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Seismic Design of Earth Retaining Structures

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Seismic Design of Earth Retaining Structures

(State of the Art Paper)

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SYNOPSIS Recent experiments, field observations and theoretical numerical studies are summarized. Where the simple conditions assumed by the Mononobe-Okabe theory are reproduced in tests, predictions and observations agree well. However, a retaining structure and surrounding soil are a complex dynamic system and behavior is far from simple. Emphasis has shifted somewhat away from dynamic stresses toward evaluation of residual displacements. With a good understanding of actual behavior, traditional methods may still be used except for large and unusual structures. The largest unknowns concern the behavior of cohesive soils and moderately dense to dense saturated sands.

INTRODUCTION

The state-of-the-art concerning seismic analysis and design of earth retaining structures is summarized by three statements.

1. Structures at waterfronts, where backfill inevitably is in large measure saturated, have frequently performed poorly during earthquakes with a number of spectacular failures. Such poor behavior has resulted from a combination of poor soils below the structures and large pore pressures developed within the backfill. The phenomenon of liquefaction has often been part of the problem.

2. Structures away from waterfronts have generally fared well during earthquakes. Examples of stability-type failures are rare, and there are only limited instances of large permanent movements. This seemingly good performance is largely the result of conservatism in design against static forces.

3. Our ability to predict just what will happen to a particular retaining structure during a major earthquake is still rather poor. Such structures are complex systems, having complicated and generally non-linear dynamic response. If simple analyses apply, it usually is only a coincidence.

In view of these statements, engineers must rely primarily on a sound understanding of fundamental principles and of general patterns of behavior. A survey of accepted standards-of-practice can be reassuring, since for some types of retaining structures there is considerable uniformity in practice. However, this situation may be very misleading - largely the result of the aforementioned conservatism and the limited experience (in the United States, at least) with really major earthquakes.

Hence the emphasis in this paper is upon the nature of the dynamic response of various types of retaining structures. The seismic design of retaining structures has been addressed in several previous state-of-the-art papers, notably Seed and Whitman (1970) and Prakash (1981). During this interval there have been:

* Additional model experiments, both in normal gravity and using geotechnical centrifuges.
* Development and use of a number of numerical techniques using discrete element representations for the soil.
* An increasing emphasis upon permanent displacements
* The beginnings of serious study of other than gravity walls and of the important effects of dynamic pore pressures.
* Only a few useful case studies based on actual experiences during earthquakes.

The next two sections summarize these studies, describing briefly what was done. Results will appear later when the behavior and design of specific wall systems are discussed.

SURVEY OF RECENT EXPERIEMNTAL RESULTS

Dry Sands

Experimental studies since about 1970 are summarized in Table 1. The column headed "gravity field" indicates whether the experiments were conducted in normal gravity or on a centrifuge at elevated gravity. In all cases the height of the wall is that of the model wall. Some of these tests, such as those by Sherif and Yong, were designed specifically to evaluate dynamic earth pressures. Others modeled particular wall systems, and measurements of dynamic pressures between soil and wall may or may not have been made.

TABLE 1: Recent experiments using dry sands

Researcher	Type of wall and support *Measured quantities*	Height of wall-mm	Gravity Field	Nature of shaking
Aitken et al.(1982)	Rigid wall sliding on sand; *applied force*	300	1g	Single pulses
Sherif et al.(1982) Sherif and Fang (1984)	Rigid wall moved outward in controlled manner during shaking; translation, rotation about top or bottom; *reactions; pressures*	1000	1g	Steady sinusoidal
Klapperich (1983)	Gravity wall sliding or tilting on sand; *pressures*	1000	1g	Steady sinusoid or earthquake motion
Sommers & Wolfe (1984)	Reinforced-earth walls; *motions only*	457	1g	Earthquake motions
Steedman (1984)	Gravity wall sliding on sand; *motions only*	90 - 135	80g	10 more or less sinusoidal pulses
	fixed base cantilever wall; bending moments in wall	175	40 - 90g	
Nagel & Elms (1985)	Reinforced-earth walls	300	1g	Single pulses
Yong (1985)	Wall moving rigidly with base; reactions; *pressures*	500	1g	Steady sinusoid
Andersen et al (1987)	Gravity wall tilting against elastic spring; *reactions*	144	80g	10 more or less sinusoidal pulses
Richards & Elms (1987)	Rigid wall pushed into sand; *motion only*	152	1g	Single pulses
Fairless (1989)	Reinforced-earth walls;*motions, stresses in reinforcing strips*	1000	1g	Single pulses
Kutter et al.(1990)	Reinforced soil and cantilever walls; *motions only*	152	30g	Earthquake-like
Neelakantan et al. (1990)	Tied-back wall in sand; *motions support force*	629	1g	Varying-amplitude sinusoids

In some tests, reactions at support points were measured in addition to or instead of direct measurements of stress between backfill and wall. I have long been skeptical of direct measurements, using pressure cells placed on or embedded within wall. However, a number of researchers appear to have obtained good results. Evaluation of the total thrust by measurement of reactions provides a useful check upon and avoids the potential difficulties with direct stress measurements, but with free standing walls such an approach is not possible.

Various forms of dynamic shaking were employed. I am a strong believer in using simple shakings either a single pulse or sinusoidal pulses. While such an approach leaves unanswered questions about just what happens when walls are shaken by complex earthquake ground motions; tests with simple inputs make it possible to observe and study basic patterns of behavior and to make meaningful comparisons between predictions and observations. These should be the primary goals for model tests.

Submerged Sands

Matsuzawa et al. (1985) have summarized several model test programs carried out in Japan, between 1956 and 1979. In all but one test, the saturated soil was in contact with a rigid wall, with dynamic water pressures being measured at the wall. Depth of water ranged from 350 to 700 mm. Sinusoidal shaking apparently was used. The grain size D_{50} of the soils ranged from 0.2 to 5 mm.

There apparently are very few experiments involving walls free to move in response to the applied forces. Steedman and Zeng (1990) describe experiments in which a rigid wall was used to simulate an anchored bulkhead. The tests were carried out on a centrifuge at 80 gravities, using a fine sand at relative densities of 55% to 80% and with silicone oil as a pore fluid. The results emphasized the increasing amplification of motions as pore pressures within the backfill increased, and also the complexity of the pore pressure behavior near the embedded portion.

Pahwa et al. (1987) report very preliminary tests in which support was suddenly removed from a wall supporting a saturated backfill. With dense backfill, there was a delay in the onset of failure owing to development of negative pore pressures. With loose backfill, a liqufaction flow failure occurred.

Field Experiences

Each new earthquake potentially yields valuable data concerning the performance of retaining walls, but generally the information is not documented in sufficient detail to permit clear conclusions.

As in many earthquakes in the past, there have been excessive movements and even failures of waterfront structures, as a result of liquefaction in backfills. Werner and Hung (1982) have provided extensive documentation of such cases. Some more recent examples are:

* Large (1.5m) residual displacement of a steel sheetpile wall in Akita, Japan in 1983 (Iai et al., 1989b). This failure was blamed on liquefaction.
* Collapse of a gravity wall and excessive movement (about 1 m) of an enclosed bulkhead in the port of San Antonio, Chile, in 1985 (Poran and Rodriguez, 1989,and personal observation of the writer). The gravity wall had already tipped outward before the earthquake, but liquefaction certainly was also a factor. There was poor compaction of the backfill for the bulkhead, and apparently the anchor rods had not been ten- sioned. In the same port, another anchored bulkhead, designed to a seismic coefficient of 0.15, experienced 0.15 m of movement and the berth remained in service.
* In Valparaiso, Chile, in 1985, movements of old gravity walls ranged from 0.05 to 0.7 m. These magnitudes were related primarily to the quality of the soils below the walls. There apparently was no liquefaction, although there may have been modest excess pore pressures (personal observations of the writer).
* Excessive lateral movements at the waterfront of the Port of Oakland during the 1989 Loma Prieta earthquake, associated with liquefaction.

In additon, there has recently been a reassessment of movements of sheet pile waterfront walls in Niigata during the 1964 earth probe, revealing lateral motions of several meters in some cases (Yasuda et al. 1989).

For retaining structures away from waterfronts, there has been remakably little documentation of significant movements. Fukuoka and Imamura (1984) present briefly observations of damage to retaining structures and data from dynamic earth pressure measurements. Grivas and Souflis (1984) report in detail concerning a bridge abutment in Greece that experienced several inches of residual movement. Ho et al. (1990) documents the behavior of 10 tied-back walls (for excavations) in Los Angeles during the 1987 Whittier earthquake. There was no evidence of loss of integrity. Kutter et al. (1990) surveyed mechanically-stabilized walls (for highways) in the region around San Francisco Bay following the 1989 Loma Prieta earthquake, and found no evidence of significant residual movements. Despite extensive damage to port facilities at Akita, Japan, 24 reinforced earth walls in the area performed very well (TAI, 1985).

THEORETICAL STUDIES

Table 2 provides what doubtless is only a partial listing of theoretical analyses using finite element or finite difference techniques to simulate the behavior of backfill and foundation soil. The exact nature of the constitutive model is not always made clear in the references. Some of these studies focus upon dynamic lateral stresses, while others aim to predict residual displacements. There is currently considerable interest in this latter topic as evidenced by the Proceedings of the Second U.S.-Japan Workshop on Liquefaction, Large Ground Deformation and Their Effects on Lifelines (O'Rourke and Hamada, 1989).

The constitutive models and computer codes usually have been developed for purposes more general than the study of retaining structures. The codes tend to be quite complex, and it appears difficult for others than the authors to apply them. They have been used to help understand and explain general behavior to explore the limits of accuracy of simpler models, and in some cases to predict (after the fact) results from model experiments and field observations. There have been few (if any) applications to actual engineering projects, but I believe use of such analysis should be considered in connection with projects of unusual size and importance.

There have also been a number of analyses using very simplificed models, using lumped masses to represent soil and/or springs to represent soil-structure interaction, e.g. Whitman (1989) and Siddarthan et al. (1990). These models have proved useful for study of limited aspects of the behavior of retaining structures, such as the phasing of wall movement and earththrust relative to base input acceleration.

EARTH PRESSURES

Papers on the subject of seismic response of retaining structures typically begin with a discussion of dynamic earth pressures. This has perhaps been inevitable. Design for static loadings always begins with an evaluation of static earth pressures, and by and large this approach has worked well. In addition, one of the earliest contributions to soil dynamics was the Mononobe-Okabe equation for earthquake-induced lateral earth pressure - and this equation has had remarkable endurance.

If anything, there has been too much emphasis upon the evaluation of dynamic earth pressures. Earth retaining structures are complex soil structure systems, and the contact stresses between soil and structure are the result of the dynamic interaction of the actual system. These stresses vary in a complex manner during earthquake shaking. For many types of retaining structures the peak contact stresses may be of little concern from the standpoint of design. However, there are also those retaining structures where peak stresses do have a major influence on design.

Active Thrust

Most interest has centered on the Mononobe-Okabe equation corresponding to the active state assuming cohesionless backfill. The equation,

TABLE 2: Recent numerical studies

Research	Problem studied	Constitutive model	Nature of shaking
Werner & Hung (1982)	Sheet pile bulkhead	Linear with strain-adjusted properties; total stress analysis	Ground motions at nearby site
Nadim & Whitman (1983)	Sliding gravity wall; dry backfill	Linear with strain adjusted properties; prepositioned failure surface	Sinusoidal; typical earthquake motions
Marciano et al. (1985)	Cantilever retaining wall; dry sand	Hyperbolic stress-strain with Masing rules	Synthetic seismogram
Fujii et al. (1989)	Anchored sheet pile quay-wall	Hyperbolic stress-strain with excess pore pressures related to cyclic strain; undrained analysis	Ground motions at nearby site
Iai and Kameoka (1989)	Anchored sheet pile quay wall	Hyperbolic stress-strain with excess pore pressure related to state parameter; undrained analysis	Sinusoidal motion
Siddarthan & Maragakis (1989)	Fixed base cantilever wall; dry backfill	Hyperbolic stress-strain with volumetric strains related to cyclic strain	Ten sinusoidal cycles
Alampalli & Elgamal (1990)	Fixed base cantilever wall; backfill	Linear shear beam for soil, non-linear interaction springs	Ten sinusoidal cycles
Al Homoud (1990)	Gravity wall resting on dry sand	Visco-elastic cap model, with gapping-sliding contact elements	Sinusoidal pulses; typical earthquake motions
Bakeer et al (1990)	Constrained motion of wall during shaking; dry sand	Hyperbolic stress-strain with Masing rules	Sinusoidal
Stamatopoulos & Whitman (1990)	Residual movement of tilting gravity wall; dry backfill	Uses empirical relations for residual strains, related to cyclic strains	Sinusoidal pulses

together with charts may be found in the aforementioned state-of-the-art papers, AASHTO (1983) U.S. Army Corp of Engineers (1989) and most texts on soil dynamics. Seed and Whitman (1970) suggested a useful approximation for the dynamic increment ΔP_{AE}:

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

where Ag is the horizontal acceleration. Note that the active thrust is increased when the acceleration is directed against the backfill, as shown in Fig. 1. If the acceleration acts in the opposite direction, the active thrust is decreased. Vertical accelerations will also affect the thrust, with downward acceleration causing a decrease.

Model tests have tended to confirm the equation, although it was not always clear whether the boundary conditions were really appropriate. The best experiments appear to be those by Sherif et al. (1982) and Sherif and Fang (1984), using a test arrangement patterned on a system developed previously in Japan. In these tests, a 1-m high wall was moved away from the backfill in a controlled manner during shaking, as shown in Fig. 2. Three situations were studied: translation of the wall, rotation about the base and rotation about the top. By measuring reactions at the top and bottom of the wall, the magnitude and line-of-action of the resultant thrust were evaluated. In some tests, pressure cells were placed on the face of the wall. The reported thrusts and stresses were the peak values once the wall had moved enough to achieve a steady-state active condition. By way of summary:

* The dynamic active thrust were very similar to those predicted by the Mononobe-Okabe equation, although somewhat larger: see Fig. 3.
* The distribution of dynamic pressure was not linear with depth, being greater against the upper portion of the wall and less against the lower portion: see Fig. 4. This was especially true for walls contrained to rotate about their

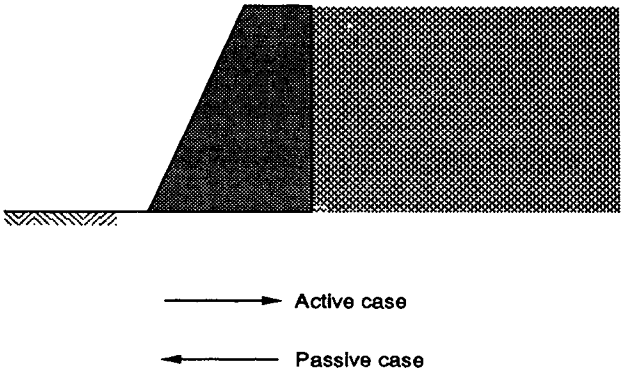

Fig. 1. Critical directions for accelerations.

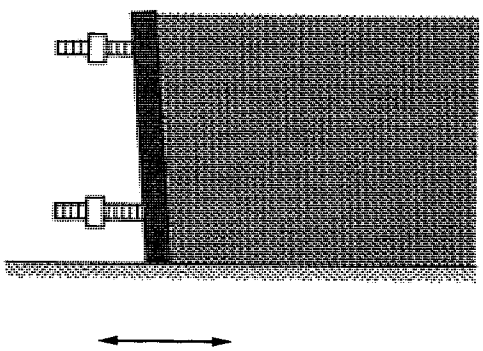

Fig. 2. Schematic of Sherif-Ishibashi tests

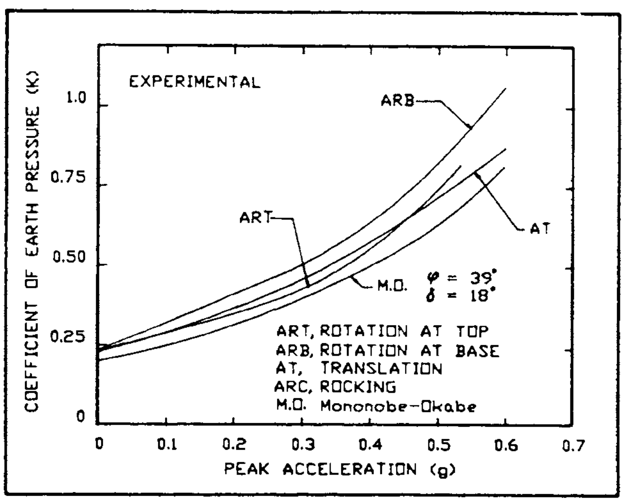

Fig. 3. Magnitude of total thrust as function of acceleration coefficient (from Bakeer et al., 1990)

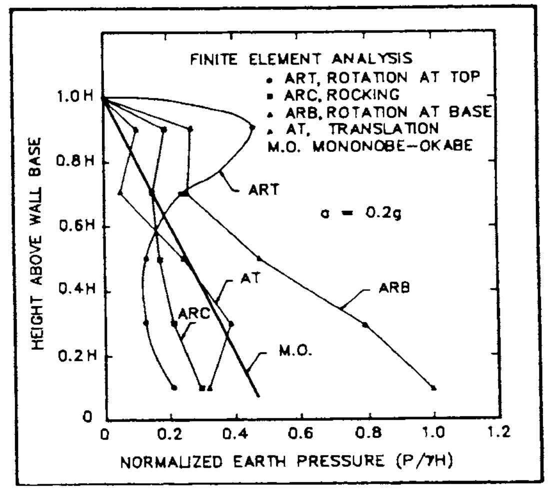

Fig. 4. Horizontal stress distribution for different motions of wall (from Bakeer et al., 1990) These are theoretical predcictions; experimental results were similar

top.

* As a result, the height of the resultant thrust increased above the lower third-point as the intensity of shaking becomes larger: see Fig. 5.

In these tests, the conditions assumed in the Mononobe-Okabe theory - active conditions and uniform horizontal acceleration - were achieved, and hence the results agreed well with theoretical predictions. However, depending upon the nature of the soil-wall system, such conditions may not occur in actual situations.

Deviations from these ideal conditions of course affect the thrust. Non-uniformity of acceleration potentially is very significant, as has been discussed by Steedman and Zeng (1989; 1990). Difference in phasing of accelerations with heighttend to reduce the thrust. If the amplitude of acceleration increases with height, then using the base acceleration in the Mononobe-Okabe equa-tion will underestimate the thrust.These effects are especially important when the frequency of shaking is close to the fundamental frequency ofthe backfill stratum.

Equations for active dynamic thrust is available for cohesive soils (Prakash, 1981; Okamoto, 1984). These equations, which predict zero thrust of the undrained shear strength are sufficiently large, have apparently not been confirmed and should be used with great caution.

Passive Thrust

There is also a version of the Mononobe-Okabe equation for passive conditions. As given in the Seed-Whitman paper, there is an error in this equation, and unfortunately this error has propagated through the literature. The correct version appears in Prakash (1981).

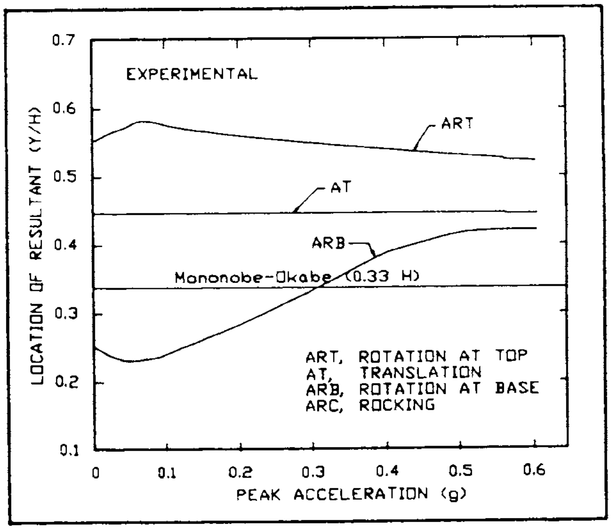

Fig. 5. Height of resultant as function of acceleration coefficient from Bakeer et al., 1990)

For this equation, a positive acceleration is directed away from the backfill as shown in Fig. 1, and acts to reduce the passive resistance below the static value. An approximate estimate for this decrement of resistance is:

$$\Delta P_{AE} = 2.125\ \gamma H^2 A \qquad (2)$$

As with the static case, wall friction affects strongly the dynamic passive resistance (see Neelakantan et al., 1990),and all of the conventional warnings concerning use of the Coulomb failure surface assumption apply.

Unyielding Walls

Theoretical results from Wood (1973), based on the assumption of modulus constant with depth, are still the standard for this case. Wood predicted a total dynamic thrust approximately equal to $\gamma H^2 A$. The dynamic horizontal stress increases with height above the base, with the resultant at a height of 0.58H. Nadim and Whitman (1983) report results from finite element studies assuming modulus increasing with depth. Although the distribution of dynamic horizontal stress differed somewhat from that found by Wood, showing smaller stresses near the surface, the height of the resultant was similar.

Model experiments by Yong (1985), with excitaation at a frequency well less than the fundamental frequency of the stratum of backfill, confirmed these theoretical results. The magnitude of the dynamic thrust was essentially that predicted by the theory, and the height of the resultant varied between 0.52H and 0.57H, with the more applicable results being at or near the upper value. The distribution of stresses appeared most similar to that predicted by Nadim and Whitman. However, when the excitation frequency was increased until a resonant condition occurred, the stresses and thrust increased. With steady, strong shaking, residual horizontal stresses developed - up to 1.8 times the initial static stresses.

GRAVITY RETAINING WALLS

In this and most of the following sections, it is presumed that excess pore pressures are insignificant. There is a final section concerning walls at waterfronts.

Gravity walls have received the greatest attention, presumably because of the apparent simplicity of this system and because the Mononobe-Okabe equation seemingly should apply directly to this case. However, tests and numerical calculations (Aitken et al., 1982; Nadim and Whitman, 1983; Andersen et al., 1987; Whitman, 1989; 1990) have shown that the actual dynamic response is far from simple. In particular:

* The resultant lateral thrust varies considerably during shaking, with values both smaller and larger than those predicted by the Mononobe-Okabe equation. The phasing of the maximum and the minimum thrusts is just the opposite of what would be expected from the Mononobe-Okabe theory. It is not clear whether the conditions assumed by this theory really apply at any time during shaking, and it seems largely fortuitous that thrusts measured in experiments involving gravity walls have been similar to those predicted by the theory.

* The height of the resultant the height is least (sometimes less than one-third H!) when the wall is moving away from the backfill, and greatest when the wall swings back against the backfill. Similarly, the mobilized wall friction varies during shaking, increasing as the wall moves outward and decreasing during reverse motion.

* Residual lateral forces develop as a result of strong shaking, together with residual displacements of the wall. The residual force may be nearly as large as the peak force during shaking.

These results emphasize that a gravity wall, its foundation and the retained backfill form a system, and the movement of the wall and the stresses between wall and soil are the result of the dynamic response of this system. These facts must be kept in mind when using simple "equivalent static" forces in design procedures.

Evaluation of Permanent Displacements

A major development in research during the past decade has been emphasis upon evaluation of permanent displacement following an earthquake shaking. This approach was given major impetus by Richards and Elms (1979), who put forth a simple method - based on analogy to Newmark's sliding block and retaining the simplicity of the Mononobe-Okabe equation - for estimating residual displacements and suggested an approach to design based on allowable movement. (see Elms and Richards, 1990). Model tests, have provided validation for the approach.

Philosophically, the Richards-Elms method is akin to that used for seismic design of ordinary buildings; that is:

* It is accepted that it is uneconomical to design for no permanent movement (for buildings, read no damage) as a result of a major earthquake.

* A seismic coefficient is selected, less than that corresponding to the peak accelerations during a major quake, which - on basis of experience or theory - is adequate to hold movement (damage) to an acceptable level.

* Design calculations are made using this seismic coefficient and the laws of statics.

Richards-Elms provided a logical and systematic method for selecting a seismic coefficient for design of a gravity wall dependent upon the permissible movement.

There has been considerable further research into this approach, as presented or summarized in Whitman and Liao (1984, 1985) and Elms and Richards (1990). There have been refinements, such as an improvement on use of a single sliding block (the Zarrabi model) and alternate equations for relating required seismic coefficient to characteristics of the ground motion and allowable displacement. The possible consequences of multi-directional shaking have been explored. Most - perhaps all - of these effects seem minor compared to three particular difficulties:

Assignment of friction angle for backfill: There are always difficulties in the way of choosing a proper friction angle to characterize sand, especially with a backfill that may not be placed under well-controlled conditions. If the sand is at all dense, then there is the additional difficulty of choosing between peak and residual friction angles. This point has been emphasized in the tests by Steedman (1984) and Aitken et al. (1982). It was clear that the effective friction angle of initially dense backfill decreased as strong shaking continued. Large displacements could be predicted using the residual friction angle, but using this angle badly overpredicted the small, initial displacements within the range of practical interest. Indeed, there were noticeable motions before a shear zone developed fully through the backfill.

Vertical variations of ground acceleration: It is well-known that peak accelerations tend to decrease signficantly with depth below ground surface. This variation is likely to be significant when fackfills become higher than, say, 30 feet. This raises the question: Just what ground motions should be used to predict permanent movement. In particular, use of the ground acceleration at the level of the base of the wall may result in serious underestimates for the movement (Nadim and Whitman, 1983). There is no clear answer to the question, but it would seem best to use the acceleration and velocity at the surface of the backfill, or perhaps an average between the surface and the base of the wall.

Tilting: The Richard-Elms approach presumes that movement results only from sliding, and recommends that walls be dimensioned so as to avoid tilting. However, it appears that actual movements are more the result of tilting. There is as yet no proven method for estimating permanent tilting, in part because of the difficulty in evaluating rotational resistance at the base of the wall. A procedure has been suggested by Al Homoud (1990), and is in reasonable accord with results of model tests and theoretical calculations.

Design

The traditional approach to design of a gravity wall involves:

* Choice of a seismic coefficient, usually 0.05 to 0.15, much smaller than the coefficient corresponding to the peak acceleration for a large design earthquake.

* Use of the Mononobe-Okabe equation to evaluate a static plus dynamic earth thrust, with the dynamic part of this thrust placed at a height of 0.6H.

* Applying an inertia force on the wall itself, based upon the seismic coefficient.

* Providing a margin of safety against both sliding and overturning.

Despite the aforementioned complexities in the actual behavior of a gravity wall, this traditional approach apparently has led to adequate designs. Earthquake-induced thrust against a wall fortuantely is modest compared to the static thrust and the inertial loading upon the wall itself. The good behavior is also likely the result of the considerable conservatism inherent in the practice of designing walls for static conditions. If this conservatism is reduced, more attention should be given to seismic behavior, especially for walls of unusual height.

CANTILEVER WALLS

A cantilever wall is basically a gravity wall where one must also worry about the bending strength of the vertical stem. Hence the magnitude and distribution of the stresses against this stem are important.

Steedman (1984) reports results from model tests using fixed-base aluminum walls with bending stiffness scaled to that of typical concrete walls. He found that the maximum bending moments were essentially those computed using the Mononobe-Okabe theory using the actual peak ground acceleration (with the resultant of static plus dynamic stresses at mid-height) plus inertial loading on the wall. Apparently typical walls will bend enough to develop active conditions. Siddarthan and Maragakis (1989) compared predictions from theory with these results and looked at the effect of varying the stiffness of the wall.

Several points are worth emphasizing. First, with a fixed base wall it is necessary to use the actual expected acceleration, and not some reduced seismic coefficient - if the aim is, as usual, to prevent yielding of the wall. Second, the actual maximum earth pressures likely will exceed those predicted by Mononobe-Okabe, but these will occur at times when they are opposed by inertial forces in the wall itself. Third, significant residual earth pressure will remain after a major shaking, and should be considered when designing for static loads. Fourth, cantilever walls may slide or tilt on their base; residual motions can be estimated using procedures developed for gravity walls. Any such slip or tilt may reduce stresses against the wall. Finally, with very high walls (say > 30 ft.) it may be unconservative to use the acceleration at the base of the wall as input.

BASEMENT WALLS

If a basement rests directly on hard rock and if the outside walls of the basement are well-braced by floors, then it would seems logical to base earth pressures according to Wood's (1973) theory and Yong's (1985) data for unyielding walls. If the aim is to avoid any yielding or cracking of the walls, the actual peak acceleration is to be used. These requirements can lead to quite large lateral earth pressures.

However, usually basements themselves move relative to the foundation soil, owing to soil-structure interaction. Any such movements reduce the lateral dynamic earth pressures. Idriss (1980), having made many dynamic finite element studies for stiff, embedded foundations concluded that stresses against the basement walls were essentially those predicted by Mononobe-Okabe together with the peak acceleration at the surface of the ground outside the structure. Chang et al. (1970) studied dynamic stress measurements made on the embedded portion of the large Lotung (Taiwan) reactor "model", during a series of actual earthquakes. They found that the vertical distribution of dynamic earth pressures changed during shaking, that the peak pressures were less than those predicted assuming elastic behavior of the soil, that residual lateral pressures developed, and that the phasing of the peak earth pressures correlated best with rocking of the structure.

My conclusion, then, is that it should suffice - except where structures are founded at a sharp interface between soil and rock - to use the Mononobe-Okabe equation together with the actual expected peak acceleration.

TIED-BACK WALLS

There appear to be several significant difficulties as regards the design of tied-back walls against the effect of earthquakes.

The first concerns the vertical distribution of dynamic lateral stresses, and the implications concerning the dynamic forces generated in the anchor rods and anchors. Model tests and theoretical studies for walls rotating about the top indicate that the resultant of the dynamic thrust does indeed act well upon the wall - nearly at 0.6H.

Second, the stiffness of the anchorage and tie-rock are potentially very important. If the anchorage is in earth much stiffer than overlying soil, and if the tie rod is oversized, then dynamic earth pressures will be increased. The difference is essentially that of using the actual peak acceleration vs. a reduced seismic coefficient that presumes some yielding of the soil.

Third, tied-back walls are often used to support cohesive soils, and we do not have proven methods for estimating dynamic earth pressures associated with such soils.

Fourth, earthquake shaking implies reduced passive resistance for the toe of a wall. Neelakantan et al.(1990) studied this problem and found that the depth of embedment often must be increased to satisfy seismic design requiresments. One important question is: should wall friction be considered when evaluating passive resistance, and if so is the sense of this friction positive or negative. Model tests by these authors suggest that positive wall friction is indeed present.

Fifth, residual earth pressures must be expected following any major earthquake.

While there have not been major problems with tied-back walls during earthquakes, these are questions that should be given serious attention for major projects. It is especially important to be conservative in the design of ties and anchors. Whitman (1990) describes briefly a project for which a special analysis was performed to evaluate potential residual forces in the rods and anchors.

MECHANICALLY-STABILIZED EARTH

Most studies and tests have to date focussed upon reinforced-earth; i.e. a system involving metal strips laid horizontally in the backfill and connected to plates that make up the face of the wall. Many of the results and methods also apply in principle to retaining structures having other types of reinforcement placed within the soil.

As regards seismic design, current design methods (for summaries, see Fairless, 1989 or TAI, 1985) are based upon model tests at UCLA during the 1970's (Fairless summarizes these tests). Seed and Mitchell in an unpublished report (see TAI, 1985) proposed a simple approach involving evaluation of the inertia force on the stabilized block of soil, dynamic earth pressures (from Mononobe-Okabe) against the back of this block, and reduction factors based on the assumption that these two forces do not peak simultaneously. All these methods are essentially working stress approaches.

However, the permissible-displacement approach can also be applied to the design of such walls, although it is essential to ensure that reinforcement should not fail during an earthquake (Elms and Richards, 1990). Model tests (Nagel and Elms, 1985; Fairless, 1985) have shown that existing procedures for locating potential failure surfaces and for computing strip forces are reasonably correct, have provided new information concerning friction on strips and distribution of stresses along strips, have suggested suitable vertical distributions for dynamic stresses, and have shown the general validity of the premissible-displacement approach.

These model tests, and also those by Kutter et al. (1990), have also indicated that transient motions at the top of a mechanically-stabilized earth walls may be greater than those atop more conventional walls. These observations can be of consequence where any form of structure (such as a sound wall) is to be placed over a mechanically-stabilized wall.

WALLS AT WATERFRONTS

In contrast to the preceding discussion of various types of walls supporting dry backfills, which have by and large performed well during earthquakes, retaining structures at waterfronts have a terrible track record. Okamoto (1984) has excellent summaries of experiences in Japan, and Whitman and Christian (1990) provide additional information. Gravity walls and caissons have typically fared very poorly, partly because of liquefaction of backfill but also because foundation soils at waterfronts so often are weak and compressible. Anchored bulkheads have not done much better, primarily because of liquefaction. There are examples of good behavior such as the previously mentioned anchored bulkhead at San Antonio, Chile. Any form of wall potentially can be safe, if proper care is taken with regard to backfill, foundations and proportions for the parts of the wall. Okamoto (1984) presents design procedures followed in Japan.

Obviously the culprit is water and excess pore pressures. It is useful to break the problem down into parts.

Liquefaction

By liquefaction is meant the build-up of excess pore pressures that remain for a time even after shaking has stopped. If these excess pore pressures are sufficiently high and the soil is sufficiently loose, the soil may lose most shear resistance and either settle considerably or - if unconfined - flow away.

If ground is level, numerous methods exist for predicting whether the build-up of excess pressures will be small, moderate or large. It would seem that these same procedures may be applied to estimate pore pressure build-up in backfills behind retaining walls (e.g. Fujii et al., 1989). These pressures add to those overturing the wall, and in the limit the backfill becomes a heavy fluid.

However, there are other aspects to the problem. As the pore pressures rise and the soil softens, the back-and-forth movements of the soil increase and this tends to throw more force against the wall. To make things worse, a resonant condition may develop and motions akin to sloshing develop. Looking at results from model tests, Steedman and Zeng (1990) suggest that critical conditions develop when excess pore pressures reach about 80% of the initial vertical effective stresses.

On the other hand, when a dense sand experiences large shear strains, pore pressures tend to decrease and can even become negative. Such changes tend to stabilize and stiffen a sand. Thus, it is possible that a sand susceptible to pore pressure build-up when the suface is flat will experience only limited lateral movement toward a slope or retaining wall. The problem today is that the likely amount of lateral movement cannot be estimated with confidence. Methods have been and are being developed (see Table 2, also National Research Council 1985), but have not yet been proven reliable. More model tests are needed, against which computational methods can be checked.

Given this situation, the tendency today is to densify backfills so as to virtually eliminate the possibility of significant pore pressure build-up. Such steps can be extremely expensive. We need rules for establishing a degree of densification sufficient to ensure that lateral movements remain within acceptable limits. Here is a major challenge for the future.

Dynamic Pore Pressures

This phase refers to pore pressures associated with the horizontal acceleration of water; they fluctuate back and forth during shaking.

There are such pressures on the waterside of a bulkhead, and their magnitude and distirubiton are given by the theory of Westergaard (1933). The total thrust from such a dynamic pressure is:

$$\Delta P_w = 0.583\ \gamma_w H^2 A \qquad (3)$$

An equation is also available for inclined surfaces (see Matsuzawa et al. 1985).

In general there are also dynamic fluid pressures within the backfill. With a very coarse soil, where the pore water can move readily relative to the mineral skeleton, they would again be given by the Westergaard theory. As a soil become finer, however, the mineral skeleton impedes movement of water under the action of inertial forces. Dynamic pore pressures decrease, but the mineral skeleton in

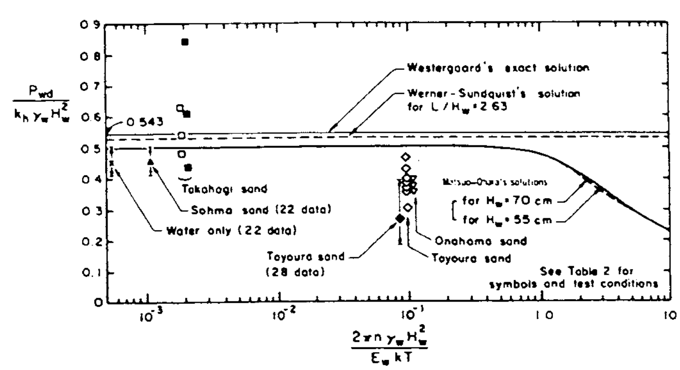

Fig. 6. Dynamic pore pressures as a function of permeability (from Matsuo and O-Hara, 1965)

effect acquires more mass. Matsuo and Ohara (1965) derived an equation for this situation. Matsuzawa et al (1985) compared available test data with this theory (see Fig. 6). The ordinate of this figure is the dynamic pore water pressure normalized by $\gamma_w H^2 A$, while in non-dimensional abscissa n is porosity, E_w is the compressibility of water, k is the permeability and T is the period of the applied accelerations. Matsuzawa et al. went on to develop a generalized apparent seismic coefficient, considering the effect of both mineral skeleton and pore fluid, and to provide an example indicating that the total lateral stress may be relatively insensitive to the permeability of the backfill.

During sinusoidal shaking, there theoretically is a simultaneous increase of pressure from the backfill and a decrease of water pressure on the waterside. During earthquake-like shaking, it seems unlikely that both peak values would occur simultaneously.

Lowering of Water Level in Harbor

As a result of a tsunami or related effects, the water level against a waterfront structure may decrease temporarily with the level inside the backfill remaining unchanged. Such a destabilizing effect should be accounted for in design.

FINAL COMMENTS

This paper has emphasized key aspects of behavior rather than reciting design rules, and has dealt with what is unknown as well as what is known. Because of better model tests and more complete theoretical analyses, much has been learned during the past decade. Hopefully this pace will continue during the next 10 years, with new significant advances concerning the most perplexing of today's problems - more economical but adequate waterfront structures.

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