SEISMIC DESIGN OF BRIDGES

SECTION 9

EARTH RETAINING STRUCTURES

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9.0 NOTATION

a	distance of roller from wall
b	width of roller
B	reaction at base of sliding wall
C_d	compressional wave speed in soil
C_o	peak ground acceleration coefficient
$C_{H\mu}$	earthquake base shear coefficient
D	dead load
d	displacement of centre of mass of wall
E_s	Young's modulus for soil
g	acceleration due to gravity
H	height of wall
K_o	at rest earth pressure coefficient
K_o'	at rest pressure coefficient for unloading $= \frac{1}{K_o}$
K_A	active earth pressure coefficient
K_{AE}	active earth pressure coefficient for combined gravity and earthquake loads
K_P	passive earth pressure coefficient
L	live load
n	number of acceleration pulses
N	threshold acceleration coefficient
P_A	force due to active earth pressure
ΔP_{AE}	increment in active earth force due to earthquake
P_{AE}	$= P_A + \Delta P_{AE}$
P_C	earth force due to compaction
ΔP_E	increment of decrement in static earth force due to earthquake
ΔP_{E1}, ΔP_{E2}	two different values of ΔP_E
P_F	earth force against forced wall
$P_{F\theta}$	earth force due to rotational forcing of wall
P_{FU}	earth force due to translational forcing of wall
P_I	inertia force on wall, abutment or bridge superstructure
P_L	maximum force transmitted from superstructure to abutment via force limiting connection
P_O	force due to at rest earth pressure
ΔP_{OE}	increment or decrement in at rest earth force due to earthquake
P_P	force due to passive earth pressure
ΔP_{PE}	decrement in passive earth force due to earthquake
P_{PE}	$= P_P - \Delta P_{PE}$
P_S	force due to static earth pressure
P_{S1}, P_{S2}	two different values of P_S
p	weight of roller per unit width
R	reaction at failure surface of sliding wedge
T	period of vibration
ΣT_{max}	sum of maximum tensions in reinforcing strips
u	displacement of forced wall
V_o	peak ground velocity
W_a, W_b	weight of abutment, bridge
W_s, W_w	weight of soil, wall
y	depth below top of wall
Z_H	probability coefficient
α_{AE}	angle of inclination of soil failure surface
α_{RE}	angle of inclination of failure surface in reinforced earth
β	coefficient for location in seismic zone B
γ	unit weight of soil
$\Delta\mu$	maximum response displacement
ε_a	axial strain
θ	angle of rotation of wall

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μ	displacement ductility factor
ν	Poisson's ratio for soil
π	numerical constant
e	earthquake induced curvature
σ_μ, σ_ν	orthogonal stresses at infinity
σ_x	orizontal pressure
ϕ	angle of internal friction of soil
ϕ_b	angle of base friction
ϕ_W	angle of wall friction

9.1 DESIGN LOADS:

9.1.1 Pseudo-static methods of analysis for determining earth pressure and structures displacements are recommended.

9.1.2 The peak horizontal ground accelerations and velocities to be used in computing inertia forces and wall displacements should be as follows:

TABLE 9.1

Seismic Zone	A	B	C
peak acceleration $C_o g$	$0.5Z_H \cdot g$	$0.5\beta Z_H . g$	$0.15Z_H g$
peak velocity V_o(m/sec)	$0.6Z_H$	$0.6\beta Z_H$	$0.2Z_H$

Coefficients Z_H and β are given in table 2.1 and figure 2.1.

9.1.3 The choice of design life and earthquake return period, which determine Z_H, should be made in accordance with section 1.3. Where a retaining wall forms part of a bridge abutment and premature yielding or excessive displacement of the wall would contribute to collapse of the bridge, the same design life and earthquake return period as used for the bridge would apply. In most other cases it is expected that the criteria set out in section 1.3.3 for "Bridges of less than average importance" would be applicable to retaining walls.

9.1.4 Vertical earthquake accelerations may be neglected.

9.1.5 The inertia force of the retaining wall itself and inertia forces transmitted to bridge abutments from the superstructure should be included in the analysis.

9.1.6 Ideally all structural components of a wall should be capable of resisting the following load combinations without exceeding the dependable strength of any member.

$$U = 1.35\ D + 1.8\ P_S + 2.25\ L \qquad (9.1)$$

$$U = 1.35\ D + 1.35\ P_S + (\Delta P_E + P_F + P_I) \qquad (9.2)$$

$$U = 0.9\ D + 1.35\ P_S + (\Delta P_E + P_F + P_I) \qquad (9.3)$$

Transitory loads should be omitted and/or any load factor taken as unity if doing so produces a worse effect.

To ensure satisfactory post-elastic behaviour in the event of design loads being exceeded, capacity design principles as set out in section 3 should be followed and potential plastic hinge regions should be detailed to meet the minimum requirements of section 5.

9.1.7 If permanent outward movement during earthquake is not acceptable the external stability of the wall should be checked using load factor equations (9.2) and (9.3) above in combination with under capacity factors for the soil not exceeding 0.9.

If permanent displacement during earthquake can be tolerated the load factors in equations (9.2) and (9.3) may be taken as unity for the designed mode of failure but a variation of at least ± 15% on the probable soil strengths should be allowed for to determine upper and lower bounds on threshold acceleration.

9.2 EARTH PRESSURES:

9.2.1 In general the total earth pressure during earthquake equals the sum of three components:

(a) Static pressure due to gravity loads.

(b) Dynamic pressure due to earthquake.

(c) Forced wall pressure due to displacement of the wall into the backfill.

These soil pressure components may be calculated by the following methods:

(a) Elastic theory.

(b) Approximate plasticity theory, e.g., Coulomb and Mononobe-Okabe.

(c) Computer analyses, modelling the soil as Winkler springs or as finite elements.

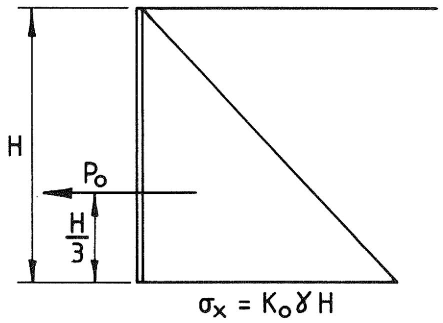
Figures 9.1, 9.2 and 9.3 show solutions for the components of earth pressure against a smooth vertical wall retaining cohesionless soil with a horizontal ground surface and the water table below the base of the wall.

A linear variation of soil pressure with wall movement between the upper and lower bound solutions may be assumed.

9.2.2 STATIC PRESSURES:

9.2.2.1 At Rest Pressure

At rest pressure, figure 9.1(a), acts against a perfectly rigid wall.

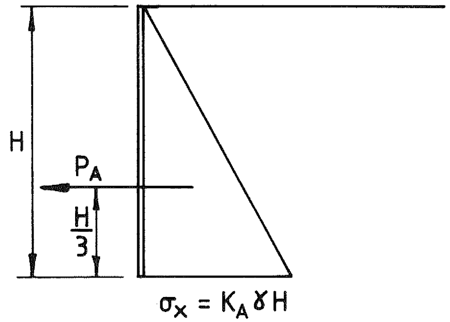


$$P_o = \frac{1}{2} \cdot K_o \gamma H^2$$

where $K_o = 1 - \sin\phi$

or $K_o = \dfrac{\nu}{1-\nu}$

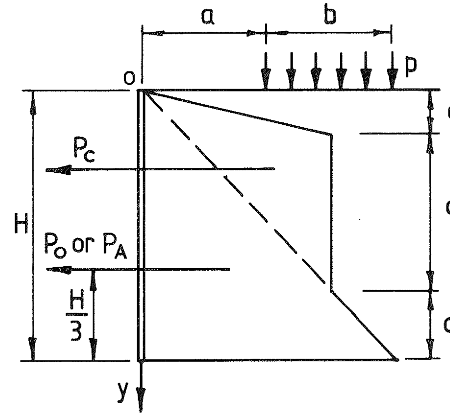
(a) AT REST PRESSURE



$$P_A = \frac{1}{2} K_A \gamma H^2$$

where $K_A = \dfrac{1-\sin\phi}{1+\sin\phi}$

(b) ACTIVE PRESSURE



$\sigma_x = K_o' \gamma y$ or $K_p \gamma y$

$\sigma_x = \dfrac{b}{a+b} \sqrt{\dfrac{2p\gamma}{\pi}}$

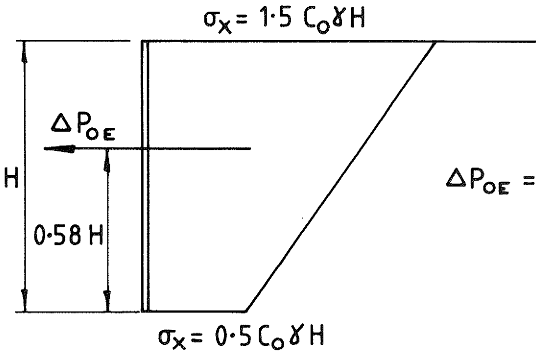
$\sigma_x = K_o \gamma y$ or $K_A \gamma y$

$K_o' = \dfrac{1}{K_o}$

$K_p = \dfrac{1}{K_A}$

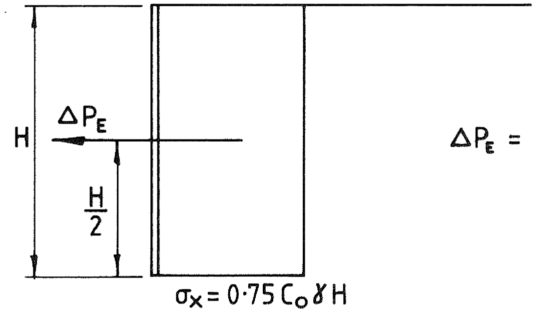
(c) COMPACTION PRESSURE

FIG. 9·1 : STATIC EARTH PRESSURES



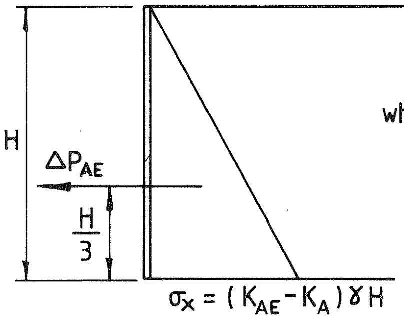
$$\Delta P_{OE} = C_o \gamma H^2$$

(a) RIGID WALL PRESSURE



$$\Delta P_E = 0{\cdot}75\, C_o \gamma H^2$$

(b) STIFF WALL PRESSURE



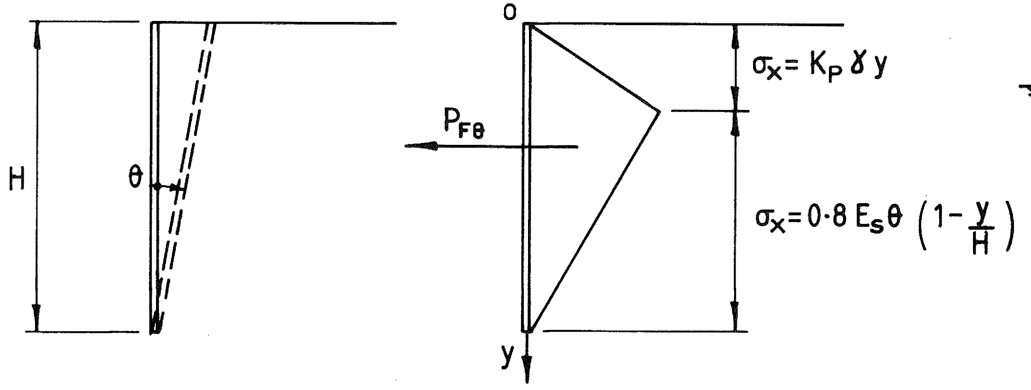
$$\Delta P_{AE} = \frac{1}{2} (K_{AE} - K_A) \gamma H^2$$

where $K_{AE} = \dfrac{\cos^2(\phi - \alpha)}{\cos^2\alpha \left[1 + \sqrt{\dfrac{\sin\phi \cdot \sin(\phi - \alpha)}{\cos\alpha}}\right]^2}$

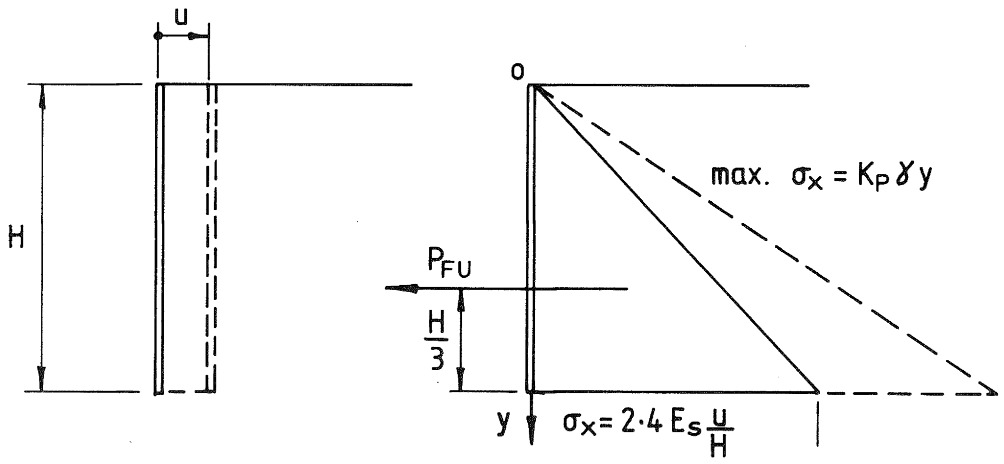
and $\alpha = \tan^{-1} C_o$

(c) ACTIVE PRESSURE

FIG. 9·2 : DYNAMIC EARTH PRESSURES

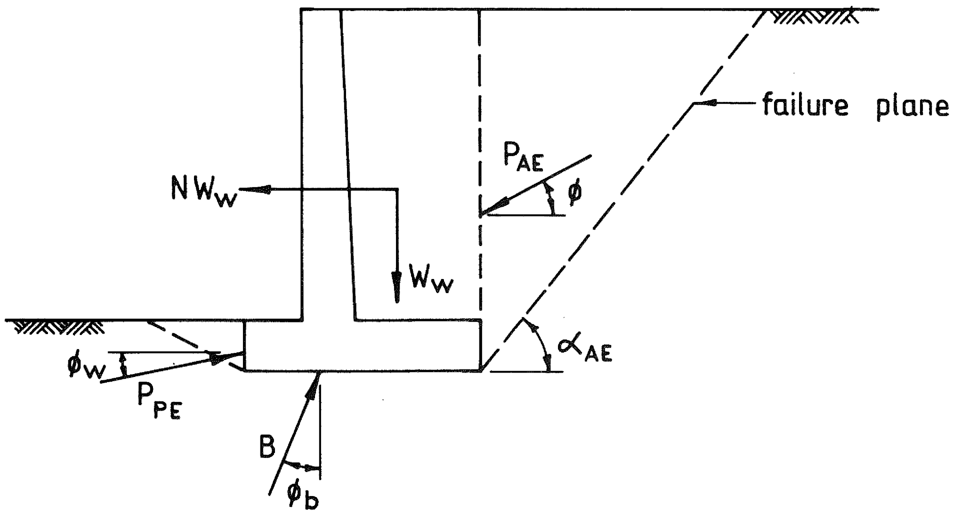


(a) WALL ROTATED

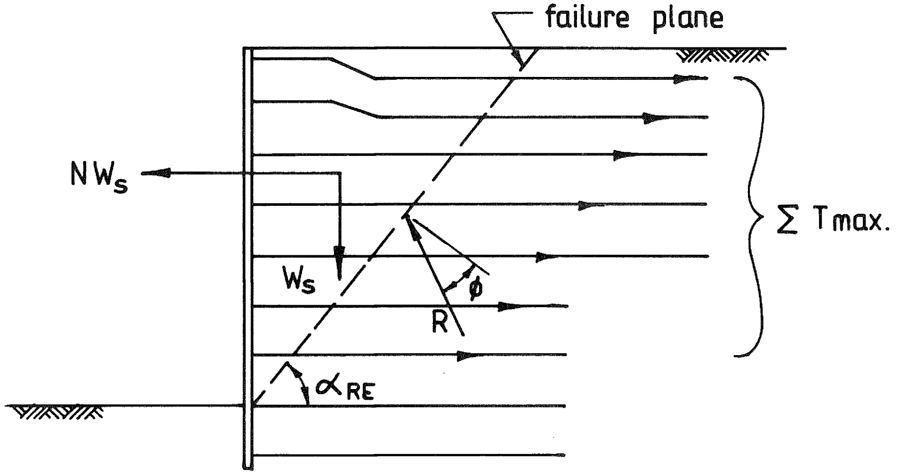


(b) WALL TRANSLATED

FIG. 9·3 : FORCED WALL PRESSURES



(a) CANTILEVER WALL



(b) REINFORCED EARTH WALL

FIG. 9·4 : DISPLACEABLE WALLS

9.2.2.2 Active Pressure

If the top of the wall moves outwards sufficiently to develop a fully plastic state of stress in the soil the pressure reduces to the active value shown in figure 9.1(b).

9.2.2.3 Compaction Pressure

If backfill is compacted in layers with a roller of weight pb travelling at distance a from the wall the pressure distribution shown in figure 9.1(c) should be assumed. For a vibrating roller pb equals the dead weight of the roller plus the centrifugal force induced by the roller vibrating mechanism.

9.2.3 DYNAMIC PRESSURES:

9.2.3.1 Rigid Wall

Figure 9.2(a) shows an approximation to the earthquake induced pressure on a perfectly rigid wall.

9.2.3.2 Deformable Wall

For a relatively stiff wall the earthquake pressure shown in figure 9.2(b) should be assumed. A movement of the top of the wall of between 0.1%H and 0.2%H, under the combined static and dynamic thrusts, would be needed to obtain this reduction from rigid wall pressure.

If the wall is sufficiently flexible for the top to move outwards by at least 0.5%H, under the combined static and dynamic thrusts, the active pressure increment shown in figure 9.2(c) may be assumed, but 0.33H above the base should be regarded as a lower bound for the position of the resultant thrust.

9.2.3.3 Displaceable Wall

If a wall is free to move outwards then subjected to a threshold acceleration Ng less than the peak acceleration of the design earthquake, $C_o g$, and the load - displacement behaviour is essentially rigid-plastic, the forward movement of the centre of mass may be estimated from:

$$d = \frac{3V_o^2}{C_o g} \left[\frac{C_o}{N} + \frac{N}{C_o} - 2 \right] \qquad (9.4)$$

If this permanent displacement is acceptable the structure need only be designed to withstand dynamic pressure due to an acceleration Ng.

9.2.4 FORCED WALL PRESSURES:

Figures 9.3(a) and (b) show the earth pressures against walls subjected to rotational and translational displacements.

In the general case an adequate estimate of the pressure against a forced wall may be made by modelling the soil as a system of Winkler springs.

It may be assumed that the pressure at any depth does not exceed the passive pressure but it should be noted that wall friction may significantly increase the passive pressure coefficient.

9.3 FREE STANDING WALLS:

9.3.1 Either of two approaches may be taken to the design of free standing walls.

(a) Design the wall to remain elastic and not suffer any permanent displacement during the design earthquake.

(b) Accept, but limit the magnitude of, permanent outward movement of the wall and design for a mode of failure which avoids yielding of structural elements wherever practicable.

9.3.2 WALLS FOUNDED ON SOIL:

For gravity, counterfort and cantilever walls founded on soil permissible permanent displacement should be adopted as the prime criterion for earthquake design.

A sliding, rather than a rotational, mode of failure should be aimed for and due account should be taken of the probable variation in soil strength when estimating the threshold acceleration to cause movement and the wall displacement.

The relationship between threshold acceleration and displacement given in section 9.2.3.3 may be assumed.

All structural elements should be designed in accordance with section 9.1.6.

The forces to be considered in the sliding block analysis of a cantilever wall are indicated in figure 9.4(a). Soil contained between the stem, base and virtual back of a cantilever or counterfort wall should be considered part of the wall mass when calculating inertia forces.

9.3.3 WALLS FOUNDED ON ROCK OR PILES:

9.3.3.1 If yielding of structural elements is to be avoided during the design earthquake, earth pressures and wall inertia forces should be based on the peak ground accelerations specified in section 9.1.2 and due account should be taken of the stiffness of such structures in estimating earth pressure.

9.3.3.2 To obtain significant permanent outward movement it will, generally, be necessary to allow yielding of the wall stem or piles. The consequent loss of serviceability, or the cost of removing backfill and repairing the wall, may be justifiable on economic grounds.

The minimum specified and probable yield strengths of reinforcement should be considered when estimating upper and lower bounds for the threshold acceleration to cause permanent deformation.

Section 9.2.3.3 may be assumed to apply, but adjustment for wall rotation is needed to determine the displacement of the top of the wall as the formula gives the displacement of the centre of mass.

9.3.4 TIED BACK WALLS:

9.3.4.1 If tie backs are restrained by some form of deadman anchor and ties are required to remain elastic during earthquake, the peak ground acceleration, from table 9.1.2, should be used to calculate the earthquake earth pressure due allowance being made for tie flexibility.

The mode of failure, in the event of overload, should be by yielding of the ties rather than failure of the wall face or connections between the ties and the wall or anchor.

9.3.4.2 If tie backs are restrained by a moveable anchor, such as a friction slab designed to slide while the rest of the structure remains elastic, limited permanent displacement could be tolerated with a consequent reduction in the acceleration to be used in computing dynamic earth pressure. The probable variation in friction slab resistance should be allowed for in determining threshold accelerations and the strength of ties and connections.

9.3.4.3 For walls of minor importance permanent displacement resulting from yielding of ties may be acceptable but particular consideration should be given to the post-earthquake effectiveness of the tie corrosion protection system in this case.

9.3.5 REINFORCED EARTH WALLS:

9.3.5.1 If the wall is to remain elastic and not suffer any permanent displacement the face panels and reinforcing strips should be capable of resisting a dynamic thrust in the range.

$$0.75\ C_o \gamma H^2 \geqslant \Delta P_E \geqslant 0.50\ C_o \gamma H^2, \qquad (9.5)$$

uniformly distributed over the height of the wall.

9.3.5.2 Alternatively the design may be based on permissible permanent displacement provided outward movement results from pull out of the reinforcing strips or, where this is impracticable, by ductile extension of the strips.

If ductile extension of the tie, rather than failure of the connection to the face panel, cannot be guaranteed the ultimate strength of the connection should exceed the tie pull out force, based on the probable apparent coefficient of friction, by a factor of at least 2.

The formula given in section 9.2.3.3, (9.4) may be used to estimate displacement.

To ensure that permanent deformations are within the capacity of facing panels to accept relative movement, the displacement of the top of the wall should not exceed 3% of the wall height.

The failure surface may be assumed to be a plane passing through the intersection of the wall face and the ground surface in front of the wall provided the top layers of reinforcing are draped, or their pull out resistance increased by other means, to prevent premature displacement of the upper section of the wall.

The angle of inclination of the failure surface should be determined by considering the equations of equilibrium of the failure wedge with the ties acting horizontally.

Upper and lower bounds on the threshold acceleration required to produce incipient failure should be calculated by considering the ties acting (a) horizontally and (b) along the failure surface and allowing for probable variations in the pull out resistance and yield strength of the ties.

The forces to be considered in the sliding block analysis of a reinforced earth wall are shown in figure 9.4(b).

9.3.5.3 External stability should be checked by considering active earthquake earth pressure (P_{AE}), acting against the interior face of the semi-rigid reinforced earth block. The peak ground acceleration ($C_o g$) should be used for walls designed to section 9.3.5.1 and the upper bound on threshold acceleration (Ng) should be used for walls designed to section 9.3.5.2.

9.4 BRIDGE ABUTMENT WALLS:

9.4.1 FORCE LIMITING CONNECTION TO SUPERSTRUCTURE

9.4.1.1 Where the connection is such that the force transmitted between the superstructure and the abutment is independent of their relative movement an upper limit can be placed on the force transmitted (P_L) and the abutment designed to withstand the forces shown in figure 9.5.

ΔP_E and P_I should be assumed in phase. P_L may, or may not, be in phase with ΔP_E and P_I.

9.4.1.2 The magnitudes of P_S, ΔP_E and P_F are dependent on the movement of the abutment wall relative to the retained soil. Limits on these forces are:

$$P_O + P_C \geqslant P_S \geqslant P_A \qquad (9.6)$$

$$\Delta P_{OE} \geqslant \Delta P_E \geqslant \Delta P_{AE} \qquad (9.7)$$

$$P_P \geqslant P_S + P_F \geqslant P_O + P_C \qquad (9.8)$$

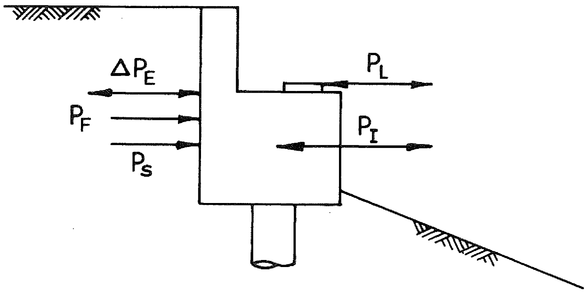


FIG. 9·5 : LOAD LIMITING CONNECTION TO BRIDGE SUPERSTRUCTURE

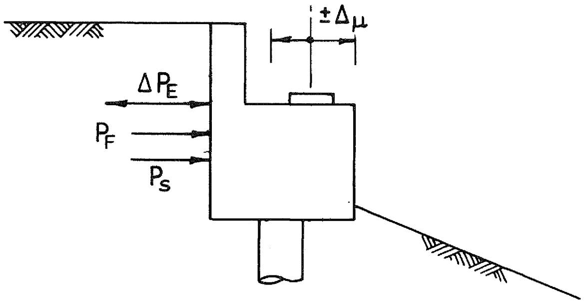
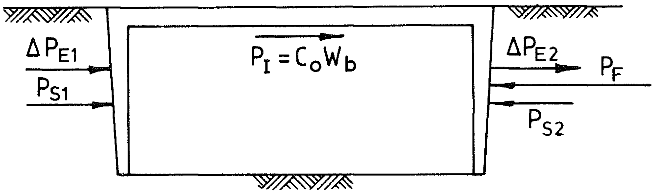
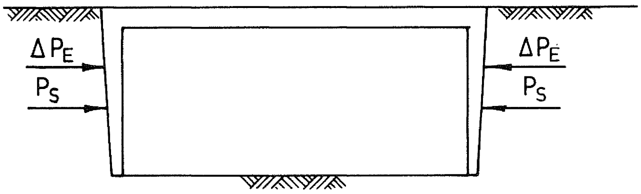


FIG. 9·6 : RIGID CONNECTION TO BRIDGE SUPERSTRUCTURE - NO SIGNIFICANT INTERACTION WITH SOIL



(a) DYNAMIC PRESSURES IN PHASE



(b) DYNAMIC PRESSURES OUT OF PHASE

FIG. 9·7 : RIGID CONNECTION TO BRIDGE SUPERSTRUCTURE - SIGNIFICANT INTERACTION WITH SOIL

If the abutment moves towards the soil it may be assumed that $\Delta P_E = \Delta P_{OE}$.

If the abutment moves away from the soil $P_F = 0$. The peak ground acceleration C_o should be used to calculate ΔP_E and P_I unless permanent outward movement of the abutment is permitted.

9.4.2 RIGID CONNECTION TO SUPERSTRUCTURE

9.4.2.1 A rigid connection may be hinged or moment resisting but no significant relative horizontal movement can develop between the abutment and the superstructure.

The two basic categories of bridges linked by rigid connections to their abutments are -

(1) Systems in which the soil pressures make no significant change to the dynamic response of the bridge.

(2) Systems where the abutment soil pressures have a strong influence on the dynamic response of the bridge.

9.4.2.2 No Significant Interaction

The maximum response displacement $\Delta\mu$ of the bridge superstructure may be calculated as specified in section 2.2.2 ignoring the influence of the retained soil, and earth pressures estimated by applying this displacement to the abutment.

The forces acting on the abutment are shown in figure 9.6. $\Delta\mu$ may, or may not, be in phase with ΔP_E. The limits on P_S, ΔP_E and P_F are as stated in section 9.4.1.2.

9.4.2.3 Significant Interaction

Two cases should be considered:

(1) Dynamic components of earth pressure in phase, figure 9.7(a):

The peak ground acceleration C_o should be used to calculate the bridge inertia force and the dynamic components of earth pressure.

The period of vibration should be calculated, taking the influence of the soil restraint on the overall system stiffness into account, and the required displacement ductility factor for the structure determined from figure 2.3, 2.3 or 2.4.

The limits on P_S, ΔP_E and P_F are as stated in section 9.4.1.2. For initial calculation it may be assumed that:

$P_{S1} = P_A$, $P_{S2} = P_O + P_C$

$\Delta P_{E1} = \Delta P_{AE}$, $\Delta P_{E2} = \Delta P_{OE}$

(2) Dynamic components of earth pressure out of phase, figure 9.7 (b):

It may be assumed that the structure remains stationary and is subjected to rigid wall pressures.

$$\text{i.e.} \quad \Delta P_E = \Delta P_{OE} \tag{9.9}$$

$$\text{and} \quad P_S = P_O + P_C \tag{9.10}$$

9.5 CULVERTS AND SUBWAYS:

9.5.1 AXIAL STRAIN

An upper bound on earthquake induced axial strain in a buried structure is given by:

$$\varepsilon_a = \pm \frac{V_o}{C_d} \tag{9.11}$$

9.5.2 CURVATURE

An upper bound on earthquake induced horizontal or vertical curvature along the length of a buried structure is given by:

$$e = \frac{C_o g}{C_d} \tag{9.12}$$

9.5.3 STRESS ON CROSS-SECTION

9.5.3.1 If the depth of soil over the structure exceeds the height of the structure, earthquake induced stresses on the cross-section may be determined by applying the static orthogonal stresses at "infinity" as shown in figure 9.8(b) and representing the soil between these planes of application and the structure, as well as the structure itself, as a system of finite elements.

The stresses at infinity are defined by:

$$\sigma_u = \pm \frac{V_o E_s}{C_d} \cdot \frac{(1 - \nu)}{(1 + \nu)(1-2\nu)} \tag{9.13}$$

$$\sigma_v = \pm \frac{V_o E_s}{C_d} \cdot \frac{\nu}{(1 + \nu)(1-2\nu)} \tag{9.14}$$

These stresses may be applied at any orientation to the structure.

9.5.3.2 If the soil cover is less than the height of the structure the loads shown in figure 9.8(a) should be used for the design of rigid structures.

9.5.3.3 Flexible corrugated steel plate structutes may be assumed to interact with the soil to produce a uniform distribution of earth pressure aroung the periphery.

9.6 STRUCTURAL DETAILING:

Particular attention should be paid to the following points when detailing reinforced concrete retaining walls:

(1) Laps in reinforcement at the base of a cantilever wall stem should be staggered.

(2) Joints subjected to opening moments should have all main bars anchored in a compression zone and diagonal bars should be provided across the joint to control cracking.

(3) Piles should be reinforced for ductile behaviour in the potential hinging region immediately below pile caps.

(4) Pile caps should be reinforced to resist the loads imposed by piles hinging.

COMMENTARY:

C9.1 DESIGN LOADS

C9.1.2 $C_o = C_{H\mu}$ for period T = 0 for each seismic zone.

C9.1.4 Vertical acceleration has a minor influence on earth pressures and since any peak horizontal acceleration may be accompanied by an upward or downward vertical acceleration there tends to be a cancelling of effects as far as wall displacement is concerned.[9.32] A nominal allowance for vertical acceleration is made in the load factor equations suggested in section9.1.6.

C9.1.5 Permanent displacement of retaining walls has been observed following earthquakes[9.7, 9.8, 9.11] and the influence of the wall inertia force on displacement has been demonstrated theoretically[9.24] and experimentally.[9.18]

C9.2 EARTH PRESSURES

C9.2.1 Only the simplest cases are illustrated in figures 9.1, 9.2, 9.3 to indicate the relative magnitudes of the earth pressure components.

The effects of backfill slope, wall friction, wall angle and water are dealt with in reference 9.19.

C9.2.2 STATIC PRESSURES

C9.2.2.3 Compaction Pressure

The expressions for compaction pressure given in figure 9.1(c) are from reference 9.14. Ingold's solution has the advantage of simplicity over the methods proposed by Broms.[9.3] If the wall can deform plastically, (eg. gabion and cribwalls), as backfill is placed compaction pressure may be insignificant.

C9.2.3 DYNAMIC PRESSURES

C9.2.3.1 Rigid and Deformable Walls
and

C9.2.3.2 The pressure distributions shown in figures 9.2(a) and (b) are approximations to solutions derived by Wood[9.33] for rigid and deformable walls assuming the soil to be in an elastic state. Reference 9.33 contains pressure distribution plots directly applicable to design of rigid, rotating and cantilever walls. Wood has shown that as the wall rotates or deflects away from the retained soil the magnitude of the dynamic increment in pressure reduces and the centre of pressure moves downwards towards the lower third point of the wall. It seems reasonable to assume a hydrostatic distribution of pressure when the soil reaches a fully plastic state but 0.33H above the base should be regarded as the lower limit for the position of the active dynamic thrust as recent research [9.18, 9.32] suggests that the location of the resultant thrust changes in a complex way during ground shaking.

It would be prudent to assume that ΔP_{AE} acts at 0.4H above the base when computing the overturning moment.

The expressions for the active dynamic thrust ΔP_{AE} given in figure 9.2(c) are simplifications of the Mononobe-Okabe formulae which are given in full in references 9.19 and 9.29.

The movement of 0.5%H to achieve an active state applies to cohesionless soil only.

C9.2.3.3 Displaceable Wall

Outward movement may result from sliding, tilting or yielding of the wall structure.

The sliding displacement mechanism is described by Richards and Elms[9.24] and the general validity of the theory has been verified experimentally by Lai and Berrill.[9.18] Studies of the tilting displacement problem are continuing.[9.1,9.32]

If a sliding block is subjected to a single triangular acceleration pulse with a peak of $C_o g$ and the block is displaced when the acceleration exceeds Ng the relative movement of the block is given by:

$$d = \frac{V_o^2}{2C_o g}\left[\frac{C_o}{N} + \frac{N}{C_o} - 2 - \frac{1}{12}\left(\frac{N}{C_o}\right)^3\right] \quad \text{for } \frac{N}{C_o} < 0.59 \qquad \text{(C9.1)}$$

where V_o is the peak ground velocity, or for n pulses

$$d \simeq \frac{nV_o^2}{2C_o g}\left\{\frac{C_o}{N} + \frac{N}{C_o} - 2\right\} \qquad \text{(C9.2)}$$

As shown in figure C9.1, a satisfactory fit to the upper bound displacement of a sliding block subjected to recorded natural earthquake ground motions is obtained when n = 6.

The sliding block model is a gross simplification of real wall- soil systems but the expression given in section 9.2.3.3 should predict the correct order of magnitude of wall displacement and further refinement is not justified at this stage.

C9.2.4 FORCED WALL PRESSURES

The approximations given in figures 9.3(a) and (b) are based on solutions derived by Wood[9.33] and Tajimi[9.31] for soil in an elastic state.

Using Winkler springs to simulate the interaction between wall and backfill Scott[9.28] obtained similar results.

Ghahramani and Clemence[9.12] give solutions for the dynamic passive pressure for wall translation and rotation about the top and bottom of the wall.

C9.3 FREE STANDING WALLS

C9.3.1 The first method will ensure a more serviceable structure but for some types of wall it may not be feasible or may be uneconomic.

The second method is a more realistic and economical approach but if outward movement cannot be obtained without yielding of structural members the cost of repairs to restore serviceability may be unacceptable.

C9.3.2 WALLS FOUNDED ON SOIL

The behaviour of gravity walls is discussed in references 9.6, 9.18, 9.24, 9.32.

The Manonobe-Okabe solution for P_{PE}, figure 9.4(a), is given in reference 9.29.

C9.3.5 REINFORCED EARTH WALLS

C9.3.5.1 References 9.4, 9.20 and 9.23 with analysis and design of reinforced earth structures for static and earthquake loads.

Static earth pressures based on measurements of stress within reinforced earth structures are generally used for design[9.20] but it is suggested that specific consideration be given to compaction pressure, as indicated in section 9.2.2.3, as it is the upper sections of the wall which are most vulnerable to damage during earthquake.[9.2,9.26]

The design earthquake loading generally specified[9.20] is an adaption of an empirical approach proposed by Richardson[9.25] on the basis of a dynamic test of a full scale wall.[9.26] Implicit in Richardson's method is acceptance of significant permanent outward movement of the wall. If permanent movement is to be prevented dynamic pressure similar to that on a relatively stiff conventional wall should be allowed for.

A uniform distribution of pressure is recommended as this approximates the distribution observed in the full scale test[9.26] and is consistent with the distribution of pressure against a relatively stiff conventional wall.

C9.3.5.2 A rational method for estimating the threshold acceleration for outward movement is described by Bracegirdle.[9.2] The limit of 3%H on permanent displacement is based on the report by Richardson et al[9.26] that their test wall tilted about 5% but there was no other observable damage or ill effect.

C9.4 BRIDGE ABUTMENT WALLS

Damage to bridge abutments during earthquakes has been well documented.[9.7, 9.8,9.11]

Stability of the approach embankments should be carefully considered[9.21] as slumping of batters, settlement or bodily movement may have a profound influence on abutment performance.

The interaction of the bridge - abutment - soil system is complex[9.1] but rational bounds can be put on earth pressure and displacements if the extent of interaction between abutment and superstructure is considered.[9.6]

C9.4.1 FORCE LIMITING CONNECTION TO SUPERSTRUCTURE

C9.4.1.1 A P.T.F.E. bearing or an energy abosorber with rigid-plastic characteristics are examples of load limiting connections.

An elastomeric bearing is not strictly a load limiting connection but the maximum force which can be transmitted through this type of bearing is generally known and this upper bound force can be used in the analysis.

Generally movement of the superstructure away from the soil will impose critical loads on the abutment foundations and movement towards the soil will impose critical soil pressures on the abutment walls.

Most abutments that are not rigidly connected to the bridge superstructure will be permanently displaced towards the superstructure during earthquake. This movement should be added to the estimated maximum response displacement of the superstructure to determine the required seismic gap.

P_L and ΔP_E are not necessarily in phase because the force transmitted via the load limiting connection would have a time-history dominated by the natural frequency of the bridge whereas the dynamic soil pressure will tend to follow the ground acceleration in magnitude and direction.

C9.4.1.2 If the abutment is forced towards the soil and the soil remains essentially elastic,

$$\Delta P_E = \Delta P_{OE} \qquad (C9.3)$$

If movement is sufficient to develop full passive pressure and the soil is being accelerated away from the wall,

$$P_S + P_F - \Delta P_E = P_P - \Delta P_{PE} = P_{PE} \qquad (C9.4)$$

The Mononobe-Okabe solution for P_{PE} [9.29] gives ΔP_{PE} approximately the same magnitude as ΔP_{OE} for average values of C_o and Ø.

C9.4.2 RIGID CONNECTION TO SUPERSTRUCTURE

C9.4.2.2 No Significant Interaction

A shallow, spill-through type, abutment propping a heavy superstructure with piers providing lateral resistance may fall into this category.

C9.4.2.3 Significant Interaction

Portal type bridges are typical of this category.

The loading shown in figure 9.7(a) is appropriate for a bridge founded on rock, since movement of the surrounding soil relative to the rock will occur during earthquake, but it is conservative for a bridge founded in soft soil, since in this case the whole structure will tend to move with the soil.

Typical portal bridges are likely to have relatively short periods of vibration initially and the soil-structure system may stiffen further as soil is consolidated against the abutments during shaking. Hence response may be close to the peak of the spectrum so that adequate ductility is required for survival.

C9.5 CULVERTS AND SUBWAYS

The expressions given in this section have been derived from elastic wave propogation theory. The application of elastic wave theory to estimate earthquake strain and stresses on underground structures is covered in references 9.5, 9.9, 9.13, 9.15, 9.17 and 9.30.

Abrupt differential ground movements that could be produced by faulting are likely to produce more severe effects. Methods of analysing long underground structures for fault movements are given in reference 9.16.

C9.5.1 AXIAL STRAIN

Expression (9.11) gives the strain arising from a compressional wave travelling in the direction of the longitudinal axis of the structure. This is an upper bound solution because:

(1) Superposition of peak particle velocities and ground accelerations over a long length of structure is unlikely, and

(2) slippage between the structure and the soil is likely to occur.

Development of frictional resistance and slippage is discussed in reference 9.30.

Values for V_o are given in section 9.1.2 and typical values for C_d are:

Compact granular soil	500 m/sec
silty sand	300 m/sec
medium clay	150 m/sec

Expression (9.11) is also applicable to above ground structures and may be used to estimate the relative displacement of bridge piers during earthquake.

C9.5.2 CURVATURE

Expression (9.12) gives the curvature arising from a shear wave travelling in a direction along the length of the structure.

This is also an upper bound solution because it is based on the assumption of no slippage between the soil and the structure.

C9.5.3 STRESS ON CROSS-SECTION

The method of applying static stresses at "infinity" appears to be accepted in Russia.[9.9,9.15] Duns and Butterfield[9.5] have shown that static solutions can give good approximations to dynamic wave propogation problems.

Alternative forms for expressions (9.13) and (9.14):

$$\sigma_u = \pm \frac{\gamma}{g} C_d V_o \qquad (C9.5)$$

$$\sigma_\nu = \pm \frac{\gamma}{g} C_d V_o \cdot \frac{\nu}{(1 - \nu)} \qquad (C9.6)$$

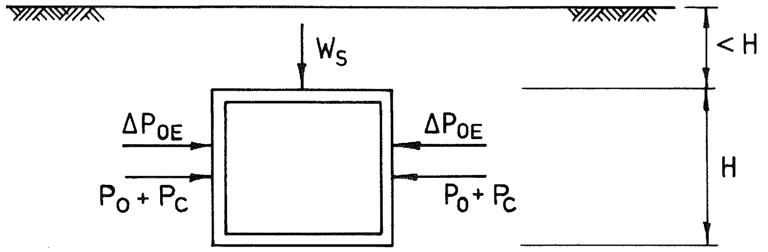
The expressions given for σ_u, σ_ν are upper bound solutions for the reasons given in section C9.5.1.

Where the finite element method is used it is recommended that the "infinity" stresses be applied at a distance of at least twice the effective "diameter" from the structure, as shown in figure 9.8. Where this is not possible, because of the proximity of the ground surface, the stress load falling above this boundary should be applied at the surface.

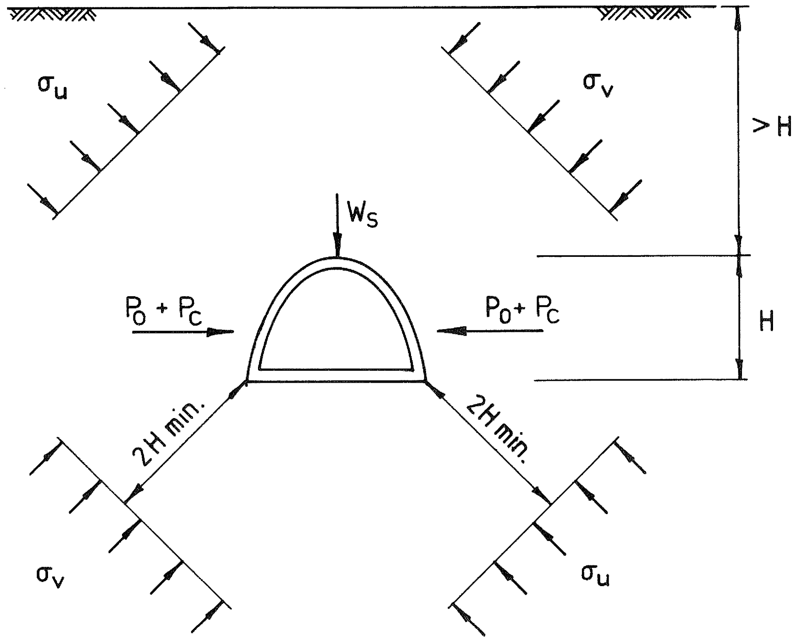
Solutions for the stresses in circular cylinders are given in references 9.5 and 9.27.

C9.6 STRUCTURAL DETAILING

Corner joints subjected to tension on the inside faces occur between wall



(a) 'SHALLOW' OVERBURDEN



(b) 'DEEP' OVERBURDEN

FIG. 9·8 : UNDERGROUND STRUCTURES

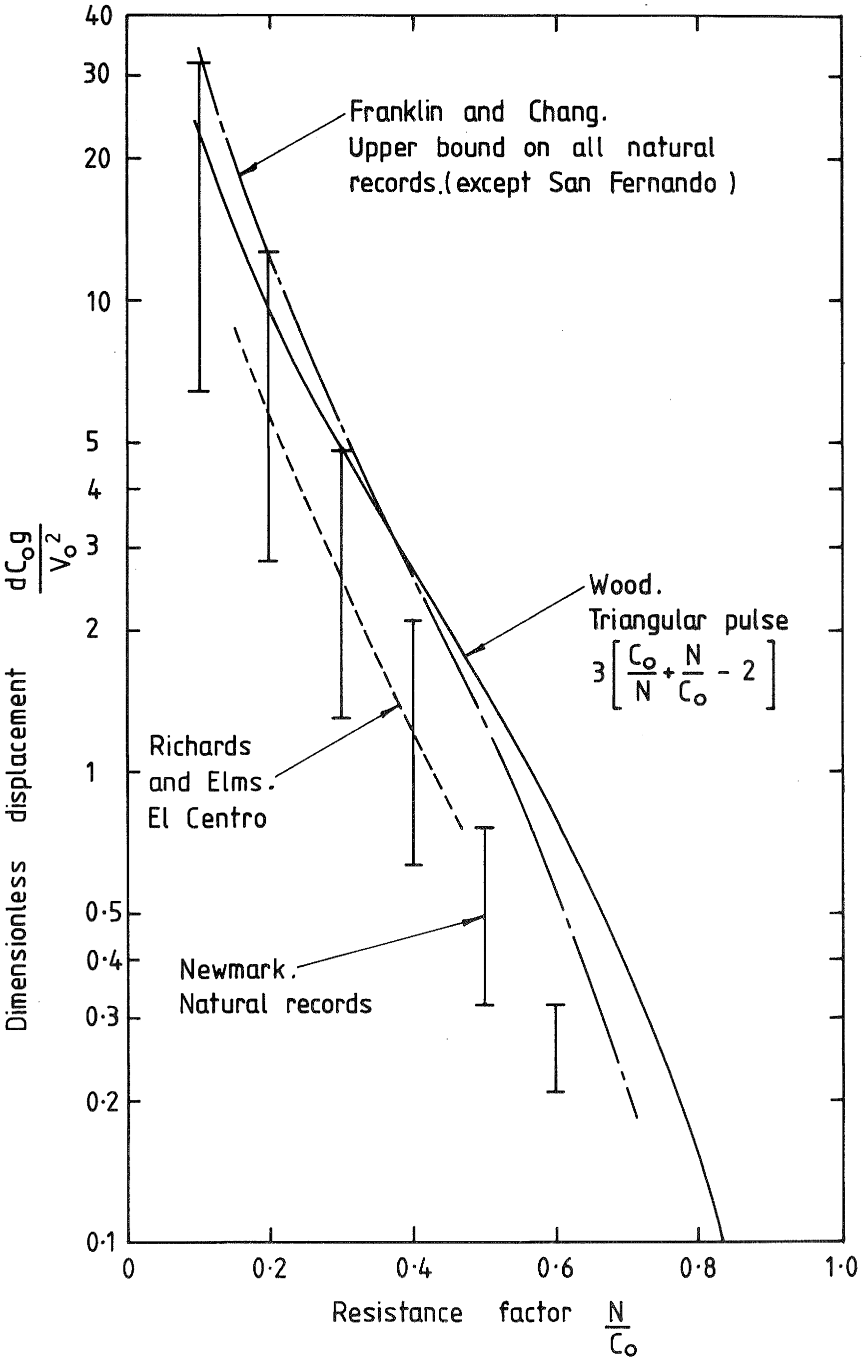


FIG. C9·1 : WALL DISPLACEMENT

bases and stems and between abutments and wingwalls. Adequate detailing is necessary to ensure that the ultimate strength of the members can be developed and cracks on the earth face are well distributed. Recommendations for detailing based on laboratory tests are given in reference 9.22.

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