Cone Penetration and Vane Shear Strength Profiles in Auckland Soils

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SUMMARY: This paper has the rather modest objective of finding the relation between the results of two in situ test procedures, the vane shear strength test and the cone penetration test, for sites with cohesive soils around the city of Auckland. Results from two sites with soil profiles typical of those found around the city are compared. Although the profiles are found to be quite variable, even when the soundings are close together, a statistical treatment of the results leads to the conclusion that the cone penetration resistance is about 11 times the undrained shear strength.

1 INTRODUCTION

A commonly used in situ investigation technique around Auckland is the vane shear test. It is used particularly for investigations of residential sites. The usual procedure is to use a manual "posthole" boring procedure and at selected depths to insert a small vane (typical dimensions are 15mm diameter and 25mm high) into the bottom of the borehole. This is a slow process. An appealing alternative is to use a cone penetrometer. With current electronic cone equipment it is posible to get a continuous profile of the soil resistance. For the types of site investigation mentioned above, a light capacity trailer mounted penetrometer rig can easily penetrate to depths greater than can be reached with the hand boring procedure. The light duty penetrometer can also obtain a complete sounding in a fraction of the time required for the manual procedure.

The objective of this paper is to compare the results of vane shear strength profiles with the cone resistance profiles and to obtain the correlation between the two test results.

2 EQUIPMENT

The field tests discussed in this paper were all done with the aid of a trailer mounted drilling rig. This is equipped with a hydraulic ram having a maximum stroke of 3m.

The vane has a diameter of 50mm and a length of 76mm, it is sufficiently robust to withstand direct thrusting into the soil profile so no hole has to be prebored to enable the vane to be advanced from the bottom of the hole. The torque is measured manually at the surface via an internal rod which is connected to the vane itself. Friction in the vane assembly due to the vertical load is reduced by a thrust bearing. Part of the measurement process is to obtain a separate reading of the torque due to friction in the system, this is small in relation to the peak torque recorded. Vane readings were obtained at 20cm intervals.

The cone used is GSCFIP 50-05-14 from Goudsche Machinefabriek of Holland. The cone and sleeve resistance are measured with strain gauge circuits.

Signal conditioning equipment by Daytronics was used. The rate of penetration was controlled manually. The target value was 2cm/sec., this was achieved with the exception of a few places in the profiles at which there were rapid changes in resistance.

An IBM PC compatible microcomputer with appropriate A-D converter was used to capture cone resistance at 20mm intervals and also to display, in real time, the profile of the cone resistance on a graphics screen.

In addition at each site a series of 100mm diameter samples was recovered by pushing thin walled brass tubes into the bottom of a borehole.

3 RESULTS

Two sites were investigated: adjacent to pier J of the Dominion Rd. overbridge in central Auckland and at the Wairau Rd. industrial development on the North Shore. At the Dominion Rd. site the soil profile consists of clays and silty clays derived from the in situ weathering of Waitemata group sandstones and siltstones. At Wairau Rd. the soil profile consists of recent alluvial silts and clays overlying pleistocene sediments.

In Figure 1 the relative positions of the vane and cone profiles are shown for the two sites.

In the following discussion the measured cone resistances have been converted to net values by subtracting the total vertical stress at the depth of the reading.

3.1 Dominion Rd.

For the Dominion Rd. site four cone profiles, two vane profiles and a water content profile are given in Figure 2. Figure 1 shows that the cone and vane test locations are relatively close. It is apparent from Figure 2 that, despite the close proximity of the soundings, there is considerable variability between the vane and cone resistance profiles. This variability requires some statistical analysis to determine the trends between the sets of data. Towards the bottom of the profiles there is a "seesaw" like variation amid a gradually increasing resistance. At the bottom of the sounding, where no further penetration was
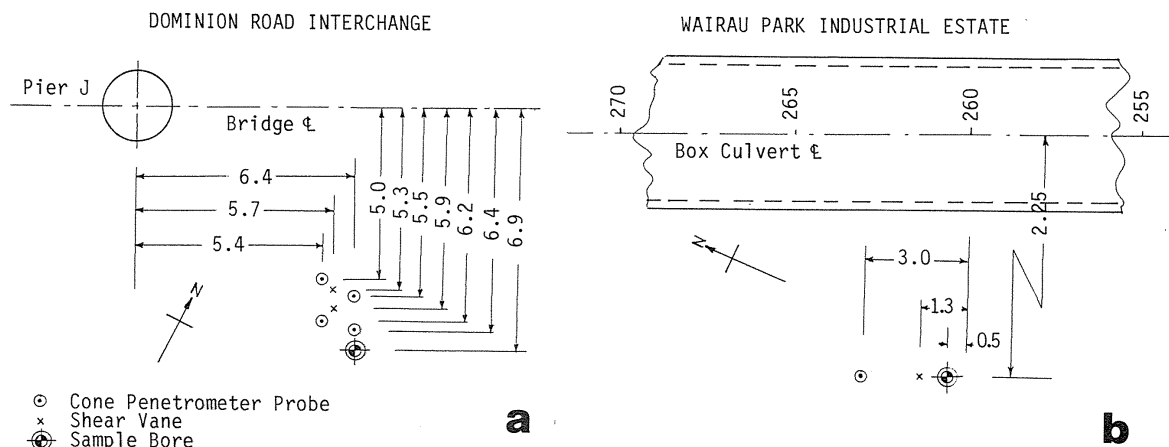
DOMINION ROAD INTERCHANGE

Pier J

Bridge ₵

— 6.4 —

— 5.7 —

— 5.4 —

5.0
5.3
5.5
5.9
6.2
6.4
6.9

N

⊙ Cone Penetrometer Probe
× Shear Vane
⊕ Sample Bore

WAIRAU PARK INDUSTRIAL ESTATE

270   265   260   255

Box Culvert ₵

2.25

N

— 3.0 —

1.3

0.5

a

b

Figure 1 Relative positions of the cone and vane profiles at the two sites. (a) Dominion Rd. (b) Wairau Rd.

Net cone resistance (MPa)

Vane Shear Strength (kPa)

Water Content (%)

Depth (m)

Depth (m)

Depth (m)

PL   LL

PL   LL
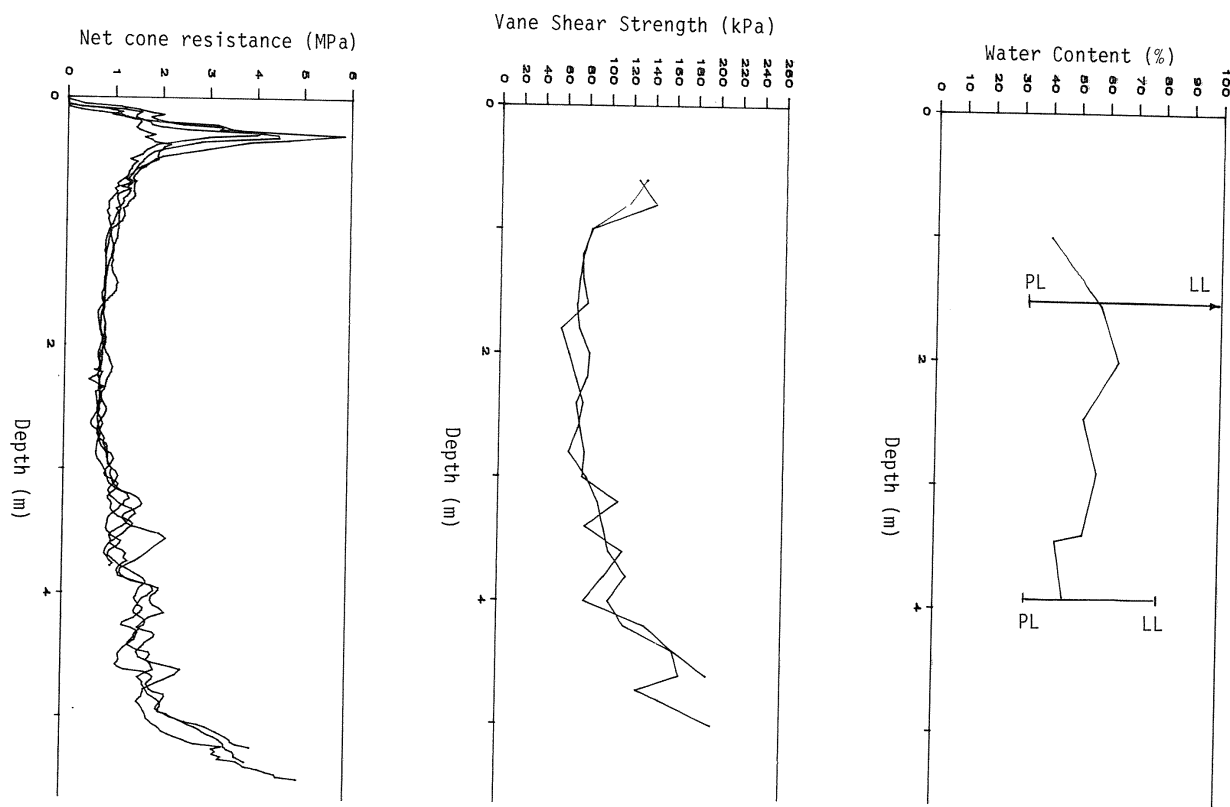
Figure 2 Net cone resistance, vane shear strength & water content profiles for the Dominion Rd. site.

possible because the load capacity of the rig had been reached, it is assumed that the top of Waitmata group materials had been encountered. The rapid fluctuations in resistance at the bottom could reflect weathering of the interlayered siltstones and sandstones character-istic of some locations in the Waitemata group materials.

Atterberg limit tests were done on samples from depths of 1.5m and 3.9m. Both sets of results {1.5m: PL 31% LL 128%, 3.9m: PL 29% LL 76%} indicate soils of high plasticity. The saturation ratio was determined on a sample from 0.9m depth, it was found to be fully saturated. During the summer in Auckland the local clays dry out and shrink consid-

erably. The fieldwork reported herein was done in early November 1987. In the preceeding months there had been enough rainfall to ensure that the water table would have been close to the surface at the time of the fieldwork. This is confirmed by the 100% saturation at 0.9m.

3.2 Wairau Rd.

Cone, vane and water content profiles for this site are presented in Figure 3. The different nature of the soil profile at this site is evident from comparison of Figures 2 and 3. The cone resistance is lower as is the vane shear strength and the water contents are correspondingly higher. Atterberg

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Net cone resistance (MPa)
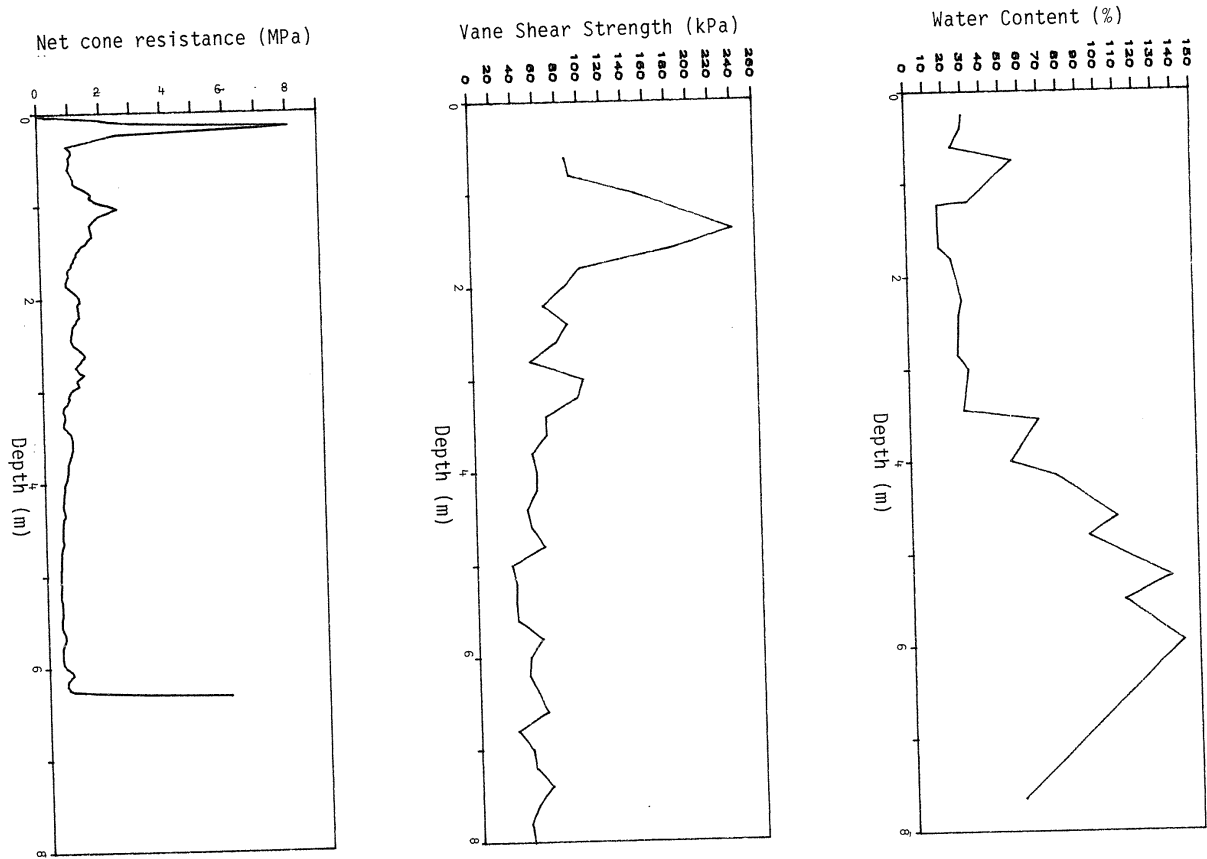
Vane Shear Strength (kPa)

Water Content (%)

Figure 3 Net cone resistance, vane shear strength & water content profiles for the Wairau Rd. site.
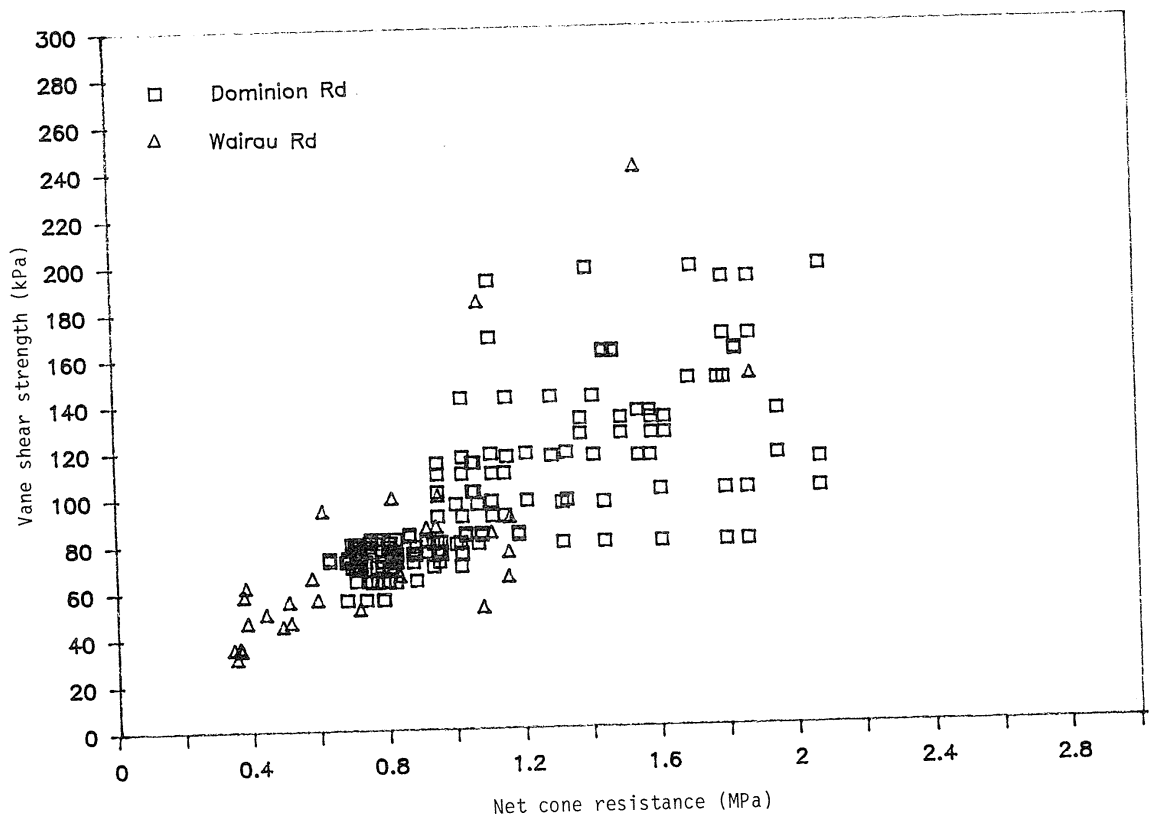
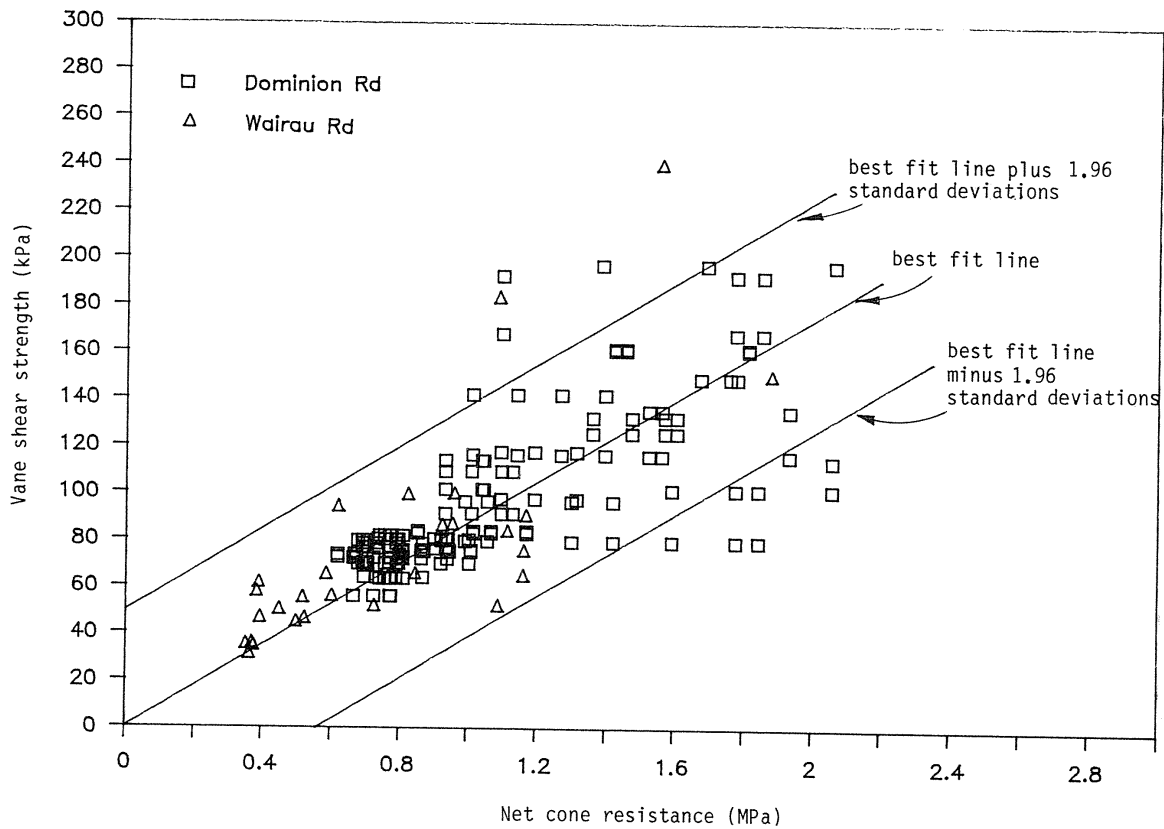Figure 4 Net cone resistance versus vane shear strength for the two sites.

Figure 5 Linear regression of data with constant variance, regression line forced through the origin.
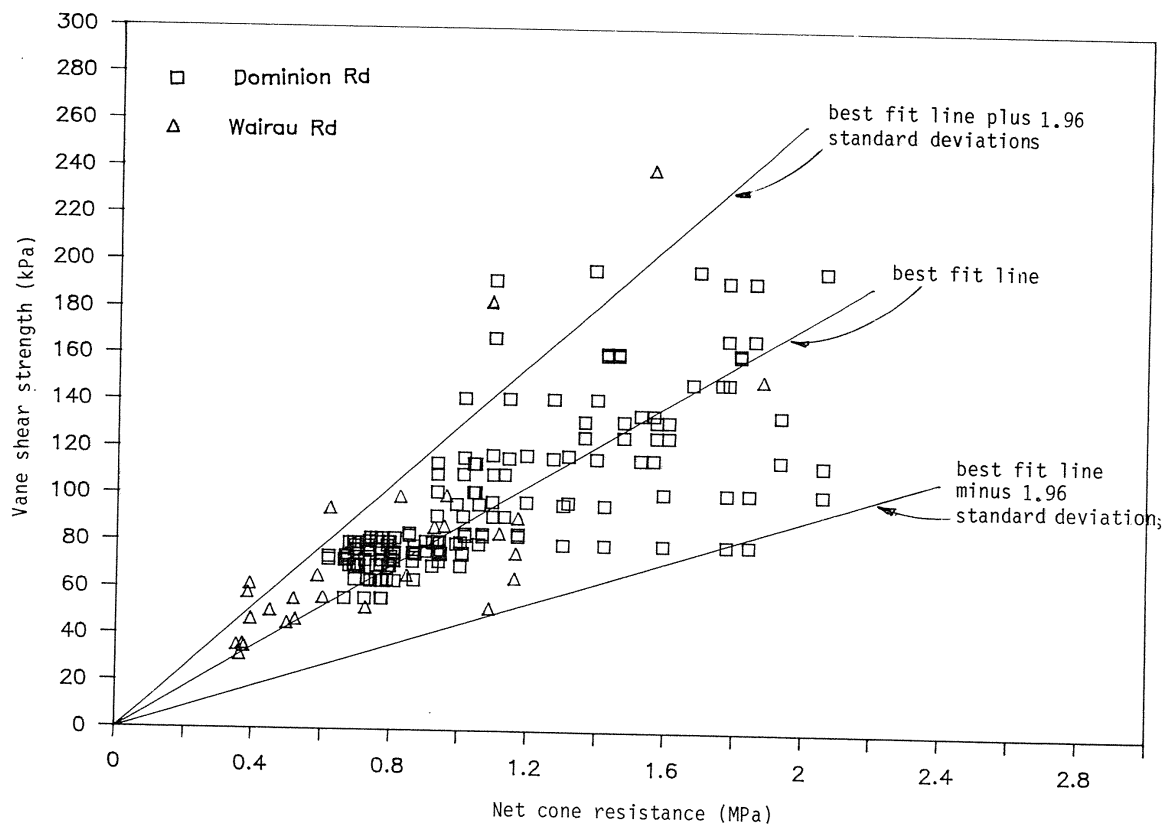

Figure 6 Linear regression of data with linearly increasing variance, regression line forced through the origin.
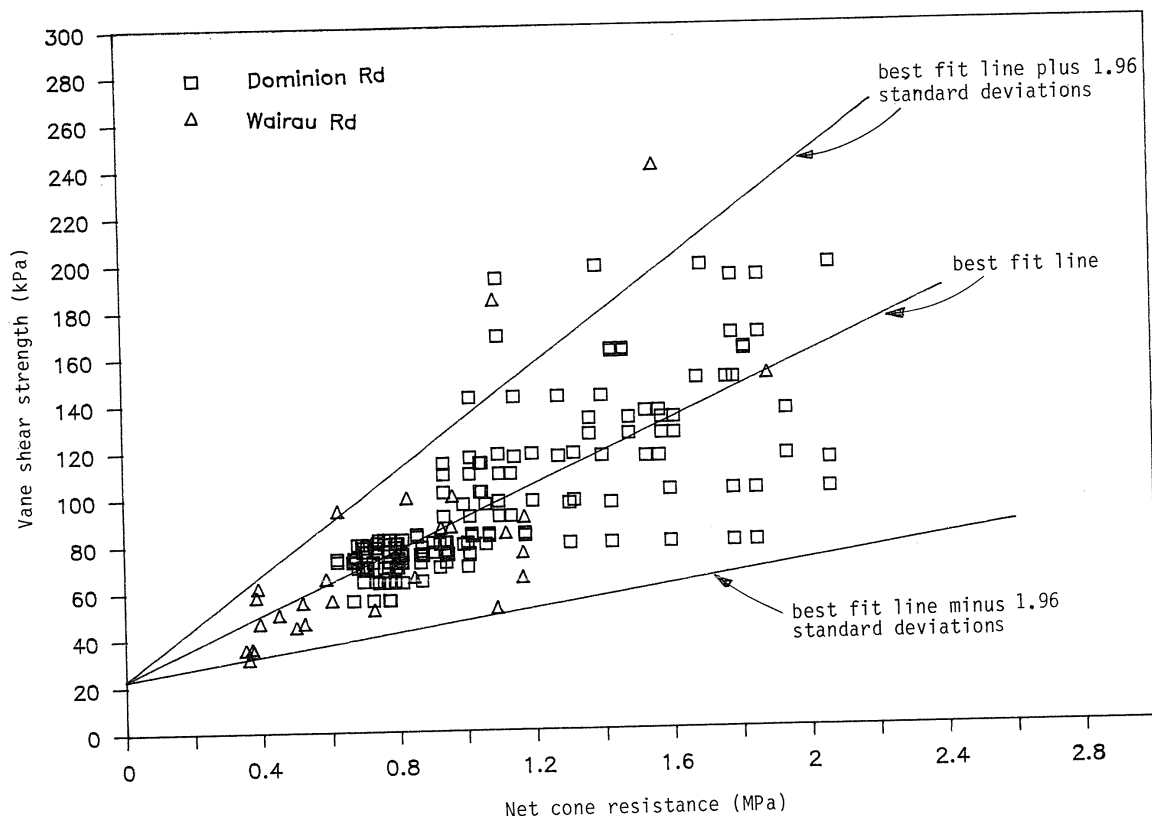
Figure 7   Linear regression of data with linearly increasing variance, regression line not forced through the origin.

limits from investigation boreholes, near the vane and cone soundings, have plasticity index values ranging between 47% and 106%.

The water table at the Wairau Rd. site at the time of the fieldwork was observed to be 0.5m below the ground surface.

4   ANALYSIS

From Figures 2 and 3 it is apparent that there is a general correlation between the cone resistance and the vane shear strength. However the variability of the profiles means that it is difficult to get a point to point comparison. For example at a depth of 4 metres at the Dominion Rd. site there are four different cone results and two vane values. At the the Wairau Rd. site considerable variability is apparent if the vane result is plotted against the cone resistance at the same depth.

In Figure 4 all the data have been plotted. For each vane result at Wairau Rd. the corresponding vane resistance has been plotted. The length of the vane is 75mm and cone readings were recorded every 20mm, thus cone readings over a depth of 80mm (5 readings in all) were averaged to get a representative value at the depth of each vane result. For the Dominion Rd. profiles there were two vane readings at each depth and four vane profiles, all of these appear as separate points in Figure 4. The difficulty with matching vane and cone resistance is highlighted as the cone resistance increases. In Figure 4 the scatter increases as the cone resistance increases. The majority of the points to the

right of the diagram correspond to the lower reaches of the Dominion Rd. cone profiles. At that level the cone resistance is seen to fluctuate rapidly and consequently the vane reading at the same depth is more likely to fall above or below some hypothetical relation between the two results.

It is assumed herein that the the scatter in Figure 4 is caused by local fluctuations in the soil profile, in other words even at the close spacing of the vane and cone soundings there are point to point variations in the soil properties. In a more homogeneous profile, unlike those at Dominion Rd. or Wairau Rd., less scatter would be expected. On the basis of this assumption the data in Figure 4 are regarded as indicating the correlation between the two test results. A linear regression process will be used to find the slope of the best fit line.

The outcome of subjecting the data in Figure 4 to a conventional linear regression analysis is presented in Figure 5. The regression has been performed subject to the additional constraint that the line must pass through the origin. The slope of this best fit line gives a factor of 11.4 to convert the vane reading to the net cone resistance with a correlation coefficient of 0.71. This regression analysis assumes that the variance of the population remains constant with "x". In Figure 5 the best fit line is plotted along with lines at ± 1.96 standard deviations from the best fit line. These lines should encompass 95% of the population if the data are normally distributed. It is clear that the data do not fit this regression model particularly well. An alternative is to assume that the variance increases linearly with "x". This procedure is set

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out by Ang and Tang (1975). The result is shown in Figure 6. The slope of the best fit line is the same as in Figure 5 but this time the Π 1.96 standard deviation lines make a better job of encompassing the data. In Figure 7 another alternative is shown, this time the variance increases with "x" but the line is not forced through the origin. Although the correlation coefficient is marginally better in this case, 0.75, the intercept on the shear strength axis, a material with an undrained shear strength of about 22kPa having zero penetration resistance, is hard to justify and so the analysis presented in Figure 6 seems to be more feasible.

Finally in Figure 8 the best fit regression line from Figure 6 is redrawn along with the 95% confidence limits on the data population as well as the 95% confidence limits on the location of the best fit line.
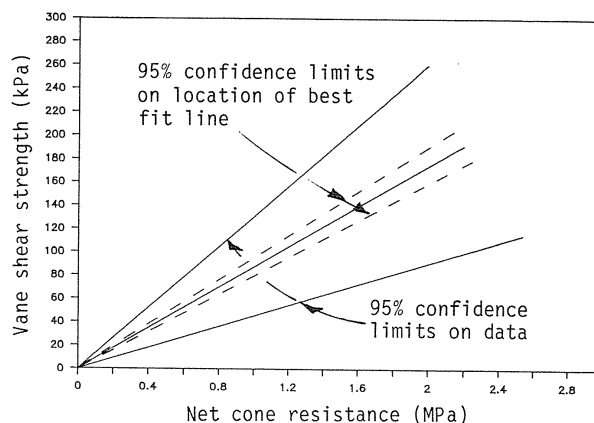
5   DISCUSSION

The classical relation between the undrained shearing strength of a saturated cohesive soil and the net ultimate bearing pressure at depth is 9. This usually denoted by the symbol N_c. The ratio between the cone resistance and vane shear strength is often denoted by the symbol N_k. Values ranging from 9 to as large as 24 have been reported in the literature for N_k. According to Muromachi (1981) Japanese experience has been that, for a cone having a friction sleeve with parallel sides and the same diameter as the cone tip, N_k is in the range 10 to 12. For cones with the earlier profile of the dutch mantle cone Muromachi reports that N_k in the range 14 to 17 is used. Carpentier (1982) reports a value of 9 for stiff fissured clays.

Lunne et al (1976) suggest that N_k is a function of plasticity index. As the PI increases N_k decreases. Others {Baligh et al (1980), Tavenas et al (1982), O'Riordan et al (1982) and Jamiolkowski et al (1982)} have confirmed this behaviour. As the Auckland clays have high plasticity index values, N_k about 11 is consistent with the trends observed elsewhere.

6   CONCLUSIONS

The vane and cone profiles discussed show considerable point to point variability. Using linear regression analysis the best fit relation between the net cone resistance and the vane shear strength gives a value of 11.4 for N_k. This value is consistent with values obtained elsewhere for clays of similar palasticity index.

7   ACKNOWLEDGEMENTS

Figure 8   Summary of regression results

8   REFERENCES

ANG, A. H. S. and TANG, W. H. (1975). Probability Concepts in Engineering Planning and Design. Wiley, Vol. 1, pp 294-297.

BALIGH, M. M., AZZOUZ, A. S., MARTIN, R. T. (1980) Cone Penetration Tests Offshore the Venezuelan Coast. MIT Report, No. R80-31.

JAMIOLOWSKI, M., LANCELLOTTA, R., TORDELLA, L. and BATTAGLIO, M. (1982) Undrained strength from CPT. Proc. 2nd. European Conference on Penetration Testing, Amsterdam, Vol. 2, pp 599-606.

CARPENTIER, R. (1982). Relationship between cone resistance and undrained shear strength of stiff fissured clays. Proc. Second European Conference on Pentration Testing, Amsterdam, Vol. 2, pp 519-528.

LUNNE, T., EIDE, O. and DE RUITER, J. (1982) Correlations between cone resistance and vane shear strength in some Scandanavian soft to medium stiff clays. Canadian Geotechnical Journal, Vol. 13, pp 430-441.

MUROMACHI, T. (1981). Cone Testing in Japan. Cone Penetration Testing and Experience, Proc. of a session sponsored by the ASCE Geotechnical Engineering Division, St. Louis, edited by G. M. Norris and R. D. Holtz, pp 49-71.

O'RIORDAN, N. J., DAVIES, J. A. and DAUNCEY, P. C. (1982) The interpretation of static cone penetrometer tests in soft clays of low plasticity. Proc. 2nd. European Conference on Penetration Testing, Amsterdam, Vol. 2, pp 755-760.

TAVENAS, F., LEROUEIL. S. and ROY, M. (1982) The piezocone test in clays: use and limitations. Proc. 2nd. European Conference on Penetration Testing, Amsterdam, Vol. 2, pp 889-894.