DESIGNING RETAINING STRUCTURES AGAINST THE EFFECTS OF EARTHQUAKES

Robert V. Whitman[1]

ABSTRACT

Theories for dynamic earth pressures are reviewed, together with experimental evaluations. The performance of retaining structures away from waterfronts continues to generally be good. Experiences with various types of structures - gravity, cantilever, mechanically-stabilized, anchored and basement - are summarized and discussed with regard to dynamic and residual forces. There have been both good and bad experiences with waterfront structures - gravity walls and caissons, anchored bulkheads and trestle piers. A study of the very important experiences at Kobe is discussed in some detail.

INTRODUCTION

In preparation for this lecture, I have reread the 1970 paper that I co-authored with Professor H. Bolton Seed (Seed and Whitman, 1970).[2] The paper begins by observing that "few cases of retaining wall movements or collapses of walls located above water table have been reported in the literature on earthquake damage", but that there have been extensive failures of quay walls and other waterfront structures. The main body of the paper focuses upon the Mononobe-Okabe equation as a tool for evaluation of lateral stresses. A final section of the paper concerning design of retaining walls contains the following comments:

"....it should be noted that the factor of safety provided in the design of the wall for static pressures may be adequate to prevent damage or detrimental movements during many earthquakes."

"Because of their special importance in times of disaster, the design of port facilities to withstand earthquake effects merits the most careful attention of the design engineer. The substantial number of quay wall failures in previous earthquakes provides little ground for complacency concerning the importance of lateral pressure effects in their design."

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[1] Professor Emeritus, Dept. Civil and Environmental Engineering, Massachusetts Institute of Technology, Cambridge Massachusetts

[2] I feel that I am free to speak well of that paper, since Professor Seed prepared almost all of the paper that dealt with retaining structures.

I am struck that these statements from 26 years ago characterize quite well the situation today.

Since 1970, there have been observations of failure and non-failure during many earthquakes, a considerable volume of theory and a number of excellent shaking tests at model scale. These results certainly have sharpened our understanding of the behavior of retaining structures during earthquakes, and thus have made possible more intelligent application of the principles set forth decades before - even though in a broad sense there has been little change in the approaches to the design of such structures.

I cannot claim to provide a complete state-of-the-art assessment, but only glimpses of some of the more interesting developments of the past decade. Since typical approaches to design involve evaluation of a "dynamic" lateral stress to be applied to a retaining structure, it is appropriate to begin with a review of what is known about lateral stresses during earthquake excitation. Then various types of retaining structures will discussed in the light of both theory and actual experiences with such structures.

THEORY AND MEASUREMENTS OF EARTHQUAKE-INDUCED LATERAL STRESSES

The Mononobe-Okabe Equation

I believe that most readers or listeners understand that the Mononobe-Okabe (M-O) equation[3] is an adaptation of Coulomb's equation for lateral earth pressure. Horizontal and vertical static forces are applied to the backfill to represent reversed acceleration forces. The resulting equation for active thrust is:

(1) $$P_{AE} = 0.5\ \gamma\ H^2\ (1-k_v)\ K_{AE}$$

where γ and H are the unit weight of the backfill and vertical height of the wall. k_v is the vertical acceleration coefficient - positive when directed upward, and hence corresponding to a downward acceleration. K_{AE} is the active stress coefficient, and is a function of (a) the geometry of the wall and backfill, (b) the friction angles of the backfill and at the wall-backfill interface, and (c) the horizontal acceleration coefficient k_h - which is positive when directed at the wall (and hence corresponds to an acceleration directed away from the wall into the backfill). Fig. 1 shows a typical plot of K_{AE} vs. k_h, for active conditions. Note that K_{AE} includes the static component of active earth

[3] This equation comes from the work of several Japanese engineers in the decade following the Great Kanto Earthquake of 1923. It appears that Professor Seed originated the name "Mononobe-Okabe equation" during his work in preparation for the 1970 paper and lectures that preceded it. The name has stuck in most of the world, but is not universally recognized in Japan.

pressure; that is, for k_h=0 we see that K_{AE} equals the Coulomb value for K_A. A simple and useful approximation for K_{AE} (Seed and Whitman, 1970) is:

(2) $$K_{AE} = K_A + (3/4)k_h$$

The M-O derivation does not by itself indicate the distribution of lateral stress over the height of a wall. By making additional assumptions, various authors (e.g. Prakash and Basavanna, 1969) have estimated the height of the resultant force. An upper limit for the location of the dynamic component of thrust results from the assumption that the backfill is uniform and elastic (Wood, 1973); in this case the dynamic thrust acts at 0.63H above the base. After reviewing the various available results, Seed and Whitman suggested applying the dynamic component at 0.6H. The height of the combined static and dynamic thrusts thus would fall between 0.33H and 0.6H, depending upon the intensity of the ground motion; 0.40H to 0.45H are often suggested as typical values.

Experimental evidence: A natural question is: How accurate is the M-O equation? There is but little evidence from measurements made during actual earthquakes. Fukuoka and Imamura (1984) report measurements using stress cells in the face of gravity walls 1.5 m high. Peak accelerations near the walls ranged from 0.04g to 0.07g. They report that dynamic earth pressures were 9-16% greater than calculated from the M-O formula. In addition a cantilever wall was constructed so that total forces acting upon the wall could be measured. Again the actual earthquakes were small: up to 0.03g peak acceleration. Results scatted considerably about values predicted from M-O. Because there is such scant evidence from the field, it is necessary to rely primarily upon small scale tests.

The best tests have been those conducted by Sherif *et al* (1982). These tests used dry sand, which is the condition for which the M-O equation was derived. The wall was slowly moved outward in a controlled fashion while the backfill was being shaken at its base. The load acting upon the wall was deduced from measurements of forces at the points where controlled outward movement was imposed This thrust showed a cyclic fluctuation superimposed upon a mean trend that decreased with movement and stabilized at the static active value. Once the mean value stabilized, the peak cyclic force was used to evaluate K_{AE}. Typical results appear in Fig. 2. Sherif and his colleagues found further than that the height of the resultant force moved upward during shaking, reaching a height of about 0.45H above the base.

The conditions existing in these experiments were just those assumed by the Mononobe-Okabe equation: essentially uniform acceleration throughout the backfill and wall movement sufficient to mobilize fully the shear resistance of the soil. Hence the results are a very good demonstration of the essential correctness of the theory. However, as will be discussed subsequently, the remain questions as to the applicability of the theory in actual problems where the outward movement of the wall is not controlled as in the experiments.

A solution for seismic lateral pressures for soils with cohesion has recently been published (Richards and Shi, 1994).

Stresses Against "Rigid Walls"

A "rigid wall" is, in this context, a wall that is firmly attached at its base to an underlying stratum, and is itself undeformable. Thus a "rigid wall" moves exactly as does the underlying stratum. Fig. 3 presents stress distributions computed assuming that the backfill is elastic - with several different assumed variation of modulus with depth, and experiencing everywhere a fixed horizontal acceleration equal to the acceleration of the base. These stresses are those caused by the base acceleration, and do not include any static stresses. For modulus constant with depth and typical values for Poisson's ratio, the total dynamic thrust against the wall is close to $\gamma H^2 k_h$ - that is, about 8/3 times the dynamic thrust from the M-O equation. The height of the resultant for this dynamic thrust lies about 0.6H above the base. For more realistic variations of modulus with depth, both the total thrust and the resultant's height are somewhat less.

The most thorough experimental investigation for this situation appears in Yong (1985). Tests were conducted on a shaking table with a wall about 1/2 m high. The results are in good agreement with theoretical solutions.

Dynamic Interaction Between Wall and Backfill

The previously-discussed solutions assume that acceleration is constant throughout the backfill, that the frequency of excitation is much smaller than the fundamental frequency of the backfill, and ignore dynamic interactions between backfill and a wall moving in response to forces exerted by the backfill. Various deviations from these simple conditions have been investigated.

<u>"Rigid wall" supporting elastic stratum:</u> If the frequency of excitation at the base of backfill approaches the fundamental frequency of the backfill (considered as an infinitely long stratum), then acceleration will generally increase up through the backfill. This amplification also affects the thrust against the wall. In these circumstances, using the base acceleration to evaluate peak thrust would underestimate this thrust.

This situation has been analyzed by Veletsos and Younan (1994a), assuming the backfill is elastic and uniform. It appears that the amplification of earth thrust at resonance is, if anything, less than the amplification of peak acceleration. The increase in thrust is controlled by radiation damping of energy from the wall to the far parts of the backfill. Thus is should be conservative to estimate thrust using Wood's results together with the peak surface acceleration.

Tilting or bending wall: From Wood's studies, it is well known that tilting of a wall causes a reduction in lateral thrust against the wall - even if the backfill is assumed to remain elastic. Veletsos and Younan (1994b) have investigated the case where a weightless wall is supported by a rotational spring. Steedman (1984) reports results from centrifuged model tests upon cantilever walls, and concludes that the dynamic earth thrusts were essentially those predicted using the Mononobe-Okabe equation. Especially with gravity walls, inertial loading upon the wall itself increases the tendency to tilt (or bend). This was very apparent in centrifuged model tests by Andersen *et al* (1991). Here the tilting support for the gravity walls was quite soft and earth thrusts at the time of maximum outward tilt were significantly smaller than those predicted using the M-O equation. In a theoretical study by Al-Homoud (1990), actual separation of wall and backfill was predicted during strong, rapid shaking.

A key "parameter" is the relative stiffness of the wall and backfill when exposed to horizontal inertial forces. If the wall is "stiffer" than the soil, dynamic lateral thrusts may be larger than the Mononobe-Okabe value - approach the value suggested by Woods. On the other hand, if the wall (because of the softness of the foundation) is less stiff than the soil, the dynamic thrust (at moments of maximum outward tilt) may be smaller than the M-O value. We do not know well the transition condition, but it may well be about at the typical stiffness of a cantilever wall.

Another important feature of both Steedman's and Andersen's tests was finding significant residual thrusts at the end of shaking. It is not clear whether these increased lateral stresses resulted from densification of the backfill or from "wedging" of the Coulomb failure wedge against the wall. The residual stresses reached values somewhat higher than the static "at rest" stress.

Sliding wall: Richards and Elms (1979) suggested an approach to evaluating the permanent displacement experienced by a gravity wall that slips on its base because of momentarily-existing seismic forces. There is a considerable literature regarding this approach and the various approximations involved; e.g. Whitman and Liao (1984).

Saturated Backfills

Westergaard (1933) provided a solution that gives the dynamic thrust against a vertical wall retaining an infinitely long reservoir of constant depth, when there is a constant (with time) base acceleration:

(3) $$P_W = (7/12)\, k_h\, \gamma_w\, h^2$$

where γ_w is the unit weight of water and h is the total depth of water. The resultant acts at 0.4H above the base.

Several authors have discussed the use of the foregoing equation to to evaluate the dynamic thrust from the pore phase of a saturated soil, and how this water thrust might be added to the thrust from the mineral skeleton Matsuzawa *et al*, 1985; Ebeling and Morrison, 1992; Ishibashi *et al*, 1994). However, the theoretical treatment of such soils is still a very confused subject. It is difficult to formulate a completely consistent set of assumptions.

Ishibashi *et al* performed shaking table tests with rigid end walls, while Whitman and Ting (1994) tested a simulated anchored wall. Tests such as these will eventually lead to useful rules for evaluating thrust from saturated soils. For the moment, the best guidance is to use either the Woods of Mononobe-Okabe equation with total unit weight.

RETAINING STRUCTURES WHERE LIQUEFACTION IS NOT A PROBLEM

It is convenient to start with discussion of situations where it is not necessary to consider liquefaction in either backfill or foundation soils. It does not exclude saturated backfills of a character or density that precludes liquefaction.

Experiences During Earthquakes

<u>Gravity walls:</u> There are few clear examples of firmly-supported gravity walls, with non-liquefiable backfill, that have performed unsatisfactorily during earthquakes. Reconnaissance reports from the San Fernando and Northridge earthquakes mention cracked abutments and one or two observations suggest some displacement of abutments or wingwalls. However, details of abutments and wingwalls are not clear, and in any case these damages were minor compared to other effects upon the bridges. The report from Northridge (Hall, 1995) states that: "At no site was there any indication of foundation or abutment failures as a primary cause of collapse." [4]

On the other hand, it is clear that not just any wall can be counted upon to perform satisfactorily during strong shaking. Grivas and Souflis (1984) describe a case of excessive movement of a concrete wingwall at a bridge abutment in Greece. This wall was 5 m high and 1 m wide at its base. It was "attached" to the main abutment along one edge, but broke away from the abutment during 3 nearby earthquakes (magnitudes 6.2-6.6, causing peak accelerations between 0.03g and 0.22g) that occurred during an 8-day period in 1981. The permanent movement ranged from 0.08m at the base to 0.15m at the top. As a second example, the EERI reconnaissance report upon the 1989 Loma Prieta Earthquake (Benuska, 1990) has a photo of a badly bulged crib wall - which is a type of gravity wall - along a roadway in mountainous terrain. Stewart et al (1994) report modest deformations of crib walls during the 1994 Northridge Earthquake.

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[4] I have seen reference to a wall that collapse on Guam, but as yet I have not recovered the reference.

There are numerous incomplete and vague mentions in the literature of bridge abutments that developed significant tilts as a result of earthquakes. Examples in recent literature (e.g. Costa Rica earthquake of April 22 1991, EERI 1991; Philippines earthquake of July 16 1990 (Schiff 1991) clearly implicate liquefaction of supporting soils. Older references do not provide sufficient information to ascertain the cause of tilting, but there certainly is the possibility that soft but non-liquefying soils did not possess sufficient stiffness when acted upon by overturning moments.

Cantilever walls: Aside from the above-cited mentions of cracking at bridge abutments, there is scant evidence of damage to cantilever walls during earthquakes. Damage to drainage channels during the San Fernando Earthquake of 1971 is often cited (Murphy, 1973). Here there were several thousand feet of open channels with concrete walls connected to concrete inverts (and also concrete box culverts). However, these channels were in zones of tectonic distortion of the ground, and crossed compression fault ridges. One channel was in an area with evidence of liquefaction effects. Undoubtedly earth pressures against the walls were large enough to yield reinforcing steel at the base of the cantilever walls - but these pressures likely could not be related to seismic earth pressure theory.

During the Hokkaido earthquake of 1993, two different walls supporting highway embankments moved outward excessively (Chung, 1995). While details are not reported, these were apparently cantilever walls, supported on piles. The walls were 4 m high. One wall rotated outward 1-2°, the other 2-3°. Cracks and settlement developed in the fill near the walls, possibly as result of poor compaction near wall.

Mechanically-stabilized walls: Mechanically-stabilized walls - whether used to stabilize excavations or as permanent treatment for cuts and embankments - have usually performed well during even strong earthquakes. Benuska (1990) makes no mention of difficulties with such walls, and a specific survey of stabilized cuts and fills (Kutter *et al*, 1990) found a few suggestions of difficulties. Sandri (1994) and White and Holtz (1994) surveyed a variety of slopes and walls, reinforced with geosynthetics, affected by strong shaking during the 1994 Northridge Earthquake; all performed very well. Stewart *et al* (1994) likewise report that reinforced soil walls did well during the Northridge event. (Crib walls, as noted above, did here and there experience excessive bulging, both during the Loma Prieta and Northridge Earthquakes.)

There was one picture from Kobe showing a reinforced soil wall that had collapsed, but I have no further information concerning this matter.

Anchored walls: The literature is also largely silent concerning the performance of f wlls supported by anchors placed deep into the soil being supported by the wall.[5] Fukuoka and Imamura (1984) report measurements made at a special modest-scale

[5] I recall a report by Fragazy upon a case successful excavation support during the Whittier Narrows, California earthquake, but again I have not as yet unearthed the reference.

tied-back wall exposed to a moderate earthquake. The dynamic stresses were small compared to those prior to the shaking.

Basement walls: There have been numerous incidents of stone foundation walls experiencing forms of failure during earthquakes. However, I have found no clear references to sound, concrete basement walls being damaged.

This is one situation for which there have been important if limited recordings of actual lateral earth pressures during earthquake ground shaking. A 1/4-scale model of a nuclear reactor structure was constructed at a site in Lotung - a seismically active region in northeastern Taiwan (Chang *et al*, 1990). The structure was embedded 4.57 m into the soil, and was instrumented extensively - including earth pressure cells on its sides and base. Measurements were made during three earthquake events causing peak accelerations at the site of 0.17g to 0.26g. The principal conclusions from analysis of these measurements were:

- Dynamic earth pressures were greatest at mid-depth.

- Dynamic thrusts were similar in magnitude to those predicted by the Mononobe-Okabe equation and much less than those predicted from Wood's theory for pressures against an unyielding wall.

- The phasing of dynamic earth pressures was related to movements of the structure relative to the soil, being greatest when the structure rocked against the backfill. Thus the phasing was similar to that observed in Andersen's tests upon tilting retaining walls.

Understanding Behavior

Thus the generally good performance of retaining walls continues, with the possible exception of cantilever walls. However, field observations have done little to advance our understanding of this good behavior, and such new understandings as we do have have come almost entirely from fundamental studies such as those described above. Understanding the evidence from the field is complicated by a variety of factors: the range of design practices with regard to "static" loadings - choice of lateral soil pressures (active vs. at-rest), safety factors - and actual pre-earthquake pressures as affected by compaction of backfill, etc.

Gravity walls: Several factors conspire to make gravity walls "good actors" during earthquakes. One is typically conservative design with regard to static loadings - conservative choice of friction angles at base of wall and in backfill, neglecting the potentially beneficial effects of wall friction, assumption of greater- than-active-condition lateral stresses. Thus, walls can sustain quite significant base accelerations before any sliding can begin. Momentary dynamic thrusts in excess of the threshold for slip usually cause only small residual displacements, and if larger motions begin to occur the thrusts can reduce dramatically. Lastly, only small wall movements are necessary to alleviate any large residual thrusts.

Thus, the standard of practice for gravity walls appears, with the possible exception of crib walls, to be adequate. This practice does imply that a seismic coefficient be applied to the mass of the wall itself; it is not enough to consider just increased lateral thrust. Engineers must be alert to ensuring adequate resistance to moment at the base of the wall.

There has been interest in a method of design based directly upon allowable permanent displacement of gravity walls (Richards and Elms, 1979; Liao and Whitman, 1984). However, the earlier discussion concerning complex and uncertain variations of lateral thrust during shaking mean that it can be difficult to make good predictions for permanent displacements. On one hand, it would appear that thrusts smaller than those given by the M-O equation may exist while permanent displacements are occurring. This may be especially true when there is tilting. On the other hand, at the end of shaking residual thrusts will remain. Some initial thoughts concerning the prediction of permanent tilt appear in Whitman (1992).

<u>Mechanically-stabilized walls:</u> Mechanically-stabilized walls are essentially designed as gravity walls, the purpose of reinforcing material being to tie together the earthen mass near the face. When well-tied together and with strong enough connections at the facing, and when backfills are free from problems associated with water and foundations are firm, such walls should behave well just as do gravity walls. Experience during large earthquakes, although still limited, suggest that current design practices are adequate.

<u>Cantilever walls:</u> A typical cantilever wall has two potential modes of failure: sliding (or tilting) of the wall plus block of soil overlying the base of the wall, and bending failure of the vertical stem. When problems arise, they typically concern the stem, which is a rather stiff element that may be overstressed in bending at its base by deformations less than those required to reach an active condition. Conservative practice that prevents lateral slippage of tilting of the wall as a whole may result in both large dynamic thrusts and significant residual thrust against the stem. Problems can become especially severe when wing walls are restrained at their tops by adjacent abutments.

Experience with cantilever walls is mixed, with enough suggestions of difficulties to provide a warning that lateral earth pressures during earthquakes may be larger than generally assumed. Where walls are rigidly restrained at their base, and where yielding and permanent rotation must be avoided, it may be more appropriate to treat walls as basement walls (see below).

<u>Anchored walls:</u> The behavior of anchored walls is generally intermediate between that of gravity and cantilever walls. The analysis of field test results by Fukuoka and Imamura (1984) indicate the potential complexity of behavior, especially when the

backfill is cohesive. While the flexibility of walls and anchorage arrangements can provide some relief from dynamic and residual thrusts, these systems are generally less forgiving than gravity walls. While the limited experience with anchored walls has been good during earthquakes, I would warn against moving too quickly to less conservative practices. Obviously, conservatism and ductile detailing of rods, connections and anchorage is especially important.

Basement walls: The problem of selecting design lateral pressures for these walls deserves special discussion. Obviously such walls, being supported near their top (and possibly at other elevations) by floors, tend to be quite rigid. Thus at first site it might be assumed that dynamic thrusts considerably greater than those from the M-O equation should be used for design. However, the limited evidence from the field indicates otherwise.

If a building is founded securely on rock, as shown in Fig. 4a, then it would seem appropriate to use a theory for dynamic lateral stresses against a "rigid" wall. Wood's simple equation might be used, but one must decide whether to use the acceleration at basement level or the acceleration at the surface of the ground - or some average. Alternatively one might use a theory that explicitly accounts for dynamic amplification of motions in the ground aside the structure.

On the other hand, if the stiffness of the earth material underlying the structure is not greatly different from that above the structure's base (Fig. 4b), then the structure may move laterally as much or more than the adjacent soil. This is the situation inferred by Chang *et al* (1990) from field observations. From a number of finite element studies of soil-structure interaction, Idriss and Moriwaki (1982) suggested that the lateral stresses could be estimated using the M-O equation with k_h corresponding to the peak acceleration at ground surface - which is consistent with Chang *et al.*

Whichever way lateral thrusts against basement walls are evaluated, here is a situation where it is appropriate to use actual expected ground accelerations rather than reduced seismic coefficients.

RETAINING STRUCTURES AT WATERFRONTS

Japanese engineers have been the leaders in developing practice for the design of waterfront retaining structures. As the discussion that follows will indicate, they often have not had good success, but they have studied the lessons of failures and thence improved their designs. Good references are Okamoto (1973, 1984) and Ports and Harbours Bureau (1991). Whitman and Christian (1990) provide a partial summary of design practices. Werner (1990) has summarized worldwide experience concerning the effects of earthquakes upon port facilities. His general conclusion is that many details of design practice - such as the choice of seismic coefficient - are unimportant compared to the problem of liquefaction.

Caissons and other gravity walls

Caisson-type quay walls typically consist of large, hollow-but-sand-filled, concrete boxes sunk into place. They have been used widely in Japan during the latter portion of this century. Fig. 5 shows a typical arrangement. Earlier, typical construction consisted of gravity walls composed of very large, dressed rocks - or perhaps of poured concrete. Typically the total unit weight of soil is used to evaluate dynamic lateral forces. Where harbor bottom soils are cohesive, caissons are placed upon a bed of sand or rubble dumped in a trough cut into the clay. Stone rubble sometimes in placed beneath the toe to increase bearing resistance in this area. Usually there has not been special treatment of the sand to achieve high relative densities. While care is often given to the particle size of backfill immediately behind the caisson or wall to ensure good drainage of tidal fluctuations, it has not been general practice to densify the bulk of the backfill.

Experience during earthquakes: Caissons and other gravity walls have almost always fared quite poorer during strong ground shaking. Large outward movements have been common, and the Japanese literature is replete with examples. However, the problem has not been unique to Japan. During the 1985 in Chile, there was a dramatic overturning of a gravity wall at the port of San Antonio, and significant movements (large enough to cause damage to crane rails and partial closure of berths) in Valparaiso. During the Northridge Earthquake of 1994, a 4-5 foot high seawall at Redondo Beach moved outward 15-20 feet as a result of liquefaction of the backfill, with damage to structures and facilities located behind the wall.

Such failures are usually attributed to liquefaction of the backfill. However, is has been suspected that failure (possibly liquefaction) of the soils on which the caissons were founded also played a major role. New experiences during two recent Japanese earthquakes are particularly instructive.

Iai *et al* (1994) summarize observations at Kushiro Port, where accelerations approached 0.5g during the large earthquake of 1993.

Design k_h	Steps against liquefaction	Lat. disp. (Δ/H)
0.15	No	20%
0.2	No	0.5-10%
0.2	Yes	<3%

Here Δ is the maximum lateral displacement, which typically was accompanied by some settlement of the caisson. H is the depth of water. Use of a larger k_h for design resulted in a greater width/height ration for the walls. To avoid liquefaction of the backfill, sand compaction piles were used well away from the caissons, with gravel drains at close distances (Fig. 6).

The experiences at Kobe Port during the Great Hanshin Earthquake of 1995 are providing important case studies of port structures (Inagaki *et al.*, 1996). At the time of the earthquake, the port had 186 quay walls, about 90% of which were of a caisson type. The peak acceleration in the area of the walls was somewhat over 0.5g, and was significantly stronger in one direction than the other. There was widespread evidence of liquefaction of the fill behind the walls.

Most of these caisson walls displaced toward the sea by about 3 m on the average, with settlement and inclination toward the sea of about 4°. The maximum lateral displacement was about 5 m. Figs. 7 and 8 show typical behavior. Displacements were greatest when a wall was positioned normal to the direction of strongest shaking. These walls had been designed using seismic coefficient of 0.10 and 0.18. The width:height ratio typically was 0.6 to 0.75. The harbor bottom clay at the locations of the walls had been replaced by granular material. The permanent displacements tended to increase with the thickness of sand placed beneath the caisson.

Two walls had been designed using $k_h = 0.25$, resulting in a width/height ratio of 1.4. The maximum outward movement of these walls was about 0.2 m. There were, however, additional circumstances that may have influenced the performance of these walls. These walls were constructed in front of an old caisson (Fig. 9), with an intervening stone backfill. In addition, these walls were oriented such that they were not affected by the largest component of ground motion.

<u>Understanding the evidence:</u> There is every reason to believe that liquefaction of backfill behind caissons increases greatly the outward thrust against sea walls. However, there no actual measurements of these increased pressures.

The paper by Inagaki *et al* (1996) discusses the role of foundation failure in contributing to permanent displacements. This is a comprehensive study, and I believe represents a landmark in the understanding of behavior of caisson-type quay walls during earthquakes. There were several different parts to the study.

- Detailed observations were made during dives in front of one wall. While it appeared that foundation rubble had been pushed out before the displacng wall (Fig.), there was no evidence of liquefaction.

-Model shaking table tests were performed, with walls 0.8 m high. Scaled permanent movements were similar to those observed in the field; the caissons inclined, pushing a rubble mound outward. Pore pressures in the sand beneath the caissons increased significantly, but not reach a liquefied state (Fig 11). The sand just behind the wall also did not reach a zero-effective-stress state. This result is in agreement with absence of sand boils directly behind the actual walls even though such boils were common further from the walls.

-An effective stress analysis using finite elements was undertaken. Fig. 12 shows the pattern of deformation within the backfill and underlying sand, and confirms again the importance of deformations in the sand supporting the caisson.

Observations and the effective stress analyses for Kobe also shed further light upon the potential effectiveness of remedial measures. There were two quays where sand compaction piles were placed into the clay beneath the caissons, with sand drains under the backfill just behind the wall (Fig. 13). (I presume the sand drains were to accelerate settlement of the portion of the backfill.) Immediately adjacent was an identical caisson where all of the clay under the caisson had been replaced by sand. The vertical settlements for the caisson over sand compaction piles were about 1/4 of that over replaced sand. Horizontal movements were reduced by 2/3, but were still substantial. The effective stress model was used to investigate three possible remedial actions: densifying sand beneath the caisson, densifying the backfill and both. Densifying only the backfill reduced horizontal and vertical displacements by about 30%; densifying the underlying sand only reduced the horizontal displacement by about 40% and vertical settlement by about 50%; densifying both reduced movements between 55% and 60%.

This work concerning was still in progress when the paper was written, and we can all look forward to further details concerning this important study. Other studies concerning the Kobe experience are also underway.

Anchored Bulkheads

<u>Experiences during earthquakes:</u> There have been many failures of anchored bulkheads during earthquakes. Such problems have been frequent in Japan, but have been observed in Chile in Puerto Montt in 1960 and San Antonio in 1985) and elsewhere. In numerous cases, bulkheads have essentially collapsed. These failures are generally attributed to liquefaction of loose backfill - it has been common to observe sand boils in lay-down areas behind failed walls - with stability possibly compromised further by liquefaction in sands below the dredge line. Either anchor bars have broken, or the anchors - being in liquefied backfill - have lost their resistance. In many other cases, outward movements at the tops of bulkheads have been large enough - 0.5-1 m - to distort crane rails and make a berth unusable. Either the anchorage was overloaded and yielded, because of increased pressure on the wall from liquefied backfill, or perhaps the anchorage was in partially liquefied soil. Iai *et al* (1995) give an example from Kushiro Port where the anchorage remained intact but the wall bulged so much as to crack (Fig. 14).

There are at least two examples of good performance. Iai(1995) describe a quay at Kushiro Port where the backfill had been stabilized by gravel drains close to the wall and sand compaction piles out to 1.7 times the water depth (Fig. 15). There was no observable effect of the large earthquake in 1993. In San Antonio, Chile, one berth with an anchored bulkhead survived the large 1985 earthquake with only about 0.1 m outward displacement, and was able to remain in service (Ortigosa *et al,* 1993). This wall had been designed according to Japanese practice using k_h = 0.2. The backfill had been densified to as to successfully avoid liquefaction.

Understanding the evidence: For most bulkheads, inertia forces on the wall itself are insignificant compared to earth pressures. Hence bad performance results either from increased earth pressures against the wall, loss of toe support owing to liquefaction of the soil in which the bulkhead is embedded, failure of the anchorage, or a combination. Zeng and Steedman (1993) present results from centrifuge model tests. At shakings small enough that liquefaction did not occur, they observed performance in accord with the common pseudo-static analysis for a bulkhead.

Iai *et al* (1966) conclude that measures against liquefaction plus overall design practice (Ports and Harbours Bureau, 1991) are adequate to provide resistance to strong earthquake shaking. As I write this, I do not have that reference available. Hence I can only suggest that the principles outlined when discussing the theory of lateral earth pressures be followed. Preventing liquefaction of backfill is, of course, a must.

Pile-supported piers

The use of a pile-supported deck - also known as a trestle pier - is increasingly common in the Americas. A typical arrangement is shown in Fig. 16. The deck extending out to the berth is supported by piling, with horizontal resistance in the longitudinal provided by batter piles. The lay-down area behind the deck typically is filled ground, with a slope under the deck - or a wall or both - as transition. The deck structure typically has been designed using a modest seismic coefficient, and quite often the filled ground has not been densified or stabilized.

Experience during earthquakes: This type of pier was present in the Ports of San Francisco and Oakland during the Loma Prieta Earthquake of 1989 (Benuska,1990). At San Francisco, there was settlement of the lay-down area behind the piled deck, and the resulting differential settlements interrupted functioning of cranes. The damage was more serious at Oakland, where the fill liquefied. Batter piles broke free from the deck, partially because of the seismic forces acting on the deck and partly because of lateral spreading of the liquefied fill. Lateral movements of the decks and settlements of the fill led to major damage to cranes, and closed portions of the port for up to a year.

An old trestle pier at Valparaiso, Chile was very badly damaged during the 1985 earthquake, primarily because of inadequate structural resistance. A pier arrangement, similar to those at San Francisco and Oakland, existed at San Antonio, Chile - where peak accelerations of about 0.5g are thought to have occurred in 1985. The pile-supported deck performed reasonably well, but there were settlements in the lay-down area - with partial disruption of operations.

Inagaki et al (1996) mention that a trestle pier at Kobe performed well.

Understanding the evidence: From these and other experiences, the use of batter piles to resist seismic forces has acquired a bad reputation. However, it is not that the concept is wrong. Rather, the problem commonly has been in the analysis of loads reaching the batter piles. In order to develop the necessary horizontal resistance, very large axial forces must occur in these piles. By providing strong moment-resisting connections between tops of some or all vertical piles and the horizontal structural members, seismic forces can also be resisted by frame action. Where analysis and design have been adequate, performance has been satisfactory.

With proper structural analysis, such designs can perform adequately during strong shaking. The need for densification of loosely-placed fills is, of course, evident.

Drydocks

Drydocks typically are U-shaped structures embedded in the surrounding soil. Since the normal ground water typically is quite high, the side walls must be designed for large water pressures as well as earth pressures. As a result, dock walls tend to be quite stiff, and hence dynamic lateral soil pressures might be large Assuring against uplift when a drydock is dewatered is another important design problem. One potential concern is that liquefaction of adjacent soil might decrease the side friction available to hold the dock down against uplift. Ebeling and Morrison (1992) have a brief discussion concerning the evaluation of lateral stresses against drydocks.

I am not aware of any failures of drydocks during earthquakes.

FINAL REMARKS

In closing, I will offer four comments concerning the future of engineering for earthquake effects upon retaining structures.

1. Obviously attention must be paid to stabilizing soils against liquefaction - both soils in backfill and soils that provide support at the toe of retaining structures. Improved practices and rules must evolve.

2. Assuming that liquefaction has been eliminated as a problem, engineering of most retaining structures will continue to rely upon "the seismic coefficient" method.

3. It is likely that there will gradually be a reduction in the conservatism used in designing for static conditions, and as a result retaining system that heretofore have fared well during earthquakes will begin to experience failures.

4. Both model testing and numerical effective stress methods for studying the performance of retaining structures increasingly offer important tools for bettering our

understanding of complex and difficult problems - but these tools must be in the hands of experienced engineers.

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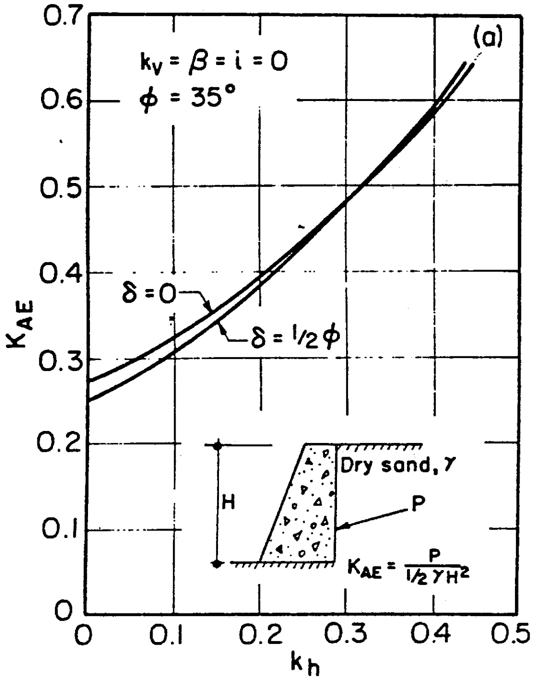

Figure 1. K_{AE} vs. Friction Angle of Backfill According to Mononobe-Okabe Equation, for Two Values of Wall Friction Angle δ (From Seed and Whitman, 1970.)

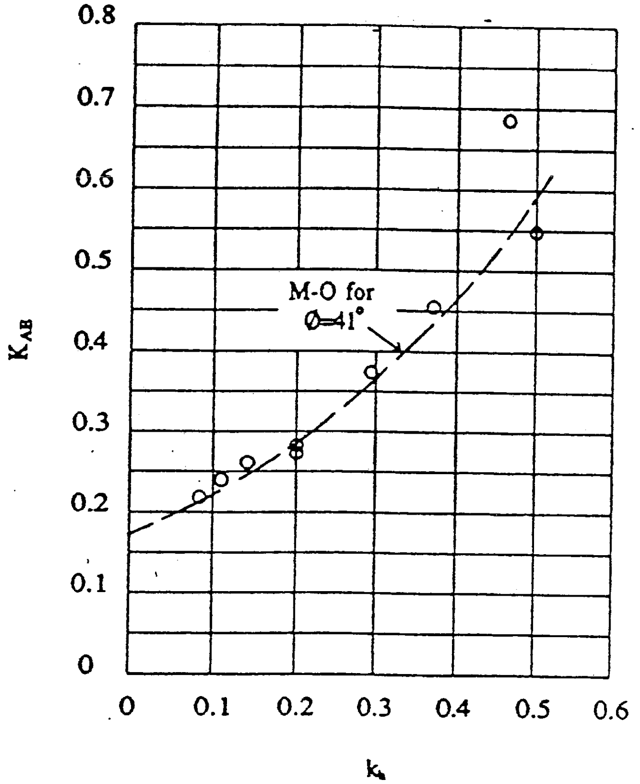

Figure 2. Predictions of Mononobe-Okabe Compared to Experimental Results. (Adapted from Sherif *et al*, 1982.)

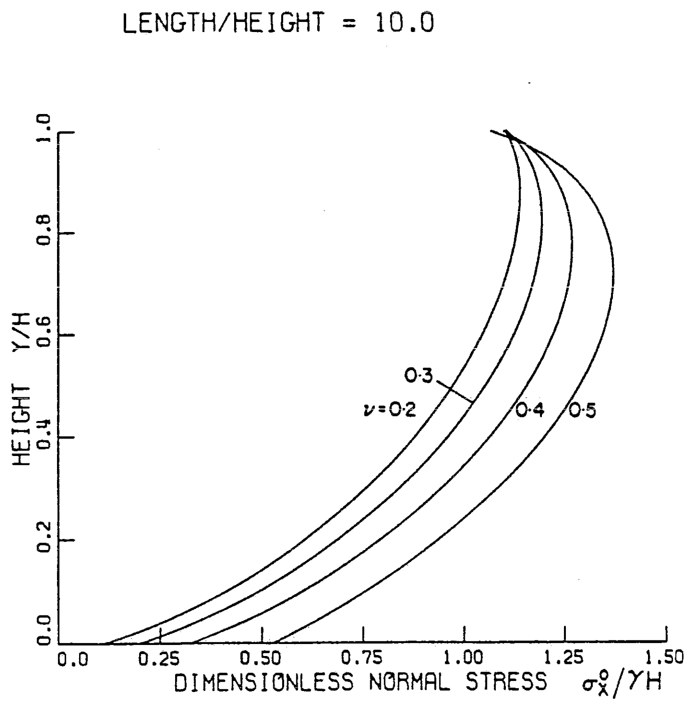

Figure 3. Pressure Distribution on Smooth Rigid Wall for One-g Static Acceleration Body Force (From Wood, 1973.)

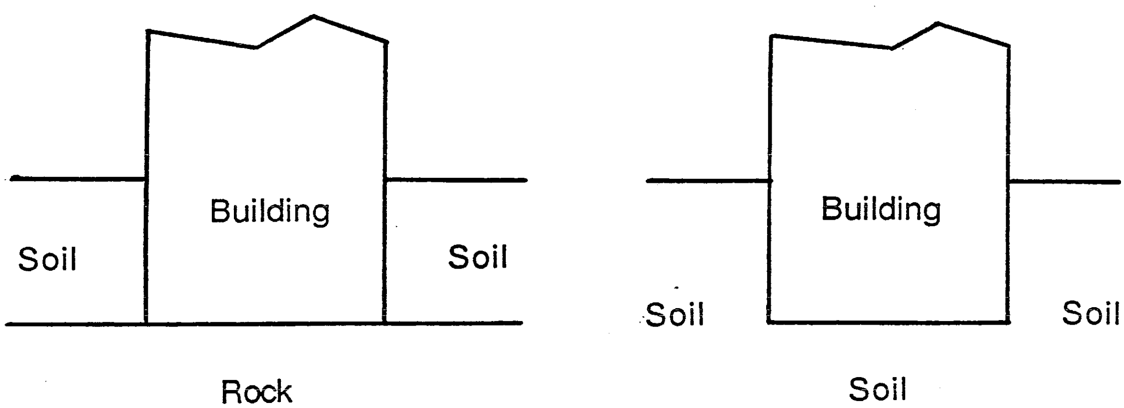

Figure 4. Embedded Foundation Founded Upon Rock (a) and Upon Soil with Same Stiffness as Backfill (b).

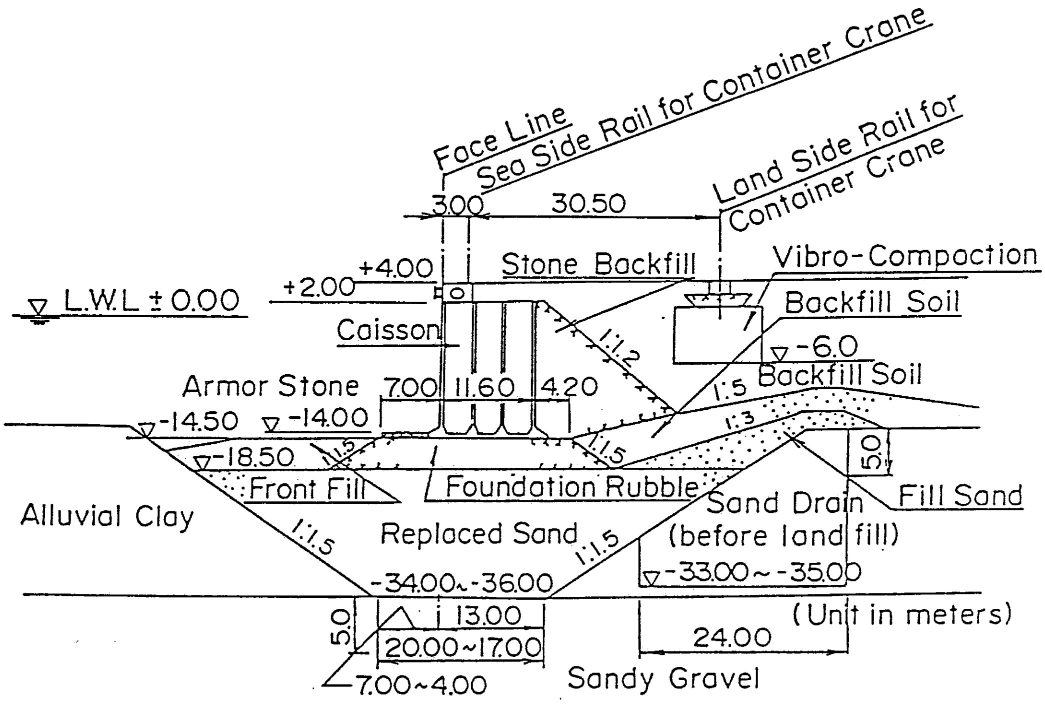

Figure 5. Typical Caisson Foundation. (From Inagaki *et al*, 1996.)

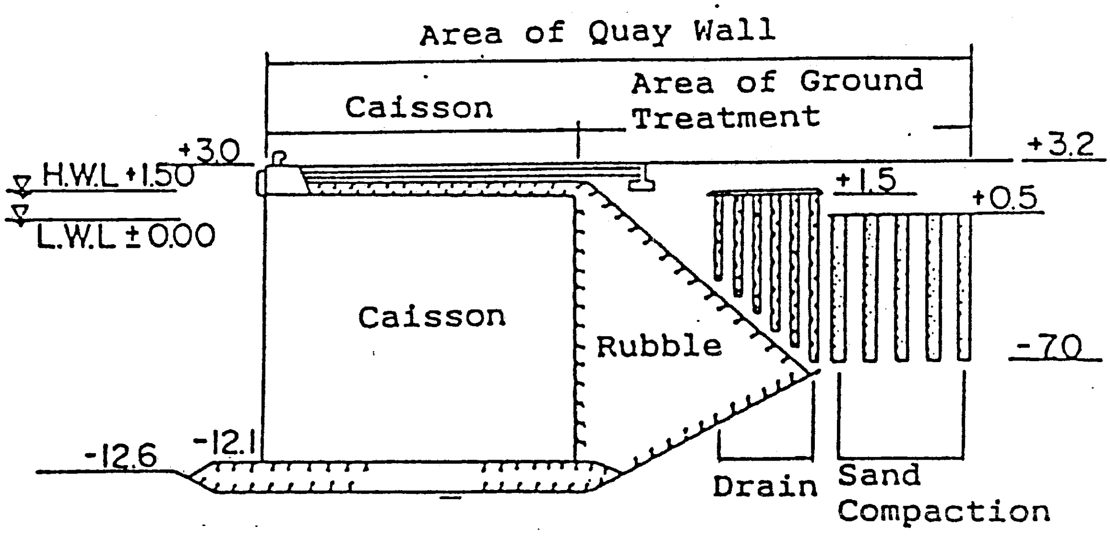

Figure 6. Use of Sand Drains and Sand Compaction Piles to Stabilize Backfill for Caisson Foundation. (From Iai *et al*, 1994.)

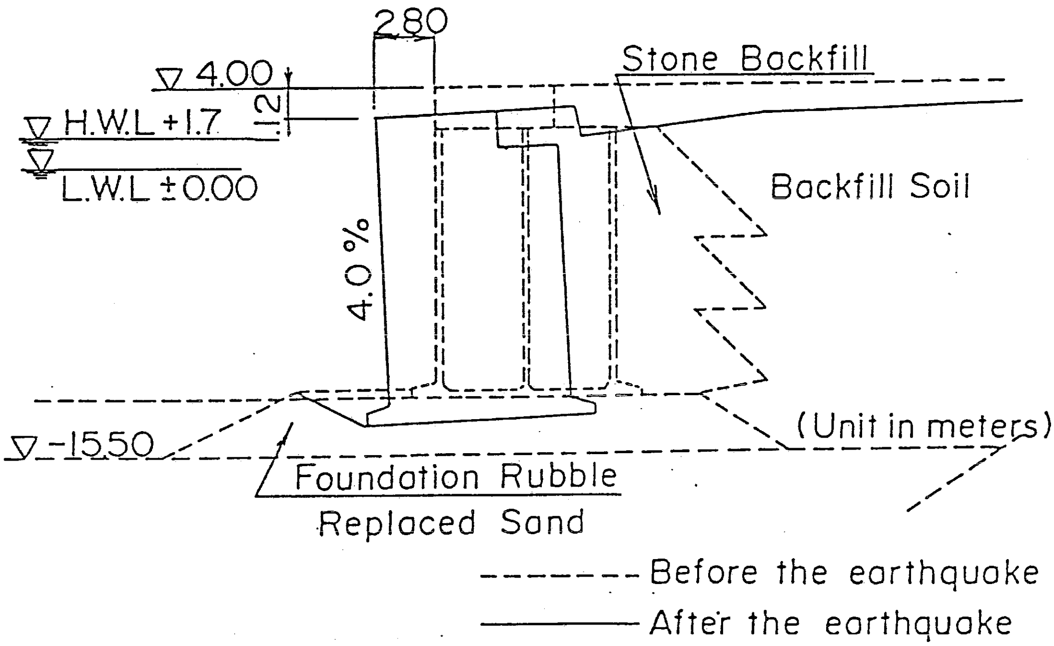

Figure 7. Typical Failure of Caisson at Kobe. (From Inagaki *et al*, 1996.)

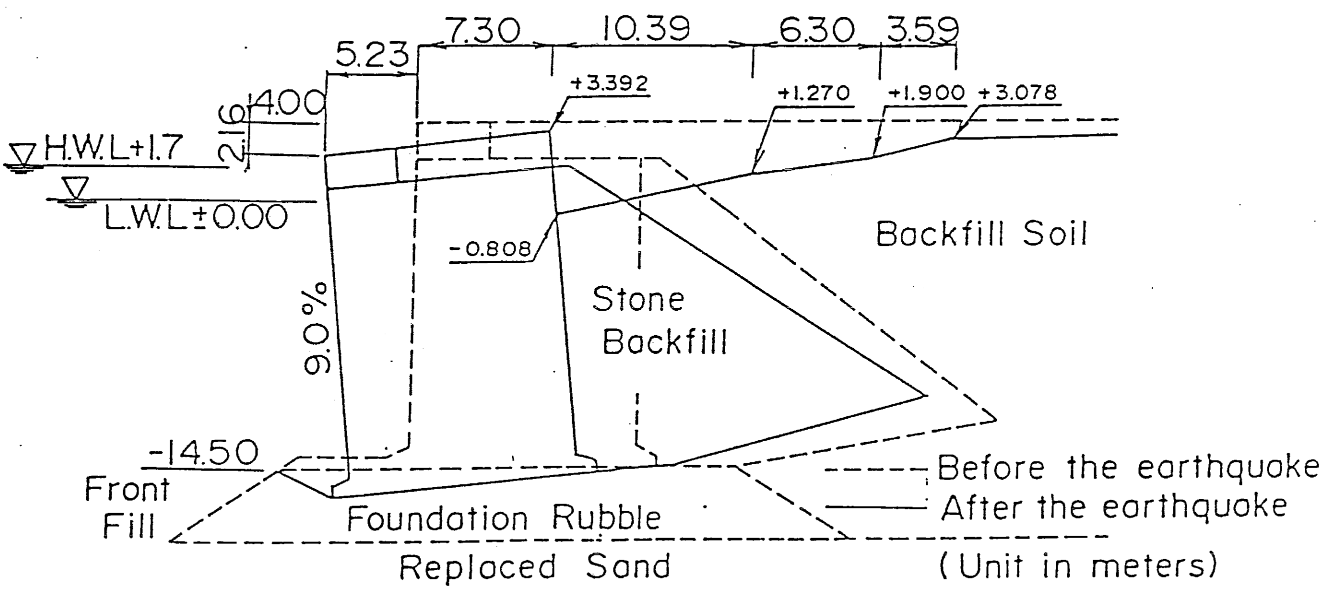

Figure 8. Typical Failure of Caisson at Kobe. (From Inagaki *et al*, 1996.)

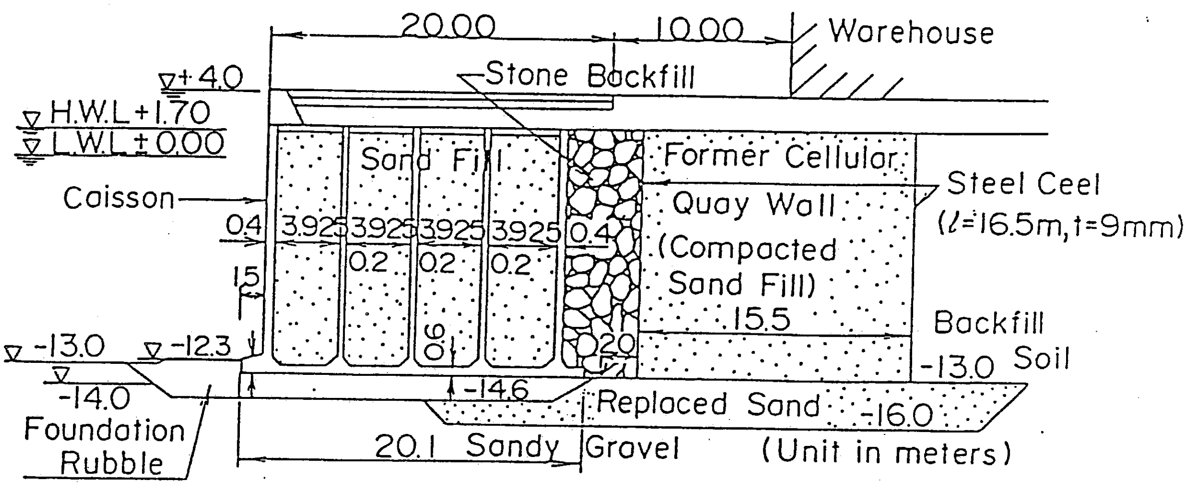

Figure 9. Caisson Quay Wall at Kobe That Performed Well. (From Inagaki *et al*, 1996.)

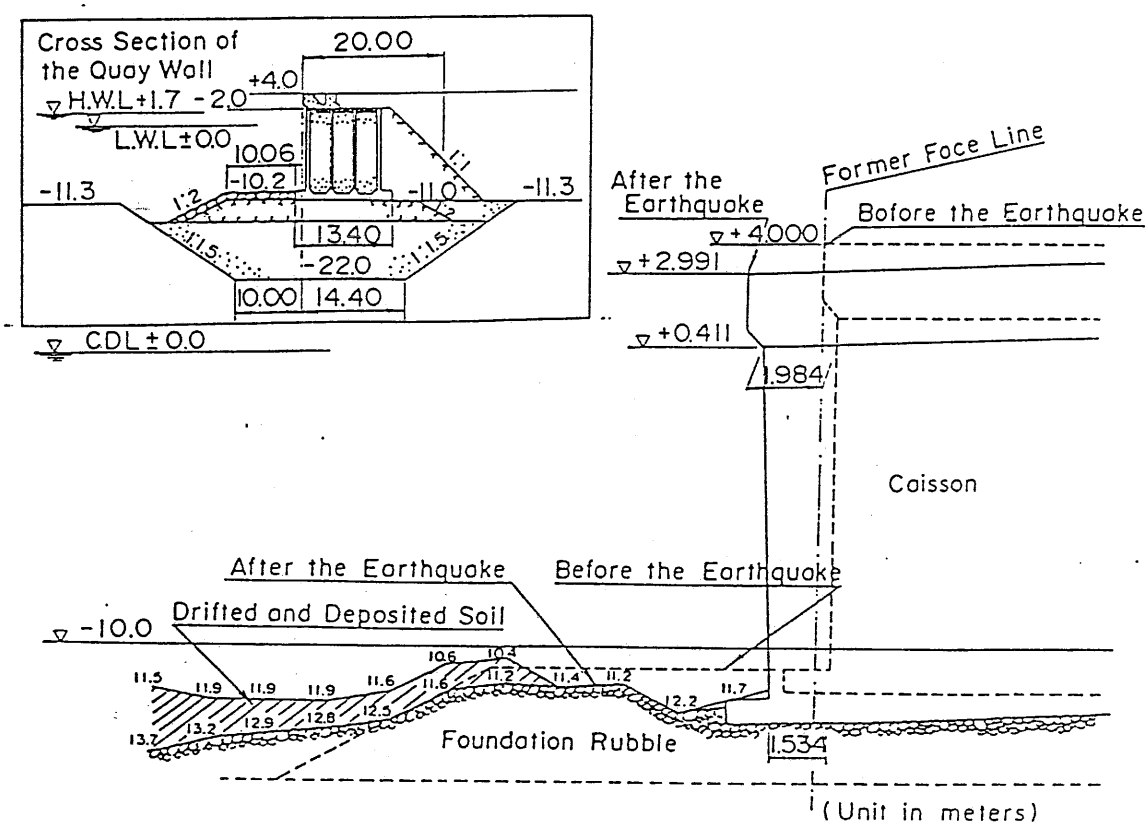

Figure 10. Rubble Pushed Ahead of Displacing Caisson, Kobe Harbor. (From Inagaki *et al*, 1996.)

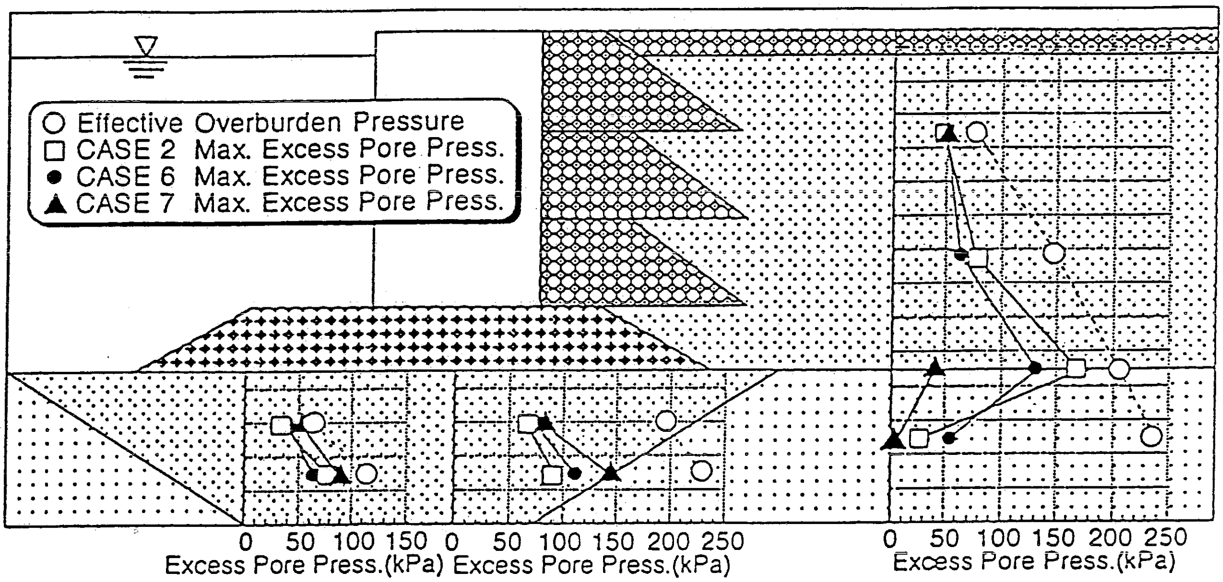

Figure 11. Excess Pore Pressures Observed in Model Test on Shaking Table. (From Inagaki *et al*, 1996.)

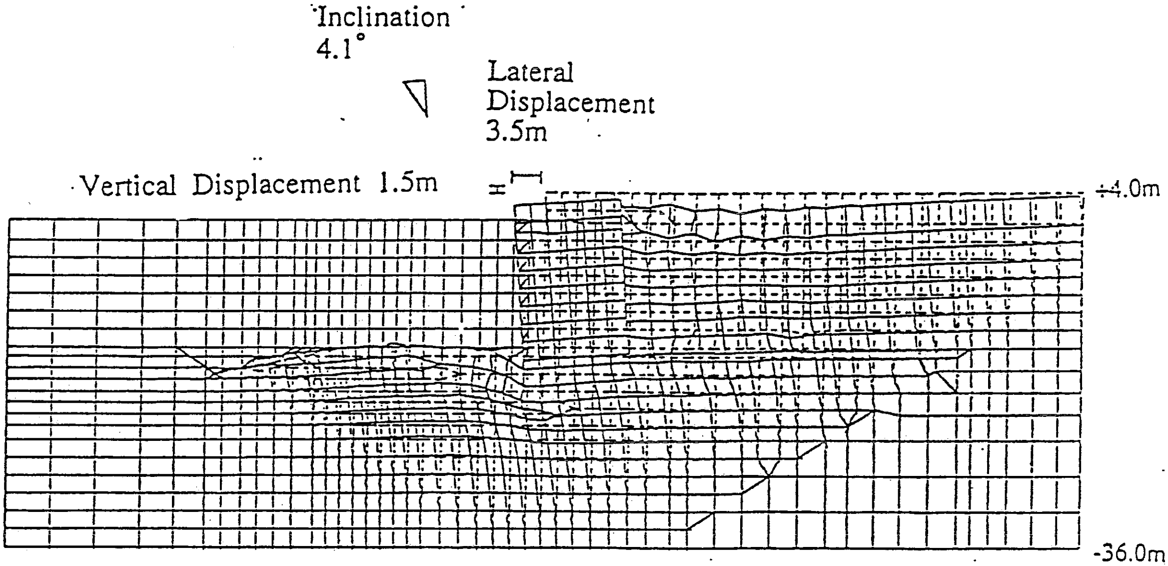

Figure 12. Deformations Below and Behind Caisson, from Dynamic Computations Using Effective Stress Model. (From Inagaki *et al*, 1996.)

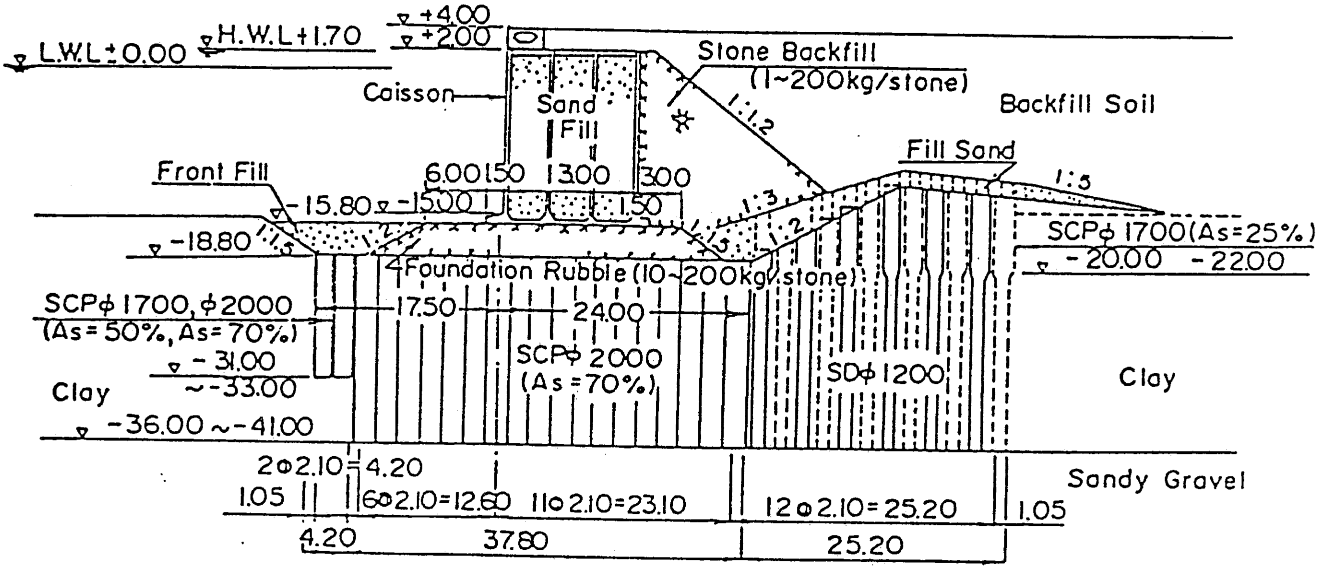

Figure 13. Caisson Foundation Treated with Sand Drains and Sand Compaction Piles, Kobe Harbor. (From Inagaki *et al*, 1996.)

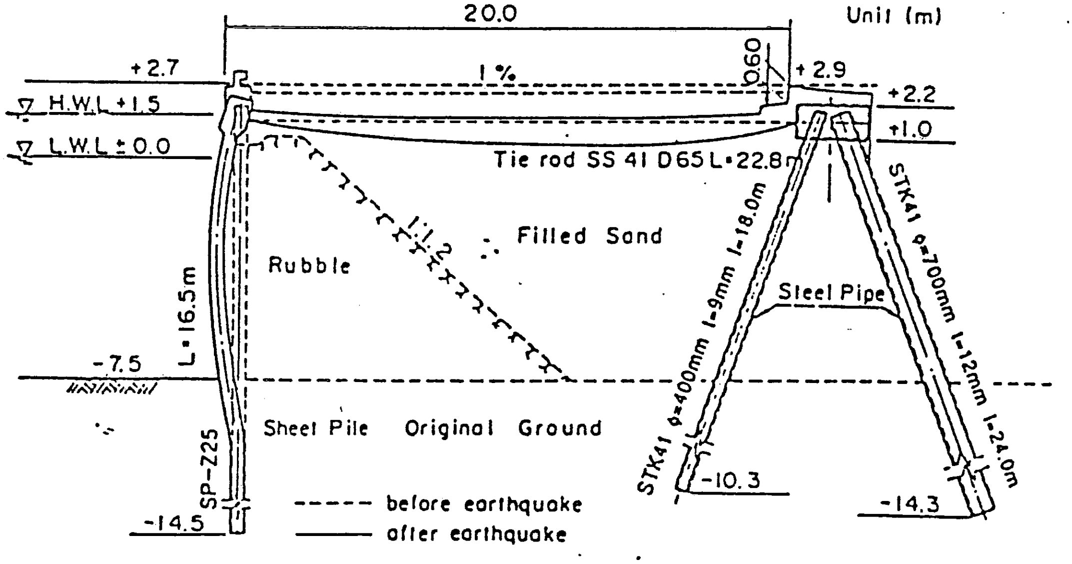

Figure 14. Anchored Bulkhead with Bulging, Kushiro Port. (From Iai *et al*, 1994.)

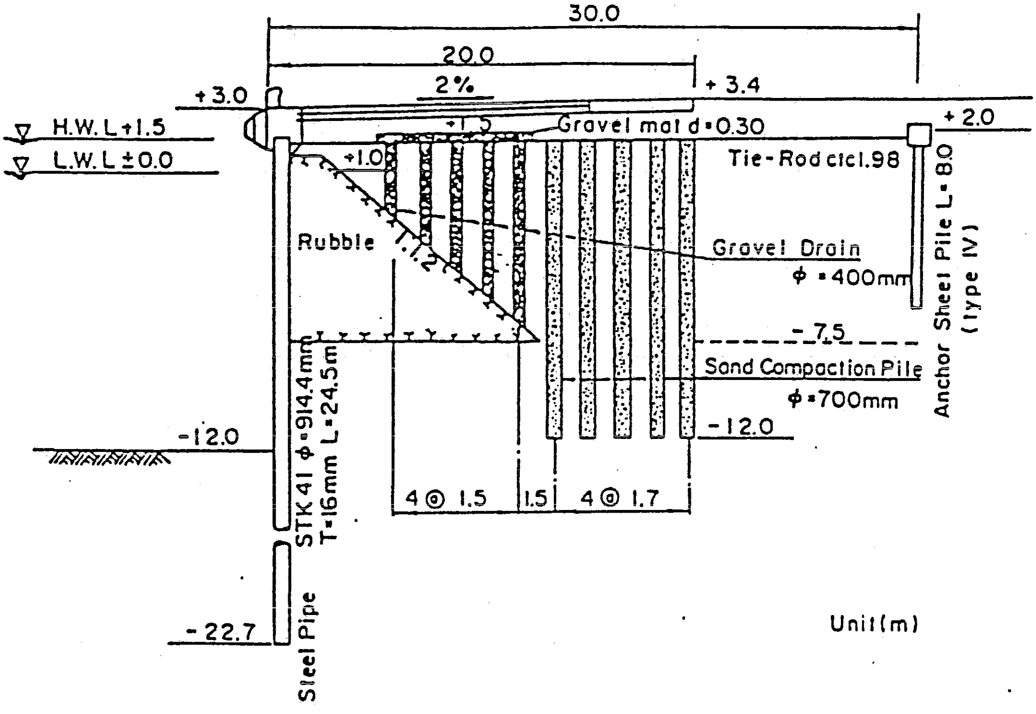

Figure 15. Stabilization of Backfill for Anchored Bulkhead, Kushiro Port. (From Iai *et al*, 1994.)

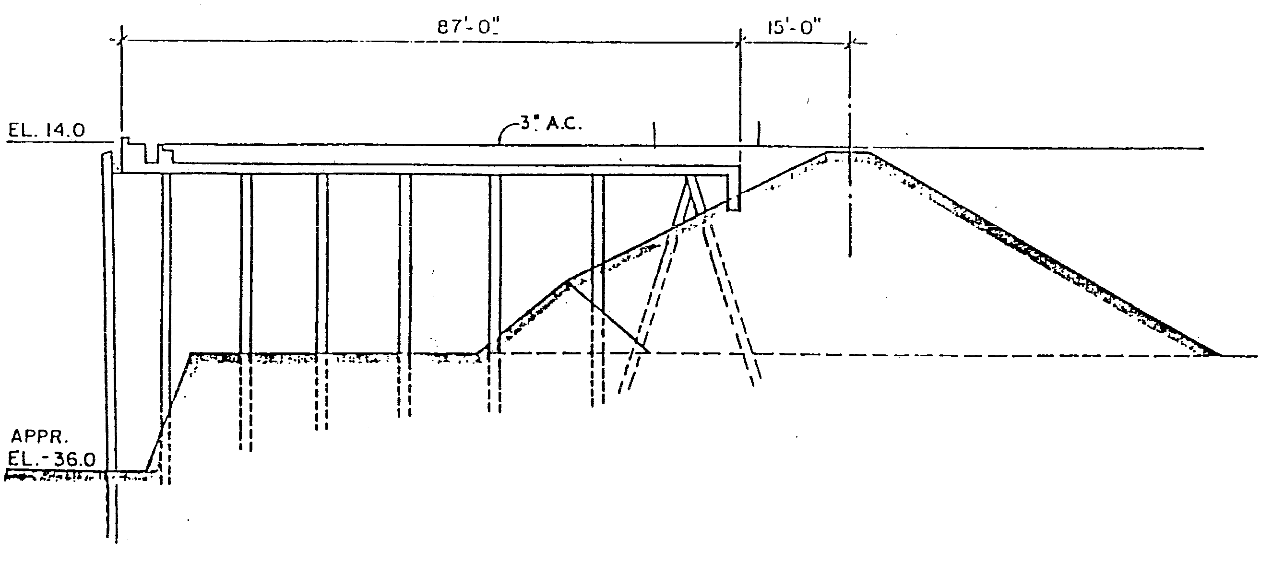

Figure 16. Typical Section for Trestle Pier. (From Benuska, 1990.)