# EVALUATION OF SEISMIC DESIGN PARAMETERS FOR THE MUSEUM OF NEW ZEALAND SITE

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ABSTRACT

Investigations carried out to evaluate the seismic design parameters, including acceleration response spectra and time-histories, for the design of the Museum of New Zealand, Te Papa Tongarewa, on the Wellington waterfront are described. The procedures used to assess the site stability under strong ground shaking and to determine the maximum likely lateral spreading and settlements are also summarised.

## 1. INTRODUCTION

The Museum of New Zealand, Te Papa Tongarewa, is to be constructed on the Taranaki Street Wharf reclamation on the Wellington waterfront (Figure 1). The building has a site plan area of about 12,000 m$^2$. There are three complete suspended floors over the full area with intermediate partial floors between the main floors. The building is approximately 160 m by 150 m in plan and 22.5 m high. Concrete frames in one direction and concrete shear walls in the other direction resist lateral loads. The building is being designed as a base isolated structure on shallow 3 m wide continuous strip footings with interconnecting beams in the orthogonal direction. Structural details and the seismic design procedures used for the structure are described by Boardman [1993].

The Building Design Brief defined the following structural performance criteria for earthquake design:

(a) The probability of the building structure exhausting available structural ductility and isolator displacement, such that collapse is likely in a major earthquake, shall not exceed 7% in 150 years.

(b) The probability of significant visible damage to the structure and/or cladding and/or finishes shall be less than 50% in 150 years.

These criteria define "collapse" and "no damage" design loading levels equivalent to earthquakes with return periods of 2000 and 250 years respectively.

In addition to the above criteria, it was found advantageous to define a 500 year return period design earthquake for both the structure and site analyses. A limiting ductility factor of 2.0 was used as the criterion to assess the performance of the structural frames and walls under this level of loading. The criterion for the site performance under the 500 year return period level was that there should be no significant liquefaction in the foundation soils and that settlement and lateral ground deformations should be limited so that there would be no adverse affect on the performance of the structures and their foundations. Under this level of ground shaking, repairable damage to underground services was considered acceptable.

The site has been reclaimed from the Wellington Harbour in two main stages. Prior to 1920, reclamation soil was deposited behind a concrete seawall to form Cable Street and some land beyond. In 1968, the main area of the site was created by end dumping clean gravel and quarry material on the seaward side of the concrete seawall. A battered rock revertment beneath the present Taranaki Street Wharf retains the most recent reclamation and only the top 2 m was mechanically compacted.

## 2. SITE INVESTIGATIONS

### 2.1 Geotechnical Investigations

In the first stage of the site investigations, seven cored-drillholes were undertaken together with borehole testing, laboratory testing and geophysical studies [Tonkin & Taylor Ltd, 1992]. A site soil model was developed by collating this information with data from earlier feasibility studies. The work identified the depth and engineering properties of the loose reclamation fill, the presence of a soft marine interface layer entrapped between the fill and the underlying competent alluvial gravels and colluvium that extends down to basement rock. The reclamation soils were found to extend to a depth of up to 13 m and the sediments to a maximum of 137 m below ground level with the underlying rock profile forming an approximately north-south trending valley resulting in a significant variation of the depth of the soils in east-west sections across the site.

To define the extent of the interface layer and to provide more details of the properties for stability and site seismic response analyses, two further investigation stages, comprising 5 boreholes and 48 combined borehole/cone penetrometer test (CPT) holes were carried out. Results indicated that the

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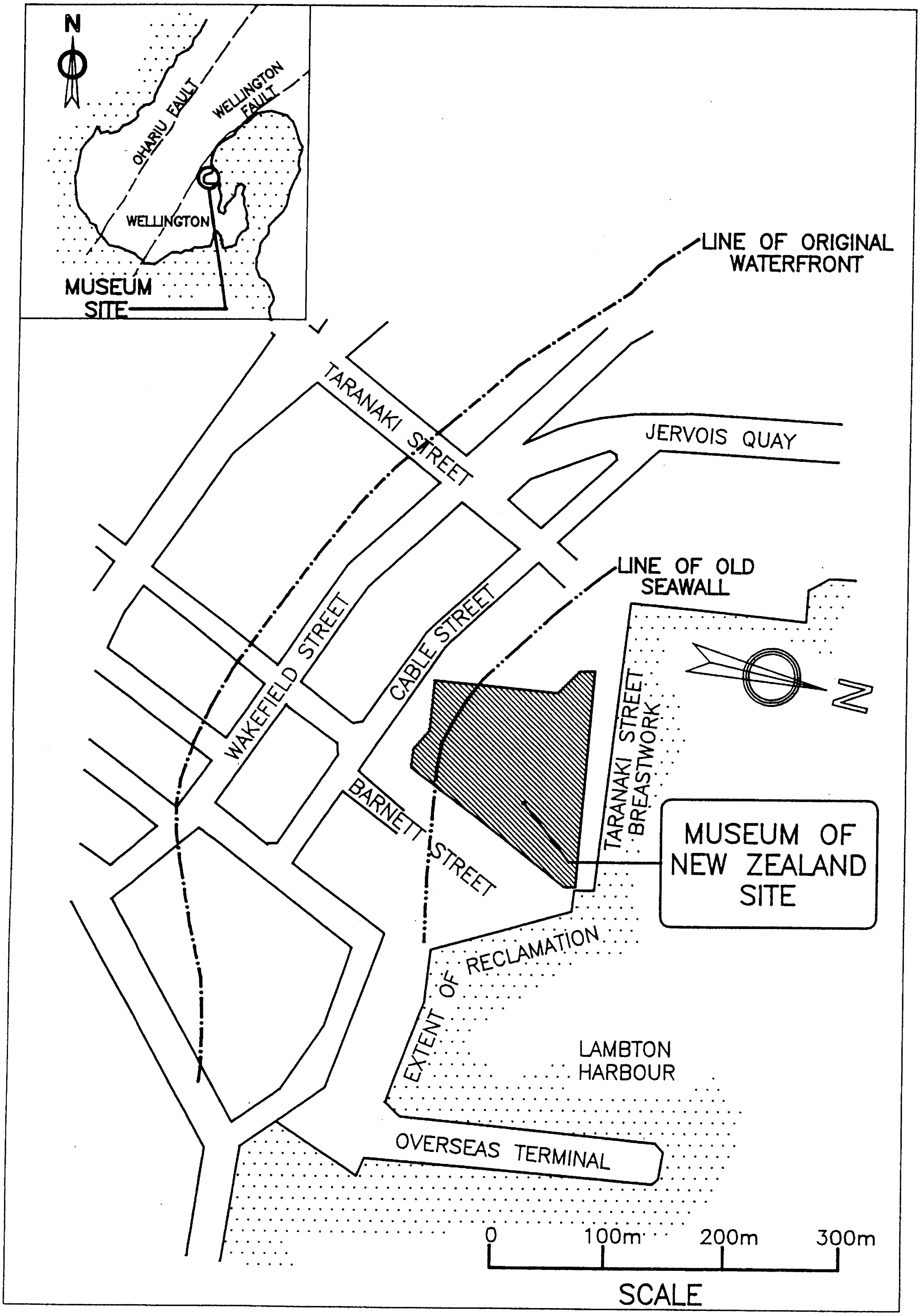


*FIGURE 1. Site Location*

interface layer materials are predominantly SILTS but ranged from silty CLAYS to sandy SILTS. The thickness of the layer ranged from 600 to 1400 mm and was essentially continuous over the site.

Evaluation of the site stability and foundation options indicated that site remedial work was required to reduce settlement and lateral spreading during strong earthquake ground shaking and to improve the strength of the interface layer to prevent sliding block type failures. A field trial of site improvement by dynamic consolidation was carried out using a 2.2 m diameter 25 t weight dropped in a grid pattern from heights up to 24 m. Test monitoring showed that the remedial work developed a very dense raft of soils to a depth of up to 8 m with reduced improvement in the deeper layers. The interface layer was reduced in thickness and increased in strength as indicated by substantial increases in the CPT cone resistance values.

### 2.2 Accelerograph Monitoring

As part of the site investigation, a film recording strong-motion accelerograph was installed in March 1989, followed by a digital accelerograph in October 1989. Twenty earthquakes were recorded up to March 1992, including fourteen also recorded by an accelerograph on a rock site at the New Zealand Seismological Observatory (NZSO) 1.3 km from the Museum site. The peak ground accelerations in the twenty records ranged from 0.003 g to 0.037 g, with only three exceeding 0.010 g.

The principal characteristic revealed by the records obtained during monitoring of the site was strong amplification with respect to rock motions, with the peak amplification usually occurring in the 0.8 to 1.0 s range. Typical peak amplifications of the 5% damped acceleration response spectra between the NZSO rock site and the Museum site were 3 to 4.

For most of the records with significant peaks in the Fourier spectra of their accelerations in the 1.0 to 1.25 Hz band, the direction of maximum energy changed dramatically between frequency bands of 0.2 Hz width centred at 1.0 Hz and 1.25 Hz. McVerry and Zhao [1991] interpreted the strongly frequency-dependent directionality of the site motions as corresponding to fundamental mode responses of the site aligned along and across the valley in the greywacke bedrock underlying the site.

## 3. SOIL PROPERTIES

A number of earthquake response analyses using models of the soil layers were undertaken as one of the procedures to assess the effects of site amplification. The methods used to establish the soil properties required for these analyses are summarised below.

### 3.1 Low Strain Shear Modulus

(a) A plot was made of all recorded SPT N-values versus depth. Data recorded in a total of 10 investigational drill holes were used.

(b) A linear regression line was fitted to the N-value plot to give:

$$N = 7.5 + 0.96\ D \qquad (1)$$

where D is the depth below the surface.

This expression was assumed to apply for depths to 58 m. Below this depth there were no test data. It was assumed that N increased linearly with depth from 58 m to give an N value of 100 at a depth of 116 m.

(c) The shear wave velocities for the site soils were estimated from the SPT N-values using the following expression from Seed et al [1986]:

$$V_s = 90\ N^{0.17}\ D^{0.2}\ \text{m/s} \qquad (2)$$

A number of direct measurements of the soil shear wave velocities were made to depths of 25 m using the cross-hole technique. These results are compared with the Seed et al [1986] correlation in Figure 2.

A number of other correlations that had been previously compared by Sykora and Koester [1988], were considered for the site. Two of these are compared in Figure 2 with the Seed et al [1986] correlation and the site measurements. At depths greater than 30 m, the Seed et al [1986] shear wave velocities lie within the range of the other correlations. At shallower depths, the Seed et al [1986] correlation equation fits the site measured values better than the other prediction methods.

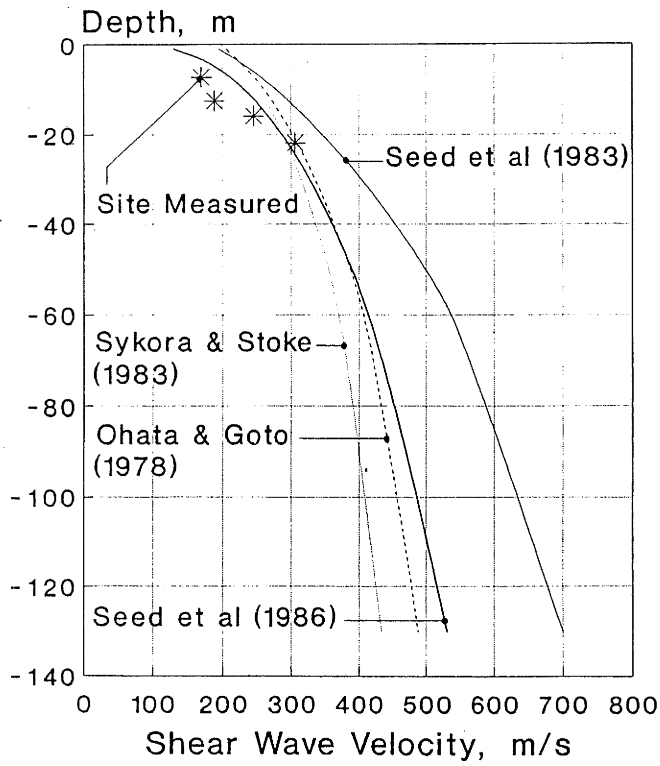


*FIGURE 2. Shear Wave Velocities for Unimproved Site*

(d) To adjust for the proposed site improvement work, it was assumed that a uniform N-value of 25 would be achieved to a depth of 18 m. N-values at greater depths were assumed to be unchanged by improvement work.

(e) The low strain shear modulus profile obtained from the shear wave velocities was verified by setting up a two-dimensional elastic finite element model of the site soil layers. The shape of the inferred valley cross-section from the investigational drilling work was modelled to a reasonable approximation. The model was divided into 13 horizontal layers with the elastic constants derived using the above procedure.

Periods of vibration were computed for both the along and across valley directions and compared with the predominant site periods of vibration measured during small earthquakes recorded at the site. The computed periods of 0.81 and 0.94 s for the across and along valley directions respectively, compared closely with measured values of 0.8 and 0.95 s.

Because test data were unavailable for the soils at depths greater than 60 m, properties in the lower part of the profile were based on experience from other deep soil sites. The finite element model verification work indicated that assumed soil properties were reasonable. The nonlinear site response analyses described later also showed that the surface horizontal accelerations were not particularly sensitive to variations made to the soil properties within the range of uncertainty.

### 3.2 Shear-Stress/Shear-Strain Curves and Soil Damping

A detailed review of published information on soil shear-stress/shear-strain curves and damping properties for cohesionless soils was carried out. Relevant information on the properties of gravels and sandy gravels found in some of the site soil layers is presented in Seed et al [1986] and Hatanaka et al [1989]. These references indicate that the shapes of the shear-stress/shear-strain and damping/shear-strain curves are not particularly sensitive to the soil grading and particle size for cohesionless soils ranging from silty sands to gravels. Extensive dynamic soil testing carried out overseas has established typical shear-stress/shear-strain curves for cohesionless soils that have been widely adopted for design applications. It was concluded that the standard S1 to S3 shear modulus/shear-strain and damping/shear-strain curves used in the computer program SHAKE88 were suitable for modelling the earthquake response of the Museum site soils.

The standard shear modulus/shear-strain and damping/shear-strain curves for sand used in the SHAKE88 analysis are shown in Figures 3 and 4. The S1 curve applies for mean effective pressures (mep) < 100 kPa; the S2 curve for mep 100 to 300 kPa, and the S3 curve for mep > 300 kPa.

## 4. WELLINGTON FAULT

The Wellington fault passes within 1.8 km from the site (Figure 1). An earthquake on this source was found to dominate the earthquake hazard at the site. Details of the other major seismic sources affecting the Museum site are summarised by McVerry et al [1991]. For these other sources, the distances to the site are greater and the annual probabilities of rupture are significantly lower.

Van Dissen and Berryman [1991] provide the following information for the Wellington fault:

| | |
|---|---|
| Horizontal Slip Rate | 6.0 - 7.6 mm/year |
| Single Event Horizontal Slip | 3.2 - 4.7 m |
| Recurrence Interval | 420 - 780 years (600 mean) |
| Elapsed Time Since Last Event | 300 - 450 years |

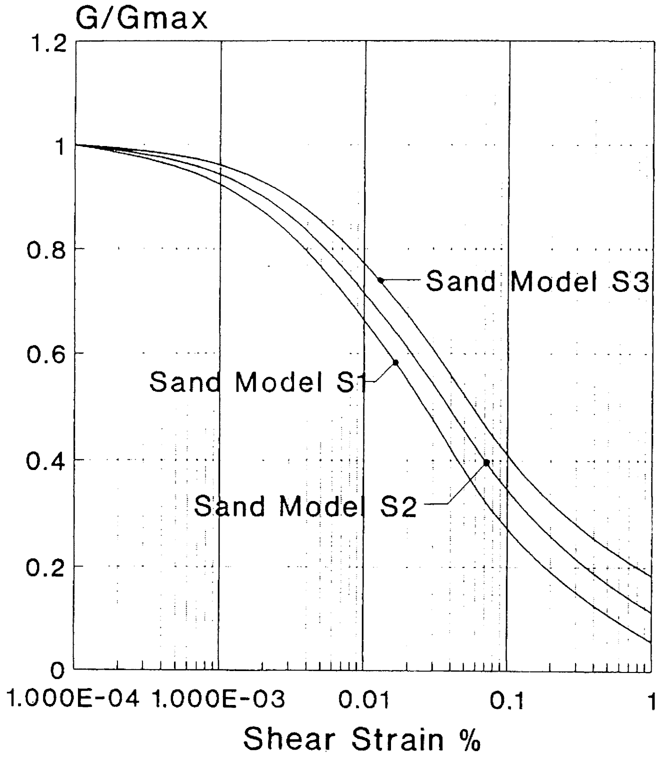


*FIGURE 3. Modulus/shear-strain Curves Used in SHAKE88*

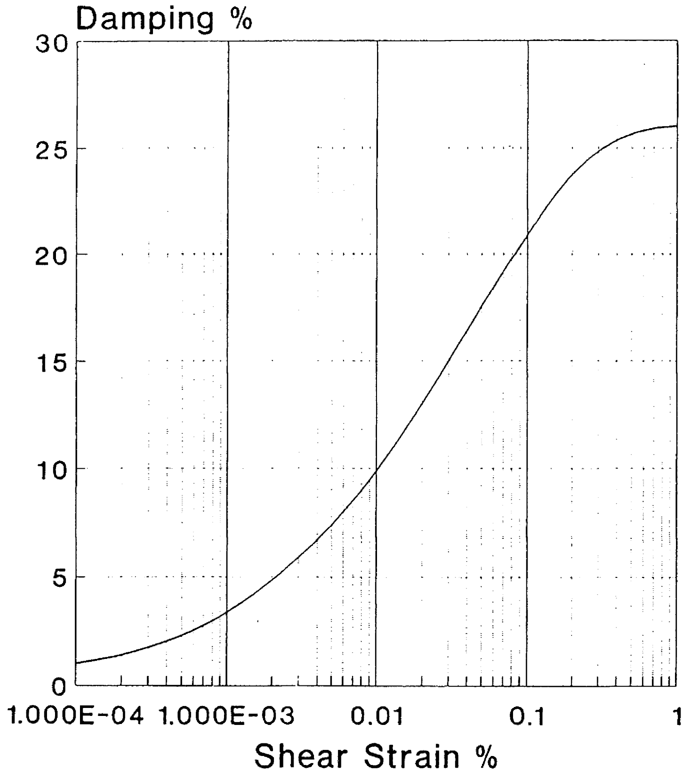


*FIGURE 4. Damping/shear-strain Curves Used in SHAKE88*

New data from field investigations have caused these recurrence interval estimates to be modified slightly from earlier estimates of 440 - 680 years, while the range of elapsed time since the latest surface fault rupture has decreased significantly from earlier estimates of 300 - 700 years reported by Beetham et al [1987].

Based on a comparison with surface ruptures throughout the world, and assuming either a 75 km long rupture, defined by the segment length, or a 4.7 m lateral displacement, a rupture of the Wellington-Hutt Valley segment is expected to be associated with a $M_s$ 7.1 - 7.8 earthquake. The Wellington-Hutt Valley segment is thought to behave in a characteristic rather than a random fashion. This implies a time dependent accumulation of energy between major earthquakes with the repeat times described by a probability curve of approximately log-normal shape.

Using the earlier preferred best estimate of the recurrence interval of 560 years, Beetham et al [1987] gave probabilities of rupture within a 150 year period of 29% and 58% for elapsed times from the last rupture of 300 and 700 years respectively. For design purposes, McVerry et al [1990] interpreted this information to give a present annual probability of rupture of the Wellington fault as 0.34% (approximately 40% in 150 years).

## 5. PEAK GROUND ACCELERATION

The mean peak ground acceleration (PGA) on the site soils from a M 7.5 earthquake on the Wellington fault was estimated using both the Joyner and Boore [1988] and Idriss [1985] prediction equations for a randomly oriented horizontal component. The Joyner and Boore equation, that relates PGA's for shallow earthquakes to moment magnitude and distance to the vertical projection of fault rupture gave a PGA of 0.60 g. A wide scatter in recorded PGA's was observed in the data used to develop the prediction equations. For example, in the Joyner and Boore prediction a variation of one standard deviation either side of a mean value of 0.6 g gives a range of 0.31 to 1.1 g.

The probability that a mean PGA value of 0.6 g will be equalled or exceeded at the site by a rupture on the Wellington fault is approximately 50%. Combining this probability with an annual probability of rupture of the Wellington fault of 0.34% gives an annual probability of exceedance of 0.17%. This probability is equivalent to a return period of about 580 years. The hazard contributions from other major earthquake sources in the region reduce this return period to about 560 years [McVerry, 1991]. A Wellington fault rupture was therefore taken for design purposes to represent approximately the 500 year return period event.

A relationship between frequency of occurrence (defined by return period) and PGA was required to scale response spectra and time-histories to the various return period design levels required for the structural and geotechnical analyses. Several different procedures were used to determine this relationship. McVerry et al [1991] determined a frequency of occurrence relationship for PGA on rock at the site from a source model consisting of the near-by active faults and the subduction zone and applying the Idriss [1985] acceleration attenuation equation. Following earthquake risk procedures suggested by Idriss, the distribution of the peak ground accelerations at the site expected from each source was assumed to be log-normal, with the standard deviation of the natural logarithm of the peak ground acceleration taken as 0.35. PGA's for the surface soil were estimated from the rock accelerations using a relationship given by Idriss [1990]. This approach, gave rock mean PGA's of 0.61 and 0.94 g corresponding to 500 and 2,000 year return periods respectively. Corresponding PGA's for the surface soil were 0.44 and 0.57 g.

The 250 year return period PGA could not be satisfactorily estimated from the major source model approach. At this return period level, events from the distributed seismicity in the region, of smaller magnitude than the characteristic events on the major sources, make a significant contribution to the probability of occurrence. To overcome this difficulty, two methods based on a uniformly distributed seismicity model with a continuous range of earthquake magnitudes were used to determine the PGA frequency of occurrence relationship. In the first of these, the Smith and Berryman [1983] return period/Modified Mercalli Intensity (MMI) relationship for Wellington was used. MMI values were converted to PGA's using a correlation relationship based on a world-wide set of accelerograms of horizontal motion [Krinitzsky and Chang, 1988] for soft soil sites and near-field ground motions. In the second approach, the frequency of occurrence of PGA's was estimated for the site surface soils using the Smith and Berryman [1983] seismicity model but with the Joyner and Boore [1981] PGA attenuation equation used instead of the Smith and Berryman MMI attenuation relationships.

The return period/PGA relationships obtained from the three methods described above are compared in Figure 5. The predictions from the uniform seismicity model using the correlation between MM intensities and peak ground accelerations gave a PGA of about 0.6 g for a 500 year return period which is in good agreement with the source model prediction. The uniform seismicity model predictions for 250, 500 and 2000 year return periods gave PGA ratios of about 0.8: 1.0: 1.3.

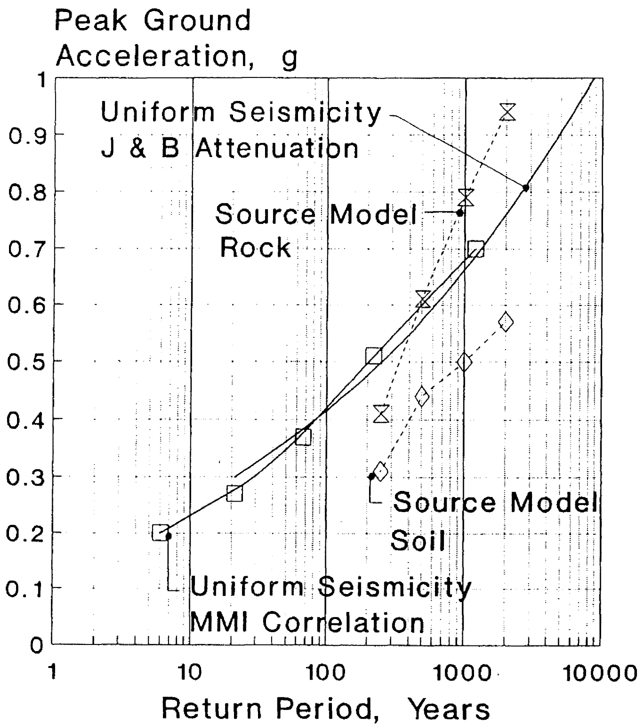


*FIGURE 5. PGA/Return Period Relationships*

## 6. ROCK HORIZONTAL ACCELERATION SPECTRA

Rock acceleration spectra were required to form the basis for selecting input acceleration time-histories for the site response analyses.

Three different prediction equations were used to derive the 5% damped rock horizontal acceleration spectra shown in Figure 6. Both the Joyner and Boore [1988] and Idriss [1985] spectra prediction methods were derived from regression analyses of spectra from North American strong motion records. The Kawashima et al [1984] spectrum was from similar studies of Japanese records. All three spectra are for a M 7.5 earthquake at locations within 3 to 5 km from the vertical projection of the fault rupture or the earthquake source. The Kawashima spectrum is for the maximum horizontal component whereas the other two are for a randomly orientated horizontal component. The data bases for the correlation studies do not contain near-source records for M 7.5 or larger magnitude earthquakes. The M 7.5 spectra are therefore based on extrapolation from records of smaller earthquakes and similar magnitude earthquakes at greater distances from the source.

For periods greater than 0.2 s, the Kawashima spectrum has spectral ordinates significantly lower than given by the other two predictions. One reason for the difference is that Japanese earthquakes mainly occur on subduction zones whereas in contrast, the North American earthquakes are mainly shallow and are associated with plate boundary surface faulting.

The Joyner and Boore spectrum is about 15% higher than the Idriss spectrum over the 1 to 3 s period range of interest for the building design. It is likely that this difference is due to the application of different regression analysis methods and the fact that the data analysed was not identical. The Joyner and Boore spectrum was adopted for the museum investigation on the basis that their regression analysis procedure had gained wide acceptance elsewhere.

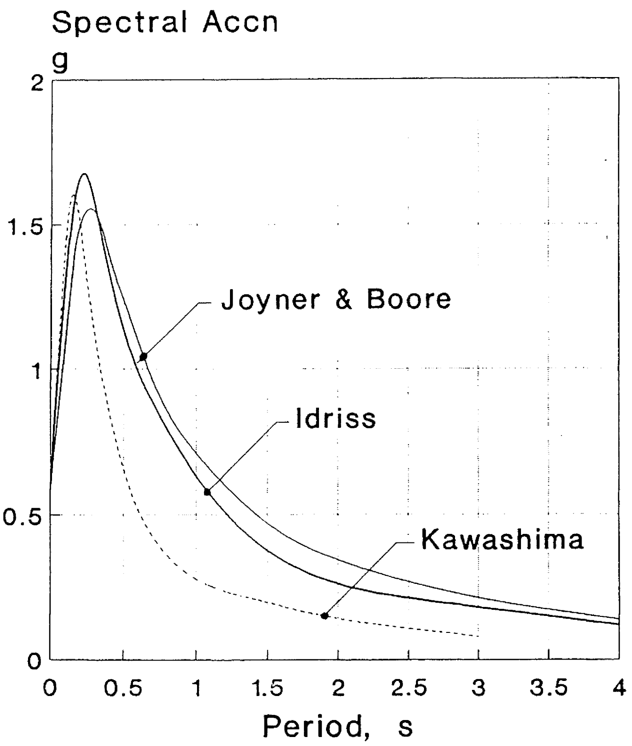


*FIGURE 6. Rock Acceleration Response Spectra Predictions*

## 7. SITE RESPONSE ANALYSES

Site response analyses were undertaken by the Engineering Seismology section of the Institute of Geological and Nuclear Sciences (IGNS, formerly DSIR) using a nonlinear two-dimensional soil model that enabled the valley on the rock interface to be approximated and by Auckland University using the SHAKE88 one-dimensional analysis method that represents nonlinear soil behaviour by an equivalent linear approach.

### 7.1 Soil Models

In final model analyses, the low strain shear moduli values from the Seed et al [1986] correlation (Figure 2) were used with the SHAKE88 S1 to S3 shear modulus/shear-strain and sand damping/shear-strain curves (Figures 3 and 4).

In the SHAKE88 one-dimensional approach, the soil valley geometry could not be modelled. In order to provide a reasonable match to the measured along valley period, the model depth was adjusted to 100 m with the soil property profile being truncated at the base of the layer. The computed low strain period of 0.97 s for the one-dimensional model compared closely with the 0.95 s value measured on site accelerographs during small amplitude earthquake motions.

In the IGNS model, the stiffening effect of the valley shape was initially incorporated by using spring elements to model assumed horizontal boundaries at the extremities of the valley. Because of this modification, it was not possible to accurately model the effects of radiation damping at the soil to rock interface. Instead, the radiation damping was simulated using an increase in viscous damping chosen to reproduce the recorded site motions using input motions recorded on rock at the NZSO about 1.3 km from the Museum site. The simplification of using a reduced height one-dimensional layer to allow for the valley stiffening was later adopted as this enabled the radiation damping effect to be more correctly included in the model.

### 7.2 Verification Analyses

Verification of both computation methods was carried out using as input the S70E component of the 19 February, 1990, Weber earthquake recorded on rock at NZSO. This input motion represents a small earthquake having spectral accelerations approximately 1/80 of those expected in the 500 year return period design level earthquake. For this level of input, the response of the models is essentially elastic.

Spectra computed from the output time-histories from both analysis techniques using the NZSO S70E input record showed reasonable agreement with the acceleration spectrum computed from the S70E Weber earthquake accelerogram recorded at the site. A comparison of the SHAKE88 output spectrum (5% damping) and the site recorded spectrum is shown in Figure 7. Except at periods less than 0.4 s, where the higher modes of the model produce amplifications that are not apparent in the site record, there is reasonable agreement. The input and site record direction of S70E corresponds approximately to the across valley direction. The model has periods of vibration more closely matching the along valley periods and better agreement could have been achieved by adjustment of the model to match the across valley periods. The reason for matching the model to the longer periods of the along valley response was to ensure a conservative result in the 2 to 3 s period range of interest for the building design.

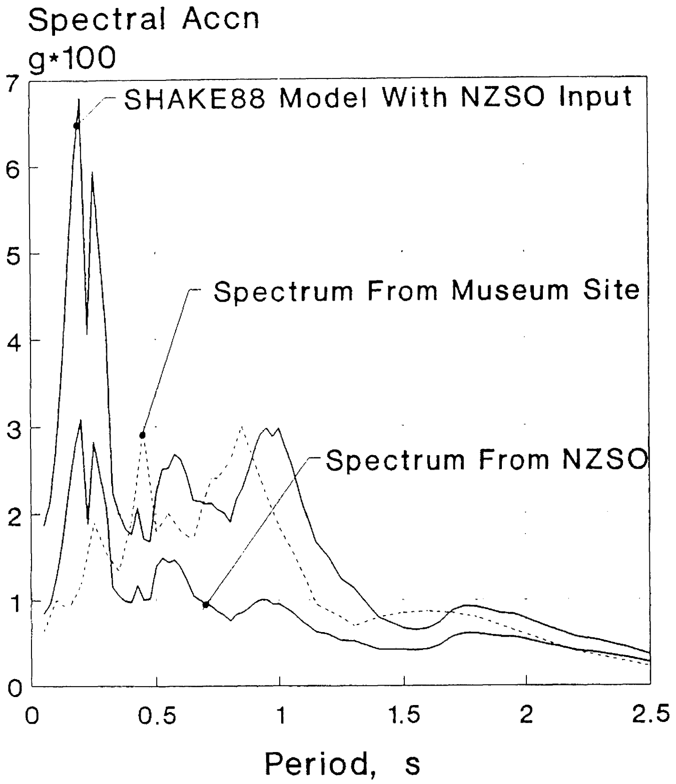


*FIGURE 7. Acceleration Response Spectra from February, 1990, Weber Earthquake, S70E Component*

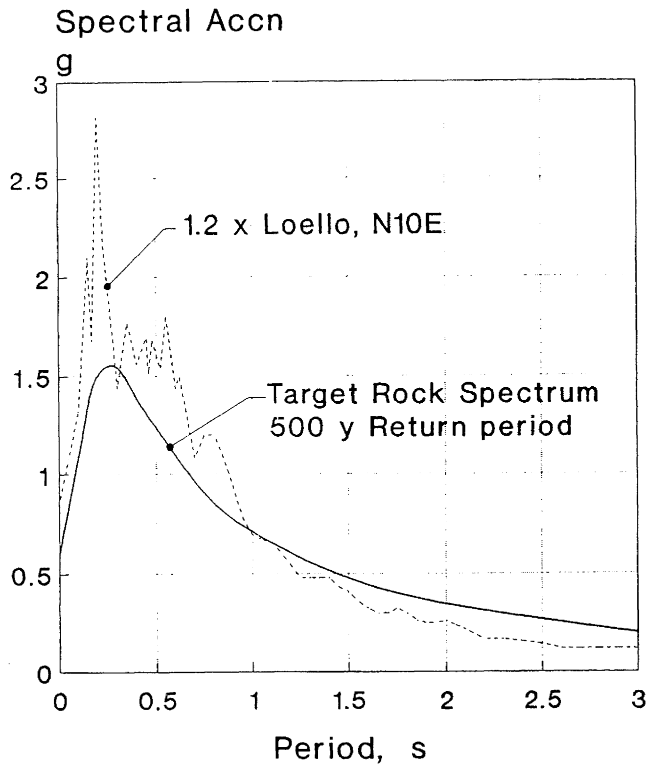


*FIGURE 8. Comparison of Spectrum from Scaled 1985 Lloello N10E Record and Target Rock Spectrum for 500 Year Return Period*

### 7.3 Input Accelerograms

Earthquake accelerograms from the 1940 El Centro, California, 1978 Tabas, Iran, and the Llolleo site in the 1985 San Antonio, Chile, earthquakes were used as inputs. The 1940 El Centro strong motions were recorded on a soil layer of 30 m of stiff clay at an epicentral distance of about 9.3 km during the $M_L$ 6.3 earthquake. Although recorded on soil, the N00E horizontal record was used as a rock input in the present study because of the very limited number of suitable strong motion rock records available. The 1978 Tabas strong motions were recorded on alluvium about 3 km from the surface fault rupture during the $M_L$ 6.6 ($M_s$ 7.5) earthquake. The Llolleo records were from a rock site. This was a $M_s$ 7.8 subduction zone earthquake with the epicentral distance of the recording site stated to be 45 km but it is thought that the recording site was immediately above the rupture.

The selected time-histories were amplitude scaled to give an approximate match with the 5% damped rock acceleration spectra for the 500 year return period design level earthquake. Scale factors of 1.8, 0.7 and 1.2 were used for the El Centro N00E, Tabas N00E and the Llolleo N10E records respectively. Figure 8 shows a comparison between the scaled Tabas N00E spectrum and the target 500 year return period rock spectrum.

### 7.4 Comparison of Results

The ratio of the computed ground surface acceleration response spectrum for 5% damping to the input spectrum for each of the three inputs was computed. These ratios indicate the level of amplification produced by the soil layers for a single-degree-of-freedom structure located at the ground surface. The spectra ratios for each model, averaged over the three inputs, are compared in Figure 9. Both the IGNS and SHAKE88 models were found to produce very similar spectral amplifications.

The amplification from both methods is greater than 1.0 for periods greater than 0.5 s; reaches a maximum value of about 2.1 at 2 s; then reduces to 1.5 at 3.5 s. The IGNS model predicts lower peak ground accelerations (zero period ordinate) than the SHAKE88, but this difference was not important for predicting the Museum response which is dominated by spectral accelerations in the 2 to 3 s range.

Also shown in Figure 9 is a spectra amplification ratio curve computed from 5% damped rock and soil acceleration spectra predicted by the Joyner and Boore [1988] equations for spectral ordinates. For periods greater than 1.5 s, the amplification predicted by Joyner and Boore is in reasonable agreement with the amplifications from the site response analyses. At shorter periods, the Joyner and Boore spectra show higher amplifications. Part of this difference can be attributed to the simplification used in the site response analyses where only vertically propagating shear waves are considered. At many sites, wave propagation through the rock and soil layers is likely to be considerably more complex than assumed in the simplified analysis procedure. Differences can also be expected from the inability to model exactly the nonlinear behaviour of the soil layers during strong shaking.

The spectral ratios shown in Figure 9 are significantly different from those between the recorded motions at the Museum site and at the NZSO rock site. The recorded motions were for low amplitudes all below 0.04 g. In modelling the much stronger motions expected from a magnitude 7.5 earthquake on the Wellington fault 1.8 km from the site, the peak amplifications are reduced from the typical 3 to 4 values in the recorded

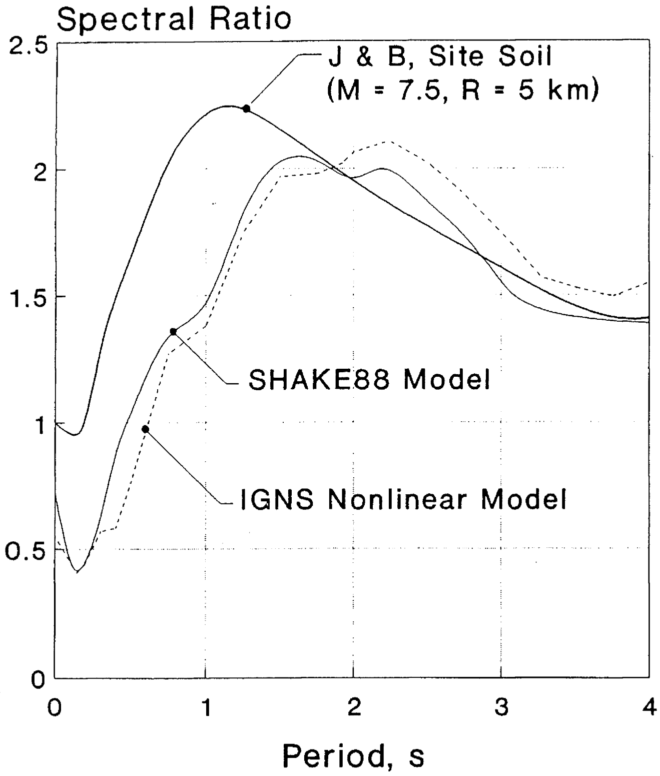


*FIGURE 9. Ratio of Acceleration Response Spectra*

motions to around 2, and the period of the maximum amplification lengthened from 0.8 to 1.0 s to around 2.0 s. The reduced amplifications and longer dominant period are the result of the nonlinear response of the model. The amplifications of the spectra of the modelled motions are greater than those of the recorded motions for periods in excess of about 1.5 s, and thus the model appears conservative for predicting ground motions for a long-period isolated structure.

### 7.5 Site Response in Loma Prieta Earthquake

In a study of the strong motion records from the $M_s$ 7.1, 1989 Loma Prieta earthquake, Borcherdt [1991] derived a correlation between the average site shear wave velocity to a depth of 30 m and the average horizontal spectral amplification (AHSA) over the period range 0.4 to 2.0 s. For the improved Museum site, the average shear wave velocity to a depth of 30 m will be about 270 m/s. At this velocity, the Borcherdt correlation gives an AHSA of 2.2. This is in reasonable agreement with the AHSA of about 2.0 indicated by the Joyner and Boore amplification curve shown in Figure 9. The Joyner and Boore amplification curve was derived for sites close to a M 7.5 earthquake. In this case, nonlinear soil effects would be expected to give lower amplifications than indicated by the Loma Prieta study which included records up to about 80 km from the epicentre.

Site amplification effects in the Loma Prieta earthquake have also been investigated by Dickenson et al [1991]. Most of the sites investigated contained significant depths of bay mud. This soil is a marine estuarine clay which is softer than the soils on the Museum site. One of the sites investigated was San Francisco Airport that has a 130 m depth of soil consisting of a relatively thin layer of Bay Mud (about 5 m) near the surface and an alternating sequence of medium stiff to hard silty clays and dense sands. The shear wave velocity profile at San Francisco Airport is similar to the Museum site but with rather lower velocities near the surface. The amplification over the period range 0 to 3 s was almost constant at a factor of about 3. Input accelerations recorded at a rock outcrop near the airport indicated that the level of shaking represented only a moderate earthquake (PGA 0.1 g). In stronger shaking, the amplification would be reduced by nonlinear soil behaviour.

## 8. SPECTRA FOR SURFACE ACCELERATIONS

To develop soil surface design spectra, comparisons were made between prediction methods developed from overseas strong motion accelerograms, spectra from near-source strong motion records, and results from the site response analyses. In addition, consideration was given to the spectral amplifications indicated by records obtained on the site from small earthquakes and moderate earthquake with relatively large epicentral distances. The adopted design spectra were based on an integration of information from all these sources rather than on any one prediction procedure.

### 8.1 Comparison of Prediction Methods

Figure 10 compares three predictions for the 500 year return period 5% damped horizontal acceleration spectrum on deep or soft soil sites and a spectrum that was developed from the site records of small earthquakes and distant moderate earthquakes. The spectrum developed from the site records took into account that the amplitudes of the 500 year motions would be much larger than those of the recorded motions. The site amplification expected in the 500 year motions with nonlinear soil response was estimated using the original IGNS model discussed in Section 7.1. Beyond 1.5 s, the ordinates of the

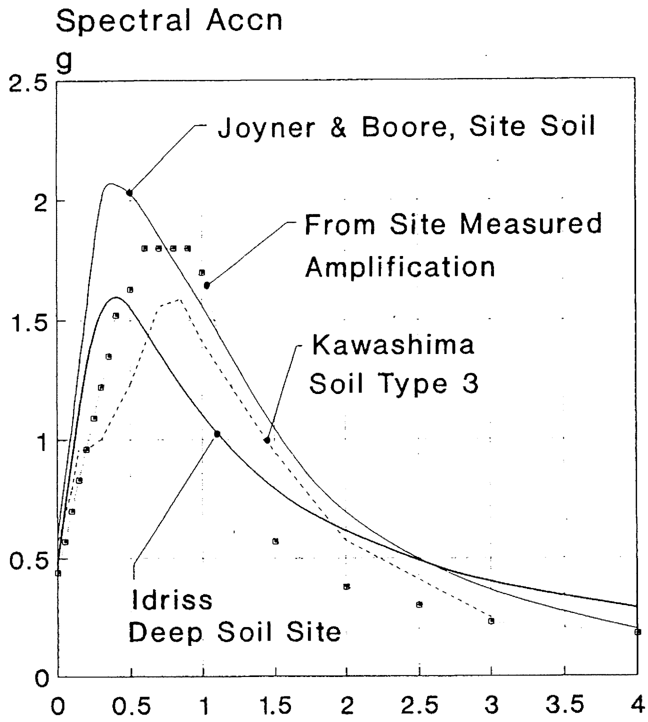


*FIGURE 10. Comparison of Predictions for Surface Acceleration Response Spectrum, 500 Year Return Period*

spectrum were increased to those corresponding to the spectral acceleration (SA) to PGA ratios of the standard Idriss [1985] deep soil spectrum.

The spectra prediction equations for soil were from the same correlation studies mentioned previously in relation to the rock acceleration spectra predictions (Section 6.). All three spectra predictions were for a M 7.5 earthquake at locations within 3 to 5 km from the vertical projection of the fault rupture or the earthquake source. The Kawashima spectrum is for the maximum horizontal component and the other two are for a randomly orientated horizontal component. The Idriss spectrum is for a deep soil site and the Kawashima spectrum for soft alluvium or sites with natural periods of 0.6 s or greater. The Joyner and Boore prediction equation includes a correlation factor based on the shear wave velocity profile at the site. This factor was applied to calculate the spectrum shown in Figure 10 by using the shear wave velocities determined from the site investigations (Figure 2).

There is a relatively large difference between the spectral ordinates of the Idriss and Joyner and Boore curves at periods less than 1.5 s. The main reason for this difference can be attributed to inclusion of the shear wave velocity correlation factor in the Joyner and Boore prediction and the lack of a similar factor in the Idriss method.

The spectrum based on the site records has spectral accelerations at periods greater than 1.4 s that are significantly lower than the other spectra. This difference results from the use of the SA/PGA factors of the Idriss standard deep soil spectral shape for periods in excess of 1.5 s. These factors were derived mostly from data from the $M_s$ 6.6 San Fernando and the $M_s$ 6.8 Imperial Valley earthquakes. The deep soil SA/PGA factors are likely to be much larger for a magnitude 7.5 earthquake than for magnitude 6.6 - 6.8 events, as indicated by comparison in Figure 10 between the Idriss deep soil curve for a magnitude 7.5 event and the standard Idriss shape used beyond 1.5 s in the curve developed from the measured site amplifications.

**8.2 Comparison With Spectrum From SHAKE Analysis**

Figure 11 compares the site spectrum computed from the Joyner and Boore 500 year return period rock spectrum using the SHAKE88 computed spectral amplification ratios (shown previously in Figure 9) and the site spectrum obtained directly from the Joyner and Boore 500 year return period prediction for the surface with the site shear wave velocity correction applied. The spectra are in good agreement at periods greater than about 1 s.

**8.3 Building Code Design Spectra**

A comparison of the Joyner and Boore 500 year return period horizontal acceleration spectrum, corrected for the site shear wave velocity, with the UBC (U.S.A. Uniform Building Code, 1988) and DZ 4203 (draft N.Z. loading code, 1988) code design spectra for 5% damping is shown in Figure 12. The UBC spectrum shape has been widely adopted in U.S.A. for building design. At the time of the study, the DZ 4203 code was still under review but it was expected that a spectrum similar to the one shown would be adopted.

For the purpose of the comparison, both code spectra have been scaled to have the same PGA of 0.61 g as the Joyner and Boore spectrum. In California, the UBC spectrum is scaled to an "effective" PGA of 0.4 g for a design level corresponding to a

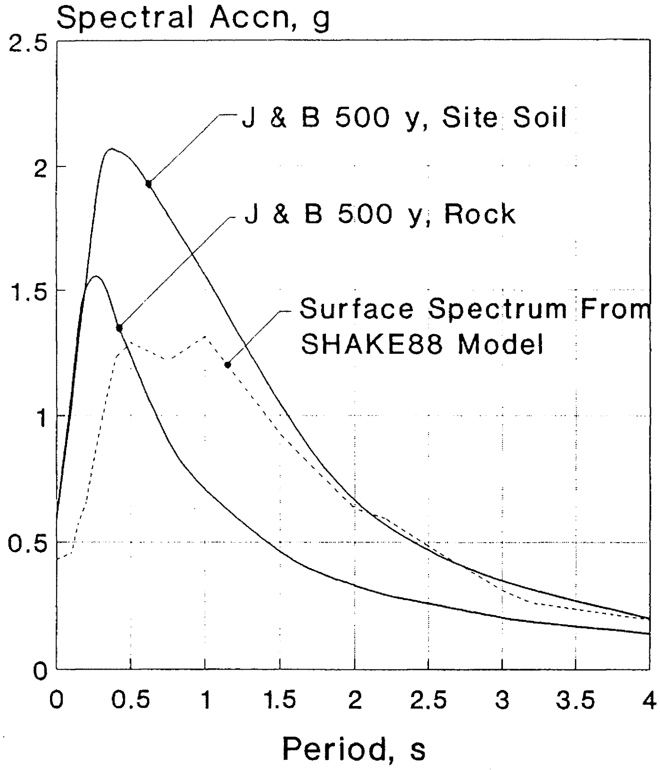


*FIGURE 11. Comparison of Spectrum from Site Response Analyses with Joyner and Boore Prediction*

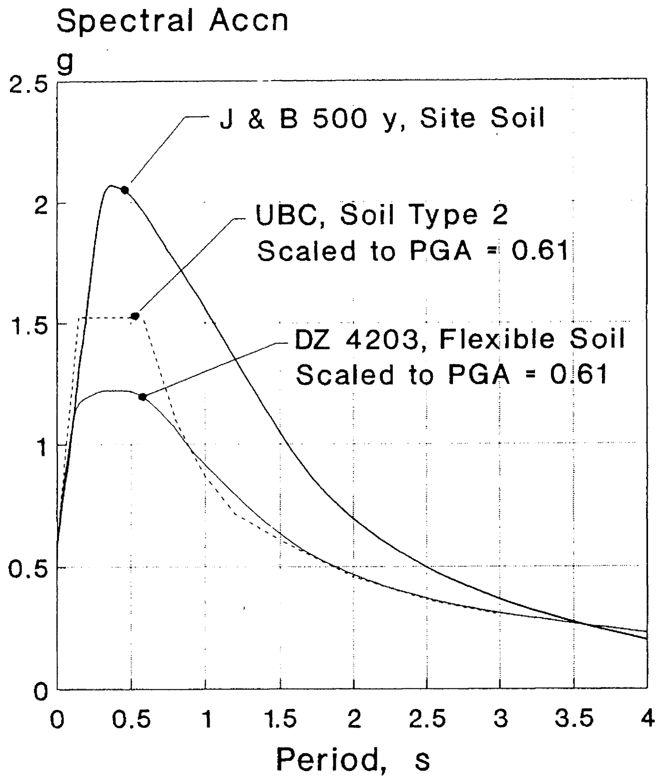


*FIGURE 12. Comparison of Code Spectra With Joyner and Boore Prediction*

10% probability of exceedance in 50 years. However, it is recognised that this "effective" PGA is less than actual peak ground accelerations expected under the design level earthquake. A recent map of acceleration coefficients prepared for the National Earthquake Hazards Reduction Program (NEHRP) provisions for buildings [Federal Emergency Management Agency, 1988] by the US Geological Survey, shows a maximum value of 0.6 g in areas close to the main faults along coastal regions of California. It is also likely that a site specific spectrum would be developed for a site in a similar situation to the Museum and a higher PGA than the "effective" value given by the UBC code would probably be used.

A PGA of 0.32 g would be used in the application of DZ 4203 code to normal building design in Wellington. A risk factor of 1.2 would be applied for the Museum to allow for a greater public risk, and in addition, a scale factor of about 1.3 might be applied to increase the 150 year return period design level assumed in DZ 4203 to the 500 year return period adopted for the Museum design.

The Joyner and Boore prediction gives spectral accelerations considerably in excess of normal design standards. Even if due account is made for the building importance and the proximity to the Wellington fault, it is unlikely that code design spectra would be scaled to have PGA's exceeding about 0.6 g. In the 1.5 to 3 s period range, the Joyner and Boore spectrum would therefore exceed normal design standards by about 50%. Bertero [1991], Dickenson et al [1991] and other researchers have recently expressed opinions that the UBC code will underestimate the shaking intensity on soil sites in regions of highest seismic risk. Thus some of the difference can be attributed to present design codes being unconservative for soil sites.

### 8.4 Adopted Design Spectra

A modified spectrum shape based mainly on the Joyner and Boore prediction for a M 7.5 earthquake at a distance to fault rupture of 5 km was adopted as the 500 year return period design spectrum. The adopted spectrum was modified from the Joyner and Boore shape to allow for site specific periods of vibration found by the site monitoring work and to account for the Joyner and Boore spectrum being more conservative than spectra from other prediction methods and spectra given in present design codes. In the period range greater than 1.5 s, the adopted spectrum was consistent with the spectral amplifications found from the site response analysis work.

The 250 year and 2000 year return period design level spectra were scaled from the 500 year spectrum using constant scale factors of 0.8 and 1.3 respectively for the period range of interest (see Section 5). Strictly speaking, the shape of the spectra should be modified for the different design levels; however, these differences were considered to be small in relation to the other uncertainties in the prediction procedure.

The adopted design spectra for the surface soil horizontal accelerations are shown in Figure 13.

### 8.5 Vertical Design Spectra

Design spectra for vertical accelerations have not been widely researched and predictive equations, based on correlations similar to the Joyner and Boore method for horizontal acceleration spectra, are not available. If vertical accelerations are to be considered in design, the usual procedure is to use the

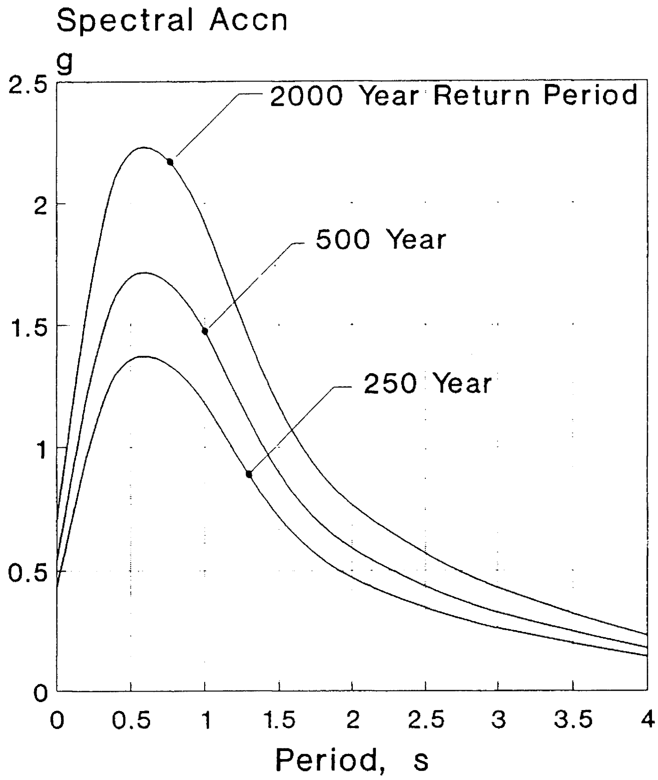


*FIGURE 13. Adopted Design Spectra for Horizontal Acceleration, 5% Damping*

horizontal spectrum scaled down by a factor of about 0.67. Although this simplification may be satisfactory in some cases, it has no rigourous basis and may result in unsatisfactory predictions for special structures where vertical accelerations may become critical. Inspection of spectra from strong motion records shows that the vertical spectra often have strong peaks that match the peaks in horizontal spectra at high frequencies but have lower spectral ordinates than horizontal spectra at periods greater than about 0.5 s. Thus the shape of vertical spectra are markedly different from horizontal spectra.

The amplification effects of soil layers on vertical accelerations is also an area which has not been adequately studied. Recent studies of strong motion data by Borcherdt [1991] have shown that soil layers produce significant amplifications of vertical accelerations but the amplification is less than occurs for horizontal accelerations. As part of the Museum study, a number of records from both the 1971 San Fernando and 1989 Loma Prieta earthquakes were reviewed. Amplification of the vertical spectral accelerations at the San Francisco Airport, a distance of about 52 km from the fault rupture in the $M_s$ 7.1 Loma Prieta earthquake, reached a maximum of 3.0 at a period of about 0.3 s. A similar amplification occurred at the Jet Propulsion Laboratory site, a distance of 31 km from the epicentre of the $M_L$ 6.3 San Fernando earthquake. In both cases, spectral amplifications were less than 1.5 at periods greater than 0.4 s. This information, for sites at moderate distances from the source, may not be particularly relevant for the Museum site responding to a Wellington fault event. In this case, the vertical response is likely to be determined more by characteristics of the source mechanism rather than the influence of soil layers. Strong vertical shaking will be generated by direct arrivals of p-waves that will not be strongly amplified as they pass from the rock into the saturated soil layers because of the relatively low p-wave impedance contrast.

In order to determine a satisfactory design spectrum for the Museum site, the strong motion vertical records from six moderate to large earthquakes were investigated and an envelope curve developed. With the exception of one of these records, the distances between the recording sites and the nearest points on the fault ruptures were 22 km or less.

In view of the proximity of the site to the Wellington fault, the design vertical PGA was taken as the 0.61 g value previously recommended for the horizontal accelerations on rock. Spectra from the six records were scaled to this PGA and a design curve sketched by enveloping most of the peaks. The vertical design spectrum developed by this procedure is shown in Figure 14.

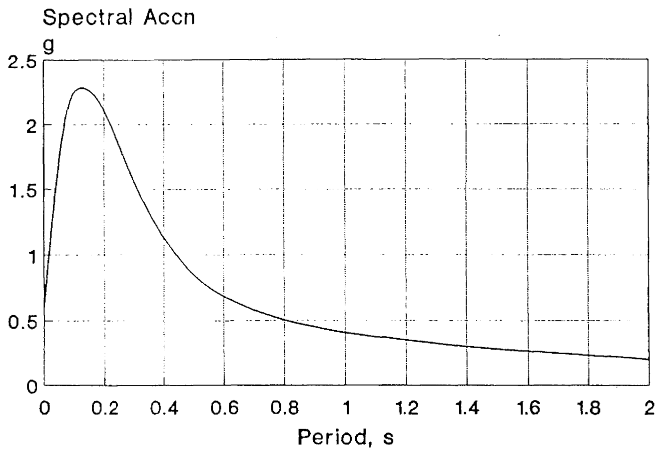


*FIGURE 14. Adopted Design Spectra for Vertical Acceleration, 5% Damping*

## 9. ACCELERATION TIME HISTORIES

Three methods were used to obtain suitable surface time histories for final geotechnical and structural design. The simplest procedure used was to scale existing records from large earthquakes to obtain a reasonable match to the design spectrum. To approximate the 500 year design spectrum, the 1978 Tabas N00E component was amplitude scaled by a factor of 1.3 and the 1976 Gazli N00E (recorded on firm sediments within 10 km of the fault causing the $M_s$ 7.0 earthquake) by 2.0. The second approach was to use the output time histories from the SHAKE88 analyses carried out for the three input motions adopted for the site response analyses. As described in Section 7.3, these inputs were scaled to match the Joyner and Boore 500 year return period design rock spectrum in the 1.5 to 3.0 s period range. The third method was to modify the frequency content of records from large earthquakes to obtain a close fit to the design spectrum. This was carried out using a special purpose program that computes a fast Fourier transform of the input, makes modifications in the frequency domain and then computes the inverse transform. A number of iterations of this process are required to give the spectrum compatible time-history [Kelly, 1992].

For the final design analyses, it was recommended that at least 5 input time-histories be used with at least one selected from each of the three methods.

## 10. SITE SLIDING STABILITY

The presence of the continuous layer of soft silty clay entrapped between the reclamation fill and underlying dense alluvium/colluvium was of concern in relation to the potential for large lateral deformations towards the harbour of the overlying fill (block slides) during strong earthquake ground shaking.

The standard procedure for estimating displacements of slope failures during earthquake ground shaking is to use the Newmark [1965] sliding block theory. Displacements are a function of a resistance factor, defined as the ratio obtained by dividing the horizontal yield acceleration required to initiate movement of the sliding block of soil by the peak ground acceleration in the earthquake acceleration time-history acting on the block. The yield acceleration in the case of the Museum site can be determined from a slope stability analysis using the residual shear strength of the interface layer and applying pressure forces from the surrounding soil and water on the assumed sliding block. The residual shear strength of the interface soils was available from the extensive CPT results obtained during the site investigations and at the completion of the dynamic consolidation trial.

Newmark [1965] produced design charts suitable for predicting outward displacements that were based on computed displacements from 4 accelerograms recorded in 3 moderate to strong Western United States earthquakes. These charts were considered to have a number of limitations for estimating the outward movements at the Museum site. The main limitation being that the accelerograms were recorded on stiff alluvium and are not representative of those expected on the deep flexible soils at the site. The charts make no allowance for the change in resistance factor that is likely to occur when the silty clay materials on the interface are remoulded by the sliding movements. Also the failure plane at the Museum site is relatively flat allowing the possibility of upslope movements producing a total downslope movement intermediate between the unsymmetrical and symmetrical sliding solutions given by the charts.

In view of the limitations of the available charts, it was decided to carry out sliding block analyses for the site based on the surface accelerograms available from the SHAKE88 site response analyses for the two horizontal components from each of the El Centro (1940), Tabas (1978) and Lloello (1986) earthquake records (Section 7.3). The outward displacements were computed using DISPLMT [Houston et al, 1987], a computer program that solves the sliding block equations of motion for any input acceleration time-history. A special feature of the program is that the yield acceleration (or sliding resistance) may be varied as a function of time or relative displacement of the sliding block.

### 10.1 Surface Accelerograms

Acceleration response spectra computed from the SHAKE88 output accelerograms for the Lloello earthquake are compared in Figure 15 with the 250 and 500 year return period design spectra adopted for the site surface. The strongest component (N10E) of Lloello accelerograms matches the 500 year design level spectrum over most of the period range of interest.

The duration of strong shaking exceeding 0.1 g in the SHAKE88 outputs was about 22, 20 and 37 s for the El Centro, Tabas and Lloello records respectively.

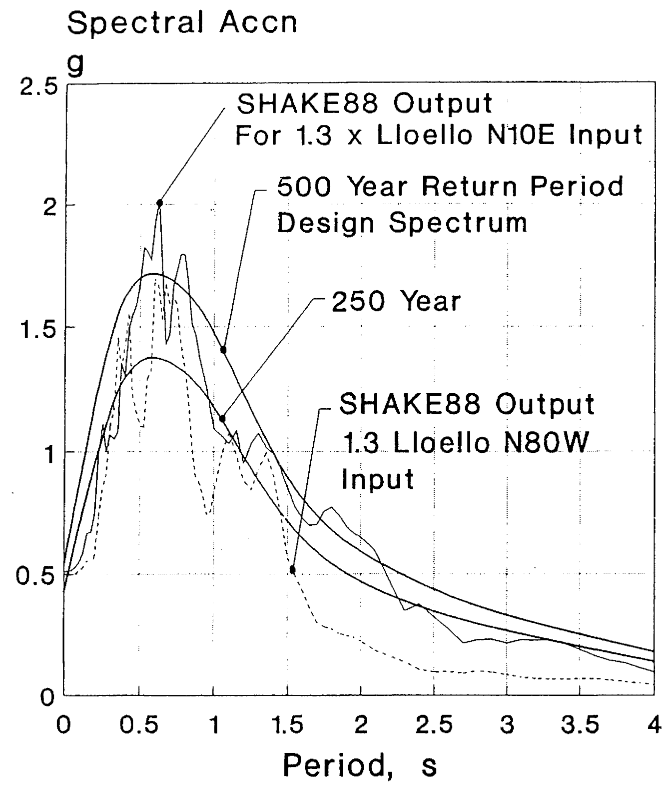


*FIGURE 15. Surface Spectra Computed from 1985 Lloello Input Accelerograms*

## 10.2 Yield Accelerations

The CPT soundings indicated that the interface layer was almost horizontal over a large part of the site north of the old seawall. The thickness of the reclamation reduces on a gradual slope from near the old seawall to the original seashore about 100 m to the south of the site.

To calculate sliding displacement on shallow slopes it is necessary to estimate the static factor of safety. This parameter is used in DISPLMT to calculate the ratio of the downslope to upslope yield acceleration. A static factor of safety for the Museum site was calculated assuming a simplified geometry consisting of two sliding blocks as shown in Figure 16. Details of the sloping interface are unknown but it is probably a stepped shape reflecting old beach levels rather than the uniform slope shown. However, the details assumed for the interface slope do not have a large influence on the computed displacements.

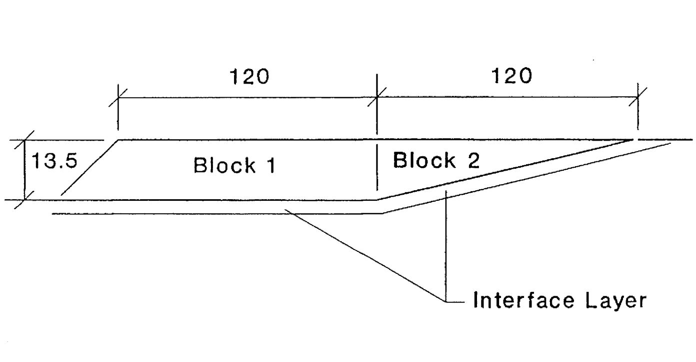


*FIGURE 16. Two-Block Model for Slope Stability Analysis*

The yield acceleration and static factor of safety for the two block model of the site were calculated using the Sarma [1979] limiting equilibrium method of slices. It was assumed that the full earthquake inertia load acted on both blocks and that the shear resistance on the sloping interface was 0.5 times the resistance on the horizontal section of the interface. This later assumption is based on the fact that a large part of this interface is beyond the site boundaries and will therefore not be improved by dynamic consolidation.

Taking the horizontal interface shear resistance as 60 kPa (indicated by the CPT results following the dynamic consolidation trial) gave a static factor of safety of 6.0 and a yield acceleration of 0.19 g.

## 10.3 Computed Displacements

Initially displacements were computed for the case where upslope yield displacements were prevented. The relative downslope displacements of the sliding block were computed for the six SHAKE88 output accelerograms over a range of resistance factors varying from 0.05 to 0.8. Each accelerogram was processed using two runs, with the sign of the accelerations in relation to the outward direction being reversed for the second run. The displacements were taken as the average of the results from the two runs.

Further analyses with upslope yield prevented were carried out assuming that the yield acceleration reduced with relative displacement of the sliding block. The yield acceleration was taken to reduce linearly during the first 50 mm of movement to a residual value 0.67 times the peak value and then remain constant. This reduction is approximately equivalent to the shear resistance on the interface layer reducing by the same factor and was thought to provide a reasonable model of the effects of remoulding in the interface soils with the 50 mm displacement being equivalent to a shear strain of about 5% in a layer of 1000 mm thickness.

A final analysis included the effects of upslope displacement. As for the previous case, the yield acceleration (in either direction) was assumed to reduce linearly during the first 50 mm of movement to a residual value 0.67 times the peak value.

Displacements for the various analyses carried out using the Lloello surface accelerograms are compared in Figure 17. The resistance factor for the cases where the peak resistance reduces to a residual value was taken as the ratio of the residual yield acceleration (after 50 mm displacement) divided by the input record peak ground acceleration. Reductions in the total displacement produced by the upslope movements were significant and found to decrease with increasing resistance factor. At a resistance factor of 0.4, the upslope yield effects reduce the displacements for the reducing yield acceleration case by a factor of about 0.7.

## 10.4 Design Earthquake Displacements

An earthquake on the Wellington fault was considered the most likely event to produce the 500 year return period design level spectral accelerations. Most of the other major earthquake sources in the Wellington region are faults expected to produce earthquakes of magnitude 7.0 or greater. In order to make a rough prediction of the duration of strong shaking for design purposes it appeared reasonable to assume that the 500 year shaking level will be produced by a magnitude 7.5 earthquake

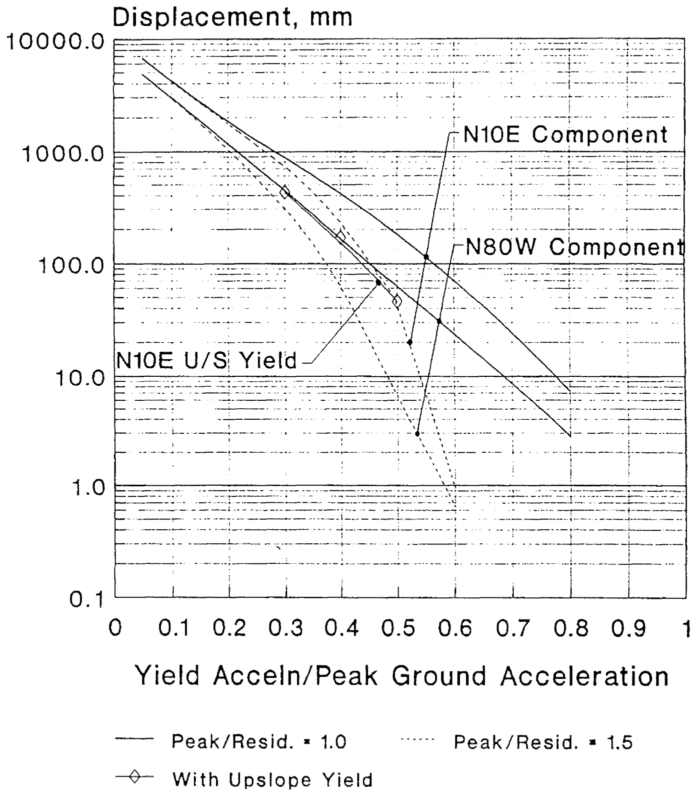


*FIGURE 17. Displacements Computed from Lloello Surface Components*

and the 250 year level by a magnitude of at least 7.0. Correlations given by Krinitzsky et al [1988] indicate that the duration of strong shaking on deep soil sites for magnitudes of 7.0 and 7.5 would be about 19 and 33 s respectively.

From the above information the maximum Lloello displacements were adopted for the 500 year return period design level. Although the Lloello strong shaking is rather longer than expected during a magnitude 7.5 earthquake, a degree of conservatism was applied to compensate for the limitation of analysing records from only a single earthquake.

The design surface PGA's for the 250 and 500 year return period earthquakes were 0.43 and 0.54 g respectively. For a yield acceleration of 0.19 g these PGA's correspond to resistance factors of 0.44 and 0.35. The total outward displacements, based on the upslope yield case with reducing yield acceleration, were 60 and 250 mm for the 250 and 500 year return period earthquakes respectively.

The displacements computed by the above procedure do not include the permanent lateral displacements that may occur in the layers above and below the interface. It is difficult to estimate the magnitude of these other components but some additional movement would be expected because of the likely development of high pore water pressures and partial liquefaction in the layers just above the interface.

### 10.5 Displacement Design Criteria

Based on a consensus of opinion of the Project management, design and peer review teams, sliding displacement limits of 200 and 600 mm were adopted for the 250 and 500 year return period design levels. The displacement analyses indicated that lateral movements would be within the adopted limits providing the strength of the interface layer was improved to give an average residual shear strength of at least 60 kPa.

## 11. LIQUEFACTION

The liquefaction susceptibility of the site was assessed using the Seed "simplified procedure". The ratio between the earthquake induced cyclic shear stress and the effective vertical stress at some depth below the surface is given by Seed and Idriss [1971] as:

$$s_{av} / r_o' = 0.65\ (a_{max} / g)\ (r_o \ / r_o')\ r \qquad (4)$$

where

| | | |
|---|---|---|
| $s_{av}$ | = | average cyclic shear stress |
| $r_o$ | = | total overburden pressure on layer |
| $r_o'$ | = | initial effective overburden pressure |
| $a_{max}$ | = | peak ground acceleration at ground surface |
| $g$ | = | acceleration of gravity |
| $r$ | = | stress reduction factor (varies from 1 at surface to 0.9 at 9.6 m) |

Stress ratios under the various design level PGA's were evaluated using equation (4). The effective stress was calculated assuming the water table to be 2.5 m below ground surface.

The average cyclic stress ratios induced by the 500 year return period design level earthquake were also obtained from the SHAKE88 site response analyses. Peak shear stresses were averaged for each of the two horizontal components from the El Centro, Tabas and Lloello earthquake inputs. The effective vertical stress component was adjusted to include the 70 kPa dead plus live load vertical stress from the building. The average stress ratios from the response analyses are shown in Figure 18. Down to a depth of 15 m there was close agreement between the average stress ratios from the response analyses and values computed by equation (4). Below this depth, the response analyses stress ratios are lower than the equation (4) values but this difference is unimportant as liquefaction was considered unlikely to occur at depths greater than about 15 m.

The liquefaction resistance of the soil was determined from the Seed empirical charts that relate the standard penetration N-value to the critical cyclic stress ratio at which liquefaction is likely to occur [Seed et al 1983]. These charts are expressed in terms of $(N_1)_{60}$, the SPT N-value corrected for overburden pressure and equipment driving energy. Average $(N_1)_{60}$ values computed from the site measured N-values are shown in Figure 19. Values to a depth of 13 m are based on the SPTs carried out following the DC trial as part of the monitoring programme to assess the effectiveness of the remedial work. The deeper values are from the 10 drill holes that formed part of the first stage of the site investigation work.

The Seed empirical charts for assessing liquefaction resistance have been presented in a variety of forms with adjustments to allow for earthquake magnitude, percentage of fines and the limiting shear strain. The empirical chart used for the Museum site evaluation was based on a magnitude 7.5 earthquake, clean sand and 10% limiting shear strain [Tokimatsu and Seed, 1987]. Critical values of shear stress ratio (liquefaction resistance) evaluated from the chart are compared in Figure 18 with the

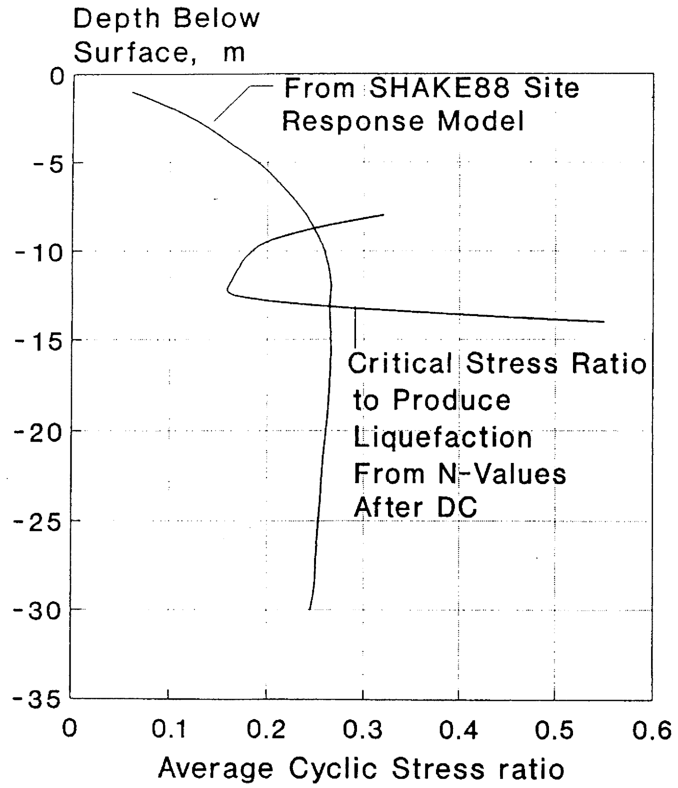


*FIGURE 18. Cyclic Stress Ratios*

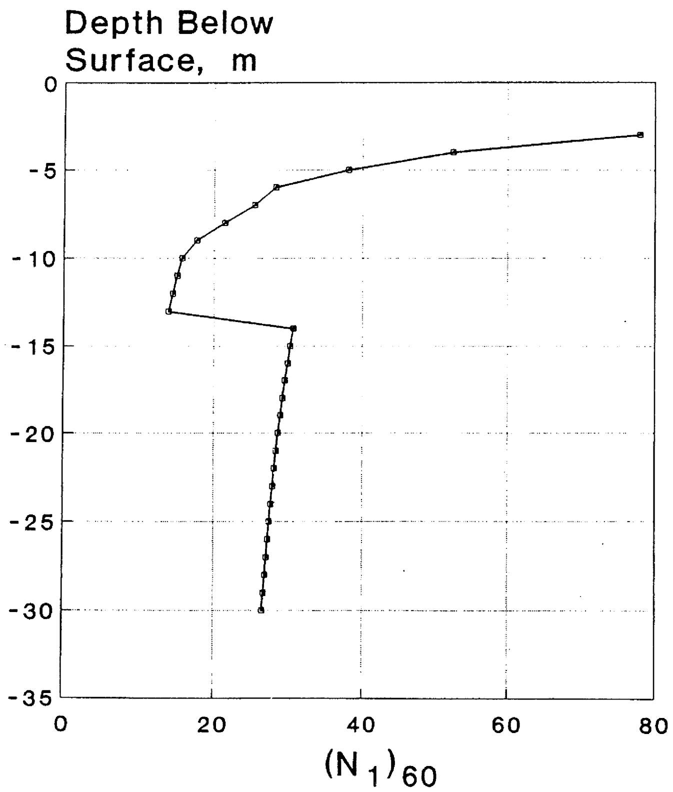


*FIGURE 19. Corrected SPT N-Values After Site Improvement*

average cyclic stress ratios induced by the 500 year return period earthquake (SHAKE88 analyses).

The comparison of induced and critical stress ratios indicates that some liquefaction is possible in the fill material over the depth range 8 to 13 m. The dynamic consolidation procedure for the final site remedial work is to be modified to enhance the improvement over this depth range. In view of the likelihood of greater soil strength improvements than shown by the dynamic consolidation trial, the wide range of grain sizes in the fill material and the "raft" effect of the dense surface layers it was concluded that significant liquefaction would be unlikely. Further, any partial loss of strength at depths greater than 8 m would have little impact on the performance of shallow foundations.

## 12. VERTICAL SETTLEMENTS

Sands and coarse grained soils tend to settle and densify when subjected to strong earthquake shaking. The primary effect of shaking in sandy soils where drainage is prevented is the generation of excess pore water pressures. Settlement will occur as the excess pore water pressure dissipates. Tokimatsu and Seed [1987] have developed an empirical procedure for estimating the amount of settlement likely to occur following the generation of excess pore water pressure in sands. Charts are used to relate volumetric strain to $(N_1)_{60}$ values and the average cyclic stress ratios (see equation 4) generated in an earthquake of specified magnitude.

Using the $(N_1)_{60}$ values shown in Figure 19 and the average cyclic stress ratios given in Figure 18 for the 500 year return period design level earthquake (magnitude 7.5) gave a total earthquake induced settlement of about 100 mm. Nearly 90% of this settlement occurs in the lower reclamation soil layers between a depth of 9 to 13 m where some liquefaction is predicted to occur. A substantial reduction in the estimated settlement is likely if the site improvement work results in greater improvement than shown by the dynamic consolidation trial. In any event, the imposition of differential settlements of the order of about 100 mm across the site was not expected to cause damage to the building structure.

## 13. CONCLUSION

The decision to locate the Museum of New Zealand on reclaimed land on the Wellington waterfront presented a number of challenging problems related to the site stability and earthquake performance of the building during strong ground shaking. By undertaking extensive investigation and applying state-of-the art techniques, a high level of confidence has been obtained in seismic design parameters specified for the building. It was concluded that a level of seismic performance that exceeds the stringent requirements of the design brief can be expected. Care in the assessment and specification of the design forces has also been an important step in the development of innovative design solutions that compensate for the ground motion amplifications expected on the deep soil site.

The detailed site testing of the marine interface layer across the site and within the dynamic consolidation trial area indicated that the weak interface soils can be improved to provide a satisfactory level of stability against seaward sliding. Provided the expected levels of improvement are achieved in the site remedial work, it was concluded that soil settlement and lateral spreading within the reclamation material during strong shaking will be within limits acceptable for the type of structure proposed.

## 14. ACKNOWLEDGEMENTS

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