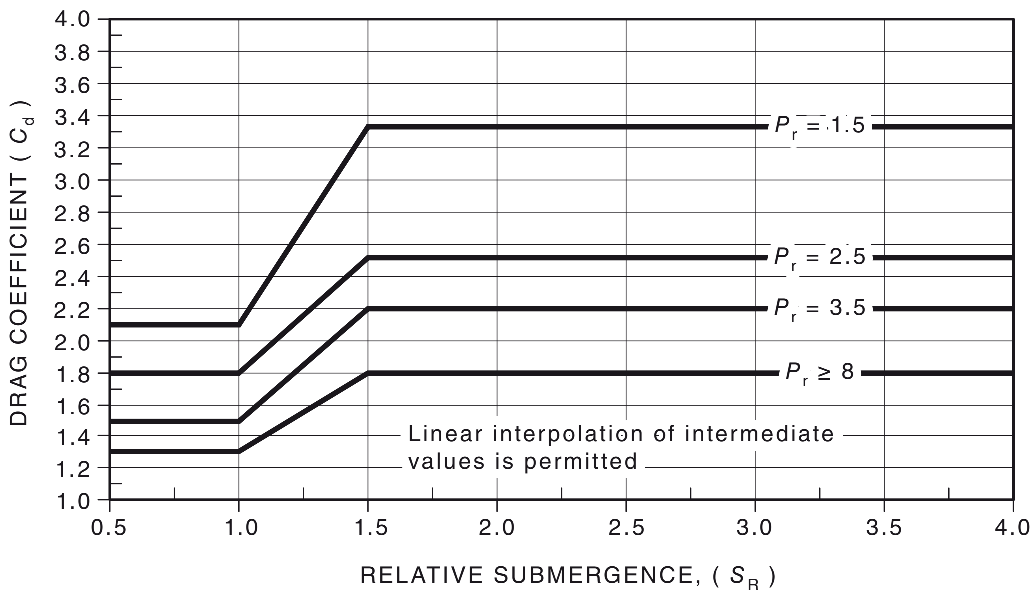


FIGURE 16.5.2(A) SUPERSTRUCTURE C_d

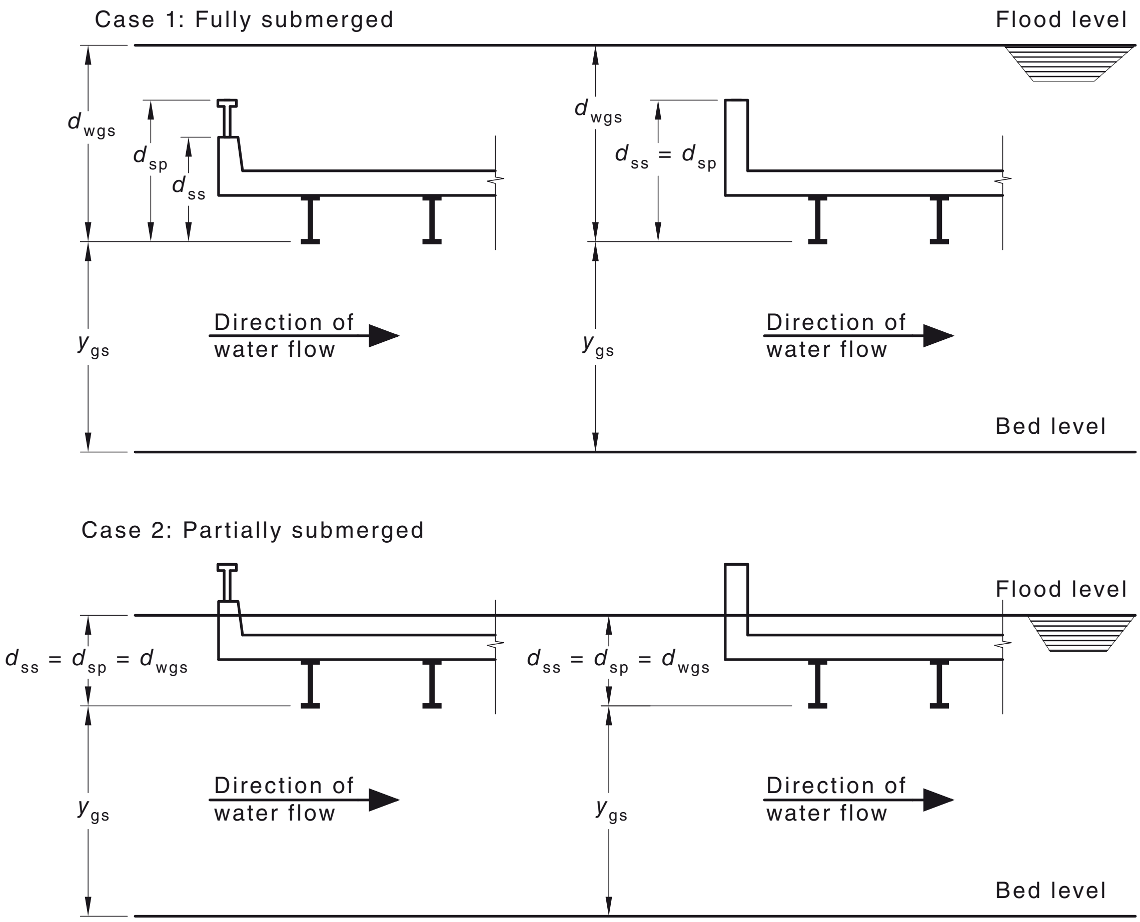


FIGURE 16.5.2(B) SUPERSTRUCTURE DRAG FORCE DIMENSIONS

16.5.3 *Lift force on superstructures*

The design lift force (F_L) on a superstructure shall be calculated as follows:

$$F_L = 0.5\, C_L V^2 A_L \qquad \ldots 16.5.3$$

where

C_L = lift coefficient

A_L = plan deck area of the superstructure

The value of C_L shall be obtained from Figure 16.5.3. An upward and downward lift force shall be calculated at each S_r.

The upward and downward lift force shall be combined with the moment as described in Clause 16.5.4 to determine the maximum uplift forces and downward forces acting on various elements of the bridge.

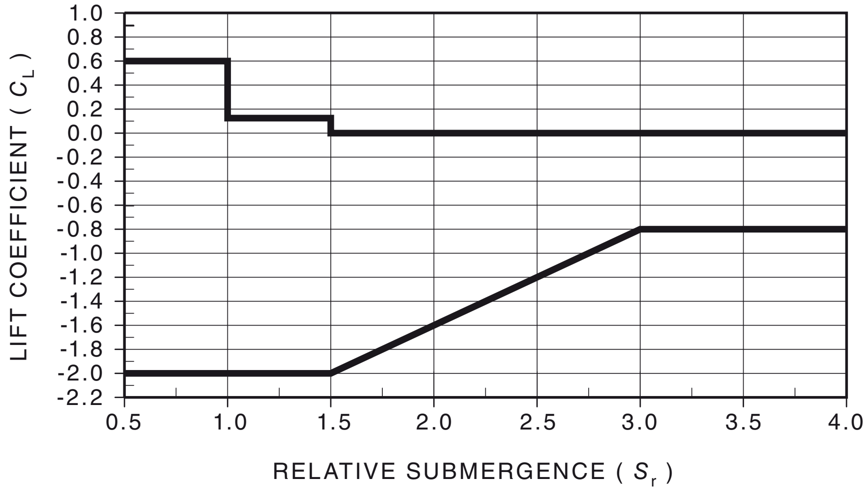


FIGURE 16.5.3 SUPERSTRUCTURE C_L

16.5.4 *Moment on a superstructure*

The drag and lift forces generate a moment about the longitudinal axis of the superstructure. The design superstructure moment due to water flow (M_g) at the soffit level at the centre-line of the superstructure shall be calculated as follows:

$$M_g = 0.5\ C_m V^2 A_s d_{sp} \qquad \text{. . . 16.5.4}$$

where

C_m = moment coefficient

The value of C_m shall be obtained from Figure 16.5.4.

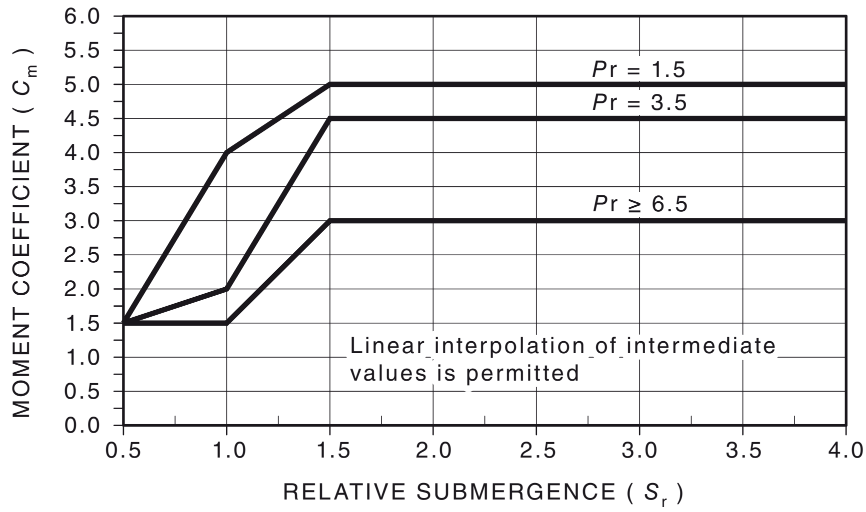


FIGURE 16.5.4 SUPERSTRUCTURE C_m

16.5.5 *Loads on superstructures with superelevation*

The loads on a superstructure with a positive superelevation (upstream face raised) of up to 4% shall be calculated in accordance with Clauses 16.5.2 to 16.5.4. The loads on a superstructure with a negative superelevation of up to 4% shall be calculated in accordance with Clauses 16.5.2 to 16.5.4, but with the following adjustments to the coefficients:

(a) The value of C_d shall be increased by 5%.

(b) The magnitude of C_L shall be increased by 20%.

(c) The value of C_m shall be the same as for a level superstructure.

If the superelevation is greater than 4%, the upward lift force shall be calculated as for wall type piers in accordance with Clause 16.4.2, except that A_L shall be taken as the plan deck area and C_L shall be taken as 0.9.

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For superelevation outside this range, study of specialist literature or physical model testing shall be undertaken.

16.6 Forces due to debris

16.6.1 *Depth of debris mat*

The depth of a debris mat varies depending on factors such as catchment vegetation, available water flow depth and superstructure span. In the absence of more accurate estimates, the minimum depth of debris mat for design shall be 1.2 m and the maximum depth shall be 3 m, or as specified by the relevant authority.

16.6.2 *Debris acting on piers*

A debris load acting on piers shall be considered for bridges where the flood level is below the superstructure. The length of a debris mat shall be taken as one-half the sum of the adjacent spans or 20 m, whichever is the lesser. The debris load shall be applied at mid-height of the debris mat, assuming the top of the debris mat is at the flood level.

16.6.3 *Debris acting on superstructures*

A debris load acting on superstructures shall be considered for bridges where the flood level is above a level of 600 mm below the soffit level. The length of the debris mat shall be the projected length of the superstructure. The debris load shall be applied at mid-height of the submerged superstructure, including any railing or barriers, where appropriate.

16.6.4 *Calculation of debris load*

The ultimate and serviceability design drag forces (F_d) due to debris shall be calculated using the following equation:

$$F_d = 0.5\, C_d V^2 A_{deb} \qquad \ldots 16.6.4$$

where

C_d = obtained from Figure 16.6.4(A), for debris acting on piers

= obtained from Figure 16.6.4(B), for debris acting on superstructures

A_{deb} = projected area of debris

NOTE: The depth of debris varies depending on the catchment vegetation.

Lateral water flow forces shall not act concurrently on parts of the bridge subject to debris loading. Lift forces and moments due to water flow or debris loading shall be considered where the superstructure is completely or partially submerged.

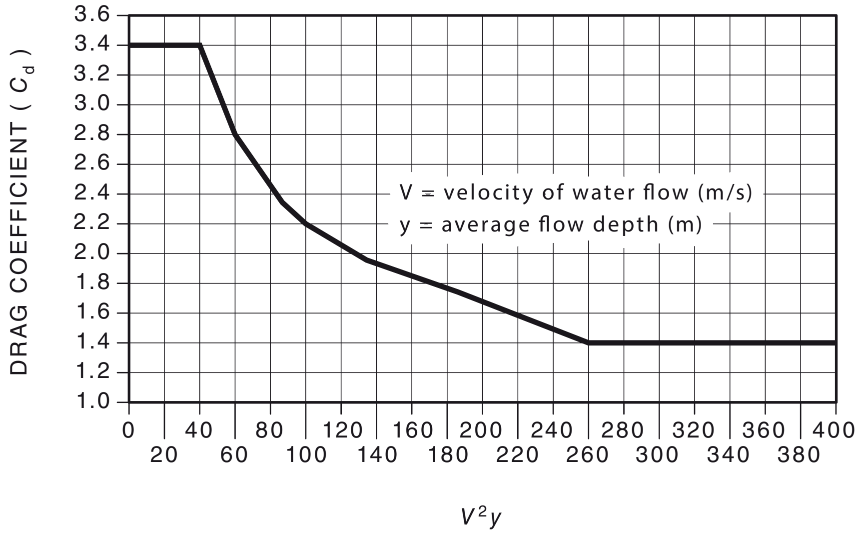

FIGURE 16.6.4(A) PIER DEBRIS C_d

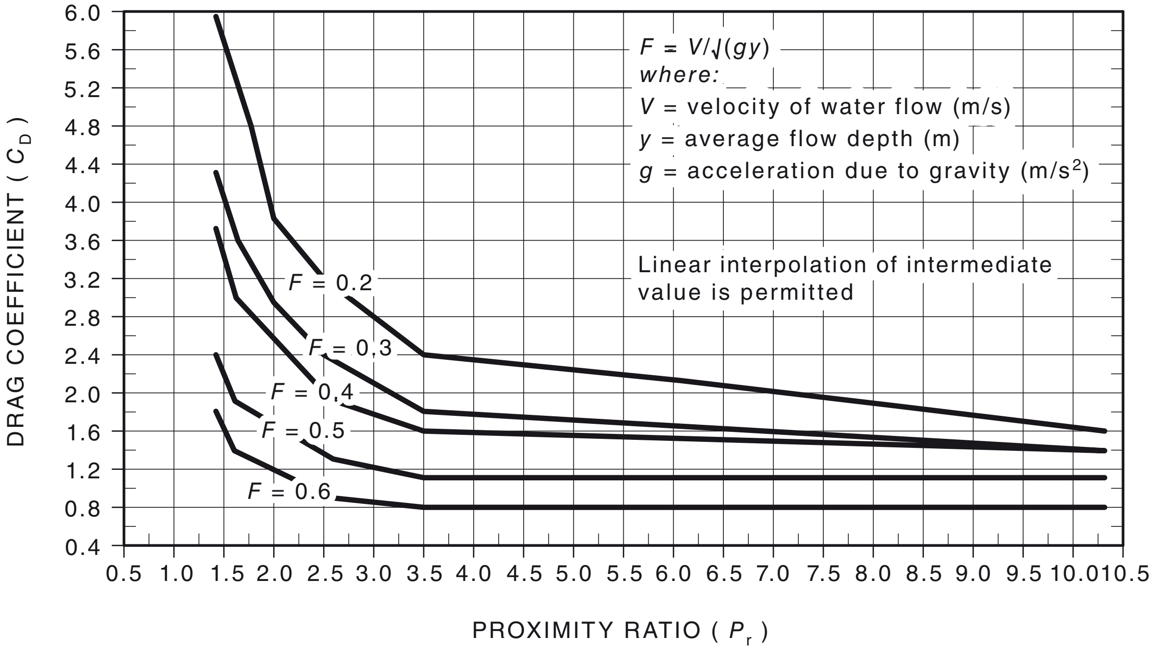

FIGURE 16.6.4(B) SUPERSTRUCTURE DEBRIS C_d

16.7 Forces due to moving objects

16.7.1 *General*

The forces due to moving objects and debris shall not be applied concurrently. Moving object impact forces shall be applied with such other water flow forces as appropriate.

16.7.2 *Log impact*

Where floating logs are possible, the ultimate and serviceability design forces exerted by such logs directly hitting piers or superstructure shall be calculated on the assumptions that a log with a minimum mass of 2 t will be stopped within a distance of 300 mm for timber piers, 150 mm for hollow concrete piers, and 75 mm for solid concrete piers. If fender piles or sheathing, to absorb the energy of the blow, are placed upstream from the pier, the stopping distance shall be increased. The design forces shall be calculated using the water flow velocity at the surface of water flow at the flood level relevant for the SLS, or for ULS, as appropriate.

16.7.3 *Large item impact*

In urban areas, the effects of impact and buoyancy from large floating items such as pontoons, pleasure craft, shipping containers, and the like, shall be considered. The type and size of large items considered shall be subject to approval of the relevant authority.

The forces due to log impact or large item impact shall not be applied concurrently.

16.8 Effects due to buoyancy and lift

In assessing the effects of buoyancy and lift on bridge structures, consideration shall be given to the following:

(a) The effects of buoyancy and lift on substructure, including piling, and superstructure dead loads. Buoyancy shall be applied concurrently with other water flow forces.

(b) The provision of effective bleed holes, which dissipate air trapped between high water level and the underside of the deck slab, and reduce the effect of buoyancy for beam and slab or box girder bridges.

(c) Provision of drainage from internal cells.

A positive tie-down system shall be provided for the superstructure if uplift occurs at any support or bearing, taking account of dead loads, buoyancy, water flow forces and debris loading.

The tie-down shall be designed for an ultimate force equal to:

$$\gamma_{\mathrm{WF}}\left(F^*_{\mathrm{Lu}} + M^*_{\mathrm{Lu}} / Z\right) + \text{Buoyancy} - \gamma_g G \qquad \text{. . . 16.8}$$

where

γ_{WF} = ultimate load factor for forces resulting from water flow, see Clause 16.3.1

γ_{g} = ultimate load factor for dead load that reduces safety, given in Table 6.2

G = dead load reaction on the support

Z = bearing layout modulus

F^*_{Lu} = ultimate design lift force

M^*_{Lu} = ultimate moment due to water flow and/or debris loading, as applicable

The bearings shall be adequately restrained in position during submergence of the superstructure.