5 Experimental Procedure

5.1 Introduction

This chapter describes the model footing test programme and the procedures employed. One main-test was performed on each soil sample. The term main-test refers to a group of events on a single sample. A typical main-test included sub-tests such as partially-drained tests, unload-reload tests, and consolidation tests. The miniature cone penetrometer tests are also described.

The partially-drained tests could be vertical load tests (VP), horizontal load tests (HSS), or moment load tests (MSS). The horizontal and moment load tests were sometimes referred to as combined load tests or sideswipe tests. The partially-drained tests were conducted at different displacement rates to investigate the effects of drainage. The vertical partially-drained tests investigated the effects of drainage on the load:deformation response of the footing. The combined load partially-drained tests investigated the effects of drainage on the combined load yield surface. In the final main-test of this investigation, the partially-drained tests were performed with alternating rates of slow and fast displacement. These multi-rate partially-drained tests were labelled VPM, HSSM, and MSSM tests (vertical, horizontal, and moment load respectively).

The unload-reload tests were performed to investigate the drained elastic response of the footing. Vertical unload-reload tests (ULRL), horizontal load-unload-reload cycles (HCYC), and moment load-unload-reload cycles (MCYC) were performed. The consolidation tests (CONSL) were vertical load tests, and were performed to investigate the rate of pore pressure dissipation.

Generally, two vertical load partially-drained tests and one horizontal load or moment load partially-drained test were performed as part of each main-test. Twenty samples were prepared and twenty main-tests were performed. One of the main-tests was performed on a dry sand sample. The rest of the tests were all on oil-saturated samples. The following number of successful sub-tests were performed: 29 VP tests; two VPM tests; eleven HSS tests; two HSSM tests; eleven MSS tests; two MSSM tests; six CONSL tests; five ULRL tests; two HCYC tests; and two MCYC tests. A chronological list of all the tests is given in Table 5.1.

The main-tests are named using four characters. The first character in all the tests is the letter

"T". The second character is a digit identifying the sand specimen. The last two digits are used to identify the number of the main-test. Three different specimens were prepared. The one dry specimen is denoted specimen "2". Only one main-test was performed on this specimen. The two saturated specimens are denoted specimen "1" and specimen "3". The majority of the main-tests were performed on these saturated specimens. Specimens 1 and 3 were reformed between tests to give 13 and 6 samples respectively. The main-test numbers range from 01 to 20. For example, test T204 is the fourth main-test and was performed on specimen 2. The main-test name and the sub-test character (usually a single digit), separated by an underscore character, identify the sub-test events that were performed within the main-tests. The events are numbered in chronological order. For example, the third sub-test in main-test T105 is labelled T105_3.

It was necessary to quit and restart the control program a number of times during some of the main-tests. On restarting the control program, a letter was attached to the end of the main-test label. For example, the background logging data files for test T313 are labelled T313, T313A, T313B, T313C, T313D, and the sub-test data files are labelled T313A_1, T313C_1, T313D_1. The first sub-test was performed during part A of the main test, the second during part C, and the third during part D. The control program of test T313 was restarted four times. All of the main-test and sub-test data files are listed in Table 5.1.

Different types of data logging were performed. Data was logged continuously by the host PC at relatively slow rates and data was logged by the ADU during the fast sub-tests. Data logging directly onto the host PC was termed "background logging". Two different files were created of this data. Files with extension .OUR contained the raw data (in bits) and files with extension .OUT contained the processed data (in engineering units). The data logged by the ADU was transferred to host PC and stored in two types of files. Files with extension .RAW contained the raw data and files with extension .DAT contained the processed data.

5.2 Test initialisation

A number of procedures were followed before commencing a test. Two pore pressure transducers were attached to the tank. A pore pressure transducer was connected to the footing and the footing was then attached to the base plate (connected to rig via the load cell). The footing was lowered onto the sand and the LVDT frame was attached to the footing. Input files for the sub-tests, and the definition file that was required to control the tests, were also produced before the QuickBASIC control program on the host PC was started.

The pore pressure transducers attached to the tank (pp_2 and pp_3) were de-aired by a pressure of approximately -60 kPa that was applied to a port located on the top of the adapters. The adapters were used to connect these transducers to the wall of specimen containers. The test rig was bolted onto the sample container and the oil level was set at approximately 50 mm above the surface of the sand. A rubber seal was placed on the pore pressure port on the underside of the footing and the port was filled with oil. The pore pressure transducer was then screwed into the footing, ensuring that air was not present in the port. After connecting the footing to the base plate, the footing was lowered into the oil and rotated to remove air from the underside of the footing. The rubber seal was then removed and the footing was lowered to the sand surface whilst the oil level was simultaneously lowered, by siphoning, to approximately 15 mm. The footing was centred and levelled. The footing was lowered at 0.001 mm/s to a vertical contact load of 10 N. This load was then held for about three minutes.

The LVDT frame was attached to the flange of the sample container and the small LVDTs were attached to the footing. The frame was levelled and adjusted to achieve the specified length between the pin joints of each of the small LVDTs. This adjustment set the lengths of the LVDTs to within ±0.01 mm of the desired values. The actual lengths were then entered in the definition file. The small LVDT measurements were referenced to their initialised values, but these in turn could be referenced to the sample container by the long LVDT measurements. The sand and oil levels were also recorded in the definition file.

The input files that were used to control some of the sub-tests were created before test initialisation. These files could be altered after test initialisation by exiting the control program or by using the "DOS Shell" function.

Once the procedures listed above were performed, the control program was started and the main-test was initialised. The three pore pressure transducer channels and the three load cell channels were set at the middle of their ranges on initialisation (the initial output was set to zero bits). At this stage there was almost no load on the footing as the small vertical contact load had inevitably been reduced due to infinitesimal movements of the footing. Thus subsequent changes in the load recorded by the load cell were actual footing loads. The output from the pore pressure transducers in the tank were processed using the oil level measurement (stored in the definition file) to give the ambient pore pressure as an initialised value. The pore pressure transducer in the footing was initialised whilst it was approximately 10 mm below the oil surface. This position was taken to have an ambient pore pressure of 0 kPa. This discrepancy was insignificant as it was of the same order of magnitude as the noise level. The output from the six LVDT channels were not offset to give an output of zero bits on initialisation. These transducers normally give an output of zero volts at the middle of their stroke, and the actual position of the armature of the small LVDTs in relation to their bodies was required.

5.3 Main-tests

The test procedure from the initialisation to the end of two typical main-tests, excluding the details of the any sub-tests, is described in this section. The main-test data was logged directly onto the host PC at rates of between one scan per second to one scan every 20 seconds. This type of data logging was termed background logging. Since the PC could not log faster than one scan per second, the sub-tests that required faster logging (sometimes up to 100 scans per second) were initially stored on the ADU. The data on the ADU was then downloaded to the host PC once the sub-tests were completed.

The main-tests were generally conducted at the drained displacement rates, which were $\dot{w}$ = 0.001 mm/s, $\dot{u}$ = 0.001 mm/s, and $\dot{\theta}$ = 0.001 deg/s. The main-test data (or background logging data) was not generally utilised as it was the sub-tests that were important. There were exceptions, however, since it was not necessary to store some of the slower sub-tests (for example the unload-reload tests and a few of the very slow combined load partially-drained tests) on the ADU. These sub-tests, which required scan rates of less than one scan per second, were stored directly on the host PC as part of the background logging data. The drained virgin-penetration stiffness was also deduced from the background logging data. Tests T101 and T114 will be described in the following sections to highlight the typical features of the main-tests.

5.3.1 Test T101

The procedure for T101 was similar to T204, T108 and T309. These tests generally contained two events, an unload-reload test from 1000 N and a sideswipe test (combined load partially-drained test, HSS or MSS) from 1500 N. The sideswipe test of T101, however, was commenced from about 1600 N. Test T108 also only contained one event, an unload-reload test. These tests were performed to derive unload-reload data that was not affected by the possible disturbance caused by previous events.

Time history plots of T101 (using the data from the processed background logging file T101.OUT) are shown in Figure 5.1. The unit of time in Figure 5.1 is seconds. The test lasted about eight hours. The plots are grouped in sets of three. The first set ([a], [b] and [c]) is of the three load components. The moment load in [c] is shown as an equivalent force (the M/2R representation is used). The second set of plots on the first page of Figure 5.1 ([d], [e] and [f]) show the three displacements measured by the long LVDTs. The rotation of the footing in [f] is shown as an equivalent linear displacement (the $2R\theta$ representation is used). The first set of plots on the second page of Figure 5.1 ([g], [h] and [i]) show the displacements calculated from the LVDT frame. The final set of plots ([j], [k] and [l]) is of the pore pressure measurements. The pore pressure measured at the centre of the footing is pp_1, and the measurements taken at the wall of the specimen container are pp_2 and pp_3.

The initialisation procedure was slightly different in T101 than that mentioned in Section 5.2. The footing was embedded to 10 N and the LVDT frame was zeroed after the control program was started. Point A in Figure 5.1[d] indicates the vertical displacement (long LVDT) associated with embedding the footing to 10 N and point B in Figure 5.1[g] indicates the zeroing of the LVDT frame. At point C in plots [d] and [g], the VLDN command (VeLocity DowN, penetration at a constant velocity to a set load limit) was used to set the footing load to 1000 N at 0.005 mm/s. This was intended to be drained penetration but point D in plot [j] indicates that pore pressure generation was associated with this displacement rate. At point E in [d] and [g], the penetration rate was reduced to 0.001 mm/s.

Although the footing was only displaced vertically, the moment load on the footing increased as indicated by point F in plot [c]. Changes in H and M with changing w were detected in most tests and were probably due to a slightly uneven sand surface. In some of the later tests, the build up in H or M (with increasing w) was reduced by rotating or sliding the footing slightly.

The pore pressure measured beneath the footing exhibited a peculiar cyclic response in test T101 only. The five sets of spikes indicated by points G_1 to G_5 in Figure 5.1[j] were not due to the behaviour of the relevant transducer and associated instrumentation. This was verified by checking the response of the load cell and displacements at the footing. The response of these transducers during the combined load horizontal sideswipe test at G_5 all mimicked that of pp_1. Also, the fact that the frequency of the spikes is fairly constant within each group (as indicated by G_1 to G_5 in Figure 5.1[j]), but differ between groups, indicates that they were probably not due to the instrumentation. Slippage in the vertical drive system did occur at the maximum vertical load but slippage was not detected at lower loads. The changing frequencies between the groups of spikes suggest that they were not caused by slippage in the drive system. The rate of rotation of the vertical stepper motor, the only moving component in the rig, during the periods of groups G_1 to G_4 was constant. It was possible that the foundation soil was shearing in a cyclic fashion because of partially-drained effects, but slippage in the drive system cannot be ruled out as a possible cause as well.

The penetration of the footing was stopped at point H in Figure 5.1[a] and [g] as the small LVDTs were approaching their limit. The LVDT frame was reset and zeroed at point I. This point is shown in plots [g], [h], and [i]. Resumption of penetration is identified by point J in plots [a] and [g]. The footing reached 1000 N at point K and was held at this load for about 8 minutes. An unload-reload loop (ULRL sub-test T101_U) was started at point L. The VLUP command was used to unload the footing to 200 N at 0.001 mm/s. The footing was then held at 200 N before the VLDN command was used (at point M in [a]) to reload the footing to 2000 N at 0.001 mm/s. The unload-reload data file (T101_U.DAT) was created from the background-logging data.

It was not possible to achieve a vertical load of 2000 N as the couple between the lead screw and the vertical stepper motor began slipping at 1700 N (indicated by point N in Figure 5.1[a]). The vertical stepper motor was stopped at point O and a horizontal sideswipe test was commenced (HSS sub-test T101_1). This is obvious from plots [b], [e], and [h]. The test was a drained sideswipe test ($\dot{u}$ = 0.001 mm/s) that was recorded directly onto the host PC as part of the background logging data. A separate file for this test was created from the background logging file (T101.OUT) and was labelled T101_1.DAT. The sideswipe test was stopped at point P and the footing was displaced back to zero horizontal displacement at point Q. The footing was then retracted at point R. The negative pore pressures induced by the retracting footing are indicated by point R in plots [j] to [l].

5.3.2 Test T114

The procedure for T114 was similar to that of the majority of the other tests. Main-test T114 contained three events, two vertical load partially-drained tests from 500 N and 1000 N (VP sub-tests T114B_1 and T114D_1), and a combined load partially-drained test from 1500 N (MSS sub-test T114E_1). Time history plots of test T114 (using data from the processed background-logging files of T114.OUT to T114E.OUT) are shown in Figure 5.2. The layout of the plots is the same as described in the previous section. The details of T114 are listed in Table 5.1.

At point A (shown in Figure 5.2[a], [d], and [g]) the VLDN command was used to set the footing to a load of 500 N at 0.001 mm/s. As in test T101, the moment load on the footing increased with vertical penetration (indicated by point B in plot [c]).

A fault occurred with the control program at point C, approximately 3200 seconds into the test, and the program was restarted as T114A after about three minutes. When the control program stopped, all the stepper motors could be stopped using the limit switches on the test rig or the stepper motor controller could be switched off. The ADU was not reinitialised on restarting. This ensured that the previous transducer settings were maintained. After restarting, the drained loading of the footing to 500 N was resumed at point D in plot [a]. The footing reached 500 N at point E and stopped automatically as an upper soft limit of 500 N was set on the vertical load. Two types of limits were set in the control program to monitor the transducer measurements (see Section 3.4). A load of 500 N was then held for a couple of minutes at point F before the small vertical stepper motor was changed to the large one. The large stepper motor was used for the very fast vertical moves ($\dot{w}$ = 5 mm/s).

The large vertical stepper motor was used in the two vertical load partially-drained tests (VP) of T114 as a penetration rate of $\dot{w}$ = 5 mm/s was required. The stepper motor controller was switched off whilst the small motor was replaced by the large motor. During this time (indicated by point G in [a]) the control program was still running and background logging was maintained. Before communication with the stepper motor controller could be resumed, the control program had to be exited and restarted. The test was restarted as T114B at point H and the load of 500 N was regained and held before commencing the first VP test at point I. This sub-test was run from input file T114_A.INP and its data was stored on the ADU.

The ADU was run in an autonomous state during the fast sub-tests. Communication between the ADU and the host PC was restricted when the ADU was running in this mode. Because of the reduced communication between the ADU and the PC, the PC did not log some of the data of the fast sub-tests at real-time. Consequently, the background logging of the PC did not capture all of the data of the fast sub-tests. Thus the background logging data of file T114B.OUT shown in Figure 5.2, during the sub-test at point I, does not show the full response of the footing. Although the vertical load recorded by background logging does not change by much at point I in plot [a], the sub-test is clearly identifiable by the sudden changes in the vertical displacement and footing pore pressure (see point I in plots [d], [g], and [j]).

Immediately after the sub-test was finished, the test data that was stored on the ADU was automatically transferred to the host PC and stored in files T114B_1.RAW and T114B_1.DAT. Whilst the data from the sub-test was being transferred, background-logging could not be maintained. The data retrieval generally did not last for more than about one minute.

The vertical stepper motors were switched once again at point J after the VP test at 500 N. This was because the drained penetration rate could not be achieved with the large motor. The test was restarted as T114C at point L, and drained vertical penetration was resumed at point M. Once the footing load reached 1000 N at point N in Figure 5.2[a], the above procedure was repeated. That is, the load was held, the stepper motors were switched, the control program was restarted, input file T114_B.INP was run (point O in [a], [d], [g], and [j]), the sub-test was transferred to the host PC (files T114D_1.RAW and T114D_1.DAT), and the stepper motors were switched back. And finally, the control program was restarted as T114E at point P.

After the VP test at 1000 N, the footing was penetrated to 1500 N at point Q and held before commencing the combined load partially-drained test at point R. In this moment sideswipe test (MSS), the footing was rotated at $\dot{\theta} = 0.2$ deg/s ($d(2R\theta)/dt = 0.524$ mm/s). This sub-test was characterised by a sudden reduction in the vertical load in [a], an increase in the moment load in [c], an increase in the rotation in [f] and [i], an increase in the vertical penetration in [g], and a reduction in the pore pressure in [j]. The sub-test was run from input file T114_C.INP and stored in files T114E_1.RAW and T114E_1.DAT. After the sideswipe, the footing was slowly retracted and the loads were reduced between points S and T. The footing was moved to its initialisation position at point T.

5.4 Partially-drained tests

The vertical load partially-drained tests (VP) were conducted at 500 N and 1000 N. The fastest of these tests (at $\dot{w} = 5$ mm/s) required the large vertical stepper motor. When the large motor was required, the motors were changed just before the VP tests. After the VP tests, the large motor was replaced by the small one.

Most of the combined load partially-drained tests were conducted from 1500 N. The combined load partially-drained tests are also referred to as horizontal or moment sideswipe tests (HSS or MSS tests). In some of the later main-tests, additional sideswipes tests were conducted at higher vertical loads than 1500 N.

The initial vertical load and the displacement rate of the partially-drained tests are listed in Table 5.1. Details of the VP, HSS, and MSS tests (including the multi-rate tests) are also listed in Table 6.6, 7.1, and 7.2 respectively. The procedures involved in the vertical and combined load partially-drained tests will be described in the following sections.

It should be noted that the footing displacements referred to were the displacements measured by the LVDT frame and not those measured by the long LVDTS.

5.4.1 Vertical load

The vertical load partially-drained tests (VP) were run from input files that were created before the main-tests were commenced. Each of these input files contained six commands, together with the parameters associated with these commands. The commands in the input file were read by the control program, which then performed the routines associated with the commands.

A typical input file is shown in Figure 5.3. This input file was used to control the VP test of T114 that was performed from 500 N (T114B_1, see Table 5.1). The first LIMITS command in Figure 5.3 was used to set the load soft limits that were active during the move event. Lower and upper limits were set. These were the loads at which the control program would automatically stop all the stepper motors. The vertical load soft limit was used to ensure that a VP test did not overshoot the vertical load from which the next sub-test was to be conducted. The effectiveness of this check was dependent on the speed of the load feedback mechanism in the control program and it was not reliable for fast tests.

The MTEST (move test) command in Figure 5.3 was used to allot and activate the test on the ADU, whilst simultaneously commanding the stepper motors to move the footing a finite distance at a specified velocity and acceleration. All three stepper motors could be moved at the same time with this command. For the VP tests, however, only parameters for the vertical stepper motor were included. The parameters in the first line of the MTEST command were used to set the test logging parameters required by the ADU. These were the test time in seconds and the number of scans per second. The time delay was used to set the delay between the start of ADU testing and the start of the footing displacements. The last three lines of the MTEST command contained the displacement parameters for the three axes of the stepper motor controller (w, u, and θ). The base speed was an extra parameter that was required by the vertical axis because two different stepper motors were sometimes used on this axis. A displacement of 1.5 mm was used in tests T101 to T107. This was reduced to 0.5 mm in the later tests because it was found that the relevant behaviour could be captured by smaller displacements. The displacement parameter in the MTEST command was also kept low so that a test would stop before reaching too high a vertical load.

A different set of soft limits was set immediately after the MTEST was complete in the input file in Figure 5.3. This was to accommodate any loads that overshot the initial limits. The ADUSTORE command was used to retrieve the test data from the ADU to the host PC. The MOVE command could be used to move the footing a finite distance. For example, back to its initial position before the VP test. But this command was only used to reset the base speed of the vertical stepper motor. At the END command, the input file was closed and the user regained access to the control program.

The scan rate chosen for each test was dependent on the displacement rate. Each test was comprised of three sections. An initial "delay" section before the move commenced (generally between $t = 0$ s and $t = 1$ s). A middle section during which the footing was displaced (the duration of this part of the test was dependent on the displacement rate). And a final part that captured the dissipation of pore pressures once the move had stopped.

The load, displacement, and pore pressure load time history of a slow VP test displaced at 0.01 mm/s is shown in Figure 5.4. The time history is that of test T317_1, which was controlled by input file T317_A (see Table 5.1). The logging and move parameters of test T317_1 are shown in Figure 5.4[b]. The ADU was set to log for 56 s and five scans were taken every second. The delay between the start of ADU logging and the start of stepper motor activity was set at one second. The three distinct sections of the VP test are indicated in plot [a].

The scan rate was adjusted to derive an adequate number of scans during the move section. In the slowest tests at $\dot{w} = 0.01$ mm/s, the number of scans during the move section ($\cong 0.5$ mm) was limited to about 250. This limit was imposed to minimise the size of the data files. The total number of scans in a VP test was limited to about 500.

The move parameters sent to the stepper motor controller are listed in the second line below the MTEST command in Figure 5.4[b]. The first number was the magnitude of the move (MR), the second was the velocity (SV), and the third was the acceleration (SA). A base speed (SB) of 0.01 mm/s was also set. These move parameters are explained in Figure 3.6. The base speed was the velocity at which the footing started accelerating up to the required velocity. Since the base speed and the velocity were the same, the acceleration time was equal to zero.

The magnitude of the move in the MTEST command in Figure 5.4[b] was set at 0.5 mm but the measured displacement increment was less. It was equal to about 0.37 mm. The displacements implied by the stepper motors were not always equal to the displacements at the footing. This difference was due to the flexibility in the rig (the rig was not infinitely stiff). The ratios of the measured displacements to the intended displacements decreased as the stiffness of the sand samples increased. The rate of increase in the load and the pore pressure in Figure 5.4 was higher at the start of the move (between $t = 2$ s and $t = 6$ s). The rates then decreased to practically constant values. This form of response was evident in most of the VP tests.

In the fastest tests at $\dot{w}$ = 5 mm/s, only 10 scans could be logged during the 0.5 mm move sections. The maximum scan rate of the ADU was 100 scans per second. The load, displacement, and pore pressure load time history of a fast VP test displaced at 5 mm/s is shown in Figure 5.5. The time history is that of test T114B_1, which was controlled by input file T114_A (see Table 5.1). The logging and move parameters of test T114B_1 are shown in Figure 5.5[b]. The relative size of the move section, and the quantity of data within, is small in relation to the rest of the test. Only about five percent of the data from the whole test is particularly useful. This data is shown in the inserts. Although the data from the very fast moves was limited, it was sufficient to define the load:deformation response. The rate of increase in the load, shown in the enlargement in Figure 5.4[a], was higher at the start of the move (between t = 1.18 s and t = 1.20 s). The rate then decreased to a constant value. As mentioned previously, this form of response was evident in most of the VP tests.

The number of scans in the first 0.05 mm of the move sections was important as it was noticed that the initial footing response was different from the subsequent response at larger displacements. In the fastest tests, only about two scans could be made during this initial part of the move sections. This is illustrated in Figure 5.5[a] by the data points in the enlargement of the move section. The ratio of acceleration to velocity was generally set at a value of 20 or more. In the fastest tests, where the duration of the move section was only about 0.1 s, a ratio of 50 was used to reduce the acceleration time. The base speed was also set at or close to the intended velocity.

<u>Multi-rate tests</u>

Vertical partially-drained tests with multiple displacement rates (VPM) were performed in main-test T120. These tests were conducted from 500 N and 1000 N and each was comprised of three sequences of events. Each sequence was comprised of two moves, a slow move and a fast move. In effect, six moves were run in immediate succession, a slow move at $\dot{w}$ = 0.01 mm/s alternating with a fast move at $\dot{w}$ = 1 mm/s. This form of displacement path was used to check the response of the footing to successive changes of displacement rate, and also to check if the response was similar to that in the normal vertical tests (VP tests).

The six daisy-chained moves of the VPM tests were run from a single input file. The input file was similar to that used in the VP tests (see Figure 5.3), but the MTEST command was replaced by three SEQTEST1 commands and the MOVE command was not included. The input file used to control the VPM tests is shown in Figure 5.6[a] (the same input file was used to control the VPM test from 500 N and the one from 1000 N). The SEQTEST1 command instructed the ADU to start logging and then ran subroutine SEQMOVE1 in the control program. The move parameters of the sequence event were not included in the SEQTEST1 command in the input file, these parameters were programmed directly into the subroutine SEQMOVE1 in the control program. This subroutine is listed in Figure 5.6[b]. Subroutine SEQMOVE1 sent a sequence, comprising two move events, to the stepper motor controller. The details of the moves are explained in Figure 5.6[b] (the move commands used to communicate with the stepper motor controller are illustrated in Figure 3.6).

Firstly, the details of the sequence are defined and sent to the stepper motor controller (lines one to eleven of the code in Figure 5.6[b]). The sequence is then executed at line twelve of the code. The total displacement of the three sequences was 0.48 mm and the total logging time was 19.3 s. Three output files were created of the events associated with input file T120_A.INP (one for each SEQTEST1 command). For the VPM test from 500 N, these are named T120A_1, T120A_2, and T120A_3. The test from 1000 N produced output files T120C_1, T120C_2, and T120C_3. A chronological list of the tests is given in Table 5.1.

The load, displacement, and pore pressure load time history of the test from 1000 N will be used to describe the typical characteristics of the VPM tests. The time history is shown in Figure 5.7. The VPM test is composed of three output files and these are plotted together in Figure 5.7. The time delay between the tests is estimated from the rate of pore pressure dissipation at the end of a test and the pore pressure at the start of a subsequent test (the pore pressure load U is shown in Figure 5.7[b]). There is a delay of about one second between the start of ADU logging and the start of the slow move in the first sequence (stored in T120C_1). The subsequent sequences do not include this time delay since ADU logging and the moves are started simultaneously. Although the displacement increment of the slow moves was set at 0.06 mm (see Figure 5.6[b]), the corresponding increment in the first sequence was only about 0.02 mm and in the last two sequences they were about 0.05 mm. This was due to the flexibility of the rig. The displacement increments of the fast moves were about 0.06 mm. These were set at 0.1 mm.

5.4.2 Horizontal load

Most of the horizontal load partially-drained tests (HSS) were run from input files. The slowest tests at $\dot{u} = 0.001$ mm/s, however, were logged directly onto the host PC as part of the background logging. The input files of the HSS tests were similar to those of the vertical load partially-drained tests (described in the previous section and shown in Figure 5.3). The only significant differences were in the LIMITS and MTEST commands. The MOVE command was also excluded from the input files of the HSS tests. In the LIMITS command, appropriate horizontal load soft limits were entered. These were generally H = -500 N to H = 500 N. The horizontal displacement parameters (u, $\dot{u}$ and $\ddot{u}$) were entered in the third line of the MTEST command.

The time history of a slow horizontal sideswipe test (HSS test T309_1) is shown Figure 5.8. The horizontal velocity in this test was 0.001 mm/s and the test lasted about 70 minutes. Test T309_1 was not run from an input file. It was manually controlled from the host PC. This was because the required logging rate was low. The PC was set to log at one scan every ten seconds during this test. The data file for this test (T309_1.DAT) was created from the background logging file (T309.OUT).

The start of the HSS is indicated by point A in Figure 5.8. The horizontal load and moment are not equal to zero at the start of the test. The initial horizontal load is about 20 N and the moment is about 40 N. These loads developed during the preceding vertical loading stage. The footing was loaded to V = 1500 N before the HSS test was started (as shown in plot [a]). At the start of horizontal displacement at point A, the horizontal load (in [b]) increases immediately even though the increase in the horizontal displacement (in [e]) is very small. The initial slope of the t:du curve in plot [e] is not linear because of the interaction between the horizontal flexibility of the rig and the changing stiffness of the footing response. After about 2000 seconds, the horizontal velocity is approximately constant. The vertical load shown in Figure 5.8[a] reduces throughout the test, but the change in the moment shown in plot [c] is small. The rotational displacement shown in

[f] increases slightly because of the increase in horizontal load. The penetration of the footing shown in [d] also changes. This was because of the vertical flexibility of the rig. The pore pressure load shown in [d] decreases at a constant rate throughout the test, but increases sharply at point B when the horizontal displacement is stopped.

The time history of a fast horizontal sideswipe test (HSS test T107A_3) is shown Figure 5.9. The horizontal velocity in this test was 0.4 mm/s and the test lasted for about eleven seconds. Test T107A_3 was run from input file T107_D.INP. The MTEST parameters of the input file are listed in Figure 5.9[a]. The ADU was set to log for 13 s and to take 40 scans every second. A delay of one second was also set. The move parameters that were sent to the horizontal axis of the stepper motor controller specified a move of three millimetres. The three sections of the HSS test are outlined in Figure 5.9[b]. These were similar to the sections of the vertical partially-drained tests.

The response of the footing in T107A_3 was broadly similar to that in the slow HSS test previously described (T309_1). The vertical load reduced, and the vertical and rotational displacements increased. A small negative moment shown in [c] was associated the HSS test. The total horizontal displacement was also less than the input value of 3 mm.

Multi-rate tests

Multi-rate horizontal partially-drained tests (HSSM) were performed in main-test T120. Two multi-rate sideswipes were performed, one from 1500 N and the other from 1700 N. The sideswipe from 1500 N was run from a single input file (T120_B.INP) and included five sequences. This input file was similar to that used for the multi-rate vertical tests. It included five SEQTEST2 commands. Each of these called a subroutine named SEQMOVE2 in the control program. This subroutine defined and executed a sequence that included two moves. There was a slow move at 0.001 mm/s and a fast move at 0.1 mm/s. Each sequence, or set of two moves, resulted in one output file. In effect, ten moves were made in immediate succession and generated files T120D_1.DAT to T120D_5.DAT.

Once the multi-rate moves had stopped and the input file had returned control to the interactive menu on the host PC, the footing was displaced horizontally at 0.001 mm/s under manual control. The data from this manually controlled section formed part of the HSSM test from 1500 N, and was extracted from the background logging file (T120D.OUT) and saved as

T120D_5A (see Table 5.1).

The sideswipe from 1700 N was run from input file T120_C.INP and was displaced in the negative horizontal direction (to the left). This file included six MTEST commands. Each MTEST command was used to perform a single move and generated a single output file. Slow and fast MTEST commands were alternated and these generated output files T120D_6.DAT to T120D_11.DAT. This form of testing was used to overcome a drawback of the sequence tests used in the sideswipe from 1500 N. In the sideswipe from 1500 N, the logging rate was fixed for the duration of each sequence. Thus there were only five scans during the fast moves. By running each move as a separate (MTEST) test, appropriate logging rates could be used. It was also possible to increase the magnitude of the move events. The only problem with the MTEST command was that there was a slight time delay between each of the MTESTs.

The sideswipe from 1700 N was continued after the events associated with input file T120_C.INP were complete. The sideswipe was continued (by manual control) at the drained displacement rate for about six minutes before input file T120_C.INP was once again run. The data of this drained section was extracted from the background logging file and stored as T120D_11A. The second execution of T120_C.INP generated output files T120D_12.DAT to T120D_17.DAT (see Table 5.1).

The time history of the HSSM sideswipe from 1700 N is shown in Figure 5.10. The filenames of the MTEST events are shown in Figure 5.10[b]. The first six files were created by the first execution of input file T120_C. The middle section between $t \cong 330$ s and $t \cong 730$ s (T120D_11A) is the manually controlled move at 0.001 mm./s. The last six filenames indicate the MTEST events associated with the second execution of the input file.

The fast events of the HSSM test resulted in a rapid decrease in the horizontal load and displacement (see Figure 5.10[b] and [e]). The changes in the horizontal load and displacement were negative because the footing was displaced in the negative horizontal direction. The pore pressure load increased during the fast events. This is evident from the spikes in the profile of U in plot [d]. The pore pressure was not equal to zero at the start of the test but the initial excess pressure was only 0.5 kPa. The response of the vertical load and displacement, shown in [a] and [d], is similar to that observed in the normal horizontal sideswipe tests (the HSS tests previously discussed). Small changes in the moment and rotational displacement are also evident.

5.4.3 Moment load

The procedures for the moment load partially-drained tests (MSS) were the same as for the HSS tests (described in the previous section). The slowest tests at $\dot{\theta} = 0.001$ deg/s ($2R\dot{\theta} = 0.0026$ mm/s) were logged directly onto the host PC as part of the background logging. In the fast tests run from input files, appropriate moment load soft limits were entered in the LIMITS command. These were generally $M = -30$ Nm to $M = 30$ Nm (or $M/2R = \pm 200$ N).

The time history of a slow moment sideswipe test (MSS test T311_3) is shown in Figure 5.11. The rotational velocity in this test was 0.0026 mm/s and the test lasted about 40 minutes. Test T311_3 was manually controlled from the host PC. The PC was set to log at one scan every five seconds. The data file for this test (T311_3.DAT) was created from the background logging file (T311.OUT).

The start of the MSS is indicated by point A in Figure 5.11. The footing was loaded to $V = 1500$ N before the MSS test was started (as shown in plot [a]). Displacement starts at point A in plot [c], and finishes at point B. The vertical load shown in Figure 5.11[a] reduces throughout the test. The change in the horizontal load shown in plot [b] is significant. In the sideswipe tests on the stiff samples, the passive component of the combined load also changed. The change in the horizontal displacement shown in [e] is negligible and the penetration of the footing shown in [d] increases by about 0.4 mm. This was because of the vertical flexibility of the rig. The pore pressure load shown in [d] decreases to a constant value, but increases at point B when the rotational displacement is stopped.

The time history of a fast moment sideswipe test (MSS test T114E_1) is shown Figure 5.12. The rotational velocity in this test was 0.524 mm/s (0.2 deg/s) and the test lasted for 26 s. Test T114E_1 was run from input file T114_C.INP. The MTEST parameters of the input file are listed in Figure 5.12[a]. The ADU was set to log for 26 s and to take 20 scans every second. A delay of one second was also set. The move parameters that were sent to the rotational axis of the stepper motor controller specified a move of three degrees. The three sections of the MSS test are outlined in Figure 5.12[c].

The response of the footing in T114E_1 was broadly similar to that in the slow MSS test previously described (T311_3). The vertical load reduced and the vertical displacement increased. There was a slight increase in the horizontal displacement as well. The horizontal load shown in [b] reduces by about 40 N. The total rotational displacement shown in [f] is also less than the input value of 7.9 mm (3 °). The pore pressure shown in [d] increases at the start of the move before reducing at an almost constant rate.

Multi-rate tests

Multi-rate moment partially-drained tests (MSSM) were performed in main-test T120. Two multi-rate sideswipes were performed, one from 1900 N and the other from 2100 N. The sideswipe from 1900 N was run from a single input file (T120_D.INP) and included five sequences. This input file was similar to that used for the multi-rate horizontal tests. It included five SEQTEST4 commands. Each of these called a subroutine named SEQMOVE4 in the control program. This subroutine defined and executed a sequence that included two moves. There was a slow move at 0.0026 mm/s and a fast move at 0.26 mm/s (0.001 deg/s and 0.1 deg/s). Each sequence, or set of two moves, resulted in one output file. In effect, ten moves were made in immediate succession and generated files T120D_18.DAT to T120D_22.DAT.

The sideswipe from 1900 N was continued after the events associated with input file T120_D.INP were complete. The sideswipe was continued (by manual control) at the drained displacement rate for about ten minutes before input file T120_D.INP was rerun. The data of this drained section was extracted from the background logging file and stored as T120D_22A. The second execution of T120_D.INP generated output files T120D_23.DAT to T120D_27.DAT (see Table 5.1).

The sideswipe from 2100 N was run from input file T120_E.INP and was displaced in the negative rotational direction (anti-clockwise). This file included eight MTEST commands. Each MTEST command was used to perform a single move and generated a single output file. Slow and fast MTEST commands were alternated and these generated output files T120E_1.DAT to T120E_8.DAT.

The time history of the MSSM sideswipe from 1900 N is shown in Figure 5.13. The filenames of the SEQTEST4 events are shown in Figure 5.13[c]. The first five files were created by the first execution of input file T120_D. The middle section between $t \cong 200$ s and $t \cong 600$ s (T120D_22A) is the manually controlled move at 0.0026 mm./s. The last five filenames indicate the SEQTEST4 events associated with the second execution of the input file.

The fast events of the MSSM test resulted in a rapid increase in the moment load and displacement (see Figure 5.13[c] and [f]). The pore pressure load increased during the first two fast events but decreased during all the other fast events. This is evident from the spikes in the profile of U in plot [d]. The pore pressure was not equal to zero at the start of the test but the initial excess pressure was only 0.6 kPa. The response of the vertical load and displacement, shown in [a] and [d], is similar to that observed in the normal moment sideswipe tests (the MSS tests previously discussed). The horizontal load was not zeroed before the MSSM test. The initial horizontal shown in Figure 5.13[b] is equal to about -40 N and this load increased slightly during the test.

5.5 Unload-reload loops

The unload-reload loops were performed to investigate the elastic response of the footing. Five vertical unload-reload loops, and two load-unload-reload horizontal and moment loops, were performed.

5.5.1 Vertical loops

The first four ULRL tests were unloaded from 1000 N and the last was unloaded from 1900 N. The ULRL tests are listed in Table 5.1. The ULRL tests were controlled manually from the host PC and the test data was extracted from the relevant background logging files. The tests were initiated from the virgin-penetration curve and the footing was unloaded and reloaded at the drained displacement rate (0.001 mm/s). The VLUP command (VeLocity UP, retraction of the footing at a constant rate) was used to unload the footing and the VLDN command was used to reload the footing. A load of 200 N was set as the lower soft limit in all the tests except the last. In T118E_U, the footing was unloaded to 190 N. The unloading of the footing stopped automatically when the lower soft limit was reached. An upper soft limit was used to control the reloading phase. The minimum load was held for a few minutes before the footing was reloaded.

5.5.2 Horizontal and moment loops

The horizontal and moment cycles (HCYC and MCYC tests) were performed in main-test T120. These tests were controlled manually from the host PC and the test data was extracted from the relevant background logging files. The data files are listed in Table 5.1. The footing was unloaded vertically from 1000 N to 500 N before the first set of cycles. And before the second set of cycles, the footing was unloaded vertically from 1500 N to 1000 N, and also horizontally from about 250 N to 0 N. The cycles were performed at the drained displacement rates and the constant velocity commands, in combination with the soft load limits, were used to control the tests. The cycles were initiated from a horizontal and moment load of zero, and the maximum and minimum soft limits were set at about ±50 N (in both types of test).

5.6 Consolidation tests

Six consolidation tests (CONSL) were performed, three in main-test T118 and three in main-test T319. The CONSL tests were run from input files and were initiated at a vertical load of 700 N. Each set of three CONSL tests were run from a single input file. The input file contained an initial LIMITS command and three HTEST commands. An ADUSTORE and an END command were also included. The HTEST command set the logging parameters for the ADU (test length = 120 s, scans per second = 2, and delay = 1) and started ADU logging. It also set the parameters for the HOLD routine in the control program, as well as starting the routine. The HOLD routine was used to load (or unload) the footing, and to hold the target load by using a load feedback mechanism. The first and third HTEST commands loaded the footing from 700 N to 900 N, and the second unloaded the footing from 900 to 700 N. The target loads were held for 120 s and a tolerance of ±3 N was set. If the load moved out of the tolerance range, the footing was moved slightly to regain the target.

The consolidation tests performed in main-test T319 are shown in Figure 5.14. These three tests were run from input file T319_B and were stored as data files T319B_1 to T319B_3. In the first test shown in Figure 5.14[a], the footing is loaded from 700 N to 900 N and the target load is reached after about 20 s. It was hoped that the footing would reach the target load very quickly, as it does in the other two tests. The effectiveness of the HOLD routine was dependent on the stiffness of the footing and the routine was programmed to work best at a particular stiffness. The routine was programmed to be effective at the elastic (or unload-reload) stiffness of the footing. Positive pore pressures are associated with the loading and reloading tests shown in Figure 5.14[b] and negative pressures with the unloading test. Most of the displacement shown in plot [c] occurs during the first test.

After the three HTEST events, the ADUSTORE command in the input file instructed the control program to download the test data from the ADU to the host PC. The END command returned the control program to manual operation.

5.7 Miniature cone penetrometer tests

The procedure for the cone tests was similar to that of the main-tests as the cone was attached to the rig. The footing was removed from the base plate and the bracket for the penetrometer was attached. The LVDT frame was not used in the cone tests. The long LVDT was used to measure the displacement of the cone. A QuickBASIC program similar to that used in the main-tests was used to control the cone tests. The cone tests were logged directly onto the host PC and one scan was usually taken every six seconds. The cone was penetrated between 100 mm and 170 mm into the sample. A penetration rate of 0.1 mm/s was used in most of the tests. This rate was calculated to ensure drained penetration, but the pore pressure was not measured so the drainage conditions could not be verified.

Cone tests were not performed on the first sample. On each of samples two to eleven, two tests were performed after the main-tests (footing tests). Between three and five cone tests were performed on each of samples 12 to 20, and some of these were before the main-tests. Tests were performed before the main-tests to check the progress of densification. The details of the cone tests are listed in Table 5.2. The different tests on a sample were performed in one of five locations. These were either at the centre or at the side of the sample. The tests at the side were either at the back, front, left, or right of the sample. The tests at the sides of the samples were located 75 mm from the tank wall, which was equivalent to 75 mm from the edge of the footing.

6 Results: Vertical Loading

6.1 Introduction

The drained load:deformation response, the consolidation behaviour, and the partially-drained behaviour of the footing is analysed. A partially-drained consolidation model is also developed.

The footing was not loaded to failure but bearing capacity estimates are compared with the drained load:deformation behaviour. The footing was subject to creep and this response is investigated. Elastic parameters are also derived from the unload-reload loops. The partially-drained consolidation model was used to deduce consolidation parameters from the consolidation tests. The consolidation tests were partially-drained. The model simulations were good when the response of the footing was mainly elastic. A framework for modelling the total stress response of the partially-drained tests is outlined.

6.2 Drained loading

The load:penetration behaviour of the footing was assumed to be fully drained at a vertical displacement rate of 0.001 mm/s. During virgin-penetration at this displacement rate, the pore pressure beneath the footing did not increase with increasing load. At very low loads, however, even small increases in the pore pressure did affect the load:deformation behaviour of the footing and this is discussed below. The tests only investigated behaviour at loads significantly below the bearing capacity, but estimates of the failure loads are discussed below. These estimates were based on the virgin-penetration stiffness measurements and the cone-penetrometer results. The creep response and the unload-reload behaviour of the footing are also investigated.

6.2.1 Bedding-in phase and associated pore pressure response

In some of the tests, the initial load:deformation behaviour of the footing, from $V = 0$ N to about $V = 100$ N, indicated that there was a thin layer of loose soil above the densified sample. The low virgin-penetration stiffness encountered at low loads could also have been caused by an uneven sample surface. A few virgin-penetration curves that illustrate the effect of a loose layer of soil on the load:deformation behaviour are shown in Figure 6.1. The response in each test became consistent above a load of about V = 100 N. The footing stiffness measured between V = 400 N and V = 500 N was used to characterise the samples and was labelled the *characterisation stiffness* (as discussed in Section 4.6).

The displacement in T101 was greater than that in T108 even though the load:deformation behaviour at higher loads was similar, as shown in Figure 6.1. When considering the load:deformation curve at higher loads, as in Section 6.2.2 below, one should subtract the displacement associated with the anomalous initial response. One way of doing this is to assume that if a loose layer was not present the initial stiffness would have been similar to the stiffness at a higher load level. This procedure was followed in the comparison of virgin-penetration curves in Section 6.2.2. The stiffness from V = 200 N to V = 400 N was used and the procedure is illustrated in Figure 6.2. Tests T101, T204, T108 and T309, the tests used to describe virgin-penetration behaviour in the following section, are shown in Figure 6.2 with suggested virgin-penetration lines extrapolated down to the displacement axis.

The anomalous initial response could not be explained by the miniature cone penetration results. For example, consider the load:deformation curves of tests T103 and T108 shown in Figure 6.1. The characterisation stiffness of these two tests was roughly similar, 112 N/mm and 99 N/mm respectively, but the load:deformation curves were different. The response of T103 indicated the likelihood of a loose layer of soil. The cone penetration curves shown in Figure 6.3, however, did not suggest that the sample used in T108 was any stiffer than that used in T103. This may have been due to a lack of sensitivity in the cone-penetrometer measurements.

The pore pressure measurements at the centre of the footing supported the idea of a loose layer of soil as the cause of the anomalous initial footing response. The vertical load, pore pressure load, and effective load responses of the tests that are shown in Figure 6.1 are plotted in Figure 6.4. It is evident from Figure 6.4 that the effective loads in the tests with an anomalous initial response, particularly T102, T103 and T105, are almost equal to zero. Test T101 cannot be used in this comparison because the footing was initially penetrated at $\dot{w}$ = 0.005 mm/s before being slowed (at $w \cong 3$ mm) to a rate of $\dot{w}$ = 0.001 mm/s, the rate used in the other tests. The pore pressure loads are calculated using Equation 2.30. A value of C_p = 0.5 was used to calculate the pore pressure loads in Figure 6.4. This value is representative of a parabolic pore pressure distribution, as discussed in Section 2.6. The initial response of tests T101, T102, T103 and T105, shown in Figure 6.4, was probably fully undrained, because of the loose layer of sand, and indicated that a value of C_p = 0.5 was appropriate. This value of C_p was deemed suitable because the effective load was greater than or equal to zero.

6.2.2 Virgin-penetration stiffness

The footing was subjected to a maximum vertical load of about V = 1500 N in most tests. Load:deformation curves of the main tests, those that did not include partially-drained sub-tests, are shown in Figure 6.5. Note that the unload-reload sub-tests, but not the sideswipe sub-tests, are shown in Figure 6.5. None of these tests except T108 exhibited softening behaviour. This indicated that the loads involved were probably well below the bearing capacity loads. In most of the tests, the virgin-penetration stiffness was still increasing at the maximum vertical load encountered. The virgin-penetration stiffness values are listed in Table 6.1. These values were measured over four load ranges in most of the tests, at 100-200 N, 400-500 N, 900-1000 N and 1400-1500 N. The values from Table 6.1 are shown in Figure 6.6. The stiffness measurements are plotted against the characterisation stiffness and best-fit lines (with zero intercepts) are included. The best-fit lines highlighted the general trend of increasing stiffness with rising load. This behaviour was not restricted to the tests on saturated samples because test T204 exhibited similar behaviour, only at loads higher than 500 N.

The characterisation stiffness values were compared to similar quantities from footing tests in the literature in Figure 6.7. The different tests should all follow a similar relationship since the initial stiffness is normalised by the radius. Some of the R_D:($k_p/R^2\gamma'$) data could be approximated by an exponential relationship. The tests of Vesic (1975) and of Gottardi and Houlsby (1995) did not fit the trend. An exponential relationship between the initial stiffness and the relative density can be postulated from bearing capacity theory if it is assumed that the initial stiffness is proportional to the bearing capacity load, and hence to the bearing capacity factor.

6.2.3 Bearing capacity estimates

The discussion of load:deformation behaviour in Section 6.2.2 indicated that the maximum loads that the footing was subjected to were probably below the bearing capacity loads. The bearing capacity loads are estimated in this section. The characteristics of the estimation are as follows:

- The estimates of the relative density from the cone-penetrometer tests, as opposed to the bulk relative density measurements (Section 4.6.2), are used in the calculation of the peak friction angle.
- The R_D:p':ϕ'_{max} relationship outlined in Section 4.5.3 is used to calculate the peak friction angle. Note that the peak friction angle in the oil-saturated specimens was reduced by 1.5° and the effective mean stress along the failure surface is assumed equal to a tenth of the contact stress.
- The bearing capacity factors of Bolton and Lau (1993) for rough footings are used.
- Estimates of the bearing capacity load are incorporated into the Hardening Rule described by Equation 2.20.

The bearing capacity estimates are shown in Figure 6.8, and are also listed in Table 6.1. The bearing capacity of the specimen in test T204 (the dry specimen) was estimated at about 10 kN. The relative density in the top layer of this specimen was 79%. The bearing capacity estimates for the tests on the oil-saturated specimens varied from 2 kN to 8 kN (that is, test T108, and tests T309 and T313). The relative density values of the oil-saturated specimens varied from 54% to 91%. The variation of the bearing capacity caused by a ±1° variation of the peak friction angle is also indicated in Figure 6.8. For the dry specimen, a ±30% change of the bearing capacity was predicted for the aforementioned variation of the peak friction angle.

Byrne and Houlsby (1999) measured the bearing capacity of a 50 mm diameter footing on dry Baskarp Cyclone sand at approximately 900 N, at a relative density of 95%. Their bearing capacity factors were similar to the theoretical factors, if one calculated the peak friction angles using the low stress adjustment to Bolton's method that was outlined in Section 4.5.3. It is probable that the bearing capacity estimates shown in Figure 6.8 are representative, since the bearing capacity factors of Byrne and Houlsby's tests are similar to those suggested by Bolton and Lau (1993).

The estimated values of the bearing capacity and the measured values of normalised initial stiffness are compared with some results from the literature in Figure 6.9[a]. The results from tests with the same size of footing (covering a range sand densities) should all follow a similar relationship since the initial stiffness is normalised by the radius. The results from tests with different sized footings should not all follow a similar relationship, however, because of the effect that scale has on the bearing capacity. The scale effect can explain the relatively larger N_γ values of the Byrne and Houlsby (1999) tests. The N_γ values of this investigation are unexpectedly larger than those from the other tests with a similar size of footing (for a constant normalised stiffness). The trend of each set of tests indicates that a linear relationship between the bearing capacity factor and the initial virgin-penetration stiffness is probable.

An indication of the displacement at bearing failure w_m for some of the footing tests was gained by considering the parameters used to fit the Hardening Law to the load:deformation curves in Figure 6.5. The estimated values of the bearing capacity factor and the displacement at bearing failure are compared with some values from the literature in Figure 6.9[b]. The lowest estimate of w_m is 10 mm and the highest is 40 mm, for tests T204 and T108. These values produced a similar trend to the results of de Beer (1970) and Vesic (1975) but the larger estimates of w_m were probably excessive.

The estimated values of V_m and the measured values of k_p were used to fit some of the virgin-penetration curves as shown in Figure 6.10. The theoretical curves were derived from Equation 2.20 and the fitting parameters are listed in Figure 6.10. These theoretical curves are used in the procedures in Chapter 7 to account for the vertical penetration that occurred during the sideswipe tests.

6.2.4 Constant-load time dependent behaviour

Creep behaviour of the footing was evident during the HOLD routines that were executed before each sub-test. For example, constant-load displacement of the footing occurred before each unload-reload sub-test as shown in Figure 6.11. Creep rates were measured at 500 N, 1000 N, and

1500 N and ranged between 0.2 mm/hr and 1.6 mm/hr as shown in Figure 6.12. The creep rate at similar loads was less in tests on stiffer soil. In each test, however, the rate increased with increasing penetration and load. This result was unexpected, as one would expect the rate to decrease with increasing stiffness. The creep values were usually measured over a period of five minutes and the creep rates should have decreased with time. If a creep rate of 1 mm/hr was assumed to be valid over a period of five hours, a typical test duration, a total creep displacement of 5 mm would have resulted. This was about half the typical total displacement during a test and was obviously not a realistic value.

Negative creep rates that resulted in heave of the footing were measured at the lowest loads during the HOLD routines in the unload-reload sub-tests, at V = 200 N in tests T101, T204, T108 and T309, as in Figure 6.11. A creep rate of approximately -0.07 mm/hr was measured in tests with saturated samples (tests T101, T108 and T309) and a rate of -0.16 mm/hr was measured in T204.

6.2.5 Unload-reload behaviour

Elastic parameters were deduced from the vertical unload-reload loops and the triaxial tests. These were compared with values in the literature.

Four vertical unload-reload (ULRL) sub-tests were performed from V_o = 1000 N and one from V_o = 1900 N. The symbol V_o represented the maximum vertical load experienced by the footing. The ULRL loops are shown in Figure 6.11 and the associated parameters are listed in Table 6.2. The ULRL behaviour of the footing was non-linear, and in the sub-tests with a low characterisation stiffness, tests T101 and T108 in Figure 6.11[a] and [c], the footing penetrated significantly during the reloading phase. The effect of creep on ULRL behaviour was checked to verify if the large penetrations, sustained during reloading of the footing on samples of low characterisation stiffness, were due to creep. A relation between the rate of creep and load was assumed for test T101. The creep rate was measured during the HOLD routines: at V = 1000 N before unloading; at V = 200 N; and at V = 550 N during reloading. A power relationship that fit the three points was assumed valid for unloading and reloading. The ULRL curve of test T101 is compared with the creep-corrected curve in Figure 6.13. It is obvious from Figure 6.13[b] that the large penetrations during reloading could not be accounted for assuming the above creep behaviour.

Two types of stiffness measurements were taken. The stiffness of the initial section of the unloading curve K'_{ul} was taken to represent small-strain stiffness at $\varepsilon_a = 0.05\%$. The gradient of a line drawn through the bottom and the top of the ULRL loop was taken to represent large-strain stiffness K'_{ulrl} at $\varepsilon_a \cong 0.2\%$. The axial strain was assumed equal to $w/2R$. The values of K'_{ul} and K'_{ulrl}, for the loops unloaded from 1000 N, are plotted against K'_{ch} in Figure 6.14. The data in Figure 6.14 indicates that K'_{ul} and K'_{ulrl} were proportional to the characterisation stiffness. The small-strain stiffness values were a factor of about two greater than the large-strain values. The ratio of K'_{ul} to K'_{eq} was also of interest as it indicated the ratio of elastic to plastic stiffness. K'_{eq} is used to represent the virgin-penetration stiffness before an event. In the case of the ULRL tests, K'_{eq} was the stiffness measured just before the footing was unloaded from 1000 N.

The stiffness values from the ULRL sub-test of T118 are not plotted in Figure 6.14 because the value of V_o and the relative size of the loop were different than in the other ULRL sub-tests. Plotting G against $V_o/V_{m\ dry}$ for a range of relative density values and for a constant value of strain resulted in a better comparison as shown Figure 6.15. This comparison included the effect of the relative density as well as the normalised stress level on the elastic shear modulus. Note that the drained bearing capacity was used, as the silicone oil does not seem to affect the elastic shear modulus. The elastic shear moduli that were derived from the unloading stiffness values are used, the values of G are listed in Table 6.2. Stiffness values derived from the relationship between G_o, p', and e that was suggested by Iwasaki *et al.* (1978), as given in Figure 2.2, are also plotted in Figure 6.15. The value of G_o was reduced by a factor of 0.2 so as to be representative of the strain level, and also to fit the ULRL test results. The match between the theoretical and experimental results was good. Lines representing constant values of V_o that were derived from the relationship of Iwasaki *et al.* are included in Figure 6.15.

The elastic shear moduli listed in Table 6.2 were calculated using the elastic solution of Poulos and Davis (1974) for the vertical displacement of a rigid disc on a finite layer.

$$G = \frac{K'_{ul}}{2\pi R(1+\nu)} I_p \qquad [6.1]$$

where I_p is an influence factor that is dependent on the Poisson's ratio and the ratio of the footing radius to layer depth. The value of h/R relevant to this enquiry is 3.3 and the drained Poisson's ratio is assumed to be 0.2. For these values the influence factor was found to be equal to 1.16. Rewriting Equation 6.1 with the values mentioned above gives,

$$G = 0.483\frac{K'_{ul}}{\pi R} \qquad [6.2]$$

The vertical stiffness of a footing on a finite layer, where h/R = 3.3, is approximately 20% greater than that of a similar footing on an infinite layer. The vertical stiffness on an infinite layer is given by Equation 2.5 and the K_1 coefficient, for a Poisson's ratio of 0.2, listed in Table 2.1.

The values of G deduced from the footing averaged 13 MPa at a relative density of 85%. This value of G is based on small-strain measurements at an effective mean stress of 30 kPa, that is for V = 1000 N and assuming σ'_h/σ'_v = 0.3. The estimates of G compare well with the theoretical values in Figure 2.2.

Elastic shear moduli for Baskarp Cyclone sand were also derived from triaxial and oedometer tests in Section 4.4. The triaxial values of G from tests BC10 to BC12, shown in Figure 4.3, are a factor of about six greater than the footing values. The mean effective stress in the triaxial tests was 87 kPa. This difference in stress level between the triaxial and footing tests cannot account for the difference in the value of elastic shear modulus. The elastic shear modulus can be assumed to be proportional to the square root of effective mean stress, as mentioned in Section 2.2.2. And an increase a factor of three in the effective mean stress should correspond with a 1.7 increase in G. The elastic shear moduli derived from the oedometer tests were a factor of about three greater than the values determined from the footing ULRL sub-tests.

The reloading sections of the ULRL loops were subject to varying degrees of plastic penetration as shown in Figure 6.11. The transition point from predominantly elastic reloading to that of predominantly plastic reloading seemed to be dependent on the ratio of V_o/V_m. This variation and the definition of the parameters involved are shown in Figure 6.16. The transition point from the reloading section of some of the sideswipe tests and from some of the combined load elastic loops are shown in Figure 6.16[a]. The penetration-corrected values of V_o are used for the sideswipe tests. The data indicated that the transition from predominantly elastic to plastic reloading occurred at not less than 70% of the V_o load, when this load was less than 50% of the bearing capacity load.

6.3 Partially-drained consolidation model

Lambe and Whitman (1969) stated that the consolidation due to an increment of load proceeds independently of all preceding and succeeding increments of load. Although clearly only true for elastic soil, this suggested that some insight into the effects of partially-drained loading could be gained by dividing the applied load into increments and superimposing the solutions for each increment. A computer program was written in VisualBasic to investigate the problem of a time-varying load. The following values were used in the model in the subsequent sections if not otherwise stated: applied total load ΔV = 200 N (Δq = 5.66 kPa); coefficient of consolidation c_v = 1.2 x 10^{-3} m^2/s; height of sample h = 0.25 m; and radius of footing R = 0.075 m. A comparison of the model predictions and consolidation tests is performed in Section 6.4.5.

6.3.1 Description of model

The consolidation solution of Davis and Poulos (1972) that is outlined in Section 2.7.3 is used to describe the relationship between the degree of pore pressure dissipation and the dissipation time for each load increment. More precisely, the solution was for the average degree of pore pressure dissipation along the centre line of the footing. This was assumed to be representative of the degree of pore pressure dissipation at the pore pressure transducer in the footing, at the centre of the base of the footing.

The solution for a linearly increasing load with a rise time of 1.5 s is illustrated in Figure 6.17. The total pore pressure at any time was equal to the sum of the current values of all the previously occurring increments. In the model, the load increment was assumed to result in an elastic total stress distribution on the base of the footing, as defined in Figure 2.25. For an elastic total stress distribution, the total stress at the centre of the footing was half the value of the average total stress. Hence the C_q value of two in Figure 6.17.

Two types of ramped loading were incorporated into the model. A *linear load* of constant rate and a *non-linear load* of decreasing rate as shown in Figure 6.18. The non-linear load, resulting in a parabolic load curve, is defined as follows,

$$V = \Delta V \left[1 - \left(\frac{t_r - t}{t_r} \right)^2 \right] \qquad [6.3]$$

where ΔV is the total applied load, t_r is the rise time, and t is the time. This particular form of non-linear load was chosen for reasons of simplicity; not because it is similar to the actual form of ramped loading observed in the physical tests. A sinusoidal form of ramped loading would have been more realistic. It is evident from Figure 6.18 that the type of loading influenced the value of the peak pore pressure u_p as well as the time at which the peak occurred t_p. Under linear loading, t_p is always equal to t_r. But under non-linear loading, t_p is always less than t_r. It was also possible to investigate the effect of partially-drained loading on the peak pore pressure and the effect of partially-drained loading on the t_{50} time.

6.3.2 Sensitivity of model

The only known solution that the model could be compared with was that for an undrained increment of load. From Figure 6.19[a], it is evident that the predictions of the model tended to the undrained solution of Davis and Poulos (1972), as outlined in Section 2.7.3, as the rise time t_r approached zero. The trend is also obvious when the comparison is made in T_v:U_p space as shown in Figure 6.19[b].

The influence of the time increment on a linearly increasing load with a rise time of 5 s is shown in Figure 6.20[a]. The difference in the pore pressure predictions with t_{inc} = 0.05 s and t_{inc} = 0.01 s is negligible. Thus it would seem that a time increment of 0.05 s was acceptable. In Figure 6.20[b], the difference between a large and a small time increment for two rise times is compared. It is evident from the t_r = 5 s and t_r = 15 s predictions with the larger time increment (t_{inc} = 1.0 s), that the time increment alone should not be used as a parameter to indicate the accuracy of the prediction. Even though the two predictions had the same value of time increment, the value of the load increments of the t_r = 5 s prediction were greater because of the lower number of increments. Dividing the applied load into 100 increments resulted in satisfactory predictions.

6.3.3 Parametric analysis using model

The partially-drained consolidation model was used to investigate the effects of load rate on the peak pore pressure and the dissipation times.

The effect of load rate on the peak pore pressure u_p is illustrated by plotting u_p/u_m against t_p/t_{50m} as shown in Figure 6.21. Note that u_m is the maximum pore pressure under undrained conditions, t_p is the time to u_p, and t_{50m} is the t_{50} time under undrained conditions. The time to peak pore pressure t_p is equal to the rise time t_r when the ramped load is linear but it is less than t_r when the load is non-linear, as shown in Figure 6.18. The data in Figure 6.21 indicates that the peak pore pressure reduces with increasing time to peak, or rise time, as expected. When t_p is equal to the undrained t_{50}, the peak pore pressure is approximately 70% and 60% of the undrained peak pore pressure under linear and non-linear loading respectively. A few points with similar rise times are also compared in Figure 6.21. It is evident that at small rise times the peak pore pressure under linear loading was greater than that under non-linear loading. This trend is reversed at large rise times.

The effect of loading rate on the t_{50} time is illustrated in Figure 6.22. Note the definition of the t_{50} times and pore pressures shown in Figure 6.23. Under non-linear loading, dissipation is assumed to commence from the rise time and not the time to peak. From Figure 6.22 it is evident that when the rise time is equivalent to the undrained t_{50} time, when $t_r/t_{50m} \cong 1$, there is a 20% increase in the dissipation time under linear and non-linear loading. At large rise times the increase in the partially-drained dissipation time is approximately 70% and 150% under linear and non-linear loading respectively.

6.4 Consolidation behaviour

Six consolidation (CONSL) sub-tests were performed, three in T118 and three in T319. In the first sub-test the footing was loaded from 700 N to 900 N. In the second sub-test the footing was unloaded from 900 N back to 700 N. And in the third sub-test the footing was reloaded from 700 N to 900 N. The load increment was maintained for two minutes to allow dissipation of the pore pressures. The consolidation tests of T118 and T319 are shown in Figure 6.24 and Figure 6.25. In these figures, du is the change in pore pressure measured at the centre of the footing, dq is the change in the average applied total stress, and dw is the change in displacement. It should be noted that the pore pressure measured at the centre of the footing did not dissipate back to its initial value. The pore pressure measurements shown in Figures 6.24 and 6.25 were corrected, by applying a weighting to ensure that the final excess pore pressure was zero (assuming full dissipation), as shown in Figure 6.26.

The consolidation tests were intended to be "true" consolidation tests; it was hoped that the load increment would be undrained. The first and second consolidation tests did not work properly because the efficiency of the HOLD routine with which the consolidation tests were performed was dependent on the stiffness of the footing (as discussed in Section 5.6). The stiffness factor programmed into the HOLD routine was applicable to the footing response in the final or reloading consolidation tests, that is in sub-tests T118B_3 and T319B_3. From the reloading consolidation tests shown in Figures 6.24 and 6.25, it is evident that the load increment of 200 N was achieved in approximately 1.5 s. Within this time one would expect approximately 30% pore pressure dissipation, assuming the T:U_p relationship outlined in Section 2.7.3 and a coefficient of consolidation of 0.8 x 10^{-3} m^2/s, as proposed in Section 4.3.3. Thus the load increment was partially-drained.

The pore pressure measurements in the tank from test T319B_3 are shown in Figure 6.27. The measurement in the footing is labelled pp_1 and those in the tank pp_2 and pp_3. One would expect the presence of the impermeable tank wall to reduce the rate of consolidation of the footing. It was not possible to clarify this but the fact that the measured coefficient of consolidation was of the same order of magnitude as that derived from the element tests indicated that the boundary effects might not have been significant.

The boundary effects could also be investigated by comparing the dissipation times of the transducers in the tank wall with the theoretical times for a semi-infinite layer. The t_{50} times of pp_2 and pp_3, from the third consolidation test shown in Figure 6.27 (test T319B_3), were 165 s and 240 s respectively. These values were compared with the solution for a triangular distribution in a half-closed layer (Craig, 1987) using the coefficient of consolidation from the element tests, c_v = 0.5 x 10^{-3} m^2/s. This solution resulted in a t_{50} of 24 s, indicating that the presence of the walls may have increased the dissipation times at the relevant points in the sand mass.

The consolidation settlements and the calculation of the coefficients of consolidation are discussed in Sections 6.4.1 and 6.4.3. The distribution of the total stress on the base of the footing is investigated in Section 6.4.2. This was necessary because the value of the total stress at the centre of the footing was required. A brief insight into the effect of consolidation on the effective loads is given in Section 6.4.4. And finally, comparisons of some of the consolidation tests and predictions generated by the partially-drained consolidation model are made in Section 6.4.5.

6.4.1 Consolidation settlement

Three stages of settlement are evident in the consolidation tests, especially in T118B_3 and T319B_3, as shown in Figure 6.28. The results from the immediate and primary settlement stages from all of the consolidation tests are listed in Table 6.3. The drained or effective stiffness parameters were derived from the results at the end of primary consolidation, after full dissipation of the excess pore pressures.

In the initial consolidation tests, T118B_1 and T319B_1, one would expect the footing to consolidate back to the virgin-penetration line. In effect, the drained stiffness of the consolidation tests should be similar to the relevant virgin-penetration stiffness. This was indeed the case considering the virgin-penetration stiffness values listed in Table 6.1. The drained stiffness values of the unloading and reloading consolidation tests also compared well with the values derived from the ULRL tests, which are shown in Figure 6.14. The similarity between the load:deformation path of the three consolidation tests of T118 and the effective load:deformation path, determined from the virgin-penetration stiffness and the ULRL tests, is illustrated in Figure 6.29. The drained unload stiffness values of tests T118B_2 and T319B_2 are higher than those measured in the ULRL tests because of the smaller strain magnitude in the consolidation tests. The similarity mentioned above indicates that the drained stiffness values determined from the virgin-penetration curve and ULRL curve could be used to estimate the stiffness parameter in the calculation of the coefficient of consolidation. This also reinforces the applicability of the theory of effective stress and indicates the absence of degrading mechanisms induced by pore pressure change. The large variation in the effective stiffness values, which was dependent on the position in load:deformation space and the nature of loading, should be noted as it results in a similarly large variation in the coefficient of consolidation (assuming a constant coefficient of permeability).

The immediate settlement parameters listed in Table 6.3 were measured over the initial "immediate" stage of the consolidation tests, as illustrated in Figure 6.28. These immediate settlements were not fully undrained as the rise time of the ramped load was about 1.5 s. The immediate stiffness values from the consolidation tests were plotted against the drained stiffness values in Figure 6.30. If the behaviour of a soil is assumed to be elastic, then the theoretical value of undrained stiffness can be used as the upper bound to the immediate stiffness, and the drained stiffness as the lower bound. The immediate stiffness under partially-drained loading should be a function of the rate of loading (or rate of displacement). The theoretical values of the undrained stiffness (1.8 times the drained stiffness) are also shown in Figure 6.30. The factor of 1.8 was derived from Equation 2.39 and the relevant values of ν and I_p given in Section 2.7.2. The partially-drained stiffness values from the consolidation tests were measured at an average displacement rate of 0.02 mm/s. This displacement rate was only a factor of ten greater than the average drained displacement rate (Table 6.3). For an undrained response, the rate of loading would have to be at least a factor of 100 greater than the drained rate of loading; this factor was estimated from the relation between the degree of consolidation and the dimensionless time factor in Figure 2.27. Thus the immediate stiffness values plotted in Figure 6.30 should be closer to the lower bound (drained values) than to the upper bound (undrained values). Considering the results in Figure 6.30, it would seem that the theoretical estimate of the undrained stiffness was only applicable to the unloading consolidation tests, tests T118B_2 and T319B_2, and was obviously not applicable in situations where there was significant plastic deformation.

If one assumes that the unloading tests represented elastic response and that the theoretical estimate of the undrained stiffness was applicable, then the question of what was the upper limit to the immediate stiffness values in the other tests arises. One possibility was that the undrained stiffness from the unloading tests, 18000 N/mm, was applicable to the loading and re-loading tests. This assumption was made in the calculation of the theoretical undrained settlements listed in Table 6.3; the theoretical undrained settlements were calculated in accordance with the methodology outlined in Section 2.7.2. The partially-drained stiffness data shown in Figure 6.30 do not provide adequate evidence for a relationship between the rate of loading, or rate of displacement, and the partially-drained stiffness to be postulated. If data was available for a range of loading rates, from loading, unloading, and reloading tests, then the influence of loading rate on the undrained stiffness could have been quantified. In addition, the effect of the degree of elasticity (the effect of the drained stiffness) would also have become apparent.

There was considerable secondary settlement as shown in Figure 6.28. The rate of secondary settlement was highly non-linear but the average rate in tests T118B_3 and T319B_3 was about 0.8 mm/hr. This rate was about half the average rate of primary settlement (or consolidation settlement), measured over a similar time scale, and was similar to the creep rates shown in Figure 6.12 and discussed in Section 6.2.4. Davis and Poulos (1968) detected significant secondary settlements in their tests of model footings on clay. Significant secondary settlements were also evident in their three-dimensional element tests but not in their one-dimensional tests (triaxial and oedometer tests respectively). The secondary settlements observed in the consolidation tests of this investigation might have been caused by creep as well as consolidation of the sand mass in areas some distance from the footing.

6.4.2 Degree of elasticity during ramped load

It was postulated from the analysis of consolidation settlements and stiffness in Section 6.4.1, that the response in the loading tests was mainly plastic, and in the unloading tests it was mainly elastic. By comparing the applied stress at the centre of the footing and the measured pore pressure, it was possible to conclude that the response was not purely elastic in most of the tests. The pore pressure response and the implied total stress at the centre of the footing, for a range of possible stress distributions (as defined in Section 2.6), are compared in Figure 6.31. If the response was purely elastic, then the pore pressures could not be larger than the total stresses, calculated assuming an elastic distribution. In most of the tests, the peak pore pressures were greater than the elastic total stress, even in unloading test T319B_2. The fact that the pore pressure was not greater than the elastic total stress in test T118B_2 and T118B_3 did not necessarily mean that the response was purely elastic. One would also have to be sure that the degree of dissipation was very low ($u_p/u_m \cong 1$) before concluding that the response was purely elastic.

The degree of dissipation during the loading stage of consolidation test T118B_3 was estimated using the findings of the partially-drained consolidation model. The rise time in the consolidation test was approximately 2 s and t_{50m} was estimated at 1.92 s from Figure 6.22, which gave a t_p/t_{50m} ratio of 1.04. From Figure 6.21, it was estimated that the peak partially-drained pore pressure u_p was about 60% of the maximum undrained pore pressure u_m. This suggested that the peak pore pressure in the consolidation test shown in Figure 6.31 was about 60% of the applied total stress.

It can be concluded that the response was mainly elastic during the ramped loading stage in T118B_3. This was based on the assumption that the applied total stress at the centre of the footing was about 67% greater than the measured peak pore pressure.

6.4.3 Estimation of undrained coefficient of consolidation

Preliminary estimates of the three-dimensional coefficient of consolidation c_3 were derived from the consolidation tests by assuming that the load increments were undrained. The consolidation tests were then compared with predictions from the partially-drained consolidation model.

From the discussion of the element tests in Section 4.3.3, it was postulated that c_1 = 0.8 x 10^{-3} m^2/s was representative of G = 13 MPa. The stiffness in the reloading footing tests, however, was approximately 8 MPa (Table 6.3). Thus the c_1 value for the footing tests was estimated at 0.5 x 10^{-3} m^2/s, assuming proportionality between c_1 and G. Converting the 1-D coefficient to the 3-D value resulted in c_3 = 0.25 x 10^{-3} m^2/s for a Poissons ratio of 0.2 (see Section 2.7.3).

Estimates of the partially-drained value of c_3 were also derived from the t_{80} times of the unloading and reloading consolidation tests. The t_{80} times were used instead of the t_{50} times because the applied loads overshot and cycled, before stabilising at the intended load increment of 200 N. The dissipation times and coefficients of consolidation are listed in Table 6.4. The average value of c_3 was 0.6 x 10^{-3} m^2/s for unloading and was 1.1 x 10^{-3} m^2/s for reloading. The effective stiffness in the unloading tests was a factor of about 2.6 greater than in the reloading tests (Table 6.3). It was expected that the values of c_3 would differ by a similar factor, assuming constant permeability. Considering the differences in the value of c_3 and the effective stiffness between the unloading and reloading tests suggests that either: the permeability in the reloading tests was about five times greater than in the unloading tests; or, the Poissons ratio in the reloading tests was about double that in the unloading tests(assuming ν = 0.2 in the unloading tests). The Poissons ratio should, if anything, have been less in the reloading tests than in the unloading tests. Thus it was possible that the permeability was not constant, and that it differed by the factor mentioned above.

The partially-drained value of c_3 measured from the reloading consolidation tests was a factor of about four greater than the estimated undrained value of c_3 from the element tests. This contradicted the findings of the partially-drained consolidation model, which indicated that $t_{50}/t_{50m} \geq 1$, and hence that the partially-drained value of c_3 must be less than or equal to the undrained value of c_3. Thus the estimate of the undrained value of c_3 from the element tests was not applicable to the footing consolidation tests.

The undrained value of c_3 was estimated from the t_{50}/t_{50m}: t_r/t_{50m} relationship derived from the partially-drained consolidation model and shown in Figure 6.22. Values of $t_r = 2$ s and $t_{50} = 2.4$ s were deduced from consolidation test T118B_3 (the t_{50} value was calculated from the measured t_{80} value). The intersection of the curve for a non-linear load in Figure 6.22 and the curve created by plotting $2.4/t_{50m}$ against $2/t_{50m}$ indicated that $t_{50m} = 1.92$ s. This was equivalent to an undrained c_3 value of 1.6×10^{-3} m^2/s.

The rate of consolidation settlement, as opposed to the rate of pore pressure dissipation, was not investigated because the magnitude of the displacements was small compared to the noise level. The average rate of settlement, however, was compared with the average rate of drainage as follows.

The rate of volume change due to settlement was compared with an estimate of the rate of fluid flow out of a volume of sand beneath the footing. The shape of the volume of sand was taken to be a hemisphere for reasons of simplicity. Two forms of pore pressure distribution were assumed. An undrained distribution was taken to be applicable during the immediate stage of settlement, or during the period of ramped loading. And a partially-drained distribution was assumed during the primary stage of settlement (Figure 6.28). With the undrained distribution, the pore pressure at the edge of the footing was assumed to be twice the pore pressure at the centre and the volume of sand was divided into two sections, experiencing different hydraulic gradients. The pore pressure distributions and flow characteristics used in the calculation of the volumetric flow rate are shown in Figure 6.32, and the results are listed in Table 6.5. Only the unload and reload consolidation tests were analysed. The primary consolidation settlements were negligible in the unload tests, thus only the immediate stage was considered in these tests. During the immediate stage of settlement, the rate of volume change derived from the settlement of the footing was, on average, a factor of two greater than that derived from the fluid flow. And during the primary stage, the values differed by a factor of about 0.35. These differences were not great considering the crudeness of the assumed flow characteristics and indicate a degree of coupling between volumetric change and flow.

6.4.4 Effective load during consolidation

The shape of the pore pressure distribution on the base of a footing founded on an elastic medium was discussed in Section 2.6. It was noted that the shape was a function of time. A parameter labelled C_p, from Equation 2.30, was introduced to account for the shape of the pore pressure distribution. Incorporating C_p into the calculation of the effective load on a footing, as opposed to assuming a uniform pore pressure distribution, results in a more rigorous calculation procedure.

The consolidation solutions of Davis and Poulos (1972) for a footing on a finite elastic medium can be used to estimate the dissipation of the average pore pressure below the centre of a footing. In practice, these solutions are used to model the dissipation of the average pore pressure assuming a uniform distribution over the whole of the base of a footing. The difference between the pore pressure at the centre of a footing and the average pore pressure is illustrated in Figure 6.33. The rate of dissipation of the average pore pressure over the whole base of the footing was significantly greater when C_p was taken to be a function of time, or when the pressure distribution was assumed to vary with time. This difference can be illustrated in terms of the effective foundation load as shown in Figure 6.34. The increase in effective load for a uniform pore pressure distribution, which is normally assumed in design, and a varying distribution is compared. It is evident that the more rigorous solution resulted in a greater rate of increase in effective load. Incorporating the effect of the change in shape of the pore pressure distribution may not be warranted considering that the common method of calculating the effective load is conservative. An implication of a greater rate of increase in effective load would be an increased rate of settlement. It should be remembered that the definition of the effect of dissipation on the pore pressure distribution, that is the definition of C_p in Section 2.6, was only postulated by the author and was not based on experimental observation.

6.4.5 Comparison with model predictions

The undrained coefficient of consolidation derived above, and the relationship between u_m and u_p, were used to produce the simulation of test T118B_3 shown in Figure 6.35. Also shown is a simulation of test T319B_3. The model predictions and the experimental data compared well, considering that the response on the footing was most probably not fully elastic. The model could not be said to be accurate at this stage; it would have to be tested against a larger experimental database consisting of tests with a range of rise times. The simulation process is summarised as follows:

- Measure partially-drained t_{50} and t_r from consolidation tests.
- Derive t_{50m} from Figure 6.22 and calculate the undrained c_3.
- Derive the ratio u_p/u_m from Figure 6.21 and calculate u_m.

It should be noted that the degree of elasticity, or the shape of the total stress distribution on the footing, could not be predicted. Therefore u_m could not be predicted. It was derived from the experimental value of u_p and the model prediction of u_p/u_m.

The model performed well but a rigorous assessment of the model could not be undertaken. An important observation was the possible applicability of the model to, not only fully elastic behaviour, but also to elastic-plastic behaviour. The model also provided a framework for differentiating between the factors affecting the pore pressure. That is, the total stress distribution on the footing and the dissipation characteristics of the system.

6.5 Partially-drained tests

The partially-drained vertical displacement events (VP and VPM sub-tests) that were performed from $V = 500$ N and $V = 1000$ N are analysed in this section.

The vertical partially-drained tests were performed using a range of displacement rates: 0.01 m/s, 0.1 m/s, 1 m/s, and 5 m/s. The stiffness of the sand specimens was not constant and therefore the vertical stiffness of the footing was also a significant variable, in addition to the displacement rate. Constant sample stiffness was intended but this was difficult to achieve. The distribution of the vertical partially-drained tests, in terms of displacement rate and characterisation stiffness, is shown in Figure 6.36. It should be noted that the displacement rates shown in Figure 6.36 are the intended rates, the actual or measured rates were slightly different because of the flexibility of the test rig. The interaction of the flexibility of the rig and the stiffness of the footing resulted in low displacement rate ratios in tests in which the stiffness of the footing response was high. The displacement rate ratio was defined as the actual rate divided by the intended rate. The effect of the footing stiffness on this ratio is shown in Figure 6.37. The measured displacement rate of each test is also listed in Table 6.6. A curve of the theoretical relationship between the displacement rate ratio and the footing stiffness is also shown in Figure 6.37[a] and [b]. This relationship was based on a system of two springs placed in series. The value of the vertical stiffness parameter K_{mf}, from Section 3.2.1, was estimated at 1944 N/mm. The stiffness parameter $K_{vp\text{-}s}$ was the stiffness of the secondary section of the partially-drained tests, discussed in Section 6.5.2. The slower tests fit the theoretical curve well but the tests at 1 mm/s and at 5 mm/s exhibited greater displacement rates than expected.

Referring back to Figure 6.36, which shows the distribution of the intended displacement rates and the characterisation stiffness values, it can be seen that only certain tests could be compared when investigating the effect of either of these variables. Although the variation in characterisation stiffness was somewhat random, most of the tests fell into narrow stiffness bands. For example, the mean characterisation stiffness of tests 17, 15, 10 and 13 was 478.5 N/mm, with a range of 13 N/mm. Therefore, with this set of tests, the characterisation stiffness could be said to be constant, and these tests could be used to investigate the effect of displacement rate.

The load:deformation behaviour of the partially-drained tests was investigated. The control parameters were displacement rate and drained (or effective) vertical stiffness. The drained vertical stiffness of the footing was related to the sample stiffness and therefore to the relative density of the sample. Hence the control parameters were analogous to loading rate and sample density.

The stiffness of the load:deformation response in the partially-drained tests was proportional to the characterisation stiffness as expected. This trend is obvious from the tests shown in Figure 6.38, where different tests, with different K_{ch} values but having the same displacement rate, are plotted together. Note that the first column is of tests performed from 500 N and the second of tests performed from 1000 N. Each row is of a set of tests at a particular displacement rate. The effect of the characterisation stiffness is observed over a range of displacement rates as shown in Figure 6.38.

The effect of displacement rate on the load:deformation response in the partially-drained tests is illustrated in Figure 6.39. Tests with similar values of characterisation stiffness are plotted together, the top row consists of tests with the lowest values of stiffness. The label in brackets adjacent to the test number indicate the displacement rate. Note that the partially-drained sub-tests in main test 20 were multi-rate tests, in which the footing was penetrated at alternating slow and fast rates. The general trend of increasing stiffness with rate is obvious from Figure 6.39, as the loads are higher in the faster tests. The multi-rate tests highlighted this trend well.

Analysis of the load:deformation behaviour of the VP tests indicated that three parameters could be used to define the observed response. An idealised shape of the partially-drained tests and the parameters involved are shown in Figure 6.40. The partially-drained load:deformation response could be separated into two distinct parts with differing stiffness values. The stiffness of the primary section is labelled $K_{vp\text{-}p}$ and that of the secondary section $K_{vp\text{-}s}$ (subscript "vp" signifies a vertical partially-drained test, and subscripts "p" and "s" signify the primary section and the secondary section respectively). The third parameter is $\Delta V_{vp\text{-}p}$, defined as the observed load increment of the primary section. Also shown is the equivalent drained stiffness K'_{eq}. This was the value of the virgin-penetration stiffness measured between 400 N and 500 N or between 900 N and 1000 N. That is, the value just before the partially-drained tests. Note that K'_{eq} is equivalent to K'_{ch} for the tests conducted from 500 N.

The shape of the idealised partially-drained load:deformation curve was similar to that observed in most of the partially-drained tests. The exceptions were the tests performed on the least stiff samples, which possessed low values of characterisation stiffness. All of the partially-drained tests on samples in which the characterisation stiffness was greater than 300 N/mm exhibited load:deformation curves similar to the idealised shape. The characterisation stiffness can be deduced from Figure 6.36. The tests in which the characterisation stiffness was less than 300 N/mm exhibited a reduction in load immediately after the primary section of the partially-drained move. After this reduction, the load:deformation curves tended to exhibit a constant stiffness. The footing in tests T102_1, T103A_1, T103A_2, and T105_2 exhibited pronounced work softening type behaviour and the total load reduced below its initial value.

6.5.1 Primary section of partially-drained tests

A distinct initial section was observed in the load:deformation curves of the partially-drained tests, as discussed in the preceding paragraphs and illustrated in Figure 6.40. The measured values of the partially-drained primary stiffness $K_{vp\text{-}p}$ and load change $\Delta V_{vp\text{-}p}$ are listed in Table 6.7, and will be discussed in this section.

Primary Stiffness

The values of $K_{vp\text{-}p}$ from the partially-drained tests are shown in Figure 6.41. There is considerable scatter in the results shown in Figure 6.41 and the control parameters, K'_{eq} and $\dot{w}$, do not seem to affect the partially-drained primary stiffness in any obvious manner. There is some evidence of proportionality in Figure 6.41[b] between the $K_{vp\text{-}p}$ and K'_{eq} in the partially-drained tests performed at V = 1000 N. Note that some of the K'_{eq} values measured between V = 900 N and V = 1000 N were not representative of the virgin-penetration stiffness, as outlined in Table 6.7. These included the two highest values of K'_{eq} in Figure 6.41[b]. The average value of $K_{vp\text{-}p}$ in the tests performed at V = 1000 N is slightly higher than that in the tests at V = 500 N.

The scatter in the values of $K_{vp\text{-}p}$ was due to a large extent on the lack of resolution in the displacement measurements used in the calculation of the stiffness values. The magnitude of the displacements over which the primary section occurred was sometimes only 0.02 mm. This resulted in a relative error of about ±40% in the values of $K_{vp\text{-}p}$, which translates to an absolute error of ±2000 N/mm for $K_{vp\text{-}p}$ = 5000 N/mm. It should also be noted that in the fastest moves, sometimes only two data points were recorded during the primary section of the moves.

It was probable that the creep that usually occurred during the holding of the vertical load had some influence on the primary stiffness (HOLD routines were performed before the partially-drained tests). Mitchell (1976) noted that creep loading usually increased the stiffness of subsequent virgin-penetration loading. The high initial values of stiffness were not due to the effects of over-consolidation. The partially-drained tests were performed from the virgin-penetration line in the majority of cases. It should be noted that a primary stiffness was evident in the preliminary tests, which were performed to verify an applicable footing size and pore pressure measurement system. In some of the preliminary tests, an altogether different loading apparatus was used. This ruled out the possibility that the loading apparatus was the cause of the observed primary stiffness.

The primary stiffness was most probably a rate dependent phenomenon due to the permeability of the soil, as there was no evidence in the literature of such large initial stiffness values in the rapid loading of dry sand. In the consolidation tests, discussed in Section 6.4.1, similarly large initial (or immediate) values of stiffness were measured. The value of the initial stiffness was also found to be proportional to the equivalent drained stiffness. That is, it was higher in the unloading (elastic) consolidation tests than in the loading (plastic) consolidation tests. It was postulated in Section 6.4.1 that there was an upper limit to the partially-drained stiffness that was equivalent to the undrained stiffness (the undrained stiffness being calculated from the elastic drained stiffness measured in an unloading test). In Figure 6.42, the partially-drained primary stiffness values from the partially-drained tests are compared with the values from the consolidation tests (the partially-drained tests were added to Figure 6.30). The partially-drained stiffness measured in the partially-drained tests was consistent with the postulate on the undrained stiffness. It was possible, however, that the upper limit to the partially-drained stiffness of the loading tests (the tests on a predominantly plastically deforming base) was approximately half that of the possible undrained limit shown in Figure 6.42. The fastest of the partially-drained tests should have been almost undrained, considering the time scale and the probable coefficient of consolidation. That is, about 0.02 s and about 1.0×10^{-3} m^2/s respectively.

Load change

It was evident that the magnitude of the change in load during the primary section of the partially-drained tests, as defined in Figure 6.40, was a significant parameter. The measured values of $\Delta V_{vp\text{-}p}$ are shown in Figure 6.43. The $\Delta V_{vp\text{-}p}$ values were measured over a displacement of about 0.05 mm.

There was a definite trend of increase of $\Delta V_{vp\text{-}p}$ with the rate of displacement in the tests conducted both at $V = 500$ N and at $V = 1000$ N. This trend can be visualised by plotting the values of $\Delta V_{vp\text{-}p}$ against each rate as shown in Figure 6.44 (plotted against the measured rate). From Figure 6.44[a], it is evident that at high values of displacement rate the sensitivity of $\Delta V_{vp\text{-}p}$ to changes in rate is reduced. Conversely, Figure 6.44[b] illustrates the behaviour at low values of displacement rate. The values of $\Delta V_{vp\text{-}p}$ tended toward constant values as the displacement rate reduced to zero. In both figures, inverse hyperbolic sine best-fit curves were included. The following expression is used to fit the data from V_o = 500 N,

$$\Delta V_{vp\text{-}p} = 23\sinh^{-1}(5\dot{w}) + 50 \qquad [6.4a]$$

and from V_o = 1000 N, the following expression is used,

$$\Delta V_{vp\text{-}p} = 46\sinh^{-1}(5\dot{w}) + 100 \qquad [6.4b]$$

Mitchell (1976) used this type of expression to model rate-dependent behaviour and it is the form of relationship expected if the rate-dependence could be modelled as a thermally activated process. The trend of the data at low displacement rates indicates the likelihood of an initial stiff response (like that of a primary stage in the partially-drained load:deformation curve) even at the drained rate of 0.001 mm/s. In the absence of stiffness enhancing rate effects, the value of $\Delta V_{vp\text{-}p}$ at the drained rate should have been approximately 10 N and 20 N for the tests conducted at V = 500 N and V = 1000 N respectively. These values were based on the average magnitude of displacement over which the values of $\Delta V_{vp\text{-}p}$ were measured, and the representative values of drained stiffness. The value of $\Delta V_{vp\text{-}p}$ at the creep rate should have been equal to zero. Taking 1 x 10^{-7} mm/s to be a representative creep rate, as determined in Section 6.2.4, suggests that the inverse hyperbolic sine functions cannot model the behaviour at very low rates of displacement.

The partially-drained tests performed at V = 1000 N also indicate a trend, although less definite, of an increase in $\Delta V_{vp\text{-}p}$ with the characterisation stiffness (Figure 6.43). This trend would have been reduced if the drained load increment were subtracted from the $\Delta V_{vp\text{-}p}$ values (the displacement increments of the primary sections were generally equal to 0.05 mm). Comparing the tests at V = 500 N and those at V = 1000 N, indicated that there was a significant increase in the values of $\Delta V_{vp\text{-}p}$ in the tests conducted at the higher load. That is, the $\Delta V_{vp\text{-}p}$ values in each test increased with penetration, or with an increase in the equivalent drained stiffness. The effect of the characterisation stiffness on the value of $\Delta V_{vp\text{-}p}$ was, however, assumed to be negligible in light of the results of the preliminary tests. In the preliminary tests, $\Delta V_{vp\text{-}p}$ values of about 250 N were measured even though the characterisation stiffness was about 2000 N/mm. That is, similar $\Delta V_{vp\text{-}p}$ values even though the stiffness was a factor of four greater than in the partially-drained tests of the main-test series. The preliminary tests referred to were TP08_1 and TP09_1A. These consisted of the rapid penetration, $\dot{w} \cong 8$ mm/s from a vertical load of about 400 N, of a similar footing on the same oil-saturated sand base as in the main series of tests.

It should be noted that the discussions above related only to partially-drained loading from the virgin-penetration curve, not from over-consolidated states. Partially-drained loading from an unloaded state, in the elastic region, should result in an initial stiff response which then softened as the partially-drained load path traversed the virgin-penetration line.

6.5.2 Secondary section of partially-drained tests

A secondary section, as defined in Figure 6.40, was observed in all of the vertical partially-drained tests. In some of the tests on the looser samples with low values of characterisation stiffness, there was considerable reduction in the load immediately after the primary section. These tests are highlighted in Table 6.7. After the reduction in load, the load:deformation behaviour tended to a straight line indicating a constant stiffness. In such cases, the secondary stiffness was taken as the gradient of the latter, and constant, part of the load:deformation curves.

The secondary stiffness of the partially-drained tests conducted from V = 500 N and V = 1000 N are shown in Figure 6.45. Note that some of the values of K^{d}_{eq} measured between V = 900 N and V = 1000 N were not representative of the virgin-penetration stiffness. Hence in Figure 6.45[b], the two rightmost points should be more to the left. The values of $K_{vp\text{-}s}$ were similar to the equivalent drained stiffness, even at the fastest displacement rates. This indicated that after the initial highly rate dependent section, the load:deformation behaviour of the partially-drained penetrating footing was rate insensitive. That is, after a strain of about 0.03%, in the tests in which work-softening was not evident, the load:deformation behaviour of the footing was not affected by the rate of displacement. Note that the above refers to the "total" load and not the "effective" load.

6.5.3 Pore pressure response in the partially-drained tests

In Sections 6.5.1 and 6.5.2 above, only the total loads were considered. The response of soil is dependent on effective stress. A methodology for estimating the effective load on a footing was outlined in Section 2.6. This required knowledge of the variation of the pore pressure distribution with time. Thereby allowing the pore pressure load and the effective load to be calculated. Realistically, only the pore pressure distribution in the latter stages of the partially-drained tests, after considerable pore pressure dissipation, could be assumed. This distribution should be parabolic in shape. Therefore the effective load could only be estimated in the latter stages of the tests. Note that the methodology outlined in Section 6.4.2 for estimating the total stress distribution, using the results of the partially-drained consolidation model, could not be used to deduce the effective stress at the centre of the footing in the partially-drained tests. This process was only applicable in cases where the total stress distribution was fairly constant over the period under consideration. That is, in cases where the response was predominantly elastic. The response in the partially-drained tests was not predominantly elastic as the moves were performed from the virgin-penetration line. In the following paragraphs, a qualitative analysis of the effects of the control parameters, the displacement rate and effective stiffness, on the pore pressure response will be undertaken. Note that the pore pressure response did not follow any clearly definable pattern.

A selection of tests with low and high drained stiffness, and at three rates of displacement (0.01 mm/s, 0.1 mm/s, and 1.0 mm/s), are shown in Figure 6.46. The first two columns include the low stiffness tests and the second two include the high stiffness tests. Tests from V = 500 N and from V = 1000 N are both included. The different rows of plots are of tests with different displacement rates. The partially-drained sub-tests from T105, T107, and T311 are taken to be representative of low stiffness samples, and T110, T315, and T317 of high stiffness samples. The characterisation stiffness of T311 is about 100 N/mm greater than the stiffness of tests T105 and T107 (Figure 6.36). The values of the characterisation stiffness of the chosen high stiffness samples are closer.

In Figure 6.46, the change in the pore pressure is compared with the change in the total stress for three possible total stress distributions. The pore pressure was measured at the centre of the footing. The estimates of the total stress at the centre of the footing were derived from the measured total load and the possible distributions. Since the distribution of the total stress on the base of the footing could not be assumed, and was probably not constant, the total stress was calculated for a parabolic (or plastic; $C_q = 0.5$) distribution, a uniform distribution ($C_q = 1.0$), and a saddle shaped (or elastic; $C_q = 2.0$) distribution. The parameter C_q was defined in Section 2.6. Thus the top dq curves in Figure 6.46 represent plastic response and the bottom dq curves represent elastic response. The response in the high stiffness tests was likely to be more elastic than plastic, and vice versa in the low stiffness tests. The initial behaviour in the fastest tests was likely to be more elastic than plastic as well. Note that in test T105_2 the footing exhibited evidence of liquefaction.

First of all, the effect of rate in the low stiffness tests shown in Figure 6.46 is considered. These tests exhibited higher relative values of du, relative to the dq values, in the faster tests. In the slowest tests, tests T105_1 and T105_2, the pore pressures dissipated. In the intermediate rate tests, tests T311_1 and T311_2, the rate of fluid flow and the rate of loading were probably similar, and hence the relative values of du are higher. And in the sub-tests of T107, the relative values of du are higher still. Tests T107_1 and T107A_1 also showed evidence of a change from an initial semi-elastic response to a more plastic response.

The effect of rate in the high stiffness tests shown in Figure 6.46 is not similar to that in the low stiffness tests. The intermediate rate tests do exhibit slightly higher relative values of du than in the slowest tests. But in the fastest tests, tests T110_1 and T110_2, the relative values of du are lower than in the slower tests, contradicting the pattern observed in the low stiffness tests. This could be explained by considering the degree of elasticity of the response. The sub-tests of T110 may have resulted in predominantly elastic response, as the sample stiffness and the rate of loading were high. The total stress at the centre of the footing may have been similar to that predicted by an elastic distribution. That is, by the bottom dq curve with $C_q = 2.0$. If the footing was penetrated further in the sub-tests of T110, the total stress distribution may have become more plastic and the relative values of du could have increased.

In summary, there was a definite trend of increasing relative pore pressure with increasing rate of displacement. This could be best illustrated by comparing tests at the two extremes of rate, at 0.01 mm/s and at 5.0 mm/s. The partially-drained sub-tests of T116 and T118, with similar values of characterisation stiffness, are shown in Figure 6.47. The displacement rates differ by a factor of 500 and relative pore pressures in T118 are significantly higher.

Comparing the low stiffness tests with the high stiffness tests in Figure 6.46, indicates a possible inverse relationship between the relative values of d*u* and sample stiffness. This is particularly evident when comparing the fastest tests. That is, the sub-tests of T107 and T110. This was most probably due to the possible differences in the degree of elasticity of the responses.

From the above analyses of the tests shown in Figure 6.46, it is obvious that the pore pressure response at the centre of the footing could only be sensibly explained with reference to the total stress at that point. Averaging the pore pressure over the base of the footing, or assuming the vertical load resulted in a uniform total stress distribution, could not explain the observed behaviour.

6.5.4 Evidence of liquefaction

In the discussion of Figure 6.46 in Section 6.5.3 above, it is mentioned that test T105_2 exhibited a liquefaction type response. A number of the partially-drained tests on samples with low values of stiffness showed a similar response. All of these tests liquefied to some degree but none showed evidence of full liquefaction. That is, the estimated effective load did not reduce to zero. It should be noted that in estimating the effective load, a parabolic shaped pore pressure distribution was assumed. This assumption was particularly applicable to the latter stages of the tests.

All the tests that partially liquefied are shown in Figure 6.48. As mentioned above, the relevant tests were all on low stiffness samples. The average characterisation stiffness was 150 N/mm. Thus the displacement rate was the only control parameter of influence. In the slowest test, the test in the first row of plots, the partial liquefaction subsides and the effective load increases. The interaction between the rate of penetration and the rate of pore pressure dissipation in the fastest test leads to progressive liquefaction and a continual reduction in the effective load. It should be noted that all of these tests were performed under displacement control. Under load control, the observed liquefaction would have been much more pronounced.

The partial liquefaction most probably commenced in the very early stage of loading and at the edge of the footing. At this stage, the pore pressure distribution could have been similar to the undrained shape, with high pressures at the edge of the footing. These high pore pressures would have resulted in substantial hydraulic gradients and the flow of pore fluid away from the edge. This flow could have caused local liquefaction.

There was a possible reason why the tests on the low stiffness samples liquefied and those on the high stiffness samples did not. Consider the definition of the effective stress at a depth δs below the edge of a footing with an upward hydraulic gradient of i,

$$\sigma'_v = \gamma_w \, \delta s \left[\left(\frac{\gamma}{\gamma_w} - 1 \right) - i \right] + q \qquad [6.5]$$

where q is the contact stress at the edge of the footing. If one assumes that the difference in the density of the samples was insignificant and that the hydraulic gradients were similar, then Equation 6.5 suggests that the contact total stress must have been greater in the tests that were not prone to liquefaction. That is, in the tests on the stiffer samples. It seemed more probable, however, that the density was the important parameter.

6.5.5 Effective load response in the partially-drained tests

Some general trends in the pore pressure behaviour are outlined in Section 6.5.3. The effective load response, deduced from estimated values of the pore pressure load and the measured total load, was also investigated. The change in the effective load dV', measured at d$w \cong$ 0.24 mm, was calculated for a range of possible pore pressure distributions. The effective load was estimated at d$w \cong 0.24$ mm because the maximum penetration in some of the partially-drained tests was not much greater than d$w \cong 0.3$ mm. The drained stiffness of the partially-drained tests was also calculated. This was calculated from the point after a move at which the pore pressure returned to its ambient value.

The range of dV' in the partially-drained tests is shown in Figure 6.49. The first column is of tests performed from $V = 500$ N and the second column of tests from $V = 1000$ N. The different rows indicate different displacement rates, the slowest tests are in the first row of plots. The circular data points indicate the values of dV' calculated assuming a uniform pore pressure distribution. Error bars are used to indicate the possible range of dV'. As illustrated in Figure

6.49[a], the top bars represent the values of dV' calculated assuming a parabolic (partially-drained) pore pressure distribution and the bottom bars represent the values calculated assuming a saddle shaped (undrained) distribution. The dotted lines indicate the values of dV' that would be expected if the penetration over the displacement increment of 0.24 mm was at the equivalent effective stiffness. That is, at the probable drained penetration stiffness.

In the absence of degrading mechanisms, such as caused by liquefaction, one would expect the change in effective load to be similar to that predicted assuming drained penetration, indicated by the dotted line. This was indeed the case in the slowest tests in Figure 6.49[a] and [e]. In the slowest tests, the pore pressure distribution was most probably parabolic in shape and the top bars should be indicative of the values of dV'. Note that the second partially-drained sub-test of T105, shown in Figure 6.49[e], shows evidence of substantial liquefaction as there is a definite reduction in the effective load in this test.

The tests at $\dot{w}$ = 0.1 mm/s, in Figure 6.49[b] and [f], also show evidence of an effective load response similar to that predicted by the drained stiffness. That is, the partially-drained sub-tests of T311 and T315. Both the partially-drained sub-tests of T103 liquefied to some extent. The pore pressure distribution was most probably parabolic in shape, as the time increment was of a similar order of magnitude to the t_{50} time. Hence the top bars should be indicative of the values of dV'.

In the fast tests at $\dot{w}$ = 1 mm/s and $\dot{w}$ = 5 mm/s, it could not be assumed that the pore pressure distributions were parabolic in shape and that the top bars were indicative of the values of dV'. Therefore it was possible that the effective load response in the fast tests was less than the expected drained response. This seemed to be the case in the tests at $\dot{w}$ = 1 mm/s. Some of the fastest tests from V = 1000 N, however, produced larger values of dV' than predicted by the expected drained stiffness, see tests 14 and 13 in Figure 6.49[h].

The discussions above indicate that the effective load response, at a specific point during the slower partially-drained tests, was similar to the expected drained response. In the faster tests, the evidence was inconclusive.

Analysis of the response at the end of the partially-drained tests indicated lower values of effective stiffness at higher rates of penetration. The overall drained stiffness of the partially-drained tests was measured between the start of the moves, and the point after the moves at which the excess pore pressures were zero. The values of the overall drained stiffness are shown in Figure 6.50. The overall drained stiffnesses at the slower rates of $\dot{w}$ = 0.01 mm/s and $\dot{w}$ = 0.1 mm/s are similar to the values of drained stiffness measured just before the partially-drained tests, indicated by the dotted lines. The faster tests produce significantly less stiff responses. This reduction in stiffness was not due to increased creep in the faster tests, as the time scales over which the measurements were taken were similar.

6.5.6 Multi-rate tests

Two multi-rate vertical partially-drained tests (VPM) were performed in main-test T120. The displacement rate was alternated between 0.01 mm/s and 1 mm/s. The details of these tests are listed in Table 6.6 and Table 5.1. The results from the constant-rate partially-drained tests that are shown in Figure 6.44, and expressed by Equation 6.4a, indicate that the load changes (or $\Delta V_{vp\text{-}p}$ values) associated with rates of 0.01 mm/s and 1 mm/s are 50 N and 100 N. That is, when the initial vertical load is 500 N. The load changes are 100 N and 200 N when the initial load is 1000N, as deduced from Equation 6.4b. One would expect similar load changes in the multi-rate tests.

The multi-rate tests are shown in Figure 6.51. The test from an initial vertical load of 500 N is shown in the plot on the left and the test from 1000 N is shown on the right. The pore pressure load U, calculated for a parabolic distribution, is also included. The extrapolated drained penetration line (or virgin-penetration line), and the partially-drained penetration lines along which the slow and fast moves tended, are shown. The symbols represent the data points along the loading sections of the fast moves. The motor stalled during the second fast move of the test from 500 N. Lines indicating the initial stiff response of the partially-drained moves are shown. The primary stiffness $K_{vp\text{-}p}$ varied between 2kN/mm and 10 kN/mm in all of the moves, but lines with a gradient of 10 kN/mm are shown in Figure 6.51. These values are similar to the values measured in the normal partially-drained tests, which are shown in Figure 6.41.

The $\Delta V_{vp\text{-}p}$ values in the test from 500 N shown in Figure 6.51[a] are 60 N and 125 N. These values are very similar to those predicted by Equation 6.4a. The values during the second and third slow moves were slightly less, but the values during both of the fast moves are consistent. The

$\Delta V_{\text{vp-p}}$ values in the test from 1000 N shown in Figure 6.51[b] are 90 N and 210 N. Again these values are very similar to those predicted, in this case by Equation 6.4b.

The findings of the multi-rate tests substantiate and extend the results of the constant-rate partially-drained tests. The multi-rate tests indicate that the rate dependent penetration lines deduced from the constant-rate tests might be unique, in so much as they are not affected by the prior $w{:}V$ state. That is, the $\Delta V_{\text{vp-p}}$ values might not influenced by prior moves at different rates.

The pore pressure at the end of each slow move increased as the tests progressed. This indicates that there might have been some degradation in the effective load since the total load at the end of each test increased in parallel with the drained penetration line.

7 Results: Combined Loading

7.1 Introduction

Sideswipes tests were performed at different rates to investigate the effect of partially-drained loading on the yield surface. The results of the drained horizontal and moment sideswipe tests are compared with tests in the literature. The drained load:deformation behaviour within the yield surface is investigated and elastic parameters are derived. The drained load:deformation behaviour of the footing during yield is also investigated, in the context of a flow rule. The effects of partially-drained behaviour, or the rate of loading, on the yield surfaces tracked by sideswipes is analysed.

7.2 Drained *V*:*H* sideswipe tests

Five drained horizontal sideswipes (HSS) were performed. The sideswipe of test T204 was performed on dry sand. In addition to these, two multi-rate sideswipe tests (HSSM) with drained components were performed in test T120. A comprehensive set of plots of the drained horizontal sideswipe tests (in the three axes of load and displacement) is shown in Figure 7.1. From these plots it can be seen that tests T118D_2 and T120D_H2 were displaced in the negative horizontal direction, or to the left. The majority of tests were commenced from V_{oi} = 1500 N, but tests T101_1, T120D_H2 and T118D_2 were commenced from V_{oi} values between 1600 N and 1700 N (see Figure 7.1[a]). Although the tests performed at a horizontal displacement rate of 0.001 mm/s were assumed to be drained, low values of pore pressure were measured at the centre of the footing. The pore pressure load shown in Figure 7.1[f], calculated assuming a parabolic pore pressure distribution on the base of the footing, varied between -5 N and -20 N. A pore pressure load of -20 N is equivalent to a pore pressure of only -2.3 kPa.

The values of V_{oi} and the intended and measured displacement rates of the horizontal sideswipe tests are listed in Table 7.1. The measured displacement rates were generally less than the intended rates because of the flexibility in the rig. This is illustrated Figure 7.2. The displacement rate ratio is plotted against the characterisation stiffness in Figure 7.2. In Section 4.6

it was suggested that the characterisation stiffness was indicative of the density of the samples. The distribution of the displacement rates and the characterisation stiffness values of the horizontal sideswipe tests is shown in Figure 7.3. From Figure 7.2 it would seem that the displacement rate ratio of the drained sideswipe tests was not influenced by the characterisation stiffness. It should be noted that the measured rates were averaged over the whole duration of each sideswipe and that some of the variations of displacement with time were not linear.

7.2.1 Shape of the yield surfaces

The sideswipes are shown in V:H space and in V_o-normalised space in Figure 7.4. The sideswipe of T101 was started from an overconsolidated state. The footing was unloaded from V = 1700 N before the sideswipe was commenced. Tests T120D_H1 and T120D_H2 were multi-rate tests in which the horizontal displacement rate was alternated between 0.001 mm/s and 0.1 mm/s. The flatter sections of the multi-rate tests represent the slower displacement rate.

The effect of the characterisation stiffness on the shape of the sideswipe curves is evident in Figure 7.4[a]. The peak values of the horizontal load are larger in the tests with the larger values of characterisation stiffness. This effect is better illustrated by plotting the tests in V_o-normalised space as shown in Figure 7.4[b]. Note that V_o was the equivalent vertical load along the extrapolated virgin penetration curve. Thus the tests shown in Figure 7.4[b] are corrected for the vertical penetration that occurred during the tests. This correction procedure is discussed in Section 2.3.3 and illustrated in Figure 2.8.

The difference between V_{oi} and V_o normalisation is shown in Figure 7.5. The magnitude of the correction is greatest in the stiffest samples, as small penetrations during the sideswipe tests resulted in relatively large increases in the value of V_o. The sideswipe of T101 was also corrected for the initial overconsolidated state. The probable degrading effect of the excess pore pressures in the multi-rate tests was not accounted for. The effect of excess pore pressures on the vertical load:deformation behaviour was discussed in Section 6.5.5.

The shapes of the sideswipe tests in Figure 7.4[b] are generally parabolic, except that of test T204_1 that was on the stiffest sample. The peak V_o-normalised horizontal load (or h_o value as referred to in Section 2.3.3) increased with increasing characterisation stiffness. A parabola defined by Equation 2.9 with $h_o = 0.12$ is also shown in Figure 7.4[b]. This parabola was representative of the V_m-normalised yield surface of the horizontal sideswipe tests of Gottardi and Houlsby (1995) (see Section 2.3.3 and Figure 2.9). But in accordance with the tests of Gottardi and Houlsby, the sideswipes do not converge to this probable yield surface when plotted in V_o-normalised space.

The shape of the sideswipe of test T101 shown in Figure 7.4 is peculiar. Cyclic shearing of the soil or slippage of the drive system may have caused the saw-tooth shaped load path (as discussed in Section 5.3.1). If this phenomenon was due to the response of the soil, then localised partially-drained shearing had probably developed. The unstable nature of the shearing might also have been influenced by the flexibility in the displacement-controlled loading rig (there was a load-controlled component in the application of the loads). A section of the saw-tooth shaped sideswipe is shown in Figure 7.6. From Figure 7.6[a] it can be seen that the unloading sections are characterised by rapid shearing and penetration, whilst the loading sections are characterised by slow shearing and penetration, which is followed by heave in some cycles. From the plot of the vertical load and pore pressure load in Figure 7.6[b], it is evident that the rapid shearing and penetration (and hence contraction) were associated with negative pore pressures. One would expect the contraction of a sand mass to be associated with positive pore pressures. Although the mass as a whole was contracting, the layer of sand immediately beneath the footing might have been dilating. Low negative pore pressures were associated with all of the sideswipe tests even though the footing was penetrating in each test (see Figure 7.1[f]). This illustrates the complex localised nature of the mechanism beneath the footing.

The analysis of the combined load tests of Gottardi and Houlsby (1995) in Section 2.3.3 indicated that the shape of the surfaces tracked by sideswipe tests was influenced by the ratio of V_{oi} to V_m. That is, the shape was influenced by the ratio of the initial vertical load to the bearing capacity load. Normalisation by V_m, as opposed to V_o, resulted in a more sensible framework for analysing the sideswipe tests.

The evidence that h_o was dependent on the value of V_{oi}/V_m was reinforced by sideswipe tests conducted by Byrne and Houlsby (1999) on dense Baskarp Cyclone sand (the same sand as used in this project). The value of h_o decreased as V_{oi}/V_m increased, or as the initial vertical load approached the bearing capacity load. The relationship between V_{oi}/V_m and h_o shown in Figure

2.10, and expressed by Equation 2.11, is assumed to be applicable to the tests in this project. This assumption could not be verified, as the bearing capacity loads of the tests in this project were not measured, they were estimated. The above postulate inferred that the aforementioned relationship was valid for a larger footing than that from which the relationship was derived. The relationship should be sufficiently generic considering that data from tests on two very different sands (in terms of grain size and shape, and also with relative densities that differed by 20%) were used to derive the relationship.

The results of this investigation indicate a similar trend to that inferred by Equation 2.11, as the values of h_o shown in Figure 7.4[b] were larger in the tests on the stiffer samples. Note that in Section 6.2.3 it was suggested that the characterisation stiffness was proportional to the bearing capacity. Therefore the value of V_{oi}/V_m for test T204_1 should be substantially less than that for test T101_1. The two tests of T118 did not fit the general trend between h_o and V_{oi}/V_m. One would expect the value of h_o in test T118D_2 to be less than the value in test T118D_1, as the value of V_{oi} was greater in test T118D_2. The value of V_m in the two tests was constant as they were performed on the same sample.

Equation 2.9 was used to fit parabolas to the drained sideswipe tests shown in Figure 7.7, so as to derive values of h_o. The measured load paths in Figure 7.7 do not represent points along a single yield surface because of the vertical penetration that occurs during the sideswipe tests. In cases where the value of V_o/V_m changed considerably during the sideswipe, where the vertical penetration was significant, the value of h_o would change accordingly.

The data in Figure 2.10 indicates that the value of h_o, and hence the shape of the V:H yield surface, is particularly sensitive at low values of V_{oi}/V_m. The values of V_o/V_m at the start and at the end of the sideswipe tests are listed in the table included in Figure 7.7. Two yield surfaces, representative of the start and the end of the sideswipes, are also shown in some of the plots where the value of h_o is expected to change significantly. By fitting parabolas to different sections of the load paths shown in Figure 7.7, it is implied that if constant vertical displacement sideswipes were conducted at the different sections they would produce parabolic shaped surfaces. The evidence from the tests of Gottardi and Houlsby (1995), of Byrne and Houlsby (1999), and from test T204_1 indicated that V:H yield surfaces at low values of V_o/V_m could not be modelled by a parabolic curve. Extrapolating a parabolic curve from the start of a sideswipe could also lead to an overestimate of the value of h_o, as the presence of an overconsolidated state would affect the initial section of a sideswipe. The values of h_o deduced from the parabolic curves, which are fitted to the start and end of the sideswipes in Figure 7.7, should be viewed as extreme values.

The values of V_o/V_m at the start and at the end of tests T204_1, T309_1, T118D_1 and T118D_2 indicated that the shape of the yield surface probably changed during these tests. The values of h_o at the end of all of the aforementioned tests, except test T204_1, are lower than at the start. This is in agreement with the expected trend between h_o and V_{oi}/V_m. The load path of test T204_1 indicates that a parabolic shape cannot be used to model the surface at very low values of V_o/V_m. The complex response observed in test T204_1 was due to strong dilatant shearing of the foundation sand. The dilatant shearing resulted in heave of the footing and a consequent increase in the vertical load. This is illustrated in Figure 7.1[g] and Figure 7.1[d] respectively. On exhaustion of the dilatant behaviour at the peak horizontal load, the vertical and horizontal load reduced. Both of these loads would probably have reduced to zero, or to a critical state, if the sideswipe were continued.

The values of h_o deduced from Figure 7.7 are compared to the results of Gottardi and Houlsby (1995) and of Byrne and Houlsby (1999) in Figure 7.8. The values of h_o for the surfaces fitted to the start of the sideswipes are generally higher than those that were indicated by Equation 2.11. This is expected since the aforementioned values of h_o are probably not representative. It should be noted that in the derivation of Equation 2.11, values of V_o/V_m at the start of sideswipes were correlated with peak values of h_o. This methodology is acceptable when the shape of the surfaces traversed by a sideswipe do not change considerably. In modelling the effect of footing penetration on the shape of the yield surface, it may be better to correlate peak values of h_o with the corresponding values of V_o/V_m. It should be remembered that the values of V_m are estimated from the cone penetrometer tests. The variability in the value of V_o/V_m of test T204_1, caused by a $\pm 1^\circ$ variation in the estimate of the peak friction angle, is shown in Figure 7.8.

The V_m-normalised sideswipes are shown in Figure 7.9. One would expect the load paths to follow a similar trend to the sideswipes of Gottardi and Houlsby (1995) (shown in Figure 2.9[b]) where the peak values of H/V_m were proportional to the corresponding values of V/V_m. It is evident from Figure 7.9, however, that the paths of test T204_1 and T309_1 cross those of the other tests. This is probably due to unrepresentative bearing capacity estimates. The bearing capacity values of tests T204 and T309 are either underestimated or those of the other tests are overestimated, or a combination of both. The range due to a ±1° variation in the estimate of the peak friction angle for tests T204 and T118 is shown in Figure 7.9. The parabola that fit the V_m-normalised results of Gottardi and Houlsby (Equation 2.9 with h_o = 0.12) is also shown in Figure 7.9. The end points of T204_1 and T309_1 are a substantial way outside of this parabola. This reinforces the idea that the bearing capacity values of these two tests are underestimated.

7.2.2 Shape of the yield surfaces on reversal of horizontal displacement

Reverse displacement sideswipes were performed at the end of some of the drained sideswipes and these are shown in Figure 7.10. These sideswipes were performed to investigate the shape of the yield surfaces after load reversal. Most of the reverse displacement sideswipes were displaced at rates greater than 0.001 mm/s, but the rates on the saturated samples were not more than a factor of ten greater than the drained rate. The pore pressure loads calculated assuming a parabolic distribution are shown in Figure 7.10[d]. The maximum pore pressures measured during the reverse-displacement sideswipes were about twice those measured during the initial drained sideswipes, and were of an opposite sign as well. The change from negative to positive pore pressure was due to the increased rate of contraction (or penetration as shown in Figure 7.10[c]) associated with the particle rearrangement. After the reversal, the pore pressures decreased and probably would have tended to steady negative values once again.

The initial and reverse-displacement yield surfaces are shown in Figure 7.10[e] and the attached enlarged view. The fitted yield surfaces in the *V*:*H* plane are representative of the end points of the initial sideswipes and those in the *V*:(-*H*) plane are representative of the end points of the reverse displacement sideswipes. The fitted surfaces were corrected for the associated vertical penetration, as well as the probable change in the shape of the surfaces. The reverse-displacement sideswipes of tests T101_1, T309_1, and T118D_2 approached the probable yield surfaces. This indicates that the findings relating to the shape of the yield surfaces are applicable even after load reversal. Yield surfaces are not fitted to the sideswipes of tests T204_1 and T118D_1. This is because parabolic yield surfaces cannot model the path tracked by a sideswipe test performed at very low values of V_o/V_m, and because the reverse-displacement sideswipe of T118D_1 was not displaced sufficiently.

The shape of the loading and reverse loading sideswipes in Figure 7.10[e] is very similar to the drained constant volume load paths investigated by Luong (1980). The $p':q$ triaxial load paths tracked by Luong were initially parabolic, but then moved upwards along a "characteristic state line" once they approached the failure surface. Tests T204_1 and T118D_2 behaved in a similar fashion. Thus the sliding line evident in footing tests may be analogous to the characteristic state line. The load paths in the undrained triaxial tests performed by Luong were also very similar to the drained constant volume paths.

7.2.3 Location of the critical state point

The point at which a footing could be displaced *continuously* at constant load and vertical displacement was labelled the critical state point. Tan (1990) referred to this point as the "line of parallel points". If a critical state point exists in $V:H$ space, one would expect load paths from high values of V/V_m to move down towards this point and for load paths from low values of V/V_m to move up towards this point. The model footing tests of Gottardi and Houlsby (1995) indicated that a critical state point did not exist in $V:H$ space at values of V/V_m greater than 0.05 (as discussed at the end of Section 2.4.2).

Consideration of the vertical load, horizontal load, and vertical displacement of the sideswipe tests (shown in Figure 7.1[d], [e] and [g]) indicates that none of the tests reached a critical state point. Consequently, from the V_m-normalised sideswipe tests shown in Figure 7.9, it can be seen that a critical state point did not exist at values of V/V_m greater than about 0.05 (assuming that the estimated bearing capacity value for test T118 was representative). Test T204_1 did achieve a temporary state of constant load and penetration after the footing heaved, but the load path reversed after reaching this state at peak horizontal load. This is clear from Figure 7.4[b]. If a critical state point does not exist in $V:H$ space, then the load path of test T204_1 shown in Figure 7.4[b] and Figure 7.9 would approach the origin after backtracking on itself. For this to happen, the shearing of the foundation soil would have to result in volumetric contraction (the dilatant shearing would have to be unsustainable). There is evidence in the literature of unstable dilatant shearing of sand at large strains. Han and Vardoulakis (1991) measured post failure volumetric contraction in the shear bands that developed in biaxial element tests.

The sideswipes that were tracked on the reversal of the horizontal displacement did not reach critical state points (the footing was displaced in the opposite horizontal direction after some of the sideswipe tests, see Section 7.2.2). At least one component of the two loads and displacements shown in Figure 7.10 is changing at the end of the reverse displacement sideswipes. Although the load paths of the reverse displacement sideswipes of T204_1 and T118D_2 shown in Figure 7.10[e] are tracking up probable yield surfaces, these paths would probably backtrack on themselves and approach the origin. That is, if they behave in a similar fashion to that observed in the positive sideswipe of T204_1.

7.3 Drained *V*:*M* sideswipe tests

Five drained moment sideswipes (MSS) were performed. In addition to these, two multi-rate sideswipe tests (MSSM) with drained components were performed in test T120. A comprehensive set of plots of the drained moment sideswipes is shown in Figure 7.11. It can be seen from these plots that tests T319D_2 and T120E_M2 were displaced in the negative rotational direction (or anti-clockwise). The tests were commenced from V_{oi} values between 1500 N and 2100 N (see Figure 7.11[a]). The V_{oi} values of each test are listed in Table 7.2.

Low values of pore pressure were measured at the centre of the footing, even though the tests performed at a rotational displacement rate of 0.0026 mm/s are assumed to be drained. The rotational displacement was defined as $2R\theta$. The pore pressure load shown in Figure 7.11[f] is calculated assuming a parabolic pore pressure distribution on the base of the footing. This load varies between zero and -20 N.

The values of V_{oi} and the intended and measured displacement rates of the moment sideswipe tests are listed in Table 7.2. Note that a rotational displacement rate of 1 mm/s is equivalent to an angular rate of 0.38 deg/s. The measured displacement rates shown in Figure 7.12 are not less than 80% of the intended rates. The variation of the displacement rates with the characterisation stiffness values of the moment sideswipe tests is shown in Figure 7.13.

7.3.1 Shape of the yield surfaces

The sideswipes are shown in *V:M/2R* space and in V_o-normalised space in Figure 7.14. Tests T120D_M1 and T120E_M2 were multi-rate tests in which the rotational displacement rate was alternated between 0.0026 mm/s and 0.262 mm/s. The flatter sections of the multi-rate tests represented the slower displacement rate.

There is a trend between the values of the characterisation stiffness and the shape of the sideswipes of tests T102, T311 and T319 shown in Figure 7.14[a]. The results from the multi-rate sideswipes of T120 do not fit this trend. Although the characterisation stiffness and the estimated bearing capacity values of tests T319 and T120 are similar, the surfaces tracked in T120 are smaller than those in T319 are. This difference is better illustrated in Figure 7.14[b] where the loads are normalised by V_o. It was possible that the response in the slow (and drained) sections of the multi-rate sideswipes was affected by the fast moves. The sideswipes of T319D_1 and T319D_2 were commenced from slightly overconsolidated states. The initial vertical loads were about 5% overconsolidated. Consequently, the sideswipes started from V/V_o values of about 0.95 (as shown in Figure 7.14[b]).

The tests shown in Figure 7.14[b] are corrected for the vertical penetration that occurred during the tests. The difference between V_{oi} and V_o normalisation is shown in Figure 7.15. The magnitude of the correction is greatest in the stiffest samples. The probable degrading effect of the excess pore pressures in the multi-rate tests is not accounted for.

The shapes of the sideswipe tests in Figure 7.14[b] are generally parabolic. The sideswipes of test T319 were stopped at the upper soft limits that were set on the moment load. These limits were set to protect the load cell. A parabola defined by Equation 2.14 with m_o = 0.09 is also shown in Figure 7.14[b]. This parabola was representative of the V_m-normalised yield surface of the moment sideswipe tests of Gottardi and Houlsby (1995) shown in Figure 2.12. But in accordance with the tests of Gottardi and Houlsby, the sideswipes do not converge to this probable yield surface when plotted in V_o-normalised space.

The analysis of the combined load tests of Gottardi and Houlsby (1995) and of Byrne and Houlsby (1999) in Section 2.3.4 indicates that the shape of the moment yield surfaces is influenced by the ratio of V_{oi} to V_m. Their results relating to V:H response indicate a similar dependency. The relationship between V_{oi}/V_m and m_o shown in Figure 2.10, and expressed by Equation 2.15, should be applicable to the tests in this project (for reasons outlined in Section 7.2.1).

Equation 2.14 is used to fit parabolas to the drained sideswipe tests shown Figure 7.16, so as to derive values of m_o. The measured load paths in Figure 7.16 do not represent points along a single yield surface because of the vertical penetration that occurred during the sideswipe tests. In cases where the value of V_o/V_m changed considerably during the sideswipe, the value of m_o would change accordingly. The similar feature relating to the horizontal sideswipes was discussed in detail in Section 7.2.1. This discussion should be applicable to the moment sideswipe results as well.

The values of V_o/V_m at the start and at the end of the sideswipe tests are listed in the table included in Figure 7.16. Two yield surfaces (representative of the start and the end of the sideswipes) are also shown in some of the plots where the sideswipes tracked into the post-peak region. That is, in the plots of tests T102A_2 and T311_3. The initial shape of the sideswipes of test T319 in Figure 7.16 is affected by the overconsolidation that was present. The load path of these tests is parabolic once the value of V_o increases beyond the overconsolidation load.

The values of m_o deduced from Figure 7.16 were compared with the results of Gottardi and Houlsby (1995) and of Byrne and Houlsby (1999), in the form of Equation 2.15, in Figure 7.17. The values of m_o deduced from the end points of the sideswipes of test T319 are similar to the values of Equation 2.15. The values of m_o of the other sideswipes are all substantially less than the values implied by Equation 2.15. Even the values of m_o that are deduced from the start of the sideswipes of T102A_2 and T311_3 are less. It is possible that the estimate of the bearing capacity of T102 is too high. The estimated value of V_m of T102 is about 50% larger than the value of T311, although the characterisation stiffness is less. The difference between the m_o values of the sideswipes of test T120 and T319 could not be explained. The measured values of characterisation stiffness and the estimated values of bearing capacity of the two different samples are both similar. It is possible that the partially-drained components of the multi-rate sideswipes of T120 had a degrading effect.

The V_m-normalised sideswipes are shown in Figure 7.18. The load paths do not follow a similar trend to the sideswipes of Gottardi and Houlsby (1995), shown in Figure 2.12[b], where the peak values of $M/2RV_m$ were proportional to the corresponding values of V/V_m. The path of the sideswipes of test T319 shown in Figure 7.18 cross those of the other tests. This was probably due to unrepresentative bearing capacity estimates. The sideswipes were all within the parabola that fit the V_m-normalised results of Gottardi and Houlsby (Equation 2.14 with m_o = 0.09). This indicates that the V_m yield surface under moment loading in this project is probably smaller than that implied by the tests of Gottardi and Houlsby.

7.3.2 Shape of the yield surfaces on reversal of rotation

A reverse rotation sideswipe was only performed at the end of test T102A_2, and not at the end of the other moment sideswipe tests. The pattern of behaviour on reversal of rotation at the end of test T102A_2 was very similar to that observed in the horizontal sideswipes. The load path approached the probable yield surface in V:($-M/2R$) space.

7.3.3 Location of the critical state point

The definition of the critical state point relevant to the footing problem was outlined in Section 7.2.3. The moment sideswipe tests of Gottardi and Houlsby (1995) indicated that a critical state point was probable at $V/V_m \cong 0.25$ (this is discussed at the end of Section 2.4.2).

Tests T102A_2 and T311_3 were approaching a critical state at the end of the tests. The vertical and moment loads shown in Figure 7.11[d] and [e] are stabilising, and the rates of penetration shown in Figure 7.11[g] are reducing. If the estimated values of V_m for tests T102 and T311 are accurate, then the V_m-normalised data in Figure 7.18 implies that the critical state point is located at $V/V_m \cong 0.1$. It is probable, however, that the estimated values of V_m are unrepresentative. If the V_m values of tests T102 and T311 are reduced so as to locate the end points of these tests at $V/V_m \cong 0.25$, the required values of V_m would be unrealistically low (equal to about 2000 N). This indicates that the critical state point suggested by the moment sideswipe tests in this project is probably located at a V/V_m ratio less than 0.25.

7.4 Drained load:deformation behaviour

7.4.1 Load:deformation behaviour within the yield surface

Two sets of horizontal and moment load-unload loops were performed in test T120. The loops were performed at load states within the *V*:*H*:*M* yield surface. The load-unload loops, and the load and vertical displacement time histories, are shown in Figure 7.19. The first set of loops shown in Figure 7.19[a] to [d] indicates that the vertical load and displacement are constant, implying that the loops are mainly elastic and that the load-unload stiffness values are representative. The vertical load reduces substantially during the second set of loops shown in Figure 7.19[e] to [h]. This indicates that the sand base was subjected to significant plastic deformation. The plastic deformation is pronounced during the negative loading section of the horizontal loop (see Figure 7.19[g] and [h]). This results in the open loop shown in Figure 7.19[e].

The vertical unload-reload stiffness associated with the first set of combined load loops was 5560 N/mm. This value is similar to the small strain stiffness values listed in Table 6.2. Thus the ratios of the combined load stiffness values to the vertical stiffness are $H/u = 0.5V/w$ and $M/(R^2\theta) = 0.3V/w$. These ratios are about half the value of those derived from elastic theory and about twice the value of those measured in the footing tests of Gottardi and Houlsby (1995). Ratios are not calculated for the second set of loops because of the plastic deformation that occurred.

The load paths preceding the loop events, the proximity of the probable yield surfaces in *V*:*H* space, and the proximity of the elastic-to-plastic transition surfaces in *V*:*H* space are investigated with the aim of explaining the difference between the two sets of events shown in Figure 7.19. The transition surfaces are deduced from the vertical reloading response that is analysed in Section 6.2.5 and illustrated in Figure 6.16. The load paths preceding the first set of loop events are shown Figure 7.20[a] to [f], and those preceding the second set are shown in Figure 7.20[g] to [l]. Although both sets of events were performed at states within the *V*:*H*:*M* yield surface, the first set of events were performed after unloading, whilst the second set were performed after unloading and some reloading. Both of the loop events are also within the *V*:*H* elastic-to-plastic transition surfaces shown in Figure 7.20[f] and [l]. These surfaces are representative of the start position of the loop events.

The yielding observed in the horizontal loading part of the second set of loop events cannot be explained by the proximity of the transition surface. This response indicates that the combined loading behaviour within the yield surface cannot be assumed to be predominantly elastic. The response of the footing during the reverse displacement moves, which were performed after some of the horizontal sideswipes, was also highly plastic, even though the load paths were within the yield surfaces. The reverse displacement moves are discussed in 7.2.2 and shown in Figure 7.10.

7.4.2 Load:deformation behaviour on the yield surface

A feature of plasticity theory is that the plastic strain increment vectors are only governed by the ratio of stresses at the relevant point, and not by the stress path. The relationship between the stress ratio and the plastic strain increment vectors is called a flow rule. The variation between the load ratio and the plastic displacement increment vectors, derived from some of the drained horizontal sideswipe tests, is investigated. It is possible to deduce plastic displacement increment vectors because there was significant vertical penetration during the sideswipe tests. The plastic load:deformation behaviour of the tests is compared with the results of footing tests in the literature.

The load ratio is defined as H/V and is presented as the angle $\tan^{-1}(H/V)$. The plastic displacements (and hence the plastic displacement increments) are calculated by subtracting the estimated elastic component from the measured displacements. A vertical elastic stiffness of 3 kN/mm and a horizontal elastic stiffness of 2.5 kN/mm were assumed (see Section 6.2.5 and 7.4.1). An illustration of the elastic and plastic displacements in w:V space is given in Figure 2.8[b]. The plastic displacement increment vector is presented as the angle $\tan^{-1}(du_p/dw_p)$.

The $\tan^{-1}(du_p/dw_p)$:$\tan^{-1}(H/V)$ results from some of the drained sideswipe tests of this investigation are compared to the results from the model footing tests of Gottardi and Houlsby (1995) in Figure 7.21. The results from a constant V test and from an overconsolidated sideswipe test are included in the data from Gottardi and Houlsby in Figure 7.21[a]. The variation between the load and displacement ratios shown in Figure 7.21[b], of the sideswipe tests of this investigation, are similar to those of Gottardi and Houlsby. The load and displacement ratios increase as the sideswipes progressed.

The load ratios shown in Figure 7.21[b] are still increasing at the end of the tests, except the ratio in test T204_1 that reduces slightly after reaching a peak value. This is consistent with the shape of the load paths shown in Figure 7.4. The load ratios would have tended to constant values at low values of V/V_m if the paths were to follow sliding lines. The displacement ratios indicate that the vertical component of the vectors tend to zero as the sideswipes progressed. The footing heaves in the sideswipe initiated from the lowest value of V/V_m (in test T204_1).

The displacement vectors shown in Figure 7.21[b] are angled between 60° and 90° to the horizontal over most of the length of the sideswipes. These angles are larger than those indicated by the plastic potentials suggested by Tan (1990) and Cassidy (1996), and shown in Figure 2.18. That is, the vertical component of the vectors measured in this research is significantly less than that suggested by the aforementioned authors. This difference is probably due to the variation in the direction of the load paths used to expand the yield surface. Tan used constant V probes and Cassidy used the constant V tests and constant load-ratio tests performed by Gottardi and Houlsby (1995). Cassidy (1999) also noted a difference in the flow rules suggested by the constant V probes and constant load-ratio tests of Gottardi and Houlsby.

The flow response of the sideswipe tests shown in Figure 7.21[b] is not appreciably affected by the wide range of the V_{oi}/V_m values between the tests. The relationship between the load ratio and plastic displacement increment vectors of the moment sideswipes were also investigated. The ratios of the displacements at the start of the moment sideswipes were equal to approximately 70°. At the end of the moment sideswipes, the load ratios were about 12° and the displacement ratios were about 85°.

The horizontal stiffness over the initial section of the horizontal sideswipes range between 2 kN/mm and 3 kN/mm (measured to 25% of the peak horizontal load). Thus the load:deformation response of the initial sections of the sideswipes is similar to the elastic response. The comparable moment stiffness was about 500 N/mm, which is a factor of about three less than the relevant elastic stiffness.

7.5 Partially-drained *V*:*H* sideswipe tests

Six partially-drained horizontal sideswipes (HSS) were performed. In addition to these, two multi-rate sideswipes (HSSM) with partially-drained components were performed in test T120. The footing was displaced horizontally at rates of 0.1 mm/s and 0.4 mm/s in the partially-drained sideswipe tests (the rate in the drained tests was 0.001 mm/s). The distribution of the displacement rates with the characterisation stiffness of the samples is shown in Figure 7.3. The values of V_{oi} and the intended and measured displacement rates of the tests are listed in Table 7.1. The ratio of the measured rates to the intended rates is plotted against the characterisation stiffness in Figure 7.2. This ratio decreased with increasing characterisation stiffness because of the flexibility in the rig.

A comprehensive set of plots of the partially-drained tests at $\dot{u}$ = 0.1 mm/s and $\dot{u}$ = 0.4 mm/s are shown in Figure 7.22 and Figure 7.23 respectively. All of the tests shown in Figure 7.22[a] and Figure 7.23[a], except T103A_3, were commenced from V_{oi} values of about 1500 N. Test T103A_3 was commenced from V_{oi} = 1340 N. Tests T103A_3 and T315C_1 were conducted from overconsolidated states. The footing was unloaded from V = 1600 N before test T103A_3. A sideswipe was attempted from V = 1500 N before T315C_1 was performed. There was an error in the input file of this initial sideswipe, which was recorded as test T315_3. The failed sideswipe resulted in expansion of the yield surface to an extrapolated virgin-penetration load of V = 1650 N.

7.5.1 Displacements and associated pore pressures

The pore pressure response of sand undergoing shear loading in element tests is dependent on the potential for volumetric strains associated with the shear strains. Sand that dilates when sheared under drained conditions results in negative pore pressures when sheared under undrained conditions. And sand that contracts when sheared under drained conditions results in positive pore pressures when sheared under undrained conditions. If the vertical and the horizontal displacements of a footing are representative of the volumetric and the shear strains respectively, one would expect the pore pressure response under partially-drained loading to be dependent on the potential for vertical penetration or heave. The pore pressure responses in the partially-drained sideswipe tests are compared with the vertical displacements in the drained sideswipe tests (tests with similar values of characterisation stiffness being comparable).

The ratio of plastic displacements from the drained sideswipe tests shown Figure 7.21[b] indicates that the rate of vertical displacement decreased as the sideswipes progressed. In the latter stages of the test on the stiffest sample (T204_1), the footing heaved. The effect of the characterisation stiffness on the vertical displacement is evident from Figure 7.1[g]. If the low pore pressures measured in the drained sideswipe tests did not affect the displacements, then under partially-drained loading one would expect an initial positive pore pressure, which should reduce as the horizontal displacement increased. The pore pressure should not decrease continuously, however, as the footings penetrate after heaving.

The pore pressure response of the partially-drained tests shown in Figure 7.22[g] and Figure 7.23[g] followed the expected trend, except the tests initiated from overconsolidated states. Tests T103A_3 and T315C_1, which were performed from overconsolidated states, did not develop initial positive pore pressures. This is understandable since overconsolidation results in increased dilatancy. This is also evident when comparing the ratio of the plastic displacements of the overconsolidated sideswipe with the normal sideswipes in Figure 7.21[a].

The coupling between the pore pressure response and the vertical displacement is evident from the effect that the horizontal displacement rate has on the slope of the u:w curves shown in Figure 7.1[g], Figure 7.22[h] and Figure 7.23[h]. The vertical penetration (comparing tests on samples with similar values of characterisation stiffness) reduced with increasing horizontal displacement rate. At very fast horizontal displacement rates, as the drainage reduces to zero, one would expect the vertical penetration to tend to a minimum.

7.5.2 Shape of the yield surfaces

The probable drained yield surfaces of each of the partially-drained sideswipe tests were deduced from the relationship between the shape of the surfaces in the drained sideswipe tests and the characterisation stiffness values. The sideswipe test results of Gottardi and Houlsby (1995) and of Byrne and Houlsby (1999) indicated that the shape of a sideswipe was dependent on the ratio of V_{oi}/V_m. This relationship was not used to deduce the probable drained yield surfaces because the V_m estimates in this investigation were unrepresentative. The normalised peak horizontal loads of the drained tests are plotted against the characterisation stiffness in Figure 7.24. The peak values in Figure 7.24[a] are not corrected for the vertical penetration, as they are normalised by the initial vertical load. The values in Figure 7.24[b] are normalised by V_o and are therefore corrected for the vertical penetration. The effect of the correction on the normalised peak horizontal load is negligible at low values of K_{ch}, and reduces the load by about 0.03 at high values of K_{ch}.

The effect of the horizontal displacement rate on the uncorrected (for vertical penetration) sideswipes was deduced by comparing probable drained and partially-drained V_{oi}-normalised sideswipes. The procedure for correcting the partially-drained sideswipes (or for choosing an appropriate value of V_o with which to normalise the loads) was not straightforward. Some possible methods for choosing the penetration corrected normalising load V_o are illustrated in Figure 7.25.

The vertical load:deformation behaviour is shown in Figure 7.25[a] and interaction of the loads is shown in Figure 7.25[b]. The sideswipe is commenced from the initial virgin-penetration load V_{oi}. The effective load V' at a point in the sideswipe is less than the total load V since a positive pore pressure load is assumed. The analysis of the vertical partially-drained tests in Section 6.5 indicates that there are unique partially-drained penetration curves. These are above the virgin-penetration curve and their heights are dependent on the rate of penetration. The stiffness $K_{vp\text{-}p}$ of the w:V path from the drained virgin-penetration curve up to the partially-drained penetration curves is higher than the drained unload-reload stiffness K'_{ulrl} (and is probably rate dependent as well). The analysis of the consolidation tests in Section 6.4 also indicates that the unload-reload stiffness is rate dependent.

Therefore it is possible that the total load V in Figure 7.25[a] should be normalised by the loads at either points $V_{o\text{-}pd2}$ or $V_{o\text{-}pd3}$, as opposed to the load at point $V_{o\text{-}pd1}$. The normalising load $V_{o\text{-}pd2}$ was extrapolated up to the partially-drained penetration curve at some drained elastic stiffness K_d and the normalising load $V_{o\text{-}pd3}$ was extrapolated up at some partially-drained (and vertical rate dependent) stiffness K_{pd}. Although the vertical penetration associated with the sideswipe shown in Figure 7.25 is zero, the plastic vertical penetration is greater than zero. Hence a plastic vertical penetration rate can be deduced.

The partially-drained sideswipes were not corrected for the vertical penetration because of the ambiguity concerning the choice of an appropriate normalising load. Although the procedure for correcting the effective load V' is straightforward (as shown in Figure 7.25[a]), this was not undertaken because the single pore pressure measurement at the centre of the footing was most probably not representative of the pore pressures on the whole of the footing. This was due to the unsymmetrical $V{:}H$ loading. The total load and effective load sideswipes are compared with probable drained parabolas in the following paragraphs.

The V_{oi}-normalised partially-drained sideswipes are shown in Figure 7.26. The tests in the left column were displaced at 0.1 mm/s and those in the right column were displaced at 0.4 mm/s. The rows represent three ranges of characterisation stiffness. The values of the characterisation stiffness are listed in the individual plots. The total load and the probable drained parabola are shown as thick lines. The thin dotted lines represent the parabolas that were fitted to the total load sideswipes. The effective load sideswipes are also shown.

The total load sideswipes shown in Figure 7.26 track surfaces that are larger than the drained parabola, except the sideswipe of T103A_3. This suggests that the size of yield surfaces tracked by horizontal sideswipe tests is rate dependent. Although two of the slower sideswipes were overconsolidated, there is a noticeable increase in the size of the surfaces tracked at the faster displacement rate as well. The parabolas fitted to the total load sideswipes indicate that the intermediate section of the partially-drained surfaces can be modelled by an increased partially-drained V_{oi} value. This is in accord with the findings of the vertical partially-drained tests.

The following two parameters were adjusted to fit the parabolas to the total load sideswipes in Figure 7.26: the intercept with the horizontal axis (or V_{oi} value); and the normalised peak horizontal load (or h_o value). The intercept value affected the size and the peak value affected the shape. The magnitudes of these values, relative to the drained parabolas, are shown in Figure 7.27. The percentage change in size and peak are plotted against the measured horizontal displacement rates. The values from the overconsolidated tests are not shown. The change in the peak value is erratic but a trend of increasing size with rate is evident in Figure 7.27. The increase in the value of V_{oi} in the fastest tests was about 12%.

The effective load sideswipes (calculated assuming a parabolic pore pressure distribution on the base of the footing) shown in Figure 7.26 do not match the drained parabolas. This suggests that the estimates of the pore pressure loads were unrepresentative or that there was effective stress hardening in the foundation sand, or both. The assumption that the effective load path under partially-drained conditions is similar to the drained load path may not be valid.

Drained load paths were followed after some of the horizontal partially-drained tests. This was done to check the position of the V:H load point after pore pressure dissipation and to check the yield surface at low values of vertical load. The loads paths followed after three of the partially-drained tests are shown in Figure 7.28. The data was taken from the background logging files. After pore pressure dissipation, the V:H load point approached the drained parabola in all of the tests. This indicates that there was not much hardening during these partially-drained sideswipes (if the effects of creep are ignored).

7.5.3 Multi-rate sideswipe tests

Two multi-rate horizontal sideswipes tests (HSSM) were performed in which the horizontal displacement rate was alternated between 0.001 mm/s and 0.1 mm/s. The details of these tests are listed in Table 7.1. The measured displacement rates varied between about one tenth and one half of the intended rates. The average measured value of the fast rates was 0.01 mm/s in the first test and 0.025 mm/s in the second test. The results from the partially-drained tests that are shown in Figure 7.27 indicate that the yield surface should expand by about three to five percent during the fast moves of the multi-rate tests. The expanded yield surface is compared with the multi-rate sideswipes in Figure 7.29. A parabola that is five percent larger than the drained sideswipes fit the peaks of the fast moves. The fast moves were not displaced for a sufficient distance to verify if the fast (partially-drained) sections would have tracked along the expanded surface. If the expanded surfaces shown in Figure 7.29 are representative, then the results indicate that the partially-drained surfaces may be unique. That is, the positions of the rate dependent partially-drained surfaces may not affected by the initial load state in V:H space.

7.6 Partially-Drained *V*:*M* sideswipe Tests

Six partially-drained moment sideswipes (MSS) were performed. In addition to these, two multi-rate sideswipes (MSSM) with partially-drained components were performed in test T120. The footing was rotated at rates of 0.026 mm/s, 0.26 mm/s, and 0.524 mm/s in the partially-drained sideswipe tests (or 0.01 deg/s, 0.1 deg/s, and 0.2 deg/s respectively). The rate in the drained tests was 0.0026 mm/s. The distribution of the displacement rates with the characterisation stiffness of the samples is shown in Figure 7.13. The values of V_{0i} and the intended and measured displacement rates of the tests are listed in Table 7.2. The ratio of the measured rates to the intended rates is plotted against the characterisation stiffness in Figure 7.12. This ratio decreased with increasing characterisation stiffness because of the flexibility in the rig.

A comprehensive set of plots of the partially-drained tests at $2R\dot{\theta} = 0.026$ mm/s and $2R\dot{\theta} = 0.26$ mm/s, and at $2R\dot{\theta} = 0.524$ mm/s are shown in Figure 7.30 and Figure 7.31 respectively. All of the tests shown in Figure 7.30[a] and Figure 7.31[a], except T317A_1, were commenced from V_{oi} values of about 1500 N. Test T317A_1 was commenced from V_{oi} = 1700 N and from an overconsolidated state. The footing was unloaded from V = 1830 N before test T317A_1. The soft load limit on the moment load was set too low in test T317_3. This test was repeated from a higher vertical load as T317A_1.

7.6.1 Displacements and associated pore pressures

The ratio of plastic displacements of the drained moment sideswipes was similar to that of the horizontal sideswipes. Thus a similar pattern of pore pressure measurements was expected. That is, an initial increase in the pore pressure followed by a reduction. The pore pressure response of the partially-drained moment sideswipes shown in Figure 7.30[g] and Figure 7.31[g] followed the expected trend. The rate of decrease of the pore pressures also reduced at large rotations. In the sideswipe on the least stiff sample (test T105_3 in Figure 7.30[g]), the pore pressure was increasing at the end of the test.

The coupling between the pore pressure response and the vertical displacement is evident from the effect that the rotational displacement rate has on the slope of the $2R\theta$:w curves shown in Figure 7.11[g], Figure 7.30[h] and Figure 7.31[h]. The vertical penetration (comparing tests on samples with similar values of characterisation stiffness) reduced with increasing rotational displacement rate. A similar pattern was observed in the horizontal sideswipe tests.

7.6.2 Shape of the yield surfaces

The probable drained yield surfaces of the partially-drained moment sideswipe tests were estimated from the characterisation stiffness. The relationship between the V_{oi}-normalised peak moments from the drained sideswipes and the characterisation stiffness values was deduced. The procedure was the same as employed with the horizontal tests. The effect of the rotational displacement rate on the uncorrected (for vertical penetration) sideswipes was deduced by comparing probable drained and partially-drained V_{oi}-normalised sideswipes. The partially-drained moment sideswipes were not corrected for the vertical penetration for the same reasons as outlined in Section 7.5.2.

The V_{oi}-normalised partially-drained sideswipes are shown in Figure 7.32. The tests in the left column were displaced at rates of 0.026 and 0.26 mm/s. The first test in the left column was displaced at 0.26 mm/s. The tests in the right column were displaced at 0.524 mm/s. The rows represent three ranges of characterisation stiffness. The values of the characterisation stiffness are listed in the individual plots. The total load and the probable drained parabola are shown as thick lines. The thin dotted lines represent the parabolas that were fitted to the total load sideswipes. The effective load sideswipes are also shown.

The total load sideswipes shown in Figure 7.32 track surfaces that are larger than the drained parabolas, except the sideswipe of T317A_1 that was overconsolidated. This suggests that the size of yield surfaces tracked by moment sideswipe tests is rate dependent, which is in accord with the results of the horizontal tests. The increase in the size of the partially-drained surfaces is more consistent in the faster tests. The parabolas fitted to the total load sideswipes indicate that the intermediate section of the partially-drained surfaces can be modelled by an increased partially-drained V_{oi} value. This is in accord with the findings of the vertical partially-drained tests.

The following two parameters were adjusted to fit the parabolas to the total load sideswipes in Figure 7.32: the intercept with the x-axis (or V_{oi} value); and the normalised peak moment load (or m_o value). The intercept value affected the size and the peak value affected the shape. The magnitudes of these values, relative to the drained parabolas, are shown in Figure 7.33. The percentage change in size and peak are plotted against the measured horizontal displacement rates. The values from the overconsolidated test is not shown. The change in the peak value is erratic but the trend of increasing size with rate is evident in Figure 7.33. The increase in the value of V_{oi} in fastest tests was about 12%. These results are very similar to those observed in the partially-drained horizontal tests.

The effective loads in the moment sideswipes shown in Figure 7.32 do not differ by much from the total loads. This is not surprising since the positive pressures on the downward half of the footing and the negative pressures on the upward half of the footing should cancel each other out. This statement is based on the assumption that there is symmetry between loading and unloading, which is not strictly true. The response of the sand subjected to unloading will be predominantly elastic and the response of the sand subjected to loading will be plastic.

The position of the *V:M* load point after pore pressure dissipation was checked for a few of the tests. The load paths followed after three of the partially-drained tests are shown in Figure 7.34. The data was taken from the background logging files. After pore pressure dissipation, the *V:M* load point approaches the drained parabola in test T114E_1 but traverses the drained parabola in the other two tests. This indicates that there was probably some degradation in tests T112E_1 and T313D_1. One would expect, however, a greater degree of degradation in the test on the less stiff sample.

7.6.3 Multi-rate sideswipe tests

Two multi-rate moment sideswipes tests (MSSM) were performed in which the rotational displacement rate was alternated between 0.0026 mm/s and 0.26 mm/s. The details of these tests are listed in Table 7.2. The measured displacement rates were approximately equal to the intended rates. The average measured value of the fast rates was 0.2 mm/s in the first test and 0.28 mm/s in the second test. The results from the partially-drained tests that are shown in Figure 7.33 indicate that the yield surface should expand by about five percent during the fast moves of the multi-rate tests. The expanded yield surface is compared with the multi-rate sideswipes in Figure 7.35. Parabolas that are eight percent and six percent larger than the drained sideswipes fit the peaks of the fast moves of the two tests (T120D_M1 and T120E_M2 respectively). These results are in accord with the results from the horizontal multi-rate tests, and reinforce the postulate that there may be unique partially-drained surfaces.

8 Conclusions

This thesis is concerned with the partially-drained load:deformation behaviour of shallow footings. A flat circular model footing was tested on a sand base under 1-g conditions. The sand was saturated with oil to reduce its permeability. The effects of partial drainage were investigated by varying the displacement rate of the footing. The effect of displacement rate on the vertical and combined load response has been analysed. The vertical virgin-penetration and unload-reload behaviour, and the combined load yield surfaces (V:H and V:M) were investigated.

8.1 Main findings

1 Rig for model testing

The displacement-controlled rig was capable of investigating the partially-drained behaviour of the model footing. For example, vertical displacement rates that differed by a factor of 5000 were investigated. Fully undrained behaviour was probably not attained, however, because of the velocity limits of the drive system. The logging speed of the data acquisition system was also not sufficient for logging the velocities required for fully undrained behaviour.

A displacement measuring system that was capable of measuring the three components of displacement to an accuracy of about ten microns was developed.

2 Sample preparation

Apparatus and procedures were developed for preparing oil-saturated (low-permeability) sand samples. An automated densification process that involved alternate application of downward hydraulic gradient and vibration was successful at preparing dense samples. It was possible to liquefy the samples by applying an upward hydraulic gradient but an automated stirring device was developed to ensured that the samples were fully fluidised.

3 Characterisation of oil-saturated sand samples

The bulk relative density measurements did not correlate well with the response of the footing. The density of the top layer of the samples was gauged from miniature cone penetrometer tests.

4 Initial virgin-penetration stiffness as a characterising parameter

The correlation between the vertical and combined load (drained and partially-drained) behaviour and the drained virgin-penetration stiffness was good.

5 R_D:p':ϕ'_{max} relationship for very low stress levels

A low stress adjustment to Bolton's (1986) method was developed. This was required because the stresses associated with the 1-g miniature cone penetrometer tests were very low.

6 Vertical load: Initial loading and risk of liquefaction

The initial loading of the footing (immediately after touchdown) on the loose samples was prone to liquefaction. This indicates that full drainage should be provided on the underside of foundations during touchdown and the initial stages of loading.

7 Vertical load: Partially-drained consolidation model

An incremental partially-drained consolidation was developed. This model was capable of simulating the partially-drained behaviour of the footing when the response was predominantly elastic.

8 Vertical load: Partially-drained unload-reload behaviour

The partially-drained unloading and reloading of the footing during the consolidation tests indicated that the unload and reload stiffness were rate dependent. The stiffness increased with displacement rate.

9 Vertical load: Partially-drained virgin-penetration behaviour

The virgin-penetration response was dependent on the rate of penetration. This investigation suggests that there are indeed unique penetration curves, which are mainly dependent on the rate of penetration and the drained penetration stiffness (or density of the sand). Two distinct load:deformation sections were evident in the partially-drained penetration curves. The primary section was highly rate dependent and the load change associated with this section could be modelled using an expression derived from rate process theory. The stiffness of the secondary section was similar to the drained virgin-penetration stiffness.

10 Vertical load: Partially-drained virgin-penetration behaviour and risk of liquefaction

Partially-drained loading from the drained virgin-penetration curve resulted in localised liquefaction in some of the loose samples. This highlights the coupling between the density and the rate on the risk of liquefaction.

11 Vertical load: Partially-drained virgin-penetration behaviour and effective stress

The overall change in the effective load between the start and the end of the partially-drained moves was checked. This indicated that the partially-drained moves resulted in degradation of the effective stiffness, as compared to the drained virgin-penetration stiffness.

12 Combined load: Drained yield surfaces and dependency of shape on V_o/V_m

Analysis of test data in the literature indicated that the shape of combined load yield surfaces tracked by sideswipe tests (horizontal or rotational displacement at constant vertical displacement) is dependent on the ratio of the current vertical load to the vertical bearing capacity load. The results of this investigation showed a similar trend. It is believed that this dependency will only be evident in cases where the foundation sand is prone to dilation. That is, when a general or local shear failure mechanism is probable.

13 Combined load: Drained yield surfaces and critical state points

Investigators have suggested that there are points on the combined load yield surfaces at which constant load and constant vertical displacement shearing occur. The analysis of the results of this investigation, and the analysis of test data in the literature, suggests that there is a critical state point along the V:M failure surface. With regard to the V:H failure surface, if a critical state point does exist, its position would be very close to the origin.

14 Combined load: Pore pressures associated with partially-drained moves

The general nature of the pore pressure response during the partially-drained combined load moves could be predicted from the flow rule deduced from similar drained moves. This highlights the coupling between the pore pressure and volumetric strain (vertical displacement).

15 Combined load: Partially-drained yield surfaces

The V:H and V:M yield surfaces expanded as the horizontal and rotational rate increased. That is, the partially-drained yield surfaces were larger than the comparable drained surfaces. This investigation suggests that there are unique partially-drained yield surfaces, which are mainly dependent on the rate and the drained response (or density of the sand). Partially-drained loading also resulted in a change in the shape of the surfaces. The effect of increasing rate in the partially-drained tests was similar to the effect of lower values of V_o/V_m in the drained tests. The effective load yield surfaces (V':H and V':M) might also have expanded.

8.2 Concluding remarks

Aspects of the (total stress) vertical and combined load behaviour of a footing on sand were shown to be rate dependent. The vertical response was stiffer, and the combined load yield surfaces expanded, as the displacement rate increased (or as the degree of drainage decreased). The rate dependent expansion of the virgin-penetration curve was modelled using an expression deduced from rate process theory. Sufficient data were not available to quantify the expansion of the combined load yield surface. But the trend of the results indicated that the initial expansion of the yield surfaces could be modelled using a similar expression to that used to model the vertical load.

The typical dimensions of shallow offshore foundations are such that, when loaded by environmental forces, the loading will be partially-drained. Current design procedures do not account for partial drainage. If the results of this investigation are representative of the prototype situation, then a design based on drained behaviour might be conservative since the partially-drained total stress behaviour is stiffer.

Good analytical models for the drained and undrained monotonic behaviour of footings exist. These models were developed from the results of experiments on model footings. The objective of this investigation was to provide experimental evidence that could be used in the development of an analytical partially-drained footing model. The ultimate objective of further research should be the development of such a model. This, however, is still not possible with the current experimental database. Much more data on partially-drained behaviour is required. Some aspects that need exploring are listed below:

— The effective stress response has to be investigated. The pore pressure distribution on the base of the footing will be required.

— The effects of sustained partially-drained loading, up to failure, on the vertical response have to be studied.

— Monotonic partially-drained loading can trigger liquefaction (or partial liquefaction) in loose samples. Pore pressure measurements across the whole area of a footing, and especially at the edge, would be required to investigate this problem.

— It is important that the fully undrained response be investigated as this, along with the drained response, help define the limits of the problem.

— More data on the effect of rate on the combined load yield surface is needed. The effect on other combined load paths and on the work-softening region has to be investigated as well. The combined load:deformation behaviour, on and within the yield surface, will also be affected by the degree of drainage.

Offshore loading is cyclic, and load reversals can have dramatic effects on the response of sand. Cyclic behaviour is highly complex and difficult to model, but the next step after the development of a monotonic partially-drained model would be the development of a cyclic partially-drained model.