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SYNOPSIS: The Mononobe-Okabe equation is still widely used in design practice to estimate earthquake induced soil pressures against earth retaining structures without differentiation of the lateral yielding or non-yielding character of the structure. Where these structures are rigid and non-yielding because of structural restraints (e.g., basement walls, bridge abutments, underground transportation, hydraulic and sanitary structures) the use of Mononobe-Okabe equation would not be appropriate and would be generally unsafe. Alternate design recommendations are proposed, based on the results of recent analytical and experimental studies by other researchers, for a nominal design earthquake expected to be representative of the New England seismicity.

INTRODUCTION

With the steady growth and redevelopment of the historical urban centers in New England and the associated need for a range of infrastructure, major underground structures have been constructed during the last decade. However, even more significant underground structures are in the planning and design phase at the present (1990). A typical illustration from downtown Boston is shown in Figure 1. High-rise buildings with up to seven levels below-grade parking garages, multi-level underground transportation structures such as Boston's depressed Central Artery, and major underground environmental structures within the scope of Boston Harbor Cleanup Project are examples of these recent and future developments.

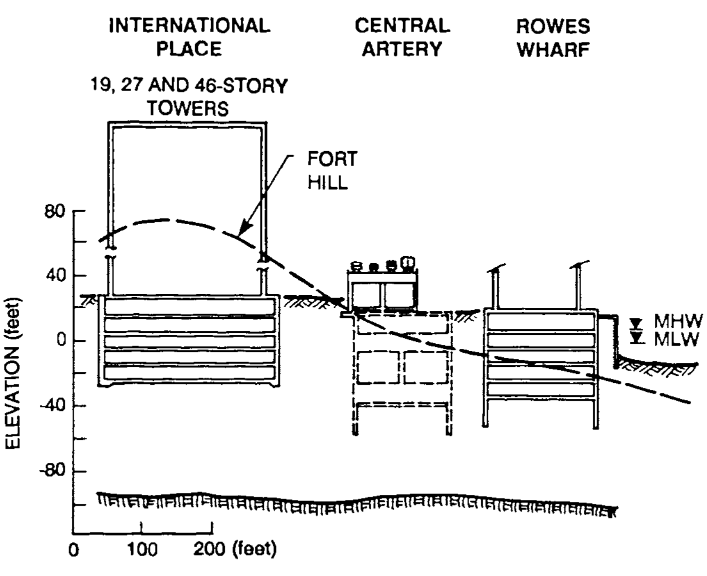

Figure 1. Schematic Cross-Section in Downtown Boston, Massachusetts

New England is a region of moderate seismicity with several major earthquakes having occurred in recent history. Accordingly, besides static loads, a proper consideration of the earthquake induced earth pressures against the above described, relatively rigid underground structures should be a relevant design requirement. In such an assessment, the regional seismicity should be represented by an appropriate design earthquake.

Primarily as a projection of the classic paper by Seed and Whitman (1970), it has been a common design practice in New England to use the Mononobe-Okabe equation for active conditions in estimating the earthquake-induced earth pressures against rigid, non-yielding basement walls. It is the theme of this paper that even though Mononobe-Okabe equation for active conditions is quite appropriate for yielding walls, it may underestimate the magnitude of dynamic incremental pressures against rigid, non-yielding earth retaining walls or structures. Whitman (1990) also indicated this important distinction.

At the outset it should be noted that in the case of a high-rise building with multilevel basements (e.g., International Place in Figure 1), dynamic earth pressures against the basement walls during ground shaking will be generated through two interactive sources. One component of the pressure is due to the inertia body forces within the soil retained by the walls, whereas the other component is associated with the earthquake induced displacements of the superstructure, that is inertia of the structure. Under such conditions, contribution of each component should be taken into account. On the other hand, in the case of a rigid underground structure without a significant above-grade portion (e.g., Central Artery in Figure 1), only the soil inertia component will be generated. Within the scope of this study only the soil-inertia generated dynamic earth pressures are addressed. Also, the structures are assumed to be founded on firm, competent ground.

NEW ENGLAND SEISMICITY

General

The New England geographical region including the states of Maine, New Hampshire, Vermont, Massachusetts, Connecticut and Rhode Island is one of the seismically more active parts of the eastern United States. Its seismicity may be characterized by frequently occurring small magnitude events, as well as occasional events of a size to cause damage to structures (Ebel 1987). Often New England and southeastern Canada are considered within the same larger seismo-tectonic region (Pulli 1982).

Seismo-tectonically northeast United States is classified as an intraplate region, and source mechanisms of the New England earthquakes are little understood at the present time. None of the documented historical earthquakes in the region is known to have been accompanied by surface fault movement, and no faults have yet been identified as active (Barosh 1979).

Historically, the following earthquakes with epicentral intensities equal or greater than MMVII, or magnitude M=5 have been recorded in New England (Pulli 1982, Algermissen 1983): November 1727 (MMVII) and November 1755 (MMVIII) Cape Ann, MA.; May 1791 (MMVIII) East Haddam, CT; October 1817 (MMVII-MMVIII) Woburn, MA; 20 and 24 December 1940 (both M=5.4) Ossipee, NH; 10 April 1962 (M=5.0) Middlebury, VT; and 18 January 1982 (M=4.8) Gaza, NH. Also, 9 and 11 January 1982 (M=5.7 and M=5.4, respectively) New Brunswick, Canada earthquakes were strongly felt in eastern Maine, with minor damage.

Recent Seismicity

Since 1975 seismic events in the region have been recorded by the seismometers of the Northeastern United States Seismic Network, and the recorded and evaluated ground motion data have been reported by Massachusetts Institute of Technology and Weston Observatory of Boston College. During this relatively short recent period, the coastal as well as central Maine, and central New Hampshire have had the major seismic activity in the region.

Based on the recently available instrumental data and historical seismicity, Ebel (1987) has identified areas of locally higher seismic activity in the northeastern United States. As depicted in Figure 2, the areas of locally higher seismic activity in New England include: Champlain Lake in Vermont (WQA: Ontario-Quebec Adirondacks); Dover-Foxcroft (DVF), Houlton (HNM), Passamaquoddy Bay (PAB), Penobscot Bay (PNB), southwestern Maine (SWM) in Maine; central Connecticut (CCT); Narraganset Bay (NB) in Rhode Island; and eastern-central New Hampshire and eastern Massachusetts (NHEM). Regarding the geographic distribution of the more recent seismic activity in New England, Perkins and Algermissen (1987) noted that it exhibits a southwest-northeast pattern in contrast to the previously suggested northwest-southwest (Charlevoix-Boston) axis.

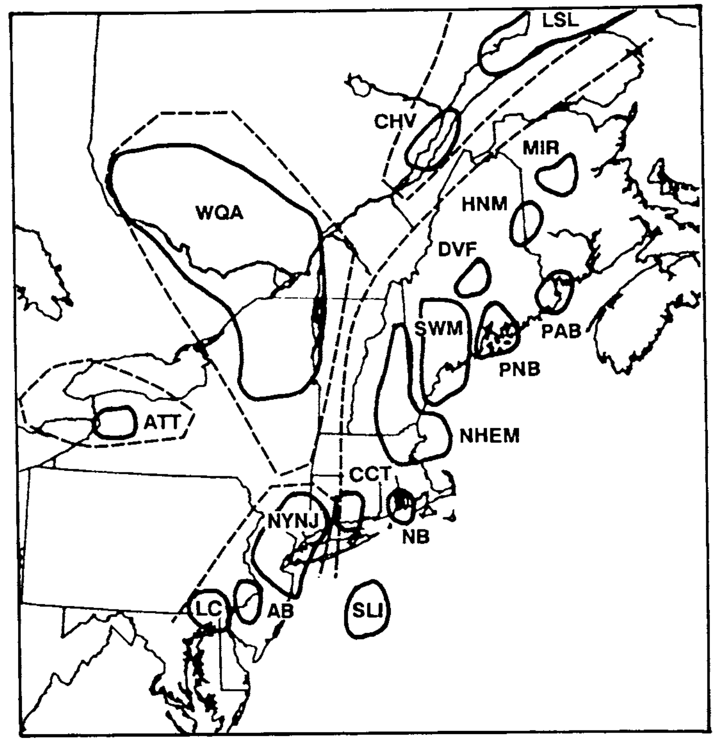

Figure 2. Seismically Active Areas in the NEUS (after Ebel 1987)

Design Earthquake for New England

In selecting a design earthquake for the purpose of this study, seismic hazard maps from current national codes and earthquake hazard reduction documents have been considered. Seismic hazard maps have been developed by the United States Geological Survey and are upgraded progressively. The maps partition the contiguous United States into zones of varying levels of potential seismic hazard based on documented historical seismicity and tectonic principles (Perkins and Algermissen 1987).

The BOCA National Building Code (1987) has been adopted by all New England states except Massachusetts which has its own building code. The BOCA Code contains a map of seismic zones for the contiguous 48 states, designated as Zone 0 through Zone 4 in increasing potential seismic hazard level. All New England states have Zone 2 designation, except for the north-central portion of Maine and extreme northern portions of New Hampshire and Vermont which are designated as Zone 1.

Uniform Building Code (1988) contains a seismic zone map of the United States with six levels of potential seismic hazard (i.e., 0, 1, 2A, 2B, 3, 4). All New England states have Zone 2A designation, except identical to the BOCA Code, the north-central portion of Maine and extreme northern portions of New Hampshire and Vermont are designated as Zone 1.

The Applied Technology Council document ATC 3-06 (1978) designates the severity of potential ground shaking at a locality by two seismic parameters: Effective Peak Acceleration coefficient (A_a), and Effective Peak Velocity-Related Acceleration coefficient (A_v). Each county in the contiguous United States has been assigned with an A_a and A_v coefficient in two respective maps. Algermissen and Perkins (1976) indicated that these maps were developed relative to peak accelerations on rock having a 10 percent probability of exceedance in 50 years. Both A_a and A_v range from a low 0.05 to a high 0.40 in increments of 0.05. All counties of the New England states have been assigned with A_a = 0.10 and A_v = 0.10, which is indicative of the same level of potential seismic hazard.

The National Hazard Reduction Program (NEHRP 1988) document provides maps more recently developed at the U.S. Geological Survey (Algermissen et.al., 1982, Perkins and Algermissen 1987) for the contiguous United States. Two maps show contours of horizontal acceleration and horizontal velocity in rock, respectively, for the contiguous United States with 10 percent probability of exceedance in 50 years, and two other maps in 250 years. A portion of the 50 year acceleration map showing the New England region is reproduced in Figure 3, because of its pertinence to this study. Perkins and Algermissen (1987), indicated that the probability levels of 10 percent exceedance in 50 years and 250 years relate to the relative levels of potential seismic hazard which might be appropriate for ordinary buildings and facilities providing critical services, respectively.

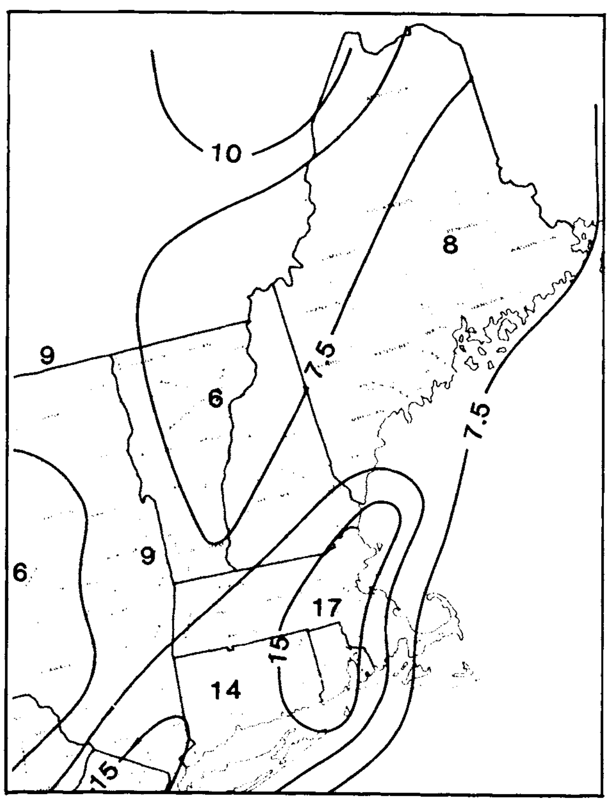

Figure 3. Map of Horizontal Acceleration (in Percent Gravity) in Rock with 90 Percent Probability of Not Being Exceeded in 50 Years (from NEHRP 1988)

The earthquake-resistant design provisions of the Massachusetts State Building Code (1988) are the first such criteria developed specifically for a jurisdiction in the eastern United States. As the outcome of a comprehensive study undertaken at the Massachusetts Institute of Technology (Krimgold 1977), Massachusetts Seismic Advisory Committee in the early 1970's prescribed a nominal design earthquake for the State, which is characterized by a peak ground acceleration of 0.12 g in firm, competent soil or rock, and an approximate epicentral intensity of MMVII-MMVIII (Luft and Simpson 1979).

Based on the presently available information on seismic hazard distribution in New England as described above, and within the context of this study, it may be concluded that: a) all New England states have an approximately equal level of seismic hazard exposure; and b) a nominal design earthquake characterized by a peak ground acceleration of 0.12 g in firm, competent soil or rock may be applicable for the whole region.

MONONOBE-OKABE EQUATION FOR YIELDING RETAINING WALLS

General

The earliest method to estimate the earthquake induced dynamic earth pressures on retaining structures was introduced by Okabe (1926), and Mononobe and Matsuo (1929). Commonly referenced as the Mononobe-Okabe (M-O) equation, it is actually an extension of the classic Coulomb (1776) theory established for static loading conditions.

The two key assumptions of the M-O formulation are: (1) during ground shaking the wall yields laterally of a sufficient amount to produce an active limiting equilibrium (or plastic equilibrium) in the soil behind the wall; and (2) the active soil wedge behind the wall behaves as a rigid body such that earthquake induced accelerations and thus the inertia (body) forces are uniform within the soil mass.

The validity of M-O equation developed on the above assumptions has been proven extensively through a great number of experimental studies, and it has been adopted universally as the standard method for evaluating earthquake-induced dynamic lateral forces in design of retaining structures (Whitman 1990).

Seed-Whitman's Simplified Mononobe-Okabe Equation

For the particular simplified conditions of horizontal ground (backfill) surface, vertical wall and zero vertical ground acceleration, Seed and Whitman (1970) proposed a simplified version of the M-O equation for evaluating the dynamic force for design analyses. Considering an average typical cohesionless soil with an angle of internal friction of 35 degrees and an average wall friction of 17.5 degrees, Seed and Whitman (1970), derived the following simple equation to estimate the magnitude of the dynamic lateral force (ΔP_{AE}):

$$\Delta P_{AE} = \frac{3}{8} \ k_h \ \gamma H^2 \qquad (1)$$

where: $k_h \ g$ = Horizontal component of the ground acceleration at the base of the wall

γ = Unit weight of the soil

H = Height of the soil against the wall

This simplified M-O equation, which will be referenced as the MO-SW equation in this paper, has been adopted by many building codes in specifying provisions for the seismic design of retaining walls. Similarly, Massachusetts State Building Code (1988) based on Equation 1 and considering a horizontal acceleration of 0.12 g specifies an earthquake force from the backfill equal to 0.045 γH^2 to be used in design analyses. The Code also requires that the dynamic force be distributed as an inverse triangle over the height of the soil.

NON-YIELDING RIGID WALLS

General

A key assumption in the M-O (or MO-SW) equation as stated earlier is that the retaining wall displaces a sufficient amount to develop a plastic stress state in the soil near the wall. The argument presented herein is that for the case of basement walls (i.e., soil retaining below-grade walls in buildings) as well as a variety of underground civil engineering structures (e.g., closed transportation and hydraulic structures, bridge abutments, power plants, pumping stations) founded on firm, competent soil or rock, the M-O method would not represent the actual conditions during ground shaking. That is these rigid and structurally restrained walls would not yield laterally and thus the soil (backfill) would not experience a plastic stress state. This in turn, would not lead to a minimum (active) lateral earth pressure regime against the wall, and therefore such a design assumption would be unsafe for rigid, non-yielding walls.

Furthermore, it will be suggested that for the case of rigid, non-yielding walls described above, and in regions of moderate seismicity such as New England, the assumption of elastic rather than plastic soil behavior would be more appropriate.

Wood's (1973) Analytical Work

Wood (1973, 1975) considered the rigid, non-yielding wall problem as shown in Figure 4 for the case of a simple rectangular boundary configuration with smooth contact (i.e., free from shear stress) between the homogeneous elastic soil and the wall interface. The lower boundary represents rigid, competent ground along which no soil displacement occurs. A uniform horizontal body force representing the soil inertia triggered by ground shaking is assumed. The magnitude of the body force (per unit soil volume) is equal to the product of the unit weight of the soil and the horizontal

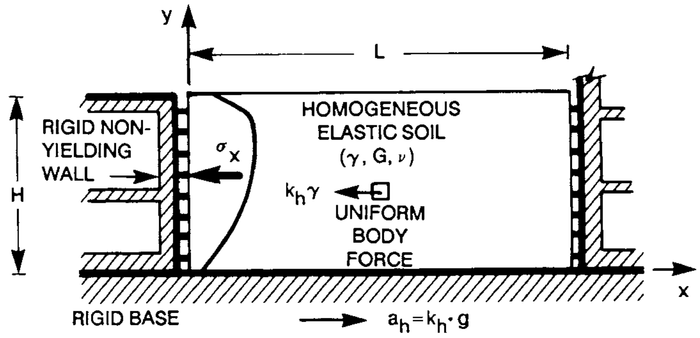

Figure 4. Wood's (1973) Analytical Problem

design acceleration normalized by the gravitational acceleration.

For the particular static problem shown in Figure 4, Wood (1973) obtained solutions equivalent to dynamic soil pressures on the wall by elasticity and finite element method. He studied the frictional wall-soil interface problem as well as the case where the soil deformation modulus is not constant but increasing linearly with depth. The effect of variations in Poisson's ratio on the solutions was also studied.

Wood's solution was used to generate static equivalent soil pressures for a horizontal design acceleration of 0.12 g adopted for New England as shown in Figure 5. A linearly increasing deformation modulus and a Poisson's ratio of 0.4 were used in the computations. The wall pressures were capped by the limiting passive pressure value. The solution illustrates the significant influence of the problem geometry defined by the ratio of the dimensions (L) and (H) on the relative magnitude of the induced wall pressures. It is also pertinent to note that the wall pressures are independent of the absolute value of the soil deformation modulus (i.e., Young's or shear modulus).

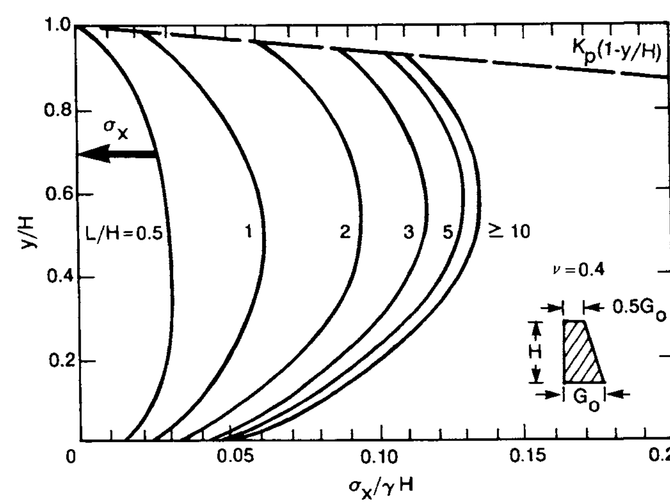

Figure 5. Dynamic Soil Pressures on Rigid Non-Yielding Wall for a_h = 0.12 g by Wood's (1973) Solution

Scott's (1973) Analytical Work

Scott (1973) used a one-dimensional elastic shear beam analogy to model soil stratum retained by a wall (Figure 6) to obtain dynamic soil pressures during ground shaking. He considered both rigid, non-yielding and deformable walls. Winkler type spring elements were used along the soil-wall interface, and Wood's (1973) solutions were used to define the character of the spring constants. A harmonic ground motion was considered at the rigid base to represent ground shaking.

First mode of Scott's solution was used to generate dynamic soil pressures (amplitude) for a harmonic ground acceleration with a 0.12 g amplitude. A uniform elastic deformation modulus and a Poisson's ratio of 0.4 were used to represent the soil stratum. The resulting maximum dynamic soil pressures for a range of (L/H) values are shown in Figure 7. Again it is pertinent to note, that the problem geometry represented by the (L/H) ratio has a significant influence on the relative magnitude of the induced dynamic soil pressures.

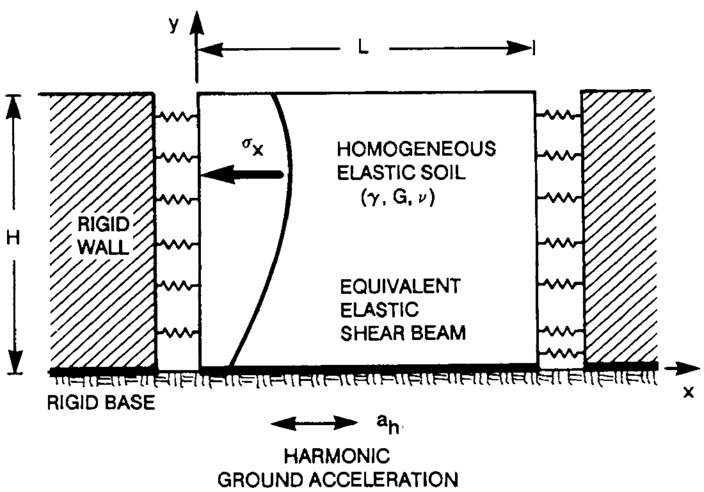

Figure 6. Scott's (1973) Analytical Problem

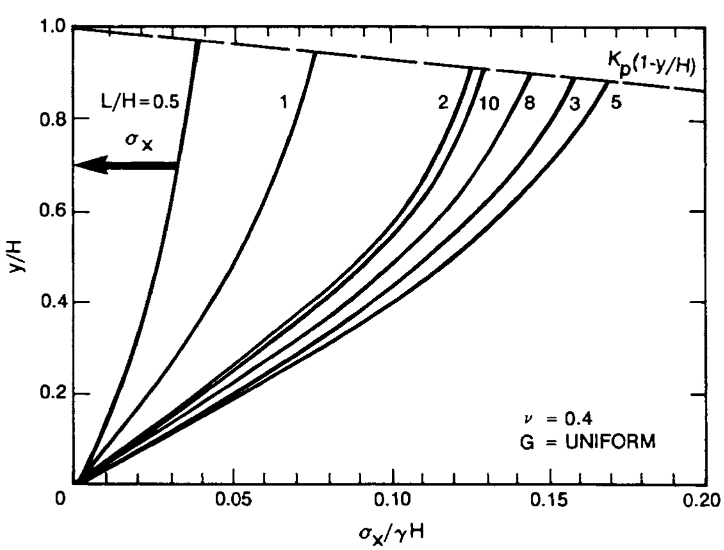

Figure 7. Dynamic Soil Pressures on Rigid Non-Yielding Wall by Scott's Solution (a_{h-max} = 0.12 g, First Mode)

Summary of Analytical Results

Analytical results obtained using Wood's (1973) and Scott's (1973) solutions for a horizontal ground acceleration of 0.12 g are summarized in Figure 8 in terms of dynamic forces. The force magnitudes for the (L/H) ratios indicated were obtained by integration of soil pressures as shown in Figures 5 and 7, respectively. Wood's (1973) results for a uniform soil deformation modulus case were also calculated and included in the figure. The progressive decrease in the dynamic thrust as depicted by the Scott's solution above an (L/H) value of about 5, may be due to the fact that only the first mode contribution was considered in the computations.

Also in Figure 8, dynamic force values computed by using M-O and MO-SW equations for a horizontal ground acceleration of 0.12 g are included for comparison. The figure indicates that the use of M-O or MO-SW equations would underestimate the seismically induced dynamic forces for rigid, non-yielding walls by 2 to 2.5 times for the particular value of ground acceleration. Only for the particular problem geometry of (L/H) ratio being about one or less M-O and MO-SW equations yield similar dynamic force magnitudes to the analytical results.

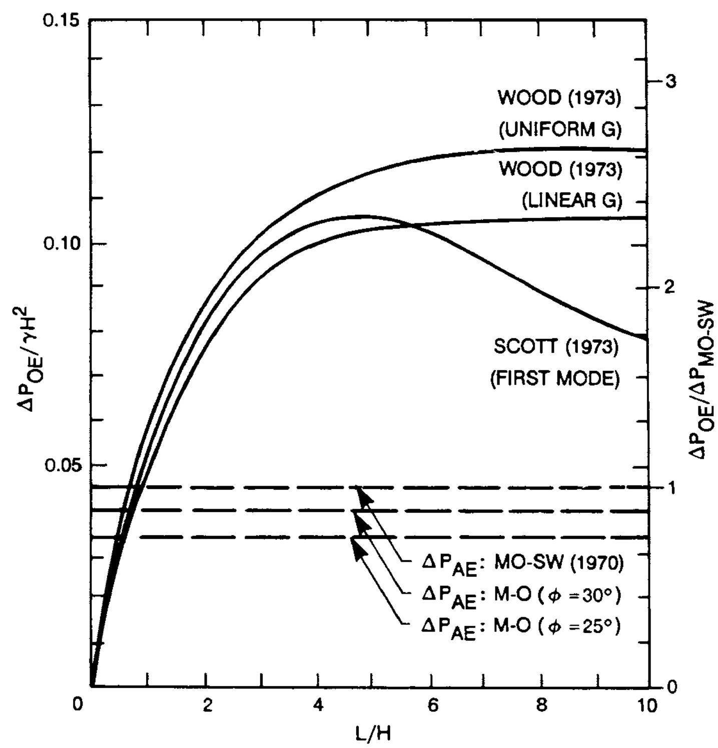

Figure 8. Summary of Analytical Results: Dynamic Forces Induced by Soil on Yielding and Non-Yielding Walls (a_h = 0.12 g)

Experimental Work by the Central Laboratories of New Zealand

Since the early 1980's, a comprehensive testing program has been undertaken at the Central Laboratories, Ministry of Works and Development, New Zealand (CLNZ), to study the behavior of earth retaining rigid walls subjected to ground shaking. A large number of shaking table tests of rigid model walls have been performed (Young 1985, Thurston 1986, 1987). A primary objective of the testing program (Elms and Wood 1987) was to provide experimental data for comparison with the previous analytical work of Wood (1973), whose results had been adopted as the basis for the seismic design of rigid earth retaining structures by the New Zealand National Society for Earthquake Engineering, NZNSEE (Matthewson, Wood and Berrill 1980).

The model wall tests were conducted utilizing a 2.44 m long, 2.44 m wide and 0.75 m high sand box mounted on a specially designed and built shaking table at the CLNZ. The model rigid wall consisted of a 25 mm thick aluminum plate, 0.6 m high by 2.24 m wide, mounted at one end of the sand box. Soil pressures on the wall were measured with five transducers located at the centerline (vertical) of the wall (Young 1985). The backfill used was a uniform sand having a D50 size of 0.25 mm. Dynamic soil pressures were measured directly (Young 1990) as the peak acceleration was increased from zero to 0.60 g in 0.05 g increments.

The measured dynamic soil pressures on the rigid, non-yielding model wall are presented in Figure 9 (Young 1985). For comparison, Wood's (1973) elastic solution, and results predicted by the M-O and MO-SW formulations have been incorporated in the figure. It is observed that experimental results agree closely with Wood's elastic solution, which have confirmed the NZNSEE design guidelines. Also, at the acceleration level of 0.12 g, representing the nominal design earthquake for New England, it may be noted that dynamic soil thrust measured on the rigid, non-yielding model wall is 2 to 2.5 times the values predicted by the M-O and MO-SW formulations.

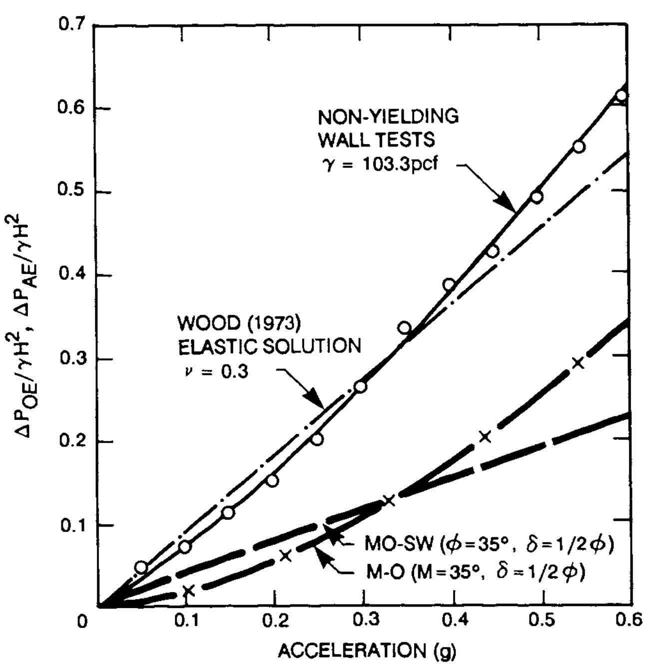

Figure 9. Dynamic Soil Pressures on Yielding and Non-Yielding Walls (after Yong 1985, Elms and Wood 1987)

In Figure 10, dynamic soil pressures measured at the centerline of the rigid, non-yielding model wall (Young 1985) at a peak acceleration level of 0.15 g are presented. Also for comparison, elastic solution for dynamic soil pressures (Wood 1973), and those computed by the MO-SW formulation have been incorporated. The figure illustrates the close agreement between the test results and the Wood's (1973) elastic solution, and the fact that the MO-SW formulation underestimates the dynamic soil pressures significantly.

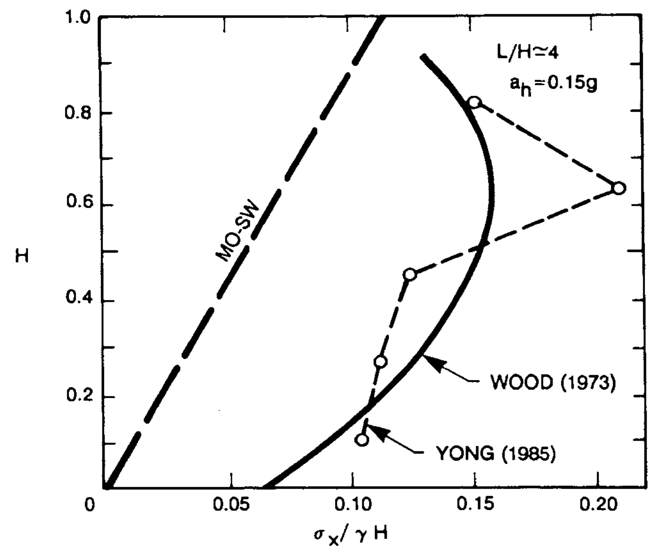

Figure 10. Comparison of Analytical and Experimental Dynamic Soil Pressures for Yielding and Non-Yielding Walls (a_h = 0.15 g)

Experimental Work by Ishibashi and Fang (1987)

Ishibashi and Fang (1987) reported the results of a series of sand box-shaking table tests in which dynamic soil pressures against a non-yielding and yielding rigid model wall were measured. The dynamic soil pressures were measured in tests with the wall being subjected to different modes of displacements at different shaking intensities. The tests were conducted using the University of Washington, Seattle, Washington, shaking table.

In Figure 11, the results of a test in which the rigid model wall was subjected to shaking with no yielding allowed at first, and then rotated about its base progressively while the shaking is continued, are shown. The measured total (i.e., initial static plus the dynamic increment) pressure profiles along the height of the wall depict the effect of wall movement (yielding) on the induced soil pressures which decrease as the wall rotation is increased. For comparison, total soil pressure profile predicted by the M-O formulation has been incorporated in the figure. It is observed that the difference in measured soil pressures for the non-yielding and yielding (active) wall conditions is rather significant. However, it should be noted that a portion of the decrease is associated with the decrease in the static soil pressure component due to shifting from an initial at-rest condition to a final active condition.

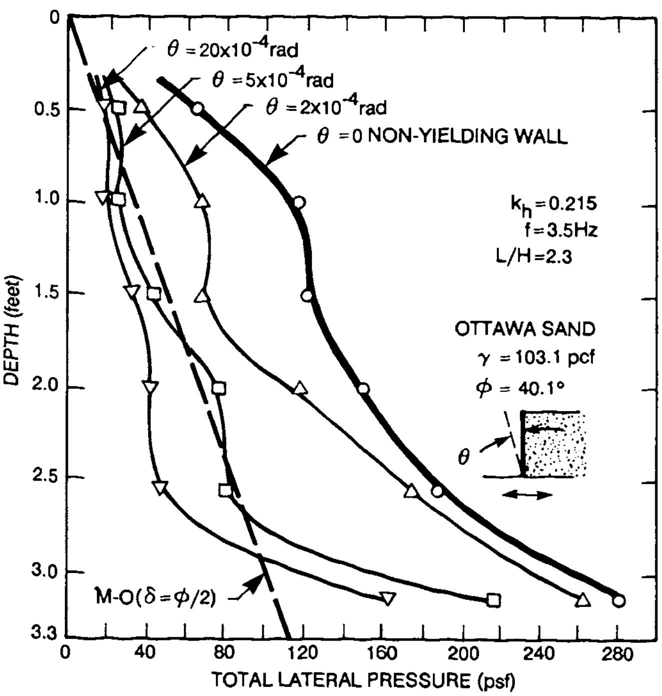

Figure 11. Total (Static + Dynamic) Soil Pressures on Wall Rotating about the Bottom (after Ishibashi and Fang 1987)

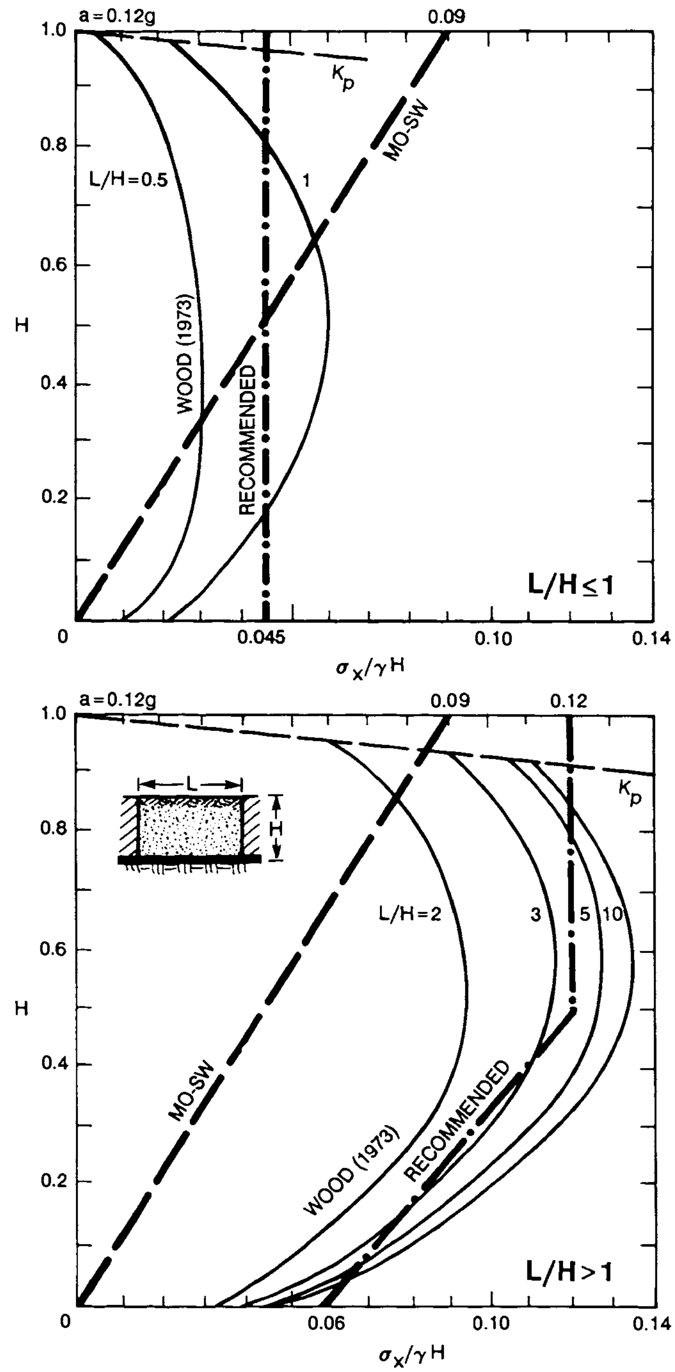

Figure 12. Recommended Dynamic Soil Pressures Against Rigid, Non-Yielding Walls in New England (a_h = 0.12 g)

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the analytical studies of Wood (1973), Scott (1973), and Nadim (1982), and the experimental work conducted at the Central Laboratories, New Zealand (Young 1985, Elms and Wood 1987) and Ishibashi and Fang (1987), it is recommended that for design practice in New England, the earthquake induced dynamic soil pressures, that is in addition to the appropriate static at-rest pressures, against rigid basement walls, bridge abutments and similar underground structures, be determined by following the guidelines presented in Figure 12. In the figure, recommendations for the dynamic soil pressures are provided for two site-structure configurations, namely up to an (L/H) ratio of about one, and for (L/H) ratios greater than one. Wood's (1973) elastic solutions were also included in the figure to allow the design engineer to deal with the (L/H) ratio effect in a more rigorous manner, if necessary.

Figure 12 also incorporates the dynamic soil pressures predicted by the MO-SW formulation. For the special range of (L/H) ratio being about one or less, the recommended total dynamic force is equal to that predicted by the MO-SW formulation, however line of actions of the dynamic forces are different. For the condition of (L/H) ratio being greater than one, the recommended dynamic forces are 2 to 2.5 times greater than that predicted by the MO-SW formulation. This observation is also consistent with the recommendations provided in AASHTO Guide Specifications for Seismic Design of Highway Bridges (1987-1988) for free standing bridge abutments or retaining walls which may displace horizontally without significant restraint (i.e., yielding), and those which are restrained from horizontal displacement by anchors or batter piles (i.e., non-yielding). The Guide Specifications further recommends the use of dynamic passive pressures for monolithic, rigid abutments which are displaced into the backfill due to inertia of the bridge superstructure, as indicated in the introduction.

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