SOIL STIFFNESS FOR SHALLOW FOUNDATION DESIGN IN THE PERTH CBD

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ABSTRACT

Foundation systems for high-rise structures in the Perth CBD include the whole range of footing types: individual spread footings, single rafts, piles, and piled rafts. Of these, raft foundations are the most common. The design of raft foundations (and indeed all foundation types) relies heavily on calculations of the anticipated total and differential settlements. For these calculations, the most crucial material parameters are the stiffnesses of the soils underlying the foundation. In the Perth CBD, the soil types consist of interbedded layers of dense to very dense sand or fine gravel, and stiff to hard clays, overlying bedrock. In the period since the 1970s, when most of the current high rise structures in the CBD were built, a number of methods of determining the soil stiffness have been used. Very little information is available regarding the actual settlement performance of these structures. However, two important publications from the 1970s provide back-analysed stiffness parameters from the measured performance of 4 moderate rise structures (up to 40 storeys high) and these are regarded as benchmark values. The paper discusses the various methods used in Perth for determining stiffness, both 'traditional' and 'modern', and the results obtained using these methods are compared to the benchmark values. Data from a number of sites, mostly at the west end of the CBD, are discussed in detail, as a number of *insitu* test methods for determining stiffness have been used at some of these sites, including seismic CPT, Marchetti dilatometer (DMT) and self-boring pressuremeter (SBP). Some comments are also included about stiffnesses of sands in other parts of the Perth area, compared to the CBD area.

1 INTRODUCTION

In this paper, the Perth CBD is taken to extend from the Mitchell Freeway in the west to Plain Street in the east, and from the shore of the Swan River in the south to the railway line in the north. A detailed study of the area was published by Andrews (1971), in which he summarised the topography and geology of the central city area and presented detailed stratigraphic logs of the area, from which he attempted to draw stratigraphic sections. Much of the rest of this introduction draws on or quotes directly from this work.

The city centre is dominated by a large dune complex to the west. Part of King's Park is situated upon this complex, which has several fairly well defined crests or peaks, the highest being Mount Eliza, which rises to over 68 m above datum[1]. From the west of the city there runs an escarpment, which is in fact an alluvial terrace of the river formed during the Quaternary Age. This escarpment is covered by dune sand, as is the rest of the area; however, there does appear to be a ridge of sand coincident with this scarp. The crest of this ridge lies between Hay and Murray Streets, with a pronounced peak (about RL 24 m) at the site of St. Mary's Cathedral, before sweeping north to another peak slightly higher at the intersection of Plain and Bronte Streets. This ridge falls gradually away towards the railway line in the north and more steeply towards the river in the south and east.

The soil conditions in this ridge area generally consist of the dune sands mentioned above, overlying alluvial Quaternary Age interbedded layers of dense to very dense sand or fine gravel and stiff to hard clays, overlying the King's Park Shale (siltstone). Along the southern (river) edge of the area, extensive reclamation has occurred in various stages since first settlement and in these areas the soil conditions can be very variable, with soft silt layers located below the fill in many areas. To the north, there are pockets of soft organic clay / peat, due to the infilling of a line of swamps that ran from the SE corner of Lake Monger to Claisebrook in East Perth.

1.1 KING'S PARK SHALE

The King's Park Shale, the bedrock under all of the CBD, consists of a calcareous sandstone, siltstone or shale, which is believed to have been formed during the early Tertiary Age, under marine conditions. Subsequent changes in relative levels between land and ocean exposed it as a land surface during the late Tertiary period. There was then a break in further sedimentary action until the start of the deposition of the younger alluvium during the Quaternary period. The

[1] It should be noted that Andrews quotes levels in feet, relative to low water mark in Fremantle. To convert to m AHD, the following relationship must be used: $\mathrm{RL}\,(\mathrm{m\,AHD}) = [\mathrm{RL(ft)} - 2.48] \times 0.3048$.

maximum proven thickness of the King's Park Shale is about 300 m. Beneath the CBD it is found at elevations varying from about RL –16 m to RL –21 m, with an average of about RL –20 m AHD. However, some borings in the east of the area along Adelaide Terrace and towards the Causeway failed to encounter it, due probably to alluvial action in the Quaternary eroding channels in it, which were subsequently filled by alluvial action.

1.2 ALLUVIAL DEPOSITS

Quoting Andrews (1971): "The (Quaternary) sequence of soils underlying the dune sand deposits (and overlying the King's Park Shale) is extremely complex, and may best be described as interbedded sands, silts and clays, which were deposited during the Quaternary geological period, mainly as a result of the alluvial action of the Swan River system. Because of the manner of their deposition, it is very difficult to clearly identify or define consistent soil layers throughout the area examined. However, some interpolation is possible from site to site." Andrews goes on to describe the stratigraphy of these alluvial layers, but, because of the complexity, the most general picture that emerges is that the upper zone is mainly cohesive with some sand layers and the lower zone is mainly alluvial sand. He states: "Almost without exception at the sites examined there is a layer, usually substantial, of this alluvial sand. Towards the west of the city, it occurs at a level of between RL 0 and RL –16 m; towards the east, it is deeper, and is first encountered at a level ranging from RL –4 m and RL –10 m and generally continues until bedrock. Occasionally, where there was no sign of the bedrock, the sand continues to much greater depths, as on the Fairlanes Bowling site on Adelaide Terrace, where grey-brown medium to coarse sand continued to the termination of the borehole at RL –45 m." (At the Hyatt Hotel site on the corner of Adelaide Terrace and Plain Street, this layer continued to the bottom of a borehole taken to RL –39 m).

This lower alluvial sand is generally dense to very dense, and is described by Andrews as fine to coarse yellow-brown and grey sand, with clay lenses and traces of (fine) gravel. The cohesive materials comprise silty clays, sandy clays and clayey sands of generally very stiff to hard consistency.

While the lower alluvial sands generally overlie the King's Park Shale directly, Andrews states that in some places there exist layers of cohesive material directly above the King's Park Shale that are too substantial to be classified as lenses or bands.

1.3 DUNE SANDS

The dune sand deposits that blanket most of the CBD area are thought to belong to the Spearwood Dune system, a system of parallel calcareous sand dunes formed in the mid-to-late Pleistocene, abutting the older (early-to-mid Pleistocene) Bassendean Dune system, which lies to the east of the CBD. The ridges in the topography referred to earlier are mainly composed of this material. Andrews states that the thickness of the dune sand along the crest of the ridge varies from about 25 m at the crest of the King's Park scarp, to about 15 m at St. Mary's Cathedral and about 18 m at the intersection of Plain Street and Bronte Street. It reduces in thickness towards the north and towards the river, to about 3 m along the Esplanade and Terrace Road. Typically, through much of the CBD it is about 10 m to 12 m in thickness. The dune sand is a fine to medium sand, varying from white through yellow to dark brown, depending on the amount and concentration of iron oxide and the degree of leaching that has taken place since deposition. It varies from very loose to loose at the surface to dense at 4 m to 6 m depth.

1.4 WATER TABLE

The underlying water table slopes gently upward from the river level at the south and is therefore well below founding level for most structures located on the high ground along St George's Terrace. However, there are frequent instances of perched water tables on top of some of the upper alluvial clay layers. The springs that gave Spring Street its name resulted from the surface seepage from one of these perched water tables as it intersected the slope down from St George's Terrace.

1.5 FOUNDATION TYPES

High-rise structures up to 262 m high (52 storeys) line St George's Terrace and Adelaide Terrace and some of the side streets leading off this main thoroughfare. The development cycle started from the boom in the iron ore industry in the 1960s, with Council House, constructed in 1962, being probably the first that could be called 'high-rise'.

Most medium rise and high rise buildings constructed since then are founded on spread footings or raft foundations, with the founding level being at least one, and often more, basement levels below street level. Major exceptions are: the Bankwest Tower (completed 1988, 52 storeys, 214 m high), founded on large diameter bored belled piles, bearing in the King's Park Shale (siltstone); QV1 (completed 1991, 40 storeys, 163 m) founded on a piled raft, with the piles finishing above the siltstone; Central Park (completed 1992, 52 storeys, 226 m), founded on piles bored into the siltstone and the Woodside Building (currently under construction, 28 storeys, 126 m), founded on auger piles bored into the siltstone. The Busport (late 1980s) and the Convention Centre (currently under construction), which are located on reclaimed land along the river, are also piled. To the north, the Telstra Exchange Tower on Wellington Street (95 m, 17 storeys, completed 1979), and the 1990s' additions to the central train station and adjoining car parking building, are also founded on piles. The GPO building on Forrest Place, constructed in 1923 is founded on wooden piles, as is the adjacent Commonwealth Bank building.

For practically all of these structures, the prime design consideration for the foundations is the magnitude of the maximum and differential settlements. Hence, measurement or deduction of the stiffness parameters for the various foundation layers is of primary interest for such structures. Over the past 30-40 years, the tools used for this purpose have included laboratory testing (oedometer and triaxial tests) and a range of *insitu* tests: SPT, CPT, pressuremeter (both insertion types and self-boring types) and seismic CPT. More recently, the DMT has been introduced, and also a technique based on measurement of surface (Rayleigh) wave velocity (the so-called SASW technique, discussed later).

For some projects, knowledge of the horizontal stress would also be very useful (basement walls, tunnels, for example). However, very little attention has been given to this aspect, and no good-quality data on horizontal stress in the Perth CBD have been published. Thus, it will not be dealt with here.

In any discussion of foundations in the Perth CBD, three publications are of historical importance. The first is a compilation of site investigation data for the area (Andrews, 1971), put together by Dr David Andrews, (then with the CSIRO Division of Applied Geomechanics), which has already been extensively referenced. This contains borehole records and SPT test results from site investigations carried out in the CBD up to that time. The second and third are proceedings of two symposia organised by the CSIRO Division of Applied Geomechanics and the WA Panel of the AGS (CSIRO, 1970, and CSIRO, 1975). In these proceedings the most important individual contribution is probably that of Fraser (1975), which reports the results of back-analyses of layer stiffnesses under three medium rise CBD buildings, obtained from settlement gauges, located at various depths below the raft foundations of these buildings, and pressure cells, located on the underside of the rafts. These data will be referred to extensively later in this paper. The three buildings are the AMP Building, NBA House and CBA House, all located along St George's Terrace in the William Street / Barrack Street area. To the authors' knowledge, there is no other similar case in the Perth CBD of such measurement of strain with depth under building foundations.

For high-rise construction, there are of course other foundation issues other than just building settlements. These include pile capacities and pile group behaviour for piled foundations, design and behaviour of retaining walls for basement construction and the effect of construction on adjacent structures (whether due to foundation interaction, or retaining wall movements during construction). Discussion of these issues is outside the scope of this paper (though of course the discussion on soil stiffness is relevant to these issues, particularly the interaction issue).

2 SOIL STIFFNESS

Before discussing soil stiffness values for use in foundation design in the Perth CBD, it is appropriate to focus on the whole question of what is soil stiffness and what affects it. One of the most important advances in geotechnical engineering in the past 20 years or so is the general realisation and acceptance of the fact that the stress-strain behaviour of almost all soils is highly non-linear, even for stiff soils in the 'elastic' region of the stress-strain response. Once this is accepted, and methods of dealing with it are devised, the whole problem of how to go about predicting deformation in such soils becomes much clearer.

Non-linear stress-strain response in the 'elastic' region has been accepted for a long time in the area of earthquake engineering. In this context, the dependence of secant shear modulus G on strain level for cyclic (dynamic) loading was illustrated by a number of researchers using laboratory resonant column testing (e.g. Hardin and Drnevich, 1972a, 1972b). More recently, the applicability of the same ideas to 'static' loading problems has been emphasised again and again.

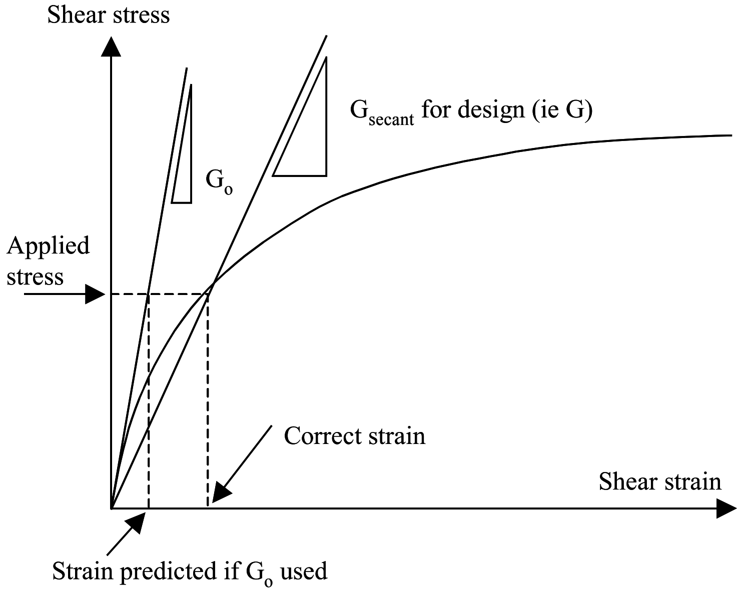


Figure 1: Illustration of non-linear stiffness of soil, and definition of initial tangent shear modulus (or small strain).

Figure 1 shows a schematic of the non-linear behaviour. When defining 'stiffness', we can focus on the initial tangent modulus (or 'small strain modulus') G_o. This stiffness is a fundamental soil property and is readily measured, as it is the stiffness that dictates the velocity of travel of shear waves. However, under a foundation in areas where there is significant shear stress, this stiffness cannot be used directly to predict deformations (as illustrated). For every level of applied stress, a different secant shear modulus (denoted G_{secant}, or simply G) must be used. There is therefore no single 'correct' value of soil stiffness for any soil layer under a foundation; the loading (strain) level in that layer must be taken into account.

Traditionally, geotechnical engineers have spent considerable time and energy searching for empirical correlations between 'the Young's modulus *E*' of the soil and various *in situ* test measurements (SPT *N*-values, CPT q_c values, etc), using the observed settlement response of footings. This value of *E* would then be used in settlement or deformation calculations. Of course, this must be of limited accuracy, because trying to fit a linear elastic model to non-linear behaviour means that the correlations obtained must relate only to that particular combination of footing type and size, soil type and level of loading. Thus, there is no logic in trying to define a unique *E*/*N* ratio or E/q_c ratio for a particular soil unless the strain range or shear stress range of interest is defined.

The way in which stiffness changes with strain level is often presented as plots of secant stiffness G versus (log of) shear strain γ, such shown in Figure 2(a), or as secant shear modulus normalised by the initial tangent value (G/G_o), versus (log of) shear strain γ, as shown in Figure 2(b).

The alternative is to plot G/G_o versus mobilised shear stress, normalised by the shear strength τ/τ_{max}, where τ_{max} is the shear strength of the soil element. This type of plot was used by Tatsuoka and Shibuya (1991) to present stiffness degradation data for a range of soils and soft rocks (though plotted as E/E_{max} versus q/q_{max}), as shown in Figure 3(a). Fahey and Carter (1993) also used this type of plot. On the basis of the shapes of curves shown in Figure 3(a), Fahey and Carter (1993) introduced a 'distorted hyperbolic model':

$$\frac{G}{G_o} = 1 - f\left(\frac{\tau}{\tau_{max}}\right)^g \tag{1}$$

In this equation, setting the 'distortion parameters' *f* and *g* to both be equal to 1, gives the straight-line hyperbolic model. Examples of the shapes of degradation curves that this model can give are shown in Figure 3(b).

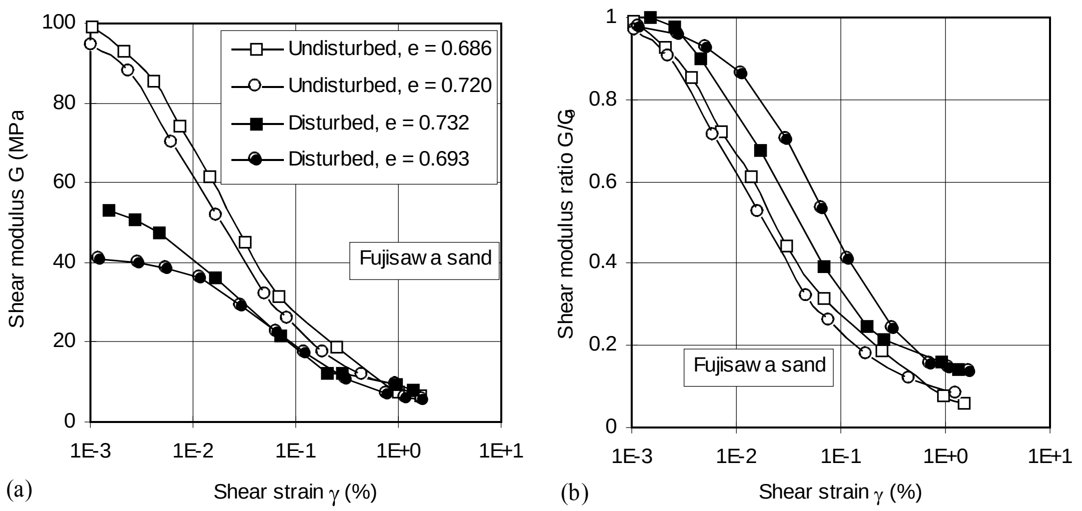
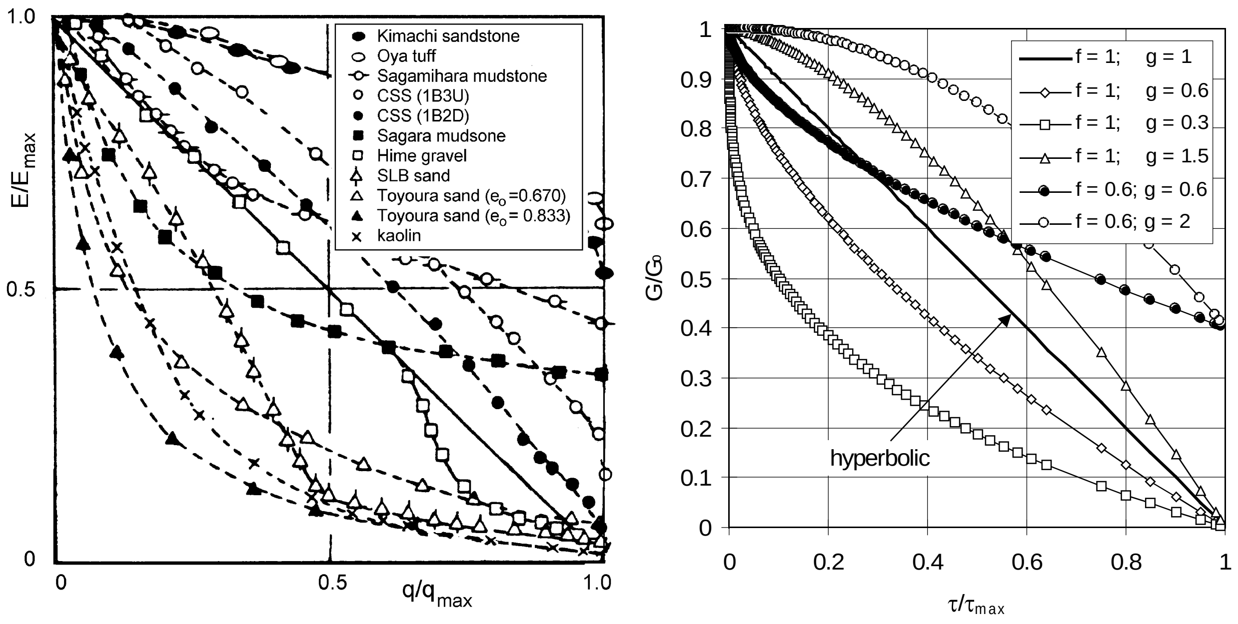


Figure 2: Modulus degradation curves for Fujisawa sand plotted as (a) G versus γ, and (b) G/G_0 versus γ (replotted from Ishihara, 1996).

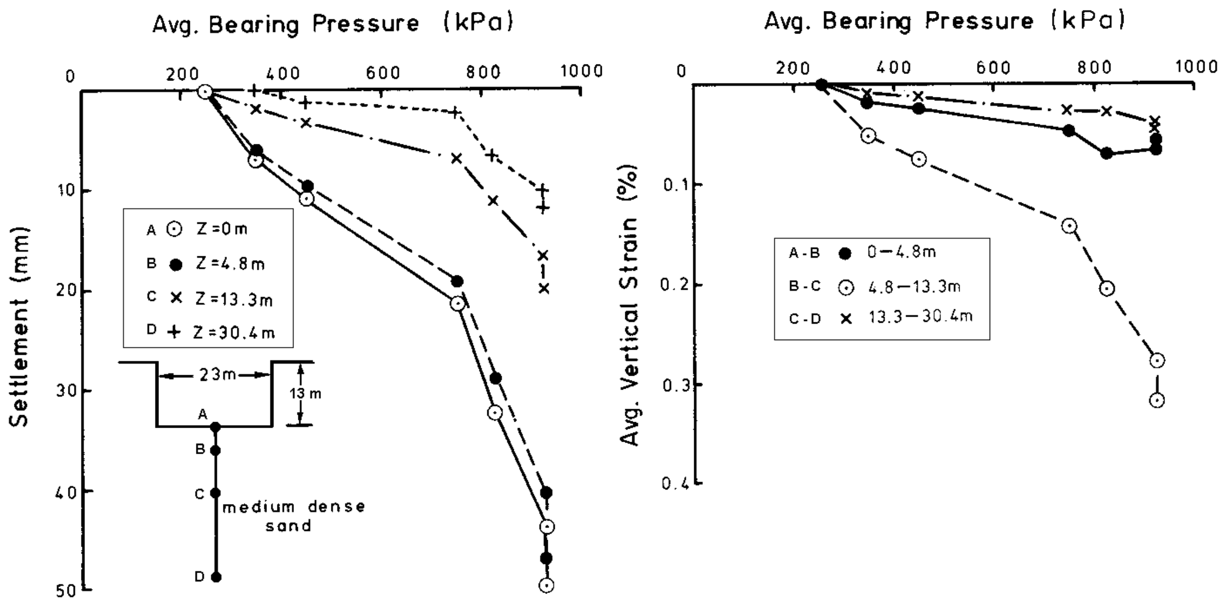


(a) (b)

Figure 3: Stiffness degradation plotted as normalised stiffness versus normalised mobilised shear stress: (a) data from various soils and soft rocks after Tatsuoka and Shibuya (1991); and (b) shapes of curves obtained using the ‘distorted hyperbolic’ model of Fahey and Carter (1993).

In his Bjerrum Lecture, Burland (1989) makes the point that the strains in the soil under ‘well designed’ foundations for most structures are generally very small. Figure 4 shows one of the examples cited by Burland. This concerns the measured settlements under a tall hotel, founded at the base of a 13 m deep excavation in medium dense sand in Berlin. Figure 4 (a) shows the settlements measured at different points beneath the centre of the heavily-loaded raft. These are interpreted to give average vertical strains over the depth intervals between the measuring points in (b). This shows that even though the bearing pressures are quite high and the raft settlement is about 50 mm, the average vertical strains are generally less than 0.1%, except in one region where they reach about 0.3%.

A further illustration of this principle is given in Figure 5, which shows the predicted major principal strains under a simple flexible raft foundation, 30 m diameter, with an applied bearing stress of 400 kPa., resting on 30 m of elastic soil with a uniform Young’s Modulus (217 MPa). This analysis is meant to be a representative of a typical simplified raft foundation in Perth (this case will be discussed further later in this paper). This figure shows the maximum principle strain to be just above 1.4 x 10^{-3} (or 0.14%), in the area shown shaded in Figure 5.



(a) (b)

Figure 4: (a) Settlements at various depths beneath the foundation raft for a tall hotel in medium dense sand in Berlin; and (b) corresponding vertical strains in the ground for the same case (after Burland, 1989).

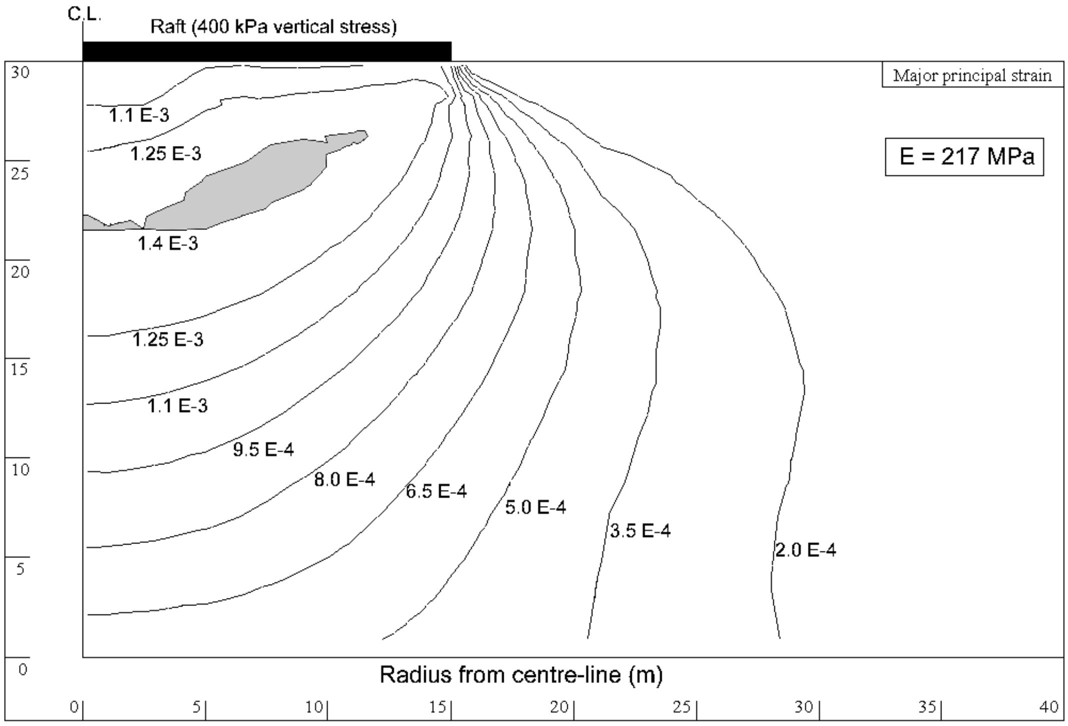


Figure 5: Contours of major principal strains under a flexible raft, 30 m in diameter, resting on a 30 m deep layer of elastic soil, with E = 217 MPa, and $\nu = 0.3$.

A number of Conferences, and a number of individual papers, have been particularly important in the development of the application of non-linear elastic concepts to routine foundation design. Some of the more important of these are:

- a number of contributions to the 10th European Conference on Soil Mechanics and Foundation Engineering held in 1991 in Florence, Italy, particularly the General Reports by Atkinson and Sallfors (1991) and Burghignoli et al. (1991).

- many of the contributions to the 1st International Conference on Pre-Failure Deformation of Geomaterials held in Sapporo, Japan, in 1994, and the follow-up 2nd International Conference on Pre-Failure Deformation of Geomaterials; Torino, Italy, 1999.
- a number of papers to the 1st International Conference on Site Characterisation, Atlanta, USA, 1998.
- a number of papers by Professor Tatsuoka and his colleagues at the University of Tokyo Institute of Industrial Research and various collaborators with this group (e.g. Tatsuoka and Shibuya, 1991), the most recent example being the Theme Lecture on the topic of pre-failure deformation properties presented at the 14th International Conference on Soil Mechanics and Foundation Engineering in Hamburg, Germany (Tatsuoka et al., 1997).
- many papers in the proceedings of the International Conference on Advances in Site Investigation Practice in London, UK (Craig, 1995).
- a number of papers on the stiffness of London Clay by researchers at Imperial College, London and others (e.g. the Bjerrum lecture of Burland, 1989, already referred to; Jardine et al., 1986; Simpson et al.., 1979; the Rankine Lecture of Atkinson, 2000).
- some papers by Italian researchers (e.g. Baldi et al., 1979; Bellotti et al., 1989; Jamiolkowski et al., 1985);
- the contributions to the Symposium in Print on Pre-Failure Behaviour of Geomaterials published in Géotechnique (Vol 47, No. 3, 1997).

3 METHODS OF MEASURING SOIL STIFFNESS USED IN PERTH CBD

Methods of determining soil stiffness for foundation design in the Perth CBD are no different from those used worldwide. These include: laboratory measurements in triaxial or oedometer tests; correlations with the results of penetration tests (SPT, CPT, Perth Sand Penetrometer); direct *in situ* measurement using the pressuremeter test, both insertion-types (Ménard, Golder's GA20, Oyo), or the self-boring pressuremeter (UWA); semi-direct *in situ* measurement using the Marchetti dilatometer (DMT) and measurement of the small-strain stiffness (G_o) using methods of determining the shear wave velocity (seismic CPT and surface wave method – the 'SASW' method).

3.1 LABORATORY MEASUREMENT

Because of the prevalence of sands in the Perth area, measurement of soil stiffness in laboratory triaxial tests for foundation design has not been much used (the authors are not aware of any case where it was used). This is clearly because of the difficulty of obtaining good-quality undisturbed samples of such soils.

However, for the Perth CBD, it is likely that good quality measurement of stiffness of the stiff clay layers could be carried out in laboratory triaxial tests. This would require high-quality sampling, and then taking into account the effects of sampling disturbance, along the lines suggested by Hight (1998). High quality sampling in this context would require, as a minimum, attention to the detail of sampling tube cutting angle, internal clearance, etc., as detailed by Hight. It would also require a high degree of precision in strain measurement in the triaxial cell, using internal submersible strain transducers. This type of instrumentation is now used at UWA as standard practice. The work over the past 20 years in London has shown that the full detail of the non-linear stiffness characteristics of London Clay can be measured very well in such laboratory tests, provided this level of care is exercised. Similar conclusions have been reached elsewhere.

It is also now reasonably commonplace to measure shear wave velocity in the triaxial cell using a range of methods ('bender elements', shear plates, for example), and from this the small strain stiffness G_o can be determined. This provides a check on the results obtained for this parameter from the precision internal instrumentation mentioned above.

The reality for many projects is that laboratory tests of this standard are unlikely to be carried out on sufficient samples to make it worth while, given the high cost of high-quality sampling and high-quality laboratory testing of this type. However, for large or important projects, it should be mandatory.

3.2 STIFFNESS OBTAINED FROM PENETRATION TEST RESULTS

Most foundation design in the Perth CBD is carried out on the basis of results from borehole logging, with SPT testing, and on CPT probing. The SPT test was the main tool used until about the early 1980s, when electric friction cone testing was introduced. However, even prior to this, some Dutch Cone testing was carried out (see for example Fraser, 1975, discussed later), but it is not clear to the authors how common such testing was.

3.2.1 SPT testing

For the past 30 years at least, SPT tests in WA have been carried out using automatic trip hammers and hence many of the problems associated with variable (and unknown) energy losses due to rope friction on the drill rig cathead, which plagues (plagued) SPT testing elsewhere, were avoided. However, more recently, the state of practice in SPT testing has not improved as it has elsewhere in the world, particularly in regard to measurement of the actual input energy[2]. This, and the use of non-rigidly coupled rods (loose-fitting bayonet-coupled rods, as used in some previous jobs in Perth), means that SPT values can vary widely depending on the equipment used. Where the SPT equipment is calibrated for input energy, the SPT N–value is corrected to the equivalent N–value for equipment with 60% efficiency (the N_{60} value). The assumption made in many countries that use trip hammers appears to be that the systems in use have an efficiency of 60%, and hence correction is not required. This assumption is highly questionable.

Fraser (1975), in a paper that will be discussed at length in the next section, carried out a comparison of some of the methods of determining settlements based on the SPT test for sites in Perth. The methods he considered were those of D'Appolonia et al. (1970), Parry (1971), and Sherif (1973)[3]. Other methods have been developed since then, the most important of which is probably that of Burland and Burbidge (1985). These methods range from very simple (Parry) to complex (Burland and Burbidge, or Schultze and Sherif).

3.2.2 CPT Testing

The electric friction cone for CPT testing began to be used in earnest in the early 1980s, and is now the most common tool for site investigation in the CBD (and elsewhere in Perth and WA). The attraction of the CPT for prediction of settlement in Perth was due largely to the fact that a detailed CPT settlement method for sands had been published by Schmertmann (1970) and Schmertmann et al. (1978). This method is based on series of tests in a large calibration chamber at the University of Florida, in which CPT tests and small-scale footing tests were performed on sands at different densities and stress levels. The method takes into account the influence on settlement of depth beneath the footing, based on a consideration of the strain level at different z/B ratios (where z is the depth below the footing, and B is the footing breadth). In simple terms, this method gives E to q_c ratios of about 2-3.

As experience with using the CPT in Perth was accumulated, it quickly became clear that the E values for sands in Perth obtained from Schmertmann's E/q_c correlations were simply much too conservative. Gradually, during the 1980s, a similar conclusion was being reached worldwide. The problem was identified as being that Schmertmann had based his correlations on the behaviour of newly prepared ('young') sand samples in the calibration chamber, which were normally consolidated, whereas most natural sands are 'aged', and are often overconsolidated.

The complexity of the relationship between E and CPT q_c value is captured by Figure 6, after Baldi *et al.* (1989). In this figure, the stiffness referred to is a drained secant stiffness (E'_s) at a 'working strain', taken to be equivalent to an axial strain in a triaxial test of 0.1%. The E'_s/q_c ratio is shown to depend on q_c and on effective overburden pressure σ'_{vo} (i.e. on $q_c/\sqrt{\sigma'_{vo}}$). For example, for a q_c value of 20 MPa, and an effective overburden pressure of 225 kPa, the value of $q_c/\sqrt{\sigma'_{vo}}$ is 1333, giving E'_s/q_c ratios varying from about 2 for 'recent NC sands' to up to 12 for 'OC sands' – a factor of 6 in the deduced value of E'_s. This shows that the correlation of Schmertmann may be correct only for 'recent NC sands', since the calibration tests were carried out in exactly such conditions.

Since the sands in Perth are generally aged (the Bassendean Sand, for example, is thought to date from the mid-Pleistocene, according to Seddon, 1972), and often overconsolidated, the relevant E'_s/q_c ratio is generally significantly higher than that proposed by Schmertmann. However, as Figure 6 clearly shows, *no single value of* E'_s/q_c *ratio applies, even for a single soil deposit,* because of the dependence of E'_s/q_c ratio on q_c and σ'_{vo}.

[2] In North America, it is now a requirement to calibrate each set of equipment to determine the actual energy delivered, and the N-values are then corrected to a standard N_{60} value, which is the value that would be obtained if 60% of the theoretical energy is delivered. A standard procedure for carrying out this calibration is laid out in the International Reference Test Procedure published by the ISSMGE (Decourt *et al*., 1988). To the authors' knowledge, no such calibration has ever been carried out for SPT equipment used in WA.

[3] The authors assume that this reference, which is to a PhD dissertation (in German), describes the same method as that usually referred to as the 'Schultze and Sherif' method, as detailed by Schultze and Sherif, 1973).

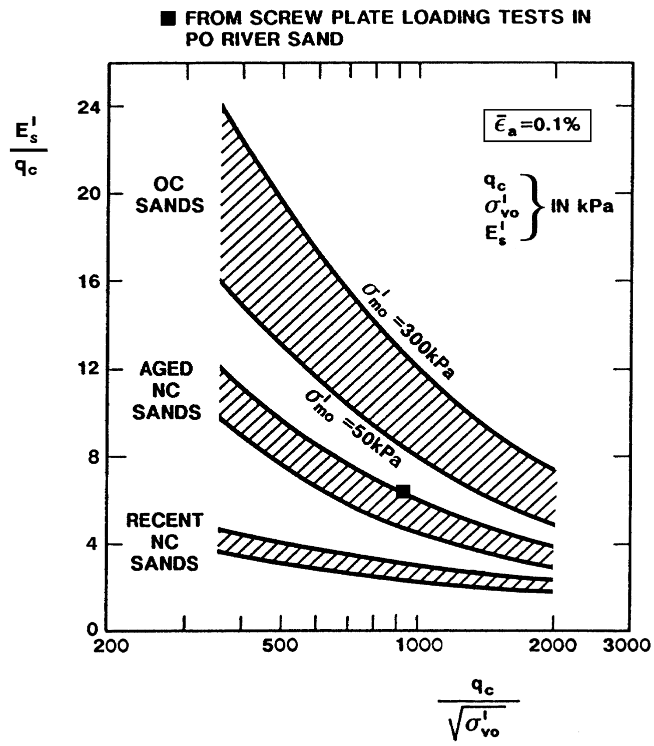


Figure 6: Ratio of 'working strain' stiffness E'_s to q_c versus $q_c/\sqrt{\sigma'_v}$ for sands with different stress histories (Baldi *et al.*, 1989).

A number of site-specific correlations of E/q_c ratio, based on measured footing load-settlement behaviour, have been carried out by individual organisations in Perth over the past decade or so, though none of these has been published, and, strictly speaking, none apply to the CBD area.

3.3 PRESSUREMETER TESTING

Pressuremeter testing has been carried out in Perth for various projects over the past 30 years or so, using both 'insertion-type' pressuremeters and self-boring pressuremeters. The insertion-type pressuremeters used include a standard Ménard pressuremeter, an Oyo pressuremeter (Oyo Corporation, Japan), a 'rock pressuremeter' from Cambridge Insitu (UK), and the GA20 pressuremeter (Golder Associates). The GA20 instrument was first described by Jewell and Fahey (1984), though the instrument has undergone considerable improvement since then.

UWA acquired a self-boring pressuremeter from Cambridge Insitu (UK) in 1976 and this instrument has been widely used for testing in the CBD and throughout WA since then. After a lapse of some years, it has recently been re-commissioned to carry out a series of tests in connection with the proposed underground rail tunnel through the heart of the CBD.

From the point of view of settlement prediction, the main attraction of the pressuremeter test is that it gives a direct value of soil stiffness. A value can be obtained from the virgin loading curve (the 'initial shear modulus' G_i) and also from unloading-reloading cycles carried out during the test (G_{ur}). Experience has shown that direct use of the G_{ur} value gives a good estimate of settlements for foundations on the stiff clays and sands of the Perth area. This apparent anomaly (that an unload-reload stiffness can predict the monotonic loading performance) will be discussed later.

The results of a self-boring pressuremeter (SBP) test in a stiff clay layer, at a depth of 29.5 m below ground level, are shown in Figure 7 (the site is at the west end of the CBD). The gradients of the unload-reload loops give values of G_{ur} of 83 MPa and 106 MPa, as shown in the left section of this Figure. Assuming a Poisson's ratio value of 0.3, these are equivalent to E values of 216 MPa and 276 MPa, respectively.

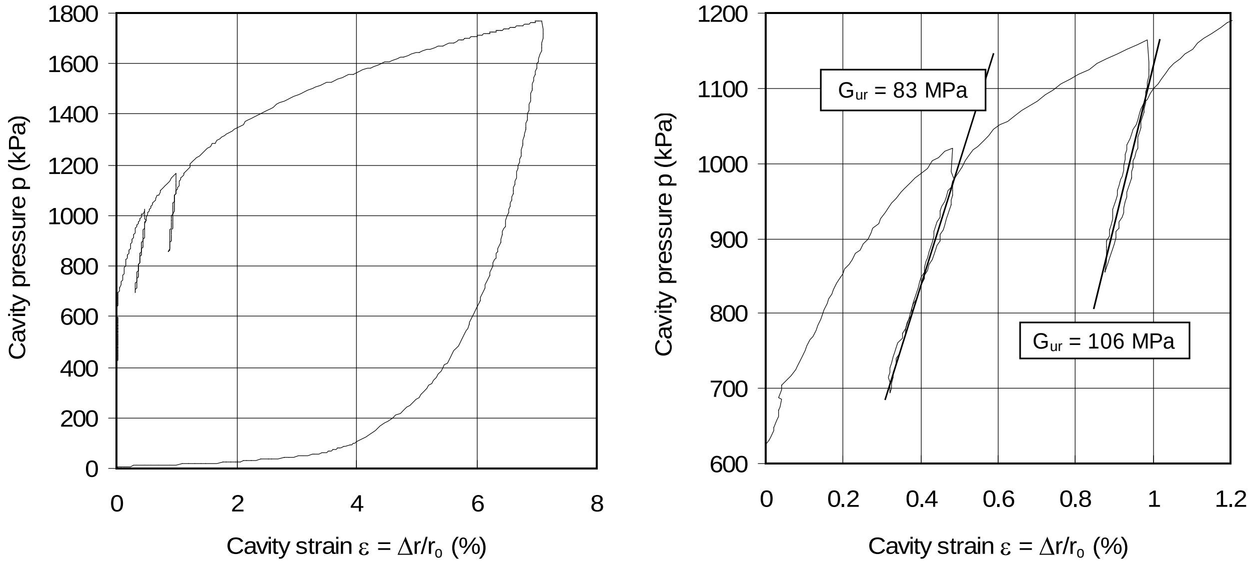


Figure 7: Results of SBP test in stiff clay layer at depth of 29.5 m, at the western end of CBD. The plot on the right shows an expanded version of the one on the left.

3.4 MARCHETTI DILATOMETER (DMT) TESTING

The Marchetti dilatometer (DMT) is a spade-like penetrometer, with a flexible steel membrane located on one face of the blade, as shown in Figure 8. It was developed by Professor Silvano Marchetti, from the University of L'Aquila, Italy, and first described by Marchetti (1980). A very extensive website, containing copies of many publications on the DMT, is maintained by Marchetti at www.marchetti-dmt.it. A DMT apparatus was recently acquired by UWA, and this has been used widely in site investigation work in Perth since then.

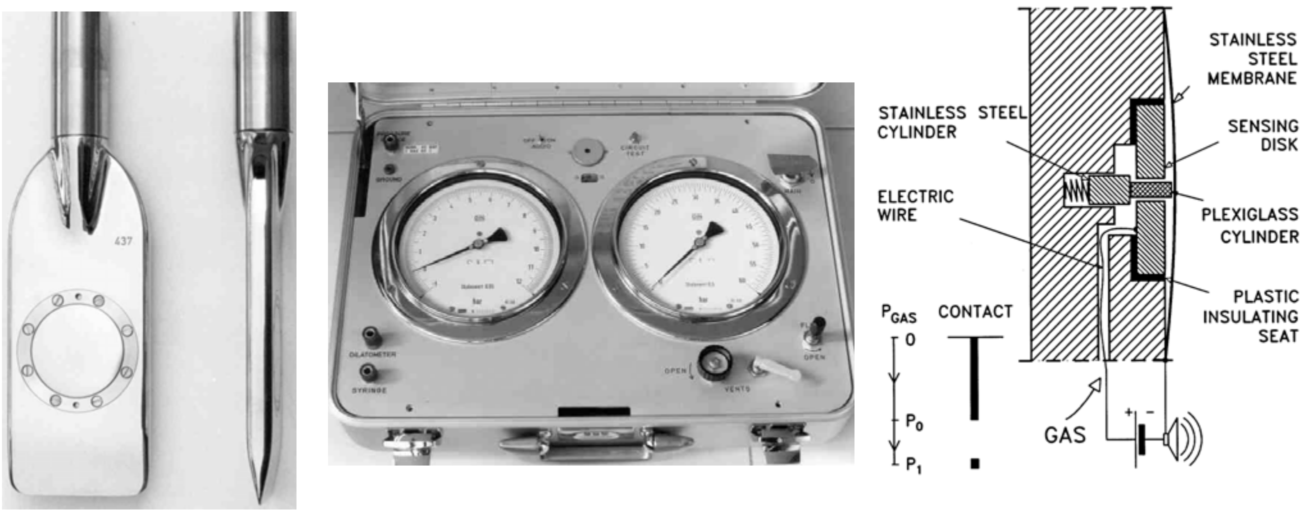


Figure 8. Marchetti dilatometer (DMT): front and side view of the 'blade' (left); view of the readout unit (centre); and schematic of the measuring system for determining 'liftoff' and displacement of 1 mm (right).

The test consists of pushing the DMT blade to the target depth, and then inflating the membrane using gas pressure, via the control unit located on the surface. The point of where the membrane 'lifts off' its seat is indicated by an audible signal, and the pressure at this point is read (the 'A' value). As the pressure is increased, the membrane starts to expand, and when it has expanded by a further 1 mm, as indicated by another audible signal, the reading at this point is also taken (the 'B' reading). A third reading (the 'C' reading) is also sometimes taken at the point during deflation when the membrane recontacts the seat.

The test is analogous to a miniature footing test, and it is obvious that the difference between the A and B pressure readings can be interpreted as some type of stiffness modulus by dividing by some appropriate average strain in the soil corresponding to a displacement of 1 mm. This is the ‘dilatometer modulus’ E_D discussed below.

Table 1 shows a list of the parameters that can be obtained from the basic DMT readings (A, B and C), with the A and B readings corrected for membrane inflation pressures (ΔA, ΔB). In particular, from the point of view of settlement calculations, Table 1 shows two modulus values, a ‘dilatometer modulus’ E_D, and a ‘vertical constrained modulus’ M. The dilatometer modulus E_D was derived by Marchetti (1980) using the theory of elasticity, and is related to the Young’s Modulus E of the soil:

$$E_D = \frac{E}{1-\nu^2} = 34.7\left(p_1 - p_o\right) \tag{2}$$

where p_o and p_1, the corrected readings obtained from the ‘A’ and ‘B’ readings, are defined in Table 1. However, Marchetti insists (see Table 1) that this equation should not be used to obtain E for settlement calculations; rather, the constrained modulus M (or M_{DMT}) should be used for this purpose, where M is derived from E_D taking into account the ‘material index’ I_D, and the ‘horizontal stress index’ K_D, as defined in Table 1.

The main attraction of the DMT is its simplicity of operation. In the testing carried out so far in Perth, a CPT truck has been used as the means of jacking in the blade to the required depths, using standard CPT rods. Testing in the CBD has been carried out to depths of up to 30 m, at a standard depth interval of 0.25 m (4 tests per m).

Table 1: Basic DMT reduction formulae (from TC16, 2001).

SYMBOL	DESCRIPTION	BASIC DMT REDUCTION FORMULAE	
p_0	Corrected First Reading	$p_0 = 1.05\ (A - Z_M + \Delta A) - 0.05\ (B - Z_M - \Delta B)$	Z_M = Gage reading when vented to atm. If ΔA & ΔB are measured with the same gage used for current readings A & B, set $Z_M = 0$ (Z_M is compensated)
p_1	Corrected Second Reading	$p_1 = B - Z_M - \Delta B$	
I_D	Material Index	$I_D = (p_1 - p_0) / (p_0 - u_0)$	u_0 = pre-insertion pore pressure
K_D	Horizontal Stress Index	$K_D = (p_0 - u_0) / \sigma'_{v0}$	σ'_{v0} = pre-insertion overburden stress
E_D	Dilatometer Modulus	$E_D = 34.7\ (p_1 - p_0)$	E_D is NOT a Young's modulus E. E_D should be used only AFTER combining it with K_D (Stress History). First obtain $M_{DMT} = R_M\ E_D$, then e.g. $E \approx 0.8\ M_{DMT}$
K_0	Coeff. Earth Pressure in Situ	$K_{0,DMT} = (K_D / 1.5)^{0.47} - 0.6$	for $I_D < 1.2$
OCR	Overconsolidation Ratio	$OCR_{DMT} = (0.5\ K_D)^{1.56}$	for $I_D < 1.2$
c_u	Undrained Shear Strength	$c_{u,DMT} = 0.22\ \sigma'_{v0}\ (0.5\ K_D)^{1.25}$	for $I_D < 1.2$
Φ	Friction Angle	$\Phi_{safe,DMT} = 28° + 14.6° \log K_D - 2.1° \log^2 K_D$	for $I_D > 1.8$
c_h	Coefficient of Consolidation	$c_{h,DMTA} \approx 7\ cm^2 / t_{flex}$	t_{flex} from A-log t DMT-A decay curve
k_h	Coefficient of Permeability	$k_h = c_h\ \gamma_w / M_h$ ($M_h \approx K_0\ M_{DMT}$)	
γ	Unit Weight and Description	(see chart in Fig. 16)	
M	Vertical Drained Constrained Modulus	$M_{DMT} = R_M\ E_D$ if $I_D \leq 0.6$ $R_M = 0.14 + 2.36 \log K_D$ if $I_D \geq 3$ $R_M = 0.5 + 2 \log K_D$ if $0.6 < I_D < 3$ $R_M = R_{M,0} + (2.5 - R_{M,0}) \log K_D$ with $R_{M,0} = 0.14 + 0.15\ (I_D - 0.6)$ if $K_D > 10$ $R_M = 0.32 + 2.18 \log K_D$ if $R_M < 0.85$ set $R_M = 0.85$	
u_0	Equilibrium Pore Pressure	$u_0 = p_2 = C - Z_M + \Delta A$	In free-draining soils

3.5 SEISMIC METHODS

The shear wave velocity (V_s) in a soil is a useful parameter for engineering purposes, because it depends on the (small strain) shear stiffness of the soil G_o:

$$V_s = \sqrt{\frac{G_o}{\rho}} \qquad (3)$$

where ρ is the bulk density (if G_o is in kPa, and ρ is in t/m^3, this gives V_s in m/s). Hence, shear wave velocity measurement can be used to determine G_o.

3.5.1 Seismic CPT

Where CPT testing is common, as in Perth, the obvious method of measuring V_s for engineering purposes is to add a seismic capability to the CPT – the so-called 'seismic CPT' or SCPT (Robertson et al., 1986). A schematic layout of the test setup is shown in Figure 9. A beam pressed firmly to the ground (e.g. by the jacks of the cone truck) is struck horizontally with a sledge hammer. This generates shear waves that propagate downwards. In the standard '1-point' cone, the travel time is determined from the surface to the cone for each successive depth by determining the arrival time of the first shear wave. The travel time between two successive depths is determined by subtracting the arrival times for these depths. In the less common '2-point' cone, the travel time over a 1 m depth interval is determined directly by comparing the arrival times at each geophone for the same event. The two methods should give the same result, but the 2-point system is preferable because the same event is being compared. Butcher and Powell (1995) show an example from a stiff clay site (Madingley) that indicates that the '2-point' method gives better data, though others claim that the '1-point' cone gives equally good results. In either method, multiple blows can be used and the results 'stacked' to improve the signal-to-noise ratio. Striking the beam at the opposite end gives shear waves of opposite sign, which are reversed numerically in the recording system before stacking. This has the advantage that whereas the reversal gives shear waves of the same polarity (and hence they are additive), it gives compression waves of opposite polarity, and hence stacking tends to cancel the compression waves.

The first seismic CPT in WA was developed at UWA in the late 1980s. This was a 'dummy' CPT, in that it did not incorporate any of the normal CPT functions. It contained two geophones, at 1 m interval, allowing 2-point tests to be carried out. Testing with this equipment was limited to a number of experimental sites (Fahey et al., 1994; Fahey and Soliman, 1994), which was carried out with the cooperation of the Water Authority of WA (using the WAWA CPT truck operated under the direction of Mr Chris Potulski). No tests with this equipment were carried out in the CBD. In recent years, seismic CPT testing has become available commercially in Perth and it has been used in quite a number of CBD sites. Some results will be presented later.

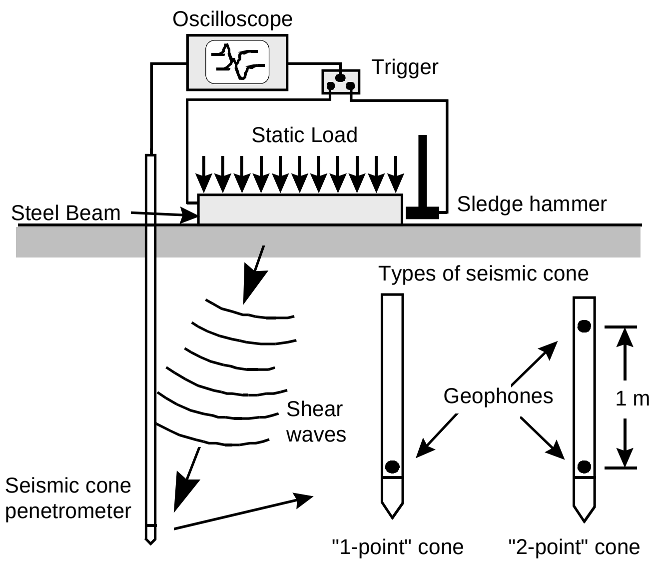


Figure 9: Schematic layout for seismic CPT test.

As made clear earlier, G_o cannot usually be used directly for deformation (settlement) calculation, since it is relevant only to areas where the strains are sufficiently small (say less than 0.001%, or at most 0.01%). Just as with E'_s/q_c ratios discussed earlier with reference to Figure 6, the ratio G_o/q_c is not constant, but depends on soil type, and on q_c, as shown in Figure 10. In this Figure, the normalisation of q_c with σ'_{vo} is slightly different from previously, but only in that the stress terms are first normalised by the atmospheric pressure (p_a) – this being done so that the normalised parameter q_{c1} is dimensionless:

$$q_{c1} = \left(\frac{q_c}{p_a}\right)\sqrt{\left(\frac{p_a}{\sigma'_{vo}}\right)} \tag{4}$$

This normalisation gives x-axis values that are a factor of 10 less than those in Figure 6.

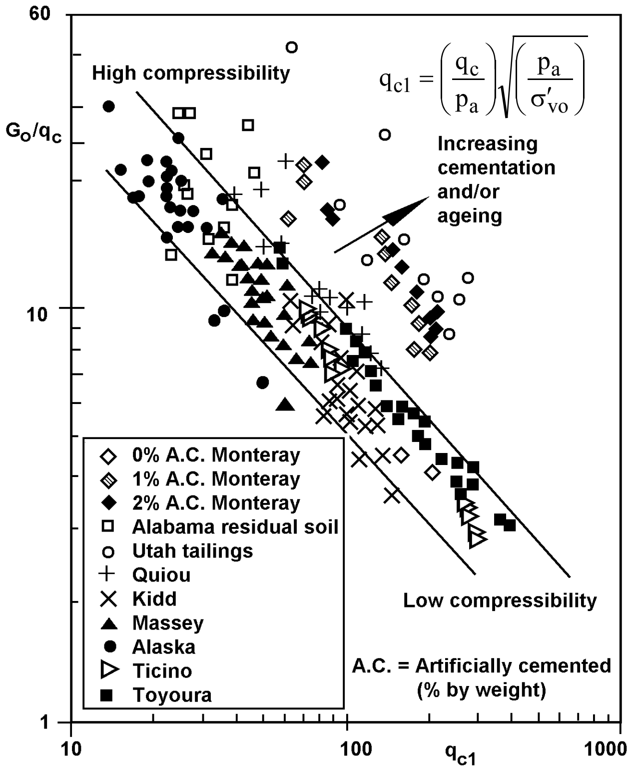


Figure 10: Plot of G_o/q_c versus q_{c1} for various sands (after Robertson, 1997, scanned and re-plotted).

Non-intrusive (surface) methods of determining G_0 profiles are also common. Rayleigh waves of different frequencies (and hence different wavelengths) can be used to determine the average G_0 value over different depth ranges, from which a profile of G_0 can be determined. The SASW method (Stokoe et al., 1989) is a refinement of this technique, in that random waveforms can be generated and inversion techniques used to determine the G_0 profile.

Once the profile of G_0 is determined, this value can be used directly in deformation calculations. In some circumstances, this will give a good prediction of the deformations, particularly in stiff or hard soils. A good example of where this was the case is given by Tatsuoka and Kohata (1994), where using the small strain stiffness directly gave a good prediction of the movement of the very heavily loaded anchor block for the suspension cables of the Rainbow Bridge in Tokyo Bay, founded on a sedimentary mudstone. However, in most cases, it is necessary to use an appropriate modulus degradation curve (such as those shown in Figure 2) in order to obtain a reasonable prediction of settlement. How this might be done is discussed later.

3.5.2 Surface (Rayleigh) waves

Surface waves, or Rayleigh waves, are waves that travel along the surface of the ground as a result of an impact on the ground, or other source of vertical vibrations. The velocity of a Rayleigh wave (V_R) is very close to, but slightly less than, the velocity of a shear wave (V_s), and V_s can be obtained from V_R.

The depth of soil that determines V_R depends on the wavelength (λ) of the Rayleigh wave – in essence, it depends on the average small strain stiffness G_o (and the bulk density ρ) of the soil between the ground surface and a depth about equal to λ. By using waves of different λ, and determining V_R corresponding to each value of λ, a profile of G_o versus depth can be determined by a process of inverse modelling, in which the continuous variation of G_o with depth is represented by a number of discrete layers with constant G_o in each individual layer.

The traditional method of doing this is to use a source of sinusoidal vibrations, in which the frequency can be controlled. Testing involves imparting vibrations at many different frequencies and determining V_R for each one. This is called Continuous Surface Wave (CSW) testing.

The alternative is to use an impact source – a sledgehammer striking a plate, or dropping a heavy mass onto the ground. This produces a complex wave, containing a wide range of frequencies (which can be determined by carrying out Fast Fourier Transform analysis). Each frequency component (each 'phase') travels at a different velocity, because of the dependence of velocity on frequency (wavelength) as explained above. The curve of phase velocity versus wavelength is called the dispersion curve, because of the spreading out (in time) of the signal arriving at each sampling point with increasing distance from the source. Using a technique called Spectral Analysis of Surface Waves (SASW), developed by Stokoe et al. (1989), the dispersion curves can be interpreted to give a profile of G_o versus depth, as with the CSW method.

Equipment for both CSW and SASW techniques has recently been acquired by UWA, and the SASW method has already been used at a number of sites in the CBD and elsewhere in Perth and WA.

4 STIFFNESS MEASUREMENTS IN A CONTEXT OF NON-LINEAR 'ELASTICITY'

Any comparison of stiffnesses obtained from different test methods must be carried out within the context of the non-linear 'elastic' stress-strain behaviour of most (all?) real soils. Thus, the stiffness parameter G_o obtained from seismic CPT (SCPT) tests is relevant to the small strain region (say less than 10^{-4}, or 0.01%); the G_{ur} values from SBP tests involve intermediate strains, as discussed earlier (Section 2).

Theoretical or finite element studies have been carried out by a number of researchers to determine the precise relationship between G_o and G_{ur} (e.g Salgado and Byrne, 1990, Bellotti et al., 1989, and Fahey and Carter, 1993). The non-linear 'distorted hyperbolic' model of Fahey and Carter (1993), which has already been discussed (Equation 1), uses two 'distortion parameters', f and g. They built this model into a cavity expansion FE program to study SBP tests in sand.

Fahey and Soliman (1994) and Fahey et al. (1994) showed how the two distortion parameters can be determined for a sand deposit (and hence the distorted hyperbolic model can be calibrated) by carrying out SCPT tests and SBP tests in the deposit, and varying the f and g parameters in the model to obtain the best possible fit to the test results. (See also Fahey 1998 and 1999). They used this process to determine these parameters for two sites in Perth, one in the Bassendean Sand at a site along the Armadale Railway Line in Bentley, adjacent to the Leach Highway Ewing Street Bridge, and the other on the north side of Vincent Street, under the Mitchell Freeway in Leederville, in a deposit that is part of the Spearwood Dune system.

Using this non-linear elastic model within the FE program AFENA (Carter and Balaam, 1990), with this set of parameters, Fahey (2001) carried out a finite element examination of the performance of a flexible raft foundation, 30 m diameter, founded on a 30 m deep deposit of sand over rock at about 1-2 m below ground level. At a bearing pressure of 400 kPa, a central settlement of about 31 mm was obtained. He also used the same non-linear model to generate SBP results in this deposit and determine G_{ur} values.

The results of this modelling showed:

- The G_{ur} values obtained from the model were about 40% of the input G_o values (the input G_o values increased with depth in accordance with $(p')^{0.5}$.
- The direct use of the G_{ur} values in a linear-elastic foundation settlement analysis overpredicted those from the full non-linear model by about 35% (which is considered not too unreasonable for settlement estimation).
- However, when G_{ur} values were used as the initial stiffness, but these were increased to take account the increase in stress due to the gradual application of the foundation load (using the 'Janbu' option in AFENA), the prediction of raft settlement was almost perfect.

Not surprisingly perhaps, the ratio of G_{ur}/G_o of 0.4 obtained in this exercise is very similar to those obtained experimentally at 8–10 m depth at the Ewing St site (Site 1 in Figure 11), since the model parameters used were obtained by fitting to the data at this site. However, Fahey (1999) showed very clearly that G_{ur}/G_o should not be expected to be the same for all sites, as the modelling showed that it depended on the softening parameter g in the model.

The results of this study must be treated with caution, as it involves only one set of model parameters, calibrated from a single site in the Bassendean Sand (and that site not even being in the CBD). However, it does give an indication of the direction in which future work could proceed – essentially the approach outlined by Fahey (1998). This study is also an important illustration of the opposing effects of increase in stiffness due to increase in mean stress level and reduction in stiffness due to increase in strain level. When considered together, these two factors lead to a near-linear load-settlement response, as is usually observed in the field.

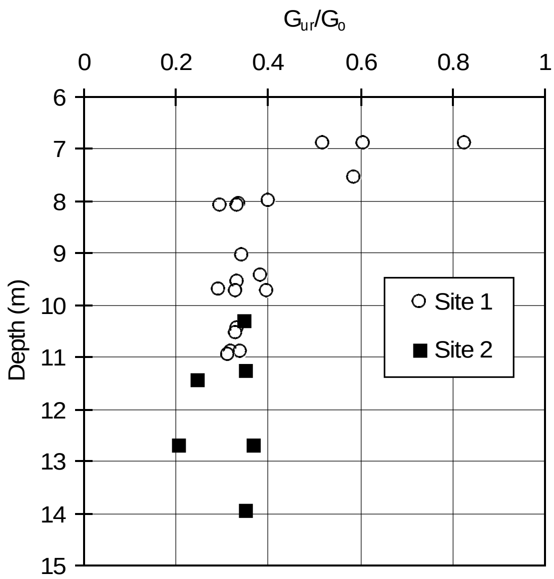


Figure 11: Ratios of G_{ur} from SBP tests and G_o from SCPT test at two sites in Perth (not in the CBD) plotted against depth (Fahey, 1998). The sand at Site 1 is of the Spearwood Dune System, and that at Site 2 is Bassendean Sand.

5 BACK-ANALYSIS OF FOUNDATION SETTLEMENT RECORDS

One of the only reliable ways of determining the appropriate soil stiffness for foundation settlement analysis is to carry out careful measurements of the actual performance of full-sized structures, and to use appropriate back-analysis techniques to determine the value of stiffness for each layer under the foundation. For this type of exercise, measurement of settlement of the foundation itself is not sufficient, but, rather, a complete profile of settlement with depth under the foundation is required. Accurate determination of the loading history is also important. All of this is rarely done.

Fortunately, in Perth, there is one excellent example of this type of measurement. The paper by Fraser (1975) referred to at the end of Section 1 presents the results of such measurement for three buildings in the CBD, constructed in the early 1970s. These three buildings are located within about 300 m of each other, along St George's Terrace, as indicated in Figure 12. They are:

- CBA House, a 12 storey (above ground level) building located on the north-west corner of the intersection of St. George's Terrace and Barrack Street; this building is founded on a 1.2 m thick rectangular raft 26.8 m by 28.7 m. The design gross pressure applied by this raft was 206 kPa.
- NBA House, a 15 storey building located on the north side of St George's Terrace, very close to CBA House, and adjacent to the entrance to London Court. It is founded on a 1.5 m thick rectangular raft, 30.2 m by 31.1 m, with a gross bearing pressure of 170 kPa.
- AMP Building, a 30 storey (131m) building, completed in 1976, located at the north-west corner of the intersection of St George's Terrace and William Street. It is founded on a 2.1 m thick raft, 42 m square in plan, with a gross bearing pressure of 330 kPa.

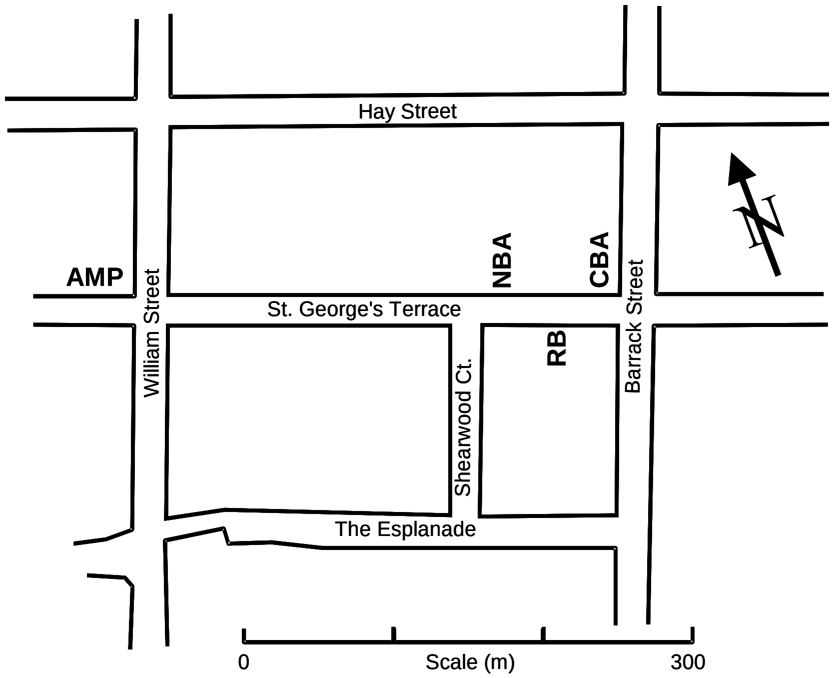


Figure 12: Locations of the 4 sites studied by Fraser (1975).

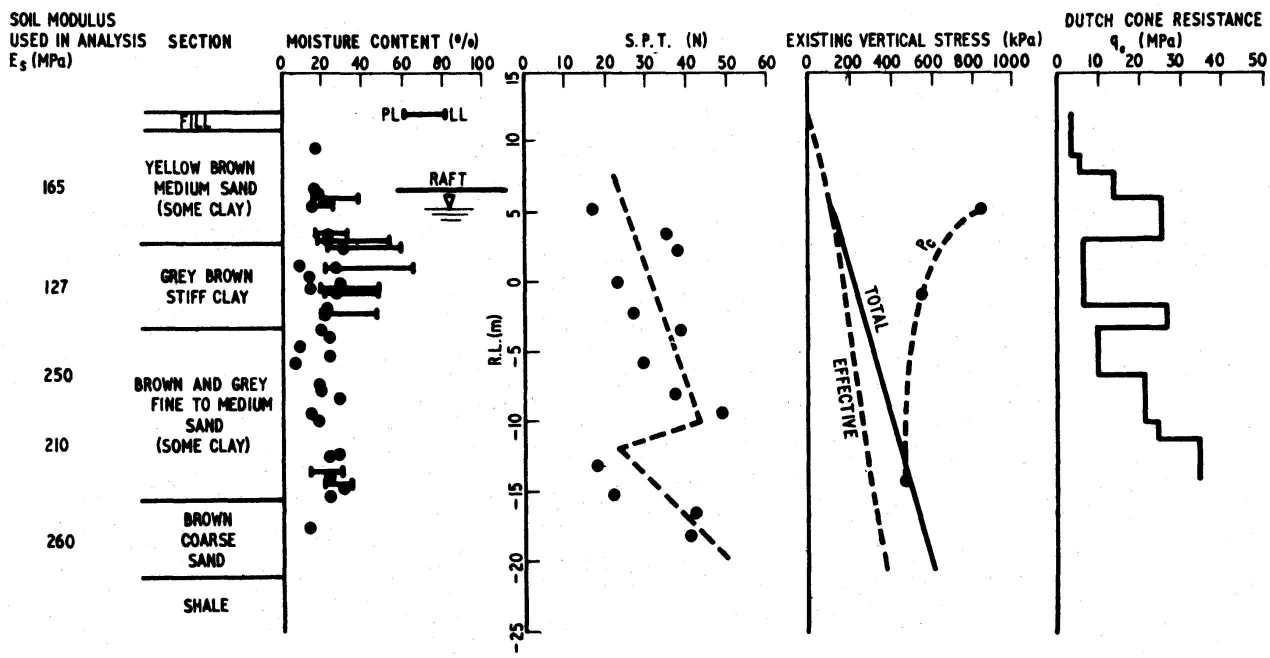


Figure 13: CBA: Soil Properties.

In fact, four buildings are discussed by Fraser but for the fourth, the Reserve Bank building (RB in Figure 12), settlement measurements were not available, so the detailed back-analysis for stiffness of individual layers could not be carried out.

The instrumentation employed included settlement markers located at different depths below the raft foundations, and also total pressure cells embedded in the underside of the rafts. Numerical analysis of each case was carried out, using the program FOCALS (Wardle and Fraser, 1975), with each soil layer being assigned a constant value of Young's Modulus E, and linear elasticity being assumed.

Three key figures from that paper are reproduced here as Figures 13 to 15. These show, for the CBA, NBA and AMP sites, details of the stratigraphy, test results and the best fit back-analysed values of E for each layer for each of the three cases. It is worth emphasising that the actual settlements in each case were relatively small, with central

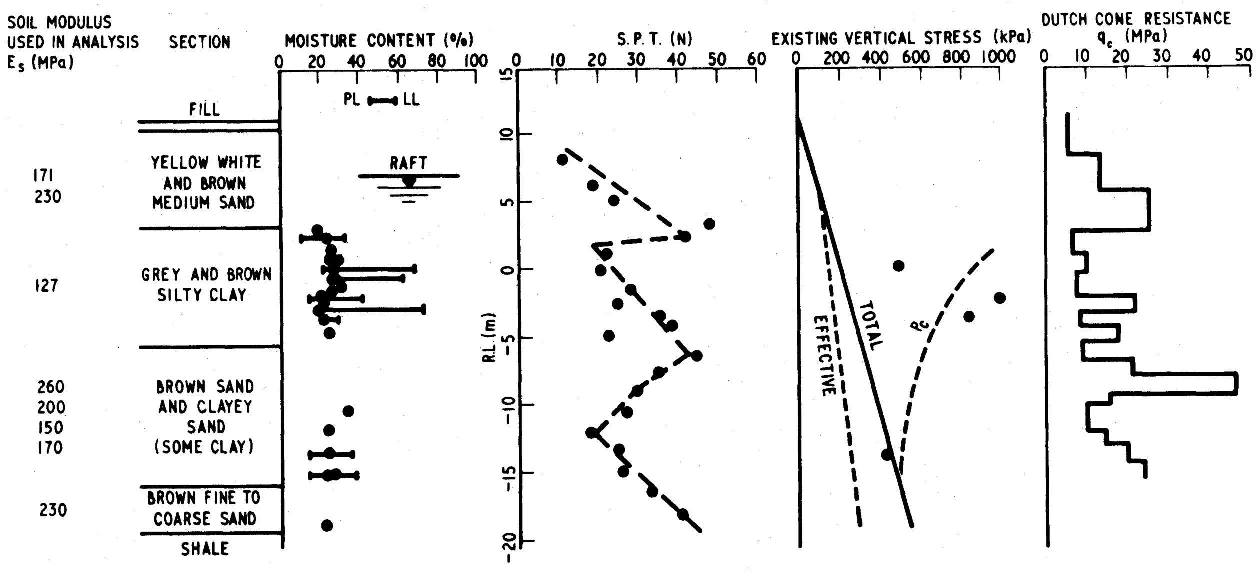


Figure 14: NBA: Soil Properties.

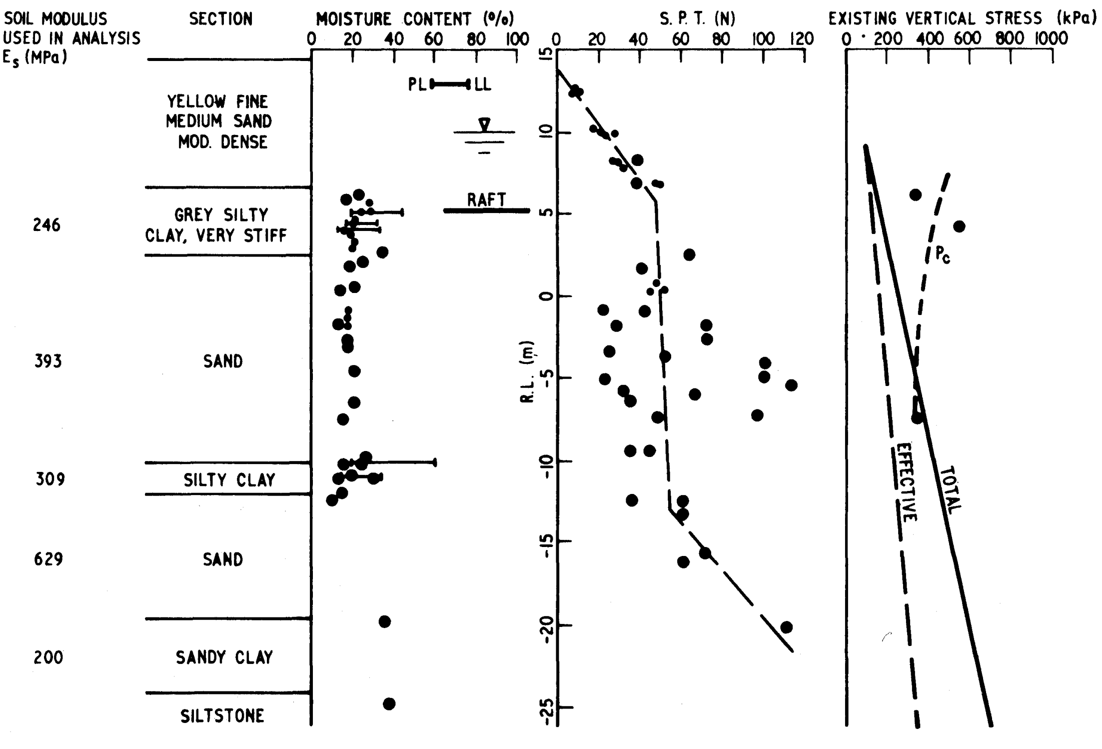


Figure 15: AMP: Soil Properties.

settlements varying from 12 mm for the CBA building, to 24 mm for the AMP building. The largest differential settlement (13 mm) was observed for the AMP building.

Fraser (1975) goes on to assess the various methods of predicting settlement that were then available, focussing particularly on the SPT test. His conclusion was that the method of Sherif (1973), which is presumed to be that described by Schultze and Sherif (1973), provides a good estimate of the stiffnesses. This observation is interesting; it suggests that the SPT test, when interpreted using the Schultze and Sherif method, is capable of providing an accurate prediction of settlements in cases such as these.

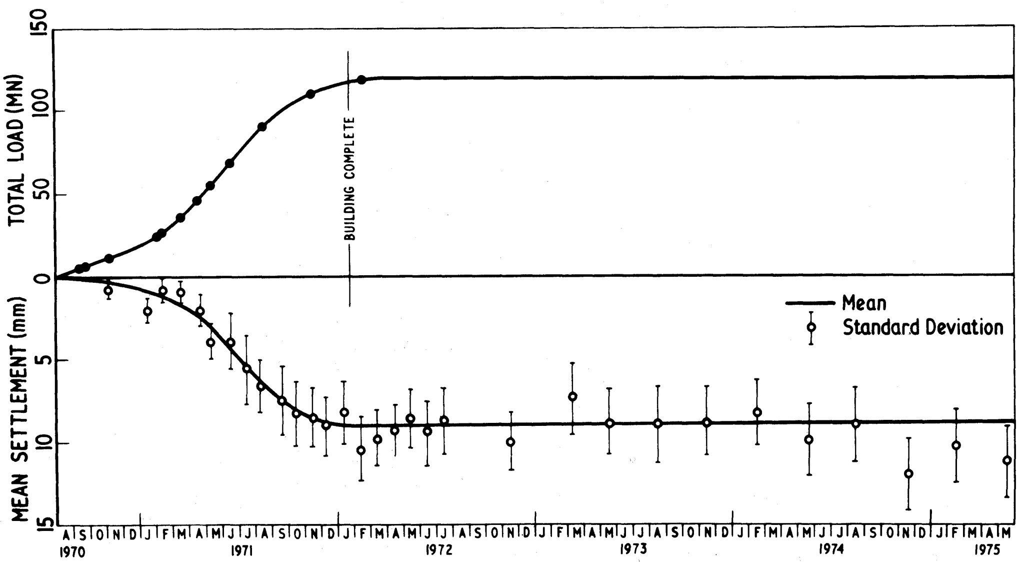


Figure 16: Loading and settlement history, NBA.

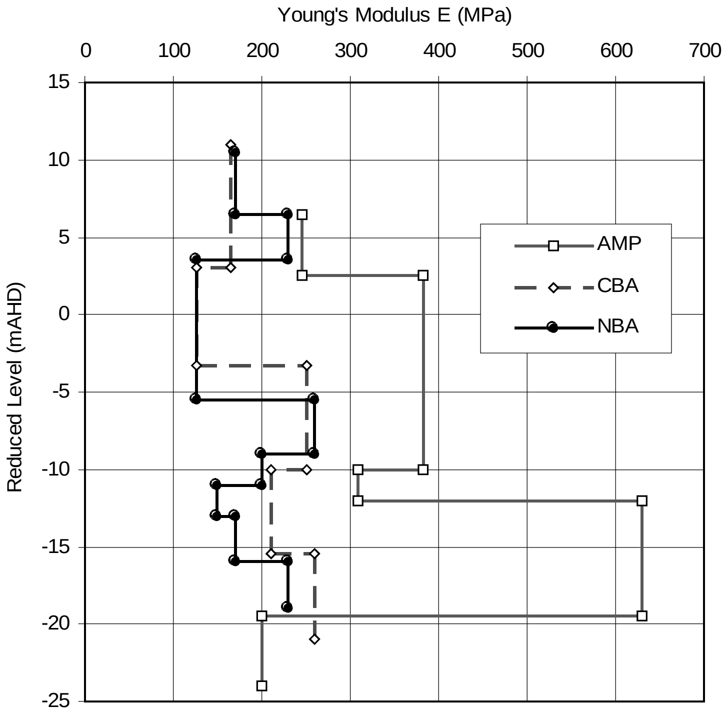


Figure 17: Stiffness profiles back-analysed for the AMP, CBA and NBA buildings (data from Fraser, 1975).

Though each of these sites contain significant thicknesses of stiff clay, Fraser (1975) showed that settlements occurred concurrently with the increase in applied loading, with very little post-construction settlement. (A plot from the paper, for the NBA building, illustrating this point, is shown in Figure 16). Hence, these clay layers are sufficiently overconsolidated that they can be treated as elastic, for the purposes of settlement analysis.

The stiffnesses obtained from each of these sites is shown plotted against RL in Figure 17, which shows that the stiffness profiles for the NBA and CBA sites are very similar, but stiffnesses are considerably higher at the AMP site.

One important point to bear in mind is that the bearing pressures used to derive these stiffness values are the gross bearing pressures, whereas in each case, due to the founding level of the raft being well below the original ground level, the net bearing pressures are considerably less than the gross values. For example, at the AMP site, Fraser states that the gross bearing pressure is 330 kPa, but the net bearing pressure is 122 kPa. For purely elastic analysis, the gross bearing pressure would be generally used, but if there is any chance that the stiffness changes (reduces) once the original vertical stress at the founding level has been exceeded, then some consideration might be given to using different stiffnesses for the portion of the loading equivalent to the original vertical stress.

However, this does not explain why the stiffnesses for the AMP site should be so much higher, when as stated earlier, it is not very far from the other two sites (300 m or so at most). The gross bearing pressure for the AMP foundation is higher than for the other two. However, when examined in the context of non-linear elasticity, this higher gross bearing pressure should result in *lower* equivalent secant stiffnesses for this site. This may be simply a reflection of the variability in stiffness with location in the CBD, with higher stiffness in the area around the AMP building – a hypothesis that is supported by some other more recent evidence from this area. There may also be some size effect at work – the raft for the AMP building is considerably bigger than those for the other two.

6 IN SITU MEASUREMENTS OF SOIL STIFFNESS

As mentioned previously, settlement predictions for buildings on shallow foundations in Perth can be made on the basis of results of SPT tests, CPT tests, SBP and other pressuremeter tests, DMT tests, and seismic tests (seismic CPT and SASW and CSW surface wave methods). In most current site investigations, CPT testing is much more common that SPT testing, so the latter will not be dealt with further (though the conclusion of Fraser, 1975, discussed above, that SPT tests can be used with reasonable accuracy to predict settlement, is worth bearing in mind).

Of all these methods, the ones that give a direct measure of stiffness are pressuremeter tests, seismic tests, and DMT tests (though the latter involves some correlation).

6.1 STIFFNESS FROM PRESSUREMETER (SBP)

In the Perth CBD, pressuremeter tests have been carried out for a number of projects. Some of these have involved testing in the King’s Park Shale, for the purposes of pile design, using insertion-type pressuremeters. The stiffnesses obtained from these tests will not be discussed here. For now, we will focus on SBP measurements made, using the UWA equipment, within the sand and clay layers above the King’s Park Shale. The sites in question are: Site M, on the north side of Mount’s Bay Road; Site W at the west end of the CBD; Site N, on the north side of St. George’s Terrace at the centre of the CBD and Site B, at the west end of the CBD.

The G_{ur} values from these SBP tests are plotted against depth below the ground surface, and against RL, in Figure 18. There are valid arguments for plotting against both depth and RL, so both have been included[4]. For any test in which two or more unload-reload loops have been performed, both values are included in this Figure. Superimposed on the SBP data are values of G obtained from the E values presented in Figure 17 deduced from the three sites by Fraser (1975), as previously. For this calculation, the value of Poisson’s Ratio was taken to be 0.3. This Figure shows that the SBP values are comparable to the values obtained by Fraser (1975). The data from Site N are particularly interesting.

[4] These arguments centre on whether the stratigraphy is likely to be horizontal, or to follow the ground contours – if the former, then plotting against RL appears to make the most sense, but if the latter, then plotting against depth is more sensible. However, even where the stratigraphy is horizontal, but the ground surface is sloping, stiffness is still, to some extent, influenced by depth – i.e. by vertical effectives stress – so even in this case, it is worth also plotting against depth.

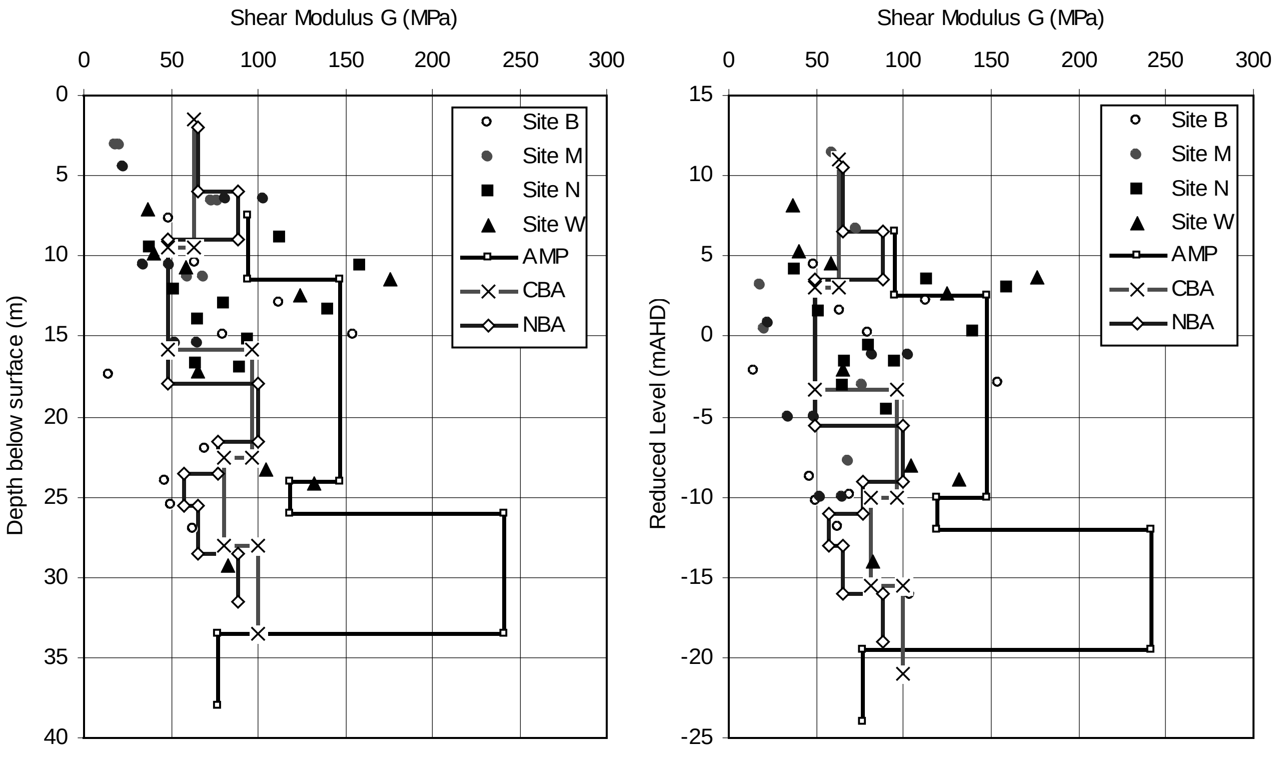


Figure 18: Shear modulus (G_{ur}) profiles from SBP tests at various sites in the CBD, compared to G values deduced from the E values from Figure 17 (assuming Poisson's Ratio $\nu = 0.3$).

As stated, this site lies close to the NBA site, between the NBA site and the AMP site. The SBP data from the Site N happen to plot very close to those from the NBA site for the most part, but with three higher values plotting very close to the AMP profile.

The importance of this is that it tends to confirm the general impression gained from this type of testing in stiff soils in Perth and in WA generally over the past 20 years – that the SBP G_{ur} values can be used directly to predict foundation settlement with a reasonable degree of accuracy. This seems to agree with the findings from the FE study by Fahey (2001) discussed earlier.

6.2 'SMALL STRAIN' STIFFNESS G_o

The seismic CPT has now been used in a number of site investigations in the Perth CBD. As was made clear in the discussion on stiffness in Section 2, these G_o values should be higher than the secant G values obtained from any other direct measurement method.

We do not yet have SCPT G_o values from any of the three sites discussed earlier for which Fraser (1975) back-analysed values of stiffness (or even from adjacent sites). In light of the apparent reasonable fit between these back-analysed stiffnesses and the G_{ur} values from SBP tests at adjacent or nearby sites, the best we can do for now is to use G_{ur} values as the best benchmark of the most appropriate 'working strain' stiffnesses for use in foundation design in the Perth CBD, and compare SCPT stiffnesses (and stiffnesses obtained from other devices) with these.

Figure 19 shows G_o values from SCPT tests compared with G_{ur} values from SBP tests at three sites. At Site M, (Figure 19a), the SBP tests were carried out as part of the original site investigation for this site, whereas the SCPT tests were carried out some 10 years later. In the intervening period, the ground surface level was reduced by excavation by about 7.5 m – i.e. from about RL + 4.2 m to RL –3.3 m. These surface elevations are indicated on Figure 19(a) by solid and dashed thick lines, respectively.

Figure 19(b) shows SCPT G_o values from Site B and Site W, and SBP G_{ur} values from the same sites, plotted against depth below ground surface. (The data from these two sites are plotted together, as they are in close proximity, at the western end of the CBD). Two SCPT G_o profiles are plotted for Site W (P1 and P2). It can be clearly seen for both sites that the G_o values are very much greater than the G_{ur} values, as expected.

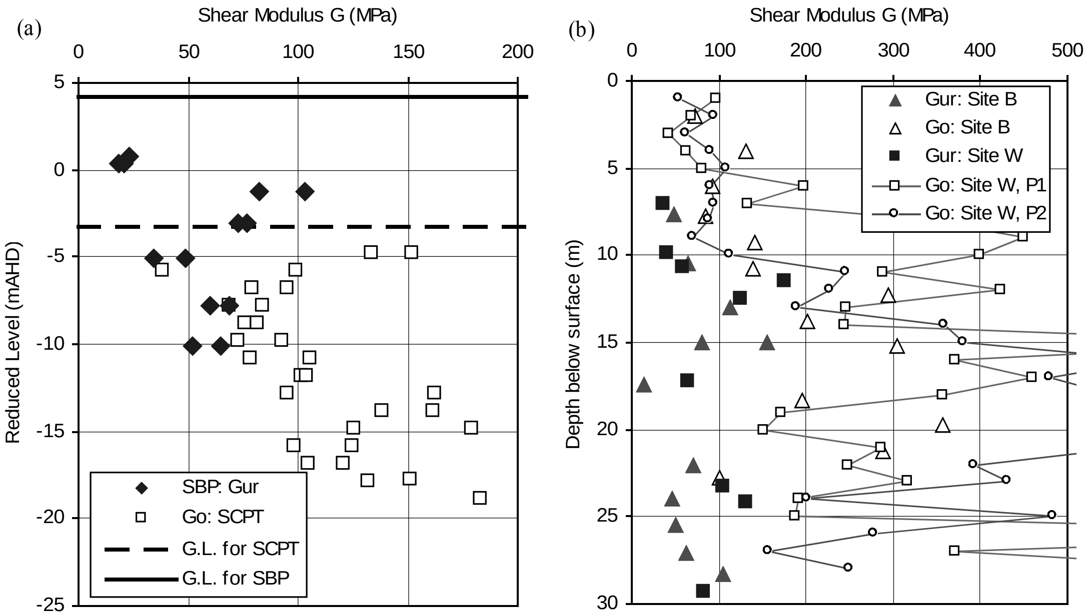


Figure 19: Seismic CPT G_o values compared to SBP G_{ur} values: (a) Site M (note excavation at site after SBP tests but before SCPT tests); and (b) Site B and Site W.

Referring again to Figure 19(a), it can be seen that because of the excavation that took place, there are only 3 SBP test results at RLs at which there are SCPT G_o values (there are actually 6 G_{ur} values shown for these three SBP tests – these are the G_{ur} values from the first and second unload-reload loops of these tests). Over this overlap interval, the G_o values are still higher than the G_{ur} values, but the overall trend in the data might suggest that the difference is not as great as that shown in Figure 19(b) for sites B and W.

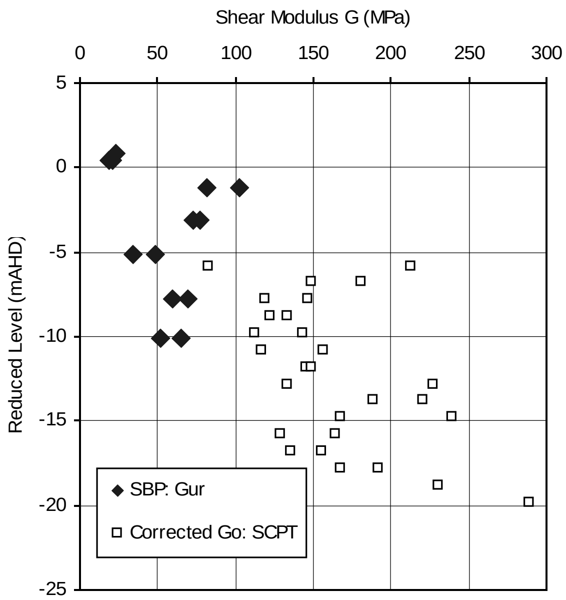


Figure 20. Comparison of corrected G_o values inferred from SCPT data and SBPT G_{ur} data at Site M (SCPT G_o values from Figure 19(a) corrected for stress level change due to excavation at site).

However, what must be taken into account in assessing Figure 19(a) is the reduction in effective confining stress that occurred due to the excavation. It is generally accepted that the small strain stiffness of sands (and clays) depends on effective confining stress to some power (0.5 is commonly assumed). Thus, for a correct interpretation of Figure 19(a), the G_o values should be increased to reflect the values that would have been obtained prior to excavation. The result of this is shown in Figure 20, in which the G_o values from the SCPT have been increased to take into account the overburden removal. This now indicates more clearly that the G_o values are indeed significantly greater than G_{ur} values for this site.

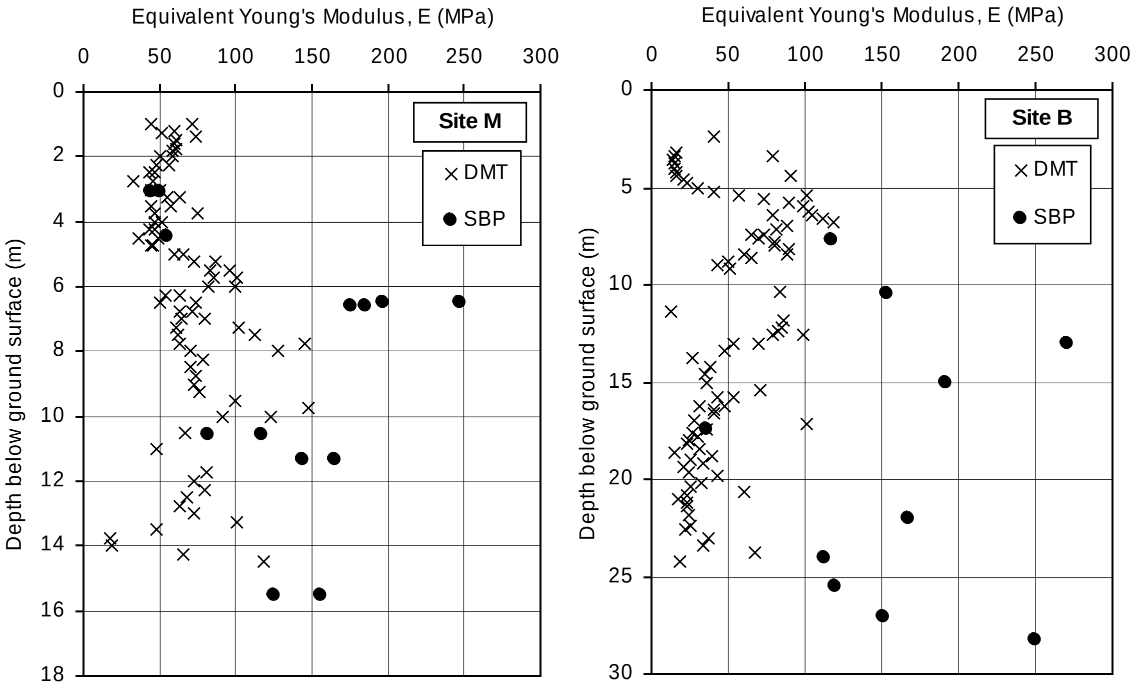


Figure 21: Equivalent Young's Moduli derived from DMT results and from SBPT G_{ur} data at Site M and Site B.

6.3 STIFFNESS FROM THE DMT

The dilatometer modulus (E_D) obtained in a DMT is a direct measure of lateral soil stiffness after installation of the device. Marchetti (1980) recommends that this modulus should be modified by an empirical factor (R_m) to obtain a soil stiffness appropriate for settlement predictions of spread foundations. The value of R_m is related to the nature and stress history of the deposit through correlations with the DMT's horizontal stress index, K_D, and is derived using the relationships provided in Table 1. An equivalent Young's modulus (E_{eq}) applicable to spread foundations is then obtained as:

$$E_{eq} = 0.8\ R_m\ E_d \tag{5}$$

Values of E_{eq} derived using the DMT in the sand strata at Site M and Site B sites are plotted on Figure 21 and are compared with equivalent Young's moduli derived from the SBP G_{ur} data obtained at these sites assuming $E_{eq}=2(1+\nu)G_{ur}=2.4G_{ur}$. It is apparent that the inferred DMT moduli are typically between 1.5 and 4 times less that those obtained from the SBP data and are also significantly lower than the back-calculated E_{eq} values plotted on Figure 17. The apparent under-estimation of equivalent stiffness from the DMT is believed to be primarily due to the higher strains induced in the ground during the DMT membrane expansion than under the raft foundations discussed in Section 5 and during the unload-reload excursions in SBP tests.

6.4 STIFFNESS FROM THE CPT

As discussed in Section 3, a single reliable proportional relationship between an equivalent Young's modulus (E_{eq}) and the CPT end resistance, q_c, should not be expected. The operational E_{eq}/q_c ratio for a raft foundation similar to those constructed at the AMP, CBA and NBA sites can, however, be obtained if the average backfigured E_{eq} value of 250 MPa deduced from Figure 17 is combined with the average CPT q_c values at other sites in the CBD. This procedure leads to E_{eq}/q_c ratios of 18 and 15 at Site B and Site M, respectively. These ratios fall towards the upper-end of the expected range for overconsolidated sands on the basis of the data shown on Figure 6. For the CBA and NBA sites, Fraser (1975) also provided data from a Dutch cone, the mechanical precursor to the modern "electric" cone penetrometer (Figures 13 and 14). Comparison with his backanalysed stiffness data in these Figures indicates a ratio between E and Dutch cone resistance (q_c) of between about 10 and 20 for the sand strata.

Some other limited correlations have been attempted between stiffness and CPT data for sites within and outside the CBD. Information presented by McInnes and Waterton (1980) suggests that a ratio E/q_c of about 5 to 6 would be required to match the measured settlement of the District Court building in Perth, while a ratio of about 8 would match the measured settlement of the King Edward Memorial Hospital.

The results of loading tests on plates of between 300 and 1000 mm diameter on various sand sites in the Perth metropolitan area yield E/q_c ratios of between 3 and 10 as reported by McInnes and Waterton (1980). If the results for the 300 mm diameter plate are ignored, plates of 750 to 1000 mm diameter yielded E/q_c ratios of between 8 and 10. The values for the larger plates are likely to be more appropriate given the effect of various factors such as disturbance and plate size.

Smith (1984) quoted the results of loading tests on plates of between 300 and 750 mm diameter at a site in South Perth. These results yielded E/q_c ratios of between 4.5 and 6.5. Settlement predictions made using these data compared relatively well with measured settlements, although the predictions appear to be generally higher than the measured values.

7 LINK BETWEEN STIFFNESSES MEASURED IN VARIOUS *INSITU* TESTS

A comparison of the stiffness values measured using various *insitu* test devices with those backfigured beneath three rafts in the Perth CBD has indicated that the moduli obtained from unload-reload loops (G_{ur}) in SBP tests (following the procedure discussed in Fahey 2000) generally approximate the backfigured values. However, stiffnesses obtained from shear wave velocity (V_s) measurements are typically 2 to 3 times higher than the SBP values, while those derived from the DMT are 2 to 3 times lower than the SBP values. It is imperative to point out, however, that under different circumstances the moduli derived from V_s or DMT data may be more appropriate. For example, Mandolini (2001) shows that G_o can be used directly to evaluate the settlement of pile groups, while the DMT E_{eq} data have been confirmed by Schmertmann (1986), Hayes (1990), and others to be suitable for the evaluation of footing settlements, where the applied load is generally a higher proportion of the foundation's ultimate capacity than in the case of a raft.

This Section investigates a link between the various stiffness parameters by combining established trends such as those indicated on Figures 6 and 10 with observations in the Perth CBD using the DMT and SBP. Central to the establishment of such a link is the recognition of the variable level of strain imposed on the soil by *in situ* test devices. In addition, consideration is also given to recognised effects of age and overconsolidation on stiffness and its rate of degradation with strain. Attention is focused on sand stiffness as, at present, most recorded data involving the SCPT, SBP and DMT in the Perth CBD have been obtained in sand.

7.1 APPROACHES FOR ESTIMATING SAND STIFFNESS

Approaches for assessing sand stiffness from (i) SCPT shear wave velocities, (ii) CPT end resistance q_c, (iii) dilatometer tests and (iv) pressuremeter tests are now examined.

7.1.1 Very small strain modulus

The variation of G_o with q_c observed in a range of sand deposits by Robertson (1997) was summarised previously in Figure 10. The bounds to all G_o data presented in this figure may be expressed as:

$$G_o = 450\sqrt[3]{q_c.\sigma'_v.p_{atm}} \quad \text{(Upper bound)} \tag{6}$$

$$G_o = 110\sqrt[3]{q_c.\sigma'_v.p_{atm}} \quad \text{(Lower bound)} \tag{7}$$

These equations are seen on Figure 22 to match the range of values recorded by the SCPT in the Perth CBD, with the lower bound being compatible with recently deposited sand fill. Equivalent expressions for the small strain Young's modulus (E_o) are obtained as follows assuming a typical small strain Poisson's Ratio of 0.1:

$$E_o = 1000\sqrt[3]{q_c.\sigma'_v.p_{atm}} \quad \text{(Upper bound)} \tag{8}$$

$$E_o = 250\sqrt[3]{q_c.\sigma'_v.p_{atm}} \quad \text{(Lower bound)} \tag{9}$$

It should be noted that the preceding equations refer to very small strain moduli at *in situ* stress levels. Higher operational values of G_o and E_o would exist beneath a loaded foundation because of the higher stress levels.

7.1.2 Relationship of Baldi et al. (1989)

Baldi et al. (1989) showed the variation with q_c of an equivalent Young's Modulus comparable to that recorded in a drained triaxial test at an axial strain (ε_a) of 0.1%. This relationship was shown previously in Figure 6. The plotted trend lines have been re-examined and were found to be well represented by the following expressions:

$$E_{eq} = (210 \pm 50)\sqrt[3]{q_c.\sigma'_v.p_{atm}} \quad \text{(Overconsolidated sand)} \tag{10}$$

$$E_{eq} = (105 \pm 25)\sqrt[3]{q_c.\sigma'_v.p_{atm}} \quad \text{(Aged normally consolidated sand)} \tag{11}$$

$$E_{eq} = (40 \pm 10)\sqrt[3]{q_c.\sigma'_v.p_{atm}} \quad \text{(Recent normally consolidated sand)} \tag{12}$$

Independent data reported by Aubeny (1992) for recent normally consolidated Toyoura sand also indicate that equation (12) is compatible with the Young's modulus measured in a triaxial test at an axial strain of 0.1%. Consequently, in keeping with findings of Jardine (1992) and Atkinson (2000), these E_{eq} values are expected to be representative of operational moduli beneath a spread foundation at a foundation settlement to width ratio (s/B) of about $3.5\varepsilon_a \approx 0.35\%$

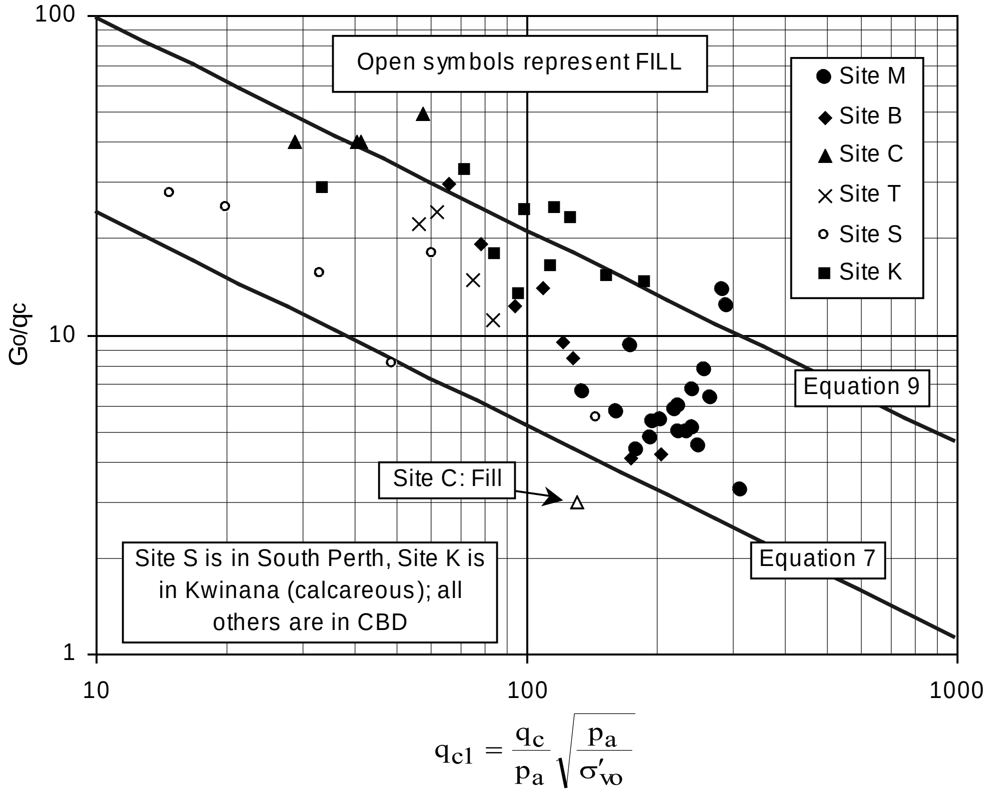


Figure 22: Relationship between G_o (from SCPTs) and q_c observed in Perth CBD sand.

7.1.3 Dilatometer modulus

The values of dilatometer modulus, E_d, measured in the CBD are plotted on Figure 23 and are seen to exhibit a similar relationship with q_c and σ'_v to that of G_o and E_o i.e. equations (6) to (9). No comparable consistent relationship for E_{eq}, derived using equation (5), was observed.

Statistical analysis of these E_d data indicated the following relationships:

$$E_d = (90 \pm 40)\sqrt[3]{q_c.\sigma'_v.p_{atm}} \quad \text{(Overconsolidated sand)} \tag{13}$$

$$E_d = (70 \pm 20)\sqrt[3]{q_c.\sigma'_v.p_{atm}} \quad \text{(Aged normally consolidated sand)} \tag{14}$$

These moduli are derived from the lateral stress required to induce a maximum lateral movement of the flexible dilatometer membrane, which has a diameter of 60mm. They may therefore be thought of as a lateral E_{eq} value at an s/B value for a rigid foundation equal to:

$$\frac{s}{B} = \frac{\pi}{4}.\frac{1}{60} = 1.3\% \tag{15}$$

7.1.4 Unload-reload modulus in SBP test

As discussed earlier, Fahey (1998) presents a procedure for obtaining appropriate shear moduli for foundation design in sands from unload-reload loops in pressuremeter tests. As discussed in Section 4, these unload-reload moduli, which correspond to equivalent moduli at a cavity strain of ≈0.1%, are generally about 40% of G_o inferred from SCPT shear wave velocity measurements.

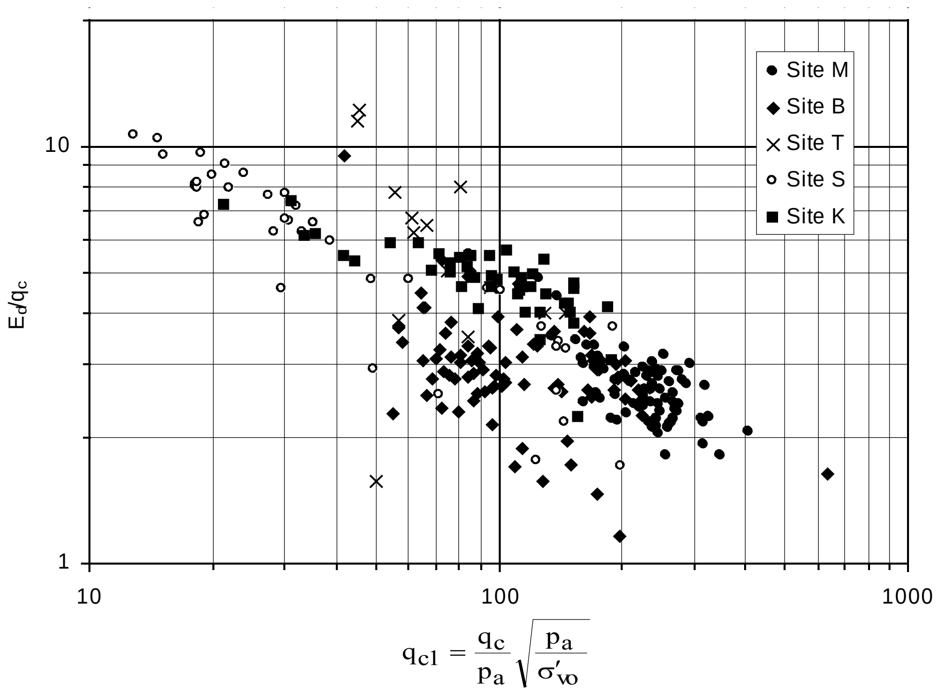


Figure 23: Relationship between the dilatometer modulus, E_d, and q_c observed in Perth CBD.

7.2 A STRAIN-DEPENDENT CORRELATION FOR E_{EQ}

The four approaches described in Section 7.1 are combined on Figure 24, which provides an approximate means of assessing E_{eq} for foundation design. The data plotted relate to:

- E_o at S/B of less than 0.001%
- E_{SBP} at S/B of 0.1%
- $E_{eq}$from Baldi *et al* (1989) at S/B of 0.35% and
- E_{DMT} at S/B of 1.3% (without correction for any disturbance/anisotropic effects).

It is apparent that the moduli given by these approaches fall into a consistent and credible pattern when the effects of strain level are considered. It is also of importance to note that E_{eq} is not a linear function of q_c and that it depends strongly on the sand state in addition to the strains induced in the soil by the foundation.

Dutch cone resistance q_c values (i.e. from a mechanical cone, rather than a modern ‘electric’ cone) are available at the CBA and NBA sites (see Figures 13 and 14). Although these q_c values may differ slightly from those obtained using a standard electric cone, their combination with the backfigured E_{eq} values in the sand layers at these sites provide a convenient means of assessing the applicability of Figure 24. Values of $E_{eq}\big/\sqrt[3]{q_c \cdot \sigma'_v \cdot p_{atm}}$ calculated in this way are plotted at the observed s/B values for the CBA and NBA rafts and evidently fall between the mean trend lines established for normally consolidated and overconsolidated aged sand.

The dependence of the rate of degradation of stiffness with strain on the overconsolidation state of the sand deposit is apparent from Figure 24 and has been confirmed in laboratory tests (e.g. Lo Presti 1994). It follows that *insitu* tests should include both a small strain stiffness measurement (i.e. using the SCPT) and a larger strain stiffness measurement (e.g. SBPT or DMT) to enable the (clearly important) effects of stiffness non-linearity to be modelled by designers.

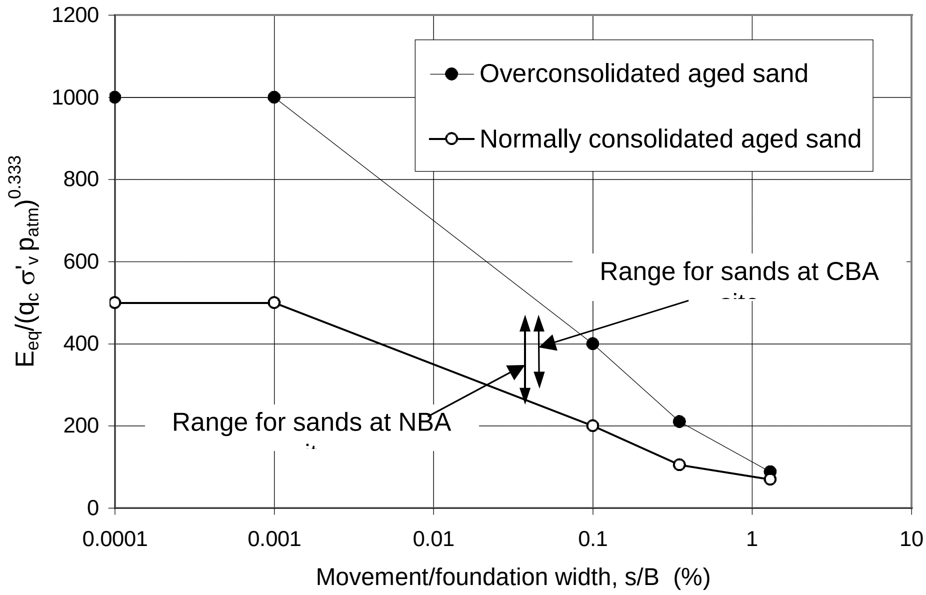


Figure 24: Proposed relationships for E_{eq} for sands in Perth CBD.

The preceding discussion indicates that correlation of the equivalent soil stiffness with cone penetration resistance is dependent on the relative strain level (the ratio of footing settlement to footing width). This effect is also evident when correlating the results of various in situ tests, which impart varying levels of strain to the ground. This effect is problematic in that a single type of in situ test will not necessarily provide the "correct" value of soil stiffness to use in design for all footings and because a single ratio of E/q_c will not be appropriate for all footing types and ground conditions.

If the relationships presented in Figure 24 are examined in the context of two different feasible footing arrangements, the following comments can be made:

- For a 30 m wide raft foundation that undergoes 20 mm of settlement, the ratio s/B = 0.07% and therefore a stiffness ratio of about 250 to 500 can be read from Figure 24.
- For a 4 m wide spread footing that undergoes 10 mm of settlement, the ratio s/B = 0.25% and therefore a stiffness ratio of 150 to 300 can be read from Figure 24.
- If the above values relate to equivalent values of q_c and σ'_v, then the equivalent stiffness is seen to be significantly different for each footing size, indicating that the value of E/q_c would vary by a factor of about 2 between the two cases.
- For typical values of various parameters, the relationships shown will lead to values of E/q_c that reduce with increasing q_c. Depending on the parameters selected, values of E/q_c of typically about 5 to 25 could be derived from Figure 24.

These comments indicate that the use of a single value of E/q_c is not appropriate for all situations. The relationships presented in Figure 24 place the various values of measured soil stiffness in context and provide a means of estimating the appropriate equivalent soil stiffness for the footing dimensions and settlement being considered. However, further assessment of these relationships against field measurements is required to evaluate their applicability for use in footing design.

When settlement calculations are performed using a fully rigorous non-linear elastic model, in which the stiffness increases with increasing mean stress and reduces with increasing strain, it is found that the overall effect is load-settlement response that is quite linear (see for example, Fahey, 2001). Because of this linearity of response, it is clearly possible to derive the correct load settlement response using a linear elastic model also, but of course this requires the choice of an appropriate linear stiffness value. The key aspect of a calculation based on linear elasticity is therefore the selection of an appropriate soil stiffness value, considering both strain level and confining stress effects. While the solution to this problem is not provided in this paper, the information presented helps to provide an understanding of these effects and indicates a potential direction for design. The information also agrees in general with the findings of Schultz and Sherif (1973) and Burland and Burbidge (1985), which also indicate that soil stiffness is not a direct linear function of cone resistance or SPT blow count.

8 CONCLUSION

The stratigraphy at any site in the Perth CBD is readily obtained from existing information, or from CPT tests or boreholes carried out specifically for the project. However, for most large projects in the CBD, the parameter of most interest is the stiffness of the various layers of soil overlying the bedrock, though for piled structures, the capacity of the piles, which does not depend only on stiffness, is a very important issue. For some projects, knowledge of the horizontal stress would also be very useful (basement walls, tunnels, for example).

We have emphasised in this paper that the stiffness of soil is not a single unique property, because of the dependence of stiffness on confining stress level and degree of loading (the level of shear strain, or the shear stress imposed compared to the shear strength). However, we have also emphasised that stiffness can be measured – both the initial tangent stiffness G_o, and larger-strain stiffnesses (G_{ur} from SBP tests, or E_D from DMT tests) – and that the relationships between these can be understood within the framework of non-linear stiffness. The full range of stiffness parameters can also be obtained from carefully conducted laboratory triaxial tests on high-quality samples. It is notable, and regrettable, that none of the advances in sampling techniques that have occurred elsewhere in the world have been adopted for use in even the most important projects in Perth.

It has been the practice for the past few decades to use correlations between penetration resistance (SPT N-value, or CPT q_c-value) and stiffness for routine design. It is now understood that these have very limited accuracy and must be used with great caution. This is obvious, since the factors that influence penetration resistance are not necessarily the same as the factors that influence the stiffness; penetration tests fail the soil, and hence mobilise the strength, whereas measurement of the stiffness relevant to working loads involves loading to well below the strength. Nevertheless, if due consideration is given to stress level, mobilised strain level, etc, appropriate linear stiffness parameters can be obtained from such tests – particularly the CPT – but the correlations to be used in such derivations will vary from site to site, and even with the size of footing within a particular site.

It appears that the best direct measure of stiffness for use in settlement prediction of foundations in the stiff sands and clays of the Perth CBD is provided by the pressuremeter, more specifically by the self-boring pressuremeter (SBP). The G_{ur} values obtained with the SBP match quite well the stiffnesses back-analysed from the measured settlements of the only buildings in Perth where appropriate instrumentation was used to monitor the performance. It is sobering to realise that this excellent monitoring work was carried out about 30 years ago and, in spite of its demonstrated usefulness, has never been repeated for any of the subsequent projects undertaken in the CBD.

Because of the practical impossibility of obtaining undisturbed samples of sands for laboratory stiffness measurement, the emphasis in this paper has been on *insitu* methods of determining stiffness. However, for the stiff to hard clays in the Perth CBD, modern high-quality sampling techniques, coupled with modern high-quality laboratory (triaxial) testing incorporating high-precision internal load and strain measurement, can provide the required stiffness information for these soils. Thus, for these soils, laboratory testing provides another acceptable method of stiffness measurement.

Our overall conclusion is therefore that for projects (in the Perth CBD and elsewhere) where the main interest is deformation/settlement, engineers should take advantage of the appropriate tools available to measure stiffness and to use these measurements in a sensible way, which requires due recognition of non-linearity and the effect of stress level on stiffness.

9 ACKNOWLEDGEMENTS

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10 REFERENCES

Andrews, D.C. (1971). *Soils of the Perth Area – The City Centre*. CSIRO, Division of Applied Geomechanics, Technical Report No. 13.

Atkinson, J.H. (2000). Non-linear soil stiffness in routine design: 40th Rankine Lecture. *Géotechnique* **50**, No 5, 487-508.

Atkinson, J.H. and Sallfors, G. 1991. Experimental determination of stress strain time characteristics in laboratory and *in situ* tests. *Proc. 10th European Conf. on SMFE*, Florence, Italy, Vol. 3, 915–956, Balkema, Rotterdam.

Aubeny, C.P. (1992). *Rational Interpretation of In Situ Tests in Cohesive Soils*. PhD Thesis, MIT, Cambridge, Mass.

Baldi, G., Bellotti, R., Ghionna, V.H., Jamiolkowski, M. and Lo Presti, D.C. (1989). Modulus of sands from CPTs and DMTs. *Proc. 12th Int. Conf. on SMFE*, Rio de Janeiro, Vol. 1, 165–170, Balkema, Rotterdam.

Bellotti, R., Ghionna, V., Jamiolkowski, M., Robertson, P.K. and Peterson, R.W. (1989). Interpretation of moduli from self-boring pressuremeter tests in sand. *Géotechnique*, **39**, No. 2, 269–292.

Burghignoli, A., Pane, V., Cavalera, L., Sagaseta, C., Cuellar, V. and Pastor, M. (1991). Modelling stress-strain-time behaviour of natural soils. Experimental determination of stress-strain-time characteristics in laboratory and *in situ* tests. *Proc. 10th European Conf. on SMFE*, Florence, Italy, Vol. 3, 959–999, Balkema, Rotterdam.

Burland, J.B. (1989). 'Small is beautiful': The stiffness of soils at small strains. Ninth Laurits Bjerrum Memorial Lecture, Canadian Geotechnical Journal, 26(4): 499–516.

Burland, J.B. and Burbidge, M.C. (1985) Settlement of foundations on sand and gravel. *Proc. Institution of Civil Engineers. Proceedings. P. 1. Design and Construction*. Vol 78, Dec., 1325-1381.

Butcher, A.P., and Powell, J.J.M. (1995). Practical considerations for field geophysical techniques used to assess ground stiffness. *Proc. Int. Conf. on Advances in Site Investigation Practice*, London, UK, 701–714, Thomas Telford, London.

Carter, J.P., and Balaam, N.P. (1990). AFENA Users' Manual (V. 4.0). Centre for Geotechnical Research, The University of Sydney.

CSIRO (1970). *Foundations on Interbedded Sands. Proceedings of a Symposium held in Perth, 1970*. CSIRO, Division of Applied Geomechanics and the WA Group of the AGS.

CSIRO (1975). *Proceedings of Symposium on Raft Foundations, Perth, 1975.* CSIRO, Division of Applied Geomechanics, and the WA Group of the AGS.

Decourt, L., Muromachi, T., Nixon, I.K., Schmertmann, J.H., Thorburn, S., and Zolkov, E. (1988). Standard penetration test (SPT): International reference test procedure. *Penetration Testing 1988: Proceedings of the 1st International Symposium on Penetrating Testing (ISPOT-1)*, Orland, Florida, Vol. 1, 3–26, Balkema, Rotterdam.

D'Appolonia, D.J, D'Appolonia, E. and Brisette, R.F. (1970). Discussion: Settlement of spread footings on sand. *Jour. SMFE Div., ASCE,* Vol. 96, No. SM2, 754–761.

Fahey, M. (1998). Deformation and *in situ* stress measurement. Invited Theme Lecture, *Geotechnical Site Characterisation: Proc. 1st International Conference on Site Characterisation (ISC '98)*, Atlanta, Georgia, Vol. 1, 49–68, Balkema, Rotterdam.

Fahey, M. (1999). Determining the parameters of a non-linear elastic model for prediction of ground deformation. *Australian Geomechanics*, Vol. 34, No. 1, March, 39–59.

Fahey, M. (2001). Soil stiffness values for foundation settlement analysis. Proc. 2nd Int. Conf. on Pre-failure Deformation Characteristics of Geomaterials, Torino, Italy, Vol. 2, 1325–1332, Balkema, Lisse.

Fahey, M. and Carter, J. P. (1993). A finite element study of the pressuremeter test in sand using a non-linear elastic plastic model. *Can. Geotech. J.*, Vol. 30, 348–362.

Fahey, M. and Soliman, A.A. (1994). Measuring the small-strain behaviour of sand *in situ*. *International Symposium on Pre-Failure Deformation Characteristics of Geomaterials (IS–Hokkaido)*, Sapporo, Japan, Vol. 1, 217–222, Balkema, Rotterdam.

Fahey, M., Robertson, P.K. and Soliman, A.A. (1994). Towards a rational method of predicting settlements of spread footings on sand. *Proc. Specialty Conf. on Vertical and Horizontal Deformations of Foundations and Embankments (Settlement '94*), Texas A&M University, USA, Geotech. Special Publication No. 40, ASCE, , 598–611.

Fraser, R.A. (1975). Review of investigation on raft foundations for major buildings on deep soils, Perth Area. *Proceedings of Symposium on Raft Foundations, Perth, 1975*. CSIRO, Division of Applied Geomechanics, and the WA Group of the AGS.

Hayes, J.A. (1990). The Marchetti dilatometer and compressibility. Proc. Seminar on *in situ* testing and monitoring, Canadian Geotechnical Society, September 1990, Ontario, 21pp.

Hight, D.W. (1998). Soil characterisation: the importance of structure and anisotropy. 38th Rankine Lecture, March, London (unpublished).

Ishihara, K. (1996). *Soil Behaviour in Earthquake Engineering*. Clarendon Press, Oxford.

Jamiolkowski, M., Ladd, C.C., Germaine, J.T. and Lancellotta, R. (1985). New developments in field and laboratory testing of soils. *Proc. 11th Int. Conf. SMFE*, San Francisco, USA, Vol. 1, 57–153.

Jardine, R.J. (1992). Nonlinear stiffness parameters from undrained pressuremeter tests. Canadian Geotechnical Journal, Vol. 29, Number 3, 436-447.

Jewell, R.J. and Fahey, M. (1984). Measuring properties of rock with a high-pressure pressuremeter. *Proceedings of the 4th Australia–New Zealand Conference on Geomechanics,* Perth, Western Australia, Vol. 2, 535–539, The Institution of Engineers, Australia.

Lehane, B. and Fahey, M. (2002). A simplified non-linear settlement prediction model for foundations on sand. *Canadian Geotechnical Journal*, Vol. 39, No. 2, 293–303.

Mandolini A. (2001). Small-strain soil stiffness and settlement prediction for piled foundations. Proc. 2nd Int. Conf. on Pre-failure Deformation Characteristics of Geomaterials, Torino, Italy (1999), Vol. 2, 1397–1404, Balkema, Lisse.

Marchetti, S. (1980). *In situ* tests by flat dilatometer. Journal of the Geotechnical Engineering Division, ASCE, Vol. 106, GT3, 299–321.

McInnes, D.B. and Waterton, C.A.O. (1980) Static Cone Penetrometers for Site Investigations, Technical Presentation to Australian Geomechanics Society, Perth, November 1980.

Parry, R.H.G. (1971). A direct method of estimating settlements in sands from SPT values. *Proc. Symposium on Interaction of Structures and Foundations*, Midlands Soil Mechanics and Foundation Engineering, Society, Birmingham, 29–37.

Robertson, P.K. (1997). Personal communication.

Salgado, F.M. and Byrne, P.M. (1990). Finite element analysis of pressuremeter chamber tests in sand.. *Pressuremeters: Proc. 3rd Int. Symp. on Pressuremeters*, Oxford, UK, 209–219, Thomas Telford, London.

Seddon, G. (1972). *Sense of Place, a Response to an Environment: The Swan Coastal Plain, Western Australia.* The University of Western Australia Press, Perth, Western Australia.

Schmertmann, J.H. (1970). Static cone to compute static settlement over sand. *J. Soil Mech. Found. Engng Div.*, ASCE, Vol. 96, No. 3, 1011–1043.

Schmertmann, J.H., Hartman, J.P. and Brown, P.R. (1978). Improved strain influence factor diagrams. *J. Geotech. Engng Div.*, ASCE, Vol. 104, No. 8, 1131–1135.

Schmertmann, J.H. (1986). Dilatometer to compute foundation settlement. *Proc. ASCE Spec. Conf. on the Use of In Situ Tests in Geotechnical Engineering*, ASCE Geotechnical Special Publication, No. 6, 303-321.

Schmertmann, J.H. (1991). The mechanical ageing of soils. *J. Geotech. Engng*, ASCE, Vol. 117, No. 9, 1288–1330.

Schultze, E., and Sherif, G. (1973). Prediction of settlements from evaluated settlement observations for sand. *Proceedings of the 8th International Conference on Soil Mechanics and Foundation Engineering*, Moscow, Vol. 1.3, 225–230.

Sherif, G. (1973). Setzungsmessungen an Industrie – und Hochbauten und ihre Auswertung. Grundb. und Bodenmech, Diss. T.H. Aachen, No. 57.

Smith, D.M.A. (1984) The Application of the Electric Friction-Cone Penetrometer to Foundation Design of South Mill Centre, Proc 4th Australia New Zealand Conference on Geomechanics, Perth.

Stokoe II, K.H., Rix, G.J. and Nazarian. S. (1989). *In situ* seismic testing with surface waves. *Proc. 12th Int. Conf. on SMFE*, Rio de Janeiro, Brazil, Vol. 1, 331–334, Balkema, Rotterdam.

Tatsuoka, F. and Shibuya, S. (1991). Deformation characteristics of soils and rocks from field and laboratory tests. Keynote Lecture, *Proc. 9th Asian Regional Conf. on SMFE*, Bangkok, Vol. 2, 101–170.

Tatsuoka, F., Jardine, R.J., Lo Presti, D., Di Benedetto, H. and Kodata, T. (1997). Characterising the pre-failure deformation properties of geomaterials. Theme Lecture, *Proc. 14th Int. Conf. SMFE*, Hamburg, Germany, Vol. 4, 2129–2164, Balkema, Rotterdam.

Tatsuoka, F. and Kohata, Y. (1994). Stiffness of hard soils and soft rocks in engineering applications. *Proc. 1st Int. Conf. Pre-Failure Deformation Characteristics of Geomaterials*, Sapporo, Japan, Vol. 2, 947-1063, Balkema, Rotterdam.

TC16 (2001). The Flat Dilatometer Test (DMT) in Soil Investigations. Report of the ISSMGE Technica1 Committee 16 on Ground Property Characterisation from *In situ* Testing, published in Proceedings of the International Conference On *In situ* Measurement of Soil Properties, Bali, Indonesia.

Wardle, L.J. and Fraser, R.A. (1975). FOCALS – Foundations On Cross Anisotropic Layered Systems. Geomechanics Computer Program No. 4/1, CSIRO Division of Applied Geomechanics, Melbourne.