SEISMIC DESIGN CONSIDERATIONS FOR GRAVITY RETAINING STRUCTURES[1]

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ABSTRACT

Major earthquakes have been observed to damage marginal wharf gravity retaining structures or walls throughout the world, and earthquakes of similar magnitude occur quite often in Canada. These earthquakes generally take place in the British Columbia coastal areas, the St. Lawrence Valley, Baffin Island and the Yukon Territory. Elsewhere, normally designed marginal retaining structures should have adequate factors of safety to withstand anticipated seismic loads. However, in earthquake-prone areas it is recommended that most marginal retaining structures be designed using the seismic coefficient method. While not all aspects of the suggested seismic design procedure are developed, detail is provided on the following: determination of the seismic exposure at the site; determination of the earth pressures due to earthquake conditions; and, recommended factors of safety for short-term conditions. Design aspects of a more specific geotechnical nature (soils investigation, liquefaction, remedial measures, tsunamis, backfill specifications) are indicated.

INTRODUCTION

Earthquakes of magnitude greater than seven have been observed to damage marginal wharf gravity retaining structures or walls throughout the world (Duke and Leeds 1963; Hayashi *et al.* 1966; Seed and Whitman 1970; Arno and McKinney 1973), and earthquakes of similar magnitude occur relatively often in Canada (Whitham *et al.* 1970). Seismic design considerations for gravity retaining structures are currently of particular interest for British Columbia Coastal areas and the St. Lawrence Valley near Quebec City. Both of these areas are seismically active with potentially high ground acceleration levels (Zone 3), and the site of considerable marine construction involving wharf expansions. Another aspect of concern is that these structures are often founded on loose or soft alluvial soils below the water table and backfilled with loose hydraulic fill. These materials have a tendency to liquefy during strong earthquake shaking with a consequent loss in bearing capacity and/or increase in earth pressure. Other contributing factors that also indicate the need to consider potential earthquake effects would include: (a) marginal retaining structures can be of substantial size as shown by the wall in Figure 1; (b) these structures can effectively steepen submarine slopes which may then be seismically unstable; (c) harbour areas may be subject to tsunamis after the earthquakes; and (d) the structures are relatively rigid so that the earthquake loadings cannot be readily absorbed through framework ductility.

At present, the National Building Code of Canada (NBC 1975) and associated design manuals do not provide sufficiently detailed guidelines on aspects such as local soil conditions, liquefaction, backfill pressures, tsunamis, short-term safety factors, etc. for the seismic design of special structures such as marginal retaining structures. In this paper, the available literature on earthquake damage to such structures is briefly reviewed and used in the development of general design guidelines. The design procedure involves a number of considerations: (a) determination of the seismic exposure at the site; (b) geotechnical investigation; (c) liquefaction analysis; (d) remedial measures to avoid potential liquefaction problems; (e) factors of safety against bearing capacity failure, sliding, overturning and deep stability for the design earthquake (seismic coefficient method); (f) tsunamis; (g) allowable short-term safety factors; and (h) backfill specifications. Topics (a), (e) and (g) which are of general interest are covered in full with design recommendations, while the other topics that are mainly of geotechnical interest will be considered more comprehensively in a companion paper for the Canadian Geotechnical Society. The complete report on this study (Thompson and Emery 1974) will be made available to the NRC Depository of Unpublished Data. Some of the general design procedures discussed may prove of assistance to those concerned with similar structures requiring seismic consideration.

DAMAGE TO MARGINAL WHARF STRUCTURES DURING EARTHQUAKES

The best documented information on damage to marginal wharf structures during earthquakes are for Chile in 1960 (Duke and Leeds 1963), Alaska in 1964 (Arno and McKinney 1973) and Niigata in the same year (Hayashi *et al.* 1966; Kawasumi 1968). The major modes of failure observed can be classified as:

(a) Tipping and/or sliding due to direct inertia effects;

(b) Tipping due to bearing capacity failures;

(c) Tipping and/or sliding due to liquefaction of the backfill;

(d) Tipping and/or sliding due to liquefaction or loss of strength of the bearing stratum;

(e) Loss of support due to submarine landslides or slippage of soft deposits, alluvial fans, etc.

(f) A wide range of combinations of the above;

(g) Tsunami effects alone, or in combination with damage listed above. The tsunami, if any, follows the earthquake shocks with a delay dependent on the epicentral distance; and,

(h) Delayed failures initiated by the earthquake damage such as increased settlements, wave action on weakened structures, after shocks, etc.

A notable quay wall failure occurred during the Chilean Earthquake of 1960 which had a magnitude of 8.4 on the Richter Scale (used throughout). This gravity retaining structure at Puerto Montt was founded on fine sand of medium density and the backfill was a very loose, saturated fine sand. The backfill material liquefied completely causing the wall to topple over (Duke and Leeds 1963). The Alaska Earthquake of 1964 with magnitude 8.3 resulted in extensive damage to marine structures due to large scale ground movements, and subsequent tsunamis. These ground movements were in the form of prolonged shaking which caused significant liquefaction of the soil (earthslides, subaqueous slides, subsidence, etc.). In general, the amount of direct seismic damage at each location was a function of: the properties of the soil; the geometry of the ground surface and soil layers; the depth of the water table; and, the intensity and duration of ground shaking. The tsunami from the Alaska Earthquake caused extensive damage to points as far distant as the head of the Alberni Canal on Vancouver Island and Crescent City, California (Arno and McKinney 1973). The 1964 earthquake at Niigata, Japan, occurred in an area containing many harbour facilities founded on loose granular soils and backfilled with hydraulic fill. The liquefaction of these soils during this magnitude 7.7 earthquake caused extensive damage along the waterfront that was subsequently aggravated by tsunamis effects (Hayashi *et al.* 1966; Kawasumi 1968). The typical failure and movements of marine structures for these three earthquakes are summarized in Table 1 together with some Canadian earthquakes.

The information available on earthquakes and damage to marginal retaining structures indicates that they may be damaged if the earthquake magnitude exceeds about seven. At least ten earthquakes of this magnitude have occurred in coastal regions of Canada during the last 100 years (Milne 1956, Hodgson 1946, 1964,1965). Consequently, it is concluded that the effects of earthquakes must be considered during the design of these walls in certain parts of Canada. Generally, it may be assumed that earthquakes of a damaging magnitude to marginal gravity walls will occur only in Zone 3 of the 1970 Seismic Zoning Map of Canada (Whitham *et al.* 1970). Elsewhere, the normal static factors of safety for foundation design should be sufficient to accommodate the dynamic loads from any anticipated seismic action. (Some parts of Zone 2 with poor soil conditions may require further consideration.) It should also be noted that while liquefaction has usually been associated with earthquakes in other countries, a re-examination of Canadian earthquake damage such as Campbell River, 1946, indicates that it has also been a problem here (Hodgson 1946).

GENERAL SEISMIC DESIGN PROCEDURE

The seismic exposure and geotechnical profile at the site are the prime design parameters. Other design information, which is a function of both seismic and normal (static) design criteria, includes: (a) the geometry, materials, function and location of the marginal gravity wall; (b) the backfill material to be used; (c) the static design procedure utilized; and (d) constraints such as tolerable movements during and after shaking (it is often assumed that movements of several inches are acceptable), surcharge loadings, submarine slopes and alluvial fans, possible fire hazards and water damage to goods, etc. Determination of the seismic exposure at the site involves an assessment of the available information on seismic activity in the area including, if possible, the anticipated ground acceleration, predominant period and duration of shaking. Pertinent local information, such as the topography and geotechnical profile, should then be examined and any amplification effects established. The geotechnical profile is also necessary for determining potential liquefaction conditions, the presence soft or sensitive clays, and the possibility of alluvial fan slips. Once the static and seismic design parameters are established, the specific steps in the design process can be followed:

(a) Determine any potential geotechnical problems, including liquefaction of the founding and backfill soils, decrease in strength of soft or sensitive clays, landslides in submarine slopes, etc.;

(b) Decide on required remedial measures if there are potential geotechnical problems;

(c) Determine the loads on the structure during the design earthquake, including the effects of the seismic forces on the horizontal and vertical pressures, and the resulting factors of safety for bearing, sliding and overturning;

(d) Establish the same factors of safety for tsunamis conditions if anticipated;

(e) Compare the calculated factors of safety with the reduced values which can be accepted during short-term earthquake loadings; and,

(f) Develop specifications for backfill materials to minimize any potential liquefaction problems.

The two basic ways for determining the seismic loading are by the seismic coefficient method or a dynamic analysis. While a dynamic analysis is more rigorous, it also requires more sophisticated techniques that are probably not warranted in most cases because of potential inaccuracies in design parameters and the rigid nature of marginal retaining structures. Consequently, the seismic coefficient method is probably more practical for gravity walls and has been adopted by countries such as Japan and India. For this study, the seismic coefficient method is used with the anticipated maximum horizontal and vertical components of the earthquake accelerations for the general design of marginal retaining structures, but a full dynamic analysis should be considered for very large structures that are somewhat more flexible and for which the potential design savings might be great.

SEISMIC EXPOSURE AT THE SITE

The first aspect to consider in any seismic analysis is whether the exposure at the site is a design factor, as earthquake activity is significant only in certain areas of Canada. The Seismic Zoning Map of Canada, 1970 (Whitham *et al.* 1970) adopted in the National Building Code of Canada (NBC 1975) and associated design manuals should be consulted first. Areas outside Zone 3 have a firm ground horizontal acceleration of less than six percent gravity (g) based on a return period of 100 years. An acceleration of less than six percent g has been observed to be lower than that required to induce liquefaction (greater than approximately 12 percent g for liquefaction is indicated by case histories) and generally the dynamic loadings for such acceleration levels can be accommodated within the normal factors of safety required for a static design (Seed and Idriss 1971; Seed and Whitman 1970). Consequently, a marginal gravity wall without seismic considerations should adequately withstand the anticipated earthquake effects in Zones 0 to 2 inclusive. In general, areas where seismic exposure may be significant (Zone 3) are the British Columbia coast, St. Lawrence Valley near Quebec City, Baffin Island and the Yukon Territory. It should be noted that Montreal and Ottawa are now in Zone 2, rather than Zone 3 as on earlier maps, even though these areas suffered at least two damaging earthquakes over 100 years ago (Hodgson 1965). Also, there is actually a transition between zones, there are confidence limits on zoning, and amplification can occur due to local soil conditions. For these reasons, sites in Zone 2 near the border with Zone 3 should be examined in more detail, particularly if amplification due to soil conditions is anticipated.

In areas of significant seismic activity, it is necessary to complete a full seismic exposure evaluation as outlined in Table 2. This detailed evaluation yields the following information on the design earthquake for the site:

(a) maximum anticipated firm ground and rock accelerations;

(b) predominant period of the underlying rock motion; and,

(c) duration of significant shaking.

Firm ground is considered to be very stiff clay, dense sand and gravel, or mixtures of these. Rock refers to bedrock, or extremely dense, rock-like strata. A check on the maximum anticipated firm ground acceleration is provided by examining the maximum anticipated underlying rock accelerations on the basis of active faults and/or sources of tectonic activity. The firm ground and underlying rock accelerations must be modified for potential amplification by less firm surface layers. A 100 year return period has been adopted as this represents a reasonable structural life, and the available Canadian seismic data is rather limited for generating longer return period levels (Whitham *et al.* 1970). These components of a detailed evaluation are explained in the following sections.

Anticipated Firm Ground Acceleration

While an initial estimate of the maximum anticipated firm ground acceleration for a 100 year return period, a_{100}, can be obtained from the very important work of Milne and Davenport (1969), it is now possible to obtain a more detailed seismic evaluation for the precise geographical location being considered. Based on their computerized catalogue of earthquake data and techniques essentially developed by Milne and Davenport (1969), the Division of Seismology, Earth Physics Branch, Department of Energy, Mines and Resources (Ottawa and Victoria), can supply the following information for a nominal fee:

(a) A table of the past earthquakes used to estimate the seismicity of the site (date, magnitude, latitude, longitude, distance, acceleration, intensity (MM)). Comments on major earthquakes, aftershocks, and known faults are usually also available. The acceleration amplitudes of the site are calculated for each earthquake using the appropriate attenuation equation.

(b) A table containing the predictions of firm ground accelerations and intensity at the site for various return periods (probability of acceleration being exceeded in one year, acceleration, intensity, equivalent return period). This data is given in Table 3 for Prince Rupert, B.C. The predictions are by extreme value statistics (Milne and Davenport 1969). A graphical presentation of these calculations is a plot of the fitted linear relationship:

[1] $$\log_e A = U - \frac{1}{a} (\log_e [-\log_e (P)])$$

where A is the acceleration amplitude in percent g having on the average a probability P of not being exceeded in one year and U (mode) and $\frac{1}{a}$ (slope) are the fitting parameters specifically related to the site. Figure 2 is such a graph for the Prince Rupert data for earthquakes with intensity greater than III in Table 4. Since the mode and slope provided by the Division of Seismology are determined directly from the list of earthquakes in the computer, they were used to develop Figure 2 rather than a new least square fit on the data in Table 4. The main concern in plotting the earthquakes is to determine significant events above the a_{100} prediction.

(c) A grid of predicted firm ground accelerations that can be used to develop contours for the region of the site as shown in Figure 3 for Prince Rupert.

The predictions of firm ground accelerations are considered to have confidence limits of about ±100 percent or a factor of 2 (Milne and Rogers 1972). For example, Table 3 and Figure 3 indicate an a_{100} (firm ground) of approximately 9.1 percent g for Prince Rupert with practical upper and lower limits of 18.2 percent g and 4.55 percent g. However, by checking major events, and particularly events that are greater than the a_{100} (firm ground) prediction, by the method given in the next section, it should be possible to improve these confidence limits on the design earthquake.

Anticipated Underlying Rock Acceleration

Seed *et al.* (1969) have presented a detailed procedure for determining rock motion characteristics during earthquakes (maximum acceleration, predominant period and duration), and this method has been applied by Khanna and Gadsby (1972) to determine the seismic exposure in Greater Vancouver including amplification effects and response spectra for structural analysis. The basic information required to assess the rock motion characteristics at a site is the distance to potential causative faults and the magnitude of earthquake that might occur on these faults. The position of active faults can sometimes be determined from tectonic maps, but the majority of earthquakes in Canada occur in regions where the exact tectonic processes are not well understood, and details of recent faulting are often obscured by deep water and/or sediments. Thus, the tectonic activity is often best related to the known centres of energy release based on the epicentres of past earthquakes. The magnitude of earthquake that might be anticipated at a fault is difficult to assess directly. In general, the higher magnitude earthquakes result from greater lengths of fault rupture (Okamoto 1973), but it is usually very difficult to estimate fault lengths. For this reason, it is usual to assign an earthquake magnitude to the fault based on maximum past events. It should be noted that magnitudes are generally considered accurate within about a quarter magnitude unit (Hodgson 1965).

Once the significant distance from the site to a zone of potential energy release and the magnitude of anticipated earthquake at that zone have been determined, it is necessary to attenuate the maximum acceleration for distance from the zone of energy release. Seed *et al.* (1969) have suggested the use of Figure 4 which is based on a number of suggested attenuation curves and procedures for earthquakes with a focal depth of 10 to 15 km. (There are also attenuation curves given by Milne and Davenport (1969) for firm ground accelerations, but not underlying rock accelerations.) Figure 4 is entered directly to give the maximum anticipated underlying rock acceleration at the site. For instance, considering Prince Rupert, the major activity is along the Queen Charlotte Faults which are approximately 75 miles to the west (Douglas, 1962). A reasonable earthquake magnitude to associate with these major faults would be 8±1/4 M since the Queen Charlotte Island Earthquake of 1964 was of this size. Extrapolating from this data in Figure 4 yields a maximum anticipated underlying rock acceleration, a (rock), of 9.4 percent g at Prince Rupert.

Anticipated Predominant Period of the Underlying Rock Motion

For amplification studies, it is necessary to estimate the range of predominant periods of rock motion at the site from the significant distance to a zone of energy release and the magnitude of anticipated earthquakes at that zone. Seed *et al.* (1969) have developed the curves in Figure 5 that provide reasonable average values for assessing predominant rock periods. For instance, the predominant period of underlying rock motion for the Prince Rupert design earthquake discussed in the previous section (distance of 75 miles and magnitude of 8.25 M) would be approximately 0.64 seconds. However, a range of earthquake magnitudes and corresponding predominant periods should be developed for use in amplification studies since potential amplification effects are a function of the predominant period.

Duration of Significant Shaking

For liquefaction potential studies it is necessary to estimate the duration of strong shaking for earthquakes of various magnitudes so that the number of significant cycles of shear stress can be determined. Seed *et al.* (1969) have adopted values suggested by Housner (1965) and these are shown in Table 5 along with the representative numbers of significant cycles given by Seed and Idriss (1971). The duration of strong shaking is the time for which the earthquake effects are potentially damaging and the number of significant cycles is the number of large amplitude peaks during the strong shaking.

Amplification by Less Firm Surface Layers

The firm ground and underlying rock accelerations must be modified for potential amplification by less firm surface layers. The simplest way of introducing surface layer effects is through empirical values such as those given in Table 6 from the National Building Code of Canada (NBC 1975), or those suggested by Newmark and Hall in Table 7 (Whitman 1970). Implicit in these site factors is the fact that most estimated ground movements are based on firm ground. It is always difficult to determine from the literature what is considered firm or soft. However, it is usual to consider the shear wave velocity, V_s, as the best indicator with shear wave velocities greater than 1000 meters/second taken to represent rock or rock-like materials, and velocities greater than about 300 to 400 meters/second to represent firm ground. Shear wave velocities for typical soils taken from Okamoto (1973) are given in Table 8, and Figure 6 shows how the shear wave velocity can be related to the N value from the Standard Penetration Test (Ohsaki 1969). It is possible to obtain more detailed shear wave data for a particular site through a geophysical evaluation.

A more detailed evaluation of amplification, and particularly the possibility of quasi resonance, can be made by considering shear waves propagating vertically upward through a surface layer or layers. When the surface layer is comprised of a horizontal single layer of uniform nature the predominant period of the ground, T_G, is given by:

$$[2] \qquad T_G = \frac{4H}{V_s}$$

where H is the thickness of the surface layer, and V_s the shear wave velocity. In the field, the ground seldom consists of a single layer and there will be a number of strata, each having its own properties. However, when there is not much difference between the properties, the predominant period may be determined by using the equivalent shear wave velocity, V_s, obtained from (Okamoto 1973):

$$[3] \qquad \frac{H}{V_s} = \sum \frac{H_i}{V_{si}}$$

where: H is the overall thickness of the surface layer;
H_i is the thickness of a constituent layer i; and,
V_{si} is the shear wave velocity within that layer.

In order to express the surface layer amplification of accelerations quantitatively, the expression developed by Karai (Okamoto 1973) on the basis of theoretical calculations and measurements is adopted:

$$[4] \qquad G(T) = 1 + \frac{1}{\sqrt{\left[\frac{1+k}{1-k}\left\{1 - \left(\frac{T}{T_G}\right)^2\right\}\right]^2 + \left(\frac{0.3}{\sqrt{T_G}} \times \frac{T}{T_G}\right)^2}}$$

and $\qquad k = \frac{\gamma_1 V_{s1}}{\gamma_2 V_{s2}}$

where: $G(T)$ is the amplification factor;

T is the predominant period of the underlying rock motion (or firm ground motion);

T_G is the predominant period of the surface layer;

γ_1 is the density of the surface layer;

γ_2 is the density of the underlying rock (or firm ground);

V_{s1} is the shear wave velocity in the surface layer; and,

V_{s2} is the shear wave velocity in the underlying rock (or firm ground).

By using Equations 2 and 4 it is possible to estimate the amplification due to a soft layer on top of firm ground, or the amplification due to a layer of firm ground on rock, etc.

The critical point to watch for is quasi resonance when $T \simeq T_G$ since this can result in amplifications of two to five depending on the values of T and T_G. Except for this case, the factors given in Table 7 are fairly realistic and generally conservative. There are more detailed methods available such as wave reflection programs or finite element programs (Khanna and Gadsby 1972). However, these programs require detailed soil properties and boundary conditions so that it is debatable if their use to develop a response spectrum is warranted except in a full dynamic analysis.

Anticipated Maximum Vertical Acceleration

While the above discussion has focussed on the horizontal acceleration to be used in the seismic design, it is also very important to recognize the effect of vertical components of earthquake motions on the seismic stability of marginal retaining structures. There is not much data available on what vertical acceleration to adopt, but Okamoto (1973) and design codes that consider vertical accelerations indicate a value of one-half the horizontal acceleration should be taken. In the vicinity of the epicentre, surface waves can contribute significant vertical accelerations. However, the design of any structure at, or very near, a significant fault requires a full dynamic analysis and this is not being considered here. The vertical acceleration will be taken as one-half the horizontal acceleration for the seismic design procedure.

Previous Experience

Valuable design data can often be gathered for a new structure from a detailed examination of structures that have been subjected to earthquakes at a close or similar site. Some data on factors such as amplification, slides, liquefaction, etc. for Canadian earthquakes is available in the references discussed in earlier sections.

GEOTECHNICAL INVESTIGATION AND LIQUEFACTION

The significant features of an investigation to determine the geotechnical conditions for a seismic design are similar to those for a static analysis, except that zones of loose sands and silts become more critical since they may be liquefied during strong seismic activity having a long duration. Specific details of the required investigation will not be given here, but the major parameters that are required to make an estimate of potential soil strength losses during an earthquake are: (a) the relative density of sands or silts; (b) the grain size distribution; and (c) the sensitivity of clays. Sensitive clays may require higher safety factors to be used.

Liquefaction of loose sands and silts, either as a general condition or in lenses, has the potential of creating the most problems in the seismic design of marginal retaining structures. There is now sufficient information available to consider liquefaction problems during the seismic design (Seed and Idriss 1967; Seed 1968; Whitman 1971; Finn 1972; Okamoto 1973). If the founding soil has the potential to liquefy during the design earthquake, remedial measures will be required. It is also advisable to prevent the backfill from liquefying, although it is possible to design the structure to resist the pressures from the liquefied soils. Further details on how liquefaction is considered in the seismic design of marginal retaining structures will be reserved for a companion paper.

EARTH PRESSURES DUE TO EARTHQUAKES

A survey of the literature and building codes for countries such as India and Japan which have regulations governing the seismic design of gravity retaining walls, reveals that the usual design approach is to apply the seismic coefficient to the static Coulomb wedge theory (Seed and Whitman 1970; Okamoto 1973; International Association for Earthquake Engineering 1970, Japan Society of Civil Engineers 1968). The National Building Code of Canada (NBC 1975) does not, however, make any provision for the design of dynamic earth pressures on marginal gravity retaining structures. Seed and Whitman (1970) have presented a state-of-the-art report on the design of simple gravity retaining structures for dynamic loads that utilizes the Mononobe-Okabe analysis. The general methods that have been adopted in the earthquake resistant design of gravity retaining structures have the following common features:

(a) The seismic coefficient method is used where the horizontal seismic coefficient, K_h, is based on specific code values. These code values are largely empirical and reflect a number of parameters such as the seismic region, importance of the structure, and subsoil conditions. Where the vertical seismic coefficient, K_v, is adopted it is taken as one-half the horizontal seismic coefficient.

(b) The resultant seismic coefficient is modified below the water table to give an apparent seismic coefficient.

(c) It is assumed that the prime purpose of the earthquake resistant design is to prevent costly damage or large movements. Structures which cannot tolerate appreciable distortions should be designed to restrain the distortions within themselves rather than trying to design for minimal earth movements.

(d) The Mononobe-Okabe analysis for dynamic lateral earth pressures, or a similar analysis is adopted. Experimental data indicates that this analysis gives reasonable values for the dynamic lateral earth pressure, but that the point of application of the dynamic portion of the earth pressure should be higher than the point of application for static earth pressures.

(e) The vertical earthquake acceleration is not considered in determining the dynamic lateral earth pressure. This is equivalent to taking $K_v = 0$.

(f) The dynamic water pressure in the backfill is not considered independently, since this is already included when the apparent seismic coefficient is used below the water table. This is based on the assumption of combined movement of the water and soil mass.

(g) The dynamic pressure of water in front of the wall is not taken into consideration. The water level at the front of the wall is usually taken at its lowest value and the water level in the backfill at its highest value.

(h) The water and/or soil contained in any cell of the structure is considered to be part of the mass of the cell.

The seismic coefficient method is recommended for the dynamic analysis of marginal retaining structures. During an earthquake, in addition to the gravitational force, the seismic force acts upon the mass as shown in Figure 7. For the case shown in Figure 7a, only the horizontal seismic force exists. Cases where the vertical seismic force acts downwards and upwards are shown in Figure 7b and 7c (Okamoto 1973). (It should be noted that since they are inertial forces, the seismic forces act in the opposite direction to the earthquake accelerations producing them.) Thus the resultant force, R, and its direction, θ, are given by:

[5] $$R = mg\sqrt{K_h^2 + (1 \pm K_v)^2}$$

[6] and $$\tan\theta = \frac{K_h}{1 \pm K_v} = K$$

where K is the resultant seismic coefficient, and the plus sign on K_v indicates a seismic force downward. Since R is a static force, the earthquake forces have been represented by a change in gravitational force from mg to R, and an inclination of the horizontal plane by the angle θ. From Figure 7 and Equation 6 it can be seen that the effect of K_v on the seismic loading will be small in comparison to the effect of K_h and becomes even less as K_h increases.

When the body is submerged in water, such as rock or soil in a backfill below the ground water table, the weight of the body is reduced by the amount of buoyancy in the water. The resultant force, R, and its direction, θ, are now given by:

$$[7] \qquad R = mg\sqrt{\left(1 - \frac{\gamma_w}{\gamma} \pm K_v\right)^2 + K_h^2}$$

$$[8] \text{ and } \qquad \tan\theta = \frac{K_h}{1 - \frac{\gamma_w}{\gamma} \pm K_v} = K'$$

where K' is the apparent seismic coefficient;
γ_w is the unit weight of water; and,
γ is the bulk unit weight of the soil.

From Equation 8 it is clear that the apparent seismic coefficient for a body of small total bulk weight takes on a large value. Equations 7 and 8 are also based on the simplified assumption that relative movement of water and soil particles during earthquakes is prevented by frictional resistance of the soil particles.

Active Earth Pressures for a Dry Backfill

The reason for starting with a dry backfill is to clarify the differences in calculations above and below the water table. For the case of the gravity retaining wall with a sloping dry backfill, shown in Figure 8, the active earth pressure, p_a, at any depth, h, is:

$$[9] \qquad p_a = (1 \pm K_v)\, C_a \gamma h$$

where:

$$C_a = \frac{\cos^2 (\phi-\theta-\psi)}{\cos\theta \cos^2\psi \cos(\delta+\psi+\theta) \left[1 + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-\theta_0)}{\cos(\delta+\psi+\theta)\cos(\psi-\theta_0)}}\right]^2} \qquad [10]$$

and: p_a = active earth pressure at depth h (includes both the static and seismic pressures);

γ = bulk unit weight of the backfill material;

h = depth from top of the structure;

θ = $\tan^{-1}K$ from Equation 6 for the three values of K_v +, -, and zero;

ϕ = angle of shearing resistance of the backfill material;

ψ = angle of inclination of the back wall;

θ_0 = angle of inclination of the backfill surface; and,

δ = angle of wall friction.

The total active force, P_a, is given by:

$$P_a = (1 \pm K_v)\, C_a \gamma \frac{H^2}{2} \qquad [11]$$

where H is the vertical height of the wall. The point of application of the active force, P_a, would be $^2/_3$ from the top of the wall. However, research indicates that a more correct distribution of the active force would result if the dynamic increment of the total active force (i.e., total active force less the static active force) is applied $^1/_3$ from the top of the wall, and the static active force $^2/_3$ from the top of the wall.

The active earth pressure, p_a (static plus seismic), at any depth, h, due to the surcharge load, q, shown in Figure 9 is given by:

$$[12] \qquad p_a = \frac{(1 \pm K_v)\cos\psi}{\cos(\psi - \theta_0)} C_a q$$

where: C_a is given by Equation 10; and,

q is the surcharge load per unit area of the inclined surface.

The total active earth force, P_a, due to the surcharge is given by:

$$[13] \qquad P_a = \frac{(1 \pm K_v)\cos\psi}{\cos(\psi - \theta_0)} C_a qH$$

and is considered to be applied 1/2 way up the wall.

The simple case of a vertical wall with a horizontal ground surface and a surcharge load of q is shown in Figure 10. For this case, the active earth pressure, p_a (static plus seismic), at any depth, h, is given by:

$$[14] \qquad p_a = (1 \pm K_v)\ (q + \gamma h)\ C_a$$

where:

$$[15] \qquad C_a = \frac{\cos^2(\phi - \theta)}{\cos\theta\ \cos(\delta + \theta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\ \sin(\phi - \theta)}{\cos(\delta + \theta)}}\right]^2}$$

Active Earth Pressures with Ground Water Table

The three examples considered up to this point have involved a dry backfill. For the case shown in Figure 11, which is more representative of marginal gravity retaining structures, the ground water table is at some depth, h_t, in the backfill. The method recommended for handling this problem is also indicated on Figure 11. The active earth pressure, p_{at}, at the ground water table and above in the backfill is calculated using Equation 9 and the resultant seismic coefficient (i.e. $\tan^{-1} \theta = K$). For the backfill above the ground water table, the bulk unit weight, γ, is used. The active earth pressure, p_{ab}, at the bottom of the wall is then calculated using Equation 9 and the apparent seismic coefficient (i.e. $\tan^{-1} \theta = K'$) which reflects the buoyancy. The buoyant unit weight, γ_b, is used below the ground water table. Then, the distribution of active earth pressure under water is obtained by joining the value of the active effective earth pressure, p_{ab}, at the bottom of the wall to the active earth pressure, p_{at}, at the ground water table, as indicated in Figure 11.

For any computation involving the vertical seismic coefficient, it will be necessary to consider the three cases of $K_v=0, +K_v$, and $-K_v$ to determine which gives critical values for use in the various steps of the stability analysis. For instance, the critical value of K_v for sliding may not be the same as for bearing capacity. As indicated earlier, most design codes consider $K_v=0$ for gravity retaining structures. However, this may not be adequate for such large walls, and all three cases of K_v should be examined.

In addition to the earth pressures acting on the marginal retaining structure during an earthquake, it is necessary to consider seismic loadings due to the mass of the structure itself. This will again involve checking the three cases of K_v=0,+K_v, and -K_v for the critical values. For this purpose, the water and/or soil contained in any cell of the structure is considered to be part of the mass of the cell. The dynamic pressure of water in front of the wall is not taken into consideration. However, it is usual to consider the water level in the backfill at its highest value. Special loads that may contribute to the instability of the structure must be included such as seismic forces due to: cranes; surcharge loads on the structure itself (surcharges on the backfill such as piles of materials or heavy buildings are included in the earth pressure calculation); and, heavy transportation equipment.

Pressure Due to Liquefied Backfill

It may be necessary on some occasions to design the gravity retaining wall to resist the earth pressures due to liquefied backfill, even though it is considered normally preferable to stop this from happening. Upon liquefaction, the backfill material will behave like a heavy fluid to the depth of liquefaction. Below the depth of liquefaction, the procedures for computing the earth pressures given in the last section are applicable. Once liquefaction occurs, the gravity retaining structure functions much like a dam holding back a heavy fluid with a unit weight equal to the bulk unit weight, γ, of the backfill soil. The design procedure is then similar to that for a dam, where the dam in this case consists of the gravity retaining structure and any rock fill placed as a berm behind the wall.

The Westergaard theory for dynamic pressures on the face of a concrete dam during earthquakes can be used to get the dynamic pressure distribution for the liquefied backfill, as shown in Figure 12 (Seed and Whitman 1970). The earth pressures p_L and p_S, at any depth, h, are given by:

[16] $$p_L = \frac{7}{8} K_h \gamma (hH)^{1/2}$$

[17] and $$p_S = \gamma h$$

where:

p_L = dynamic earth pressure due to liquefied backfill at depth h;

p_S = static liquefied earth pressure at depth h;

K_h = horizontal seismic coefficient (vertical accelerations not considered);

γ = bulk unit weight of the liquefied backfill material; (typically 100 to 110 lb/cu.ft.);

h = height below the surface; and,

H = total height of liquefied zone (assumed to be the height of the wall in Figure 12).

The total liquefied fill pressures, P_L and P_S are then

[18] $$P_L = \frac{7}{12} K_h \gamma H^2$$

[19] $$P_S = \frac{1}{2} \gamma H^2$$

where P_L is the total dynamic force, and P_S is the total static force due to the liquefied backfill. The total force on the wall due to the liquefied backfill is thus:

[20] $$P_{LS} = \frac{7}{12} K_h \gamma H^2 + \frac{1}{2} \gamma H^2$$

P_L is taken to act 3/5 from the top of the wall and P_S to act 2/3 from the top of the wall. From Equation 20 it can be seen that the liquefied fill results in extremely large earth pressures.

FACTORS OF SAFETY FOR BEARING CAPACITY, SLIDING, OVERTURNING AND DEEP STABILITY

After determining the forces applied by the backfill to the structure during the design earthquake, the factors of safety of the marginal retaining structure against bearing capacity, sliding, overturning and deep stability are calculated in the same way as for static conditions. The stability of slopes created or steepened by the construction of the structure must also be examined.

While tsunamis have not been considered here, they may influence the design. Wiegel (1970) has provided a state-of-the-art paper and a comprehensive list of references that can be used to consider tsunamis and their effects. It should be noted that the travel time of tsunamis is significantly longer than that of seismic waves. This is very important as it allows the effects of tsunamis to be considered separately from the direct seismic effects.

It is a generally accepted procedure to reduce the normal factors of safety for combinations of loadings that are unlikely to occur simultaneously and for which the tolerable distortions can be fairly large. An effective reduction in the factor of safety for seismic conditions, by allowing a greater allowable bearing capacity (or by applying a reduction factor to design loads), is adopted in a number of countries as indicated in Table 9. Since there is a great variation in the values adopted, it is suggested that the criteria applied by a country such as Japan in which there has been considerable experience with the seismic design of marine structures should be used as the basis for reduced factors of safety in Canada (Japan Society of Civil Engineers 1970). The basic Japanese reductions for gravity quay walls and foundations are given in Table 10 along with the recommended reductions in more generalized terms.

APPLICATION

While it is not possible to give a complete design here, typical results for the marginal wharf retaining wall shown in Figure 1 are given in Table 11. In this case, the pressures have been calculated for a site near Prince Rupert, B.C., and for a hypothetical backfill that can liquefy. For the actual site, a_{100} (firm ground) was found from Figure 3 to be 9.5 percent g, so that K_h is 0.095 and K_v takes the values ±0.0475 or 0. From the non-liquefied backfill cases, it can be seen that the vertical acceleration has little influence. The drastic reduction in the factor of safety when the backfill is allowed to liquefy indicates the importance of avoiding this condition for economic designs. While the design methods adopted for tsunamis have not been detailed, this can also be a severe condition due to the large unbalanced water pressures that develop.

CONCLUSION

The seismic design of marginal wharf gravity retaining structures involves a number of steps, and it has only been possible to cover adequately the determination of seismic exposure, earth pressures developed during shaking and allowable short-term safety factors here. Geotechnical considerations involving the soils investigation, liquefaction, remedial measures and backfill specifications, and the consideration of tsunamis are also important elements in the design that must be considered.

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TABLE 1 Typical failures and movements of marine structures

Earthquake and Date	Magnitude	Harbour and Distance from epicentre	Damage	Approximate Movement
Gulf Islands, BC 11 Jan. 1909[1]	VI (MM)	Anacortes 15 miles	Piers damaged.	-
Estevan, BC 6 Dec. 1918[2]	7.0	Uclulet 90 miles	Several weak piles broken.	-
Graham Island, BC 26 May 1929[1]	7.0	Epicentral area	Beach fissures and slip (liquefaction?).	-
Campbell River, BC 22 June 1946[3]	7.3	Kildonan 70 miles	Alluvial fan slipped damaging seaward end of pile supported plant.	-
		Maple Guard Spit 45 miles	Piles distorted (liquefaction?).	-
		Comox Lake 25 miles	Mooring pile popped out (liquefaction?).	-
		Goose Spit 25 miles	Jetty sagged and substantial skidway damaged (liquefaction?).	-
Chile 22 May 1960[4]	8.4	Puerto Montt 70 miles	Complete overturning of gravity walls.	15 feet
			Outward movement of anchored bulkheads.	2 to 3 feet
Alaska 27 March 1964[5]	8.3	Achorage 76 miles	Pile supported dock almost completely destroyed.	6 feet
		Seward 80 miles	Circular-cell sheet pile, bulkhead collapsed.	-
			Other waterfront facilities almost totally destroyed.	-
Niigata 16 June 1964[6]	7.5	Niigata 32 miles	Tilting of gravity wall.	10 feet
			Outward movement of anchored bulkheads.	1 to 7 feet
			Settlement of revetment.	6 feet

[1] Milne 1956.
[2] Denison 1919.
[3] Hodgson 1946.
[4] Duke and Leeds 1963; Seed and Whitman 1970; this reference also contains illustrations and photographs of typical failures and movements.
[5] Arno and McKinney 1973.
[6] Hayashi *et al* 1966; Seed and Whitman 1970.

TABLE 2 SEISMIC EXPOSURE EVALUATION

Information Required	How Obtained	Application
1. General seismic exposure at the site - Zone 0,1,2 or 3 (approximate maximum anticipated *firm ground* acceleration for a 100 year return period).	New Seismic Zoning Map of Canada, 1970. (Whitham *et al* 1970), or National Building Code 1975 (NBC 1975).	To determine if a full evaluation is required. Full Evaluation required for Zone 2[1] near border with Zone 3 and Zone 3 sites.
2. Maximum anticipated *firm ground* acceleration for a 100 year return period.	(a) General data from Milne and Davenport, 1969. or, preferably (b) Detailed data for the site provided by the Earth Physics Branch.	Amplification studies, liquefaction potential, bearing pressure calculations, stability against sliding, overturning and landslides.[2]
3. Maximum anticipated underlying *rock* acceleration.	From distance to significant faults on which earthquakes of a certain magnitude might be generated. Figure 4 then used.[3]	Same as 2 above. This method basically provides a check on 2(b) above, and the underlying *rock* accelerations for amplification studies.[2]
4. Predominant period of underlying rock motion.	Same as 3 above. Figure 5 then used.[3]	Amplification studies. The predominant period is considered to be that at which the spectral acceleration is a maximum. Estimate only.
5. Duration of shaking.	From magnitude of earthquake that might be anticipated. Table 5 then used.[3]	Liquefaction potential. Estimate only.

[1]Main concern is with potential amplification at sites in Zone 2 near border with Zone 3.

[2]Procedures 2 and 3 are used together to determine a *firm ground* or *rock* acceleration and are then used in conjunction with Procedures 4 and 5, and knowledge of the geotechnical conditions, to determine the amplification (actual acceleration at the structure) and the liquefaction potential of the soil.

[3]Seed *et al*. 1969.

TABLE 3 Predicted firm ground accelerations for Prince Rupert

Probability of acceleration being exceeded in one year	Acceleration percent g	Intensity (MM)	Equivalent return period, years
0.333	0	II	3.
0.100	1	IV	10.
0.033	2	VI	30.
0.020	4	VI	50.
0.010	9	VII	100.
0.005	20	VIII	200.
0.003	33	IX	300.
0.001	133	XI	1000.

TABLE 4 Plotting positions for earthquakes with Intensity > III, Prince Rupert[1]

Year	Intensity MM	A_m[2] %g	$\log_e A_m$	m	$P_m = \frac{m}{N+1}$	$-\log_e(-\log_e[\frac{m}{N+1}])$
-	≤ III	0	-	1-64	-	-
1900	IV	1	0	65	.891	2.22
1956	IV	1	0	66	.905	2.35
1958	IV	1	0	67	.919	2.52
1899	V	1	0	68	.932	2.69
1945	V	1	0	69	.946	2.92
1929	V	2	0.69	70	.960	3.22
1948	V	2	0.69	71	.974	3.65
1949	VII	7	1.94	72	.988	4.43

[1]Earthquakes between 1899 and 1970 inclusive, 72 year period (N=72). Least squares fit supplied by Division of Seismology:

$$\log_e A = -3.12 + 1.16\ (-\log_e[-\log_e(P)])$$

Mode = -3.12

Slope = 1.16

[2]Accelerations provided in whole percent g units.

TABLE 5 Duration of strong shaking[1]

Earthquake magnitude M	Duration of strong shaking, seconds	Number of Significant cycles
8	50 or more	30
7.5	not given	20
7	25 to 30	10
6	15	not given
5	5	not given

[1]Seed *et al.* 1969.

TABLE 6 Foundation factors[1]

Type and depth of soil	F
Rock, dense and very dense coarse-grained soils, very stiff and hard fine grained soils; compact coarse-grained soils and firm and stiff fine-grained soils from 0 to 50 ft deep.	1.0
Compact coarse-grained soils, firm and stiff fine-grained soils with a depth greater than 50 ft; very loose and loose and soft fine-grained soils from 0 to 50 ft deep.	1.3
Very loose and loose coarse-grained soils, and very soft and soft fine-grained soils with depths greater than 50 ft.	1.5

[1]NBC 1975.

TABLE 7 Newmark's site factors[1]

Site condition	Factor
Soft ground	1.5
Firm ground: soft rock	1.0
Hard rock	0.67

[1]Whitman 1970.

TABLE 8 Shear wave velocities[1]

Soil	Shear wave velocities, V_s, meters/second
Sand	60
Reclaimed land	100
Sandy clay	100-200
Clay	250
Sand-bearing gravel	300-400
Moist sand	340
Gravel	1000 or above

[1]Okamoto 1973.

TABLE 9 Increase in allowable bearing capacity for short term considerations[1]

Country	Short term allowable bearing capacity / Long term allowable bearing capacity	
Algeria	Rocks	3
	General soil conditions	2
	Saturated loose soils	1
Argentina	1.25	
Canada	Dead plus live plus earthquake	1.33
	Dead plus live plus earthquake plus temperature, settlement distortion, etc.	1.5
Germany (West)	1.50	
Greece	1.50	
India	Having a bearing pressure greater than $45t/m^2$	1.5
	Having a bearing pressure greater than $20t/m^2$ and equal to or less than $45t/m^2$	1.3
	Having a bearing pressure greater than $10t/m^2$ and equal to or less than $20t/m^2$	1.0-1.3
Japan	2.0	
Portugal	2.0	

[1]International Association for Earthquake Engineering 1970.

TABLE 10 Recommended reductions in factors of safety for loadings during earthquakes

Stability Conditions under consideration	Reduction in Factor of Safety: Japanese Practice[1]	Reduction in Factor of Safety: Recommended for Design of Marginal Wharf Gravity Retaining Structures
Bearing Capacity	50% of Static.	50% of Static resulting in value of 1.5 except for sensitive clays which increases to 2, to reflect potential loss of strength from shaking.
Sliding	From 1.2 for static to 1.0 for seismic.	80% of Static (with a minimum of 1.0)
Overturning	The eccentricity of the loading may increase from 1/6 of base width for static to 1/3 for seismic.	The eccentricity of the loading may increase from 1/6 of base width for static to 1/3 for seismic.
Slope Stability	From 1.3 for static to 1.0 for seismic	80% of Static (with a minimum of 1.0)

[1]The Japan Society of Civil Engineers 1968.

TABLE 11 Typical calculated and allowable stability values for the retaining wall in Figure 1[1]

Loading Condition	Factor of Safety Against Sliding		Eccentricity		Average Effective Bearing Pressure
	Calculated	Acceptable (from Table 10)	Calculated	Acceptable (from Table 10)	
Static	1.86	1.5	0.04	0.167	5.8
Non-Liquefied Backfill K_v+	1.05	1.2	0.18	0.33	9.2
Non-Liquefied Backfill K_v-	1.06	1.2	0.17	0.33	8.4
Non-Liquefied Backfill K_v0	1.04	1.2	0.17	0.33	8.6
Liquefied Backfill K_v0	0.48	1.2	0.55	0.33	∞
Tsunami	0.76	1.2	0.34	0.33	22.4

[1]In this case, the pressures have been calculated for a site at Prince Rupert, B.C. and for a hypothetical backfill.

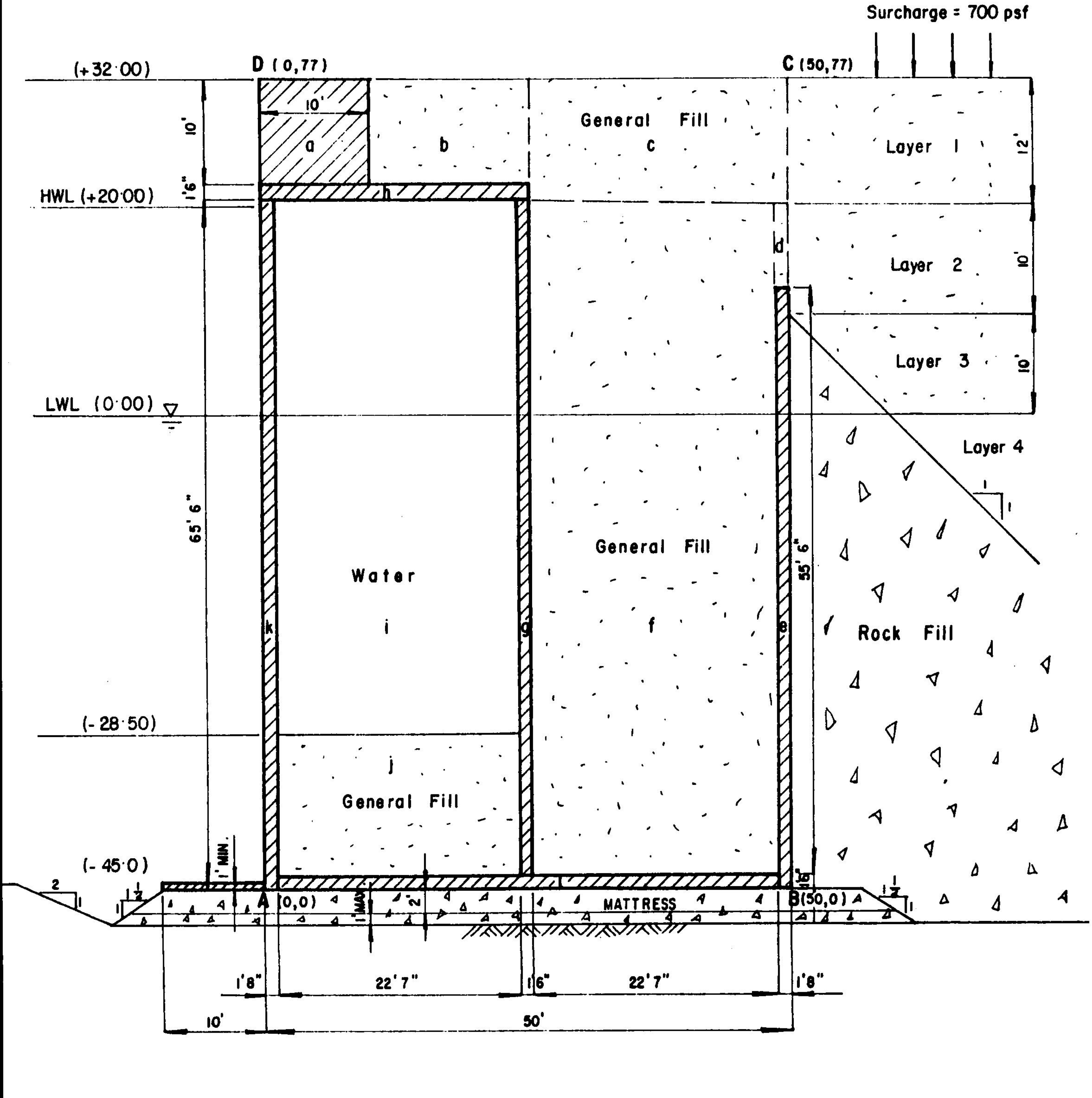


FIG. 1 ASSUMED CROSS SECTION OF MARGINAL WHARF RETAINING WALL AND BACKFILL

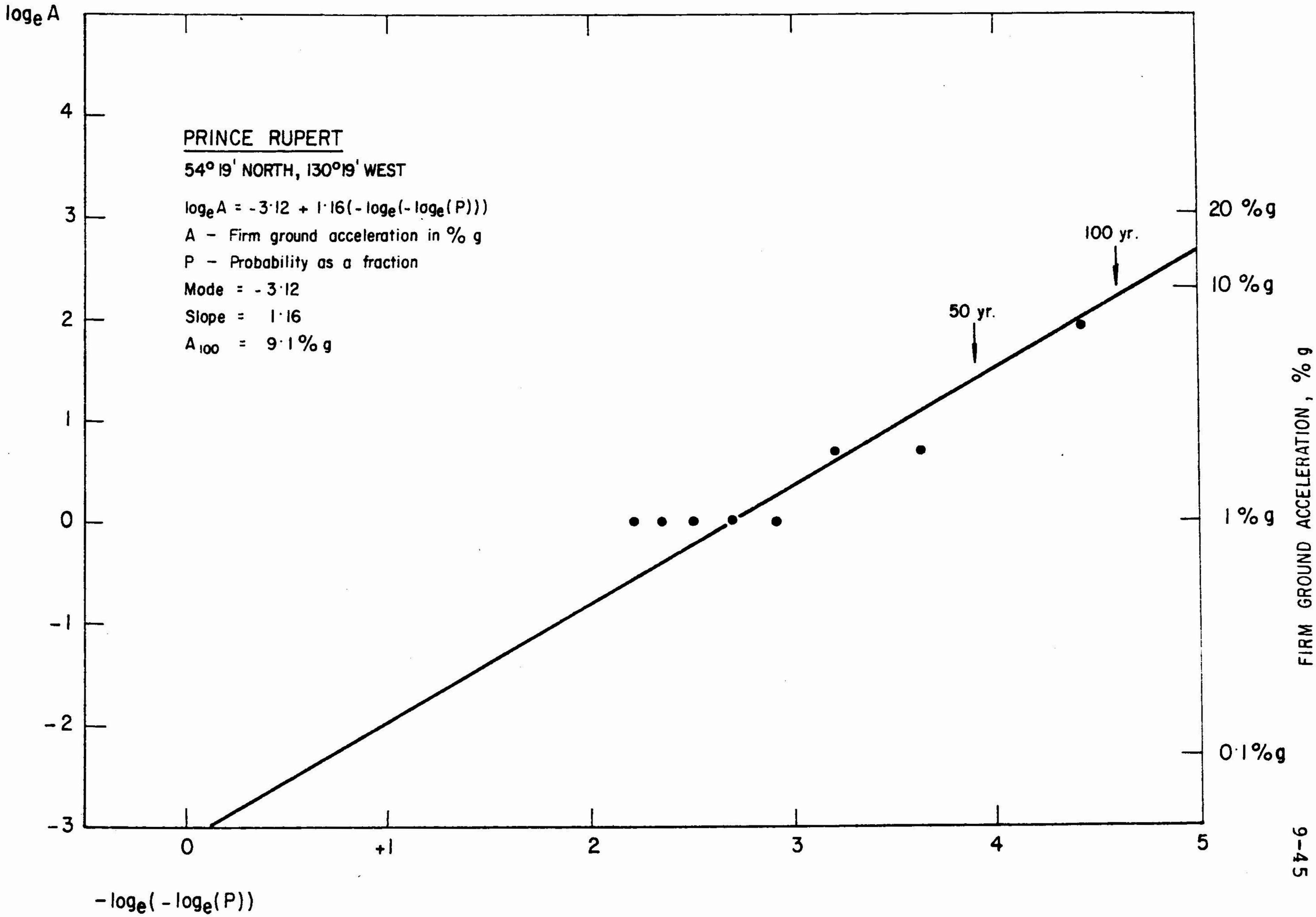
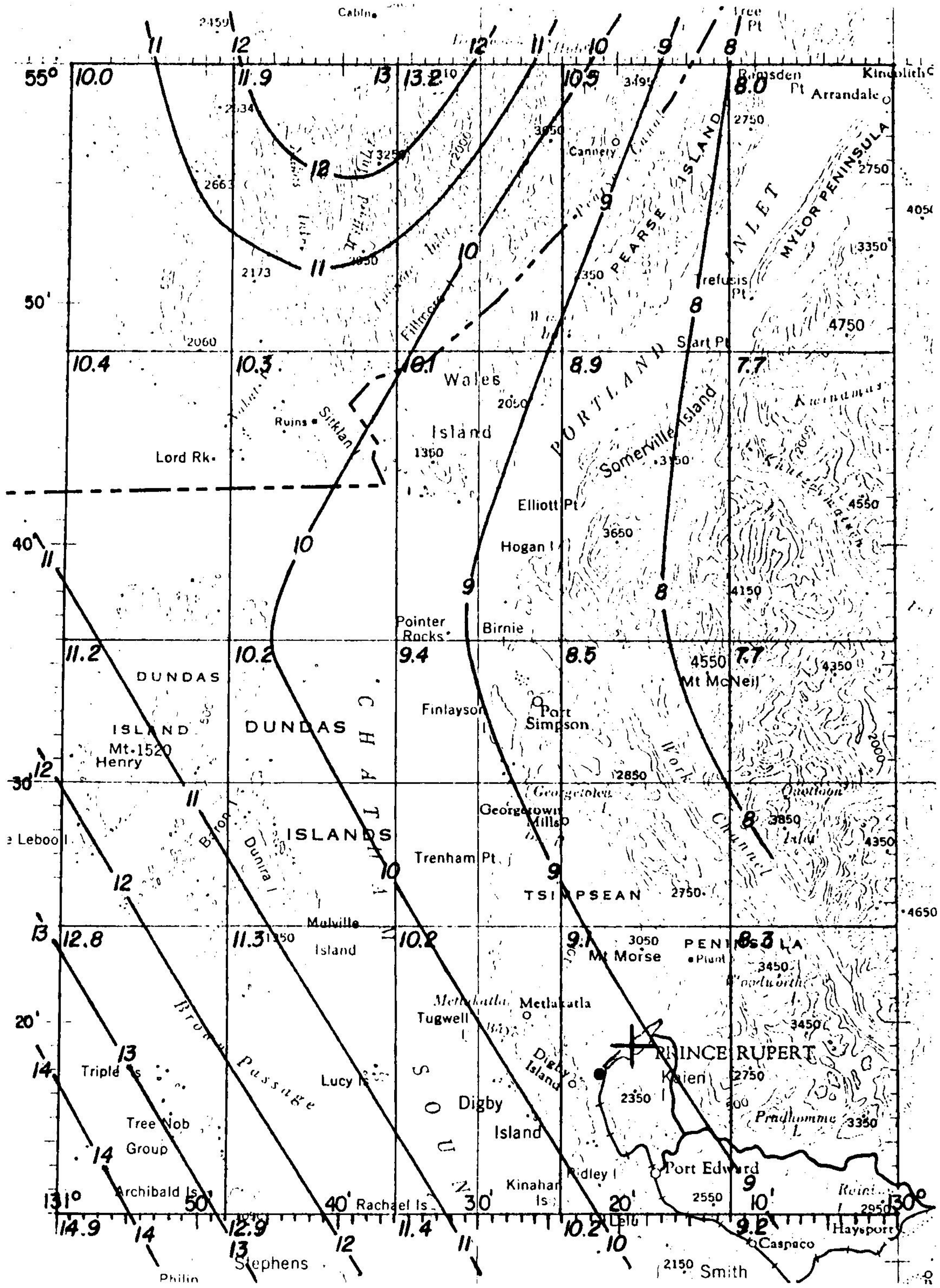


FIG. 2 FIRM GROUND ACCELERATION DATA FOR PRINCE RUPERT



10		20		30		40	MILES	

SCALE 1 : 500,000

FIG. 3 FIRM GROUND ACCELERATION CONTOURS FOR PRINCE RUPERT AREA

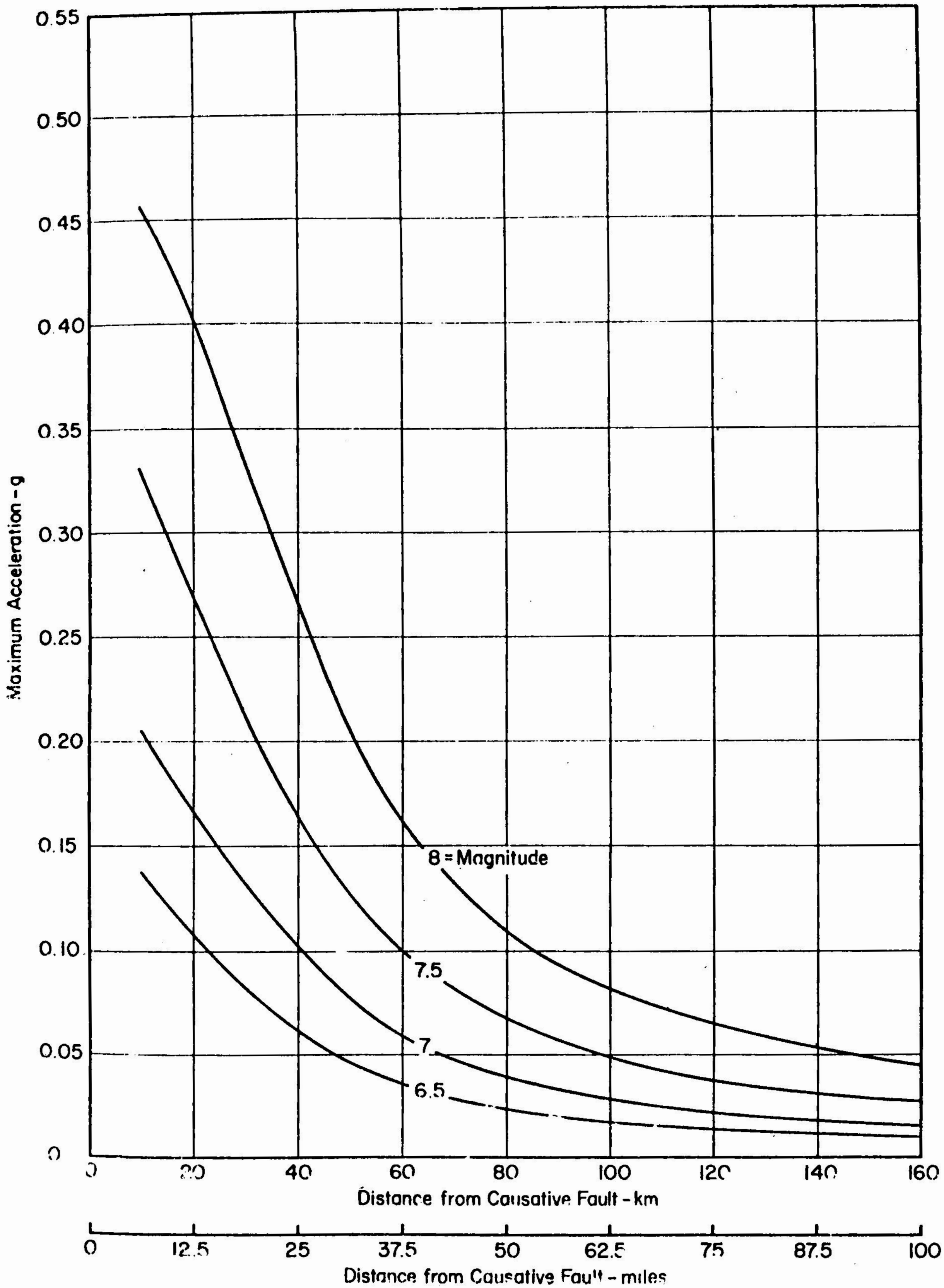


FIG. 4 VARIATION OF MAXIMUM ACCELERATION WITH EARTHQUAKE MAGNITUDE AND DISTANCE FROM CAUSATIVE FAULT

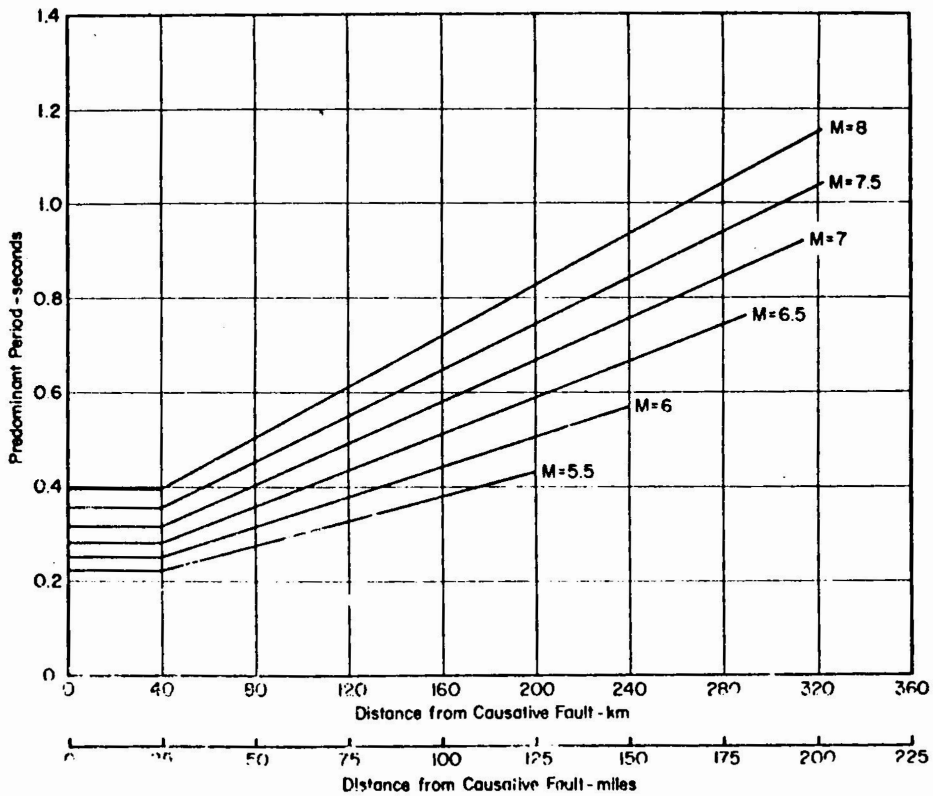


FIG. 5 PREDOMINANT PERIODS FOR MAXIMUM ACCELERATIONS IN ROCK

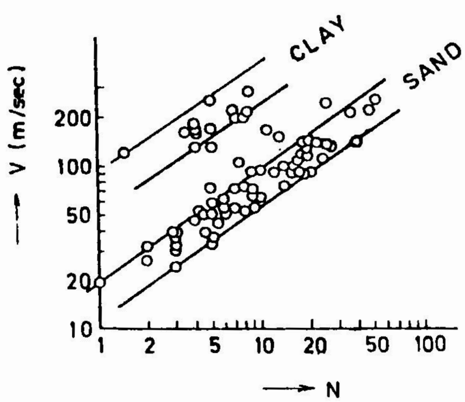


FIG. 6 SHEAR WAVE VELOCITY AND N-VALUE
OF STANDARD PENETRATION TEST

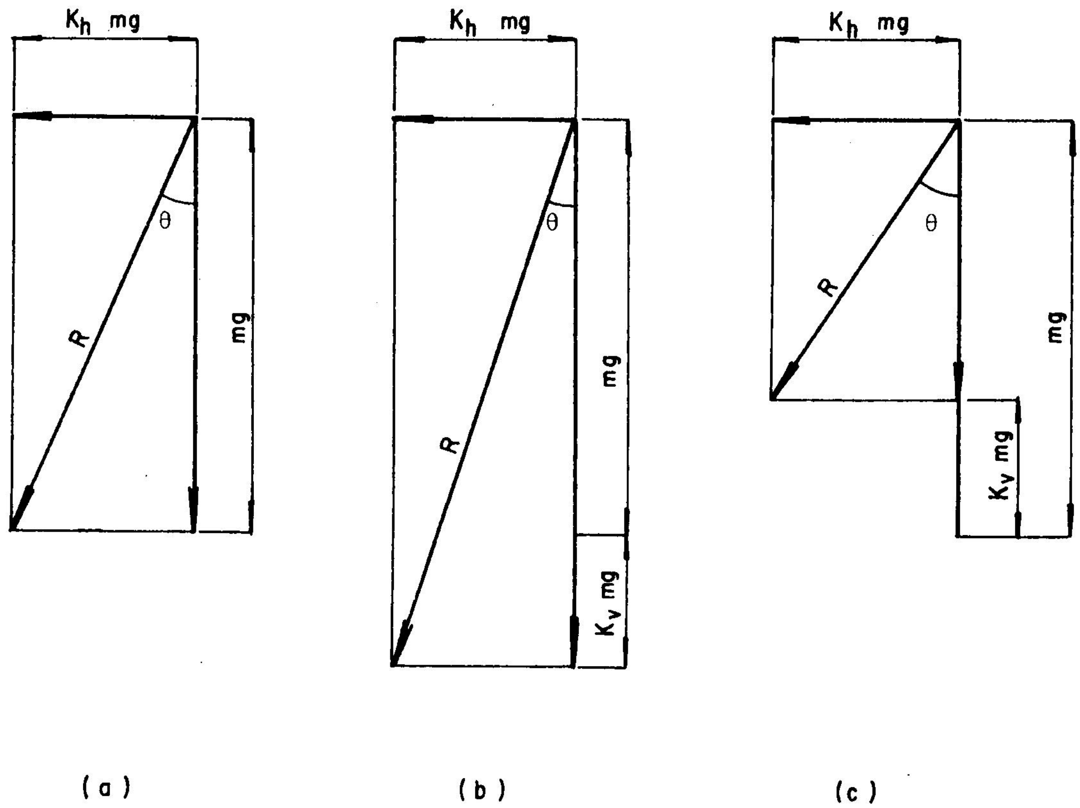


FIG. 7 SEISMIC COEFFICIENTS AND FORCES

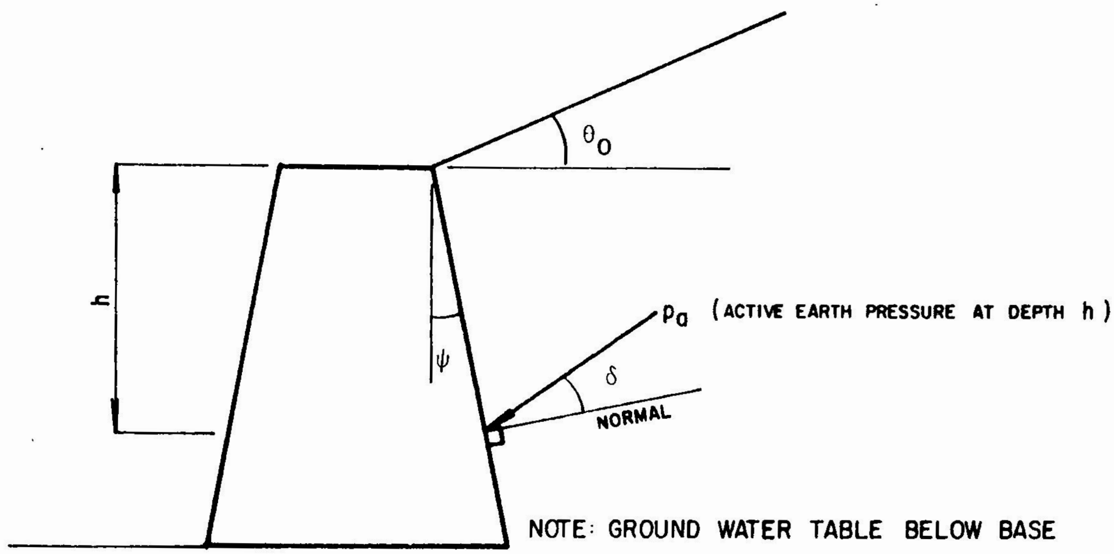


FIG. 8 DYNAMIC ACTIVE EARTH PRESSURE AT DEPTH h FOR A SLOPING BACKFILL

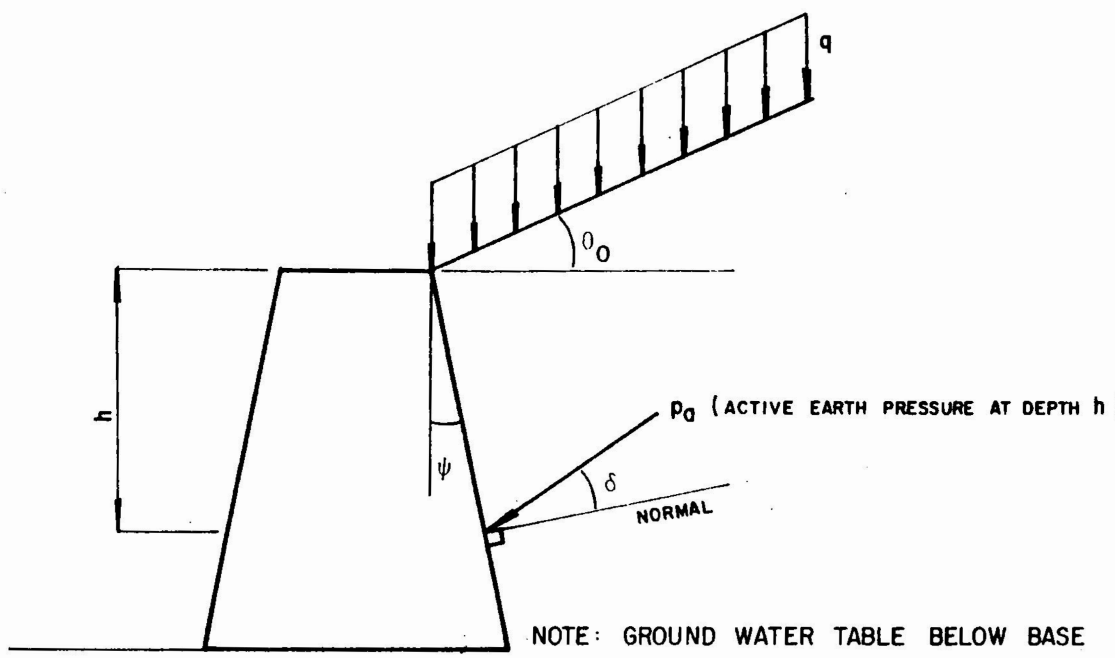


FIG. 9 DYNAMIC ACTIVE EARTH PRESSURE AT DEPTH h DUE TO SURCHARGE

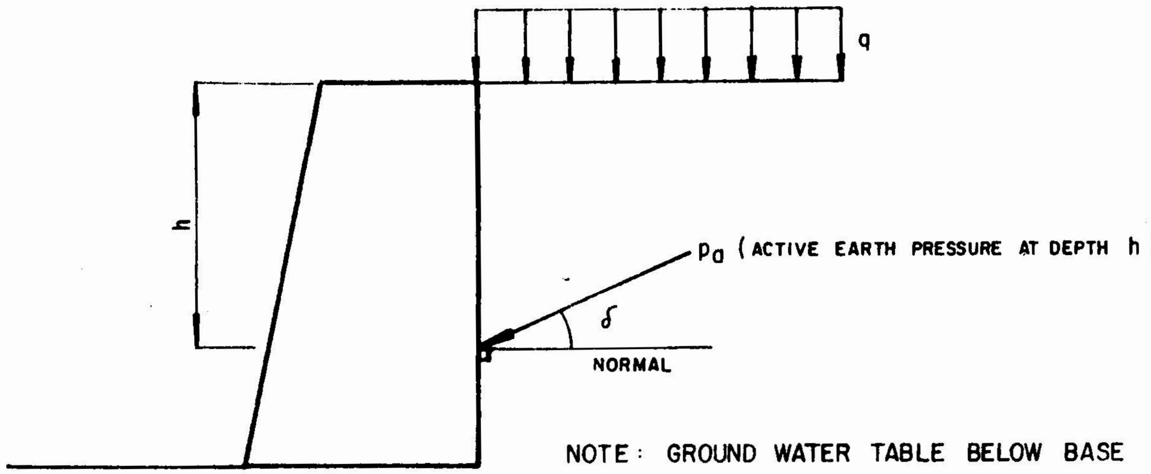


FIG. 10 DYNAMIC ACTIVE EARTH PRESSURE AT DEPTH h ON A VERTICAL WALL

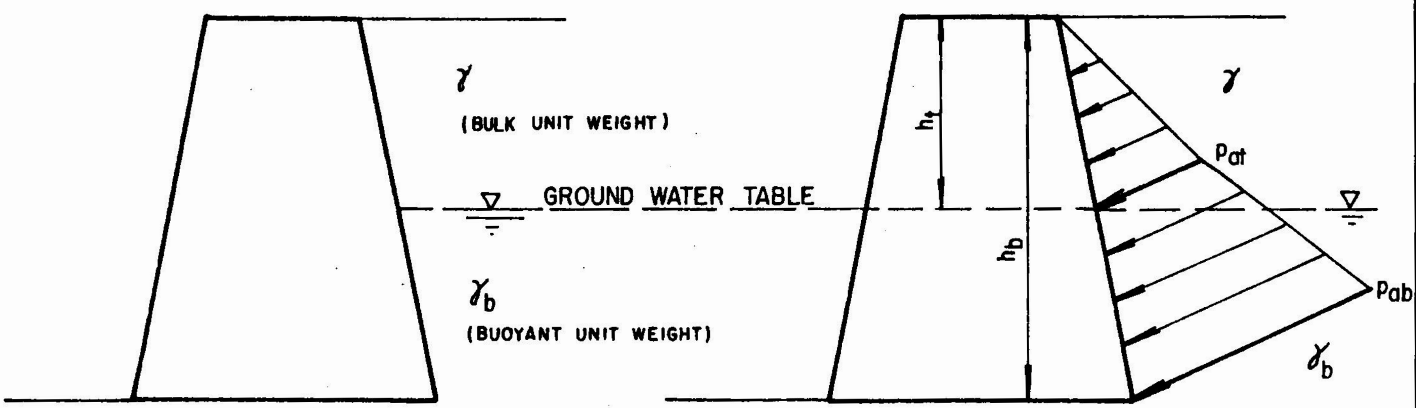


FIG. 11 DYNAMIC ACTIVE EARTH PRESSURES WITH GROUND WATER TABLE

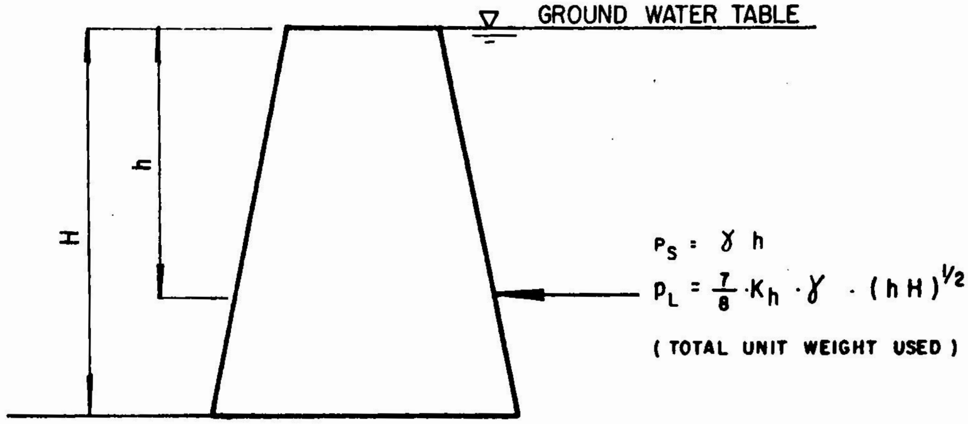


FIG. 12 DYNAMIC EARTH PRESSURE AT DEPTH h DUE TO A LIQUEFIED BACKFILL (WESTERGAARD THEORY)