Seismic Response of Tieback Retaining Walls

Phase 1

Final Report
December 1987

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16. ABSTRACT

The current design practice used by WSDOT for the design of permanent tieback walls is to assume that the static design of a tieback wall retaining clayey soils provides an adequate reserve of strength to prevent failure during seismic loading. This design procedure is based largely on the assumption that the soil and the wall move together during ground shaking and that significant dynamic loads are not produced. For tieback walls retaining sandy soils, it is assumed that dynamic loads are produced. Mononobe-Okabe dynamic soil pressures are added to the static design pressure to account for the dynamic load. The validity of these assumptions and the resultant design practices is evaluated in this study.

A pilot numerical study was conducted on a forty foot high wall with three levels of tiebacks using the program FLUSH. It was found that the wall and the soil tend to move in-phase and only negligible dynamic tie forces are generated. However, the soil above and below the excavation level tends to move out-of-phase, leading to significant dynamic pressures and bending moments in the wall and near the excavation level. It appears that in least some cases, tieback walls with an adequate static safety factor may suffer significant damage or fail during seismic loading and that the use of Mononobe-Okabe dynamic pressures may be conservative.

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ABSTRACT

Permanent tieback walls are used extensively on sections of Interstate 5 and Interstate 90 in Western Washington. Additional tieback walls are planned for new sections of I-90 on Mercer Island and I-5 near Olympia. During an earthquake, failure of one or more of these walls could block essential transportation corridors at a time when disaster relief operations are in progress.

It is current Washington State Department of Transportation (WSDOT) design practice to assume that the static design of a tieback wall retaining clayey soils provides an adequate reserve of strength to prevent failure during seismic loading. This design procedure is based largely on the assumption that the soil and the wall move together during ground shaking and significant dynamic loads are not produced. For tieback walls retaining sandy soils, it is assumed that dynamic loads are produced. Mononobe-Okabe dynamic soil pressures are added to the static design pressure to account for the dynamic load. The validity of these assumptions and the resulting design practice is evaluated in this study.

The results of a literature review clearly show that very little work has been done on the seismic response of tieback walls. No justification was found for the assumption that a static design is sufficient for clayey backfills, nor was any found that Mononobe-Okabe pressures are valid for sand backfill. Correspondence with researchers, design engineers and contractors substantiated the results of the literature review. All respondents agreed that this is an important topic which requires additional research.

A pilot numerical study was conducted. A forty foot high wall with three levels of tiebacks was analyzed using the program FLUSH. For this particular example problem it was found that the wall and the soil tend to move in-phase and only negligible dynamic tie forces are generated. However, the soil above and below the excavation level tends to move out-of-phase, leading to significant dynamic pressures and bending moments in the wall near the excavation level. It appears that in at least some cases, tieback walls with an adequate static safety factor may suffer significant damage or fail during seismic loading. It also appears that the use of the Mononobe-Okabe dynamic pressures may be conservative. Recommendations are made for additional research to evaluate the magnitude of the problem and to aid in the development of design procedures for seismic regions.

SUMMARY

Currently there are no generally accepted methods of analysis and design of tieback retaining walls for seismic loading. WSDOT designs their tieback walls for static loading alone when clayey soils are retained. They add a dynamic soil pressure, calculated using the Mononobe-Okabe equations, when a sandy soil is retained. The rationale for this approach is the commonly held belief that, during an earthquake, a tieback wall will move with the soil it is retaining and no significant dynamic loads will develop in clayey soil and dynamic loads in sandy soils will be limited to Mononobe-Okabe pressures. However, this assumption is not backed up by field observations or research reported in the literature. The study reported herein is designed to determine if this assumption is justifiable and, if not, what additional research is needed to improve WSDOT's design procedures.

Four methods were used to evaluate current design procedures. These include: (1) a review of literature; (2) a survey of researchers, design engineers and contractors; (3) a study of potential failure modes; and, (4) a pilot numerical study of seismic response of tieback walls.

The literature review confirmed the opinion that very little research related to seismic response of retaining walls has been conducted. With the exception of a relatively unsophisticated numerical study in 1975, a recent numerical study in 1987, and two laboratory model test studies in the early 1960's, there has been no work done on tieback walls subjected to seismic loading. Studies on cyclic loading of anchors do provide data useful for the understanding of potential failure modes of tieback walls; however, additional work is required to determine the magnitude of

potential cyclic loadings. The only report of field testing found in the literature was related to behavior of walls subjected to blast loading. This study concerned a wall constructed in rock which further reduces its applicability to the subject of this report - response of tieback walls in soil.

Correspondence with researchers, design engineers and contractors confirmed the results of the literature review. In addition, all respondents agreed that work should be done to evaluate this problem.

The pilot numerical studies shed a great deal of light on the problem. For the particular wall-soil system chosen for the analysis, it was found that the assumption that the wall and soil move together appears to be valid. However, out-of-phase movement of the the soil above and below the excavation level appears to be a significant problem. Substantial dynamic pressures are generated in the lower portions of the wall between the bottom tie rod and the excavation level. Large dynamic bending moments are produced in the wall at the tie rod connections and at the excavation level. Peak dynamic bending moments range up to approximately 20% of the static bending moment at the tie rod locations and approximately 300% of the static moment at the the excavation level for the particular wall studied. Dynamic bending moments such as these could lead to excessive bending of the wall and perhaps failure. In addition, large vertical accelerations in the wall were observed even though only horizontal input base accelerations were used. These high vertical accelerations lead to increased bearing pressures in the soil immediately below the base of the wall and shearing pressures on the back of the wall.

While the results from the numerical study are specific to the wall-soil system studied, they clearly show that there is a possibility that significant dynamic forces might be generated during seismic events. It appears that for some tieback walls static design alone may not provide a sufficient reserve of strength to prevent failure. There is also evidence that the Mononobe-Okabe method may overpredict dynamic pressures above the excavation, but underpredict net dynamic pressures below the excavation level.

Recommendations are made for additional research to evaluate the magnitude of the problem, and to aid in the development of new design procedures. These recommendations are divided into immediate research needs and long term needs.

CONCLUSIONS

The conclusions listed below are based on a literature review and pilot numerical studies. The numerical work is based on the analysis of a specific wall-soil system subjected to a specific earthquake record. Generalizations based on these numerical studies, therefore, are still somewhat tentative. Based on the work conducted for this project, the following conclusions are made:

1. There is very little information in the literature on the seismic response of tieback walls.
2. The work which is reported in the literature neither supports not refutes the assumption that a static design for walls retaining clayey soils provides an adequate margin of safety for seismic conditions. Neither does it support or refute the assumption that Mononobe-Okabe dynamic pressures should be used for walls retaining sandy soils.
3. Correspondence with other researchers, design engineers and contractors indicates little to no knowledge or experience with tieback walls under seismic loading.
4. Pilot finite element analyses show that, for the tieback system studied, the wall and the retained soil move together and no significant dynamic anchor loads develop.
5. Pilot finite element analyses indicate that the soil above and below the excavation level do not move together and often accelerate in opposite directions. This out-of-phase motion of the soil is a major factor in the development of high soil pressures on the wall and bending moments in the wall.

6. The numerical studies also show that high vertical accelerations develop in both the soil and the wall leading to large dynamic bearing pressures at the base of the wall.
7. Although this situation was not specifically analyzed, it is likely that anchors placed below the level of the excavation would be subjected to large dynamic loads and possibly load reversals if the soil above and below the excavation level move out-of-phase. This is especially likely when the lower soil is stiffer than the upper soil.
8. Additional research is required to determine the effects of out-of-phase motion and vertical accelerations on the behavior of the tieback system.

RECOMMENDATIONS FOR FUTURE RESEARCH

The results of this study clearly show the possibility that significant damage to permanent tieback walls might be caused by earthquake loading. This possibility exists despite the apparent confirmation that the wall and soil backfill move essentially in-phase. Because the potential for significant damage has been demonstrated, additional research is needed to study, in more detail, the principle parameters affecting wall behavior. The following recommendations are divided into two groups. The first set of recommendations should be carried out as quickly as possible to identify the extent of the potential problem and to begin to address the remedial measures, if any, required. The second set of recommendations reflect long-term goals leading to a comprehensive understanding of the seismic loading problem.

Immediate Research Needs

The following research tasks are deemed to be of critical importance and should be initiated as soon as possible:

1. Additional numerical studies should be conducted using the FLUSH program. These studies should be designed to evaluate the effects of the following:

    a) Period ratio (retained soil to soil below excavation)

    b) Sloping ground

    c) Tieback below the excavation level

    d) Varying soil stiffness

    These analyses should also be used to evaluate the extent of wall displacement during seismic loading. If feasible, an actual WSDOT wall could be modeled with earthquake records appropriate for the Puget Sound area.

2) An instrumentation package for a new and/or existing wall should be designed and priced. It is important to validate the predicted tieback behavior with actual performance records and to evaluate the actual importance of the problem.

3) Current design guidelines should be evaluated and modified on the basis of the work described in this report and the results of the tasks listed under item #1 above.

4) A preliminary review of existing WSDOT walls should be carried out in the light of the new design guidelines to identify those walls which are susceptible to earthquake-induced damage.

The above recommendations could be carried out immediately and would take between 6 and 12 months to accomplish. At that time WSDOT would have a much better understanding of the effects of seismic loading on their permanent tieback walls and would be in a position to incorporate any needed design changes in future construction. The groundwork for additional work, both numerical and field would be firmly established.

Long-Term Research Needs

Although the numerical studies discussed in this report and proposed above will result in a major advance in our understanding of seismic response of permanent tieback walls, the importance of tieback walls in maintaining high use urban corridors during an earthquake emergency is such that much additional study is warranted/justified. The following recommendations are divided into numerical, field, instrumentation and laboratory work. They are not meant to be all-inclusive, but rather first steps in the comprehensive investigation of the problem.

1) A comprehensive numerical parametric study should be conducted. The work described in this report and recommended above is a significant step in understanding the behavior of tieback walls, but a detailed analysis covering all significant parameters is necessary. A more complete evaluation of the effects of soil type and ground motion is particularly important.

2) Field testing of anchors should be conducted. Two specific test series are recommended. The first would be conducted on short anchors and variable grout pressure. Pull-out tests would be used to determine the relative importance of grout pressure on anchor size, anchor shape, and anchor capacity. In the second series of tests, either on existing anchors or new short anchors, cyclic loading would be applied to determine strength degradation. These tests should include stress reversals if the numerical studies indicate that this is possible.

3) Instrumentation packages should be installed on new and/or existing walls. The information gained from such instrumentation, even from small earthquakes, would be very useful in verifying numerical models. As an additional benefit, data could be collected on long term creep behavior of anchors.

4) Laboratory and centrifuge tests should be conducted to supplement the other studies and to provide a method of broadening verification of the numerical results. Laboratory studies of cyclic loading of pressure grouted anchors are needed, as well

as centrifuge studies of model walls subjected to earthquake motion. Some aspects of the problem might also be investigated using shaking table tests.

INTRODUCTION

The Washington State Department of Transportation (WSDOT) makes extensive use of permanent tieback walls, especially in the Puget Sound area. Table 1 lists completed walls on I-90 to illustrate the extent of tieback wall use in Washington. In addition to those walls already in service, new portions of I-90 on Mercer Island and on I-5 through Olympia currently being designed and constructed by WSDOT contain several miles of tieback walls. Both cohesive and granular soils will be retained by these walls and the ground surface behind the walls will frequently be sloping.

Because the Puget Sound region ranks as one of the more active earthquake areas in the United States, it is prudent to consider dynamic loading in the design of transportation structures such as bridges, tunnels and retaining walls. Current WSDOT design procedure related to seismic analysis of tieback walls retaining cohesive soils is to assume that the wall and retained soil move together and no significant additional stresses are imposed on the wall or the tiebacks. It is therefore assumed that the safety factor for static conditions provides an adequate safety factor under seismic loading and no additional analyses are employed. These assumptions are based on a general feeling by WSDOT and many outside Geotechnical engineers, rather than by research reported in the literature.

The current WSDOT design procedure for permanent tieback walls retaining cohesionless soils is to use the Mononobe-Okabe method to determine dynamic soil pressures against the wall. These pressures are then added to the static pressures and the wall is designed to resist the combination. Again, this procedure is not based on any research reported in

Table 1. WSDOT Tieback Retaining Wall Inventory - Interstate 90

Mile Post	Year Built	Wall Area (sq. ft)	Wall Length (ft)	Max. Wall Height (ft)
8.34	1985	10,620	510	31
5.14	1986	9,785	53	42
5.30	1986	8,712	242	36
8.40	1986-87	133,990	4,545	42
3.40	1986	48,000	2,302	34
4.08	1987	55,100	2,436	32
4.80	1987	28,210	690	50
0.04 - 0.59	1986	9,736	272	41
0.04 - 0.59	1986-87	7,574	225	43
0.04 - 0.59	1986-87	41,522	1,050	51
0.04 - 0.59	1986	4,343	339	20
0.04 - 0.59	1987	2,223	217	36
1.25 - 1.42	1986	7,215	470	20
1.25 - 1.42	1986	4,540	212	24
1.25 - 1.42	1986	5,885	310	30
1.25 - 1.42	1986-87	2,024	46	44
1.25 - 1.42	1986-87	9,054	298	32
1.02 - 1.22	1986	30,863	1,112	39
1.02 - 1.22	1986	21,214	833	35
1.02 - 1.22	1986	2,363	106	36

the literature.

In order to validate and/or improve this current design methodology, WSDOT has funded the first phase of a three-phase project entitled "Seismic Design of Permanent Tieback Walls." This report presents the results of Phase I. Phase II consists of more detailed studies of the behavior of tieback walls under seismic loading and development of design procedures. Phase III consists of implementation of the results of these studies in the form of a design manual.

The objectives of Phase I are to:

1. Determine the state-of-the-practice in the design of tieback walls for seismic loading.
2. Identify potential failure modes.
3. Conduct pilot dynamic numerical analyses of the response of tieback walls to seismic loading.
4. Determine if current WSDOT design procedures adequately address seismic loading conditions.

To accomplish the first objective, a review of the technical literature was conducted. In addition, letters were sent to researchers, design engineers and contractors in several countries requesting any information they might have related to this topic. Approximately 40 references were received and reviewed and 6 responses to the letter were obtained. Based on this work, a review of the current state-of-the-practice is presented in the following section.

The next section of the report presents the results of pilot numerical analyses which were performed for a typical wall with a silty sand backfill. Based on these analyses and the results of the literature review, several possible failure modes are hypothesized and described in the subsequent section. The final section presents preliminary recommendations based on the results of Phase I.

STATE-OF-THE-ART EVALUATION

It has been known for over 50 years that retaining structures located in seismically active areas can be subjected to high dynamic loads during earthquakes. The most common method of incorporating seismic stresses in the design of cantilever and gravity retaining walls is to add a pseudo-static component to the static pressures on the wall. This approach was originally developed by Mononobe and Matsuo (15) and Okabe (20) and is commonly referred to as the Mononobe-Okabe (M-O) method. It assumes that the an active Coulomb wedge, which acts as a rigid body, develops behind the wall. Appropriate seismic coefficients are applied to this soil wedge to compute the horizontal and vertical inertia forces caused by the earthquake loading. The inertia forces are then superimposed on the static stresses. Originally, it was assumed that the dynamic increment of earth pressure acts at the same location as the static component, i.e., at a distance 1/3 H from the base of the wall. However, more recently Seed and Whitman (25) suggested an approach to calculating the point of application of the resultant based on summation of moments about the base of the wall due to the static and the inertia force. The static force is applied at 1/3H and the dynamic increment is applied at 0.6H. Other researchers have suggested locations for the dynamic force ranging from 0.5 to 0.67 H.

WSDOT currently uses AASHTO's "Guide Specifications for Seismic Design of Highway Bridges--1983" (ATC-6) as their design manual for retaining structures. ATC-6 identifies gravity, cantilever and non-yielding walls in their description of retaining structures. The M-O method with a seismic coefficient of one-half the anticipated maximum ground acceleration is recommended for cantilever walls. The Newmark sliding wedge method is recommended for gravity walls which can undergo some lateral displacement. Because it has been shown that the earth pressures for non-yielding walls is underestimated by the M-O method, a seismic coefficient of 1.5 times the peak anticipated ground acceleration is recommended.

Unfortunately, ATC-6 does not cover the seismic design of tieback walls. The term tieback is used in Chapter 6 in connection with non-yielding walls, but it is not clear from the context exactly what type of wall is being described, except that it is not a standard tieback wall with anchors placed in the soil backfill.

Literature Review

There are several recent books and FHWA reports which address the design and analysis of tieback walls and permanent ground anchors (5,7,19,23,29). Of these, only Hanna's book (7) specifically discusses seismic loading of anchors and only two paragraphs are devoted to the topic. The following quote from this book gives a good summary of the current state of knowledge:

> "The subject of anchored structures in earthquake regions is not well documented and, in the future projects in such regions, anchored structures should be monitored to assess the adequacy of present design methods, load levels in anchors and how the structure behaves under seismic loading. Until this is done there must be some uncertainty about how best to design prestressed anchor systems for these loading conditions."

There have been a few papers which discuss seismic loading of tiebacks and/or cyclic loading of anchors. These are divided into numerical studies, laboratory and field studies and are discussed below.

Numerical Studies

In what appears to be the first numerical study of seismic response of tieback walls, Rutledge (22) used a pseudo-static force in conjunction with a static finite element method to design tieback walls with sloping ground surface. Anchor forces equal to the dead load and a 0.2g horizontal pseudo-static load were applied to the wall in the finite element analysis. Soil elements were checked for failure and the procedure repeated until a satisfactory stress distribution was obtained. At this point the design load for the anchors was increased by a factor of 1.5 and the soil wall system reanalyzed using a limit equilibrium analysis to ensure that the wall could withstand a seismic coefficient of at least 0.25g. He also concluded that there is no standard method of analysis or literature available on the seismic design of tieback walls.

The only other numerical study located by our search was recently described by Siller *et al* (26). The authors performed a dynamic finite element analysis of a wall subjected to a vertically propagating half-sine pulse of displacement. The excitation pulse had a frequency of 6.5 Hz and an amplitude of 0.5 g. The model wall was 36.4 feet high, including 10.4 feet below the excavation level. Two levels of tieback anchors were used. The tiebacks were modeled by incorporating a spring support for the wall at the tieback locations. The tieback prestress was modeled by applying equal but opposite nodal forces at the two ends of each tieback. The tieback stiffness was 1.0 kip/ft which models a 50 foot tieback length with 1.9 square inches of steel, placed at a spacing of 8 feet longitudinally. It was not clear from the paper if the tiebacks were horizontal or inclined.

The authors used both linear and nonlinear soil models in their study. They concentrated the discussion of their results on the differences in permanent displacements of the wall and total force acting on the wall which are predicted using the two soil models. They concluded that nonlinear behavior leads to smaller displacement occilations and an accumulation of permanent deformations of the wall toward the excavation. They also concluded that the total force acting on the wall is lower for a nonlinear soil due to the permanent deformation of the wall.

Laboratory Studies

Murphy (17) carried out an experimental study on a model wall in sand. The wall was made of 3/4 inch thick solid rubber and the tie-rod was made of strands of round sectioned rubber. After vibrating the wall in a shaking table for 20 seconds, it was noted that the wall had translated horizontally while remaining vertical. At this stage the strands were released in order to simulate anchor failure. Planes of shear failure were observed during vibration and after tie release. Murphy concluded that active stresses under dynamic conditions are higher than those under static conditions and that the planes of shear failure are at a lower angle than Rankine's state. This was true for active failure behind the wall as well as the passive conditions at the toe.

Kurata, Arai and Yokoi (12) performed shaking table tests on flexible anchored model walls in sand. They used a single anchor and densified the sand by an initial stage of vibration before they applied the accelerations. They concluded that the bending moments and the tieback forces consist of two parts: an oscillating part during the vibration and a residual part that remains after the shaking ceases. They showed that the residual stresses were considerably higher than the oscillating stresses. They also showed the effect of soil modulus at the toe of the wall. As expected, the bending moments near the toe increase with an increase in the soil modulus and decrease with a reduction of the soil modulus.

Anchor studies utilizing cyclic loads have been carried out in the past, although most of these were performed in the laboratory on dead anchors (anchors with zero pre-stressing are termed dead anchors). Prestressing the anchor overconsolidates the soil between the wall and the anchor, thereby reducing the rate of deformation under subsequent loading (Trollope *et al*, 28). Also, the higher confining pressures tend to stabilize sandy backfill materials, (Morgan, 16). Hanna *et al* (8) indicate that the general lack of related research may be due to the common belief that preloading allows for any adverse effects caused by subsequent cyclic loads. They note the work of Carr (4) and Abu Taleb (1). Carr (4) subjected a plate shaped anchor to repetitive loads showing that the displacements increased with the application of cyclic loads but the ultimate pullout capacity remained the same. Abu Taleb (1) performed tests on prestressed anchors. He showed that repeated loads decreased the prestressing force in the tie and that the higher the initial prestress load the lower the anchor displacement per load cycle.

Hanna *et al* (8) performed 46 laboratory tests on plate shaped anchors, only two of which were preloaded. Although none of the tests were performed on prestressed anchors, their results show general trends of anchor behavior. They concluded that:

a) Dead anchors undergo permanent movements when subjected to repetitive loads.

b) Movements per cycle of load decrease as the number of loading cycles progresses; however for very large load ranges instability may occur.

c) Alternating loads (positive to negative & vice-versa) cause much more severe conditions than repetitive loads (sign remains the same).

d) Alternating loads reduce the fatigue life of an anchor tremendously as compared to repeated loads.

e) Prestressing does not completely eliminate movement under subsequent loading; a small movement always occurs.

f) It is expected that prestressed anchors will behave similarly to dead anchors, although the fatigue life of dead anchors may be smaller.

In the current codes of practice little attention is paid to the effects of cyclic loads.

<u>Field Testing</u>

The only reference found which describes field testing deals with the effects of nearby blasting on prestressed anchors, and is reported by Littlejohn, *et al* (13). They concluded that only nominal fluctuations occurred in the pre-stressing loads even when the anchor heads were only five meters from the first line of charge holes. However, the anchors were being used to stabilize a rock slope so the applicability of the results to tieback walls in soil may be minimal. Also, blast loading differs from seismic loading in many respects including type, direction and magnitude of ground motion. For instance, ground motion during an earthquake may consist predominantly of shear waves propagating upwards from underlying bedrock, whereas during close proximity blast loading the energy may predominantly be transmitted through horizontally propagating waves.

Correspondence

Letters were sent to 12 researchers, contractors and design engineers who have significant experience with tieback walls. They were asked for any information they might know of regarding studies done on seismic response of tieback walls, what they felt the potential failure modes might be under seismic loading conditions and whether they felt research was needed on this topic. Six individuals, listed in Table 2, responded to our letter and all confirmed the lack of research on seismic response of tieback walls. Although none could cite any occurrence of damage done to a tieback wall due to earthquake loading, the consensus of opinion was that a study of this problem would be very useful.

Table 2. List of Respondents

Name	Affiliation
1. Prof. T. H. Hanna	University of Sheffield
2. Mr. K. Ronald Chapman	Schnabel Foundation Company
3. Mr. David E. Weatherby	Schnabel Foundation Company
4. Prof. N. R. Morgenstern	University of Alberta
5. Prof. G. S. Littlejohn	University of Bradford
6. Peter J. Nicholson	Nicholson Construction Company

POTENTIAL FAILURE MODES

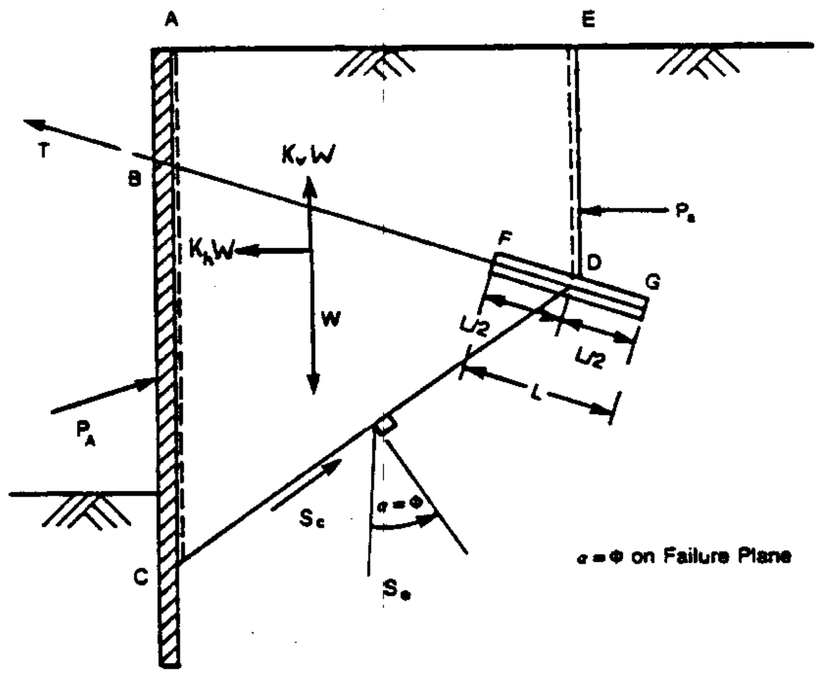
Introduction

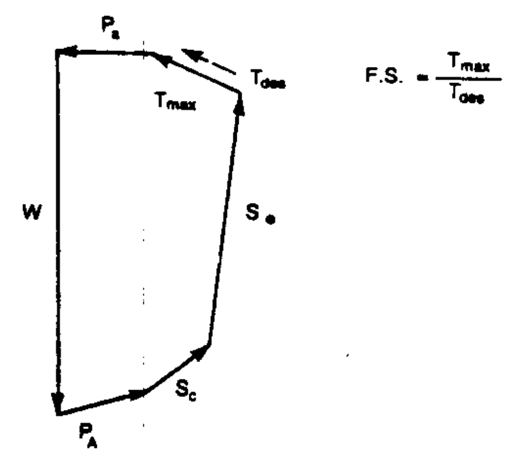
Potential failure modes under seismic loading were initially identified from an examination of static failure modes. The rationale for this approach is that the static modes should all be adversely affected by seismic motions and/or forces. In addition, other failure modes unique to seismic loading conditions were hypothesized and analyzed. In the discussion below, the associated static failure modes are covered first, followed by the hypothesis failure modes.

Overall Stability

The overall stability of tieback walls and the retained soil is a major design consideration. This is composed of two parts - internal stability (the wall-soil system) and external stability (the slope including the wall). Although, according to Anderson, *et al.* (3), the analysis of overall stability for static conditions is not clearly understood, methods are available for both internal and external stability analysis. For example, FHWA report DP-68-1 (5) presents a simplified method based on the work of Kranz (11) and Ranke, et al. (21), for internal stability analysis. This method is illustrated in Fig. 1. To adapt this method to include seismic forces, the inertia of the soil wedge ACDE, due to horizontal and vertical ground accelerations may be included in the force polygon.

To analyze the overall external stability of the wall, (illustrated in Fig 2), Cheney (5) suggests that a circular arc or wedge method, or a





1. W - Weight of soil mass within the failure surface.
2. P_a - Design force acting on the surface DE. A driving force due to water must be considered when below the water table. While P_a has been drawn horizontally, it could have been an inclined force.
3. S_Φ - Frictional component of soil resistance. This force is applied at an angle, $\alpha = \Phi$ (full obliquity) to the normal base of the soil mass. It should be noted that α cannot be greater than the internal friction angle of the soil. Mobilized shear resistance acting along the plane is (S_Φ cos Φ) tan Φ.
4. S_c - Component of soil resistance due to cohesive soil strength.
5. P_A - Active earth force between point A and point C. Point C is the point of zero shear.
6. T - Tieback force.

Figure 1 Vector Diagram for Internal Stability Analysis

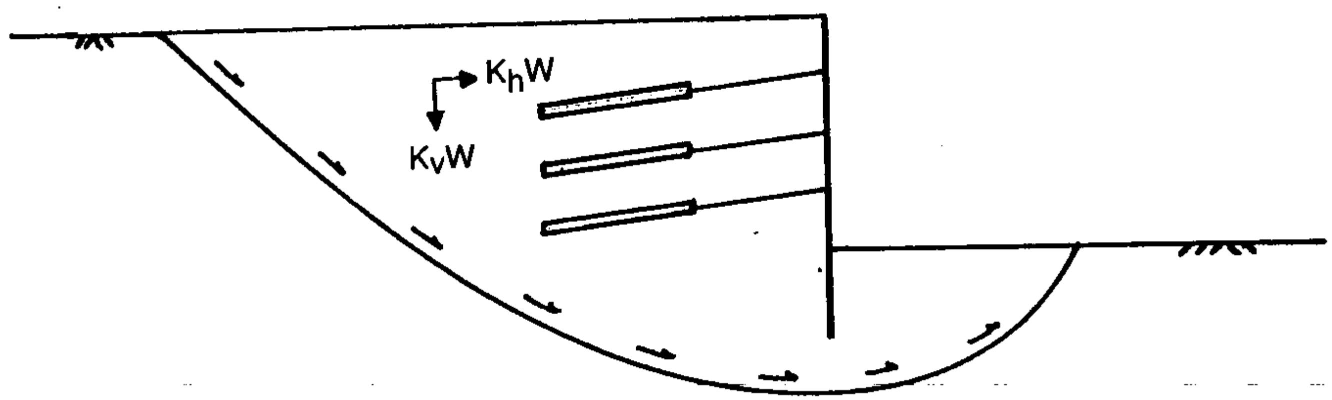


Figure 2 Illustration of Overall Stability Analysis

combination of the two, may be used. A horizontal inertia force can easily be included in such an analysis, as it often is in standard slope stability investigations, to account for seismic forces.

Anchor Failure

Failure of individual anchors is a relatively common occurrence during installation and proof testing. Although failure of a few anchors will generally not lead to wall failure, excessive anchor failure will.

The major detrimental effects of seismic motion on anchor integrity are increased anchor loads and loss of adhesion and/or frictional resistance in the soil due to cyclic loading. The possible effects of increased anchor loads during earthquake loading are obvious; however, the potential causes of these increased loads are complex. For anchors placed at small angles from the horizontal such that the anchor is located at or above the bottom of the excavation, increased anchor loading will come from relative movement between the wall and the anchor. If the wall tends to move with the soil, significant dynamic anchor loads might not occur. However, for anchors located below the excavation, out-of-phase motion of the soil above and below the excavation might lead to extremely high dynamic anchor loads, and possibly load reversals, even though the wall and the soil are not significantly out-of-phase. The pilot numerical studies described below shed considerable light on this failure mode.

The effects of vibration on sandy soils can be divided into two distinct problems. First, cyclic straining of sands can be accompanied by a reduction in peak strength which will lead to decreased frictional capacity. The possible reduction in anchor load might be estimated by laboratory cyclic triaxial or direct shear tests on the backfill material.

A second possible adverse condition is considerably more speculative. It is possible that the high grout pressure (typically 150 psi) used during anchor installation causes residual stresses in the surrounding soil. It seems likely that some increase would occur due to arching of the soil, although the extent of any build-up of in-situ stresses around a grouted anchor is not known at this time. Assuming, however, that this is the case, an unknown portion of the total anchor capacity comes from this residual state of stress. If, during a seismic event, the soil around the grouted zone densified there would be a reduction in the normal stress acting on the anchor and, consequently, a reduction in anchor capacity. Cyclic loading of existing anchors in sandy soil might be useful for determining the feasibility of this type of failure.

In the case of clayey soils surrounding the anchors, strain softening may occur during cyclic loading, thus leading to a reduced anchor capacity. Overconsolidated clays are particularly likely to undergo significant strength reduction due to cyclic loading. The magnitude of strain softening which might occur could be studied by both laboratory and field testing. Cyclic loading of existing anchors in clayey soils could be accomplished relatively easily and should provide important information regarding the potential problems related to strain softening.

Bearing Capacity

Static vertical loads are transmitted to the ground at the base of the wall. These loads come from the weight of the wall itself and the vertical component of the tie forces. The static forces alone can be quite significant. Earthquake motion can significantly increase the vertical loads transmitted to the ground through vertical acceleration of the wall and the vertical component of the dynamic tie force. The pilot numerical studies described in this report clearly indicate that vertical ground accelerations are a significant factor in the behavior of a tieback wall.

Excessive Bending Moments in the Wall

Structural failure of the wall may be caused by excessive bending moments. Even in cases where the wall and soil move in-phase and there is no relative movement between the soil and the wall, dynamic bending moments can be induced. For example, when there is out-of-phase movement of the soil mass above and below the excavation level, bending of the wall will take place, resulting in increased moments. The pilot numerical studies very clearly show increased moments in the wall, although little to no relative movement between the wall and the soil takes place and anchor loads are not significantly different.

Wall Rotation and Lateral Movement

Both wall rotation and lateral movement are encountered during construction of a tieback wall. Some additional movement is expected after construction is complete; however, it is typically rather small. Transitory wall rotation and translation should, of course, be expected to accompany earthquake loading. Since some permanent ground displacement after an earthquake is common, it is likely that a tieback wall will also undergo some permanent movement.

Tie Rod Failure

The tie rod is an essential feature of the tieback wall. At least three distinct failure modes in the tie rods can lead to failure of the wall: bonding between the tie rod and the anchor, fracturing of the rod (tensile failure) and shear failure of the rod. It is not known how cyclic stresses might affect bond strength; however, it is clear that increased anchor loads could cause the bond to be overstressed even if there is no cyclic strength degradation. Fracturing of the tie rod is also associated with increased anchor load. Large relative vertical movements of the anchor and the wall could lead to shearing of the tie rod and/or the tie rod-wall connection.

Sloping Ground

All of the above failure modes are adversely affected by sloping ground. In particular, external stability problems related to out-of-phase motion between upper and lower soils would be particularly affected. Since many of the WSDOT walls are located in areas of significant ground slope, it is important to thoroughly investigate this aspect of the problem.

Conclusions

Although other failure modes may be possible, it is our opinion that those listed above are the most probable. The pilot numerical studies described in the next section provide some evidence that there are circumstances when an adequate static design might not be sufficient to prevent failure under seismic loading conditions.

PILOT NUMERICAL ANALYSES

As part of the work conducted during Phase I, pilot numerical studies have been performed. The intention of this work is to help evaluate the significance of seismic loading on various failure modes and to aid in the identification of other failure modes. The work is not intended to be comprehensive, but rather to highlight areas which need additional study.

Description of the Analyses

The behavior of tieback retaining walls under earthquake conditions is modeled using the finite element method. For this purpose, a dynamic finite element computer program FLUSH (14) is used. The program FLUSH only provides the dynamic increment of stresses; therefore, a static finite element program SOIL-STRUCT (6) is also employed to obtain the initial stresses in the soil and the wall. Details of both computer programs are given in Appendix A.

Wall-Soil System

A one foot thick concrete wall with three levels of ties is used in all the analyses. For the purposes of this study, there is no difference between this wall and a soldier pile wall with an equivalent stiffness. Wall height is 40 feet with a penetration of 10 feet below the bottom of the excavation. The ties are spaced vertically at 10 foot centers, horizontally at 7 foot centers and are inclined at an angle of 14 degrees from the horizontal. The angle was selected to keep the anchors above the bottom of the excavation. The total unbonded length of the tie rods is approximately 20 feet and the bond length is 20 feet. The anchor diameter is 12 inches. A scale drawing of the wall is shown in Figure 3.

Static Analysis

Initial stresses in the soil wall system due to gravity and the static stresses induced by the construction sequence are simulated using SOIL-STRUCT. The state of stress in the soil-wall system, after the final sequence of construction, is used to compute input parameters for FLUSH, in particular the initial shear moduli for the soil elements. Bending moments, displacements of the wall, and forces in the tiebacks, as obtained from static analysis, are added to the dynamic increments to obtain the total value.

Boundaries on both the left and the right sides are fixed in the horizontal directions only, whereas, the bottom boundary is fixed against vertical movement as well. The finite element mesh used with SOIL-STRUCT is shown in Fig. 4. It contains 238 elements and 270 node points.

The soil which is modeled in the analysis is a homogeneous silty sand. Its properties are listed in Table 3. Further details of the static input parameters are discussed in Appendix A.

Dynamic Analysis

In order to incorporate the stresses obtained from the static analysis into the dynamic analysis, the dynamic finite element mesh was chosen to be as similar as possible to the mesh used in the static analysis. The mesh used for FLUSH, shown in Fig. 5, contains 214 elements

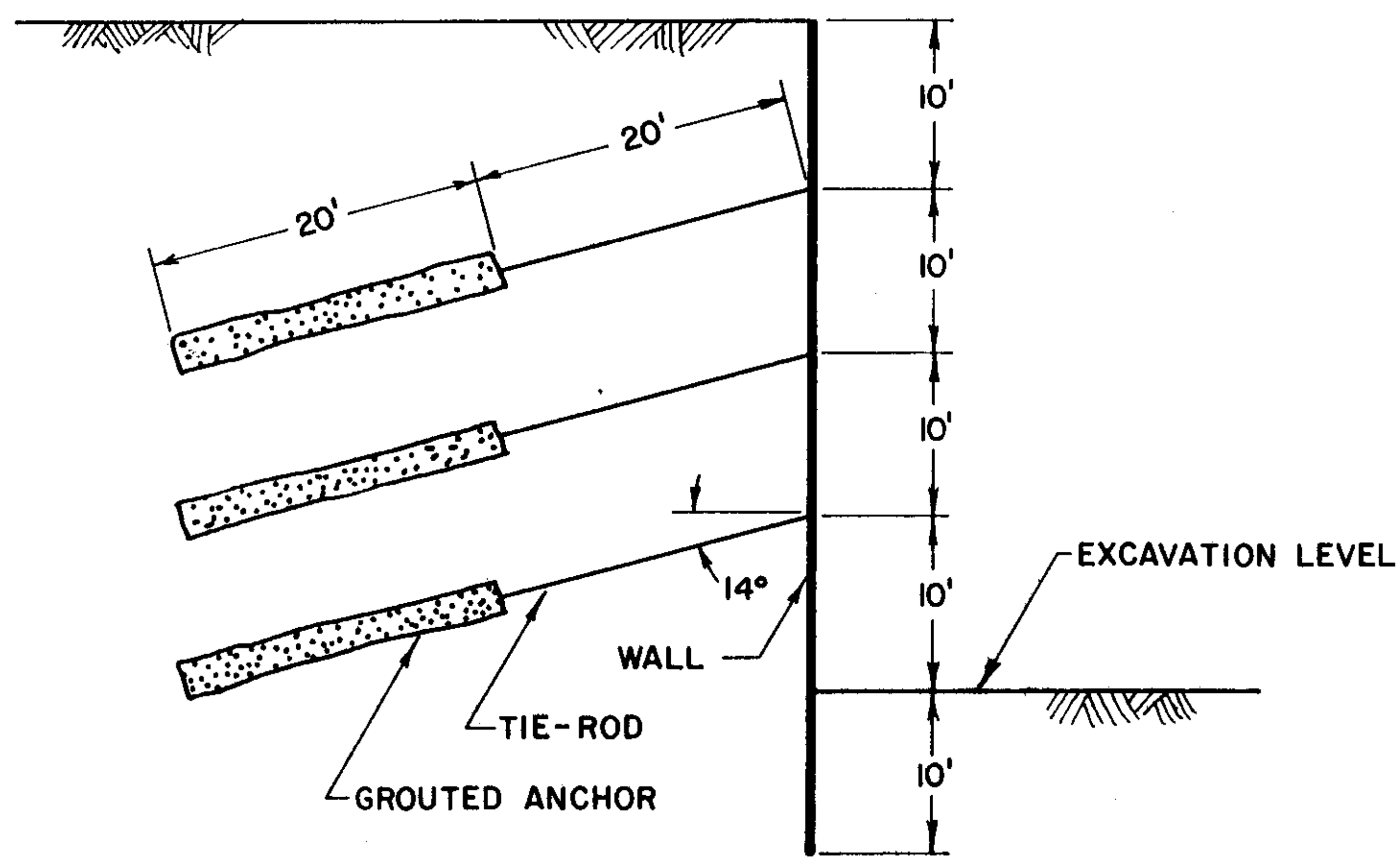


Figure 3 Tieback Wall Used in Numerical Analyses

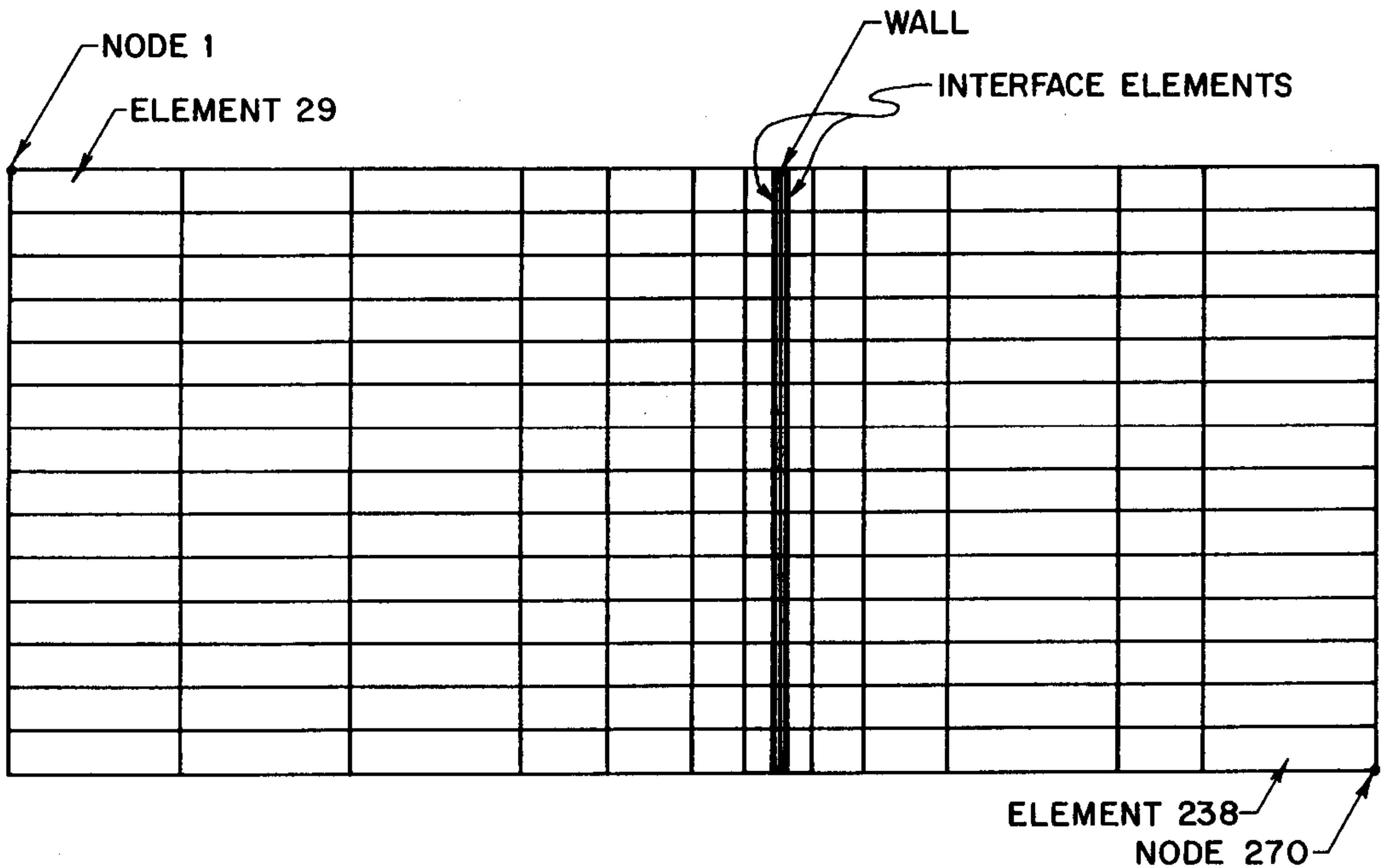


Figure 4 SOIL-STRUCT Finite Element Mesh

Table 3. Soil Parameters Used in the Numerical Analyses

Parameter	**Computer Program**	
	SOIL-STRUCT	FLUSH
Unit Weight	125 pcf	125 pcf
Poisson's Ratio	0.3	0.3
Shear Modulus	-	$G = K_2(\sigma_m)^{1/3}$ *
Initial Tangent Modulus	$E_i = P_a K_m(\sigma_3/P_a)^n$ **	-
Unload-Reload Modulus	$E_{ur} = P_a K_{ur}(\sigma_3/P_a)^n$	-
At Rest Earth Pressure Coeff.	0.4	-
Friction Angle	36°	-
Cohesion	100 psf	-

* See Appendix A for details

** $n = 0.5$

P_a = 2,120. psf (atmospheric pressure)

K_m = 500.

K_{ur} = 700.

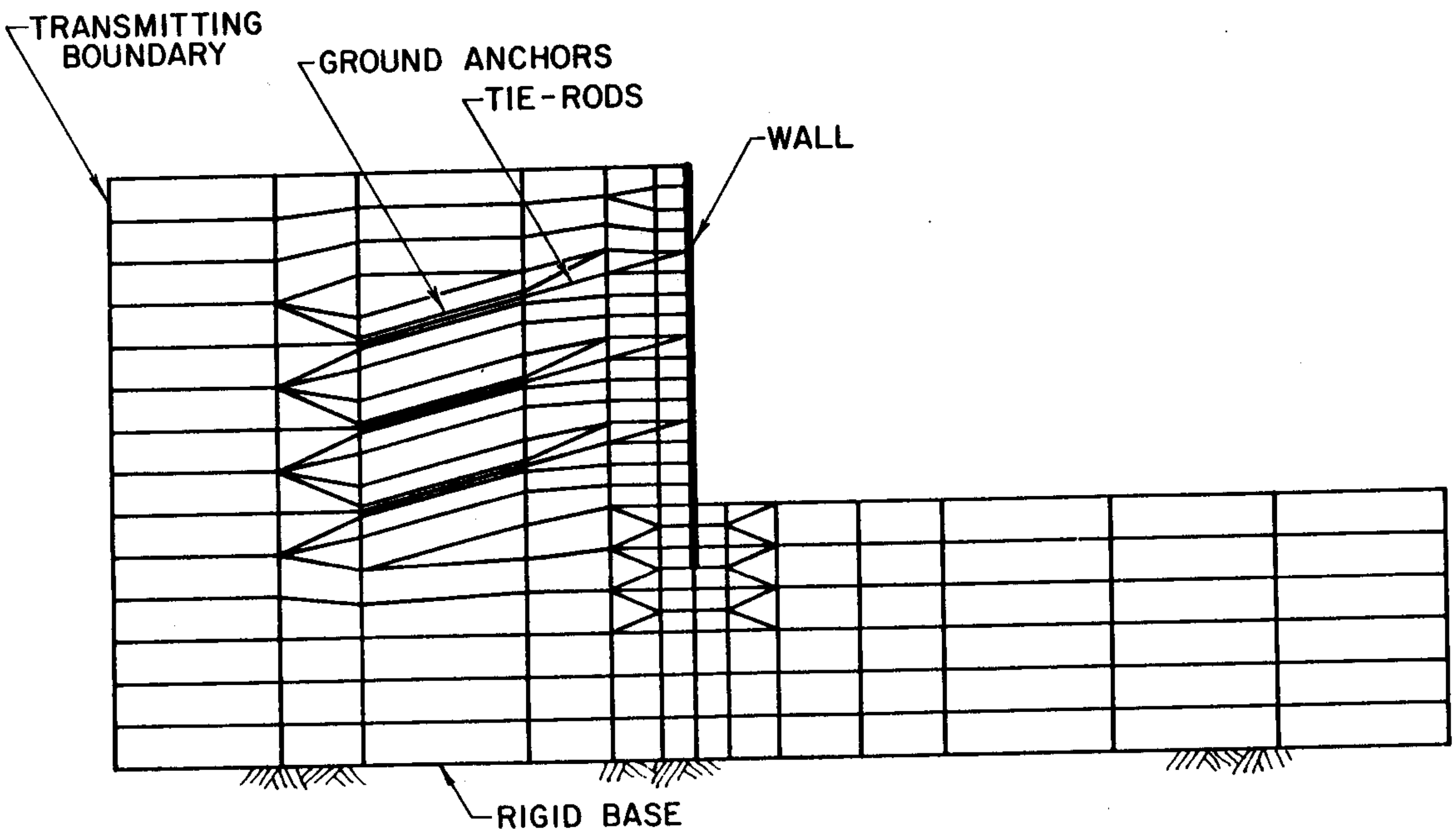


Figure 5 FLUSH Finite Element Mesh

and 230 node points. Some of the major differences between the two meshes are:

- the ground anchors are modeled by solid elements in FLUSH and are not modeled in SOIL-STRUCT.
- interface elements were not able to be used as they are not available in FLUSH.
- the right boundary of the dynamic mesh is placed at a greater distance due to certain modeling constraints as described later.

These differences are not considered to detract significantly from the results. They may be evaluated in Phase II part of the study.

The Dynamic Model

In the FLUSH mesh, each ground anchor is modeled by two linear elastic concrete elements. Ties connecting the wall to the ground anchors and the wall itself are both simulated by one dimensional beam elements. A transmitting boundary is used on the left side while the right side is free to move in the horizontal direction only. The right side boundary is placed at a distance of two and a quarter times the height of the excavation, thereby reducing the effects of the boundary conditions on the area of interest; i.e., the region close to the wall. Because the boundary on the left side is a transmitting boundary, it can be placed closer to the wall. The horizontal boundary at the bottom represents a rigid base or bedrock, and is the location of the input accelerations.

Dynamic Input Parameters

Input parameters for FLUSH include soil, concrete and steel properties, and a record of ground motion. Material properties include

Poisson's ratio, unit weight, shear modulus at low strain, shear modulus for the first iteration (for soil) and damping ratio for the first iteration. The actual values used for the FLUSH analyses are given in Appendix A along with a detailed description of how they were obtained.

The earthquake record used in the analyses was provided with the FLUSH program. The time increment of the provided record is 0.02 seconds and the duration of the record is 7.68 seconds. A number of different time steps were used and finally 0.15 seconds was chosen, as it produced the maximum dynamic amplification in the soil-wall system. The maximum input acceleration was set at 0.15g. The predominant period of the record is 0.32 seconds. A time history of the input motion used is shown in Fig. 6. A more detailed sensitivity study to correlate the period ratio between the soil system and the earthquake record is recommended in the next phase.

RESULTS

General

Although the numerical studies conducted to date are preliminary in nature, they do shed considerable light on the problem. Of particular importance is the insight they give to the question of "in-phase" vs. "out-of-phase" motion. One of the reasons often given for not considering seismic loading separately for tieback walls is the intuition that the wall and the retained soil are "in-phase." This means that the wall and the retained soil tend to move together and there is little to no relative movement between them. It is often assumed that this will result in negligible increases in load on the wall.

Although this study tends to confirm the in-phase behavior of the wall and the retained soil, it clearly points out a second, perhaps more

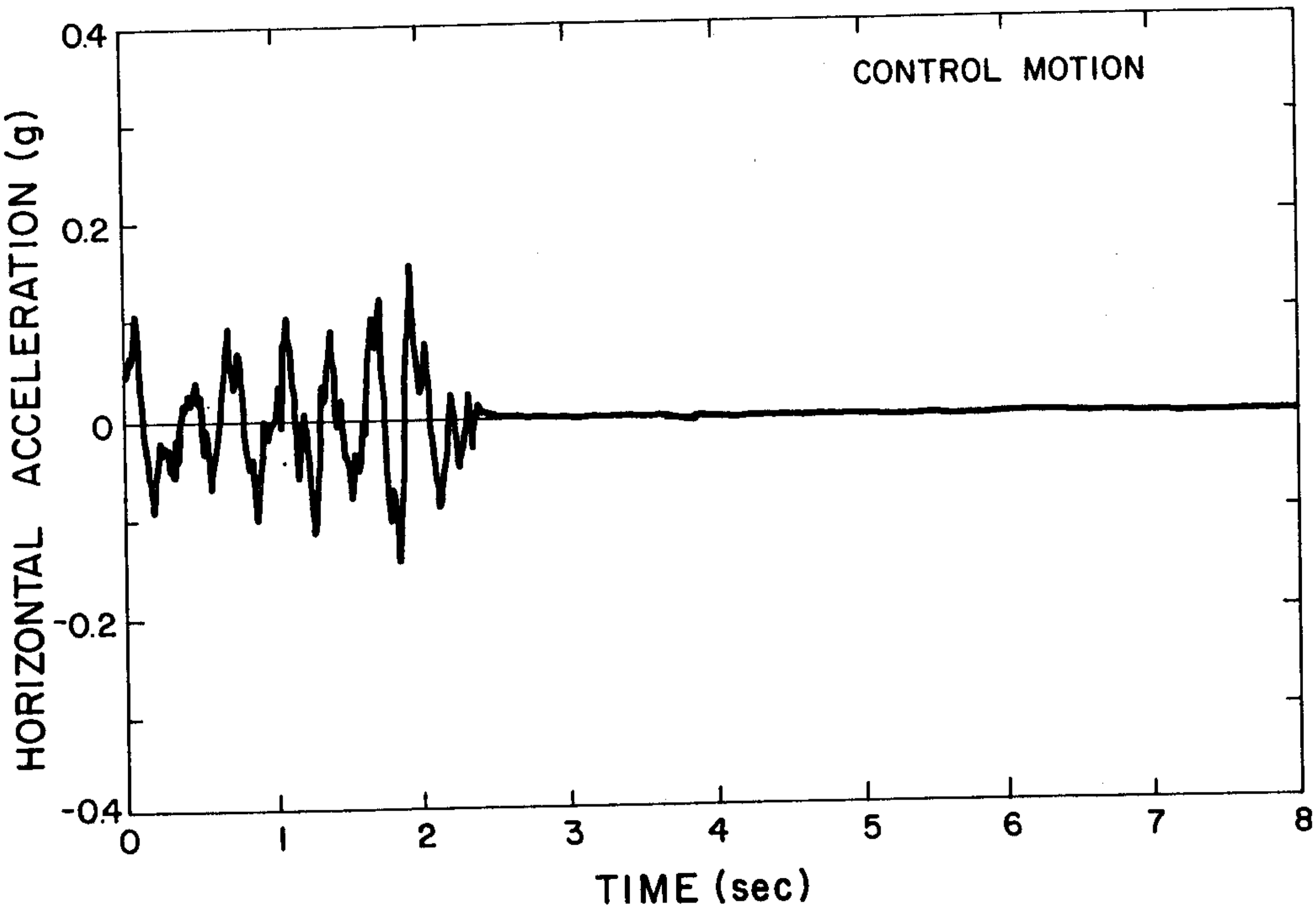


Figure 6 Input Motion Used in the Numerical Analyses

serious, consideration. On the basis of these numerical studies, it appears that the soil above and below the bottom of the excavation may not move in-phase. The analyses show that these two "layers" of soil move in opposite directions during a significant portion of the shaking. This can be seen in Fig. 7 which shows the horizontal acceleration of each node point 1.965 seconds after the beginning of shaking. The vertical dashed lines represent zero horizontal acceleration. The solid lines represent the horizontal accelerations of the soil at that location in the mesh. Where the solid line is to the left of the corresponding dashed line, the soil is accelerating to the left, where the solid line is to the right of the dashed line, the soil is accelerating to the right. The magnitude of the acceleration is given by the distance between the solid and dashed line. Although actual magnitudes of acceleration are difficult to pick out, it is clear from this figure that the upper and lower portions of the soil deposit are accelerating in opposite directions. This out-of-phase behavior leads to the development of high horizontal pressures and bending moments.

It was also found that relatively high vertical accelerations are induced in the wall and soil, even though the input motion is entirely horizontal. This increases the bearing pressure at the base of the wall and lowers the safety factor against bearing capacity failure.

Finally, it was found that dynamic amplification of the base motion was produced and peak horizontal ground accelerations of 0.32g were produced, compared to a peak input acceleration of 0.15 g.

The three factors discussed above all influence the response of the wall-soil system. The overall results are presented below for each of the major parameters of wall behavior: displacements, horizontal accelerations,

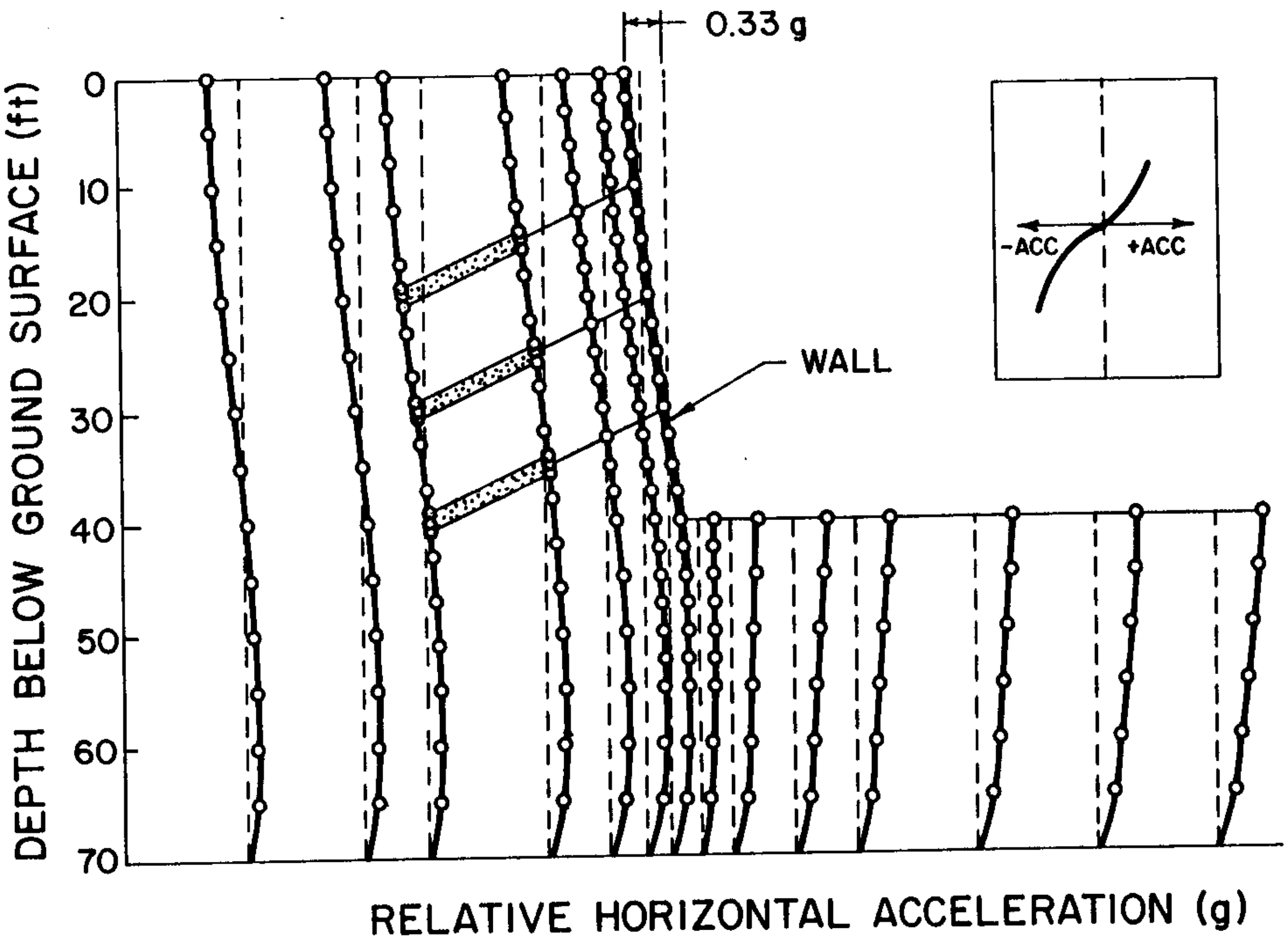


Figure 7 Horizontal Acceleration at Time = 1.965 Seconds

pressures on the wall, bending moments, vertical accelerations and tie forces.

Wall Displacements

The FLUSH program does not calculate absolute displacements of the system, but rather pseudo displacements. However, based on these data it is clear that some residual displacement does occur due to shaking. Additional studies will have to be conducted to determine the potential magnitude of these displacements.

Horizontal Acceleration

Maximum horizontal accelerations in the wall occur at the top where they are slightly more than twice the maximum input motion. The wall acceleration decreases with depth and at the bottom of the wall it is only slightly higher than the peak input acceleration. Amplification of horizontal accelerations in the free field is lower than that observed both in the soil near the wall and the wall itself. This is illustrated in Fig. 8 which shows time histories of horizontal acceleration at various locations.

Soil Pressures on the Wall

Large dynamic horizontal pressures are induced on the wall near the bottom of the excavation. This can be seen in Fig. 9 which shows the static pressure and the maximum dynamic pressure on the wall vs. the depth below the ground surface. Below the excavation level the net soil pressure on the wall is plotted. The upper 30 feet of wall are subjected to only negligible dynamic pressures, but beginning at the level of the lower

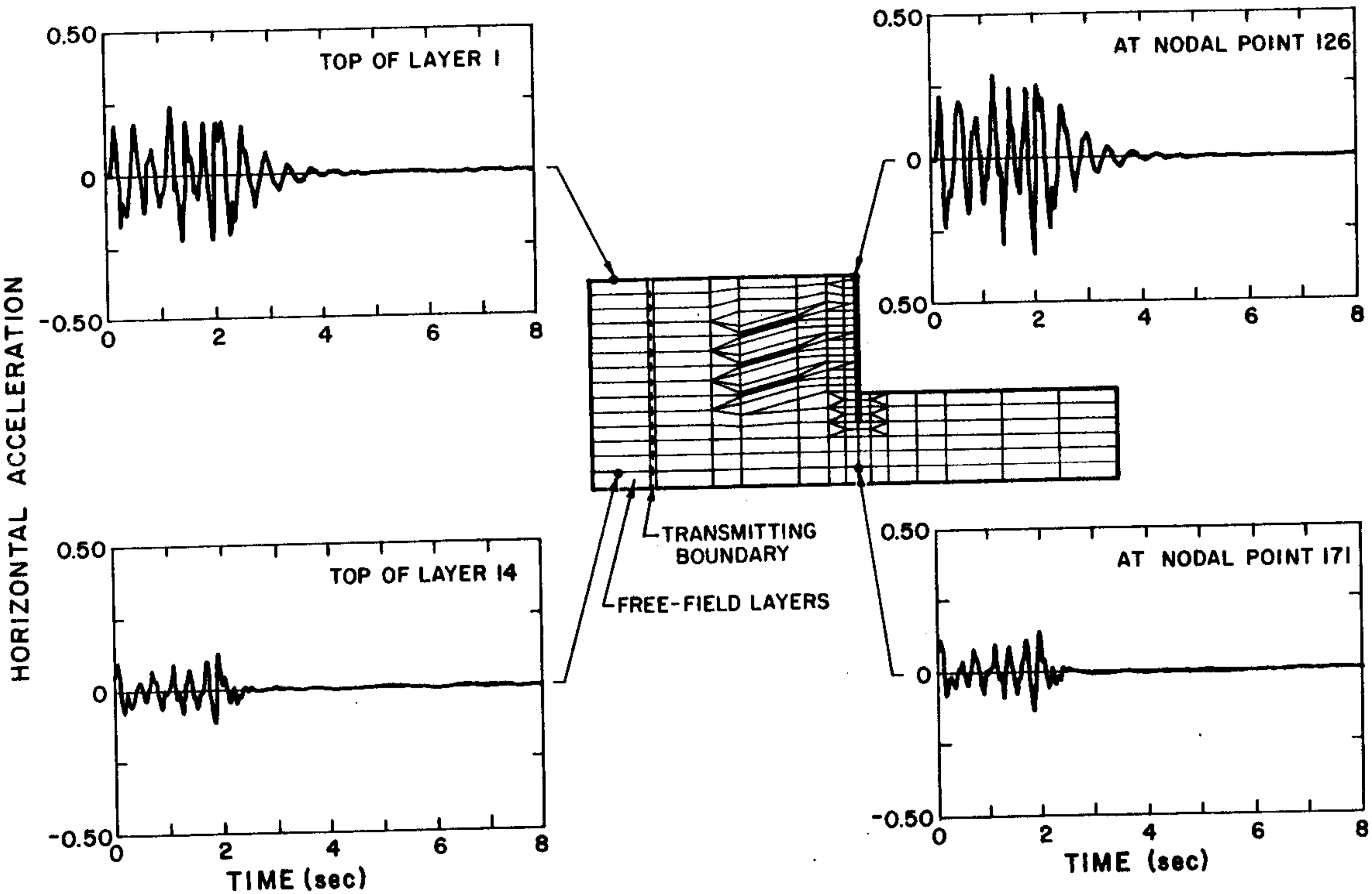


Figure 8 Time Histories of Horizontal Ground Motion at Four Locations

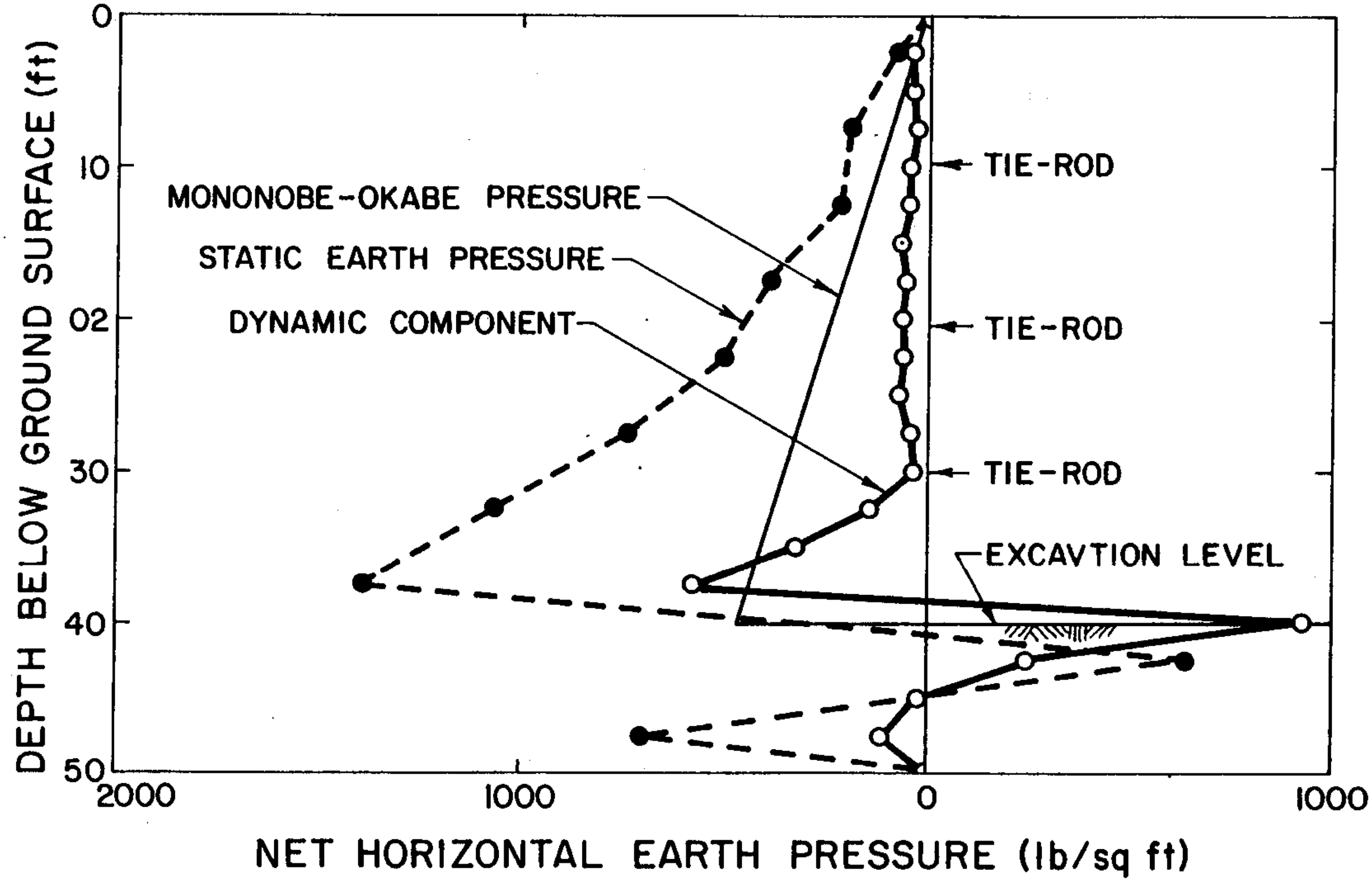


Figure 9 Earth Pressures Acting on the Wall

tieback the pressures increase dramatically to a peak of approximately 600 psf two feet above the excavation. This dynamic increment is approximately 40% of the static pressure at this location. At approximately 7 feet below the excavation level the net dynamic pressure is 725 psf, more than 7 times the static value. The reason for these large increases in pressure is the out-of-phase motion of the upper soil vs. the lower soil.

The dynamic earth pressure predicted using the Mononobe-Okabe equations are also plotted on Fig. 9. A wall friction angle of 18 degrees and a horizontal ground acceleration of 0.15 g were used to obtain this pressure diagram. In this case, the Mononobe-Okabe method overpredicts the dynamic earth pressure by a significant amount except for a small 2 foot interval near the excavation level.

Bending Moments

The movement of the wall and the dynamic pressures generated by the out-of-phase behavior of the soil above and below the excavation level lead to large dynamic bending moments in the wall, as illustrated in Fig. 10. In this figure the static bending moment and the peak dynamic bending moment plotted vs. depth below the ground surface. The absolute value of the bending moment is plotted in this figure to make the comparison easier to see. The maximum static bending moment is approximately 5,500 lb-ft per foot and occurs just below the upper tie rod. The dynamic moment at this location is 1,100 lb-ft per foot, approximately 20% of the static value. The peak dynamic bending moment is approximately 4,600 lb-ft per foot and occurs at the excavation level. This is almost 3 times the value of the static moment at the excavation level. The combined static and dynamic

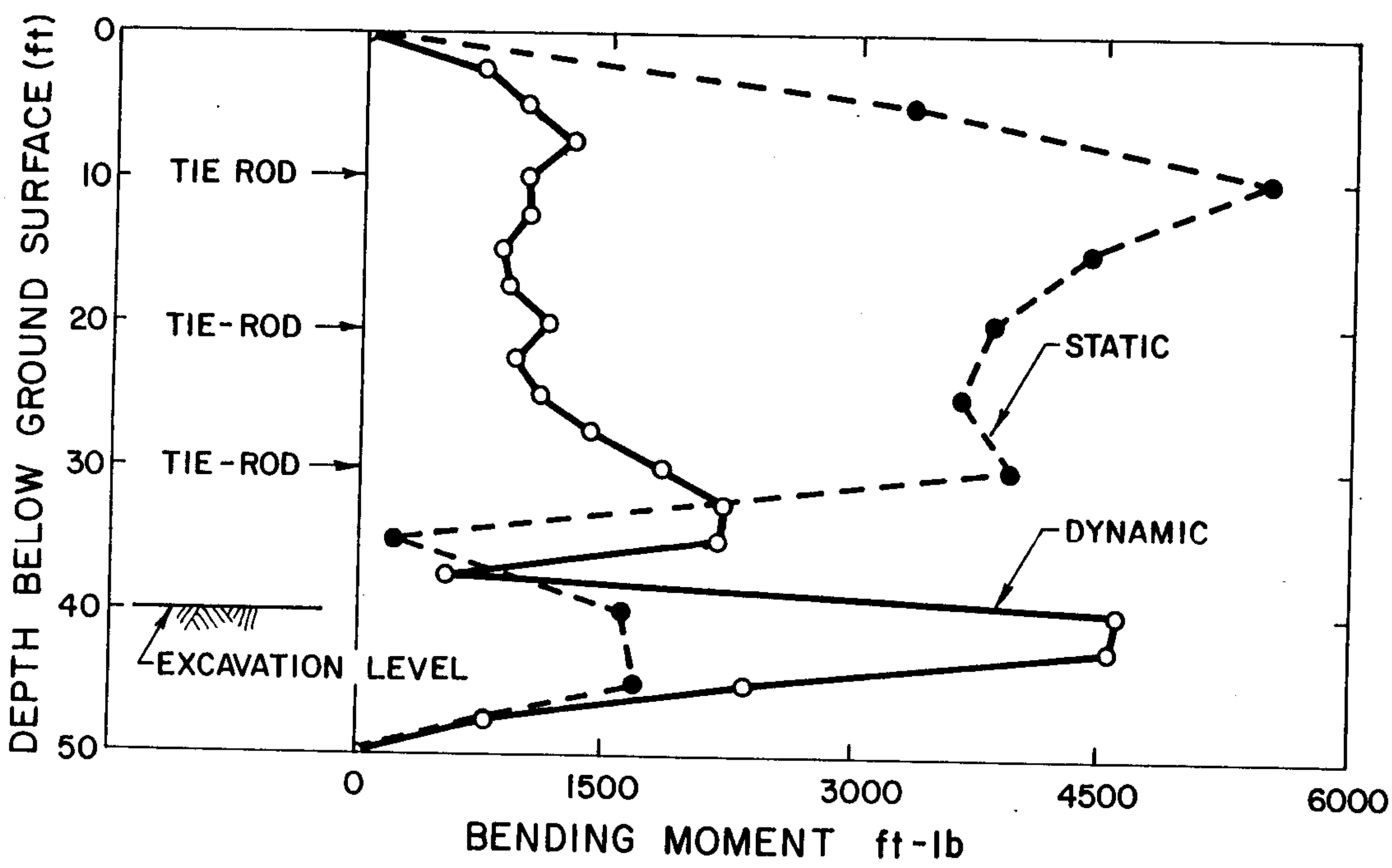


Figure 10 Static and Peak Dynamic Moments in the Wall

moment at this location is approximately 6,300 lb-ft per foot, 15% higher than the peak static moment in the wall.

Vertical Accelerations

Very large vertical accelerations, approximately equal to the maximum horizontal input motion, are induced in the region between the wall and the anchors. This is illustrated in Fig. 11 which shows the time history of vertical acceleration at node point 101, located at the ground surface behind the wall. Sitar and Clough (27) also noted high vertical accelerations when analyzing the seismic response of steep slopes in cemented sands. It is felt that these accelerations are real and not due to difficulties in the numerical procedure. Rocking of soil-wall system is the most likely cause of the vertical accelerations.

The wall itself also undergoes significant vertical accelerations as shown in Fig. 12. These accelerations cause large axial stresses to develop in the wall which are transmitted to the ground. This is shown in Figs. 13 and 14. Fig. 13 is a plot of maximum dynamic axial force in the wall as a function of depth. The peak axial force of 18,000 lbs per foot is about 2.4 times the weight of the wall.

Fig. 14 shows the peak dynamic vertical stress in the ground immediately below the wall. A value of over 2000 psf is obtained in a small region at the base of the wall with a significantly larger region subjected to over 1000 psf. It is possible that the lack of an interface element between the wall and the soil might affect the predicted values of vertical pressures beneath the wall. To estimate the possible effects of slippage between the wall and the soil, an analysis was conducted in which the stiffness of the soil immediately adjacent to the wall was reduced to a

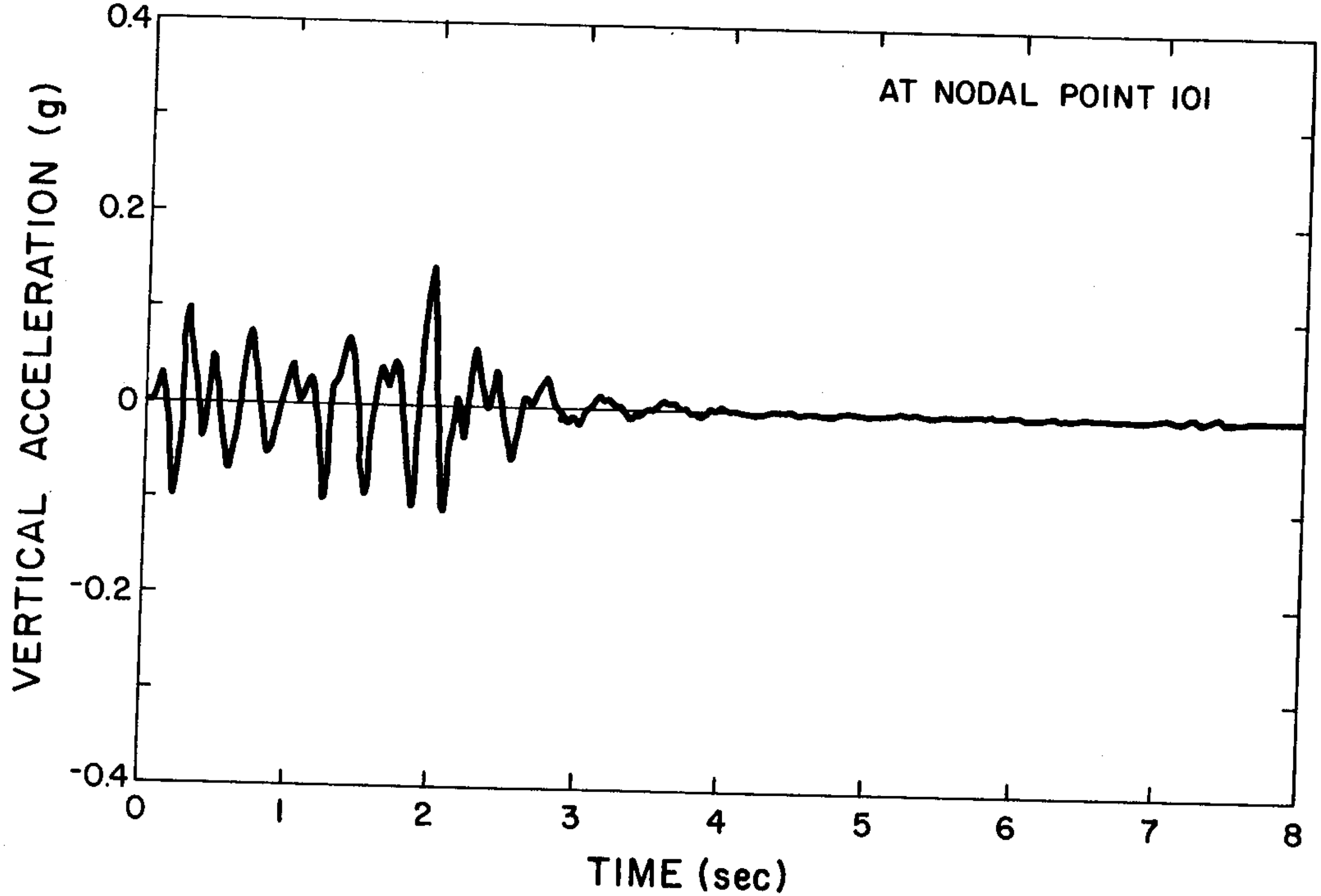


Figure 11 Time History of Vertical Acceleration at Node 101

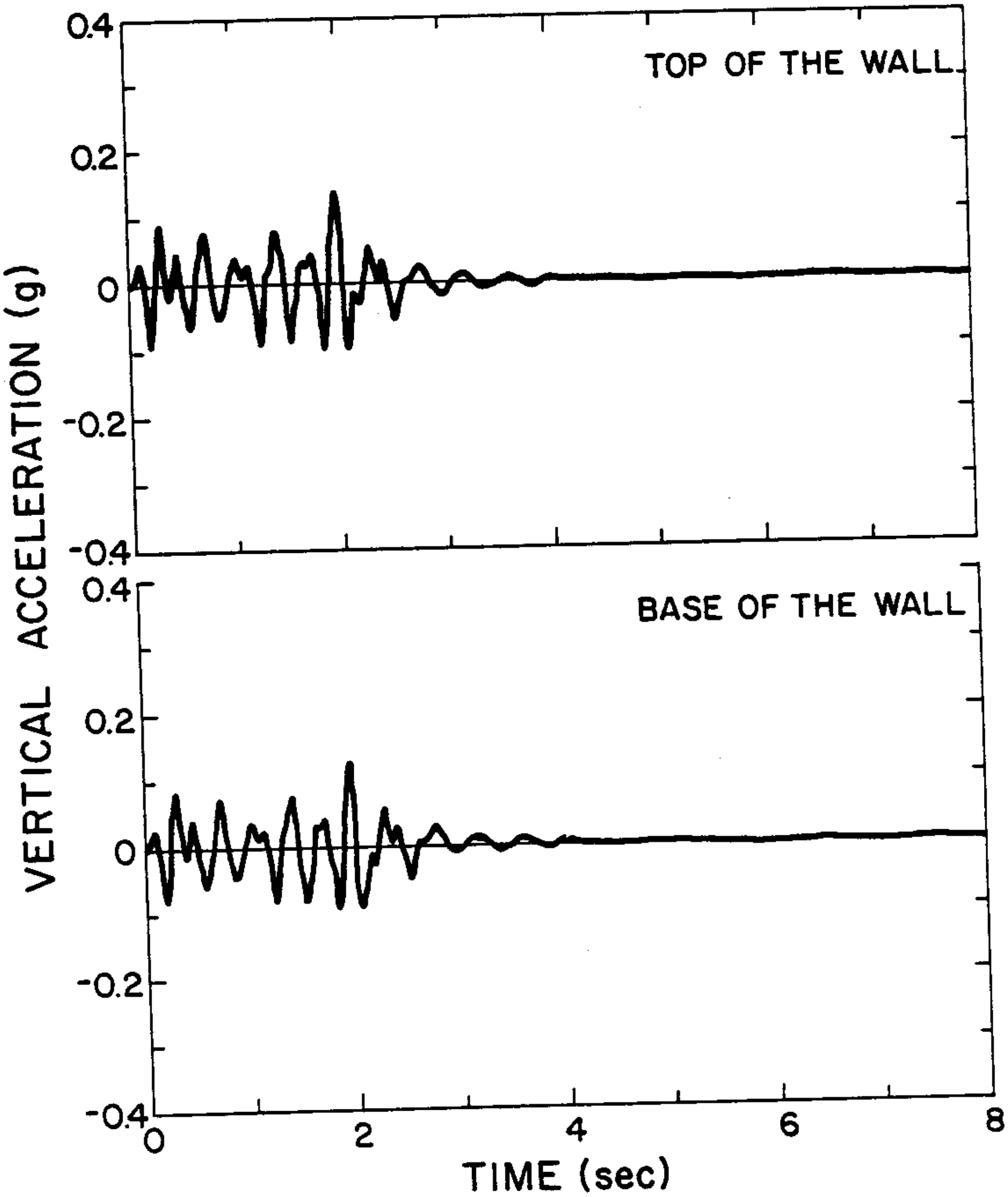


Figure 12 Time Histories of Vertical Acceleration at Top and Bottom of the Wall

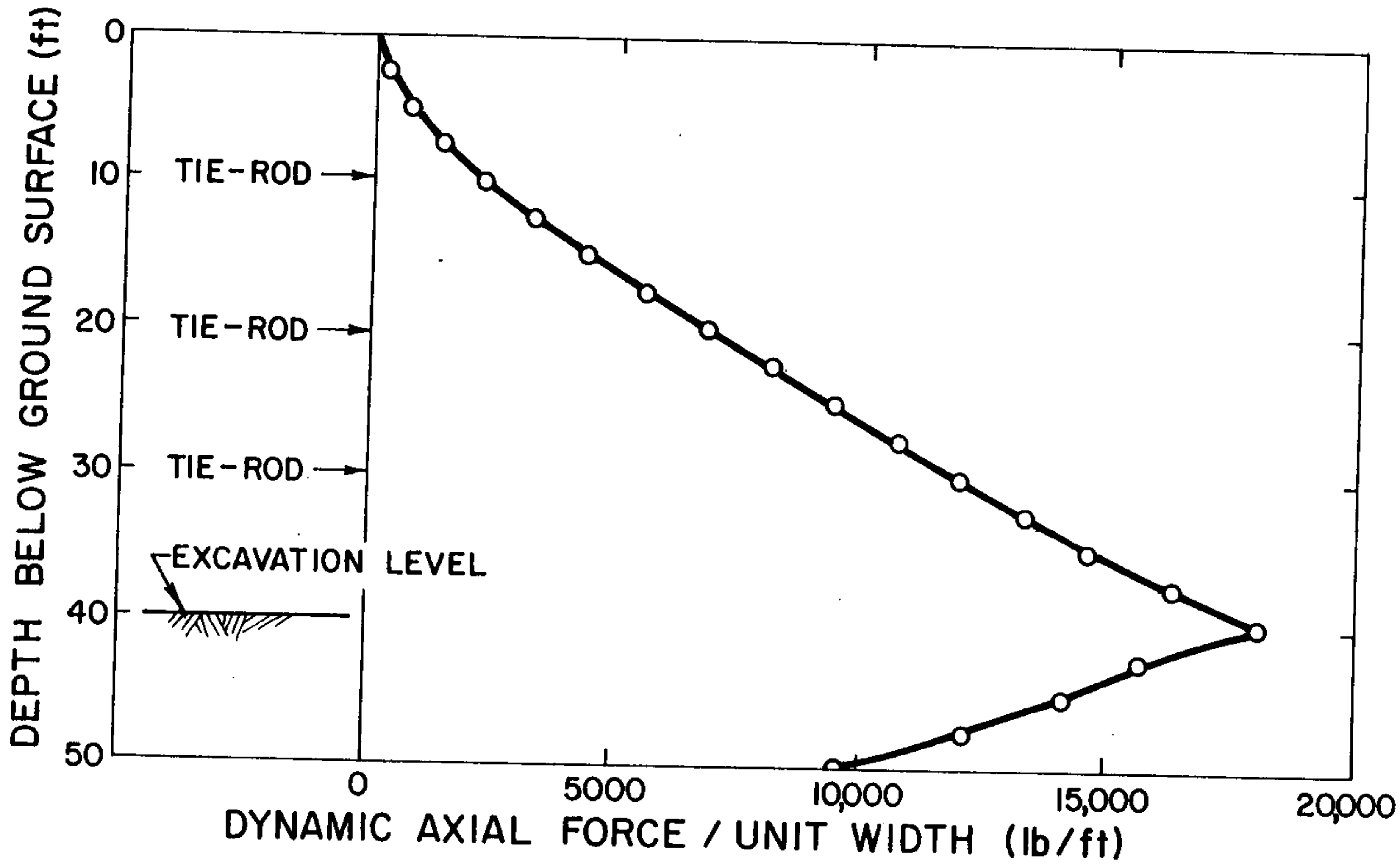


Figure 13 Peak Dynamic Axial Force in the Wall

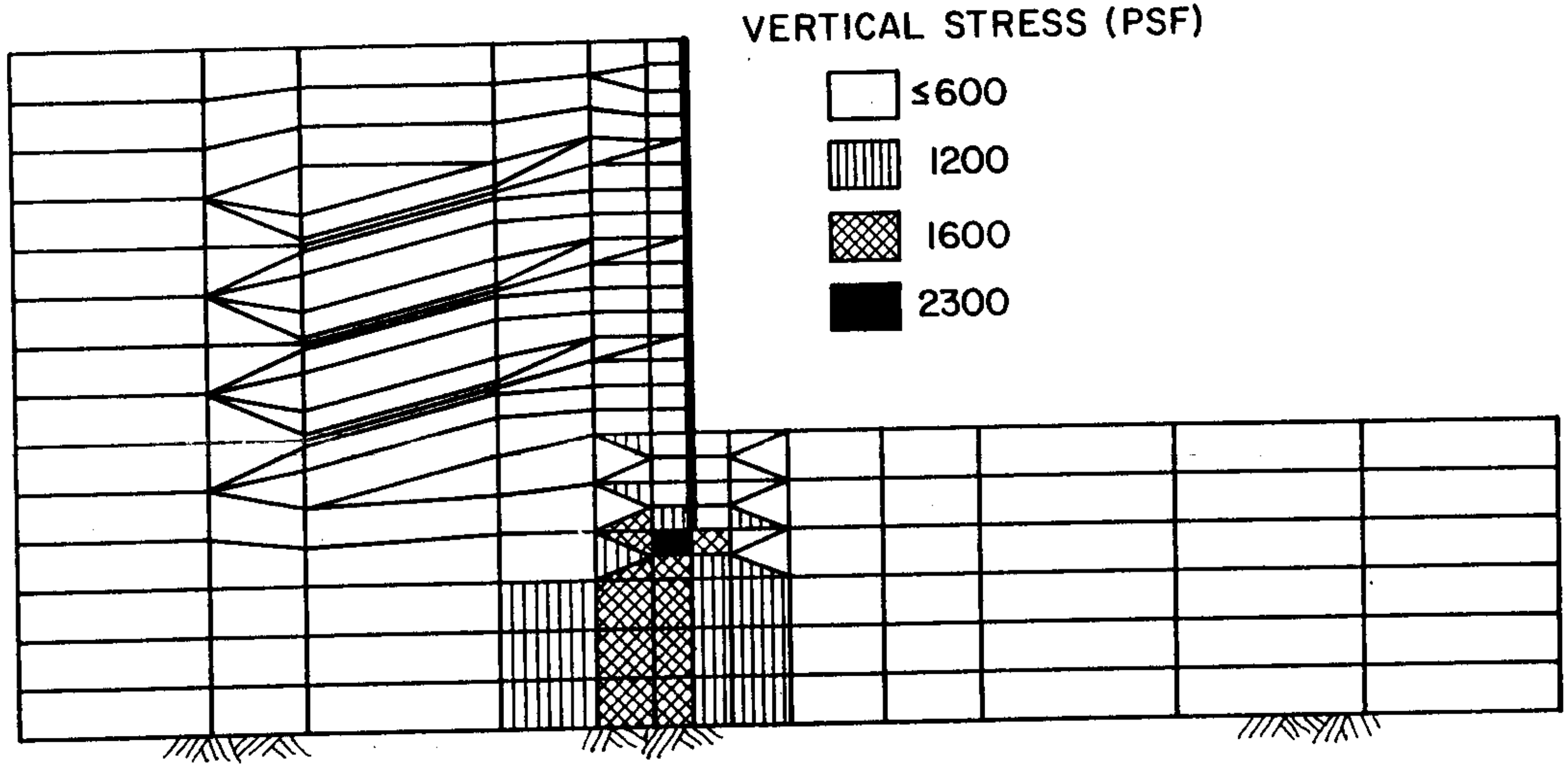


Figure 14 Peak Dynamic Vertical Stresses beneath the Wall

negligible value. The results indicate a negligible effect; thus indicating that the lack of an interface element does not appear to be a problem.

Tie Forces

In these analyses negligible dynamic load increments are induced in the tie rods. The peak dynamic loads are -428 lbs, -434 lbs, and 413 lbs for the upper, middle and lower tie rods, respectively. In comparison, the prestressing force applied during construction is 11,000 lbs. It should be remembered, however, that all three levels of tie rods are located in the upper layer of soil above the base of the excavation. There is little relative movement between the wall and the soil in this layer. It is probable that significant tie forces would develop in the tie rods in situations where the anchors are located in the lower soil layer. An even more important consideration in this case is the possibility of load reversals occurring if the anchor is moving out-of-phase with the wall. As discussed in the literature review, significant loss of anchor strength may develop in such a case.

Discussion of Results

It is important to realize the limitations of the numerical study described above. Because the objective was to do pilot work to determine if a more detailed numerical study was justified, only one wall and soil profile was considered. It is with care, therefore, that general conclusions should be made regarding other wall geometries, input motion and soil profiles. This study does show that for at least one set of conditions significant dynamic loading can occur.

It is likely that there are worse combinations of wall geometry, soil profile and input motion so the results of this study cannot be taken as an upper bound. An example of a case in which the loads on the wall might be much more severe would be a wall which penetrates through a soft layer into a very stiff one. Considerably lower forces might be generated when the soil deposit is relatively uniform and the wall height is small compared to the depth of the deposit.

Due to the two dimensional nature of the FLUSH program, it was necessary to model a continuous concrete wall, rather than the more common soldier pile wall. Although the wall properties were selected such that the stiffness of a typical soldier pile wall was used, it is likely that a soldier pile wall would behave somewhat differently. Perhaps the largest difference would be in the predicted lateral soil pressures below the excavation level. Horizontal pressures on the soldier piles are likely to be higher than on a continuous concrete wall. This might lead to higher than predicted lateral movement of the wall.

Perhaps the major limitation of this study comes from the selection of a single soil profile. It appears that the major factor in the response of the wall to seismic loading is the out-of-phase behavior of the upper and lower soil layers. In any future work a parametric study will be needed to determine the range of influence this phenomenon can have. The investigation might be done in terms of period ratio; that is the ratio of the fundamental periods of the two layers versus the fundamental period of the input motion.

The location of the anchors relative to the bottom of the excavation may also be very important. In an extreme case, an anchor embedded in a very stiff layer below the excavation might be subjected to load reversals when the upper soil is much softer. As described in the literature review, load reversals significantly affect the capacity of anchors and could lead to failure of the anchor much more quickly than repeated loading without load reversal.

The situation of sloping ground behind the wall is also very significant because of the increased ground stresses, and especially in light of the number of tieback walls retaining sloping ground already in place in Washington. Investigation of this problem requires some modifications to the FLUSH finite element mesh and is beyond the scope of this project. Any additional work should include an investigation of the influence of sloping ground on wall behavior.

PRELIMINARY RECOMMENDATIONS

The results of the pilot studies described above indicate that static design of tieback walls may not, in all cases, provide adequate safety against seismic loading. It appears clear that the magnitude of dynamic forces acting on the wall will be greatly influenced by the relative motion of the soil above and below the excavation level. In cases where they tend to move in phase, dynamic loading on the wall may be relatively low. However, when out-of-phase movement occurs, there may be danger of wall failure. Because of this, it is difficult to say whether or not an existing wall is adequately designed for seismic loading without doing some ground motion studies.

Because of the preliminary nature of the work done to date, it is premature to suggest any immediate changes in WSDOT design procedures. However, additional work clearly is necessary and the recommendations for immediate future research suggested in this report should be carried out as soon as feasible. This additional work should lead to specific recommendations for both existing and future tieback walls.

APPENDIX A

Computer Program SOIL-STRUCT

SOIL-STRUCT (6) is a finite element program for analyzing problems involving arbitrary increments of the construction sequence, such as excavation, anchored wall construction, slurry wall placement, concrete or fill placement and dewatering. The non-linear stress dependent stress-strain behavior of the soil and the linear elastic bending of the wall are modeled using two dimensional isoparametric elements. Soil-structure interaction between the wall and the soil mass is simulated by incorporating one dimensional interface elements. Tie-rods and braces are modeled by one-dimensional bar elements.

In addition to the stresses in the wall and the soil, the output includes wall and soil displacements, changes in the tie-back load, stress levels and soil properties. Soil properties consist of the tangent modulus and poisson's ratio for each element after the last increment of the construction sequence.

Computer Program FLUSH

FLUSH (14) is a finite element program for approximate three dimensional analysis of dynamic soil-structure interaction problems. Two dimensional plane strain elements are used to simulate the soil mass and one dimensional beam elements are used to model structural materials. Also, void elements are available to model underground voids or cavities. An equivalent nonlinear technique in the frequency domain is utilized to model nonlinear effects. Additional capabilities are summarized as follows:

- transmitting boundaries are available to simulate the infinite extent of the soil mass.
- 3-D approximation possible by incorporating viscous boundaries.
- deconvolution of the input motion inside the program allows the location of excitation to be at any depth.

The program has some disadvantages for use in analyzing the seismic response of tieback walls. It does not have any interface elements to model relative movement between the wall and the soil and the anchor and the soil. Also, since the displacements and the velocities are computed in the frequency domain they are not very realistic or reliable. While these shortcomings might be significant for a complete parametric study of the problem, it is felt that they are not so important as to invalidate a pilot study such as this.

Soil Properties

In the finite element program FLUSH, the required material properties are:

1. Poisson's ratio (A poisson's ratio of 0.3 was used in all the analyses.)
2. Unit weight of soil (A unit weight of 125 pcf was used)
3. Shear modulus at low strain
4. Shear modulus for the first iteration
5. Damping ratio for the first iteration

The shear modulus used in this study for the first iteration is equal to seventy five percent of the shear modulus at low strain; therefore, only the shear modulus at low strain and damping ratio for the first iteration are discussed in detail in this section.

<u>Shear modulus at low strain</u>

Strains of 1 x 10^{-4} percent are considered as low strains. The equation used to estimate the shear moduli of sands at this strain level is:

$$G = 1000\ K_2\ (\sigma_m')^{1/2}$$

where

σ_m' = Mean principal effective stress in lb/ft^2

K_2 = Empirical parameter that takes account of the the material type, its relative density and the strain amplitude.

This relationship was introduced by Seed and Idriss (24) based on a comprehensive study by Hardin and Drenevich (9,10). They showed that the parameter K_2 is most influenced by the initial relative density or void ratio and the strain amplitude and least influenced by the effective angle of friction ϕ', coefficient of earth pressure at rest K_o and the effective vertical stress σ_v'. Shear modulus decreases as the strain amplitudes increases therefore a set of reduction curves is needed in the analysis. Such reduction curves are shown in Figs. 15, 16, and 17. Fig. 15 is especially useful when field data are obtained using the standard penetration test. Reduction curve for 90% relative density is used for the material surrounding the anchors and 75% relative density curve was used for the rest of the material. Higher relative density is chosen to reflect the effects of grouting pressures and prestressing loads.

For this study the magnitude of K_2 used are 61.5 and 70 for realtive density of 70% and 90% respectively. The mean principal stress is computed by the following formula:

$$\sigma_m' = (\sigma_x' + 2\sigma_y')/3$$

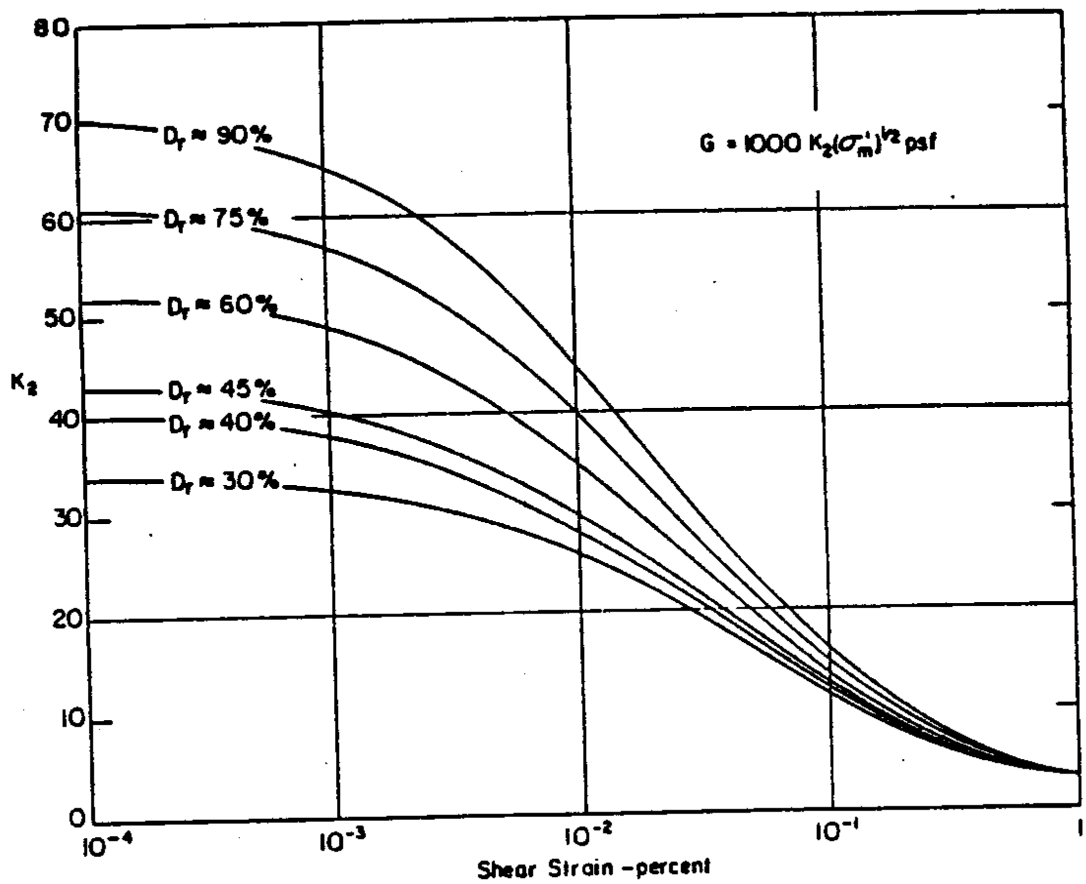


Figure 15 Shear Moduli of Sands at Different Relative Densities (After Seed and Idriss 1970)

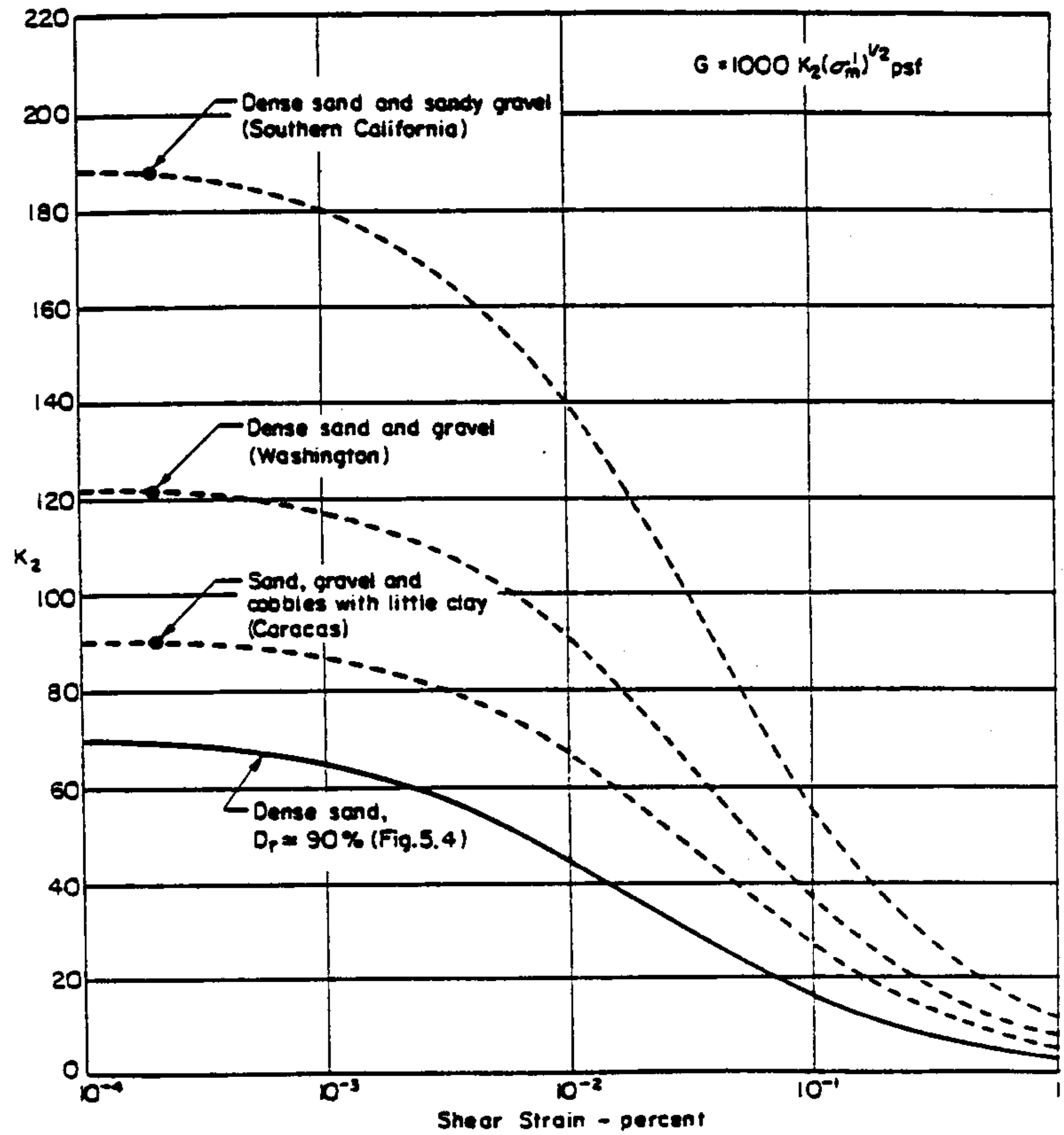


Figure 16 Moduli Determinations for Gravelly Soils
(After Seed and Idriss 1970)

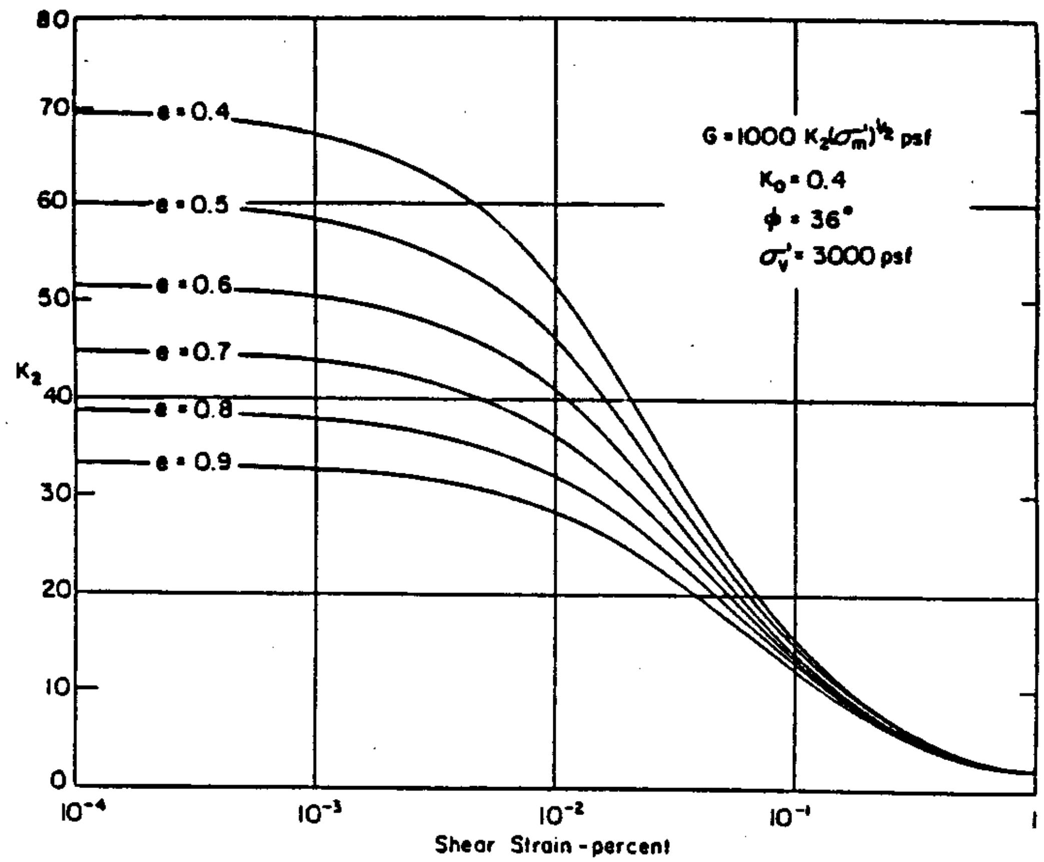


Figure 17 Shear Moduli of Sands at Different Void Ratios (After Seed and Idriss 1970)

where σ_x' and σ_y' are horizontal and vertical effective stresses respectively; their values are obtained from SOIL-STRUCT output.

Damping ratios for sands

Hardin and Drenevich (9,10) showed that shear strain amplitude, initial void ratio, effective mean principal stress and the number of applied cycles were important factors in the determination of damping ratios, whereas the angle of friction, octahedral stress and the degree of saturation were least important. An equation to estimate the maximum damping ratio was presented as:

$$\beta_{max} = 30 - \log_{10}N$$

where N = Number of cycles.

Seed and Idriss (24) pointed out that β_{max} is most influenced by the first five cycles. For the usual range of interest, say $5 \leq N \leq 30$, the effect of the number of cycles is minimal. They calculated the effective mean principal stress using K_o and effective vertical pressure and went on to show that K_o was a minor factor for a wide range of values and that the effective vertical stress was only significant for values less than 500 psf. For a typical soil with a unit weight of 125 pcf, such low pressures are only important in the top four feet. Figure 18 was provided by Seed and Idriss (24) based on a number of investigations as typical curves for damping ratio versus strain amplitude.

Damping ratio for the first iteration, β_i is estimated from an equation provided by Hardin and Drenevich as follows:

$$\beta_i = \beta_{max} \{1-(G_i/G_{max})\}$$

where G_i = Shear modulus for the first iteration

G_{max} = Shear modulus at low strain

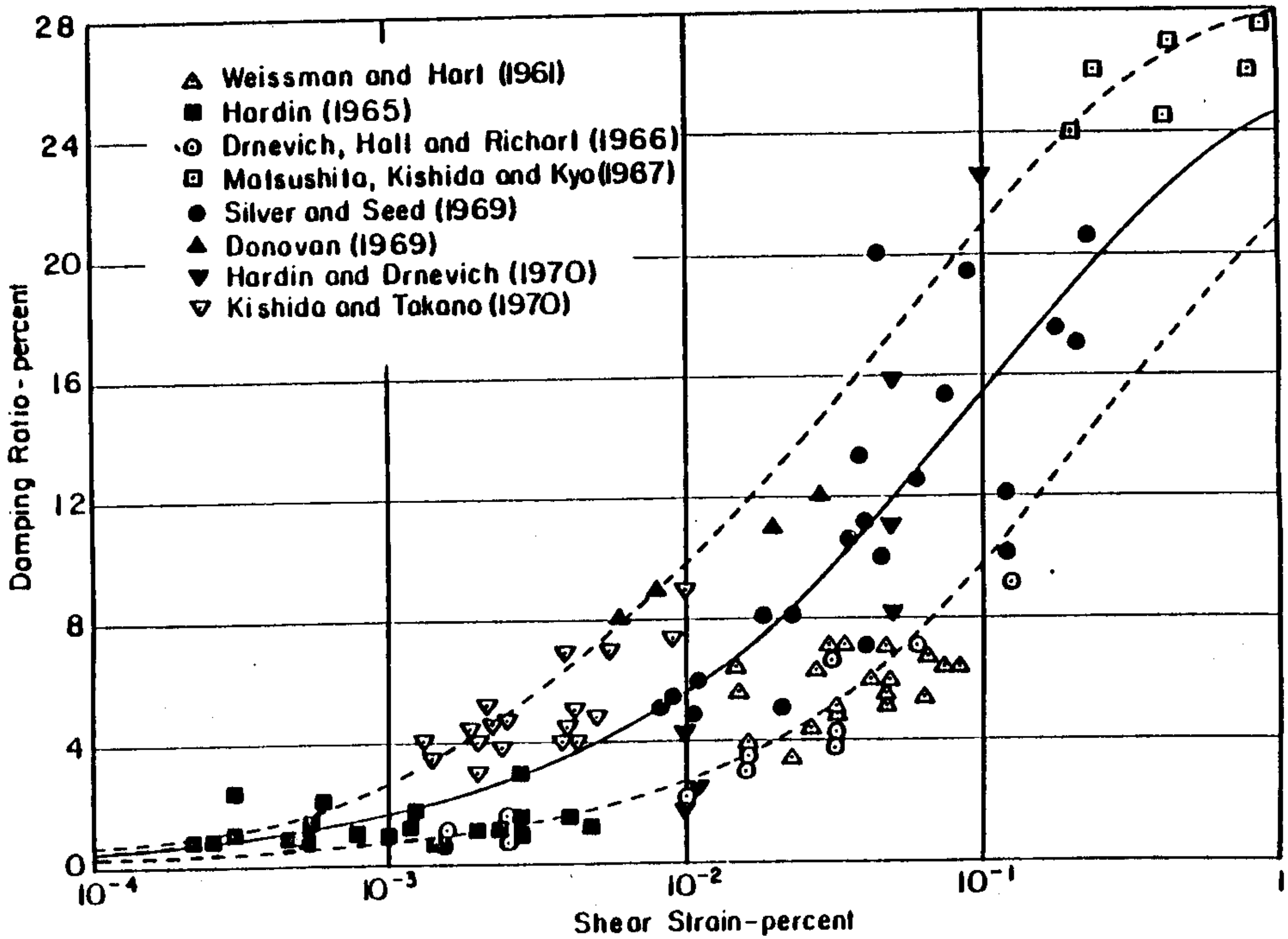


Figure 18 Damping Ratios for Sands (After Seed and Idriss 1970)

β_{max} is calculated using an N value greater than five.

The present study was limited to the use of sands only, although curves for shear moduli and damping ratios of saturated clays are available. Seed and Idriss note that the data on clays show significant scatter due to the inherent problems of sample disturbance in laboratory tests and relatively small strains associated with field tests.

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