Seismic Stability of the Sulphur Point Wharf

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ABSTRACT: The Sulphur Point Wharf development is a major expansion of the Port of Tauranga Ltd's port facilities in Tauranga Harbour involving an area of some 60 hectares. Initial development of the reclaimed site involves the construction of some 600 metres of wharf with a design dredge depth of 14.5 metres.

The Tauranga Harbour consists of deep deposits of predominantly sands interspersed with layers of silts and fine gravels. The wharf structure consists of a reinforced concrete deck supported on driven precast prestressed concrete piles. The sands at the site which form the marginal slope under the open piled wharf structure are protected with rock buttressing.

This paper describes the seismic design philosophy developed for the wharf structures and the design criteria adopted. "Operating" and "contingency" design earthquakes are defined together with performance expectations relative to the design life of the facility. The features and considerations associated with the assessment of seismic stability in the geotechnical engineering design process are presented.

Initial investigations at Sulphur Point indicated that the recent sands did have a liquefaction potential. Site investigations and the evaluation of soil properties with respect to seismic shaking is described. Extensive piezo cone penetrometer testing together with static and dynamic triaxial testing of undisturbed piston samples has enabled the behaviour of the soils to be assessed. Insitu geophysical testing including shearwave velocity measurement is described.

The wharf structure/soil interaction associated with the marginal slope and soil behaviour have been addressed in the design process. Wharf stability and estimated deformation behaviour under the operating and contingency design earthquakes is presented.

1. INTRODUCTION

The Port of Tauranga is a major port facility located in the Bay of Plenty area of the North Island of New Zealand (Figure 1). Wharf construction has concentrated on the Mount Maunganui side of the harbour since development of the port commenced in 1950. Presently there is some 2000m of concrete wharf at Mount Maunganui providing a nominal 11 berths.

To enable further expansion of the Port of Tauranga the development of facilities at Sulphur Point on the Tauranga side of the harbour has been initiated. The construction of 600m of wharf and associated land facilities involving an area of some 60 hectares is now well advanced.

Works Consultancy Services Ltd has assisted the Port of Tauranga Ltd with the development of the wharf design philosophy, site investigations and the geotechnical evaluation of the site. Assistance and advice has been provided by Works Consultancy Services Ltd on geotechnical design aspects.

Sulphur Point has been developed with extensive reclamation utilising sands dredged from the harbour. The extensive deposits of recent sediments in the Tauranga Harbour at Sulphur Point present a significant challenge for the development of the site. This paper describes the seismic design consideration for the development of Sulphur Point. Stability of the wharf structures is discussed.

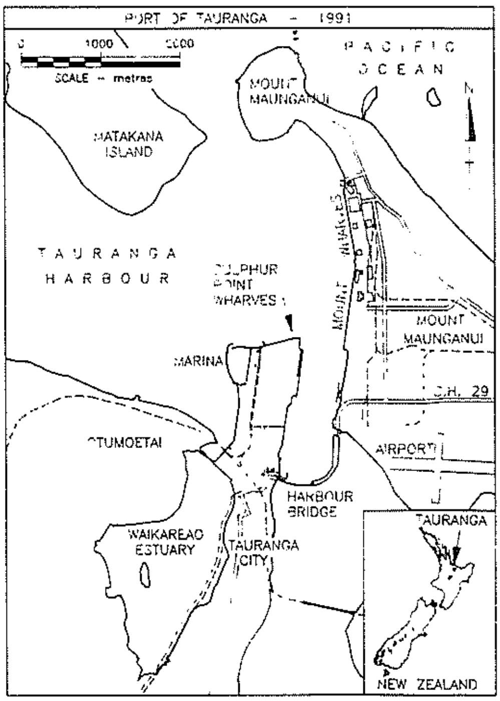

Figure 1 : Location Plan

2. SEISMIC DESIGN CRITERIA

A design philosophy for the Sulphur Point development was adopted which reflected the operational requirements of the Port of Tauranga Ltd. Aspects considered included:

- seismic hazard
- seismic design philosophy
- geotechnical aspects
- structural aspects
- performance requirements
- design life

For the Sulphur Point facilities two levels of earthquake design loading were adopted. The two levels have been described as the operating level earthquake and the contingency level earthquake.

Operating Level Earthquake: This loading was assessed to have a probability of exceedence of 50% during the 50 year design life of the facility. As a result of this event it is expected that minor repairable damage would be experienced and the facility would remain operational.

Contingency Level Earthquake: This more severe loading was assessed to have a probability of exceedence of 10% during the 50 year design life. In the case of this event extensive damage would be expected but the facility would be resistant to collapse.

Seismic design parameters adopted for the design [based on Matuschka *et al* (1985)]are summarised in Table 1.

3. GEOLOGY

The Port of Tauranga is located in the Tauranga Harbour, a drowned Pleistocene river basin. The basement to the basin consists of a number of volcanic rock units formed principally in the time interval 1 to 6 million years before present day (Houghton and Hegan, 1980). Soft, weakly consolidated fluvatile and estuarine sediments (the Tauranga Formation) consisting of deep deposits of predominantly sands (often pumiceous) interspersed with shells, layers of silts and occasional layers of fine gravels.

Investigations have shown this description to be typical of the Sulphur Point site where borehole K1 encountered these materials to a depth of 49.5m. The soils have been broadly grouped:

Upper materials	Recent beach sands
Lower materials	Older alluvial silts, sands and gravels

The interface between the soil groups typically varied between RL -12m and RL -18m.

4. SOIL PROPERTIES

A variety of investigations were carried out for the wharf development including:

- boreholes (both washdrilled and cored)
- standard penetration tests (SPT)
- piezo cone penetration tests (uCPT)
- geophysical profiles
- seismic shear wave velocities
- laboratory tests (grading, classification, static and dynamic triaxial tests)

Samples for triaxial testing were recovered by thin walled piston sampler. Excellent samples were recovered and sampling from the HQ3 triple barrel coring was also remarkably effective.

The location of tests along the wharf structure are indicated on Figure 2.

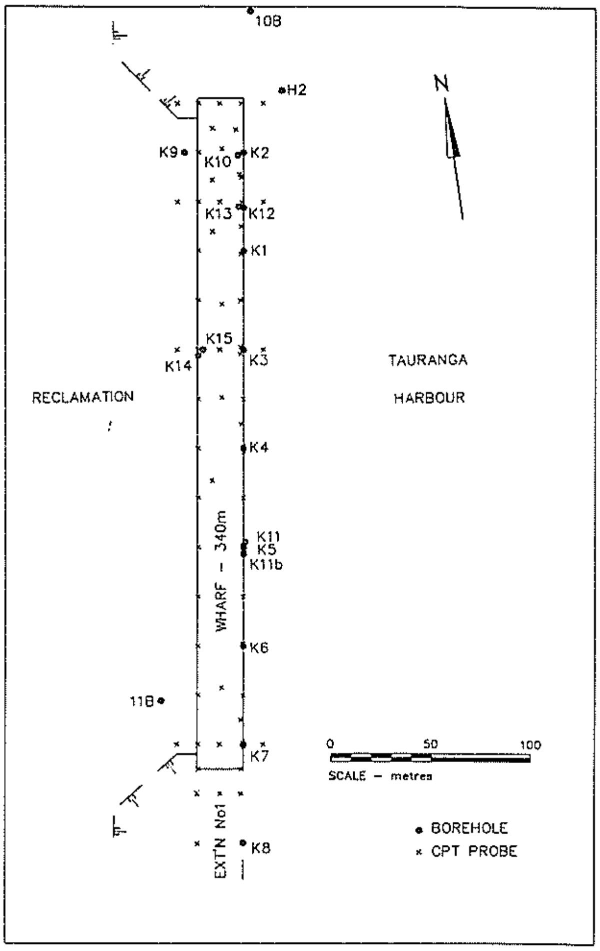

Figure 2 : Location of Tests

Event	Probability of Exceedence during design life	Return Period (t_r)	Peak Spectral Acceleration	Design Peak Ground Acceleration	Magnitude M
Operating Level	0.50	70 years	0.5g	0.20g	6.5
Contingency Level	0.10	450 years	0.9g	0.36g	7.5

Table 1 : Seismic Design Parameters

Typical soil data at location K3 are presented in Figure 3.

A feature of the sands identified at Sulphur Point was their relatively sharp grain profiles. This was particularly noticeable where the sands contained significant volcanic glass.

5. LIQUEFACTION POTENTIAL

Liquefaction is a situation where cohesionless soils lose their shear strength through the generation of pore water pressures when subjected to (earthquake) shaking. There are a number of techniques for evaluating the liquefaction potential for cohesional soils. Based on the site investigation data available these techniques were applied to the Sulphur Point soils.

5.1 Grading Criteria

The data, plotted in Figure 4, indicates that the upper SANDS (ie down to RL -12m to RL -18m) consistently satisfy the grading criteria for easily liquefiable sands (Iwasaki, 1986). The fines content increases in the lower soils and this will provide a higher resistance to liquefaction.

5.2 CPT Evaluation

An evaluation of the level ground liquefaction potential of the soils at the site based on CPT cone resistance, q_c, data has been made using the approach presented by Franklin (1986) based on an evaluation of q_{c1} (where q_{c1} is the limiting cone resistance at 100 kPa overburden stress). Liquefaction potential has been assessed for the case behind the wharf, ie formation level RL +4.5m and the limiting CPT values are shown on Figure 3.

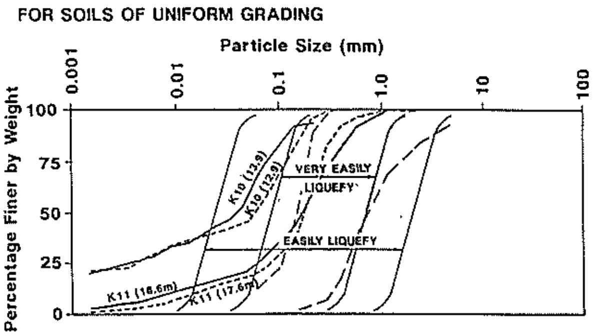

Figure 4 : Soil Particle Size

From this appraisal it is apparent that:

i) For the operating earthquake there are layers which are weak and will experience liquefaction to a depth of RL -15m (cf K7).

ii) For the contingency earthquake the CPT evaluation indicates that liquefaction will be extensive and extends to a depth in excess of RL -25m.

5.3 SPT Evaluation

The SPT results indicate similar liquefaction potential to that derived from the CPT data. However there is large scatter in the SPT results and this complicates the interpretation and raises concerns over the reliability of the data and the method of interpretation (Martin, 1991).

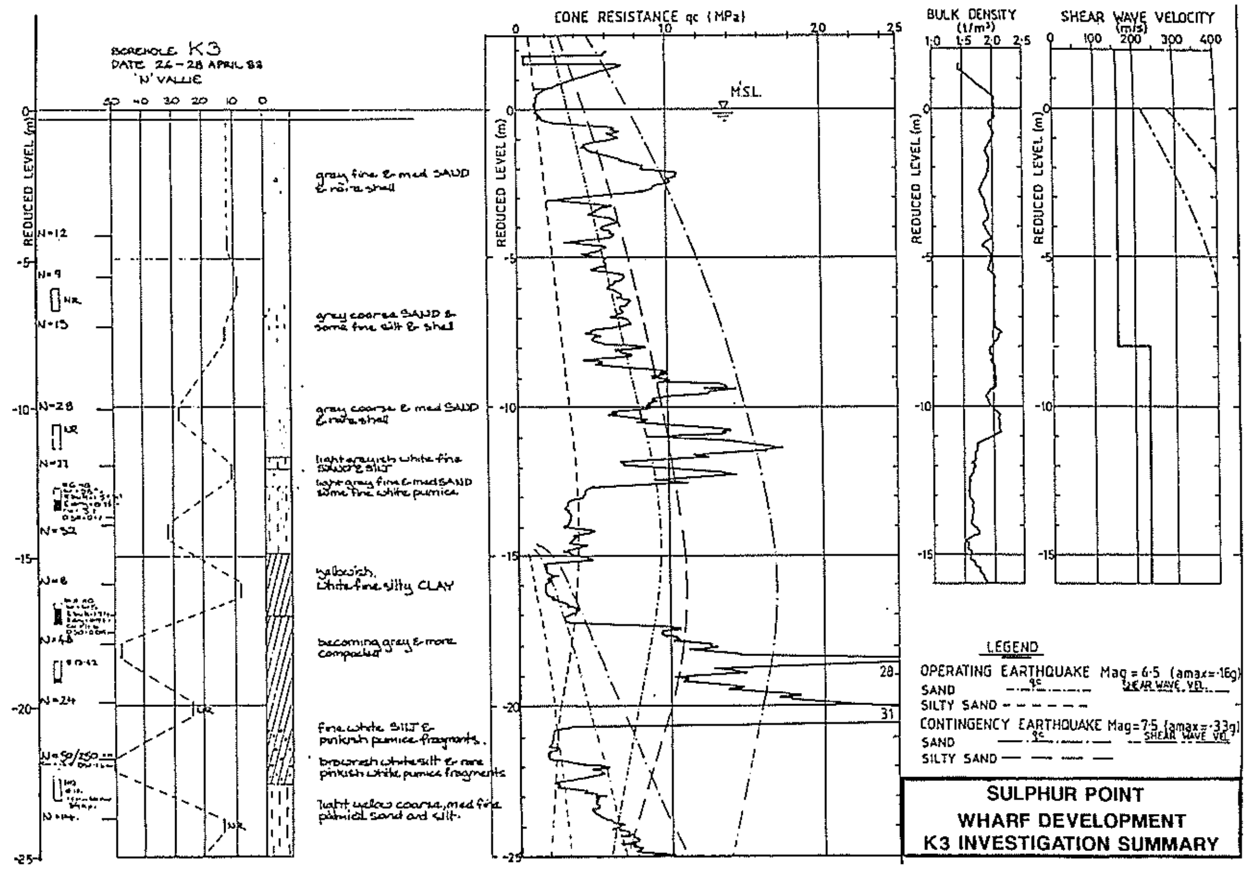

Figure 3 : Typical Soil Data

5.4 Shear Wave Data

Typical shear wave velocity for sands ranges are presented in Table 2. The data from Sulphur Point was consistent with this and typical results are shown on Figure 3.

Material Type and Age	Shear Wave Velocity, Vs
Very recent non-compacted sands	90-200 m/s
Other Holocene sands (<10,000 yrs)	150-300 m/s
Pleistocene sands (>10,000 yrs)	180-450 m/s

Table 2 : Typical Ranges of Vs in Standard Sands (after NRC, 1985)

From laboratory testing it has been found that there is a threshold shear strain, γc, of 0.01% below which there is no excess pore water pressure generated (Dobry, 1985 and NRC, 1985). Using this approach shear wave velocities have been calculated for a threshold strain of 0.01% for the operating and contingency earthquakes as illustrated in Figure 4.

From an assessment of the shear wave velocity data in Figure 3 it is observed that there is an extensive pore pressure generating potential at various levels under both the operating earthquake and the contingency earthquake.

5.5 Dynamic Triaxial Data

Dynamic triaxial isotropically consolidated tests were undertaken on a range of soil types obtained by piston sampling. The pore pressure response is summarised on Figure 5.

The pore pressure response is consistent with that expected from the literature. The recent beach sands are more susceptible to liquefaction than the recent silty sands.

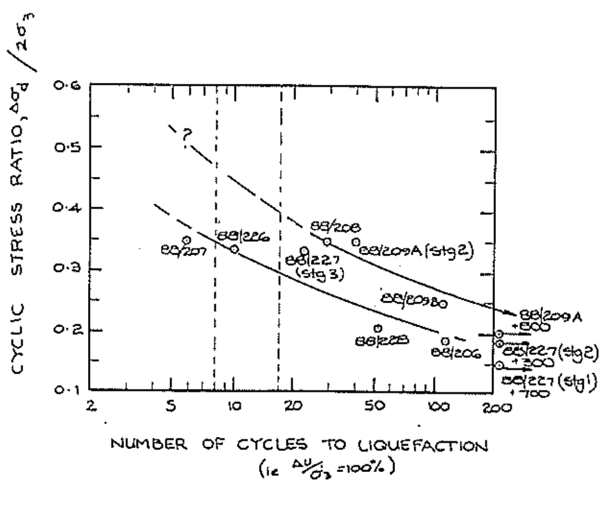

Figure 5 : Dynamic Triaxial Data

From the dynamic triaxial pore pressure response data presented on Figure 5 factors of safety against liquefaction have been calculated:

a) Operating earthquake:

$$FS = \frac{\text{Test strength at 8 cycles}}{\text{Design shear stress}}$$

$$= \frac{0.35}{0.33} = 1.06$$

b) Contingency earthquake:

$$FS = \frac{\text{Test strength at 15 cycles}}{\text{Design shear stress}}$$

$$= \frac{0.28}{0.60} = 0.47 < 1.0$$

The dynamic triaxial data is consistent with the other observations. It indicates that performance under the operating design earthquake will be marginal and significant liquefaction will occur for the contingency design earthquake.

5.6 Discussion

From the evaluation of the investigations data presented in the previous sections it is observed that the soils at Sulphur Point do have a level ground liquefaction potential.

Under the operating earthquake it is observed (CPT data) that some layers of material behind the wharf have a liquefaction potential whereas under the contingency earthquake liquefaction potential is extensive.

6. WHARF STRUCTURE FORM

A variety of wharf structures were considered for the Sulphur Point site., The traditional suspended concrete deck supported by concrete piles, with raked piles providing lateral load resistance, was discarded in favour of the preferred ductile structure (Figure 6).

Vertical prestressed concrete piles are detailed for ductility and during earthquake loading the wharf is restrained in position by the sloping marginal embankment. The marginal embankment is excavated and then lined with rock fill for protection and reinforcement. The displacement performance of the wharf is dependent on the behaviour of the slope.

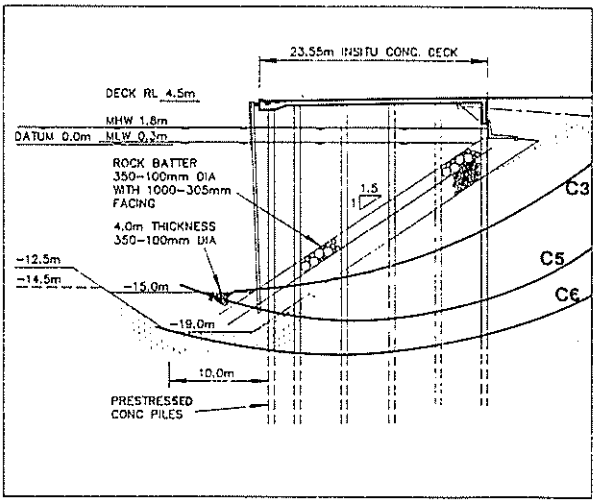

Figure 6 : Wharf Structure

Tie backs have been used to increase the lateral stiffness of the wharf against service loads arising from shipping.

A tied back sheet pile wharf option was appraised in detail. However there were technical and cost problems coping with the liquefied soil pressures during earthquake loading both in terms of sheet pile strength and deadman design for walls with dredged depths of 14.5m alongside.

7. WHARF STRUCTURE STABILITY

The stability of the rock buttressed slope and its associated stability under seismic loading was assessed. The stability of the rock slope was considered for two situations:

i) Stability under seismic loading without increased pore pressure.
ii) Post seismic stability with increased pore pressures.

The stability of the slope is complicated by the presence of the piles which will provide dowel action and some strengthening. For the purposes of the preliminary appraisal the piles were ignored.

The stability analysis indicated:

i) The critical acceleration for the rock slope assuming a $\phi = 36°$ for the sands is approximately 0.07g (ie the horizontal acceleration to reduce the stability to Factor of Safety = 1.0).

   With this critical acceleration slope displacements have been estimated [Bracegirdle (1980)] as follows:

   - operating earthquake-displacement $\approx$ 0.05m
   - contingency earthquake-displacement $\approx$ 0.2m
     (assuming no increase in pore pressures)

ii) A pore pressure rise of approximately 50% in the slope materials reduces the stability to a factor of safety of less than 1. In this situation load is transferred to the piles and full dowel action would be developed. The need for a more detailed study to address this was identified.

8. COLLAPSE STABILITY

8.1 Procedure

While it is accepted that, provided the dynamic loading is severe enough, all sands can experience a 100% pore pressure ratio only "loose" sands will suffer large displacements. This large displacement behaviour associated with very low undrained (residual) shear strength is the catastrophic failure (flow slides) usually associated with liquefaction (Casagrande, 1975). Moderately dense to dense sands exhibit "cyclic mobility", ie only very small strains develop before shearing and dilatancy cause a reduction in the pore water pressure which represents a recovery of the effective stress.

It is difficult to quantify the actual soil behaviour and whether or not collapse behaviour will develop. Poulos *et al* (1985) presents a procedure for evaluating the stability of a soil mass subject to shear stress, such as a slope or embankments. This procedure provides a rational basis for determining the post liquefaction stability of the wharf structure based on the soil characteristics.

Poulos *et al* (1985) describes the calculation of a factor of safety against liquefaction, F_L, (ie factor of safety against catastrophic instability) as:

$$F_L = \frac{\text{Undrained steady-state shear strength}}{\text{Shear stress required to maintain static equilibrium}} = \frac{Su}{\tau_d}$$

Using limit equilibrium stability analyses the driving shear stress, τ_d, can be easily computed. Values in the range τ_d = 19 to 31 kPa have been calculated for the Sulphur Point Wharf.

The undrained steady state shear strength (Su) is considered to be a function of void ratio only (Poulos *et al*, 1985). (This reflects the observation that under severe loading and displacement all soil structure is destroyed and the void ratio is the fundamental property.) The critical step in the procedure for liquefaction evaluation is the determination of the insitu void ratio and the correction of laboratory-measured undrained steady state strengths to take into account changes in void ratio of the soil during sampling and testing. In the case of Sulphur Point a wide range of insitu void ratios exist and no definitive measurements of insitu void ratio have been made. It is considered that sample disturbance associated with the thin walled piston samples was minimal and the samples are representative of the insitu void ratios.

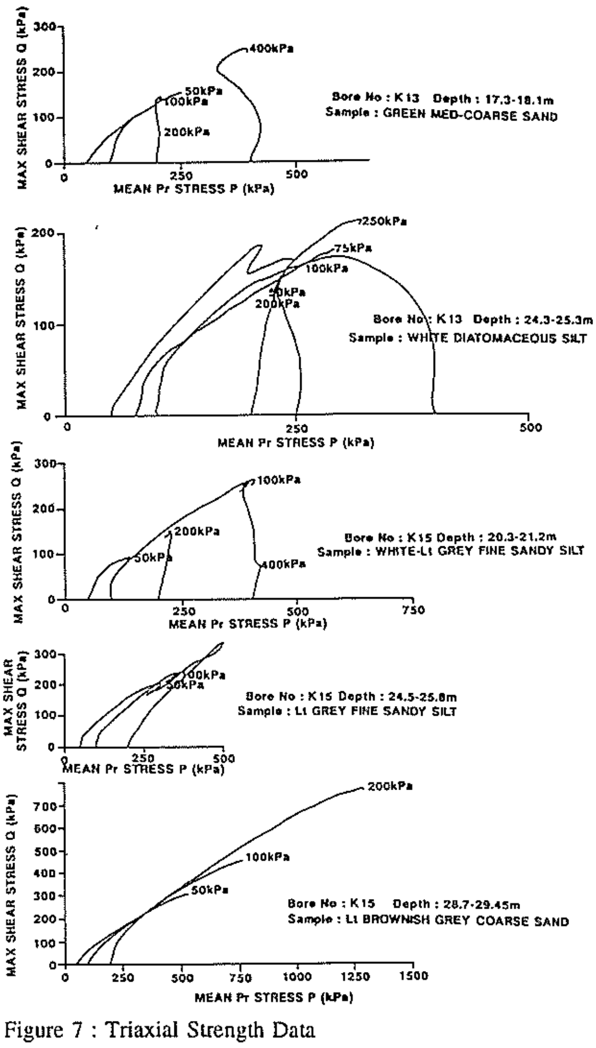

Figure 7 : Triaxial Strength Data

Bore No.	RL (m)	Depth (m)	Description	Void Ratio e	Undrained Strength Su
K13	-14.8 to -15.6	17.3-18.1	Grey green med-course pumiceous sand	1.25 to 1.99	>100 kPa
K13	-22.4 to -22.8	24.9-25.3	White diatomaceous silt	1.95 to 3.89	>100 kPa
K15	-18.3 to -18.2	20.3-21.2	White-light grey fine sandy silt	1.23 to 1.40	≈100 kPa
K15	-21.5 to -22.8	24.5-25.8	Light grey fine sandy silt	1.38 to 1.42	≈150 kPa
K15	-25.7 to -26.45	28.7-29.45	Light brownish grey coarse sand, some gravel	0.55 to 0.67	>300 kPa

Table 3 : Triaxial Undrained Strength

The sample of silt (K12, depth approximately 25m) which was tested by dynamic triaxial testing was also subjected to static undrained triaxial loading with the 42% residual dynamic pore pressure. This sample indicated an undrained strength of some 80 kPa.

8.2 "Undrained Strength" of the Sulphur Point Soils

From conventional consolidated undrained triaxial tests the steady state "undrained" strength can be determined (Poulos *et al*, 1985). An examination of the triaxial results for Sulphur Point indicate a significant range of "undrained strengths" determined from an examination of the stress paths (Figure 7). Undrained strengths have been evaluated as summarised above in Table 3.

These results indicate a characteristic undrained strength of 80 kPa would be appropriate (conservative) in the consideration of post seismic stability.

It is interesting to compare the strength of these Sulphur Point soils with the tentative relationship proposed by Seed (1987) which is presented as Figure 8. Sulphur Point SPT "N" values are typically in the range 16-20 (although higher and lower values are recorded) which indicates residual strengths (Figure 8) in the range 36 to +54 kPa. Based on this, it was considered prudent to check stability for a lower strength of 36 kPa.

8.3 Factor of Safety Against Collapse

Poulos *et al* (1985) consider that $F_L = 1.1$ is acceptable where the insitu void ratios and steady state conditions are well defined and appropriate corrections have been made. Noting that the steady state strength is sensitive to void ratio (ie small changes in void ratio can result in large changes in undrained strength) and that no corrections to the undrained strength have been made it was considered that $F_L > 2.0$ should be achieved for the Sulphur Point stability.

Based on the measured undrained strength values and the calculated mobilised shear stress values (surfaces shown on Figure 6) the following factors of safety are calculated:

Circle C3 $F_L = \frac{80}{23} = 3.5$ $(F_L = \frac{36}{23} = 1.50)$

Circle C5 $F_L = \frac{80}{19} = 4.1$ $(F_L = \frac{36}{19} = 1.9)$

Circle C6 $F_L = \frac{80}{31} = 2.6$ $(F_L = \frac{36}{31} = 1.2)$

This analysis indicates that the Sulphur Point Wharf has an acceptable factor of safety against collapse failure (without considering the strength contribution provided by the piles). It is noted that the stability results in parenthesis based on Figure 8 (Seed, 1987) indicate marginal stability (ie, it is less than $F_L > 2.0$).

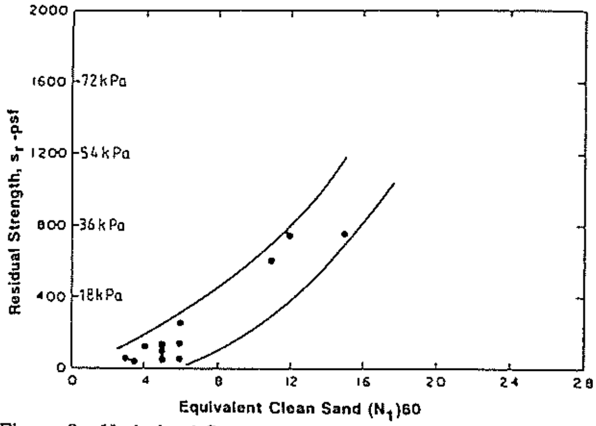

Figure 8 : Undrained Strength Data (after Seed)

9. STRUCTURE DISPLACEMENT

Structure horizontal displacement has been assessed on the basis of the measured soil properties and published data. Displacement estimate are summarised in Table 4.

Event	Axial Strain (Dynamic Triaxial)	Estimated Shear Strain	Displacement
Operating Earthquake	2%	1 - 3%	0.2 - 0.4m
Contingency Earthquake	20%	3 - 20%	2 - 4m

Table 4 : Summary of Wharf Displacement

A structural assessment based on these displacements indicates that the wharf structure will not collapse.

10. CONCLUSIONS

The assessment of seismic stability of structures constructed over recent sediments is complex. While engineering criteria exist for the assessment of liquefaction and the stability of structures on level ground there is less information regarding the determination of the stability of sloping sites.

Based on level ground methodology it is observed that the Sulphur Point site does have a significant liquefaction potential without any closer consideration of the soil properties. This suggests that the wharf will experience collapse.

The procedure developed by Poulos *et al* (1985) provides a rational basis for the assessment of seismic stability based on the consideration of the undrained strength of the soil. Using this procedure it is demonstrated that the Sulphur Point Wharf will remain stable against collapse even under the contingency earthquake.

Displacement behaviour of structures under seismic loading is difficult to assess. Estimates of displacement have been made based on measured and published data.

By consideration of the actual properties of the soils at Sulphur Point it has been possible to demonstrate that the wharf structure adopted for the development will satisfy the seismic design philosophy and criteria adopted.

11. ACKNOWLEDGEMENTS

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12. REFERENCES

Bracegirdle A (1980): "Seismic stability of reinforced earth retaining walls", Bulletin NZNSEE Vol 13, No.4, December 1980.

Casagrande A (1975): "Liquefaction and Cyclic Deformation of Sands - A Critical Review", Harvard Soil Mechanics Series No. 88, Harvard University, 1975 (Also Proc 5th Pan Am Conference on SM & FE, Buenos Aires, 1975).

Dobry R (1985) unpublished data - see NRC (1985).

Franklin A G (1986) : "Use of the piezocone for evaluating soil liquefaction potential", Proc 8th European Conference on Earthquake Engineering, Lisbon, 1986.

Houghton B F and Hegan B D (1980): "A preliminary assessment of geological factors influencing slope stability and land slipping in and around Tauranga City", DSIR NZ Geological Survey Engineering Geology Report No.EG348, October 1980.

Iwasaki T (1986): "Soil Liquefaction Studies in Japan: State-of -the-Art" Soil Dynamics and Earthquake Engineering, Volume 5, No.1, January 1986.

Martin G R (1991): "A practical assessment of site liquefaction effects and remedial measures", Proc 2nd Int Conf on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St Louis, March 1991 (also NZ Geomechanics News No.42, July 1991.)

Matuschka *et al* (1985) : "New Zealand seismic hazard analysis", Bulletin of NZNSEE, Vol 18, No.4, December 1985.

NRC (1985): "Liquefaction of Soils During Earthquakes", Committee of Earthquake Engineering, National Research Council, Washington, 1985.

Poulos S J, Castro G, and France J W (1985): "Liquefaction Evaluation Procedure", Proc ASCE Journal Geot Eng Vol III, No.6, June 1985.

Seed H B (1987): "Design Problems in Soil Liquefaction", Proc ASCE Journal of Geot Eng, Vol 1B, No.GT8, August 1987.