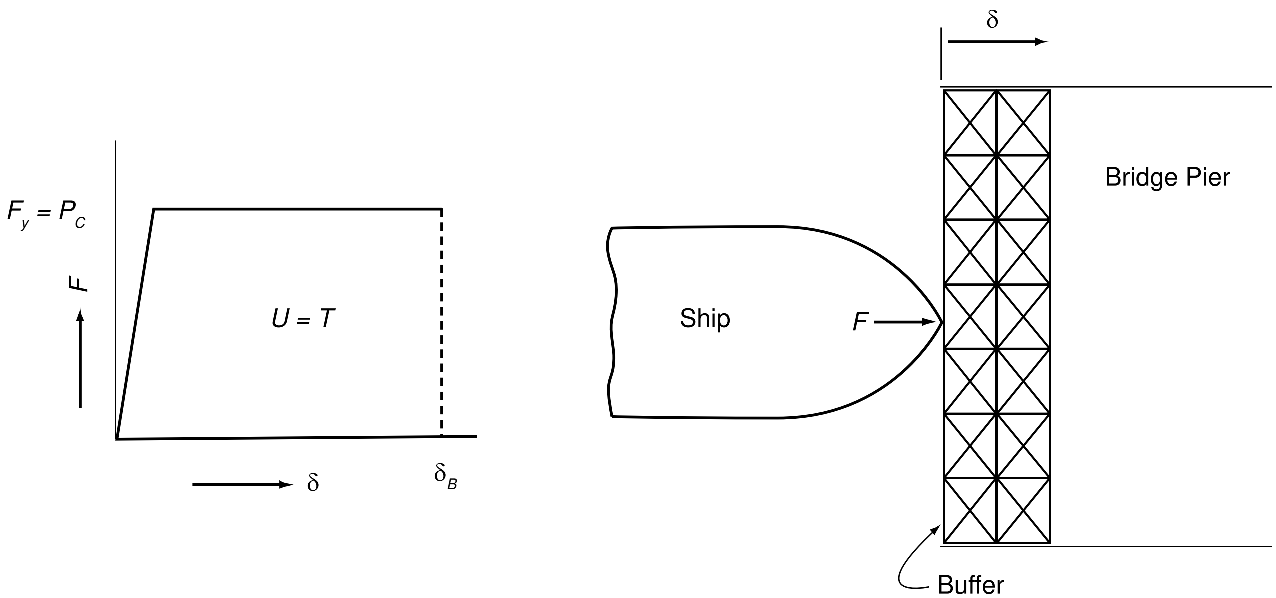


Figure C7.3.1.4-2—Load-Deformation Relationship of Framed Steel Fender Wall

7.3.2 Pile-Supported Systems

Pile-supported structures can be used to absorb collision impact loads. Pile groups connected together by rigid caps provide one method of generating high levels of protection resistance to vessel impact forces. Free-standing piles and piles connected by relatively flexible caps are also used for bridge protection. The pile groups may consist of vertical piles, which primarily absorb energy by bending, or batter piles which absorb energy by compression and bending. As a result of the high impact design loads associated with vessel collision, plastic deformation and crushing of the pile structure is permitted provided that the vessel is stopped before striking the pier or the resulting impact is less than the resistance of the pier and foundation.

The pile-supported protection structures may be either free-standing away from the pier or attached to the pier itself. Fender systems may be attached to the pile structure to help resist a portion of the impact loads. Timber, steel, or concrete piles may be utilized depending on site conditions, impact loads, and economics.

C7.3.2

Single-standing piles or pile groups of wood, steel, and concrete have long been used for vessel mooring operations. These structures are designed to elastically resist the mooring and berthing forces imparted by merchant vessels. In contrast to mooring operations in which the relatively low impact energies can be absorbed elastically by piles, the far greater energies associated with ship collision can usually only be absorbed by plastic deformation and crushing of the pile structure. After the collision, all or parts of the destroyed structure usually require replacement.

P. Tambs-Lyche discusses an example of a pile-supported protection system used for the Tromsø bridge in Norway which has a main span of 260 ft (Tambs-Lyche, 1983). The main piers of the bridge were originally protected by concrete piles supporting a rigid concrete beam as shown in Figure C1. In separate accidents, the western fender was destroyed by a collision with a 10,000-*DWT* vessel in 1961, and the eastern fender was destroyed by collision of a 1,560-*DWT* ore ship in 1963. The capacity of the original fenders was estimated to stop a 10,000-*DWT* ship drifting at a speed of 1 knot.

Following these accidents, an investigation recommended that the protection system be replaced with a stronger pile-supported structure capable of stopping a 12,000-*DWT* ship impact at a speed of 8 knots. The construction costs were so expensive, however, that the Norwegian Bridge Administration decided to reduce the protection criteria to stopping a 7,000-*DWT* ship at 8 knots and to require vessels larger than this to use an alternate navigation channel available in nearby Sandnes Sound. The new protection structure shown in Figure C2 was constructed in 1975 and consists of a ring- shaped rigid concrete beam encircling the pier with the beam supported by steel pipe piles filled with concrete. The clearance between the inside face of the concrete ring and the main columns varied from approximately 17 to 23 ft.

As part of the Tasman Bridge pier protection investigation in Australia, Maunsell and Partners evaluated the pile-supported system shown in Figures C3 and C4. The system consisted of eight 10-ft diameter prestressed concrete piles tied together by a rigid cap beam. During the design impact of a 35,000-*DWT* ship at 8 knots, the piles would form plastic hinges at the top and bottom to absorb the impact energy through rotational deformation (Maunsell and Partners and Brady, 1978).

The Rosario-Victoria crossing of the Paraná River in Argentina is a cable-stayed bridge with a main span of 1,150 ft. All piers in the Paraná River rest on approximately 100-ft deep pile foundations. The vessel impact study resulted in a governing design ship of 43,000 *DWT*, with a speed of 9 knots, and a computed impact force of 26,500 kips. The study took into account a variation of the water level of up to 20 ft and local scour of up to 40 ft, resulting in a potential total free length of pile reaching up to 138 ft. With the specific geotechnical and hydraulic situation the foundations themselves could not be designed to withstand the high impact forces economically within the elastic range. Therefore, pile-supported systems, designed as sacrificial structures by exploiting their plastic capacities, were the appropriate solution as shown in Figure C5. The independent pile-supported concrete beams, around the foundations of the Orinoco River in Venezuela, shown in Figure C6 is another example. (Svensson, 2006).

Derucher developed the following dynamic analysis method for the design and analysis of pile-supported protective structures (Derucher and Heins, 1979). The analysis assumes that the pile and fenders remain in the elastic range, and that the ship is a non-deformable rigid body. The pile structure/ship system is modeled as a spring and weight and a distribution factor, *DF*, is introduced into the spring constant to account for the influence of walers and adjacent piles in the structure (Figure C7). Assuming a fender attached to the pile structure, Derucher's method yields:

$$P = KYC \qquad \text{(C7.3.2-1)}$$

$$K = \frac{(K_p)(K_f)}{(K_p + K_f)} \qquad \text{(C7.3.2-2)}$$

where:

P = applied force to structure (kips),

K = equivalent spring constant of pile and fender (kip/in.),

Y = maximum system deflection (in.),

C = vessel coefficient,

K_p = spring constant of pile (kip/in.), and

K_f = spring constant of fender (kip/in.).

The stiffness of a cantilevered pile with a unit lateral load on top can be computed by:

$$K_p = \frac{1}{\Delta_p} \tag{C7.3.2-3}$$

$$\Delta_p = \left(\frac{L^3}{3EI_p}\right) DF \tag{C7.3.2-4}$$

where:

Δ_p = pile deflection due to unit load (in./kip),

L = length of pile above fixity (in.),

E = modulus of elasticity (ksi),

I_p = moment of inertia of pile (in.^{4}), and

DF = distribution factor.

The vessel coefficient, C, accounts for the eccentricity, configuration, and hydrodynamic mass coefficient of the vessel. For head-on impact, $C = C_H$ as defined in Article 3.8. The distribution factor, DF, developed by Derucher (1979) can be computed as:

$$DF = \left[-6.0 \times 10^{-7}(D_x) + F\right] L^{-0.006} \tag{C7.3.2-5}$$

where:

$$F = -3.5 \times 10^{-13}\left(D_y\right)^2 + 3.1 \times 10^{-7}\left(D_y\right) + 0.335 \tag{C7.3.2-6}$$

D_y = vertical stiffness = EI_p/S_p,

S_p = vertical pile spacing (in.),

D_x = horizontal stiffness = EI_w/S_w,

I_w = waler moment of inertia (in.^{4}), and

S_w = horizontal waler spacing (in.).

The maximum system deflection, Y (in.), and frequency, λ, can be computed as:

$Y = V/\lambda$ (in.)

$\lambda = (K/M)^{1/2}$ (sec^{-1})

where:

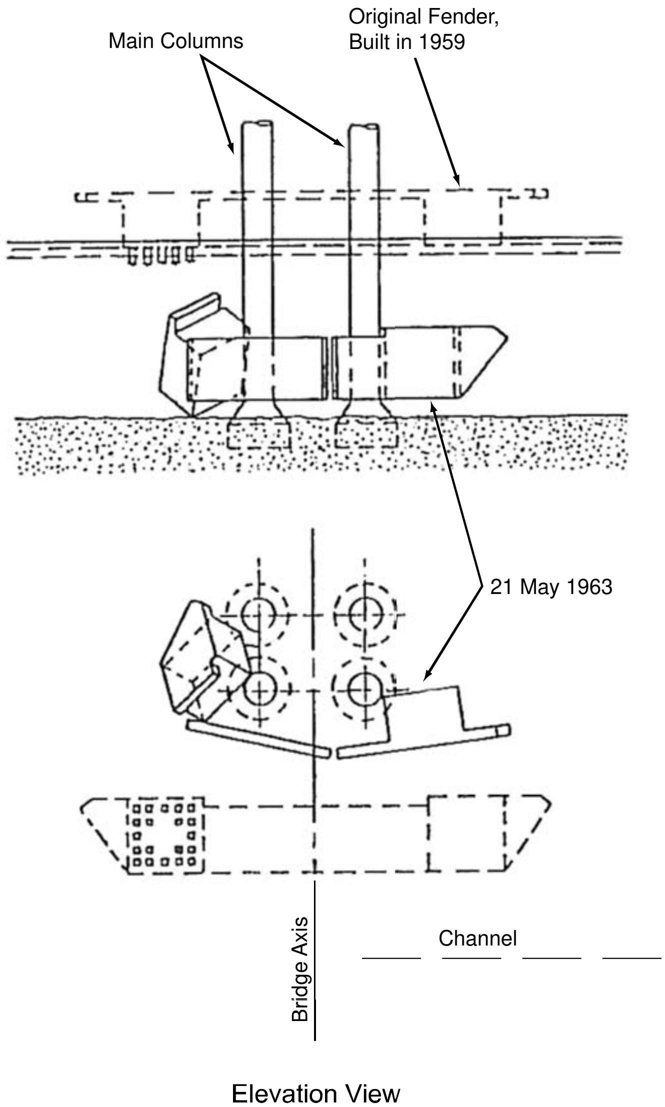
V = impact velocity (in./s), and

M = mass of vessel (kip-s^2/in.).

The acceleration, a, and stopping time, t, can be determined as follows:

$a = V\lambda$ (in./s^2)

$t = (\pi/2\lambda)$ (sec)



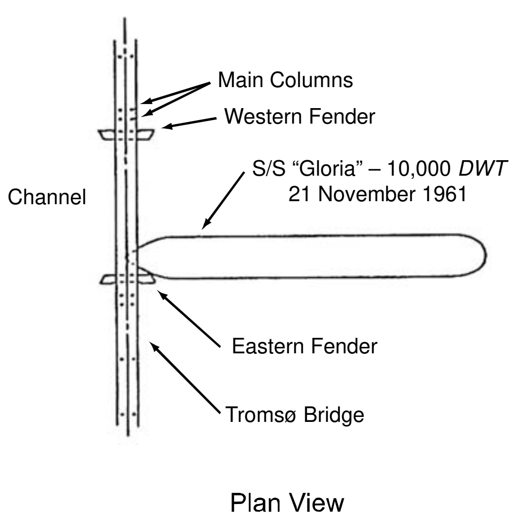


Figure C7.3.2-1—Detail of Destroyed Pile-Supported Fender of the Tromso Bridge due to a 1963 Ship Collision

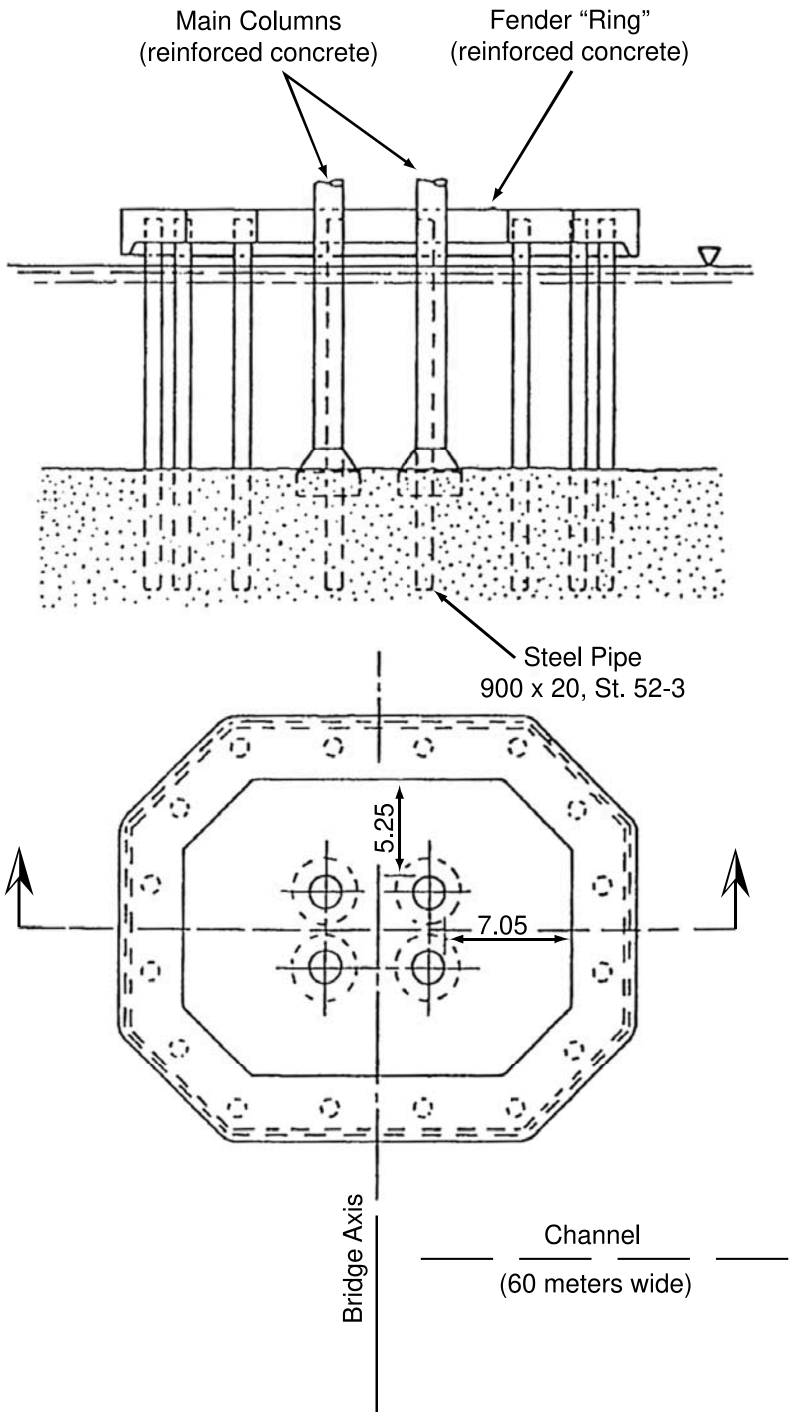


Figure C7.3.2-2—Pile-Supported Protection System for the Tromsø Bridge, Norway (All Units Are Metric)

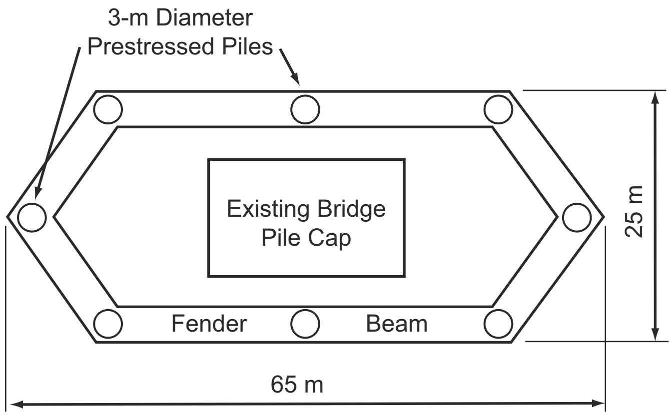


Figure C7.3.2-3—Plan of Pile-Supported Pier Protection System Evaluated for the Tasman Bridge, Australia (All Units Are Metric)

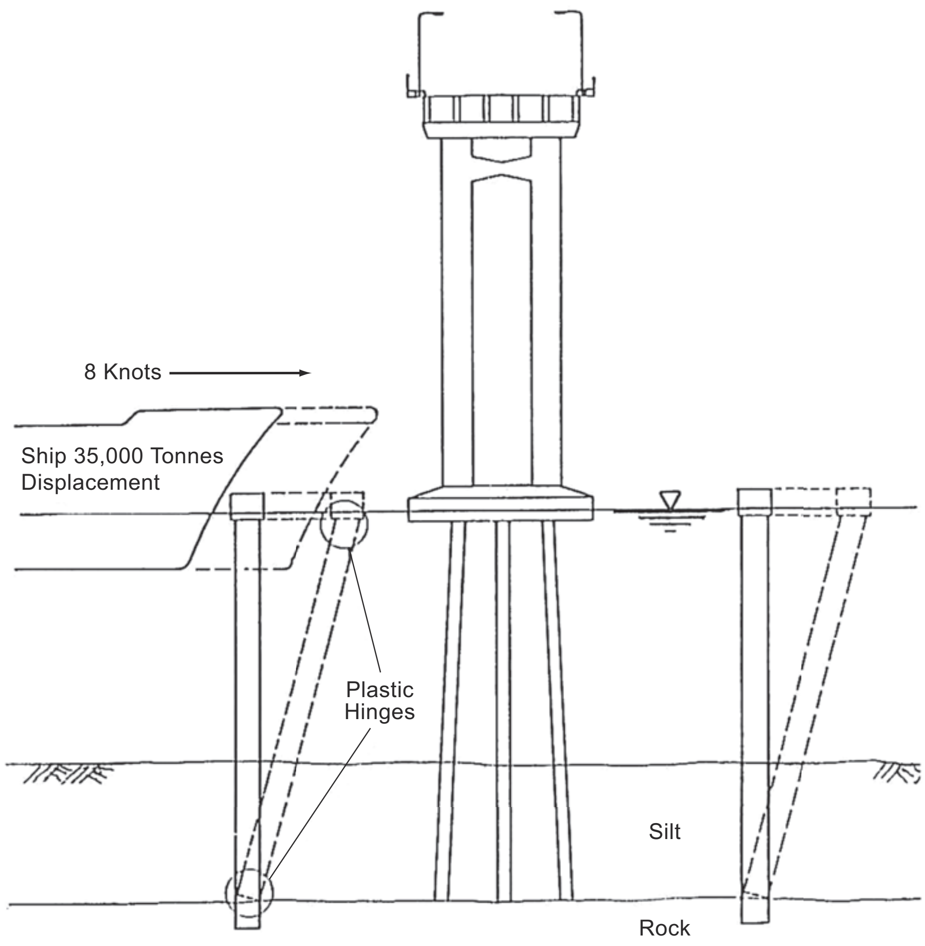


Figure C7.3.2-4—Section of Pile-Supported Pier Protection System Evaluated for the Tasman Bridge

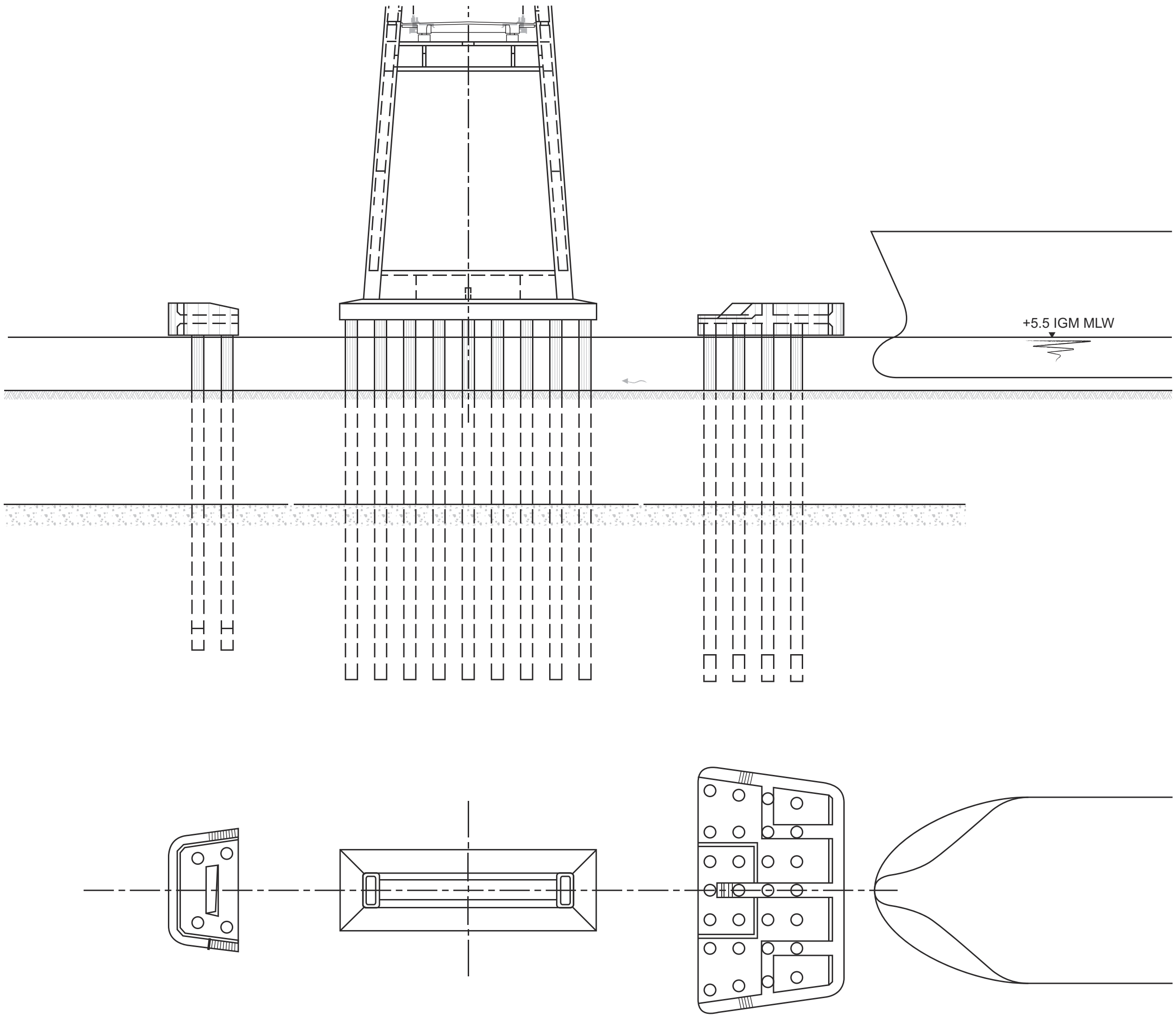


Figure C7.3.2-5—Pile-Supported Protection System for the Rosario-Victoria Bridge, Argentina

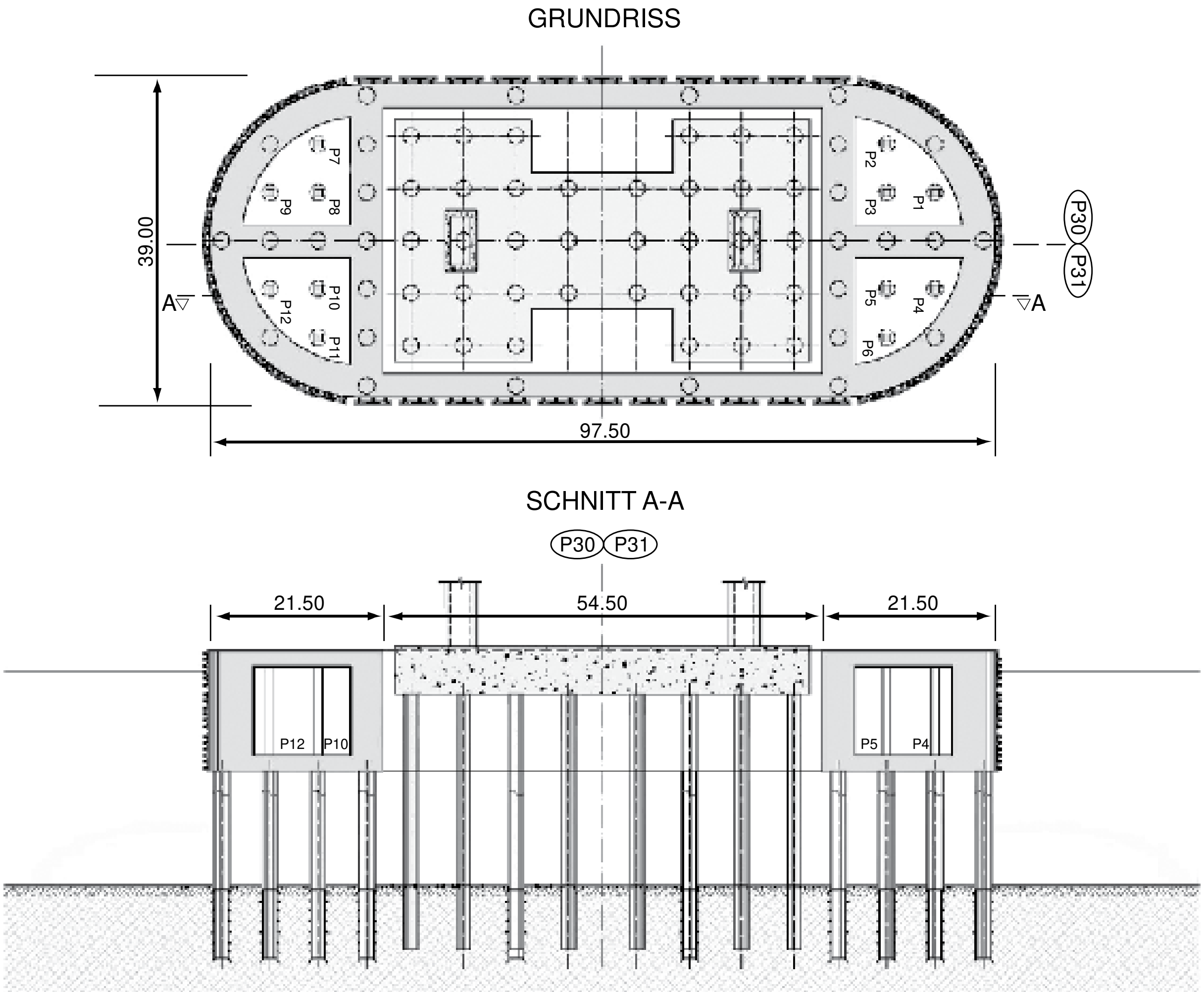
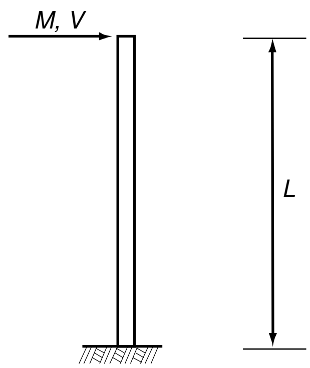
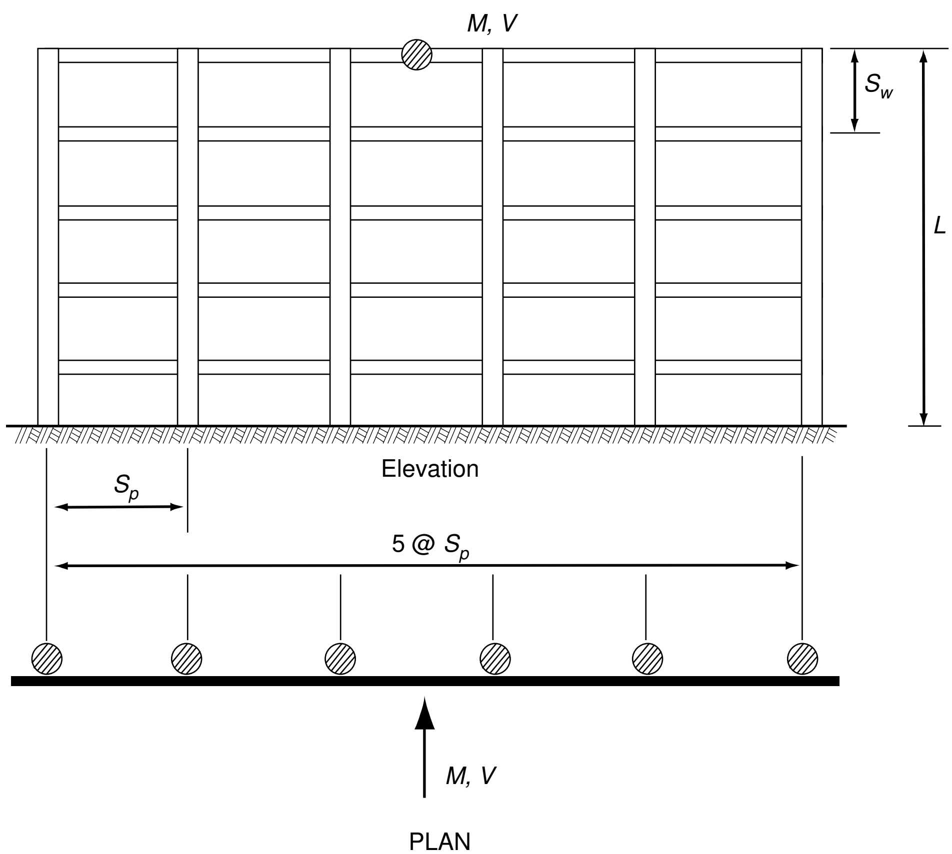


Figure C7.3.2-6—Pile-Supported Protection System for the Orinoco Bridge, Venezuela



a. Single Pile Elevation



b. Multiple Pile Fender Structure

Figure C7.3.2-7—Typical Pile-Structure Geometry for Derucher's Dynamic Analysis