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Earthquake Commission Research Foundation - Project 99 / 419

Effect of Vertical Earthquake Shaking on the Displacement of Retaining Structures

Research Report

for the Earthquake Commission

EQC

Opus: *an accomplished work, a creation, an achievement*

Earthquake Commission Research Foundation – Project 99 / 419

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Research Report

for the Earthquake Commission

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Abstract

Research was carried out to assess the displacement performance of retaining walls, and the significance of vertical earthquake shaking, which can occur in near fault areas. The research included a literature review and numerical analyses using a numerical model of a Reinforced Earth wall.

A literature review indicated that vertical earthquake shaking has not been seriously considered until recently. Recent papers highlight the importance of vertical shaking to the assessment and design of retaining walls. No model studies have been carried out to verify the effects of vertical shaking or to assess parameters that might be important.

The study showed that a wide range of displacements can occur in different earthquakes, even when they have similar peak ground accelerations. The energy content was found to be a more influential parameter to assess wall displacements in earthquakes.

The vertical shaking had a significant effect on the displacement of the wall, when the energy content of the earthquake was significant, or where the frequency of the earthquake shaking was such that it was similar to the natural response frequency of the wall. This indicates that the frequency content of the earthquake and resonance effects can be important. The magnitude of the displacements depend on both the energy content of horizontal and vertical shaking and the amplitude of shaking. Where the horizontal and vertical shaking are not significantly large (say distant from earthquake sources), then the vertical accelerations are shown to have little effect.

The results suggest that there is a possibility that vertical shaking may increase the flexibility of the retaining structure, and modify its natural period. Where this shifts the period of a structure to a frequency with significant energy content in an earthquake, this can lead to resonance effects and hence greater displacements.

Currently design is based on pseudo-static methods using horizontal peak ground accelerations. The study shows the importance of the energy, frequency content and vertical shaking of earthquakes. This is important for design of retaining systems supporting other structures. Further research is recommended to assess the performance of different wall systems under earthquakes with different characteristics, and to develop appropriate design parameters and methods where vertical shaking is likely to be a major component.

Technical Abstract

Research was carried out to assess the displacement performance of retaining walls, and the significance of vertical ground motions, which can occur in near fault areas. The research included a literature review and numerical analyses using a finite-difference FLAC model of a Reinforced Earth wall. The model analyses were carried out for a wall with an aspect ratio of one (the ratio of reinforcement length to height of wall). Four different earthquake time history records from California were used in the analyses and were modified to obtain a greater range of earthquake shaking. The study was effective in assessing the displacement performance of walls and the parameters that have a significant influence on wall behaviour, under vertical ground shaking.

A literature review indicated that vertical earthquake shaking has not been seriously considered until recently. Recent papers highlight the importance of vertical shaking to the assessment and design of retaining walls. These recent studies have involved pseudo-static analyses. No model studies, either physical or numerical, have been carried out to verify the effects of vertical shaking or to assess parameters that might be important.

This was the first known model study of the influence of vertical shaking and was carried out using a finite-difference numerical program FLAC and incorporated both vertical and horizontal shaking.

The analyses confirmed the vulnerability of the upper strips to pullout during earthquake shaking, a factor which has long been recognised in the practical design of reinforced earth structures.

Displacement of this robust wall structure was less than 25 mm for a modest energy of shaking associated with the four earthquakes chosen, and up to 200 mm for larger shaking with a higher energy. The maximum displacement (other than at the top where reinforcement pullout occurred) with horizontal shaking alone from the analyses was only 23 mm.

The study showed that a wide range of displacements can occur in different earthquakes, even when they have a similar peak ground acceleration. The sum of the Power Spectral Density (PSD), which represents the energy content, was found to better relate to the variation of displacements in these different earthquakes.

The vertical shaking had a significant effect on the displacement of the wall. The vertical shaking generally had very little effect on wall displacements when earthquake shaking of modest energy and peak ground accelerations similar or only slightly larger than the design accelerations were used. However, earthquakes with different frequency characteristics can lead to significantly larger displacements than with horizontal shaking alone. This was observed in the case of the Loma Prieta earthquake record used in the study. This may be due to the frequency content of the earthquake in relation to the

natural response frequency of the retaining structure. This indicates that the frequency content of the earthquake and resonance effects can be important.

The magnitude of the displacements depends on both the energy content of horizontal and vertical shaking and the amplitude of shaking.

An appraisal of the results has led us to a hypothesis that the vertical shaking could increase the flexibility of the retaining structure, and modify its natural period. Where this shifts the period of a structure to a frequency with significant energy content in an earthquake, resonance effects and hence greater displacements can result.

Currently design is based on pseudo-static methods using horizontal peak ground accelerations. The study shows the importance of the energy, frequency content and vertical shaking of earthquakes to the displacement performance of retaining structures. This is important to the design of retaining systems supporting other structures, particularly in near-field areas where vertical shaking can be strong.

Further research is recommended to assess the performance of different wall systems under earthquakes with different characteristics, and to develop appropriate design parameters and methods where vertical shaking is important.

1 Introduction

Opus International Consultants Limited (Opus) has undertaken research on the effect of vertical ground shaking on the displacement of earth retaining structures during earthquakes. The research was supported and funded by the Earthquake Commission's (EQC) Research Foundation. This report presents the results of this research, for publication by EQC.

Competently designed and constructed retaining walls have been observed to perform well in earthquakes. However, there have also been some instances where walls have failed or undergone undue outward displacement. Properly detailed retaining structures can be designed to undergo displacement during large earthquakes without collapse. However, a good understanding of the likely displacements is essential for detailing the walls to displace, and to provide appropriate clearance to adjacent structures. In New Zealand, it is common practice to support structures such as bridge abutments, on fill retained by reinforced soil structures. If the displacement of such walls can be predicted with reasonable confidence, then its effect on the supported structures can be accommodated with more confidence.

Past studies have generally concentrated on the displacement of retaining structures due to horizontal ground accelerations only. In practice, the effect of vertical ground shaking has been disregarded in the estimation of wall displacements. However, given that commonly used retaining structures rely on gravity loads to provide resistance against earth pressures and also earthquake inertia loads, there has been concern over how vertical ground shaking would affect retaining wall performance, particularly in terms of displacements. Recent research has suggested that vertical ground shaking may lead to larger wall displacements. This is of particular concern in areas close to fault rupture (near field) where the vertical component of shaking could be high. Consideration of vertical accelerations in the design of an abutment for the Newlands Overbridge, near Wellington, located close to a major active fault and supporting an important bridge, highlighted the significance of vertical ground motions in the design of retaining structures. This provided the impetus for this research.

While the general thrust of this research is applicable to a variety of gravity walls, the analysis has been carried out for reinforced soil walls, which are increasingly commonly used in New Zealand and worldwide. This research has been carried out using a finite difference computer model subject to dynamic analyses, to assess wall performance. The FLAC (Fast Lagrangian Analysis of Continua) software was used for this purpose. Records from real, past earthquakes were used for the dynamic analyses.

This report presents a summary of this research, a review of relevant literature, the results from the analyses and a discussion of the findings.

2 Objectives

The objective of this research was to assess the displacement performance of retaining walls, and the significance of vertical ground motions, which can occur in near fault areas. This will lead to a more reliable and confident assessment of performance and design of retaining structures, and the structures supported by them.

3 Literature Review

3.1 Introduction

A review has been carried out of literature associated with the observed earthquake performance and research on the displacement of retaining structures and in particular where consideration has been given to the effect of vertical ground motions. A literature search was carried out with the help of TeLIS, Opus' in-house library and information service. Selected literature was sourced and reviewed, and the relevant literature is listed in Section 11.

3.2 Observed Performance

3.2.1 *Historical Reports*

Performance of retaining walls in earthquakes had seldom been reported in detail in the literature, until recently. Many papers give a general indication only, and walls that do not fail are rarely reported. More recently, particularly after the 1995 Hyogoken Nanbu earthquake in Kobe, Japan, there have been a number of reports on the earthquake performance of retaining structures.

As early as 1924, Mononobe is reported to have quoted the poor performance of retaining walls in the 1923 Kanto earthquake in Japan. He also attributed the retaining wall failures to the contribution of significant vertical ground motions in the area.

3.2.2 *1989 Loma Prieta Earthquake, San Francisco, USA*

Collin et al (1992) documented observations of the performance of geogrid reinforced soil structures in the 1989 Loma Prieta earthquake in San Francisco, USA. Reinforced soil structures are reported to have experienced little damage, and performed well in that event.

3.2.3 *1995 Hyogoken Nanbu Earthquake, Kobe, Japan*

Tatsuoka et al (1995 and 1996) provided a summary of the performance of retaining walls associated with railway embankments in Kobe, in the 1995 Hyogoken Nanbu Earthquake (Japan). They reported that masonry walls, leaning unreinforced concrete retaining walls and unreinforced concrete gravity retaining walls, that were located in the areas of greatest shaking in Kobe, performed very poorly during this earthquake. A number of these walls constructed 60 years or more ago collapsed or experienced severe damage. Reinforced concrete cantilever retaining walls generally performed better, but still were significantly damaged and many had to be demolished. The gravity and cantilever walls were apparently designed using pseudo-static methods, for a peak ground acceleration of 0.1g to 0.2g, representing only modest earthquakes.

Comparatively, geogrid reinforced retaining structures are reported to have performed much better, and experienced little damage other than some displacement or deformation. However, these structures were generally in areas with a lesser earthquake shaking, had the benefit of recent seismic design and were generally less than 3 years old at the time of the earthquake. It is noted that these walls apparently had reinforcement lengths of only 35% of their height, which is much shorter than that commonly used in New Zealand.

Tatsuoka et al (1995) drew attention to the Tanata wall, which was in an area of intense ground shaking, and generally performed well, although it displaced by up to 100 mm at the base and 260 mm at the top, see Illustration 1. The up to 6 m high wall was built with geogrid reinforcement, and a rigid reinforced concrete facing. The facing suffered only limited cracking.

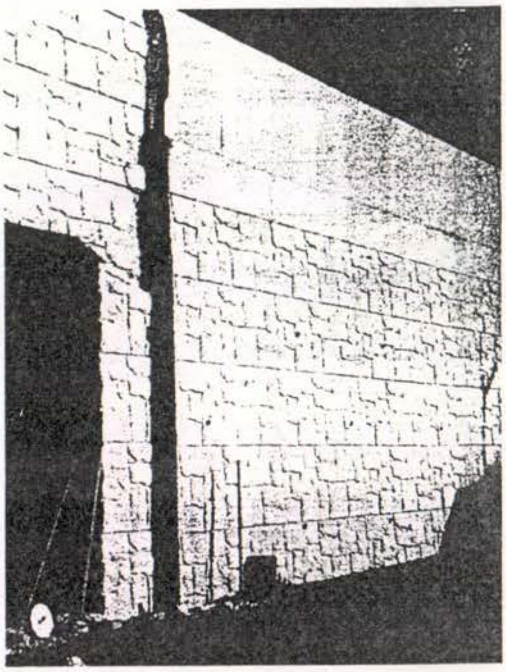


Illustration 1 - Displacement of Tanata Wall in the 1995 Hyogoken Nanbu Earthquake
(after Tatsuoka et al, 1996)

Koseki et al (1999) considered the contribution of vertical accelerations on the factors of safety against failure of this wall, and stated that the effect of the vertical accelerations was much smaller than the horizontal accelerations. However, Ling and Leshchinsky (1998) back-analysed the Tanata wall, and noted that the observed displacement is unlikely to have occurred without the significant contribution of vertical ground shaking, as discussed later in Section 3.4. Tatsuoka et al (1996) described the performance of reinforced soil walls with steel reinforcement (Reinforced Earth® or Terre Armee®) during the 1995 Hyogoken Nanbu earthquake, and observed that their performance was similar to the geogrid reinforced soil retaining structures, with comparable displacements and deformation, but no failure or collapse.

Nishimura et al (1996) also described the good performance of geogrid reinforced retaining structures during the 1995 Hyogoken Nanbu earthquake, although these were subjected to ground accelerations as high as 0.3g to 0.7g. These researchers considered the design methods used for geogrid reinforced soil retaining structures, and suggested that the length of the reinforcement should be increased to improve performance. They have suggested that increasing the length in the upper section would be adequate.

3.2.4 *1999 Chi-Chi Earthquake, Taiwan*

Koseki and Hayano (2000) report on the performance of retaining walls during the 1999 Chi-Chi earthquake in Taiwan. They have observed damage to a large number of retaining structures including concrete walls and reinforced soil walls, and attribute damage to a range of factors such as permanent ground displacement, slope movement, loss of bearing capacity, excessive inertia force and/or insufficient compaction. They suggest the spacing and length of reinforcements as being factors that may have had an influence on the failure of reinforced soil walls. Dobie (pers comm) visited the area affected by the Chi-Chi earthquake and observed walls that failed and those that performed well. He suggested that many walls have performed well in areas of strong horizontal and vertical shaking, and those that failed had additional factors. He suggests that vertical shaking may have allowed blocks to displace allowing the gravel backfill to run out of walls with modular block facing, which sometimes led to failure. He notes that the performance of connections of modular block walls under vertical shaking may require consideration.

3.3 Observed Vertical Motions

Earthquake motions during earthquakes are generally recorded along three orthogonal directions, generally N-S, E-W and vertical, and are reported in the literature. The vertical motions are generally smaller than motions in the principal horizontal direction, and are commonly stated to be of the order of $^2/_3$ of the corresponding horizontal motions. However, the relative sizes of the vertical component of accelerations vary from earthquake to earthquake and also with distance from the epicentre. It is generally recognised that larger vertical motions are likely in near-field areas close to the epicentre or fault rupture.

Bozorgnia et al (1996 and 1998) assessed the vertical motions recorded during the 1994 Northridge Earthquake in Los Angeles, USA, and showed that the vertical motion is sensitive to the distance from the epicentre of the earthquake. They found that the ratio of vertical to horizontal response spectra is high in the near field region, and can exceed the commonly assumed ratio of $^2/_3$. However, it reduces to less than $^2/_3$ at long periods, see Illustration 2.

The illustration shows that the highest vertical to horizontal spectral ratio occurs at a period of less than 0.1 second. Retaining structures are also likely to have a short period of this order.

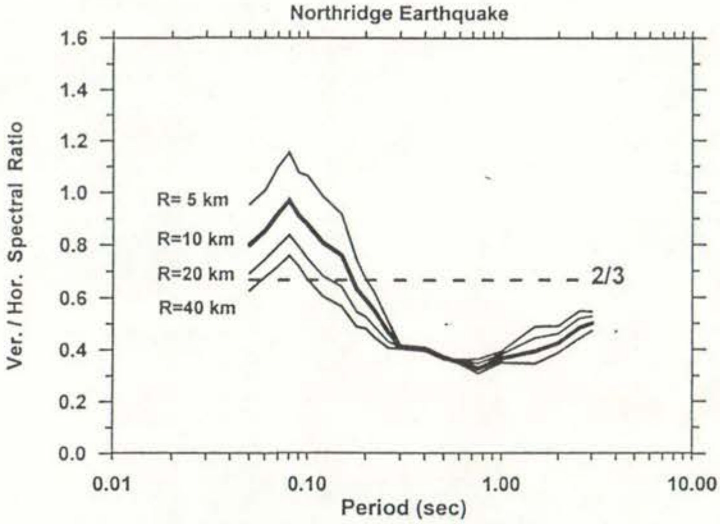


Illustration 2 Vertical to Horizontal Spectral Ratio for the Northridge Earthquake
(after Bozorgnia et al, 1998)

A similar assessment of earthquake records from the Smart-1 Array in Taiwan and the 1989 Loma Prieta Earthquake in San Francisco, USA also showed similar patterns (Niazi and Bozorgnia, 1992; Bozorgnia and Niazi, 1993). These analyses confirm that vertical ground motions would be important for retaining structures in near-field areas, particularly given that the retaining structures are likely to have a short period of response, where the maximum vertical to horizontal spectral ratios occur.

3.4 Past Research on Retaining Wall Displacements

3.4.1 Early Development of Seismic Design

Gravity retaining structures rely on the weight of the wall (including backfill that adds weight) to resist earth pressures. Vertical earthquake motions that are inherently cyclic in nature, can therefore be expected to increase the resistance to sliding when accelerating upwards, and decrease the resistance to sliding when accelerating downwards.

Early work on the seismic analysis and design of retaining walls was carried out in Japan by Okabe (1924) and Mononobe and Matsuo (1929). The Mononobe-Okabe method provides a means of assessing the earthquake increment of earth pressure (over and above the earth pressure under static conditions) on retaining walls. This method or variations of it are widely used to this day.

It is understood that a combination of horizontal and vertical ground motions led to severe damage to earth structures during the 1923 Kanto earthquake in Japan (Ling and Leshchinsky, 1998). In 1924, Mononobe is reported to have highlighted the importance of vertical ground motions in the design and performance of retaining structures.

Chopra (1966) considered the effect of vertical accelerations on earth dams. He considered earthquake records with vertical/horizontal acceleration ratios of 0.2 to 0.31, and concluded that the effect of vertical accelerations were significant for analysing dams. Chopra also suggested that this would be important where the vertical component was unusually large.

3.4.2 *Development of Displacement Based Design*

Earthquake shaking can have a significant effect on the design of retaining walls. Design is largely based on stability criteria. Deformation or displacement of walls leads to cracking of walls and an adverse effect on structures and facilities located on or in the ground supported by the walls. Therefore displacement is an important criterion for the performance or serviceability of the wall.

Newmark (1965) proposed a method of assessing permanent displacements of earth structures by considering a rigid sliding block approach. The approach involves assessing the critical horizontal acceleration at which the factor of safety against sliding reduces to one, when the structure begins to slide, and then integrating over the time periods when the earthquake acceleration pulse exceeds the critical acceleration. Illustration 3 (Cai and Bathurst, 1996) shows this approach. The effect of vertical acceleration was not considered in this method.

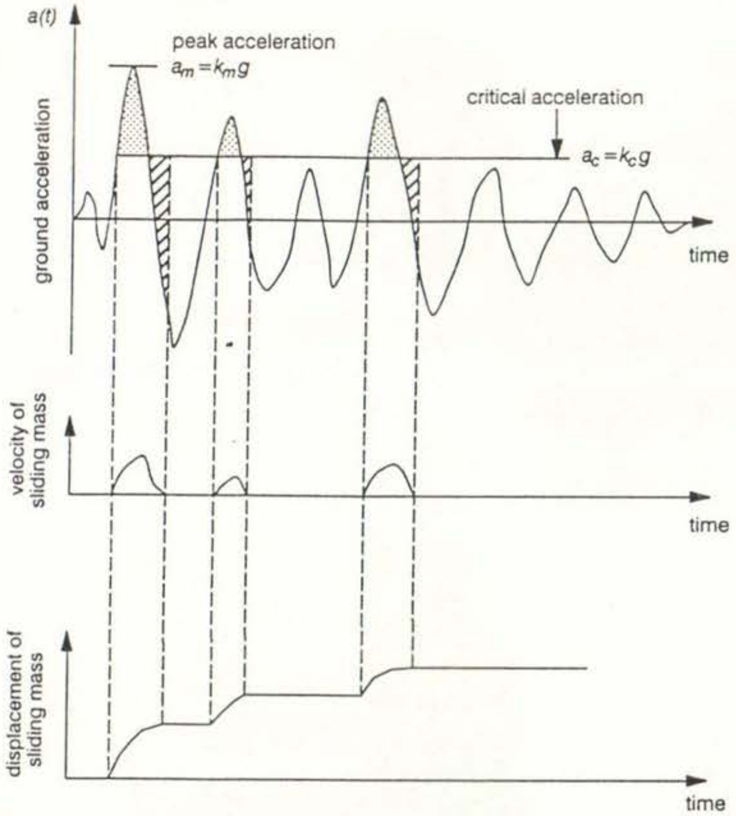


Illustration 3 - Sliding Block Approach to Estimate Earthquake Induced Displacement
(after Cai and Bathurst, 1996)

With the development of codes of practice requiring design for significant earthquake accelerations, it became apparent that design considering the full loads from earthquakes can lead to gravity walls that are very large. From a design point of view, allowing for some horizontal displacement in large earthquakes can enable the wall to be designed for a smaller earthquake load and hence lead to a more economical wall design.

Elms and Richards (1979) proposed a method of design with displacement as a criterion for design, using Newmark (1965)'s sliding block theory as a basis to estimate displacements. The approach involves deciding on an acceptable amount of displacement to ensure adequate performance for the particular case, and allowing that selected limited amount of displacement to occur. This method did not include consideration of vertical earthquake accelerations.

Following the development of reinforced soil retaining structures, Bracegirdle (1980) applied the use of displacement design to the seismic design of Reinforced Earth® retaining walls. Again he only considered horizontal accelerations from earthquakes.

Wood and Elms (1990) reviewed and summarised the results of research on the seismic design of earth retaining walls. They also presented design guidelines that are widely used in New Zealand. The methods do not include consideration of vertical ground motions, and are based on the use of the peak horizontal ground accelerations estimated from the code of practice, currently NZS 4203 : 1992 (Standards New Zealand, 1992).

Ling and Leshchinsky (1996) proposed seismic design of geosynthetic-reinforced soil structures based on permanent displacement limits, and not on a pseudo-static approach alone.

Murashev (1998) reviewed design methods for geosynthetic-reinforced soil structures for use in road projects in New Zealand. He compared the design solutions that would result from using different methods that are in use for the design of geosynthetic-reinforced soil structures. However, he has not considered displacements from vertical earthquake shaking, and its importance for design.

3.4.3 *Model and Numerical Analyses of Earthquake Induced Displacements*

Conventional Gravity Walls

One of the early finite element analyses of the earthquake performance of retaining walls was carried out by Nadim and Whitman (1983). They used a linear-elastic model with slip elements along the inferred failure planes, base and the wall/soil interface. They took into account the amplification of ground motion within the backfill. Displacements were assumed to occur at the slip planes, and the model did not take into account deformations within the soil mass. They considered amplification to be important, depending on the ratio of the dominant frequency of the earthquake motion to the fundamental frequency of the backfill.

Steedman (1984) carried out centrifuge modelling to assess the earthquake performance of retaining walls, but this appears to have not taken into account the effect of vertical ground motions. Based on further centrifuge testing, Steedman and Zeng (1991) verified theoretical analyses showing that dynamic amplification in flexible wall structures can lead to much smaller critical accelerations and hence larger displacements, than indicated by pseudo-static methods. Initially stiff structures that suffer softening can also show similar characteristics.

Whitman (1990) indicated that the deformation of the backfill is important, and can lead to larger wall displacements.

Siddharthan and Norris (1991) used a finite element model to analyse the earthquake behaviour of retaining walls. They used a 2-dimensional elastic-perfectly plastic model with slip elements at the wall-soil and base interfaces and the inclined plane where slip is expected to occur. They suggested that the Elms and Richards model under-predicts the wall displacement at earthquake frequencies close to the resonant frequency of the wall.

Siddharthan et al (1992) proposed that the displacement of the top of gravity retaining walls is a combination of two modes of movement - sliding and tilting, see Illustration 4.

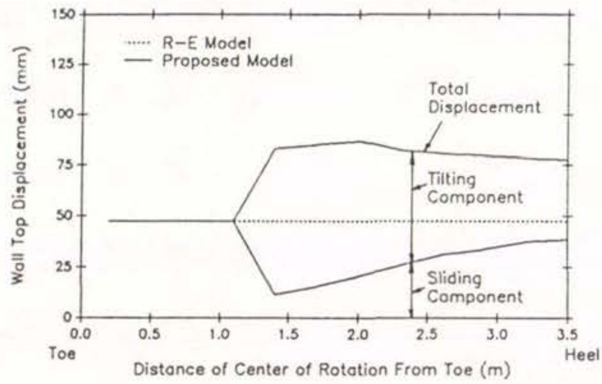


Illustration 4 - Components of Wall Top Displacements
(after Siddharthan et al, 1992)

They extended the Elms and Richards (1979) (R-E) method to include the effects of tilting of walls. They also indicated that contribution to wall top displacement from tilting is less if the foundation soil is stronger, and above an angle of internal friction of the foundation soil of 35°, the effect of tilting is absent.

Steedman (1998) discussed the performance of retaining structures and in particular the waterfront quay walls that underwent significant displacements in the 1995 Hyogoken Nanbu earthquake in Kobe, Japan. He identified the degradation of the strength and stiffness of the foundation materials as being important factors that led to large permanent outward displacements. The displacements were associated with deformation of the foundation materials rather than sliding along the base of the caisson walls.

Reinforced Soil Retaining Structures

Reinforced Earth® walls are proprietary reinforced soil structures, with concrete panel facing and steel reinforcement strips, and have been increasingly used since the 1970s. Shaking table tests on models of Reinforced Earth® walls were carried out at the University of Canterbury by Nagel (1985) and Fairless (1989). The shaking table only facilitated testing with motions in a single horizontal direction. Fairless (1989) considered the strip forces, the failure surface and also the horizontal displacement of the wall. He suggested that the block sliding models provide a reasonable basis for assessing the earthquake induced displacements.

Geosynthetic materials have found increasing use in the reinforcement of earth retaining structures in the 1990s. Modular block facing has also been promoted for use with geosynthetic reinforcement. Much research over the past decade has focussed on reinforced soil walls with more flexible geosynthetic reinforcement. Geogrids comprising geosynthetic mesh are commonly used as reinforcement in such structures.

Cai and Bathurst (1995) used a 2-dimensional finite element code (modified TARA-3) to assess the performance of modular block face walls with geosynthetic reinforcement. Analyses using the model indicated that slips occurred at facing block interfaces indicating the importance of interface shear properties. The model also indicated that a large proportion of the displacement could occur in the lower half of the wall.

Bathurst and Hatami (1998) carried out numeric analysis of a geosynthetic-reinforced soil retaining wall with a stiff facing, using the finite difference package FLAC (Fast Lagrangian Analyses of Continua). They demonstrated that FLAC can be used for modelling walls by comparison with previous finite element model analyses by others. They showed that a larger reinforcement length to height ratio does lead to smaller displacements of the wall. They also compared walls restrained from sliding at the base with ones that are free to slide. Their results indicated that walls that were able to slide at the base lead to smaller deformations at the top of the wall than the ones fixed at the base. This could be due to the smaller earthquake loads that act on a wall that can slide, and the consequential smaller deformation in the upper part of the wall.

Matsuo et al (1998) studied the earthquake performance of geogrid-reinforced walls using shaking table model testing as well as block displacement analyses. They suggested that the sliding block analysis gave one-fourth the displacement from shaking table model tests. However, Simonelli et al (2000) who also carried out shaking table model tests, have indicated that the model tests confirm the general validity of the sliding block analytical models.

3.4.4 *Early Research on the Effect of Vertical Ground Motions on Wall Displacement*

Until recently, the contribution of vertical ground motions was given limited attention in the research on displacement of retaining walls. For example, Seed and Whitman (1970) considered the effect of vertical accelerations to be insignificant in the analyses of retaining walls, based on their assumption that vertical accelerations are generally small.

The effect of vertical motions on the seismic stability of retaining walls was considered by Wolfe et al (1978), based on shaking table model testing. Both horizontal and vertical shaking was considered, using sinusoidal as well as earthquake base accelerations. Considering ±0.2g horizontal and vertical accelerations, they found that the displacements with both horizontal and vertical accelerations were not appreciably higher than with horizontal accelerations alone. Considering earthquake ground motions with a peak vertical acceleration of 0.35g and a peak horizontal acceleration of 0.5g, they found that the displacement with vertical and horizontal accelerations was only slightly more than that with horizontal accelerations alone. On the basis of these results, they concluded that vertical motions could be disregarded in the assessment of the displacement of walls.

Elms and Richards (1979) considered the effect of vertical accelerations in their description of the approach for the seismic design of retaining walls. They showed that even modest vertical accelerations of up to 0.2g have an important effect on the earthquake forces on the wall. However, they suggested that the effect of vertical accelerations is relatively minor when the horizontal acceleration is low (up to 0.2g). They did not consider the effect of vertical ground motions, when they presented a method for assessment of the displacement of walls.

Elms and Richards (1990) discussed the effects of transverse and horizontal accelerations on wall displacement considered by Sharma (1989). They concluded that the effect of vertical accelerations on displacements was negligible. It was suggested that there would be little correlation between the vertical and horizontal components of earthquake shaking, and the effects of downwards and upwards vertical accelerations would cancel each other.

The effect of transverse accelerations was found by Elms and Richards (1990) to be significant. For example, for the 1940 El Centro earthquake, the displacements were 67% higher when the transverse accelerations were also considered.

3.4.5 *Recent Research on the Effect of Vertical Shaking on Displacements*

Conventional Gravity Walls

Siddharthan et al (1992) considered the effect of vertical ground motions on a gravity wall, using a real earthquake record from the 1940 El Centro earthquake. They calculated the displacements for a gravity wall, considering both sliding and tilting. The earthquake record was scaled to a 0.3g horizontal acceleration for both N-S and E-W components, and the same scaling factor was applied to the vertical acceleration. The actual magnitude of the vertical acceleration is not stated.

The analyses considered the wall displacements for :

- N-S horizontal component (scaled to 0.3g) alone
- E-W horizontal component (scaled to 0.3g) alone
- N-S horizontal component (scaled to 0.3g) and vertical component (with same scaling)
- E-W horizontal component (scaled to 0.3g) and vertical component (with same scaling)

The displacements presented were significantly different for the N-S and E-W components, which had both been scaled to 0.3g peak acceleration. Their analyses indicated that the inclusion of vertical accelerations can make a difference of -4% to +60% depending on the roughness of the wall and the N-S or E-W direction of horizontal component used, see Illustration 5.

For the case they considered, it is interesting to note that although the E-W component alone gave significantly larger displacements than the N-S component alone, the percentage increase in displacements when combined with vertical accelerations was higher for the N-S component (42% to 60%) than the E-W component (-0.4% to 9%). That is the vertical acceleration had a much larger effect (42% to 60%) on the wall displacements, when used in conjunction with the N-S horizontal component.

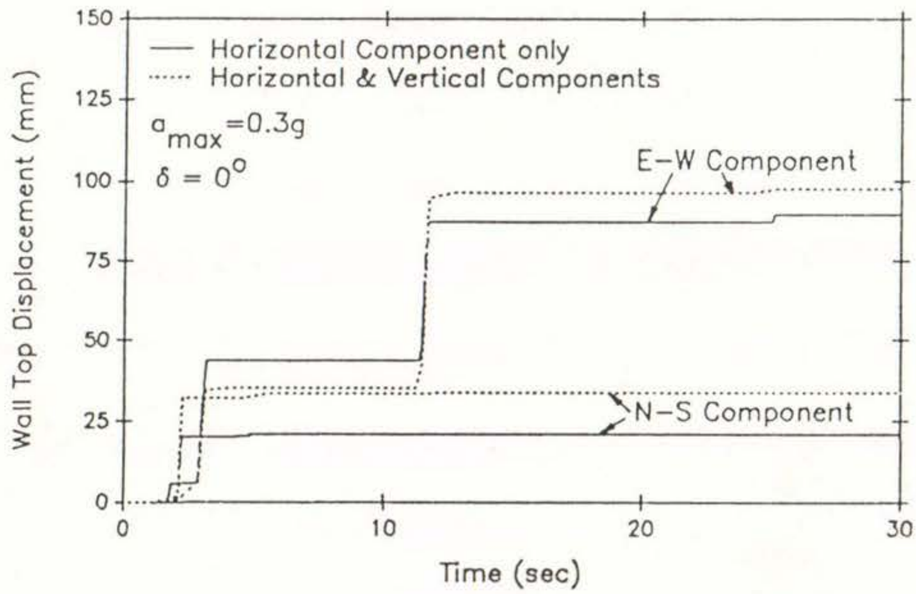


Illustration 5 - Displacements of Top of wall, with Vertical and Horizontal Shaking
(after Siddharthan et al, 1992)

Siddharthan et al (1991) assessed the likely permanent wall displacements in a similar manner for five different earthquakes with magnitudes between 6.4 and 7, representing moderate to large earthquakes. They considered the records in the two orthogonal horizontal directions as well as the vertical accelerations. The assessed displacements varied widely from less than 5 mm to over 212 mm, even though the horizontal accelerations were all scaled to the same peak ground acceleration of 0.3g. This variation was attributed to the frequency content of the records.

The wide variability shows that the peak ground acceleration is not a parameter that can be used effectively to assess earthquake displacements. The actual vertical accelerations for these records were not stated. It is possible that the magnitude of vertical accelerations (and hence the scaled vertical accelerations) also varied and may have been another significant factor contributing to the diverse displacements calculated.

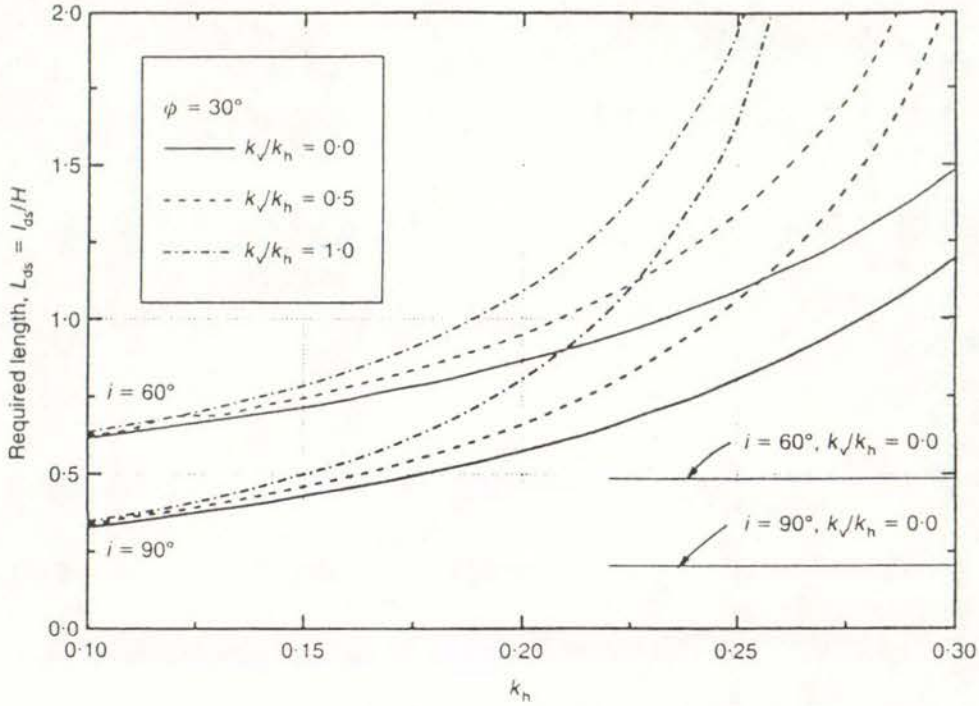
Reinforced Soil Retaining Structures

Cai and Bathurst (1996) assessed modular block faced geosynthetic-reinforced retaining walls using pseudo-static methods based on the Mononobe-Okabe theory and displacements based on the sliding block approach of Newmark (1965). They present charts for estimation of the critical acceleration (to initiate movement) and hence displacement. They show that use of a high vertical acceleration in the analyses of sliding, gives a lower critical acceleration. This would lead to a larger displacement.

Ramakrishnan et al (1998) have carried out model tests using a shaking table, to assess the earthquake performance of geotextile reinforced walls, with either segmental block facing or wrapped facing. They suggest that the critical acceleration for segmental block faced walls can be twice that for wrapped face walls. Although vertical acceleration was not used in the model tests, based on sliding block analyses, Ramakrishnan et al demonstrate that the vertical acceleration will reduce the critical horizontal acceleration, implying that this can lead to a larger displacement.

Recent research by Ling and Leshchinsky (1998) considered the effect of vertical ground motions on the seismic design of geosynthetic reinforced earth retaining structures. They carried out pseudo-static analyses using a limit equilibrium approach, considering both horizontal and vertical accelerations. The results of their parametric study indicated that upwards vertical acceleration required an increase in geosynthetic reinforcement length, and a downward vertical acceleration led to a reduced length of reinforcement being required. On the other hand increased tensile reinforcement is required with a vertical acceleration in a downward direction, although the effect of vertical acceleration is not very significant. However, intuition would suggest that the reinforcement strength and length requirements may be less for the given directions of vertical accelerations. We have contacted one of the authors, and from his response (Leshchinsky - pers comm), it appears that this confusion arises from the way it is described. Therefore, we conclude that their study probably indicated that a downwards earthquake acceleration which reduces reinforcement friction requires increased reinforcement length and an upwards acceleration which increases earth pressures requires an increased tensile capacity.

Illustration 6 shows the effect of vertical accelerations on the length of reinforcement required to resist direct sliding. It indicates that for horizontal accelerations higher than 0.2g, the effect of vertical accelerations can be very significant, even when vertical accelerations are only half the horizontal acceleration.



Note : l_{ds} – length of bottom reinforcement required to prevent direct sliding; H – height of slope or wall
k_v vertical seismic coefficient; k_h – horizontal seismic coefficient
Ø – angle of internal friction of soil; i – slope angle of face from horizontal

Illustration 6 - Effect of Earthquake Accelerations on Geosynthetic Length Required to Resist Direct Sliding, for Face Slopes of 60° and 90°
(after Ling and Leshchinsky, 1998)

Ling and Leshchinsky indicate the effect of vertical acceleration to be significant where the horizontal acceleration is greater than 0.2g.

Since permanent displacement is considered to be a more rational criterion for designing reinforced soil structures, Ling and Leshchinsky also considered the critical acceleration and permanent displacement, using a case study of a wall in the 1995 Hyogoken Nanbu earthquake in Kobe, Japan. This 6 m high geosynthetic-reinforced wall (Tanata wall) is located in the centre of Kobe City, and is described by Tatsuoka et al (1995) as mentioned in Section 3.2.3. The wall slid out relative to the adjacent piled culvert structure, by about 100 mm at the bottom of the wall. The maximum horizontal (normal to the wall face) and vertical acceleration recorded at an adjacent site was 0.42g and 0.38g respectively.

Neglecting vertical acceleration, the wall was assessed to have a critical acceleration of 0.47g, and no displacement would have been expected. Their assessment considering the vertical and horizontal accelerations, gave a critical acceleration of about 0.27g to 0.29g, and a displacement of 108 mm to 170 mm, depending on whether a rigid wall with friction at the base, or a wall with no facing is assumed. This compared favourably with the displacement of 100 mm observed at the base of the wall.

General Model

Very recently, Elms (2000) presented refinements to the Newmark sliding block model, based on analyses to assess the effect of including transverse (parallel to the wall face) and vertical earthquake motions, in addition to the longitudinal (normal to the wall face) earthquake motions that are usually considered. He indicated that the inclusion of transverse earthquake motions could give significantly larger displacements than using the longitudinal motions alone. The analyses were based on a sine wave and hence the results are only indicative. While he also suggested that similarly vertical accelerations can also increase the displacement, this was not presented. From discussion with Elms (pers-comm) it is understood that he has also considered the effect of vertical accelerations in a similar manner, although this has yet to be published. His work confirms that vertical accelerations can have a significant effect on the displacement of walls.

3.5 Summary of Past Research

The importance of displacements to the performance and design of walls has long been recognised. The significance of vertical ground motions is understood to have been recognised by Mononobe as far back as 1924. However, research in the 1970s and 1980s concentrated on the effect of displacement caused by horizontal earthquake motions alone and the effect of vertical ground motions received little attention. Limited analyses had led to the belief that the effect of vertical ground motions was insignificant. This was based on analyses for small horizontal and vertical accelerations less than 0.2g and the belief that vertical and horizontal ground motions are unrelated and the effects of upward and downward accelerations would offset the effects of each other.

Recent research in the 1990s, notably by Siddharthan et al (1991 and 1992) and Ling and Leshchinsky (1998), have demonstrated the importance of vertical accelerations, and that they could lead to significantly larger wall displacements than predicted from consideration of horizontal accelerations alone. Related issues that have arisen are the possible significant effects of transverse accelerations and the contribution of frequency of the earthquake motions. Analyses have led to a wide range of displacement predictions, depending on the other components of acceleration (transverse and vertical) and the frequency or type of earthquake shaking. The energy content of strong shaking (which includes accelerations as well as duration of each pulse) will also have a significant effect.

The wide variation in displacements assessed by Siddharthan et al (1991) for earthquakes scaled to the same peak ground acceleration, confirm that earthquake acceleration alone is a poor indicator to assess wall displacements. This is not surprising considering that peak ground acceleration may reflect a single peak value in the earthquake time history.

In recent research studies, displacements from vertical accelerations have been considered based on analytical models using the Mononobe-Okabe theory and the Newmark sliding block analyses. No model or numerical analyses appear to have been carried out to verify the effect of vertical ground motions.

4 Research Design

4.1 Basis for this Research

Recent research has given credence to the likelihood of vertical accelerations being significant to the displacement performance and hence design of retaining wall structures. Given the importance of this to the design of walls, the effect of vertical ground motions has been studied. This is particularly important in New Zealand, where the walls are used to support other structures such as bridge abutments and buildings in areas with hilly terrain.

4.2 Nature of Modelling

As the recent past research into vertical ground motions has been based on analytical methods, it was considered useful to base this research on an independent model. The different possible approaches considered were :

- *Centrifuge modelling* : no facilities are available in New Zealand and this is very expensive.
- *Shaking table tests* : the feasibility of this was considered using the shaking table facilities at Opus Central Laboratories or at Canterbury University, but they were not readily capable of imparting horizontal and vertical accelerations together and also would still be expensive and would take significant time.
- *Numerical modelling* : this was more readily available and could be used at modest cost. It also has the advantage that analyses can be carried out for a number of different earthquake records and combinations, and possibly for different types of walls in the future.

Different types of numerical models were considered and the Fast Lagrangian Analyses of Continua (FLAC) (Itasca Consulting Group,1993) was chosen, because :

- it is based on a finite difference algorithm and is capable of readily accommodating large displacements.
- it was readily available, and one of the authors was familiar with its use, and hence was cost effective.
- FLAC had recently been successfully used and tested by other researchers (Bathurst and Hatami, 1998) for assessing the earthquake behaviour of reinforced soil retaining walls.

The version of software used was FLAC 3.23 with the dynamic analysis module (Itasca Consulting Group, 1993).

4.3 Choice of Wall Type

A number of different wall types were considered for the research. After careful consideration, the research was based on a Reinforced Earth® retaining wall. The reasons which led to this choice include :

(a) The large reinforced earth block will displace mainly by sliding. This will avoid the added complication of a wall that can slide as well as tilt.

(b) Reinforced soil walls are the most common gravity type walls in New Zealand, for a reasonable height of more than 3 m.

(c) Reinforced Earth® wall was chosen in the first instance as these walls with inextensible (steel) reinforcements are less likely to undergo significant wall deformation, and hence are more suited to situations where structures are supported near the face, where the displacement of walls will be more important.

A wall with a height of 7.5 m was chosen as displacements would be significant and critical for this height, and this is also a common order of wall height for bridge abutments, where displacements would be important.

A wall with cohesionless (gravel) backfill was chosen as this is the backfill required for Reinforced Earth® walls. Also a dense gravel foundation was assumed which would not give rise to additional effects such as foundation deformation or failure, or significant amplification of earthquake shaking.

5 Development of the Model

5.1 The FLAC Program

FLAC (Fast Lagrangian Analyses of Continua) is a two-dimensional explicit finite difference code which simulates the behaviour of structures built of soil, rock or other materials that may undergo plastic flow when their yield limit is reached (Itasca Consulting Group, 1993). Materials are represented by elements that are configured as a grid by the user to the shape to be modelled. Each element behaves according to a prescribed stress/strain law (linear or non-linear) in response to the applied forces and boundary conditions. Yielding can be modelled in the grid in "large-strain" mode. A "Lagrangian" calculation scheme is used in FLAC, which is thus well suited for modelling large distortions. There are several built-in constitutive models that permit the simulation of the highly non-linear, irreversible responses typical of geological and similar materials.

5.2 Description of Model

The model used is shown on Illustration 7 and Figure 1. It consists of a foundation 12 m deep with a 7.5 m high Reinforced Earth® (RE) wall. The Reinforced Earth® wall is typical of those built in the past in New Zealand, with the dimensions chosen to be similar to a section of a wall at a State Highway 1 overbridge that is 7.5 m high with 7.5 m long reinforcement strips. Cruciform reinforced concrete facing panels are modelled (five 1.5 m by 1.5 m panels give a 7.5 m tall wall) with four reinforcement strips per panel. For static stability, the foundations of RE walls are generally buried to a depth of 1% of the height below the ground surface, and so this was also modelled.

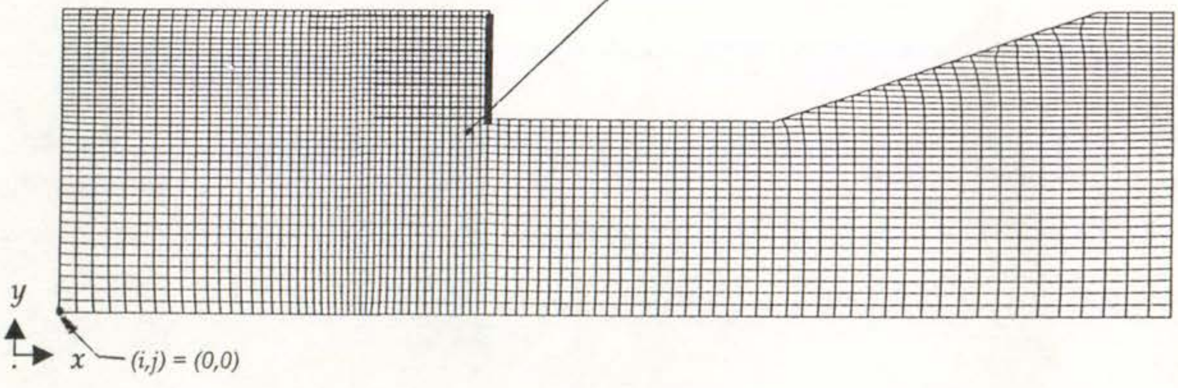


Illustration 7 - FLAC Model of Reinforced Earth Wall

With the version of FLAC used in the modelling, there is uncertainty in the manual as to whether far-field boundaries can be modelled at different heights, at the left and right hand sides of the model. There were also difficulties experienced during modelling that may have been due to differing boundary heights. Therefore the left and right hand side boundaries were made to be of equal height by placing a 3 horizontal to 1 vertical slope from near the wall base, see Illustration 7 and Figure 1.

The model was "built" in stages. The foundation was established and at-rest stress conditions imposed, followed by short equilibration stepping. Facing panels were then placed one at a time, with their attached cables and soil, and each stepped to equilibration. After equilibration of each panel, elastic soil strengths were adjusted for the increase in confining stress, before the next panel was installed. During construction, reinforcement strip friction, which increases with confining stress, was given a nominal average value. After completion, the reinforcement strip friction was reset as discussed in Section 5.5.2.

5.3 Elements

Soil elements used were rectangular, smaller in areas of interest or stress concentration and larger nearer the boundaries. The sizes of the elements were increased gradually toward the boundaries (e.g. each column was 1.05 times the width of the previous one). Sudden increases in element size cause smaller time steps to be used, so uniformly changing element dimensions provides for an optimal solution. The slope was formed by slightly distorting the grid.

The facing panels were modelled using beam elements. Each panel was made up of four beam elements. Each beam element can accept moments and forces at each end, including both compressive and tensile forces. Each beam (facing panel) was independent of the others, so they could move and rotate separately. The mass of each panel was modelled using a gravity load at the lower node of each of the four elements comprising the beam.

Reinforcement strips were modelled using FLAC's cable elements. Each strip was modelled as a cable consisting of 15 cable elements. The cables (strips) were attached to the beams (facing panels) at the nodes between beam elements 1 and 2, and between beam elements 3 and 4. Cables (strips) are designed as tension structures, do not accept moments and can be "grouted" into the soil. FLAC's cables are circular in section.

In prototype walls, there are regularly spaced ribs on the reinforcement strips to increase the strip-soil friction. The ribs effectively force the shearing surface on strip pullout to be almost entirely within the soil rather than at the soil-strip interface. This is advantageous because the soil internal friction is significantly greater than the soil-steel interface friction.

Cable elements in FLAC have a grout annulus around them so they can be grouted into the soil or rock. Pullout of a cable can occur either at the grout-soil interface or at the grout-cable interface. By suitable choice of properties, the user can model the appropriate type of pullout.

To model pullout through the soil, typical of prototype reinforcement pullout, the "grout" annulus around the cables was given the properties of the surrounding soil and a high bond strength was set for the grout–cable interface. In this way the grout–soil interface was in fact modelled as a soil–soil interface.

5.4 Boundary Conditions

In numerical simulation, it is important that boundary effects are minimised. In dynamic analysis, this means that the model boundary needs to simulate free-field conditions. There should be no reflection of outward-radiating waves back into the model by the boundary.

One way to simulate the necessary energy radiation is to use a large model. Material damping will absorb most of the energy in the waves reflected from distant boundaries. However, large computational time and costs arise with larger models. The alternative is to use quiet (viscous) boundaries, for which several formulations have been proposed.

In FLAC, the viscous boundary developed by Lysmer and Kuhlemeyer (1969) is used. It is based on the use of independent dashpots, and is almost totally effective for body waves at an angle of incidence greater than 30°. It is less effective at lower angles of incidence, but has the advantage of being suitable for use in time domain analyses.

Dashpots are attached to the boundary in the normal and shear directions and provide viscous normal and shear tractions given by :

$$t_n = -\rho C_p v_n$$

$$t_s = -\rho C_s v_s$$

where v_n and v_s are the normal and shear components of the velocity at the boundary,
ρ is the mass density, and
C_p and C_s are the P- and S-wave velocities.

In FLAC, the tractions t_n and t_s are calculated and applied at every time step in the same way as the boundary loads. It is desirable to have free-field conditions at the left and right boundaries of the model to simulate a semi-infinite half space, that is to minimise the effect of the boundaries. To simulate free-field boundaries at the sides of the model, FLAC has an *APPLY FREE-FIELD* command. It involves the execution of a one-dimensional free-field calculation in parallel with the soil system analysis. Free-field boundaries have the characteristics of quiet boundaries (ie to absorb the energy and not reflect incident waves), and have a one dimensional column of unit width simulating the behaviour of the extended medium. The lateral boundaries are coupled to the free-field grid by viscous dashpots (effectively a quiet boundary). Static equilibrium conditions prior to the dynamic analysis are transferred to the free field when the *APPLY FREE-FIELD* command is invoked.

Quiet boundaries in the x- and y-directions (i.e. horizontal and vertical directions respectively) were applied at the base of the model. Thus the model had free-field boundaries on the sides and a quiet boundary on the base.

Acceleration and velocity inputs are incompatible with quiet boundaries. Stress inputs can be used instead. A velocity record is transformed into a stress record using:

$$\sigma_n = 2(\rho C_p)v_n$$
$$\sigma_s = 2(\rho C_s)v_s$$

where σ_n = applied normal stress
σ_s = applied shear stress
ρ = mass density
C_p = velocity of p-wave propagating through the medium
C_s = velocity of s-wave propagating through the medium
v_n = input normal velocity, and
v_s = input shear velocity.

The factor of two in the equations results because the applied stress must be doubled to overcome the effects of the quiet boundary. This process was used in the simulations reported here, with the stresses applied along the bottom boundary.

During simulation, the energy radiated in-plane is reasonably absorbed by the quiet boundaries. However, energy radiated out-of-plane must also be damped. In FLAC, this is accomplished using the so-called 3-D damping, which absorbs the difference between the actual particle velocity under the structure and the free-field velocity around the model region. This type of damping was used in the simulations reported.

5.5 Material Properties

5.5.1 *Soil*

The Mohr-Coulomb constitutive model available in FLAC was used for the soil, which was cohesionless with a friction angle of 36°, dilation angle of 7.5° and density of 2.0 tonnes per cubic metre. These values were chosen as being representative of many sandy gravels that might be in foundations and fills behind Reinforced Earth® walls.

The soil's Young's modulus, *E*, was modelled as a function of confining stress, σ, using :

$$E = E_0(\sigma/\sigma_a)^\alpha$$

where σ_a is the atmospheric pressure and α is a property of the material.

For sandy gravel with few fines, α is about 0.70 so this value was used. An initial Young's modulus E_0 of 600 MPa was used and some comparative runs were done using Young's Modulus E_0 of 400 MPa.

Using a Poisson's ratio of 0.3, the bulk modulus and shear modulus, which are required by FLAC, were then calculated. The resulting shear and compression wave velocities in the model are as shown in Figure 2 to Figure 5.

It is difficult to reproduce hysteretic damping (i.e. independent of frequency) numerically because of the problem with path dependence, which makes results difficult to interpret (FLAC manual). In time-domain programs, Rayleigh damping is commonly used to circumvent the problem. Rayleigh damping consists of two viscous components, velocity- (or mass-) proportional and stiffness-proportional. The velocity-proportional damping acts on the lower frequency modes of the system, while the stiffness-proportional component damps higher frequencies. Lower frequencies are usually associated with the movement in unison of several zones or gridpoints, while higher frequencies are normally inter-zone vibrations.

It was found that some damping was necessary to prevent high frequency resonance, particularly in the y-direction when vertical motions were used. Thus stiffness-proportional Rayleigh damping was used. Because there is significant damping already in the soil (which is a frictional material), only a very small percentage of critical damping is required. In the model used in this research, 0.1% of critical damping was used.

5.5.2 *Reinforcement Strips*

Prototype reinforcement strips are steel and 60 mm by 5 mm, with 5.5 mm thick ribs. The strips have a steel cross-sectional area of 300 mm^2, and a surface area on which the friction is transferred to the soil of 152 mm^2, taking into account the ribs. This assumes that friction is transferred to the soil on a surface, level with the outer edge of the ribs. The FLAC model uses cable elements (that are defined in FLAC as circular) and were modelled with the same surface area of 152 mm^2 / mm length along which the friction is transferred in the case of the prototype strips. The friction was assumed to be transferred along the outer surface of the 'grout' annulus (48.4 mm diameter), with the grout annulus being given a nominal thickness of 5 mm. This gave a cable diameter of 38.4 mm and steel cross sectional area of 1157 mm^2. Given that this steel cross sectional area is larger than the prototype strip steel cross sectional area of 300 mm^2, the Young's modulus of the cable was scaled by a factor of 300/1157. This would ensure that the elongation of the cable was the same as that of the prototype strip, for the same load.

The shear behaviour of the grout annulus is represented as a spring-and-slider system located at each cable node (i.e. between individual cable elements). K_{bond} is the grout shear stiffness (the spring) and defines the shear behaviour of the grout. S_{bond} is the cohesive strength of the grout with the soil (slider strength) and defines the maximum shear force per unit cable length that can develop between the grout and the soil.

The grout shear stiffness (K_{bond}) per unit thickness is given by :

$$K_{bond} = \frac{2\,\pi\,G_g}{\ln\,(1\,+\,2t\,/\,D)}$$

where G_g = grout shear strength, in this case the same as the soil shear strength,
t = grout thickness,
D = reinforcement diameter.

During construction of the wall (in static conditions) a nominal average value was used for K_{bond} for all strips. At the completion of construction, K_{bond} was reset for each reinforcement strip using the average soil shear modulus around that strip. Thus there was one value of K_{bond} per cable (reinforcement strip).

To calculate the cohesive strength of the grout with the soil (S_{bond}), the normal Reinforced Earth® strip friction equations were used:

$$f^* = f_0^* - (f_0^* - \tan\phi)\,(\frac{y}{6}) \quad for \quad 0 \le y \le 6$$

$$f_0^* = 1.2 + \log CU$$

$$f^* = \tan\phi \quad for \quad y > 6$$

$$\tau_{\max} = \sigma_{yy}\, f^*$$

where CU = the coefficient of uniformity (3 was used)
τ_{max} = maximum shear stress at the interface on pullout.

The shear stress τ_{max} was then converted to the maximum bond (shear) force per unit cable length, S_{bond}, using the area of the grout-soil interface.

Since τ_{max} is proportional to the confining stress and the confining stress was expected to vary with the earthquake input, S_{bond} was updated during dynamic stepping using the current average value of the vertical stress for each cable. Thus there was a value of S_{bond} for each cable.

Normally FLAC cables are unable to accept compression forces, although there is a parameter that sets the maximum compression force that the cable will accept. Initially this was set to zero, but during stepping it was found the top strips were going into compression occasionally. The result was that the strips became shorter. To prevent shortening, the maximum compression force was set equal to the maximum tension (i.e. yield) force.

It was also found during stepping that the top two cables were pulling out and the facing panel was falling outwards. Attempts were made to prevent this by lengthening those two cables to 1.5 times the length of the rest of the cables. This was unsuccessful. In prototype walls, many designers specify that the upper two layers of strips are draped down to increase the confining stress and hence the friction. This was not attempted in the modelling, as it was felt that the pullout at the wall crest had little effect on the overall outward displacement as measured at the base and mid-height.

5.5.3 *Facing Panels*

The facing panels were modelled as reinforced concrete cruciform panels with a Young's modulus of 23.94 MPa, moment of inertia of 8.79 x 10^{-4} m^4 and area 0.075 m^2. Vertical gravity loads of 0.17 kN were applied at the nodes at the base of each beam element to simulate the weight of the facing panel.

On the rear surface of all facing panels (beams), an interface was used to allow the soil to slide on the panel. One was also used on the front of the bottom panel where it is buried. A wall friction angle of 20° was used. The normal and shear stiffnesses of the interface were set to the values recommended in the FLAC manual, 10 times the equivalent stiffness of the stiffest neighbouring zone. The apparent stiffness (in stress-per-distance units) of a zone in the normal direction is

$$\max\left[\frac{(K+\frac{4}{3}G)}{\Delta z_{\min}}\right]$$

where K and G are the bulk and shear moduli respectively and Δz_{min} is the smallest width of an adjoining zone in the normal direction.

The maximum value is taken over all the zones adjacent to the interface. All interfaces had normal and shear stiffness values set to 2295 MPa, based on the initial trial (laboratory scale and thus low) K and G values used. Little movement was observed at the interfaces.

5.6 Calibration / Verification

The Newmark sliding block analogy is used to predict permanent displacements of retaining walls and soil blocks in earthquakes. The displacements depend on two factors, the critical acceleration and the earthquake. The critical acceleration is a property of the wall, so for model and prototype walls, the permanent seismic displacements should be the same if the critical acceleration is the same.

Fairless (1989) measured displacements in model testing of reinforced earth walls on a shaking table. The critical (yield) accelerations above which permanent displacements were initiated were of the order of those expected in prototype walls, 0.16g to 0.3g. The displacements measured were of the order of a few millimetres prior to formation of a failure surface in the model. Observed yield accelerations also varied for each earthquake.

The model used in this research was not calibrated directly by comparison with displacements measured in earthquakes or in specific scale models, principally because displacement data for RE walls in earthquakes is very sparse. It was considered appropriate to proceed when the displacements were of the order of those expected under horizontal acceleration - a few millimetres - because the primary interest of this research was the relative effect of vertical ground motions and the percentage increase of displacements when vertical ground motions were introduced. The actual displacements will in any case vary depending on the particular wall characteristics and soil properties.

6 Earthquake Records

6.1 Choice of Earthquake Records

Earthquake records with significant vertical ground motions were required, as the primary objective was to assess the effect of vertical shaking. Since vertical ground motions are generally high in near-field areas, earthquake records generally within 10 km of the epicentre were considered desirable for the study. However, due to the lack of suitable records that were readily available, earthquake records up to 20 km from the epicentre were accepted.

A search was carried out on the Internet for earthquake records. The criteria used to obtain appropriate earthquake records were :

- vertical accelerations were significant in comparison with the horizontal accelerations
- near-field records, from within about 20 km of the epicentre
- magnitude was greater than about 6.5
- earthquake record is associated with rupture of a normal fault.

Four suitable earthquake records were chosen. The location and properties of the records and the characteristics of the associated earthquakes are summarised in Table 1. The larger of the two horizontal components and the vertical component were used in the FLAC model simulations. Power spectral densities of the records used are presented in Figure 6 to Figure 9.

Table 1 - Earthquake records chosen for the simulations

Earthquake Record	Magnitude	Hypocentral Distance	Peak Acceleration	
			Component	PGA (g)
Landers, Jun 28, 1992 11:57:34 UTC at Joshua Tree Fire Station	$M_S = 7.5$ $M_W = 7.4$	9 km	90 Deg 0 Deg Up	0.28 0.27 0.18
Big Bear, Jun 28, 1992 15:05 GMT at Big Bear Lake - Civic Center Grounds	$M_L = 6.6$	7 km	270 Deg 360 Deg Up	0.48 0.54 0.19
Northridge, Jan 17 1994, 12:30:55.4 GMT at Castaic Old Ridge Route	$M_L = 6.6$ $M_W = 6.7$	18 km	360 Deg 90 Deg Up	0.51 0.56 0.22
Loma Prieta, Oct 17, 1989 00:04:02 GMT at Corralitos Eureka Canyon Rd, Santa Cruz Mtns	$M_L = 7.0$ $M_S = 7.1$	18 km	90 Deg 0 Deg Up	0.48 0.63 0.44

6.2 Baseline Correction

When recorded earthquake accelerations are double-integrated, they often do not finish with the acceleration, velocity and displacements all equal to zero. It is necessary to correct for this before using the records in numerical simulation. The process involves the addition of a half-wavelength (i.e. low frequency) sine wave to the record. This was done for all records used.

To check the baseline correction, each record was run on a model consisting of a simple box of square elements and modified as necessary to minimise the displacement and velocity at the end of shaking. With the box of elements, there is no perturbation due to geometric or other effects in the model, so the entire model displaces essentially as one block. When the baseline correction was optimal, the displacement of the entire box was small and the velocity was zero or very small at the end of shaking.

Even though all records were baseline corrected, there was always residual displacement of the entire model at the end of shaking. Usually this was significantly larger than the displacement measured in the simulated walls. This difficulty was dealt with in this study by deriving net displacements as discussed in Section 7.4.

6.3 Scaling

The earthquakes were converted to stresses and input as outlined in Section 5.4. The objective was to achieve preferred earthquake motions at the base of the wall, against which the wall displacements could be compared. Scaling was used to :

- Overcome the attenuation of ground motions due to the model's elements that were located between the base of the model and the base of the wall.
- Overcome the induced motions of elements in orthogonal directions (eg vertical motions induced by horizontal motions).
- Enable modification of the magnitude of the motion to assess the effect of changes in magnitude on the wall displacements. In particular vertical accelerations were modified by factors to assess their effect.

The horizontal and vertical earthquake shaking inputs were scaled by multiplying the input stress history by the relevant scaling factor.

Scale factors were used to produce peak accelerations at the wall foundation level that were close to those in the field earthquake records. Scale factors were also used to modify the motions for other model runs to assess the effect of earthquake shaking with the same frequency characteristics, but different amplitudes of vertical and horizontal shaking. The scaling factors for each earthquake component were determined in trials with both horizontal and vertical motions input together. Model accelerations at the wall foundation that approximately match accelerations from the earthquake records, were achieved by adjusting the scaling factors. An exception to this was the horizontal accelerations from the 1992 Landers Earthquake, which was relatively small at 0.28g, see Table 1. In order to get accelerations of reasonable size, and be comparable to other earthquakes, scaling was used in this instance to obtain initial peak horizontal accelerations of about 0.55g, see Table 2.

6.4 Earthquake Motion Combinations for Analyses

Table 1 shows the earthquake records chosen for use in the FLAC model simulations. Two groups of earthquake motions were used in the model analyses. Group I had an energy input and accelerations similar to that from the earthquake records for horizontal and vertical shaking. The following runs were used with Group I earthquake shaking :

- Horizontal record only (with the amplitude roughly matching the earthquake record, except for Landers Earthquake)
- The same horizontal input plus vertical input scaled to roughly match earthquake record
- The same horizontal record plus ½ to 2 times the vertical input.

The vertical records were scaled by a factor of up to 2, to enable an assessment of the effect of different levels of vertical shaking on the displacement of the walls, given the same earthquake characteristics, such as frequencies, duration and number of large pulses. The accelerations considered as input to the wall are those measured during each run at node (42,20), in the wall foundation (location shown on Illustration 7).

Group II earthquake inputs were obtained from the same records as used for Group I, but were scaled to have a larger energy input, and associated larger horizontal and vertical accelerations. The larger scaled horizontal as well as vertical accelerations are consistent with values that have been reported in the literature in different earthquake events, although suitable earthquake records could not be readily sourced for this study.

Since the earthquake is applied at the base of the model and measured at the wall base for use in assessing the results, the required scaled vertical and horizontal accelerations were difficult to achieve. The effects of changing the scale factors varied between the earthquake records. Scaling was used to obtain accelerations close to those required.

The actual horizontal and vertical accelerations measured during the analysis runs at the foundation of the wall are given in Table 2, in Section 8.

7 Model Analyses

7.1 Measurement of Changes

FLAC provides the ability to record a history of the values of various parameters at selected intervals and locations during simulation stepping. This capability was used to record histories of the following at selected locations :

- accelerations
- reinforcement strip (i.e. cable) axial forces and friction (the FLAC property S_{bond})
- displacements and velocities
- lengths of cable elements
- confining stress within a soil zone
- net displacements (i.e. node displacement minus overall model displacement).

Some of the above histories required programming using the built-in FLAC-ish programming language, FISH. These are discussed in the following sections. The same histories were recorded for each run.

7.2 Acceleration Histories

Acceleration histories in the x- and y- directions (horizontal and vertical respectively) were recorded at several locations. They provide a record of the input and of the response, as well as the uniformity of the acceleration input at wall foundation level. Input accelerations were recorded at nodes two rows beneath the wall foundation and response accelerations were recorded at three locations behind the wall facing, as shown in Figure 10.

The earthquake record acceleration histories for the four chosen earthquakes are given in Figure 11 to Figure 14. A set of acceleration histories recorded at five locations beneath the foundation of the wall for one of the earthquakes are given in Figure 15 to Figure 19. It is clear that the records are very similar at this level of investigation.

7.3 Reinforcement Strip Forces and Friction

Reinforcement strip force histories were recorded at three locations on the lower four strips, two on the fifth strip and at the facing on the remainder. Typical sets of plots are shown in Figure 20 to Figure 25. To take these histories, FISH programming was necessary to access the FLAC data lists.

Because strip friction (S_{bond}) was updated during stepping, histories of S_{bond} were recorded for each strip during each run. Typical sets of plots are shown in Figure 26 to Figure 28. These histories required FISH programming to access the FLAC data lists.

7.4 Total and Net Displacement

Histories of total displacements, in the x- and y-directions, were recorded at the locations shown in Figure 29. A typical example is shown in Figure 30.

At the end of each run, a plot was produced showing the total displacements, but this was of little value because the displacements of interest were lost in the usually much larger model displacements. A typical example is shown in Figure 31. Of more use is a plot of the net displacements, where the model displacement is subtracted from the total displacements. Several ideas were tried to arrive at the overall model displacement to subtract. These included the average for a row of nodes, the average for a number of nodes in the wall foundation (e.g. at row j=20), and the displacement for just one node, preferably one close to the wall foundation. The chosen method was to subtract the displacement at node (44,20) below the wall base, as it was the most efficient way to achieve a representative net displacement of the wall with respect to the ground below the reinforced soil block. A set of plots showing the typical net displacement vectors are presented in Figure 32 to Figure 35.

While this approach provided a rational method to assess the displacement of the wall face in relation to the wall foundation, under the limitations of the model, the derived displacements are not expected to be accurate given the complex displacement of the model. This is highlighted by the very small negative net displacements indicated when horizontal shaking alone was applied using the Loma Prieta base records. However, the orders of magnitude of displacement and the important trends indicated from the analyses when different input records are applied are of primary importance in this study.

Of most value to the objectives of this research were histories of net displacement at the locations shown on Figure 36. These histories were generated by subtracting the displacement at node (44,20) from the displacements at the locations shown. Typical examples are shown in Figure 37.

7.5 Cable Element Lengths

As mentioned in Section 5.5.2, shortening of the upper strips was observed in the model simulations. To help resolve the problem, the histories of the lengths of the affected cable elements were recorded and compared to the histories of the axial forces in those elements. It was clear that the shortening occurred when the cable was in compression, because the maximum compression force had been set to zero.

7.6 Confining Stress

The strip friction parameter S_{bond} was updated during stepping depending on the confining stress. To verify this, histories were taken of the confining stress at two locations, in zones (44,22) and (44,26), which are located two columns of zones behind the facing. Row 22 is beneath the lowest reinforcing strip and row 26 is above the fourth strip.

7.7 Difficulties in the Model Analyses

Several difficulties were encountered during the running of the models using the input records. These have required the model to be adjusted to obtain results that can be used with confidence. A brief discussion of these issues is presented in Appendix A.

7.8 Power Spectral Analyses

The power spectral density (PSD) of an earthquake record is a measure against frequency of the energy in an earthquake. It is calculated from the square of the Fourier amplitude spectrum. The definition used here for the power spectrum is one of several possible, and is adapted from Press et al (1992) by Itasca (1993).

$$P_0 = \frac{1}{N^2} * (| f_0 |)^2$$

$$P_k = \frac{1}{N^2} * \left[(| f_k |)^2 + (| f_{N-k} |)^2 \right.$$

$$P_{N/2} = \frac{1}{N^2} * (| f_k |)^2$$

where N is half the number of points in the original data field,

P is the power spectrum output,
f is the result of the Fast Fourier Transform of the original data, and
k varies from 0 to N/2.

By then summing the power in all frequency bins, a measure of the total energy content of the earthquake shaking (Σ PSD) is obtained, separately for the horizontal (ΣH PSD) and vertical (ΣV PSD) directions. The sum of the power spectral density has units of (velocity)^2/time, or power per unit mass, and is presented in $m^2 s^{-3}$ in the results in Table 2.

Earthquake parameter(s) are chosen to characterise the shaking when analysing the displacement. Peak ground acceleration is easy to derive and is commonly used but is not particularly good at characterising earthquakes.

We might expect that earthquakes with more energy at the site would result in more displacement of the structure being shaken. The total energy can also be viewed as one parameter that combines or contains two other parameters associated with an earthquake, that is, the duration of shaking and the intensity of shaking. The PSD, summed through all frequencies, provides a measure of that energy.

To enable comparison with the displacements, the PSD was calculated for each run from the acceleration history measured at node (42,20) in the foundation of the wall. In this way, we obtain a good estimate of the energy of the shaking actually experienced by the wall. The Fast Fourier Transform routine used was supplied with FLAC and was adapted from Press et al (1992).

8 Results

8.1 FLAC Analysis Results

The results of the FLAC analyses including the specific parameters recorded during the analyses were plotted to check the behaviour of the models. Specific examples of the FLAC output are presented in the figures referred to in Sections 5 to 7 of this report. The full outputs are not presented. The results relevant to the outcomes of this study have been collated from the output and are summarised in Table 2.

8.2 Wall Base Accelerations

The peak horizontal (H pga) and vertical (V pga) ground accelerations recorded at the base of the wall at node (42,20) are summarised in Table 2. Note that these are different from the accelerations associated with the input earthquake records at the base of the model.

Table 2 also presents the Acceleration Ratio (AR), which is calculated as :

$$AR = \frac{Vertical\ Peak\ Ground\ Acceleration\ \ [V\ pga]}{Horizontal\ Peak\ Ground\ Acceleration\ \ [H\ pga]}$$

Group I has peak horizontal and vertical accelerations recorded at the base of the wall (42,20) with the input accelerations at the base of the model scaled to achieve accelerations as close as possible to the earthquake records, and with the vertical accelerations scaled by factors of 0.5 to 2. Group II has larger horizontal and vertical accelerations than those from the chosen records from the four past earthquakes.

8.3 Power Spectral Density of Wall Base Motion

The sum of the Power Spectral Density (Σ PSD) derived from the acceleration time histories in the horizontal (ΣH PSD) and vertical (ΣV PSD) directions, recorded at the base of the wall (42,20) are also summarised in Table 2.

Table 2 also presents the Sum of the Power Spectral Density Ratio (PSDR), calculated as :

$$PSDR = \frac{Sum\ of\ Vertical\ Power\ Spectral\ Density\ \ [\Sigma\ V\ PSD]}{Sum\ of\ Horizontal\ Power\ Spectral\ Density\ \ [\Sigma\ H\ PSD]}$$

Note that the sum of the horizontal Power Spectral Density for the Group II runs are much larger and have a larger energy content than those for Group I.

The power spectral density has been derived as it may provide a better parameter to relate to wall displacements, than the peak ground acceleration, which represents a single peak value in the time history.

8.4 Displacements

The net permanent displacement of the wall at the end of each of the runs, is summarised in Table 2, along with the corresponding horizontal and vertical peak ground accelerations and sums of power spectral density. The net displacements shown are end-of-earthquake displacements calculated by subtracting the displacement at node (44,20) below the base of the wall, from those for the locations shown.

The displacements are presented at the facing, at the

- top of the wall
- mid height
- bottom of the wall.

Table 2 also presents the Displacement Ratio (DR), calculated as :

$$DR = \frac{\textit{Displacement of the Wall with Vertical and Horizontal Accelerations}}{\textit{Displacement of the Wall with Horizontal Acceleration Only}}$$

The displacement ratio is presented for the displacements at mid height of the wall and the bottom of the wall.

Table 2 Model Analyses Results

Earthquake	Node Location in Model	Parameters			Units	Group I – Lower Energy Input					Group II – Larger Energy Input					
						Horizontal	Horizontal and Vertical Input				Horizontal	Horizontal and Vertical Input				
						H	H + ½V	H + V	H + 1½V	H + 2V	H					
1992 Landers Earthquake At Joshua Tree Fire Station	(42,20)	Peak Ground Acceleration	Horizontal	H pga	ms^{-1}	0.55 g	-	0.50 g	0.57 g	0.55 g	0.61 g	-	0.73 g	0.83 g	0.97 g	H pga
	(42,20)		Vertical	V pga	ms^{-1}	0.04 g	-	0.21 g	0.32 g	0.43 g	0.04 g	-	0.80 g	1.01 g	1.03 g	V pga
	(42,20)	Acceleration Ratio		V pga / H pga	-	0.07	-	0.42	0.56	0.78	0.07	-	1.10	1.22	1.06	V pga / H pga
	(46,41)	Wall Face Displacements		Top of Wall	mm	16	-	17	18	21	181	-	158	246	282	Top of Wall
	(46,31)			Mid Height	mm	9	-	9	9	9	8	-	45	70	172	Mid Height
	(46,23)			Bottom of Wall	mm	4	-	4	4	4	0	-	28	40	53	Bottom of Wall
	(46,31)	Displacement Ratio	Mid Height	DR - Middle	-	1.00	-	1.01	1.03	1.01	1.00	-	5.7	9.0	22.1	DR – Middle
	(46,23)		Bottom	DR - Bottom	-	1.00	-	1.04	1.10	1.04	1.00	-	70.8	100	132.5	DR - Bottom
	(42,20)	Sum of Power Spectral Density	Horizontal	ΣH PSD	m^2s^{-3}	0.088	-	0.086	0.085	0.084	0.124	-	0.122	0.118	0.118	ΣH PSD
	(42,20)		Vertical	ΣV PSD	m^2s^{-3}	0.000	-	0.027	0.061	0.107	0.000	-	0.041	0.093	0.168	ΣV PSD
	(42,20)	Power Spectral Density Ratio		ΣV PSD/ΣH PSD	-	0.00	-	0.32	0.72	1.27	0.00	-	0.34	0.79	1.42	ΣV PSD/ΣH PSD
1992 Big Bear Earthquake At Big Bear Lake Civic Centre	(42,20)	Peak Ground Acceleration	Horizontal	H pga	ms^{-1}	0.58 g	-	0.59 g	0.62 g	0.72 g	0.62 g	-	0.65 g	0.67 g	0.77 g	H pga
	(42,20)		Vertical	V pga	ms^{-1}	0.06 g	-	0.22 g	0.36 g	0.45 g	0.05 g	-	0.56 g	0.84 g	1.04 g	V pga
	(42,20)	Acceleration Ratio		V pga / H pga	-	0.10	-	0.37	0.58	0.63	0.08	-	0.86	1.25	1.35	V pga / H pga
	(46,41)	Wall Face Displacements		Top of Wall	mm	45	-	55	59	69	51	-	105	111	123	Top of Wall
	(46,31)			Mid Height	mm	14	-	11	13	15	15	-	23	30	42	Mid Height
	(46,23)			Bottom of Wall	mm	8	-	6	7	8	9	-	11	13	14	Bottom of Wall
	(46,31)	Displacement Ratio	Mid Height	DR - Middle	-	1.00	-	0.81	0.90	1.04	1.00	-	1.48	1.98	2.77	DR - Middle
	(46,23)		Bottom	DR - Bottom	-	1.00	-	0.76	0.84	0.97	1.00	-	1.24	1.45	1.62	DR - Bottom
	(42,20)	Sum of Power Spectral Density	Horizontal	ΣH PSD	m^2s^{-3}	0.016	-	0.016	0.016	0.015	0.021	-	0.019	0.019	0.019	Σ H PSD
	(42,20)		Vertical	ΣV PSD	m^2s^{-3}	0.000	-	0.006	0.013	0.024	0.000	-	0.009	0.020	0.035	Σ V PSD
	(42,20)	Power Spectral Density Ratio		ΣV PSD/ΣH PSD	-	0.00	-	0.39	0.87	1.54	0.00	-	0.47	1.04	1.80	ΣV PSD/ΣH PSD
1994 Northridge Earthquake At Castaic Old Ridge Rd	(42,20)	Peak Ground Acceleration	Horizontal	H pga	ms^{-1}	0.62 g	-	0.61 g	0.65 g	0.69 g	0.6 g	-	0.70 g	0.92 g	0.98 g	H pga
	(42,20)		Vertical	V pga	ms^{-1}	0.09 g	-	0.29 g	0.39 g	0.51 g	0.15 g	-	0.82 g	1.09 g	1.04 g	V pga
	(42,20)	Acceleration Ratio		V pga / H pga	-	0.15	-	0.48	0.60	0.74	0.25	-	1.17	1.18	1.06	V pga / H pga
	(46,41)	Wall Face Displacements		Top of Wall	mm	52	-	41	56	73	79	-	208	210	270	Top of Wall
	(46,31)			Mid Height	mm	23	-	14	23	25	22	-	44	69	111	Mid Height
	(46,23)			Bottom of Wall	mm	14	-	5	12	13	12	-	25	26	43	Bottom of Wall
	(46,31)	Displacement Ratio	Mid Height	DR - Middle	-	1.00	-	0.61	0.99	1.09	1.00	-	2.01	3.12	5.02	DR - Middle
	(46,23)		Bottom	DR - Bottom	-	1.00	-	0.34	0.84	0.91	1.00	-	2.02	2.11	3.50	DR - Bottom
	(42,20)	Sum of Power Spectral Density	Horizontal	ΣH PSD	m^2s^{-3}	0.031	-	0.029	0.030	0.030	0.039	-	0.039	0.040	0.040	ΣH PSD
	(42,20)		Vertical	ΣV PSD	m^2s^{-3}	0.000	-	0.011	0.024	0.042	0.000	-	0.016	0.037	0.070	ΣV PSD
	(42,20)	Power Spectral Density Ratio		ΣV PSD/ΣH PSD	-	0.00	-	0.37	0.81	1.40	0.00	-	0.42	0.91	1.74	ΣV PSD/ΣH PSD
1989 Loma Prieta Earthquake At Corralitos	(42,20)	Peak Ground Acceleration	Horizontal	H pga	ms^{-1}	0.59 g	0.61 g	0.83 g	0.69 g	-	0.62 g	0.75 g	0.71 g	0.87 g	-	H pga
	(42,20)		Vertical	V pga	ms^{-1}	0.07 g	0.25 g	0.64 g	0.82 g	-	0.07 g	0.88 g	0.49 g	1.21 g	-	V pga
	(42,20)	Acceleration Ratio		V pga / H pga	-	0.12	0.41	0.77	1.19	-	0.11	1.17	0.69	1.39	-	V pga / H pga
	(46,41)	Wall Face Displacements		Top of Wall	mm	80	93	132	155	-	102	145	143	173	-	Top of Wall
	(46,31)			Mid Height	mm	2	9	14	18	-	1	6	10	35	-	Mid Height
	(46,23)			Bottom of Wall	mm	-2	5	8	11	-	-3	2	7	22	-	Bottom of Wall
	(46,31)	Displacement Ratio	Mid Height	DR - Middle	-	1.00	3.56	5.67	7.63	-	1.00	5.91	9.36	31.7	-	DR - Middle
	(46,23)		Bottom	DR - Bottom	-	1.00	-2.75	-4.94	-6.53	-	1.00	-0.53	-2.3	-7.4	-	DR - Bottom
	(42,20)	Sum of Power Spectral Density	Horizontal	ΣH PSD	m^2s^{-3}	0.065	0.067	0.069	0.071	-	0.082	0.087	0.107	0.090	-	ΣH PSD
	(42,20)		Vertical	ΣV PSD	m^2s^{-3}	0.000	0.006	0.025	0.056	-	0.00	0.038	0.012	0.096	-	ΣV PSD
	(42,20)	Power Spectral Density Ratio		ΣV PSD/ΣH PSD	-	0.00	0.10	0.36	0.79	-	0.00	0.44	0.11	1.06	-	ΣV PSD/ΣH PSD

9 Appraisal of Results

9.1 Top of Wall Displacement

The displacement at the top of the wall from the model analyses is much larger than that along the lower part of the wall. The wall top displacement is generally larger than at the bottom of retaining walls, due to tilting of gravity walls (Siddharthan et al, 1992), and deformation of the wall in the case of mechanically stabilised earth walls, such as the Reinforced Earth® wall used in the model analysis in this instance.

However, in this study, pull-out of the uppermost strips was observed, which is likely to have led to the significantly larger displacements observed. This is considered to be due to the lack of confining pressure at the top of the wall to generate sufficient frictional resistance. This is a recognised problem and is overcome in practice by bending the uppermost strips and taking them to a lower level, so that the confining pressure is greater. However, this complication was avoided in the FLAC model used in this study, as this would further complicate the variation of confining pressure with change in vertical acceleration. A variation to the modelling of the upper strips could be considered in any further research on wall displacements.

Nevertheless, the results of the model confirm the vulnerability of the uppermost strips to pullout, and hence the importance of taking the uppermost strips to a lower level in practical designs. The pullout of the upper strips would be exacerbated when vertical accelerations are present, as this further reduces the confining pressure when the vertical acceleration acts downwards. Given that the displacements observed at the top of the wall may not materialise in a practical design, the displacements at the top of the wall have not been considered in this appraisal, except when considering horizontal shaking alone.

9.2 Magnitude of Displacements

The magnitude of the displacements observed in the results of the analyses (except those at the top of the wall discussed in Section 9.1), are generally :

- less than 25 mm in the case of Group I analyses, and
- less than 200 mm in the case of the Group II analyses where the applied ground shaking was quite large.

These relatively limited displacements are not surprising, given that this is for a very robust wall, where the wall aspect ratio (ratio of reinforcement length to wall height) is one. The wall modelled is based on an actual wall, designed in an area of high seismicity in New Zealand. The wall was designed to allow no gross permanent displacement under the design load case, that is a peak ground acceleration (horizontal) of 0.42g, as determined from the loadings code NZS 4203:1992 (Standards Association of New Zealand, 1992).

The model analyses shows that, where no displacements are allowed for using current design standards, the actual order of displacements at the lower half of the wall could be :

- Up to 25 mm, in earthquakes giving Group I horizontal and vertical shaking,
- Up to 175 mm, in earthquakes giving Group II horizontal and vertical shaking.

It can be seen from Table 2 that the maximum displacement assessed for horizontal shaking alone, is only 23 mm along the lower half of the wall, even with the larger Group II earthquake shaking considered. This highlights the importance of considering vertical ground shaking from earthquakes.

9.3 Effect of Horizontal Ground Shaking Alone

It is useful to initially consider the effect of horizontal ground shaking alone on the observed displacement of the wall in the model analyses. Illustration 8 shows the variation of the displacement at the top of the wall with horizontal peak ground acceleration, when horizontal shaking alone is applied (that is, no vertical shaking).

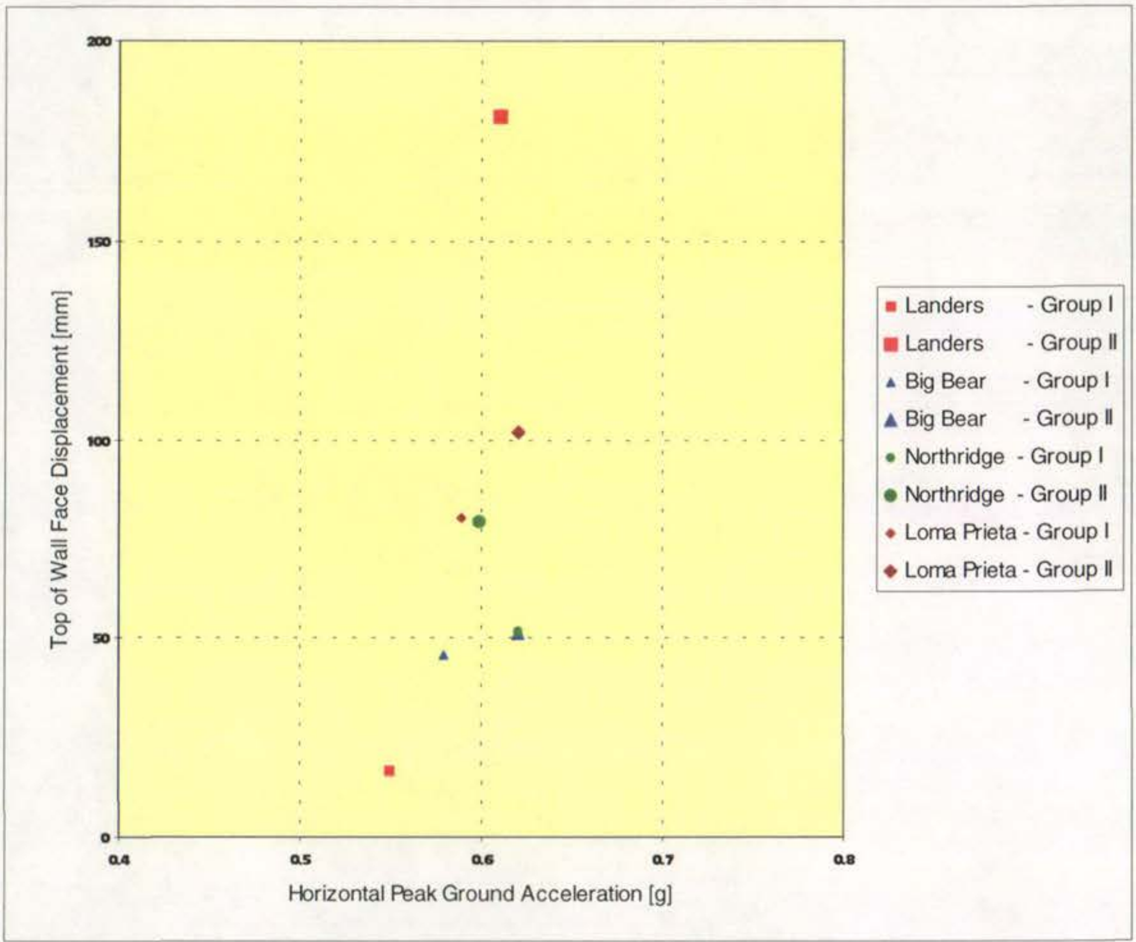


Illustration 8 - Top of Wall Face Displacement v Horizontal Peak Ground Acceleration

Illustration 8 indicates the wide range of displacements of the wall, even when the horizontal ground shaking applied does not vary significantly, and is in the narrow range of 0.55g to 0.62g. This indicates that peak ground acceleration alone is a poor parameter to assess wall displacements. This is not surprising given that peak ground acceleration only represents the highest peak acceleration, and may not be representative of the range of peak accelerations in a given earthquake record. It also does not represent the duration and the energy content of the earthquake record, or of individual pulses.

Wall displacements may be more related to the energy content of earthquake records, which includes representation of the number and size of large pulses, and the duration of shaking. Energy content of earthquakes may provide a better parameter to assess displacements. To verify this, the wall displacement at the top of the wall was plotted against the sum of the horizontal peak spectral density, which represents the energy of the earthquake record. This is presented in Illustration 9.

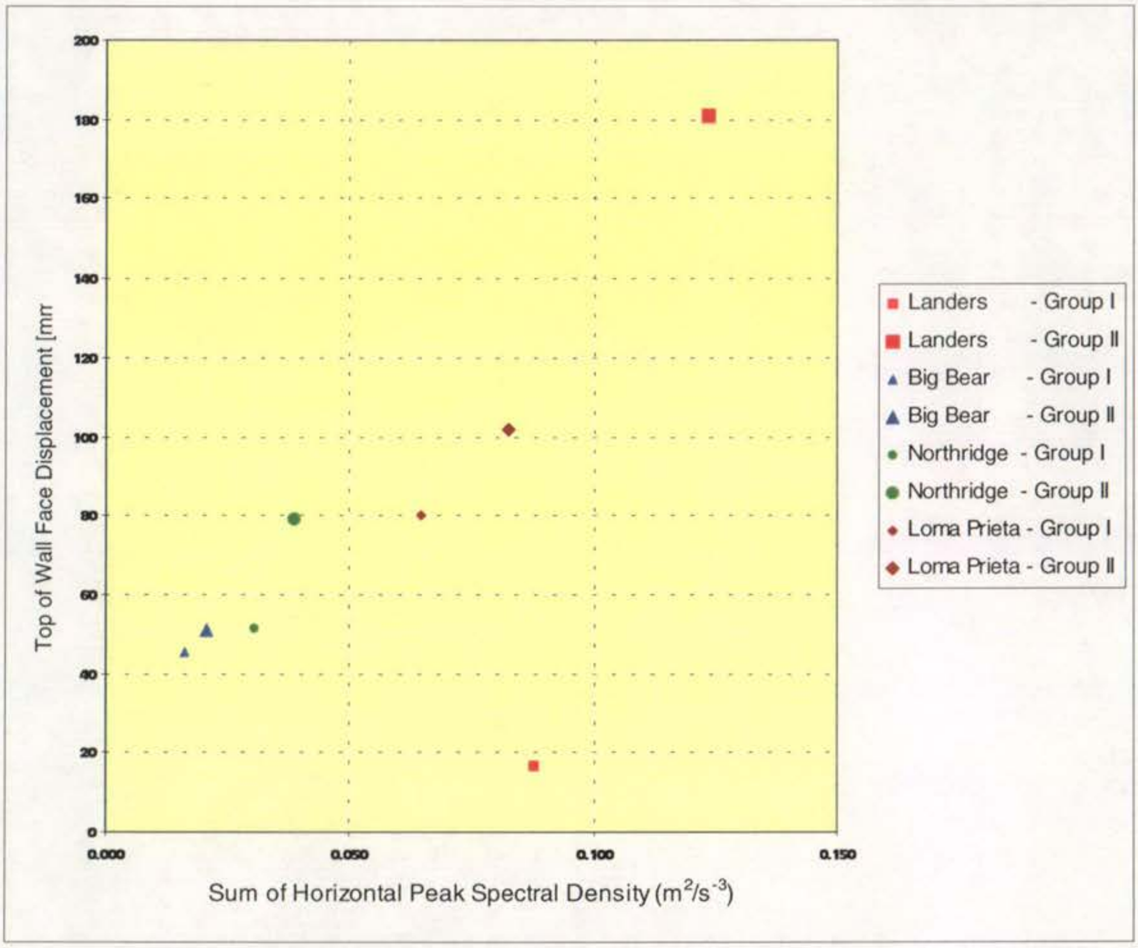


Illustration 9 - Top of Wall Face Displacement v Sum of Horizontal Peak Spectral Density

Considering horizontal ground shaking alone, Illustration 9 shows that the wall displacements show a good correlation with the sum of the horizontal peak spectral density. Only the displacement from Group I Landers Earthquake shaking does not correlate well with the sum of horizontal PSDs. This may be due to the earthquake record having a large energy, but with most of the energy being concentrated at a lower acceleration amplitude for a greater part of the earthquake duration, where the critical force required to initiate wall movement is not available.

Illustration 10 shows the variation of the displacement at mid height of the wall with the sum of the horizontal power spectral density, when horizontal shaking alone is applied (that is, no vertical shaking).

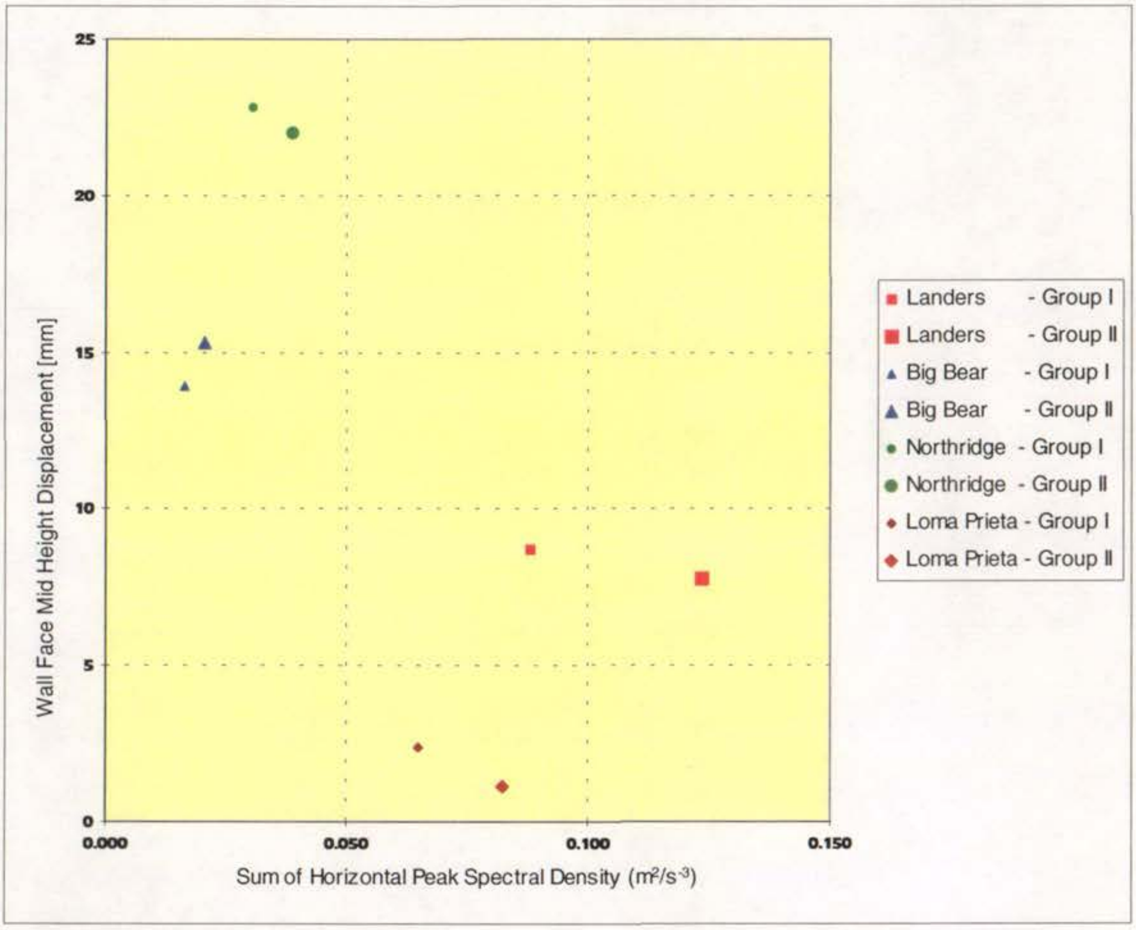


Illustration 10 - Wall Face Mid Height Displacement v Sum of Horizontal Peak Spectral Density

The displacements at mid height of the wall do not show a good correlation with the sum of the horizontal PSD as seen with the top of the wall displacements. The reason for this is not very clear. This may be due to the frequency content of the earthquakes, and the influence of the frequency content of earthquakes is discussed further in Section 9.7. The smaller displacements recorded with horizontal shaking alone are also more sensitive to the inaccuracies in the model.

9.4 Effect of Vertical Accelerations on Wall Displacement

The wall displacements at mid height, with the application of both horizontal and vertical accelerations, are shown against the peak vertical acceleration in Illustration 11. The wall displacement is represented in dimensionless form as the displacement ratio at mid height of the wall, which is the displacement with vertical shaking divided by the displacement without vertical shaking, as discussed in Section 8.4.

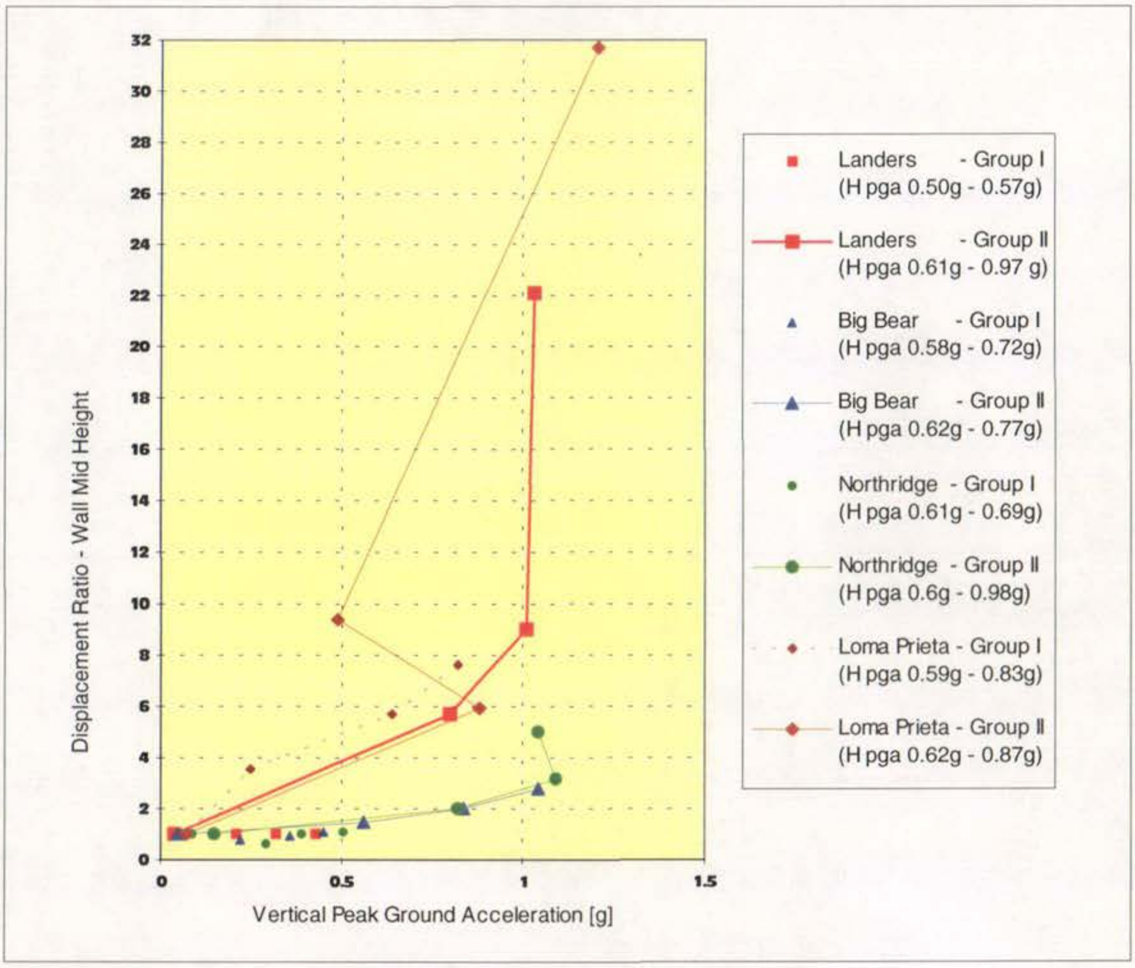


Illustration 11 - Displacement Ratio (Mid Height) v Vertical Peak Ground Acceleration

Illustration 11 shows that the displacement changed very little with the application of vertical ground shaking in addition to horizontal shaking for the Group I earthquake shaking, with the exception of the records from Loma Prieta Earthquake. In the case of the Group I shaking based on the Loma Prieta Earthquake, the displacement was up to almost eightfold, with the application of vertical ground shaking accelerations of up to 0.82g. Even for a modest vertical peak ground acceleration of 0.25g, the displacement was three fold of that without the vertical acceleration. The increases in displacement ratios in the Loma Prieta earthquake based records may be sensitive to small inaccuracies in the displacement with horizontal shaking alone, which is very small at about 2 mm.

In the case of the larger energy Group II earthquake shaking, the displacement significantly increases with the application of vertical shaking. In the Group II shaking based on the Big Bear and Northridge earthquakes, the displacements are about 1.5 to 5 times larger when vertical shaking is introduced, whereas with the Landers and Loma Prieta earthquake based shaking, the displacements are very much larger (up to 32 times). Again the large displacement ratios for the Loma Prieta based shaking, may be sensitive to inaccuracies in the very small displacement of about 1 mm with horizontal shaking alone.

The displacement ratio is shown against the dimensionless Acceleration Ratio (vertical pga divided by horizontal pga) in Illustration 12.

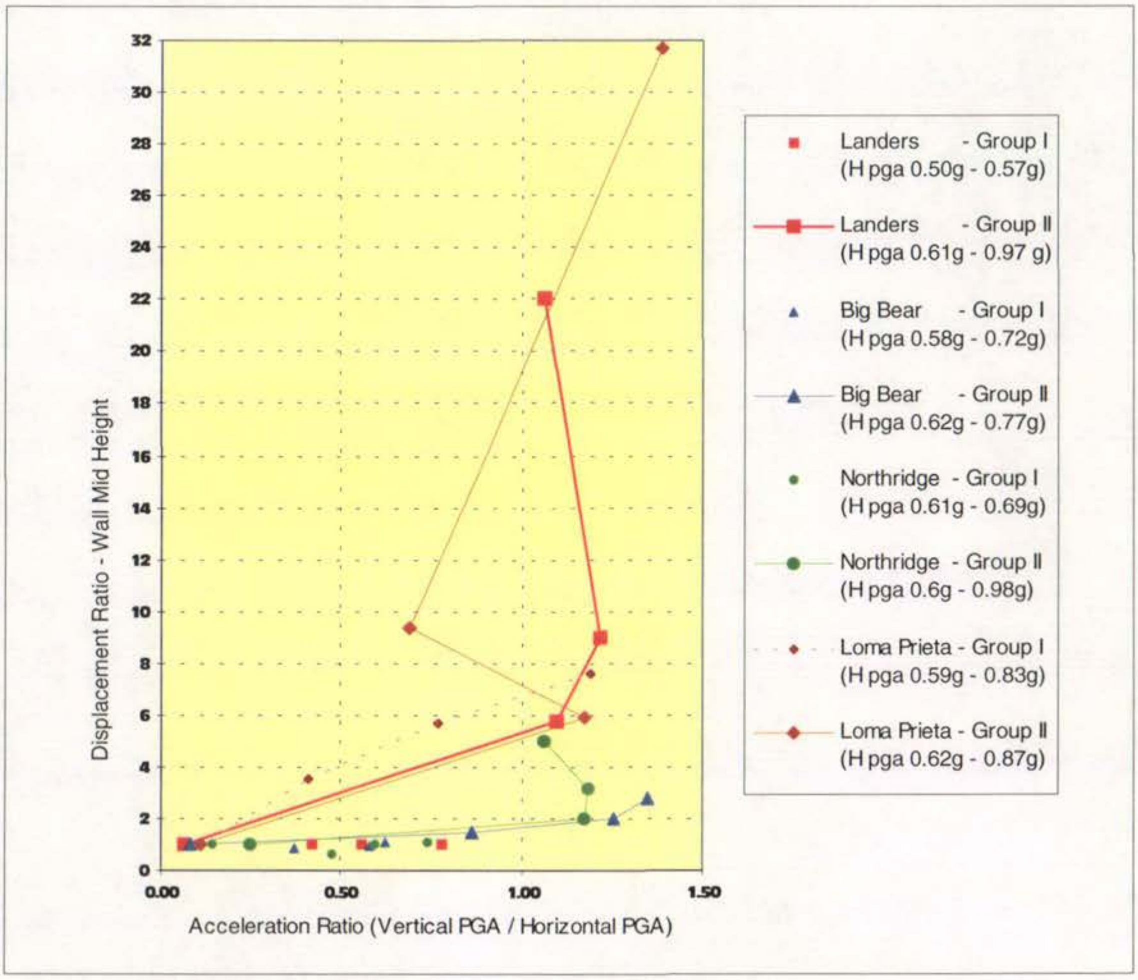


Illustration 12 - Displacement Ratio (Mid Height) v Acceleration Ratio

The diverse range of displacement ratios for comparable levels of horizontal and vertical peak ground accelerations, suggest that peak ground accelerations may not provide a good parameter to assess wall displacements.

9.5 Influence of Power Spectral Density on Wall Displacements

The peak ground accelerations do not appear to be a good parameter to assess wall displacements as discussed above. One of the reasons can be the energy content of these four earthquakes. The energy contents of the earthquakes, represented by the average sum of the horizontal power spectral density used in the analyses, are compared in a bar chart in Illustration 13.

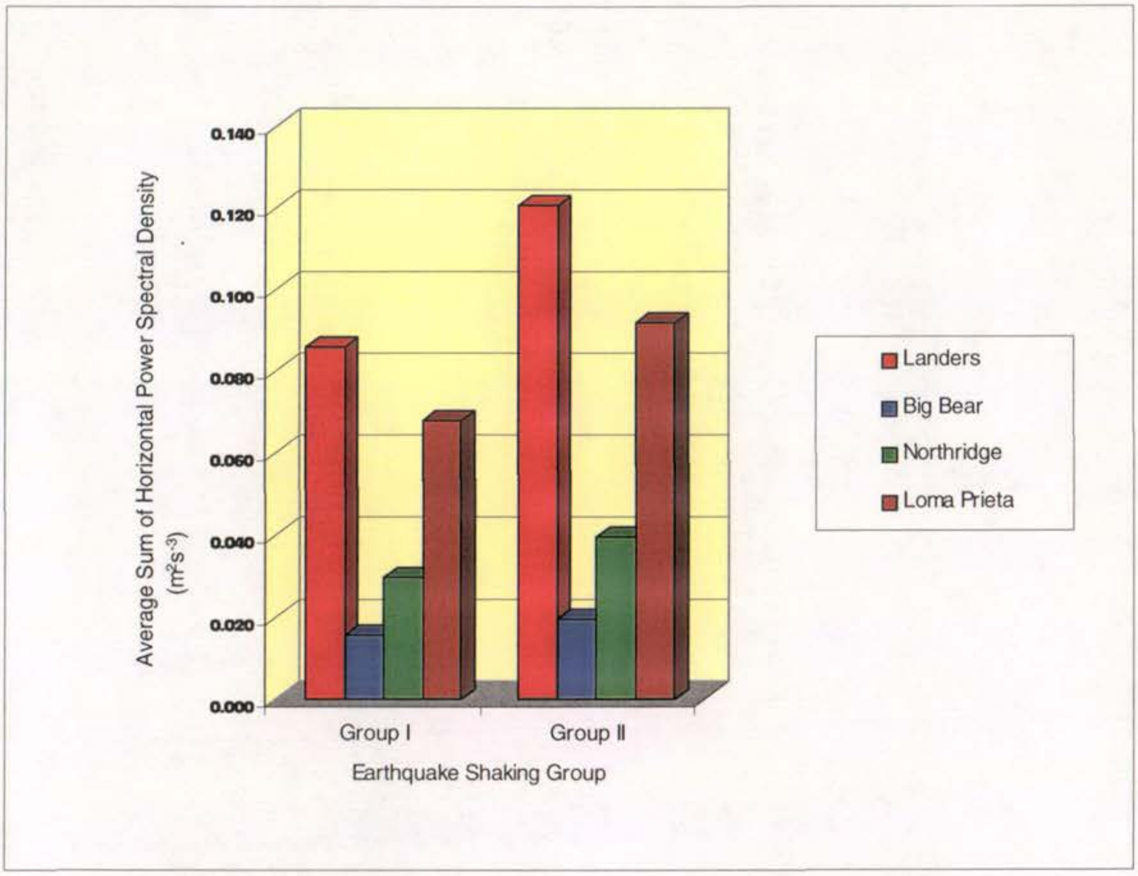


Illustration 13 - Average of Sum of Horizontal Power Spectral Density

The larger energy content of the Landers and Loma Prieta earthquake based time histories used in the study, as indicated by Illustration 13, could explain the large displacements, when combined with vertical shaking.

Illustration 14 shows that larger the horizontal power spectral density, the larger the likely wall displacement, when vertical shaking is applied. It also shows that for a given sum of horizontal power spectral density and similar levels of horizontal peak ground acceleration (shown in the legend), the displacements can still vary widely.

This illustrates that horizontal shaking peak ground acceleration and energy do not explain the variation in displacements, and shows that strong vertical shaking does have a very significant effect on the displacement of walls.

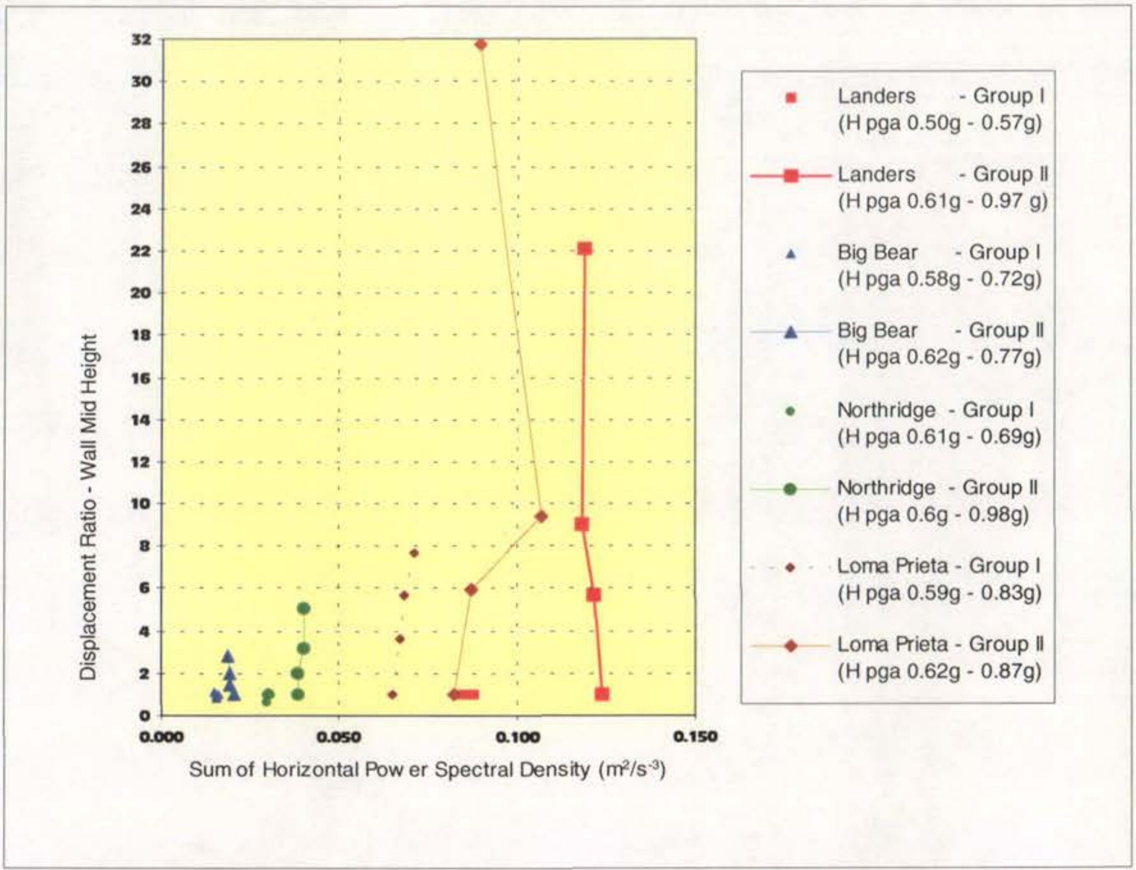


Illustration 14 - Average of Sum of Horizontal Power Spectral Density

The effects of the energy associated with vertical shaking on the wall displacements are considered in Section 9.6 below.

9.6 Effect of Vertical Power Spectral Density

The effect of the sum of vertical Power Spectral Density on wall displacements is explored in Illustration 15.

Illustration 15 shows the Displacement Ratio against the Sum of the Power Spectral Density (PSD) in the vertical direction, for each of the earthquakes. As noted earlier, except for the Loma Prieta based earthquake shaking, the Group I earthquake shaking does not give any appreciably larger displacements for records with a larger sum of vertical PSD. For the greater energy Group II earthquake shaking, the displacement clearly increases with an increase the energy of vertical shaking (represented by the sums of vertical PSD). This relationship is stronger compared to the relationship with peak vertical ground acceleration.

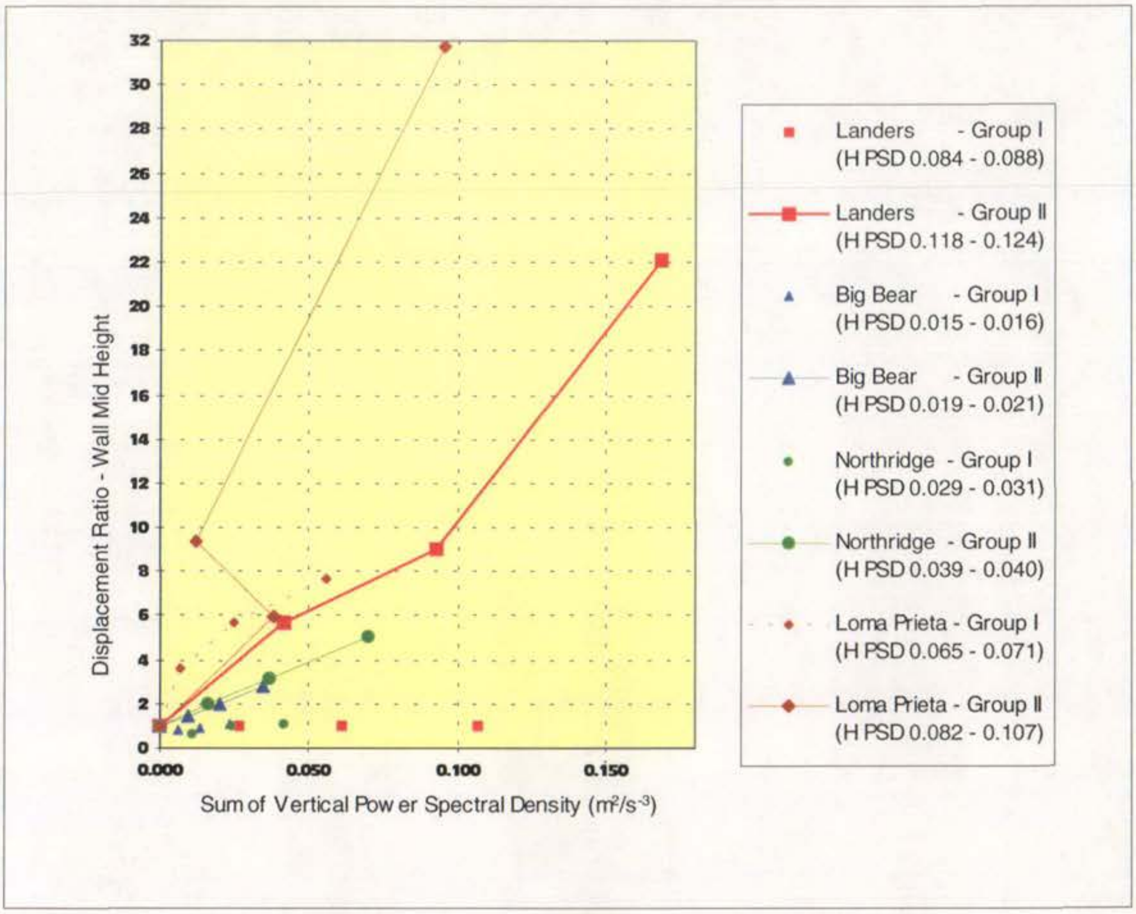


Illustration 15 - Displacement Ratio (Mid Height) v Sum of Vertical Power Spectral Density

The differences in the displacement ratios between the different earthquakes are apparent when the sum of the horizontal PSDs of the earthquakes are also considered. The horizontal PSDs are also given in the legend of Illustration 15.

The illustration shows that :

- The effect of vertical shaking is very small for the smaller energy Group I shaking, except for the Loma Prieta based earthquake shaking
- The larger displacements from the Loma Prieta based shaking may be due to the larger horizontal PSD (energy) compared to the Big Bear and Northridge based shaking
- The vertical shaking has a very significant effect on wall displacements when the earthquake shaking has a larger energy content (Group II shaking used)
- The magnitude of the displacements depends on the energy content of both the vertical and horizontal shaking
- The large displacement ratios for the Loma Prieta based shaking may be sensitive to the displacements from horizontal shaking alone, given that they are very small.

The effect of vertical shaking on wall displacements can be better expressed in dimensionless form by plotting the wall displacement ratio against the ratio of the vertical and horizontal power spectral density, as shown on Illustration 16.

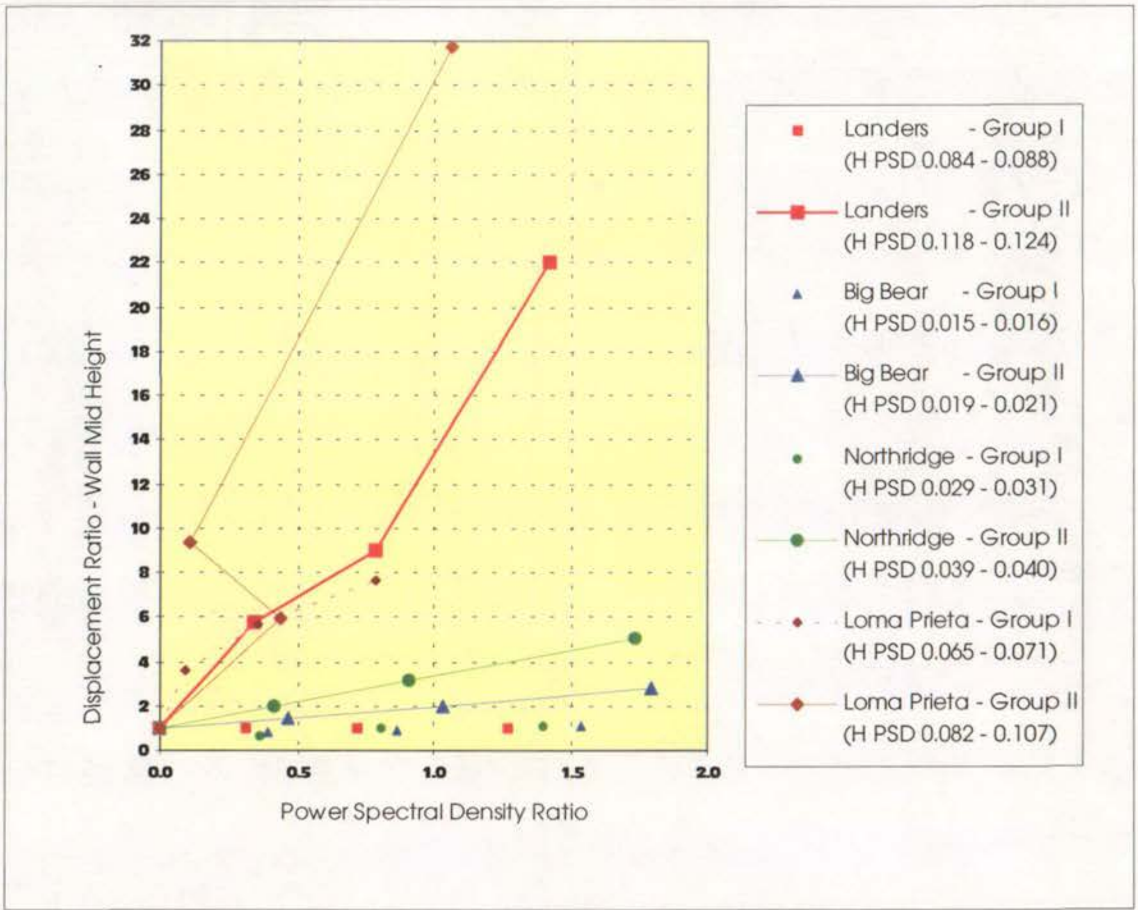


Illustration 16 - Wall Displacement Ratio v Power Spectral Density Ratio

The commonly assumed ratio between vertical and horizontal shaking of two-thirds, gives a displacement of about twice that assuming horizontal shaking alone, given that the earthquake is one with a significant energy content. This is based on the Group II shaking based on Big Bear and Northridge records. With larger energy shaking such as from Group II shaking based on Landers earthquake, or Loma Prieta, the displacement can be much higher.

For near field areas close to the epicentre, the ratio between vertical and horizontal shaking can be greater than one, and there could be a correspondingly larger displacement. The differences in the displacements are likely to depend on the proportion of larger amplitude shaking within the time history. The vertical acceleration may have limited effect where the wall is distant from earthquake sources, where the amplitude of shaking may be less than or similar to the critical acceleration of the structure, and the vertical shaking is small.

9.7 Influence of Frequency Content

Illustration 10 in Section 9.3 indicated that horizontal earthquake shaking with greater energy (Landers and Loma Prieta based shaking) led to smaller displacements than those with a smaller energy content (Big Bear and Northridge). However, with vertical shaking, Landers and Loma Prieta shaking gave much larger displacements than the lower energy Big Bear and Northridge earthquake shaking.

A similar phenomenon is indicated by the results published by Siddharthan et al (1992), as discussed in Section 3.4.5. This may be related to the frequency content of the different earthquakes. The frequency content of the shaking based on the four earthquakes can be seen in the Power Spectral Density plots against frequency, presented in Illustration 17, and Figure 6 to Figure 9.

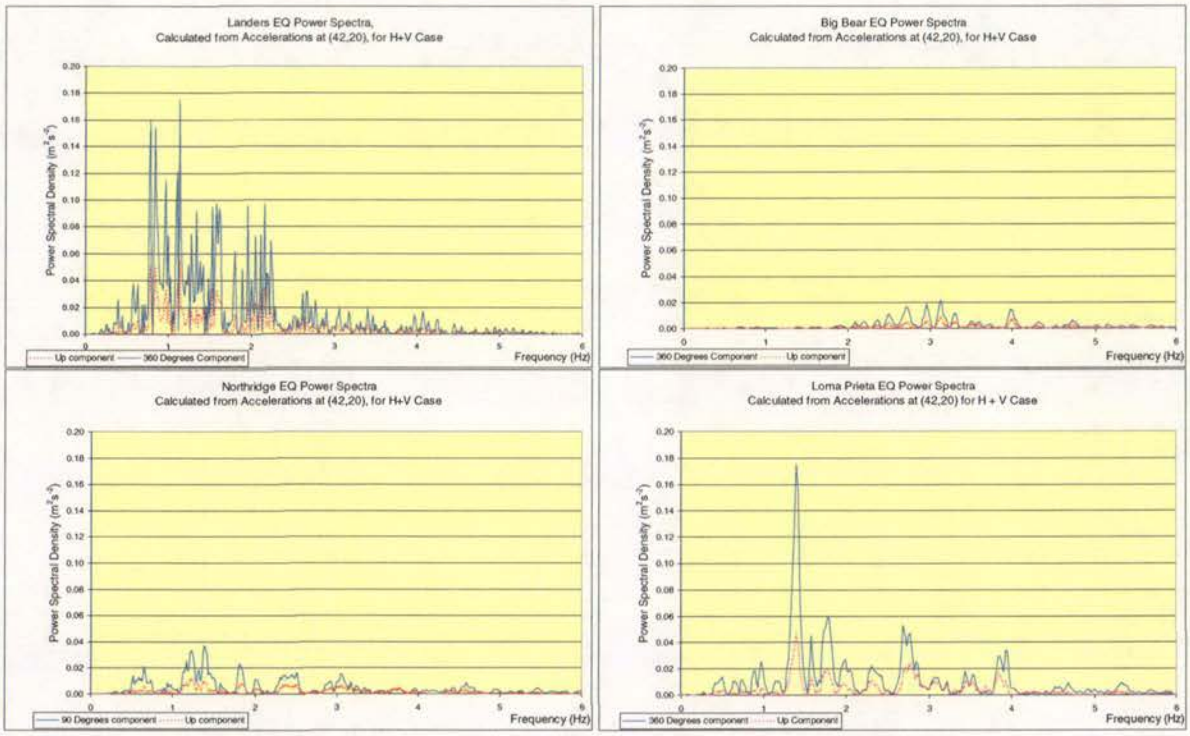


Illustration 17 – Power Spectral Density – Frequency Charts

The Power Spectral Density versus Frequency charts show the wide difference in energy-frequency distributions for the four earthquakes. The figures show that the energy associated with the vertical and horizontal components have a similar frequency distribution. The relationship between the frequency content of the earthquake and the natural period of the wall structure may influence the behaviour of the wall under different earthquakes.

The reinforced earth wall modelled probably has a short period of response, say less than ¼ second (or a frequency of 4 Hz to 5 Hz). The energy content of these earthquakes is low at these frequencies, see Illustration 17, and therefore resonance effects are not likely.

However, when vertical earthquake shaking is applied, it is possible that the period of the reinforced soil wall changes as the flexibility of the wall is increased. This may happen as a consequence of two effects, which will be cyclic in nature, as with the vertical shaking. These effects are :

(a) Change in the friction on the strips due to the variation in the vertical overburden stress with vertical acceleration. This was incorporated in the FLAC model, and will also happen in actual walls. Change in the strip friction can alternatively lead to more slippage increasing flexibility, and less slippage reducing flexibility.

(b) Change in the flexibility of the soil due to change in the yield curve of the soil during horizontal shaking. The Mohr-Coulomb soil strength model depends on vertical effective stress, which is influenced by vertical shaking. This effect is also likely to occur in real walls. The change in the yield curve (that is, the onset of plasticity) can alternatively lead to more deformation when the yield strength is reduced, increasing flexibility, and less deformation in the opposite cycle, decreasing flexibility.

While these effects are cyclic, it is possible that the reduction in flexibility is more significant than the increase in flexibility in the opposite cycle, because the displacement or deformation leading to a reduction in flexibility will be more dominant than the increased stiffness in the opposite cycle. Therefore, it is plausible that vertical shaking leads to an overall increase in flexibility and hence an increased period of response, or reduced frequency.

Such a change in the period of response could have a profound effect on the behaviour of the structure and hence the displacement, depending on the frequency content of the earthquake. If the period of the structure increases to say ½ or 1 second (frequency of 2 or 1 Hz), then the period may coincide with the large energy content of, for example, the Landers and Loma Prieta earthquake shaking, see Illustration 17, leading to resonance. This could in turn lead to significantly larger displacements. This hypothesis appears to explain the behaviour of the model when exposed to the large energy shaking, with and without vertical shaking, that is :

- small displacements resulted from Landers and Loma Prieta earthquake shaking (compared to that from Big Bear and Northridge cases) with horizontal shaking alone,
- relatively large displacements (compared to horizontal shaking alone and also the Big Bear and Northridge cases) result when vertical shaking is introduced.

Further research may help explore this hypothesis in more detail, and hence confirm its importance for the design of wall structures.

9.8 Reinforcement Strip Forces

The main focus of the study was on the displacement behaviour of the wall. Therefore, the strip forces have not been studied in detail. However, it was observed from the model analyses that strip forces at the end of earthquake shaking were significantly greater than those before the earthquake shaking, particularly near the bottom of the wall. In addition to being of importance for the design of the strips, this could also have a significant effect on creep displacements along the reinforcement-soil interface, particularly if a more cohesive soil or geosynthetic-reinforcement were to be used. This could lead to ongoing deformation after an earthquake.

From a brief perusal of some of the results, the effect of vertical shaking on strip forces appeared to vary, and no clear trend was apparent.

9.9 Implications for Design

Retaining structures are presently designed based on pseudo-static design methods, which take into consideration of horizontal ground accelerations. A displacement-based design approach is used for the more critical structures in areas of high seismicity, particularly in New Zealand. Displacements are generally estimated from peak ground accelerations based on the loadings code NZS 4203:1992 (Standards Association of New Zealand, 1992). In New Zealand, walls support important structures such as bridge abutments and buildings in steep terrain. Therefore the earthquake displacements are very important for good performance of these structures.

The current research confirms that vertical accelerations can have a significant effect on wall displacements, depending on the energy content and size of earthquakes. In particular, structures located in areas near significant sources of earthquakes are likely to experience larger displacements than currently assumed. In these near-source areas, the energy content of earthquakes and the vertical component are likely to be larger. Where the horizontal and vertical shaking are not significantly large (say distant from earthquake sources), then the vertical accelerations are shown to have little effect.

A better understanding of the effect of vertical shaking from earthquakes will enable a more robust design of walls, and where necessary, detailing of supported structures to withstand larger displacements.

This research highlights the important effect of vertical shaking, and will help this to be taken into consideration in the design of structures. However, further research would help consider the effect of vertical shaking of different characteristics on different types of retaining structures, and will enable the selection of parameters and methods that can be applied in design.

9.10 Further Research

Further research on a number of issues associated with the displacement behaviour of retaining structures, particularly in the presence of vertical shaking, would be valuable to better understand the earthquake behaviour of walls and enable more reliable design.

Some of the issues worthy of further consideration are :

(a) The performance of the walls under earthquakes with different characteristics, and consideration of the effect of the earthquake frequency content.

(b) The hypothesis that a change of the period of the wall occurs in the presence of vertical shaking, and that resonance effects lead to much larger displacements, where the frequency content of the earthquake is unfavourable.

(c) The effect of vertical earthquake shaking on reinforcement forces.

(d) The effect on walls with different aspect ratios (reinforcement length to height), and hence critical accelerations

(e) Extension of the model analyses to include other types of walls and in particular geosynthetic reinforced walls and modular block facing.

(f) Improvement of the model, and baseline correction of earthquakes to reduce the gross model displacement, which masks the wall behaviour, and hence improve accuracy.

(g) Development of parameters and methods for design taking into account the significant effect of the energy content of earthquakes.

10 Summary and Conclusions

The objective of this research was to assess the displacement performance of retaining walls, and the significance of vertical ground motions, which can occur in near-fault areas. The research included a literature review and numerical analyses using a finite-difference FLAC model of a Reinforced Earth wall. The model analyses were carried out for a wall with an aspect ratio of one (reinforcement length to height of wall), and robust seismic design for an acceleration of 0.42g horizontal shaking. Four different earthquake time history records from California were used in the analyses and were modified to obtain a greater range of earthquake shaking. The study was effective in assessing the displacement performance of walls and the parameters that have a significant influence on wall behaviour, under vertical ground shaking.

The following are the key outcomes from this study :

(a) The importance of vertical accelerations has been known since the 1920s. However, this was not given serious consideration until recently. Recent papers by Siddharthan et al (1992), Ling and Leshchinsky (1998) and Elms (2000) highlight the importance of vertical shaking to the assessment and design of retaining walls.

(b) The recent studies have involved pseudo-static analyses. No model studies, either physical or numerical, have been carried out to verify the effects of vertical shaking or to assess parameters that might be important. This was the first known model study.

(c) The study confirmed the effectiveness of using the finite-difference numerical program FLAC in the earthquake analysis of retaining structures incorporating both vertical and horizontal shaking. This required overcoming a number of difficulties with such modelling. Further refinement of the models will facilitate ease of further research and probably use of the model to analyse important structures for design purposes. The recent new version of FLAC may facilitate this.

(d) The analyses confirmed the vulnerability of the upper strips to pullout during earthquake shaking, a factor which has long been recognised in the practical design of reinforced earth structures.

(e) The maximum displacement (other than at the top where reinforcement pullout occurred) with horizontal shaking alone from the analyses was only 23 mm. When vertical shaking is also applied, the displacement of this robust wall structure was less than 25 mm for a modest energy of shaking associated with the four earthquakes chosen, and up to 200 mm for larger shaking with a higher energy. The wall was designed for no displacement under a design acceleration of 0.42g.

(f) The wall displacement at the top, with horizontal shaking alone, showed that a wide range of displacements can occur in different earthquakes, even when they have a similar peak ground acceleration. This is not surprising given that the peak ground acceleration reflects the peak amplitude of a single pulse only, and does not adequately represent the predominant period, strong motion duration or energy content of the earthquakes. The sum of the Power Spectral Density (PSD) was found to better relate to the variation of displacements in these different earthquakes, which had very different sums of peak spectral densities, although peak horizontal ground accelerations were similar.

(g) The vertical shaking had a significant effect on the displacement of the wall. Again the vertical sum of power spectral density was a better parameter to assess wall displacements than peak vertical acceleration.

(h) The vertical shaking generally had very little effect on wall displacements when earthquake shaking of modest energy and peak ground accelerations similar or only slightly larger than the design accelerations were used. However, earthquakes with different frequency characteristics can lead to significantly larger displacements than with horizontal shaking alone. This was observed in the case of the Loma Prieta earthquake record used in the study. This may be due to the frequency content of the earthquake in relation to the natural response frequency of the wall. Loma Prieta had a significant energy content at a frequency of about 4 Hz (period of ¼ second), which may roughly be the natural period of the structure. This indicates that the frequency content of the earthquake and resonance effects can be important.

(i) The magnitude of the displacements depends on both the energy content of horizontal and vertical shaking and the amplitude of shaking.

(j) Landers and Loma Prieta earthquake shaking gave smaller displacements with horizontal shaking alone (smaller than the Big Bear and Northridge shaking), but gave relatively large displacements when vertical shaking was also applied (compared to Big Bear and Northridge shaking). This is interesting, and one hypothesis is that the vertical shaking increased the flexibility of the retaining structure, and hence the period of the structure, to a period where these earthquakes had a significant energy content, leading to resonance effects and hence greater displacements.

(k) Currently design is based on pseudo-static methods using horizontal peak ground acceleration alone. The study shows the importance of the energy and frequency content and the vertical shaking of earthquakes to the displacement performance of retaining structures. This is of significance to the design of retaining systems, particularly where they support structures and are close of sources of earthquakes where vertical shaking can be high.

(l) Further research is recommended to assess the performance of different wall systems under earthquakes with different characteristics, and to develop appropriate design parameters and methods where vertical shaking is important.

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Figures

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Beam plot

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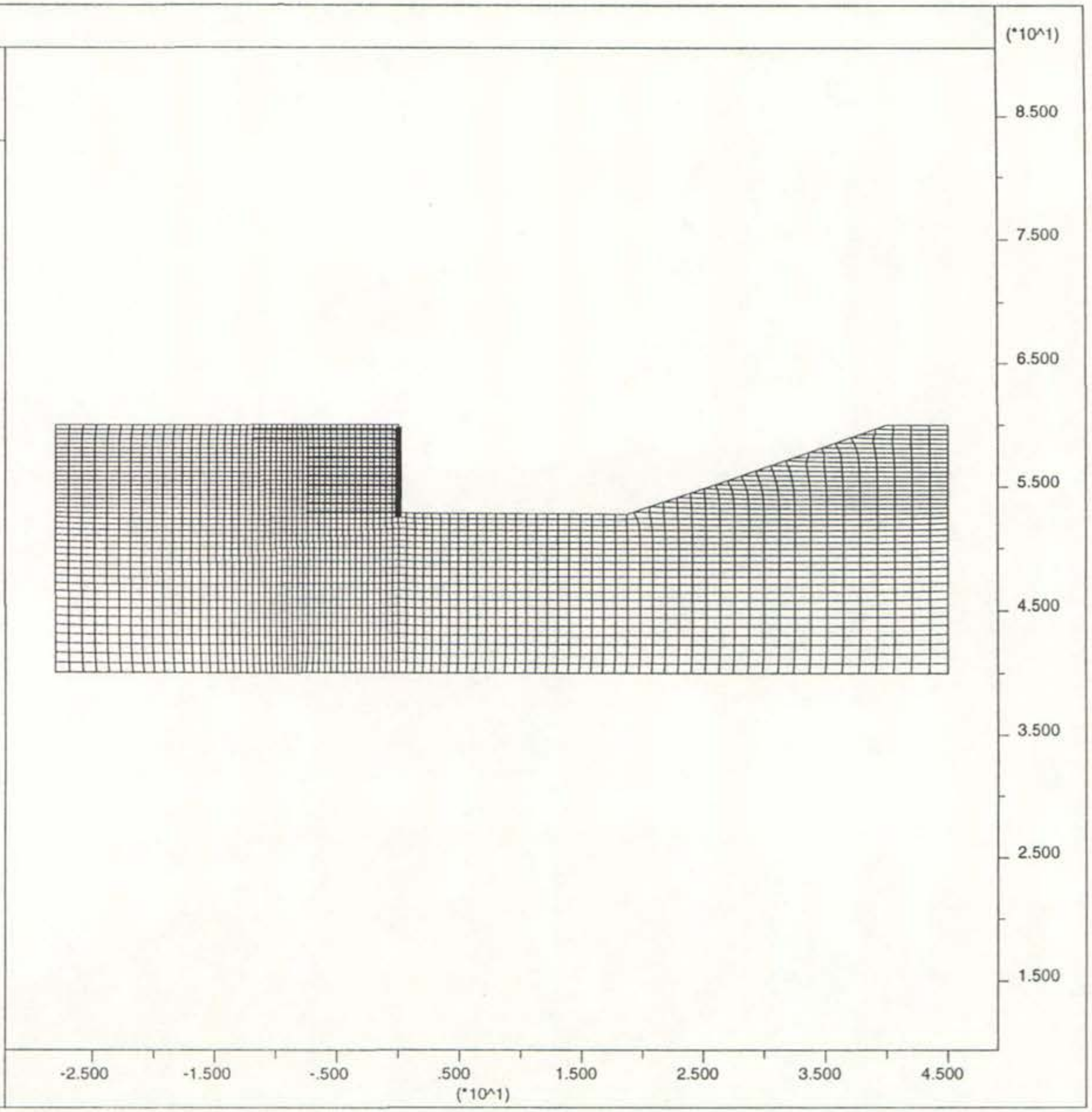
Wellington, New Zealand



OPUS International Consultants

Title: The FLAC model

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 1

JOB TITLE : Shear Wave Velocities with E0=600 MPa

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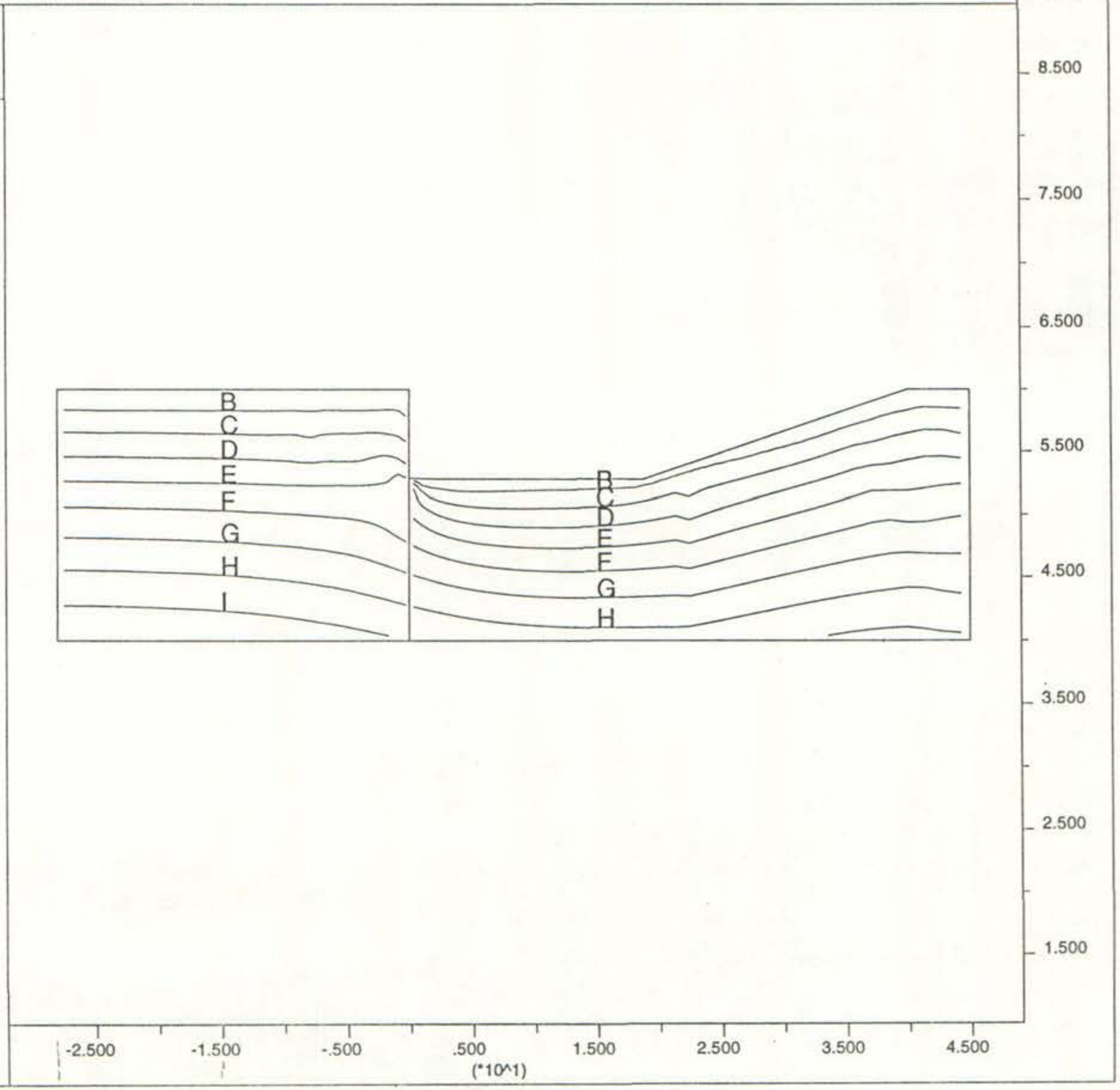
Opus International Consultants
Wellington, New Zealand



OPUS

Title: Shear wave velocities for E_o = 600 MPa

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5c2818.00

Date: June 2000

Figure: 2

Title: Compression wave velocities for E_0 = 600 MPa

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5c2818.00

Date: June 2000

Figure: 3

JOB TITLE : Compression (P) Wave Velocities with E0=60 0 MPa

FLAC (Version 3.23)

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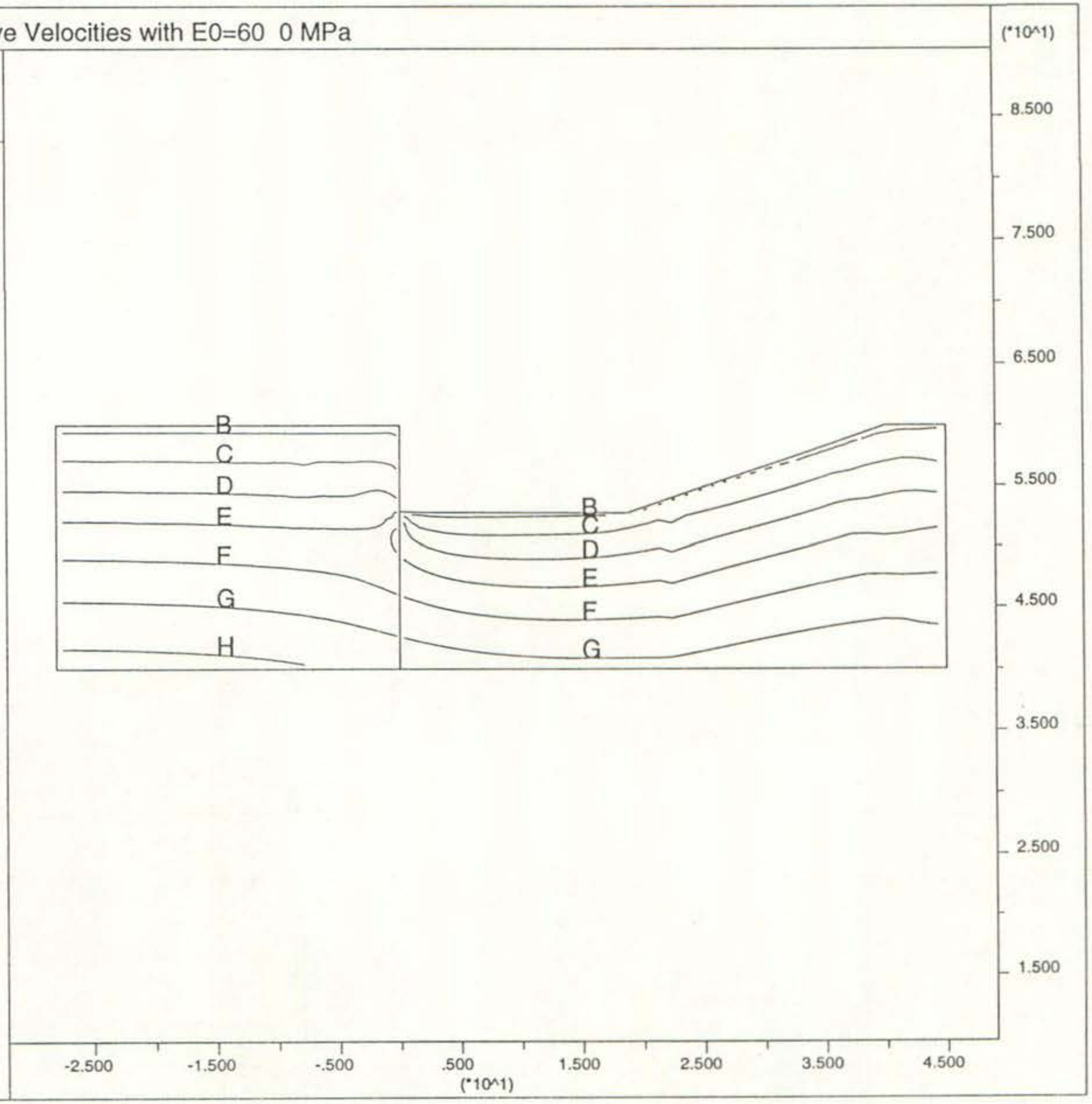

Opus International Consultants
Wellington, New Zealand

JOB TITLE : Shear Wave Velocities with E0=400 MPa

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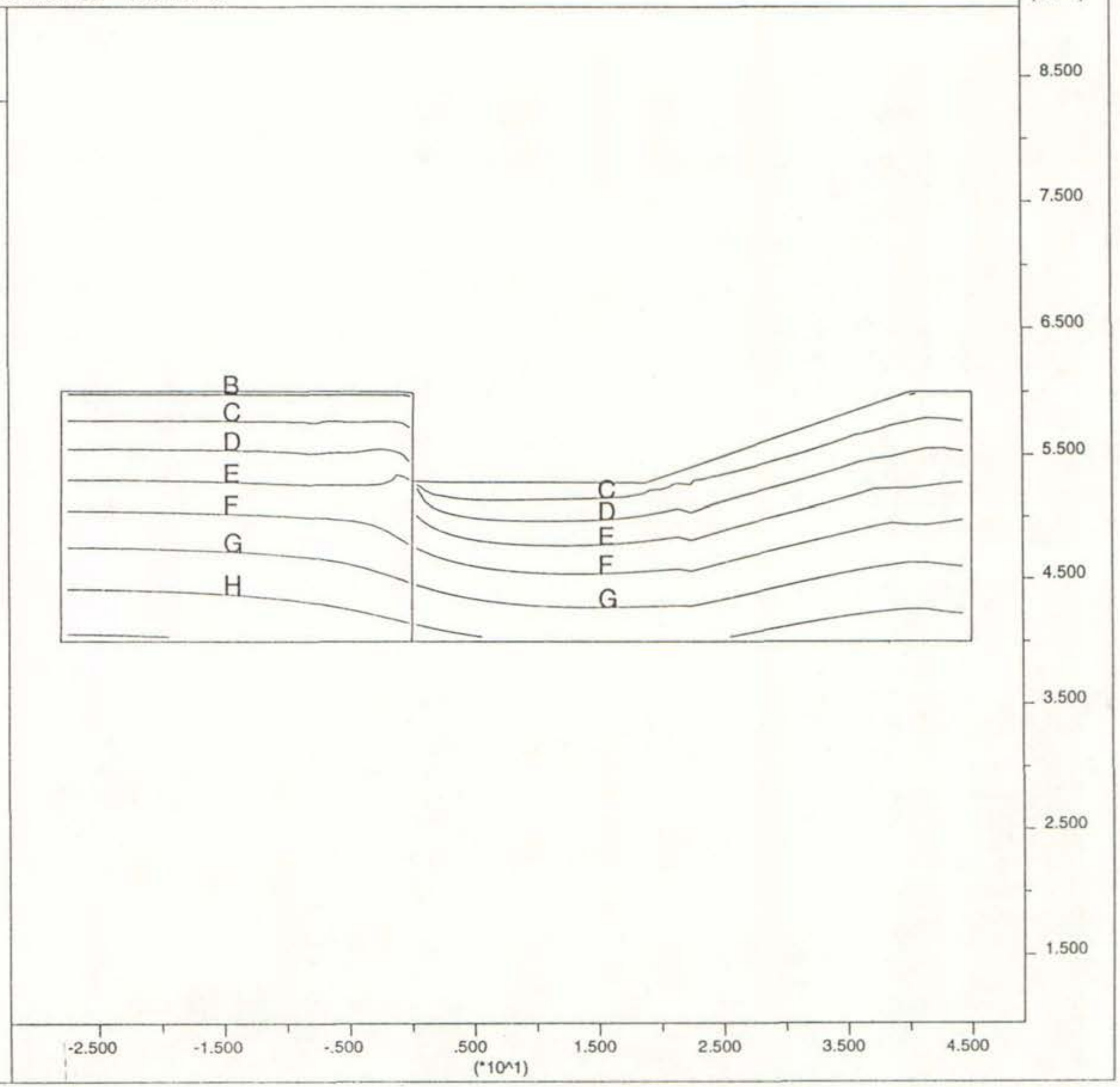

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Wellington, New Zealand

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Title: Shear wave velocities for E_0 = 400 MPa

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5c2818.00

Date: June 2000

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JOB TITLE : Compression (P) Wave Velocities with E0=40 0 MPa

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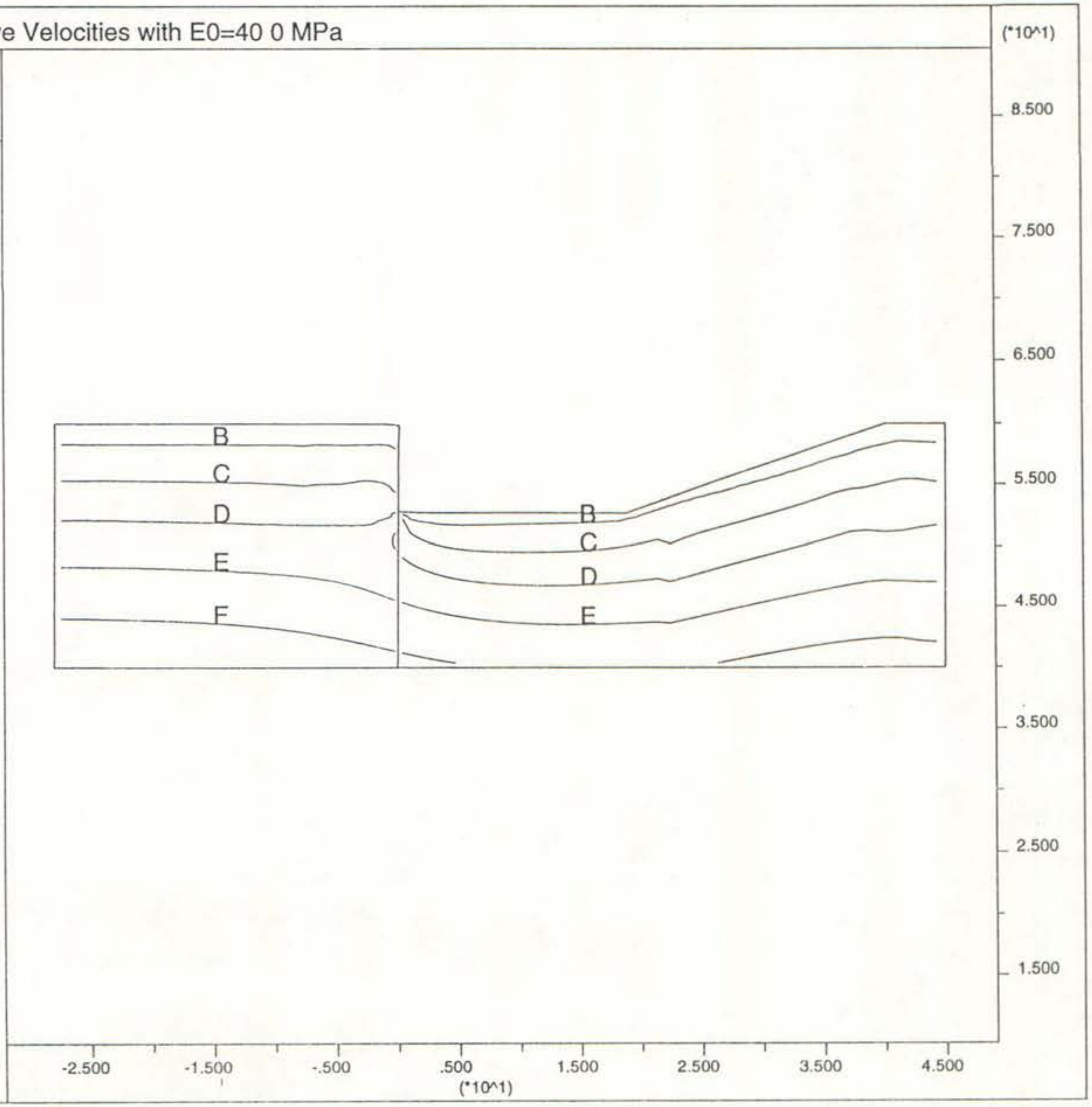

Opus International Consultants
Wellington, New Zealand

OPUS INTERNATIONAL CONSULTANTS

Title: Compression wave velocities for E_0 = 400 MPa

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5c2818.00

Date: June 2000

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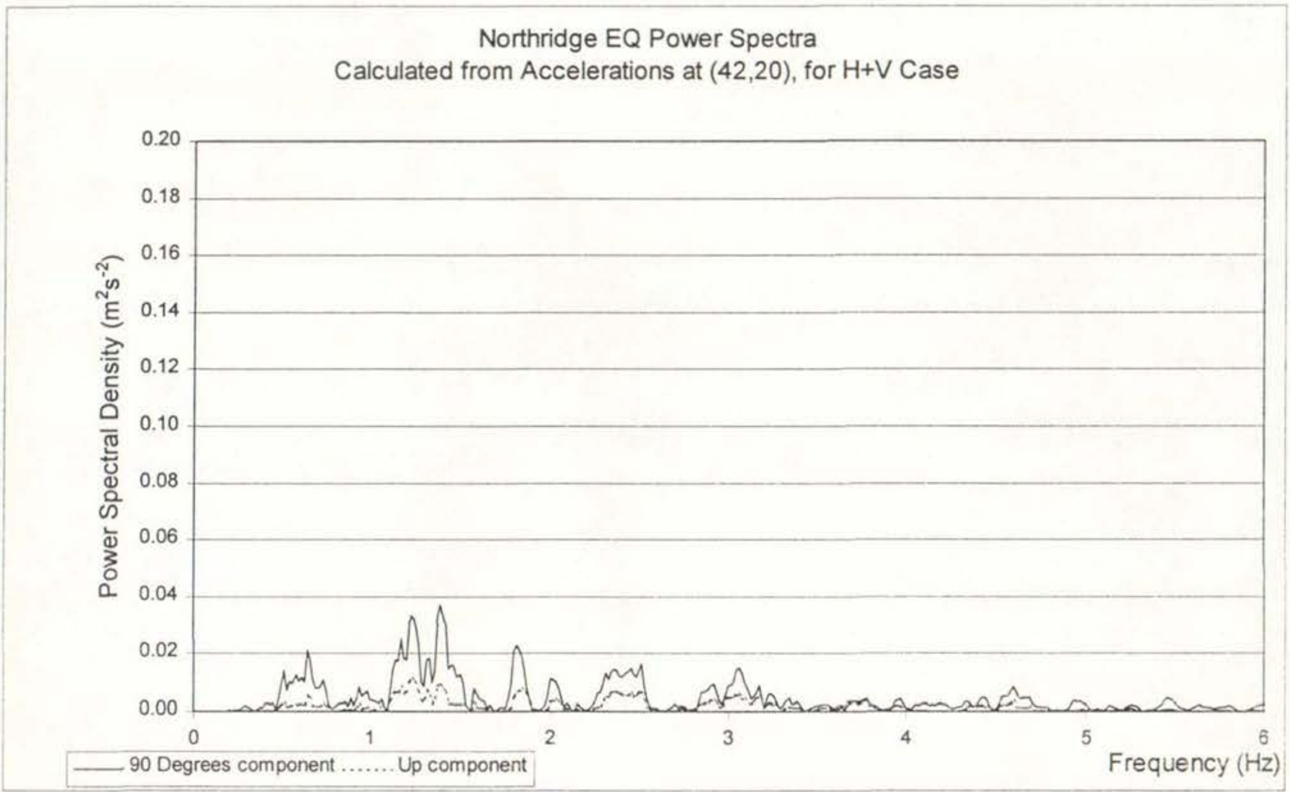

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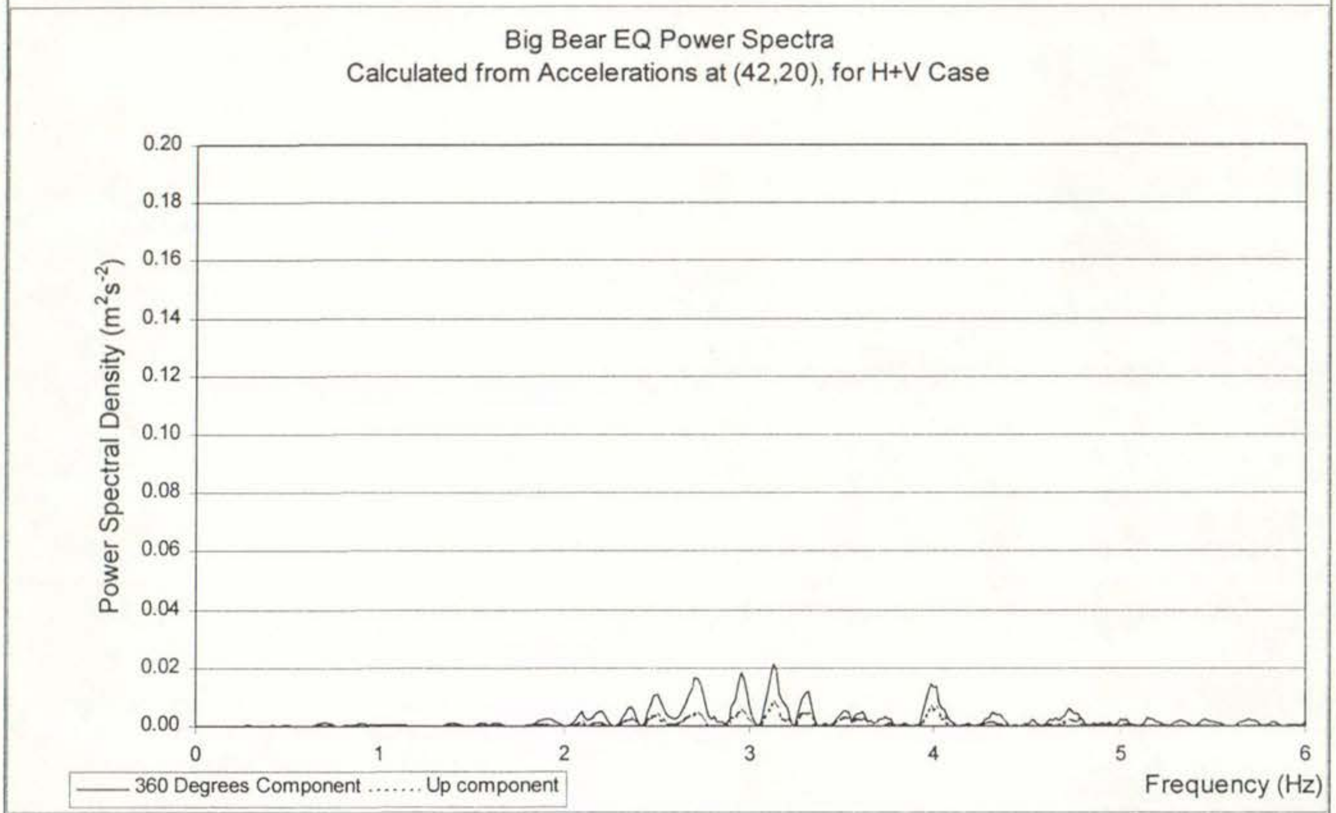
Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures	*Job No.:* 5C2818.00	*Date:* August 2000	*Figure:* 6

OPUS INTERNATIONAL CONSULTANTS



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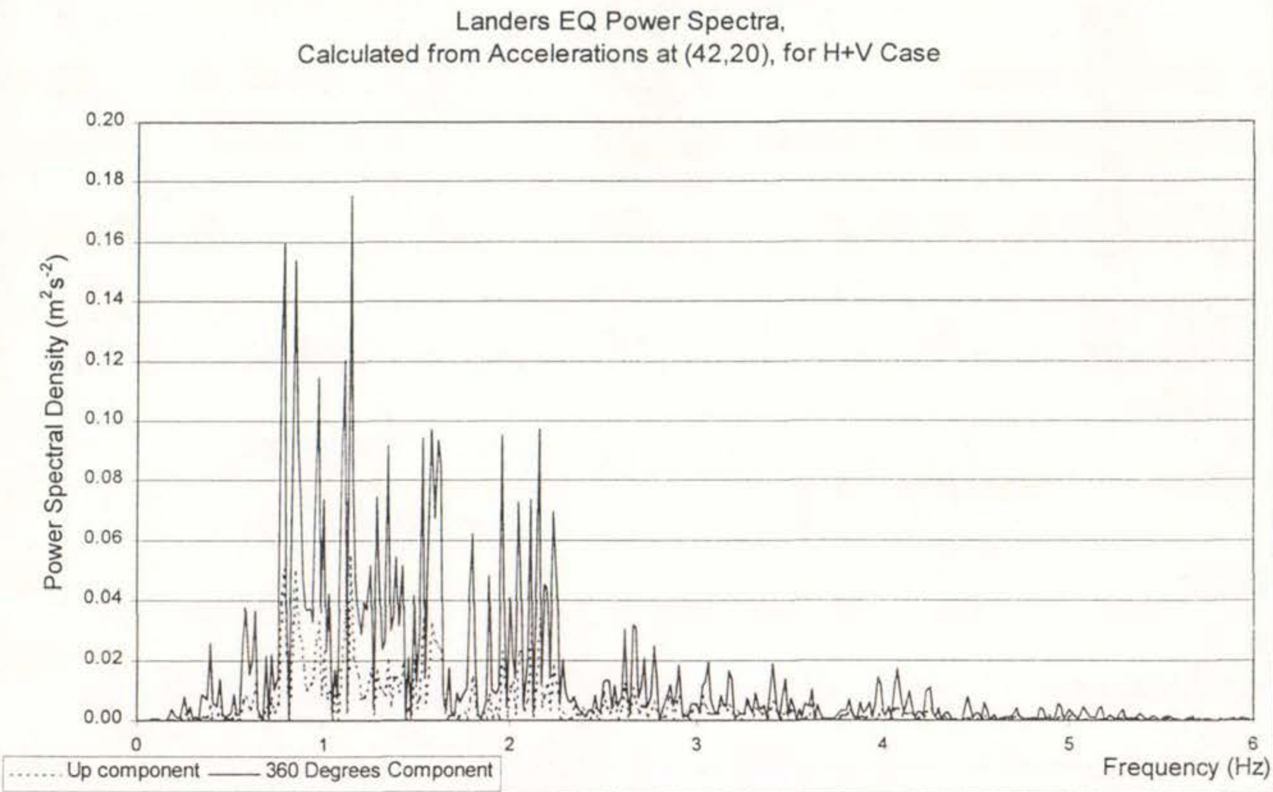
	Project: **Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures**	*Job No.:* 5C2818.00	*Date:* August 2000	*Figure:* 7



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Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures		*Job No.:* 5C2818.00	*Date:* August 2000	*Figure:* 8

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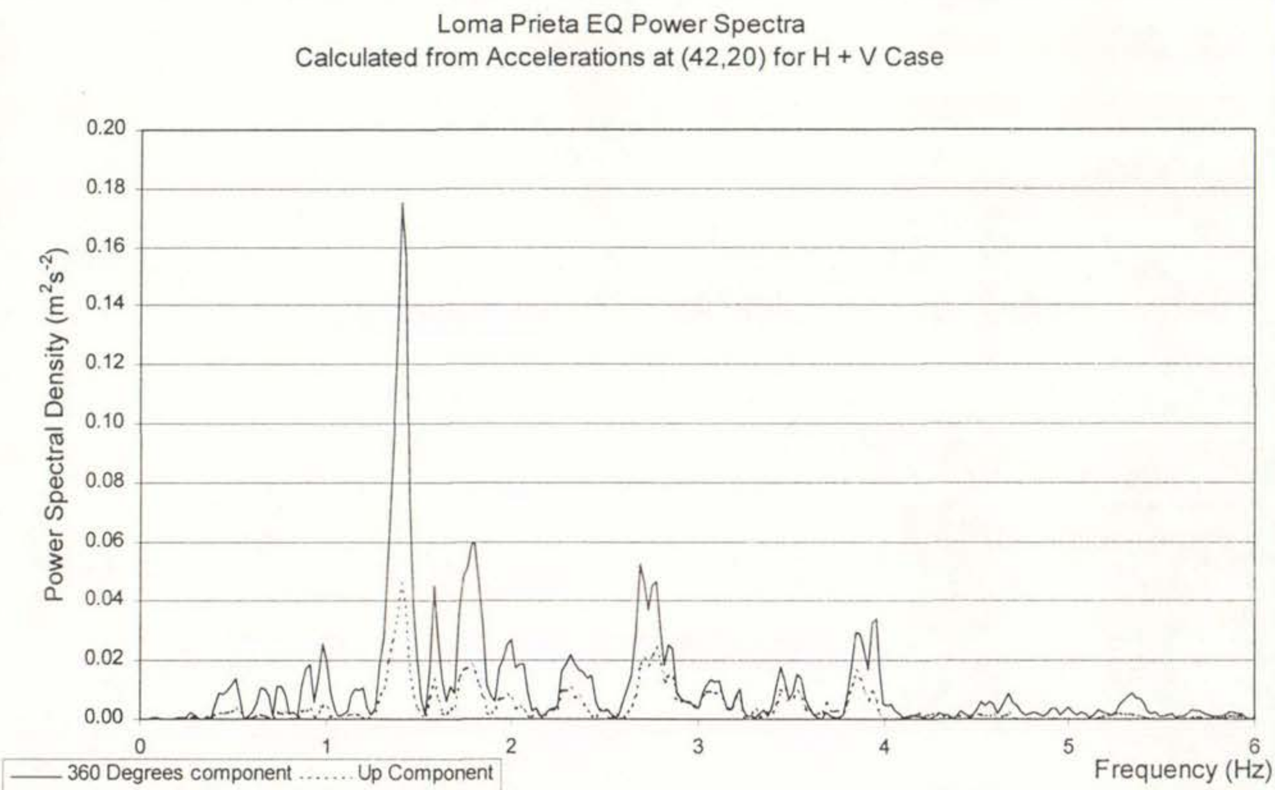

OPUS INTERNATIONAL CONSULTANTS	*Title:* Power spectral density for Loma Prieta Earthquake, 0 degree and up components	*Client:* Earthquake Commission		
	Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures	*Job No.:* 5C2818.00	*Date:* August 2000	*Figure:* 9

JOB TITLE : Completed wall showing reinforcement and facing. Acceleration history locations

FLAC (Version 3.23)

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Acceleration History Locations

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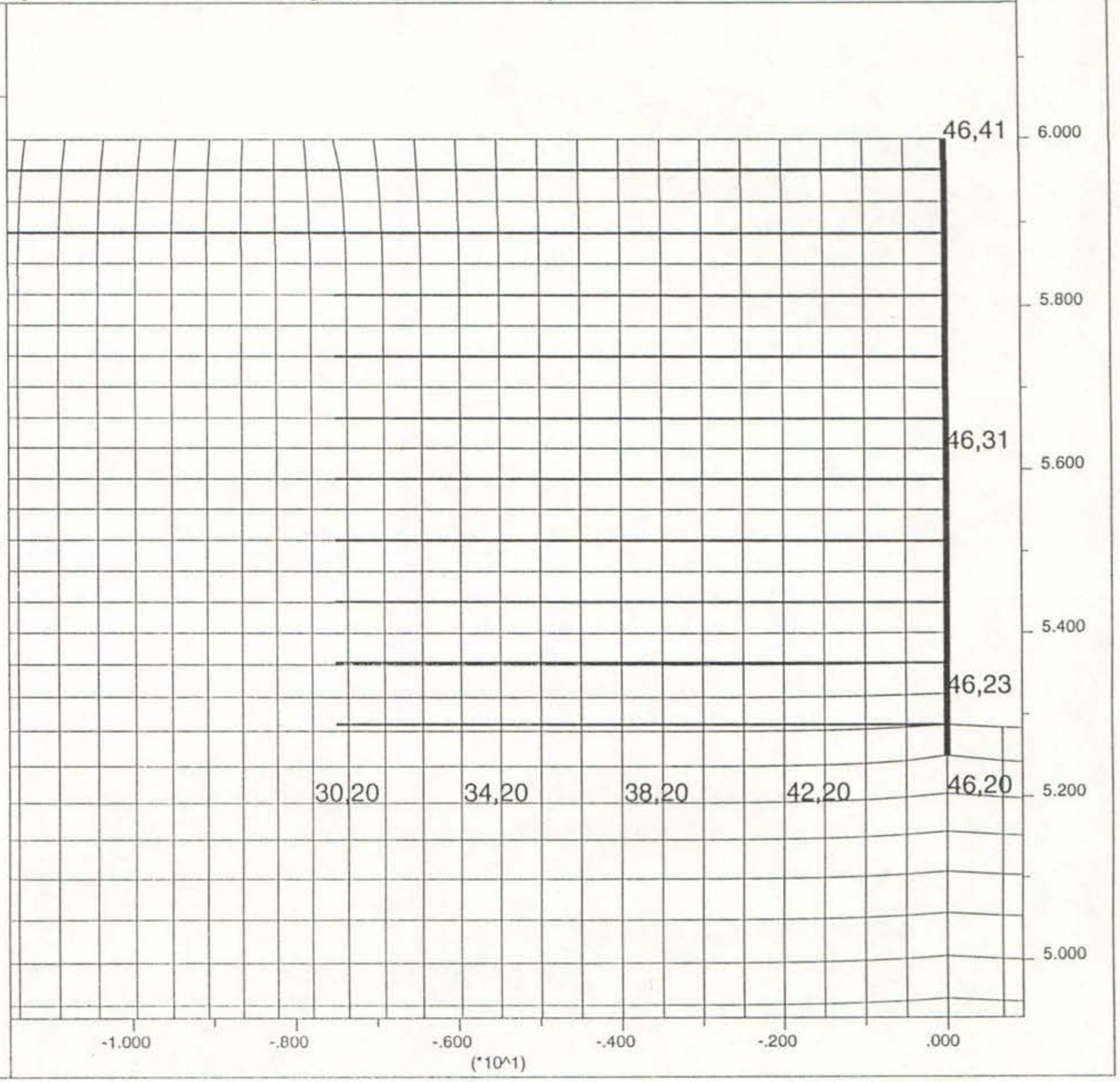


OPUS International Consultants

Title: Acceleration history recording locations

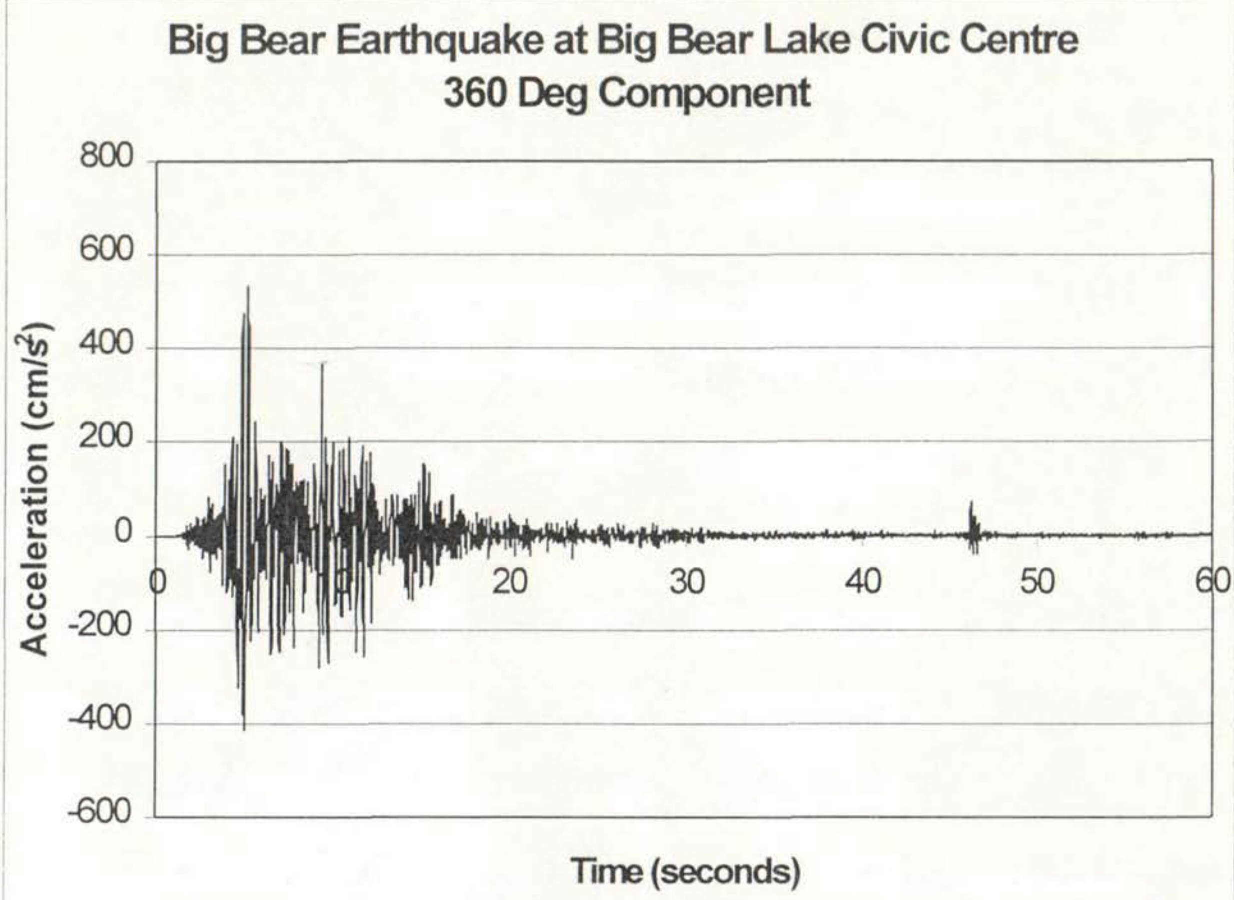
Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

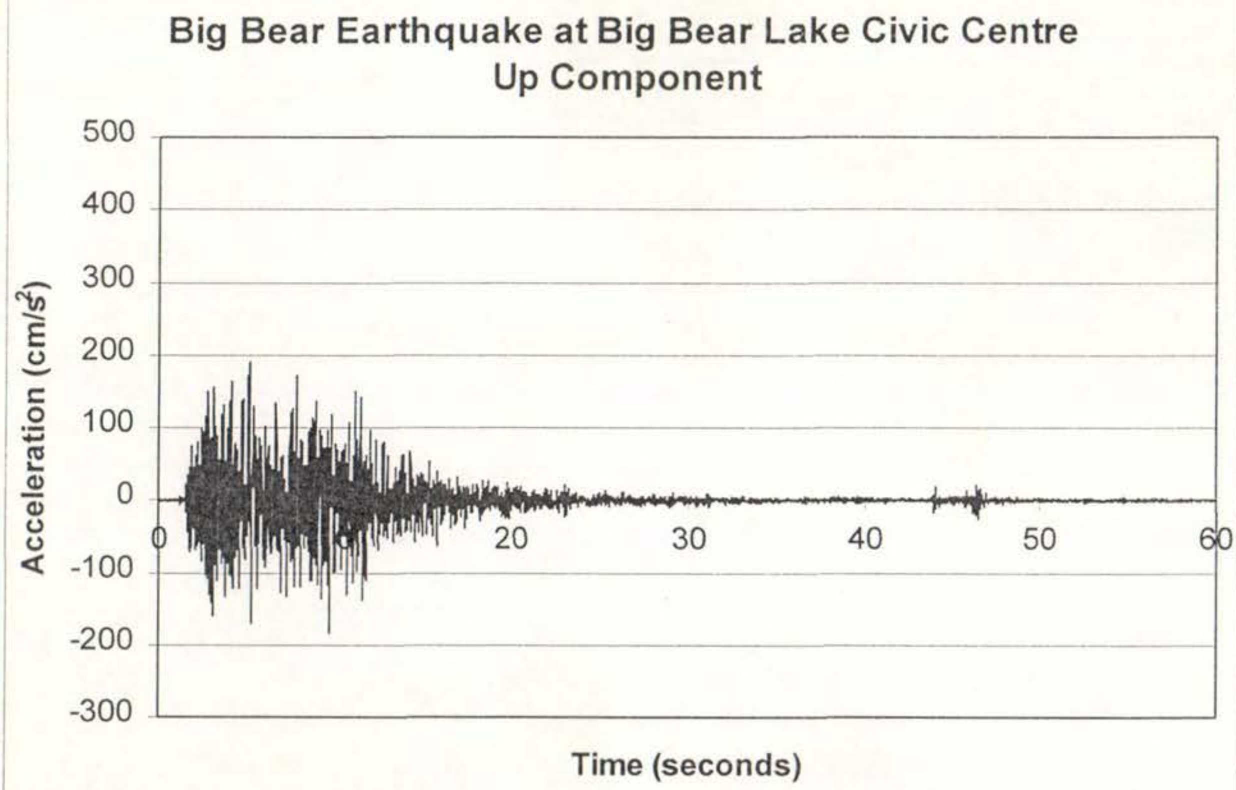
Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

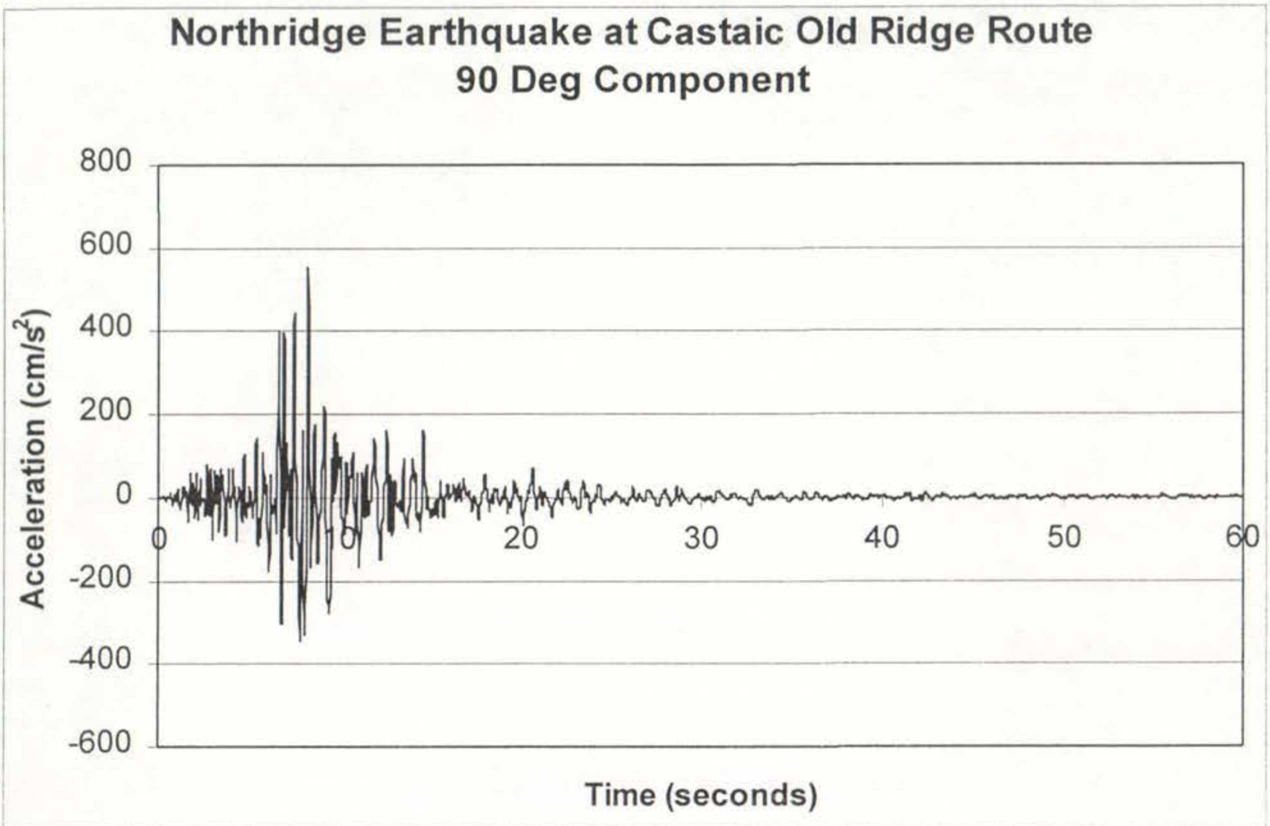
Figure: 10

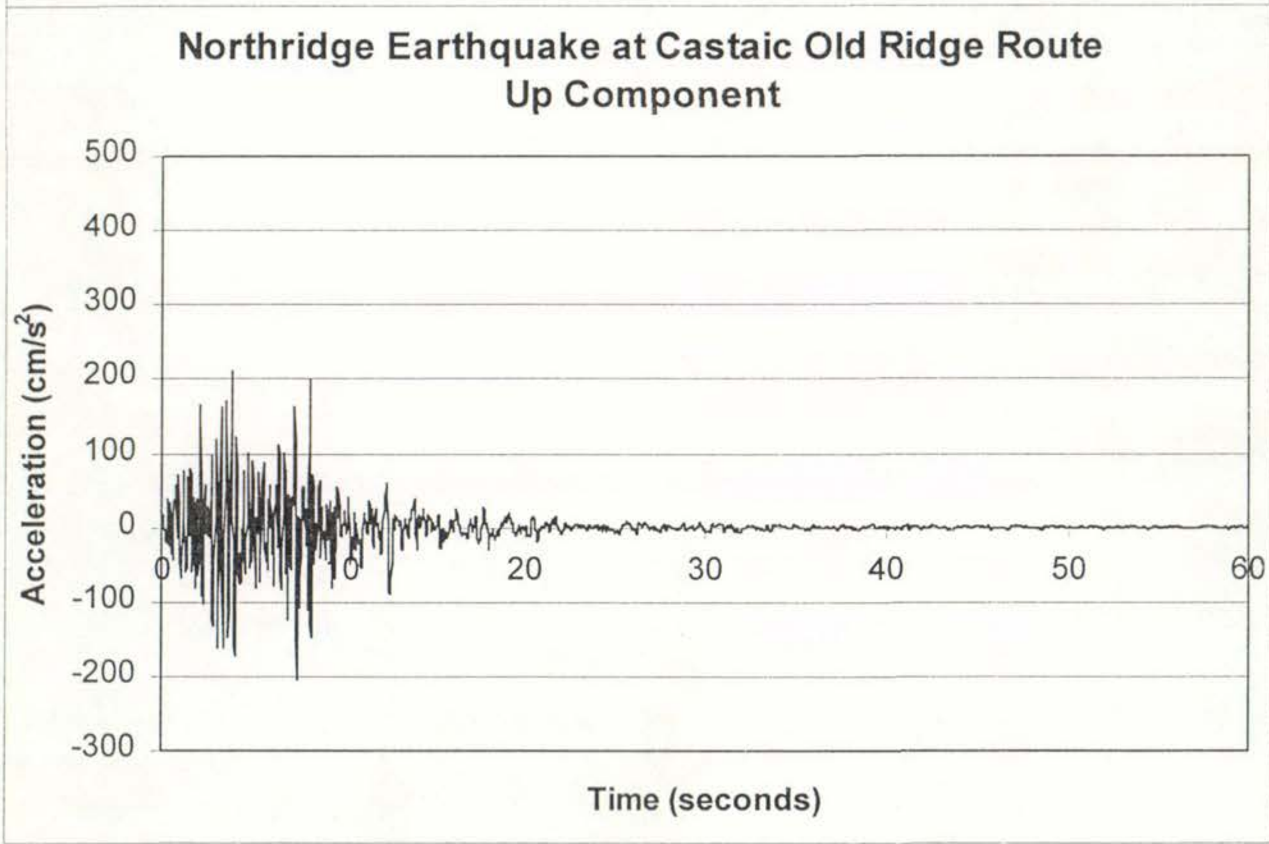




OPUS INTERNATIONAL CONSULTANTS	*Title:* Acceleration records used for Big Bear Earthquake, recorded at Big Bear Lake Civic Centre grounds	*Client:* Earthquake Commission

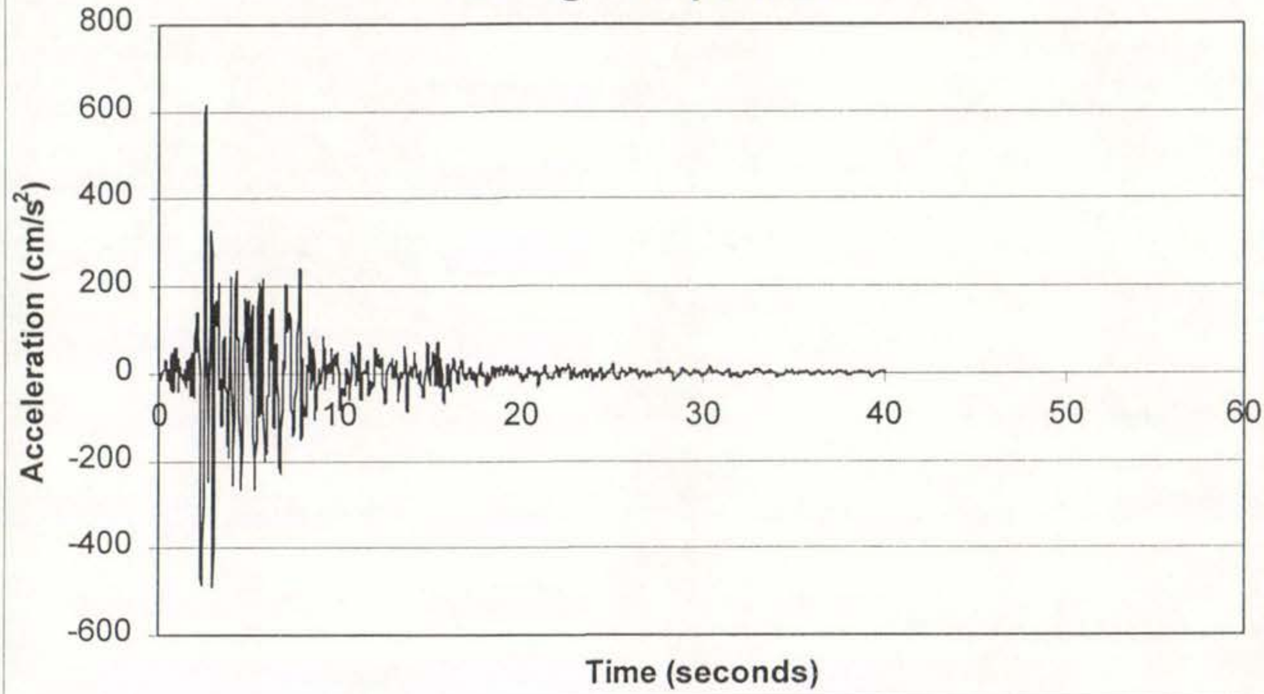
Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures	*Job No.:* 5C2818.00	*Date:* August 2000	*Figure:* 11



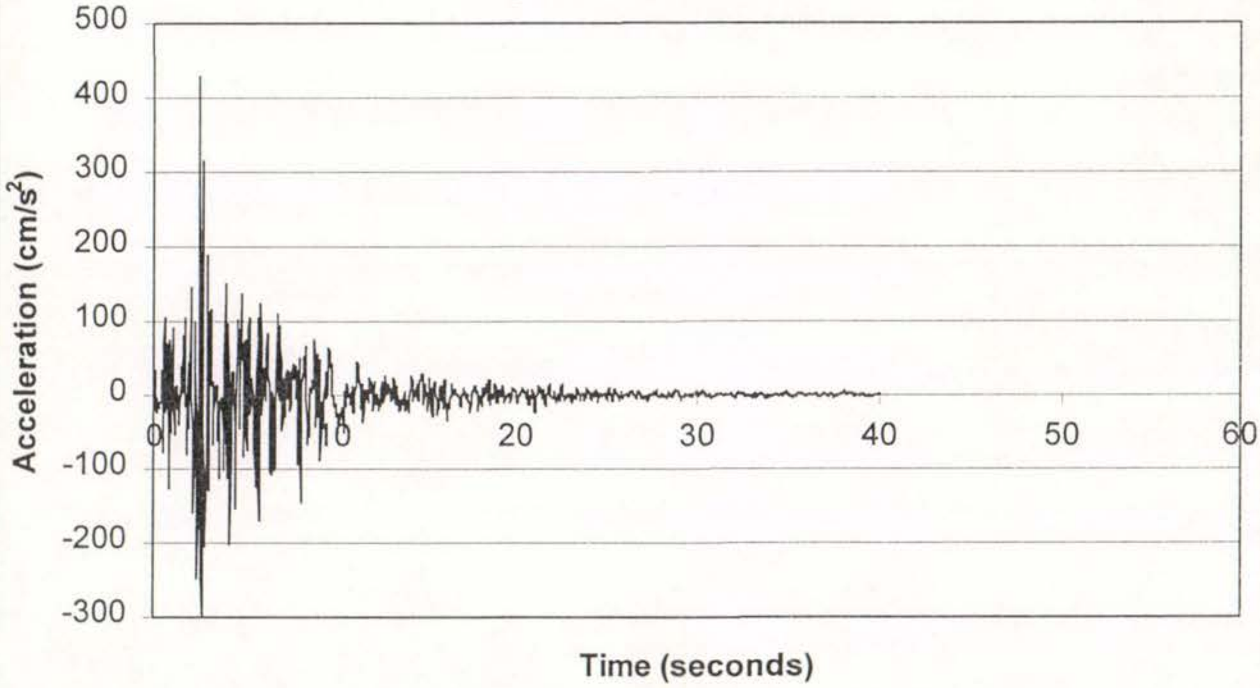


OPUS INTERNATIONAL CONSULTANTS	*Title:* Acceleration records used for Northridge Earthquake recorded at Castaic Old Ridge Route	*Client:* Earthquake Commission		
	Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures	*Job No.:* 5C2818.00	*Date:* August 2000	*Figure:* 12

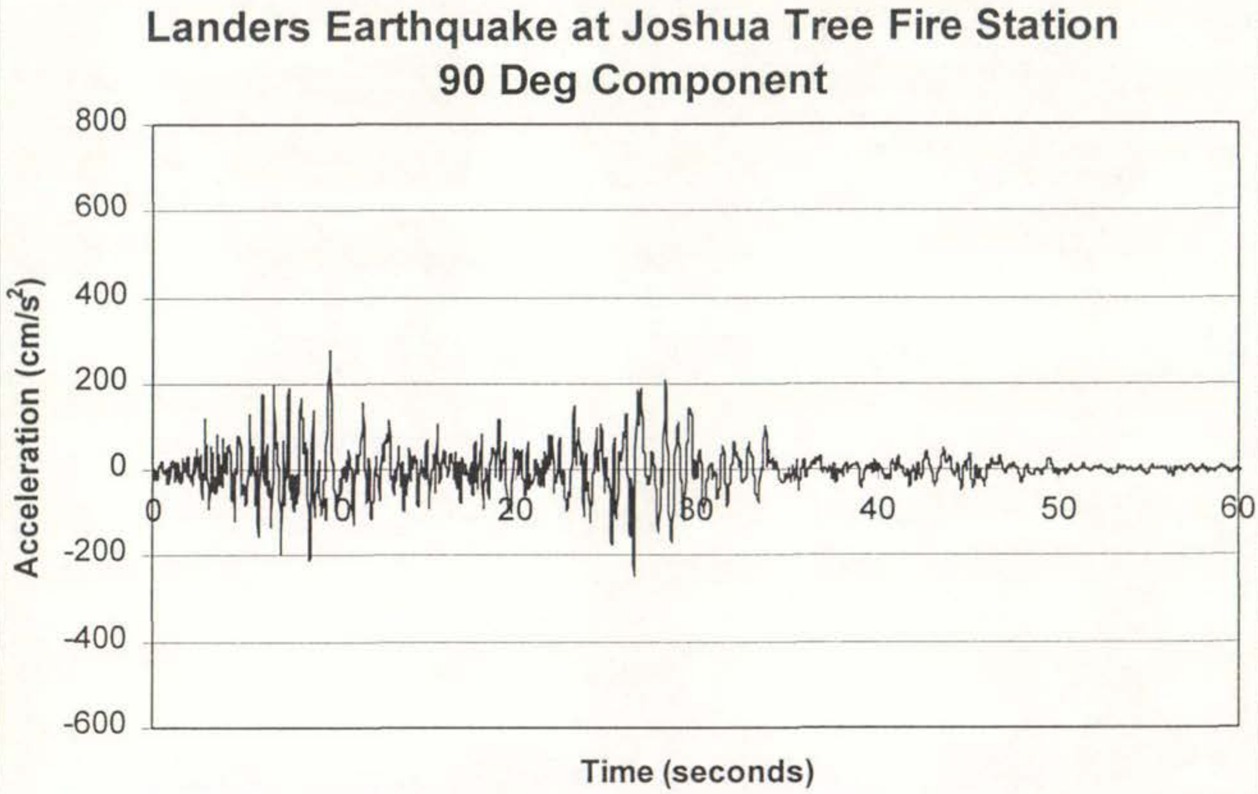
Loma Prieta Earthquake at Corralitos
90 Deg Component

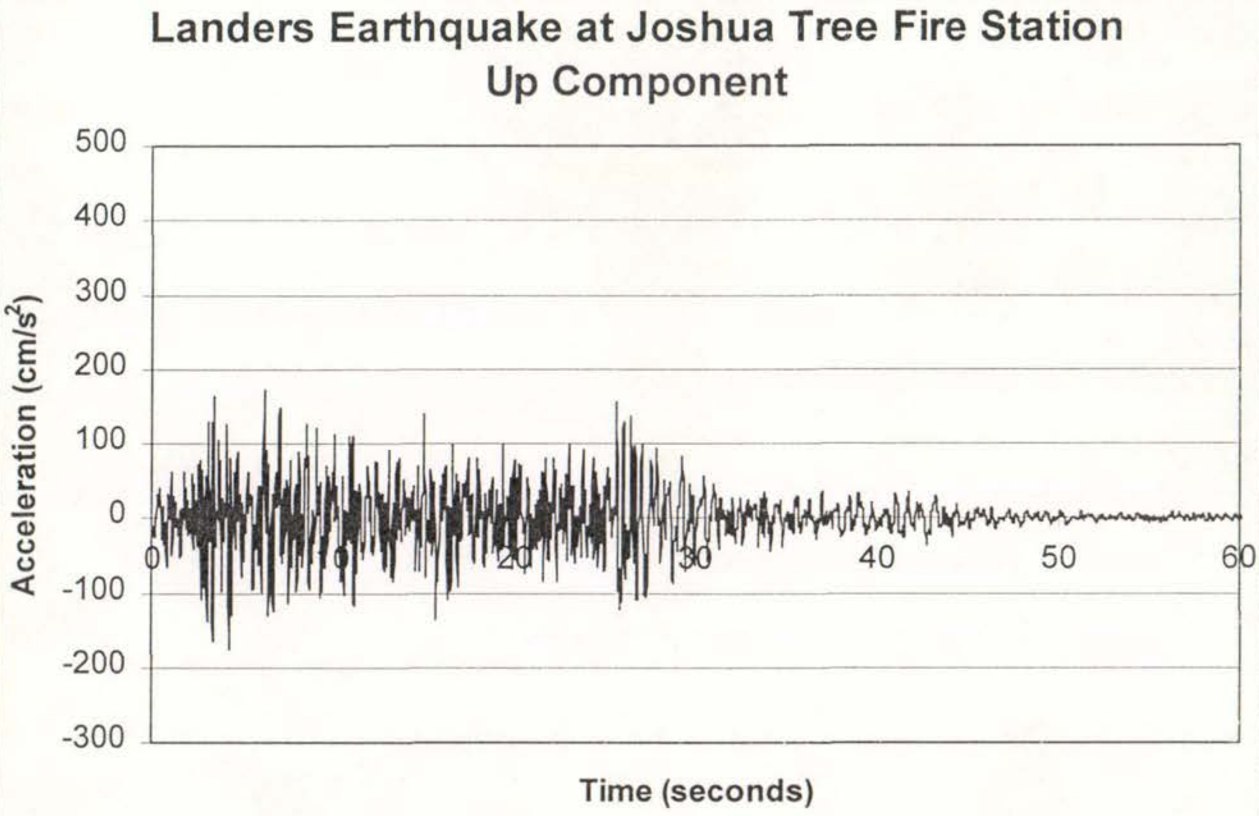


Loma Prieta Earthquake at Corralitos
Up Component

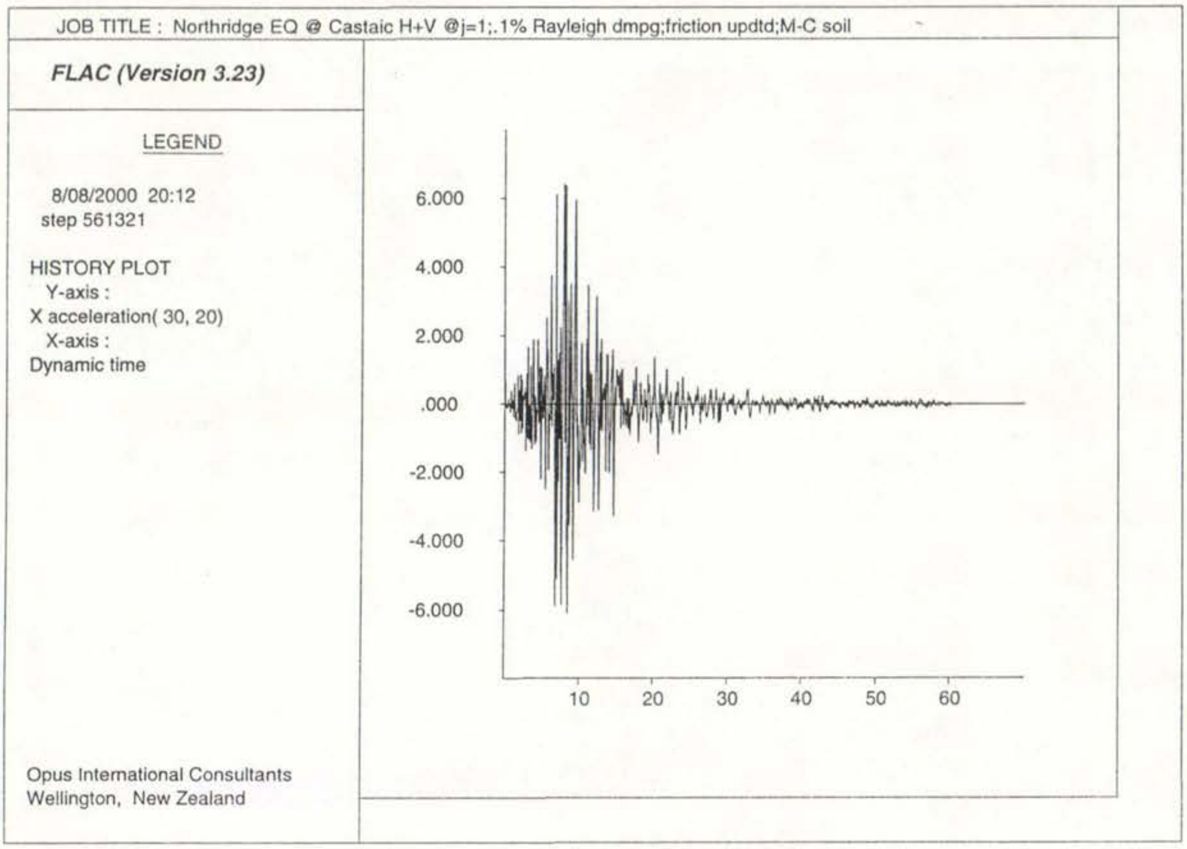


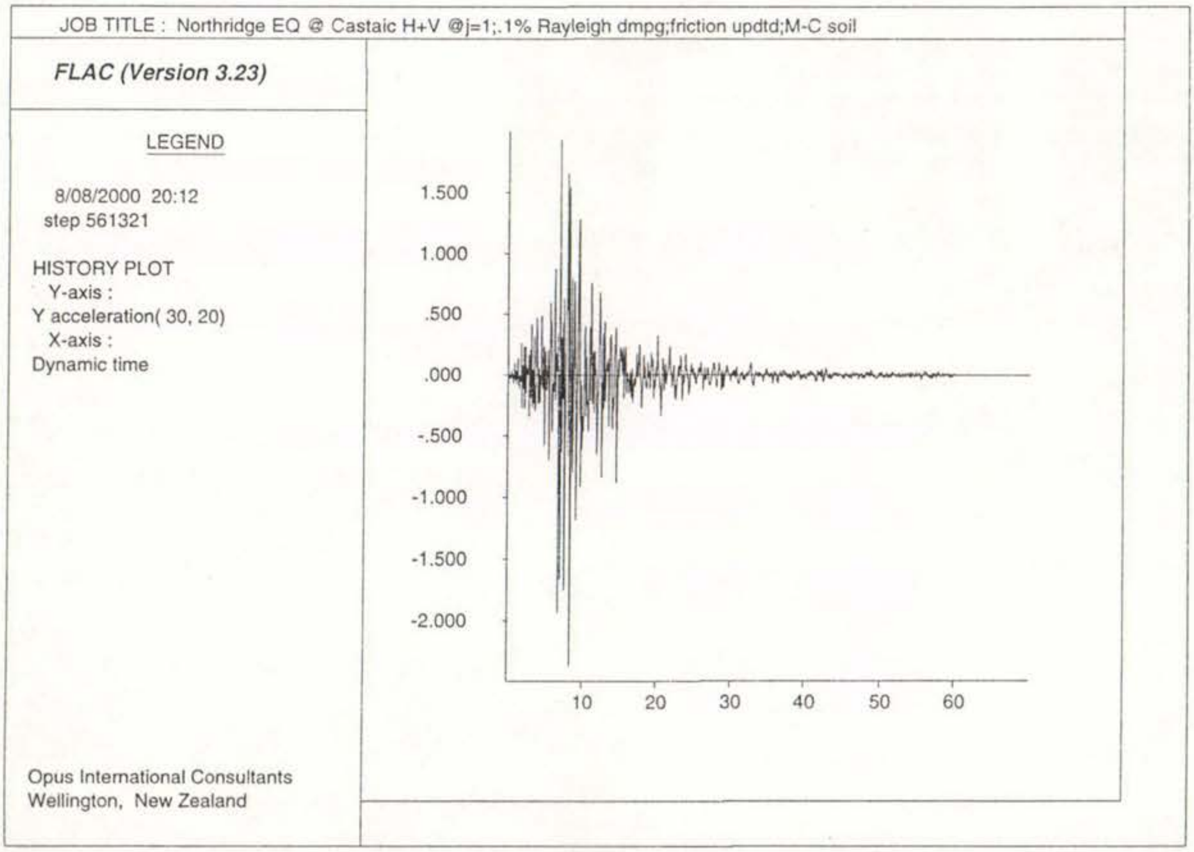
OPUS INTERNATIONAL CONSULTANTS	*Title:* Acceleration records used for the Loma Prieta Earthquake recorded at Corralitos	*Client:* Earthquake Commission		
	Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures	*Job No.:* 5C2818.00	*Date:* August 2000	*Figure:* 13



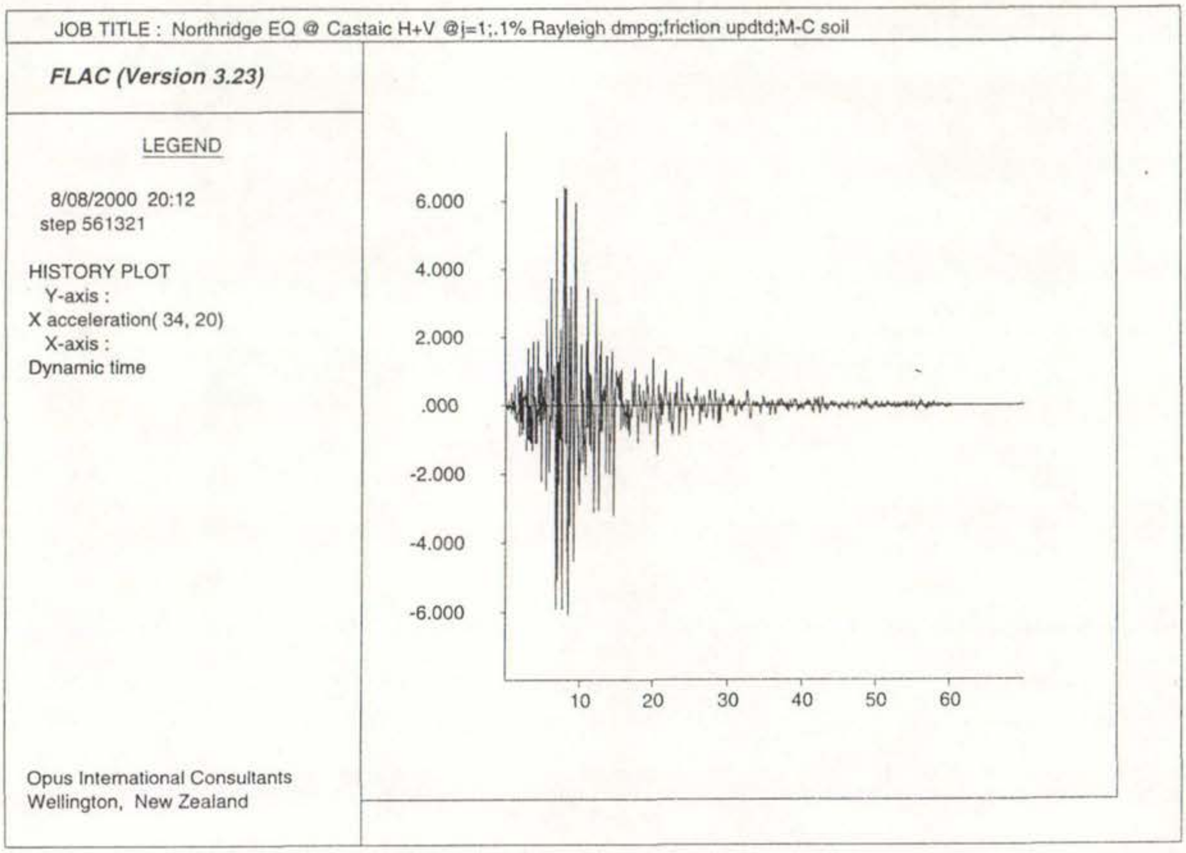


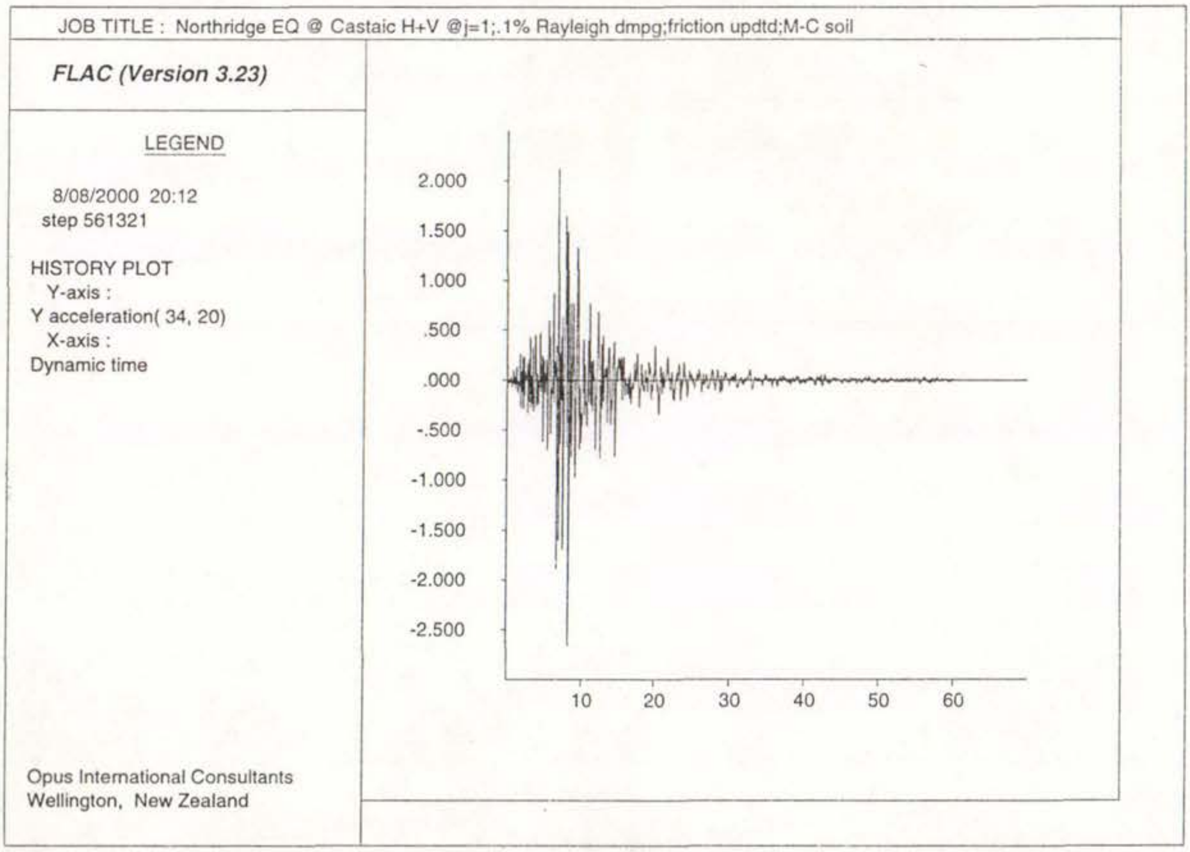
OPUS INTERNATIONAL CONSULTANTS	*Title:* Acceleration records used for the Landers Earthquake recorded at Joshua Tree fire station	*Client:* Earthquake Commission		
	Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures	*Job No.:* 5C2818.00	*Date:* August 2000	*Figure:* 14





OPUS INTERNATIONAL CONSULTANTS	*Title:* Accelerations recorded at node (30,20) for the Northridge Earthquake model	*Client:* Earthquake Commission		
	Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures	*Job No.:* 5C2818.00	*Date:* August 2000	*Figure:* 15





OPUS INTERNATIONAL CONSULTANTS	*Title:* Accelerations recorded at node (34,20) for the Northridge Earthquake model	*Client:* Earthquake Commission		
	Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures	*Job No.:* 5C2818.00	*Date:* August 2000	*Figure:* 16

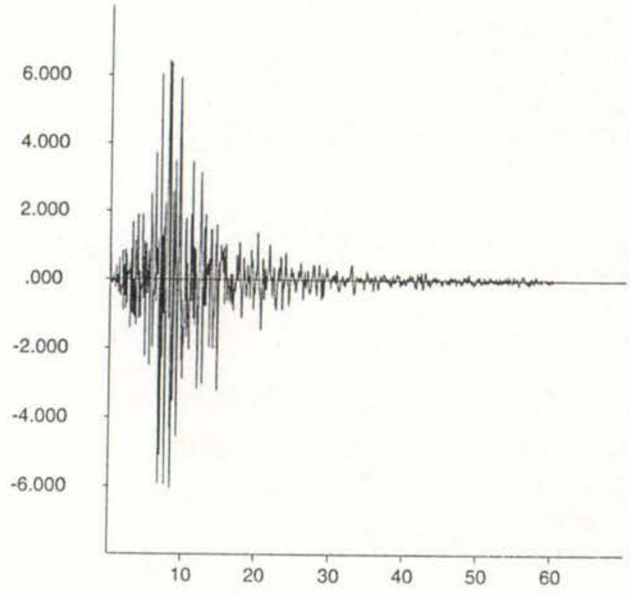
JOB TITLE : Northridge EQ @ Castaic H+V @j=1;.1% Rayleigh dmpg;friction updtd;M-C soil

FLAC (Version 3.23)

LEGEND

8/08/2000 20:12
step 561321

HISTORY PLOT
Y-axis :
X acceleration(38, 20)
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

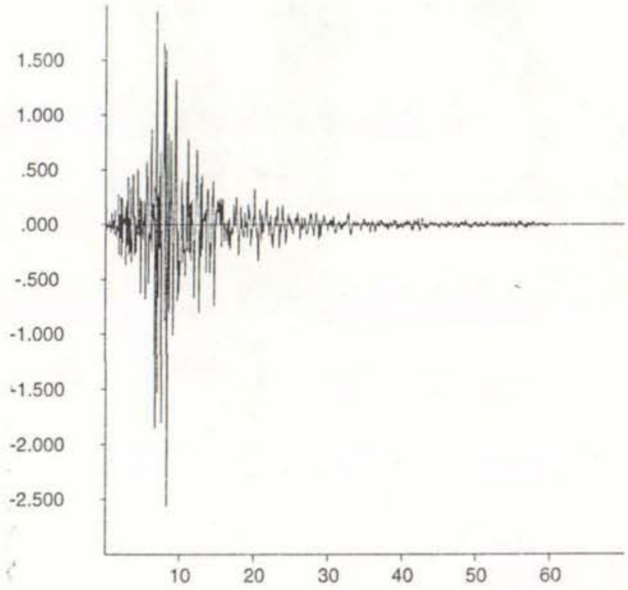
JOB TITLE : Northridge EQ @ Castaic H+V @j=1;.1% Rayleigh dmpg;friction updtd;M-C soil

FLAC (Version 3.23)

LEGEND

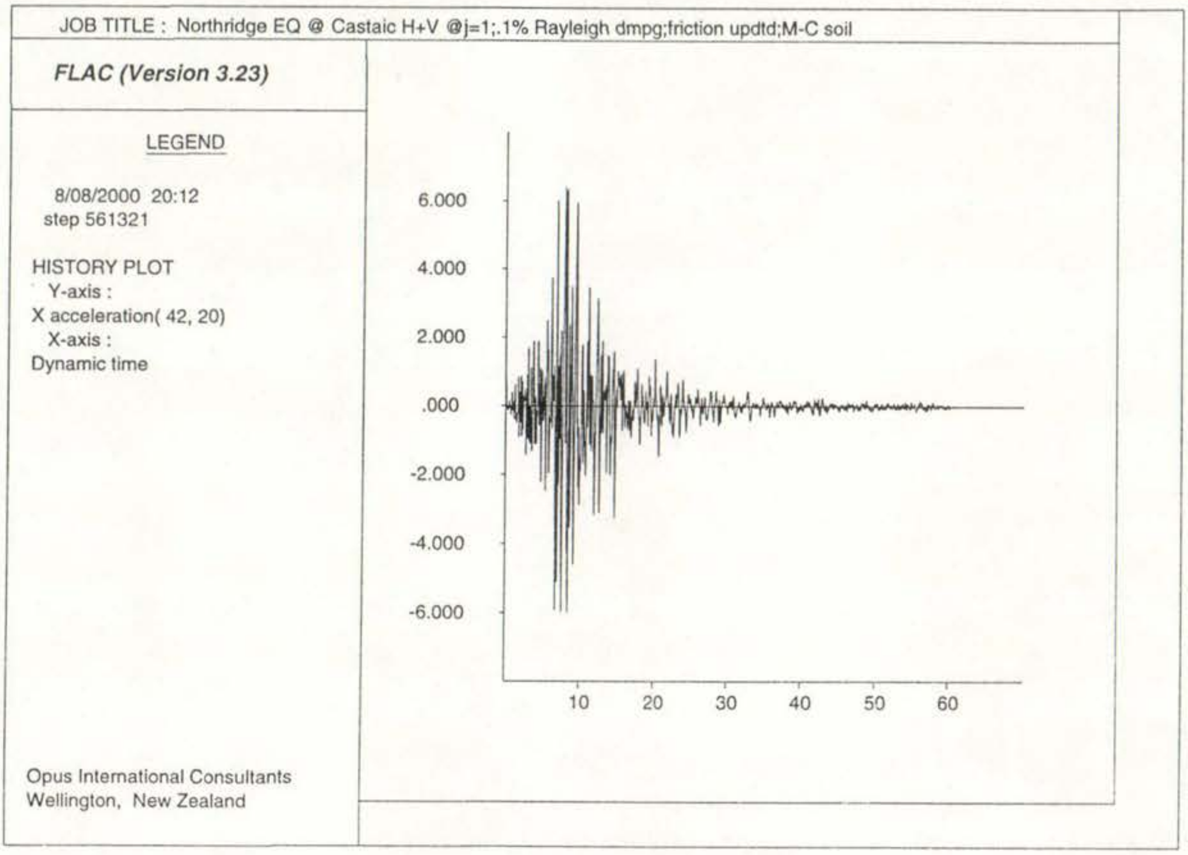
8/08/2000 20:12
step 561321

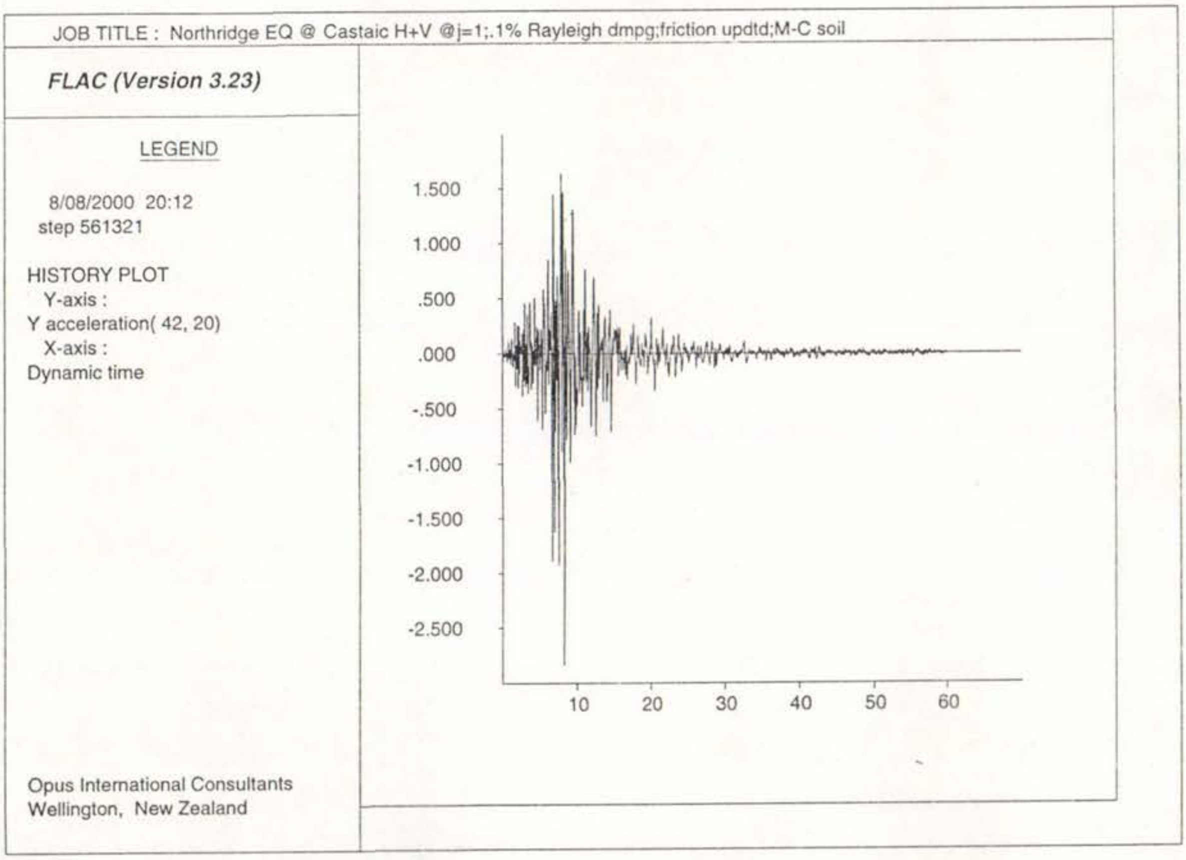
HISTORY PLOT
Y-axis :
Y acceleration(38, 20)
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

Title: Accelerations recorded at node (38,20) for the Northridge Earthquake	*Client:* Earthquake Commission			
Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures	*Job No.:* 5C2818.00	*Date:* August 2000	*Figure:* 17	





OPUS INTERNATIONAL CONSULTANTS	*Title:* Accelerations recorded at node (42,20) for the Northridge Earthquake	*Client:* Earthquake Commission		
	Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures	*Job No.:* 5C2818.00	*Date:* August 2000	*Figure:* 18

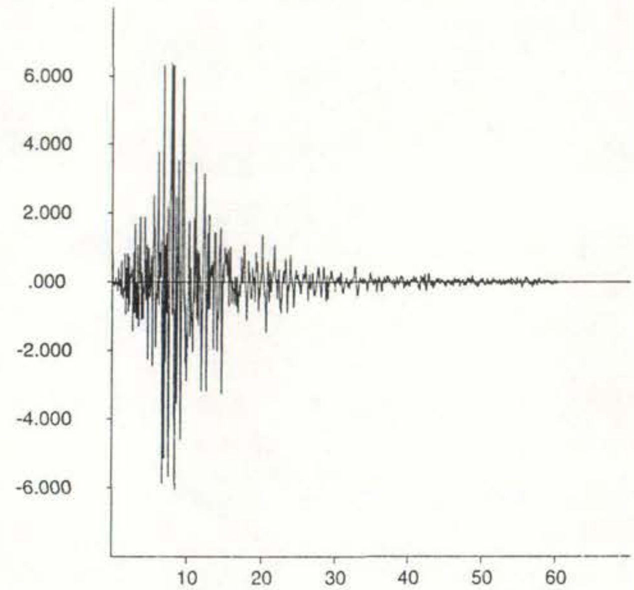
JOB TITLE : Northridge EQ @ Castaic H+V @j=1;.1% Rayleigh dmpg;friction updtd;M-C soil

FLAC (Version 3.23)

LEGEND

8/08/2000 20:12
step 561321

HISTORY PLOT
Y-axis :
X acceleration(46, 20)
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

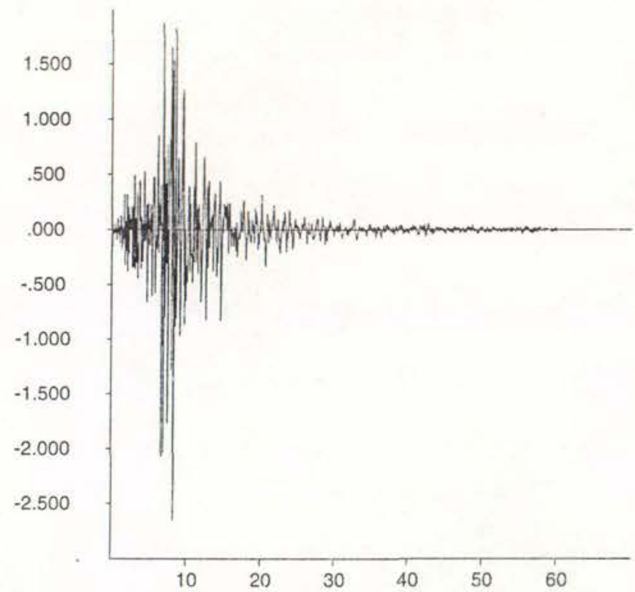
JOB TITLE : Northridge EQ @ Castaic H+V @j=1;.1% Rayleigh dmpg;friction updtd;M-C soil

FLAC (Version 3.23)

LEGEND

8/08/2000 20:12
step 561321

HISTORY PLOT
Y-axis :
Y acceleration(46, 20)
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

OPUS INTERNATIONAL CONSULTANTS	*Title:* Accelerations recorded at node (46,20) for the Northridge Earthquake	*Client:* Earthquake Commission		
	Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures	*Job No.:* 5C2818.00	*Date:* August 2000	*Figure:* 19

Title: Reinforcing strip forces in the lowest strip during the Loma Prieta earthquake

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 20

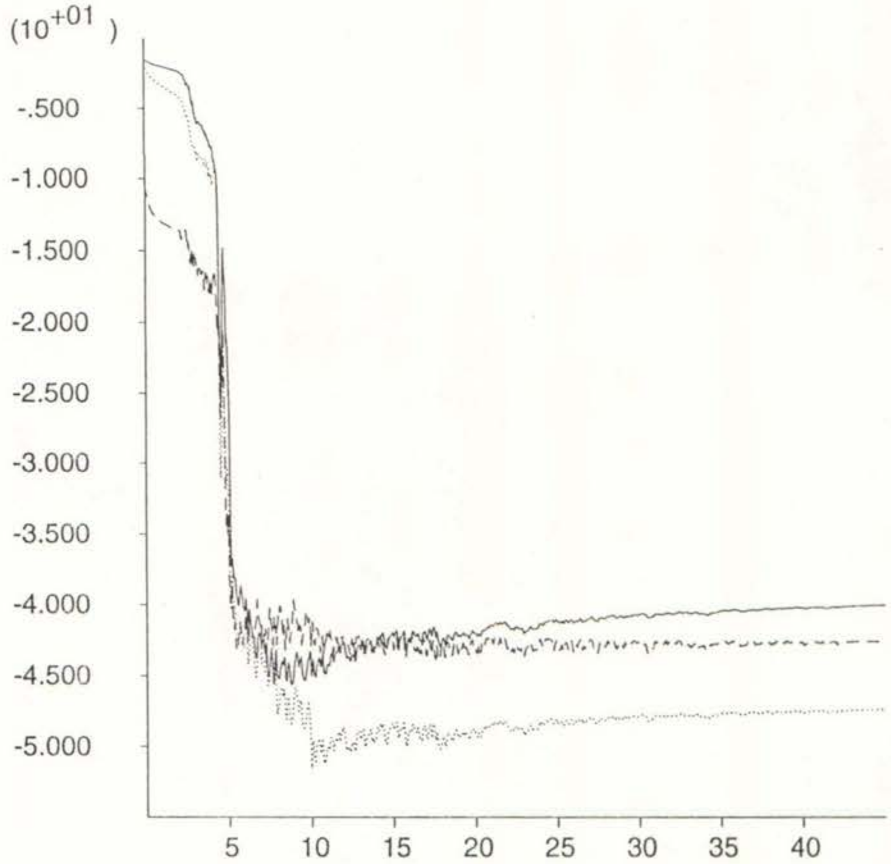
JOB TITLE : Santa Cruz Mountains EQ H+V components

FLAC (Version 3.23)

LEGEND

8/07/2000 16:18
step 400654

HISTORY PLOT
Y-axis :
Axial force in element 13 (solid line)
Axial force in element 16 (dotted line)
Axial force in element 19 (dashed line)
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

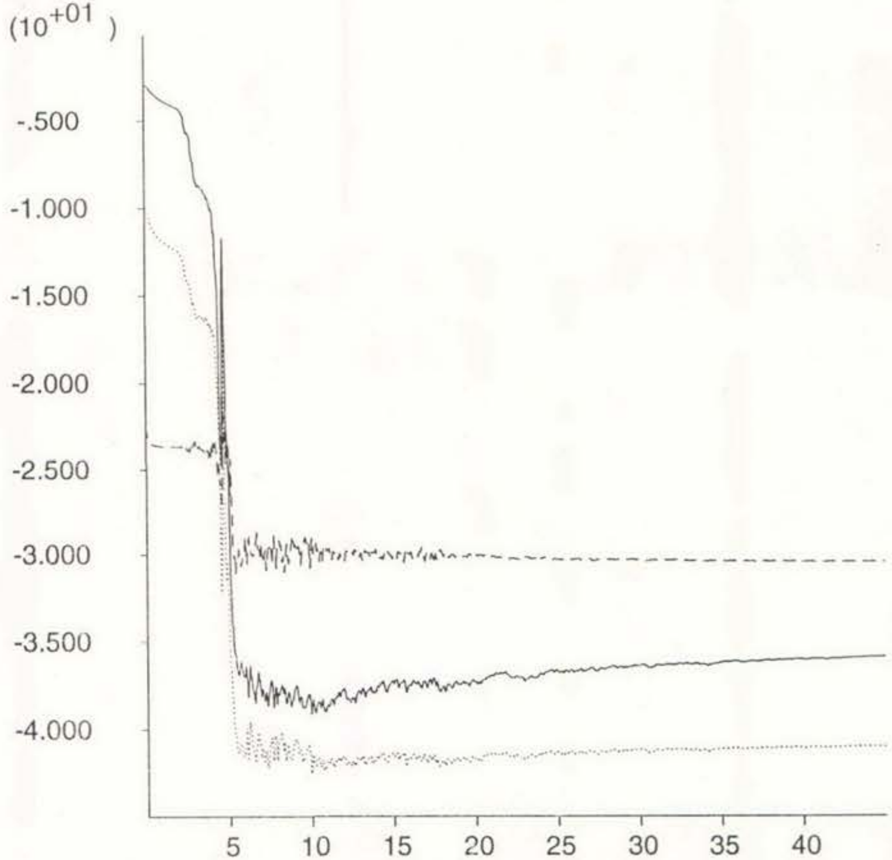
JOB TITLE : Santa Cruz Mountains EQ H+V components

FLAC (Version 3.23)

LEGEND

8/07/2000 16:18
step 400654

HISTORY PLOT
Y-axis :
Axial force in element 28
Axial force in element 31
Axial force in element 34
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

OPUS INTERNATIONAL CONSULTANTS

Title: Reinforcing strip forces in the second-to-lowest strip during the Loma Prieta earthquake

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 21

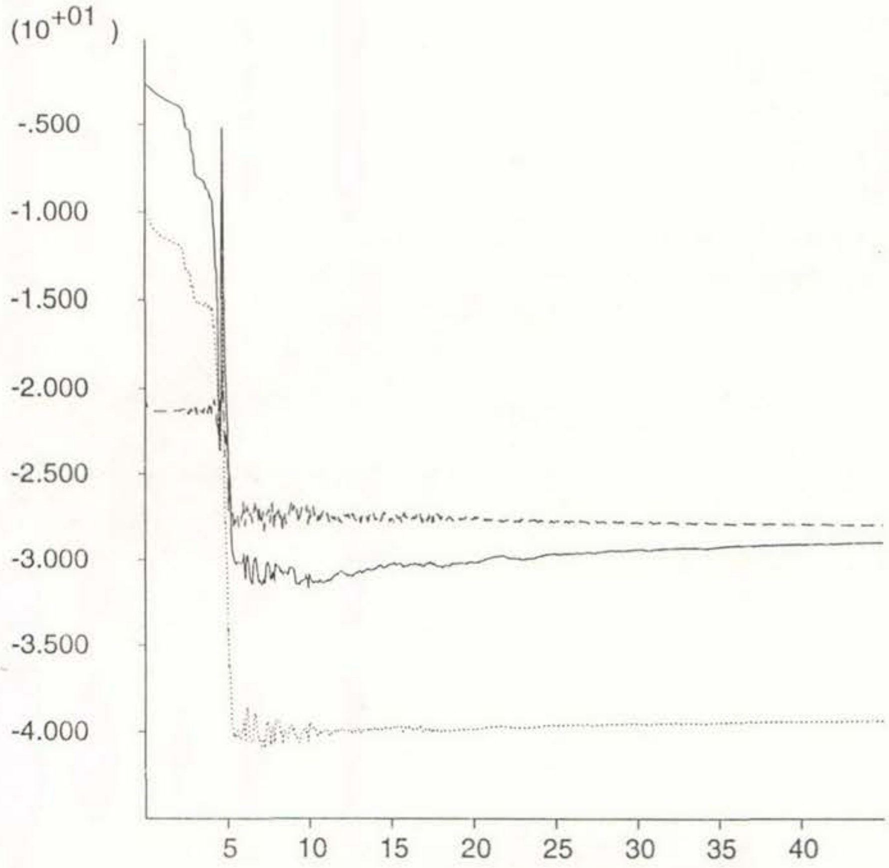
JOB TITLE : Santa Cruz Mountains EQ H+V components

FLAC (Version 3.23)

LEGEND

8/07/2000 16:18
step 400654

HISTORY PLOT
Y-axis :
Axial force in element 45 (solid line)
Axial force in element 49 (dotted line)
Axial force in element 53 (dashed line)
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

OPUS INTERNATIONAL CONSULTANTS

Title: Reinforcing strip forces in the third-to-lowest strip during the Loma Prieta earthquake

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 22

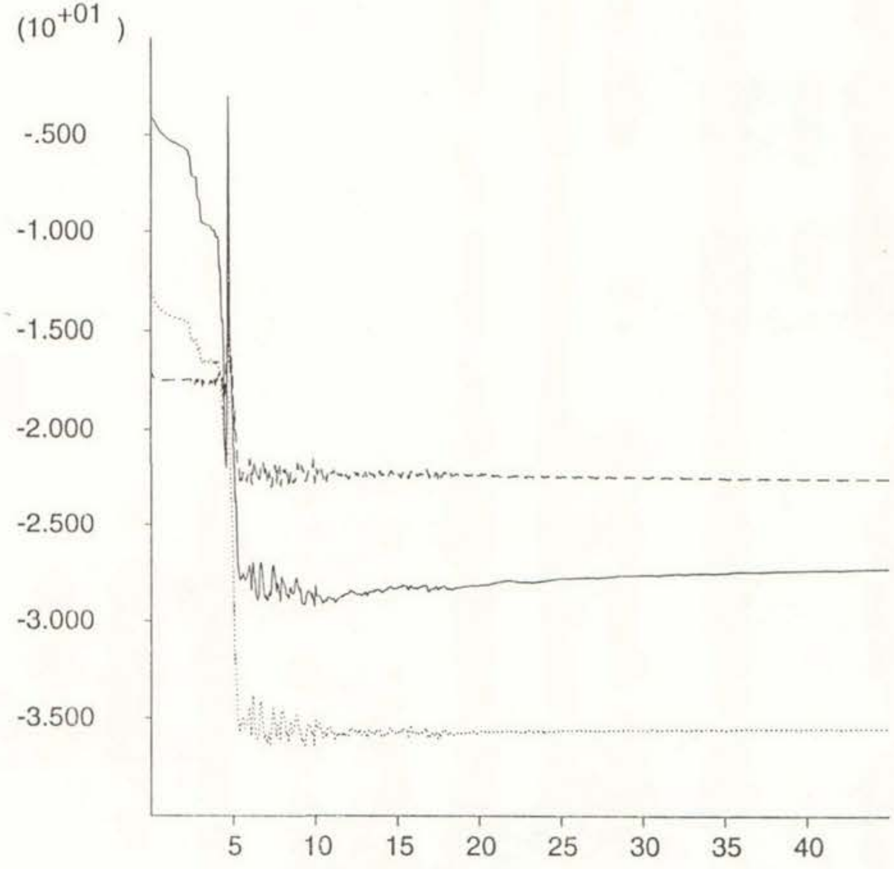
JOB TITLE : Santa Cruz Mountains EQ H+V components

FLAC (Version 3.23)

LEGEND

8/07/2000 16:18
step 400654

HISTORY PLOT
Y-axis :
Axial force in element 60
Axial force in element 64
Axial force in element 68
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

OPUS INTERNATIONAL CONSULTANTS

Title: Reinforcing strip forces in the fourth strip during the Loma Prieta earthquake

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 23

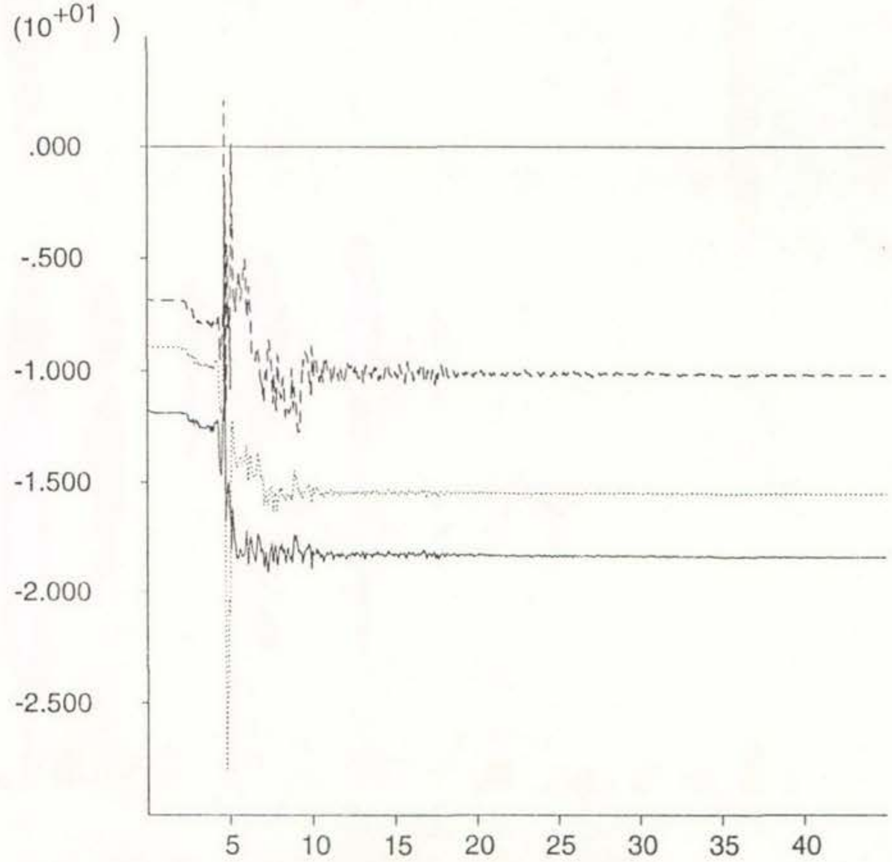
JOB TITLE : Santa Cruz Mountains EQ H+V components

FLAC (Version 3.23)

LEGEND

8/07/2000 16:18
step 400654

HISTORY PLOT
Y-axis :
Axial force in element 102
Axial force in element 121
Axial force in element 136
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

Title: Reinforcing strip forces adjacent to the facing in the sixth to eighth strips during the Loma Prieta earthquake

Client: Earthquake Commission

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Job No.: 5C2818.00

Date: August 2000

Figure: 24

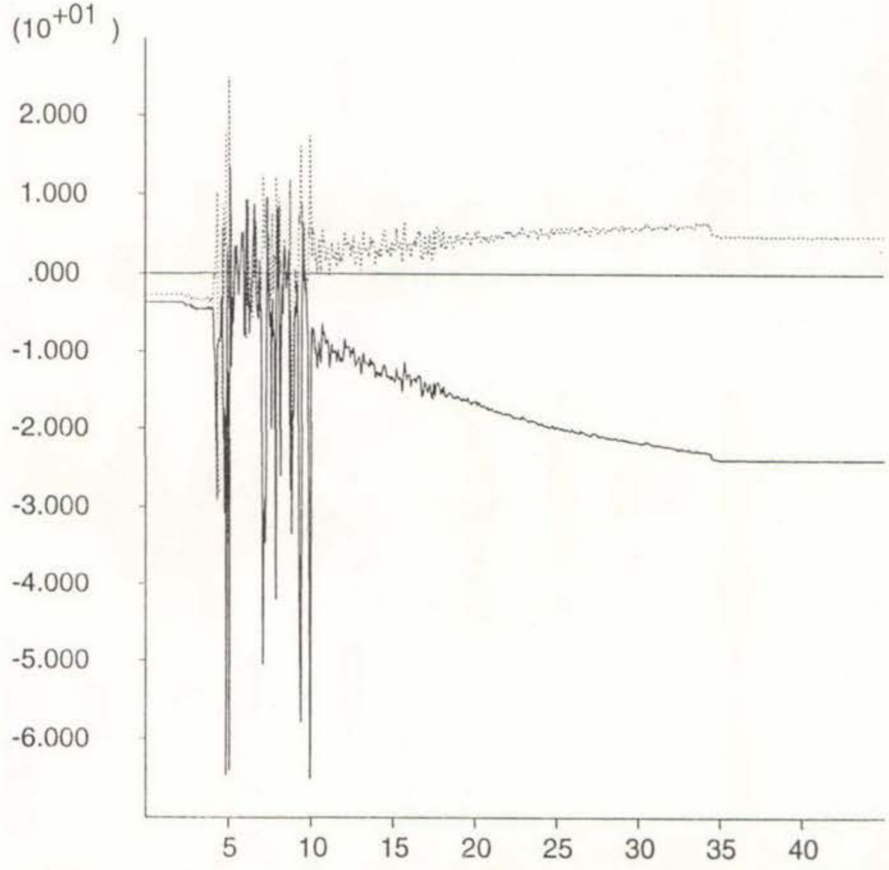
JOB TITLE : Santa Cruz Mountains EQ H+V components

FLAC (Version 3.23)

LEGEND

8/07/2000 16:18
step 400654

HISTORY PLOT
Y-axis :
Axial force in element 164
Axial force in element 188
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

Title: Reinforcing strip forces adjacent to the facing in the ninth and top strips during the Loma Prieta earthquake

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 25

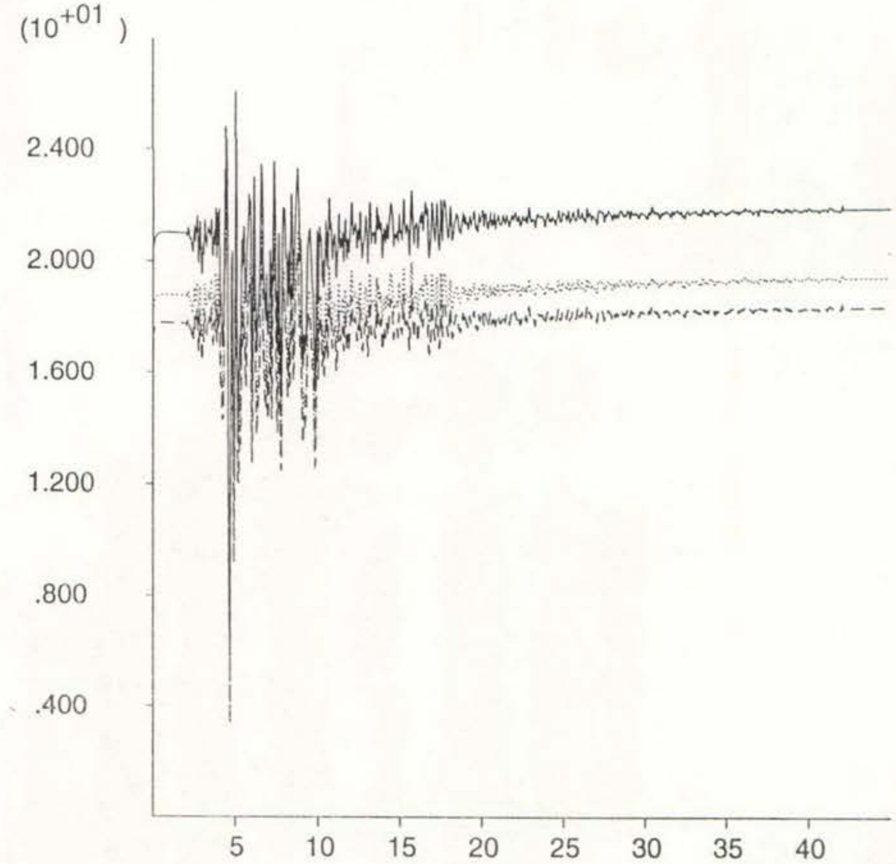
JOB TITLE : Santa Cruz Mountains EQ H+V components

FLAC (Version 3.23)

LEGEND

8/07/2000 16:18
step 400654

HISTORY PLOT
Y-axis :
Friction in Strip 1
Friction in Strip 2
Friction in Strip 3
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

Title: Histories of S_{bond} in the lower three strips during the Loma Prieta earthquake

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 26

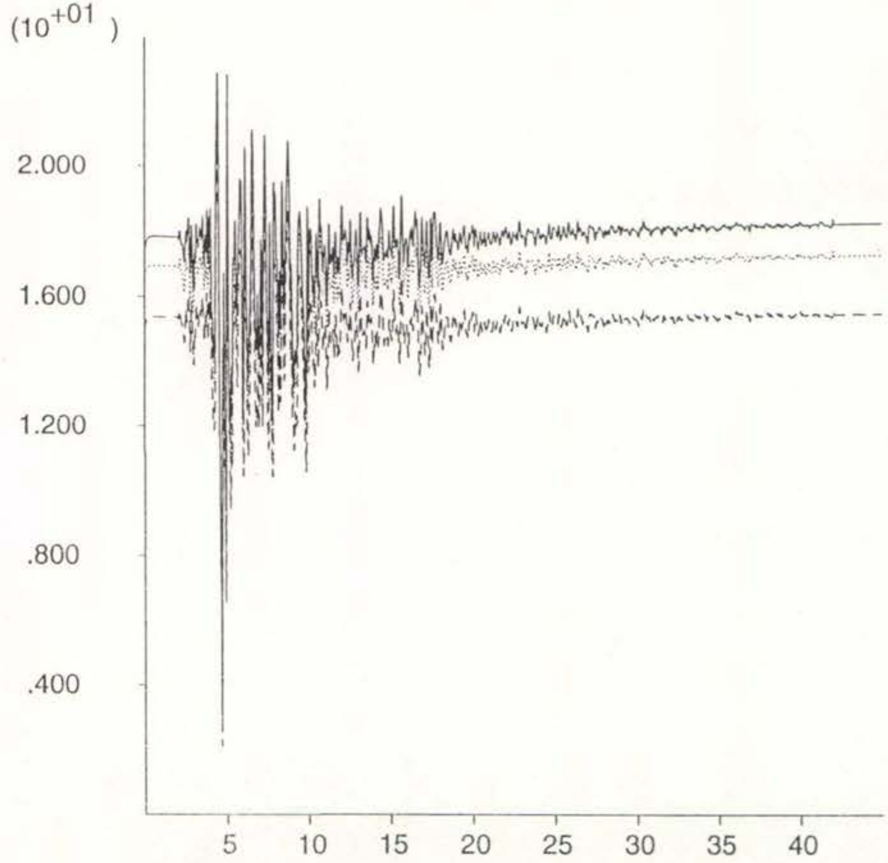
JOB TITLE : Santa Cruz Mountains EQ H+V components

FLAC (Version 3.23)

LEGEND

8/07/2000 16:18
step 400654

HISTORY PLOT
Y-axis :
Friction in Strip 4
Friction in Strip 5
Friction in Strip 6
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

OPUS INTERNATIONAL CONSULTANTS

Title: Histories of S_{bond} in the fourth to sixth strips during the Loma Prieta earthquake

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 27

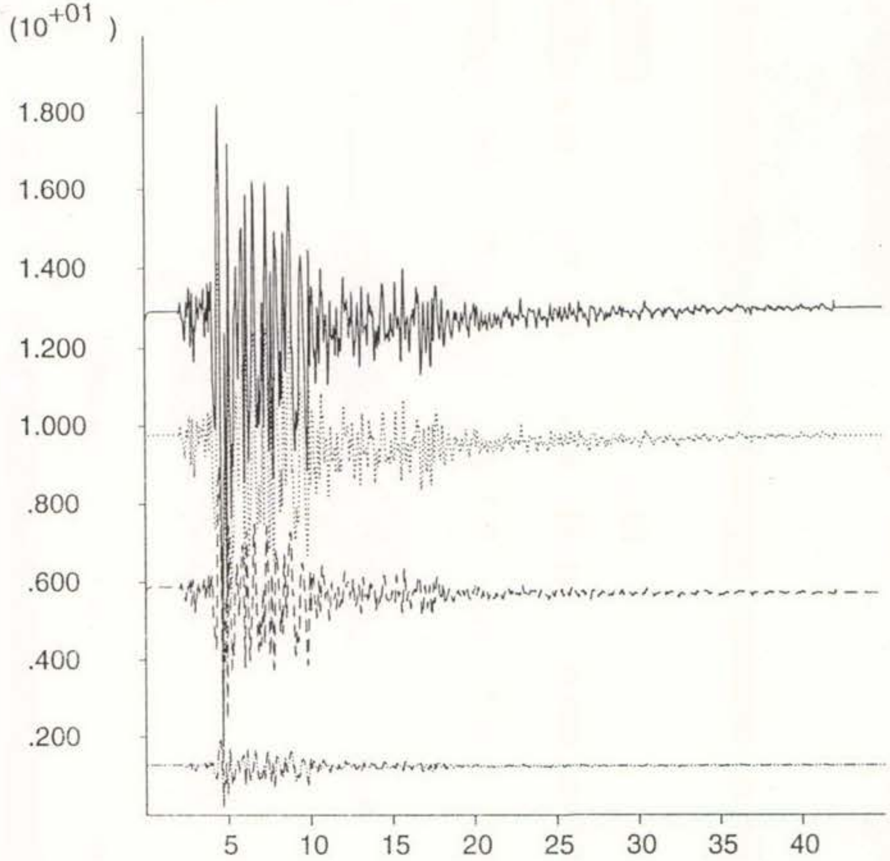
JOB TITLE : Santa Cruz Mountains EQ H+V components

FLAC (Version 3.23)

LEGEND

8/07/2000 16:18
step 400654

HISTORY PLOT
Y-axis :
Friction in Strip 7
Friction in Strip 8
Friction in Strip 9
Friction in Strip 10
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

Title: Histories of S_{bond} in the seventh to top strips during the Loma Prieta earthquake

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 28

JOB TITLE : Completed wall showing reinforcement and facing. Displacement history locations

FLAC (Version 3.23)

LEGEND

8/11/2000 12:33
step 23617
-1.151E+01 <x< 9.199E-01
4.927E+01 <y< 6.169E+01

Grid plot

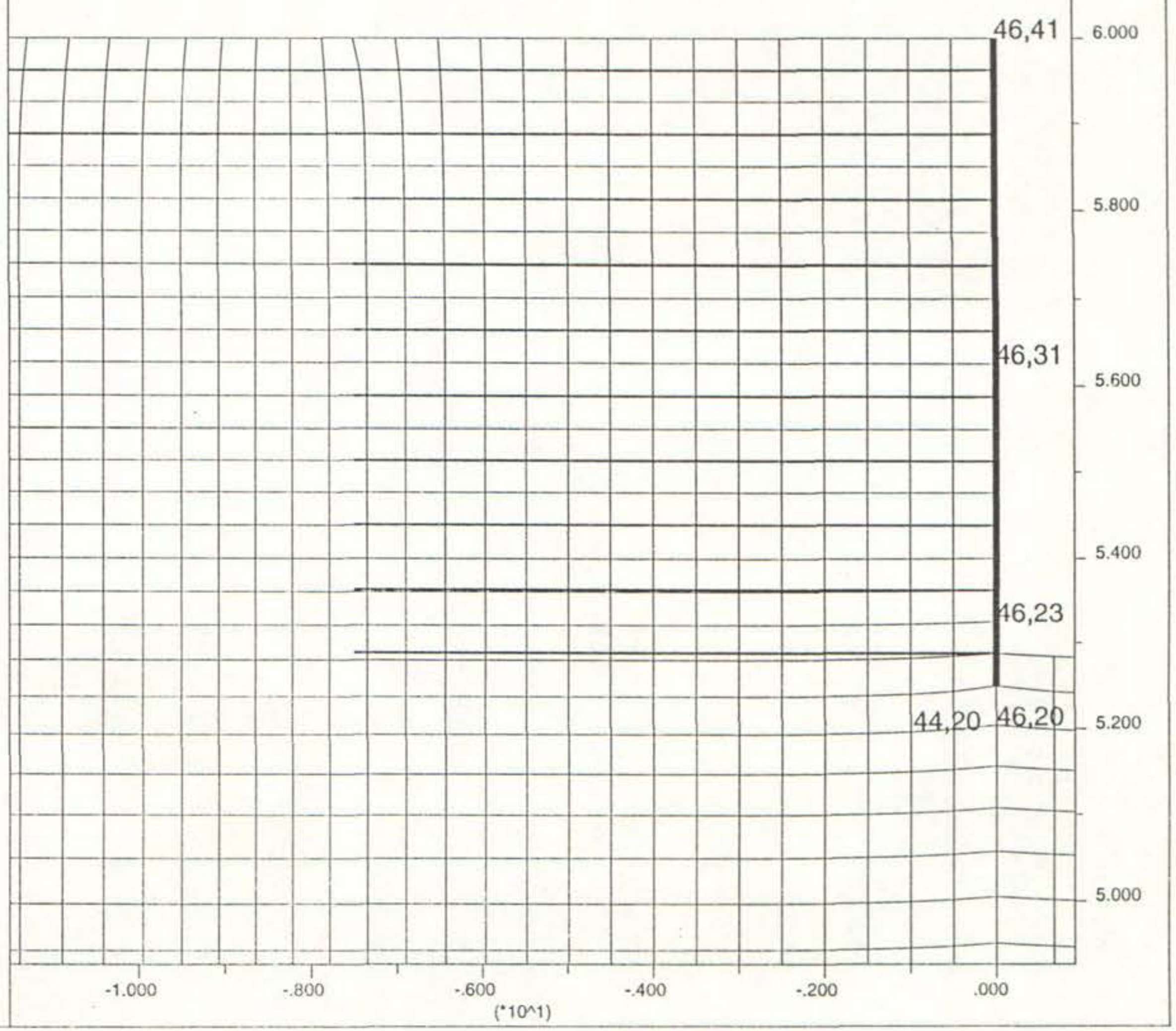
0 2E 0

Cable plot

Beam plot

Displacement History Locations

Opus International Consultants
Wellington, New Zealand



OPUS INTERNATIONAL CONSULTANTS

Title: Locations of displacement histories

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 29

Title: A typical total displacement history

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 30

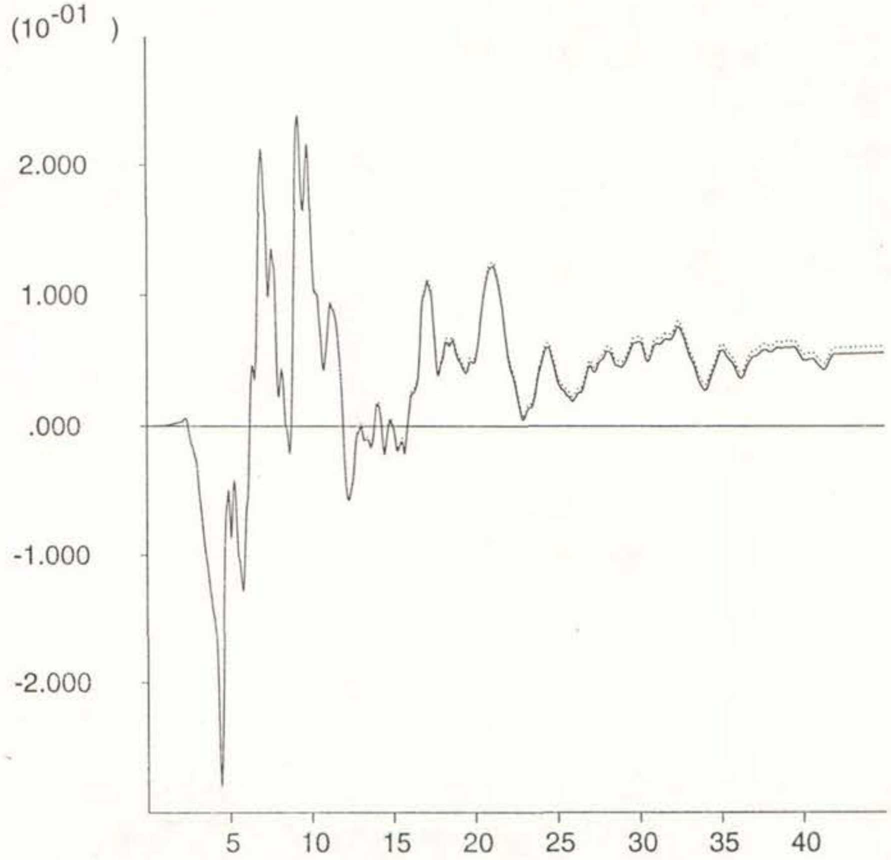
JOB TITLE : Santa Cruz Mountains EQ H+V components

FLAC (Version 3.23)

LEGEND

8/07/2000 16:18
step 400654

HISTORY PLOT
Y-axis :
X displacement(46, 23)
X displacement(46, 31)
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

JOB TITLE : Landers EQ H+V; .1% Rayleigh dmpg; correct friction updtd; M-C soil; large acceln

FLAC (Version 3.23)

LEGEND

8/11/2000 11:32
step 711820
-3.215E+01 <x< 5.008E+01
8.847E+00 <y< 9.107E+01

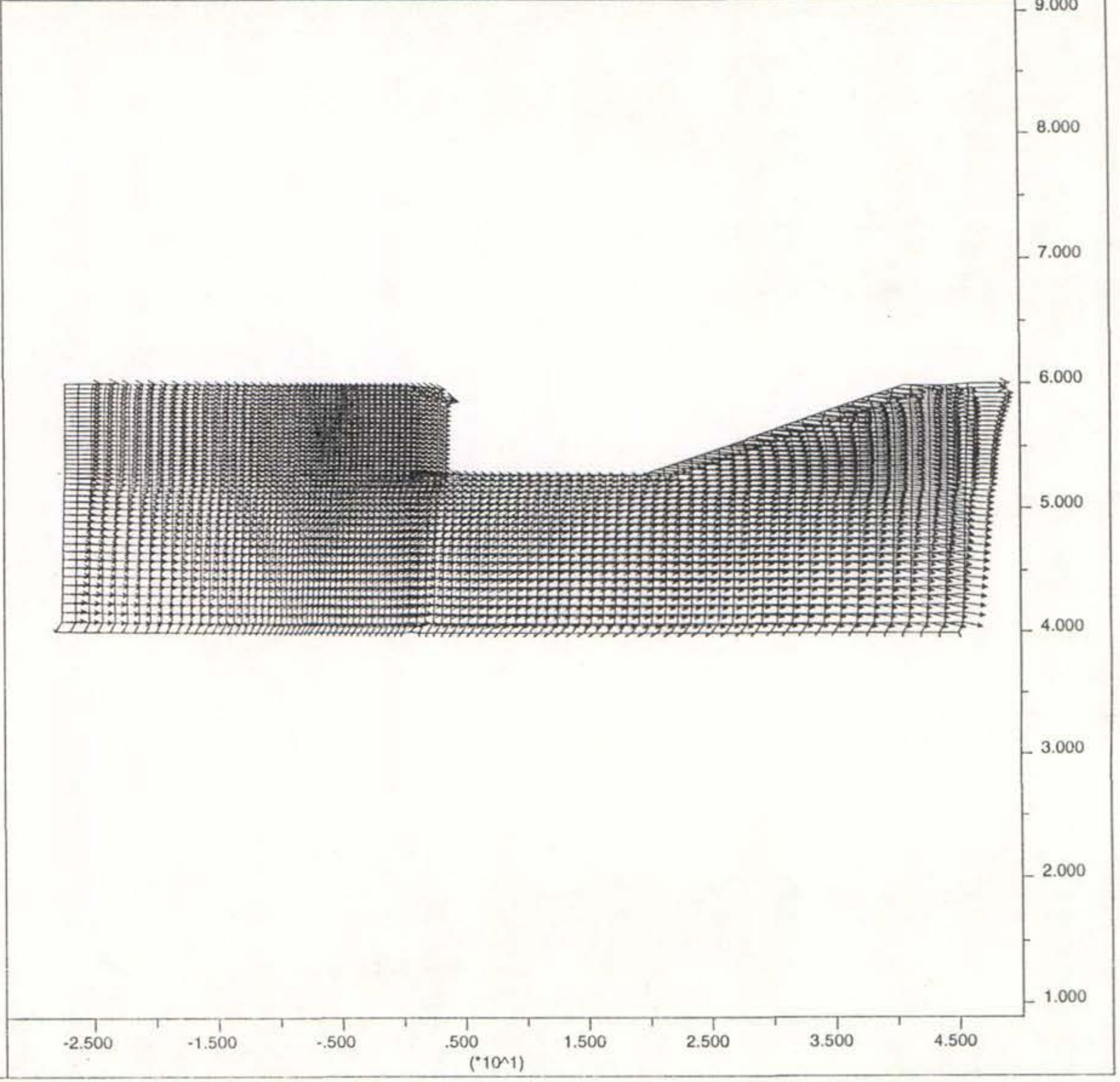
Grid plot

0 2E 1

Displacement vectors
Scaled to Max = 2.000E+00
Max Vector = 1.000E+00

0 5E 0

Opus International Consultants
Wellington, New Zealand



OPUS INTERNATIONAL CONSULTANTS

Title: A typical end-of-earthquake displacement vector field

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 31

JOB TITLE : Santa Cruz Mountains EQ H+V components

FLAC (Version 3.23)

LEGEND

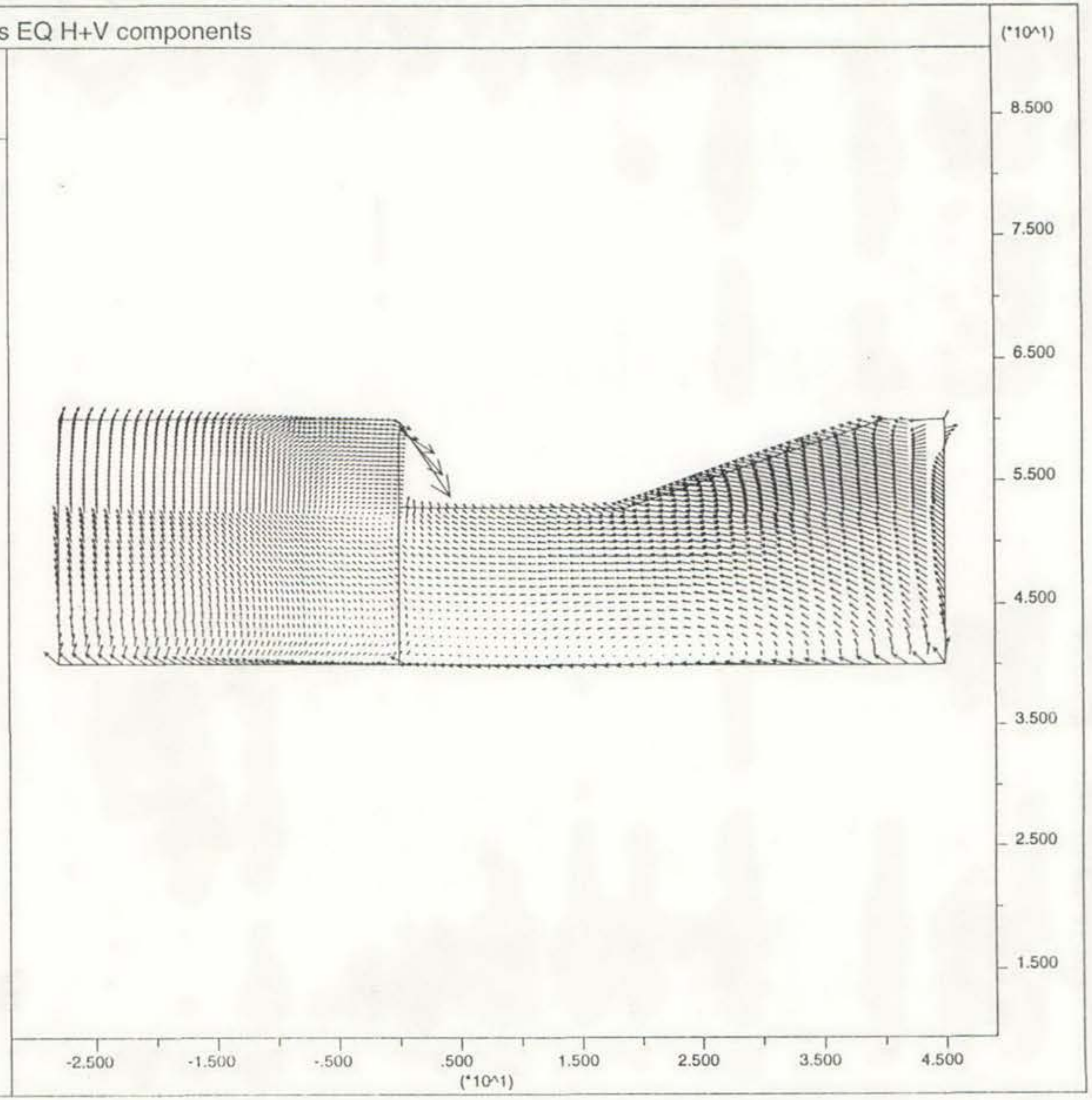
8/07/2000 16:28
step 400654
-3.205E+01 <x< 4.913E+01
9.395E+00 <y< 9.058E+01

Boundary plot

0 2E 1

Net displacements
Max Vector = 2.343E-01

0 5E -1



Opus International Consultants
Wellington, New Zealand

OPUS INTERNATIONAL CONSULTANTS

Title: End-of-earthquake net displacement vectors for the Loma Prieta earthquake horizontal plus vertical simulation

Client: Earthquake Commission

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Job No.:	*Date:*	*Figure:*
5C2818.00	August 2000	32

JOB TITLE : Northridge EQ H+V; .1% Rayleigh dmpg; friction updtd; M-C soil. Net displacements.

FLAC (Version 3.23)

LEGEND

8/11/2000 13:28
step 561321
-3.206E+01 <x< 4.928E+01
9.352E+00 <y< 9.069E+01

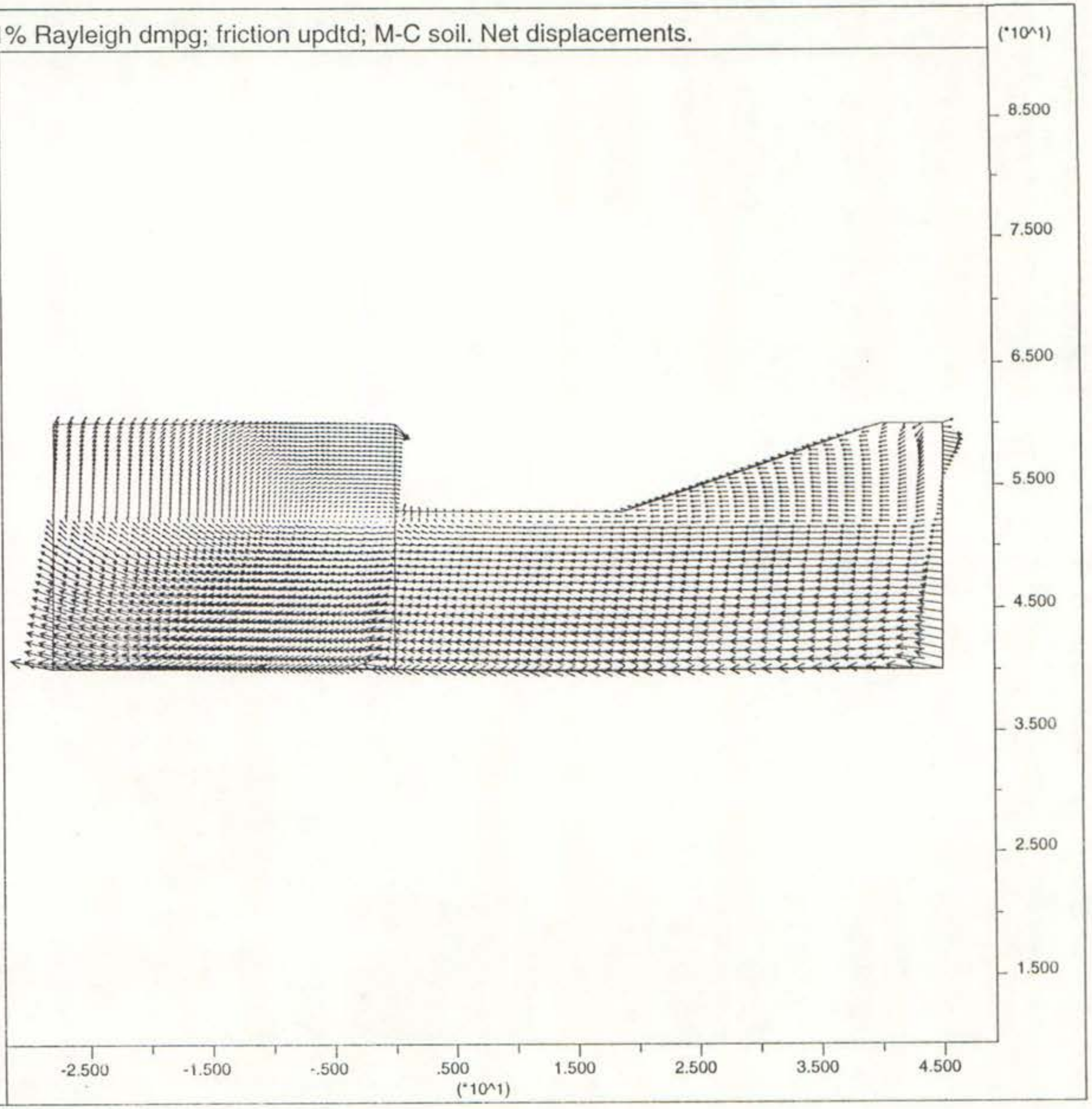
Boundary plot

0 2E 1

Net Displacement Vectors
Scaled to Max = 3.000E-01
Max Vector = 1.500E-01

0 1E 0

Opus International Consultants
Wellington, New Zealand



Title: End-of-earthquake net displacement vectors for the Northridge earthquake horizontal plus vertical simulation

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

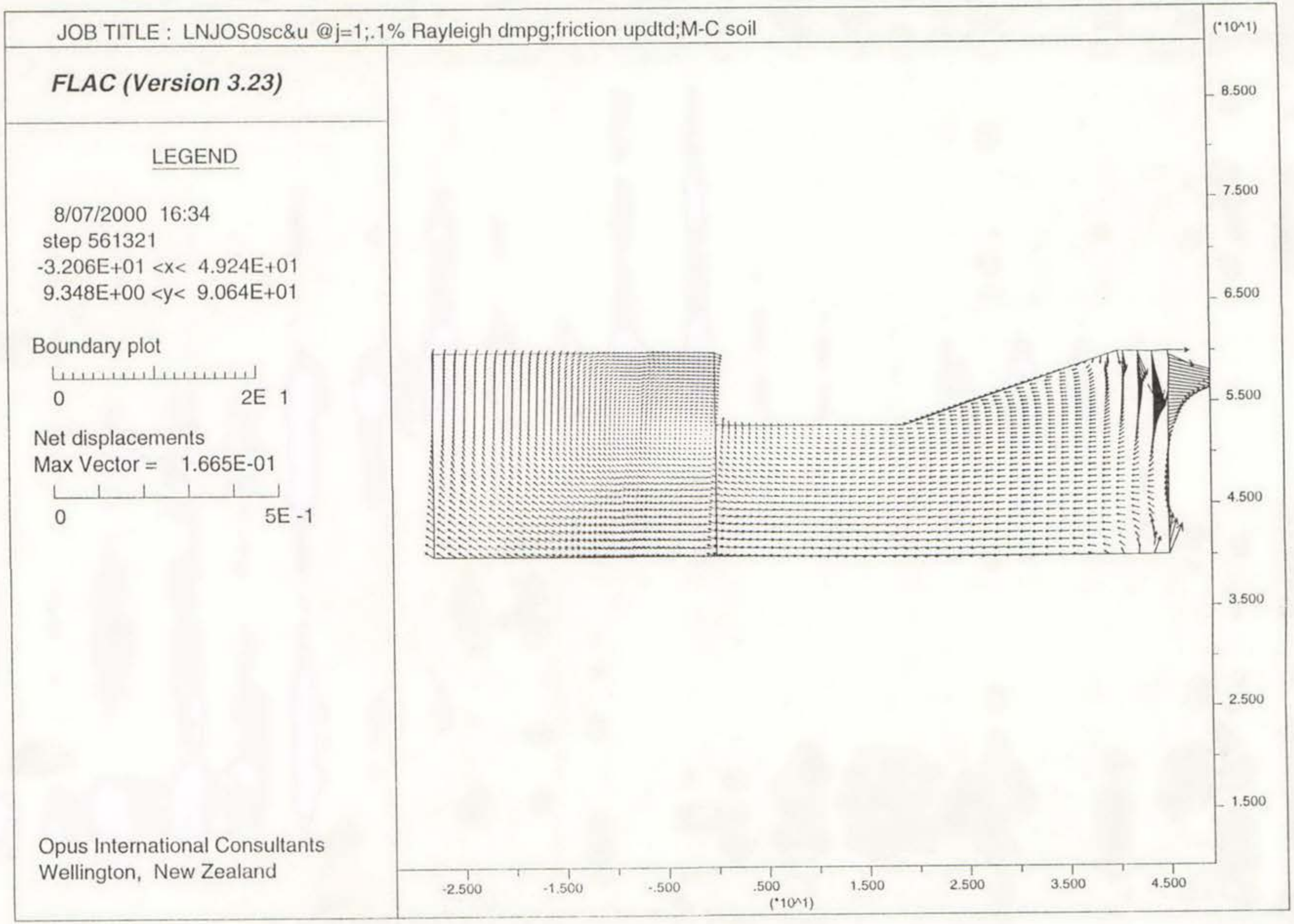
Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 33

OPUS INTERNATIONAL CONSULTANTS



Title: End-of-earthquake net displacement vectors for the Landers earthquake horizontal plus vertical simulation

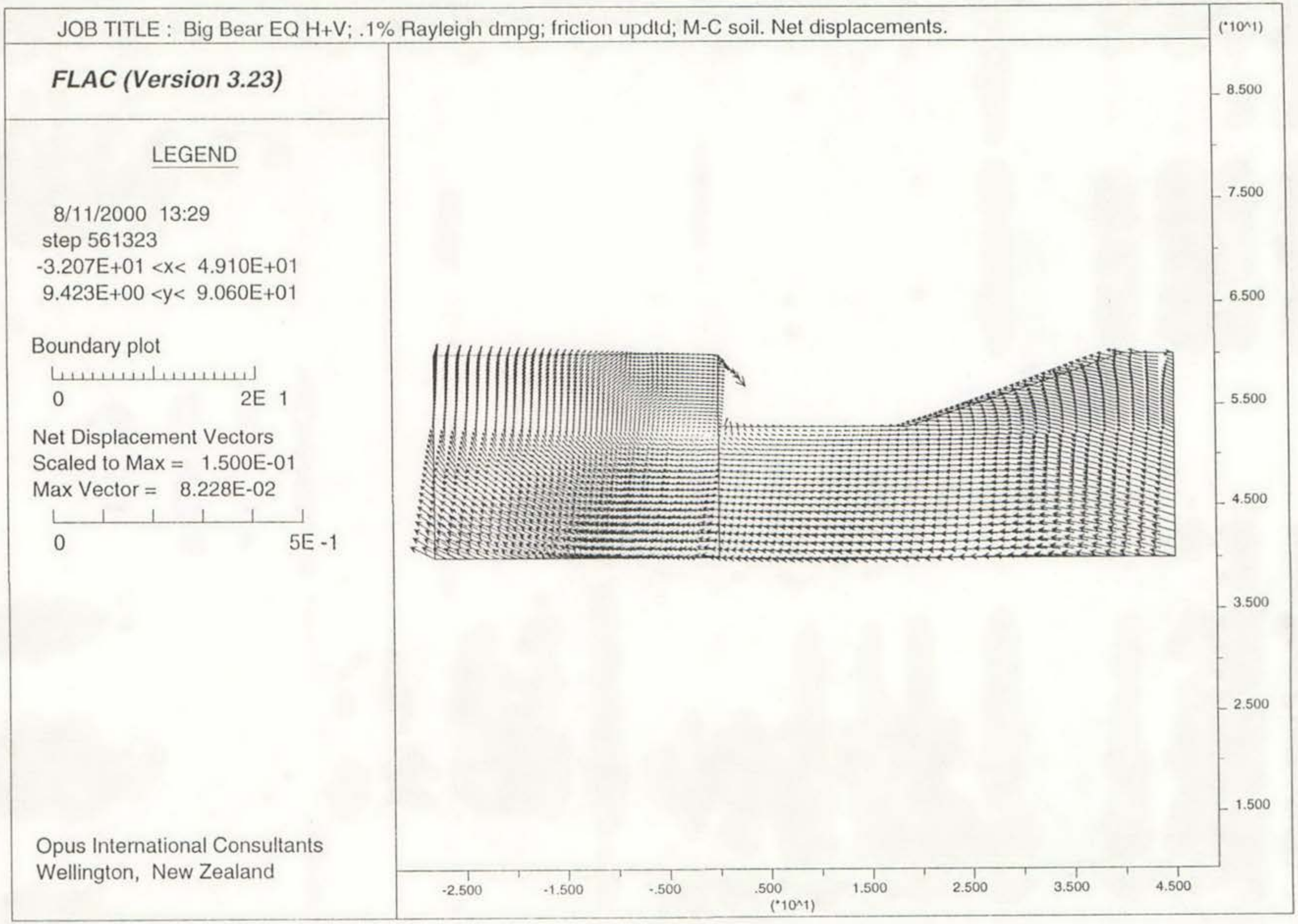
Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 34



OPUS

Title: End-of-earthquake net displacement vectors for the Big Bear earthquake horizontal plus vertical simulation

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 35

JOB TITLE : Completed wall showing reinforcement and facing. Net displacement history locations

FLAC (Version 3.23)

LEGEND

8/11/2000 12:33
step 23617
-1.151E+01 <x< 9.199E-01
4.927E+01 <y< 6.169E+01

Grid plot

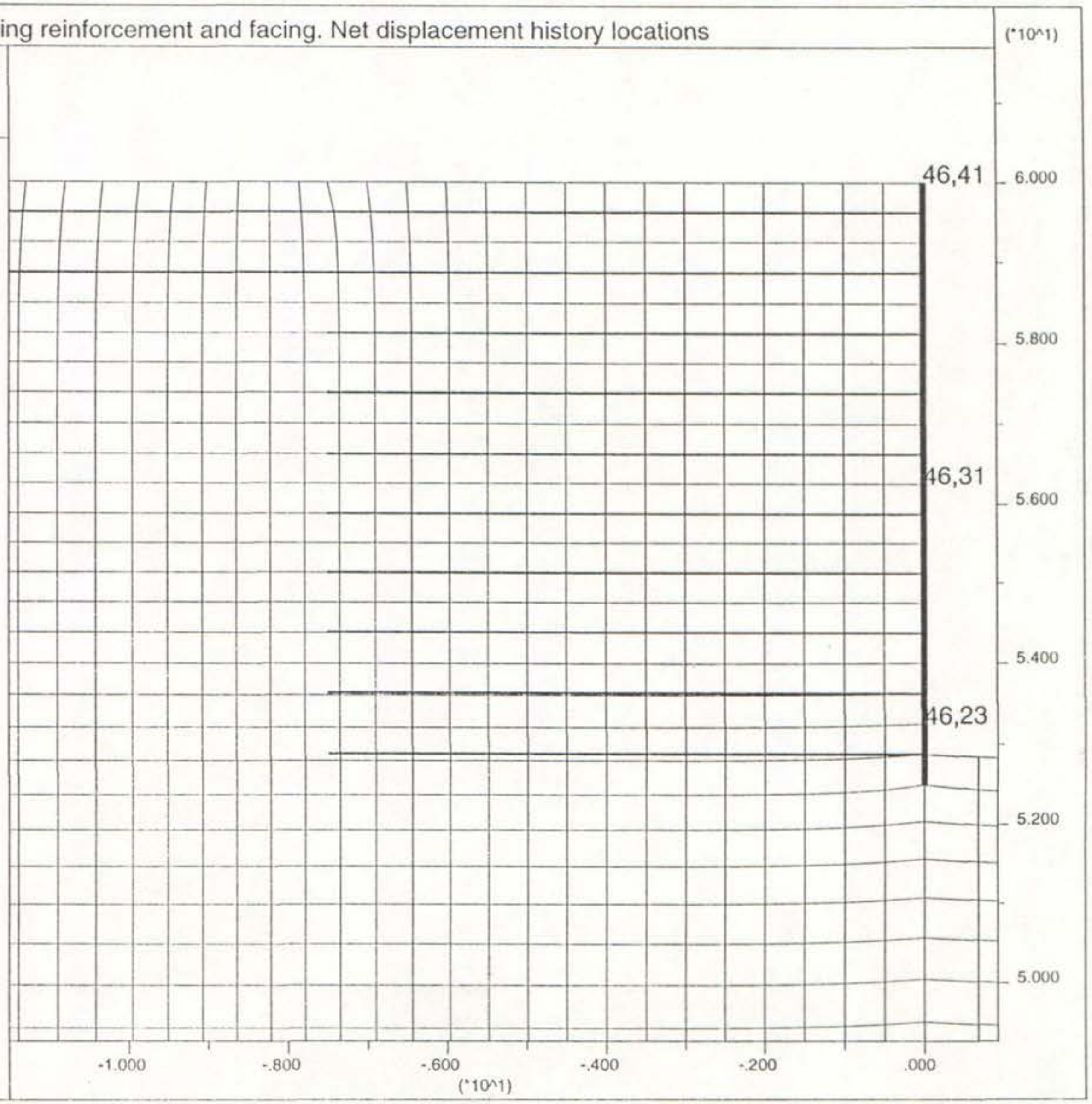
0 2E 0

Cable plot

Beam plot

Net Displacement History Locations

Opus International Consultants
Wellington, New Zealand



OPUS INTERNATIONAL CONSULTANTS

Title: Locations of histories of net displacement

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 36

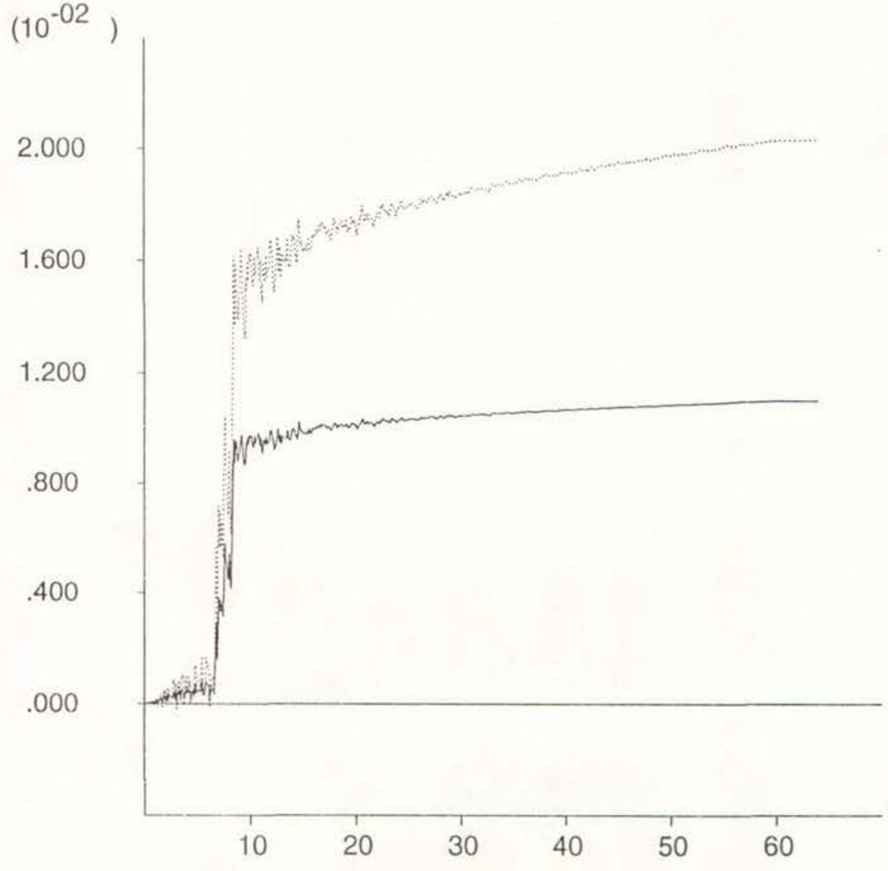
JOB TITLE : Northridge EQ H+V; .1% Rayleigh dmpg; friction updated; M-C soil

FLAC (Version 3.23)

LEGEND

8/11/2000 13:55
step 561321

HISTORY PLOT
Y-axis :
Net x-displacement at (46,23)
Net x-displacement at (46,31)
X-axis :
Dynamic time



Opus International Consultants
Wellington, New Zealand

Title: Typical net displacement histories

Project: Effect of Vertical Shaking on the Earthquake Displacement of Retaining Structures

Client: Earthquake Commission

Job No.: 5C2818.00

Date: August 2000

Figure: 37

Appendix A

Difficulties in the Model Analysis

Appendix A

Difficulties in the Model Analyses

Several difficulties were encountered during the running of the models using the input records. These have required the model to be adjusted to obtain results that can be used with confidence. A brief discussion of these issues is presented below.

Model Displacement

The entire model displaced during shaking, apparently no matter how good the baseline correction of the input record. This was seen as a severe problem for a considerable time. Initially the model was fixed in the x- and y-directions, but this is not permissible because the quiet and free-field boundaries, which rely on movement to absorb energy in the dashpots, are then ineffective. Therefore, it was decided to accept the overall model displacement and correct for it as necessary, i.e. to calculate net displacements. Baseline correction is then able to be less rigorous.

While holding the boundaries fixed in x and y, difficulties were experienced inputting the earthquake. It was found that when feeding in vertical records, the zones immediately above the input grew taller, resulting in the entire model from that point upwards being moved upwards. Various ways of feeding in the earthquake within the model foundation, were tried, without success. Several foundation sizes were also tried, including one so large that the earthquake had been damped to almost nothing by the time it arrived at the wall foundation. All of these problems were overcome when the x and y fixity was removed.

Cable Element Shortening

The upper two or three cables had elements shortening near the facing during periods of compression during shaking. This does not occur in prototype walls and no buckling of upper reinforcement strips was observed in model testing by Fairless (1989). To resolve this problem, the compression yield force (i.e. the maximum compression force) in the cables was set to the same numerical value (but of opposite sign of course) as the tension yield force. On reflection this may be causing the strips to push themselves out during compression (apparently pulling out) because the elastic modulus of the steel is much greater than that of the soil. Using a maximum compression force (input as compression yield force) calculated as numerically equivalent to the value of Young's modulus adopted for the soil might resolve the problem.

Upper Cables Pulling Out

During shaking the upper cables pulled out and the facing panel tilted away from the soil. There is no report of observation of such behaviour in past seismic or simulated seismic shaking, either for model or prototype walls. Designers of Reinforced Earth walls usually drape the upper one or two layers of reinforcement strips down lower into the fill, to increase the confining stress and hence the strip friction, in an attempt to prevent pullout.

This was not attempted in this work, because it would have meant changing the way the strip properties were defined - the same properties were used for the entire cable, rather than having separate properties for each cable element. This would have also required changes to the method of updating the strip friction parameter S_{bond} during the analyses.

It was decided that the apparent failure of the top of the wall was having little or no significant effect on the overall displacement of the walls during simulation. Model studies (e.g. Fairless, 1989) have shown that the most critical strips in resisting overall seismic displacement are those in the lower third of the wall, so upper strips pulling out should have little effect on overall wall displacement. It is also observed that the wall analyses have not shown signs of excessive tilt.

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