Fig. 15-2 — Typical Backwall Damage

Fig. 15-3 — Stepped Pier Cap, No Damage

Fig. 15-4 — Longitudinal Separation of Bridge and Approach Fill

The majority of concrete bridges can be adequately designed by the methods presented in the AASHO "Standard Specifications for Highway Bridges" or the more refined methods, used by several state highway departments, that relate the design forces to the mass and stiffness of the structures.

EARTHQUAKE RESISTANT DETAILS

As mentioned before, earthquake loading is rarely the critical loading condition for design of normal overcrossing structures or short-to-medium span bridges. Structural details are the critical factors for earthquake resistance of these structures. Relatively short structures are really integral portions of the roadway section rather than independent units, and must be detailed accordingly.

Details necessary to resist the differential earth movements likely during major earthquakes may be divided into two groups: (1) details to prevent differential movement in the plane of the superstructure, and (2) details to isolate the structure from the approach fills so that differential movement will create a minimum of damage to the structure. The type of framing system of a structure has a considerable influence on its resistance to earthquake damage. The following framing systems are normally used for concrete bridges. Details that will increase the earthquake resistance are recommended for each system.

Rigid Frames

Rigid-frame structures with retaining type abutments may be severely damaged by differential ground movements, though the structures may be usable after a major earthquake. Resistance of these structures to damage is increased (at large expense) by designing the retaining abutments for the passive earth pressures of the soils retained. A more practical method of preventing damage is to provide "crumple" sections between the walls and the approach fill. The "crumple" sections should be at least six in. thick, and the walls should be designed for the ultimate strength of the "crumple" sections. Corrugated sheet metal or expanded polyethylene resistant to chemical attack by the soils make effective "crumple" sections.

Rigid frame structures with spill-through type abutments have excellent earthquake resistance. They will oscillate with the ground, yet allow differential approach fill movement with little or no damage to the structures. Abutments should have minimum backwall areas in contact with the ground and should be designed for passive earth pressures or provided with "crumple" sections in regions of high earthquake activity.

Continuous Slab and Beam Bridges

Superstructures that are continuous over, but not rigid with, the substructure units have good earthquake resistance. Long-span structures of this type should have their bearings fixed (to horizontal movement) at as many piers as expansion stresses will allow and have relatively free movement at the abutments. These structures will thus be somewhat isolated from the differential approach fill movements. Damage from high-intensity earthquakes for these structures may range from cracked abutment backwalls and wingwalls to punching of the beams through the backwalls and tilting and cracking of the pier shafts. The structures would, with few exceptions, be usable after the shocks and repairable at reasonable cost.

Short-span continuous superstructures should be fixed at all supports. Connections should have a capacity not less than the passive earth pressure on the vertical contact area of the abutment cap and backwall.

Simple-Span & Cantilever/Suspended-Span Structures

Simple-span and cantilever/suspended-span structures should have as many spans as possible fixed (to horizontal movement) at piers and should be detailed to allow expansion at the abutments. Earthquake resistance of a long series of spans that are 60 ft or less in length may be greatly increased by fixing alternate spans at both ends. End spans should be fixed at the first interior piers and free at the abutments.

Ends of beams should be square and in line with the beams of the adjacent spans to prevent damage or transverse displacement from oscillation. In areas of extreme earthquake activity, the space between beam ends should be filled with a replaceable material, such as a soft wood, that will allow the slow expansion movements of the structure but resist the quick movements of an earthquake.

Stepped pier caps should be avoided. If they are used, either the deeper beams should be fixed at the pier or sufficient room should be provided on the cap to prevent contact of the beams with the face of the step (Fig. 15-3).

Skewed And Curved Structures

Longitudinal movements of skewed or curved structures cause differential transverse movements at the superstructure joints. Beam bearings for these structures should be detailed to prohibit any transverse movement. Effective restraint to transverse movement will make these structures as resistant to differential fill movement as comparable square structures.

EXCEPTIONS

Long-span structures (over 200-ft span length) or structures with very large angles of skew may behave quite differently from the structure types listed in this report. Oscillation may be transverse rather than longitudinal and a dynamic analysis of the entire structure may be a mandatory requirement for resistance to damage. Several long-span truss bridges were damaged during the Alaskan earthquake. The damage was from longitudinal oscillation or longitudinal displacement of the piers and abutments. The trusses were relatively stiff for their span lengths, however, and were supported by solid-shaft concrete piers so their reactions to the earthquake approximated the action of shorter-span structures previously described.

Passive soil pressures have been recommended in this report as design loads to prevent damage from differential horizontal soil settlements during earthquakes. Use of these pressures should not be considered a guarantee of resistance to damage. Certain types of soil or soils in various conditions (frozen, cemented, highly surcharged, etc.) may not yield quickly enough to limit the load to the passive pressure during the rapid earthquake movements. Designing for forces greater than the passive pressures, however, does not appear to be reasonable without further investigation.

CONCLUSIONS

Damage to bridges during the Alaskan earthquake of March 27, 1964, was primarily due to longitudinal differential settlement of the approach fills or river alluvium (crowding together of substructure units) and acceleration forces along the longitudinal axes of the bridges.

The resistance to earthquake damage of the normal concrete bridge is more dependent on the selection of type of framing and detailing of connections and joints than on the structural design.

The recommendations in the body of this report are related to the various types of framing normal for concrete bridges. The goal of these recommendations is to provide one of two conditions: isolation of the superstructures from the approach fills, or integration of the structures with the roadway sections. The fulfillment of either of these conditions will provide bridges that are highly resistant to damage from major earthquakes.

Ends of beams should be equal and in line with the centers of the adjacent beams to prevent damage at transverse displacement from oscillation. In areas of high earthquake activity, the space between beam ends should be filled with a replaceable material, such as a soft wood, that will allow the slow expansion movements of the structure but resist the quick movements of an earthquake.

Stepped pier caps should be avoided. If they are used, either the deeper beams should be fixed at the pier or sufficient room should be provided on the cap to prevent unseating of the beams with the loss of the span (Fig. 15-2).

Skewed And Curved Structures

Longitudinal movements of skewed or curved structures cause differential transverse movements at the substructure joints. Bearing devices for these structures should be detailed to prohibit any transverse movement. Effective restraint of transverse movement will make these structures as resistant to differential movement as comparable square structures.

EXCEPTIONS

Long-span structures (over 300-ft span length) or structures with very large angles of skew may behave quite differently from the structure types listed in this report. Oscillation may be transverse rather than longitudinal and a dynamic analysis of the entire structure may be a mandatory requirement for resistance to damage. Several long-span truss bridges were damaged during the Alaskan earthquake. The damage was from longitudinal oscillation or longitudinal displacement of the piers and abutments. The trusses were relatively stiff for their span lengths, however, and were supported by solid-shaft concrete piers so their reactions to the earthquake approximated the actions of shorter-span structures previously described.

Passive soil pressures have been recommended in this report as design loads to prevent damage from differential horizontal soil settlements during earthquakes. Use of these pressures should not be considered a guarantee of resistance to damage. Certain types of soil or soils in saturated conditions (that is, cemented, highly saturated, etc.) may not yield quickly enough to limit the load to the passive pressure during the same earthquake movements. Designing for forces greater than the passive pressures, however, does not appear to be reasonable without further investigation.

CONCLUSIONS

Damage to bridges during the Alaskan earthquake of March 27, 1964, was primarily due to longitudinal differential settlement of the approach fills or their movement (crowding together of substructure units) and acceleration forces along the longitudinal axes of the bridges.

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The recommendations in the body of this report are related to the various types of framing normal for concrete bridges. The goal of these recommendations is to provide the use of two conditions: isolation of the superstructure from the approach fills, or integration of the structures with the roadway sections. The fulfillment of either of these conditions will provide bridges that are highly resistant to damage from major earthquakes.

ASEISMIC DESIGN OF REINFORCED CONCRETE CAISSONS FOR BRIDGE FOUNDATIONS

By KOICHI TAMURA

In the aseismic design of a reinforced concrete caisson, its stability and stress have heretofore been checked by Dr. Mononobe's formula in Japan. This method is recommended when the ground is cohesionless stiff soil. But, for a caisson excavated in cohesive soil and based on a hard layer, the design method governed by its ultimate condition is proposed. In soft ground, however, if ground movement is restrained by a caisson in an earthquake, the earth pressure acting on the caisson should also be considered as a load.

Keywords: bending moments; bridge foundations; bridges (structures); caissons; dynamic loads; earthquake resistant construction; loads (forces); pressure; reinforced concrete; shear; soil (material); soil mechanics; structural design.

Reinforced concrete caissons are widely used for bridge foundations, when the ground is composed of soils and a deep scouring around a pier in flood is anticipated. In Japan, the design of a caisson is greatly influenced by the effect of earthquake. In design it is commonly assumed that the ground around a caisson resists its movements in an earthquake and Dr. Mononobe's formula is used, in which the horizontal earth pressure is assumed to increase in proportion both to the depth and to the amount of the movement of a caisson.

However, in cohesive soils, the horizontal earth pressure due to the movement of a caisson does not always increase in proportion to the depth. Therefore it seems unsuitable to use the formula in cohesive soils. Further, in soft ground, it has been recognized that the ground moves in an earthquake, the amount of which varies according to the depth and the stiffness of soils. Therefore, if the ground motion is restrained by the existence of a caisson, the earth pressure acts on the caisson as a load in the same direction as that of the seismic force of the caisson.

The No. 1 Pier of Shinano-gawa Railway Bridge inclined by 8 deg in the direction of the river in the Niigata Earthquake in 1964. It had a caisson foundation and was situated near the quay wall. A soft artificial sand layer from 15 ft to 30 ft in depth slid toward the river and pushed the upper part of the caisson. This phenomenon is explained by means of a simple assumption that the caisson is influenced by the passive earth pressure due to the sliding layer.

The following are suggested as practical methods for aseismic design, insofar as the dynamic analysis of the ground and foundation has not been fully developed yet.

SLENDER CAISSONS IN COHESIONLESS SOILS
(Mononobe's formula)

Assumptions

(1) Horizontal earth pressure increases in proportion both to the depth and to the amount of the movement of a caisson.

KOICHI TAMURA, Deputy Director, Structure Design office, Japanese National Railways is a Structural Engineer and a graduate of Tokyo University. He is experienced in the maintenance of railway structures and has engaged in the design and construction of bridges on soft ground.

(2) The caisson rotates about a point as a rigid body and the seismic force of a pier and superstructures are balanced by horizontal earth pressures in the side of the caisson.

Horizontal Earth Pressure (Fig. 16-1a)

The horizontal earth pressure, p, is

$$p = \frac{p_1 y}{{y_1}^2}(2y_1 - y) \tag{1}$$

Maximum p occurs at the depth of y, or ℓ as follows:

$$p_1 = \frac{k_o w_o \ell + H_o}{y_1 - \frac{\ell}{3}}\left(\frac{{y_1}^2}{2b\ell^2}\right) \leq Cw(y_1 + z_1) \tag{2}$$

$$p_2 = \frac{p_1 \ell (2y_1 - \ell)}{2b{y_1}^2} \qquad \leq Cw(\ell + z_1) \tag{3}$$

where

$$y_1 = \left(\frac{\ell}{2}\right)\frac{4M_o + 3H_o\ell + k_o w_o \ell^2}{6M_o + 4H_o \ell + k_o w_o \ell^2}$$

w = unit weight of a soil (submerged unit weight in water)

k_o = horizontal seismic coefficient (ratio of acceleration between horizontal earthquake and gravity)

w_o = weight of a caisson per unit length

M_o = bending moment acting on a caisson at the ground surface

H_o = horizontal force acting on a caisson at the ground surface

C = coefficient of passive earth pressure

Design for Longitudinal Direction

The bending moment and the shearing force at the depth z in the longitudinal direction can be obtained according to the seismic force and the earth pressures above.

$$M_z = M_o + H_o z + \frac{k_o w_o}{2} z^2 - 2b\int_o^z p(z-y)dy \tag{4}$$

$$S_z = H_o + k_o w_o z - 2b\int_o^z p\,dy \tag{5}$$

Compressive or shearing stress in concrete and tensile stress in reinforcement of a caisson in the longitudinal direction can be obtained from the combination of the bending moment or shearing force and the vertical force at the same depth.

Design for Transverse Direction

In order to decide the cross-section of a caisson in an earthquake, it is assumed that the design loads are the horizontal earth pressure in the opposite direction of the seismic force and the active earth pressure or earth pressure at rest in the lateral direction of the seismic force (Fig. 16-1b).

The walls of a caisson are assumed as reinforced concrete one way slabs with horizontal reinforcements which are the members of a box-section rigid frame, hollow circular wall, etc., according to the shape of a horizontal cross-section of a caisson. The stresses of reinforced concrete walls at a certain depth are calculated from the shape and the load above-mentioned in the horizontal direction. These stresses should be calculated at some depths related with earth pressures and amounts of reinforcements.

SHORT CAISSONS IN COHESIONLESS SOILS

Assumption

The general assumption is the same as that of slender caissons. Besides, base pressure is proportional to the sinking of a caisson. Tensile stress between the bottom of a caisson and the soil underneath it does not take place.

The Case of Base Reaction Lying in the Core

If a caisson rotates about the point m and inclines θ radian, horizontal earth pressure p and base pressure q are as follows, assuming the horizontal and vertical spring constants (the coefficient of subgrade reaction) of $K_1\ell$ and $K_2\ell$ at the bottom1 (Fig. 16-2a):

$$p = K_1 y(2y_1 - y)\theta \tag{6}$$

$$q = K_2 \ell x\theta + \frac{N_o + w_o\ell}{A} \tag{7}$$

Then the maximum earth pressure at the side and the bottom is as follows, considering the equilibrium of horizontal and vertical forces and moments.

$$p_1 = \frac{3\left\{k_o w_o \ell^4 + 3H_o\ell^3 + 4M_o\ell^2 + 8\alpha K a^3(k_o w_o\ell + H_o)\right\}^2}{4b\ell^3(\ell^3 + 24\alpha K a^3)(k_o w_o \ell^2 + 4H_o\ell + 6M_o)} \leq Cw(y_1 + Z_1) \tag{8}$$

$$\left.\begin{matrix} q_1 \\ q_2 \end{matrix}\right\} = \frac{N_o + w_o\ell}{A} \pm \frac{3\alpha K(k_o w_o \ell^2 + 4H_o\ell + 6M_o)}{b(\ell^3 + 24\alpha K a^3)} \leq \begin{matrix}\text{permissible} \\ \text{soil pressure}\end{matrix} \tag{9}$$

where

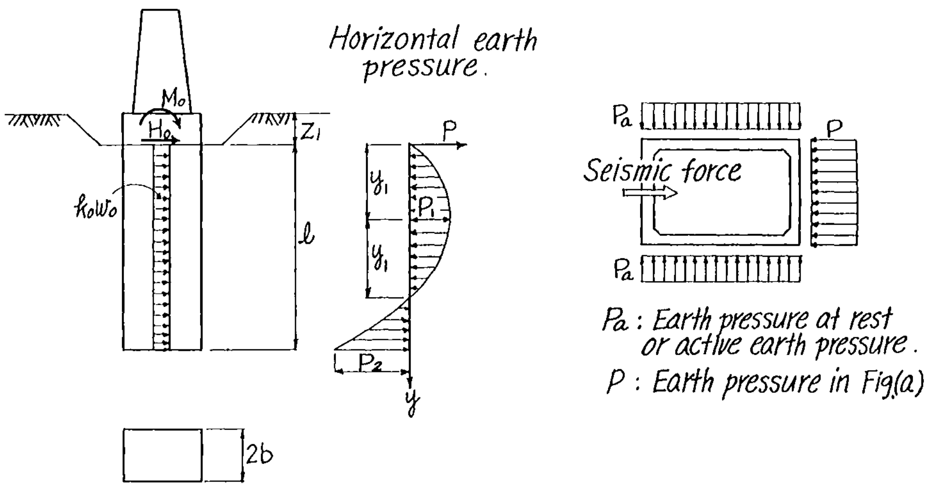
$$y_1 = \frac{k_o w_o \ell^4 + 3H_o\ell^3 + 4M_o\ell^2 + 3\alpha K a^3(k_o w_o \ell + H_o)}{2\ell(k_o w_o \ell^2 + 4H_o\ell + 6M_o)}$$

$$K = \frac{K_2}{K_1}$$

$$\alpha = \frac{3}{2ba^3}\int_{-a}^{a} b_x x^2 dx \quad \left(\text{for instance, } \alpha = 1 \text{ or } \frac{3\pi}{16} \text{ for rectangular or circular section}\right)$$

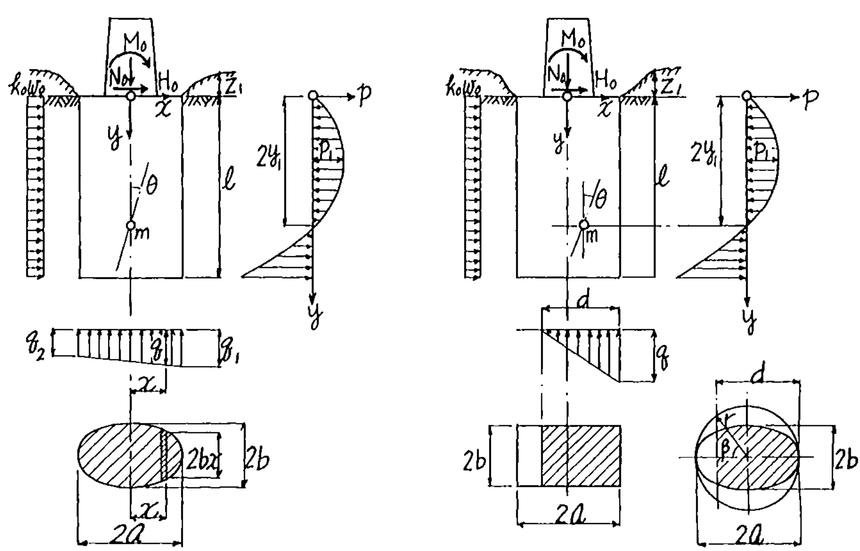
A = area of the bottom of a caisson

Other notations are the same as that of slender caissons or are shown in Fig. 16-2.



a) Longitudinal direction. b). Transverse direction.

Fig. 16-1 - Loads and Horizontal Earth Pressures Acting on a Slender Caisson



a). Base reaction lying in the core. b). Base reaction lying outside of the core.

Fig. 16-2 - Loads and Earth Pressures Acting on a Short Caisson

The Case of Base Reaction Lying Outside of the Core

For a rectangular section, (Fig. 16-2b),

$$p = K_1 y(2y_1 - y)\theta \tag{10}$$

$$q = d\theta K_2 \ell \tag{11}$$

The following equations are obtained from the equilibrium of forces and moments.

$$\theta = \frac{N_o + w_o \ell}{d^2 K_2 \ell b} \tag{12}$$

$$y_1 = 3\ell + \frac{d^2 K(H_o + w_o k_o \ell)}{2(N_o + w_o \ell)\ell} \tag{13}$$

$$M_o - \frac{k_o w_o \ell^2}{2} + \left\{\frac{7\ell^3}{2Kd^2} - \left(a - \frac{d}{3}\right)\right\}(N_o + w_o \ell) + \frac{2}{3}\ell(H_o + w_o k_o \ell) = 0 \tag{14}$$

The value of d can be obtained from Eq. (14), then y_1, θ, q and p are calculated by Eq. (10)-(13). Notations in the equations are shown in Fig. 16-2 or in the previous section.

For a circular or elliptical section, the following equations are deduced similarly to the case above.[1]

$$p_1 = \frac{3\left\{k_o w_o \ell^4 + 3H_o \ell^3 + 4M_o \ell^2 + 8m_2 K a^3 (k_o w_o \ell + H_o)\right\}^2}{4b\ell^3(\ell^3 + 24m_2 K a^3)(k_o w_o \ell^2 + 4H_o \ell + 6M_o)} \leq Cw(y_1 + Z_1) \tag{15}$$

$$q = \frac{3aK(k_o w_o \ell^2 + 4H_o \ell + 6M_o)}{b(\ell^3 + 24m_2 K a^3)}(1 + \cos\beta) \leq \text{permissible soil pressure} \tag{16}$$

$$y_1 = \frac{k_o w_o \ell^4 + 3H_o \ell^3 + 4M_o \ell^2 + 8m_2 K a^3 (k_o w_o \ell + H_o)}{2\ell(k_o w_o \ell^2 + 4H_o \ell + 6M_o)} \tag{17}$$

From the equilibrium of vertical forces,

$$N_o + w_o \ell = \frac{6Ka^2 m_1 (k_o w_o \ell^2 + 4H_o \ell + 6M_o)}{\ell^3 + 24m_2 K a^3} \tag{18}$$

where

$$m_1 = \frac{1}{3}\sin^3\beta + \frac{1}{2}\sin\beta \cdot \cos^2\beta + \frac{\pi - \beta}{2}\cos\beta$$

$$m_2 = \frac{3}{2}\left(\frac{\pi - \beta}{8} + \frac{1}{3}\sin^3\beta \cdot \cos\beta + \frac{\sin 4\beta}{32}\right)$$

The values of m_1 and m_2 are obtained from the condition satisfying Eq. (18), assuming the value of β. Then the values of p_1, q, and y_1 are calculated.

The method for designing the longitudinal section of a caisson and the method of combination of earth pressures to decide the cross-section are the same as that of slender caissons.

CAISSONS IN COHESIVE SOILS AND SUNK ON HARD LAYERS(2)

Stability Calculation

When the ultimate condition of the stability of a caisson is considered in an earthquake, it is assumed that the seismic force of a caisson and the active earth pressure around it act in the same horizontal direction and the passive earth pressures around the caisson act in the opposite direction. It is supposed that the passive earth pressure is, near the surface of ground, the same as used for design of retaining walls and, at the deep place of the ground, the same as the common ultimate bearing capacity of a soil.

Ultimate bearing capacity of a soil, according to Dr. G. G. Meyerhof,[3] is written

$$q = CN_c + \gamma D_f N_q + \gamma \frac{B}{2} N_\gamma \tag{19}$$

If Eq. (19) is applied in the horizontal direction, it is supposed that the third term is zero and γD_f is equivalent to the active earth pressure acting in the opposite direction at the same depth as the passive earth pressure. Therefore, the effective earth pressure, p_e, acting on a caisson is the difference of passive and active earth pressure as follows:

$$P_e = P_p - P_a = CN_c + P_a(N_q - 1) \tag{20}$$

When the angle of internal friction is neglected in cohesive soils, the value of N_c and N_q for strip foundations, according to Dr. Meyerhof, are approximately 8 and 1. Therefore the effective earth pressure is written 8C from Eq. (20), in which C is the cohesion of a soil.

A reinforced concrete caisson based on a hard layer through cohesive soils is assumed to rotate as a rigid body about its bottom in an earthquake. In the ultimate condition, it is assumed that the effective earth pressure is 2C at the surface and increases to a maximum value of 8C at the depth of 2b, then decreases linearly from the depth of $\ell/2$ and becomes zero at the bottom, considering the rotation of a caisson (Fig. 16-3a).

The base reaction is equal to the resultant normal force at the bottom of a caisson and is distributed in the form of a trapezoid or triangle, the maximum value of which is obtained from the common ultimate bearing capacity theory. Then the base reaction and resisting base moment are obtained:

$$R = N_o + w_o\ell = \int_d^x q\, dA \tag{21}$$

$$M_r = Re \tag{22}$$

When the base reaction of a circular caisson lies outside of the core, the resultant force, R, and the resisting base moment, M_r, are written as follows:

$$R = N_o + w_o\ell = \frac{2r^2q}{1 + \cos\beta}\, m_1 \tag{23}$$

$$M_r = Re = \frac{r^3q}{1 + \cos\beta}\, m_2 \tag{24}$$

where

$$m_1 = \frac{1}{3}\sin^3\beta + \frac{1}{2}\sin\beta \cdot \cos^2\beta + \frac{\pi - \beta}{2}\cos\beta$$

$$m_2 = \frac{\pi - \beta}{4} + \frac{\sin 4\beta}{16} - \frac{2}{3}\sin^3\beta \cdot \cos\beta$$