28 Apr 1981, 2:00 pm - 5:00 pm

A Simple Method of Estimating Seismic Pressures from Cohesive Soils Against Basement Walls

P. W. Taylor
University of Auckland, New Zealand

Z. Indrawan
University of Auckland/ The Christian University of Indonesia

Proceedings: First International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, April 26 - May 3, 1981, St. Louis, Missouri

A Simple Method of Estimating Seismic Pressures from Cohesive Soils Against Basement Walls

P. W. Taylor
Professor of Civil Engineering, University of Auckland, New Zealand

Z. Indrawan
Research Student, University of Auckland, on leave from The Christian University of Indonesia

SYNOPSIS Although this problem arises frequently in design practice, there is little guidance for the designer in current literature. The proposed method entails estimating the free-field soil deformation caused by a horizontal acceleration. Dynamic increase in earth pressure against an (effectively rigid) wall is assumed to be proportional to the free-field deformation, relative to the base of the wall, with an upper limit equal to full passive pressure. Dynamic pressures calculated using this method are compared with field evidence from published records of observations made on a building in Yokohama during an earthquake.

INTRODUCTION

In his State-of-the-Art paper, Prakash (1977) noted that "information regarding dynamic passive pressure is quite limited". This is certainly true, and the information that is available on dynamic pressures is almost exclusively related to cohesionless backfills. Moreover, much of the published research has been concerned with independent retaining walls which can be displaced to some extent (by rotation or translation) whereas the deformations of a wall which forms the basement of a large building are strictly limited. Thus the designer, confronted with the problem of estimating seismic pressures from cohesive soils against basement walls has very little guidance. The ATC publication 'Tentative Provisions for the Development of Seismic Regulations for Buildings' (1978) expresses this clearly: "It is left for the foundation engineer to determine the design lateral pressure under dynamic loading."

PASSIVE PRESSURE

The maximum pressure that could occur, with any given wall/soil movement is the full passive pressure. Considering the case of a saturated cohesive soil ($\phi = 0$ for total stress analysis) and a level ground surface without surcharge, the passive stress may be estimated (neglecting wall adhesion) from:

$$p_p = \gamma z + 2c_u$$

where γ is the soil density
z is the depth below the surface, and
c_u is the soil cohesion.

Passive pressures on areas narrow in width, compared to their depth, such as pile faces, can attain higher values, (over $9c_u$) but this effect will be insignificant for basement walls.

A literature search failed to reveal any laboratory tests on passive pressure measurements taken to failure with cohesive soils. Carder, Murray and Krawczyk (1980) describe a test on a 1 m high wall against a compacted silty clay, but this was discontinued before failure.

To design for the full passive pressure would certainly be safe, but it would also be uneconomic, particularly for firm clays. Experience has shown that dynamic pressures on basement walls are greater for soft clays than for firm clays (ATC) which indicates that, for the firmer soils, passive pressure is not usually attained.

SOIL-WALL DEFORMATION

It is well-known that (considering the soil to be stationary) the wall deformation required to attain passive pressure is considerably greater than that to attain actual pressure. Moreover, the vertical distribution of pressure is strongly dependent on whether the movement of the wall (assumed rigid) is by translation, by rotation about the top edge, or by rotation about the base. James and Bransby (1971) using a velocity field method for sands, confirmed by experimental results, found a stress distribution of the form shown in Figure 1(a) for rotation of the wall about the top.

The same authors give expected distributions for rotation about the base and for translation as shown in Figures 1(b) and (c). Applying a non-linear finite element approach, and using drained triaxial test results for a normally consolidated clay, Yudhbir and Varadarajan (1974) derived passive pressure distributions for rotational movements similar to 1(a) and (b). The roughly parabolic distribution of 1(b) is confirmed by field measurements of earth pressures on bridge abutments (Broms and Ingleson, 1971) and on lock walls (Smoltczyk et al., 1971) resulting predominantly from

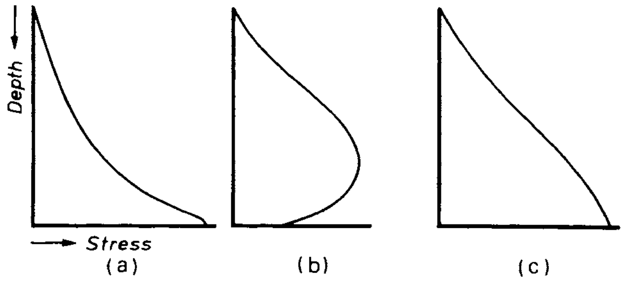

Figure 1. Passive Pressure Distributions
(a) Rotation about the top
(b) Rotation about the base
(c) Translation

seasonal rotation about the base. A theoretical investigation of dynamic passive earth pressures by Ghahramani and Clemence (1980) also leads to the distributions shown in Figure 1.

Rowe and Peaker (1965) found that the horizontal translation required to attain maximum passive thrust, in laboratory tests on walls 0.46 m high ranged from about 4% of wall height for dense sand, to over 20% for loose sand.

For the case of wall rotation about the base, Figure 2(a) (reproduced from James and Bransby 1971) shows, for four different tests, the rotation required to attain peak normal stress, as a function of the depth ratio d/H (where d is depth below the surface and H the total depth). When replotted as displacement ratio δ/H against depth ratio, as in Figure 2(b) it is seen that maximum passive pressures (when $\delta = \delta_c$) are attained at an approximately constant ratio $\delta_c = 0.043$ H (for Leighton Buzzard sand at void ratio of 0.5).

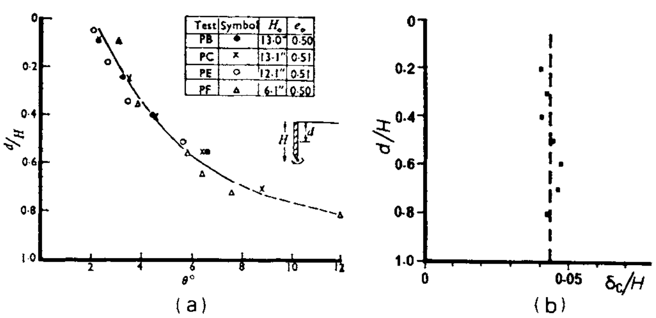

Figure 2. (a) Rotation to attain peak stress (James and Bransby)
(b) Displacement to attain peak stress

While the investigations reviewed above are related principally to sands, the concept of a 'critical displacement', required to attain full passive pressure, is utilised in the method proposed.

PROPOSED METHOD

The method is restricted to cohesive ($\phi = 0$) soils and, as it is a dynamic problem, with no opportunity for dissipation of pore pressures, analysis is in terms of total stress.

For the design horizontal acceleration, (assumed constant with depth) the horizontal deformations of the soils above the base of the wall are estimated. Seed and Idriss (1971) have shown that because of dynamic effects, there is a reduction of horizontal acceleration with depth. At 10 m depth, for example, the effective value is only 86-96% of that at the surface, but this reduction is ignored here. Ideally, the shear modulus (G) of the soils would be determined from dynamic tests. Such test results are seldom available from routine investigations. Approximate values of shear modulus may however be estimated from the undrained cohesion (c_u) by assuming a suitable value for the ratio G/c_u.

The deformations calculated are the "free-field" deformations, that is, those that would occur at a location remote from any obstruction. The basement of a building is usually a box-like structure, very rigid in comparison with the soils surrounding it. The base of the wall must move with the soils at that level, so that relative deformation is zero.

The assumption is made that the seismic pressures are dependent on the relative deformation between the soil and the wall. At base level there is no relative deformation so the soil pressure should not increase above its static (at rest) value. In this respect the seismic deformations bear some similarity to the case of a rigid wall being rotated about its base.

Above this level, there will be an increase in earth pressure. It is assumed that the earth pressure attains its full passive value when the relative deformation δ equals or exceeds the critical value, δ_c. The increase is taken to be proportional to δ for values below δ_c. Thus, at any particular depth,

if p_p is the passive pressure
p_o is the static pressure

then the dynamic pressure increase is given by

$$p_d = \frac{\delta}{\delta_c} (p_p - p_o) \qquad (\delta/\delta_c \leqq 1)$$

FIELD OBSERVATIONS

Fortunately, seismic pressures on the basement walls of a building in Yokohama have been observed (Yuukou Ikuta et al., 1979). This has enabled a comparison to be made between the earth pressures calculated, as described above, with observed values and provided a basis on which to assign values of G/c_u and δ_c.

The Yokohama Tenri Building has two basement floors and 27 stories above ground level. The foundations comprise cast-in-place piles supporting the central core and basement walls extended to 26-28 m depth forming a continuous piling wall supporting the perimeter. The authors (Ikuta et al.) had instrumented the perimeter basement wall to enable earth pressure (at 7 points) and water pressures (at 4 points) to be measured. Triggered at an acceleration of 0.02 g, the records from all instruments were recorded throughout the earthquake of 12 June 1978 (magnitude 7.4, epicentral distance 380 km) which caused a maximum acceleration of 0.125 g at the site. The earth pressure meters record total stress against the wall. During the earthquake, dynamic pressures up to 37% of those under static conditions were observed. The authors note that both dynamic pressures and the ratio of dynamic/static pressure tended to be larger near the ground surface.

The soil profile is predominantly silt, with unconfined compression strength increasing with depth from 25 kPa near the surface, to 92 kPa at 21 m. Below that are layers of fine sand and clay. Sufficient information is given to enable the proposed method of estimating dynamic pressure to be applied. Assuming $G/c_u = 400$, the deformation caused by a horizontal acceleration of 0.125 g was determined by dividing the soil profile into a number of layers, assumed to be of uniform properties. As shown in Figure 3, the total deformation, over 26 m depth is about 0.05 m. Static pressures observed on the South side were consistent with a submerged density of 5.1 kN/m^3, water table at 4 m depth and a coefficient of earth pressure at rest, $K_o = 0.58$. Static pressures (on both South and East sides) were determined on this basis and are shown, together with the passive pressures (taking soil density as 15 kN/m^3). Taking $\delta_c = 0.025$ H, where H is the total wall height, gives good correlation between observed and calculated values, particularly on the South side, where the dynamic pressure distribution is as expected (Table I). On the East side, the observed distribution is somewhat anomalous. There also, observed static pressures were lower than used in the calculation. If, however, the excess pressure is determined from the observed static pressure, the comparison ratios are improved (0.72; 1.40; 1.04).

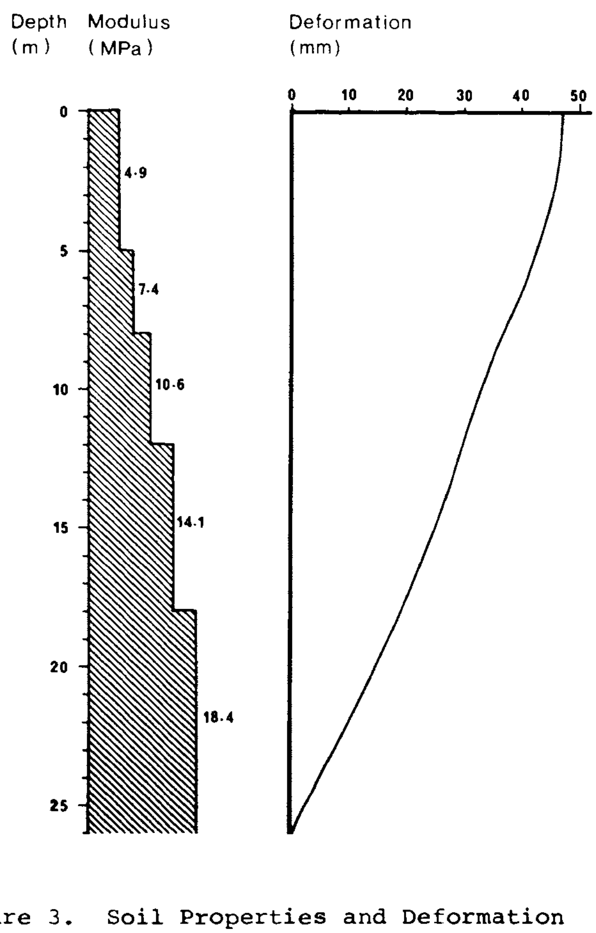

Figure 3. Soil Properties and Deformation

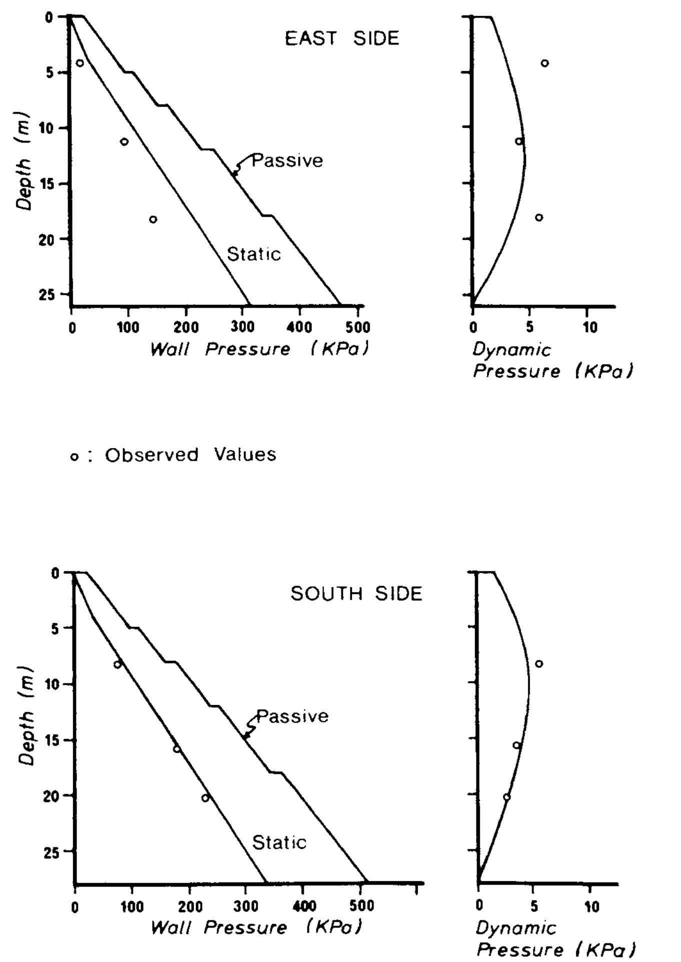

Figure 4. Calculated and Observed Pressures

It will be seen that the dynamic pressure increase for this 0.125 g earthquake was nowhere greater than 7% of the maximum possible. The distribution of dynamic pressure, tending to be greater near the surface, is in accord with the theory outlined.

TABLE I. Earth Pressures - Yokohama Tenri Building

Location	Depth (m)	Recorded Values: Static Pressure (kPa)	Recorded Values: Dynamic Pressure (kPa)	Calculated Pressures: Passive Pressure (kPa)	Calculated Pressures: Excess Pressure (kPa)	Calculated Pressures: Dynamic Pressure (kPa)	Comparison: Ratio: Calculated/Observed
East Side H = 26 m	4.2	17.1	6.4	87.5	50.4	3.3	0.52
	11.2	105.0	4.2	220.9	94.6	4.8	1.14
	18.2	145.5	5.9	365.1	149.5	4.1	0.70
South Side H = 28 m	8.3	72.1	5.7	177.4	72.3	4.3	0.76
	15.8	176.2	3.5	307.6	122.6	3.9	1.13
	20.3	246.1	2.7	405.6	154.3	2.7	0.99

DESIGN RECOMMENDATION

It is considered that the correlation, for the Yokohama Tenri Building, is sufficiently close to justify the use of the method in practice. In view of the uncertainties entailed, it is recommended that for design a reduced value of $\delta_c = 0.01$ H be used. This results in higher, and therefore more conservative design pressures. The method has been applied to a multi-storey building in Auckland, New Zealand.

FURTHER DEVELOPMENT

More accurate estimation of the free-field deformations may be obtained from a computer analysis of site response, preferably by non-linear methods (Taylor and Larkin, 1978). Instead of the linear relationship between dynamic pressure and relative deformation between soil and wall, a nonlinear relationship would be more realistic.

SUMMARY

The method outlined, for the estimation of seismic pressures from cohesive soils, is considered to be suitable for routine design practice. It is simple to apply and requires no additional data beyond that normally available from routine investigations. As further field evidence becomes available, refinements in the method will undoubtedly be made.

REFERENCES

ATC (Applied Technology Council)(1978), "Tentative Provisions for the Development of Seismic Regulations for Buildings" ATC 3-06

Broms, B.B. and I. Ingelson (1971), "Earth Pressure against the Abutments of a Rigid Frame Bridge" Geotechnique, Vol. 21, No. 1, pp 15-28

Carder, D.R., R.T. Murray and J.V. Krawczyk (1980), "Earth Pressures against an Experimental Retaining Wall Backfilled with Silty Clay" Transportation and Road Research Laboratory, TRRL Lab. Report No. 946

Ghahramani, A. and S.P. Clemence (1980), "Zero Extension Line Theory of Dynamic Passive Pressure", Proc. ASCE, Vol. 106, No. GT6, pp 631-645

Ikuta, Y., M. Maruoka, T. Mitoma and M. Naganou (1979), "Record of Lateral Pressure taken during Earthquake" Soils and Foundations, Vol. 19, No. 4, pp 85-92

James, R.G. and P.L. Bransby (1971), "A Velocity Field for some Passive Earth Pressure Problems", Geotechnique, Vol. 21, No. 1, pp 61-83

Prakash, S. (1977), "Seismic Response of Soil Deposits, Embankments, Dams and Structures", in "Soil Dynamics and its Application to Foundation Engineering"(Yoshimi et al.), Proc. 9th ICSMFE, Tokyo, Vol. 2, pp 605-650

Rowe, P.W. and K. Peaker (1965), "Passive Earth Pressure Measurements", Geotechnique, Vol. 15, No. 1, pp 57-78

Seed, H.B. and I.M. Idriss (1971), "Simplified Procedure for Evaluating Soil Liquefaction Potential", Proc. ASCE, Vol. 97, No. SM9, pp 1249-1273

Smoltczyk, U., K. Hilmer, E. Franke and B. Schuppener (1977), "Earth Pressure Variations due to Temperature Change", Proc. 9th ICSMFE, Tokyo, Vol. 2, pp 725-733

Taylor, P.W. and T.J. Larkin (1978), "Seismic Response of Nonlinear Soil Media", Proc. ASCE, Vol. 105, No. GT3, pp 369-383

Yudhbir and A. Varadarajan (1974), "Lateral Earth Pressure Analysis using Relevant Soil Modulus", Soils and Foundations, Vol. 14, No. 2, pp 89-95