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NUMERICAL METHOD FOR DYNAMIC ACTIVE EARTH PRESSURE DISTRIBUTION

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SUMMARY

For aseismic design of retaining structures knowledge of earth pressures is needed. This problem is statically indeterminate. The earth forces obtained by Coulomb's and Mononobe-Okabe theories agree fairly with test data and hence form basis of proposed method. These theories neglect moment equilibrium and fail to give pressure distribution. To overcome this, proposed numerical method considers equilibrium of discrete elements of rupture wedge to obtain pressure along rupture surface and wall back using moment equilibrium.

INTRODUCTION

Knowledge of earth pressures is needed for aseismic design of retaining walls of road and railway embankments, bridge abutments, quay walls etc. Research on the topic dates back to 1776 when Coulomb presented classical earth pressure theory and that for dynamic case dates back to 1926 when Mononobe-Okabe theory was presented. These theories predict earth forces in good agreement with test data for active case (3, 4, 5). As such this forms a reasonable basis for the proposed study.

These theories neglect moment equilibrium and fail to predict pressures. To overcome this, they recommend linear variation of pressure with depth. This is not always true for static case. Dynamic tests indicate nonlinear pressure variation with resultant acting with in middle third height (3, 5). Pressures by Basavanna's method are in error due to erroneous assumption that vertical stress at a point within rupture wedge is same as that due to soil column there. In fact, each column standing on rupture surface receives some weight from adjacent column on fill side and transfers some weight to adjacent column on wall side.

Force equilibrium conditions suffice to obtain magnitudes of soil reaction and earth force. Remaining moment equilibrium is not adequate to evaluate their points of action. Hence, the system is indeterminate. The proposed numerical method overcomes this by considering equilibrium of descrete elements of rupture wedge of cohesionless fill to obtain pressures along rupture surface and wall with due regard to moment equilibrium. It is impossible to present all details of studies carried out for want of space. To highlight merits, some results are presented in terms of dimensionless factors to make them independent of units of measurements.

PROPOSED METHOD

It assumes rigid wall of known wall friction retaining cohesionless dry uniform fill with plane surface carrying no loads. It is further assumed that wall movement is adequate to realize active case, rupture surface is planer and rupture wedge is rigid. Figure 1 shows soil-wall system, discretization of wedge and force diagram which are easy to understand. *For wall leaning away from fill equilibrium of wedge ADB gives* angle ηdb for force on face DB. Rankine's theory of infinite slopes indicates that force on vertical plane at C acts parallel to A C. Angle ηkl on KL is given by :

$$\tan(\eta kl) = \tan(\beta) + [\tan(\eta db) - \tan(\beta)] \cdot (CL/CB)^{z} \tag{1}$$

Variation of power z from 1 to 3 leads to less than 4% variation in lever arm of earth force, Ea, about B. Hence, z is not a significant parameter and its value in this study is assumed to be unity.

Force equilibrium of each element is considered to obtain reaction on base along rupture surface. From this, pressures along BC can be obtained by assuming pressure at C to be zero and pressures to vary linerarly along base of each element. This is reasonable if number of elements considered is adequately large. Error in soil reaction and its moment about B using 10 and 20 elements with respect to those for 30 elements is less than 0.007% which is negligible. As such, 10 elements are considered adequate. Knowing pressures along BC, point of action of soil reaction can be obtained. This is used in considering mement equilibrium of entire wedge to obtain point of action of earth force. Earth pressure px at a depth x below wall top is assumed as :

$$Px = \gamma \cdot Ca \cdot x^{N} \tag{2}$$

Earth pressure constant Ca and power factor N are evaluated knowing magnitude and point of action of earth force. Dimensionless factors used in this presentation are listed in Table 1. Definitions of Cfah, Cprah and Cmoah are similar to those for Cfa, Cpra and Cmoa except that instead of px its horizontal component is used. Increasing Cfa and Cma indicate increasing earth force and overturning moment. Increasing Cha indicates upward movement of point of action of earth force above B which in turn indicates increasing pressures near top and decreasing pressures near bottom end of the wall.

Table 1 Dimensionless Factors

Factor	Symbol	Definition
Depth factor	Cda	x/H
Pressure factor	Cpra	$px/(\gamma.H)$
Point of action factor	Cha	Hp/H
Earth force factor	Cfa	$2\ Ea/(\gamma.H^2)$
Moment factor	Cma	$6Hp.Ea/(\gamma.H^3)$
Mobilization factor	Cmoa	$px/(\gamma.x)$

RESULTS OF PARAMETRIC STUDIES

Effect of Seismic Coefficients : Horizontal seismic coefficient, αh, is the most significant parameter. Figure 2 shows increasing pressures near top end and decreasing pressures near bottom end as αh increases which is in agreement with observations based on test data reported by Ishii et al (3) and by Matsuo and Ohara (5). Increasing top width of wedge with increasing αh and the resulting larger inertia forces near the top also support this observation. This is further supported by larger mobilization factor, Cmoa, as indicated in Fig. 3 for larger values of αh. This highlights the need for strengthening the top portion of wall for better earthquake resistance.

Vertical seismic coefficient, α_v, alters effective unit weight of fill and does not affect Cha when α_h is zero. As cited in Fig. 4, this is so for proposed method and for Basavanna's method. As α_h increases, changes in α_v show stronger influence of Cha which is expected. The figure also shows Cha obtained as per Indian Standard Code of Practice (IS: 1893-1984) which shows largest influence of α_v when α_h is zero and decreasing influence of α_v as α_h increases which is not correct. Besides, for any α_h, as α_v decreases, Cha as per IS Code decreases which is opposite of that indicated by proposed method and by Basavanna's method. All this suggests that recommendations of this code of practice for dynamic pressures are not based on sound engineering principles and needs proper rivision.

Effect of Wall Frection : Figure 5 shows that as angle of wall friction, δ, increases nonlinearity of pressure variation increases with pressure near top end reducing and those near the bottom and increasing for static case. This agrees with observations of Terzaghi (6). For dynamic case also pressure at any point decreases with increasing δ which is in order as δ represents resisting forces. For smooth wall, static pressures vary linearly which is expected.

Effect of Angle of Shearing Resistance of Fill : Angle of shearing resistance, ϕ, represents resisting forces. As such, as shown in the Fig. 6, increasing ϕ improves wall performance by decreasing Cha, Cma and Cfah particularly for higher values of α_h. This highlights the need for improving the fill properties by proper compaction to improve the performance of the wall during earthquakes.

Effect of Angle of Surcharge : Surcharge of fill is additional source of destabilizing forces. As shown in Fig. 7, Cha, Cma and Cfah increase with increasing angle of surcharge, B, which is in order. This also indicates increasing pressures near top end with increasing β. Therefore, level fills or fills with gentle surcharge slopes are preferable under seismic conditions.

Effect of Angle of Wall Back : From Fig. 8 it may be noted that as angle of wall back, α increases Cfah, Cma and Cha decrease. This is most pronounced for Cma and at higher values of α_h. Increase in Cha with increasing α indicates relatively sharper increase in pressure in upper half compared to thos in lower half of the wall. So, vertical wall backs are better than wall backs leaning away from the fill.

Earth Pressure Constant Ca and Power Factor N : The constant Ca decreases linearly with increasing δ and α whereas it decreases nonlinearly with increasing ϕ. Values of N increase nonlinearly with increasing δ and α whereas they may increase or decrease with increasing ϕ. The effect of δ and α are shown in Fig. 9 and Fig. 10 respectively. Such charts are useful in obtaining pressure distributions without performing the proposed numerical analysis which will save cost and time of design and analysis.

CONCLUSIONS

Earth pressure problem is statically indeterminate. Theory of Coulomb for static case and that of Mononobe-Okabe for dynamic case neglect moment equilibrium and fail to give pressure distribution. They predict earth forces which fairly agree with test data. As such, failure wedges predicted by them have been adopted for the proposed numerical method. The method devides rupture wedge into smaller elements and examines equilibrium of elements to obtain pressures along rupture plane. This together with moment equilibrium of wedge is used to obtain point of action of earth force and hence pressures along wall back using Eqn. 2.

The proposed method satisfies all the three equilibrium conditions. Observations from predicted results are in good agreement with those based on experimental data. Presentation of results in terms of dimensionless factors makes them independent of the size of the problem and units of measurements which is desirable. Results also highlight drawbacks of IS Code of Practice for dynamic earth pressure distribuion.

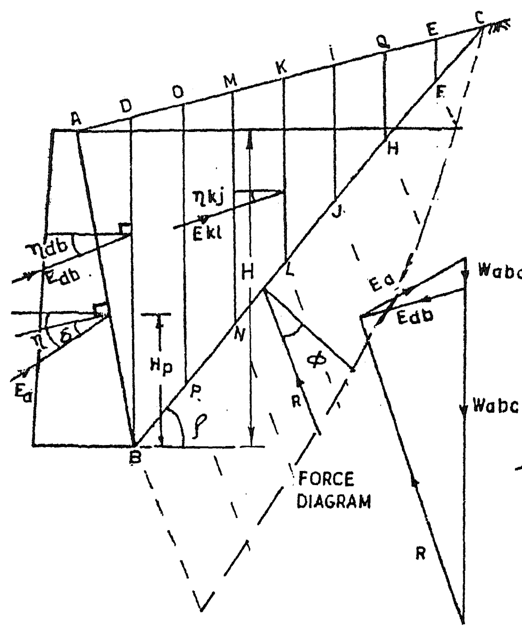


Fig. 1 Soil-Wall System

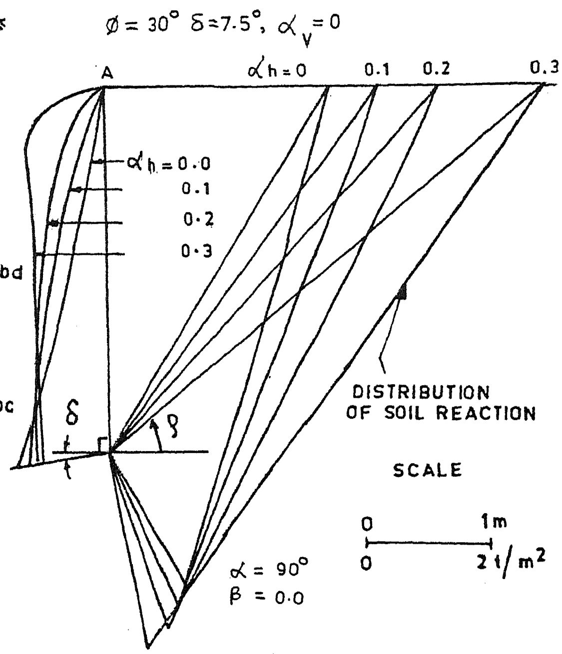


Fig. 2 Distribution of Earth Pressure and *Soil Reaction*

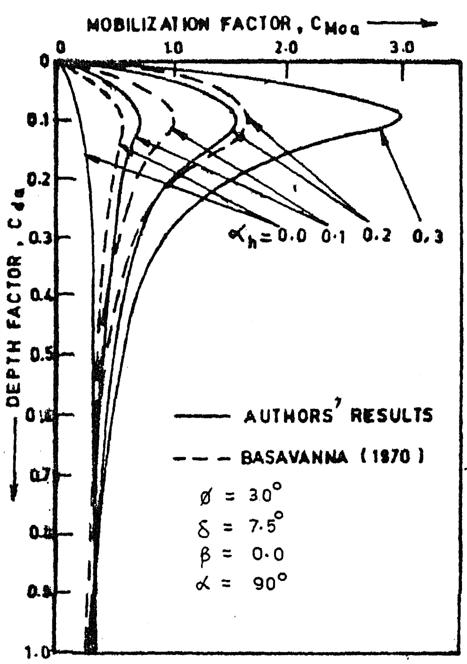


Fig. 3 Variation of Mobilization Factor with Depth Factor.

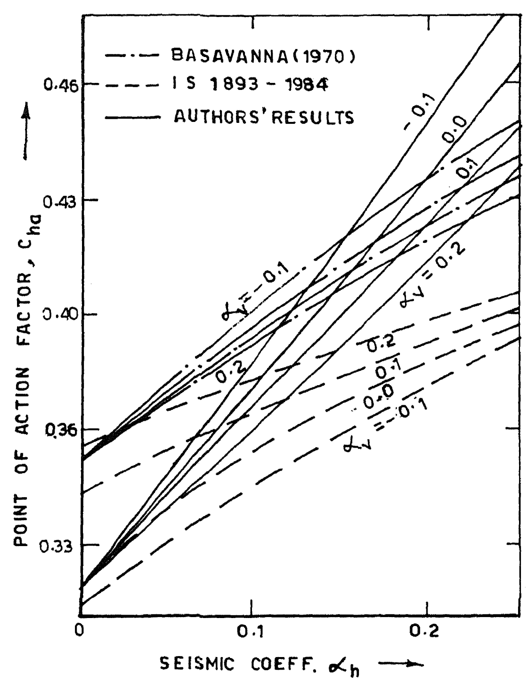


Fig. 4 Variation of Cha with Seismic Coefficients.

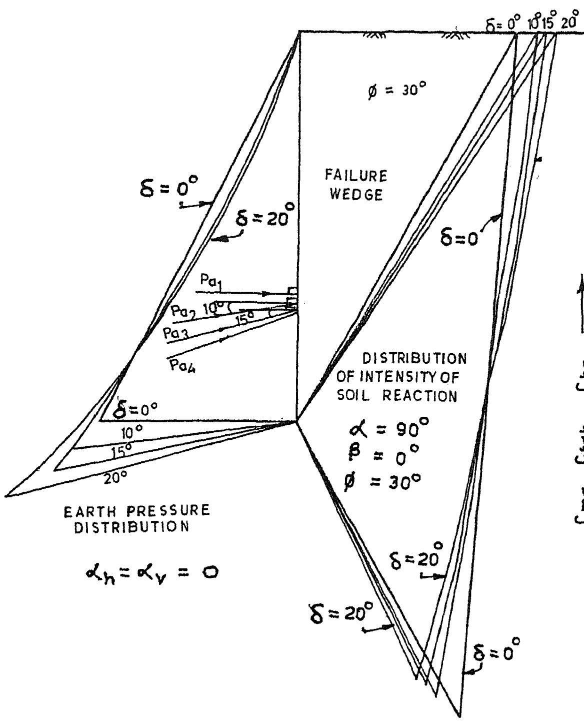


Fig. 5 Effect of Wall Friction on Distribution of Earth Pressure and Soil Reaction

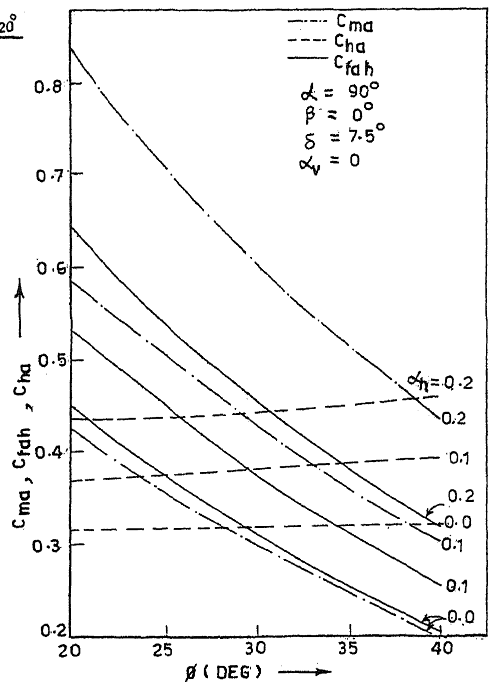


Fig. 6 Variation of Cha, Cma and Cfah with Angle of Shearing Resistance, ø

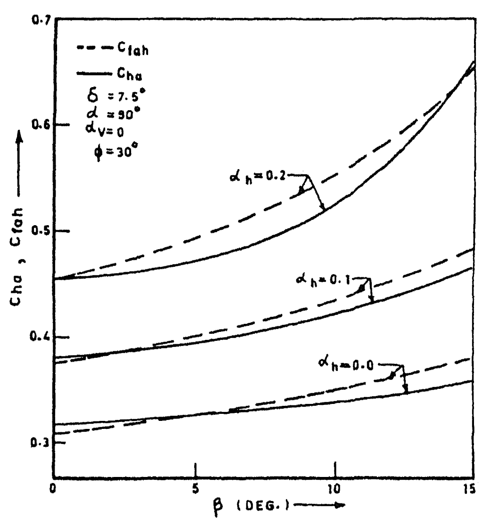


Fig. 7 Variation of Cfah and Cha with Angle of Surcharge, β

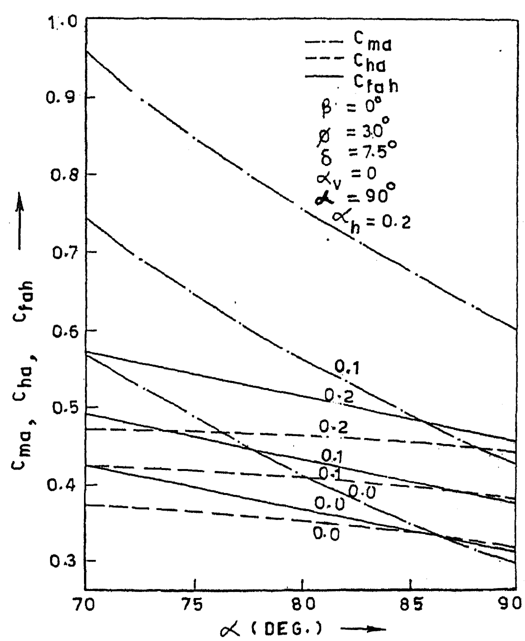
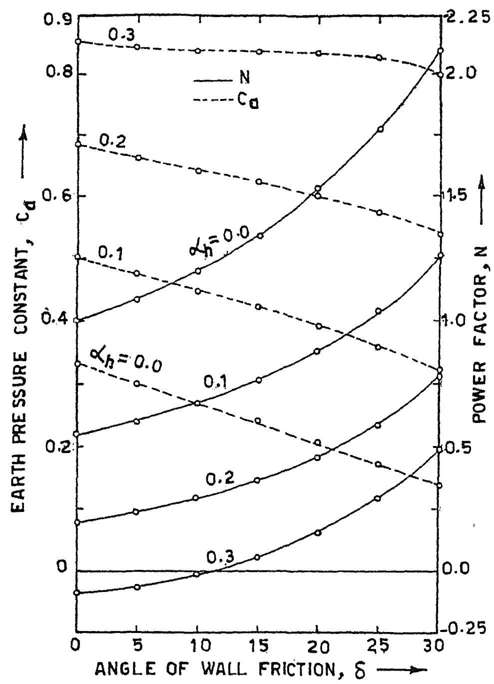
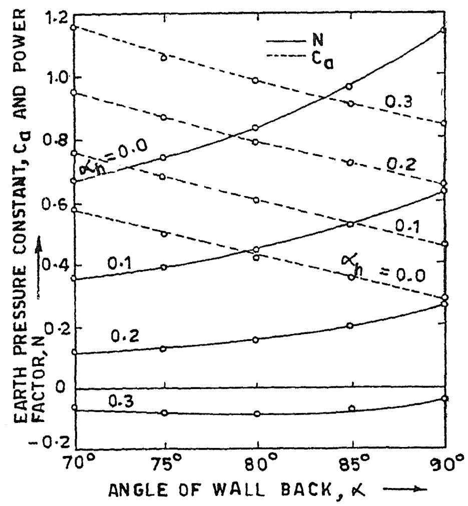


Fig. 8 Variation of Cma, Cha and Cfah with Angle of Wall Back, α



$\beta = 0, \ \phi = 30°$

Fig. 9 Variation of Ca and N with Angle of Wall Friction, δ



$\beta = 0, \ \phi = 30°$

Fig. 10 Variation of Ca and N with Angle of Wall Back, α

Based on predicted results is concluded that earth pressure vary nonlinearly with depth especially for higher values of seismic coefficients. Pressure mobilization near top end warrants strengthening of wall near top end. Near vertical walls fills with nearly level surfaces and rough walls with as dense a fill as possible are also desirable.

The computer programme for the proposed analysis is small with only 500 statements and is very economical to run.

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