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VOLUME 1
Issue 1 (G)

THE NEW ZEALAND GEOMECHANICS SOCIETY

PROCEEDINGS OF A WORKSHOP

ON

LATERAL EARTH PRESSURES AND RETAINING WALL DESIGN

held during the N.Z.I.E. Conference in Wellington, 1974

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Proceedings of a workshop on lateral earth pressures and retaining wall design, held during the N.Z.I.E. Conference, February 1974

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INTRODUCTION

The participation of the N.Z. Geomechanics Society at the 1974 N.Z.I.E. Conference in Wellington took the form of a workshop session on Lateral Earth Pressure and Retaining Wall Design. A workshop session consists of the presentation of papers together with rather informal discussions on a particular topic. The Proceedings record the papers presented and the discussion which took place, under the chairmanship of Mr. J.H.H. Galloway, Chairman of the N.Z. Geomechanics Society. The venue was Victoria University of Wellington.

Proceedings Editors : J.P. Blakeley
I.M. Parton

A REVIEW OF STATIC EARTH PRESSURE THEORY

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Synopsis

A broad review is given of the classical theories of earth pressure and some of the difficulties in applying the theories to practical problems.

1. INTRODUCTION

This paper reviews the classical theories for determining earth pressure and conditions under which the theories can rightly be applied. The paper then goes on to discuss some difficulties involved in applying the theories to practical problems and some of the consequent approximations which must be made. No attempt is made to set out in any detail the theories for determining the static earth pressure acting on a retaining structure. This can be found in any soil mechanics text book.

2. OUTLINE OF THE COULOMB AND RANKINE THEORIES

The two classical theories of earth pressure in common use today were developed by Charles Augustus Coulomb (1736-1806) about 1776 and W.J.M. Rankine (1820-1872) about 1857. There has been very little significant contribution made since this time.

The Coulomb theory was the first of the "general wedge theories" and the Rankine theory the first of the "stress type theories". In a general wedge theory the shear strength is only developed along a single rupture surface at the base of the failure wedge (which acts as a rigid body) and the combination of the forces acting on the wedge (including wall friction) determines the pressure applied to the wall adjacent. Experimental evidence suggests only a small movement is needed along the rupture surface to develop the assumed forces.

In a stress type theory the shear strength must be developed throughout the whole soil mass which is thus in a state of plastic equilibrium. Sufficient lateral movement in the soil mass must be possible to develop this state. Gravity is the only force assumed to be acting on individual soil particles within a soil mass. Obviously by taking the sum of forces at a number of points on a straight line within the soil mass, the total force acting on the line can be determined. The problem is that the soil mass is assumed to be "semi-infinite" and thus the development of forces within the mass must not be influenced by the presence of a wall.

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Gravity retaining walls (including crib walls) cannot generally meet this condition. In practical problems it can only be achieved for a cantilever wall, assuming the earth pressure to act on a "virtual back" through the heel of the retaining wall, and therefore that the earth above the base of the wall acts as part of the wall. With these assumptions wall friction acting on the back of the wall need not be considered.

The Coulomb theory assumes a straight line rupture surface on the base of the wedge. This does a slight injustice to the laws of statics. However in the active case, the error is only of the order of 2%. Coulomb carried out a series of experiments to try and verify his theory which was developed only for active pressure. He did not attempt to extend the theory to passive pressure (which was done by others at a later date using the same theory).

In the case of passive pressures the assumption of a straight line rupture surface leads to a considerably greater error than 2%. However, as discussed later in this paper the magnitude of the error is still likely to be small compared with error involved in predicting the correct shear strength parameters to be used in the analysis.

In summary, the Coulomb theory is an approximate theory only as it is assumed that the failure wedges have a straight rupture surface; whereas the Rankine theory is an exact method providing that the assumed conditions can be met. However it can be said that the Coulomb method can more generally be applied and hence is rather more useful.

Neither method will give a true picture of the actual forces on a retaining wall unless the failure stress state is achieved in the soil mass. This requires a state of plastic equilibrium in the soil mass in the case of Rankine, or the development of full shear strength along a rupture surface in the soil in the case of Coulomb.

3. USING THE COULOMB AND RANKINE THEORIES

3.1 Mathematical Formulae

For a cohesionless granular backfill with a level surface or uniformly sloping surface and no surcharge loads, mathematical formulae have been developed for determining earth pressure without the need to resort to the use of time consuming trial wedges and graphical solutions using a polygon of forces. The Rankine formula can also handle a uniformly distributed surcharge load on top of the fill but for level backfill surfaces only. The Rankine formula can also be applied to a cohesive soil but again for level backfill surfaces only.

However these formulae cannot handle:

(i) An irregular (non uniform slope) backfill surface profile

(ii) Surcharge loads on backfill (except for uniformly distributed surcharge using the Rankine formula).

(iii) Line loads on the backfill surface.

(iv) Cohesive soils (except for the Rankine formula with a level backfill surface only).

Where these factors are involved then the trial wedge method must be employed as discussed below.

3.2 Trial Wedge Methods

As outlined in Section 3.1 above, where the backfill surface profile is irregular, surcharge loads or line loads are applied, or the soil is cohesive, then the method of trial wedges is generally the only method available to determine earth pressure acting on a retaining wall. A variation of the trial wedge method is the Culmann graphical solution which by rotating the direction of the forces acting on the wedge through a constant angle enables the polygon of force diagram to be superimposed on the trial wedge diagram.

The trial wedge method can be employed using either the Rankine or the Coulomb assumptions. The use of Coulomb assumptions is readily understandable as the Coulomb theory is a general wedge theory. However the use of Rankine assumptions in the trial wedge method, although commonly done, is more questionable since the Rankine theory is a stress type theory (as discussed in Section 1). The decision whether to use Rankine or Coulomb assumptions in the trial wedge method is generally basedon whether the Rankine or Coulomb conditions can more nearly be satisfied by the wall (as discussed in Section 3.3 below).

In summary the main differences between using the Coulomb and Rankine assumptions in the trial wedge method are:

(i) For Rankine the resultant pressure on the wall acts parallel to the slope of the backfill surface. In the case of a non uniform backfill surface the average slope is taken over a distance equal to twice the height of the wall.

On the other hand for Coulomb, friction on the back of the wall is assumed to be the main factor governing the direction in which the resultant pressure on the wall acts. The result pressure is taken to act at an angle δ to a normal through the back surface of the wall. The angle δ is the angle of wall friction, usually assumed to equal 2/3 ϕ but its true value is rather uncertain.

(ii) The weights of the trial wedges used in the two methods are slightly different. In the case of Coulomb, the wedges are bounded by the backfill surface, the back of the wall and the trial rupture surface. In the case of Rankine, instead of being bounded by the back of the wall, the trial wedges are bounded by a vertical line through the heel of the wall ("virtual back").

(iii) When using Coulomb, an additional force called wall adhesion should be assumed to act parallel to the back of the wall. This is usually taken to be 2/3 of the soil cohesion times the area of the back of the wall.

3.3 Choice of Rankine or Coulomb

In choosing between the Rankine and Coulomb assumptions the following factors should be considered:

(a) For the Coulomb theory to be applied, the back of the wall must be plane or so nearly plane that it can be assumed to be so.

(b) For the Rankine theory to be applied, the wall must not interfere with the outer plane of rupture (which acts at an angle to the vertical as defined on Fig. 1) and also the soil in the wedge bde on Fig. 1 must not slide on the back of the wall but must move with the wall. With reference to Fig. 1, this condition is satisfied if R, the resultant of Pa and Wabe mades an angle ω with the normal to the back of the wall such that $\omega < \delta$ where δ is the angle of wall friction.

In Summary

(i) If the back of the wall is plane, either Rankine or Coulomb can be used depending on:

- if the outer plane of rupture cannot form without interference from the wall, Coulomb conditions should be used
- if the outer plane of rupture can form without interference from the wall, then Rankine conditions apply if $\omega < \delta$ and Coulomb conditions if $\omega > \delta$.

(ii) If the back of the wall cannot be assumed to be a plane surface, then Coulomb conditions cannot apply but Rankine conditions can be used if the outer plane of rupture can form without being obstructed by the wall.

(iii) In general solid gravity retaining walls of usual proportions satisfy Coulomb conditions, especially if it is assumed that $\delta = 2/3\ \phi$.

Cantilever and counterfort retaining walls do not generally satisfy the conditions of either theory, but walls of usual proportions can be assumed to satisfy Rankine conditions without introducing serious errors in the computed pressure on the back of the wall.

(iv) The Rankine conditions are often assumed for all walls under 25ft high, as they are simpler and generally conservative.

(v) In general, the Coulomb and Rankine conditions lead to only a 10-15% difference in the calculated earth pressure force but a very different line of action.

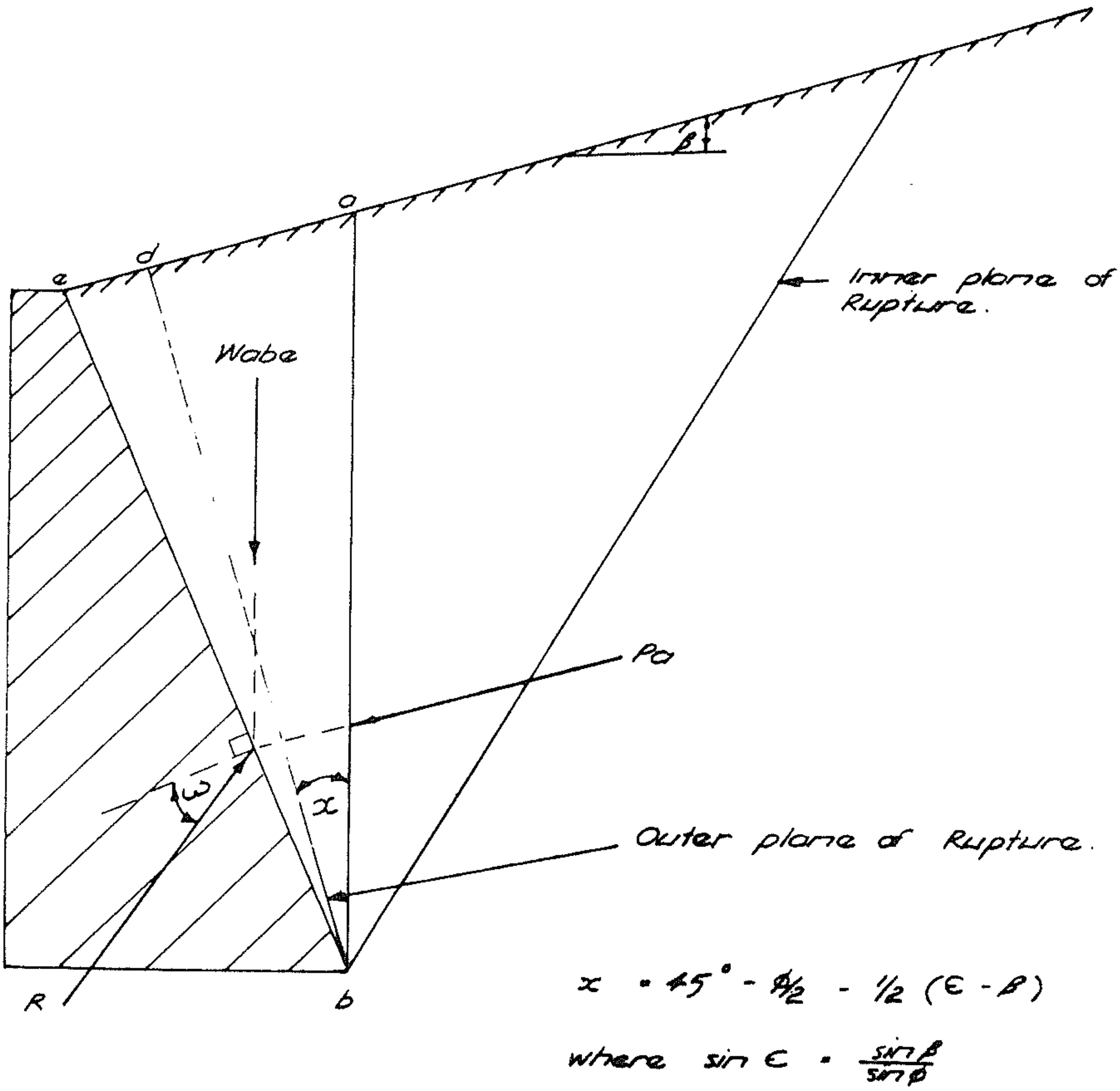

Fig 1. Factors Influencing Choice Between Rankine and Coulomb.

4. SELECTION OF PARAMETERS FOR THE ANALYSIS

4.1 Importance of Shear Strength Parameters

The most important single factor in determining the pressure acting on a retaining wall is the correct selection of suitable shear strength parameters for the backfill material. This will have a much larger effect on the computed pressures than the difference between Rankine and Coulomb, as it is very difficult to predict either c or Ø to within closer than 20-30%, which is at least twice the normal difference between Rankine and Coulomb computations of earth pressure.

In the case of Rankine, correct shear strength parameters are necessary to predict the values of principal stresses in a state of plastic equilibrium; and in the case of Coulomb, the correct shear strength parameters are necessary to calculate the shear strength mobilised along the assumed failure surface at the base of the wedge.

4.2 Effective Stress v Total Stress Parameters

Considerable differences of opinion still exist on the relative merits of using total stress and effective stress shear strength parameters. As long ago as 1943 Terzaghi (Reference (1) page 23) stated that he believed effective stress parameters should give a better prediction of the true orientation of the surfaces of sliding, but in fact effective stress parameters do not generally lead to a good prediction of the geometry of the sliding surface when using the trial wedge method.

Also the effective stress method requires an accurate prediction of the distribution of pore pressure along any failure wedge, which is not easy for short term conditions following construction of the wall, when deformation within the backfill may still be occurring which leads to a buildup in pore pressures. However for the long term (steady state) conditions, the pore pressure may be assumed to be dependant on the height of the ground water table, which is usually likely to vary seasonally.

On the other hand, if the total stress method is used, it is also very difficult to simulate conditions in the backfill behind the retaining wall when carrying out the triaxial test.

It is the author's opinion that for free draining backfills, effective stress parameters can safely be used, as pore pressures will not be built up as the embankment deforms during and following construction. However for clay backfills or other non-free draining materials, the post-construction earth pressures should be computed using total stress parameters (unless an accurate prediction can be made of pore pressure buildup); but for steady state earth pressures, a better prediction should be possible using effective stress parameters. Usually the post construction stage will be more critical, and for clay backfills it is generally reasonable to assume that $\emptyset = 0$ using total stress analysis providing that the backfill is likely to become saturated or nearly so under unfavourable conditions.

4.3 Effect of Water Pressure

As well as being of importance in predicting pore pressures in the case of effective stress analysis, when using effective or total stress analysis the height of the ground water-table is also very important in determining water pressure which may act on the wall. Beneath ground water-table, the submerged weight of soil is used in the determination of earth pressures, and water pressure is taken to act separately, being reduced neither by friction or cohesion as is the case with the soil mass. The effect is therefore that water pressure can as much as treble the load acting on the wall.

For this reason it is important to use free draining backfill and provide drainage at the base of a retaining wall if at all practicable, in order to keep the ground water-table level as low as possible. Free draining backfill has the additional advantage of preventing a buildup of pore pressure as the backfill deforms (as described in Section 4.1) and also much less deformation is necessary to achieve the active pressure state (see Section 6.4).

4.4 Testing to Obtain Shear Strength Parameters

Tables and charts are commonly available from which the resultant earth pressure on a wall of a given height can be read off for various "typical" types of soil backfill. In the opinion of the author, the use of such charts is likely to lead to gross error, especially if the user is not familiar with the assumptions inherent in the charts.

Shear strength parameters for the proposed backfill should either be assumed or else obtained by mean of triaxial testing, and the resultant earth pressure should then be calculated from first principles in the manner which is broadly outlined in this paper.

For walls up to 10ft (3 metres) high, expenditure on triaxial testing may not be justified, but it certainly can be for higher walls where significant savings in the cost of the wall may result from reliable information on soil shear strength.

For compacted soil backfills, triaxial tests should ideally be carried out on samples compacted in the laboratory over the range of dry densities and moisture contents expected in the field, which are then allowed to soak to an equilibrium moisture condition. When a wall is retaining natural ground, the best possible undisturbed samples should be obtained for testing, preferably at a moisture content as high as is likely to occur under wet ground conditions; and also because of the likely variation in natural ground both with depth and over an area, as many samples as practicable should be tested in order to obtain likely average values of shear strength parameters along a rupture surface.

During construction a careful visual examination should be made, possibly supplemented by additional triaxial testing, to ensure that the properties assumed in design are correct. (This should be carried out not only for the backfill but also for the foundations for the wall). In particular, the compaction of backfill behind a retaining wall should be closely supervised, as a weak backfill will lead to much higher pressures on the wall, and in addition unsightly settlement may occur in the soil behind the wall. If necessary, the design of the wall should be checked after the visual examination is made during construction.

4.5 Other Parameters to be Considered in Computing Earth Pressure

(a) Soil Bulk Density

Variation of soil bulk density can considerably affect the pressures computed on a retaining wall. Ideally both the saturated density, and the bulk density at natural moisture content should be obtained for all soils that may be used. Saturated density should be used for clay backfill where ground water-table level is likely to be high, and from this the submerged density can be obtained for use in earth pressure computations beneath ground water-table level.

(b) Wall Friction and Adhesion

As previously discussed, the angle of wall friction δ is usually taken to be 2/3 ϕ and the wall adhesion taken to be two thirds the soil cohesion. However, as far as the author is aware little experimental information is available to justify these assumptions.

(c) Stress-Strain Relationship

This is of some importance in determining the amount of wall yield which will be necessary to fully mobilise shear strength and hence develop the active pressure condition. This is discussed further in Section 6.4.

(d) Swelling and Softening of Clays

Clay soils will shrink as moisture content decreases and shrinkage cracks will develop. Conversely, if moisture content increases the clay will swell. This is particularly likely to happen if shrinkage cracks fill with water. The pressure exerted by a swelling clay on a retaining wall may be many times the computed active pressure used in design. Also, in stiff fisured clays, there may be a drastic reduction in strength as the swelling occurs. Such effects can generally be at least partially overcome by using free draining backfill adjacent to the wall which is drained at the base.

Clays with a plasticity index greater than 20 are more likely to be prone to these undesirable swelling effects.

(e) Permeability of Soil

This is of importance in assessing the likely rate of dissipation of pore pressure as a backfill deforms behind a retaining wall. Also it is important in evaluating whether drainage can readily be achieved to keep the ground water-table low and hence minimise water pressure acting on a retaining wall.

On the other hand, an impermeable backfill may be of advantage in a structure subject to underseepage, such as a flood wall.

5. THE EQUIVALENT FLUID PRESSURE METHOD

The equivalent fluid pressure method for designing retaining walls is very widely used by engineers, probably because the concept of a fluid of a density which will give the same earth pressure as a given soil is easily understood. The method is really a modification of the Rankine assumptions for a wall retaining a backfill with a horizontal ground surface. In fact, for a cohesionless backfill the equivalent fluid density is simply equal to the soil density divided by $(\frac{1 + \sin \phi}{1 - \sin \phi})$. However for a cohesive backfill, the equivalent fluid density is more complicated as it should be dependent on the height of the wall. In particular, the equivalent fluid pressure method can lead to gross error if applied to a wall retaining a sloping backfill surface.

The use of Rankine assumptions, if properly understood, is simple enough for there to be no good reason for using the equivalent fluid method even if the ground surface is horizontal. Since the equivalent fluid method is based on Rankine assumptions it is inapplicable under the same conditions, and departs even further from reality. Also a soil must be allowed to yield to develop the active state of stress, whereas such a movement would not be required if the soil was a fluid. The equivalent earth pressure cannot be recommended as it gives the inexperienced user a false sense of security regarding his understanding of the theory of earth pressure.

Huntingdon (Reference (2) page 139) states that "unless the assumptions on which the method is based are understood, there is a danger of misapplication leading to improperly designed Structures".

6. SOME PRACTICAL PROBLEMS IN COMPUTING EARTH PRESSURES

6.1 Surcharge Loads

The mathematical formulae can only handle surcharge loads uniformly distributed over the whole of a level backfill surface and then using the Rankine formula only.

The trial wedge method, using either Coulomb or Rankine assumptions, can conveniently handle uniformly distributed surcharge loads over part or all of the backfill surface and also line loads.

However, in the case of point loads on a backfill surface, because of the three dimensional effect within the backfill, the load applied to the wall due to the presence of the point load cannot be determined by means of the trial wedge method, and the procedure generally adopte is to compute the stresses acting in an elastic medium using the Boussinesq equations. This procedure is described in Reference (3) pp 297-303 and Reference 4 Fig. 31. There is a fundamental fallacy in taking this approach, in that when computing active earth pressure the backfill is assumed to be in a state of plastic equilibrium (in the case of Rankine) or in a state where failure is about to take place along a rupture surface (in the case of Coulomb). Hence the soil mass cannot be assumed to act elastically. However, it appears that the elastic theory is used because at present no better method is available of tackling this problem.

Reference (4) page 23 also suggests the use of the elastic theory for line loadings, when the line load is small in comparison with the active earth pressure.

6.2 Tension in Backfill

When computing active earth pressure on a retaining wall due to a cohesive backfill, the length of the rupture surface should be assumed to be reduced because of the presence of tension cracks at the surface of the backfill. In theory, the depth of these cracks will increase directly with soil cohesion and hence a stronger cohesive soil can be expected to have deeper tension cracks. Because the cohesion can be expected to vary with seasonal wetting and drying, the question arises as to what value of cohesion should be taken in the analysis of earth pressure. In the author's opinion, although the lowest expected value of cohesion will lead to smaller tension cracks than the highest expected value, it is more likely to lead to a higher value of earth pressure on the retaining wall because of the reduced cohesion along the remainder of the rupture surface.

In a cohesive backfill a tensile force is actually generated near the top of the backfill which can balance out compressive force at a lower level to give a zero resultant force. However this tensile force is not generally assumed to act in computing earth pressure for two reasons. The first is the possibility of tension cracking, and the second is that in order for this tensile force to reduce pressure on a retaining wall, adhesion must develop between the backfill and the wall, and this cannot generally be relied upon. Hence the tensile force is usually ignored.

Reference (4) page 21 suggests that under seismic loading conditions, the presence of tension cracks in cohesive soil may be ignored since the lateral compression at the ground surface due to the dynamic increment will offset the tensile stresses. This is discussed further in Section 6.6.

6.3 Mixed (or Limited) Backfill

A commonly encountered problem when a retaining wall is designed to support natural ground is that the backfill material will be partly a compacted fill (usually free draining cohesionless soil) and partly the natural ground behind. Even if the same soil is used to backfill behind the wall, the recompacted soil may be significantly weaker or stronger than the natural soil behind. The problem is whether to use the shear strength properties of the fill or of the natural ground in computing earth pressure on the wall. This problem is discussed in Reference (2) pp 98-103 and Reference (4) pp 17-18. Briefly, if mathematical formulae are used the position of failure planes in both the fill material and the natural soil should be determined (e.g. for a cohesionless backfill with a level surface the failure plane will rise up at an angle of $45^\circ + \frac{\phi}{2}$ to the horizontal). The same should also be done if the trial wedge method is used. If both planes fall within the limit of the backfill, then obviously the backfill shear strength parameters should be used; and if both planes fall within the natural ground, then the natural ground shear strength parameters should be used. If a critical failure plane falls within both materials then the shear strength parameters which give the greater pressure on the wall should be used. If for both materials the critical failure plane lies within the other material, Reference (4) page 17 suggests that the backfill may be assumed to slide along the physical boundary between the two materials and the shear strength of the backfill material should be used in the analysis.

6.4 "Non-Yield" of Retaining Wall

As stated in Section 1, neither the Rankine or Coulomb theory will give a true picture of the actual forces on a retaining wall unless the failure stress state is achieved in the soil behind the wall. This will depend on the wall being able to yield far enough to develop this failure stress state. The amount of yield required will depend on the stress-strain properties of the soil and as much as 10 times as much yield will be required for a cohesive backfill as for a cohesionless backfill. Reference (3) page 334 suggests values of yield (or wall tilt) necessary to develop active pressure for various soil types.

The wall tilt may be achieved either by the inherent flexibility of the wall, or by deformation under the pressure on the base of the wall. If a wall is to be founded on rock it may necessary to incorporate an earth pad under the toe.

For tie back walls anchored at the top, there will often be insufficient yield in the ties to develop active pressure. If for any reason it is believed that insufficient yield will occur to develop active earth pressure than the co-efficient of earth pressure at rest Ko should be used. Little experimental evidence seems to be available on the true value of this factor, but values of 0.4 to 0.5 are commonly taken. Reference (5) is a recent paper discussing Ko but the discussion is confined mainly to sands. Even less appears to be known regarding values of Ko for cohesive soils.

For a cohesionless soil the difference between "at rest" and active earth pressure will not be large unless the angle of internal friction ϕ is assumed to be greater than 30°. However for cohesive soils there could be a large difference between "at rest" and active earth pressures, depending on the value of Ko which is chosen. In the author's opinion, for cohesive soils a reasonable method is to compute both active and passive earth pressure and then to select a value for "at rest" earth pressure somewhat closer to the active case than the passive case.

6.5 Temperature Stresses in Retaining Structure

Additional earth pressure on a retaining structure can result from structural members undergoing expansion or contraction due to temperature effects. A particular case in point is a bridge abutment where long beams are in direct contact with a wall. Any temperature effects of this nature should be carefully considered in computing earth pressures.

6.6 Seismic Effects

Although this paper is concerned only with static earth pressure some brief comments are made below regarding seismic effects.

(i) A trial wedge or other form of analysis should always be carried out for the structure along the line outlined previously in this paper and ignoring earthquake forces, in order to obtain earth pressure P_A.

(ii) A similar but separate analysis should then be carried out, adding the earthquake forces to those previously considered. For cohesive soils, tension cracks should be ignored in this particular analysis. An earth pressure P_{AE} will be obtained. The difference between P_{AE} and P_A is termed the dynamic increment.

(iii) If the backfill or foundations for a retaining wall are in loose sand below water-table level, there is a risk of severe or total loss of strength in these materials due to liquefaction in a major earthquake.

6.7 Point of Application of Earth Pressure

The point of application of earth pressure on a retaining wall is equally as important as the magnitude of the earth pressure in determining the bending moment on the wall.

The following rules are generally accepted:

(i) If there are no surcharges or line loads and the backfill has a uniform slope, then the earth pressure acts at one third the height of the wall above the base (or one third the height of the "virtual back" for Rankine conditions).

(ii) If the backfill has a level surface and a uniformly distributed surcharge load, then if the Rankine formula is used, the additional pressure on the wall due to surcharge is assumed to act half way up the wall.

(iii) If the backfill has an irregular slope or surcharge load but no line loads, then determine the centre of gravity of the failure wedge plus the surcharge load and draw a line through this point parallel to the assumed failure surface to intersect the wall. This will be the point where the earth pressure is assumed to act.

(iv) If there is a line load on the backfill, the increase in earth pressure ΔPa due to the line load should be calculated. Reference (6) pp. 205-210 describes a method to determine the point of application of ΔPa.

(v) If there is a point load on the backfill, pressure on the wall due to th point load is calculated using elastic theory, and there will be an irregular distribution of pressure on the wall which can be assumed to act at the centre of the pressure diagram.

(vi) Where the ground water-table levelis within the backfill, the centre of gravity of the failure wedge should be calculated using the submerged density below water-table level, and the point of application of earth pressure is then obtained in the same manner as in (iii) above. Water pressure will act separately at one third the height of the ground water-table above the base of the wall.

(vii) For earthquake forces, the dynamic increment (as defined in Section 6.6) can be assumed to act at two thirds the height of the wall.

7. SUMMARY

(a) In order to predict correctly earth pressures using either Coulomb or Rankine assumptions, the retaining wall must be capable of yielding sufficiently to develop the full shear strength of the soil. The amount of yield required is dependent on the stress-strain properties of the soil and is much greater for a cohesive than a cohesionless soil. If this amount of yield is not possible, then the co-efficient of earth pressure at rest should be used.

(b) The most important single factor in correctly computing earth pressures on a retaining wall is to make a good prediction of the shear strength properties of the backfill. This will involve correctly simulating field conditions in the triaxial test if total stress analysis is used, or correctly predicting field pore pressures if effective stress analysis is used. The backfill in the field must be at least as good as that assumed in design or else the error involved will be much greater than the difference between Coulomb and Rankine assumptions.

(c) Charts available for predicting earth pressures behind retaining walls for certain standard soil types are not recommended, even for retaining walls of limited height, unless the user has a very good understanding of the assumptions on which the charts are based. For the same reason the equivalent fluid method of determining earth pressure is not recommended.

(d) Backfill behind a retaining wall should always be drained if at all possible as this has a twofold effect. Firstly, water pressure acting on a wall is generally much more significant than earth pressure; and secondly, earth pressure can be reduced in a drained fill as the shear strength parameters are likely to be increased.

8. CONCLUSION

For most retaining structures (excluding very high walls) designers will generally prefer to keep their calculations as simple as possible and use their engineering judgment. However a good knowledge of the assumptions made in earth pressure theories, together with the difficulties in applying the theories to practical problems is most important in developing engineering judgment.

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DYNAMIC PRESSURE ON EARTH RETAINING STRUCTURES

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INTRODUCTION

This paper illustrates some of the problems associated with dynamic soil pressure as it is at present understood, and briefly reviews available design methods. New Zealand is in a seismic zone and therefore dynamic design is just as important in soil structures as it is in other forms of structural design, methods for which are already well developed and included in design code requirements.

The uncertainties of ground motion cannot be under-estimated. In a recent statistical study of the problem D. Vere-Jones (Ref. 1) pointed out that stastical formulae for the probability of failure are convenient, but gloss over the important but difficult problem of variable response due to soil type and topography and the possibility of non-stationarity. It is a matter of experience that soil type and soil properties are of primary importance in the behaviour of soil retaining structures and in fact any structure above the ground as well.

There are other aspects of earthquake behaviour, which at present are completely omitted from design considerations and in fact have received scant attention in the research fields. These factors could have very significant effects on structural behaviour, but they are not included in design methods because our fundamental knowledge of them is almost nil and there are no recorded measurements. In general our knowledge of soil and structure interaction is largely observational and it is pertinent to draw attention to some of the gaps that exist in the seismic design field.

1. What is the effect of horizontal wave velocity and the impact of wave energy on a structure, particularly for structures with a length comparable to or greater than a seismic wave length? Long retaining walls are particularly vulnerable in this regard.

2. The influence of the rigidity of a structure is considerable. This factor is included in the seismic design of above-surface structures. It is being investigated in the research field for earth retaining structures, is being found to be important but as yet is not included in design methods for earth structures.

3. Periodic phasing of motion is very possible. This is the effect of a sudden amplification of motion due to the seismic frequencies coming into phase with the fundamental frequency of a structure for a short period of time.

4. Distance from epicentre is important, which leads to recognition of the different localised effects created by short period high frequency vibration near an epicentre as compared with longer period lower frequency vibrations at a greater distance from an epicentre. The big question here is knowing in advance where an epicentre is likely to be in relation to the structure being designed.

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5. Of great importance, but seldom recognised in seismic design criteria, are the localised microzone effects such as changes in geological formation in the subsurface materials along the length of a structure, such as across a river bed under a bridge or under a long retaining wall.

CAUSES OF DYNAMIC EARTH PRESSURE

Vibration in soil from any cause, if great enough, can create dynamic earth pressure effects. Seismic action is primarily responsible for the major effects of this sort and the evidence, after any earthquake, observed on retaining walls and bridge abutments is sufficient to show the existence of dynamic earth pressure. Estimation of the magnitude of the dynamic pressure is very much more difficult than recognising its existence. Growing interest in seismic research has to some extent focused attention on soil behaviour and the generation of dynamic pressure. The engineering reality of the problem has been appreciated for at least fifty years but in this time no finality has been achieved in design methods. Japanese research in the 1920's produced the Mononobe-Okabe formula, which has been widely used but is not the complete answer.

EFFECTS OF VIBRATION ON SOILS

In general soils will undergo considerable changes when vibrated. Depending on their nature, characteristics and degree of confinement, they may do any of the following: densify, consolidate, liquefy, expand or spread.

Earthquakes are responsible for dramatic earth pressure effects, but less obvious although insidious causes may be at work. Small repeated vibrations of traffic such as heavy locomotives on bridge abutment fills can gradually change the earth pressure on the abutment over a period of time.

The degree of confinement or restraint has a very great influence on the resulting pressure effects under seismic action. High levels of restraint can increase the magnitude and alter the distribution of dynamic pressure.

The type of material has a major influence on its behaviour under dynamic conditions.

1. Gravels and graded cohesionless material will tend to densify and so increase residual pressure in addition to any immediate dynamic effects.

2. Sands, particularly saturated uniform sands, may liquefy if not constrained by sufficient overburden pressure.

3. Clays and silts can deform plastically and under dynamic conditions the cohesion and angle of friction may be much lower than the static state conditions.

GROUND ACCELERATION

The problems of dynamic pressure can become very complex and some research workers (Refs. 2 and 3) have produced some very elegant and highly mathematical solutions to the problem.

The primary considerations are very simple to state and most difficult to solve:

1. What ground accelerations are possible?

2. What accelerations should be adopted as design criteria?

From the first question, it is possible to have horizontal (and perhaps vertical) accelerations exceeding 1.0g under some geological and site conditions near an epicentre. This has been noted in several earthquakes including Inangahua. Ambraseys (Ref. 4) has quoted five strong motion records with peak accelerations ranging from 50% to 124% g. Depending on the nature of the subsurface material Ambraseys suggests that alluvial deposits have a self-limiting capability for transmitting high accelerations. His suggested limits are 0.25g to 0.35g for high plasticity deposits, and for saturated sandy clays and medium dense sands this figure could be 0.5g to 0.6g. Weak layers in stratified material can reduce this to 0.3g.

It appears then that if a mathematically accurate dynamic analysis is justified the choice of acceleration values must be carefully made to suit the materials. It is not difficult to make several trials with finite element computer programmes because for an elastic analysis the end result is in linear proportion to the input acceleration, hence simple scaling factors can be applied to output results.

Currently used computer programmes cannot cope with non-elastic, or post-elastic behaviour of ground materials.

Due consideration must be given to the interaction of the structure and the ground if the structure has a natural period within the seismic period range. Seed and Idriss (Ref. 5) showed that for the El Centro earthquake the acceleration induced in a simple structure varies with the fundamental period of the structure. The maximum acceleration in the El Centro earthquake was 0.339g. From their study it was found that induced accelerations were as follows:

Structure period	Induced acceleration
0.2	1.0g
0.5	1.02g
1.0	0.48g

This indicates that for this particular earthquake a resonance condition would be created at periods of about 0.5 to 0.2 seconds, e.g. frequency range 2 to 5 Herz, which is very commonly found in seismic frequencies.

MAGNITUDE AND DISTRIBUTION OF DYNAMIC PRESSURE

An increasing amount of research has been aimed at determining these factors. The widely used Mononabe-Okabe formula, for example, provides a means of finding a magnitude, but the point of application of the resultant dynamic force has been questioned and is thought to be incorrect.

Indian research with model tests has indicated that the combined static and dynamic distribution of pressure is approximately parabolic with a centre of pressure a little above 0.6 of the height of a wall. This has been recently substantiated by Scott (Ref. 3) with a mathematical analytical method which indicates that the M-O formula may give a reasonable definition of magnitude in some cases, but in general is not adequate and the point of application of the resultant force should be at about 0.64 H.

The magnitude and distribution of dynamic pressure depend very much on the type of restraining structure, its flexibility and the degree of fixing at the bottom and top. For example, restraint at the top by ties or props (e.g. superstructure of a bridge) can create particularly large forces at the point of restraint. Tied or propped walls or abutments will crack at points of highest stress (Ref. 6). Without any restraint the combined static and dynamic forces appear to be nearly uniform as illustrated by the action of a crib wall on the wingwall of Brown Ck bridge during the Inangahua earthquake. This cribwall moved almost horizontally for a distance of 3 ft 9 ins with no change in front batter and very little apparent damage.

In general it can be assumed for the design of walls not propped or tied at the top, that the centre of dynamic pressure is between 0.6H and 0.66H and the static pressure resultant is at 0.33H. In combination these create an almost rectangular pressure distribution.

A close study of the damage to five bridges after the Inangahua earthquake was made by the author (Ref. 6) and this indicated that the forces on the abutments would have been three to four times the normal active pressure.

Seed and Whitman (Ref. 7) have published an excellent "state of the art" survey of dynamic earth pressure and methods of design. In general they support the M-O formula for design, but also point out various important influences not taken into account in this formula, such as sloping surcharged backfilling and the very great effect of water saturation of the backfilled material.
The design methods described by them are based on wide experience and a range of relevant published data.

In an analytical method using a one dimensional shear beam Scott (Ref. 3) finds a most probable deflection and a most probable pressure distribution at all levels. With this method the total pressure resultant is significantly higher than the M-O formula.

A finite element solution by Aggour and Brown (Ref. 8) shows that a wide range of dynamic pressures and distribution is possible depending on the geometry and material properties. They also emphasise the aggravation to damage if the soil structure system has a resonance frequency within the seismic frequency range. In order to prevent wall and filling separation the initial static pressure must exceed any dynamic tension that could occur from oscillations. Consequently adequate compaction of backfilling material is very important to dynamic stability and overall coherence of the soil structure system.

DESIGN METHODS

The assumptions to be made and the refinements of method depend mainly on the magnitude of the project and the amount of economy of materials that might be achieved by careful consideration of the material properties and dynamic behaviour. A large project is worthy of complete site and materials investigation and refined design methods leading to the most economical solution, whereas for a small earth retaining structure the use of simple conservative assumptions is quite justifiable.

For a small project it may be assumed for combined static and dynamic pressure that:

1. Earth pressure is uniform, i.e. centre of pressure is at mid-height (or a little higher).

2. The magnitude of the pressure resultant is three times the normal active design pressure from Coulomb or Coulman line analyses.

3. Allowable stress increases of up to 50% would be reasonable.

This approach would be conservative for a flexible wall, but may not be adequate for walls tied or propped near the top.

Another design approach for simple cases of vertical walls and horizontal backfills is proposed by Seed and Whitman (Ref. 7). This appears to be simple to apply and approximates the M-O total pressure but applies the resultant at 0.6H above the base.

For walls supporting sloping backfill a more detailed analysis is required and Seed and Whitman (Ref. 7) show that a backfill with a 20° slope would have approximately double the pressure effect of a horizontal backfill.

For large projects a complete analytical treatment is suggested:

1. The properties of all the materials under and behind the wall must be correctly known. These include: in-situ density, shear modulus, elastic modulus and Poisson's ratio for all materials.

2. Complete geological geometry of the site should be known to bedrock under the site.

3. The nature of the input vibration and its region of application must be known or assumed (e.g. seismic input at bedrock if possible).

4. The problem can then be solved with a finite element computer programme of the type proposed by Carr and Moss (Ref. 9), Scott (Ref. 3) or Aggour and Brown (Ref. 8).

5. Alternatively the non-computer methods given by Seed and Whitman (Ref. 7) may be applied using a number of design charts.

MATERIAL PROPERTIES

A complete site investigation is essential to obtain subsurface data. In-situ and laboatory tests can provide the parameters required. Silts and clays can be tested in a dynamic triaxial machine. Field tests can provide densities and in-situ dynamic parameters can be obtained from wave velocity tests.

INPUT

A variety of digitised real earthquake recordings and artificial vibrations are available for computer use. Once a programme has been set up various inputs can be tried to test the effect of varying the input. The application region of the input is one of the most difficult things to decide. Theoretically it should be at bedrock with all overlying strata identified, with known dynamic properties. However, simplifying assumptions may be justified with an input vibration at a much higher level even up to the foundation of the wall. Not a great deal is known about the effect of varying the input vibration positions.

CONCLUSIONS

Although the ultimate methods of design for dynamic pressure on earth retaining structures have not yet been achieved because many unknown and questionable aspects are unresolved, sufficient is now known about dynamic behaviour to enable a rational approach to be made to this problem. If seismicity is to be considered as a real problem in engineering design then it is just as important that earth structures be designed for this, as is the use of already codified procedures applying to above-surface structures.

Seed and Whitman (Ref. 7) draw attention to the lack of provisions in various national codes for the adequate seismic design of earth retaining structures. Only five countries - Portugal, Turkey, India, Greece and Japan - have some requirement for increased seismic pressure and in the last three the M-O formula is the recommended design method.

The U.S.A. has no national code provisions but T.V.A. (from 1939!) has had remarkably comprehensive seismic requirements. Not only do they require the use of the M-O formula, but the ground saturation is also taken into account, with the resultant force point of application to be taken between 0.58H and 0.67H. However, the acceleration at 0.18g appears to be rather low. Stress increase allowances are 50% in concrete and for overturning analyses the line of resultant force may fall as far out as the quarter point of the base.

Some similar provisions should be embodied in New Zealand codes to provide guidelines for the design of earth retaining structures.

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COMMENTS ON EARTHQUAKE-INDUCED SOIL PRESSURES

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The basic input parameters required for the analysis of a dynamic soil-structure interaction problem are seldom known with any degree of precision. Many soil-retaining structures will be of insufficient importance to warrant more than a very basic soil investigation and only in exceptional circumstances would a soil investigation be sufficiently detailed to enable a good prediction of the soil behaviour under dynamic loading. Uncertainty exists regarding the magnitude and frequency composition of incoming earthquake waves at any particular site. At the present time the nature of the mechanism generating the waves is not precisely known and no satisfactory method exists for predicting the modifying effects of the geology along the travel paths. Even if generation and modification of the earthquake waves could be accurately modelled, uncertainty would exist regarding the relative location of possible epicentres for major earthquakes. In view of the limitations in the basic input parameters, the estimation of earthquake-induced pressures by approximate methods such as the Mononobe-Okabe can be frequently justified. However, it is important to appreciate the basic assumptions of the Mononobe-Okabe method and the limitations of its applicability.

BASIC ASSUMPTIONS OF MONONOBE-OKABE METHOD

1. The wall is assumed to displace laterally a sufficient amount to produce a state of plastic equilibrium behind the wall.

2. The soil is assumed to satisfy the Mohr-Coulomb failure criterion.

3. Failure in the soil is assumed to occur along a plane surface through the toe of the wall and inclined at some angle to the horizontal.

4. The wedge of soil between the wall and the failure plane is assumed to be in equilibrium at the point of incipient failure, under gravity, earthquake and the boundary forces along the wall and failure surface. The forces acting on the soil wedge of weight W are shown in Fig. 1 for the case of cohesionless soil.

5. The effect of the earthquake is represented by equivalent static horizontal and vertical forces $k_h W$ and $k_v W$ applied at the centre of gravity of the wedge.

6. The method gives the magnitude of the total force acting on the wall but does not give its point of application or the pressure distribution. In the initial development of the method it was apparently assumed that the total force acted at point 1/3H above the base of the wall of height H. In view of model test results and more recent refinements of the method, Seed and Whitman (Ref.1) have recommended that the force increment on the wall due to the earthquake load be assumed to act 0.6H above the base.

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LIMITATIONS OF MONONOBE-OKABE METHOD

1. For a fully plastic stress state to develop in the soil behind the wall it is necessary for the top of the wall to deflect outwards about 0.5% of the wall height. (The amount of outward displacement required is dependent on the soil properties.) Although this condition may be satisfied by many cantilever walls and other relatively flexible wall types this may not be the case for more rigid structures and structures founded on piles or on rock foundations. An example of a structure of sufficient rigidity to prevent development of a plastic stress state is shown in Fig. 2.

   Finite element analyses show that if a wall is sufficiently rigid to prevent the development of a fully plastic stress state, the earthquake-induced forces may be significantly in excess of the Mononobe-Okabe value. The results of a static linearly elastic finite element analysis of a slender rigidly founded cantilever wall are given in Fig. 3. The pressure distributions shown are for a 1.0g horizontal body force in the soil layer (no gravity loading), and demonstrate the influence of the soil-wall stiffness ratio, S, which is defined by

$$S = \frac{E_s H^3}{E_w I_w} = \text{soil-wall stiffness ratio}$$

   where E_s = Young's modulus for soil

   E_w = Young's modulus for wall

   I_w = second moment of area of the wall per unit length

   The Mononobe-Okabe pressure distribution for a soil having an angle of internal friction of 35° is also plotted in Fig. 3. The distribution shown is the earthquake increment for a 1.0g horizontal coefficient and is assumed to be triangular for the purpose of comparison with the finite element solutions.

   As the stiffness of the wall is decreased the finite element solution shows that tension develops at the top of the wall and progresses downwards. Because in general the wall will separate from the soil in the tension zone the solutions are not exact for this case. The possibility of appreciable inelastic behaviour of the soil as the wall stiffness is decreased also limits the applicability of the elastic solutions. However, this very basic analysis shows that the magnitude and shape of the earthquake-induced pressure distribution is likely to be quite sensitive to the relative stiffness between the wall and soil. It is of interest to note that reasonably good agreement exists between the triangular Mononobe-Okabe distribution and the elastic solution for an S value of about 100.

2. In the Mononobe-Okabe method it is not in general possible to include the influence of the dynamic behaviour of the structure on the soil pressures. For example, the large power-house structure shown in Fig. 2 will vibrate by rotation on its elastic rock foundation and the pressures generated by movement of the soil-retaining wall towards the soil layer cannot be obtained from the Mononobe-Okabe method. In certain cases these pressures resulting from movement of the structure relative to the soil need to be superimposed with pressures resulting from inertia body forces in the soil.

3. Although the assumption of a plane failure surface appears reasonable, its validity has been based on a very limited number of test and field observations. More research employing numerical methods is required to check this assumption.

4. The Mononobe-Okabe method is essentially a static method, requiring the selection of suitable earthquake coefficients k_h and k_v which are used to determine equivalent static inertia forces on the wedge. No account is taken of resonance effects or the amplification of the earthquake motions that might occur as a result of the propogation of the motion through a relatively soft soil layer behind the wall.

   In general, coefficients are chosen that are significantly less than the peak accelerations to be expected in a suitable design earthquake, apparently on the assumption that some permanent outward movement of the wall can be tolerated. There appears to be no rational basis for the magnitude of the reduction made. Reasonably exact solutions of dynamic plasticity problems are undoubtedly difficult but nevertheless it would seem desirable to improve the Mononobe-Okabe approach by accounting for dynamic effects in an approximate manner. The method used by Newmark (Ref. 2) to study the stability of dams and embankments during earthquakes could be easily applied in the study of wall problems and would be a convenient way of extending the Mononobe-Okabe method to include basic plastic-dynamic behaviour. In Newmark's method the sliding displacement of a soil mass resulting from peak inertia forces exceeding the sliding resistance, is computed by using a simplified form of acceleration pulse. If this approach is used the design would be based on choosing an acceptable limit for the permanent displacement of the wall rather than the selection of an earthquake force coefficient. Newmark's results demonstrate that if the peak accelerations in the design earthquake are significantly greater than the earthquake coefficients used to compute the Mononobe-Okabe force, appreciable displacement of the wall may occur.

   If the basic Mononobe-Okabe assumptions are unlikely to be satisfied for a particular wall structure it is recommended that earthquake pressures be estimated by the finite element method. Because of the imprecision of the basic input parameters, simplifications regarding the soil properties and problem geometry can often be justified. For relatively rigid wall structures the results obtained by Wood (Ref. 3) from elastic theory may be appropriate.

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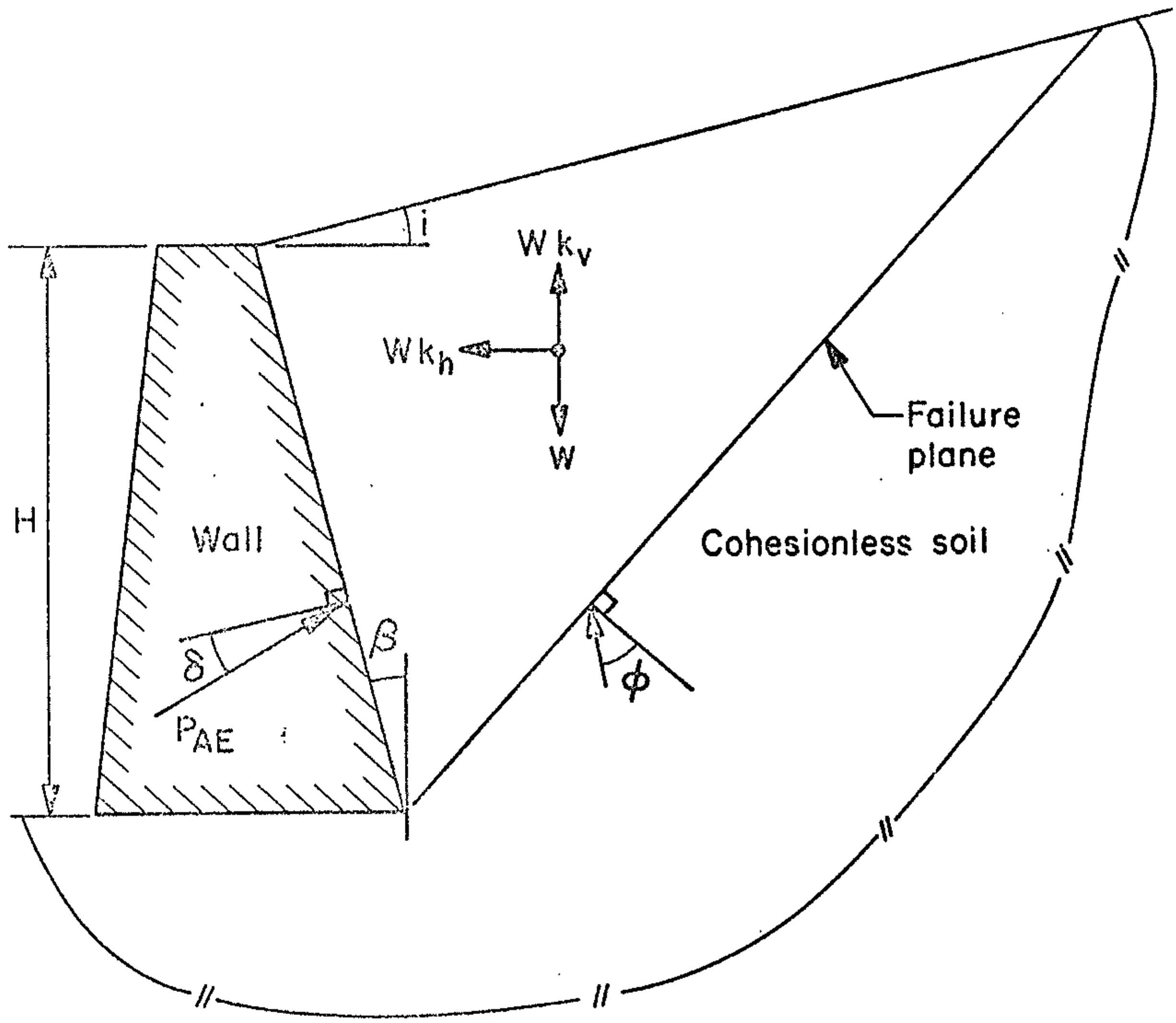
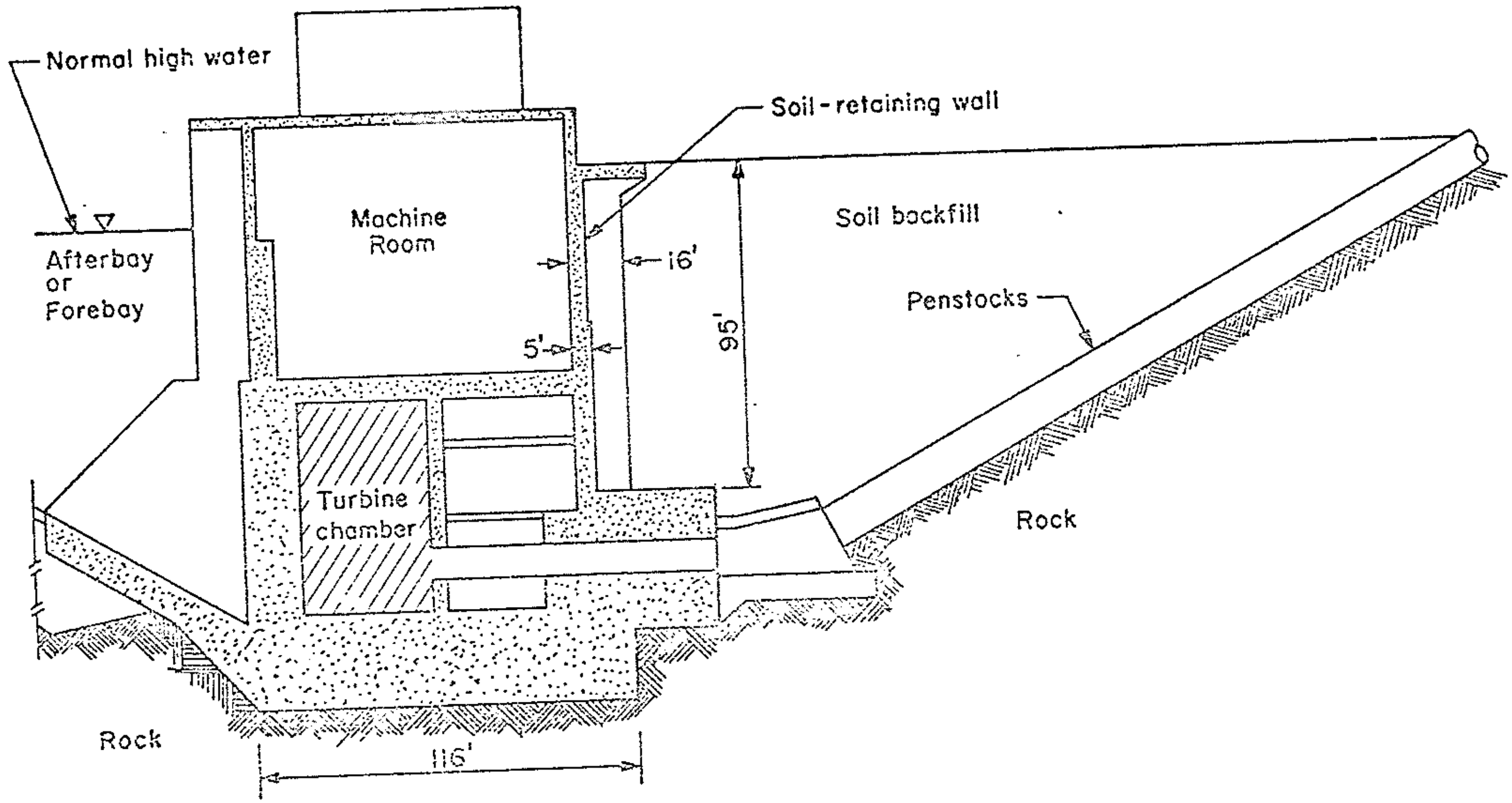


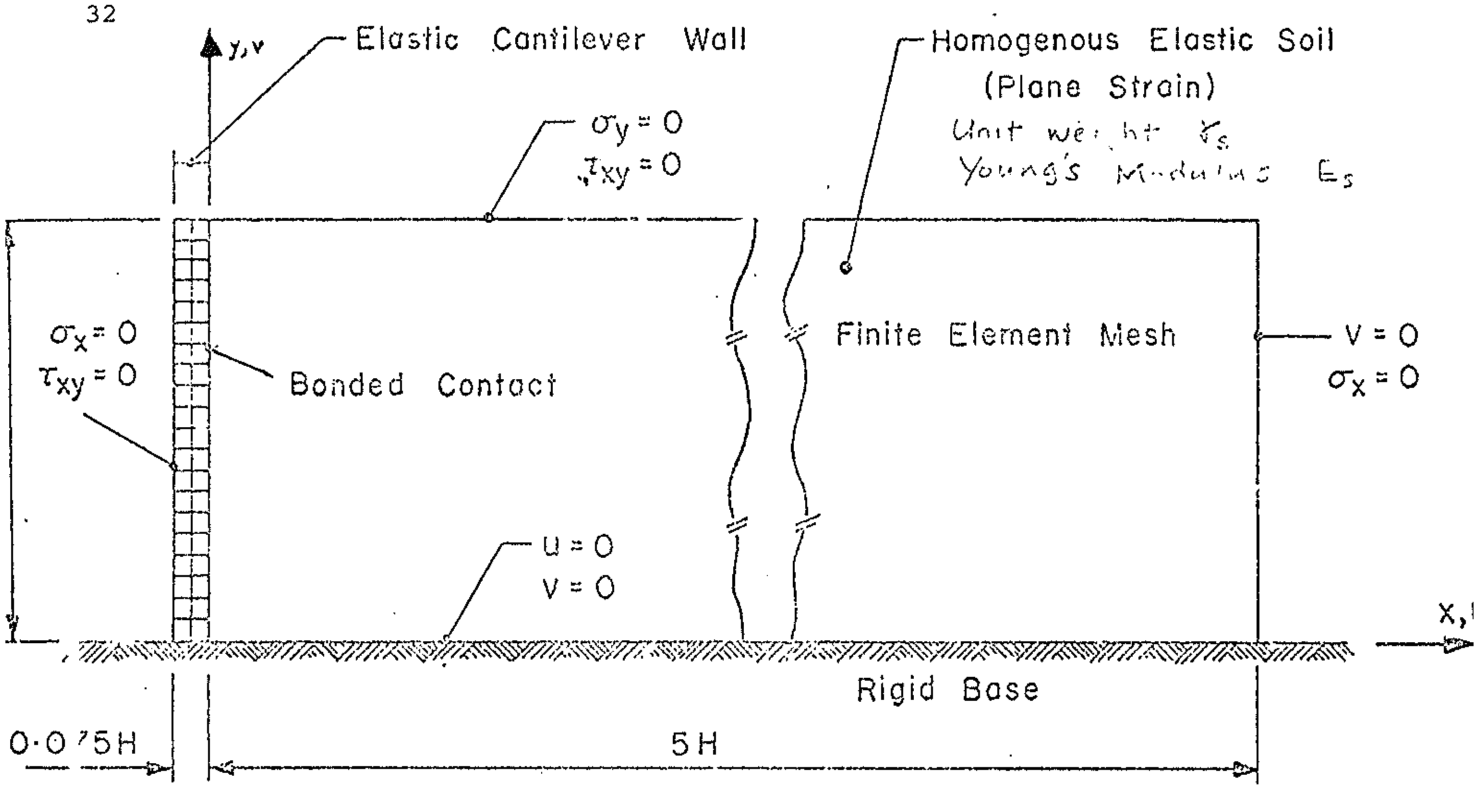
MONONOBE–OKABE ANALYSIS

Figure I



CASTAIC POWERHOUSE
Typical Cross-section

Figure 2



CANTILEVER WALL PROBLEM

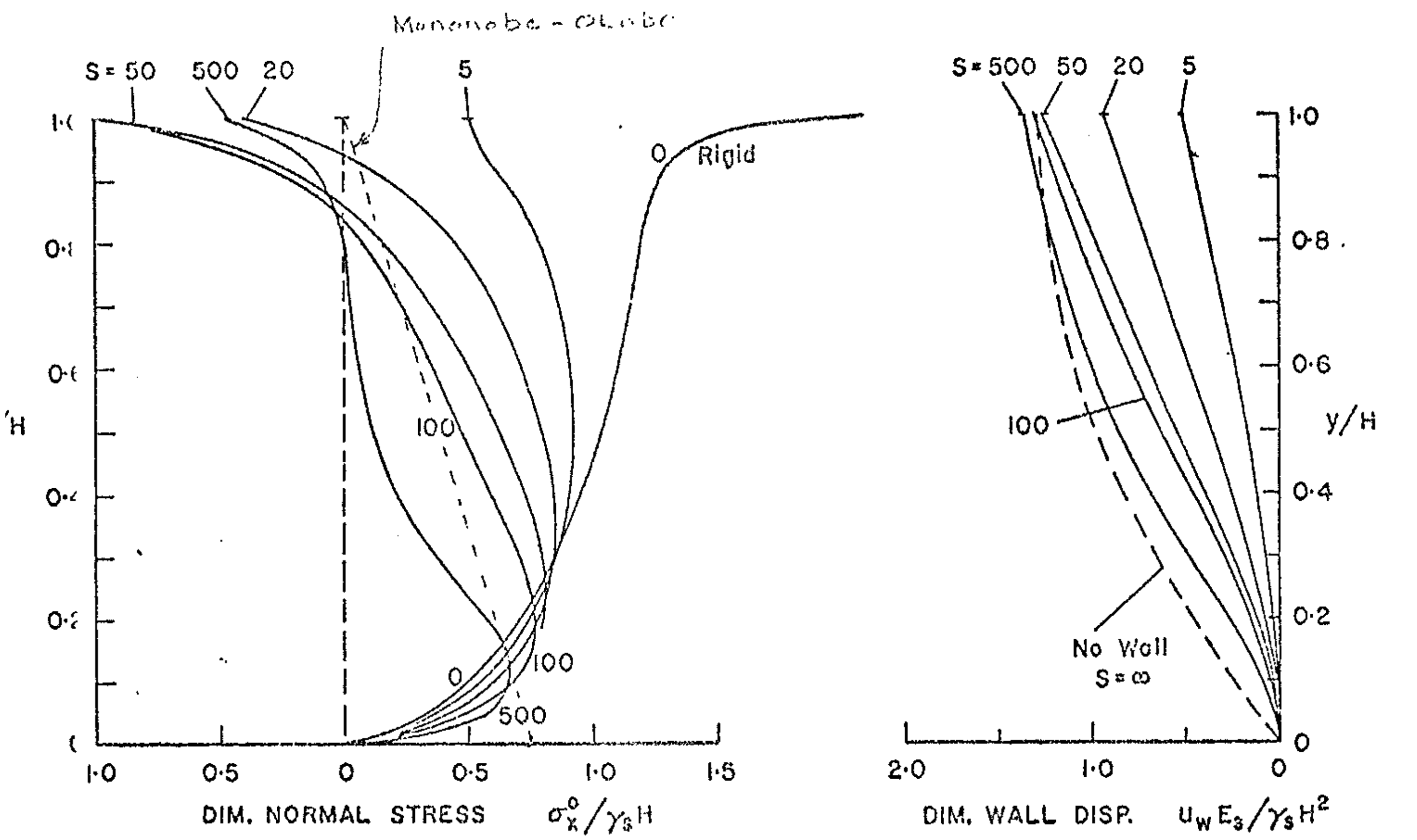


Figure 3 Pressure distributions and displacements for cantilever wall. One-g static horizontal body force. Poisson's Ratio = 0.3 . σ_x^0 = normal stress on wall. u_W = wall displacement

AN INTRODUCTION TO THE M.W.D. RETAINING WALL DESIGN NOTES

J.C. RUTLEDGE* and A. KENNAIRD**

J.C. RUTLEDGE

The Retaining Wall Design Notes are intended as a guide for use in the design and construction of retaining walls and similar earth retaining structures. They have been developed from notes used in Wellington Urban Motorway design office in 1970. By using the "Notes" it is hoped that junior and inexperienced staff can learn more quickly how to design sound, economical walls and that a more uniform design approach will result.

The "Notes" are not intended as a soil mechanics textbook.

The emphasis in the "Notes" has been towards important aspects of wall design such as:

1. The provision for earthquakes.
2. The bearing capacity of foundations.
3. The selection of backfill material.
4. The importance of wall movements.
5. Water effects.
6. The design of crib walls.

Some of these points such as earthquake loading and foundation bearing capacity under eccentric inclined loads, are not readily available in standard textbooks and are covered in more detail in the "Notes".

A very important section of the "Notes" is that concerning the calculation of foundation bearing capacities. In the past the traditional Terzaghi formula has been commonly used. This method if used for retaining walls gives inaccurate answers that are not safe.

Bearing capacity calculations should be made using methods which allow for eccentric inclined loads with allowances where appropriate for foundations that are inclined or on a slope. Such methods have been developed by Meyerhof and J.B. Hansen. The authors personally favour the use of Hansen's method, particularly for foundation materials with angles of internal friction of less than 30°.

The following example shows the importance of not using the Terzaghi formula.

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A 38 ft high counterfort wall on foundation material having a cohesion of 5 psi and angle of internal friction of 25°. With earthquake loading the calculated factor of safety against ultimate bearing failure would be:

Using Terzaghi	7.8
Using Meyerhoff	0.8
Using Hansen	1.5

It should be appreciated that bearing pressure failure and sliding failure are not separate failure mechanisms but are linked.

The "Notes" emphasise the importance of allowing for water pressures and in providing a designed drainage system that includes considerations of permeability and filter gradings. In most cases the drainage system cannot be inspected or replaced and so it must function correctly.

Crib walls are one of the most common types of wall used in New Zealand. However, there have been a number of crib wall failures. The main reasons for the failures seem to be:

(i) The use of single depth walls when double or triple depth walls should have been used. This indicates inadequate attention to design or the wrong evaluation of site calculations.

(ii) The use of cohesive and impermeable backfill and infill materials.

(iii) Not using a reinforced concrete foundation slab to prevent bearing failure under units and differential settlements.

(iv) Poor workmanship in the manufacture of units and the construction of walls.

(v) Inadequate drainage.

(vi) Inadequate compaction of backfill and infill material.

Information provided by crib wall manufacturers should be used with care and always checked by calculations.

The Ministry of Works and Development has recently revised specification MOW 7562 which is a specification on the manufacture of crib units and the construction of crib walls. Crib walls designed according to the "Notes" and manufactured and constructed according to the revised specification should be sound, safe walls.

Some comments have been made about the extra costs that will result from using the "Notes". Excluding earthquake loadings the "Notes" should mean no extra cost because such factors as eccentric and inclined loads should always have been allowed for. With regard to earthquake loadings some examples may be interesting.

In the following examples the live load surcharge is 2 ft and the seismic coefficient is 0.2.

For conventional cantilever or counterfort walls with level backfills there is no increase in cost for wall heights of up to about 24 ft.

For wall heights up to 36 ft the increase in cost is about 5-8%.

For crib walls with level backfills there is no increase in cost for walls up to about 17 ft in height. At 23 ft the increase in cost is about 3%.

However, for walls with sloping backfills, particularly crib walls, the effect is much greater.

For example:

For 6 ft crib walls there is no increase in cost.

For 9 ft crib walls the increase is 30%.

For 13 ft crib walls the increase is 50%.

A. KENNAIRD

The aim of writing the "Notes" was to show in practical terms the factors which affect earth pressure and how the effect of each should be estimated for a particular wall.

An accurate estimation of soil properties is basic to the whole exercise of earth pressure calculations. In some situations using $C = 0$ and $\emptyset = 30^{o}$ would perhaps be all that was required to give a conservative design for granular backfill. However, this is often used in cases where it is not at all applicable. The tables give typical soil properties for a range of materials but it is always preferable to estimate the values from tests on the particular material involved. Since there are a number of test methods which could be used to determine the soil properties, the designer of the earth retaining structure should take an interest in the soil testing so that values applicable to the situation are used.

Where there is a choice of backfill, a free draining granular material should be used if economically feasible. Many walls are still built using impermeable clayey backfill but this produces high earth pressures and drainage problems. Where a clay bank apparently has a high strength it may at first appear that little support is necessary. However, the clay may be subject to softening which could reduce the shearing strength to a small fraction of its original value over a period of time. In this case a better design could include excavation and backfilling with a granular material.

The increase in lateral earth pressure or the decrease in bearing ability which results from liquefaction of sandy soil layers can be cause for concern in areas subject to earthquake or other shocks. It should be noted, however, that this phenonemon is not as common as would appear from recent literature. A number of factors must all be present before the soil would be prone to liquefaction. Perhaps the most important of these factors is that the soil must be saturated.

Mr. Blakeley's paper has touched on most of the aspects of calculation of static earth pressures covered by the "Notes". His paper mentioned the use of wall adhesion in the equation for Coulomb's theory. The "Notes" recommend that this should be ignored since it cannot be relied upon. It is not always sufficient to simply use the classical theories to calculate the static lateral earth pressure. Limitations on their application must be understood if a realistic estimation of earth pressure is to be obtained.

The limiting equilibrium theories all require that the maximum shearing strength of the soil is mobilised at failure of the system. This requires a deformation in the soil which may not eventuate. If sufficient information is available it is possible to determine lateral earth pressure from the available wall movement. For example, Fig. 1 shows the relationship between wall rotation and earth pressure coefficient for sands. A major point of interest is the small displacement required to reduce the earth pressure to the fully active state. On the other hand the movement required for full passive pressure is much larger. For clayey soils, the movement required to produce active pressure can be up to ten times that for sand.

The amount of wall movement is dependent on the rigidity of the wall and the foundation. If the wall is designed to slide or tilt so that the earth pressure is always reduced to the active value, this minimum pressure can be used for stability and structural calculations. However, it is often desirable or necessary to hold the wall in place which will mean that pressures remain in excess of the active values. In the absence of more extensive information, the design "Notes" suggest that for rigid walls on rigid foundations the static lateral earth pressure should be the at-rest value. For walls of intermediate rigidity the earth pressure should be half way between the at-rest and active values. Walls on soil foundations and flexible cantilever walls can be designed for active pressure.

There is a lack of practical methods for calculating the at-rest lateral earth pressure. However, for the case of a vertical wall and a horizontal ground surface, the coefficient of at-rest earth pressure K_o is usually taken as equal to $1-\sin\phi$ for normally consolidated or lightly compacted materials. For this case a linera pressure distribution is indicated by elastic theory. For sloping backfills, elastic finite element solutions indicate that the lateral pressure distribution becomes more trapezoidal, i.e. point of application higher up the wall. The total pressure may be estimated from an equivalent triangle given by assuming that K_o varies proportionally with K_a.

Compaction of backfill behind a restrained wall may produce lateral earth pressures even in excess of the at-rest values particularly over the top 6 ft (2 metres). Broms (Ref. 1) investigated this and has proposed a design distribution for granular material compacted against a rigid wall (See Fig. 2). Experimental measurements of pressure against bridge abutment walls, recently reported by Casagrande, also show a non-linear distribution with lateral pressures in excess of the at-rest value near the top of the wall. There is therefore often a need to design for earth pressures greater than active or at-rest values near the top of a restrained wall. Where walls are designed for earthquake effects, the design earthquake earth pressure will produce this additional strength at the top.

In general allowance should only be made for earthquake effects for important structures. For many walls the cost and effort involved in designing for earthquake would not be economically warranted. The "Notes" recommend importance categories for level of seismic design. It should be noted that walls adequately designed for static earth pressures, especially those with an allowance for live load, will automatically have the capacity to withstand at least moderate earthquake ground motions and in many cases earthquake loading would not be a critical design case.

The most common method of obtaining lateral earth pressures due to earthquake loading is the pseudo-static seismic coefficient method. In this method a force equal to the weight of a wedge of soil multiplied by a specified value of seismic coefficient is assumed to act statically at the centre of gravity of the wedge. A horizontal seismic force only need be used since typical vertical accelerations have a very small effect on earth pressure. The seismic coefficients given in the "Notes" are determined without regard to the dynamic characteristics of the retaining structure or soil. There are obviously a number of approximations in this approach. However, it has been used extensively overseas and for the normal type of free standing wall has been shown by experiment to give reasonable answers.

Mononobe and Okabe derived equations based on Coulomb's theory for active and passive earth pressure in cohesionless soil with the additional seismic forces (both vertical and horizontal) included as set out in Fig. 19 of the "Notes". Originally, it was assumed that the pressure distribution was triangular with the resultant acting at one-third the wall height from the base. This has since been shown to be incorrect and a method whereby the dynamic increment is applied at the top third point (as shown in the "Notes) is now generally used.

The seismic coefficient method can be used in conditions where the wall can move so as to develop the full shearing strength of the soil, but for rigid walls or rigid foundation conditions it is not applicable. The "Notes" make some recommendations for these cases but further work is needed in this field. This problem is discussed in Dr. Wood's paper.

REFERENCE

1. Broms B. (1971) : "Lateral Earth Pressures due to Compaction of Cohesionless Soils". Proc. 4th Budapest Conf. on Soil Mechs. and Fdn. Eng. pg 382.

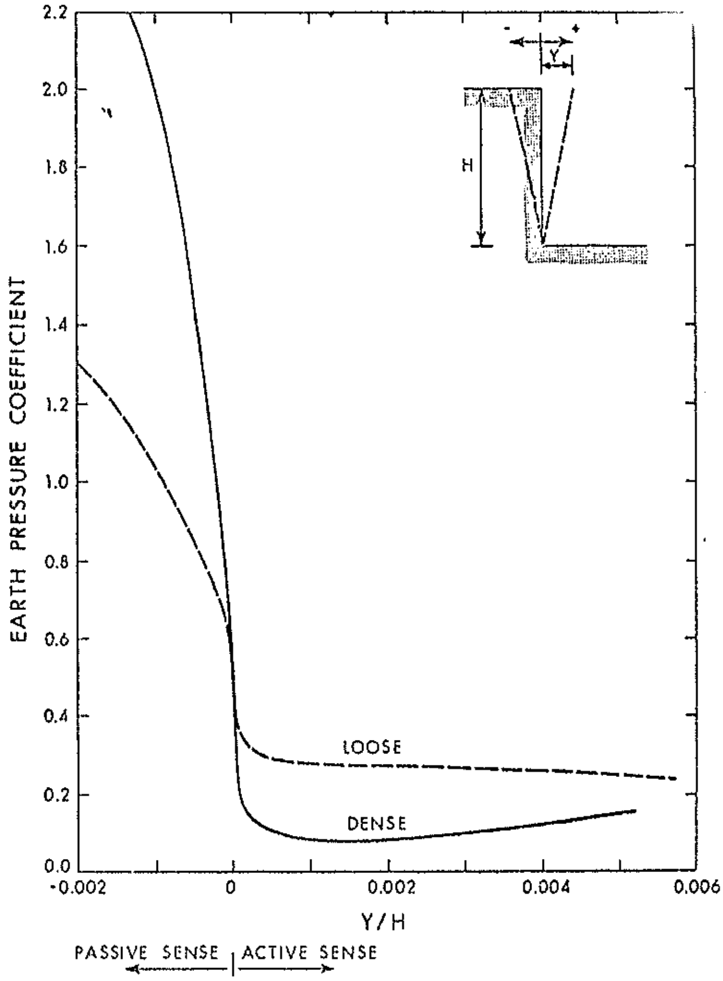

Figure 1

Relation between Movement of a Wall and Earth Pressure for Different Densities (after Terzaghi, 1954)

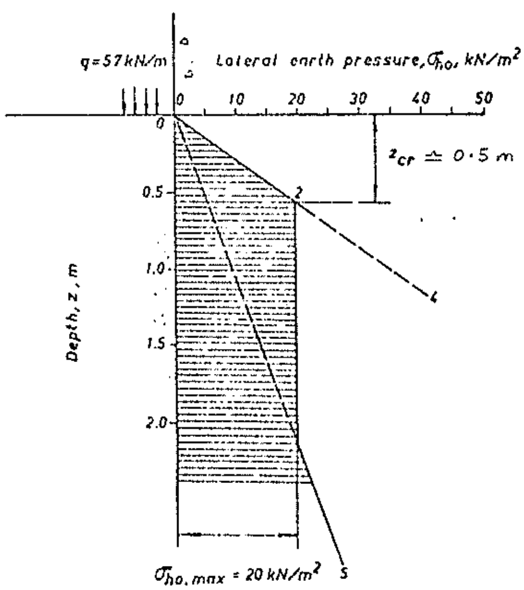

Fig. 2. Proposed design of earth pressure distribution

THE PRACTISING ENGINEER'S VIEWPOINT

G.A. PICKENS*
B.E., B.A., M.N.Z.I.E.

1. INTRODUCTION

The formal aspects of retaining wall design are well covered by the M.W.D. "Retaining Wall Design Notes". The writers of the "Notes" are to be commended for collating design information so comprehensively, and in particular for bringing into focus points such as bearing capacity under inclined and eccentric loads and the existence of at-rest earth pressures. With the design procedures so well covered by the "Notes", this presentation will attempt to demonstrate the engineering judgement which seems to play a large part in practice.

2. THE DESIGN EARTH PRESSURE

It is relatively easy to make a rational assessment of bearing capacity for wall foundations and to design the wall structure itself. The earth pressure, the most important factor in the whole business, is usually impossible to estimate with any degree of accuracy or confidence. How often does one encounter clean granular material with a predictable friction angle, as the medium retained?

Walls range from being underdesigned to almost superfluous. Some walls designed for active pressures are in fact subjected to at-rest pressures but do not fail because of the various factors of safety incorporated in design (e.g. safety against overturning, non-recognition of wall friction, high safety in reinforced concrete design). They may lean slightly but show no other signs of distress. In other cases the retaining wall serves only as a revetment preventing erosion and receives no earth pressure because the bank is naturally stable. At-rest pressures are likely to develop behind a wall which has limited capacity to yield, is founded in relatively stiff unyielding soil but which retains fairly plastic soil dipping towards the wall. Alternatively, materials such as welded tuffs and competent soft rocks may never exert any soil pressures on a facing wall. Where a wall is built close to a naturally competent or stable cut face, the maximum lateral pressure is likely to be the silo pressure generated by the backfill in the gap between wall and soil, and it is suggested that there are cases when this type of pressure distribution can be adopted.

3. CLAYS AND DRAINAGE

Most of our problems in assessing lateral pressures arise from our dealing with clays, steep cuts restricted by site boundaries and inability to place more than a narrow drainage course between the wall and the cut face. The natural, and sometimes unpredictable, soil beyond the sphere of construction governs soil pressures. If these cohesive soils could support tension then theoretically most cohesive banks would stand vertically without being

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retained. We know that in most situations such banks will fail. However, in some cases the "tension" does exist; a vertical face of plastic clay some 10 ft high is still stable below a roadway in Auckland, although an old retaining wall support of timbers and metal tie rods has long lost its structural strength. What design pressure should be taken for the replacement wall?

The reason for the stability of the above bank and many other lightly designed walls retaining clay is considered to be the low ground water level together with a surface protection against seasonal moisture changes which cause tension cracks. Water filled tension cracks initiated by clay soil shrinkage are a source of unusually high earth pressure behind walls retaining clay type soils. The shrinkage crack could occur immediately behind any permeable backfill zone so that the full water pressure is transmitted to the wall. A water-filled tension crack say 5 ft deep can be assumed as an extra design loading case. However, it is preferable to guard against this situation by adequate drainage where the problem is likely to be significant (See Fig. 1). Unexpected water pressures have affected many walls and the "Notes" do well to point out the vital importance of adequate drainage; the type of detail shown on Figs. 33 and 34 of the "Notes" is strongly supported.

4. CATERING FOR HIGH EARTH PRESSURES

In some instances, e.g. basement walls, design for at-rest pressures must be faced. Techniques can be adopted to reduce design pressures in some cases and two examples are described.

(i) A large retaining wall is founded on relatively stiff weathered Waitemata formations and retains very plastic Pleistocene clays which are expected to creep with time and build up high pressures against a wall of limited flexibility. A crib wall was selected because of its large capacity to deform. Furthermore the construction batter behind the wall was eased and the space backfilled with granular fill having predictable characteristics. Active earth pressures were assumed, based on characteristics of the backfill and the assumption was made that the wall type would be sufficiently flexible to yield continuously and thereby relieve potential higher pressures caused by soil creep.

(ii) A house builder cut a platform for his house near the foot of a steep hillside (see Fig. 2). The building inspector expressed just concern and requested a retaining wall designed by a registered engineer. The slope was underlain by competent sandstone exposed on the building platform but highly plastic weathered soil overlay the sandstone. The hill slope showed obvious signs of soil creep. A rigid wall would ultimately be subjected to extremely high pressures and would be very costly. A crib wall was recommended, again because of its flexibility, but no guarantee was given that the wall would not move within the lifetime of the house. The wall might tilt and move with time but it was judged that this risk and the potential costs of rebuilding the wall in the future were worth facing because of the large savings afforded by this type of wall.

As a footnote on high pressures, the opinion is offered that we should measure the at-rest pressure in-situ where the wall constitutes a significant structure. Techniques exist for doing this and the measurements can be made as part of the investigation. Alternatively, an approximation can be made using the methods outlined by Brooker & Ireland (Ref. 1). Their laboratory studies suggest relationships between plasticity index, overconsolidation ratio and coefficient of earth pressure at rest. By means of consolidation and index tests, the k_0 value can thus be estimated.

5. CRIB WALLS

The writer's experiences in estimating and allowing for earth pressures, particularly where the retained material is of clay type, have led to a personal preference for the crib type wall where the wall is free standing and there is adequate construction space. The crib wall is used extensively but it is contended that our knowledge of how it works as a structural unit is inadequate.

How many designers have satisfied themselves that commercially available units will withstand with safety the loads imposed on them? How critical is it if units crack and tensile reinforcement rusts away long term? Who can assess how horizontal shears are transmitted to foundation level? One method of analysis assumes silo pressures within the crib cells, and dependent on unit dimensions and infill properties, computations can show that the loads are transferred entirely to the intersection points of units. In this case the foundation is subjected to line loads and a 6 inch thick mesh reinforced slab is theoretically inadequate.

It is interesting to note that very few designers depart from the ¼:1 wall slope which probably developed from traditional road cutting batters plus the fact that it is impracticable to erect units at a steeper slope. Providing the unit proportions ensure that infill material is retained, a crib wall can be built at different slopes, even vertical, but the designer will be obliged to design and not use handy charts. It is held that wall slopes closer to the vertical than ¼:1 can sometimes give the least construction cost where there is a steep slope retained. Finally the writer considers that some design charts put out by crib unit manufacturers can be misleading in uninformed hands and that manufacturers should be discouraged from issuing such charts.

6. TEMPORARY RETAINING WALLS

Temporary retaining walls for construction purposes frequently give trouble. They tend to be underdesigned because of their temporary nature and also tend to be treated lightly for the same reason. Specifications most often place the entire responsibility for design on the contractor. The argument against the designer detailing temporary retaining structures is that the design is bound not to suit the contractor's approach and/or special equipment. The writer tested the latter claim for a job and was told by contractors that they never hold large timbers, steel beams etc. in stock.

It is suggested that it is time that the designer looked seriously at the temporary retaining structure where this is of significance. A design incorporated in documents would at least give a common basis for tender and force any bidder offering a design alternative to treat his alternative design as seriously as the original. Some caution should be exercised because unless there is strict site control, temporary works are frequently modified without any engineer's knowledge. It is also suggested that higher than normal design stresses could be allowed in ductile materials used for temporary wall construction. Should design stresses for temporary works be codified?

Ref. 1   Canadian Geotechnical Journal Vol II February 1965

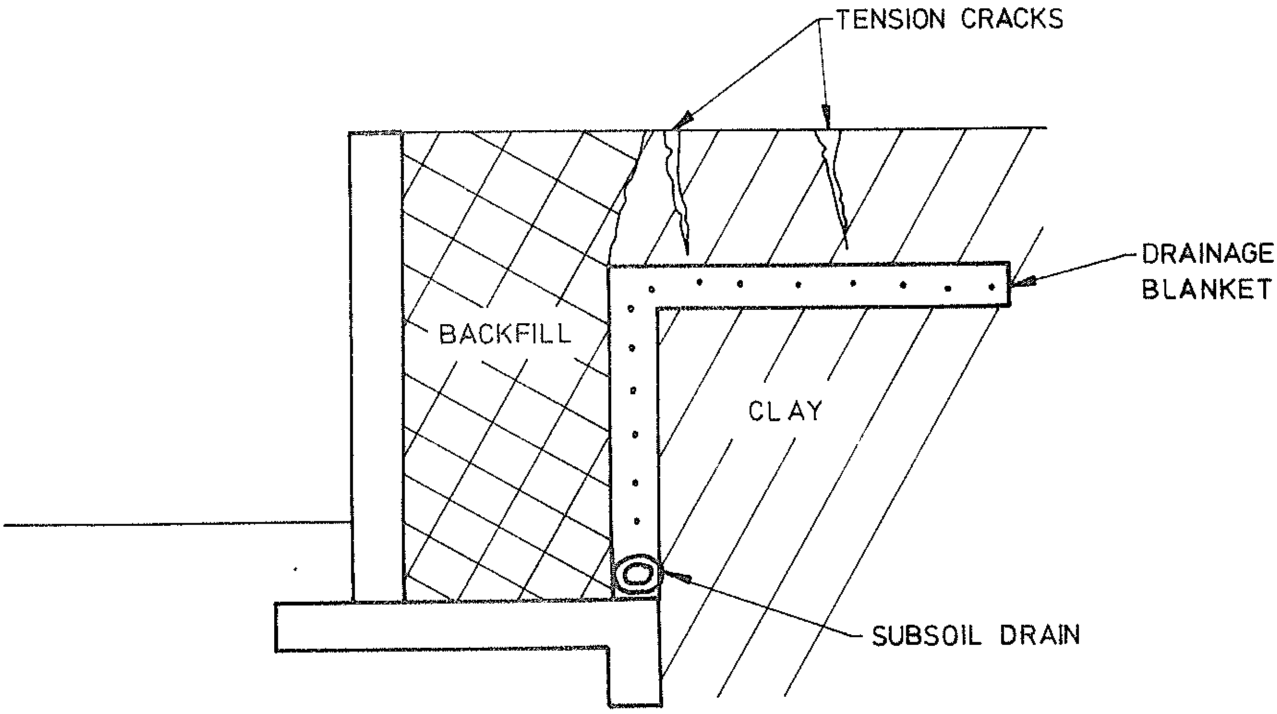
TENSION CRACKS
DRAINAGE
BLANKET
BACKFILL
CLAY
SUBSOIL DRAIN

Fig.1

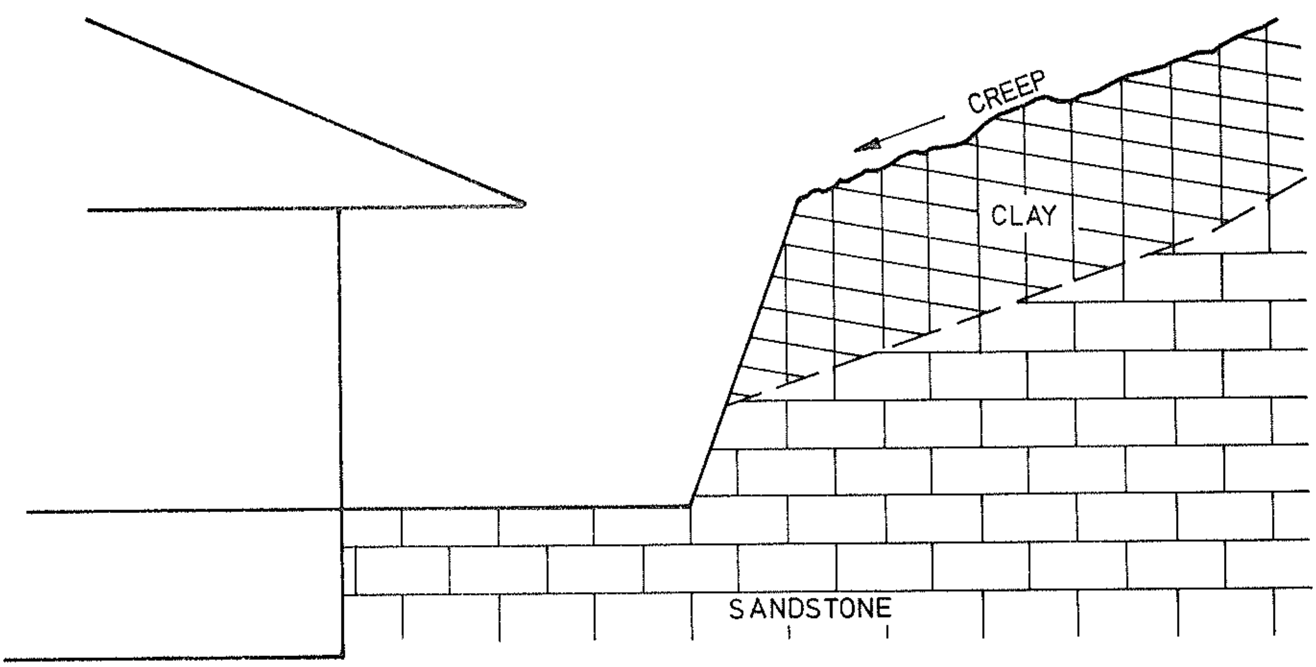
CREEP
CLAY
SANDSTONE

Fig.2

THE ACADEMIC VIEWPOINT

P.W. TAYLOR*
B.Sc., B.E., Ph.D., M.N.Z.I.E., M.I.C.E., M.ASCE

1. INTRODUCTION

Speaking last on the list of invited contributors has its disadvantages. It is likely that at this stage of events everything noteworthy will have been said and this is probably the case in this session. Also the author had been asked to comment from the academic viewpoint and the very word "academic" carries with it connotations of uselessness - something that is utterly impractical.

This paper will examine from a very simple viewpoint the design philosophy used for retaining walls, because this philosophy is unique in civil engineering structures. Whereas other structures are designed for the largest loads which could possibly be imposed, a retaining wall is designed for the smallest loads which can possibly be imposed. This point has been well covered in the M.O.W. Retaining Wall Design Notes, together with the consequences. In this paper the author will examine in a fundamental way the conventional method of designing for active pressures. The justification for this is that before the wall fails it must yield. If it yields the pressures will reduce to active values and therefore the wall can be designed for active pressures. This is the rather woolly basis on which retaining wall theories have been developed - in other words it is assumed that sufficient deflection can occur so that the active Rankine state may be attained and the wall can be designed for this active pressure.

2. COMPARISON OF DESIGN LOADS FOR A BRIDGE AND A RETAINING WALL

In Fig. 1 a comparison is made between the loads that a bridge and a retaining wall are designed for. For a bridge, the operating range is between no live load and some fraction of full live load. When this is compared with a retaining wall, the design load will be conventionally the active earth pressure. Now assume for simplicity that the wall is rigidly held while the backfilling is taking place. The pressure against the wall rigidly held will be earth pressure at rest, or if the fill is solidly compacted could be very considerably above earth pressure at rest. Assuming that initially the wall is rigidly held and then the props holding the wall rigid are removed, the loads will then reduce to some value which must be above the design load. In other words, the operating range will be typically above the design load as is shown on Fig. 1. Now this is not the usual situation in structural design and it means that the effective load factor (which for the bridge is the ratio between the collapse load and the operating load and is generally quite high) is likely to be very much smaller for the retaining wall even though the same factors of safety may have been used as for the bridge design. This suggests that either (as suggested in the M.O.W. Notes) a value of earth pressure should be used which is more likely to approximate the actual earth pressure, or else if the concept of designing to active pressure is to be adhered to, a much higher factor of safety should be assumed. The end result will be the same.

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However, the concept of holding a retaining wall rigid while a backfilling is placed behind it is not likely to pertain in practice. What is more likely is as shown on Fig. 2. If the fill is solidly compacted, the actual earth pressures may well exceed "at rest" values. Also, if the fill is cohesive, the pressure does not need to be zero at the surface. Fig. 2 (a) shows backfill only part way up the wall and before any significant deflection in the wall has occurred. If the compaction process is continued up to the top of the wall as shown on Fig. 2 (b), the actual earth pressure could vary as indicated on the diagram for a reasonably flexible type of wall with a fixed base.

3. RELATIONSHIP BETWEEN WALL MOVEMENT AND LOAD ON WALL

In Fig. 3 the relationship between horizontal load from the fill and wall movement is shown. This is a property characteristic of the fill. However, the load-deflection characteristics of the wall must also be considered and this can be plotted on the same diagram (i.e. wall resistance versus wall movement). The wall cannot provide any resistance until it has deflected to some extent, and the graph will have some characteristic slope. The design load and the collapse load of the wall are also plotted. The point where the two graphs cross will determine the actual load on the wall and movement of the wall. If the load-deflection characteristics of the wall are much more rigid than shown on Fig. 3, it may be possible that the collapse load may be reached before the wall has yielded sufficiently, i.e. the two graphs do not intersect and the wall fails. On the other hand, if the load-deflection characteristics of the wall are much more flexible, then it may be possible for the actual pressure on the wall to be reduced to active pressure although this would be rather unusual.

4. TIED BACK RETAINING WALLS

Two other items which the author would like to mention are both covered in a paper by L. Casagrande ("Comments On Conventional Design Of Retaining Structures" Jour. Soil Mechanics and Foundation Engineering Division ASCE, Vol. SM2, pp 181-198, Feb 1973).

The first item is the special consideration required for an anchored retaining wall. This topic is not included in the M.O.W. Notes. Retaining walls which are anchored or tied back near the top are becoming more common as construction techniques using earth or rock anchors improve. L. Casagrande, based on a study of a number of case histories, considers that except for uncompacted coarse granular backfill such as gravel or crushed stone, pressures on retaining walls tend to <u>increase with time</u> as a result of cyclic variations, such as variations in temperature, fluctuations of ground water level, vibrations from traffic or other sources. Casagrande suggests that this is a long term cumulative effect and that these minor fluctuations will lead to higher earth pressures over a period of time, with the exception of retaining walls supporting cohesionless coarse grained uncompacted material. He suggests that where fine grained backfill is used, designs should be based on "at rest" pressures (with the usual factor of safety) or else a factor of 2.5 if active pressures are used in design. The M.O.W. Notes (pages 12-13) cover this point very well.

With regard to the design of anchorages for tied retaining walls, Casagrande considers that designs based on currently accepted practice tend to yield excessively at anchorage level and that to prevent this, both the tie rods and the anchorages should be designed for not less than twice the conventionally calculated values.

However, typical soil mechanics practice has been to increase the calculated tie rod force by only 15-25% whereas Casagrande now considers increasing this force by 100%, and the reason for this is found in the shape of the curves shown on Fig. 4. Movement of a wall away from the fill leads to active pressures and only a small amount of movement is required. On the other hand, movement of an anchor towards the restraining soil leads to passive pressure but a very much greater amount of movement is required. Thus it has been suggested by Casagrande that in the design of anchorages, a very much higher factor of safety is necessary and hence his recommendation that the tie rod force be doubled.

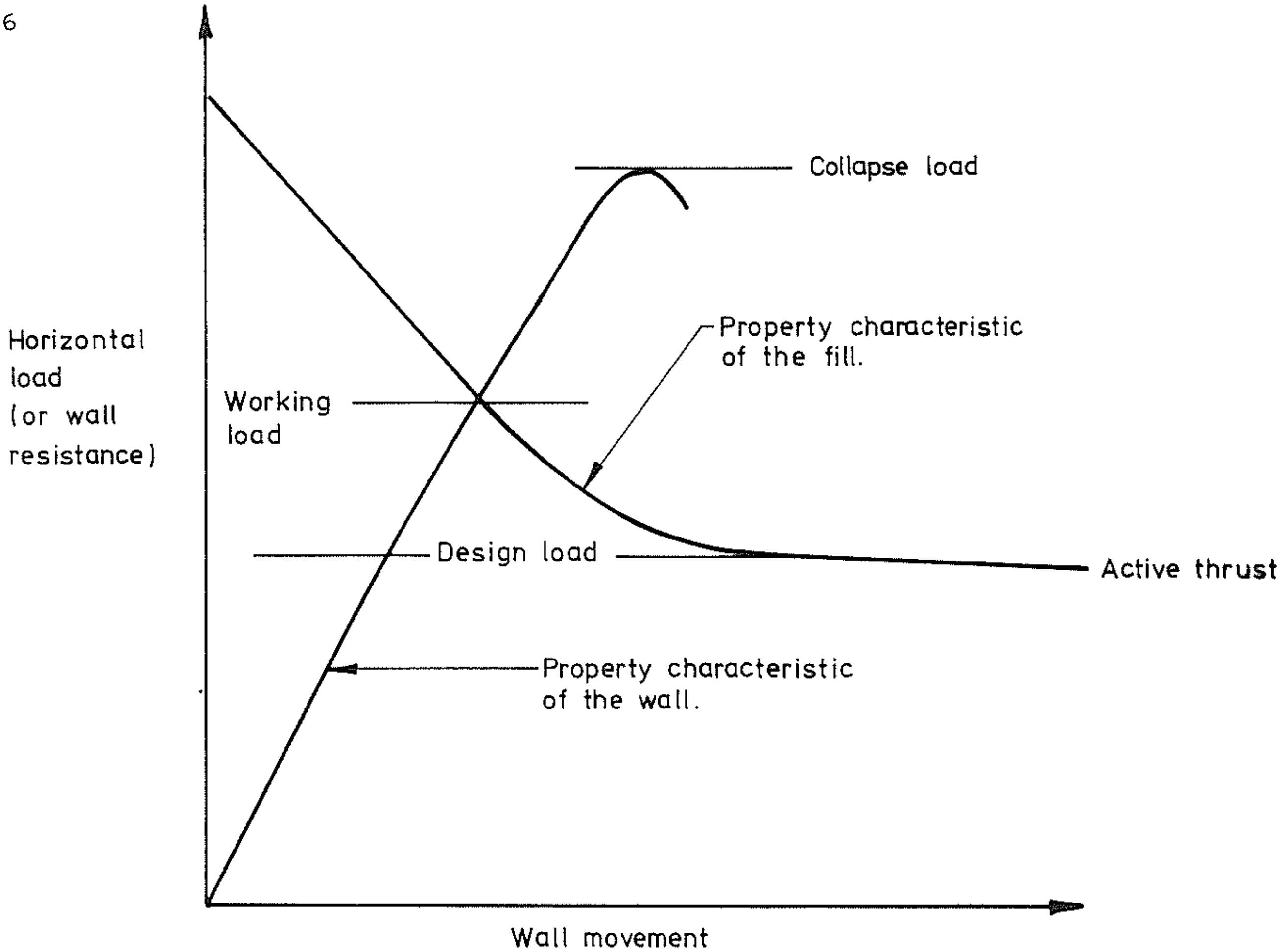
Collapse load
Horizontal load (or wall resistance)
Property characteristic of the fill.
Working load
Design load
Active thrust
Property characteristic of the wall.
Wall movement

Fig. 3

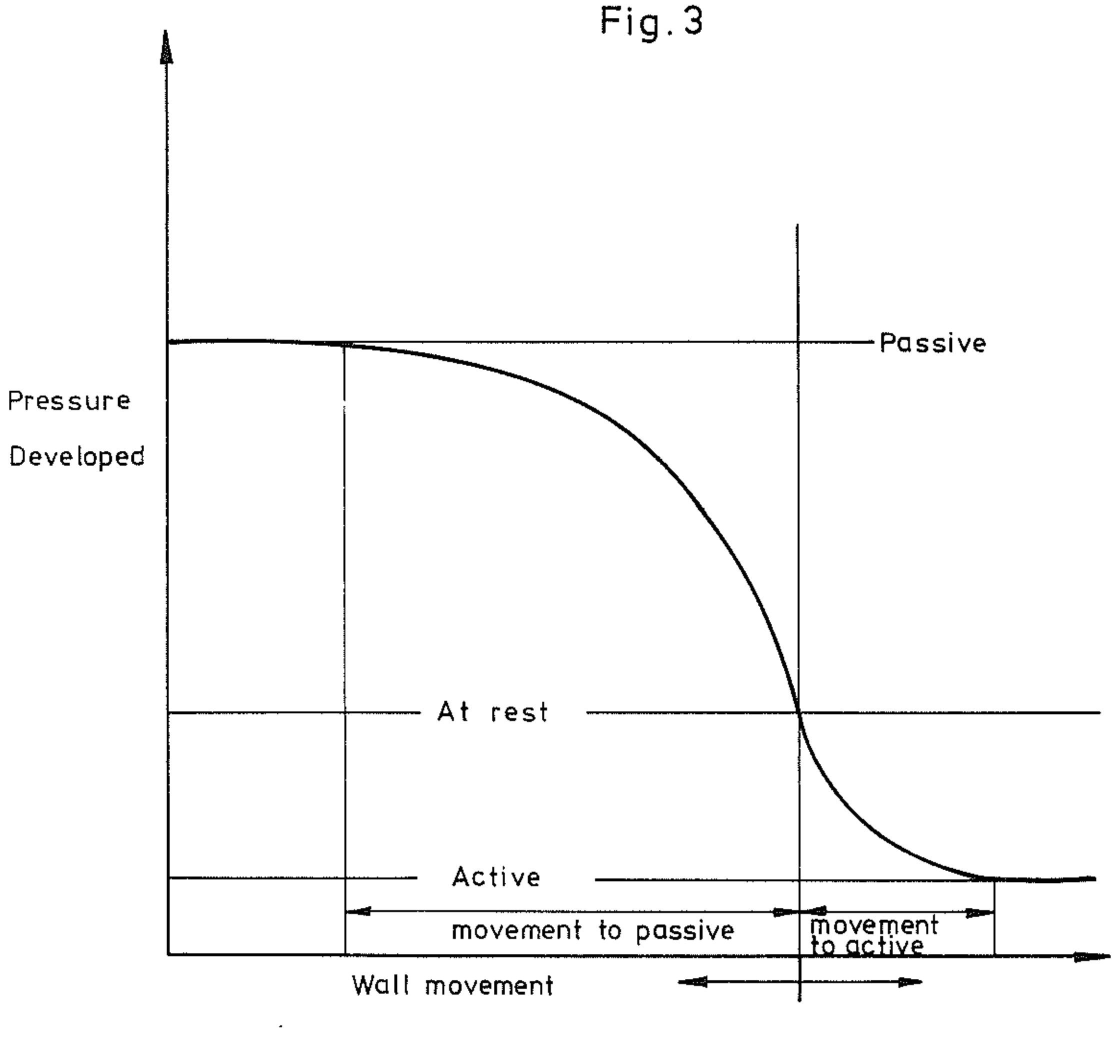
Passive
Pressure Developed
At rest
Active
movement to passive
movement to active
Wall movement

Fig. 4

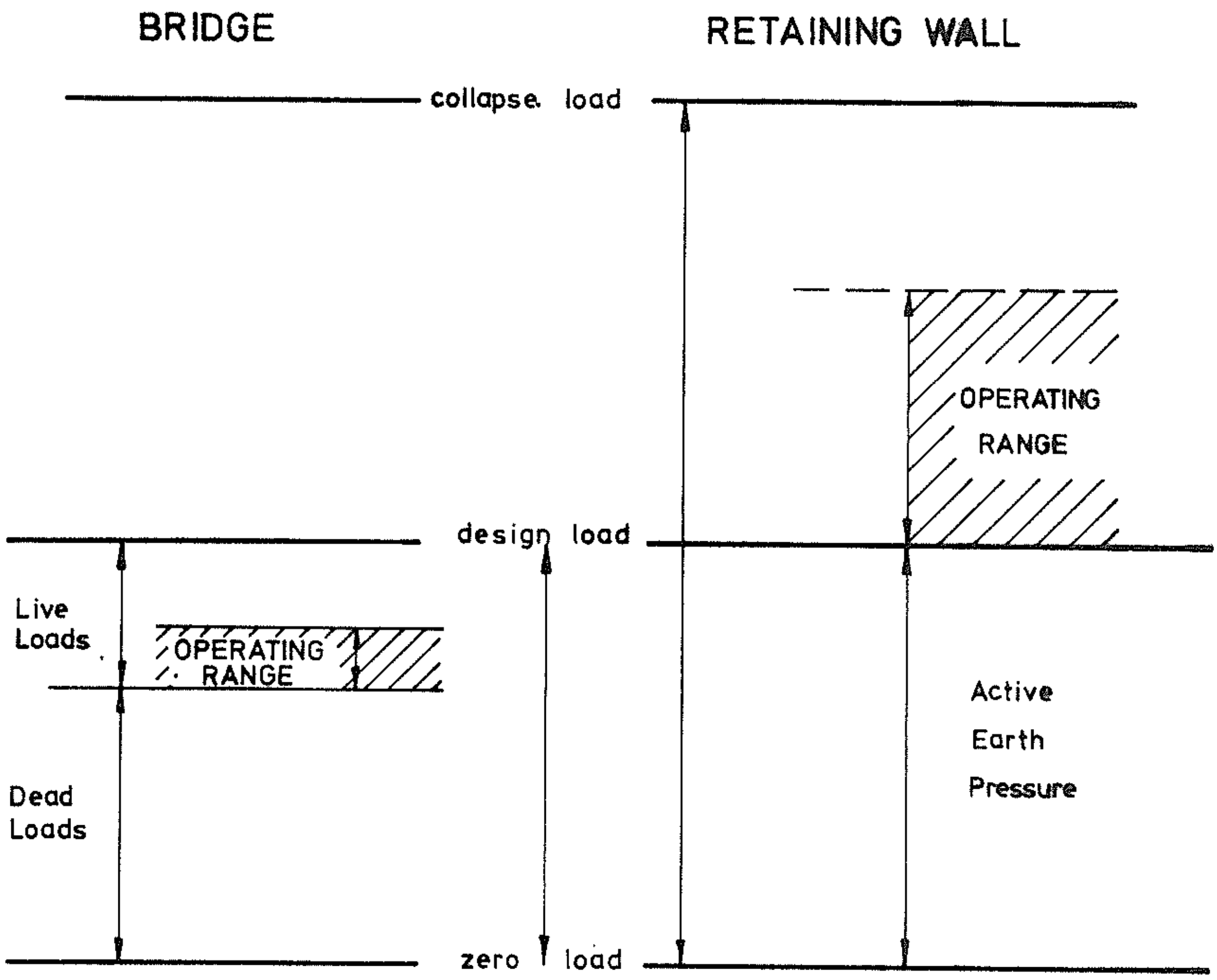
BRIDGE
RETAINING WALL
collapse load
OPERATING
RANGE
design load
Live
Loads
OPERATING
RANGE
Dead
Loads
Active
Earth
Pressure
zero load

Fig.1

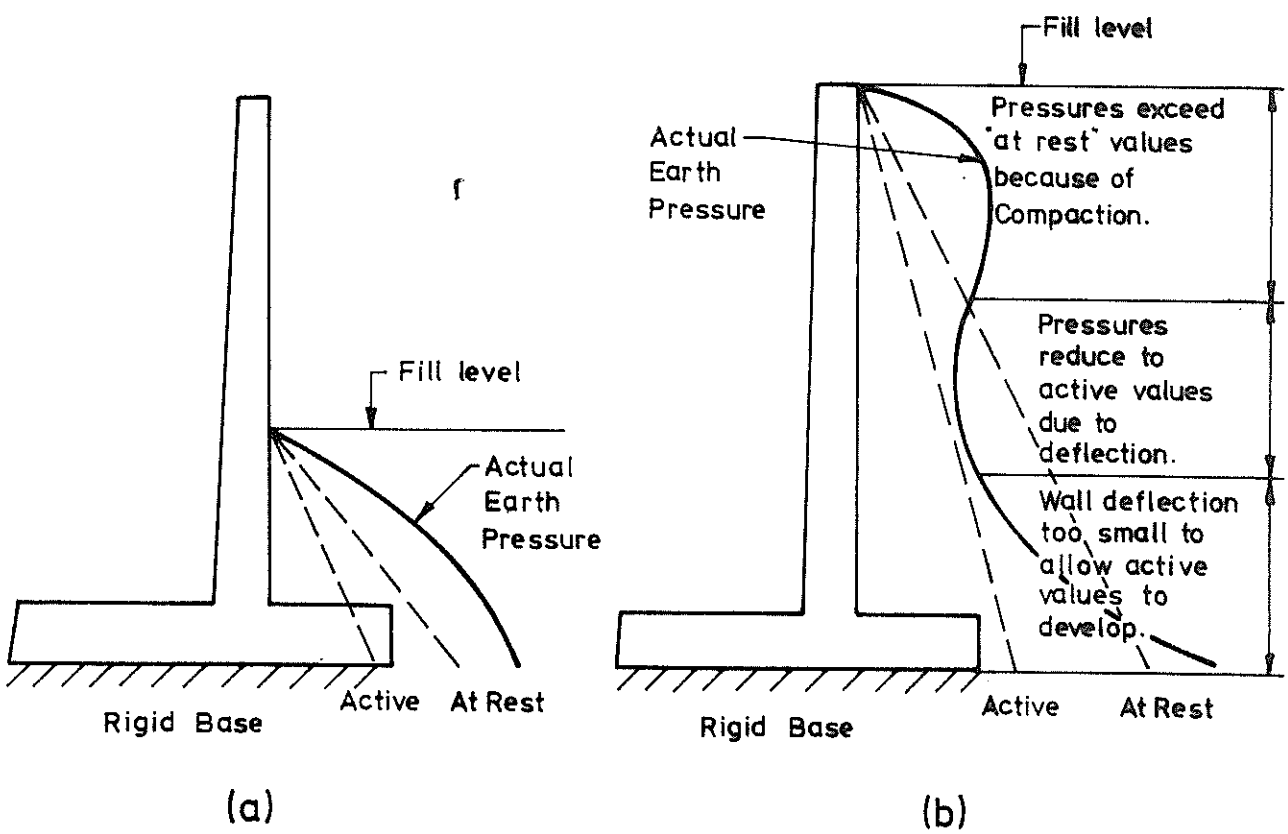
Fill level
Actual
Earth
Pressure
Rigid Base
Active
At Rest
(a)
Fill level
Actual
Earth
Pressure
Pressures exceed
"at rest" values
because of
Compaction.
Pressures
reduce to
active values
due to
deflection.
Wall deflection
too small to
allow active
values to
develop.
Rigid Base
Active
At Rest
(b)

Fig. 2

DISCUSSION

Following the presentation of papers the Chairman, Mr. J.H.H. Galloway, called for discussion on the presented papers and the Retaining Wall Design Notes.

MR. M.T. MITCHELL (Waikato Technical Institute, Hamilton)
spoke on several sections of the Retaining Wall Design Notes.

Section 8.1.3: If ultimate strength concrete design is used on retaining walls then accordingly the maximum earth pressure should be designed for.

Section 5: Discussing the effect of surcharge, Mr. Mitchell stated that differentiation should be made between the effects of surcharge on rigid and flexible retaining walls. With the flexible case, elastic relationships, are assumed to apply, while with the rigid case the elastic solution must be doubled due to mirror image effects.

Section 3.4.2: Where coarse grained soils were used with high compaction behind retaining walls, Mr. Mitchell thought it was possible to exceed k_o conditions (at rest pressures).

Mr. Mitchell also referred to the references of Casagrande and Tschebotarioff as quoted by Professor Taylor.

MR. J.C. RUTLEDGE (Ministry of Works and Development, Wellington)
referred to Casagrande's paper "Comments on Conventional Design of Retaining Structures" quoted by Professor Taylor. With an increase in the number of tied-back retaining walls being built in New Zealand, Mr. Rutledge said it should be noted that Casagrande was referring to anchored bulkhead type walls (such as back-filled sheet pile walls) with unstressed mild steel ties when he commented that tie loads could be substantially higher than assumed.

Mr. Rutledge said this was not the case with the tied-back walls being designed in New Zealand at present. These walls are placed against the country and held back by stressed high tensile ground anchors.

MR. J.W. PARNHAM (Wellington City Council)
spoke of creep of clay fill behind cribwalls and asked members present at the Workshop if this phenomenon had been experienced.

MR. RUTLEDGE showed slides of cribwalls which had been backfilled with clay. Invariably the wall units themselves (stretchers and headers) had failed, rather than soil creep occurring, due to high pore water pressures.

MR. D.M. COOK (Ministry of Works and Development, Twizel)
asked if any of the speakers had used finite element methods for design of walls and what faith they place in the analysis.

DR. J.H. WOOD (Ministry of Works and Development, Wellington)
said he had considerable faith in the analysis if the correct element types and mesh were used. Results compare well with exact analytical solution for simple cases if care is taken. Dr. Wood stressed the need to find and use the relevant soil properties. Without this data a finite element analysis was not worthwhile.

MR. P.G.M. IMRIE (Kingston, Reynolds, Thom and Allardice, Wellington)
said all the speakers had spoken of forces generated by backfilling behind walls and the relevant factors of safety. A special case is where retaining walls form part of a building basement and under earthquake conditions the walls transfer shear forces from the ground to the building. High pressures, approaching passive pressure levels, may well be reached. Mr. Imrie said a study he had carried out indicated that pressures may reach 1.25 γ H or approximately three times at-rest pressures.

MR. G. EVANS said in reply that this condition, with a retaining wall fixed at the top, was similar to that of bridge abutments.

DR. J.H. WOOD said that dynamic analysis of retaining walls to determine earth pressures under earthquake conditions were very complicated. Resonance of the wall and soil may be experienced.

MR. J.B.S. HUIZING (Ministry of Works and Development, Wellington)
spoke of the importance categories presented in Table 9 of the Retaining Wall Design Notes. Mr. Huizing was interested to receive comments from participants at the workshop regarding the suggested coefficients. If applied to many structures existing today, the minimum requirements might not be met. The seismic coefficients were used on the Wellington Urban Motorway walls where the designers had a particular task to accomplish.

MR. G.A. PICKENS (Tonkin and Taylor, Auckland)
replied that in his opinion walls were not generally integral parts of structures, and did not warrant expensive design. It may be more economical to have the walls fall down in an earthquake and repair them afterwards.

MR. HUIZING replied that importance category 2 of the Retaining Wall Design Notes covered that case. However, the Wellington Motorway would be vital in time of a major earthquake.

MR. D.K. TAYLOR (Tonkin and Taylor, Auckland)
said that Wellington was perhaps a special case in that a fault line existed alongside the motorway. In other areas earthquakes were not such real risks.

MR. HUIZING replied that the discussion generated was the type of comment desired on the "Notes".

Discussion followed from many participants. Generally, all agreed that the suggested coefficients were a positive attempt to provide design coefficients and in some cases should be treated as only minimum values.

MR. J.H.H. GALLOWAY (Ministry of Works and Development Central Laboratories, Lower Hutt)
Written discussion received is presented on the following page.

MR. J.C. RUTLEDGE spoke to Mr. Pickens paper and agreed that temporary support should be specified in contract documents to a greater extent than is common in New Zealand. The usual situation of leaving the standard of temporary support to the contractor to decide is unsatisfactory for the adjoining owners, for the client, and for the contractor. The case may arise where one contractor submits a low tender price (based on unsatisfactory temporary support) and be awarded the contract whereas a second contractor, because his price includes adequate temporary support, cannot be competitive.

Mr. Pickens mentioned providing detailed design and drawings for the temporary support. An alternative which the Ministry of Works and Development has included in a recent contract, with a large amount of temporary support, was to specify the design methods and standards (in a similar way to the Retaining Wall Design Notes) together with the soil parameters to be used in design. Tenderers were then asked to design their own temporary support to these standards. It was hoped that this would give tenderers more flexibility in using equipment and methods of construction that they preferred than would be the case if detailed drawings and specifications were provided.

MR. K.H. GILLESPIE (Brickell, Moss Rankine & Hill, Lower Hutt)
felt that the method of specifying temporary support, spoken of by Mr. Rutledge, where design methods and parameters had been specified, was not successful from the contractor's viewpoint. Mr. Gillespie had been involved with several contractors interested in tendering.

The Chairman closed the meeting at 5.30 p.m.

J.H.H. GALLOWAY (WRITTEN CONTRIBUTION)

Though in my opening remarks as chairman I suggested that Mr. Blakeley's fortuitous use of the cypher "Wabe" for the weight of the wedge abe in Fig. 1 of his paper (1) might have a looking-glass flavour, I fear that few of us had the courage to get into that mysterious area. Perhaps three slithy toves were glimpsed, but mostly we shuddered, shut our eyes and moved on. This, I feel, is a pity because it was certainly my hope that one or two would be slain. The gap between our ability to do calculation and to recognise the actual problems that exist in a particular case is still too large for comfort. For example, I do not recollect that anyone mentioned the problems of determining the properties of the backfill actually used on a particular job. Until these problems are tackled and the responsibility for properly specifying materials and techniques, and checking that backfilling is done correctly is taken seriously by the designer, it is little exaggeration to say that sophisticated calculations about the wall are a waste of time. Were cantilever walls proportioned on the basis of "base width = 1/3 H, stem thickness 1/8 H + 2", 0.5% steel" they would be of comparable overall accuracy.

One slithy tove that was brought to our notice was the question of earthquake-induced pressures on retaining walls, and Mr. Evans (2) suggested that much higher seismic factors than are usual should be used. He mentioned the things he had seen after the Inangahua earthquake and voiced the opinion that spreading of approach fills was a very important factor in bridge abutment distress. I also had the chance to make a very quick tour of the area shortly after the earthquake and came to much the same conclusion. My impression, and I only had a few hours in which to inspect the area, was that most distress was due, not to the bridges battering their abutments, but to the abutments being forced inexorably against the bridges. Piles had been damaged not by the bridge rocking to and fro, but by the backfill behind the abutments moving through them, and the wingwalls had been forced off culverts by the filling leaning very hard against them. I cannot see that these phenomena can be explained by seismic accelerations directly. It seems highly unlikely that the four bridges and two culverts that I saw should have been so oriented with respect to the earthquake accelerations that signs of distress always indicated movement along the structure. Further, why did the two large culverts which lost their wingwalls show no signs of structural distress in their bodies? Had the forces which broke the wingwalls been caused by liquefaction of the fill, the pressure on the culvert body would have crushed it like an eggshell! I am therefore forced to dismiss both "acceleration reversed" and "liquefaction" as dominant factors and so have to postulate something else. This could be a sort of "ratchet action". In this the wall momentarily accelerates away from the backfill leaving a gap into which the backfill moves, thus preventing the wall returning to its original position and raising the earth pressures somewhat. Further cycles of acceleration will compound the effect until either the wall fails structurally, starts to overturn, or slides on its base. The method of "failure" which will occur will depend on the circumstances of the case. Obviously the effect is only present in retaining structures with substantial flexibility with respect to the adjacent fill and thus spares the culvert body from large pressures. If this hypothesis is roughly true it implies designing bridge abutments to to withstand passive earth pressures, i.e. one order higher than commonly used. Alternatively, the abutment could be tied back into the fill with earth anchors (designed to resist maximum seismic acceleration of the abutment plus bridge) so that the fill always remains in intimate contact with it. Obviously prestressed anchors would not be appropriate and the abutment develops into a box type of adequate mass to resist passive earth pressures from behind by friction along the base. The resulting expense is very considerable and rarely warranted and in my report (3) I suggested that the abutments of the majority of bridges should be designed with little more than the usual lip service to seismic effects, and in view of the rarity of major earthquakes, just be written off when badly damaged. Only for a few particularly long or important bridges could the expense of proper antiseismic abutments be afforded.

An aspect of antiseismic design that was not dealt with, but which I believe in the earth pressure context is particularly important, is the state of things once the earthquakes shaking has ceased. With very little exaggeration it can be said that few structures are destroyed by earthquakes, but many are left in a condition where they destroy themselves. The Turnagain Heights area in Anchorage, Alaska, is an excellent demonstration of this statement. The earthquake shaking led to a dramatic loss of strength of soil layers underlying the area. The whole area then moved towards the unsupported seaward scarp and broke up in the process. Photographs (4) of the area clearly showed the abrupt edges of blocks and minor scarps, edges which would not have remained clearly defined had they been subjected to severe shaking. The implication is that for important earth and earth retaining structures it is not sufficient merely to consider their stability under high earthquake accelerations. One has also to consider the new conditions which may apply. Will there be a change in groundwater level or seepage pattern? Will drain have become choked? Will surface cracks allow the entry of stormwater? Will there be liquefied or greatly weakened layers? Such possibilities have to be evaluated and the safety of the structure checked under those conditions which appear relevant. This re-evaluation will also have to allow for the near certainty of significant seismic action (aftershocks) in conjunction with the other relevant conditions.

This leads to one problem that was actually given a brief airing. This was the question of classifying retaining walls into different categories of importance and applying different design standards to each. This is touched on in the "Design Notes" (5) and Mr. Huizing's request for comment on these suggestions provoked little reaction. Feeling that the matter was too important to pass over, I made a few quick comments from the Chair. These I would now like to amplify, as follows:

(a) I am greatly impressed by the method of Hollings (6) as a means of assessing the amount of effort that should be put into providing greater security. I believe that, applied thoughtfully, it will give good guidance in the purely commercial area. Where life or amenity are involved, monetary values have to be assigned to these and this raises difficulties. It is virtually impossible to assign these values publicly without raising policital red herrings as to what they should be, but "in camera" very good guidance can be obtained with essentially arbitrary values. One finds that one's instinctive horror at massive loss of life checks one from assigning very low values to an individual life, while figures of $100,000 or even $500,000 per life do not contribute much to the overall loss when the risk to life is carefully assessed. The reason is that this risk is, in statistical terms, normally infinitesimal.

Let us postulate a series of retaining wall failures which immediately lead to

(i) a massive release of radioactivity from a large nuclear power station;

(ii) failure of a major dam;

(iii) failure of a spillway structure leading in turn to the failure of a medium sized dam;

(iv) partial collapse of a multi-storey building;

(v) collapse of a modest sized railway bridge over water;

(vi) collapse of a medium sized railway bridge over water;

(vii) collapse of a major highway bridge over land.

Table 1 gives my assessment of the risk to life and direct loss involved in each of the above cases. Case histories are rare, but I think those quoted support the assessments. Other figures can be postulated but I do not think these will upset the absolute dominance of water as a risk to life in all cases except nuclear disaster. Consequential losses and costs could change the picture somewhat, but not drastically. I therefore suggest that only those retaining walls whose failure would precipitate a water disaster should be in the high risk category so far as loss of life is concerned.

) The second aspect concerns the retaining wall whose collapse could lead to the severing of a vital link. Such links could be a water supply line, electric power line, motorway, railway or major road. I am very sceptical of the word "vital". It is an emotive word and tends to stifle cool evaluation of the problem. One should never say "this feature is vital" till one has said "what will the situation be when this feature fails?". In the vast majority of cases it will be found that the feature is nowhere near "vital" and its failure will only lead to some inconvenience, economic loss and perhaps a few deaths.

If a link is vital, I suggest it should be dealt with in the following ways in order of preference.

(i) Its "vital" nature should be abolished by removing the particular dependency that makes it so (e.g. disperse a city over a wide area). This is only rarely economically (or politically) possible once the dependency has been created, but it is a powerful argument for wise control of development.

(ii) It should be secured by creating a mesh of alternative links. This is usually very attractive in economic terms once a major dependency exists. It can be politically acceptable too as many subsidiary benefits (e.g. better general communication, less congestion) accrue.

(iii) The dependent area can be provided with adequate stocks of resources to survive the loss of the vital link till it is restored. Such resources as food, fuel, medical supplies etc. are readily stockpiled but water resources, particularly for fire fighting, are often difficult.

(iv) It can be secured by more adequate design. In the case of retaining walls this is frequently utterly unacceptable as it often means that a retaining wall solution is inappropriate and sufficient land has to be taken to allow safe batter slopes or in a bridge abutment (see above) it could lead to an extra span at each end so that approach fills are avoided. My feeling is that "vital" retaining walls cannot really be secured merely by adopting higher seismic factors. The whole context of the problem has to be looked at.

) Any "vital" link is made up of a series of features the failure of any one of which could lead to severance or severe restriction of the link. To evaluate this type of problem, some of the techniques used to assess nuclear safety appear relevant (7). Essentially these involve assigning both a probability and an associated effect to all deficiencies that could conceivably develop in the link. For example, the collapse of a particular retaining wall could block three lanes of a four lane motorway (3/4 blockage). The probability of this happening in any one year is, say, one in one hundred. The "probable deficiency" in the link is thus 3/4 x 0.01 = 0.75% per annum. All other probable deficiencies are assessed in like manner and tabulated. This tabulation immediately shows up those features which contribute the bulk of the total probable deficiency, and thus what must be tackled to make significant improvement to the overall security of the link. Probabilities may be difficult to define but for retaining walls two criteria dominate. These are rainfall and earthquakes, and fortunately their probabilities of exceeding design values can be assessed with tolerable precision.

There is at least one case that requires special consideration. This is where a retaining wall or fill supports a pipeline or other facility which could release large quantities of liquid. In this case minor movement could cause leakage to develop. This leakage could give rise to fluid pressures in the fill which in turn aggravate the movement and lead to a rapidly worsening situation ending in collapse. This special case can, of course, be included in the probable deficiency tabulation and obviously will warrant special treatment. It is to be noted, however, that this arises not from either the pipeline or the feature below the wall being "vital", but from the especially self-aggravating nature of the content.

A useful extention of the technique, which I believe is new, would be to assess the time necessary to correct each probable deficiency and restore acceptable serivce. This time, multiplied by the probable deficiency would give a measure of the "probable loss of service". Tabulation of these values would produce a different picture of need. For example, washout of a culvert could lead to complete severance of a link, but adequate service could be restored within a week, and complete service within six months. On the other hand, seismic liquefaction and bulk movement of a natural sand bed could destroy 60% of a multi-span trestle type road bridge which would take five years to restore (or might even cause the whole link to be abandoned). Assigning a 3% annual probability to the washout and 0.5% annual probability to the liquefactic gives probable deficiencies of 3.0% per annum and 0.5% per annum respectively. However, the probable losses of service are:

3% per annum x 7 days = 0.21 days per annum for the washout, and 0.5% per annum x 5 years = 9.12 days per annum for the liquefaction. Thus the rarer liquefaction causes a much more serious probable loss of service and may have to be dealt with ahead of the much more frequent washout.

The voluminous literature on these probablistic techniques indicates that they provide very useful relative information and give clear guidance in finding weaknesses in a scheme. As with Hollings' method they run into serious red herrings if an attempt is made to establish public standards of risk, performance or loss of service. Only those used to disaster planning appear likely to be able to give useful guidance in establishing suitable standards.

One final point is the question of drainage and seepage behind retaining walls. There is a very simple minded idea current, that drainage behind a retaining wall abolishes seepage forces! This is, of course, but a pious hope. The most that a typical drainage system can do is to change the point of application and direction of the seepage force. Mostly these changes are beneficial, but this is not

necessarily so. The seepage forces remain very much alive and lie in wait for deficiency to develop in the drainage. Thus, the drains must remain effective for the life of the structure, and under all conditions. "All conditions" is a very sweeping context and includes such things as the effects of prolonged rainfall, the possibility of surface water getting into cracks, and changes in ground water levels, flows and pressures as a result of earthquake.
The latter can be very significant and last for appreciable times. Much of the massive landslip damage I saw at Inangahua I would attribute to such effects rather than to actual accelerations.

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TABLE 1

Case for definition (see text)	People Affected				Assessed Loss $M		Remarks
	Total	Killed %	Serious Injury %	Slightly Affected %	Facilities	Lives (@ $0.5M per life)	
(i)	10^3-10^5	30	30	40	5 x 10^3	15 x 10^2	Radiation deaths, damage and contamination
(ii)	10^3-10^4	20	10	70	5 x 10^2	250	c.f. Vaint & Malpasset; drowning and flood damage
(iii)	10^2-10^4	1	1	98	3 x 10^2	2.5	c.f. Baldwin Hills. Several hours' warning
(iv)	10^2-10^3	10	20	70	5	2	c.f. Hollings data. Ronan Point Flats
(v)	10^2-10^3	50	10	40	0.5	50	c.f. Tangiwai; drowning
(vi)	30-300	40	20	40	1	20	c.f. Silver Bay Bridge, U.S.A.; drownings
(vii)	30-300	5	25	70	1	0.5	Includes consequential collisions, c.f. multiple accidents in fog on U.K. motorways

Data Sources:

(i) Hypothetical - there are no case histories yet!
(ii) Vaiont: E.N. Record October 31, 1963; 2000 killed. Malpasset: Catalogue of Dam Disasters; Babb A.O.: U.S.B.R. 1968: 396 killed
(iii) Baldwin Hills: Civil Engineering (U.S.A.) Feb 1964: 5 killed
(iv) Collapse of Flats at Ronan Point, Canning Town, H.M.S.O. 1968: 4 killed
(v) Tangiwai Bridge: Encyclopaedia of New Zealand: 151 killed
(vi) Silver Bay Bridge: Facts on File 1967: 46 killed
(vii) Accident on M.1 at Watford Gap 18.3.1974: 5 dead, 36 injured, 150 vehicles (NZBC)
7 dead, ? injured, 160 vehicles (Evening Post 20.3.74)