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Seismic design of a highway in pumiceous land

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ABSTRACT: A case study of seismic design for a highway upgrade in pumice-rich soils is presented. Seismic design in such soils is difficult because of the unique characteristics and lack of existing test information for pumiceous soils. Furthermore it has been demonstrated that conventional measurements of in situ material properties do not adequately characterise the static and dynamic soil properties. Alternative means of determining the actual soil properties are discussed.

Numerical modelling was used to calculate the factor of safety, deformations and excess pore pressure generated during the design earthquake for embankments subjected to seismic loading. The numerical model output is compared with predictions based on Makdisi and Seed's simplified procedure. Liquefaction potential analysis via conventional methods is also presented, with checks via numerical analysis.

Loadman portable falling weight deflectometer, in situ CBR test, plate bearing test and nuclear densometer proved to be reliable in situ testing for the pumiceous sands while various other tests were found to be unreliable. Particle size analysis and Cone Penetrometer Tests (CPT's) do not adequately determine the liquefaction potential for a pumice sand, which is strongly dependent on relative density. A rigorous finite element analysis may provide an alternative to laboratory testing. Makdisi and Seed's simple procedure for predicting permanent deformation of an embankment proved to be applicable to the pumiceous sands.

1 INTRODUCTION

The Rangiriri to Ohinewai four laning project is part of Transit New Zealand's strategy for upgrading State Highway 1 between Auckland and Hamilton (Figure 1). These works are designed by Sinclair Knight Merz and constructed by the Fletcher Higgins Joint Venture. The project comprises the duplication and upgrading of existing highway to modern dual carriageway standards, the construction of a number of intersections and one grade separated junction with associated ramps. The ground conditions of the site, which is predominately low lying ground adjacent to the Waikato River, are characterised by pumice alluvial soils and high groundwater table, particularly in winter. The northern part of the alignment is at grade but south of Ohinewai the road climbs to cross McDermotts Ridge and allow a future extension over the main rail line to the east. Low-lying areas are given over to farming and a network of ditches and pumps control a high water table. The new embankments, ranging from 1 m to 8 m high, have been constructed from imported fill of completely weathered greywacke or reworked pumice sands. The principal's requirement for this project specifies 0.1 g as a design seismic load. However, studies by various authors suggest that a higher seismic load may be appropriate, as discussed further in Sections 3 and 4.

2 GEOLOGY

2.1 Regional Geology

Pumice sands, silts and gravels of the volcanically derived Taupo Pumice Alluvium dominate the majority of the highway alignment. Small areas of swamp and peat deposits occasionally overlie these alluvial sands. Towards the south, slightly higher ground (McDermott's Ridge) is formed by undifferentiated units of the Walton Sub-group and Hamilton Ash which have weathered to form soft silts and clay soils.

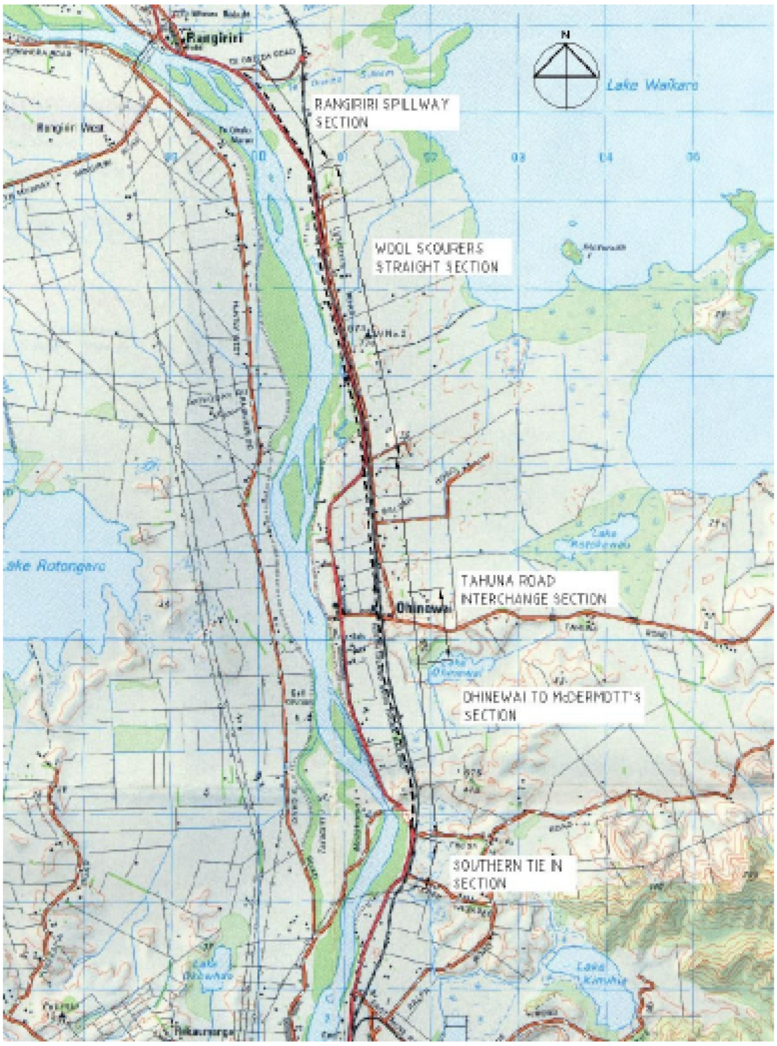

Figure 1 Location Plan - Rangiriri to Ohinewai

2.2 Taupo Pumice Alluvium

At least 18m of alluvial pumice silty sands and gravels of the Taupo Pumice Alluvium formation underlie the alignment. These comprise of light brown or grey, homogeneous pumiceous fine to coarse sands and silts which range in density from loose to dense. The pumice content of the sands decreases with depth whilst the quartz content increases. They have a high permeability ($K = 10^{-3}$ to 3×10^{-6} m /sec) and are usually saturated close to the original ground surface in winter. Occasional organic and plastic silt layers occur and become more common with depth.

2.3 Groundwater

The contract area is low-lying with groundwater within a few meters of the existing ground surface. Levels in the Waikato River, some farm pumping and seasonal rainfall influence the watertable. Runoff from the existing highway enters side ditches and rapidly soaks away, when the watertable is low. Equally high levels in the adjacent Waikato River are rapidly reflected in groundwater tables beneath the new road. In a flood event the river will rise, driving a corresponding rise in the watertable and in extreme events the water level will be at or above ground level.

3 LOCAL SEISMICITY

A review of the historical earthquakes of magnitudes between 4 and 6.9 between 1840 and 1994 was completed using the National Earthquake Information Database for epicentre data, for a polygonal area extending 100km from Rangiriri. Table 1 shows the data obtained from this search.

Table 1. Shallow Seismicity 1840-1994

No	Magnitude	No. of Earthquakes
1	4.0 – 4.9	64
2	5.0 – 5.9	4
3	6.0 – 6.9	0

The Rangiriri site appears to fall in a region with a low level of activity between the Taupo Volcanic Zone, and the seismically quiet area of western North Island. The most recent large earthquake (155km away) was the 1987 Edgecombe Earthquake of magnitude 6.5.

The nearest known active fault capable of generating earthquakes to the site is the Kerepehi fault, which runs in a north-northwest direction through the middle of Hauraki basin in the Firth of Thames. The distance to the fault from the site ranges from 40km to 70km. The average recurrence interval for major earthquake events is approximately 6000 years. Table 2 shows historical data on seismic events on the Kerepehi Fault, reported by the Institute of Geological and Nuclear Sciences. It appears that the fault segment 3 can generate a maximum earthquake of magnitude 6.9.

Table 2. Magnitude and Recurrence Interval of Kerepehi Fault

Fault Segment	Segment Length, km	Calculated Maximum Earthquake Magnitudes	Displacements	Last Earthquake Occurrence	Average Recurrence Interval	Epicentral Distance from the Site, km
Kerepehi 1	36	6.2	-	-	-	40
Kerepehi 2 (Te Poi Segment)	25	6.1	-	-	-	40
Kerepehi 3 (Waitoa Segment)	25	6.9	1.8m to 3-4m	1,800 –4,800 yrs	4, 500 – 9,000 yrs	50
Kerepehi 4 (Elstow Segment)	23	6.5	0.2m –0.7m	1,400, 5,600 and 9,000 yrs	1,200 – 4,200 yrs	70

The regional seismicity indicates that the seismic source in the Firth of Thames has a record of generating a number of earthquakes up to Magnitude 4.9 with occasional occurrence of earthquakes between Magnitude 5.0 and 5.9.

4 SEISMIC GROUND MOTIONS

Peak ground acceleration (PGA) is the most common measure of the severity of ground motions from seismic activity. Attenuation relationships are generally used to estimate the PGA. Most attenuation relationships consider that PGA is a log-normally distributed function of the energy source, distance between source and site of interest, with adjustments for local geology (soil or soil site), fault type and other factors. Attenuation relationships are derived from regression analyses of strong motion data from past earthquakes.

McVerry et al (1995) developed simple attenuation relationships for peak ground acceleration from strong motion records obtained since 1966. This data has been used to estimate the PGA for this site. A design PGA of 0.15g was adopted for the pseudo-static analysis, which approximately corresponds to an average return period of 100 years. Given the conservative nature of pseudo-static analyses, use of the 100 year average return period provides an adequate design basis for a new highway embankment.

A geological hazard model has been developed by the Institute of Geological and Nuclear Sciences Limited to estimate the future seismic hazards. This indicates that the site will have a 10% probability of peak ground acceleration exceeding 0.2 g within 50 years.

5 CHARACTERISTICS OF PUMICEOUS SAND

Marks et al (1998) provided a brief summary of the general characteristics of pumiceous sands;

- Lightweight
- Highly frictional
- High water absorption
- Coarser grains are highly degradable, compressible, and erodable.

These characteristics are largely due to the vesicular nature of the pumiceous sand grains. Site observations, laboratory and field-testing and subgrade field trials undertaken for the design of the works indicate that the Taupo Pumice Alluvium shares these characteristics at this site. The results of the laboratory, field testing and subgrade field trials for this project are discussed in the following sections.

5.1 Laboratory, Field Testing and Subgrade Field Trials

Due to the difficulty of recovering undisturbed samples of pumice sands, in common with most sands, the amount of laboratory testing completed was limited to primarily classification and compaction tests with most of the strength testing done in situ. Classification tests comprised moisture content, solid density and measuring the gradation of the soils. Re-compaction properties were tested using standard and heavy compaction tests and CBR tests for potential fill use.

Field ground investigations involved boreholes, trial pits, Scala penetrometers and Cone Penetration Tests (CPT's) at intervals along the route. Prior to the start of the main earthworks full-scale subgrade compaction trials were undertaken to determine the properties of in situ and re-compacted sands. The following laboratory and field tests were used, providing a unique opportunity to measure the performance of test methods and compaction techniques;

- In situ CBR
- Plate bearing test
- Nuclear densometer

- Loadman portable falling weight deflectometer
- Clegg Hammer
- Humboldt Geo-gauge
- Scala penetrometer
- Moisture content
- Sand replacement density
- Grading
- Compaction

Testing using Clegg Hammer and Scala penetrometer proved unreliable in determining the in situ density and strength, probably due to crushing of the sands. CPT testing results were poorly correlated with other density tests for similar reasons. The in situ CBR tests performed better, but due to the small diameter of the plunger crushing effects are considered to have slightly affected the results. The Humboldt Geo-gauge was unable to effectively measure stiffness because of the very low stiffness and high porosity, unique to the pumice sand. Results from the grading indicate the pumice sand is predominantly fine to medium grained, occasionally coarse grained.

The most effective in situ tests for measuring compaction improvement proved to be the nuclear densometer, sand replacement, Loadman portable falling weight deflectometer and plate bearing tests. The nuclear densometer was calibrated against the sand replacement tests and was found to be a quick, reliable indicator of the dry and bulk density of the pumice sands. The plate bearing, in situ CBR and Loadman tests provided good resolution of stiffness, and there appear to be some form of correlations between the tests, as shown in Figures 2 and 3.

Figure 2 Loadman vs Plate Bearing Elastic Modulus

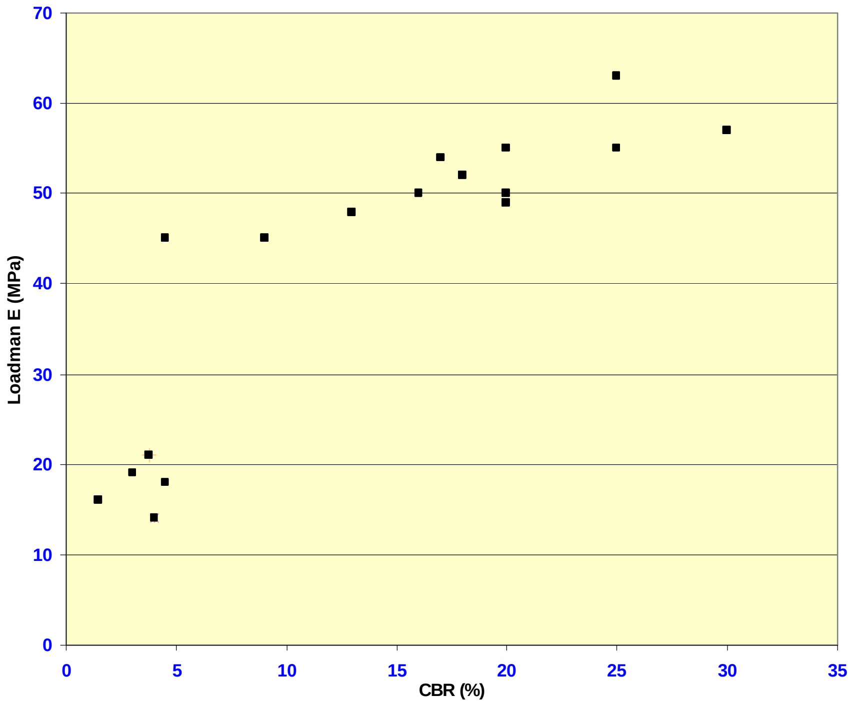

Figure 3 CBR versus Loadman

The Loadman, being a dynamic test, provided higher stiffness values than the plate bearing tests as would be expected. The plate bearing tests were back analysed using bearing capacity and elastic theory to determine the effective cohesion c' and friction angle ϕ'. These values, used in the finite element analysis, were similar to other published values. The parameters used for the design are summarised in Table 3. The parameters below equate to unimproved soil. A range of parameters were tested to evaluate the effects of ground improvement. It was concluded that the design was acceptable for the range of parameters analysed.

Table 3. Summary of Assumed Parameters for Pumiceous Sands

Soil Type	**Bulk Density** g **(kN/m^3)**	**Effective Cohesion,** **c' (kN/m^2)**	**Friction Angle** f**' (°)**
Taupo Pumice Alluvium	15	5	45

A large number of field verification tests were performed during the construction of the highway. The field verification tests were generally in good agreement with the trend of CBR versus Loadman modulus shown in Figure 3

6 LIQUEFACTION ANALYSIS

With the high groundwater table and presence of loose, low density pumice sands the potential for liquefaction of foundation soils under seismic conditions was considered using three methods;

6.1 Particle Size Distribution

The liquefaction potential of the pumiceous sands was evaluated using the particle size distribution analysis proposed by Tsuchida (1970). The results are shown in Figure 4.

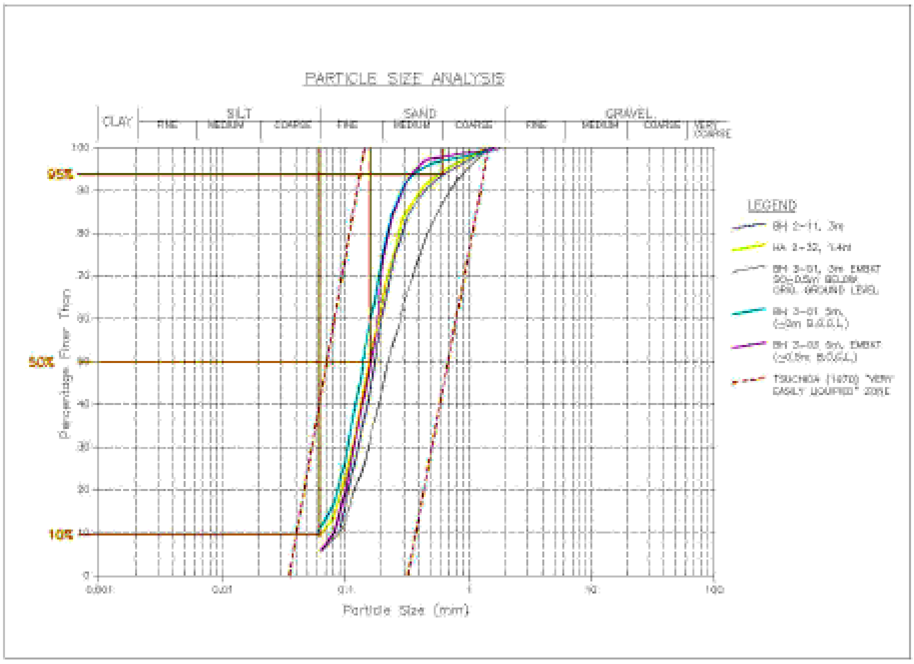

Figure 4 Particle Size Distribution Analysis

The particle size analysis indicates that the pumiceous sands are well within the highly liquefiable particle size range. However, Tsuchida's particle size analysis was developed for normal hard-grained quartz sands, and there is limited experience of its application to highly frictional pumiceous sands. Therefore the liquefaction potential was further evaluated by other means as described below.

6.2 **Cone Penetration Test Correlations**

There are several published methods for evaluating liquefaction potential based upon CPT correlations in common use. These methods, however, have been developed from analysis of normal hard-grained quartz sands and do not consider the unique characteristics of volcanic pumiceous sands. Wesley et al (1998) demonstrated that published relationships between cone resistance, relative density, and confining stress are unreliable for a pumiceous sand. The CPT data, despite its inability to distinguish between loose and dense deposits, was used to assess liquefaction potential using the NCEER method (Youd and Idriss, 1997) for a magnitude 6.5 earthquake with a peak ground acceleration of 0.2 g. This method showed that the pumice sands were susceptible to liquefaction. The reliability of this method with regard to this type of material is questionable, though, at this moment there is no better method.

6.3 **Finite Element Analysis**

Since the in situ testing indicated that there was a potential for liquefaction, the ground was improved by undercutting and recompaction. The ground improvement increased the density and stiffness sufficiently to minimise the risk of liquefaction and provide a uniform subgrade. In addition to the in-situ testing correlation, finite element analysis was carried out as a check analysis. The review of the calculated strains and excess pore pressure from the analysis confirmed that liquefaction would not be an issue in the improved area. A hyperbolic type stress-strain soil model was used in the analyses.

7 SEISMIC DESIGN OF EMBANKMENTS

Analysis of embankment stability was initially carried out assuming the pseudo-static load of 0.1 g, which is the principal's requirements for the project. A second analysis was subsequently carried out using the higher pseudo-static load of 0.15 g, which is based on the authors' assessment of the regional seismicity.

7.1 Scope of Analysis

The response of a new embankment (Tahuna Interchange) to seismic loading was modelled using finite element code PLAXIS. The finite element analysis of the embankment comprised two parts. Firstly the embankment subjected to pseudo-static horizontal loads of 0.1 g and 0.15 g was modelled. Secondly the horizontal ground displacement records of a past M6 earthquake (PGA = 0.2 g) were imposed on the base of the model embankment as the boundary condition, and the response of the embankment was computed. As noted earlier, earthquake magnitude of this order is expected to occur within foreseeable future, and there is about 10% probability that earthquake with a PGA exceeding 0.2 g will occur at the site within the design life. The results of the finite element modelling of the embankment are compared to the theoretical responses calculated after Makdisi and Seed (1977), which is based on Newmark's sliding block model (1965). The finite element model of the 8 m high embankment at Tahuna Road Interchange is shown in Figure 5.

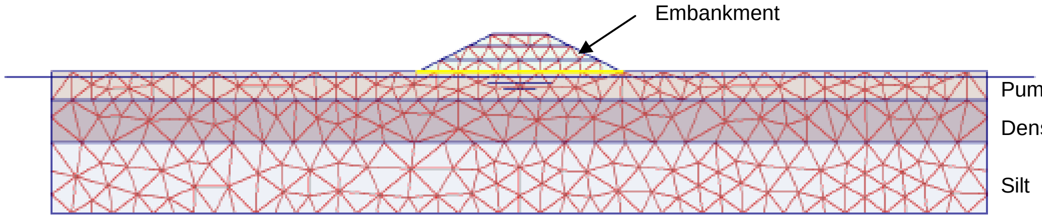

Figure 5 Finite Element Model for the 8 m High Embankment

7.2 Pseudo-Static Analysis

The factor of safety for the embankment was checked by applying a pseudo-static load of 0.1 g and 0.15 g to the finite element model. The pseudo-static loads were applied to the entire embankment model as horizontal gravitational forces. To determine the factor of safety for each case, shear strength parameters c' and ϕ' of all soils were incrementally reduced until a failure mechanism developed. The ratio of c' to ϕ' was maintained at a constant value during this artificial strength reduction process. The factor of safety is defined as the ratio of the initial shear strength parameters to the artificially reduced strength parameters that induce failure. The factor of safety was calculated for two alternative embankment construction options (Table 3).

Table 3. Calculated Factor of safety for the Embankment

Embankment Fill Material	Factor of Safety	
	0.1 g	0.15 g
Option 1 – Greywacke Fill	>1.5	1.2
Option 2 – Pumice Sand Fill	>1.5	>1.5

The calculated factor of safety is adequate in all cases. Nevertheless, various authors have demonstrated the limitations of the pseudo-static approach to predict the seismic behaviour of embankments (Makdisi and Seed, 1977).

7.3 Permanent Deformation of the Embankment

The expected permanent deformation of the 8 m high embankment was also calculated using Makdisi and Seed's simplified procedure. The procedure was applied directly to the first embankment material option, that is, the greywacke fill. For the alternative pumice sand fill option, the permanent deformation was calculated from the same procedure, but the non-linear relationship between shear modulus and shear strain for a pumice sand published by Marks et al (1998) was used instead of Makdisi and Seed's relationship for quartz sands. The calculated permanent deformation for each embankment material option is presented in Table 4. The ranges of the displacement calculated from the Makdisi and Seed procedure are due to a scatter in the past earthquake data used in the procedure.

Table 4. Calculated Permanent Horizontal Displacement of the Embankment

Embankment Fill Material	Maximum Embankment Height	Permanent Horizontal Displacement at Crest	
		Makdisi & Seed	Finite Element Analysis (M6 Earthquake Motion)
Option 1 – Greywacke Fill	8 m	2 ~ 50 mm	5 ~ 15 mm
Option 2 – Pumice Sand Fill	8 m	~ 0 mm	1 ~ 20 mm

Despite some simplifications made in the Makdisi and Seed procedure, the calculated permanent embankment deformations are reasonably consistent with the finite element analysis results, indicating that simple analysis procedures for predicting the dynamic response of an embankment such as this are sufficient for these small embankments. Furthermore Makdisi and Seed's procedure has been demonstrated to give reasonable estimates of permanent deformation of embankments in pumiceous soils provided the soil characteristics are determined by appropriate means.

The displacements calculated were consistent with observed displacement in the recent Port Villa earthquake for similar sized embankments on soft soils.

8 CONCLUSION

A series of in situ testing of volcanic pumiceous sands was carried out at the Rangiriri to Ohinewai highway upgrade site, New Zealand. Among the common testing methods performed, plate bearing test, in situ CBR and Loadman portable falling weight deflectometer provided useful correlations of the characteristics of the pumiceous sands. Nuclear densometer testing also provided a quick, reliable means of measuring the in situ density of the soil. The other methods, which include Clegg Hammer, Scala penetrometer, and Humboldt Geo-gauge proved unreliable in the pumiceous sands.

Liquefaction assessment methods developed for ordinary quartz sands, such as particle size analysis and CPT correlations, are not applicable to pumiceous sands because of the pumice sands unique characteristics. Marks et al (1998) demonstrated by laboratory testing that the liquefaction potential of pumiceous sands is largely dependent on relative density, which cannot be accurately measured by CPT's. Finite element analysis may provide an alternative to laboratory testing of reconstituted samples. For this purpose, a robust analysis would be required to account for cyclic degradation of the soil and specific strain-dependent behaviour of pumiceous sands.

Whilst pseudo-static analysis of the embankment provided an indication of the factor of safety under seismic loading, pseudo-static approaches have been demonstrated to be of limited use by various authors. A simple analysis procedure proposed by Makdisi and Seed was tested in this case study. The calculated permanent deformation of the embankment was consistent with finite element analysis, implying that simple procedures such as that of Makdisi and Seed are useful and applicable for predicting dynamic response of embankments in pumiceous soils, provided that the characteristics of the soils are determined by appropriate means.

9 ACKNOWLEDGEMENTS

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