IS 4651 (Part 2) : 1989

(Reaffirmed 1995)

Indian Standard

PORTS AND HARBOURS — PLANNING AND DESIGN — CODE OF PRACTICE

PART 2 EARTH PRESSURES

(First Revision)

UDC 627·2/·3:624·131·53

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October 1990

Price Group 4

Ports and Harbours Sectional Committee, BDC 66

FOREWORD

This Indian Standard (Part 2) (First Revision) was adopted by the Bureau of Indian Standards on 24 May 1989, after the draft finalized by the Ports and Harbours Sectional Committee had been approved by the Civil Engineering Division Council.

A great need has been felt for formulating standard recommendations relating to various aspects of water front structures. This standard is one of a series of Indian Standards formulated on this subject. This part (Part 2) deals with earth pressures. This standard was first published in 1969 mainly to cover provisions of earth pressure for the design of port and harbour structures.

In the first revision, the subject has been covered comprehensively to include all aspects related to earth pressure needed for the design of ports and harbours structures. The clause on earth pressure has been revised.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test of analysis, shall be rounded off in accordance with IS 2 : 1960 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

PORTS AND HARBOURS — PLANNING AND DESIGN — CODE OF PRACTICE

PART 2 EARTH PRESSURES

(*First Revision*)

1 SCOPE

1.1 This standard (Part 2) deals with earth pressures on waterfront structures required for ports and harbours.

2 REFERENCES

1893 : 1984 Criteria for earthquake resistant design of structures (*fourth revision*)

3 TERMINOLOGY

3.0 For the purpose of this standard, the following definitions shall apply.

3.1 Angle of Wall Friction

The roughness of retaining walls causes the active earth pressure and passive earth resistance to be inclined. The angle of this inclination to the normal to the wall is known as the angle of wall friction.

3.2 Arching Earth Pressure

When the top of the wall is restricted and there is freedom of movement towards the base, the earth pressure exerted by the backfill is known as arching earth pressure.

3.2.1 For movement away the backfill, it is termed arching active earth pressure.

3.2.2 For movement towards the backfill, it is termed arching passive earth resistance.

3.3 Earth Pressure

The pressure exerted by the backfill on the wall. The nature and extent of the wall movement alters the lateral pressure of the backfill exerted on it.

3.3.1 *Earth Pressure at Rest*

If there is no wall movement, the pressure exerted by the backfill is termed the earth pressure at rest.

3.3.2 *Active Earth Pressure*

If the wall moves sufficiently away from the backfill by translatory motion or rotation about the base of their combination, lateral pressure of the backfill is reduced and is termed active earth pressure. The movements required are small and about 0·001 (*see* **4.1**) for sand and 0·05 H for clay are sufficient to mobilize the active pressure.

3.3.3 *Passive Earth Resistance*

If the wall moves sufficiently towards the backfill, by translatory motion or rotation about the base of their combination, the backfill offers resistance which is termed passive earth resistance (also, less accurately, passive earth pressure).

3.4 Retaining Wall

A wall which holds back a soil mass is a retaining wall. The soil mass is referred to as a backfill.

4 SYMBOLS

4.1 For the purpose of this standard, the following symbols shall apply:

c, c_1 = apparent cohesion;

c', C'_1 = effective cohesion;

F = factor of safety;

H = height;

H_1 = equivalent height of soil for uniform surcharge q or a load;

i = slope angle;

K_0 = coefficient of earth pressure at rest;

K_A = coefficient of active earth pressure;

K_p = coefficient of passive earth resistance;

$N\phi$, $N\phi_1$ = the flow value $= \tan^2 (45° + \phi/2)$ or $\tan^2 (45° + \phi_1/2)$

P_0 = the (total) earth pressure at rest per unit length;

P_A = the (total) active earth pressure per unit length;

P_P = the (total) passive earth resistance per unit length;

P_a = the (total) earth pressure per unit length when arching is present, for example, in braced open cut;

p = pressure intensity;

p_o = intensity of earth pressure at rest;
p_A = intensity of active earth pressure;
p_P = intensity of passive earth resistance;
Q = point load;
q = intensity of uniformly distributed load or surcharge intensity;
z = depth from top of retaining wall;
α = inclination of the back of retaining wall with the horizontal;
γ, γ_1 = bulk unit weight of soil (soil particle + water + air);
γ', γ'_1 = submerged unit weight of soil;
γ_d = dry (bulk) unit weight of soil;
γ_{sat} = (bulk) unit weight of soil in fully saturated condition;
γ_w = unit weight of water;
δ = angle of wall friction;
σ = normal stress;
$\bar{\sigma}$ = normal effective stress;
$\bar{\sigma}_z$ = normal effective stress at depth Z;
ϕ, ϕ_1 = apparent angle of shearing resistance; and
ϕ', ϕ'_1 = effective angle of shearing resistance.

5 WALL MOVEMENTS

5.1 In water front structures, wharves constructed as gravity retaining walls have sufficient outward movement to mobilize active pressure and Coulomb's pressure distribution obtains.

5.2 Wharves constructed as sheet-pile retaining walls, either dredged in the front or backfilled, sufficiently yield to justify the adoption of Coulomb's active pressure distribution.

5.2.1 In highly plastic clays, pressures approaching at rest condition may develop unless wall movement can constitute with time.

5.3 Walls in the locks and in the dry docks, being monolithically constructed with the floors may not yield and earth pressure at rest in these cases should be taken.

5.4 Gravity wharf walls funded on cohesionless subgrade may not yield forward sufficiently as to mobilize full passive resistance. Corresponding to the yield for active pressure not more than half of the passive resistance may be available.

5.5 Gravity wharf walls founded on clayey subgrade may eventually sufficiently yield as to fully mobilize the passive resistance.

5.6 Usually the backfill in a gravity wharf wall settles more than the wall and the angle of wall friction for active earth pressure is positive. The cases of gravity wharf walls on clayey foundations require to be individually examined for settlement and if the wall should settle more with reference to the backfill, negative angle of wall friction will result.

5.6.1 Angle of wall friction for sheet-pile walls should be judged with reference to the deposition of the backfill and the relative movements of the wall and the backfill.

5.7 For passive resistance, as the earth normally heaves up due to wall movement, the angle of wall friction is generally positive.

5.7.1 But when the wall moves up relative to the backfill as it may happen in a sheet-pile wall which is strongly held back by anchor at the top and which, deflecting, moves out and upward in the embedded portion, negative angle of wall friction will result.

NOTE — Convention of sign for the angle of wall friction is given in Fig. 1.

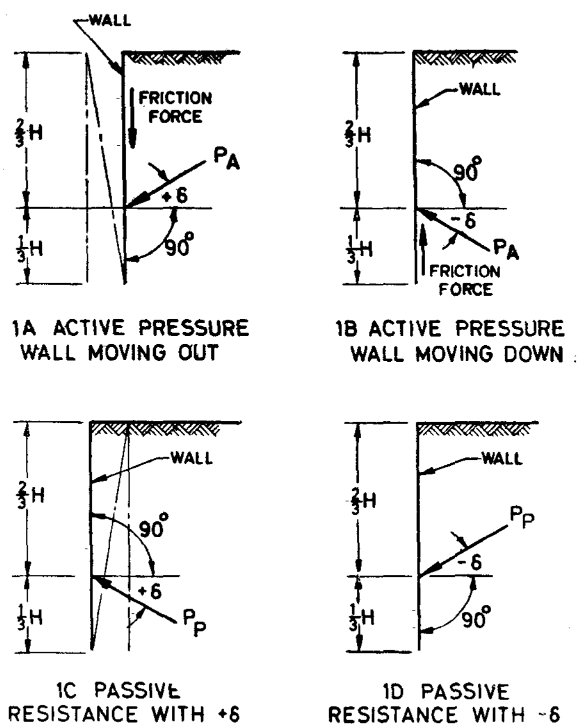


FIG. 1 CONVENTION OF SIGN FOR THE ANGLE OF WALL FRICTION δ

6 EARTH PRESSURE AT REST

6.1 For a horizontal backfill, the intensity of earth pressure at rest at a depth z is given by:

$$p_0 = K_0 \gamma z \quad \text{...(1)}$$

The field values of K_0 are given in Table 1.

Table 1 Field Values of Coefficient of Earth Pressure at Rest, K_0

(*Clause* 6.1)

Sl No.	Soil	K_0
(1)	(2)	(3)
i)	Loose sand	0·4
ii)	Dense sand	0·5 to 0·6
iii)	Sand, well tamped	0·8
iv)	Soft clay	0·6
v)	Hard clay	0·5

7 ACTIVE EARTH PRESSURE

7.1 In waterfront structures, the backfill is usually horizontal. With a uniform surcharge intensity q and uniform backfill:

$$P_A = \frac{1}{2}\left(\gamma + \frac{2q}{H}\right) H^2 \frac{K_A}{\cos\delta} - 2cH\sqrt{\frac{K_A}{\cos\delta}} + \frac{2c^2}{\gamma} \quad \ldots(2)$$

where

$$K_A = \frac{\sin^2(\alpha + \phi)\cos\delta}{\sin^2\alpha \sin(\alpha - \delta)} \times \frac{1}{\left\{1 + \sqrt{\frac{\sin(\phi + \delta)\sin\phi}{\sin(\alpha - \delta)\sin\alpha}}\right\}^2} \quad \ldots(3)$$

and

$\delta = \phi$ for stepped back walls,

$= \frac{2}{3}\phi$ for other cases of walls.

7.2 For broken back wall or irregular cohesionless backfill surface and loading, one of the graphical methods should be applied.

7.3 For a horizontal earth surface, smooth vertical back retaining wall, a two-layered backfill carrying surcharge and with water table, it is convenient to draw pressure diagram using the following equation (*see* Fig. 2).

$$\left.\begin{array}{l}
\text{For } 0 \leqslant Z \leqslant D,\ p_A = \dfrac{\bar{\sigma}_z}{N\phi} - \dfrac{2c}{\sqrt{N\phi}} \\
\quad \text{where } \bar{\sigma}_z = q + \gamma Z \\
\text{For } D \leqslant Z \leqslant h,\ p_A = \dfrac{\bar{\sigma}_z}{N\phi} - \dfrac{2c}{\sqrt{N\phi}} + \gamma_w (Z - D) \\
\quad \text{where } \bar{\sigma}_z = q + \gamma D + \gamma' (Z - D) \\
\text{For } h \leqslant Z \leqslant H,\ p_A = \dfrac{\bar{\sigma}_z}{N\phi_1} - \dfrac{2c_1}{\sqrt{N\phi_1}} + \gamma_w (Z - D) \\
\quad \text{where } \bar{\sigma}_z = q + \gamma D + \gamma' (h - D) + \gamma'_1 (Z - h)
\end{array}\right\} \ldots(4)$$

Negative value of p_A should be ignored.

7.3.1 This method may be used for more than two layers also.

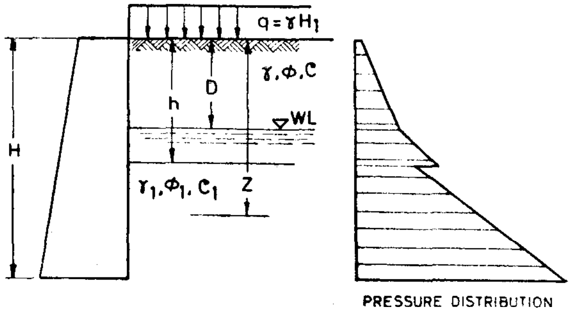


Fig. 2 A Surcharged Two-Layered Soil with Water Table

7.3.2 With rough wall and/or inclined-back retaining wall, equation (2) should be applied to find pressures for different heights and the total resultant should then be evaluated.

7.4 In case of vertical wall with horizontal ground surface, the active earth pressures should be computed in accordance with the procedure given in Annex A.

7.5 If the surface of the backfill carries a line load (such as, a rail line) W_L per unit length parallel to the crest of the wall, the simple approximate method suggested by Terzaghi and Peck, and indicated in Fig. 3 will be found to suffice (*see* 7.6.2). If point d is located below the base of the wall, the effect of the line load may be neglected.

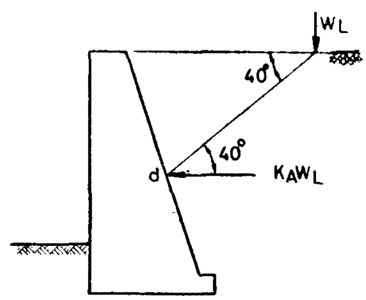


Fig. 3 Magnitude and Line of Action of Pressure Due to Line Load

7.6 In case of surface of the backfill carrying isolated load (such as, shear leg) the tentative approximate method given in **7.6.1** is suggested.

7.6.1 The line of action of the resultant force should be obtained by a construction similar to that for line load (*see* Fig. 3), the 40° line being constructed from the centre of the loaded area. It is assumed that if the length of the loaded area L and the distance between the back of the wall and the near edge of that area be X, the resultant lateral thrust will be distributed along a length of wall equal to $L + X$. Then if W_p be the load on the area, the resultant thrust per unit length of the wall will be

$$K_A \frac{W_p}{L + X}$$

7.6.2 Concentrated point loads or heavy line loads are usually carried on separate foundations, such as, piles transmitting pressures effectively below the base of the wall. If it is, however, necessary to load the backfill, the method shown in Fig. 4 based on Boussinesq equation modified by experiment, is recommended, and this is a more refined alternative to **7.5** and **7.6.1**.

7.7 For braced open cuts, the practice is to place the bracings and insert the struts as the excavation proceeds. This amounts to restraining the lateral movement at the top and permitting increasingly greater movement with increasing depth. The permissible movement, thus, does not conform to that necessary for Coulomb's pressure distribution. Actual pressure distribution and, therefore, the position of the resultant earth pressure and also magnitude will depend upon the actual mode of construction even for uniform soil. Empirical recommendations given in Fig. 5 and Table 2 may be followed.

7.8 Effect of External Causes on Earth Pressure

7.8.1 *Effect of Rainfall*

Hydraulic pressure due to rainfall should be taken into account. In a freely draining backfill, if the water entering it fills up all the voids and the water is being drained out, the hydraulic pressure will be one-half of the full static hydrostatic pressure $\left(0.5\ \gamma_w \frac{H^2}{2}\right)$.

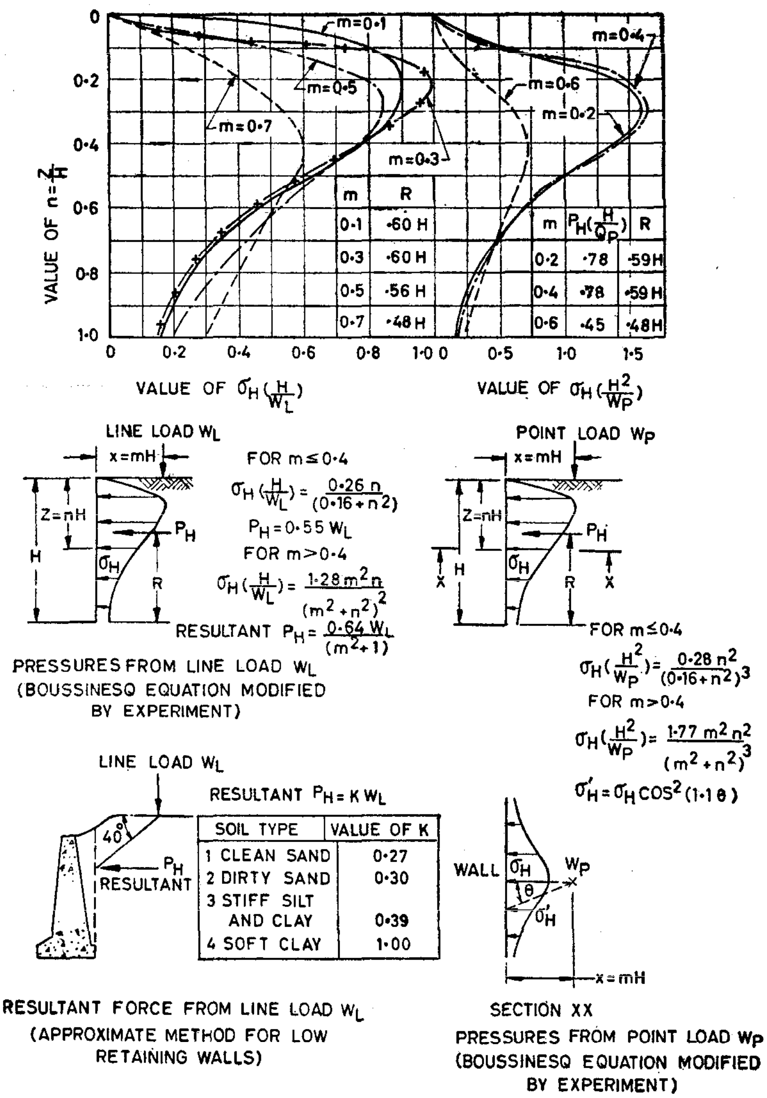
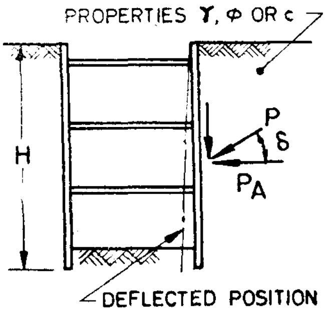
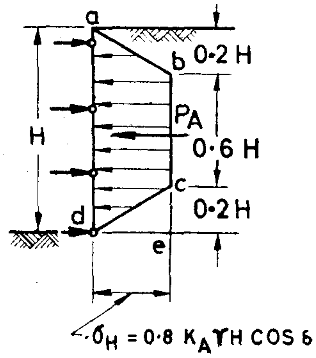


FIG. 4 HORIZONTAL PRESSURE ON WALL DUE TO SURCHARGE



Sheet piling or soldier beams driven. As excavation is deepened wales and braces placed in sequence. Wall deflection increases with depth, resulting in trapezoidal pressure distribution.



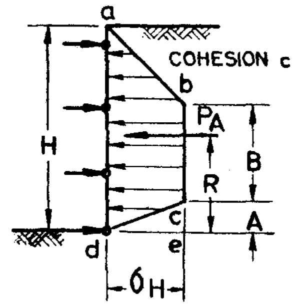
EXCAVATION IN SAND

Area *abcd* is pressure distribution in dense sand.

Resultant $P_A = (0{\cdot}64)\, K_A \gamma_H{}^2 \cos \delta$ acting 0·5 H above base of cut.

Area *abcd* is pressure distribution in loose sand. Resultant $P_A = (0{\cdot}72)\, K_A \gamma_H{}^2 \cos \delta$ acting 0·48 H above base of cut.

K_A is the coefficient of active earth pressure including effect of wall friction.



EXCAVATION IN CLAY

Area *abcd* is pressure distribution. Shape of pressure diagram and magnitude of pressures depends on value of stability number:

$$N_0 = \frac{\gamma_H}{C}$$

FIG. 5 PRESSURE IN BRACED CUTS

Table 2 Values of P_A, σ_H, A, B and R for Excavation in Clay
(Symbols are as in Fig. 5 under 'Excavation in Clay')

(*Clause* 7.7)

	$2 < N_0 < 5$	$5 < N_0 < 10$	$10 < N_0 < 20$	$20 < N_0$
P_A	0·78 $H\sigma_H$	0·78 $H\sigma_H$	$(2{\cdot}1 - 0{\cdot}055\, N_0) \times H\sigma_H$	0·5 $H\sigma_H$
σ_H	$\gamma H - 1{\cdot}5\,(1 + N_0)\, C$	$\gamma_H - 4C$	$\gamma_H - (8 - 0 - 4\, N_0)\, C$	γH
A	0·15 H	0·15 H	$(0{\cdot}3 - 0{\cdot}015\, N_0)\, H$	0
B	0·55 H	0·55 H	$(1{\cdot}1 - 0{\cdot}055\, N_0)\, H$	0
R	0·46 H	0·46 H	0·38 H	0·33 H

If the amount of water entering the backfill is less than sufficient to fill up all the voids but the water is being drained out, the hydrostatic pressure will be less than one-half of the full static hydrostatic pressure. And if, on the other hand, the amount of water exceeds the capacity of the backfill to drain it out, full static hydrostatic pressure will build up.

7.8.2 *Consolidation Pore Pressures*

Where impervious clay behind a wall is loaded, hydrostatic excess pore pressure may develop in the clay and exert additional pressure.

7.8.3 *Effect of Compaction*

Compaction of the backfill in the Coulomb's wedge behind the wall increases the earth pressure.

7.8.4 *Hydraulic Fills*

Account should be taken of the increase in earth pressure when the backfill is placed by hydraulic fill. The coefficients of active earth pressure for loose hydraulic fill material range from about 0·35 for clean sands to 0·50 for silty fine sands.

7.8.5 *Effect of Traffic*

If the backfill is subject to heavy traffic vibrations, reduce tan ϕ and tan δ by 20 percent.

7.8.6 *Effect of Earthquake*

For effects of seismic forces on earth pressures, IS 1893 : 1975 may be referred to.

8 PASSIVE RESISTANCE

8.1 When a retaining wall is pushed out due to active thrust of the backfill and the surcharge thereon, partial or full passive resistance will be mobilized in the front soil. In waterfront structures, the front soil will usually be dredged and there may be downward inclination. However, the depth in the fairway channel or berth should be taken for calculations, and

$$i = 0 \qquad \ldots(5)$$

should be adopted

8.2 Waterfront structures are usually vertical in the front and the passive resistance is given by:

$$P_p = \frac{1}{2}\gamma H^2 \frac{K_p}{\cos\delta} + 2\,cH\sqrt{\frac{K_p}{\cos\delta}} \qquad \ldots(6)$$

where

$$K_p = \frac{\cos^2\phi}{\left[1-\sqrt{\dfrac{\sin(\phi+\delta \sin\phi}{\cos\delta}}\right]^2} \qquad \ldots(7)$$

8.2.1 It is recommended to take generally $\delta \leqslant \phi/3$ and, in which case, equation (7) shall apply. If the wall movement outward should be more than what is essential to mobilize active earth pressure, δ may exceed $\phi/3$, equation (7) is developed with plane surface of rupture and the error for true curvilinear surface of rupture is insignificant up to about $\delta = \phi/3$. For values of δ exceeding this value, therefore, curvilinear surface of rupture should be taken and the values of K_p may be taken from tables on the subject (Tables for the calculation of passive resistance, active pressure and bearing capacity of foundation by Caquot and Kerisel).

8.2.2 In case of gravity wharf wall $\delta = +\,\phi/3$ may be adopted unless, as in case of clayey foundation, greater outward movement is expected, when

$\delta \not> \frac{3}{4}\phi$ may be taken.

8.2.3 For sheet pile wharf wall $\delta = 0$ should be taken, except when it is to be strongly held back by anchor at the top and is likely to move upward in the embedded position when appropriate negative δ should be adopted.

8.2.4 There is no likelihood of surcharge for passive resistance.

8.2.5 Front soil of wharves in which passive resistance could be mobilized, is generally submerged, and invariably so at high water.

8.2.6 For gravity wharf wall, the question of layered front soil should not arise. For a sheet pile type, it may arise and for horizontal earth surface, smooth vertical front and a two-layered front, soil a pressure diagram should be drawn (*see* Fig. 6).

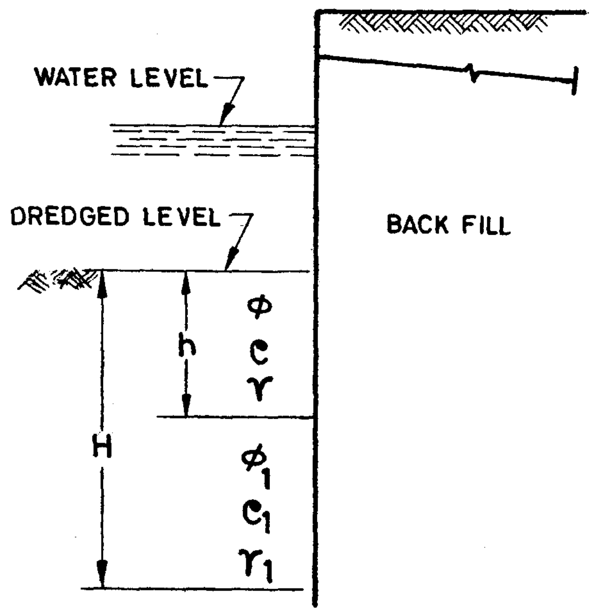


FIG. 6 SHEET PILE WHARF IN A TWO-LAYERED FRONT SOIL

$$\left.\begin{aligned} &\text{For } 0 \leqslant Z \leqslant h \quad p_p = \overline{\sigma_2} N\phi + 2\,c\sqrt{N\phi} \\ &\quad \text{where } \overline{\sigma_2} = \gamma' Z \\ &\text{For } h \leqslant Z \leqslant H \quad p_p = \overline{\sigma_2} N\phi'_1 + 2c'_1\sqrt{N\phi'_1} \\ &\quad \text{where } \overline{\sigma_2} = \gamma' h + \gamma'_1 (Z-h) \end{aligned}\right\} \quad \ldots(8)$$

8.3 In case of vertical wall with horizontal ground surface, the passive earth resistance should be computed in accordance with the procedure given in Annex B.

ANNEX A

(*Clause* 7.4)

ACTIVE EARTH PRESSURE FOR VERTICAL WALL WITH HORIZONTAL GROUND SURFACE

A-1 COHESIONLESS SOILS

A-1.1 The active earth pressure at any depth, Z, is calculated by the following formula:

$$p_A = K_A\,\gamma Z$$

The values of K_A for different values of angle of shearing resistance ϕ and angle of wall friction δ are given in Table 3.

A-1.2 The effect of wall friction on the values of K_A is not very large and may be neglected for sheet pile structures, that is, take $\delta = 0$. The error resulting from this is on the safe side.

A-2 COHESIVE ($\phi = 0$) AND MIXED (C − ϕ) SOILS

A-2.1 The active earth pressure at any depth,

Z is computed by the formula:

$$p_A = K_A \gamma Z - K_{Ac} c$$

The values of coefficients, K_A and K_{Ac} for different values of angle of shearing resistance ϕ, angle of wall friction δ and the ratio of adhesion to cohesion are given in Table 4.

A-2.2 The effect of wall friction on the values of K_A and K_{Ac} is smaller for sheet pile structures; that is, take $\delta = 0$. The error is on the safe side. The wall adhesion C_W for calculation of active pressure should be taken as being equal to C up to a maximum of 50 kN/m².

A-2.3 It is advisable to neglect any negative values of earth pressure which represent tension. However, allowance should be made for intrusion of water in the tension cracks between the wall and the soil which will produce the full hydrostatic pressure over the crack depth.

A-3 LATERAL EARTH PRESSURE DUE TO SURCHARGE LOADS

A-3.1 If the backfill carries a uniformly distributed superimposed load q, the lateral active earth pressure is given by:

$$p_A = K_A (q + \gamma z) \text{ for granular soils, and}$$

$$= K_A (q + \gamma z) - K_{Ac} c \text{ for cohesive and mixed soils.}$$

A-3.2 If the surface of the backfill carries a line load (such as a rail line) W_L per unit length parallel to the crest of the wall, the simple approximate method suggested by Terzaghi and indicated in Fig. 3 may be used. For a more accurate procedure, the wedge theory may be used.

Table 3 Values of K_V for Cohesionless Soils — Vertical Walls with Horizontal Ground Surface

(*Clause* A-1.1)

Values of δ	Values of ϕ				
	25°	30°	35°	40°	45°
0°	0·41	0·33	0·27	0·22	0·17
10°	0·37	0·31	0·25	0·20	0·16
20°	0·34	0·28	0·23	0·19	0·15
30°	—	0·26	0·21	0·17	0·14

Table 4 Values of K_A and K_{Ac} for Cohesive and Mixed Soils

(*Clause* A-2.1)

Coefficient	Values of δ	Values of C_W/C	Values of ϕ					
			0°	5°	10°	15°	20°	25°
K_A	0	All values	1·00	0·85	0·70	0·59	0·48	0·40
	ϕ		1·00	0·78	0·64	0·50	0·40	0·32
K_{Ac}	0	0	2·00	1·83	1·68	1·54	1·40	1·29
	0	1	2·83	2·60	2·38	2·16	1·96	1·76
	ϕ	½	2·45	2·10	1·82	1·55	1·32	1·15
	ϕ	1	2·83	2·47	2·13	1·85	1·59	1·41

ANNEX B

(*Clause* 8.3)

PASSIVE EARTH RESISTANCE FOR VERTICAL WALL WITH HORIZONTAL GROUND SURFACE

B-1 COHESIONLESS SOILS

B-1.1 The passive resistance at any depth, Z, is computed by the formula :

$$P_p = K_p \gamma Z$$

The values of coefficient K_p for different values of angle of shearing resistance ϕ and angle of wall friction δ are given in Table 5.

B-1.2 In calculating passive earth resistance in the design of sheet pile walls, it is usual to take $\delta = 2/3\ \phi$, except in the case of certain silty sands where a reduced value of δ should be taken ($\delta = 1/3$ to $2/3\ \phi$).

B-2 COHESIVE ($\phi = 0$) AND MIXED (C—ϕ) SOILS

B-2.1 The passive resistance at any depth Z, is computed by the formula:

$$P_p = K_p \gamma_z + K_{pc}\ C$$

The values of the coefficient K_p and K_{pc} for different values of angle of shearing resistance ϕ, angle of wall friction δ and the ratio of adhesion to cohesion are given in Table 6.

B-2.2 In calculating passive earth resistance in the design of sheet pile walls, it is usual to take $\delta = 2/3\ \phi$, except in the case of certain silty sands where a reduced value of δ should be taken ($\delta = 1/3$ to $\frac{1}{2}\ \phi$). The wall adhesion C_w should be taken as being equal to c up to a maximum of 50 kN/m².

Table 5 Values of K for Cohesionless Soils — Vertical Walls with Horizontal Ground Surface

(*Clause* B-1.1)

Values of δ	Values of ϕ			
	25°	30°	35°	40°
0°	2·5	3·0	3·7	4·6
10°	3·1	4·0	4·8	6·5
20°	3·7	4·9	6·0	8 8
30°	—	5·8	7·3	11·4

Table 6 Values of K_p and K_{pc} for Cohesive and Mixed Soils

(*Clause* B-2.1)

Coefficient	Values of δ	Values of C_w/C	Values of ϕ					
			0°	5°	10°	15°	20°	25°
K_p	0	All	1·0	1·2	1·4	1·7	2·2	2·5
	ϕ	Values	1·0	1·3	1·6	2·2	2·9	3·9
K_{pc}	0	0	2·0	2·2	2·4	2·6	2·8	3·1
	0	½	2·0	2·6	2·9	3·2	3·5	3·8
	0	1·0	2·6	2·9	3·2	3·6	4·0	4·4
	ϕ	½	2·4	2·8	3·3	3·8	4·5	5·5
	ϕ	1·0	2·6	2·9	3·4	3·9	4·7	5·7

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Amendments Issued Since Publication

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Eastern : 1/14 C. I. T. Scheme VII M, V. I. P. Road, Maniktola CALCUTTA 700054	37 86 62
Northern : SCO 445-446, Sector 35-C, CHANDIGARH 160036	2 18 43
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Printed at New India Printing Press, Khurja, India