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CHAPTER 11

TRIAXIAL COMPRESSION TESTS

11.1 INTRODUCTION

The objectives of the triaxial compression tests were:-

a. To determine the shear strength characteristics of soil-cement for use in design and for comparison with the results of the unconfined compression and direct shear tests.

b. To investigate the effect of the variations in compaction and curing conditions, examined in the leaching test program, on the shear strength characteristics of soil-cement.

c. To investigate the effect of leaching in the leaching cells and leaching bath on the shear strength of soil-cement.

The consolidated undrained triaxial test with pore pressure measurement was used for all tests in order to obtain the maximum information from the test program.

A total of 46 sets of specimens were scheduled for testing. However, as discussed in chapter 9, considerable deterioration of the inlet end of specimens subject to leaching in the leaching cells occurred. Surface deterioration of the specimens in the leaching bath was also observed.

As discussed in section 9.3.5, it was felt that strength testing of badly deteriorated specimens would give meaningless results. This decision is reinforced by the conclusion in chapter 9 that leaching is a surface effect and that leaching by percolating water is unlikely to lead to deterioration in a massive soil-cement embankment. In addition, the accurate final dry mass of these test specimens was considered essential for interpretation of the leaching test results and it was felt that triaxial testing could lead to significant material loss. The limiting penetration of 1 mm for specimens to be subject to triaxial testing, was thus selected. Only unleached specimens and leached specimens for which penetration was within this limit were tested.

In the light of the conclusions in chapter 9 and on the basis of observations of the deformation of leached specimens tested triaxially, only the data for the unleached specimens, a total of 19 sets, was processed and is presented in this chapter. In most cases, sets contained six specimens but a number of three specimen sets were tested to investigate additional factors.

Curing conditions and ages investigated were determined by the leaching test program. As noted in chapter 9, various factors delayed the triaxial program and as a result no triaxial tests were conducted at ages less than 290 days.

Cell pressures used in the test program were derived from consideration of the analysis discussed in chapter 3 applied to theoretical embankments up to 150 m high, including an alternative soil-cement structure for the Rondebosch Dam. On the basis of this analysis, it was concluded that the very high cell pressures used by certain other workers in this field were unrealistic and that a maximum confining stress of the order of 2200 kPa was adequate for the size of soil-cement dam that could reasonably be expected to be constructed in the foreseeable future.

A back pressure of 200 kPa was applied to the specimens during saturation and consolidation and the total confining pressures during testing were 300, 450, 700, 1030, 1600 and 2400 kPa. In the case of sets containing three specimens, only the three lowest cell pressures were used. Two high pressure triaxial cells and pressure supplies were designed and built to accommodate the higher pressures. Pressure supplies were also built for the low pressure cells and gauges for measurement of volume change were built for all cells.

Strain rates and test procedures were selected after studying a number of references, including Jardine (1963), Jackson (1965), El-Rawi (1967) and Abboud (1969).

The strain rate of 0.02 %/min used by Jardine (1963) and Jackson (1965) was used. This was slower than that used by Abboud (1969) and El-Rawi (1967) but was found to give an acceptable time to failure. Because of the relative stiffness of the soil-cement specimens, a considerable deflection of the proving rings occurred before failure. Accordingly, it was necessary to adjust the rate of displacement of the loading ram to accommodate this additional deflection.

11.2 APPARATUS

Standard constant strain rate compression testing machines were used for the triaxial tests. Stiffer mountings and side bars were required for the two machines with lowest capacity to accommodate the required loads, but the machines were otherwise completely standard.

Load measurement was by means of proving rings. Three additional proving rings of 40 kN capacity were fabricated from roller bearing shells, a fourth shell was repeatedly loaded to check linearity and hysteresis and then loaded to destruction in order to determine suitability and safe working load.

Deflection measurement was by means of dial deflection gauges and pore pressure measurement by means of electrical pressure transducers. Linearly variable displacement transducers (LVDT's) were obtained and brackets made to enable them to be mounted in parallel with the dial gauges for load and deflection measurement. Screened cabling and power supplies were installed to permit all readings to be taken at a central point by means of an electronic data-logger. However, problems with the data-logger necessitated manual recording of all data.

A needle punched geofabric with pronounced longitudinal fibre orientation and low lateral tearing strength was used to form the side drain around the specimen. Comparison with conventional strip drains indicated no increase in strength resulting from the use of the fabric. This was attributed to the low lateral strength of the fabric and the low radial strain at failure. This material provided excellent side drainage during de-airing, saturating and consolidation. Sintered brass porous disks were used to provide rigid end drains. A disk of geofabric was placed between the porous disk and the specimen to prevent clogging. Conventional slotted filter-paper drains were used for the unstabilized specimens.

Because of the high pressures used, two latex membranes of the type manufactured for the leaching tests were used to enclose the specimen. These were held in place at each end with two O-rings.

11.2.1 Triaxial Cells

The triaxial cells used for the lower cell pressures were standard cells manufactured by Wykeham Farrance Ltd with a maximum working pressure of 1030 kPa. These cells were modified by replacing the standard "Klinger" packed gland plug cocks with ball valves with neoprene seals which provided the same zero volume change characteristic with better sealing and smoother operation at lower cost.

Two high pressure triaxial cells were built to accommodate the higher pressures required. These were designed for a maximum working pressure of 7 MPa and were fabricated from mild steel in order to simplify machining and minimise cost. To prevent corrosion, a cell fluid of 90% water 10% soluble machinists cutting oil was used. This was found to be clean to work with and did not leave a significant oily deposit on the cells and membranes. No short-term deterioration of the membranes was noted. One of the assembled cells, together with the major components of the second cell, is illustrated in plate 11.1.

PLATE 11.1
HIGH PRESSURE TRIAXIAL CELLS

The high pressure cells were modelled on the standard Wykeham Farrance cells and made use of the standard cell base. This was modified by the replacement of the plug cocks with high pressure ball valves. Holes were drilled for bolts fastening the base of the high pressure cells to the standard base. In normal use, the cell base was left bolted to the standard base. The cell was machined from thick walled steel tubing and the top cap was similar in design to the standard top cap. A 25 mm polished steel piston with an O-ring seal was used. The resistance of the O-ring was considered to be insignificant in comparison to the axial loads applied. The top cap was fitted with a ball valve for venting the cell.

In use, the specimen was assembled on the pedestal, the cylinder was placed over it and the top cap located. The seats for the O-ring seals for the cylinder were machined to provide accurate alignment of the cylinder, top cap and base. The top cap was held down by six 16 mm diameter bolts, three of which passed through spacer tubes machined to ensure that the top and bottom of the cell were parallel and to provide the required compression in the O-rings. A large C-spanner was used to hold the top of the cell while the bolts were tightened.

In all other respects, operation of the cell was the same as for the standard triaxial cell.

11.2.2 Cell Pressure Supply

A number of alternative sources of pressure supply were considered for the high pressure triaxial cells. These included the use of hydraulic regulator systems similar to those used by Golder and Akroyd (1954) and Jardine (1963). However, the maximum cell pressure of 2400 kPa was within the limits of the pressure available from high pressure air regulators and gas in cylinders. It was accordingly decided to use accumulators to pressurise the high pressure cells. Nitrogen was selected as the pressure source due to its low solubility and because it was already being used in the leaching and direct shear programs.

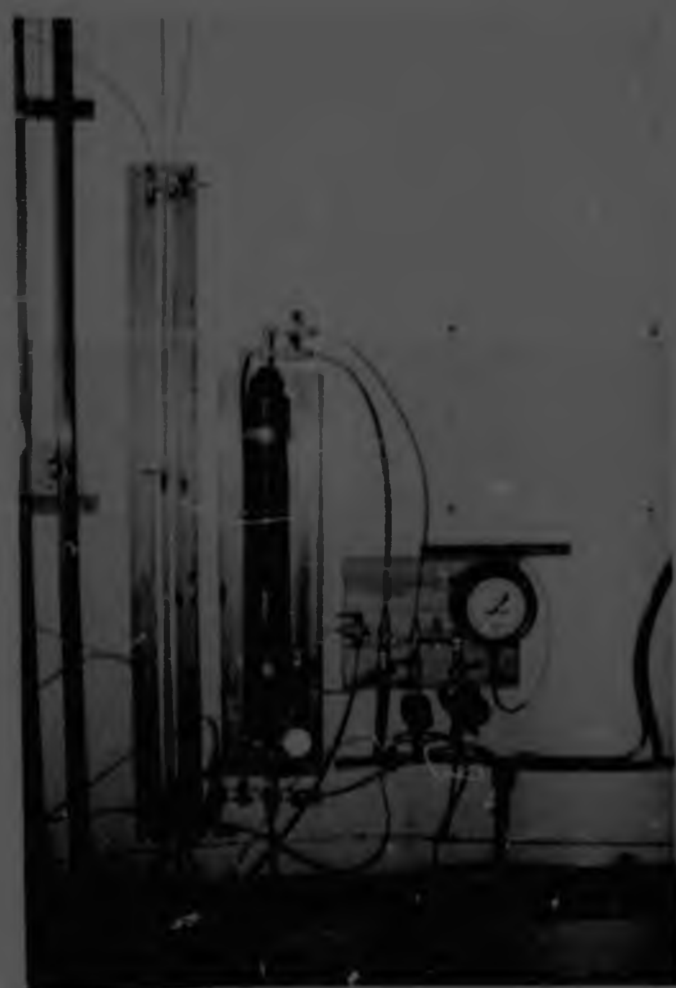
Plate 11.2 gives a general view of one of the high pressure accumulators and related equipment. The cylinder was assembled using standard high pressure steel steam pipe and fittings and coated internally with three layers of an epoxy coal-tar paint each cured at 30°C for twenty four hours. The soluble oil solution referred to earlier was stored in the accumulator and transferred to the cell for testing. A visual level gauge was connected to the cylinder which was positioned so that the fluid surface corresponded to the mid-height of the cell when the cell was full.

PLATE 11.2
HIGH PRESSURE ACCUMULATOR AND CONTROL PANEL

Valves at the bottom of the accumulator connected to the cell, the de-aired water supply and to waste. Valves at the top of the accumulator connected to the pressure supply (top valve in plates 11.2 and 11.3) and the vacuum supply (lower valve). The vacuum connection was used for rapidly draining the cell after testing referred to earlier was stored in the accumulator and transferred to the cell for testing. A visual level gauge was connected to the cylinder which was positioned so that the fluid surface corresponded to the mid-height of the cell when the cell was full.

PLATE 11.2

HIGH PRESSURE ACCUMULATOR AND CONTROL PANEL



Valves at the bottom of the accumulator connected to the cell, the de-aired water supply and to waste. Valves at the top of the accumulator connected to the pressure supply (top valve in plates 11.2 and 11.3) and the vacuum supply (lower valve). The vacuum connection was used for rapidly draining the cell after testing and for degassing the fluid in the accumulator after draining. The fluid was stored under vacuum between tests. The two-way valve on the left of the control board provided for connection of the vacuum to the top cap on the specimen to hold the specimen firmly during cell assembly and to aid in de-airing. This valve also connected the top drain to two additional valves which connected it either to waste or to the back pressure, volume change apparatus discussed in section 11.2.3, below.

Two regulators were provided for use where pressures under and over 1030 kPa were required. A precision high pressure gauge was connected to the high pressure regulator (on the right in plate 11.2) and the low pressure regulator was connected to a precision low pressure gauge on the original control board (not visible). A two-way valve on the low pressure regulator allowed connection to the compressed air supply when pressures under 500 kPa were required and connection to the nitrogen supply for pressures above this.

The existing mercury-pot pressure system was found to be unreliable at pressures in excess of about 500 kPa and was also relatively time-consuming to use. Four low pressure systems of similar design to the high pressure system described above were built. These were designed for a maximum cell pressure of 1030 kPa and the controls were identical to the low pressure side of the high pressure accumulators described above. The accumulators were fabricated from galvanised steel water pipe and fittings and coated internally with epoxy coal-tar paint. De-aired distilled water was used as the cell fluid.

Plate 11.3 illustrates a low pressure installation. The control board is on the right with the accumulator in the center.

PLATE 11.[illegible]

[illegible] ACCUMULATOR AND CONTROL PANEL

11.2.3 Volume Change Measurement

As there were insufficient conventional paraffin volume change gauges, an alternative system was devised using rigid nylon pressure tube with close dimensional tolerances. The volume change boards are visible to the left of the accumulators in plates 11.2 and 11.3. Tubes with external diameters of 1/8 and 3/8 inch (3.175 and 9.525 mm) were used, giving capacities of 4.2 and 39.4 ml/m. The tubes were mounted in parallel with ball valves to select the size required. Each tube was supplied with a millimeter scale and the larger tube was fitted with a sliding perspex pointer to eliminate parallax error. Potential error due to manufacturing tolerances and volumetric changes due to expansion under pressure were calculated to be of the order of 0.5% and accordingly no calibration was considered necessary.

Back pressure was applied by means of regulated compressed air monitored on a precision pressure gauge. The tubing connecting the volume change gauge to the specimen was flushed with de-aired distilled water during setting up. The length of the connecting tubing was approximately 2 m with a bore of 4.[illegible] mm and it was considered that diffusion of air into the specimen during the saturation and consolidation period was unlikely. A screw pump was provided in the water supply line to accommodate large volume changes.

11.3 TEST PROCEDURES

Sets of six specimens, or two sets of three specimens, were tested simultaneously using the procedure described below. De-aired distilled water, "water", was used in all steps discussed in the following section.

Sealed and humid cured specimens were soaked in water for a period of 24 hours and then placed under water in a desiccator. A vacuum was applied to the desiccator using a high-vacuum pump with gas ballast. The specimens were maintained under vacuum for a period of at least two hours, by which time no significant evolution of gas was occurring. The specimens were then transferred to the triaxial cells. This procedure was based on that used by Jardine (19[illegible]3).

Leached specimens were placed directly in the triaxial cells as it was considered that they were likely to be saturated. It was also considered that de-airing in a desiccator would lead to loss of material from the surface.

The side drains and porous disks were de-aired under vacuum and stored in de-aired distilled water when not in use.

A bead of water was run onto the pedestal, the specimens were wrapped in the side drain, placed in the inner membrane and placed on the porous disk on the pedestal, care being taken not to trap air. The second membrane was fitted together with two O-rings around the pedestal. Water was run through the bottom drain around the specimen to flush out air and the top porous disk and loading cap were then positioned. The specimen was again flushed, the top O-rings positioned and water flushed through the top drain. The top drain was then connected to the vacuum supply so that the specimen was firmly held on the pedestal. The cell was assembled and the loading ram brought into contact with the top of the specimen until the proving ring dial gauge registered a deflection of approximately 30 divisions. A pressure of 100 kPa was applied to the accumulator and the cell filled.

Water under the static head from the de-aired distilled water reservoirs (approximately 70 kPa) was then allowed to flush through the bottom drain, past the specimen and out the top drain under vacuum for a period of at least six minutes. Air bubbles in the connecting tube were dislodged by gentle tapping. The top drain was then switched from vacuum to waste. Flow to waste was permitted to flush the specimen for a further period of about six minutes.

The level in the 9.5 mm level tube was adjusted to the center of the specimen, and a pressure of 200 kPa applied. The cell pressure was increased to 250 kPa and the level tube connected to the top and bottom drains on the specimen to apply the saturating back pressure. Level readings were taken before and after connecting and at intervals thereafter. It was found that all significant volume change occurred within a period of 2 hours but the specimens were left to saturate under back pressure overnight for convenience.

A final level was then taken and the top and bottom drains isolated. The cell pressure was increased to 450 kPa and the pore pressure noted. Further readings were taken over a period of about one hour. In all cases, the pore pressure responded instantaneously but did not register the full increase in the cell pressure. Generally, no significant subsequent decay of pore pressure was observed. This was consistent with the findings of Abboud (1969) who reported values of Skemptons B parameter in the range of 0.5 to 0.7. Abboud investigated this phenomenon in some depth and concluded that it was consistent with the nature of soil-cement and that, provided no decay of pore pressure occurred, the specimen could be considered to be saturated.

The cell pressure was then increased to its final value and the pore pressure behaviour noted. The volume change gauge level was recorded, the top and bottom drains opened and a further reading taken. The specimens were allowed to consolidate against the back pressure of 200 kPa for a period of at least two hours. Generally, very little consolidation occurred after the first hour and total consolidation was small.

The specimen drains were then closed, zero load readings of load, deflection and pore pressure taken and the testing machine engaged. Specimens were loaded to failure with regular recording of time, load, deflection and pore pressure.

The cells were stripped, the mode of failure of the specimen described and sketched and the specimen oven dried to determine its final moisture content.

11.4 TEST RESULTS

Failure of specimens tested triaxially after leaching in the leaching cells was generally by way of plastic deformation of the inlet end of the specimen. The conclusions in chapter 9 indicate that leaching is a surface effect and this is confirmed by the unconfined compression tests on specimens cured in the leaching bath discussed in chapter 10. It was thus decided that the triaxial compression test data for specimens subject to leaching was of no real value and the data was accordingly discarded. Only the results for the sealed and humid cured specimens were processed and these are discussed in this section. The results for one set of three specimens which were tested n the leaching bath as part of the investigation into non-standard curing conditions were processed.

No investigation into the effect of moisture content at test or degree of hydration for individual sets was undertaken. It is considered that the investigation into these factors with regard to unconfined compressive strength, presented in chapter 10, provides sufficient insight into these factors and gives an indication of possible causes of observed scatter in the results discussed here.

11.4.1 Stress - Strain and Pore Pressure - Strain

The stress - strain characteristics, Mohr envelopes and P-Q diagrams for soil-cement have been examined in detail by Jardine (1963), El-Rawi (1967), Abboud (1969) and others. Jardine and Abboud also investigated the pore pressure - strain characteristics of the material. The stress - strain characteristics, Mohr envelopes and P-Q diagrams for dry lean concrete were examined by Price (1977).

In view of the number of available references and in order to save time, it was decided not to produce stress - strain - pore pressure diagrams for all tests. A selection of representative plots are presented in graphs 11.1 and 11.2. Deviator stress is plotted against strain in the upper part of the graphs and pore pressure in the lower part. The same vertical scale is used for both stress and pore pressure to facilitate comparison. Strain scale limit is 6% and stresses are calculated in terms of nominal initial area as the strain at failure was small in most cases.

Graph 11.1 depicts the data for a complete set of six specimens compa cement content of 10% and standard compaction conditions, humid cured for 7 days and sealed until testing at age 353 days. This set of data is considered to be representative of most of the tests conducted as part of this program. The shape of these curves is generally similar to those presented in the references cited above. The rounded peaks were associated with a smooth shear failure of the specimen whereas the specimen tested at a confining pressure of 0.30 MPa experienced a sudden brittle failure.

It will be noted that the results exhibit a trend to increasing strain at failure with increasing confining pressure, and hence deviator stress, suggesting that the secant modulus at failure remains reasonably constant. The results for the specimen subject to a confining pressure of 0.30 MPa do not agree with this trend, and exhibit a steeper slope and brittle failure at a lower strain. This anomaly of apparently greater stiffness at low confining pressure was observed with most of the test results and may be due to the low pressure. However, a number of problems were experienced with deflection measurements on the apparatus used for tests at this confining pressure in the early stages of the test program. In view of this, the deflection related data for specimens subject to this confining pressure have been omitted in certain of the sections that follow.

It is particularly notable that the peak pore pressure occurs well before peak deviator stress in most cases and that pore pressure at failure is generally insignificant. In addition, maximum pore pressure is generally less than 10% of maximum deviator stress. Pore pressure at failure as a percentage of maximum principal effective stress is presented in table 11.1. With the exception of the unstabilized specimens, this ratio did not exceed 3% and was generally less than 1% except at low confining pressures. A similar relationship is evident in most of the data presented in the references cited previously.

GRAPH 11.1

DEVIATOR STRESS AND PORE PRESSURE vs STRAIN

FOR SPECIMENS WITH 10 CEMENT TESTED AT AGE 353

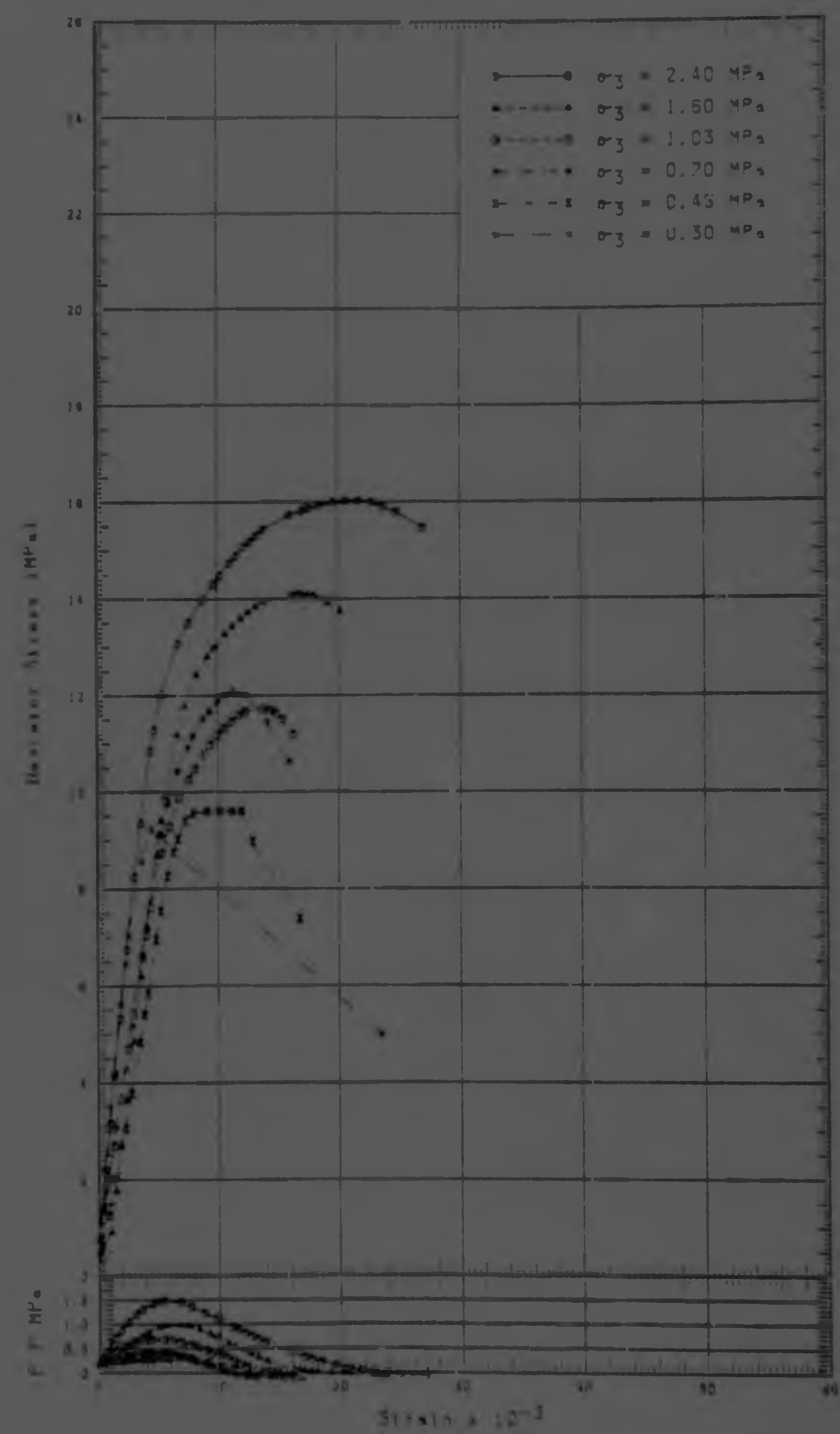


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TABLE 11.1
SUMMARY OF RESULTS -- TRIAXIAL COMPRESSION TESTS

Code	Specimen Description (Nominal curing periods)	No in set	Cement (%)	Age (days)	UCS (MPa)	C′ (MPa)	Phi′ (deg)	Reg'n Coeff (r^2)	Average Pore P (% σ_1)	Average Modulus (GPa)
A	Standard Cure:-	6	0.0	337	0.290	0.071	27.01	0.99	38.41	0.088
B	7 days humid,		5.0	293	5.229	1.056	36.49	0.98	0.62	0.815
C	287 days sealed.		10.0	295	10.233	2.247	37.27	0.99	0.83	1.768
D				320	10.383	2.466	36.01	0.99	0.63	2.040
E			15.0	297	15.250	3.251	39.18	1.00	0.88	2.504
F			20.0	290	20.166	4.439	34.71	0.97	0.77	2.541
G	7d Humid, 343d sealed	6	10.0	353	10.563	2.242	37.47	1.00	0.90	2.042
H	924d sealed			929	12.343	2.621	37.70	1.00	1.04	1.807
I	924d sealed			931	12.347	2.803	36.61	0.99	0.81	2.330
J	28d Humid, 266d sealed	3	10.0	291	N/A	1.962	40.94	1.00	1.33	2.169
K	Full period humid cure					1.537	45.40	0.95	0.42	1.562
L	7d Humid, 21d leach bath			292		1.789	42.83	1.00	1.65	1.619
M	Leach bath from 7 days					1.605	36.31	0.97	1.57	1.484
N	1 series W/D, humid cure	6				2.281	36.13	1.00	0.94	1.581
O	sealed cure			294		2.063	34.65	1.00	0.64	1.458
P	Compacted 1% dry of OMC	6	10.0	292	11.090	2.041	38.33	0.98	0.37	1.960
Q	1% wet of OMC			291	8.841	1.666	39.92	0.99	-0.32	1.167
R	2% above MDD			290	10.965	2.351	38.50	0.99	0.31	1.909
S	Microconcrete, c/w 1.0	6	18.2	291	14.731	2.651	40.48	1.00	1.67	2.593
T	c/w 1.2		22.0	293	21.449	3.238	47.11	0.95	1.09	3.703

In view of the time required to saturate and consolidate specimens and conduct tests with pore pressure measurement, it is considered that the use of "quick" triaxial tests on unsaturated specimens, as used by El-Rawi, should be adopted for routine testing with a limited number of consolidated undrained tests with pore pressure measurement being made initially to confirm the above relationship for a particular soil. It is further considered that the results of triaxial compression tests on soil-cement could be reported in terms of total stresses without significant error. However, as the necessary data was available, the data in this chapter is presented in terms of effective stresses.

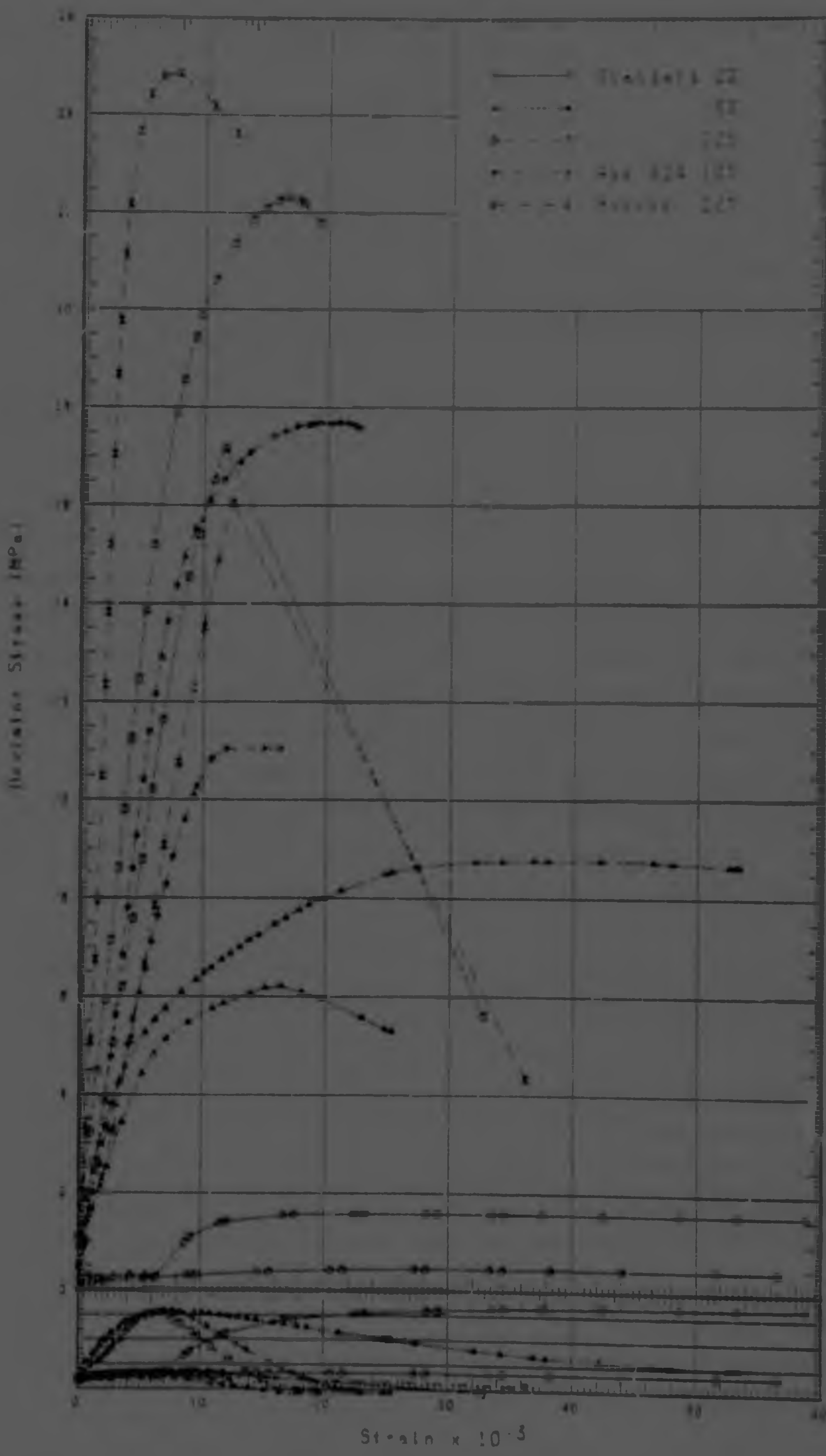
Graph 11.2 depicts the deviator stress - strain and pore pressure - strain curves at confining pressures of 2.4 and 0.45 MPa for a number of other test conditions. This data is plotted to provide an indication of the general trend of results. The lower limit confining pressure of 0.45 is used in preference to that of 0.30 for the reasons referred to above.

The data plotted in graph 11.2 is for unstabilized specimens, standard soil-cement specimens with 5 and 20% cement and microconcrete specimens containing 22% cement, all tested at a nominal age of 2?4 days together with specimens with 1?% cement tested at age 930 days. The general trend is similar to that described above with strain at failure decreasing with increasing cement content and hence increasing strength.

It is notable that the specimens with 5% cement failed plastically at the highest confining pressure as evidenced by the nearly horizontal portion of the stress - strain curve that results from the increasing cross-sectional area of the specimen. The unstabilized specimens also exhibit plastic failure. Sudden brittle failure occurred at the confining pressure of 0.45 MPa in the case of the microconcrete and 20% soil-cement.

GRAPH 11.2

DEVIATOR STRESS AND PORE PRESSURE v STRAIN AT
CELL PRESSURES OF 2.4 AND 0.45 MPa FOR VARIOUS CONDITIONS



The maximum pore pressure is of similar magnitude for all stabilized specimens tested at the same confining pressure and occurs at similar strain. Pore pressure decay however, occurs more rapidly as cement content increases. In many cases, negative pore pressures developed at, or shortly after, peak deviator stress. Pore pressure in the unstabilized specimens develops more slowly and reaches a similar magnitude to the maximum stress some time after the maximum stress has been reached confirming the plastic nature of the failure. The microconcrete specimens exhibit very low pore pressure development as a result of the greater stiffness of the material.

11.4.2 Shear Strength Characteristics

As mentioned above, the shear strength characteristics of soil-cement have been investigated in detail by other workers in this field, accordingly, it was not considered necessary to plot Mohr envelopes. P-Q diagrams are presented in graphs 11.3 to 11.5 and the data is summarised in table 11.1. This table also summarises the properties of the test specimens. The test specimens generally correspond to other sets tested in the leaching program and in many cases to sets tested in unconfined compression. Table 11.1 also lists the character codes used in graphs 11.3 to 11.9 to designate the individual sets.

Price (1977) found that the P-Q diagram for dry lean concrete could be approximated by a straight line and Lambe and Whitman (1969) report a similar observation for soils. Inspection of the data in Abboud (1969), El-Rawi (1967) and Jardine (1963) indicated that a similar approximation could be used for soil-cement, over the range of confining pressures used, without significant error. In addition, the limited number of data points, a maximum of six, obtained in these tests, did not provide sufficient resolution to attempt to fit curved envelopes, as suggested by Abboud, or two stage linear envelopes, as suggested by El-Rawi. A linear regression was therefore applied to the P-Q data points and these lines are reflected together with the data points in graphs 11.3 to 11.5.

The regression coefficients (r^2) are summarised in table 11.1 from which it will be seen that a good fit was generally obtained. All data points for each set were included in these regressions. Examination of the individual data points indicates a tendency for the slope to decrease as stresses increase although this is not pronounced, as indicated by the regression coefficient. While this has little bearing on the results in general, it does indicate that the P-Q lines for the three specimen sets, which were conducted at the three lowest cell pressures, are not directly comparable with those for the six specimen sets.

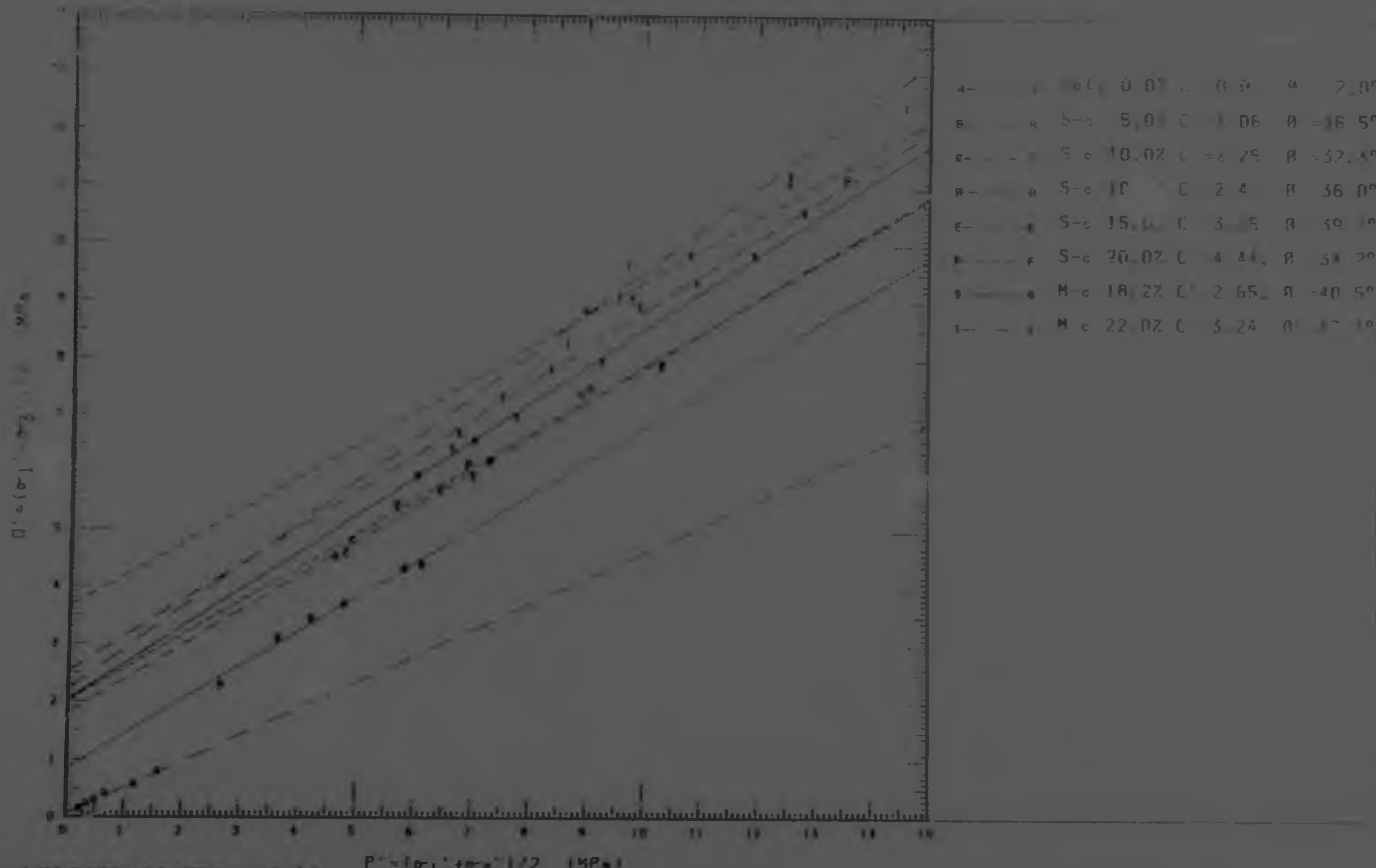
The parameters P' and Q used in these graphs are calculated in terms of the major and minor effective principal stresses. From the linear regressions on these points the effective stress shear strength parameters of cohesion (C') and friction (Ø') were computed using the normal formulae (Lambe and Whitman 1969). These values are presented on each graph to facilitate interpretation and are summarised in table 11.1.

Graph 11.3 presents the P-Q data for the standard soil-cement specimens with cement contents of 0, 5, 10, 15 and 20% and the microconcrete specimens, all tested at an age of approximately 29 days. A progressive increase in cohesion with increasing cement content is evident and a pronounced increase in slope occurs from 0 to 5% cement. Thereafter, no particular trend in slope with increasing cement content is apparent for soil-cement, while an increase occurs with the microconcrete specimens.

Graph 11.4 presents the P-Q data for the specimens subject to extended cures of from approximately 1 to 2.5 years together with those subject to compaction 1% wet and dry of optimum moisture content and those compacted to 2% above standard proctor maximum dry density. The data for the standard 10% set, C, is replotted on this graph with a solid line for reference purposes. A slight

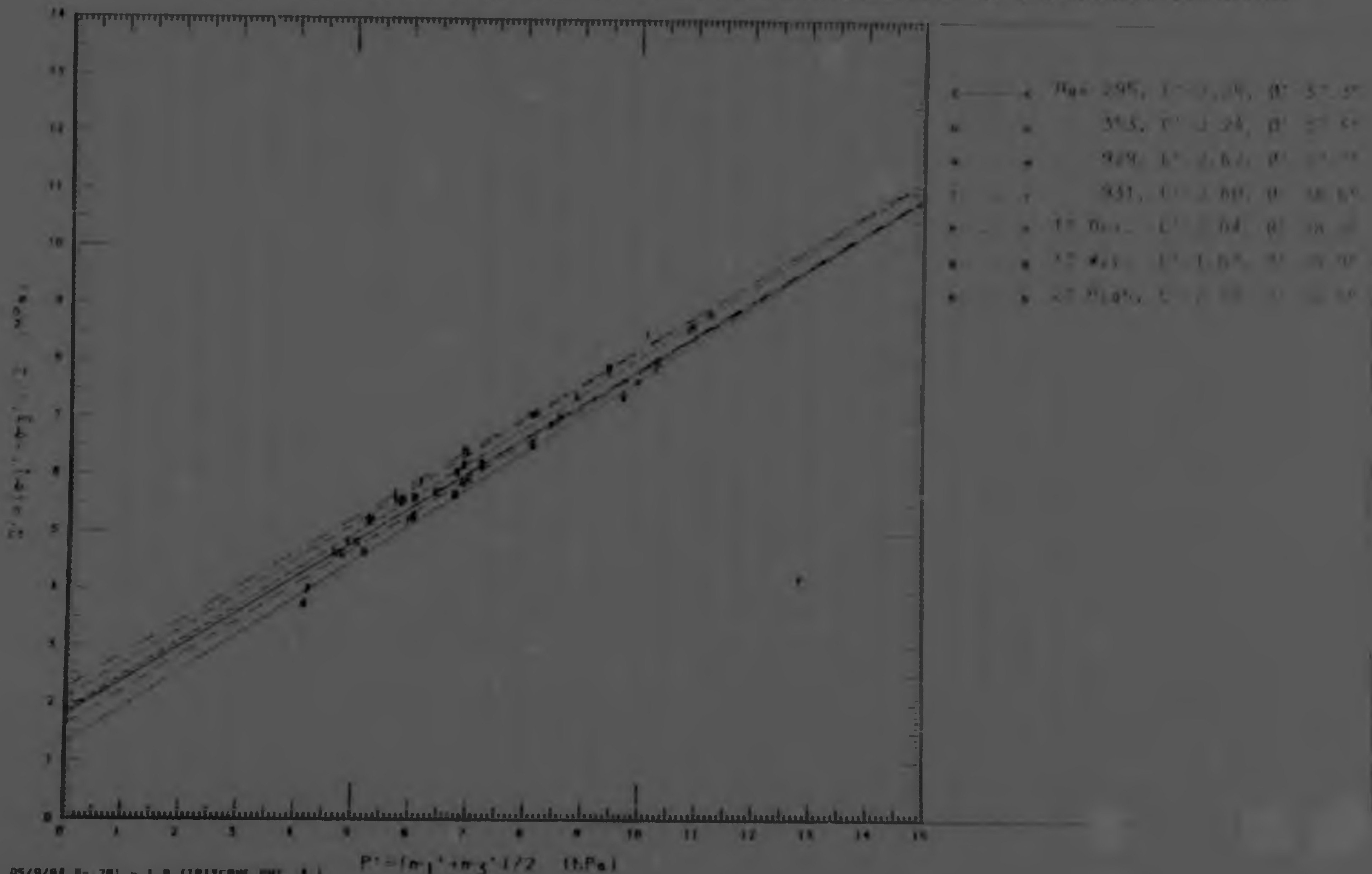
GRAPH II.5

P-Q DIAGRAM FOR STANDARD SOIL CEMENT AND MICROCONCRETE SPECIMENS



GRAPH 11.4

P-Q DIAGRAM FOR SPECIMENS WITH IDS CEMENT SUBJECT TO EXTENDED CURE AND NON-STANDARD COMPACTION



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increase in cohesion is evident with extended cure and a possible slight decrease for specimens compacted wet of optimum. There is no indication of any significant variation in slope and the data is closely distributed about that for the reference set.

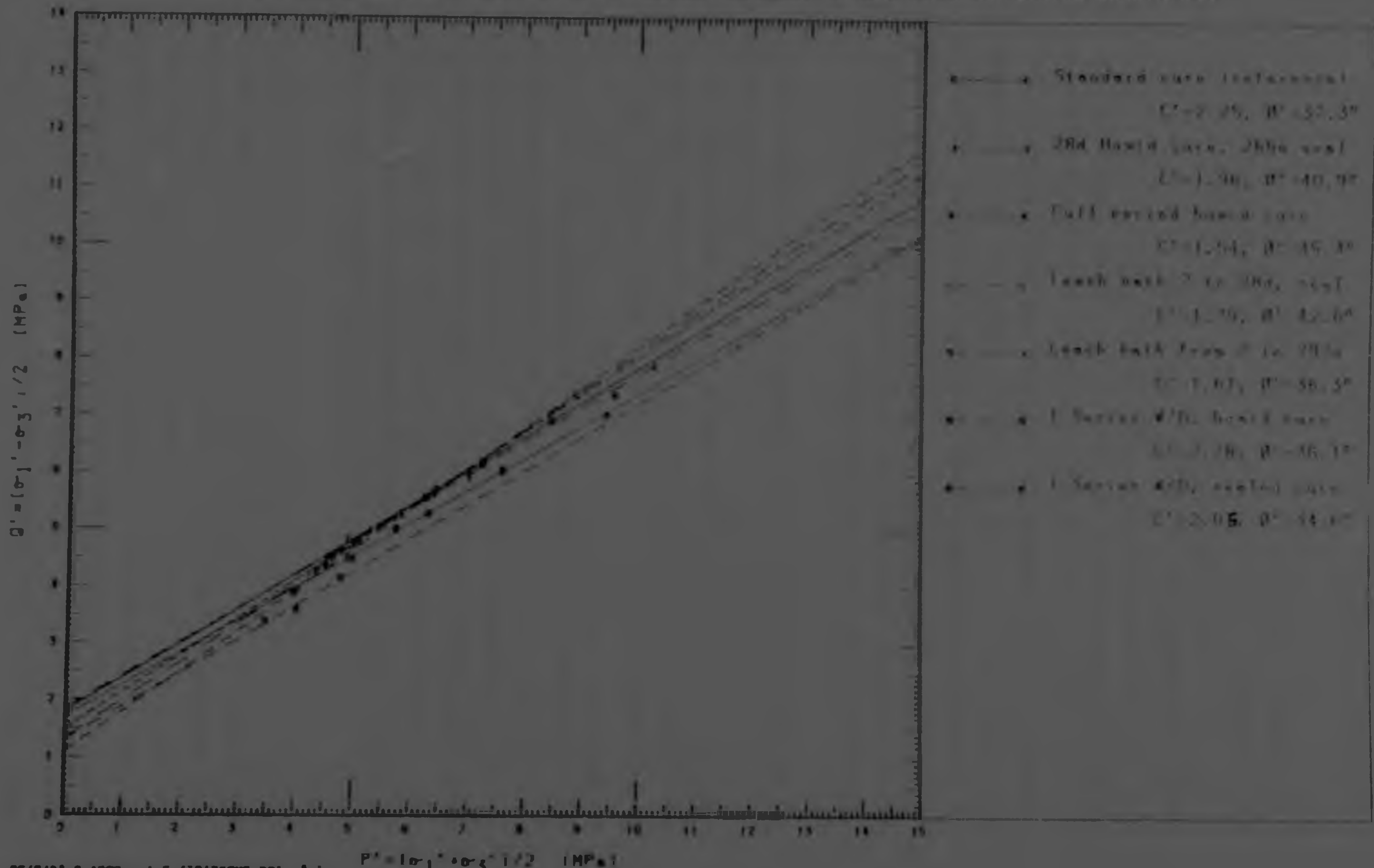
Graph 11.5 presents the P-Q data for the specimens subject to non-standard curing conditions together with that for the standard 10% set (C) for reference. Some variation in cohesion is again apparent. It is notable that of the four three specimen sets, J, K, L and M, the first three have appreciably higher estimated values of friction angle. This is considered to reflect the curvature referred to above and in view of the limited number of specimens, it is considered that this does not represent a significant deviation. It is however significant that the fourth set, that placed in the leaching bath from seven days, exhibits an angle of friction considerably less than the average for all sets. While this appears to indicate that leaching reduces the angle of friction, which may be the case in those zones from which virtually all stabilizer has been removed, it should be noted that calculation of stresses in terms of the effective cross-section could alter the slope. The effective length of the specimen will also be reduced. As this effect is not considered to have any practical significance, it was not investigated further.

11.4.3 Elastic Modulus

In section 11.4.1 it was noted that the secant modulus from start of loading to maximum deviator stress appeared to remain reasonably constant with increasing confining pressure. Inspection of graphs 11.1 and 11.2 also indicates that in most cases, the slope of the linear portion of the stress - strain curve tends to increase with increasing confining pressure, a situation that must occur if the first observation is correct.

GRAPH 11.5

P-Q DIAGRAM FOR SPECIMENS WITH 10% CEMENT SUBJECT TO NON-STANDARD CURING CONDITIONS



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In order to obtain some indication of the elastic modulus for comparative purposes, a variety of options were considered. In view of the shortage of time and the decision not to produce a complete set of stress - strain curves, it was decided not to use the hyperbolic representation, developed by Duncan and Chan and used by Abboud (1969), to estimate the initial tangent modulus. Consideration of the stress - strain curves in graphs 11.1 and 11.2, together with those in the references cited previously, indicated that the slope of the stress - strain curve was generally linear to about 70% of the maximum deviator stress. It was therefore decided that the slope of this portion of the curve would provide a reasonable value for comparative purposes as well as providing a reasonable estimate of the initial tangent modulus.

It was further decided to calculate the slope on an incremental basis thus eliminating the need for zero corrections of deflection. After consideration of the data and a few trial calculations, the average slope at a point corresponding to half the deviator stress at failure was adopted. This slope was determined as the average of five slope increments centered on the appropriate stress. Determination of the average tangent modulus at one third of the stress at failure was also considered as this was more consistent with a factor of safety of 3 on compression. However, it was found that, in many cases, there were insufficient data points before the third point to permit determination of the moving average and inspection indicated that the slope at the two points did not differ significantly.

Since half the deviator stress at failure is equal to the parameter Q, the resulting modulus has been designated the "Q-modulus" to differentiate it from other moduli which are in general use. Trial calculations indicated that the Q-modulus is of similar magnitude to both the secant modulus at failure and the initial tangent modulus determined by conventional means. The Q-modulus was found to exhibit considerably less variation and more consistent trends than the other two moduli for the cases considered.

The data was processed using a computer spreadsheet program as noted in appendix A. In processing the data for the complete stress - strain curves, it was found that calculation of the incremental modulus (slope) together with its five-point moving average provided a useful numerical technique for correcting zero error and validating data without the need to plot the results. It is suggested that this technique could be of value in processing the output of many types of experimental ,)gram.

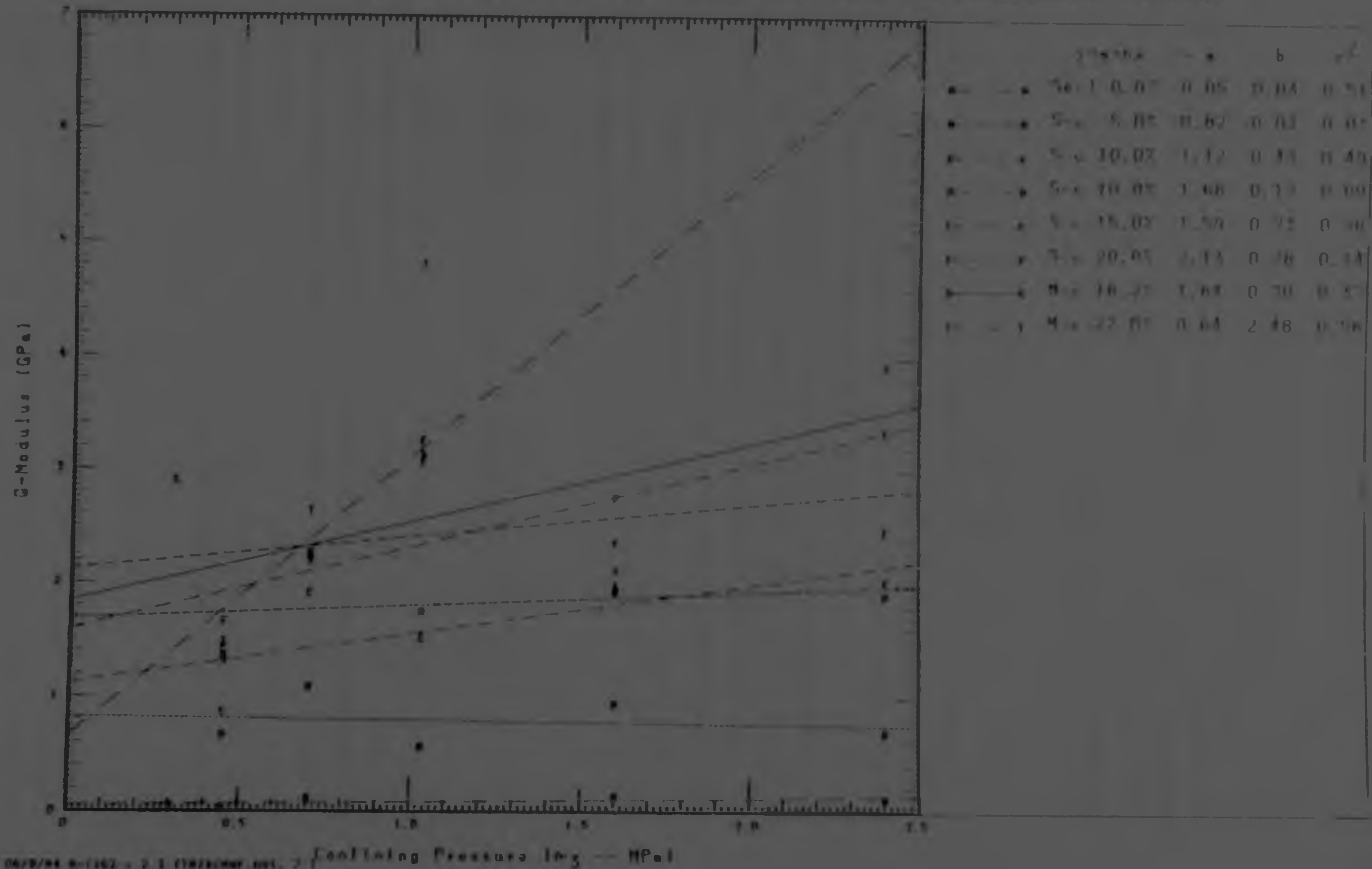
As noted above, a tendency for the slope of the stress - strain curve to increase with increasing confining pressure is apparent. Abboud (1969) reported a linear relationship between the logarithm of initial tangent modulus and logarithm of confining pressure. As noted in section 11.1, this trend is not entirely consistent for the present data, particularly at low confining pressures.

Graph 11.6 depicts the Q-modulus data for the standard specimens plotted against confining pressure. In view of the reservations expressed above concerning the deflection data for the specimens tested at a confining pressure of 0.3 MPa, this data has been omitted for most of the sets plotted. While in most cases there is a tendency for modulus to increase with confining pressure, not all sets agree with this trend and a considerable amount of scatter is evident as indicated by the very poor regression coefficients tabulated in the legend. The nature of the scatter is such that no significant improvement in fit was obtained using regressions of log(modulus) vs log(confining pressure) as suggested by Abboud (1969).

Low regression coefficients were observed for all sets and accordingly Q-modulus vs confining pressure plots were not prepared for the remaining data. In view of the observed scatter and poor correlation, it was concluded that rejection of the modulus value at the 0.3 MPa confining pressure was not warranted. The use of the average modulus for each set, including the low confining pressure data, was therefore adopted to provide a value for comparative purposes and is presented in table 11.1.

GRAPH II.6

G-MODULUS vs CONFINING PRESSURE FOR STANDARD SOIL CEMENT AND MICROCONCRETE SPECIMENS



This summary data is plotted against cement content on a log scale in graph 11.7 from which it can be seen that a reasonably close relationship exists for the standard specimens containing 10% cement and tested at an age of 294 days (r^2=0.94). It is notable that the microconcrete specimens containing 18.2% cement lie on the same line and that the specimen with 22% cement is reasonably close.

11.4.4 Relationship of Cohesion to Cement Content

The effective cohesion data in table 11.1 is plotted against cement content in graph 11.8 from which it can be seen that a close relationship exists for the standard soil-cement specimens at age 294 days (r^2=0.995). Cohesion increases with increased curing period (H and I) and decreases with compaction wet of optimum (Q). The low values for the points J, K and L reflect the steeper slopes for the the three specimen sets referred to above while that for the specimen placed in the leaching bath from seven days reflects the reduced cross-section referred to in chapter 10.

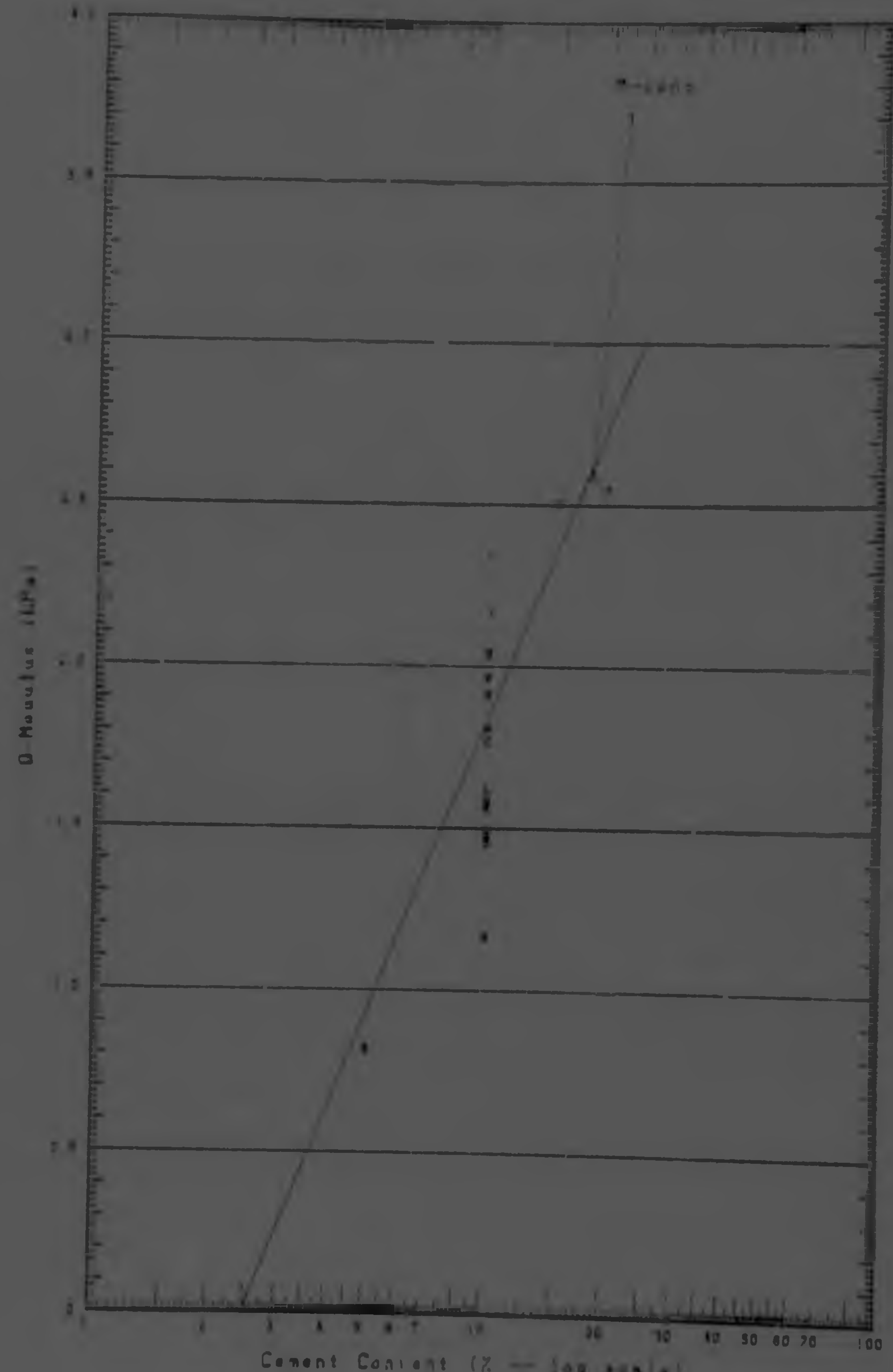
11.4.5 Effective Angle of Internal Friction

As noted in section 11.4.2, the angle of internal friction increases substantially for an initial addition of 5% cement but thereafter is independent of cement content. This agrees with the data presented by Nussbaum and Colley (1971) which reflects an increase in angle of friction at low treatment levels but no particular trend at levels above about 5% depending on soil type. Most of the increase observed by Nussbaum and Colley occurred with the first one or two percent cement addition. A plot of the data in table 11.1 indicates similar behaviour.

The average angle of friction for the standard soil-cement specimens in table 11.1 (B to I) is 36.9° with a standard deviation

GRAPH 11.7

Q-MODULUS vs CEMENT CONTENT



——— Regr'n A,B,C,D,E,F: y=1.18+0.307 log x : r^2=0.94

For individual codes refer to table 11.1.

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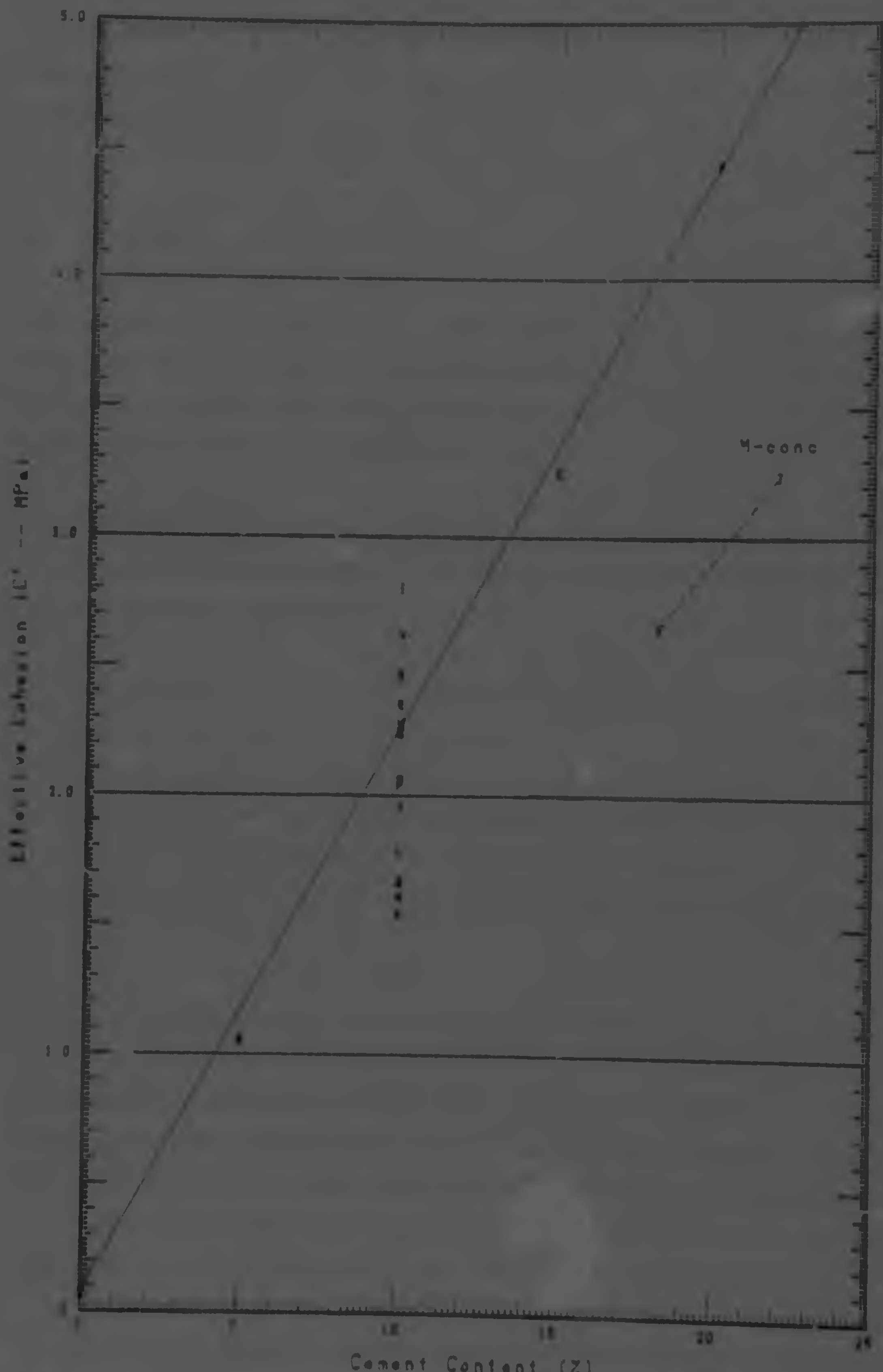
GRAPH 11.8

EFFECTIVE COHESION vs CEMENT CONTENT



—— Regr'n A,B,C,D,E,F : y=0.089+0.219 x : r^2=0.995

For individual codes refer to table 11.1.

TS C-CEM PLT -- 08/8/84-8-1284-- | S :TRIXCONF DAT. 10 |

of 1.3 while that for all soil-cement specimens in table 11.1 is 38.1° with a standard deviation of 2.8. An angle of internal friction of 37° is therefore considered to be a reasonably representative value for design with this soil and cement contents above 5%.

11.4.6 Effect of Age at Test

The data in table 11.1 was plotted against age at test on linear and log scales and regressions performed. As there were only five data points and no data was available at early ages, the following observations must be regarded as tentative:-

a. The angle of internal friction is independent of age in the range considered, this is consistent with the observations in the previous section.

b. The effective cohesion increases with age as observed in section 11.4.4.

c. The Q-modulus may increase slightly with extended curing but the scatter in the data is such that any increase cannot be considered to be of any significance for design purposes.

11.4.7 Cohesion and Unconfined Compressive Strength

Aboud (1969), Mitchell (1976d) and others have indicated a close relationship between cohesion and unconfined compressive strength and this relationship was theoretically defined in section 4.3 of the paper reproduced in chapter 3. This relationship is also suggested by the observations in the previous sections.

The unconfined compressive strength for the standard soil-cement specimens was determined from the data presented in graph 10.9 and that for the non-standard specimens and microconcrete from the individual test results for corresponding sets. No data was available for specimens subject to non-standard cures. This data is presented in table 11.1.

The effective cohesion is plotted against unconfined compressive strength in graph 11.9 from which it will be seen that good agreement exists. This plot indicates a reasonable basis to account for different curing ages, treatment levels, etc. In addition, the points for the microconcrete specimens lie closer to the line for soil-cement than they do in graph 11.8. The regression intercept of -0.30 cannot be considered to differ significantly from zero so the relation can be expressed as:-

Cohesion = 0.221 x Unconfined Compressive Strength

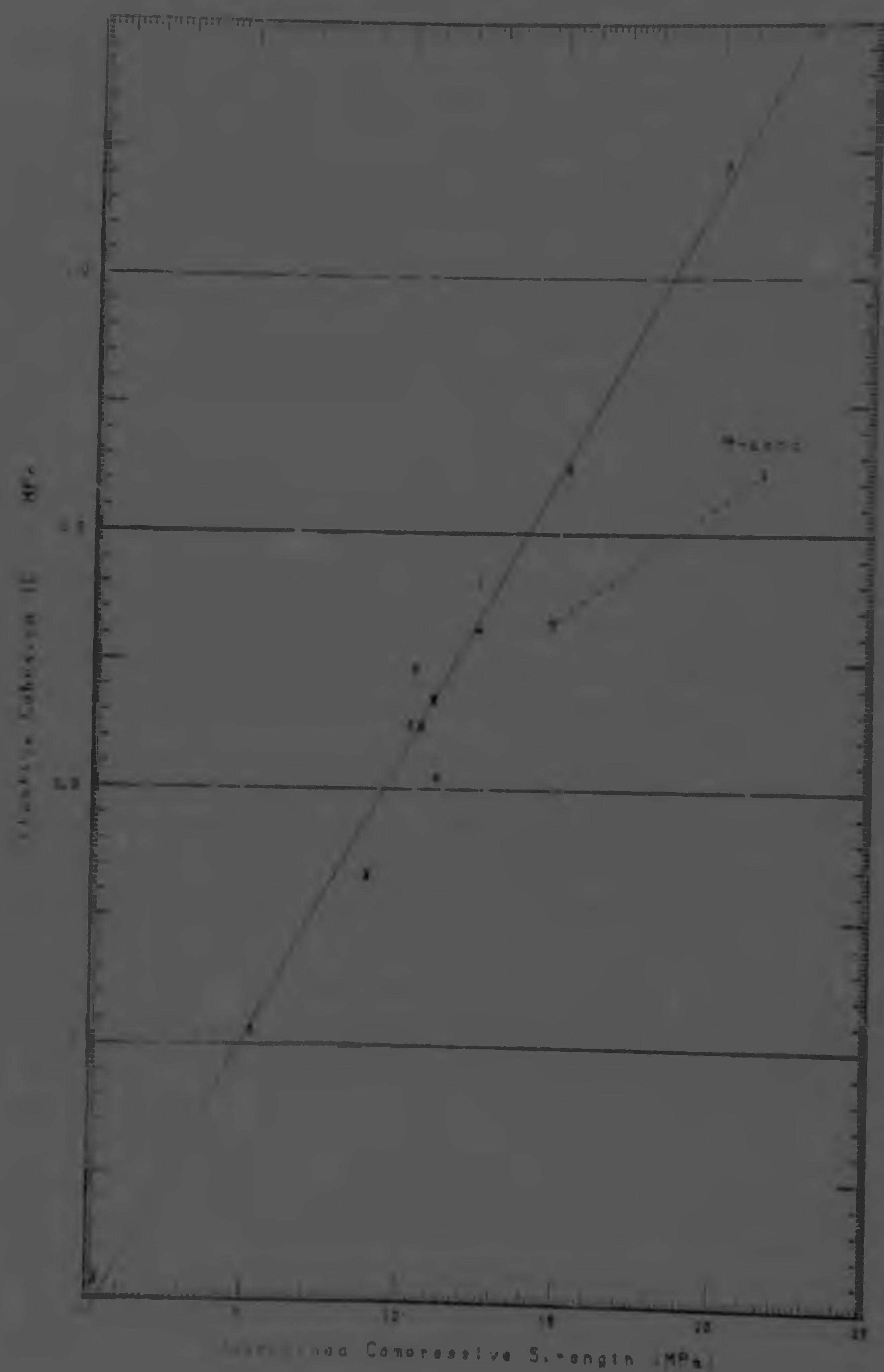
Applying the value of 37° proposed for the angle of internal friction in section 11.4.5 to equation 1, in the paper in chapter 3, yields a value of 0.249 which compares favourably with the value of 0.221 above. It is therefore concluded that the unconfined compressive strength is a good measure of the effective cohesion of soil-cement once a reasonable estimate of the angle of friction has been obtained. In particular, since the angle of friction does not change appreciably above a certain minimum cement content, the unconfined compressive strength provides a reasonable measure of the effect of variations in curing and other conditions on the shear strength of soil-cement.

It should be noted that the theoretical equation overestimates the effective cohesion by approximately 11% in this case. This can probably be considered to be within experimental error but further research into this aspect is required. This discrepancy can be compensated for by modifying the constant factor in equation 1 from 0.5 to 0.445 to give:-

$$C' = 0.445\, \sigma'_u\, [\cos \phi' - \tan \phi' (1 - \sin \phi')]$$

GRAPH 11.9

EFFECTIVE COHESION vs UNCONFINED COMPRESSIVE STRENGTH



— ... ; $y=0.080+0.221x$; $r^2=0.980$

... table 11.1

11.4.8 Q-Modulus and Unconfined Compressive Strength

Abboud (1969) reports a linear relationship between the logarithm of modulus in compression and the logarithm of unconfined compressive strength. Graph 11.10 presents the Q-modulus plotted against unconfined compressive strength, both on log scales. A strong correlation is evident for all data points.

It is particularly notable that the results for the microconcrete (points S and T, joined by the dashed line) are effectively identical to those for the soil-cement specimens.

11.5 CONCLUSIONS

Analysis of the triaxial test data for specimens subject to leaching in either the leaching bath or leaching cells was not undertaken as the results of the analysis in chapters 9 and 10 indicated that this data would be of no real value.

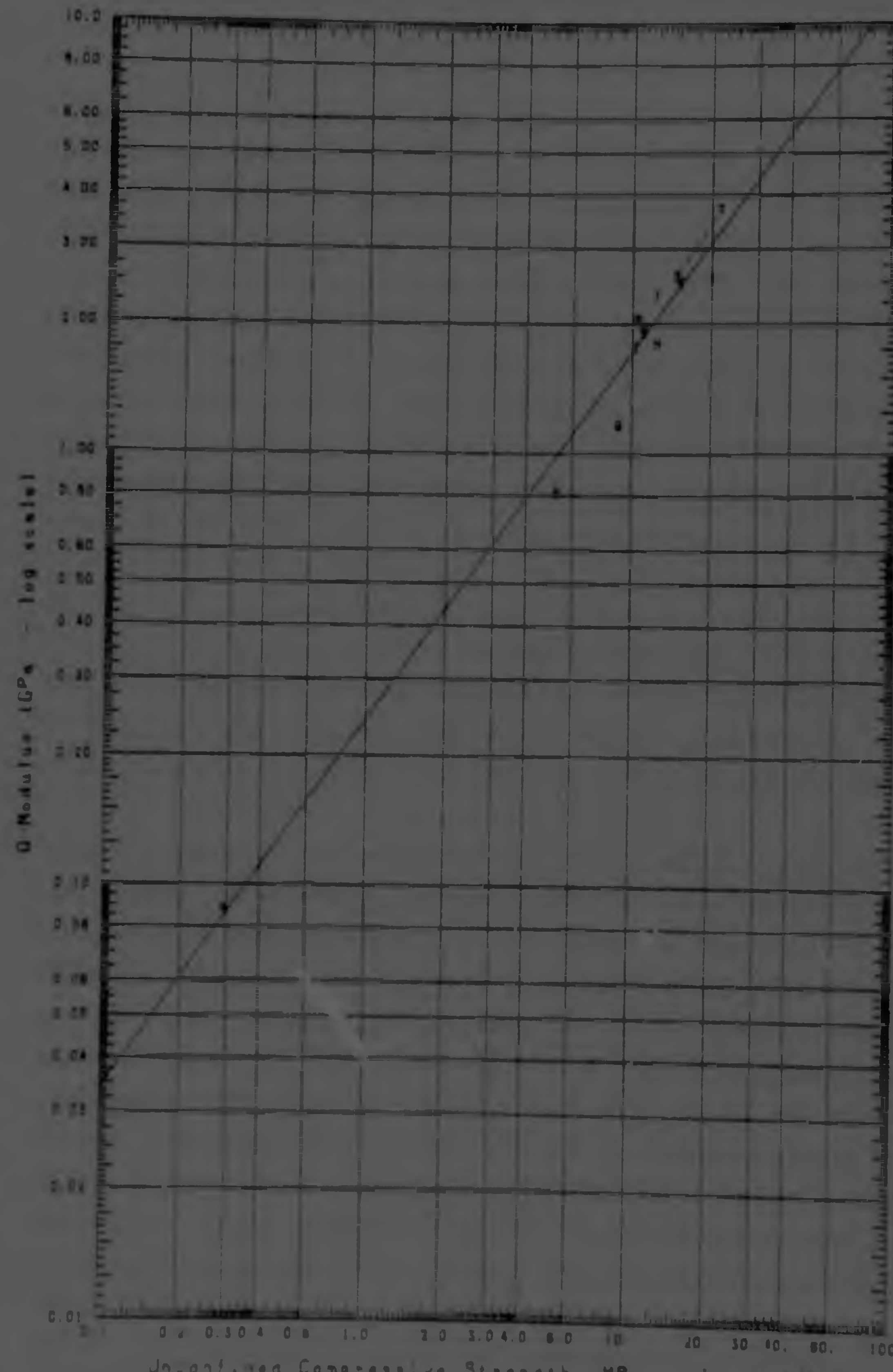
A detailed analysis of the stress - strain and pore pressure - strain characteristics was not undertaken due to the large amount of existing data in various references. Sample plots for this data exhibit similar characteristics to those previously reported.

Stabilized specimens were found to fail smoothly in shear at high confining pressures and exhibit brittle failure at low pressures. Specimens with low cement contents exhibited plastic failure at high pressures.

The peak pore pressure was found to occur well before failure for all stabilized and microconcrete specimens. Pore pressure at failure was less than 3% of maximum principal stress for all stabilized specimens. It is therefore concluded that the use of unconsolidated undrained triaxial tests on unsaturated specimens can be adopted as the standard triaxial test technique for most stabilized soils. Initial trial consolidated undrained tests with

GRAPH 11.10

Q-MODULUS vs UNCONFINED COMPRESSIVE STRENGTH



Regr'n : log y = -0.619 + 0.652 log x ; r^2=0.976

For individual codes refer to table 11.1

TSMODULS ... -- 13/9/84 ... STRIK... PNT ...

pore pressure measurement on saturated specimens should, however, be conducted to confirm this for a particular soil type and in the case of cement contents below 5%.

Owing to the limited number of specimens tested, six in each set, it was not possible to fit the curved or two stage failure envelopes proposed by Abboud (1969) and El-Rawi (1967). Only P-Q diagrams were prepared for the data and the envelopes obtained by linear regression on all data points. High values of regression coefficient were obtained in all cases and it is concluded that this approximation is acceptable over the range of confining pressures used in this program. A slight curvature was however apparent and led to a higher estimated angle of internal friction for those sets containing only three specimens as these were tested at the three lowest confining pressures.

The angle of internal friction was found to increase from 27° to 37° with the addition of 5% cement to the soil, thereafter no further increase was noted. This value is apparently unaffected by age over the interval considered and no significant change with changes in curing and compaction conditions was observed.

Cohesion was found to increase with cement content and age at test and a linear relationship between unconfined compressive strength and cohesion was confirmed. On the basis of these results, the theoretical relationship proposed by Robertson and Blight (1978), in the paper reproduced in chapter 3, is modified by the introduction of an additional factor of 0.89 thus reducing the constant in the equation from 0.5 to 0.445. Given the constant nature of the angle of internal friction for cement contents above 5%, this provides a reasonable basis for relating the cohesion determined in the triaxial test to the unconfined compressive strength. It is therefore concluded that the unconfined compressive strength provides an acceptable means of investigating the effect of variations in curing conditions etc on the shear strength of soil-cement.

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An elastic modulus for comparative purposes was defined as the average of five incremental modulus values centered on a value of half the deviator stress at failure. Since this value corresponds to the value Q associated with the standard P-Q diagram, it is referred to as the "Q-modulus". This modulus was found to be of similar magnitude to both the secant modulus at failure and the initial tangent modulus and was found to be less variable than these parameters for the data investigated.

No particular relationship between Q-modulus and confining pressure was determined and the average modulus for a set was thus adopted as the representative value. The logarithm of this value was found to increase linearly with the logarithm of the unconfined compressive strength. The results for the micro-concrete specimens were found to be coincident with those of the soil-cement.

Calculation of the incremental slope (modulus) during data processing was found to be a valuable means of correcting zero error in deflection readings and validating data. This technique is recommended for any application involving the processing of large amounts of manually recorded experimental data.

It is strongly recommended that further work in this field should make maximum use of unconfined compression tests as a quick and reliable means of evaluating the effect of changes in specimen properties on the shear strength characteristics of soil-cement. These tests should be supported by initial consolidated undrained triaxial tests on saturated specimens to confirm the pore pressure relationship referred to here. Thereafter, unconsolidated undrained triaxial tests on unsaturated specimens should be used where additional triaxial testing is required to establish the relationship between cohesion and unconfined compressive strength.

CHAPTER 12

DIRECT SHEAR TESTS

12.1 INTRODUCTION

The objectives of the direct shear tests were as follows:-

a. To simulate as closely as possible sliding failure on a horizontal or near horizontal plane which is one of the major potential failure situations. The direct shear test simulates this condition better than any other.

b. To conduct at least one series of strength tests on specimens produced with soil containing the coarse fraction (+4,75 mm) excluded from the soil used in other tests. The direct shear equipment permitted the testing of specimens with a maximum particle size of 19 mm (3/4") which represented 86% of the total soil as compared to 62% for the minus 4.75 mm fraction.

c. To examine the strength of the material on construction joints on a reasonably large specimen as previous research (De Groot 1976) was confined to specimens 100 mm square and 50 mm deep tested without application of vertical load.

The test program was divided into two stages, the determination of the properties of the intact material and the examination of the relative merits of a number of construction joint treatments.

The standard cement content of 10% was used together with 5% and 20% for the tests on the intact material. As other test programs had been restricted to material passing the 4,75 mm sieve, it was decided to work in terms of cement contents expressed as a percentage of this fraction for these tests. This was justified on the basis that it is the soil mortar which, when mixed with cement, provides a stabilized matrix containing the coarser particles and that the "concentration" of cement in the mixture is a function of the specific surface. The specific surface area of the fraction retained on the 4.75 mm sieve was estimated at 0.0167 m^2/m^3, or 0.014% of the area of the fraction passing the 4.75 mm sieve, on the basis of data in Lambe and Whitman (1969). The cement contents as a percentage of total dry soil mass were 3.2, 6.4 and 12.8% respectively.

Age at test was 28 days to be consistent with the rest of the program. Intact specimens were sealed immediately after compaction and cured at 25 °C.

The test conditions for the jointed specimens were basically the same as for the intact specimens. All specimens were compacted at a cement content of 10% of minus 4.75 mm fraction with a 48 hour delay between compacting the first and second layers. Specimens were cured in a closed chamber during this period with twice daily applications of water using a mist spray. The second layer was sealed immediately after compaction. Testing took place at an age of 28 days after completion of the specimen.

Three joint preparations were examined; joints "as compacted" without treatment; application of neat cement before compaction at the rate recommended by De Groot (1976); and wire brushing of the joint surface in an effort to simulate the technique used by the United States Bureau of Reclamation and the Portland Cement Association.

All tests were limited to four specimens by the number of box sets for the testing rig.

The range of vertical stresses applied was selected to correspond to the confining pressures used in the triaxial tests. As a result of various problems with the apparatus, the distribution of normal stresses was not uniform and did not correspond to the intended values. The values used were 0.54, 1.17, 1.83 and 2.04 MPa.

12.2 APPARATUS

12.2.1 Direct Shear Apparatus

The direct shear apparatus used for these tests was originally built for the testing of rock samples, plate 12.1, shows a general view of the machine in use.

PLATE 12.1

DIRECT SHEAR APPARATUS IN USE

Modifications were made to the box sets used with the machine to permit their use with soil-cement, this included the fabrication of a collar and platen to permit the material to be statically compacted in the boxes. In order to produce a zone in which the failure plane could form, the box sets were modified to produce a separation of approximately 12 mm during testing. Shear plane area was 380 mm long by 160 mm wide and overall specimen depth was 260 mm.

Vertical and shearing forces were applied by means of 30t capacity hydraulic jacks. Vertical load was regulated by means of high pressure nitrogen acting on the surface of hydraulic oil in a receiver. This system provided sufficient gas volume to maintain a constant load during specimen dilation.

Horizontal load was applied by means of an air-hydro pump with a hand pump used for initial load application. Rate of loading was controlled by means of a regulator on which the setting was continuously increased by means of an electric motor driving through a reduction gearbox. This controlled the air pressure supplied to the air-hydro pump and hence the rate of loading. A loading rate of 5.2 kN/minute was used for all tests. This control mechanism worked effectively, producing consistent loading rates. This can be seen from the load - deflection and time - deflection graphs in section 12.4.

In order to observe the behaviour of the material after failure, the rate at which horizontal deflection occurred was controlled by means of a needle valve inserted in the hydraulic line from the air-hydro pump to the jack.

Monitoring of the applied horizontal load during all stages of the test was performed using a pressure gauge with lazy pointer. The air-hydro pump assembly is shown in plate 12.2. Withdrawal of the horizontal load jack was accomplished by applying a vacuum to the hydraulic line.

PLATE 12.2

AIR-HYDRO PUMP ASSEMBLY

Measurement of horizontal load was intended to be by means of a 300 kN load cell which was located between the jack and the reaction frame. However, the original load cell was found to be non-linear and the cell built to replace it was found to be temperature sensitive. All horizontal load readings were thus obtained by measurement of hydraulic pressure.

Measurements of both horizontal and vertical deflections were made using dial deflection gauges reading directly to 0.001 inch (0.0254 mm).

12.2.2 Apparatus for Compaction of Specimens

Specimens were prepared by static compaction using a Macklow Smith 1750 kN compression testing machine. In order to permit filling and compaction, a collar and platen were fabricated from 50 mm steel plate. The collar was fitted on top of the box and the platen was attached to the cross-head of the machine. The cradle shown in plate 12.3 was used to move the box set under the cross-head of the machine.

PLATE 12.3
CRADLE FOR POSITIONING SHEAR BOXES ON
TESTING MACHINE FOR COMPACTION OF SPECIMENS

12.2.3 Curing of Test Specimens

In agreement with other tests in the program, direct shear specimens were cured for a period of 28 days at a temperature of 25 ^{o}C. A curing chamber was built using dry laid bricks to produce a double brick wall that was lined with polythene sheet to exclude draughts. The top of the chamber was covered with 75 mm polystyrene slabs.

An electric fan heater with thermostatic control was used to circulate heated air in the chamber. The temperature was monitored using maximum and minimum thermometers and was generally within the limits of 23 to 28 °C.

12.3 TEST PROCEDURES

The procedures used in the various components of the direct shear test program are described below.

12.3.1 Soil Preparation

The blend soil including coarse fraction, which was stored after initial blending, was used for these tests. The original intention was to conduct a series of tests using unstabilized material to investigate the effect of maximum particle size and minimum box dimension on the results. However, owing to delays in other parts of the program, it was necessary to eliminate these items.

A review of literature was undertaken with no definitive indication as to acceptable maximum particle size. Marachi et al (1969) provided the only information of relevance. After due consideration, it was decided to use the procedure used by the PCA for wet-dry tests. viz replacement of the material retained on the 19 mm sieve with the same mass of material passing the 19 mm sieve and retained on the 4.75 mm sieve.

The fine material was air dried to constant moisture content and then stored in sealed drums. The coarse aggregate was stored in drums until required. One drum of coarse aggregate was filled with water and refilled after every set was compacted to ensure that the aggregate was saturated at the time of batching.

Moisture content samples were taken from the fine material and oven dried at 110 °C on the day prior to batching to ensure close control over batch moisture content.

Saturated surface dry moisture content of the aggregate was determined at the start of the test program and thereafter adjustments in this allowance were made on the basis of variations in batch moisture content. It was found that because of the large amount of coarse material (13kg) required for each batch, it was not practical to use absorbent pads to dry the surface prior to batching. Consistent results were obtained by spreading the required amount of material to drain on a large 4.75 mm sieve immediately before starting to batch the other components. It was then possible to judge by eye when the desired moisture condition was achieved. Occasional agitation using a scoop ensured uniform drying.

12.3.2 Preparation for Compaction

The boxes were prepared for compaction by cleaning with cotton waste and then reassembled as required. The base plate was greased to provide protection against rusting and then covered with a sheet of polythene to facilitate removal. The inside surfaces of the boxes were greased to prevent rusting, reduce side friction during compaction and to facilitate extrusion.

In the case of the tests on intact material, the top boxes were bolted to the bottom boxes with 9.5 mm spacer bars clamped in place. In the case of the jointed specimens, only the bottom box was assembled and a set of 4.75 mm spacer bars were used to support the collar at the time of compaction in order to locate the construction joint at the center of the box separation.

12.3.3 Batching and Compaction

The material was weight batched and mixed in a pan mixer as shown in plate 12.4. A timed mixing sequence of hand and machine mixing lasting nine minutes was followed.

In the case of the intact specimens, one batch produced one specimen, whereas for the jointed specimens it produced two half specimens. Placement was in three equal layers for each box half. Each layer was partially compacted by applying two passes of overlapping blows with a 25 mm square steel tamping bar.

PLATE 12.4
PAN MIXER AND BATCHED MATERIAL

The cross-head was then brought into contact with the material in the box and load was applied by the testing machine to bring the cross-head, which formed the flange of the compaction plug, into contact with the collar on the box set. On closure, the load was transferred to the box set and this condition was maintained for a period of one minute to permit consolidation.

In the case of the intact specimens, the surface of the specimen was sealed with two layers of polythene held in place with packaging tape. The jointed specimens were not sealed. The box set was then placed in the curing chamber.

12.3.4 Treatment of Jointed Specimens

Jointed specimens were cured in the curing chamber for two days during which time they were sprayed twice daily with distilled water starting on the evening of compaction. An application rate of 750 g/m^2 was used. This was arbitrarily determined to give a limited amount of free water on the surface of the specimen immediately after spraying, with a view to simulating the condition achieved by a water tanker under site conditions.

The frequency of application was chosen to maintain a reasonable degree of moisture on the surface of the specimen in an effort to ensure repeatability.

The two day delay between compaction of the first and second layers was selected on the following grounds:-

i. Discussion with M.R. Dunstan, while he was in South Africa, indicated that in his investigations into high flyash roller compacted concrete, he considered the maximum acceptable delay to be two days. In cases where a greater delay was unavoidable special precautions were taken to ensure that bond was achieved.

ii. Hansen (1979a) in reply to enquiries with regard to construction delays in the placing of the soil-cement embankments for the cooling water reservoirs for Barney M. Davis power station, refers to delays of between 0.5 and 3.5 hours between layers with the embankment being placed in relatively short (+/- 400 m) lengths. This was done with the object of avoiding major delays.

iii It was considered that in constructing a large embankment dam, the area of placement would be substantial and that delays of one or two days would be relatively commonplace. A delay of two days was felt to represent an upper limit of routine delays. The need to investigate the effect of other delay periods is however stressed.

The surface treatments applied to the individual construction joints are described below.

12.3.4.a Untreated Joint

The surface, as compacted, is shown in plate 12.5 from which it can be seen that the surface produced by static compaction was generally smooth and intact. This surface was lightly brushed with an absorbent cloth, immediately before placing the second layer, to remove loose material. It should be noted that the spraying of the joint on the morning of compaction produced a surface condition similar to the saturated surface dry condition used for coarse aggregate. This was consistent with the finding of De Groot (1976) that "a condition similar to SSD (saturated surface dry) seems to be marginally better."

12.3.4.b Cement Treated Joint

De Groot (1976) states that "where bond between layers is considered necessary, lift line treatment with cement is recommended". The rate recommended is 0.2 kg/m^2 and this rate was used in these tests. In the light of the results, it is felt that a higher rate should be considered. De Groot in fact observes that "it may be desirable to use slightly higher amounts to account for construction variation." Specimens were prepared in the same way as those with untreated joints and the cement was applied immediately before the material for the second layer was placed. A 12 mm paint brush was used to spread the cement as evenly as possible over the joint surface. Application of the cement in paste form is recommended for future laboratory work

PLATE 12.5
SPECIMEN SURFACE AS COMPACTED

12.3.4.c Wire Brushed Joint

Wire brushing of construction joints is the treatment that is typically applied by the USBR and recommended by the PCA as a means of achieving bond on joint planes in soil-cement facings. De Groot (1976) found that "the removal of the smooth bonding plane strengthens the joint between the layers." Various other authors have referred to the merits of brushing of joint planes and are referred to in chapter 3. Plate 12.6a, obtained with correspondence from Warren (1979), shows a typical broomed soil-cement surface.

It was originally intended to fabricate apparatus to simulate the actual brooms used in practice as closely as possible, however, owing to the delays previously mentioned, the rig was not built and the tests were restricted to one set with wire brushed joints.

In order to simulate the wire brushed joint surface, a number of tests were conducted using a selection of wire brushes obtained from hardware stores. These were held in an electric hand drill with a rotational speed of approximately 1000 rpm. The brush finally used had an overall diameter of 104 mm and contained crimped steel bristles with a free length of 27 mm and diameter 0.27 mm. Peripheral velocity was approximately 330 m/min as opposed to 380 m/min for a typical rotary broom used by the USBR (Warren 1979, Brace 1979). The surface produced using this brush is shown in plate 12.6b. It is felt that this is a reasonable approximation of the surface shown in plate 12.6a. Note that the width of the specimen in plate 12.6b approximates the length of the pen in plate 12.6a.

In judging the suitability of the surface produced, the surface shown in plate 12.6a was considered together with that shown in other photographs received from the same source. In addition, the descriptions referred to in chapter 3 were considered. With regard to the surface described by Davis et al (1973) who state that brooming "leaves a striated surface

12.3.4.c Wire Brushed Joint

Wire brushing of construction joints is the treatment that is typically applied by the USBR and recommended by the PCA as a means of achieving bond on joint planes in soil-cement facings. De Groot (1976) found that "the removal of the smooth bonding plane strengthens the joint between the layers." Various other authors have referred to the merits of brushing of joint planes and are referred to in chapter 3. Plate 12.6a, obtained with correspondence from Warren (1979), shows a typical broomed soil-cement surface.

It was originally intended to fabricate apparatus to simulate the actual brooms used in practice as closely as possible, however, owing to the delays previously mentioned, the rig was not built and the tests were restricted to one set with wire brushed joints.

In order to simulate the wire brushed joint surface, a number of tests were conducted using a selection of wire brushes obtained from hardware stores. These were held in an electric hand drill with a rotational speed of approximately 1000 rpm. The brush finally used had an overall diameter of 104 mm and contained crimped steel bristles with a free length of 27 mm and diameter 0.27 mm. Peripheral velocity was approximately 330 m/min as opposed to 380 m/min for a typical rotary broom used by the USBR (Warren 1979, Broce 1979). The surface produced using this brush is shown in plate 12.6b. It is felt that this is a reasonable approximation of the surface shown in plate 12.6a. Note that the width of the specimen in plate 12.6b approximates the length of the pen in plate 12.6a.

In judging the suitability of the surface produced, the surface shown in plate 12.6a was considered together with that shown in other photographs received from the same source. In addition, the descriptions referred to in chapter 3 were considered. With regard to the surface described by Davis et al (1973) who state that brooming "leaves a striated surface

PLATE 12.6a

SOIL-CEMENT SURFACE AFTER BROOMING IN FIELD

PLATE 12.6b

BRUSHED SOIL-CEMENT SURFACE PRODUCED IN LABORATORY

with narrow grooves approximately 1/2 inch (1.27 cm) apart", Hansen (1979a) observes "very hard to get grooves 1/4" deep, 1/16" is closer to reality". De Groot (1976) described the surface as similar to a cut surface and used cutting to produce the surface tested in his research.

Brushing was undertaken after curing for four hours and again immediately before assembling the top half of the box set for compaction of the second layer.

12.3.5 Conduct of Direct Shear Tests

The box sets were placed on the sliding platen of the testing machine and a vertical stress of approximately 2 MPa was applied in increments. The boxes were then separated by means of jacking bolts and the packing pieces removed leaving a separation of approximately 12 mm.

The 2 MPa confining pressure during box separation compares with the pressure of between 0.9 and 1.4 MPa required to extrude specimens from the boxes after testing and the 3.1 to 3.4 MPa required for flange closure during compaction.

The receiver pressure was then adjusted to give the required vertical stress and horizontal loading commenced. Initial loading was by hand until the lower limit of the air-hydro pump was reached. Further loading took place under load rate control until failure, at which point strain rate control was initiated.

On completion of the test series, specimen halves were extruded from the boxes using a hydraulic jack.

12.4 TEST RESULTS

Graphs 12.1 to 12.6 reflect the results of the direct shear tests. All results are plotted against horizontal deflection as conversion to shear strain was not felt to be relevant. Adjustment to remove the bedding-in portion of the loading curve has also not been applied in order to maintain the separation of the individual results. The curves thus reflect the test history of each specimen from the commencement of application of horizontal load.

The significant features shown in these graphs are as follows:-

a. The elastic portion of the loading curve, i.e. the initial portion after bedding-in, is generally linear and the slopes in all tests are similar.

b. The rate of loading to failure is generally uniform and consistent.

c. The post-failure portion of the results is well developed and the strain rate is reasonably consistent.

d. The loading curves for the intact material show a progressive development from a relatively plastic failure mode at the low cement content (5%) to a relatively brittle mode at the high cement content (20%).

e. The specimens with construction joints generally show a smaller drop between ultimate and residual stresses.

f. The dilation experienced by the jointed specimens is substantially less than that experienced by the corresponding intact specimens (10% cement). In a number of cases virtually no vertical displacement occurred and in the case of the wire brushed and untreated joints, significant compression occurred

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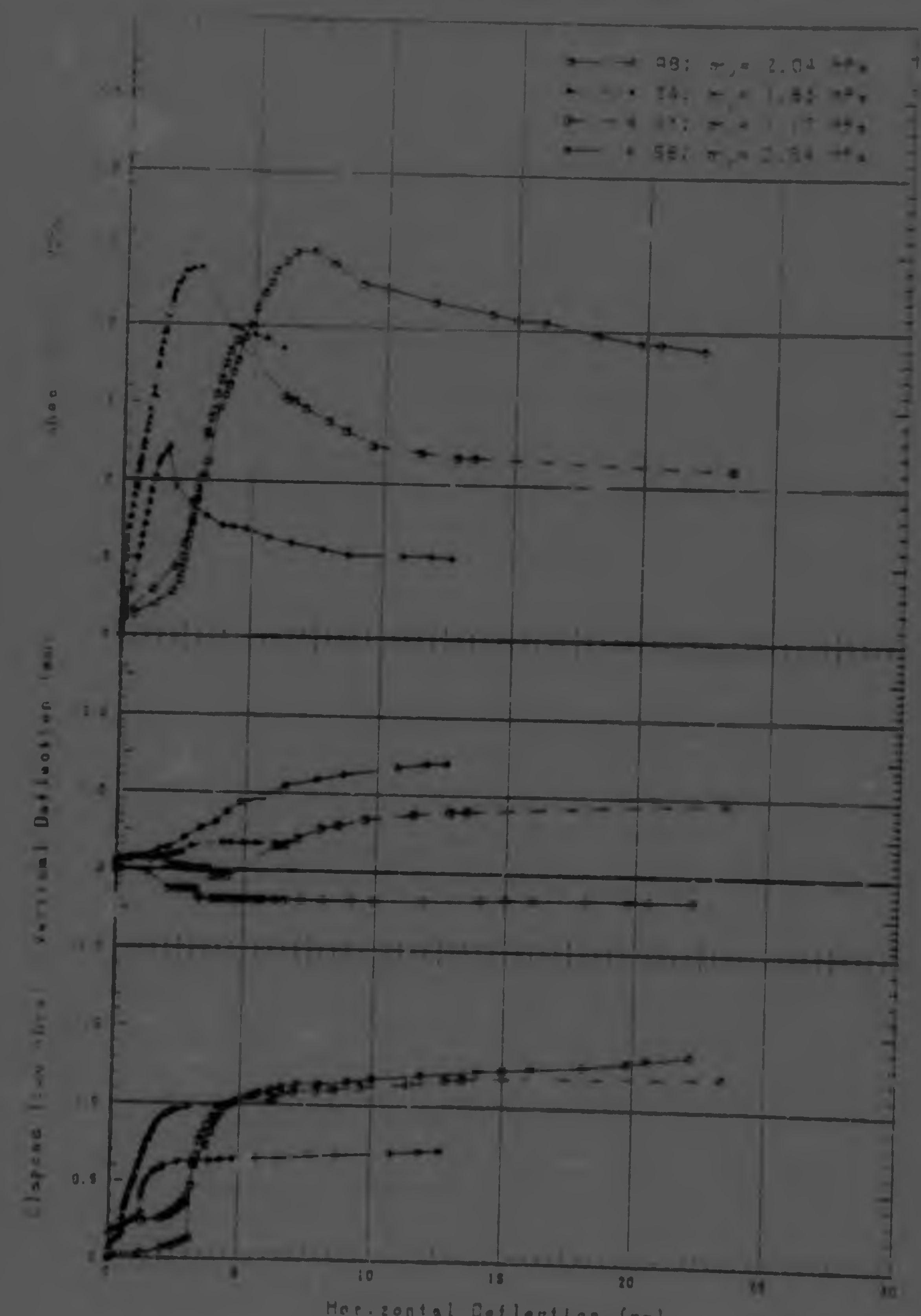
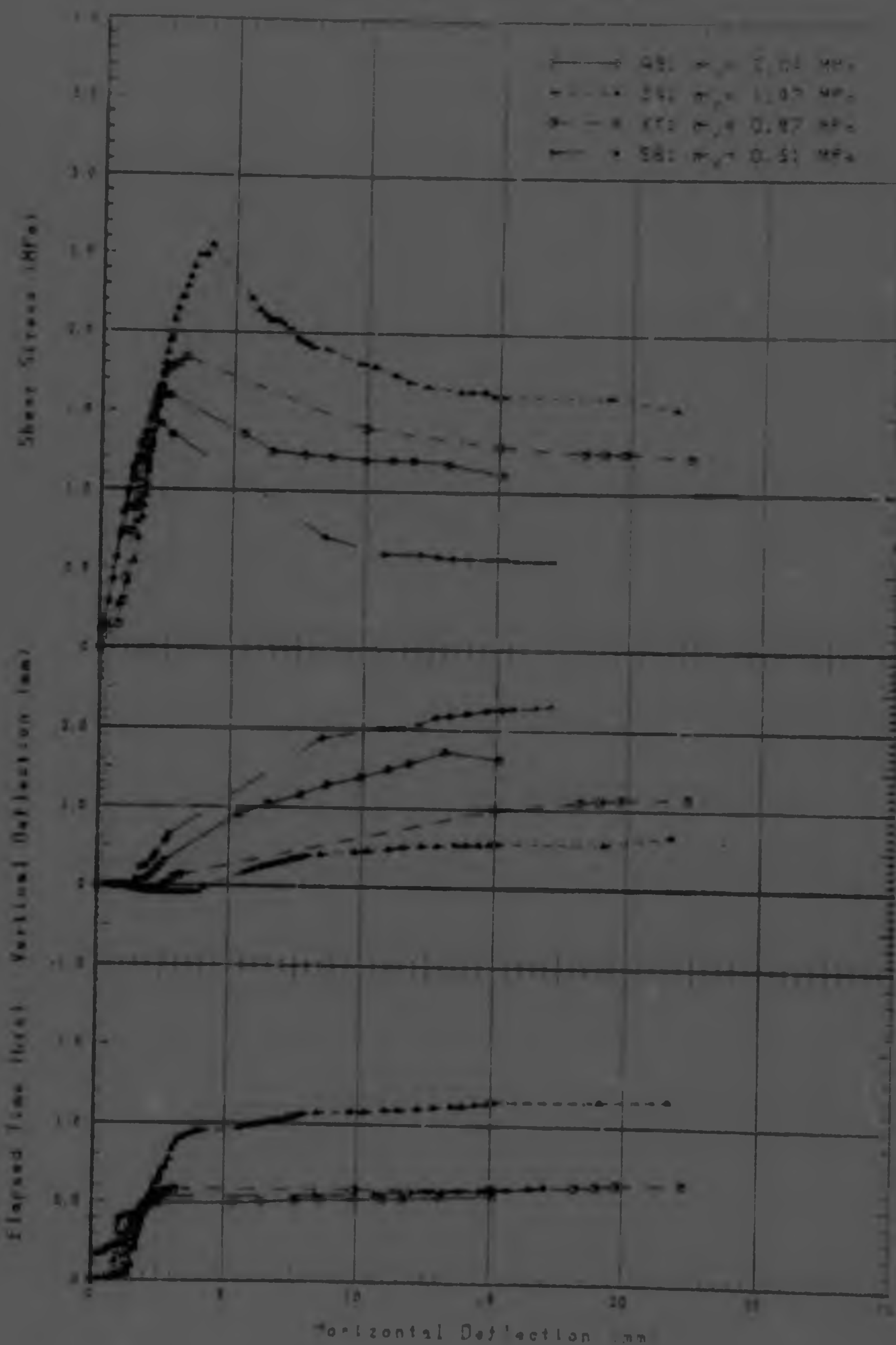
.RAPH .2.

N - SPECIMENS 5% CEMENT



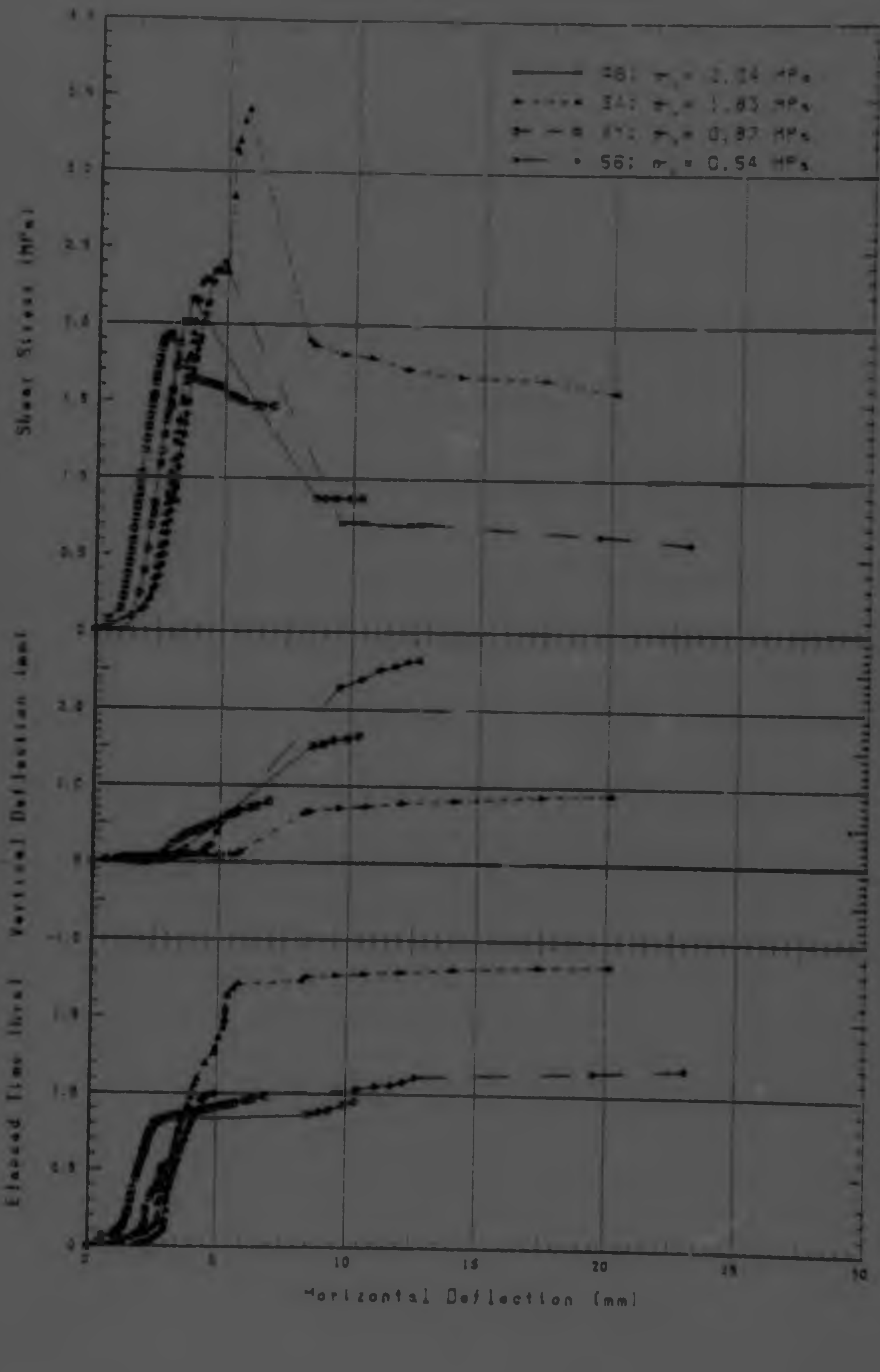
GRAPH



Shear Stress (MPa)
Vertical Deflection (mm)
Elapsed Time (hrs)
Horizontal Deflection (mm)
0
5
10
20

GRAPH 12.5

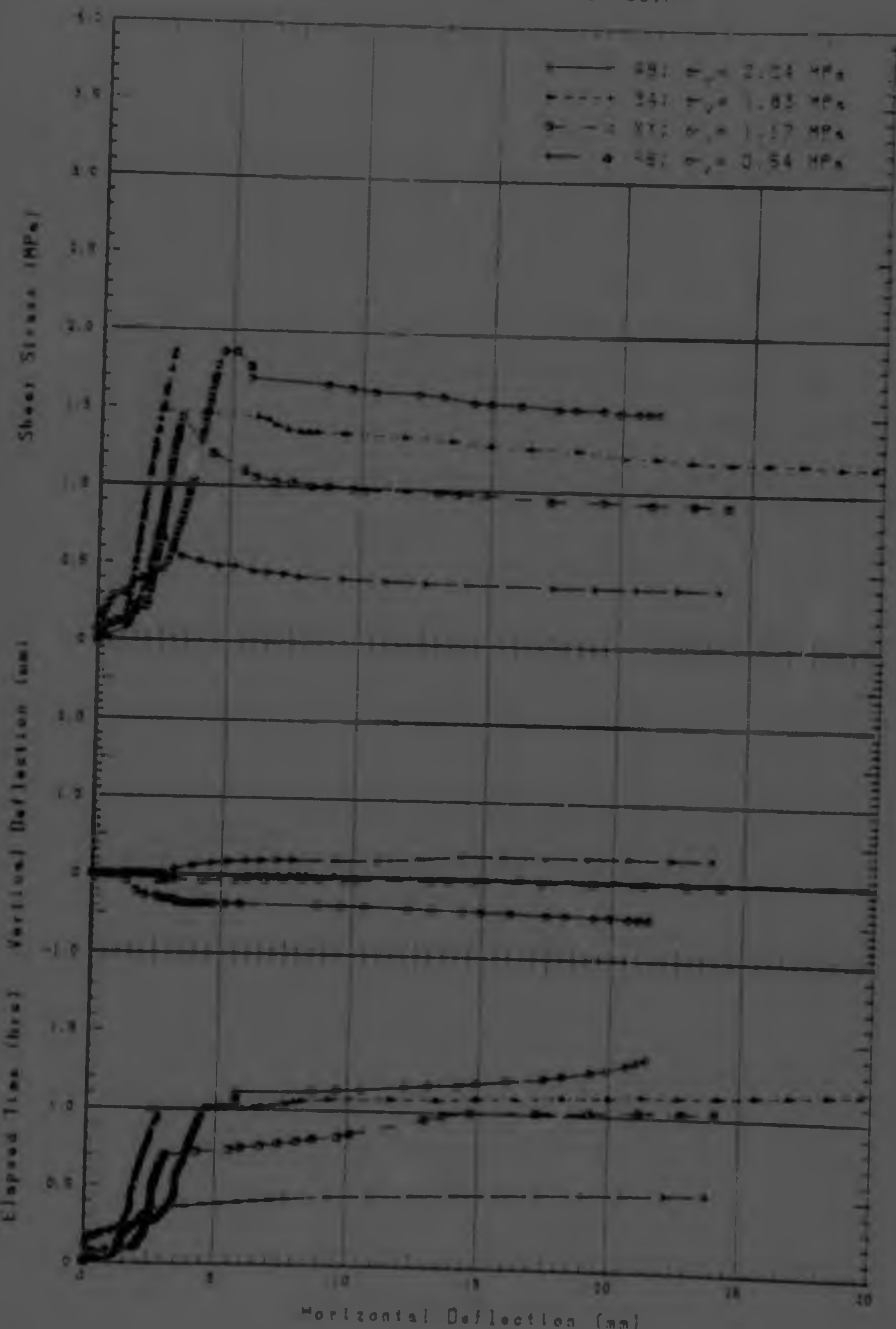
INTACT SPECIMENS 20% CEMENT



1720-1.PLT

GRAPH 12.4

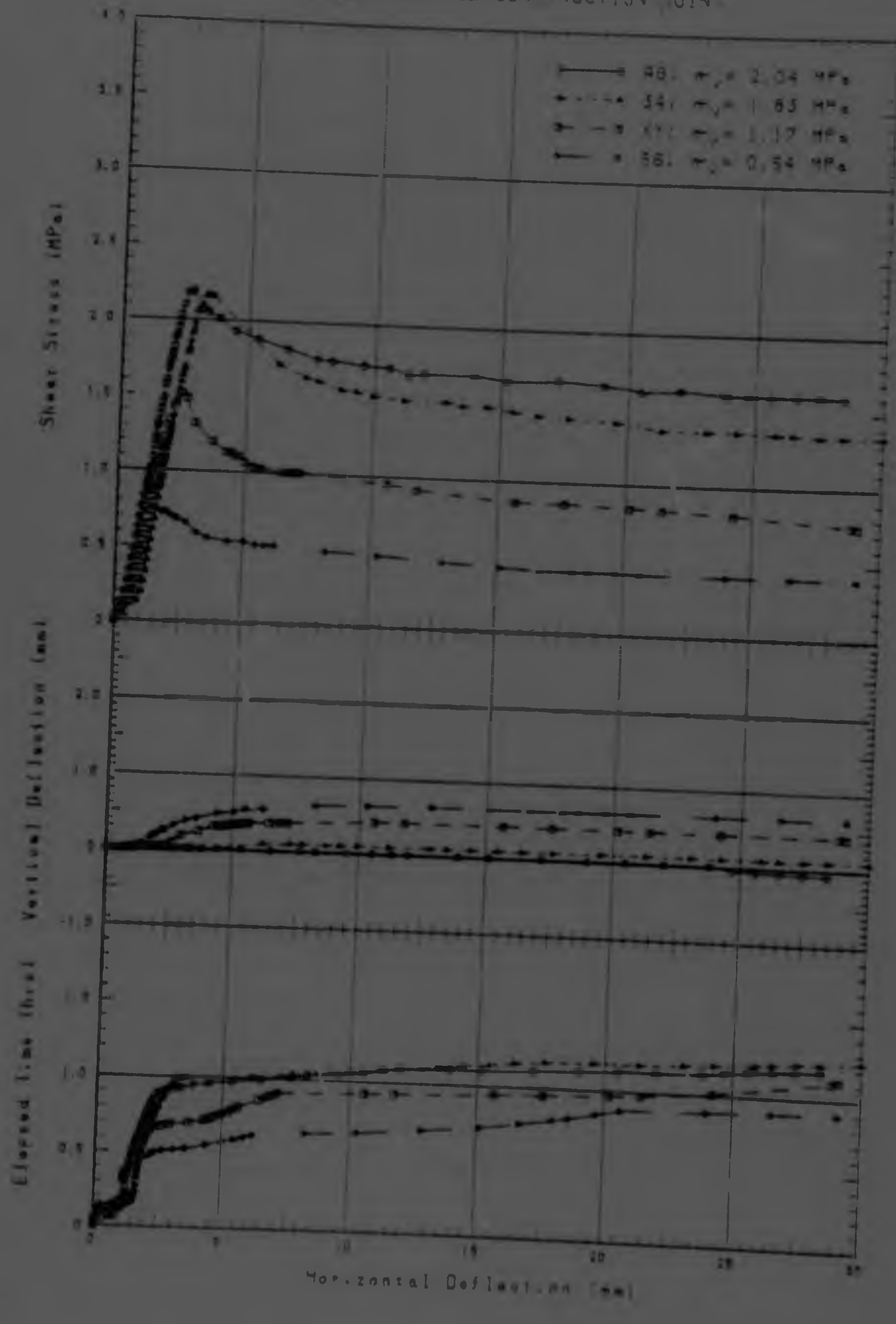
UNTREATED CONSTRUCTION JOINT



UNTR-1.PLT — 27/10/83-7e209-- 1.0

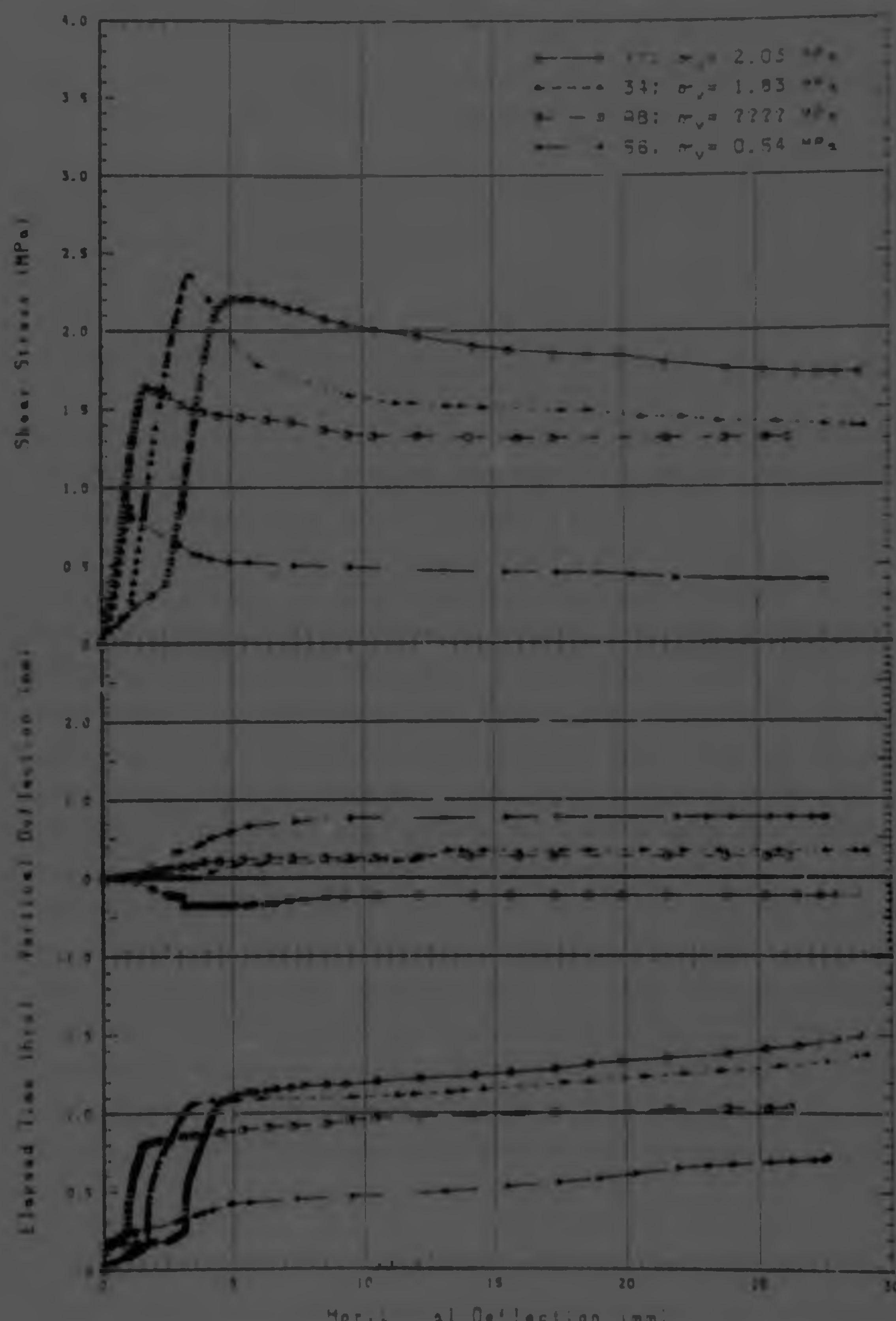
GRAPH 12.5

CEMENT TREATED CONSTRUCTION JOINT



GRAPH 12.6

WIRE BRUSHED CONSTRUCTION JOINT



BRSH-1 PLT -- 2

in the specimens subject to the greatest vertical stress. Of the intact specimens, only that with 5% cement experienced compression under this stress.

g. As is to be expected, dilation decreases with increasing vertical stress while time to failure increases.

h. Dilation increases with increasing cement content as does strength and time to failure.

12.4.1 Failure Envelopes

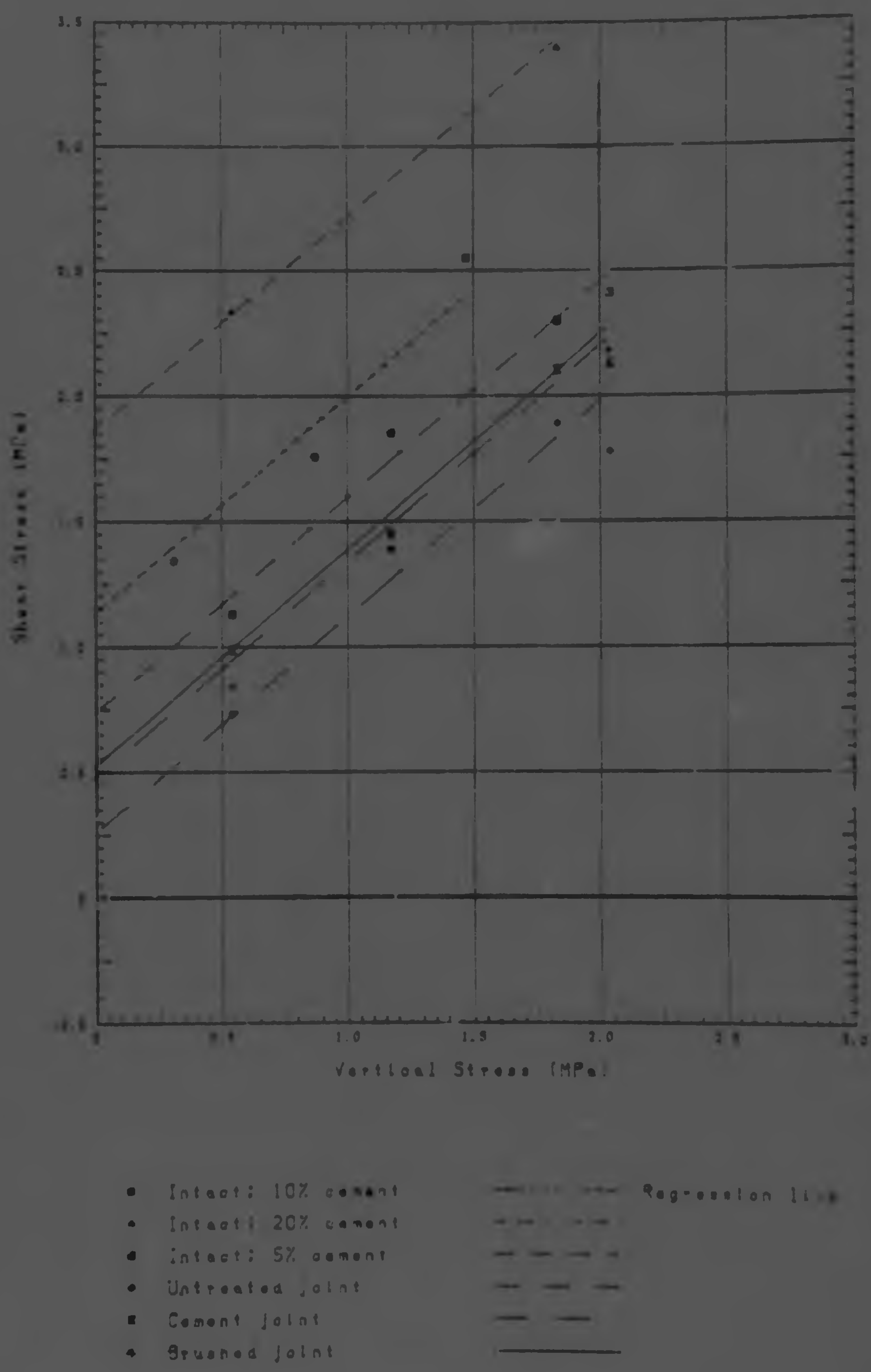
From these results, a graph of failure shear stress against vertical stress was plotted in order to obtain the failure envelopes, this is presented in graph 12.7.

Owing to various problems during the test program, it was necessary to discard a number of results, only those considered valid are reflected on the graph. From a visual inspection of the data points, it was apparent that there was no basis for fitting other than straight lines of equal slope to all data sets, this was consistent with the results of the triaxial tests presented in chapter 11 and with the findings of other researchers. Accordingly, a transformation was performed whereby all data sets were translated to align their centers of gravity with the center of gravity for all the data (Chatfield 1975). A linear regression was then performed on the combined data to determine the angle of internal friction for the material and the translation constants were used to calculate adjusted cohesion for each set of tests. The regression coefficient r^2 was calculated as 0.96 indicating a good fit. The results of the regression are summarised in table 12.1.

Graph 12.7 indicates an increase in cohesion with cement content. This is consistent with the results of the triaxial and unconfined

GRAPH 12.7

FAILURE ENVELOPES FOR DIRECT SHEAR TESTS



SHR-10% PLT -- 04/11/83-7=218-v 1.1

TABLE 12.1
SUMMARY OF ULTIMATE STRENGTH RESULTS
FOR DIRECT SHEAR TESTS, RANKED ON COHESION

TEST	Specimen No	Vertical Stress MPa	Shear Stress MPa	Corrected Cohesion MPa
UNTREATED JOINT				
	AB=06.4C/401	2.038	1.777	
	34	1.828	1.889	
	XY	1.169	1.390	
	56	0.538	0.732	0.250
CEMENT TREATED JOINT				
	AB=06.4C/443	2.038	2.131	
	34	1.828	2.105	
	XY	1.169	1.449	
	56	0.538	0.987	0.471
WIRE BRUSHED JOINT				
	XY=06.4C/478	2.032	2.183	
	34	1.828	2.299	
	56	0.538	0.846	0.517
INTACT 05% CEMENT				
	AB=03.2C/317	2.038	2.410	
	34	1.828	2.299	
	XY	1.169	1.852	
	56	0.538	1.129	0.726
INTACT 10% CEMENT				
	34=06.4C/249	1.470	2.549	
	Y	0.871	1.757	
	56	0.306	1.342	1.125
INTACT 20% CEMENT				
	34=12.8C/283	1.828	3.395	
	56	0.538	2.334	1.849

Angle of Internal Friction = 40.7°

Group Average Cohesion = 0.72 MPa

compression tests. Cohesion is plotted against cement content on graph 12.8 and a linear relationship, as found with the triaxial tests, is indicated by the data points and solid line.

The results for the jointed specimens are however not as encouraging. In all cases, the ultimate shear strength is substantially less than that for the intact material with the same cement content (10%) and also less than that for the intact material with 5% cement. There is no significant difference between the results for the specimens with cement treated and wire brushed construction joints but they are substantially stronger than the specimens with untreated joints.

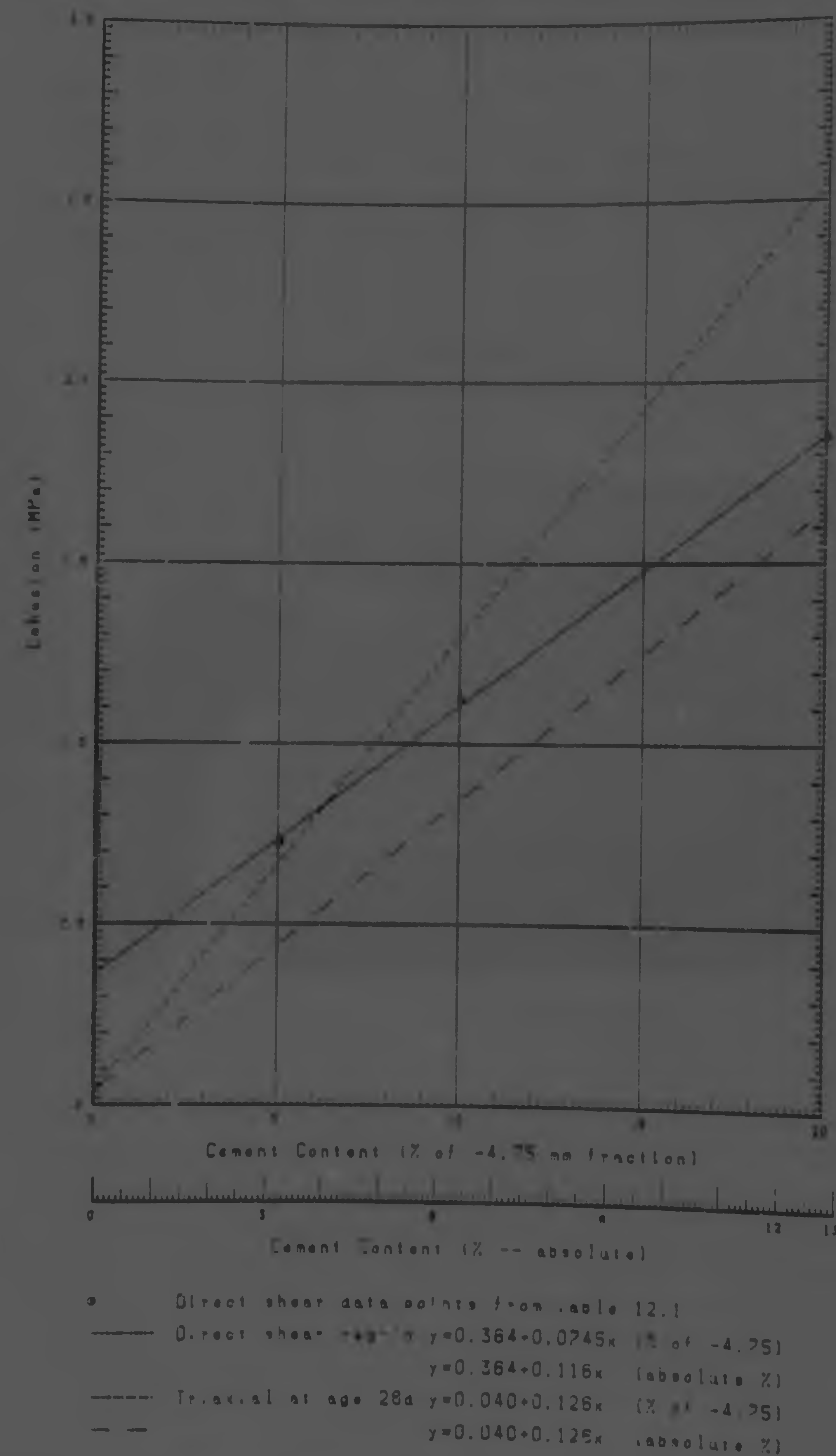
These results differ substantially from those reported by De Groot (1976) and there is thus cause to question the compaction procedure used. The reduced dilation of the jointed specimens also suggests that compaction on the joint plane was inadequate. It appears likely that the tamping of the material, as performed, was not sufficient to ensure intimate contact on the joint plane and the use of an impact rammer in future tests should be considered with a view to simulating the high contact pressures that would occur under a sheepsfoot roller.

It should be noted that in dealing with a material containing coarse aggregate, where the shear strength of the aggregate may exceed that of the stabilized matrix, it is unlikely that the full shear strength of the intact material will be developed on construction joints. Other forms of test, such as tensile tests on the joint and shear tests without applied normal pressure, as used by De Groot, will not necessarily be affected by the absence of coarse aggregate across the joint plane and poor bond between the matrix and aggregate particles could lead to the joint plane appearing stronger than the intact material.

Failure of the jointed specimens treated with cement and by brushing was generally through the material adjacent to the joint

GRAPH 12.6

COHESION vs CEMENT CONTENT OF INTACT SPECIMENS



C-VS-CEM.PLT -- 11/08/84-w=1271-v 1 5 (8 SHR-CSNF DAT, 8) plane rather than on the joint plane itself. The failure surface in the intact material was generally dished in appearance with shearing of weaker aggregate particles while stronger particles gouged channels in the soil-cement matrix. Plate 12.7 shows typical failure surfaces for the different joint conditions, from front to back:- untreated joint, cement treated joint, wire brushed joint and unjointed specimen. Cement content of all specimens was 10% of fine fraction.

PLATE 12.7
TYPICAL FAILURE SURFACES

The indicated joint cohesion in graph 12.7 for cement treated and brushed joints is approximately 0.5 MPa which, while not as great as that of the intact material, is nevertheless a significant strength. This bond could be sufficient to resist wave forces and account for the behaviour observed in slope protection. The work reported by De Groot (1976), used soils which contained no significant material retained on the 1.19 mm sieve and conseplane rather than on the joint plane itself. The failure surface in the intact material was generally dished in appearance with shearing of weaker aggregate particles while stronger particles gouged channels in the soil-cement matrix. Plate 12.7 shows typical failure surfaces for the different joint conditions, from front to back:- untreated joint, cement treated joint, wire brushed joint and unjointed specimen. Cement content of all specimens was 10% of fine fraction.

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Note that the effective shear strength of construction joints can be increased by placing the material so that the joint plane slopes up towards the downstream face. A reduction in the delay between placing of layers is likely to improve the bond on construction joints. This requires further investigation to determine the maximum permissible delay without joint preparation.

In view of the problems experienced in simulating compaction in the laboratory, it is felt that field trials similar to the subgrade restraint tests of Bofinger and Sullivan (1971) should be considered. This test procedure would involve placing a number of layers of soil-cement using the techniques and equipment to be used on site. A block of cured soil cement would be excavated to the level of the joint plane using a paving saw and the block sheared while subject to a suitable vertical stress. A number of blocks could be tested at different vertical stresses. Load and strain rate control could be applied using the apparatus developed for this test program. This technique would permit the examination of a large number of variables in a relatively short time.

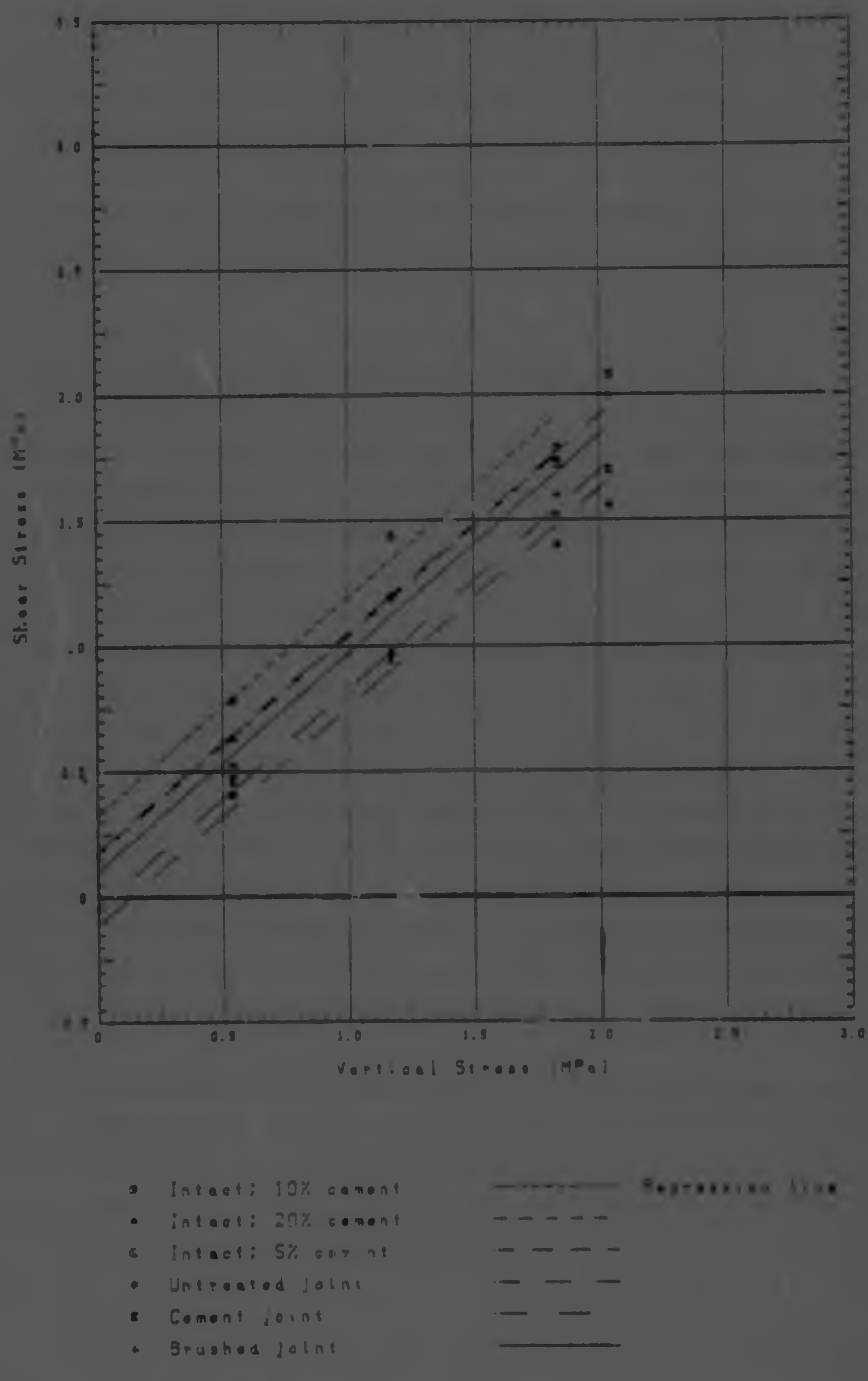
12.4.2 Residual Strength Envelopes

It is apparent from the load - deflection curves that the difference between ultimate and residual strength for the jointed specimens is substantially less than that for those without joints. Accordingly, shear strengths at displacements of 5, 10 and 15 mm past failure were determined and the data subjected to the same transformation and regression applied to the ultimate strength data. Graph 12.9 reflects the strength envelopes at a displacement of 5 mm past ultimate.

GRAPH 12.9

CONSOLIDATION OF DIRECT SHEAR DATA AT

A DISPLACEMENT OF 5 mm PAST FAILURE DISPLACEMENT



In all cases a regression coefficient of 0.98 was obtained with angle of internal friction ranging from 41° through 39.75° to 38.77° for the three displacements considered, indicating a progressive polishing of the failure plane. Calculated group average cohesion varied from 0.074 through 0.014 to 0.002 MPa for the same range. This cannot be considered to differ significantly from zero, confirming that once failure has occurred, the residual strength is purely frictional.

These results indicate that the residual strength of the jointed material is essentially identical to that of the unjointed.

12.4.3 Ratio of Ultimate to Residual Strength

From the inception of this test program, it was considered that design on the basis of residual shear strength should be investigated as a potentially more reliable technique on the assumption that residual strength was likely to be less variable in practice than ultimate strength. It was for this reason that the horizontal load control apparatus was designed to supply strain rate control in the post-failure zone and this decision was vindicated by the results discussed above.

With a view to expressing these results in terms of a design philosophy, the ratio of ultimate to residual strength was examined. This ratio can be considered to represent a measure of the factor of safety that would apply to a design on the basis of the residual strength relative to the ultimate strength. Considering the extreme cases for the intact material:-

> The maximum ratio varies from 3.7 at a displacement of 5 mm past failure to 4.5 at a displacement of 15 mm for the specimen with a cement content of 20% subject to the lowest vertical stress (0.54 MPa). For the specimen subject to 1.8 MPa this reduces to between 1.95 and 2.18.

The **minimum** ratio varies from 1.2 to 1.35 for the specimen with a cement content of 5% subject to the highest vertical stress (2.04 MPa). For the corresponding specimen with untreated joint, the ratio varies from 1.08 to 1.18. For the specimen with 5% cement subject to a stress of 0.54 MPa the ratio ranges between 1.8 and 2.

From consideration of the available data, it appears that the ratio is proportional to cement content, confirming the increasingly brittle nature of the material with increased cement content.

12.4.4 Factors of Safety Customarily Applied

Robertson and Blight (1978) section 4.3.2, reproduced in chapter 3, refer to the apparent anomaly in the customary factors of safety against sliding failure used in design of earthfill and concrete gravity dams. In the first instance, a factor of safety of 2 is generally considered adequate whereas in the second a minimum factor of 5 is specified in cases where the simplified method of analysis cannot be applied. Note that the USBR analysis referred to in section 3.2.d is somewhat less conservative. The simplified analysis requires merely that the "sliding factor", the ratio of horizontal to vertical forces, should lie in the range of 0.65 to 0.75 for concrete on rock, as discussed, this requires a minimum angle of internal friction of 33°. This empirical method relies entirely on the cohesion of the material to provide a safety factor against sliding.

The safety factors referred to above are applied to the ultimate strength of the material i.e. the ultimate strength is divided by the relevant factor to arrive at a design strength which is compared with design stresses to determine the adequacy of the material. Relating this to the ratio results summarised above:-

a. The weak material has a true factor of safety of residual over ultimate streng[illegible]e range 1.2 to 2.0. This compares well with the value of 2 applied to soil. The value of 1.2 corresponds to an enhanced factor of safety on residual strength where design is undertaken with a safety factor of 2 on ultimate strength.

b. In the case of the stronger material, the factor of safety of residual over ultimate strength is in the range 1.95 to 4.5. This represents an enhanced factor of safety on residual strength where design is based on a safety factor of 5 on ultimate strength. It should be noted that in concrete the ratio of ultimate to residual strength may well exceed 5.

12.4.5 Proposed Design Philosophy

The above suggests that the apparent anomaly, referred to above, represents a design philosophy which is in fact based on consideration of the residual strength of the material, i.e. on the basis of experience, the factors of safety that have come to be associated with the two materials, soil and concrete, in fact reflect the approximate ratio of ultimate to residual strength commonly developed by the respective materials.

This relationship is by no means exact, the ratio of 1.2 above indicates that in the case of soil, the factor of 2 also includes an allowance for the greater variability of the material. Likewise, a consideration of the higher strength concretes indicates that the factor of safety of 5 allows for a percentage of the cohesion to be taken into account. However, in terms of the stresses actually encountered in a concrete gravity dam, this percentage is generally small; applying equations 1 and 2 in Robertson and Blight section 4.3 and assuming an angle of internal friction of 40°, an unconfined compressive strength of 26 MPa is indicated before the factor of 5 makes allowance for cohesion.

On the basis of the above discussion, taking account of the fact that soil-cement can be expected to vary in strength from that of a strong soil to a moderate quality concrete, depending on the application, it is suggested that design should be based on the residual (i.e. frictional) strength of the material with a factor of safety of say 1.2 applied to take account of material variability.

At the same time, it is acknowledged that additional tests are required to support this proposal.

It should be noted that, as the angle of internal friction has been found to be independent of cement content for cement contents above 5% (3.2% of whole soil), this design philosophy does not provide a basis for determination of cement content. It is therefore proposed that a minimum cement content of 3.2% of whole soil should be specified until further tests are conducted at low cement contents and that a minimum factor of safety of 2 should be applied to the ultimate strength.

12.4.6 Comparison with Results of Triaxial Tests

The triaxial tests yield an estimated angle of internal friction of 37° as opposed to the value of 40.7° for the direct shear tests presented in table 12.1. It is likely that the presence of coarse aggregate will cause an increase in angle of friction and it is considered that this 3.7° increase results from this. It should however be noted that the maximum principal stress in the direct shear tests is considerably lower than that occurring in the triaxial tests. Consequently, an increased angle of internal friction would be expected from the curvature of the Mohr envelope referred to in section 11.4.2.

In order to compare the estimated cohesion obtained from the direct shear tests with that for the triaxial tests, which were conducted on specimens at a nominal age of 294 days, an adjustment was required. Graph 10.9 indicates a linear relationship between unconfined compressive strength and logarithm of age and graph 11.9 indicates a linear relationship between cohesion and unconfined compressive strength. From the regression lines for different cement contents plotted in graph 10.9, it was determined that the average ratio of unconfined compressive strength at an age of 28 days to that at 294 days was 0.576 with a standard deviation of 0.009. Applying this factor to the regression coefficients presented in graph 11.9 gives an estimate of the cohesion - cement content relationship at age 28 days for the standard cylinders in the triaxial test:-

$$\text{Cohesion (MPa)} = 0.040 + 0.126 \times \text{Cement Content (\%)}$$

This data was obtained for specimens containing only the soil fraction passing the 4.75 mm sieve whereas the direct shear specimens also contained the fraction retained on this sieve. As discussed in section 12.1, it was decided to specify the cement content of the direct shear specimens in terms of the the soil fraction passing the 4.75 mm sieve and this convention has been used througnout this chapter. The primary cement content axis on graph 12.8 is plotted on this basis whereas the secondary axis reflects the equivalent absolute cement content for the direct shear specimens in terms of the total soil component.

The solid line on graph 12.8 reflects the cohesion - cement content relationship for the direct shear test results in terms of cement content expressed on either basis. The cohesion intercept of 0.364 MPa is considerably above the value of 0.040 estimated for the triaxial results. This indicates that the presence of coarse aggregate causes a significant increase in shear strength at low cement contents where the strength of the aggregate is high relative to that of the soil-cement matrix.

The short-dashed line on graph 12.8 reflects the cohesion - cement content relationship derived from the the triaxial results plotted in terms of the fraction passing the 4.75 mm sieve. With the cement content expressed on this basis, the results indicate that while the cohesion intercept is significantly increased, the rate of increase of cohesion with cement content is significantly decreased. This occurs to the extent that above a cement content of approximately 6.2% the strength of specimens prepared without the coarse fraction exceeds that of specimens containing it.

As the soil being investigated was a residual soil formed by weathering of the in situ rock, the degree of weathering, and hence the strength, of the coarse aggregate varied considerably. Consequently, as the strength of the matrix increases with increased addition of cement, an increasing proportion of the coarse aggregate is weaker than the matrix and thus the average strength of the material increases more slowly than if the coarse aggregate were not present.

While this effect is probably not significant for cement contents under about 12%, it does indicate that care should be taken in applying the results of tests on specimens containing only the minus 4.75 mm fraction if the cement content is reckoned on this basis. It is likely that the relative slope of the results for the material containing the coarse fraction will vary considerably depending on the properties of the coarse aggregate and further investigation of this effect is required.

These results suggest that the beneficiation of the soil either by the replacement of a proportion of the natural aggregate with; or by the further addition of; an intact, unweathered aggregate, such as quarried rock, could improve the strength of the soil-cement considerably by increasing the cohesion intercept, increasing the slope of the cohesion - cement content relationship and possibly also increasing the angle of internal friction. Given the relatively low strength of the soil-cement matrix, it is likely that a relatively small addition of high strength aggregate would be required.

The above discussion indicates that the initial assumptions made in estimating the equivalent cement content for the comparison of specimens prepared with and without the coarse fraction, were erroneous. An alternative basis of comparison is in terms of the absolute cement content of the soil-cement mixture, i.e. the cement content expressed as a percentage of all soil in the mixture irrespective of maximum particle size. The long-dashed line on graph 12.8 represents the adjusted cohesion - cement content data for the triaxial test results expressed on this basis i.e. plotted relative to the lower cement content axis.

On this basis, the slope of the cohesion - cement content relationship for both tests is very similar, differing by less than 6%. It is possible that the convergence of the two lines with increasing cement content reflects the increasing strength of the matrix relative to the more heavily weathered particles of coarse aggregate.

Expressed in these terms, the results of the two test programs are comparable. For the soil used in this research, it is apparent that design on the basis of the actual cement content of test specimens prepared using the soil fraction passing the 4.75 mm sieve will underestimate the strength available from soil-cement prepared with the complete soil. This provides a conservative basis for design but additional investigation of this relationship is required.

It is likely that the variation in degree of weathering of the sand and gravel fractions is similar. Consequently, if the strength of soil-cement is determined by the average aggregate strength, it is likely that similar results will be obtained irrespective of maximum particle size over the range considered. It is considered that this is a likely explanation for the results observed here.

It should be noted that the results discussed in this section do not have any significant effect on the design philosophy proposed in section 12.4.5 which is based on the frictional component of shear strength.

12.5 CONCLUSIONS

It is proposed that design for sliding stability of soil cement should be based on the residual (frictional) strength of the material with a factor of safety of 1.2 to account for the variability of the material. In addition, a minimum cement content of 3.2% of total soil and a minimum factor of safety of [illegible] on ultimate strength are proposed. Further large scale direct shear tests are required to provide statistical support for this proposal. In the case of subsequent analysis in this program, an angle of internal friction of 40^{o} is applicable.

Further tests are also required to determine a reliable method for developing the full ultimate shear strength of the material on construction joints. It is noted that the techniques investigated in this research developed the full residual strength of the material but did not develop the full ultimate strength. It is suggested that the use of an impact rammer on the first layer of material placed on the joint plane could remedy this problem. It is also suggested that testing of isolated blocks excavated in trial slabs compacted using full-scale construction equipment would provide an excellent means of rapidly examining the effects of different joint treatments.

At this stage, it is considered that the lower ultimate strength of the jointed specimens may be more a result of experimental technique than of material properties. This is based on the observation that other workers in this field have found the two methods of joint preparation examined here, cement treatment and wire brushing, to provide full bond on construction joints. It is however noted that in dealing with a material containing coarse aggregate it is unlikely that the full shear strength of the intact material will be developed on construction joints and that no results are available for comparison. Further investigation of this aspect is required.

Uniform application of cement to joint surfaces at the rate of 0.2 kg/m^2 recommended by De Groot (1976) was found to be impractical and it is considered that a rate of about 0.4 kg/m^2 is likely to be the practical minimum. With a layer thickness of 0.2 m, this represents an increase in the cement content of the embankment of the order of 0.1%.

If the intact shear strength of the material cannot be reliably mobilised on construction joints, construction with a crossfall sloping up towards the downstream face should be specified.

The effect of variation in construction delay requires further investigation. In particular, the maximum delay which will permit construction without joint treatment should be investigated.

While the development of a device to simulate the rotary steel brooms used in practice is still considered desirable, it is felt that adequate results can be obtained using the techniques used in this research.

Evaluation of the cement content of the material in terms of the fraction passing the 4.75 mm sieve does not provide a reasonable basis for extending the results of strength tests conducted on specimens prepared using this fraction to the whole soil. The basis for comparison should be the cement content in terms of the entire soil content of the material. This will slightly underestimate the strength available from the soil containing the coarse fraction as the coarse aggregate causes a slight increase in the angle of internal friction and cohesion at all cement contents. Thus, when cement contents of 5% cement are referred to throughout this thesis with reference to specimens produced from soil passing the 4.75 mm sieve, it should be assumed that the same cement content would be used for the soil containing the coarse fraction i.e. the equivalent cement content to be used for the soil with coarse fraction is also 5%. The effective cement contents for the direct shear specimens is therefore 3.2, 6.4 and [illegible].8% respectively.

Further research into the relationship between results obtained with and without coa[illegible] aggregate and the influence of degree of weathering of coarse aggregate are required.

It is suggested that the shear strength of soil-cement could be significantly improved by the use of relatively small quantities of high strength aggregate either added to the soil or used to replace a proportion of the natural aggregat[illegible] Further research into this proposal is required.

CHAPTER 13

OEDOMETER TESTS

13.1 INTRODUCTION

The objective of these tests was to investigate the settlement characteristics of soil-cement under gradual loading simulating construction and subsequent behaviour under cyclic reservoir operation. Short-term tests were conducted for comparison with the long-term tests and further tests were conducted to examine the effect of variations in loading conditions and moisture content.

The number of tests was limited by the number of oedometers available. The duration of the short-term tests was 14 days and that of the long-term tests approximately 60 weeks. A total of 70 specimens were tested as part of this program.

The stresses applied in these tests were selected to be similar to those likely to occur in practice with the degree of over-stressing limited to permit an examination of the behaviour under cyclic loading. The maximum stress was selected to correspond to the 97.5% confidence limit estimate of the 28 day soaked, unconfined compressive strength presented in graph 6.3. A factor of safety with regard to compressive stress (Sc) of 3, as suggested by the USBR and referred to in section 3.2 (d) was applied to this figure. This provided a higher stress level than the value of Sc=4 referred to in section 3.1.

Examination of the characteristics of gravity sections similar to those proposed in chapter 3 indicated that for the maximum principal stress at the toe of the embankment, the ratio of reservoir empty to reservoir full stress was approximately:-

$$0.86 / \log_{10}(\text{unit weight})$$

For the material used in this research. with an as-compacted unit weight of 20.6 kN/m^3, the ratio is [illegible]. This factor was used to determine the reservoir empty stress for use in the tests.

It should be noted that this reservoir empty stress is the maximum principal stress at the toe and not the maximum stress that will occur under the center of the embankment at the end of construction. The reservoir full stress is however the maximum compressive stress that will occur in the embankment. These values therefore bracket the maximum end of construction stress that can be expected at any point in the embankment. The maximum stress used for the specimens with 20% cement was limited to 2700 kPa by the capacity of the testing machines.

The number of long-term settlement tests was limited to two sets of two specimens. For the Rondebosch Dam with Sc=3, the indicated cement content in the toe area is 5.7%. For the same conditions, a cement content of 13% is indicated for an embankment of 100 m. In order to be consistent with the rest of the test program, cement contents of 5 and 15% cement were selected for the long-term test program.

The standard short-term tests involved loading corresponding to construction followed by cyclic loading corresponding to reservoir operation. After the second "filling", the specimen was flooded with water to examine the effect of the availability of extra moisture. The stresses used for the standard cement contents of 5, 10, 15 and 20% are summarised in table 13.1. Where time permitted, additional loading cycles were undertaken after the initial standard 14 day test in order to further examine the effect of cyclic loading. Short-term tests were conducted on specimens subject to the standard curing conditions and tested at ages of 56, 140 and 252 days.

TABLE 13.1

LOADING SEQUENCE FOR OEDOMETER TESTS

Day	Loading State	Stress (kPa) for Cement Content 5%	10%	15%	20%
1	Start of test	0	0	0	0
	Nominal load	11	12	12	12
		56	135	206	269
2		189	403	618	807
3	End construction	433	929	1442	1907
4		545	1174	1786	
pm	Reservoir full	667	1418	2198	2702
5		667	1418	2198	2702
6		667	1418	2198	2702
7		545	1174	1786	
pm	empty	433	929	1442	1907
8		545	1174	1786	
pm	full	667	1418	2198	2702
9	Inundate	667	1418	2198	2702
10		545	1174	1786	
pm	Reservoir empty	433	929	1442	1907
11		545	1174	1786	
pm	full	667	1418	2198	2702
12		545	1174	1786	
pm	empty	433	929	1442	1907
13		189	403	618	807
14		56	135	206	269
pm	Nominal load	11	12	12	12
15	End of test	0	0	0	0

Note that intermediate loads between reservoir empty and full were only applied when circumstances permitted

The long-term test specimens were loaded with a constant daily increment for five days and held at constant stress over weekends to monitor additional creep deflection. The same limiting stresses used in the short-term tests were applied. The stress histories for the long-term specimens are depicted on graphs 13.1 and 13.2. Loading rate for the 5% specimens was 11.1 kPa per day effect of cyclic loading. Short-term tests were conducted on specimens subject to the standard curing conditions and tested at ages of 56, 140 and 252 days.

TABLE 13.1

LOADING SEQUENCE FOR OEDOMETER TESTS

Day	Loading State	Stress (kPa) for Cement Content 5%	10%	15%	20%
1	Start of test	0	0	0	0
	Nominal load	11	12	12	12
		56	135	206	269
2		189	403	618	807
3	End construction	433	929	1442	1907
4		545	1174	1786	
pm	Reservoir full	667	1418	2198	2702
5		667	1418	2198	2702
6		667	1418	2198	2702
7		545	1174	1786	
pm	empty	433	929	1442	1907
8		545	1174	1786	
pm	full	667	1418	2198	2702
9	Inundate	667	1418	2198	2702
10		545	1174	1786	
pm	Reservoir empty	433	929	1442	1907
11		545	1174	1786	
pm	full	667	1418	2198	2702
12		545	1174	1786	
pm	empty	433	929	1442	1907
13		189	403	618	807
14		56	135	206	269
pm	Nominal load	11	12	12	12
15	End of test	0	0	0	0

Note that intermediate loads between reservoir empty and full were only applied when circumstances permitted

The long-term test specimens were loaded with a constant daily increment for five days and held at constant stress over weekends to monitor additional creep deflection. The same limiting stresses used in the short-term tests were applied. The stress histories for the long-term specimens are depicted on graphs 13.1 and 13.2. Loading rate for the 5% specimens was 11.1 kPa per day during the 5 day loading week, equivalent to a rate of construction of approximately 0.52 m per day, which is unlikely to be exceeded in practice. Loading rate for the 15% specimens was 27.5 kPa per day, equivalent to 1.29 m/d. Loading rate for reservoir operation was doubled as it was considered that reservoir filling could potentially proceed more rapidly than construction. Loading of long-term specimens commenced at a specimen age of 3 days.

All specimens tested in this program were disks 71.4 mm diameter and 20 mm deep in order to permit the use of standard oedometers. It had been observed in other parts of the test program that specimens tended to swell slightly during the initial stages of curing, thus suggesting that a considerable degree of restraint would be present in specimens compacted and tested in standard sample rings. Since the compressive strength of the material was considerably greater than the stresses to be applied in these tests, the specimens were tested unconfined. This provided minimum vertical restraint and lateral confinement and was expected to lead to maximum settlement.

In order to examine the effect of restraint and confinement, a number of tests were undertaken using specimens compacted and cured in specimen rings. Additional specimens, both confined and unconfined, were tested with top and bottom platens of reduced diameter -- a form of plate loading test. Platen diameters of 50 and 39 mm were used.

Specimens cured in the leaching bath and subject to leaching in the leaching cells described in chapter 9 were also tested as part of this program. Further specimens were tested after soaking in water for 7 days and after being subject to saturation under vacuum as described in chapter 11.

A number of unstabilized specimens were tested for comparison with the results for the stabilized specimens. Two specimens were tested unconfined at the same stress levels as used for the specimens with 5% cement. Two confined specimens were tested at the stress levels corresponding to the specimens with 15% cement. This maximum stress corresponds to the maximum vertical stress that would occur at the base of a 100 m high earthfill embankment.

13.2 APPARATUS

Standard, soil mechanics, oedometers were used for these tests. The machines were modified by the addition of a second loading platform on the load hanger to facilitate interchange of weights. Certain of the older machines were modified by attachment of the supporting pillar for the deflection gauge directly to the cell in order to minimise errors due to movement of the frame of the testing machine.

The cells were sealed against evaporation of water by several layers of the same laboratory film used to seal the leachate collection bottles described in chapter 9. This film was covered with packaging tape to exclude light and hold the film in place. Tape was also used to seal the film to the top platen around the loading ball so that it did not interfere with the ball seat. Water for flooding the specimens was supplied in the normal manner through bottom connections to the cells. Plate 13.1 illustrates two cells and testing machines.

The 71.4 x 20 mm disk specimens were statically compacted in a similar manner to the standard 37.5 x 75 mm cylinders used in other parts of the program. A mould was designed that permitted specimens to be compacted and extruded in the same way as the standard cylinders or to be compacted directly into the standard oedometer sample rings. Additional rings were fabricated to permit specimens to be compacted and cured in the rings. The mould is illustrated in plate 13.2 and additional details are furnished in appendix C.

PLATE 13.1
LONG-TERM SETTLEMENT TESTS IN PROGRESS

PLATE 13.2
MOULD FOR COMPACTION OF DISK SPECIMENS

13.3 TEST PROCEDURES

Specimens were statically compacted, humid cured for 7 days and then sealed in foil and wax until required for testing. Long-term specimens were removed from the humid chamber after 3 days and placed in the testing machine.

Prior to testing, cells were prepered by filling the distribution channels beneath the porous disks with water. The porous disks were soaked and surface dried in order to prevent desiccation of the specimens. The specimens were placed in the cells and the cells assembled. Plastic annuli were used to locate the 39 and 50 mm diameter porous disks concentrically with the specimens in the tests with reduced platen diameter. The outer shells of the oedometer cells were positioned and the covering of laboratory film and packaging tape was sealed to the top of the platen.

The cell was then placed in the testing machine and a zero load reading taken. The nominal load was applied and loading commenced in accordance with the sequence in table 13.1 for the short-term specimens or as described in section 13.1 for the long-term specimens.

Initially, deflection readings were taken immediately after application of load and at intervals thereafter. However, it was found that most of the settlement occurred virtually instantaneously and readings were subsequently taken before the application of the next load increment, or daily, as required.

The long-term tests included periods of constant stress of 2 to 4 weeks at reservoir full and empty in order to monitor creep effects. These specimens were accidentally inundated after first "filling" instead of after second filling as applied to all short-term specimens.

13.4 TEST RESULTS

In the light of the results for the leaching tests described in chapter 9, the data for the specimens cured in the leaching bath and for those subject to leaching in the leaching cells was not processed.

The settlement results in this section are presented in terms of strain as it was felt that this provided the best basis for comparison with concrete and the results of the triaxial tests. The use of soil mechanics conventions and void ratio was felt to be inappropriate as the volume of the hydrated cement paste was unknown. Void ratios for the material, calculated on the basis of a specific gravity of cement of 3.15 (USBR 1975), are 0.427, 0.436, 0.444 and 0.452 for 5, 10, 15 and 20% cement respectively.

Load and settlement are plotted in the form of stress - strain curves with strain on the horizontal axis and compression positive. If viewed from the right, these plots conform to the soil mechanics convention of pressure on the horizontal axis and compression negative.

In interpreting these graphs it should be noted that the data was observed with dial gauges reading to 0.002 mm per division which corresponds to a strain of 0.1×10^{-3}. This is half a division on the strain axis of graphs 13.1 and 13.2 and one division on graphs 13.3 to 13.11. Small visible deviations on these graphs are therefore of similar magnitude to the limits of repeatability for deflection readings. Although no error analysis was performed, it is considered that variations of similar magnitude could result from temperature effects. Elastic deformation of loading balls and platens and porous disks may also have a sl[illegible]ht influence on the observed results but are considered to be [illegible]ificant.

The small observed deflection changes were al[illegible] susceptible to disturbance of the testing machines by accidental bumping or vibration and to inaccuracies due to friction in dial gauges and testing machines.

It is considered that the above factors account for a number of the random trends shown in the accompanying graphs and that the apparatus used is insufficiently sensitive for the measurement of the small secondary deflections observed with this material. However, the primary deflections are well defined. Tests with longer specimens would be required for examining creep behaviour in detail.

The data observed on one testing machine is consistently random and variable for all tests conducted on it. Accordingly, while the data is plotted on the graphs, it has been ignored in determining the modulus values presented at the end of this section. The affected specimens are indicated in the corresponding discussion.

It should be noted that the observed deflections can be significantly affected by the presence of loose particles on the surface of the specimen which could lead to high settlements under the first few load increments.

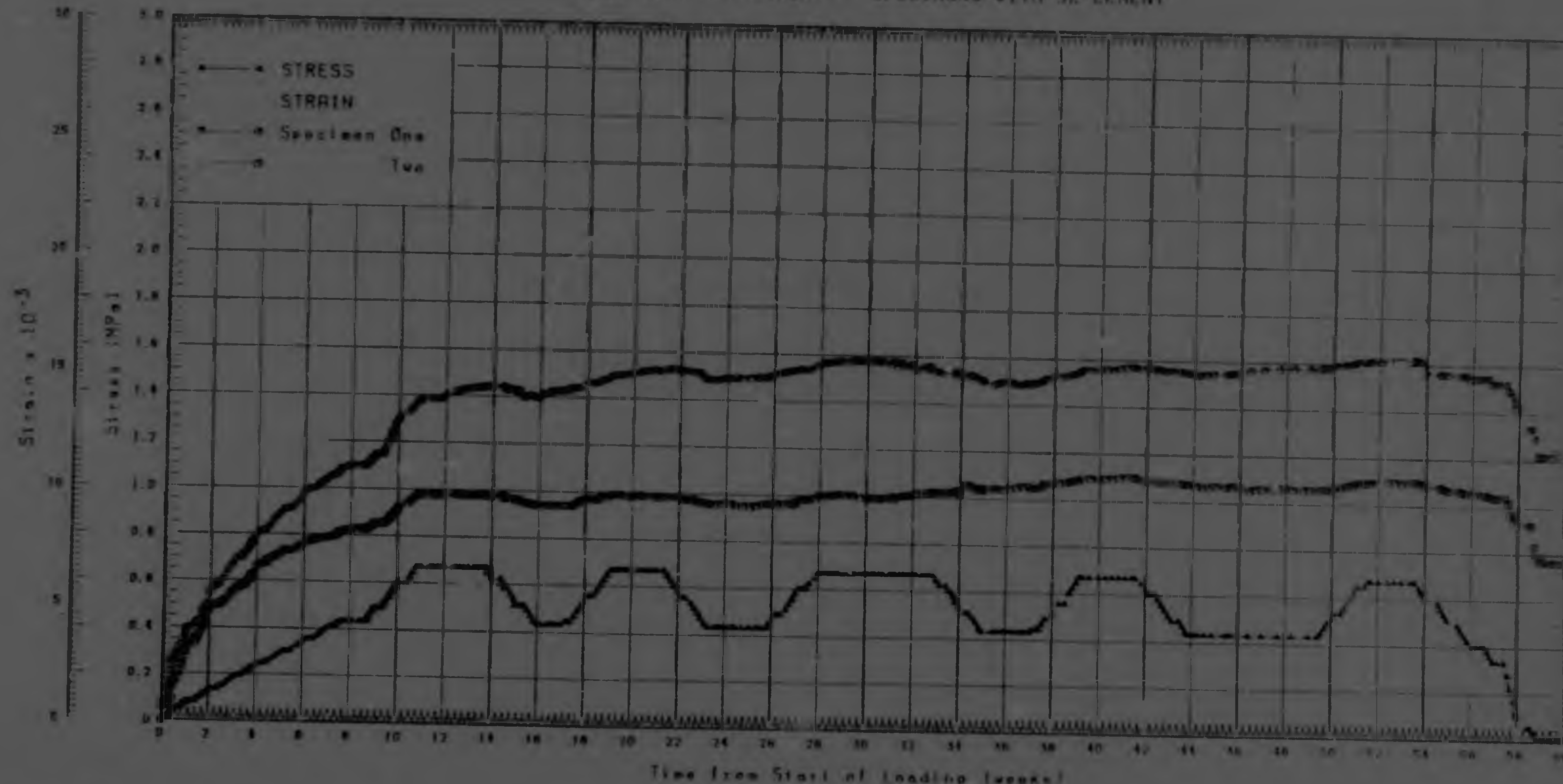
13.4.1 Long-Term Settlement Tests

Graphs 13.1 and 13.2 depict the stress - time and strain - time data for the long-term tests plotted on the same axes for specimens with 5 and 15% cement respectively. The stress - strain data for these tests is plotted on graph 13.3

As would be expected, rate of settlement decreases with time during initial loading as curing of the material proceeds. Between 75 and 88% of settlement to first filling occurs by end of construction. Thereafter, cyclic reservoir operation generates relatively small changes in strain. The 5% specimens show a small progressive increase in strain which levels off after the

GRAPH 13.1

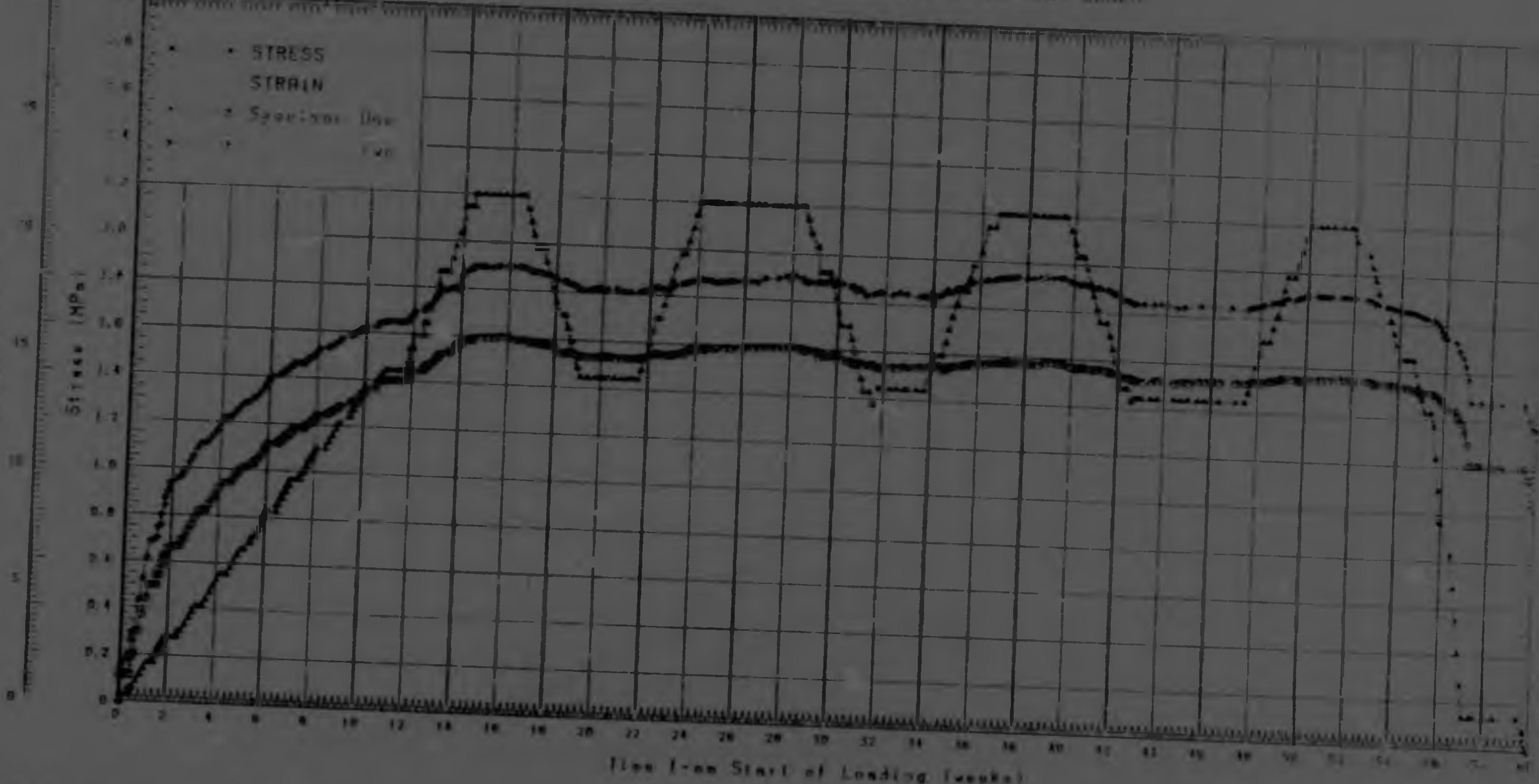
LONG-TERM SETTLEMENT -- SPECIMENS WITH 5% CEMENT



LT-STRAIN.PLT -- 13/8/88-8-038-- 1.8 LTCEMF.DAT, 7.1

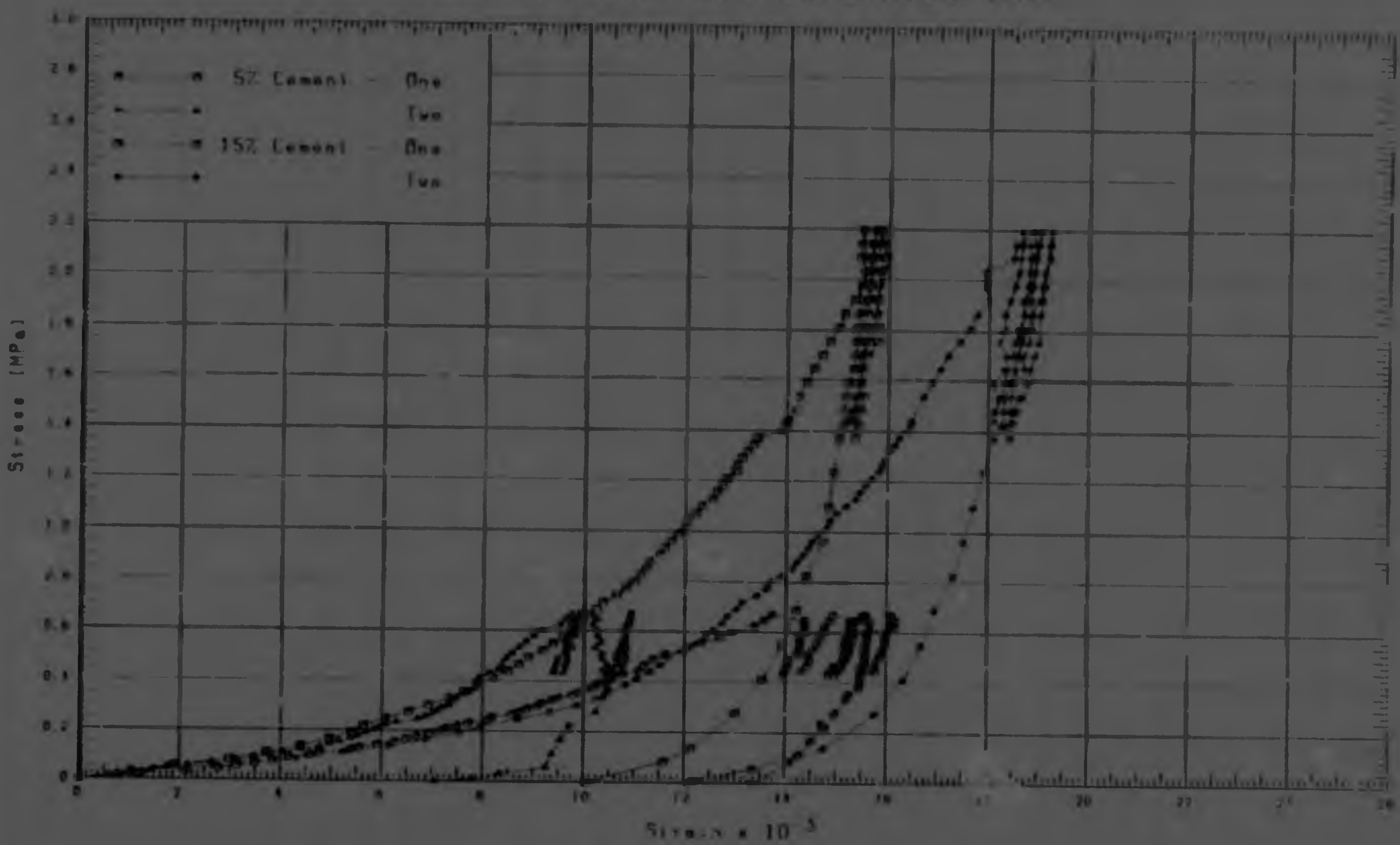
GRAPH TS.2

LONG-TERM SETTLEMENT — SPECIMENS WITH TS2 CEMEN



GRAPH 13.3

STRESS vs STRAIN FOR LONG TERM SETTLEMENT TESTS



fourth cycle, no particular trend is evident for the 15% specimens. The unloading curves indicate a considerable degree of irrecoverable consolidation and / or creep which is consistent with the above observation.

The stress - strain plots for the reservoir cycles for specimens with 15% cement confirm the lack of any significant creep trend under cyclic loading and it is considered that the spread between the repeat loading and unloading curves largely reflects the limit of repeatability referred to previously.

The stress - strain plots for the 5% specimens reflect a greater susceptibilty to machine disturbance due to the lower applied stresses as reflected by a number of discontinuities. However, it is considered that a degree of creep is indicated as a progressive displacement with each reservoir cycle is evident. This compares with the 15% specimens where the last reservoir cycle lies to the left of a number of preceding cycles.

A slight change in strain under constant stress is evident in both cases, but is small and is unlikely to be of significance in a soil-cement embankment dam.

It is notable that the initial stress - strain curves for specimens with 5 and 15% cement shown on graph 13.3 are effectively identical. However, in view of the different rates of loading applied, it is doubtful whether any significance can be attached to this observation.

13.4.2 Standard Short-Term Tests

The standard short-term tests involved the application of the loading sequence listed in table 13.1 to unconfined specimens loaded uniformly over the entire surface of the specimen. Where time permitted, specimens were subject to additional loading cycles after completion of the standard sequence.

Graph 13.4 depicts the short-term results for the unstabilized specimens. A compressed strain axis, with maximum value 160 x 10^{-3} is used for this plot. Strain axis limit for all other stress - strain plots is 26 x 10^{-3}. Graphs 13.5 to 13.8 depict the short-term data for the stabilized specimens according to cement content and tested at various ages.

As mentioned above, the data obtained on one testing machine was ignored in interpreting the results. The affected specimens are:-

a. The second specimen with 5% cement tested at age 252 -- the data plotted with an x on the right of graph 13.5.

b. The second specimen with 10% cement tested at age 140 -- the randomly distributed data plotted with a diamond symbol on graph 13.6.

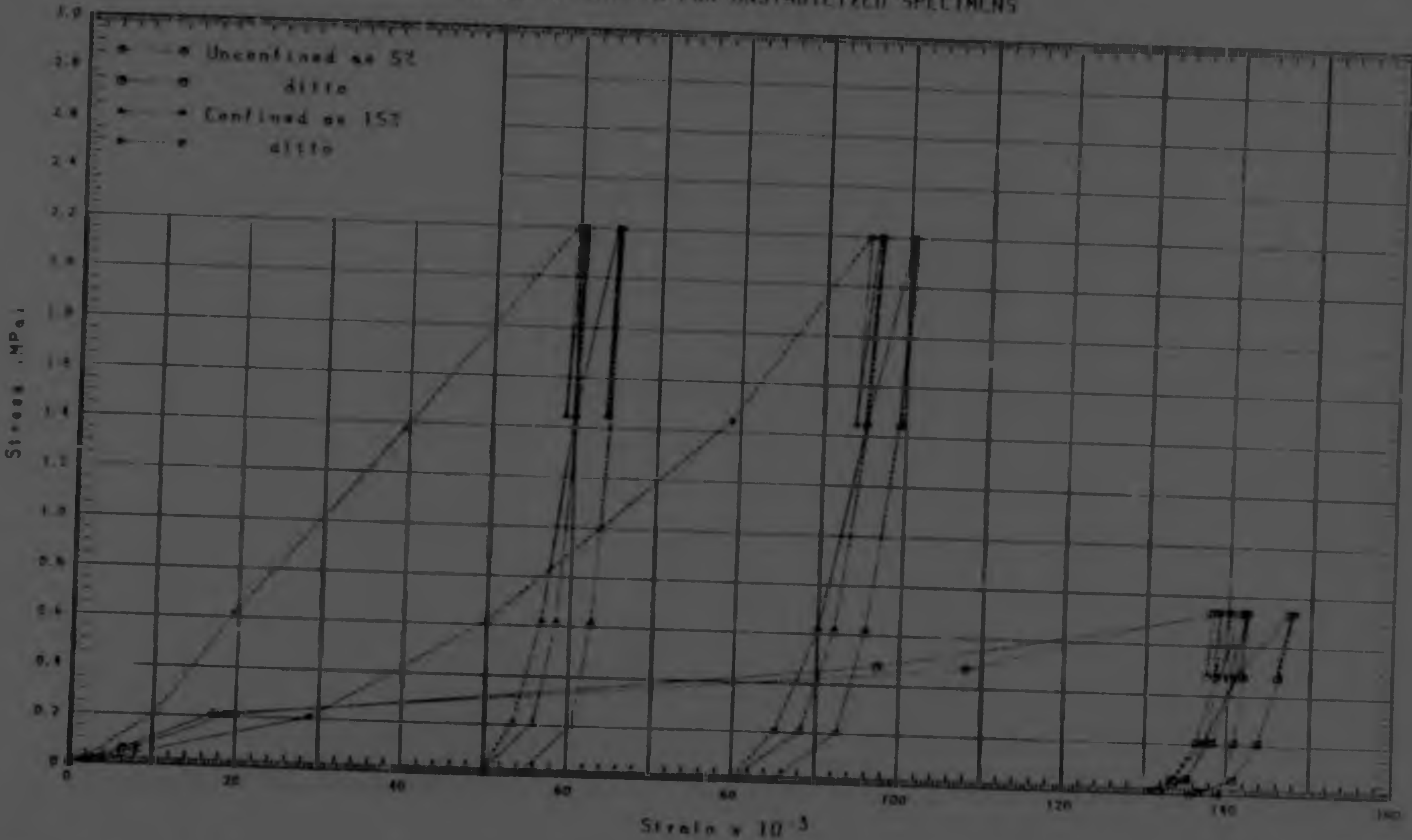
c. The second specimen with 15% cement tested at age 140 -- the randomly distributed data plotted with a diamond symbol on graph 13.7.

The unconfined, unstabilized specimens exhibit considerably more deflection than the confined specimens and observed deflection for the confined unstabilized specimens is 3 to 5 times the maximum for the unconfined stabilized specimens.

General form of all results is similar to that for the long-term test results. It is notable that all results for stabilized specimens lie in a similar band irrespective of cement content. The maximum strain increases slightly with increasing cement content but this increase is small in comparison to the range of observed results. There is a tendency for maximum strain to decrease with increasing age at test but this trend is masked by the variation between nominally identical specimens.

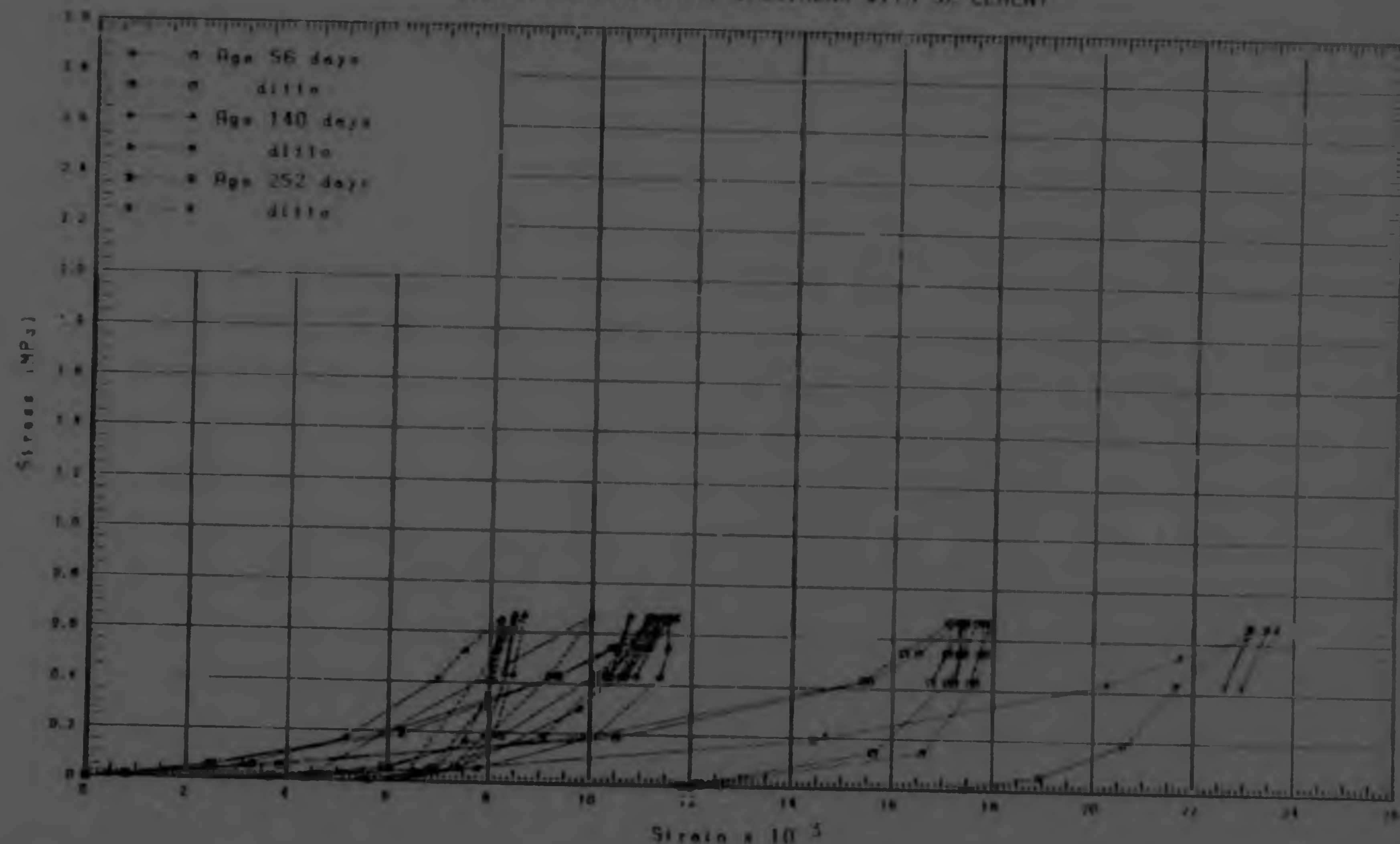
GRAPH 13.4

STRESS vs STRAIN FOR UNSTABILIZED SPECIMENS



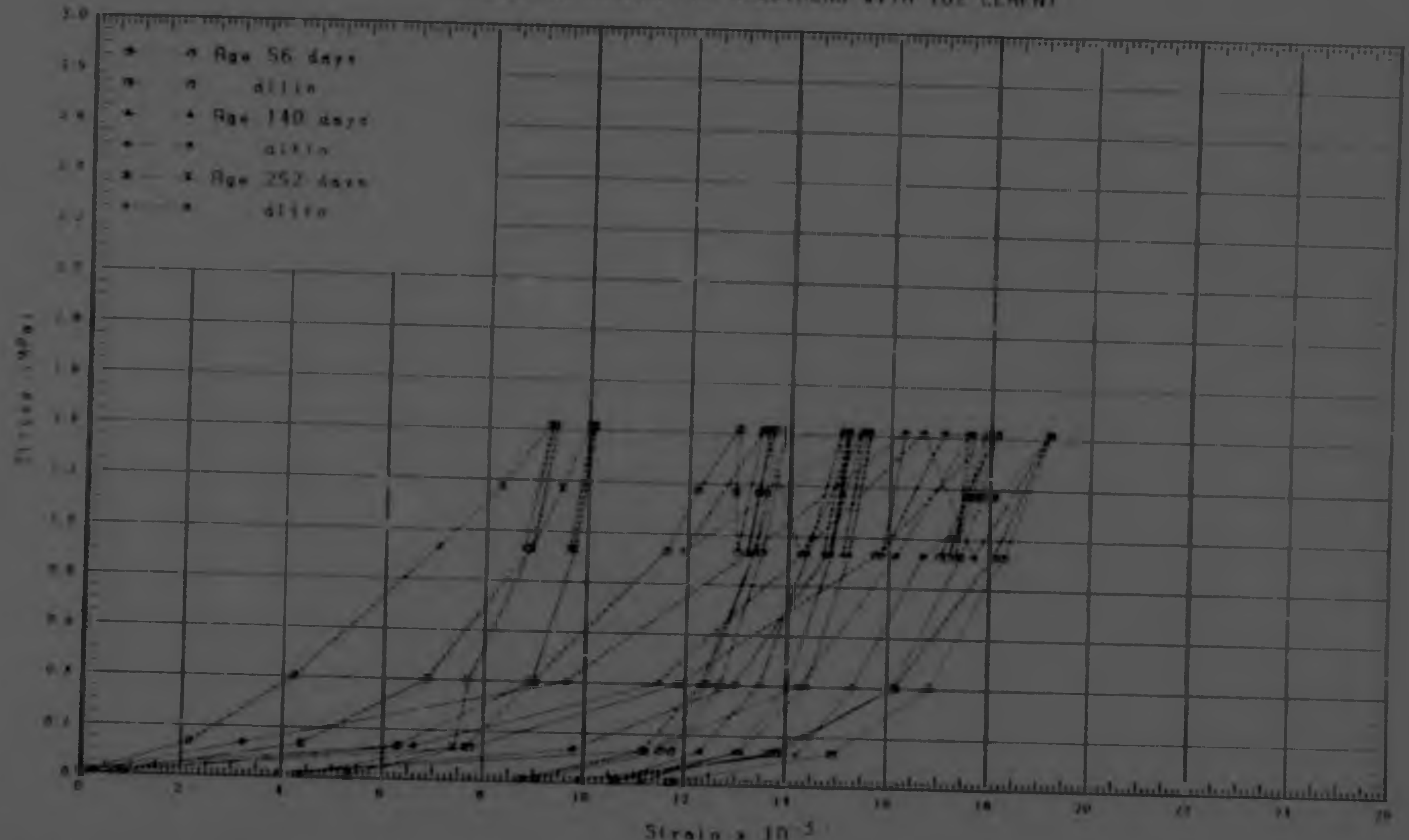
GRAPH 13.5

STRESS vs STRAIN FOR SPECIMENS WITH 5% CEMENT



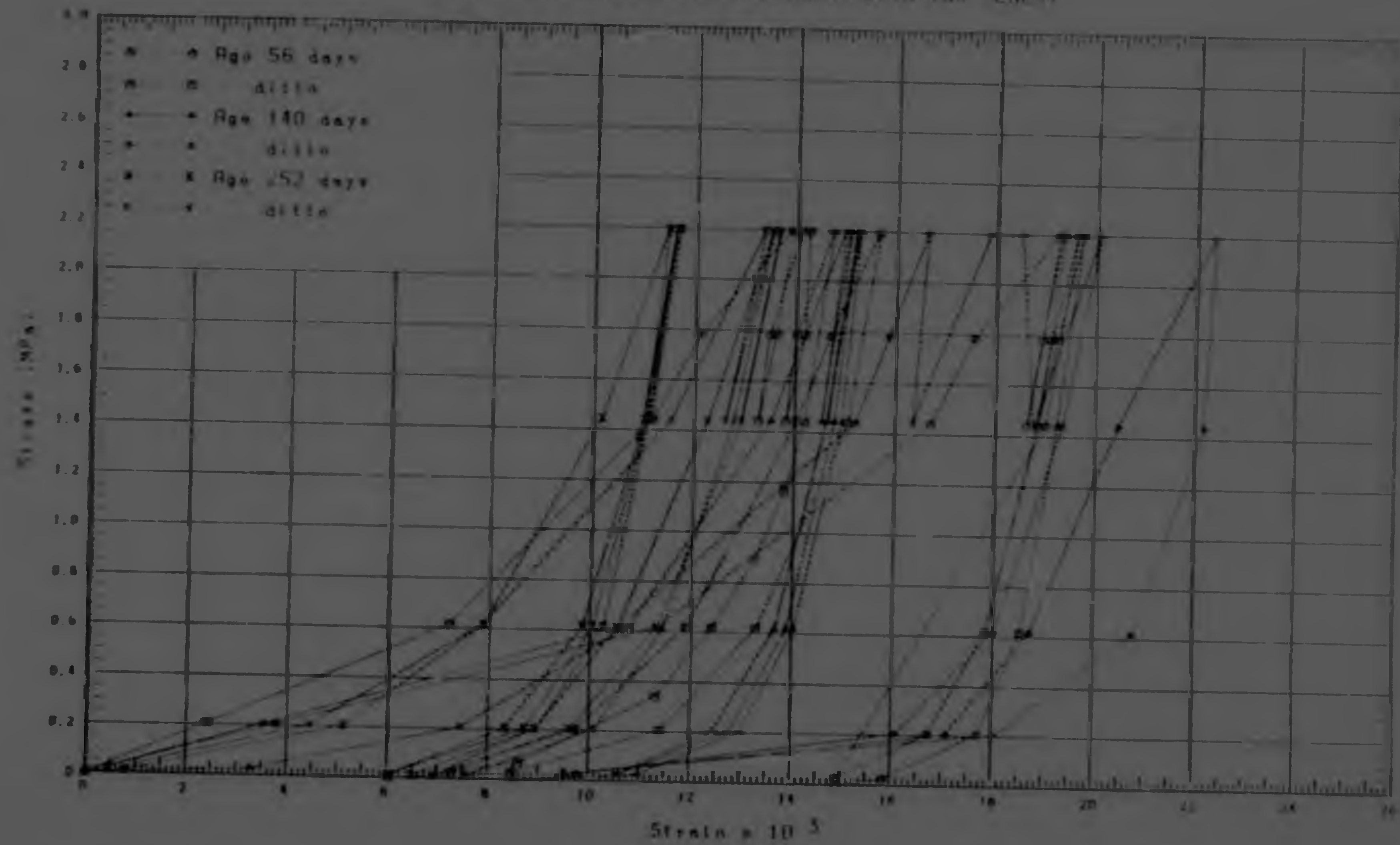
GRAPH 13.E

STRESS vs STRAIN FOR SPECIMENS WITH 10% CEMENT



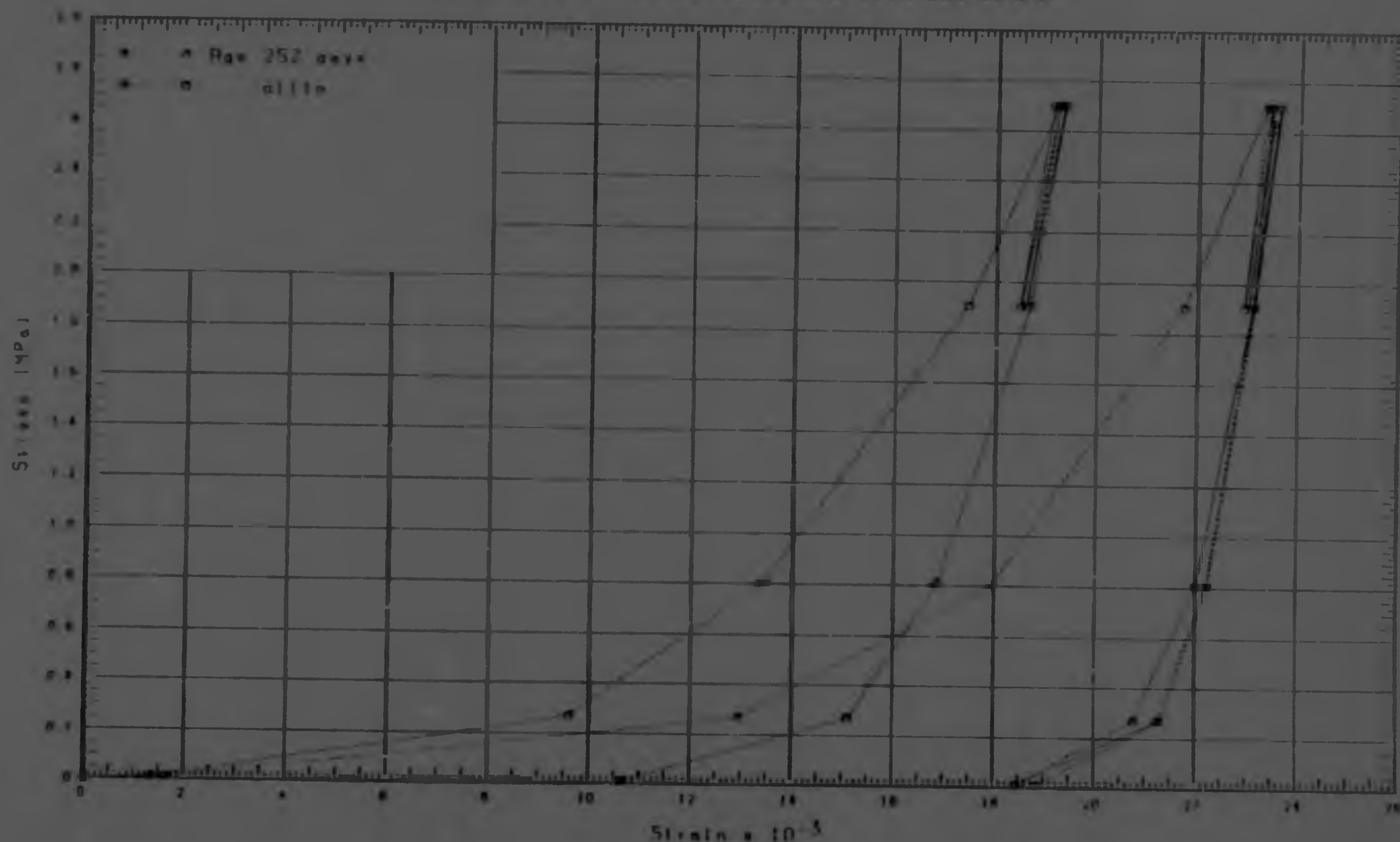
GRAPH 13.2

STRESS vs STRAIN FOR SPECIMENS WITH 15% CEMENT



GRAPH 13.8

STRESS vs STRAIN FOR SPECIMENS WITH 20% CEMENT



The maximum strain in all cases is less than 24×10^{-3}, or 2.4%, most of which occurs by the end of construction. Since the cement content in most zones of the embankment will be determined by factors other than compressive strength, it is expected that average consolidation in most zones of the embankment will be of the order of 1% or less.

The spread of results for reservoir cycles is generally small but some data exhibits sudden displacements which are attributed to machine disturbance. A slight hysteresis loop is evident for specimens subject to reloading from zero and this is sometimes associated with a significant additional displacement which may reflect an additional consolidation or may be attributed to machine disturbance.

No increase in strain following flooding is discernible and it is considered that the incorrect flooding of the long-term specimens after first filling is unlikely to have had any effect on the results. This is consistent with the low permeability reported in chapter 9 which would be expected to be reduced as a result of the consolidation indicated in these tests.

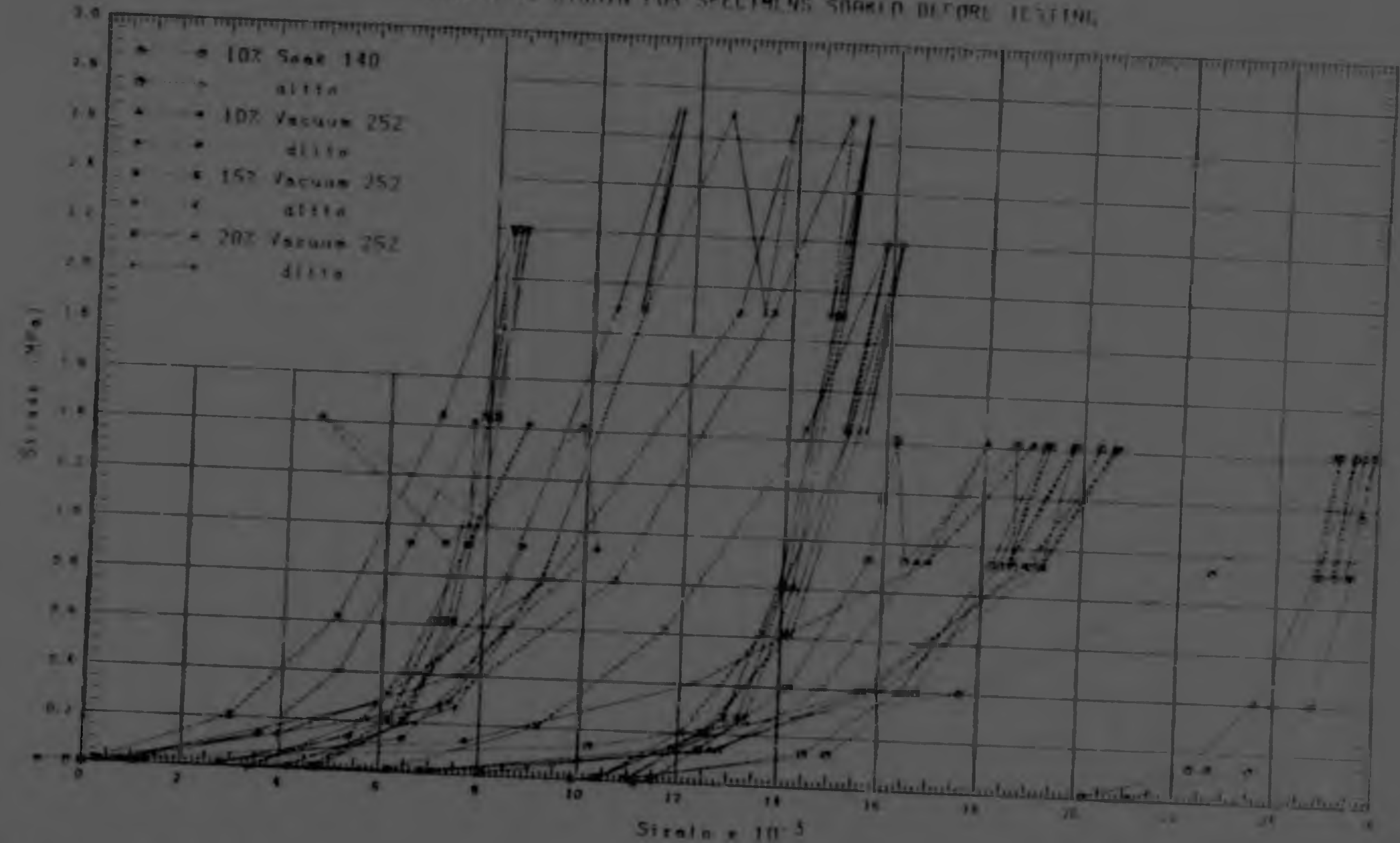
The spread of results covers the range of the long-term test results and the stress - strain curves are of generally similar shape. It is therefore considered that the standard test at an age of 56 days provides an acceptable measure of the long-term settlement characteristics of the material.

13.4.3 Specimens Soaked Before Testing

A number of specimens were soaked before testing as control specimens for the leached and leach bath cured specimens the results of which were not processed. The results for these specimens are depicted on graph 13.9.

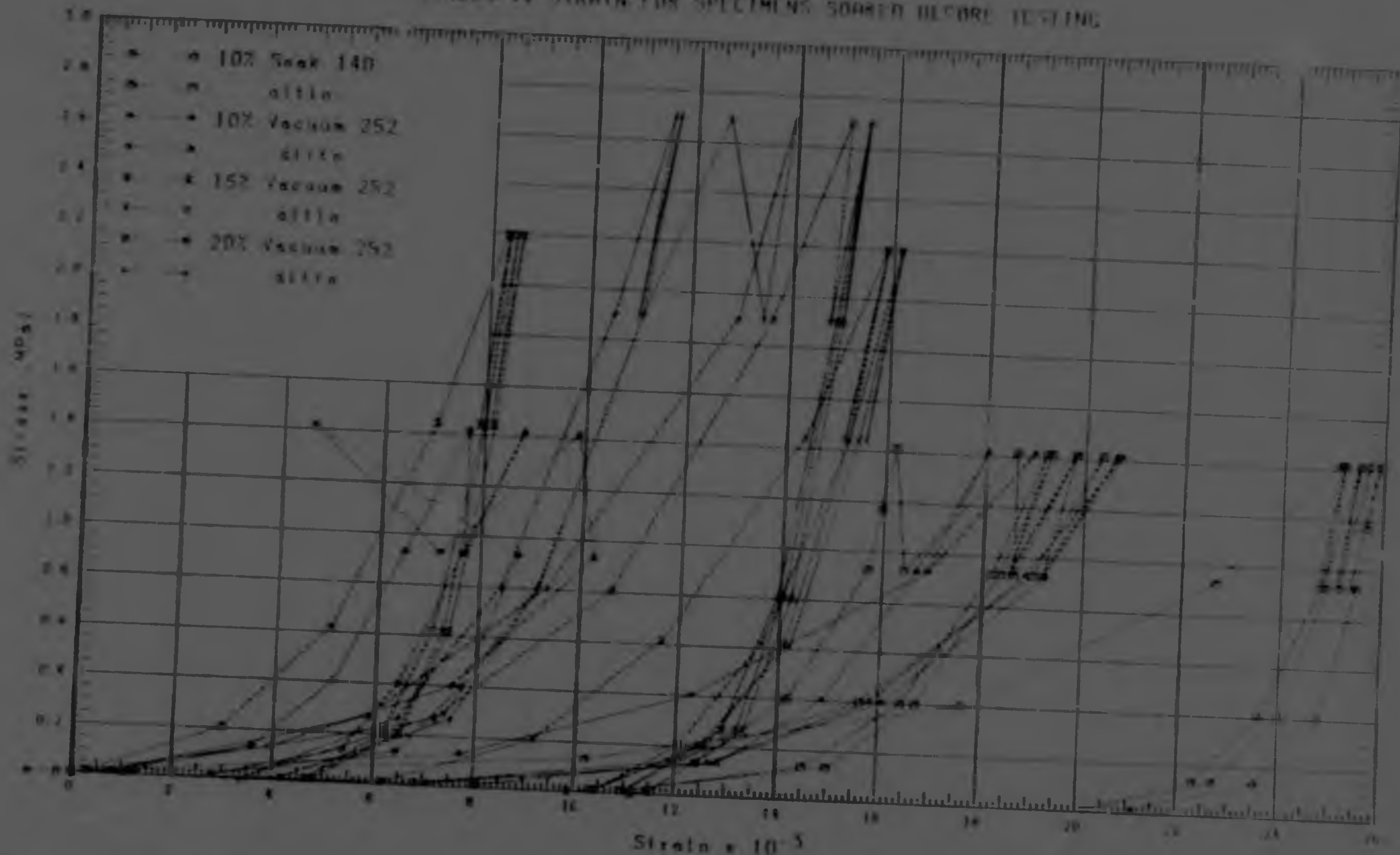
GRAPH 13.9

STRESS vs STRAIN FOR SPECIMENS SOAKED BEFORE TESTING



GRAPH 13.9

STRESS vs STRAIN FOR SPECIMENS SOAKED BEFORE TESTING



Two specimens with 10% cement were soaked for 7 days from age 133 prior to testing and these exhibit somewhat higher deflections than the corresponding standard specimens and the specimens subject to vacuum saturation. It is considered that this probably results from surface softening as a result of leaching.

The second specimen with 10% cement subject to vacuum saturation, plotted with a diamond symbol on graph 13.9, was tested on the machine referred to earlier. This data exhibits random behaviour and was accordingly ignored.

The results for the remaining specimens subject to vacuum saturation and tested at age 252 lie in the same band as those for the standard test specimens. It is therefore concluded that increasing the moisture content of the material has no significant effect on its consolidation characteristics.

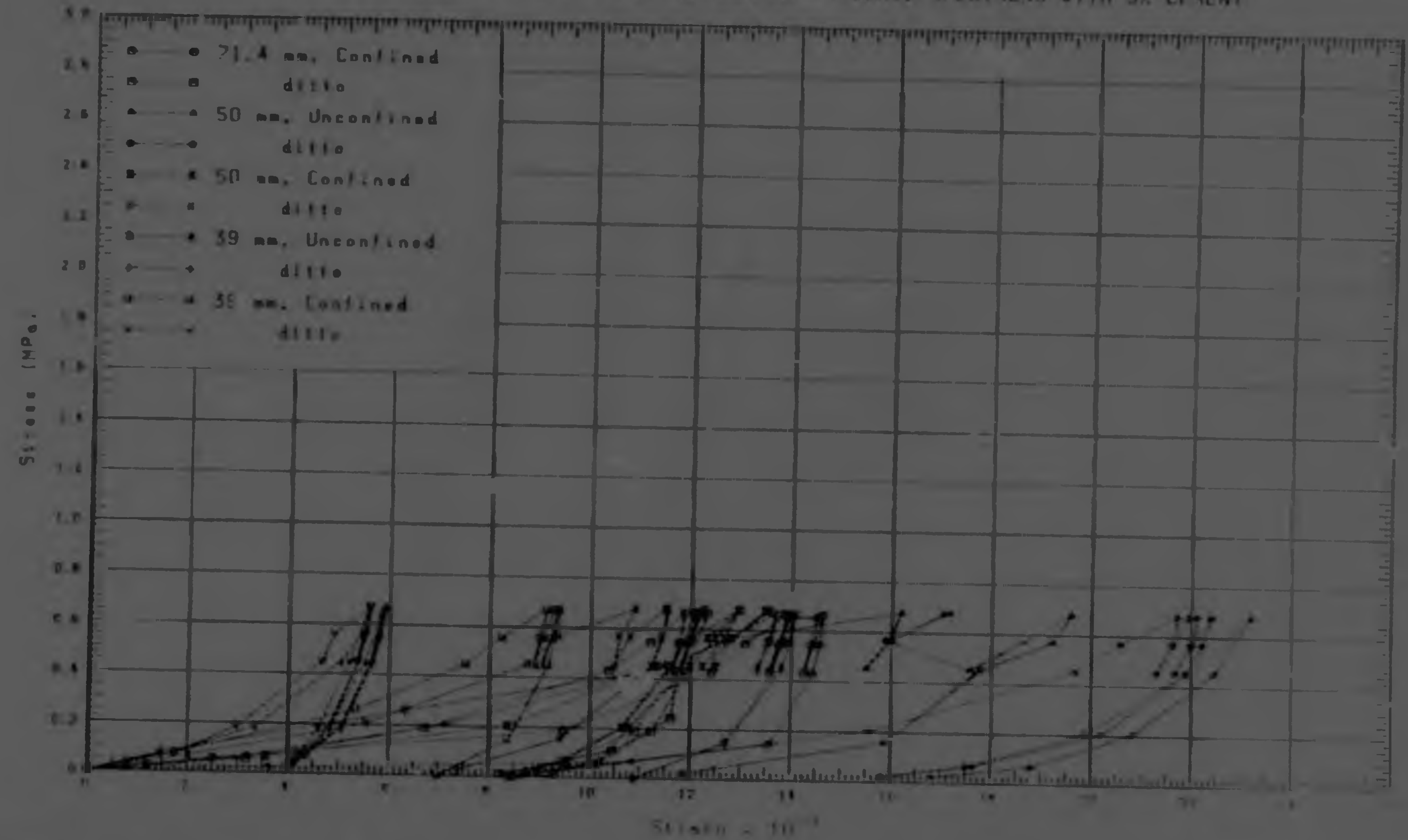
13.4.4 Non-Standard Loading Conditions

Tests with non-standard loading conditions were conducted at cement contents of 5 and 15% cement to correspond to those used for the long-term tests. The results for these tests are depicted on graphs 13.10 and 13.11.

With the exception of the two 5% specimens tested unconfined with a 50 mm platen, all results lie in the same band observed for the standard and soaked specimens. In the case of the first of these 5% specimens, the test record indicates that the machine may have been disturbed on at least two occasions and this may account for the random distribution of part of the data. In the case of the second specimen, very large settlements were noted on application of the nominal load and the first load increment and this zero error accounts for the excessive deflection reflected on graph [illegible].

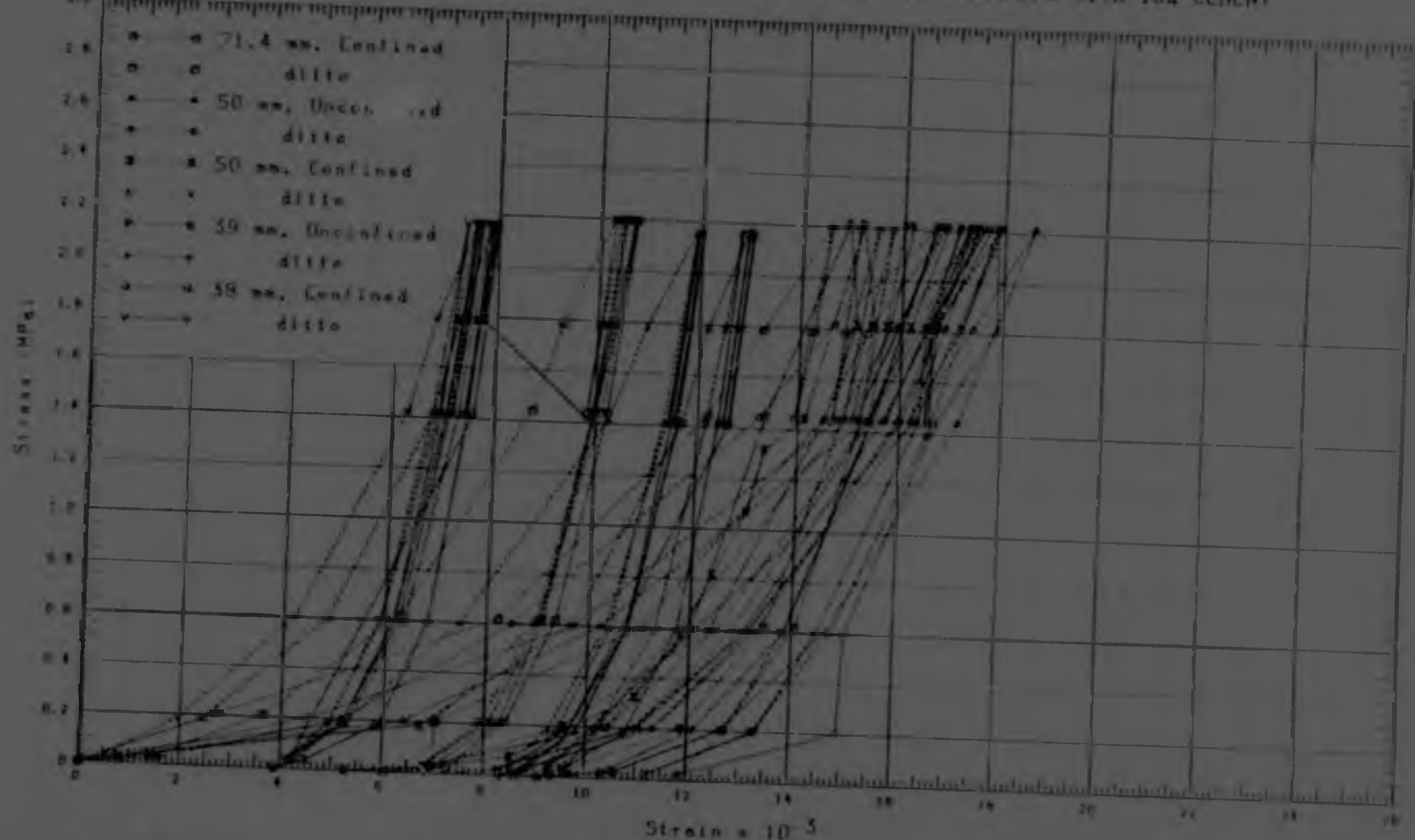
GRAPH 13.10

STRESS vs STRAIN FOR NON-STANDARD TEST CONDITIONS, SPECIMENS WITH 5% CEMENT



GRAPH 13.11

STRESS vs STRAIN FOR NON STANDARD TEST CONDITIONS, SPECIMENS WITH 15% CEMENT



While the results indicate a possible tendency for deflections to reduce with reduced platen diameter and increased confinement, this tendency is masked by the scatter between nominally identical specimens. It is however apparent that these factors have a relatively minor influence on the consolidation behaviour of the material in these tests. It is therefore considered that the standard loading conditions adopted in this program provide a reasonable assessment of the settlement characteristics of the material.

13.4.5 Characteristic Moduli

From an examination of the data in graphs 13.3 to 13.11, it was concluded that the consolidation behaviour of soil-cement could best be represented by three characteristic moduli for design purposes. These moduli were defined as follows:-

a. The modulus corresponding to the interval -- start of construction to end of first filling -- the "construct and fill" modulus. This was taken as the secant modulus from zero load to end of consolidation after application of the reservoir full load for the first time and was extracted from the experimental data. This modulus gives an indication of the total settlement to be expected.

b. The modulus corresponding to the interval -- end of construction to end of first filling -- the "first filling" modulus. This was taken as the secant modulus from end of consolidation after application of the final embankment load to end of consolidation after application of the reservoir full load and was extracted from the experimental data. This modulus gives an indication of the settlement to be expected when the reservoir is first filled and can be used in conjunction with the previous modulus to estimate the end of construction settlement.

As discussed in section 13.1, it should be noted that the end of construction stresses applied in this test program are not the maximum stresses that would occur under the center of the embankment. This modulus thus provides a basis for estimating the settlement at any point in the embankment relative to the reservoir full stress at that point.

It should also be noted that these tests assume a vertically applied compressive stress whereas the maximum principal stress at the toe of an embankment is inclined outwards, thus leading to reduced confinement and possibly increased settlement. However, the results of the tests on specimens with increased confinement indicate that this factor is unlikely to be of significance.

c. The modulus corresponding to repeated filling and draining of the reservoir -- the "reservoir operation" modulus. This was estimated by eye from the plotted data using a parallel ruler to estimate the average slope for all cycles plotted, greater weight was given to later cycles. This modulus gives an indication of the largely elastic movement to be expected during reservoir operation.

The average values of these moduli for each pair of test specimens, excluding those specimens rejected previously, are listed in table 13.2.

From consideration of the discussion in the previous sections, it is apparent that the initial consolidation is largely independent of cement content if the stress applied bears a constant relationship to the unconfined compressive strength of the material. In addition, increased curing and increased confinement, both of which result in an increase in compressive strength also lead to a reduction in consolidation. Both these factors indicate that the modulus of the material increases in proportion to the compressive strength of the material. As discussed in section 11.4.3, a linear relationship on logarithmic axes has been

TABLE 13.2
SUMMARY OF RESULTS -- OEDOMETER TESTS

Code	Graph No	NOTES	Moduli (MPa) Construct and Fill	First Filling	Reservoir Operation	Comp Str MPa
1	13.3	Long-term, 5% cement, start age 3	57	112	952	N/A
2		15% cement	127	355	1415	N/A
3	13.4	0%, unconfined, load as 5%, age 252	5	8	431	0.3
4		0%, confined, load as 15%, age 252	30	44	971	N/A
5	13.5	Standard, 5% cement, age 56	39	135	1161	3.5
6		age 140	69	154	987	4.5
7		age 252	57	101	1322	5.1
8	13.6	Standard, 10% cement, age 56	98	517	959	6.9
9		age 140	92	429	1500	8.9
A		age 252	148	333	1390	10.0
B	13.7	Standard, 15% cement, age 56	136	288	1412	10.5
C		age 140	147	528	2069	13.3
D		age 252	173	503	1292	14.1
E	13.8	Standard, 20% cement, age 252	29	479	1669	20.8
F	13.9	Soaked from age 133, tested 140, 10% cem	64	282	808	8.9
G		Vacuum saturated age 252, 10% cement	75	236	330	10.0
H		15%	202	555	1790	14.1
I		20%	187	682	1909	20.8
J	13.10	5% cem, age 56, 71.4 mm platen, confined	52	154	926	3.5
K		50 mm platen, unconfined	37	136	500	3.5
L		confined	58	163	497	3.5
M		39 mm platen, unconfined	81	181	835	3.5
N		confined	96	185	969	3.5
O	13.11	15% cem, age 56, 71.4 mm platen, confined	182	406	1463	10.5
P		50 mm platen, unconfined	129	260	697	10.5
Q		confined	130	305	1199	10.5
R		39 mm platen, unconfined	175	540	3039	10.5
S		confined	280	645	1875	10.5

Standard case is unconfined specimen with loading platen 71.4 mm in diameter.
Codes in first column refer to graphs 13.12 to 13.14.

TABLE 13.2
SUMMARY OF RESULTS -- OEDOMETER TESTS

Code	Graph No	NOTES	Moduli (MPa) Construct and Fill	First Filling	Reservoir Operation	Comp Str MPa
1	13.3	Long-term, 5% cement, start age 3	57	112	952	N/A
2		15% cement	127	355	1415	N/A
3	13.4	0%, unconfined, load as 5%, age 252	5	8	431	0.3
4		0%, confined, load as 15%, age 252	30	44	971	N/A
5	13.5	Standard, 5% cement, age 56	39	135	1161	3.5
6		age 140	69	154	987	4.5
7		age 252	57	101	1322	5.1
8	13.6	Standard, 10% cement, age 56	98	517	959	6.9
9		age 140	92	429	1500	8.9
A		age 252	148	333	1390	10.0
B	13.7	Standard, 15% cement, age 56	136	288	1412	10.5
C		age 140	147	528	2069	13.3
D		age 252	178	503	1292	14.1
E	13.8	Standard, 20% cement, age 252	129	479	1669	20.8
F	13.9	Soaked from age 133, tested 140, 10% cem	64	282	808	8.9
G		Vacuum saturated age 252, 10% cement	75	236	330	10.0
H		15%	202	555	1790	14.1
I		20%	187	682	1909	20.8
J	13.10	5% cem, age 56, 71.4 mm platen, confined	52	154	926	3.5
K		50 mm platen, unconfined	37	136	500	3.5
L		confined	58	163	497	3.5
M		39 mm platen, unconfined	81	181	835	3.5
N		confined	96	185	969	3.5
O	13.11	15% cem, age 56, 71.4 mm platen, confined	182	406	1463	10.5
P		50 mm platen, unconfined	129	260	697	10.5
Q		confined	130	305	1199	10.5
R		39 mm platen, unconfined	175	540	3039	10.5
S		confined	280	645	1875	10.5

Standard case is unconfined specimen with loading platen 71.4 mm in diameter.
Codes in first column refer to graphs 13.12 to 13.14.

reported by Abboud (1969) and was found to provide a good fit to the data obtained in the triaxial tests as presented in graph 11.10.

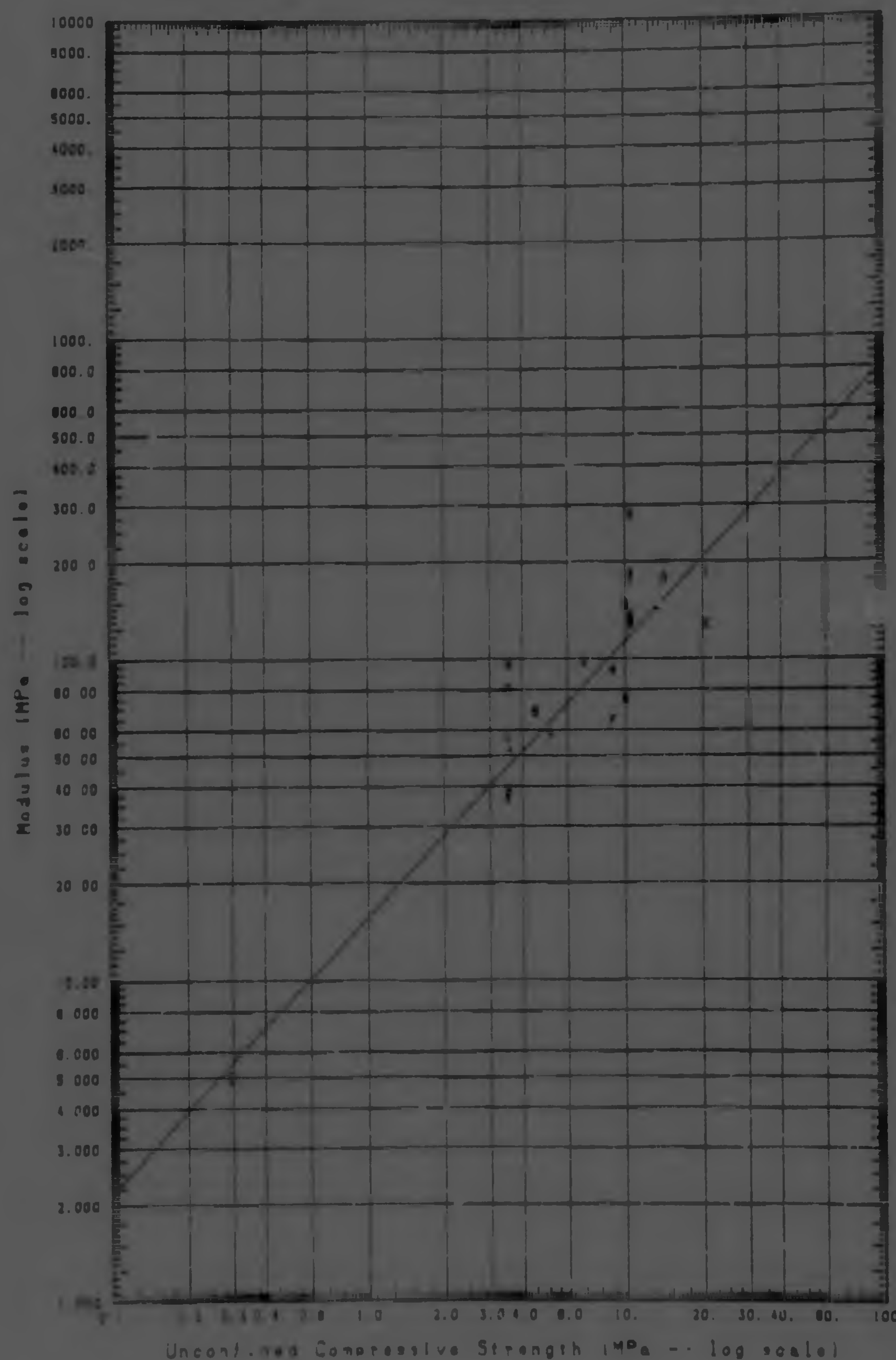
Accordingly, the soaked unconfined compressive strength for the material at the appropriate ages and cement contents was estimated from graph 10.3 and is presented in table 13.2. The three moduli discussed above are plotted against this strength in graphs 13.12 to 13.14 with log scales used on both axes. Table 13.2 also provides a key to the codes used in these graphs. Since no estimate of the confined compressive strength for the method of confinement used, was possible, these results are also compared with the standard unconfined compressive strength as are those for the saturated and soaked specimens. The regression lines on these graphs are calculated for the standard specimens subject to standard test conditions (points 3, 5 to 9 and A to E). The coefficients for these regressions, are considered acceptable in view of the scatter of results observed in this test program.

The regression lines for the three moduli discussed above are presented as dashed lines on graph 13.15 for comparison. The tendency for the modulus to increase with cement content is strongly apparent for all three moduli and particularly for the construct and fill and first filling moduli both of which correspond to the first loading cycle. Higher values are observed for the first filling modulus than for the construct and fill modulus. This is consistent with the definition of the two moduli and the behaviour referred to previously.

The regression line for the reservoir operation modulus is considerably flatter than that for the other two moduli and examination of the scatter of results reflected on graph 13.14 indicates that further observations for unstabilized specimens could indicate a more horizontal relationship. The observed scatter results from the fact that the values for this modulus were

GRAPH 13.12

MODULUS -- START OF CONSTRUCTION TO RESERVOIR FULL



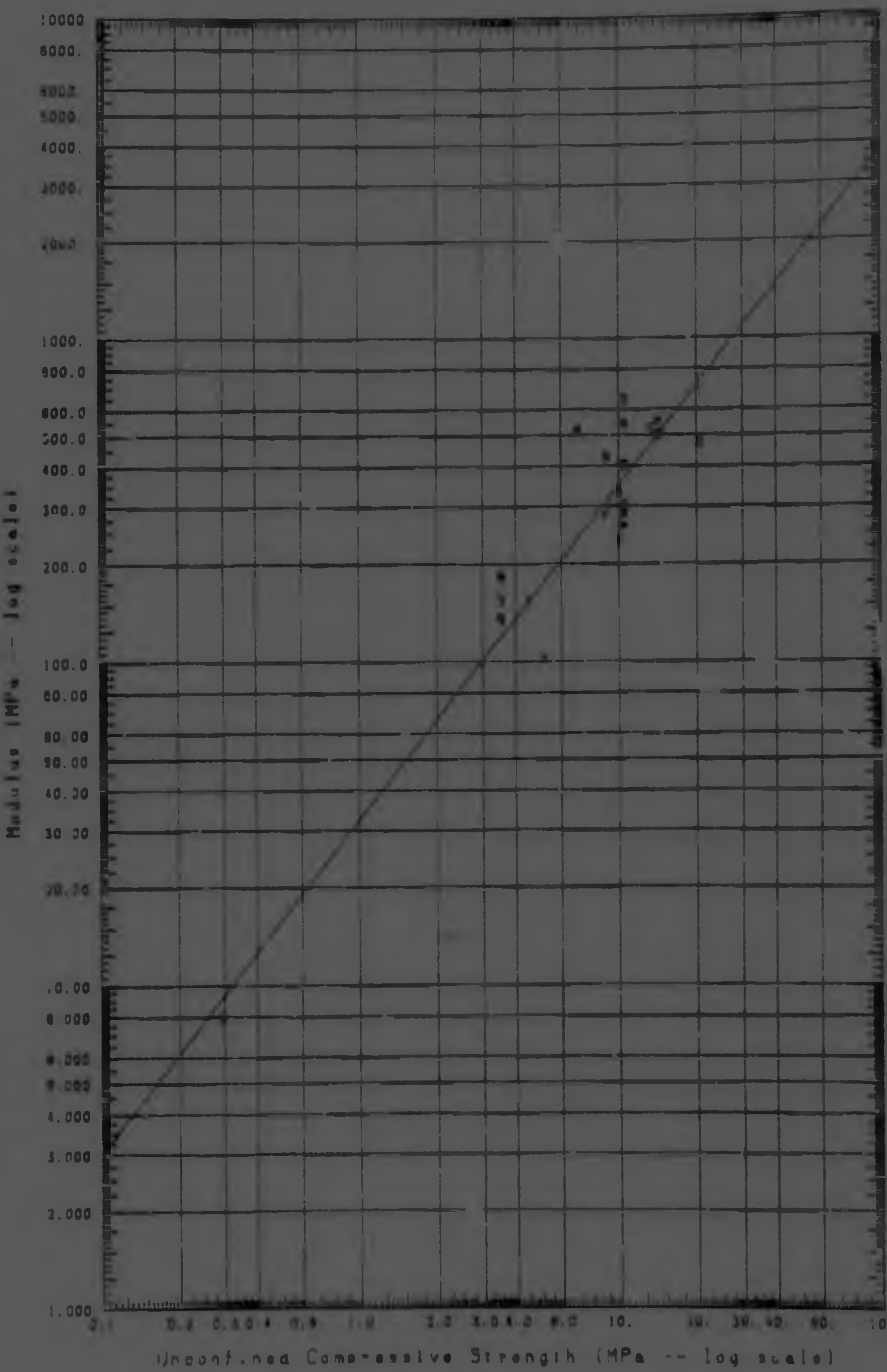
Regr'n : log y = 1.194 + 0.860 log x r^2=0.953

For individual codes see table 13.2

CONSMOD PLT -- 13/8/84

GRAPH 13.13

MODULUS -- FIRST FILLING FROM END OF CONSTRUCTION



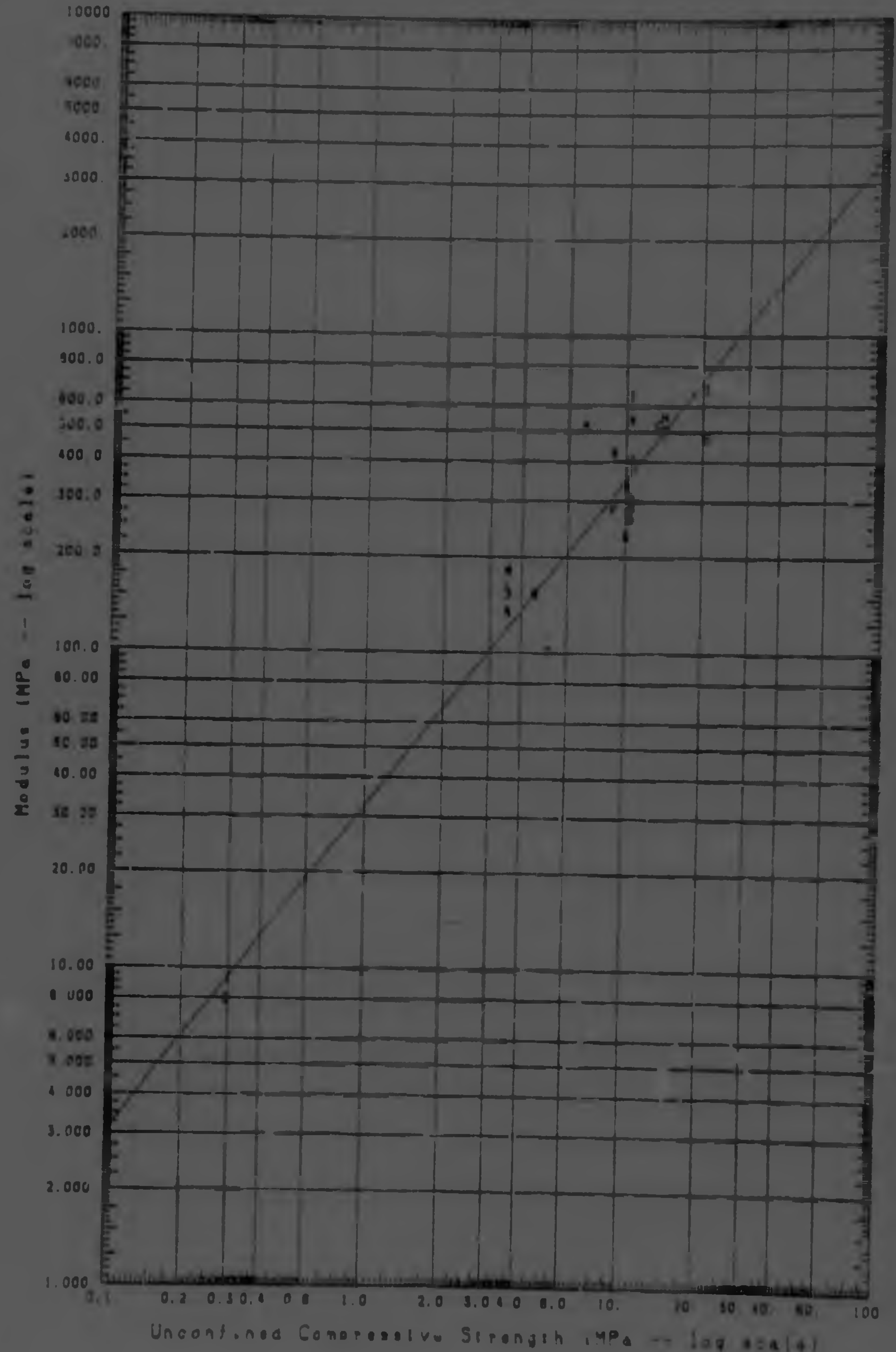
——— Regr'n : log y = 1.509 + 1.033 log x : r^2=0.92

For individual codes see table 13.2

LFILLMOD.PLT -- 13/8/84-8=1279-v 1 2 (SETLCOMF.DAT, 17)

GRAPH 13.13

MODULUS -- FIRST FILLING FROM END OF CONSTRUCTION

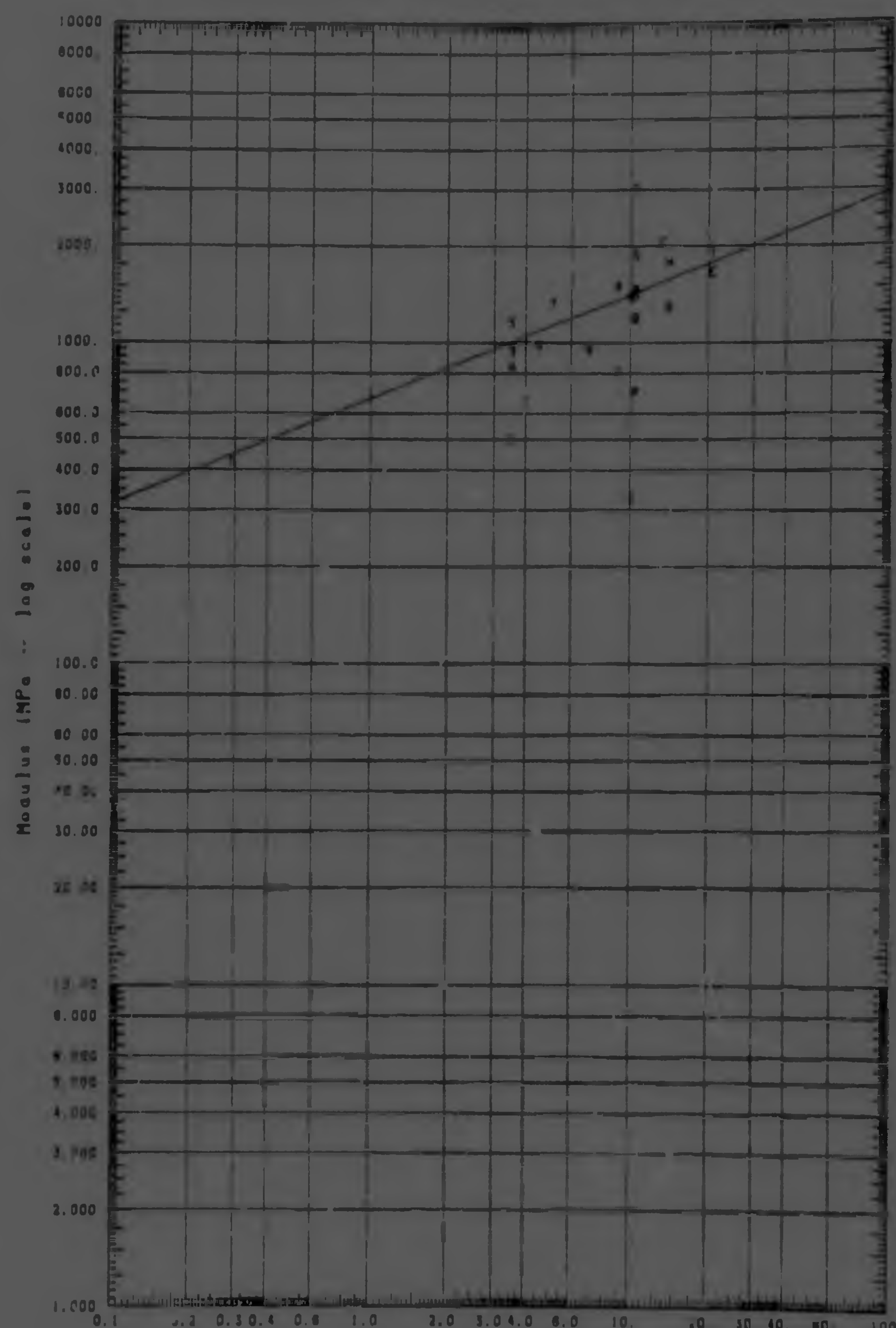


——— Regr'n : log y = 1.509 + 1.033 log x : r^2=0.92

For individual codes see table 13.2

LFILLMFO PLT -- 13/8/84-8=1279-- 1 2 (SETLCOMP DAT, 17)

GRAPH 13.14
MODULUS -- CYCLIC RESERVOIR OPERATION

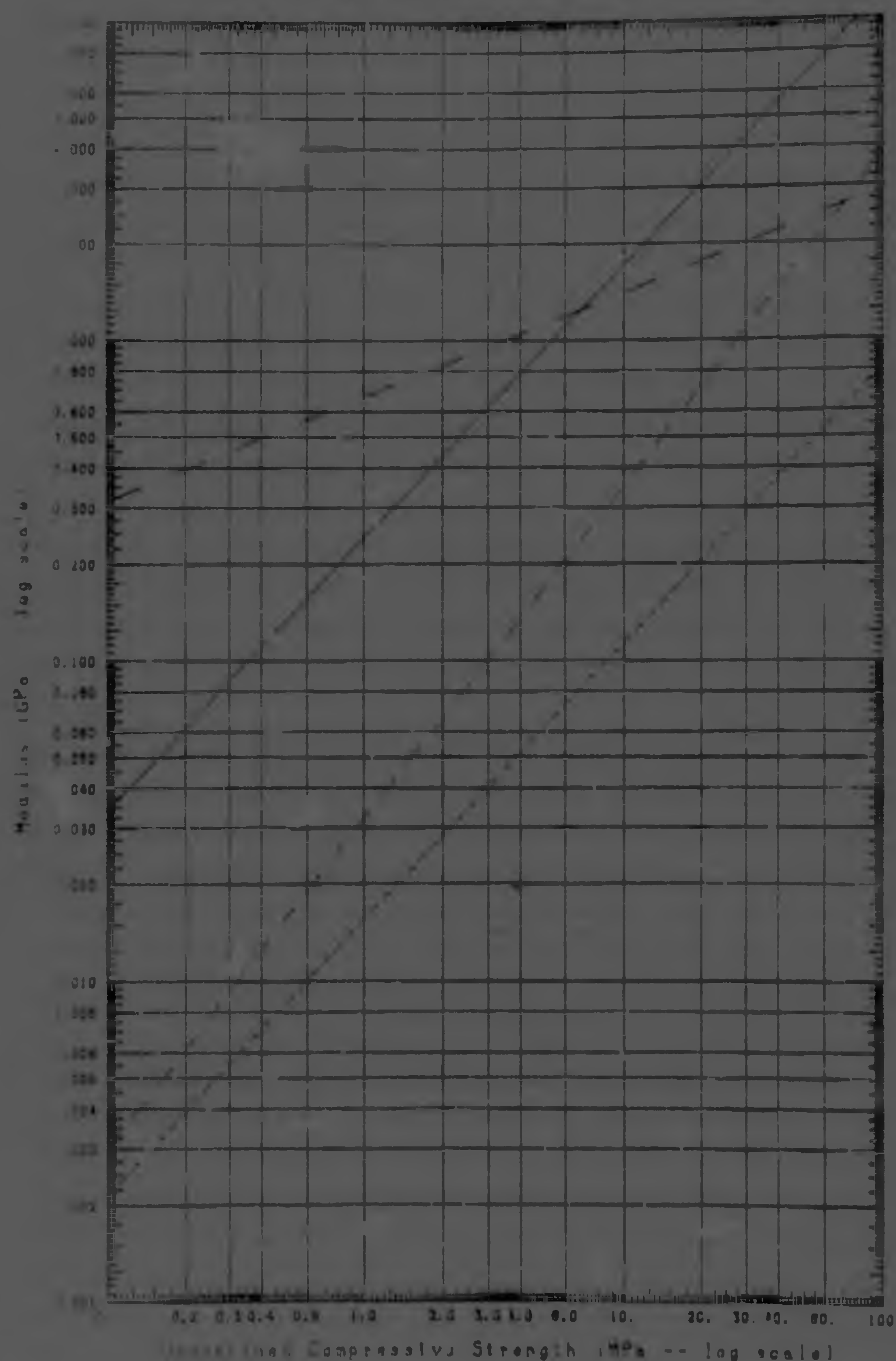


Reg'n : log y = 2.825 + 0.324 log x : r^2=0.84
For individual codes see table 13.2

LCTC MOD PLT -- 13/9/84-8=1275-v 1.3 (SETLCONF.DAT, 17)

GRAPH 13.1[illegible]

COMPARISON OF OEDOMETER AND TRIAXIAL MODULI



manually obtained from the plotted data. In addition, the relatively high values (steep slopes) of this modulus make it susceptible to small errors in deflection measurement.

The results for the reservoir operation modulus suggest that its value is less dependent on the degree of stabilization than on the properties of the particular soil. A possible explanation for these results is that settlement under a particular stress occurs by way of creep and / or conventional consolidation of the matrix until an appropriate level of interparticle contact of the stiffer particles in the soil is reached. It appears that the level of interparticle contact may be virtually independent of the compressive strength of the matrix and that the reservoir operation modulus is a measure of the elasticity of the fabric of the soil.

It should be noted that, as the applied stresses used in this test program were determined in terms of 28 day unconfined compressive strength, this introduces a tendency for the modulus results to be related to the strength of the material.

In this regard, it is particularly relevant that the tests on the unconfined unstabilized specimens were conducted at the same stress levels as used for the specimens stabilized with 5% cement. This confirms the trend for a cement increase from 0 to 5% cement. However, the data for the unstabilized specimens, point 3 on the graphs, appears to exert considerable leverage on the regression results. This suggests that if the unstabilized specimens had been tested at a different stress level, the observed moduli could have differed significantly from those presented here and this could have had a significant effect on the regression results. Examination of the effect of excluding the data for the unstabilized specimens from the regression indicates a slight reduction in slope and a reduction in correlation in all cases. However, the relationship between the three lines and their location on the graph is not significantly affected and the observations made in the previous paragraphs are therefore confirmed.

Further tests on unstabilized specimens and stabilized specimens with different cement contents, all tested at the same stress levels should however be conducted to investigate the significance of the stress levels used here.

13.4.6 Comparison with Results of Triaxial Tests

Graph 13.15 also presents the regression line of Q-modulus vs unconfined compressive strength from the triaxial test results presented in graph 11.10. It is notable that the slope is similar to that of the first filling and construct and fill moduli which is consistent with the fact that the triaxial specimens had not been previously loaded, the consolidation stresses applied in the triaxial tests were small relative to the stress at which the modulus was determined. It is considered that this correlation confirms the validity of the trend observed for these moduli.

The triaxial results are somewhat above those for the construct and fill and first filling moduli and it is considered that this probably reflects the different rate of load application in the two tests. Creep can be expected to lead to an increase in settlement which would have the effect of reducing the modulus determined in the triaxial test. Other factors which may contribute to this difference are:-

the higher stresses in the triaxial test;
the different shapes of the test specimens;
the different levels of confinement in the tests;
possibly some pre-consolidation of the triaxial specimens.

The triaxial modulus is of similar magnitude to the reservoir operation modulus over the range of unconfined compressive strengths likely to be used for design.

As the oedometer test results allow for the effects of creep, it is considered that they are more appropriate for the design of a structure in which loads are to be applied gradually and maintained for a considerable period of time.

13.5 CONCLUSIONS

The test apparatus used is not sufficiently sensitive for detailed measurement of the long-term creep behaviour of the material, larger specimens and more robust apparatus are required. The apparatus used was particularly susceptible to zero error and machine disturbance. These tests however provide a good indication of the maximum settlement likely to be encountered in practice. The standard test with unconfined specimens loaded over their full area and tested at an age of 56 days provides a reasonable estimate of the long-term consolidation characteristics of soil-cement. Further tests with thicker specimens are however considered desirable to confirm the results of this investigation.

Settlement under the load increments applied in this test program was effectively instantaneous and it is therefore considered that all settlement, other than that resulting from creep, will be complete at the end of initial loading.

Flooding of the cell containing the specimen and saturating the specimen under vacuum prior to testing, had no significant effect on the characteristics of the material. The presence of water is therefore not expected to affect the settlement behaviour of a soil-cement embankment.

Maximum settlement was found to be reasonably constant for all cement contents where the applied stress was determined as a fixed percentage of unconfined compressive strength. The effect of different stress levels requires further investigation but the application of a lower stress level will lead to reduced settlement. Maximum observed consolidation was 2.4% and in most cases was considerably less. In a soil-cement embankment, cement content will, in most cases, be determined by criteria other than compressive strength. Accordingly, stress levels lower than those used in these tests will prevail and it is thus expected that average consolidation in any zone of an actual structure will be less than 1%. Consolidation can be limited in critical areas, such as around foundation irregularities, by choice of a higher cement content than may otherwise be indicated. The use of accelerated curing, as suggested in chapter 10, might also be investigated.

Three moduli, the "construct and fill", "first filling" and "reservoir operation" moduli are defined for design purposes. A good correlation between the logarithm of these moduli and logarithm of unconfined compressive strength exists. This relationship takes account of increases in both cement content and age.

No distinct increase in stiffness with increased confinement and restraint was observed and it is concluded that increases in stiffness in the body of an embankment, while present, will not be particularly significant. The modulus values obtained in this test program are found to be comparable to those obtained from the triaxial tests when allowance is made for the differences in test conditions, particularly the difference in rate of load application and stress levels.

It is proposed that the construct and fill modulus should be used to estimate the settlement at the end of first filling of the reservoir at any point in the embankment and that the first filling modulus should be used to estimate the end of construction settlement. The reservoir operation modulus can be used to evaluate the movement that will occur during cyclic operation of the reservoir. It is felt that the modulus obtained from the triaxial tests is not appropriate to the design of this type of structure.

Progressive consolidation under cyclic stresses corresponding to reservoir operation occurs. This is small relative to the initial consolidation and is unlikely to be significant. Increased cement content or accelerated curing can be used to reduce this effect in critical zones.

CHAPTER 14

HEAT GENERATED DURING HYDRATION

14.1 INTRODUCTION

The objective of this investigation was to determine whether the generation of heat by cement hydration, secondary hydration or pozzolanic reaction, represented a significant threat to the structural integrity of the soil-cement dam by way of crack development.

Two forms of temperature related crack development occur in mass concrete, thermal cracking within a concrete monolith when a temperature differential in excess of 24 °C occurs and restraint cracking at the cold joint (Dunstan and Mitchell 1976). The former can be prevented by reducing lift height and by insulation. Restraint cracking can be prevented by reducing the lift height or by increasing the rate of placement, both of which reduce the temperature differential across the joint. Dunstan and Mitchell found temperature increase to be a function of increase in lift depth, a low grade hearting concrete showed a gain of about 8 °C for a lift of 0.8 m increasing to about 17 °C at 1.6 m. Rate of increase for higher grade concretes was similar.

These results indicate that temperature gain in soil-cement placed in layers of 200 to 300 mm is unlikely to represent a major problem in the construction of soil-cement dams. Schrader (1977) supports this view with regard to roller compacted concrete on account of the relatively small volume in one placing and the large surface area of each layer. He observes, however, that thermal studies are still necessary and that particular attention should be paid to temperature gains from insolation. Hansen (1979a) indicated that no studies of heat gain due to hydration were carried out for the solid soil-cement embankments at Barney M. Davis power station but that he believed that an investigation had been carried out previously which indicated that temperature gain was not a problem.

On the basis of the above information, only a limited test program was undertaken with a view to confirming the above observations and investigating secondary hydration and pozzolanic reaction.

An insulated enclosure was erected in the constant temperature laboratory established for the leaching test program. This enclosure contained compartments for six specimens in AASHO moulds. The number of compartments was limited by the extent of the required insulation.

Specimens were compacted with cement contents of 0, 5, 10 (2 specimens), 15 and 20% cement. Six thermocouples were embedded in each specimen and daily temperature readings were taken over a period of six months. A head of percolating water was connected to the specimens after three months to investigate whether the supply of additional water caused the evolution of further heat.

14.2 APPARATUS

The specimens were compacted in steel AASHO moulds 152.3 mm diameter and 152.3 mm long using standard AASHO compactive effort. These moulds were fitted with steel end plates with gaskets and clamping bolts to provide a watertight seal. Connections were provided at top and bottom for water from the leaching supply. A layer of geofabric was placed at either end of the specimen to permit uniform flow.

Copper-Constantan thermocouples with glass braid insulation were used. These were welded using an argon shielded arc to ensure consistency and were sealed against ingress of water with heat-shrink tubing. The thermocouples entered the specimen through a plastic sleeve in a compression fitting in the end cap. The sleeve was filled with silicone rubber adhesive once compaction was completed and the compression fitting was tightened once the silicone had cured, prior to flooding the specimens.

The soil-cement was compacted in four layers with a thermocouple placed at the center of each joint and adjacent to each end plate. A sixth thermocouple was placed at the edge of the center joint.

Additional thermocouples were placed on the front and rear faces of the insulated enclosure and in the center of the body of the insulation between the two middle compartments. A fourth thermocouple was provided to monitor the air temperature adjacent to the thermometer.

An electric thermometer reading directly to 0.1 $^{\circ}$C, (Comark model 1621) was used. A total of forty thermocouples were connected to two thermocouple switch boxes produced by the manufacturer of the thermometer. Temperature compensating wire and connectors were used to connect these boxes to the thermometer.

The enclosure consisted of a frame covered with hardboard and filled with expanded polystyrene granules. Hardboard compartments projecting into the polystyrene were provided for the test specimens. Packing pieces of expanded polystyrene sheet were placed around the cell to provide maximum insulation and a close fitting block of polystyrene was used to close the entrance of the compartment.

Owing to space limitations, it was not possible to construct the enclosure to provide equal cover in all directions. The compartments were located with 300 mm of cover above and below and 200 mm behind the specimen. A 50 mm slot was left between the rear of the enclosure and the wall to allow circulation of constant temperature air. Space between compartments was 350 mm and between the end compartments and the ends of the enclosure 450 mm. The front panel sloped inwards from top to bottom giving cover of approximately 250 mm at the top of the specimen and 200 mm at the base.

Diamant (1969) reports coefficients of thermal conductivity (k) of 0.033 W/m/^{o}C for polystyrene, 0.52 for "stony normal soil" and 1.21 for "very damp, tightly packed soil". On the basis of these figures, for equal coefficients of heat transmission (U), 200 mm of polystyrene insulation is equivalent to between 3.1 and 7.3 m of soil. On the expectation that not more than two layers of soil-cement, each not thicker than 300 mm, would be placed in one day, this insulation was considered sufficient.

14.3 TEST PROCEDURES

Soil-cement was batched and mixed in a timed sequence similar to that used for other specimens. Temperature readings of the mixture and of the layers once placed, were taken at intervals during mixing and compaction. The specimens were compacted in four layers and moisture content samples taken after each layer. The thermocouples for each layer were positioned and held in place with a straight-edge until the soil-cement was placed in the mould. Plate 14.1 shows the center and edge thermocouples on the center joint prior to placing the third layer.

Compaction took place in the general laboratory which was not temperature controlled as the vibration of compaction could not be tolerated near the oedometers and other machines in the constant temperature laboratory. Temperature in this laboratory was about 22 ^{o}C during compaction.

The cells were assembled and transferred to the constant temperature laboratory with temperature readings taken at intervals. Plate 14.2 shows an assembled cell together with polystyrene packing pieces prior to insertion in the insulated compartment.

PLATE 14.1

THERMOCOUPLES ON MIDDLE JOINT

PLATE 14.2

ASSEMBLED CELL AND INSULATION PACKING

Time from addition of water to transfer of the specimen to the constant temperature laboratory was approximately one hour.

Temperature readings were taken at intervals during the first days after compaction and daily thereafter. It was observed that the calibration of the electric thermometer appeared to change if it was left on for any length of time and accordingly it was only switched on when readings were taken. Two readings were taken for all thermocouples in sequence to compensate for any minor fluctuations. It was also observed that the thermometer appeared to be sensitive to changes in air temperature caused by operation of the temperature control system.

Owing to time constraints imposed by other parts of the test program, specimens were compacted on consecutive days with the exception of a two day delay between the first and second specimens. Specimens were compacted in the sequence:- 0, 5, 10, 10, 15 and 20% cement content and placed in consecutive compartments. This sequence was adopted on the expectation that any heat gain registered between compartments would be similar, due to the corresponding increase in cement content, and would be indicated by the results for the two adjacent 10% specimens.

After approximately three months, de-aired distilled water was connected to the specimens to determine whether the availability of additional water led to further heat evolution due to secondary hydration or pozzolanic reaction. In most cases, no significant flow was observed over a further three months. In those cases where flow was observed, it occurred almost immediately the water supply was connected indicating that it was occurring along the seepage path formed by the thermocouples.

14.4 TEST RESULTS

The temperature of the soil-cement mixture in the mixing bowl reached a maximum approximately 20 minutes after addition of water, while the specimen was being compacted. In the case of the specimen with 20% cement, the maximum temperature of the mixture was 26.9 °C after 19 minutes, with initial, dry mixture temperature of 22.8° and air temperature of 22°. The corresponding peak specimen temperature was 25.1° for the third layer 23 minutes after the start of mixing with initial mould temperature of 21.1°. One hour after mixing, prior to transfer to the constant temperature laboratory, specimen temperature ranged from 23.1° on the surface to 23.8° in the center with air temperature of 22.2° indicating rapid decay to ambient. Similar temperatures one hour after mixing were observed for all the test specimens.

Fulton (1977) reports a typical rate-of-heat-evolution characteristic for cement which peaks after about 7 minutes and decays to near zero after 10 minutes. A second peak starts after about 2 hours and peaks at about 5% of the initial peak value after 10 hours before decaying by about 85% after 30 hours.

The general form of this initial characteristic is consistent with the observations discussed above as it can be expected that the heat capacity of the soil would have the effect of delaying and attenuating the peak. It is considered that the rapid temperature decline of the specimen, 1.3° in 37 minutes, gives an indication of the rapid reduction in temperature that can be expected to occur in the field. It should be noted that evaporation from the unsealed surface of the material during placing and compaction would be expected to accelerate the dissipation of the initial temperature gain. It is also likely that the initial gain would peak before the soil-cement was spread, spreading should thus further assist heat dissipation. It is therefore considered unlikely that the initial hydration heat gain would have any effect on the temperature of the compacted material. Heat gain due to insolation can be expected to be of greater importance.

Peak temperature of the specimen containing 20% cement was 30 °C and occurred 27 hours after start of mixing. This agrees with the general form described by Fulton with the reduced heat dissipation, resulting from improved insulation, clearly apparent. The delay in reaching maximum temperature is again ascribed to the heat capacity of the soil and mould. The data for the individual thermocouples in this specimen reflected an average drop in temperature of 0.15 °C from the center to the surface over a distance of approximately 76 mm. The difference between the specimen and the air in the laboratory was 6.5°. Assuming that this drop occurred across the minimum 200 mm thickness of polystyrene, the coefficient of thermal conductivity was estimated as follows:-

Drop in temperature across 76 mm of soil cement = 0.15°
Drop in temperature across 200 mm of polystyrene = 6.5°
Therefore to maintain temperature difference:-

$$\text{Thickness required} \quad t_s = (6.5 / 0.15) \times 0.076$$
$$= 3.29 \text{ m of soil-cement}$$

Coeff of thermal conductivity of polystyrene (k_p) = 0.033 W/m/°C
Coeff of heat transmission (U) = k/t W/m²/°C
For equivalent insulation:-

U of soil-cement (U_s) = U of polystyrene (U_p)

Therefore:-

$$k_s/t_s = k_p/t_p$$

or

$$k_s = (0.033 / 0.2) \times 3.29$$
$$= 0.54 \text{ W/m/°C}$$

This compares favourably with the value of 0.52 for "stony normal soil" suggested by Diamant (1969) and referred to in section 14.2, above. The gradient through the polystyrene confirms the effectiveness of the insulation provided.

Since the variation in temperature through the specimen was small relative to that through the insulation, the average temperature for the six thermocouples in the specimen was used for further analysis of the data. The initial temperature rise effectively decayed within four weeks and thus only this data was processed in detail.

In order to determine absolute temperature gain, a reference was required. Inspection of the data for the unstabilized specimen indicated no initial or subsequent temperature gain confirming the suitability of this specimen as a reference. The temperature profiles for the unstabilized specimen and the three reference thermocouples are plotted in graph 14.1. These thermocouples were mounted at the center of the front and rear faces of the enclosure and in the center of the body of insulation i.e. midway between the front and rear faces and midway between the two central compartments at the midheight of the insulation.

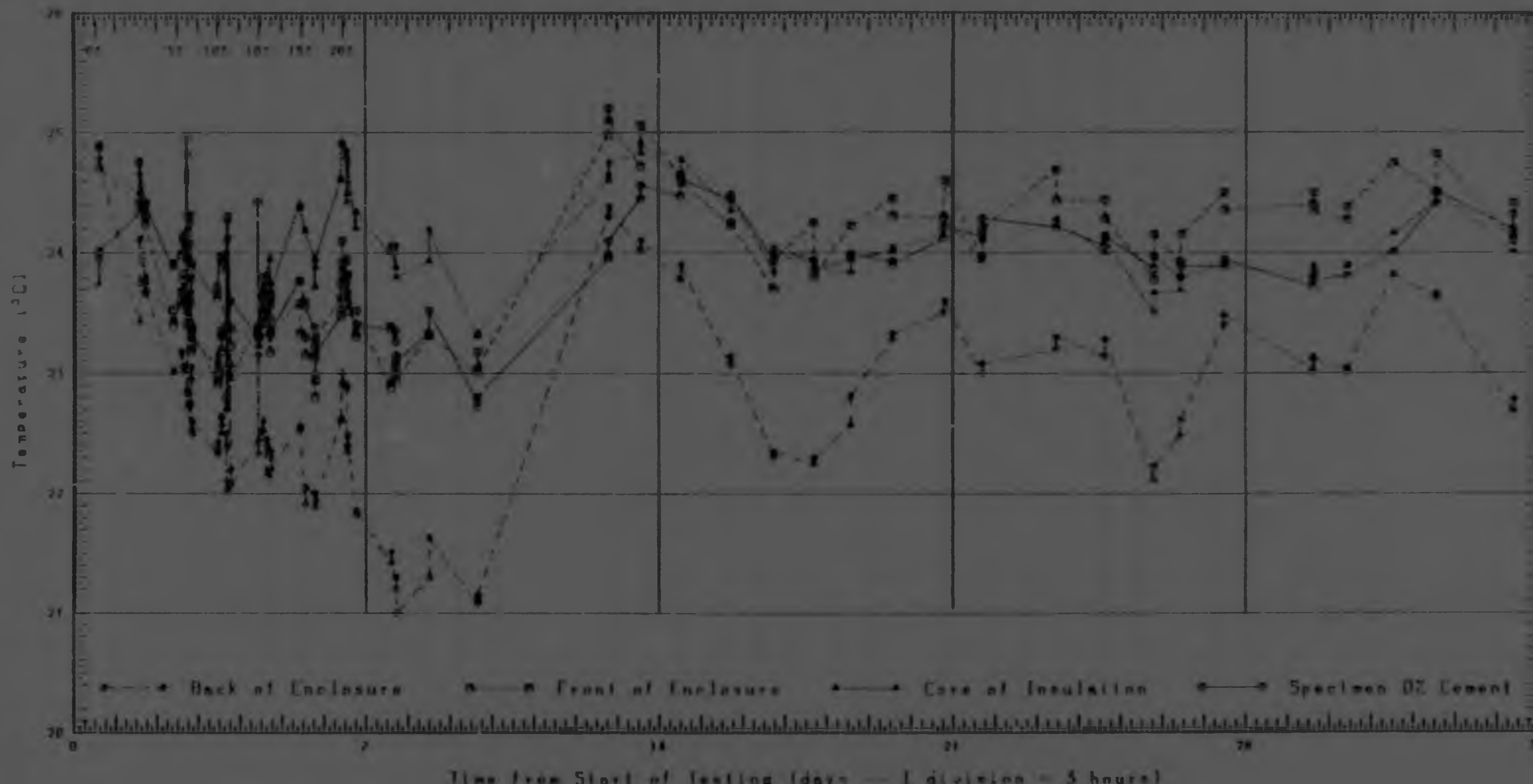
The data plotted on graph 14.1 covers the five week period starting at 0h00 on the day the unstabilized specimen was compacted. The time of compaction of each of the six test specimens is indicated at the top left hand corner of the plot. An exaggerated vertical scale with origin at 20 ^{0}C is used to obtain data separation.

The profiles for the four data sets plotted on graph 14.1 show similar overall trends. The temperature at the rear of the enclosure was consistently lower because of the narrow air gap between the insulated enclosure and the wall of the laboratory which restricted the flow of heated air. The remaining profiles lie in a narrow band, generally between 23 and 25 ^{0}C which agrees well with the temperature records for the laboratory over the test period. A maximum daily range of approximately 2 ^{0}C was achieved throughout the period of this research and was the limit available from the thermostats used.

The daily range of temperature is particularly notable on the days on which specimens were compacted. On these days, a large number of readings were taken, spread throughout the day and the extremes of thermostat operation, together with the effect of continual access to the laboratory, are apparent. In all cases, the scatter of results for the core thermocouple and unstabilized specimen is considerably less than that for the surface thermocouples.

GRAPH 14.1

COMPARISON OF TEMPERATURE PROFILE OF UNSTABILIZED SPECIMEN WITH TEMPERATURE AT VARIOUS POINTS AROUND INSULATED ENCLOSURE



It was observed that the thermometer calibration appeared to change slightly while readings were taken. This effect was not confirmed or quantified but it is considered that it could have contributed to the observed variations, some of which appear excessive in view of the degree of insulation. All readings were taken with the same thermometer and all thermocouples were read consecutively, so that the relative temperature data, discussed below, should be unaffected by this error.

The electric thermometer was not available during the eleventh and twelfth days and readings were taken with a digital voltmeter, however it was not possible to obtain a reliable calibration with this instrument and the data for this interval was thus discarded.

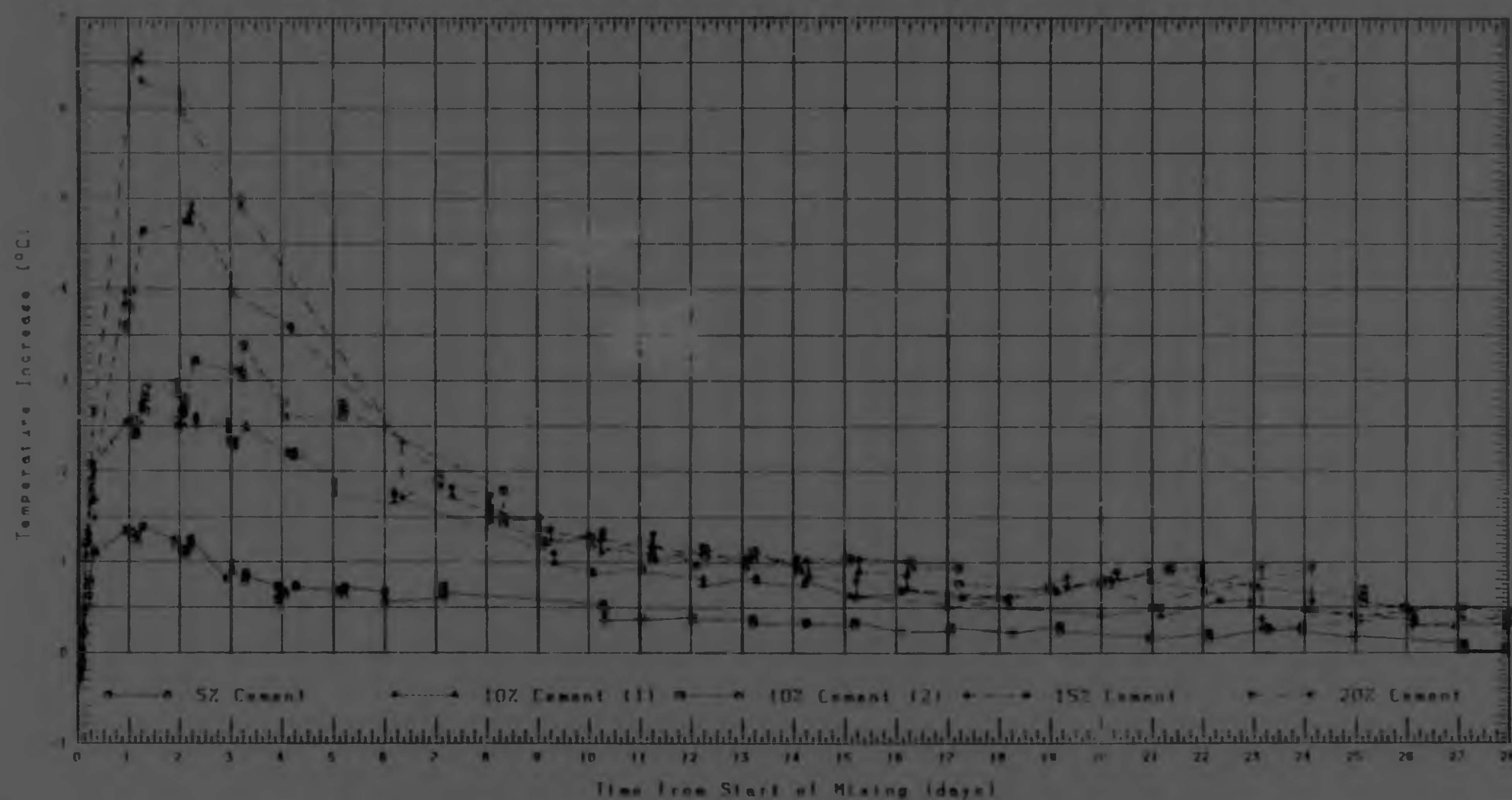
The effect of lateral heat transfer within the body of the insulation is evident in the data for the unstabilized specimen which lies above that for the core in the days immediately following the compaction of the specimen with 5% cement. This effect is also apparent in the data for the core thermocouple following the compaction of the second specimen with 10% cement. This data lies significantly above that for the unstabilized specimen and front thermocouple for a period of 5 days. Thereafter, the data for these three sensors is generally similar.

The profiles for the temperature increase of the stabilized specimens relative to the unstabilized specimen are plotted on graph 14.2. The temperature increase was calculated for corresponding sets of data. The time axis reflects the elapsed time from the addition of water to the dry soil and cement mixture. Specimens were compacted at intervals of one day.

A further indication of the heat transfer between specimens can be obtained from the results for the second specimen with 10% cement. This data follows exactly the same path as that for the first 10% specimen for approximately the first 27 hours and thereafter shows a significant further heat gain. It is considered that this

GRAPH 14.2

TEMPERATURE PROFILES OF STABILIZED SPECIMENS RELATIVE TO UNSTABILIZED SPECIMEN



HM TEMP PLT 28/02/84 4-135 v 2 1 IM M-CONF NRL 3 1 additional heat gain reflects heat transfer from the adjacent 15% specimen. This deviation is similar to that measured by the core thermocouple as discussed above. The 20% specimen shows a much sharper peak than that for other specimens as it was the last specimen compacted and consequently did not receive any substantial further heat input. Since the temperature of previously compacted specimens peaked at about the time that the next specimen was compacted, it is considered that heat transfer from previously compacted specimens did not have a significant influence on the peak temperature achieved.

While the exact form of the decay curve is distorted by the heat transfer mentioned above, it is considered that the decay curve for the 20% specimen is reasonably representative as no heat input from specimens with higher cement content occurred. The exact shape of the decay curve will in any event be a function of the degree of insulation and specimen size. Heat transfer from adjacent specimens with lower cement contents can be considered to represent an increase in the effective size of the specimen. On this basis, the results indicate that most of the heat gain decays in a period of about 10 days and the peak is significantly attenuated in comparison with that reported by Fulton. This is again ascribed to the heat capacity of the soil and mould and the high degree of insulation. It is noted that temperature decay is still occurring after 28 days. It is notable that the initial rate of temperature increase is very similar for all specimens.

Since virtually all heat gain occurs within 24 hours and not more than two layers, each of 200 to 300 mm thickness, are likely to be placed in this period, no significant temperature differential in the embankment due to hydration is expected.

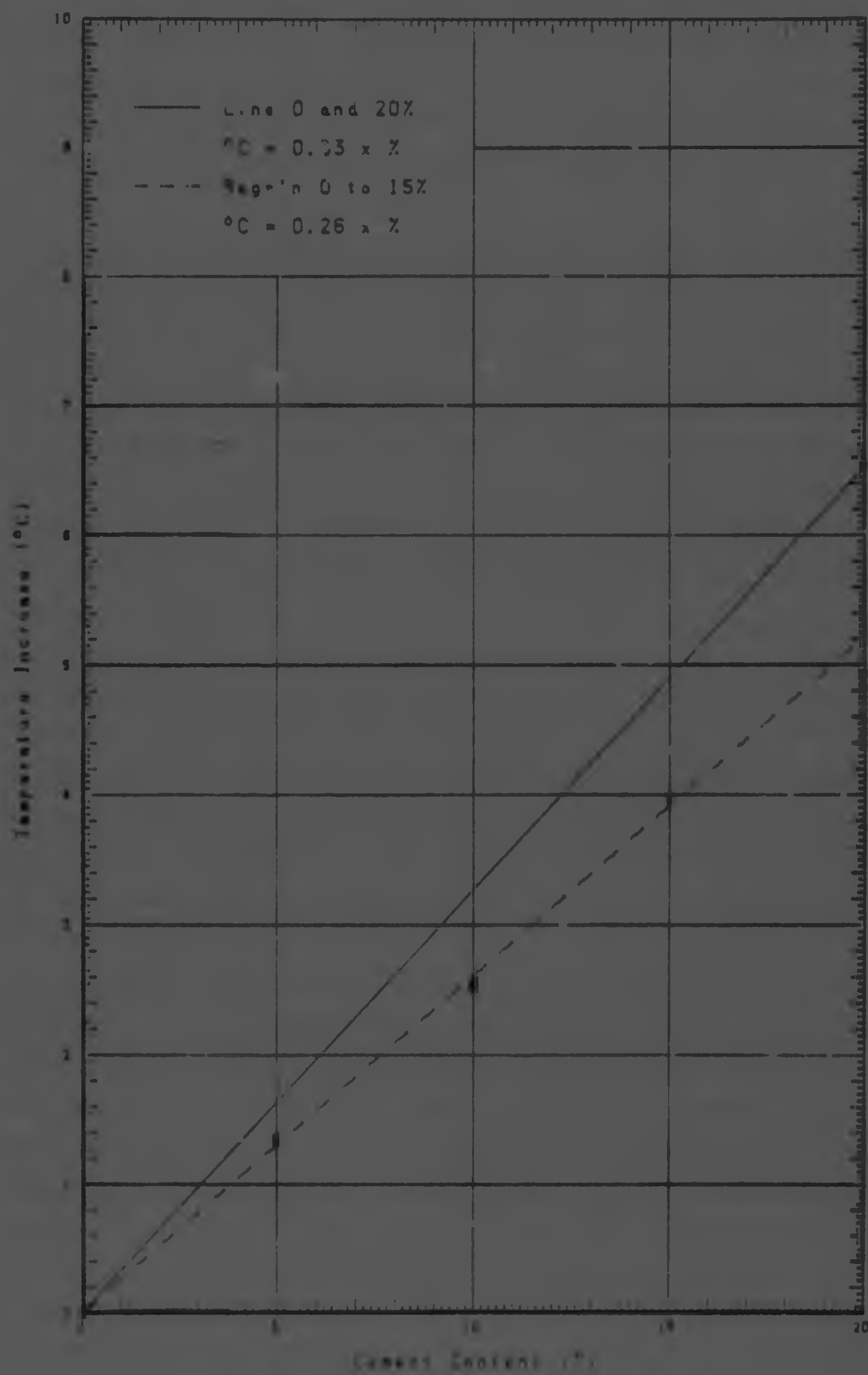
On the basis of the form of the peak for the 20% specimen, the peak temperature for individual specimens without heat transfer was assumed to occur at 24 to 27 hours. In all cases, a levelling off or decrease in temperature occurred after this interval. These peak values are plotted against cement content on graph 14.3. The values for specimens with cement contents up to 15% exhibit a very linear trend, regression on these points yields a regression coefficient of 0.998 and slope of 0.26. However, the point for the specimen with 20% cement lies well above the regression line. A line through this point and the origin indicates a slope of 0.33. These lines are plotted on graph 14.3.

Dunstan and Mitchell (1976) observed that temperature gain increased with placing temperature. It is therefore possible that the heat gain from the 15% specimen raised the temperature of the 20% specimen, immediately after placing in the insulated compartment, sufficiently to cause a significant increase in temperature gain. Other explanations for this discrepancy may exist or it may simply represent an accumulation of experimental errors. The primary finding of this investigation is that heat gain due to hydration will not be sufficient to lead to thermal cracking and consequently a detailed investigation of this anomaly is not warranted. The exact thermal behaviour of the material can only be further investigated by means of a trial embankment. It is however considered that the available information is sufficient to indicate that further investigation of this aspect prior to construction of a working structure is not warranted. If an estimate of temperature gain in an insulated mass is required, it is suggested that the value of 0.33 $^{o}C/\%$ cement could be used.

No temperature change was observed on connection of the water supply after three months. It is therefore concluded that in view of the low permeability of the material, the rate of advance of the wetting front is so slow that the volume of soil-cement affected at any time is insufficient to lead to any significant temperature rise. Consequently, secondary hydration and pozzolanic reaction do not pose any threat to the integrity of the structure by generation of thermal stresses.

GRAPH 14.3

TEMPERATURE GAIN vs CEMENT CONTENT



CEM-TEMP.PLT -- 28/0?/84 4:35

14.5 CONCLUSIONS

The temperature increase characteristics of soil-cement are found to agree with the rate-of-heat-evolution characteristics for cement reported by Fulton (1977) with the peaks delayed and attenuated by the heat capacity of the soil. The initial peak occurs approximately 70 minutes after addition of water and the secondary peak after approximately 27 hours.

Heat dissipation from the soil-cement during mixing and handling was found to be considerable and also restricted the temperature gain of the mixture. It is concluded that the initial heat gain will peak before the soil-cement is placed on site and that handling and evaporation will result in effectively all initial heat gain being dissipated by the time placing of a layer is completed. Spraying to replenish moisture loss, as normally applied, will lead to further evaporative cooling. Heat gain due to insolation will probably be more significant than that due to initial hydration.

The coefficient of thermal conductivity of the soil-cement was found to be of the order of 0.54 W/m/°C comparing well with that suggested by Diamant (1969) for soil. The 200 mm minimum depth of expanded polystyrene insulation used in the test apparatus is thus equivalent to approximately 3.3 m of soil-cement applied approximately one hour after mixing.

The secondary temperature gain was found to peak at between 24 and 27 hours. Most of the secondary temperature increase decayed in a period of 10 days. The temperature increase was found to lie in the range 0.26 to 0.33 °C/% cement content for the specimen size and degree of insulation used. As the maximum cover that might be expected to be placed in a 24 hour period was one to two layers of soil-cement, each 200 to 300 mm thick, it is concluded that no significant residual temperature increase can be expected. Evaporation from the surface of the soil-cement can also be expected to assist heat dissipation.

Heat transfer was found to occur between test specimens and affect the shape of the time - temperature curve recorded. As specimens were compacted with a 24 hour delay between specimens and in sequence of increasing cement content, it is considered that heat transfer did not significantly affect the peak temperatures recorded. This factor is thus not considered to have any bearing on the results of the test program.

No temperature gain was observed when additional water was supplied to the test specimens after curing for three months. It is concluded that because of the low permeability of the soil-cement, only a limited volume of material is affected by increasing moisture content at any time and consequently no significant heat evolution occurs.

It is therefore concluded that for a soil-cement embankment, raised at the rate envisaged, temperature effects due to hydration will be of no significance and heat gains due to insolation will be of greater importance. In the light of the observation by Dunstan and Mitchell (1976) that wet curing leads to significant reductions in the surface temperature of mass concrete, it is suggested that the normal procedure of spraying layers of soil-cement at intervals will further assist in dissipating heat gains due to hydration and insolation.

It is suggested that if further information is required on the thermal behaviour of mass soil-cement, this will only be obtained by the construction of a trial embankment. However, in the light of these results together with the experience with soil-cement and other materials placed in thin layers, it is concluded that the construction of a trial embankment simply for the purpose of monitoring thermal behaviour is not warranted.

CHAPTER 15

SUMMARY OF RESULTS OF LABORATORY PROGRAM

This chapter reviews the results of the experimental programs presented in chapters 8 to 14. The parameters extracted for design purposes are summarised in section 16.1 and recommendations for further research are summarised in chapter 17.

15.1 WETTING AND DRYING DURABILITY TESTS (Chapter 8)

The soil from the Rondebosch Dam site is found to be suitable for use as slope protection. Cement contents of 2 and 2.4% for soil with and without coarse fraction satisfy the standard mass loss criteria. A cement content of 4% is indicated for slope protection over the expected range of reservoir levels.

The hydration correction of 25% of cement content, recommended by the PCA (1976c), is found to be conservative for the material used in this research and a factor of 12 % is considered more appropriate.

The extension of the test program to include a second series of twelve wetting and drying cycles provides useful additional information. At cement contents above 16%, additional mass gain occurs with every wetting and drying cycle indicating continuing hydration with every wetting period. A diminishing rate of material loss is found for specimens containing more than 6% cement, suggesting convergence on a stable, no loss, situation. A zero loss condition occurs at a cement content of 15%. Greater mass gains were observed for the 37.5 mm diameter cylinders used as standard in the rest of this program than for the standard wet-dry test specimens.

An alternative test procedure is proposed and is reviewed in Chapter 17.

Leaching tests on specimens subject to wetting and drying cycles indicate an increase in permeability. An increase in resistance to leaching is observed despite this increase in permeability. Losses in the wet-dry test are found to be comparable to those observed in the leaching tests for equivalent cumulative soaking / leaching periods and it is suggested that the wet-dry test is essentially a measure of resistance to surface leaching.

The unconfined compressive strength of specimens subject to wetting and drying cycles is found to exceed that of comparable standard soil-cement specimens and it is concluded that the drying phase of the standard test at 72 ^{o}C leads to accelerated curing of the soil-cement. These results suggest that the severity of the test could be increased by reducing the temperature of the drying phase or dispensing with it altogether and conducting the test using a leaching bath as used in this research. This would considerably reduce the time required to conduct the test.

A further proposal resulting from the tests on specimens subject to the standard wet-dry test cycles is that strength and leaching resistance in critical areas can be improved at early ages by the use of black plastic sheeting to provide a moisture barrier and elevated temperatures. Areas where this technique might be employed include those regions which are at risk from overtopping during construction as well as spillway surfaces.

It should be noted that the freeze-thaw test is usually more severe than the wet-dry test (PCA 1976c, Marais 1984). Freeze-thaw tests were not conducted as part of this program due to a lack of suitable equipment and since freeze-thaw testing was not considered appropriate to South African conditions. The proposed method of increasing the severity of the wet-dry test could be calibrated to provide a more appropriate test of equal severity to the freeze-thaw test.

15.2 LEACHING TESTS (Chapter 9)

It is concluded that, for a homogeneous embankment subject to a low gradient of the order of unity, retaining natural water and where the elapsed time between start of construction and first filling is likely to exceed one year, leaching by percolating water does not present a problem. The above conditions can be expected to be fulfilled by most structures of any significance and the results indicate that even for considerably more severe conditions than those referred to above, no major problem is likely to be encountered.

Surface leaching and erosion is considered to be the most significant form of leaching attack but the extent of this deterioration is unlikely to exceed the construction tolerance on profile during the life of the structure. The leaching bath, a tank flushed regularly with distilled water, is found to provide a simple alternative test for investigating leaching resistance.

As mentioned in the previous section, the results indicate that the standard ASTM wetting and drying test is essentially a surface leaching test and as such it is found to provide a reasonable measure of the durability of soil-cement in water retaining structures. This is consistent with the large volume of experience in the use of soil-cement slope protection.

The use of high hydraulic gradients to accelerate the leaching test is found to lead to an overestimate of both permeability and leaching rate and could lead to the rejection of material that experience indicates to be adequate. Specimens subject to leaching at a hydraulic gradient of 10 m/m were found to exhibit very low flows and leaching losses and a mass gain was observed in one case. Difficulties were experienced in establishing flow under these conditions.

It was found that pH does not provide a reliable measure of calcium concentration of leachate due to the presence of other alkaline metal salts in the cement. It is suggested that mass determinations of dissolved salts following evaporation of leachate should provide a simple and low cost means of evaluating the leaching - time characteristics of test specimens.

The presence of dissolved gases was found to reduce the permeability of the material considerably and further reductions in permeability were observed with the natural reservoir water which was found to be in approximate chemical equilibrium with the soil of the reservoir basin. Absorption of magnesium from this water and retention of magnesium, present in the leachate of unstabilized specimens, by stabilized specimens, was observed indicating that the soil-cement had a significant capacity for magnesium.

Water saturated with carbon dioxide was found to result in low permeabilities due to the presence of dissolved gases with a tendency for permeability to increase as leaching progressed. The equilibrium CO_2 content of the solution was lower than might be expected in a reservoir where anaerobic decay was taking place and leaching under these conditions is expected to be more severe. This might be simulated using a leaching bath with a continuous flow of CO_2.

The leaching tests do not provide any basis for selection of a cement content for use in slope protection other than those indicated by the results of the wetting and drying tests, although an optimum cement content of 10% at higher gradients is indicated. Cement contents of 6% and 15%, related to the level of reducing loss and no loss in the double wetting and drying test, are suggested in view of the steep slopes proposed.

Permeability of the soil-cement tested was found to be of the order of 1×10^{-9} m/s for de-aired distilled water at 25 ^{o}C under a hydraulic gradient of 50 m/m. Permeability of unstabilized soil under these conditions was found to be of the order of 1×10^{-10} m/s. A progressive reduction in permeability approaching an asymptote with extended percolation was generally observed.

Permeability of microconcrete (concrete mortar) specimens was considerably less than that of soil-cement but was found to fulfil the mix design requirements. It is suggested that this lower permeability reflects the effect of wet curing of the concrete specimens as opposed to the humid cure of the soil-cement specimens. Soil-cement specimens wet cured from seven days exhibited a significant reduction in permeability.

Disk specimens exhibited very high permeabilities initially but these decayed to levels similar to those for the cylindrical specimens. Similar behaviour was observed for cylindrical specimens with longitudinal tension cracks where permeability decayed to that of the intact material.

Compaction 1% dry of proctor optimum moisture content (OMC) results in a considerable increase in permeability and leaching loss under flow. Specimens compacted 1% wet of OMC exhibit reduced permeability and leaching loss. Compaction at optimum moisture content and density 2% above proctor maximum does not lead to any significant change in the properties of the material. No substantial difference in leaching bath and wet-dry test losses is observed for these conditions and it is considered that, provided a surface leaching condition exists, tolerances on compaction conditions do not need to be excessively strict. However, where a relatively high gradient exists, care should be taken to avoid compaction unduly dry of optimum. A limitation of 0.5% or even 0.25% dry of optimum might be appropriate in certain circumstances.

A model for leaching in the leaching cells, leaching bath and wet-dry test is proposed and a theoretical depth of deterioration is defined as the depth to which all stabilizer is removed assuming no stabilizer loss in the remainder of the specimen. This was compared with the penetration measured using a standard bitumen penetrometer and a relationship of penetration approximately equal to 25% of theoretical depth of deterioration was determined for the standard leaching conditions.

The depth of deterioration was found to provide a valuable tool for comparing the results of leaching tests with different specimen geometries and test conditions.

15.3 UNCONFINED COMPRESSION AND TENSION (Chapter 10)

Unconfined compressive strength and split tensile strength is found to increase linearly with cement content and logarithm of age. The unconfined compressive strength data is summarised in graph 10.9 which can be used for design purposes. The split tensile strength is found to be approximately 10% of the unconfined compressive strength indicating a true tensile strength of the order of 10.8% of the compressive strength.

Soaking of the specimen prior to test is found to significantly reduce strength and, in general, the moisture content at test is found to have a significant effect on measured strength. Strength is also found to be related to degree of hydration, as measured by residual (oven dry) moisture content. These factors account for a number of anomalies in the test results.

Compaction wet of optimum moisture content is found to lead to a reduction in strength, while compaction dry of optimum or above proctor maximum dry density has no significant effect. Compaction at optimum is therefore to be preferred although compaction dry of optimum may be indicated where reduction of shrinkage is required.

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Strength reductions of specimens leached in the leaching bath support the observation in the previous section that this is a surface leaching effect.

The unconfined compressive strength of the soil-cement used in this program compares favourably with that determined in other research and with that of concrete used at Dworshak Dam and rollcrete proposed for Willow Creek Dam although higher quantities of cementitious material are indicated. It is however concluded that these differences are such that soil-cement will be found to be competitive on suitable sites. Higher strengths are obtained for the soil-cement used in this research than for the micro-concrete mixes at ages in excess of about 200 days.

A discrepancy between the strength of soil-cement considered adequate for slope protection of earthfill embankments and that proposed for the facing of Willow Creek Dam is noted. It is suggested that if this discrepancy is eliminated, the higher cementitious contents for soil-cement, referred to above, will prove largely unnecessary.

15.4 TRIAXIAL COMPRESSION TESTS (Chapter 11)

Stabilized specimens failed smoothly in shear at high confining pressures with sudden brittle failure at low pressures. Specimens with low cement contents exhibited plastic failure at high pressures.

Peak pore pressure was found to occur well before failure for all stabilized specimens and pore pressure at failure was less than 3% of maximum principal stress. The use of unconsolidated undrained tests on unsaturated specimens is therefore recommended for routine testing with a limited number of consolidated undrained tests with pore pressure measurement on saturated specimens being conducted to confirm this relationship for different soil types and low cement contents.

Linear regressions on P-Q data were found to give acceptable results for the confining pressures used in this research although slight curvature of the envelope was observed.

The angle of internal friction was found to increase from 27^{o} to 37^{o} with the addition of 5% cement to the soil. Thereafter, angle of friction was found to be independent of cement content and curing and compaction conditions.

Cohesion was found to increase with cement content and age and a linear relationship between unconfined compressive strength and cohesion was confirmed. The constant in the theoretical relationship between cohesion and unconfined compressive strength derived in chapter 3 is amended from 0.5 to 0.445 and it is suggested that further investigation of this relationship is desirable. It is however considered that these results provide a reasonable basis to justify the use of the unconfined compressive strength as the routine parameter for investigations into the effects of different curing conditions, etc on the shear strength of soil-cement once the angle of internal friction has been determined.

An incremental modulus for comparative purposes, the Q-modulus, was defined and found to be of similar magnitude to the initial tangent modulus and the secant modulus to failure. No particular relationship between this modulus and confining pressure was observed. The relationship between logarithm of Q-modulus and logarithm of unconfined compressive strength was found to be linear and the same relationship was observed for both soil-cement and microconcrete specimens.

Calculation of the incremental slope of data during data processing is recommended as a useful technique for correcting zero error and validating data.

15.5 DIRECT SHEAR TESTS (Chapter 12)

It is proposed that design for sliding stability of soil-cement should be based on the residual (frictional) strength of the material with a factor of safety of 1.2 to account for material variability. An angle of internal friction of 40° is indicated. In addition, a minimum factor of safety of 2 on ultimate strength and a minimum cement content of 3.2% of whole soil is proposed. Further investigation of these proposals is required.

Further tests are required to determine methods of mobilising the full shear strength of the material on construction joints, modifications to the test procedure used are recommended. The dosage rate for cement treatment of construction joints should be at least 0.4 kg/m^3 in order to achieve reasonably uniform distribution and application in paste or slurry form is recommended. An investigation into the effect of variation in construction delay between layers is required. The use of in situ testing of soil-cement layers is recommended for further tests.

Inclusion of the soil fraction retained on the 4.75 mm sieve leads to an increase in cohesion and angle of internal friction and it is suggested that the shear strength of soil-cement could be improved by the use of high strength aggregate either to supplement or replace a portion of the gravel fraction in the soil.

Where tests are conducted on specimens prepared using the soil fraction passing the 4.75 mm sieve, design cement contents for the soil containing the coarse fraction should be calculated as the same percentage of the complete soil and not of the minus 4.75 mm fraction. Thus, where the unconfined compression tests on specimens containing only the fine fraction indicate a cement content of 5%, a cement content of 5% of the soil including coarse fraction should be used in design. Further investigation of this relationship is required.

15.6 OEDOMETER TESTS (Chapter 13)

The test apparatus used was not sufficiently sensitive for detailed measurement of long-term creep behaviour, larger specimens and more robust equipment are required to investigate these aspects. These tests however provide a good indication of maximum likely settlement.

The standard test used unconfined specimens 71.4 mm in diameter and 20 mm thick loaded over their full area. Maximum stress was determined as one third of the soaked 28 day unconfined compressive strength and a loading cycle related to reservoir operation was used. The short-term test of two weeks duration on 56 day old specimens was found to give a reasonable estimate of long-term behaviour.

Settlement under load increme s was found to be largely instantaneous and no significant settlement is expected beyond end of loading. Progressive consolidation under cyclic reservoir operation is not considered to be significant.

Flooding of the cell, saturating the specimen prior to testing, variation of specimen restraint and loading conditions did not cause any significant change in settlement characteristics.

Maximum settlement was found to be reasonably constant for all cement contents and is related to the approximately constant ratio of applied stress to unconfined compressive strength that was applied. Maximum consolidation was 2.4% and was generally considerably less. As cement content will generally be determined by factors other than settlement requirements, stress levels will generally be less than used in this test program. Consequently, it is anticipated that settlement is unlikely to exceed 1% in any part of a soil-cement embankment.

Three moduli, the construct and fill, first filling and reservoir operation moduli are defined for design purposes and a good correlation between logarithm of modulus and logarithm of unconfined compressive strength is established. These relationships are summarised on graph 13.15. The modulus values from this program are found to be comparable to those established in the triaxial test.

15.7 HEAT OF HYDRATION (Chapter 14)

The temperature increase characteristics of soil-cement are found to agree with those of cement with the peaks delayed and attenuated by the heat capacity of the soil.

Heat dissipation during mixing and handling was found to be considerable and it is considered that no residual heat gain in the embankment from the initial hydration peak is to be expected. Insolation is considered to represent a greater problem with regard to initial heat gain but it is considered that normal spraying of the construction surface will provide sufficient evaporative cooling.

Coefficient of thermal conductivity of the soil-cement was found to be of the order of 0.54 W/m/^{o}C. The depth of insulation used in this test was thus approximately equivalent to 3.3 m of soil-cement applied one hour after mixing.

Secondary temperature gain peaked at between 24 and 27 hours and was found to lie in the range 0.26 to 0.33 ^{o}C/% cement content for the specimen size and degree of insulation used. In view of the limited cover that might be applied in normal construction and the likely effect of evaporation, it is concluded that no significant residual temperature increase can be expected.

No temperature gain was observed when percolating water was applied to the specimens after curing for three months and it is concluded that no heat gains due to secondary hydration or pozzolanic reaction will occur.

It is considered that there is no basis to anticipate thermal problems in a soil-cement embankment. If further investigation is required, it should be conducted by means of a trial embankment but construction of a trial embankment simply for the purpose of monitoring thermal behaviour is not considered warranted.

CHAPTER 16

DESIGN OF THE SOIL-CEMENT DAM

This chapter summarises proposals for the design of the soil-cement dam, reviews recent developments in dam construction and examines the design and costing of a soil-cement dam.

16. MATERIAL PROPERTIES FOR DESIGN

The results of the laboratory program with regard to specific values for use in the design of an alternative soil-cement embankment dam for the Rondebosch, Middelburg, site are summarised below. It should be noted that a number of proposals have been made for further research into certain of these properties. These recommendations are summarised in chapter 17.

16.1.1 Soil Classification

The soil is classified as A-2-4 according to the ASTM system (SC-SM according to AASHO). The soil grading is represented by the dashed line on graph 6.1 (soil 2). Atterberg limits are liquid limit 24% and plasticity index 5.5%. Linear shrinkage is 3.7% and specific gravity 2.7.

For the whole soil containing coarse fraction, approximately 23% passes the 200 mesh sieve (0.074 mm) and 87% passes the 9.5 mm sieve.

16.1.2 Compaction Characteristics of Soil-Cement

Maximum dry density of soil-cement produced with soil passing the 4.75 mm sieve is 1905 kg/m^3 at optimum moisture content of 13.75%. For the whole soil, maximum dry density is 1985 kg/m^3 at optimum moisture content of 12.6% giving an as-compacted density of 2235 kg/m^3.

If a gain in mass due to hydration of 12.5% of cement content, as indicated by the results of the wetting and drying and leaching tests, is applied, then dry masses for determination of over-turning stability range from 1993 kg/m^3 at 3.2% to 2026 kg/m^3 at 20% cement. A value of 2000 kg/m^3 is considered reasonable for design. Note that because of the low permeability of the material, it is unlikely that a significant reduction in moisture content after placing will occur provided regular spraying of lift surfaces is applied.

16.1.3 Durability of Slope Protection

In terms of the PCA (1976c) recommendations, a cement content of 4% is required for slope protection over the expected range of reservoir levels with a cement content of 2% being sufficient in the areas not subject to wave action. In terms of the requirements for uniform mixing, a minimum cement content of 3% cement must however be applied throughout the embankment.

It should also be noted that the critical durability test in the United States is generally the freeze-thaw test which was not considered appropriate for South African conditions. The use of this test would probably indicate higher cement contents than those indicated by the wet-dry test and this should be considered in designing slope protection.

The leaching tests do not provide any basis for selection of a cement content for use in slope protection. Cement contents of 6% and 15%, related to the level of reducing loss and no loss in the double wetting and drying test are suggested in view of the steep slopes proposed. Further investigation, preferably by way of a trial embankment is required. These allowances might be reduced by 1% in each case to allow for the presence of the coarse fraction.

The wet-dry test results indicate that the resistance to erosion is increased by the inclusion of the coarse fraction of the soil and Nussbaum and Colley (1971) indicate that the resistance to erosion in the field is similarly improved by the addition of coarse material. They consider gravel additions such that up to 30% of the resulting soil is retained on the 6.35 mm (1/4 inch) sieve. The soil used in this research contains a gravel component of this magnitude and consequently should not require beneficiation. However, in the light of the findings of the direct shear tests, it is possible that replacement of a portion of the gravel fraction with a high strength aggregate might be beneficial on the spillway facing.

16.1.4 Strength of Soil-Cement

For design purposes, graph 10.9 should be used to determine the cement content required to provide a particular unconfined compressive strength at a given age. Note that these results are for tests on the minus 4.75 mm fraction and that the direct shear test results indicate a probable increase in strength for material containing the coarse fraction. Design with the values in graph 10.9 should therefore be conservative.

While design for tensile strength of soil-cement is to be avoided, the tensile strength of soil-cement can be estimated as 10.8% of the unconfined compressive strength.

The effective angle of internal friction for the unstabilized material determined in the triaxial test is 27° increasing to 37° at a cement content of 5%. Thereafter it is effectively constant for changes in cement content, age, curing and compaction conditions. These values were determined for the soil fraction passing the 4.75 mm sieve.

The effective cohesion in the triaxial test is 22% of the corresponding unconfined compressive strength for the soil-cement tested in this program and the general relationship is similar to that proposed in chapter 3, equation 1, with the constant amended from 0.5 to 0.445.

A linear relationship exists between logarithm of Q-modulus and logarithm of unconfined compressive strength and is presented in graph 11.10. This modulus applies only in the case of rapidly applied loads.

Design for sliding failure should use an angle of internal friction of 40° where soil including the coarse fraction is used. Design should be based on the residual (frictional) strength of the material with a factor of safety of 1.2 applied for material variability. A minimum cement content of 3.2% of whole soil should be used and a minimum factor of safety of 2 on ultimate strength should be applied. The cement content to give the required cohesion can be estimated from graph 12.8 from which the following relationship was derived:-

$$\text{Cohesion (MPa)} = 0.364 + 0.116 \times \text{Cement Content (\%)}$$

It should be noted that this relationship was established in the range of cement contents from 3.2 to 12.8% and that extrapolation below 3.2% is not advisable.

If it is found that the ultimate shear strength of the material cannot be mobilised on construction joints, joint surfaces should be designed with a crossfall sloping up towards the downstream face of the dam. Joint treatment of brooming as normally specified for slope protection and application of 0.4 kg/m^3 of cement slurry should be specified for cold joints. The acceptable delay between placing layers should be determined in the field.

16.1.5 Long-Term Settlement

Graph 13.15 presents values of three moduli related to unconfined compressive strength for use in estimating settlement in any part of the embankment at any stage during the life of the dam. It is considered that maximum settlement is unlikely to exceed 1% where applied stress does not exceed one third of 28 day unconfined compressive strength.

The "construct and fill" modulus can be used to estimate settlement up to the end of first filling. The "first filling" modulus can be used to estimate settlement at any stage during first filling relative to the settlement calculated for the end of first filling. The "reservoir operation" modulus provides an estimate of movement during reservoir filling and draining. A reduction in estimated settlement can be expected where applied stresses are considerably less than one third of the 28 day unconfined compressive strength.

16.1.6 Thermal Behaviour

The coefficient of thermal conductivity of the soil-cement is of the order of 0.54 W/m/$^{\circ}$C.

Temperature gain for the specimen size and degree of insulation used in the test program was of the order of 0.33 $^{\circ}$C/% cement content.

Initial and secondary heat gain due to cement hydration are not expected to be significant and insolation is expected to be more troublesome. It is considered that regular spraying of the construction surface, as normally applied to aid curing, will be sufficient for evaporative cooling to prevent significant heat gains in the body of the embankment.

16.2 REVIEW OF RECENT DEVELOPMENTS

Owing to a variety of factors, there was a considerable delay between the completion of the laboratory program and the processing of the results. During this period, the paper by Schrader (1982) referred to in section 3.2 was obtained and data from this source is referred to in several of the preceding chapters. Copies of the CIRIA reports on the work undertaken by Dunstan were also obtained during this period (Dunstan 1981a, b and c).

In order to ensure that the conclusions of this research were relevant and took account of the most recent developments in the field, letters were directed to a number of the correspondents who had been particularly helpful in the earlier stages of this research. Information was obtained from Moffat (1984), the Cement and Concrete Association (1984), Hansen (1984a) and the USBR (Pedde 1984). A review of literature in the PCI library was also undertaken. A number of papers published while the laboratory program was underway were also obtained.

Significant developments in this period include the CIRIA "International Conference on Rolled Concrete for Dams" held in June 1981 as a result of Dunstan's research and the work undertaken by numerous organisations in the United States, particularly under the auspices of ACI committee 207 which issued a number of reports on roller compacted concrete (RCC or rollcrete). The culmination of this work was the design of Willow Creek Dam which has been referred to previously. A number of further developments in the use of rollcrete have been reported.

A number of reports which are felt to be of particular relevance to this research are reviewed below and discussed in terms of the results of this research. This is by no means a comprehensive review of recent developments and many of the documents referred to here provide additional references. It is however felt that these papers are reasonably representative of the scope of major recent developments.

16.2.1 Developments Reported Prior to 1981

Johnson and Chao (1979) review the use of rollcrete at Tarbela and report that no cracking was observed in tunnels excavated after one year.

ACI committee 207 reviewed the state-of-the-art of rollcrete and suggested guide-lines for design and construction (ACI 1980). They concluded that the frictional strength of the material was sufficient to resist sliding. This supports the design philosophy proposed in section 12.4.5.

16.2.2 The CIRIA Conference on Rolled Concrete

A copy of the discussion and summing up of the CIRIA conference referred to above (CIRIA 1981) was obtained from Moffat (1984). In the discussion, Moffat notes that a 25 reduction in construction time can result in a 10% reduction in cost and that computed values of primary stress are of the order of 2 to 3 MPa for gravity sections up to 100 m. He queries the need for the high strengths proposed by Dunstan and others. This is in agreement with the observations in section 10.4.14. Moffat also suggests that the most appropriate method of design is to include an upstream impermeable facing.

In the same discussion, J. Lowe queries the need for bond and tensile strength at lift joints since these requirements are not required at foundation level. In the final summing up of the conference, J. D. Llewellin refers to the points raised by Moffat, Lowe and Schrader with particular reference to the doubts as to the necessity of bond on construction joints and the importance of minor discontinuous imperfections. Llewellin also refers to the aesthetics of the "rough and ravelly" slopes of Willow Creek Dam and observes that such structures may well blend with the surroundings in a manner which is not possible with conventional concrete structures. He suggests that "a new method of construction must develop its own aesthetic concepts and not try to imitate previous ones". It should be noted that any advantage which can be claimed for the rollcrete dam in this regard is even more relevant to the soil-cement dam which can be expected to have a more natural colour.

Llewellin also presents a table comparing the various mixes discussed at the conference on the basis of paste volume and water / cementitious ratio. Paste volume ranges from 125 to 207 l/m^3 and w/c ratio from 0.88 to 4.3, flyash content of these mixes ranged from 0 to 85% of total cementitious content. For the soil used in this research, paste volume ranges from 270 to 355 l/m^3 for cement contents from 3.2 to 20% and corresponding w/c ratios range from 4.03 to 0.76. It is apparent that while w/c ratios are comparable to those discussed at the conference, paste volumes are considerably higher. This reflects the higher moisture content required for compaction of the soil-cement and this in turn reflects the relatively high fines content, 23% passing the 200 mesh sieve. It should however be noted that this moisture content is virtually independent of cement content and that the compaction characteristics of the soil-cement are more dependent on the soil than on the paste. It should also be noted that some reaction between the soil and cement appears to occur as indicated by the absorption of magnesium by soil-cement reported in section 9.4.5.f.

16.2.3 Developments Reported During 1982 and 1983

Dunchak and Hansen (1982) provide a useful summary of developments in the field of rollcrete and recommend the use of a richer, wetter bedding mix with smaller maximum particle size where the construction delay exceeds 8 to 24 hours. They suggest that the uncompacted surface of the downstream face can be regarded as sacrificial for curing purposes. Cementitious contents of rollcrete reported range from 60 to 300 kg/m^3 which compares favourably with the 95 to 331 kg/m^3 investigated in this research.

Mass (1983) reports on the use of rollcrete at the Guri Hydroelectric Project in Venezuela to construct a cofferdam. He reports that "inspection of the cofferdam after exposure to flows of 12 000 m^3/s showed no erosion damage of the unformed face. Also, no visible cracks have been observed over the entire length of the structure". Maximum aggregate size was 76 mm and cement content 100 kg/m^3.

16.2.4 Willow Creek Dam

The Willow Creek Dam, referred to previously, was constructed during 1982/83. Schrader and McKinnon (1984) report that the rollcrete used at Willow Creek did not agree with the normal relationship between water / cement ratio and strength, they note that "increasing the water to get better compactablity actually can cause higher strength with higher water / cement ratios". It is therefore considered that the figures offered by Llewellin do not necessarily provide the ideal method for comparing different mixes. In view of the high water demand of the soil itself, it is therefore suggested that allocation of the entire moisture content of the soil to paste volume is meaningless in the context of soil-cement and rollcrete containing fine material. Equally, in cases where a reaction occurs between the cement and fine fraction, some attempt should be made to evaluate the contribution of this reaction to strength development and to make allowance for this factor in determining equivalent paste volume and water / cementitious ratio.

It is considered that the construction of Willow Creek Dam, as reported in the paper by Schrader and McKinnon referred to above, constitutes an important landmark in the development of new techniques for the construction of dams. The design and execution of the project made maximum use of the advantages inherent in the material as proposed by Raphael and others and summarised in chapter 2 and all aspects of design and construction were reviewed from first principles. The envisioned savings in time and money were realised. The structure was "functionally complete one year after the work was advertised for bidding" as compared with the estimated three years for a rockfill dam and the bid price of US$14 million represented a $13 million saving. "If conventional concrete had been used for the alternate design, the savings would have been considerably more", average cost for rollcrete was about $29/m^3 including the facing as opposed to an estimated $85/m^3 for conventional concrete. Certain points from this paper which are felt to be relevant to this research are discussed below.

Schrader and McKinnon note that "attention to detail during the design (including construction, design, and scheduling details) and effective responsible management during construction were essential to the project". It is considered that this observation is of major significance in obtaining the maximum benefits from the new technique. In addition, specifications were relaxed to permit more flexible construction control. In conducting this review, it was disturbing to note that certain designers are already moving towards less flexible specifications and more elegant solutions which can only reduce the benefits of this technique. It is felt that this tendency should be avoided at all costs.

The aggregate at Willow Creek was stockpiled during winter to minimise temperature gains, this suggests that, in the South African context, efforts should be made to construct embankments during the winter as far as possible, possibly with restrictions on placing during the heat of the day if insolation is likely to present a problem during summer.

Up to 25% of the silty, sandy, gravel overburden was included in the quarried aggregate at Willow Creek and it was not necessary to waste material from the quarry. The final product was allowed to contain up to 10% of non-plastic material passing the 200 mesh sieve. This compares with the 23% present in the soil used in this research which exhibited a limited degree of plasticity. It should be noted that this soil was obtained by hand excavation and that the trial holes for Lund's design (Lund 1974) were excavated using a relatively small machine. Examination of a number of erosion gullies in the proposed borrow area indicated that the upper strata of the underlying partially weathered rock was highly fractured and it is considered that the coarse aggregate content of the soil could be substantially improved by ripping this material with a large crawler tractor.

This technique would probably produce an aggregate with a considerably reduced fines content and increased maximum particle size. Crushing to limit particle size would however be required. It is likely that the increase in coarse aggregate would significantly increase strength for a given cement content and that the resulting soil-cement would differ little in properties from rollcrete produced with either quarried or natural aggregate. At this level the distinction between the two materials becomes blurred and it is possible that the proposed dam might be better described as a rollcrete rather than a soil-cement dam.

In reviewing the available literature it was apparent that what was referred to as rollcrete in certain cases might have been classified as soil-cement under different circumstances. It appears that the resistance of engineers to accepting the idea of "soil" in their "concrete" is such that a stage may soon be reached where the term "rollcrete" will encompass the better quality soil-cement mixes produced with low plasticity soils containing a relatively large proportion of coarse aggregate.

Placing specifications at Willow Creek required that a minimum of one lift should be placed in three shifts in order to minimise cold joints with a maximum of three lifts in two shifts in order to limit problems related to heat generation. Two eight hour shifts were worked six days a week in order to minimize cold joints with a limit of exposure of 871 centigrade degree hours before surface preparation and the application of a bedding mix was required. Problems were however experienced with maintaining joint cleanliness to the extent that while bond was not required for sliding resistance, a limited amount of seepage was observed on the downstream face. It is suggested that a conveyor belt system might be used to transfer material to the top of the embankment in order to eliminate the tracking of dirt by construction vehicles. Shear tests on large blocks confirmed that bond was not required to develop adequate sliding resistance.

It was observed that "the presence of the gallery near the upstream face created a short flow path which allowed more seepage than desired". However, the rate of seepage reduced with time confirming the observations of De Groot (1971a) referred to in chapter 1 and the results of the leaching tests reported in section 9.4.7.k.

A program of drilling was undertaken to examine cores and limited grouting of the foundation was found to be necessary. In conjunction with this, grouting of the joints near the upstream face was undertaken to reduce seepage. Pressure tests in the boreholes indicated the in situ permeability to be less than expected.

In the light of examination of the cores referred to above, Schrader and McKinnon (1984) suggest that the sand fraction of the mix (passing the 9.5 mm sieve) should be further increased above the 35 to 40% used for the 76 mm maximum aggregate size mix and 50 to 55% used for the 38 mm mix. The corresponding percentage for the soil used in this research is 87% for what is essentially a minus 38 mm grading, the ripping program suggested above for the Rondebosch site would have the effect of reducing this percentage and would probably bring it into line with the levels suggested by Schrader and McKinnon. It is also reported that "tests made by allowing the natural silt content to increase showed this to further increase the RCC strength".

The downstream slope was constructed at 0.8 to 1 as the limit at which control could be maintained without special equipment. The slight overbuild provided "additional stability and eliminated the inconvenience, complication, and cost of forming a steeper slope which structural analysis would have allowed". The reinforced earth technique of supporting the vertical upstream face was modified to use larger panels (1.2 by 4.9 m) and a reduced number of anchor bars with a resulting additional saving of $600 000.

An integral spillway was used. This was constructed with a conventional mass concrete cap but the facing had the same surface as used for the rest of the downstream face.

16.2.5 After Willow Creek

Oliverson and Richardson (1984) report on the design of the Upper Stillwater Dam where they found the rollcrete gravity dam to be the lowest cost solution after considering earth embankments, concrete faced rockfill, conventional concrete gravity and massive head buttress alternatives. They report that internally vibrated concrete will be used for foundation levelling up to 4.6 m thick in order to provide a 30 m platform for initial placement of rollcrete. The levelling concrete will also be used to initiate the crossfall for the rollcrete layers, typically 0.02:1. Conventional concrete slipformed facings, as proposed by Dunstan, are to be used on both faces.

It was noted that a crossfall was used in a number of cases in order to increase sliding resistance and to provide drainage on the construction surface.

International Engineering Company (1984) in a leaflet on the design of the Middle Fork Dam indicate a crossfall of 0.05:1. They report that "the computer analysis, which utilized thermal characteristics determined from laboratory testing, indicate that no special measures are needed to prevent cracking of the RCC". Thermocouples will be installed to monitor thermal behaviour. The facing concrete, which is placed behind temporary forms immediately before placing of the rollcrete is a conventional mix designed for freeze-thaw resistance.

PCA information sheets supplied by Hansen (1984b) indicate five rollcrete dams currently under construction, including the two mentioned above, and a further four planned for bidding in 1985.

Casias and Howard (1984) have reviewed the twenty year performance of soil-cement facings on a number of USBR dams and report that the facings have generally performed satisfactorily. Problems that have been experienced have largely resulted from poor interlayer bond which has permitted separation and breakout of slabs of soil-cement. It should be noted that with the steep slopes proposed for the soil-cement dam, the exposed layer edge will be considerably reduced and a degree of compression on the upstream facing will be present at all times thus resisting any tendency for opening of construction joints. Casias and Howard report that the USBR is currently investigating means of improving interlayer bond by reducing construction delays, brooming joint surfaces and treating the surface with a neat cement slurry immediately prior to placing the next layer. So far, the results of the investigation appear to be giving promising results.

A conference on "Materials for Dams 84" is to be held in Monte Carlo in December and the May 1985 ASCE Spring Convention in

Denver will include two symposia on "Roller Compacted Concrete - - A-State-of-the-Art" It is therefore apparent that the use of rollcrete is rapidly becoming established. Accordingly, it is suggested that, in view of the shallow, sandy residual soils present on many dam sites in South Africa and the current drought and shortage of funds, the use of the soil-cement / rollcrete dam using materials produced by ripping should be investigated as a matter of urgency.

16.3 PROPOSALS FOR THE RONDEBOSCH SOIL-CEMENT DAM

A brief analysis of the required section for the Rondebosch Dam site was undertaken to enable the general form of a possible soil-cement solution to be examined. The analysis was undertaken using the gravity analysis program and techniques used to generate the data presented in chapter 3.

16.3.1 Selection of Section

The section dimensions are based on those proposed by Lund (1974) and allow for increased freeboard so that an ungated spillway can be used. It was considered that the use of an ungated spillway was consistent with the objective of simplicity in the soil-cement dam. However, should the cost of acquiring additional land to accommodate the higher flood lines warrant it, a gated spillway could undoubtedly be incorporated. In this case, a reduction in section would result in reduced cost for the soil-cement solution. Volumes are based on those for the earthfill embankment proposed by Lund which was sited at an angle to the the buttress dam actually constructed. The soil-cement dam could probably be located on the same axis used for the concrete dam and a reduction in volume could therefore be achieved. It should also be noted that overtopping of the soil-cement dam can be tolerated under the extreme flood so that a smaller freeboard allowance could be accepted. It is therefore considered that the estimate of embankment volume used here is conservative.

Dimensions of the earthfill embankment were:-

Crest width	6.0 m
Height of spillway crest	35.4 m
Maximum reservoir depth at design flood	39.4 m
Maximum height	40.9 m
Volume of the proposed earthfill embankment	890 000 m^3

It was noted in section 3.2.c that further consideration should be given to the use of a vertical upstream face in view of the limiting slope of 0.8:1 established at Willow Creek for placing without support. Dinchak and Hansen (1982) also report that the use of a vertical upstream face was necessary before roller compacted concrete was found to provide a competitive solution for the Zintel Canyon Dam. Accordingly, optimum sections for both the standard gravity and equal slope sections were determined. For the soil-cement used in this research with analysis for overturning stability under the maximum design flood with full uplift and dry density, a limiting downstream slope of 0.66:1 was determined for the dam with vertical face whereas for the case of equal upstream and downstream slopes, a value of 0.43 applied. The volume of the equal slope option is thus 22% greater than that with vertical upstream face. Maximum stress at the toe is also marginally increased. It was therefore concluded that the use of a vertical upstream face was to be preferred and further analysis is of this section.

Analysis of this section indicated that with zero uplift, an increase in theoretical downstream slope to 0.47:1 was possible. Increasing the unit weight from 20 kN/m^3 to 22 kN/m^3 indicated an increase in slope to 0.6:1 and the two factors combined permit an increase to 0.44:1. Since it is considered unlikely that significant drying of the material to any depth in the embankment will occur, this indicates an increase in safety factor. In addition, provided the foundation is adequately grouted and no tension occurs on the upstream face, it is considered unlikely that anything approaching full uplift will occur under the entire structure. The true stable section would thus lie in the range 0.66 to 0.44:1. However design to a slope steeper than 0.66 is not considered prudent.

In selecting a downstream slope, it was noted that the Willow Creek Dam was constructed with a slope of 0.8:1 for ease of construction without forms. The Upper Stillwater Dam is being constructed with a stepped section, the lower portion of which has a slope of 0.6:1 and is retained using slipformed curbs as proposed by Dunstan (Oliverson and Richardson 1984). In the present case, a reduction in volume of 26 000 m^3 results from the use of the steeper 0.66:1 slope. From the available data for the Upper Stillwater Dam, it was estimated that a volume of 33 900 m^3 of high grade conventional concrete would be required for slipforming of both faces, a total saving of 59 900 m^3 of soil-cement. For this case, the ratio of volume of soil-cement saved to volume of high grade concrete used was 1.77. For the mixes used at Upper Stillwater, the ratio of cost of the high grade conventional concrete facing mix to that of the best quality rollcrete mix used on the project was 2.03, for the hearting rollcrete this increased to 2.52 (PCA 1984), it was therefore concluded that the 0.8:1 slope was more economic to construct as well as having the added advantage of ease of construction and increased resistance to overturning.

The absolute worst case for overturning stability would involve a massive lens of water under the structure developing the full reservoir head over its entire area. This would result in a rectangular pressure distribution under the structure so that only the submerged unit weight was effective. For the maximum flood with downstream slope of 0.8:1 and unit weight of 12 kN/m^3, the line of action of the resultant is 0.224 m outside the third point and a tensile stress of 15 kPa is indicated at the heel. Since some degree of composite action within the structure can reasonably be expected and since no situation in which the entire structure will be "floating" can be conceived, it is considered that there is no need for a complex system of relief wells and drains thus further reducing the complexity of the structure. It is considered that a basic program of foundation monitoring in conjunction with a limited grout program will be entirely sufficient.

16.3.2 Seepage

The problem of seepage on construction planes at Willow Creek was referred to in the previous section. It is considered that if suitable precautions are taken to avoid tracking of dirt onto the construction surface, this problem can be largely overcome at source. The upstream facing system used at Willow Creek is felt to be appropriate to the soil-cement structure proposed here. If the material adjacent to this facing is not subject to traffic by construction plant, the joint surfaces are brushed and slushed as necessary and a reasonable quality soil-cement mix is used, an upstream facing which is effectively impermeable can be obtained at low cost.

In any event, the degree of seepage reported by Schrader and McKinnon is such that it is unlikely to lead to alarm and certainly will not lead to significant deterioration of the structure. Flow down the uncompacted downstream face is unlikely to be particularly noticeable and could assist the establishment of the natural plant cover suggested by Llewellin (CIRIA 1981). Alternatively, the use of a no-fines mix at the downstream face, could provide the necessary drainage with no overbuild since the full section is not required by structural considerations.

16.3.3 Spillway

The structures referred to in the previous section all make use of conventional overflow spillways incorporated in the embankment.

The spillway at Willow Creek uses a conventional mass concrete cap but the downstream face of the spillway is the same uncompacted material used for the rest of the downstream face. The erosion resistance of the coffer dam at Guri, referred to in the previous section (Mass 1983), supports this design solution. It is proposed that this technique should be considered for the soil-cement dam. Further research is however required to determine its suitability. Aspects requiring investigation are the effect of maximum aggregate size, grading and aggregate characteristics and the effect of cement content.

Murray and Schultheis (1977) report on the use of fibre reinforced concrete and polymerisation of concrete to increase the resistance to cavitation of the spillway at Dworshak Dam. It is possible that fibre reinforcement of soil-cement could be of benefit and that a suitable polymer could be introduced into the soil-cement during mixing with polymerisation of the facing soon after compaction.

Alternative methods of spillway surfacing might include the use of a horizontally slipformed interlocking facing of high strength concrete, as referred to previously. This might be tied into the embankment using reinforced earth techniques if necessary. Further alternatives might be precast units or cast in situ mass concrete, both tied into the embankment if required.

These alternatives could all be investigated by means of a trial embankment and it is considered that this represents the only realistic method of investigation. The ideal solution would involve the construction of a weir downstream of the spillway of an existing dam located so that maximum flow velocity was experienced.

It should be noted that cavitation is generally not a problem for velocities of less than 12 m/s or falls of less than 30 m (Murray and Schultheis 1977). This suggests that the problem could be reduced by locating the spillway towards the ends of the embankment rather than at its center and using soil-cement lined channels to train the water into the river bed. This could be implemented as a form of side channel spillway designed so that the length of the embankment was not increased.

It should also be noted that the relatively flat 0.8:1 downstream slope is likely to lead to a spillway crest that is broader than the nappe profile thus resulting in positive hydrostatic pressure along the contact surface (USBR 1976a). Under these conditions uplift forces on the crest or spillway lining are unlikely to occur and special precautions to tie the spillway lining to the embankment should not be necessary. While this spillway design is not the most efficient, it will be simpler to construct in soil-cement and the required capacity should be available from an increase in spillway length or reservoir depth.

16.4 DESIGN OF THE RONDEBOSCH SOIL-CEMENT DAM

16.4.1 Values for Design

Values for design obtained from the above analysis are as follows:-

Reservoir Empty

Maximum stress at heel .... 1.018 MPa
Maximum principal stress at toe .... 0.035 MPa

Normal Operation, Reservoir Full

Maximum effective principal stress at toe ..... 0.511 MPa
Maximum effective stress at heal (no uplift) .. 0.727 MPa
Horizontal force .... 6.147 MN/m
Vertical force (full uplift, dry unit weight) 11.567 MN/m

embankment rather than at its center and using soil-cement lined channels to train the water into the river bed. This could be implemented as a form of side channel spillway designed so that the length of the embankment was not increased.

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Maximum effective stress at heal (no uplift) .. 0.727 MPa
Horizontal force .... 6.147 MN/m
Vertical force (full uplift, dry unit weight) 11.567 MN/m

Maximum Design Flood

Maximum effective principal stress at toe ..... 0.692 MPa

Maximum effective stress at heel (no uplift) .. 0.617 MPa

Horizontal force .... 7.614 MN/m

Vertical force (full uplift, dry unit weight) 10.807 MN/m

16.4.2 Strength Required for Gravity Stability

In terms of the USBR recommendations for "Design of Gravity Dams" referred to in section 3.2.d (USBR 1976a), the factor of safety for compressive loads is 3 under usual and 2 under unusual conditions:-

For maximum stress at the toe:-

Usual loading = 0.511 x 3 = 1.53 MPa

Unusual loading = 0.692 x 2 = 1.38 MPa

Required toe strength = 1.53 MPa

Required cement content from graph 10.9:-

At 28 days = 2.5%

90 days = 1.7%

For maximum stress at the heel:-

Usual loading = 1.018 x 3 = 3.06 MPa

No unusual loading condition

Required cement content from graph 10.9:-

At 28 days = 5.0%

90 days = 3.6%

16.4.3 Strength Required to Resist Sliding

The USBR (1976a) requires a factor of safety of 3 for usual loading and 2 for unusual loading with a value of r=1 in equation 3, in chapter 3 (section 4.3.2 of Robertson and Blight 1978).

For usual loading:-

$$3 = (11.567 \tan 40 + 1 \times 38.72\ C') / \ 6.147$$
$$= 1.58 + 6.14\ C'$$
$$C' = 0.23 \text{ MPa}$$

For unusual loading:-

$$2 = (10.807 \tan 40 + 1 \times 38.72\ C') / \ 7.614$$
$$= 1.19 + 5.09\ C'$$
$$C' = 0.16 \text{ MPa}$$

Required C' = 0.23 MPa

Graph 12.8 indicates that this cohesion is obtainable from the unstabilized soil including coarse fraction but this is determined by extrapolation.

Section 16.1.4 proposes:-

Factor of safety on residual (frictional) strength ... 1.2
Factor of safety on ultimate strength 2.0
Minimum cement content 3.2%

Residual strength:-

The values of 1.58 and 1.19 above represent the frictional strength component of safety factor under usual and unusual conditions, residual strength is therefore adequate (>= 1.2). Note that values are determined with dry unit weight of soil-cement and full uplift.

Ultimate strength:-

From the previous calculations with Q=2, the required cohesion is 0.068 MPa ((2 - 1.58)/6.14) under usual and 0.16 MPa under unusual conditions. As above, zero cement content is indicated.

Therefore minimum cement content of 3.2% applies.

Note that the discussion in section 4.3.2 reported a value of sliding safety factor of 5 with value of r=0.5 as being customarily applied. Applying these values indicates a required cement content of 6.5% for design on the 28 day strength.

16.4.4 Durability Requirements

Section 16.1.3 indicates a recommended cement content for facing layers of between 6 and 15% and suggests that this can be reduced by 1% for soil containing the coarse fraction. With a view to selecting a cement content for design, the discussion presented in section 7.1 was reviewed. It was concluded that the observations made in that section remained valid. In addition, no tests on specimens containing 10% cement indicated any basis to suggest that the material was in any way inadequate. The leaching tests indicate that 10% cement provides the optimum treatment level. Accordingly, it is proposed that a cement content of 10% should be used for the soil-cement behind the upstream facing units and on the downstream face. A cement content of 15% should possibly be considered for the spillway facing but further investigation of this aspect is required.

A cement content of 10% is therefore specified for the facing material. A 3 m horizontal facing width is considered adequate since no significant deterioration of the facing is expected. This width should be sufficient to allow placing without special equipment.

16.4.5 Thermal Considerations

As previously noted, no problems are expected from heat gain due to hydration of cement. A suitable construction schedule, such as that used at Willow Creek Dam, should be followed and steps taken to limit heat gain of aggregate and compacted layers by insolation. A limited program of thermal monitoring should be undertaken. Construction during winter may be specified under South African conditions if there is concern with regard to insolation gains. Overhead irrigation of stockpiles to raise moisture content to the required level and encourage evaporative cooling may also be employed. Blight (1962) provides information on the control of moisture content under arid conditions.

16.4.6 Specification of Cement Contents

The required minimum cement contents indicated in the previous sections are:-

For compressive strength of	toe	3.0%
	heel	5.0%
For sliding resistance		3.2%
For durability of facings		10.0%

Cement contents for structural considerations have been determined in terms of the 28 day strength of the material and can therefore be considered to be conservative.

As the use of soil-cement in this application is untried and a degree of resistance to its use is anticipated, it is suggested that a minimum cement content of 5% should be specified at this stage. This will provide factors of safety against sliding and compression of similar magnitude to those formerly applied to concrete as summarised in chapter 3 (section 4.3). This also avoids the need for a gradation of cement content to avoid overstressing at the heel inside the facing material.

The cement content specification of the soil-cement dam therefore:-

Hearting mixture 5.0
Upstream and downstream facing to a width of 3 m 10.0
Spillway facing to a width of 3 m 15.0

A few layers of 10% cement content material may be considered desirable over the foundations to assist in distribution of stresses and increase resistance to seepage. Conventional mass concrete may be required for foundation levelling.

A conventional concrete cap is proposed for the spillway in order to obtain the correct profile and prevent exfoliation of soil-cement layers if poor bond occurs on construction joints.

16.4.7 Settlement

For the cement contents proposed, unconfined compressive strength at various ages can be estimated from graph 10.9 and the various moduli can then be estimated from graph 13.15. To obtain a reasonable estimate of settlement, it would be necessary to determine settlements for increments of height based on a proposed rate of construction as the moduli increase with compressive strength and hence age. The pressure distribution at the base of the embankment also changes considerably during reservoir filling and this would also require an incremental analysis to estimate total settlement.

As an indication, the construct and fill modulus for the material with 5% cement increases from 0.04 GPa at 28 days to 0.06 GPa at 200 days, for the material with 10% cement the range is 0.08 to 0.12 GPa. For the reservoir operation modulus, the corresponding values are 0.95 to 1.15 and 1.25 to 1.4 GPa for 5 and 10% respectively. Incremental settlement for loading to end of construction heel stress of 1.02 MPa ranges from 2.3 to 1.7% at 28 and 200 days for material with 5% cement to 1.4% to 0.8% for material with 10% cement.

These figures suggest that a problem may arise due to differential settlement between the 10% facing material and 5% hearting material which could lead to additional stresses being transferred to the facing. This could necessitate a more gradual change in cement content between facing and hearting but it is considered that the strength of the material is sufficient to accommodate this and that no particular problems will result. The higher stress level in the facing may assist in reducing permeability.

It is considered unlikely that compression of any layer in the dam will exceed 1% and that this is unlikely to cause any significant distress to the structure. It should be noted that if the proposal to increase the coarse aggregate content of the soil is adopted, the compressibility of the soil-cement is likely to be significantly reduced. It should also be noted that the maximum stresses calculated here will apply over a limited length of the embankment and that some degree of redistribution of stresses across the valley can be expected.

16.5 COST COMPARISON FOR RONDEBOSCH DAM

A realistic cost comparison is not possible without detailed designs and estimates for each solution, however an attempt is made here to evaluate relative costs in a variety of ways in order to obtain some indication of the viability of the structure proposed in the previous section. Costs quoted in the following discussion have been adjusted to the third quarter of 1984 as far as possible.

In order to assess the approximate cost of the structure proposed above, an estimate of the average cement content was required. Since the average valley tends to be more parabolic than triangular in profile, it is considered that the average cement content at a section corresponding to two thirds of the maximum embankment height will be reasonably representative. On this basis, the volume of 5% hearting mix is estimated at 131 000 m^3 and that of the 10% facing mix at 72 000 m^3. Average cement content is therefore 6.8% or 126 kg/m^3 (213 lb/yd^3).

16.5.1 Comparison in Terms of United States Data

The PCA "Guide for Estimating Cost of Soil-Cement Slope Protection (for Construction in 1982)" (PCA 1982) indicates a cost of processing soil cement in excess of 100 000 cubic yards of US$8.2?/$yd^3$. This translates to $10.71/$m^3$. The USBR "Construction Cost Trends" index shows no change over the period 1982 to July 1984 (USBR 1984), accordingly the above figure is considered a reasonable estimate of current US prices. The June 1984 report on "Materials prices" in Engineering News Record indicates bulk cement costs in the range 52 to 73 US$/ton. Adopting a price of $70/ton and assuming a short ton (2000 pounds) translates to $77/t (metric tonne = 1000 kg).

For the cement content of 126 kg/m^3 the cost of cement is $9.71/$m^3$ giving a total cost of $20.42/$m^3$. The cost of cement at the Willow Creek Dam site was $117/t ($106.20/ton) indicating a cement cost of $14.79/$m^3$ and total cost $25.50/$m^3$. This price range compares favourably with the average $24.68/$m^3$ ($19.00/$yd^3$) of Willow Creek Dam (Hansen 1984b).

Volume of concrete actually used in the Rondebosch Dam was 100 000 m^3 (Manson 1984). Hansen (1979b) provides a graph for estimating the "Cost of Mass Concrete in Dams" which takes account of buttress, arch and gravity dams, from this the 1979 cost of mass concrete is estimated as $55/$yd^3$. The USBR cost index increases from 112 to 155 over this period (USBR 1984) to give a cost of $99/$m^3$ ($76/$yd^3$) which translates to a total cost of concrete of $9.9 million. The 203 000 m^3 estimated for the soil-cement dam at

Rondebosch gives an embankment cost in terms of the figures in the previous paragraph of between $4.14 and $5.18 million a saving in excess of 50% on the basic structure.

Assuming that the placing cost of earthfill is half the processing cost of soil-cement indicates a cost for the earthfill embankment design of $4.77 million. This is comparable to the figures for the soil-cement alternative but does not take account of the cost of the slope protection, spillway, cofferdams and outlet works most of which can be expected to be more costly than their equivalents on the soil-cement dam. The use of half the soil-cement processing cost is arbitrary and probably underestimates the cost of earthfill particularly when the much longer haul distances for earthfill, referred to in section 5.1, are taken into account. It is therefore considered that the soil-cement option would probably be significantly more economic than the earthfill alternative on this site. It should be noted that the Merritt Dam cost comparison in chapter 3 (section 5.2 of paper) shows a major cost advantage for soil-cement over the earthfill embankment erected on that particular site.

The above comparisons indicate that, in terms of current US costs, soil-cement is probably competitive with roller compacted concrete and significantly less costly than the concrete buttress and earthfill alternatives.

16.5.2 Comparison in Terms of South African Data

A summary of the costs for the Rondebosch Dam were obtained from Mr E. Manson of Stewart, Sviridov and Oliver, consulting engineers (Manson 1984). The dam was constructed in the period September 1974 to August 1977 and median date for costing purposes was taken as February 1976. Construction indexes for this type of construction obtained from Central Statistical Services in Pretoria average 116.6 in February 1976 and 373.3 in July 1984, a compound rate of inflation of close to 15% per annum. The adjusted costs for the Rondebosch concrete buttress dam with P and G, escalation and contingencies allocated appropriately, in millions of South African Rands, were:-

Excavation	2.5
Concrete, shuttering, reinforcing and joints	11.0
Pipework	1.4
Radial Gates	2.6
TOTAL	17.5

As a first approximation, it is considered that the cost of excavation will be comparable for the soil-cement dam -- the area requiring preparation will be greater but the degree of preparation will probably be less. It is also anticipated that the cost of pipework will be of similar magnitude although an increase for the soil-cement dam is possible. As discussed in section 16.3, the analysis of the soil-cement alternative is based on a structure with ungated spillway. The R2.6 million item for radial gates therefore does not apply. It should be noted that this saving may be offset by an increase in cost of expropriation to accommodate higher flood lines but that inclusion of radial gates would lead to a reduction in the height of the structure and hence a reduction in cost.

Escalation over the course of the contract contributed R3.6 million to the above figure of R17.5 million, 21% of the total cost of the structure, the reduction in construction period of the soil-cement dam thus represents a potential saving of several million rand.

An estimate for soil-cement processing (borrow, stockpile, haul, mix and compact) of R4.60/m^3 was obtained from a major civil engineering construction company (Macdonald 1984). As they had no experience with central plant mixing of soil-cement, and did not have the equipment on which to base such costs, the estimate is based on roads experience using a spreader box and plough for mixing. Delivered cost of cement on site was estimated at R125/t giving a total soil-cement cost of R20.40/m^3 and total embankment cost of R4.14 million which is approximately 37% of the concrete cost of the dam as built.

It is considered that the processing cost may be something of an underestimate as the soil-cement dam requires a tighter specification and more sophisticated processing than generally used on roads in South Africa. Doubling the above figure to R9.02/m^3 gives a total soil-cement cost of R5.08 million, 46% of the concrete cost, still a considerable saving.

Marais (1984) reports costs for cement stabilized gravel sub-base ranging from R4.56 to R9.99/m^3 and cement costs of R90 to R111/t depending on area and nature of material. These values compare favourably with the range considered above.

For the earthfill alternative, average haul distances are more than double and it is considered that this would more than offset the cost of plough mixing of stabilizer. Cost of the order of R4.1 million for the earthfill alternative is therefore indicated at R4.60/m^3.

While these estimates do not take account of the cost of the facing elements for the soil-cement dam and other secondary costs, it is apparent that a large saving can be achieved. The absence of radial gates and reduction in construction period should also lead to reduced construction costs.

16.5.3 Cost Comparison -- Conclusion

As previously noted, no cost comparison can be reliable unless it is based on detailed designs prepared by the same design team thoroughly familiar with the characteristics of the site. If these designs are submitted to open tender, as at Willow Creek Dam, then a true indication of the relative merits of the various solutions will be obtained.

It is however considered that the general financial viability of the soil-cement dam has sufficient theoretical validity to justify its consideration on suitable sites. The above cost comparisons serve to illustrate that soil-cement is almost certainly considerably less costly than conventional concrete on a large variety of sites and can also be expected to be less costly than earthfill in most cases where foundation conditions are adequate. In particular, a considerable cost saving would almost certainly have been effected on the Rondebosch Dam site if soil-cement had been specified.

16.6 CONCLUSION

The above comparison indicates that the soil-cement used in this program is likely to produce a structure of similar cost to a rollcrete dam. The relative merits of the two solutions is likely to be very much a function of a particular site and, as noted in section 16.2.4, the distinction between the two materials is becoming increasingly blurred. Whether soil-cement produced on the Rondebosch Dam site using material including a significantly increased coarse aggregate fraction produced by ripping is truly soil-cement or should be regarded as roller compacted concrete is a matter for debate. If soil is defined as "naturally occurring weathered rock material that can be excavated using earthmoving machinery", then the material under discussion is surely soil-cement. However, if "roller compacted concrete produced using locally available aggregate with relaxed specifications and without blasting" is a more acceptable term to the general public, and many engineers, then that is a perfectly acceptable definition of the material proposed here.

Whatever the terms used to describe the material, the soil-cement investigated in this research has been shown to be suitable for the construction of large dams designed on gravity principles and the suitability of the material with increased coarse fraction can only be improved.

CHAPTER 17

RECOMMENDATIONS FOR FURTHER RESEARCH

Throughout the discussion of the results of this test program, aspects requiring further investigation have been noted. Many of these investigations would be relatively minor and would serve to enhance the results of this research or confirm assumptions but are unlikely to contribute significantly to knowledge in this field. However, it is considered that additional investigation in certain areas would be valuable and these proposals are summarised in this chapter.

17.1 CONSTRUCTION OF TRIAL EMBANKMENT

In considering these proposals, it should be noted that it is the author's opinion that the results of this research, together with the existing body of knowledge presented in the list of references and bibliography, provide sufficient grounds to justify the construction of a trial embankment. This embankment might take the form of a structure erected within the reservoir of a dam now under construction or of a weir downstream of an existing dam, but it is considered that there is sufficient information to justify the construction of a trial embankment as a basis for tenders for the construction of a soil-cement dam as an alternative solution for a proposed structure. It is felt that this would be particularly appropriate in the present South African context of drought and a depressed economy.

The construction of a trial embankment is considered to be the most valuable form of further research that could be conducted. While the relatively high cost of such a trial is accepted, the observation of Holtz and Hansen (1976) with regard to the Bonny Reservoir Test Section is appropriate; "it is pertinent to note here that this expenditure" (US$80 000 in 1953) "has been repaid many times, as is so often true of sound research ventures".

17.2 ALTERNATIVE WETTING AND DRYING TEST

An alternative wetting and drying test procedure making use of the standard 37.5 mm diameter by 75 mm long test specimens used in this research is proposed. The reduced size of the specimens accelerates the mass gain during repeated wetting and drying cycles for specimens with 10% cement. It is postulated that at lower cement contents, the loss of material will be similarly accelerated as the results of the leaching test program indicate that deterioration in the wetting and drying test is primarily due to surface leaching.

An alternative test technique might involve the use of these specimens with a five hour soaking period and twenty four hour cycle period without abrasion. Abrasion testing of the specimens would only occur at the end of twelve cycles and would be conducted using the vibrating tumbler described in appendix B.

A technique based on these proposals will require considerably less time to conduct and permit more specimens to be tested at the same time. The procedure would also require considerably less technician time and lend itself to a significant degree of automation.

The results of the leaching tests suggest that the test could be made more severe, or accelerated, by reducing the temperature during the drying phase or dispensing with the drying phase altogether. It is possible that comparable results could be obtained using a leaching bath similar to that used in this research and leaching the specimens for a period which might be as short as two to three days. Losses might be determined simply in terms of change in oven dry mass relative to control specimens or by abrasion in the tumbler referred to above. Unconfined compressive strength might provide an alternative basis for comparison.

The depth of deterioration, as defined in section 9.4.12, is suggested as a means of comparing losses for tests using different specimens and test procedures.

Considerable research is required to extend the results of this program in order to establish whether the observed scale effects are consistent and to establish a test procedure that provides a reliable correlation with the standard test. This investigation should also examine the possibility of developing a test of equal severity to the freeze-thaw test but more appropriate to applications where freezing seldom occurs.

17.3 ALTERNATIVE METHODS OF SEALING TEST SPECIMENS

The method of sealing test specimens used in this program involved wrapping the specimen in aluminium foil and dipping it in paraffin wax to obtain a complete seal.

Reactions between the foil and cement occurred in those specimens with high cement contents and where the specimens had been soaked prior to sealing. This reaction occurs when aluminium is exposed to moisture in a highly alkaline environment as is present when cement hydrates. While there was no indication that this reaction was in any way detrimental to the test specimens, alternative methods of sealing the specimen without allowing the wax into direct contact with the soil-cement should be investigated. This is considered desirable in all cases where soaking or saturating of the test specimen is required.

Possible alternatives include the use of self sealing plastic kitchen film or laboratory film, possibly followed by dipping in wax and the use of closely fitting plastic bags with self sealing "zips" or sealed by heat welding or adhesive tape.

17.4 ALTERNATIVE LEACHING RATE DETERMINATION

The method of calcium analysis used in the leaching test program was costly and only a limited number of leachate samples could be analysed.

Evaporation of leachate samples under controlled conditions could permit gravimetric determination of the dissolved solids content of the leachate. This could be associated with a limited program of detailed analyses to determine the composition of the dissolved salts.

The leachate samples from this test program are still available and tests could be conducted using these samples.

17.5 RELATIONSHIP BETWEEN COHESION AND UCS

The results of the triaxial test indicate a correction of the order of 11% to the theoretical relationship between cohesion and unconfined compressive strength proposed in equation 1 in chapter 3. While this discrepancy is probably within the range of experimental error, further investigation of this aspect might be of interest.

17.6 EROSION RESISTANCE

While the erosion resistance of soil-cement with regard to wave action on slope protection is well established, the proposal to include an overflow spillway as part of the soil-cement embankment dam indicates the need for further investigation of the erosion resistance of soil-cement when subject to high flow velocities.

It is necessary to investigate means of obtaining maximum erosion resistance of soil-cement as well as alternative methods of spillway construction that would still permit construction of an integral spillway.

In order to investigate the erosion resistance of soil-cement subject to high flow velocities, the construction of a trial overflow weir downstream of a major dam would provide the most definitive solution. With reasonable construction control it should be possible to construct an embankment with each length of, say three meters constructed with a different soil-cement mix, all placed consecutively and compacted simultaneously. Variations to be investigated might include different cement contents and gravel fractions, fibre reinforcement and polymerisation. In addition, the effect of accelerated curing using black plastic sheeting might also be investigated. The use of slipformed and other facing elements and the provision of a sacrificial layer might also be investigated.

17.7 BOND ON CONSTRUCTION JOINTS

Further tests to determine techniques for mobilising the full shear strength of the material on construction joints are required.

These tests could be conducted using the apparatus used in this program with suitable modifications to the compaction procedure to ensure adequate compaction on the construction joint. Alternatively, tests could be conducted by placing a number of layers of soil-cement using the equipment and techniques normally used on site. Blocks of material above the construction joint could then be isolated and tested in situ with a range of vertical stresses.

Tests are also required to determine the effect of variations in construction delay between layers and the maximum permissible delay without joint treatment.

17.8 EFFECT OF THE PRESENCE OF COARSE AGGREGATE

The results of the direct shear tests indicate an improvement in the strength of soil-cement containing the gravel fraction retained on the 4.75 mm sieve and suggestions are made for relating the results of tests on specimens prepared using the fraction passing the 4.75 mm sieve to the material prepared using the whole soil.

Further investigation of the effect of the coarse fraction on the strength in direct shear and compression is required.

Beneficiation of the soil with high strength aggregate used either to replace part of the gravel fraction or to supplement it, should also be investigated. Investigation of this factor should be included in any program of in situ shear tests such as those proposed above.

17.9 USE OF OTHER STABILIZERS

In view of the extent of this laboratory program, it was not possible to investigate the use of other stabilizers and soil types.

As previously mentioned, it is considered that the documents presented in the bibliography provide a reasonable basis for extending many of these results to other soils and stabilizers.

This research is currently being extended by G. M. Bentel in the Department of Civil Engineering at the University of the Witwatersrand who is investigating different soil and stabilizer types.

Further investigation into certain aspects may well be indicated by the results of Bentel's research.

CHAPTER 18

CONCLUSIONS

The suitability of soil-cement for use as a material in dam construction has been confirmed. None of the material properties investigated give any grounds to doubt its suitability.

While further research into certain aspects of the properties of soil-cement is still required, it is considered that these aspects would be best investigated by means of a trial embankment constructed on the site of a proposed dam with the specific objective of providing the information necessary for the design and construction of a soil-cement dam on that site.

It is suggested that a higher coarse aggregate content than conventionally used with soil-cement will have a beneficial effect on material properties but is by no means essential to produce material suitable for dam construction. This material will differ little in properties from roller compacted concrete and in the extreme case can be regarded as such. The relative merits of roller compacted concrete and soil-cement will depend on individual sites.

The design used for Willow Creek Dam is considered to be an excellent example of an engineered structure which is simpler and cheaper to construct while at the same time being at least as safe as conventional solutions which require elaborate precautions to ensure their long-term stability. It is considered that this type of design is equally well suited to the soil-cement dam.

The soil used in this research is ideally suited to stabilization with cement for use in dam construction and is characteristic of the shallow residual soils available on many potential dam sites in Southern Africa. It is estimated that a saving in excess of fifty percent of the cost of the concrete buttress dam erected on the site investigated would have been effected if the proposed soil-cement alternative had been employed. A considerable reduction in construction time would also have been possible.

In view of the current drought in South Africa and the shortage of funds for development, the use of soil-cement should be investigated with a view to immediate implementation in order to obtain maximum benefit from the available financial resources together with a rapid increase in available reservoir capacity.

REFERENCES

The following section lists all references cited in the text. Items of correspondence are cited as "correspondence with author" and details are supplied in the following section, **Correspondence.** A **Bibliography** of documents consulted during the research but not cited in the thesis is included after the list of **Correspondence.**

It is regretted that, due to pressure of time, it was not possible to study all documents as thoroughly as I would have liked. Consequently, it should be noted that the contents of this thesis do not necessarily take account of all the information presented in the documents listed in the list of **References** and the **Bibliography.**

The following abbreviations are used in the list of **References**, list of **Correspondence** and **Bibliography**:-

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ASCE	American Society of Civil Engineers, New York.
ASTM	American Society for Testing and Materials, Philadelphia.
BSI	British Standards Institution, London.
CEPSSI	Proceedings of the Continuing Education Program on Soil and Site Improvement, University of California, Berkeley 1976.
CIRIA	Construction Industry Research and Information Association, London.
CSIR	Council for Scientific and Industrial Research, Pretoria, South Africa.
CSIRO	Commonwealth Scientific and Industrial Research Organization, Syndal, Australia.
ECCD	Proceedings of the Engineering Foundation Research Conference on 'Economical Construction of Concrete Dams', Asilomar 1972. Published by ASCE.
FHA	Federal Highway Administration of the United States Department of Transport, Washington.

HRB	Highway Research Board of the National Academy of Sciences -- National Research Council, Washington.
HRBB	Highway Research Board Bulletin of the HRB.
HRR	Highway Research Record, formerly HRBB.
MCR	Magazine of Concrete Research.
PCA	Portland Cement Association, USA.
PCI	Portland Cement Institute, South Africa.
PROC__ICOLD	Proceedings of the __ International Commission on Large Dams.
PROC__SM&F	Proceedings of the __ International Conference on Soil Mechanics and Foundation Engineering.
RCCD	Proceedings of the Engineering Foundation Research Conference on 'Rapid Construction of Concrete Dams', Asilomar 1970. Published by ASCE.
RRL	Road Research Laboratory, formerly TRRL.
SAICE	South African Institution of Civil Engineers, Johannesburg.
STP	ASTM Special Technical Publication.
TRB	Transportation Research Board, National Academy of Sciences, Washington.
TRR	Transportation Research Record, formerly HRR.
TRRL	Transport and Road Research Laboratory of the Department of the Environment, Crowthorne, England.
USBR	United States Bureau of Reclamation, Denver, Colarado.
USCOLD	United States Committee on Large Dams.

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Items in the **Bibliography** have been divided into two categories, indicated by a superscript 1 or 2 after the date, e.g. "ANDAY M.C. (1961)[2]" indicates a category 2 item. The two categories are defined as follows:-

1. **Category 1** includes all those documents which were consulted in depth and which are considered to have made a direct contribution to the research as conducted. This includes theses, reports, papers, etc with some bearing on the research conducted and which were referred to in making decisions with regard to scope of experimental programs, designing experimental apparatus and procedures, performing background calculations or interpreting results. General reference books that were sources of specialist data used in design of apparatus etc are included. In the interests of brevity, those documents listed in the References section are not listed here. Personal communications with the author are listed in the section entitled **Correspondence**.

2. **Category 2** includes all those documents that were encountered as part of the literature survey, obtained from correspondents or acquired once the experimental program was under way, and which did not have a direct influence on the research in terms of specific input. These documents constitute the broad background of existing knowledge which together served to influence the direction of the research. The list includes documents relating to methods of soil stabilization other than

stabilization with cement, which were investigated prior to limiting the scope of the research to soil-cement. This information is included in the hope that it will be of assistance to others conducting research in this and related fields.

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APPENDIX A

COMPUTER PROGRAMS

A number of computer programs were written by the author specifically for this research and a number of commercial software packages were purchased for data processing etc. These programs are briefly discussed below.

The gravity analysis discussed in chapter 3 was performed using a program running on a Hewlett Packard model 97 programmable calculator. The program accepted input of the design parameters discussed in chapter 3 and output the stresses, factors of safety, etc discussed in that chapter.

The slope stability analysis discussed in chapter 3 was performed using a program running on a Hewlett Packard model 9830 desktop computer. The program calculated the factor of safety for slope stability on a slip circle as discussed in chapter 3. A manual iterative technique was used to find the center of the critical circle. An algorithm was developed to assist in this procedure but was not implemented in the program.

All data processing of the results of this research together with plotting of graphs and word processing of the thesis was undertaken on an Exidy Sorcerer microcomputer with 56k user memory and twin, single sided, soft sectored, disk drives formatted with 77 tracks, 16 sectors to give a capacity of 296k per drive.

The processing of the results of the research program was undertaken using the computer program **Supercalc**^{2} (TM), produced by Sorcim Corporation. Data files on disk can be supplied to

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All graphs were plotted with a Watanabe WX 4671 X-Y Plotter using a program written by the author using the **Microsoft BASIC** (TM) language, revision 5.2. The graph parameters and data files can be comprehensively and interactively defined by the user and the program is largely self documenting. Source code is comprehensively annotated and comprises a total of approximately 120k bytes of code in fifteen overlays. Copies of source code can be supplied to genuine research organisations for their own use, subject to receipt of an undertaking with regard to copyright and payment of costs.

This thesis was composed, edited and printed using the **Spellbinder** (TM) word processing program produced by Lexisoft Incorporated, version 5.12. This program was also used for program editing, for editing of tables and for editing of Supercalc data output to produce the data file format required for plotting. **Spellcheck** (TM) by Lexisoft was used for checking spelling in the text of the thesis.

APPENDIX B

VIBRATING TUMBLER FOR ABRASION TESTING

A vibrating tumbler used for the abrasion testing of stabilized briquettes of coal and ores for metallurgical purposes was referred to in section 8.4.4.

This tumbler was developed by Prof G.E. Blight at the Department of Civil Engineering at Witwatersrand University in the mid-seventies and was used successfully for tests for specific applications over a number of years. No report on this research has been published (Blight 1984) and, accordingly, brief details are provided here. The apparatus was developed because the accepted abrasion tests, e.g. the Los Angeles abrasion test (ASTM 535) and the ASTM tumbler test for coke (D294) required excessively large samples.

The apparatus consists of a closed drum with an internal diameter of 150 mm and 150 mm long. A longitudinal shelf projects from the inside of the drum which is suspended by a number of radial springs at its base. The drum slowly rotates about its horizontal axis with a frequency of 0.1 Hz and simultaneously vibrates perpendicular to this axis with a frequency of 30 Hz and amplitude of 2mm. A general view of the apparatus, with the cover of the drum removed, is presented in plate B.1.

Pairs of briquettes were tested after initial weighing by vibrating in the drum for intervals of five minutes. The specimens were then weighed, the drum cleaned out, and the specimens subject to a further period of five minutes vibration. This sequence was repeated a number of times to obtain a loss - time curve for the material.

In proposing apparatus of this type for a modified wetting and drying test using smaller specimens, it is envisaged that a single period of vibration, determined to produce results comparable to those of the standard test, would be used. In view of the higher strength of soil-cement, it is possible that a greater amplitude or frequency of vibration might be required to achieve the desired results in an acceptable time. A larger drum diameter may also be required to accommodate the 37.5 mm diameter by 75 mm cylinders used in this research and it might be found convenient to accommodate a set of six specimens in the drum simultaneously. It is however felt that the general principle has merit as a means of simplifying the standard wetting and drying test procedure.

PLATE B.1

VIBRATING TUMBLER FOR ABRASION TESTING

APPENDIX C

SUPPLEMENTARY INFORMATION ON APPARATUS

This appendix contains additional information with regard to the apparatus built specifically for this research. Much of the apparatus used in this test program was designed and built specifically for this project. Other items were significantly modified and others, while already in existence, were themselves custom built at some stage. As a result of the extensive scope of this investigation and the large numbers of certain items required, considerable design effort was expended in minimising costs by the use of alternative, low cost materials, simplification of design, relaxation of machining tolerances, etc. Existing apparatus and components were also modified where possible.

The object of this appendix is to provide additional information on some of the apparatus referred to in the body of the thesis in order to clarify operating procedures and provide additional information to those who might wish to build similar apparatus for any reason. The apparatus is illustrated by way of plates as dimensions are generally not critical and were frequently determined by existing equipment, space and available materials. Copies of the workshop sketches and drawings for most of this apparatus, together with more detailed photographs, can be supplied to any research institution interested.

In the following sections, occasional references will be found to specific manufacturers, these are made where a particular product has certain characteristics that suit it to the application in question or in cases where considerable difficulty was experienced finding a suitable product. Reference to these manufacturers does not necessarily mean that they produce the only acceptable product, there are almost certainly others producing equivalent articles.

The sequence adopted in this chapter corresponds to the sequence in which the tests for which the apparatus was used are discussed. Headings reflect the section to which the apparatus concerned is related.

C.1 Mercury Displacement Cell (6.3)

PLATE C.1
MERCURY DISPLACEMENT CELL AND SOIL DISKS

The mercury displacement cell was used for measuring the volume of disks of compacted soil produced to compare the effects of different methods of compaction. The 75 mm long, 37.5 mm standard cylindrical specimens were divided into six equal portions by means of polythene disks placed as the mould was filled. After compaction, the apparatus was used to determine volume for calculation of density to examine the variation within the specimen.

Plate C.1 illustrates the apparatus, without mercury together with a number of the soil and polythene disks. The cell was produced from perspex tubing with a turned base cemented to it. The cap was machined with an internal cone rising towards the center and ending in a vertical vent through which excess mercury drained. Four radial grooves were machined in the cone and lengths of perspex rod with spherical ends were cemented in them. The resulting ridges provided support to prevent damage to the soil disk, while allowing mercury to flow around it. The vent outlet was surrounded by a circular, sharp overflow weir for the excess mercury, which then drained down a channel and through the spout on the side of the cap. The weir provided a precise, reproducible level for the mercury.

C.2 Mould for Standard Cylinders (6.3 and 9.3.1)

The mould fabricated in accordance with BS 1924 (1975) which was used for compacting all the 75 x 37.5 mm specimens used in this research is illustrated in plate C.2. The mould dimensions were reduced for the specimen size used, the British Standard is for 100 x 50 mm specimens. This size was determined by the available triaxial apparatus and available head for the long term seepage tests. The mould was further modified by a 50% reduction in the height of the spacing collar indicated by the standard. This was found necessary to achieve equal plug projection prior to compaction which was found to be desirable to obtain the most uniform density distribution through the specimen.

The funnel illustrated was not indicated by the standard but was necessary to prevent particles sticking to the upper bore of the mould which interfered with insertion and removal of the top plug.

The funnel was formed from a length of steel tubing, with the OD machined to provide a snug fit in the mould bore, soldered to a funnel formed from galvanised steel sheet. The extrusion plug and tamping bar are also illustrated.

PLATE C.2

CYLINDER MOULD AND ACCESSORIES

C.3 Compaction Workstation (9.3.1)

Plate C.3 illustrates the general arrangement for compaction of cylindrical and disk specimens. The apparatus from left to right is:

1. Domestic food mixer with special mixing hook illustrated in plate 6.1.
2. Cylindrical mould and accessories (plate C.2).

c. large funnel [illegible]
soil-cement [illegible] was placed [illegible]
mixing.

d. balance (0.1 g), batching [illegible]
watch for batching and mixing control [illegible]

e. moisture content sample tins, felt [illegible]
of specimens and balance for weig[illegible]
moisture content specimens to 1 mg.

f. humidor; tin with close fitting lid, [illegible]
grid, in which specimens were placed [illegible]
the humid chamber for initial [illegible]

PLATE C.3
COMPACTION WORKSTATION

C.4 Sealing of Specimens (9.3.1)

Plate C.4 illustrates a partially sealed cylinder specimen. The circumferential foil has been wrapped around the specimen with a length of plastic covered wire (twist-tie) beneath and held in place with a self adhesive label with the specimen code on it. The end of the wire projects below the label for use in tearing the foil and wax on stripping. The disks of aluminium foil (heavy kitchen foil) are to the right in the picture. The disks were placed on the ends of the specimens and the circumferential foil folded over. The specimen and foil was then dipped in wax to provide the moisture seal. The foil prevented the wax from coming into contact with the specimen and possibly affecting its permeability.

PLATE C.4
SEALING OF SPECIMENS FOR CURING

C.5 Compressed Air Pumping Vessel for Distilled Water (9.2.8)

Distilled water was obtained from the Department of Chemistry where it was tapped directly from bulk storage tanks into drums carried in a Departmental vehicle. From these drums, the water was gravity fed to the pumping vessel shown in plate C.5 through the clear pipe at bottom left. The vessel was fabricated from 315 mm nominal OD high density polyethylene (HDPE) piping rated at 1200 kPa with 25 mm steel end plates faced with 6 mm UPVC.

PLATE C.5

COMPRESSED AIR PUMPING VESSEL FOR DISTILLED WATER

Compressed air at a pressure of 500 kPa was used to lift charges of water to storage tanks in the water store at the top of the building. The air was passed through a 5 micron filter to prevent contamination. Check valves on the inlet and outlet permitted operation by alternate venting and pressurising the vessel using the two valves on the top plate. The curved black pipe originating from the valve on the top left of the vessel directs vented air against the wall to dissipate its energy.

C.6 De-airing and Carbonating Vessel (9.2.8)

The de-airing and carbonating vessel is illustrated in plate 9.5 in chapter 9. Details of its construction and operation are discussed below.

This vessel was fabricated from 600 kPa 315 mm OD HDPE pipe with 6 mm steel end plates faced with UPVC. The three way valve on the top plate permitted connection to the vacuum supply and alternate venting to the low pressure nitrogen atmosphere of the de-aired water reservoirs. The float valve on the vacuum line prevented accidental flooding of the vacuum pumps. An H shaped manifold at the bottom of the vessel was connected to a pump used for pumping the de-aired or carbonated water to the appropriate storage reservoirs. Ring manifolds at the top and bottom were fitted with micro mist sprays.

For de-airing, a full vacuum, limited by the vapour pressure of water, was established and the distilled water was drawn into the tank through the top manifold, the mist sprays providing the necessary atomisation for de-airing. The water was held under vacuum in the vessel until required for recharging the de-aired distilled water reservoirs. The vacuum was then relieved by gradually venting the vessel to the low pressure nitrogen atmosphere and the water was pumped to the reservoirs. The next charge of water was then de-aired.

The bottom ring manifold was used for carbonating the de-aired water when required for recharging the supply for that element of the test program. After the water was de-aired, carbon dioxide was slowly introduced through the mist nozzles at the bottom of the tank until the vacuum was dissipated and a pressure of approximately 100 kpa was developed. This pressure was maintained for a period of several hours to ensure saturation. The pressure was reduced to atmospheric and the water allowed to stand overnight to allow excess CO_2 to dissipate, the water was then pumped to the storage reservoir.

The storage tanks in the water store were covered and air drawn into the tanks as they were tapped was filtered to prevent contamination. The water was drawn from these tanks to the de-airing vessel through a 5 micron water filter to remove any suspended matter, spores etc that could contaminate the water supply system or block the micro mist nozzles in the de-airing tank.

C.7 Water Reservoirs, Constant Head Tanks, etc (9.2)

Plate 9.5 also illustrates a number of reservoirs, a constant head tank and flow indicator. Plate C.6 shows the main reservoirs and constant head tanks, located to the left of those depicted in plate 9.5. Constant head tanks were located at different levels to accommodate the different levels of the leaching cell racks and different heads. Standard head was 3.75 m for the cylindrical specimens.

Sealing of the plastic drums was achieved by means of heavy aluminium tape backed with bitumen putty, as used in general industrial and domestic sealing applications (Flashband / Ditsit). This tape was pressed around and over the junction of the lid and drum and the clamping ring closed. Silicone rubber sealant was applied to the sealing surfaces before the lid was positioned and the tape applied.

PLATE C.6

WATER RESERVOIRS, CONSTANT HEAD TANKS, ETC

C.8 Leachate Sample Storage Rack (9.2.12)

Plate C.7 illustrates one of the racks built for storage of leachate samples. The rack is covered with heavy black polythene sheet and is fitted with a roller blind with side clamps to exclude light. This was done to prevent deterioration of samples or bottles due to exposure to direct sunlight and to prevent algal growth from any spores which might be present from the soil. Other racks were located in the constant temperature lab which was kept dark when not occupied so that additional light proofing of the racks was not required.

PLATE C.7

LEACHATE STORAGE RACK

C.9 Pipe and Fittings -- Leaching and Triaxial Equipment

Considerable difficulty was experienced initially in finding suitable sources of plastic tubing and brass fittings for much of the apparatus required for the long term seepage / permeability tests. Fittings were required that could be rapidly and reliably assembled and in some cases were required to hold significant pressures. In the case of the low pressure nitrogen system, fittings were required that would seal first time as the number of connections and the low pressure would make leak detection extremely difficult.

A source of brass pipe fittings was found but the fittings were weak as they had been machined from poor quality brass rod with a pronounced grain which tended to split when taper threads were tightened or pressure was applied and it was therefore necessary to find alternative sources. A large variety of pipe and fittings were ultimately used for low pressure applications. These included UPVC pressure pipe and fittings for manifolds, garden hose and low density polyethylene (LDPE) agricultural piping for vacuum lines and large supply and drain lines.

The fittings used for higher pressure applications, including all connections to the triaxial cells, were manufactured by two companies. High pressure fittings for plastic and copper tubing were manufactured by Enots Ltd (P O Box 22, Lichfield, Staffs WS13 6SB, England) represented in South Africa by Balco Ltd. These fittings were used on all connections to the triaxial cells because they could be repeatedly disconnected without loss of seal.

Pressure fittings for plastic tubing and matching coloured tubing (polyflo), transparent low pressure plastic PVC tubing, high pressure nylon tubing (nylo-seal) and pressure rated brass pipe fittings were manufactured by Gould Imperial-Eastman, Valve and Fittings Division (6300 West Howard Street, Chicago, Illinois 60648, USA), represented in South Africa by Beckman Instruments Ltd. Polyflo tubing and fittings were used for many of the connections in the leaching apparatus where small diameters (up to 12.7 mm OD) and colour coding were required. Clear PVC tubing was used for the leachate collection tubes and for visual level gauges on reservoirs and constant head tanks. Nylo-seal tubing, which is translucent, was used for level gauges in pressure applications such as cell pressure accumulators, de-airing tank and pumping vessel. Nylo-seal tubing was also used for all piping for the triaxial cells including nitrogen distribution, level gauges and volume change gauges. This tubing is manufactured to close tolerances, has a high pressure rating and is very stiff. Gould brass fittings were used in all high pressure gas applications. Gould needle, ball and check valves were also used.

A variety of ball valves were used in the triaxial apparatus depending on pressure, availability and price. Those manufactured by Klinger were used for the low pressure triaxial cells. Low pressure ball valves manufactured by Legris were used for the volume change apparatus and high pressure valves from the same source were used on the high pressure triaxial cells.

C.10 Adaptors for LVDT's on Proving Ring (11.2)

PLATE C.8
LVDT's MOUNTED ON PROVING RING

The adaptors designed for mounting linearly variable displacement transducers (LVDT's) in parallel with dial deflection gauges were referred to in section 11.2. Plate C.8 illustrates two LVDT's mounted using these brackets. The brackets were produced from flat aluminium bar with a hole bored to accommodate the transducer and a slot to provide the clearance for clamping. Two tapped holes at the back of the clamp allowed a variety of brackets to be attatched for mounting purposes.

C.11 Vertical Load Control for Direct Shear Tests (12.2.1)

The nitrogen - oil accumulator referred to in section 12.2.1 is illustrated in plate C.9 connected to the vertical load jack of the direct shear apparatus. Nitrogen from the cylinder at bottom right was tapped through a needle valve connected directly to the cylinder until the required pressure was achieved in the accumulator. Vertical load was applied in stages and a number of fine adjustments of the accumulator pressure were required as the nitrogen readjusted to ambient temperature. The accumulator was vented through a needle valve on the side to release vertical load.

PLATE C.9

VERTICAL LOAD CONTROL FOR DIRECT SHEAR TESTS

The accumulator had been previously used in a similar application but was cleaned and pressure tested before use. It was machined from steel bar with an ID of 142 and OD of 178 mm with a steel plug welded into the top. Plug thickness was 39 mm top and 42 mm bottom. The accumulator was pressure tested to 41 MPa and used at a maximum working pressure of 7 MPa. All connections were made using hydraulic fittings rated at 70 MPa.

In use, the accumulator was filled with sufficient oil to lift the jack to the end of its stroke leaving a few millimetres in the bottom of the vessel. This maximised the gas volume and provided the necessary cushion to accommodate vertical displacement of the loading arm.

C.12 Horizontal Load Rate Control for Direct Shear Tests (12.2.1)

PLATE C.10
HORIZONTAL LOAD RATE CONTROL FOR DIRECT SHEAR TESTS

The horizontal load control system referred to in section 12.2.1 is illustrated in plate C.10. The electric motor on the left drives the 900:1 reduction gearbox through a vee belt and cone pulleys giving three rates of loading. The gearbox drives a conventional pressure regulator through a 200 mm pulley mounted directly on the regulator shaft using a poly-vee belt with four vees. This belt is tensioned by the spring loaded arm at the rear. Once turned on, this device steadily increases the air pressure available to the air-hydro pump illustrated in plate 12.2 thus increasing the horizontal shearing force.

C.13 Curing Chamber for Direct Shear Specimens (12.2.3)

PLATE C.11
CURING CHAMBER FOR DIRECT SHEAR SPECIMENS

The curing chamber for direct shear specimens discussed in section 12.2.3 is illustrated in plate C.11. Bricks placed along the sides of the chamber support the shear boxes to form a duct. The boxes were placed together and a fan heater with a thermostatic control was placed at one end. Air was passed down the duct below the boxes and recirculated above them. Temperature was generally maintained in the range 23 to 27°C.

C.14 Brush Used for Direct Shear Joint Preparation (12.3.4.c)

The wire brush used to produce the joint surface illustrated in plate 12.6.b in chapter 12, is illustrated in plate C.12, together with the electric drill with which it was used. The drill was held with its axis horizontal and parallel to the long axis of the specimen. The brush was moved across the joint surface in overlapping bands with steady pressure applied to maintain the width of the brush footprint.

PLATE C.12
WIRE BRUSH AND DRILL FOR JOINT PREPARATION

C.15 Components of Disk Mould (13.2)

Plate C.13 illustrates the components of the disk mould shown assembled in plate 13.2. The split ring at bottom right is the spacing collar used while filling the mould. A standard oedometer sample ring is to the left. The oedometer ring is inserted into a recess in the bottom half of the mould, at top right, and the top half of the mould is clamped in place. The specimen is then statically compacted and may be left in the sample ring by separating the two halves of the mould or may be removed from the mould using the extrusion plug shown on the left in plate 13.2.

PLATE C.13
COMPONENTS OF DISK MOULD

APPENDIX D

PERMEABILITY AND LEACHING RATE DATA FOR INDIVIDUAL SPECIMENS

This appendix contains the graphs of permeability and calcium leaching rate data for individual test specimens, grouped by set, as referred to in chapter 9.

This data is included as it is not always possible to distinguish individual groups of results in the composite plots presented in graphs 9.1 to 9.21. The graphs include other test data tabulated as part of the legend. This data is included for comparison with the plotted data and to give an indication of the spread of results observed within sets of nominally identical specimens.

The data below the graphs is discussed in section 9.4.7. The column headings are:-

a. Imc % -- the initial moisture content at the time the specimen was placed in the leaching cell, no particular correlation between this value and initial permeability is noticeable.

b. Int %/d -- leaching rate in percent per day of initial dry mass, calculated in terms of change in oven dry mass of the specimen at the end of leaching relative to the estimated oven dry mass of the specimen on compaction. This value does not allow for hydration and as such will tend to underestimate the loss. It should be noted that a negative value in this column indicates a loss of material whereas the leaching rate, by definition, represents a loss when positive. This value is generally of similar magnitude to the calcium leaching rate data, note that in comparing the numerical data with the calcium leaching rate axis, the tabulated data must be multiplied by 1000 as it is expressed in units of %/d whereas the axis is in %/d x 10^{-3}.

c. **Ctl %/d** -- leaching rate in percent per day of control dry mass, calculated in terms of change in oven dry mass of the specimen at the end of leaching relative to the mass calculated from the initial mass adjusted by the amount indicated by the hydration calibration discussed in section 9.4.8. This value allows for the gain in final mass as a result of hydration and is the "Adjusted Loss" used throughout chapter 9. As the hydration correction is calculated in terms of control specimens, it assumes that all partially and fully hydrated constituents of the cement are leached in a constant proportion. It is unlikely that this is so and in comparing this value with the calcium leaching rate, it should be noted that this value may tend to overestimate the average calcium loss. The observations in paragraph b, above, with regard to sign and scale also apply to this value.

d. **Pen mm** -- the penetration at the end of leaching determined using the bitumen penetrometer.

e. **Flow l** -- the cumulative flow through the specimen over the duration of the test, expressed in litres.

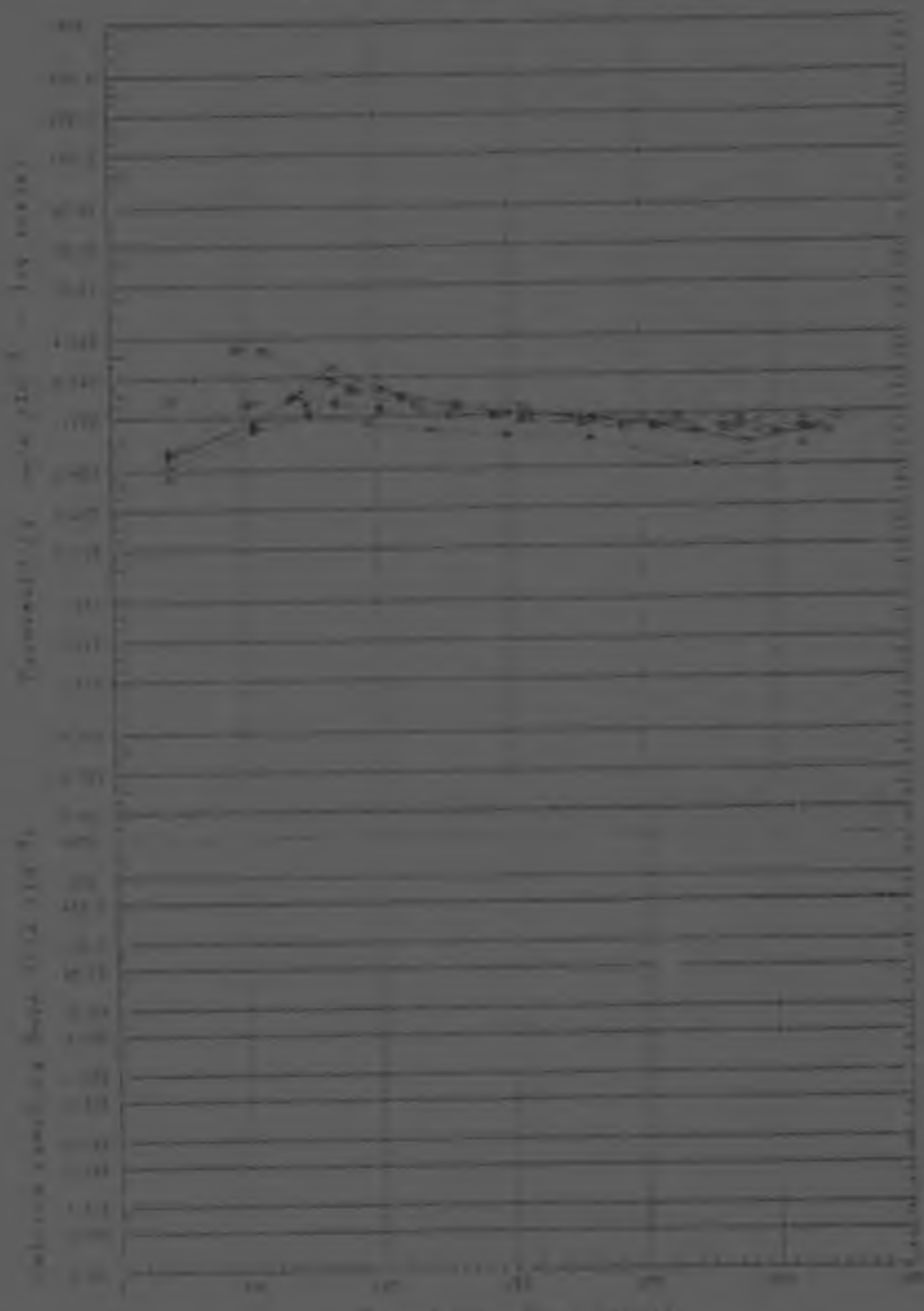
In examining these results, it should be noted that the average values in the legend do not always correspond exactly with the values in table 9.3. The graphs in the appendix include data for all specimens including those where notes during testing indicated possible material loss or other grounds for ignoring certain data points. These suspect data points were rejected in calculating the summary data used in plotting graphs 9.22 to 9.34 and tabulated in table 9.3.

The principle of default titling has been applied to the graphs in this appendix with only the non-standard conditions for a particular set referred to in the graph title. The standard conditions apply to all the leaching tests conducted as part of this research. Generally only one of the parameters was varied for a particular set of specimens. The standard conditions are listed in table D.1

TABLE D.1

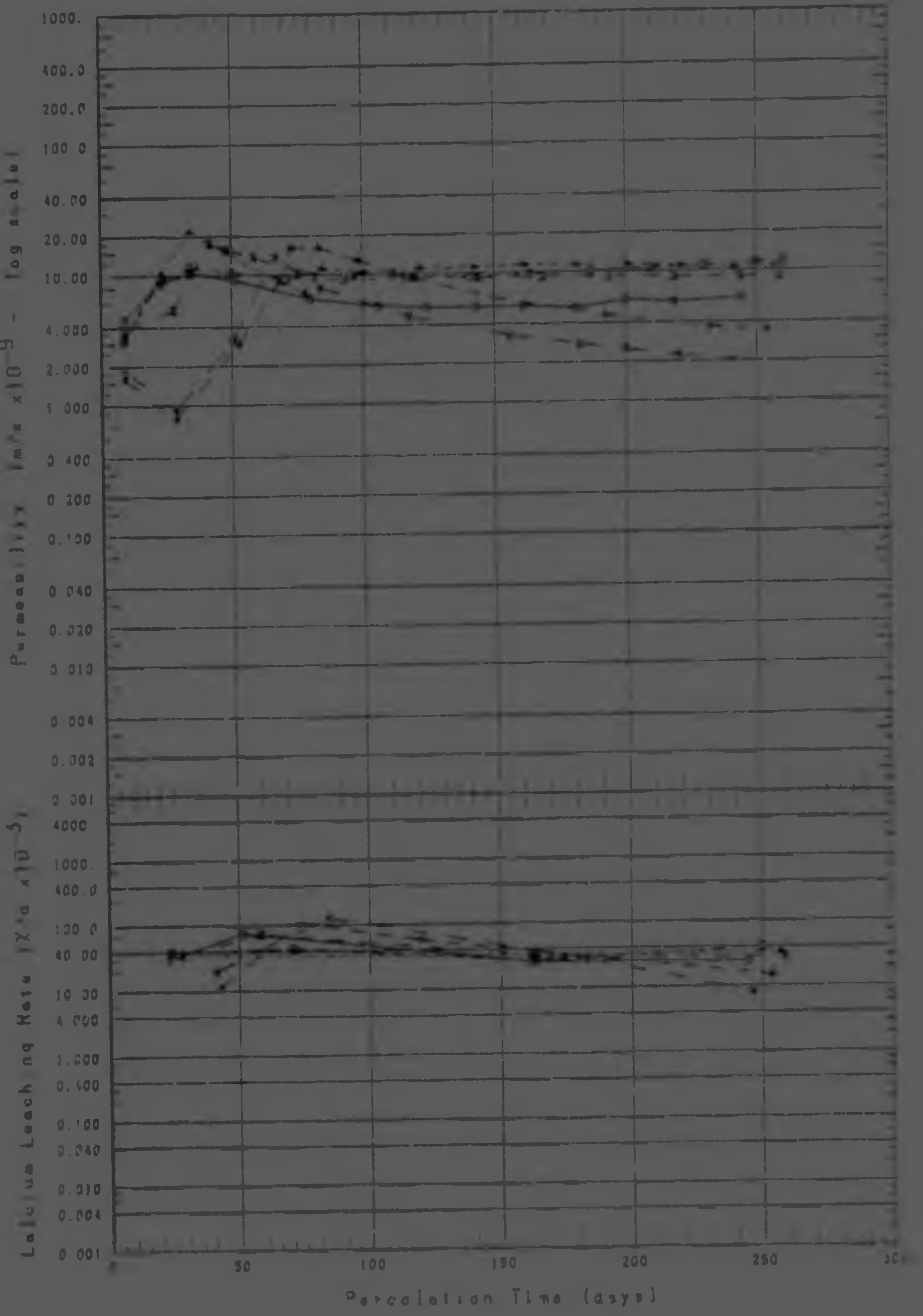
STANDARD CONDITIONS FOR LEACHING TEST PROGRAM

Item	Standard Conditions
Specimen	Cylinder 37.5 [illegible] diameter and 75 mm long.
Material	Soil-[illegible]nt c[illegible]mpacted to Standard Proctor densi[illegible] 1905 kg/m^3) and moisture content (13.[illegible]
Cement	10% Cement by dry mass of soil, secondary standards, 5, 15 and 20%.
Leaching Water	De-aired distilled water.
Gradient	50 m/m along axis of specimen i.e. head of 3.75 m.
Cell Pressure	Nominal head of 10.2 m on middle tier, variation +/- 0.5 m for top and bottom tiers.
Period	Nominally 266 days (38 weeks).
Cure	7 days humid cure followed by 21 days sealed in foil and wax.



GRAPH D.2

STANDARD CYLINDERS, 5% CEMENT

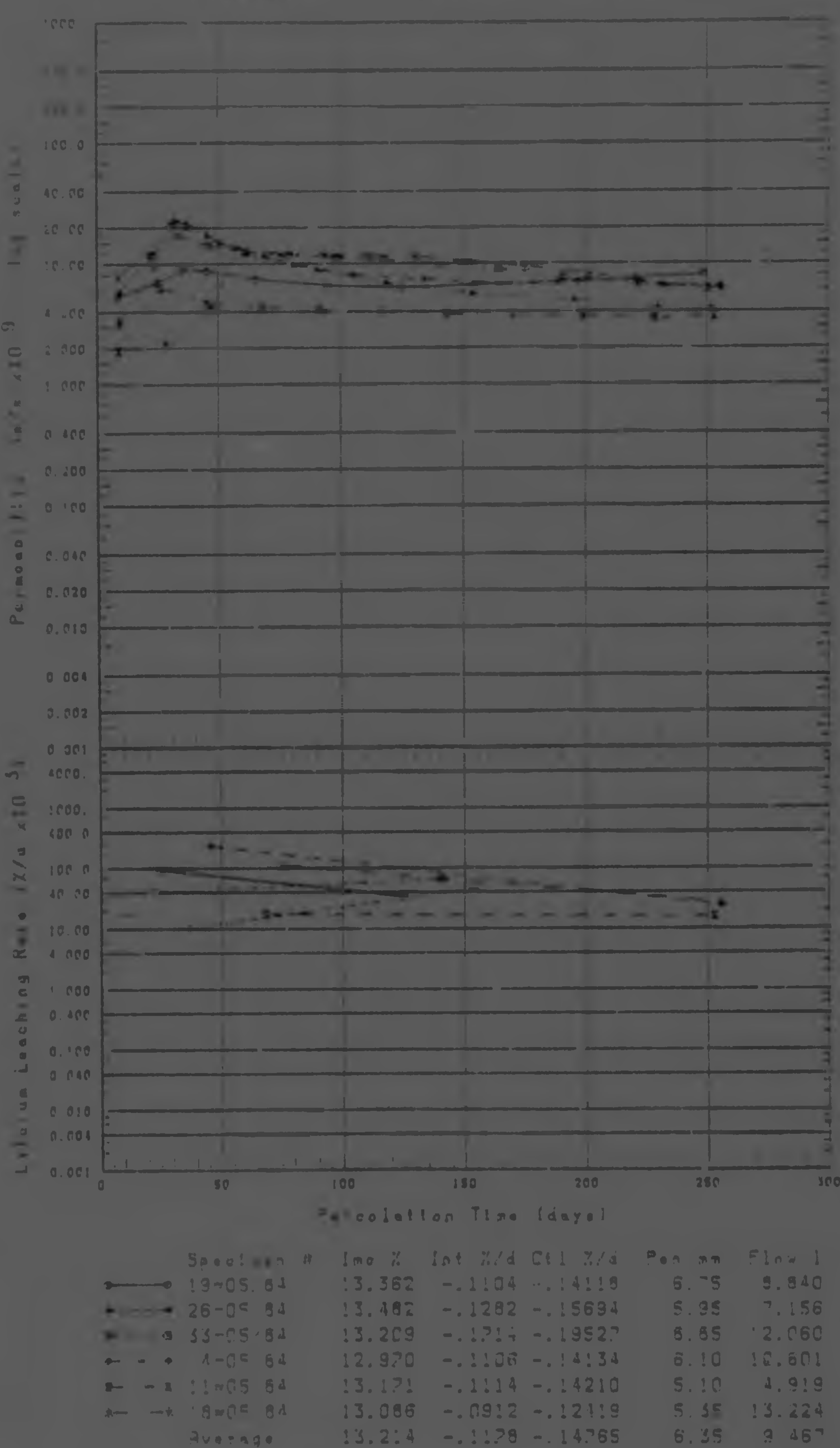


	Specimen #	Inc %	Int %/d	Ctl %/d	Pen mm	Flow l
	2=05/84	13.194	-.0956	-.12808	4.05	7.914
	8=05/84	13.341	-.1262	-.15523	8.35	11.596
	15=05/84	13.193	-.1388	-.16640	4.30	11.387
	22=05/84	13.439	-.0734	-.10835	8.15	7.334
	29=05/84	13.476	-.1090	-.13993	6.45	11.598
	36=05/84	13.212	-.0607	-.09706	5.05	4.560
	Average	13.309	-.1005	-.13230	5.76	9.165

STD-05.PLT -- [illegible] 1.1 FLCH-CONF DAT. 3

[illegible] D.3

[illegible] CYLINDERS, [illegible] CEMEN[illegible] SECOND SET.

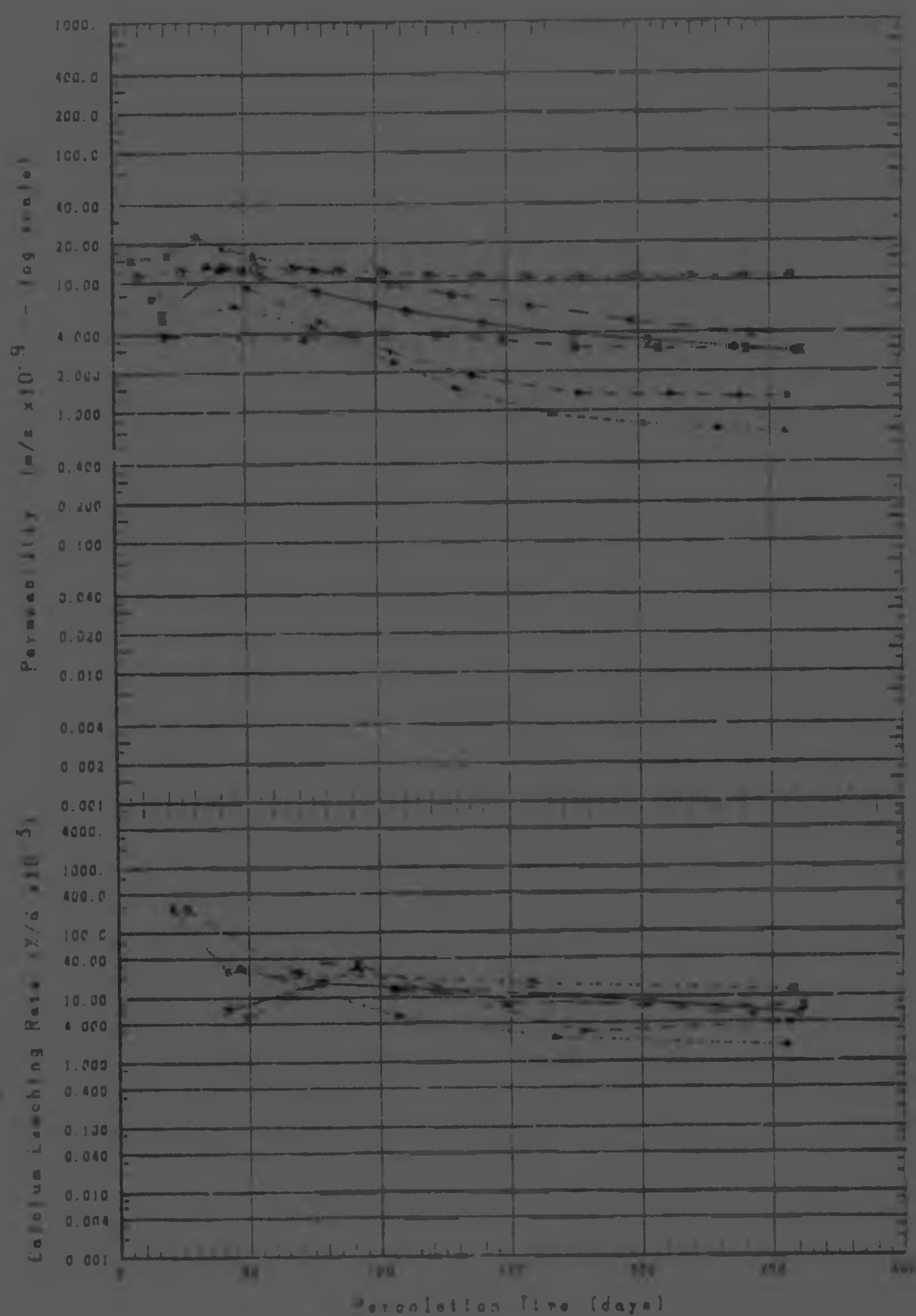


	Specimen #	Imo %	Int %/d	Ctl %/d	Pen mm	Flow l
o——o	19-05.84	13.362	-.1104	-.14118	6.75	8.840
•——•	26-05.84	13.482	-.1282	-.15694	6.35	7.156
■——■	33-05.84	13.209	-.1214	-.19527	6.85	12.060
◆– – ◆	4-05.84	12.970	-.1106	-.14134	6.10	10.601
▲– – ▲	11-05.84	13.171	-.1114	-.14210	5.10	4.919
— —	18-05.84	13.086	-.0912	-.12119	5.35	13.224
	Average	13.214	-.1128	-.14765	6.35	9.467

CM-CONF.DAT, 2]

GRAPH D.4

STANDARD CYLINDERS, 10% CEMENT

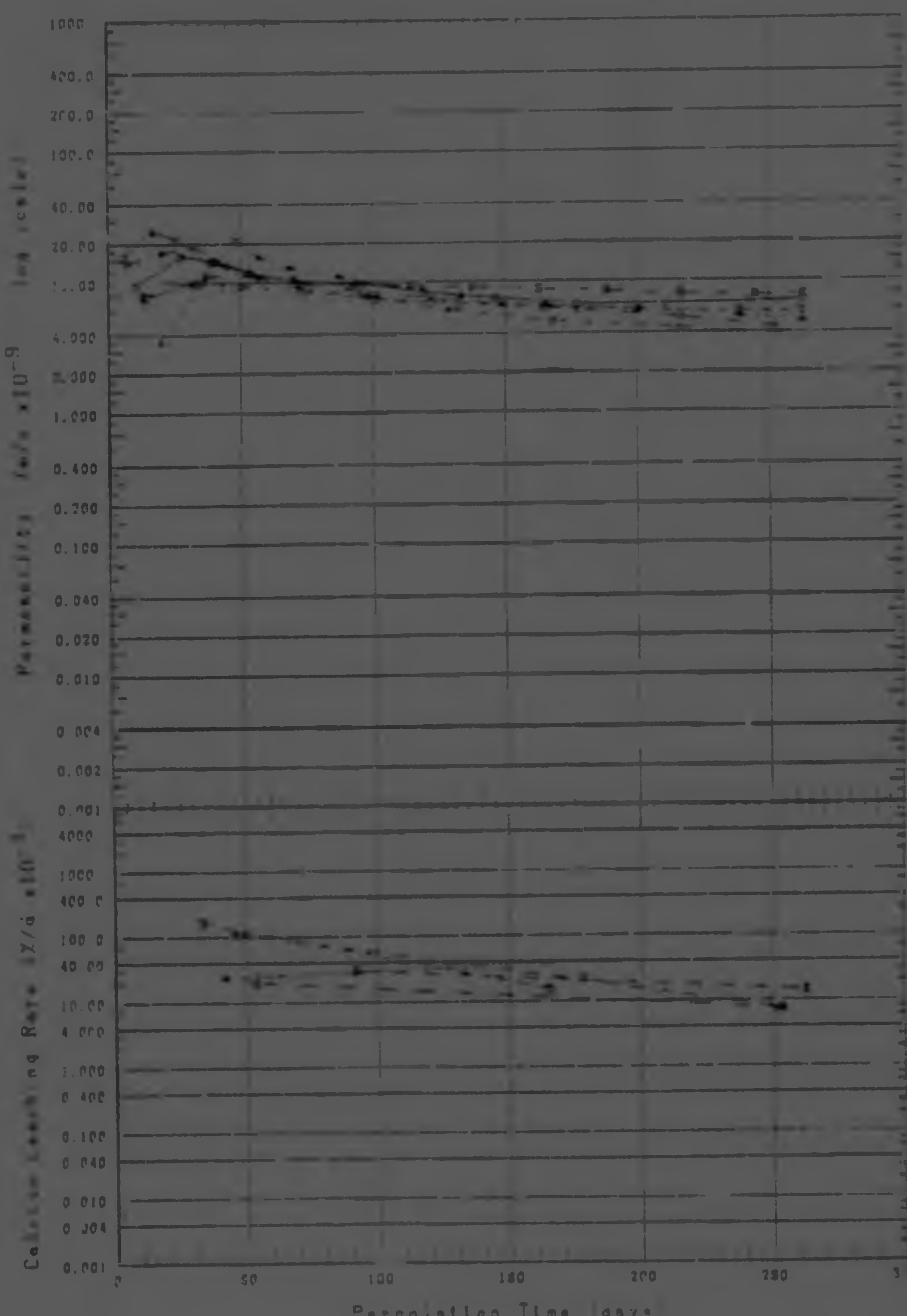


Specimen #	Imc %	Int %/d	Ctl %/d	Pen mm	Flow l
1=10/63	13.317	-.0518	-.09053	4.00	7.071
6=10/63	13.062	-.0135	-.05679	3.15	3.229
15=10/63	13.176	-.0817	-.11686	7.80	14.571
22=10.63	13.040	-.0376	-.07822	4.10	4.844
29=10/63	13.160	-.0140	-.05722	3.55	4.41_
36=10/63	13.218	-.0794	-.11465	7.00	11.875
Average	13.162	-.0464	-.08575	4.90	7.662

STD-10.PLT -- 13/03/84-9=1197-- [illegible]

GRAPH 7.5

STANDARD CYLINDERS, 10% CEMENT [illegible]

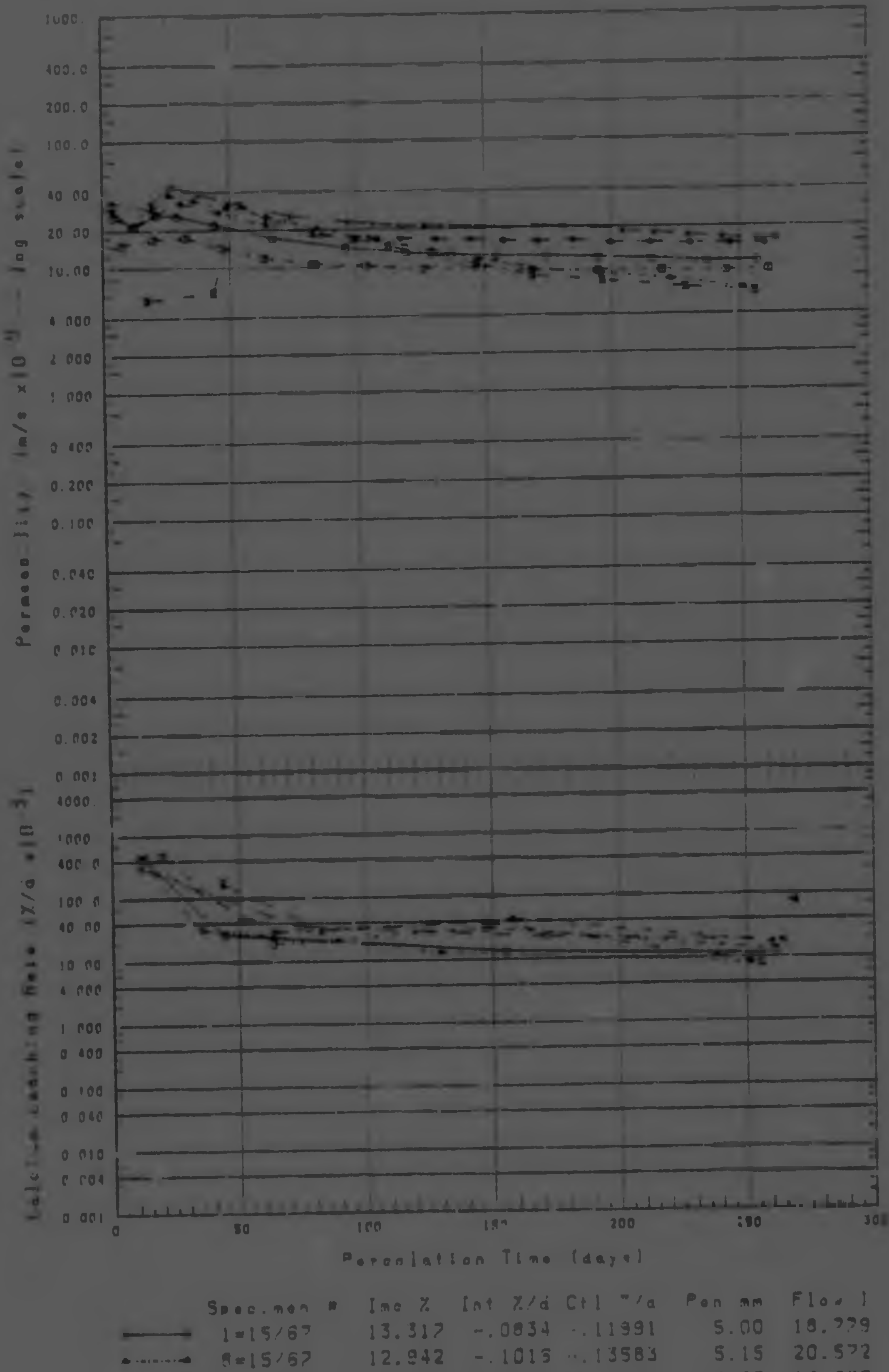


Specimen #	Im %	Int % d	Ctl % d	Pen mm	Flow l
13-10/63	13.337	-.0717	-.10793	2.45	11.201
20-10/63	13.012	-.0717	-.10794	5.25	9.916
27-10/63	13.17.	-.0570	-.09497	3.20	9.097
34-10/63	13.059	-.0821	-.11712	4.30	9.918
5-10/63	13.039	-.0601	-.09777	4.05	11.425
12-10/63	13.014	-.0751	-.110[illegible]	7.25	11.298
Average	13.111	[illegible]0709	-.10734	4.42	10.476

SIG-10 PLT 15/03/94-9=1200 1 'LCH-CONF.DAT', 2 1

GRAPH D.6

STANDARD CYLINDERS, 15% CEMENT

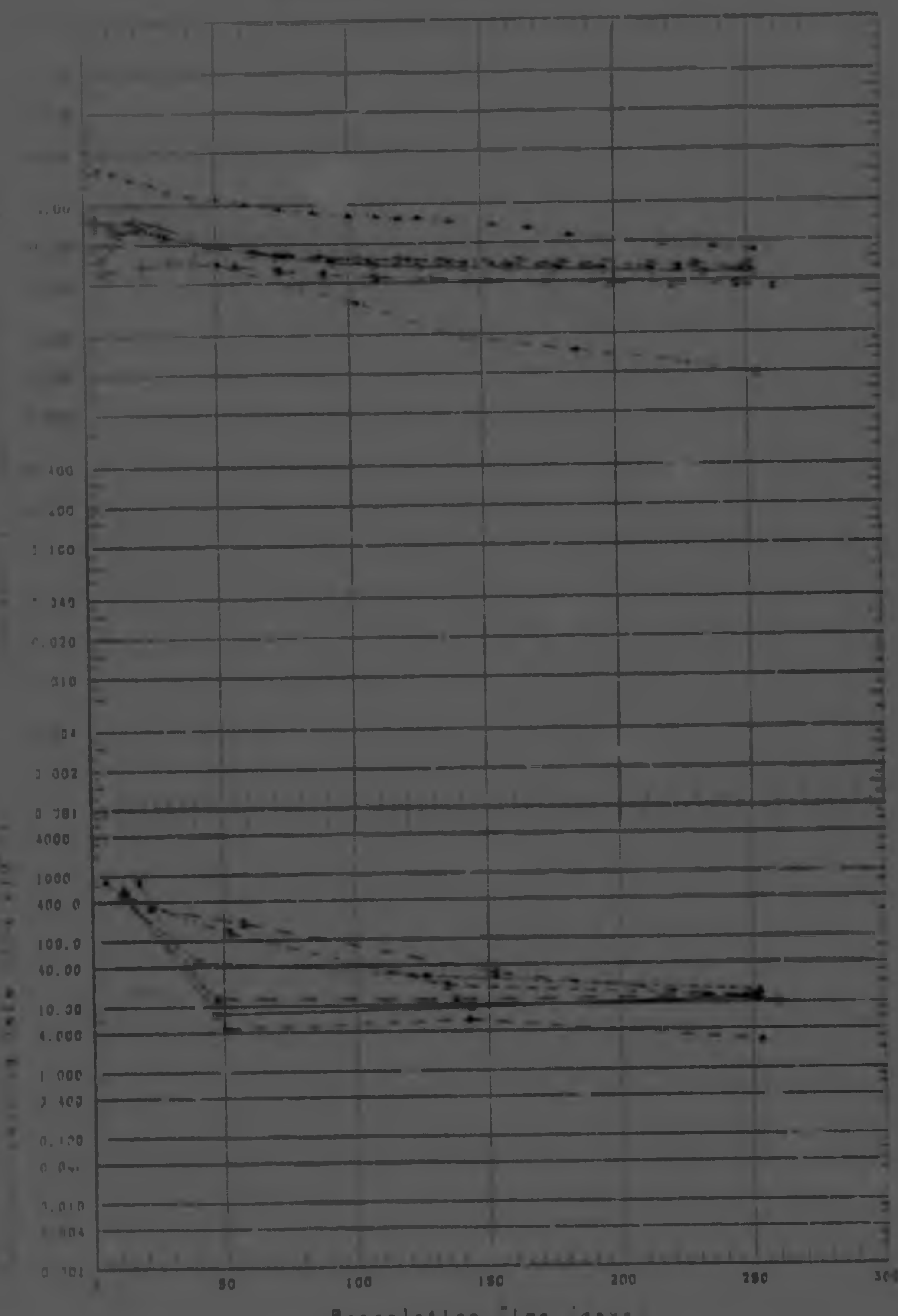


	Specimen #	Imc %	Int %/d	Ctl "/a	Pen mm	Flow l
	1=15/67	13.317	-.0834	-.11991	5.00	18.779
	8=15/67	12.942	-.1015	-.13583	5.15	20.572
	15=15/67	12.439	-.0821	-.11679	6.05	13.835
	22=15/67	13.152	-.0964	-.13128	5.35	23.711
	5=15/67	13.111	-.0596	-.09905	5.25	13.317
	12=15/67	12.940	-.1165	-.14891	6.30	29.185
	Average	12.995	-.0899	-.12556	5.52	19.950

STC 15 PLT 5/03/84-9=1200 v 1 0 (LCM-CONF DAT 2

GRAPH D.7

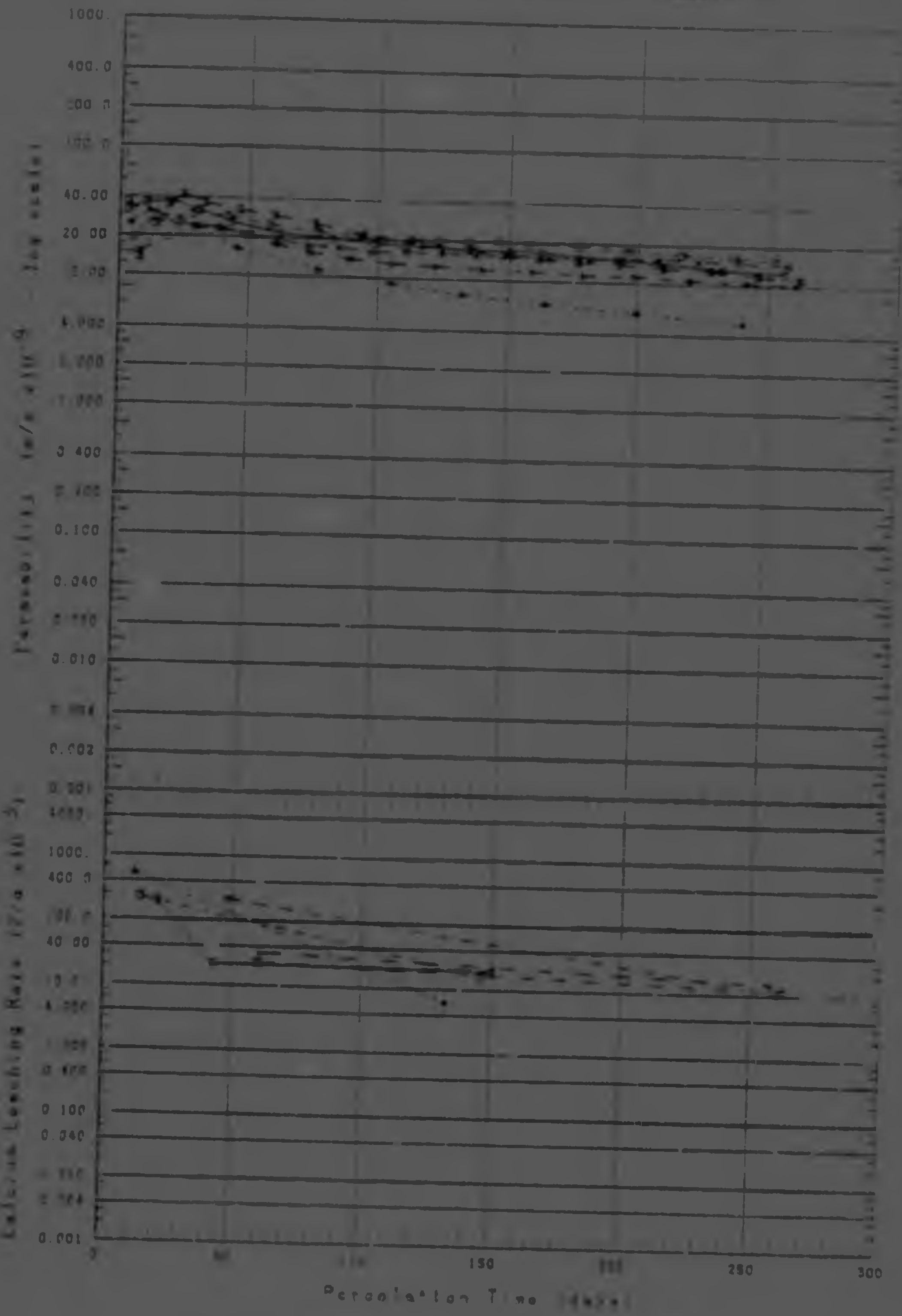
STANDARD CYLINDERS, 20% CEMENT



	Specimen #	Tie %	Int %/a	Ctl %/a	Pen mm	Flow l
	1=20/88	13.213	-.0837	-.12230	3.20	19.129
	8=20/88	12.795	-.1428	-.17373	7.10	41.629
	15=20/88	12.791	-.0908	-.1284[illegible]	3.05	20.403
	22=20/88	12.671	-.0554	-.09771	5.80	9.769
	29=20/88	12.777	-.0603	-.10[illegible]	5.05	13.967
	6=20/88	13.179	-.0857	-.1[illegible]	3.80	20.825
	Average	12.904	-.0864	-.12462	4.67	20.954

GRAPH D.8

STANDARD CYLINDERS, 20% CEMEN

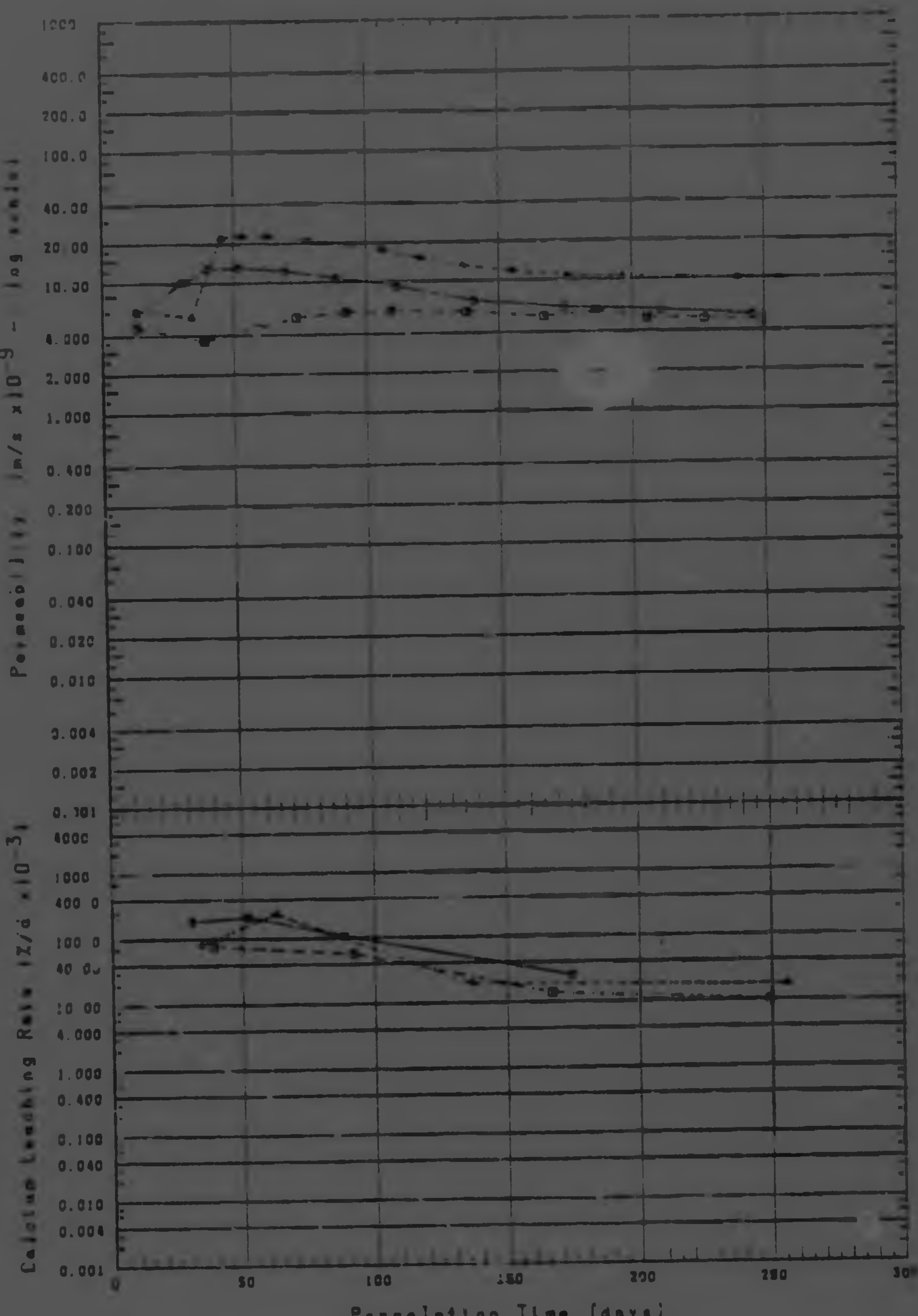


Specimen #	Ime %	Int %/d	Ctl %/d	Pen mm	Flow l
13=20/88	12.879	-.0998	-.13634	3.45	24.431
27=20/88	12.790	-.0724	-.11244	4.85	13.408
11=20/88	12.853	-.1024	-.13859	3.65	16.415
13=20/53	12.827	-.0827	-.12019	4.90	17.674
3=20/53	12.857	-.1013	-.13636	5.50	24.961
24=20/53	13.101	-.0917	-.12799	4.35	22.301
Average	12.884	-.0916	.01284	4.62	[illegible]

.16/03/84-8=1201 1 0 .CM-CONF DAT.

GRAPH D.3

28 DAY HUMID CURE, 10% CEMENT

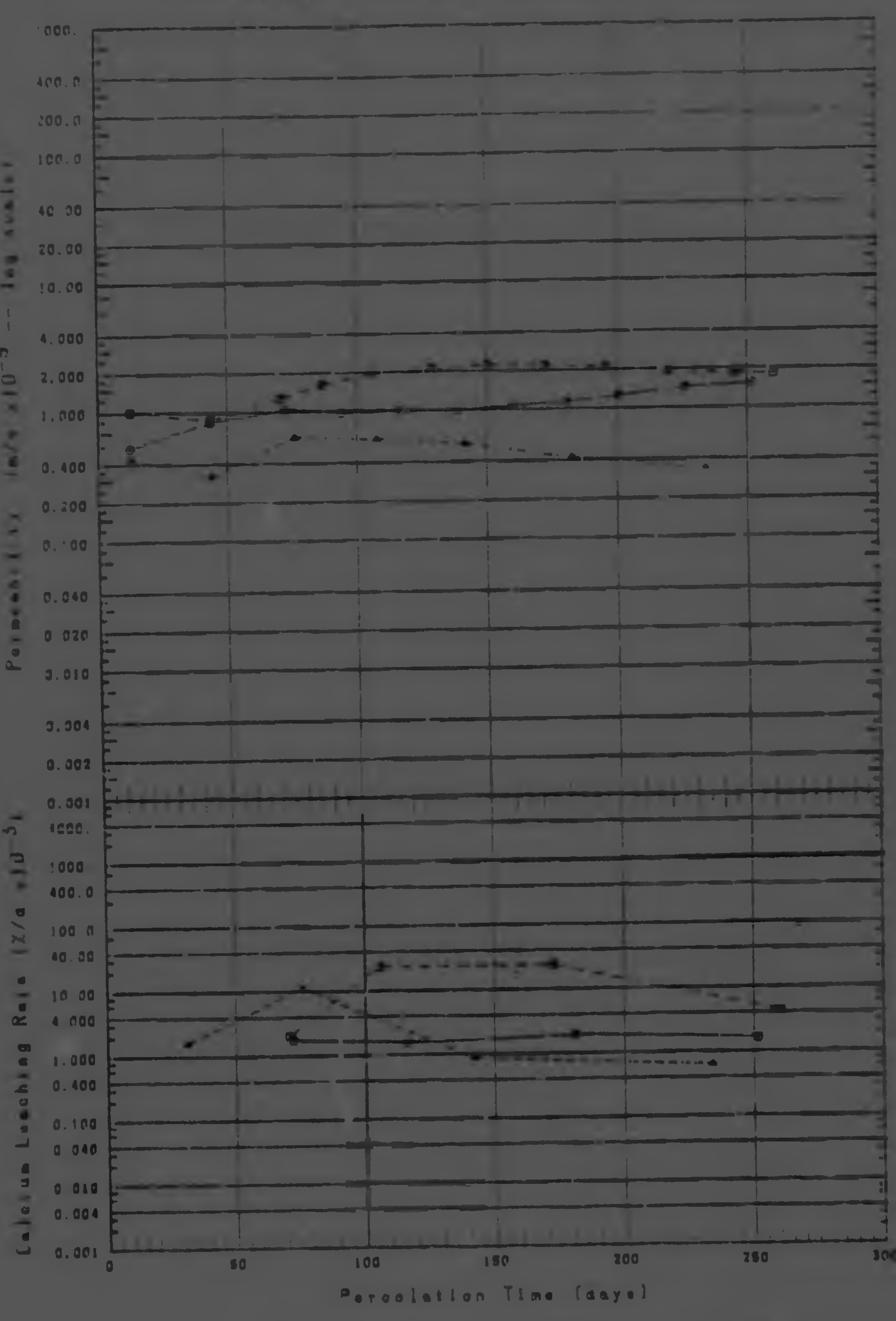


	Specimen #	Imc %	Int %/d	Ctl %/d	Pen mm	Flow l
	1=10/98	12.275	-.0081	-.05286	6.20	9.828
	15=10/98	12.117	-.0855	-.12101	6.60	16.124
	11=10/98	12.175	-.0623	-.10061	4.25	6.374
	Average	12.189	-.0519	-.09142	5.68	10.775

H4L-10.PLT — 16/03/84-A=120]-v 1.0 [LCM-COMP DAT. 2

GRAPH D.10

7 DAY HUMID, 21 DAY LEACH BATH CURE, 10% CEMENT

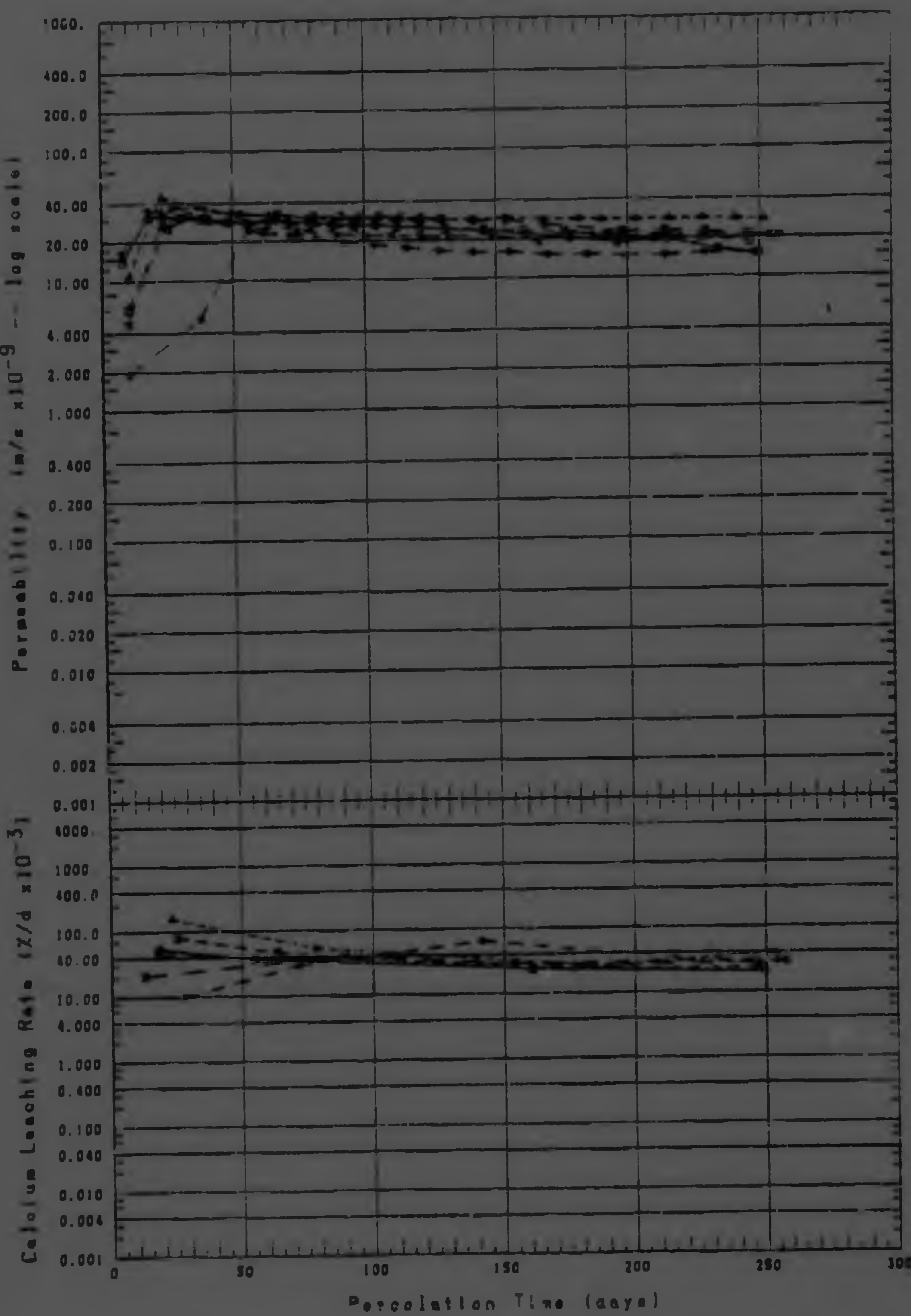


	Specimen #	Imo %	Int %/d	Ctl %/d	Pen **	Flow 1
	8=10/98	15.598	.00275	-.04520	1.45	1.323
	4=10/98	15.405	.01667	-.03094	2.05	.565
	18=10/98	15.504	-.0213	-.06441	.60	2.126
	Average	15.502	-.0007	-.04614	1.37	1.338

GRA10.PLT -- 10/03/84-9=1201-v 1.0 CLCM-COMP.DAT, 2 1

GRAPH D.11

7 DAYS HUMID CURE AND 12 CYCLES WET-DRY, 10% CEMENT

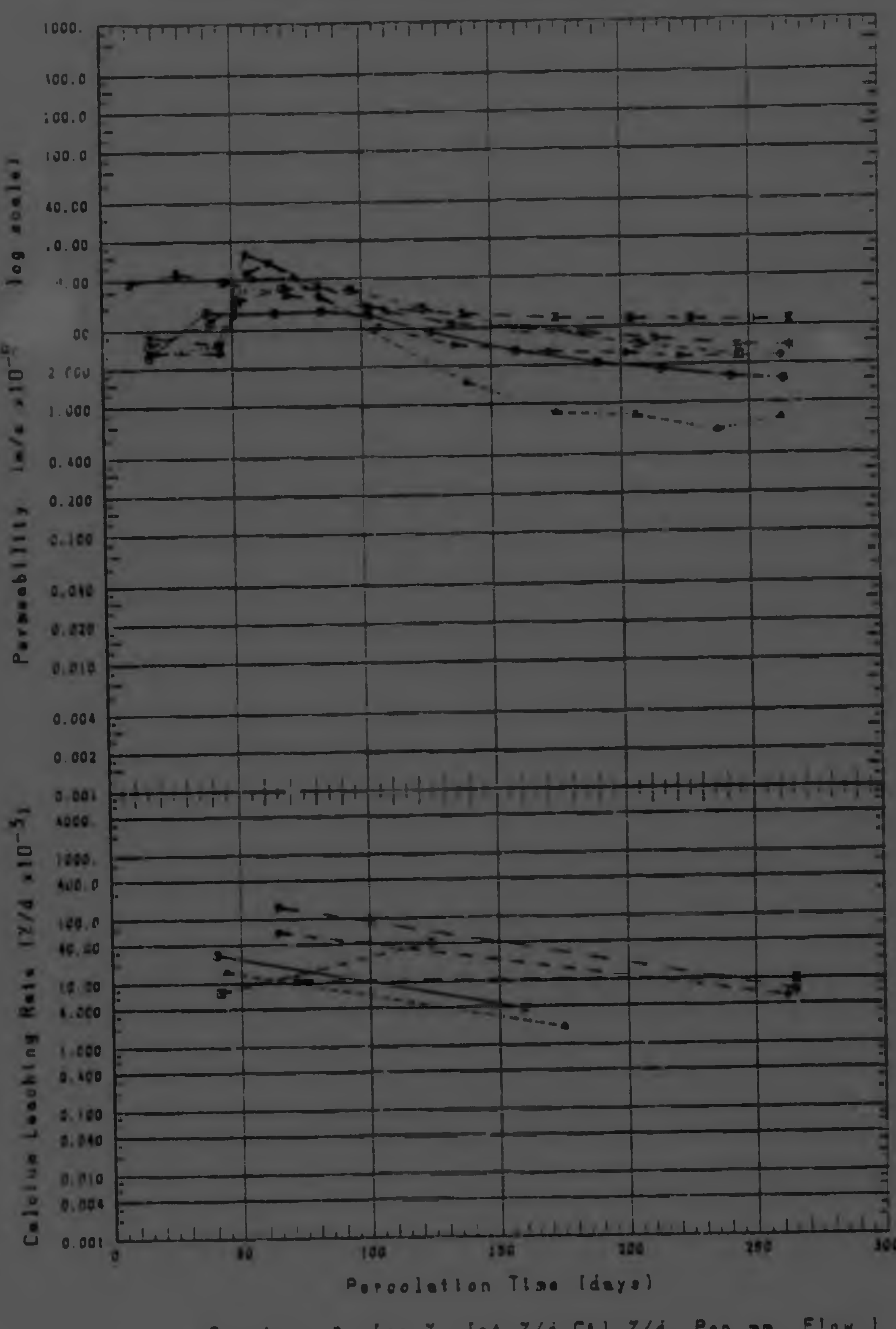


	Specimen #	Imc %	Int %/d	Cil %/d	Pen mm	Flow l
●——●	31=10/100	2.326	-.0367	-.11027	5.25	28.195
———	2=10/100	2.441	-.0515	-.12196	6.55	35.568
■---■	9=10/100	2.763	-.0203	-.09732	4.30	27.923
◆- - -◆	16=10/100	2.262	-.0165	-.09426	4.65	17.595
▲- —▲	23=10/100	2.734	-.0387	-.11187	4.25	33.999
●— —●	30=10/100	2.306	-.0397	-.11263	7.00	26.478
	Average	2.472	-.0335	-.10674	5.37	28.293

MDL-10.PLT — 18/03/88-8=[20]—v 1.0 (LCM-CONF DAT. 2)

GRAPH D.12

7 DAYS HUMID AND 77 DAYS SEALED CURE, 10% CEMENT

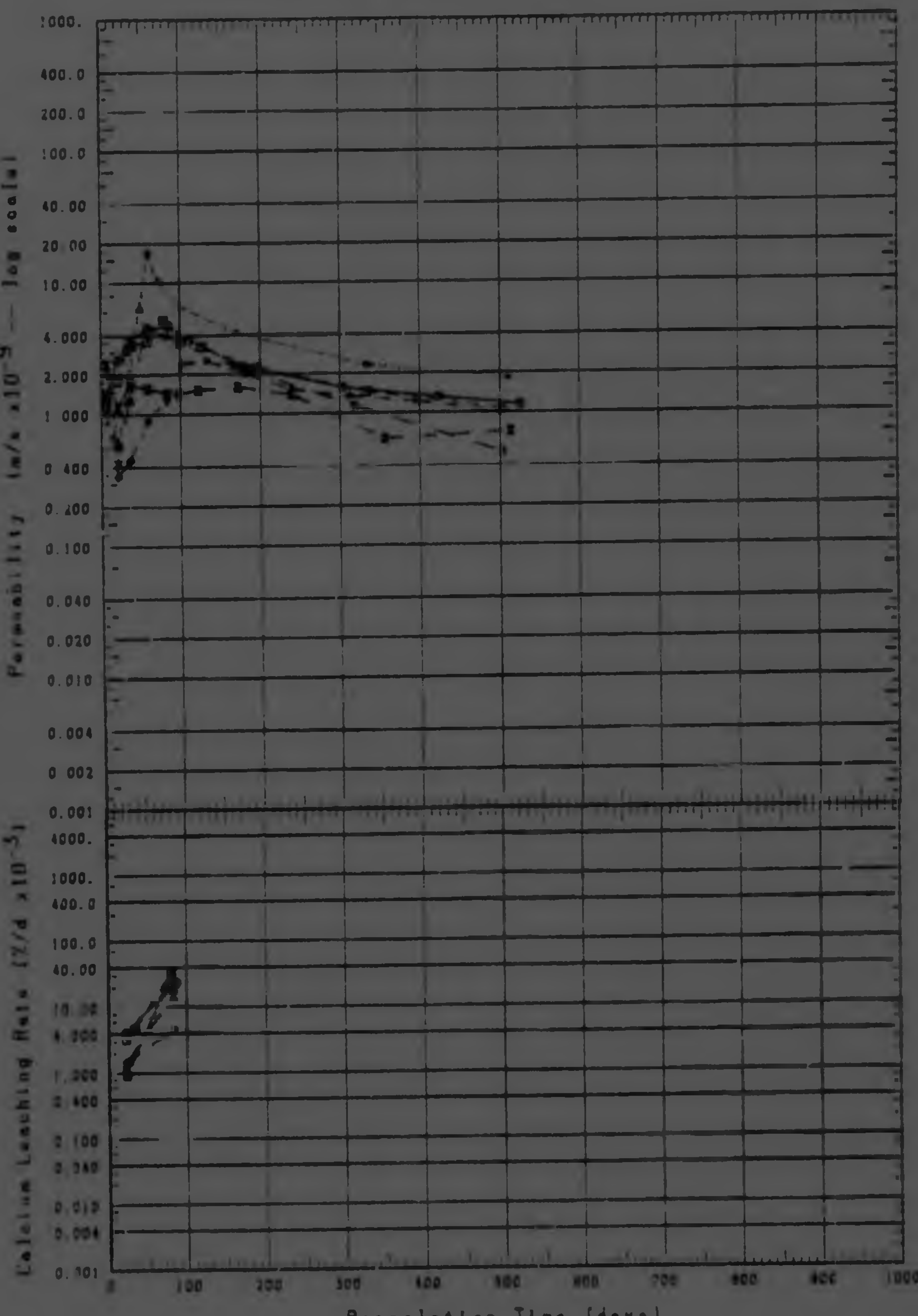


Specimen #	Imc %	Int %/d	Ctl %/d	Pen mm	Flow l
1=10/09	13.012	-.0129	-.05680	3.80	4.130
8=10/09	13.084	-.0018	-.04710	3.35	3.637
15=10/09	13.203	-.0103	-.05459	1.75	5.889
22=10/09	13.683	.00756	-.03886	3.40	4.877
5=10/09	13.032	-.0370	-.07803	3.30	8.048
12=10/09	14.240	.01714	-.03045	3.80	6.237
Average	13.371	-.0063	-.05096	3.23	5.470

911-10.PLT — 10/03/84-8-1201-v 1.0 (LCM-CONF DAT. 2)

GRAPH D.13

7 DAYS HUMID AND 343 DAYS SEALED CURE, 10% CEMENT

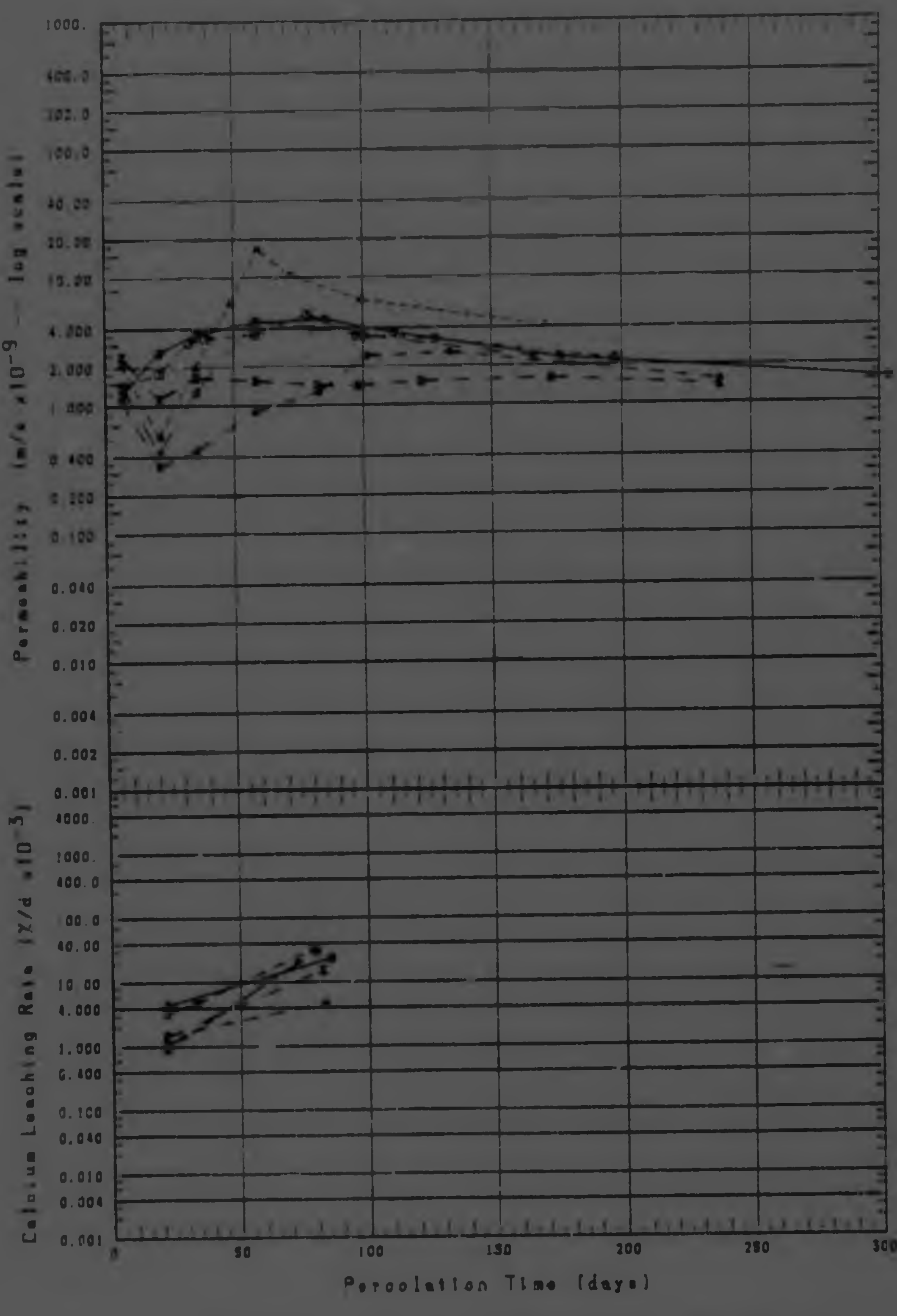


	Specimen #	Imo %	Int %/d	Ctl %/d	Pen mm	Flow 1
	13=10/08	13.164	.00221	-.02102	1.78	5.672
	2=10/08	12.994	-.0127	-.03424	4.00	8.988
	9=10/08	12.646	-.0013	-.02406	1.51	5.331
	18=10/08	13.040	.00947	-.01584	1.79	3.696
	5=10/08	13.391	.01118	-.01325	1.09	2.994
	12=10/08	13.032	.00785	-.01614	1.75	4.259
	Average	13.133	.0042	-.01925	1.99	5.157

948-10.PLT -- 18/03/84-8=1202-v 1.0 (LCH-COMP DAT. 4)

GRAPH D.13a

7 DAYS HUMID AND 343 DAYS SEALED CURE, 10% CEMENT

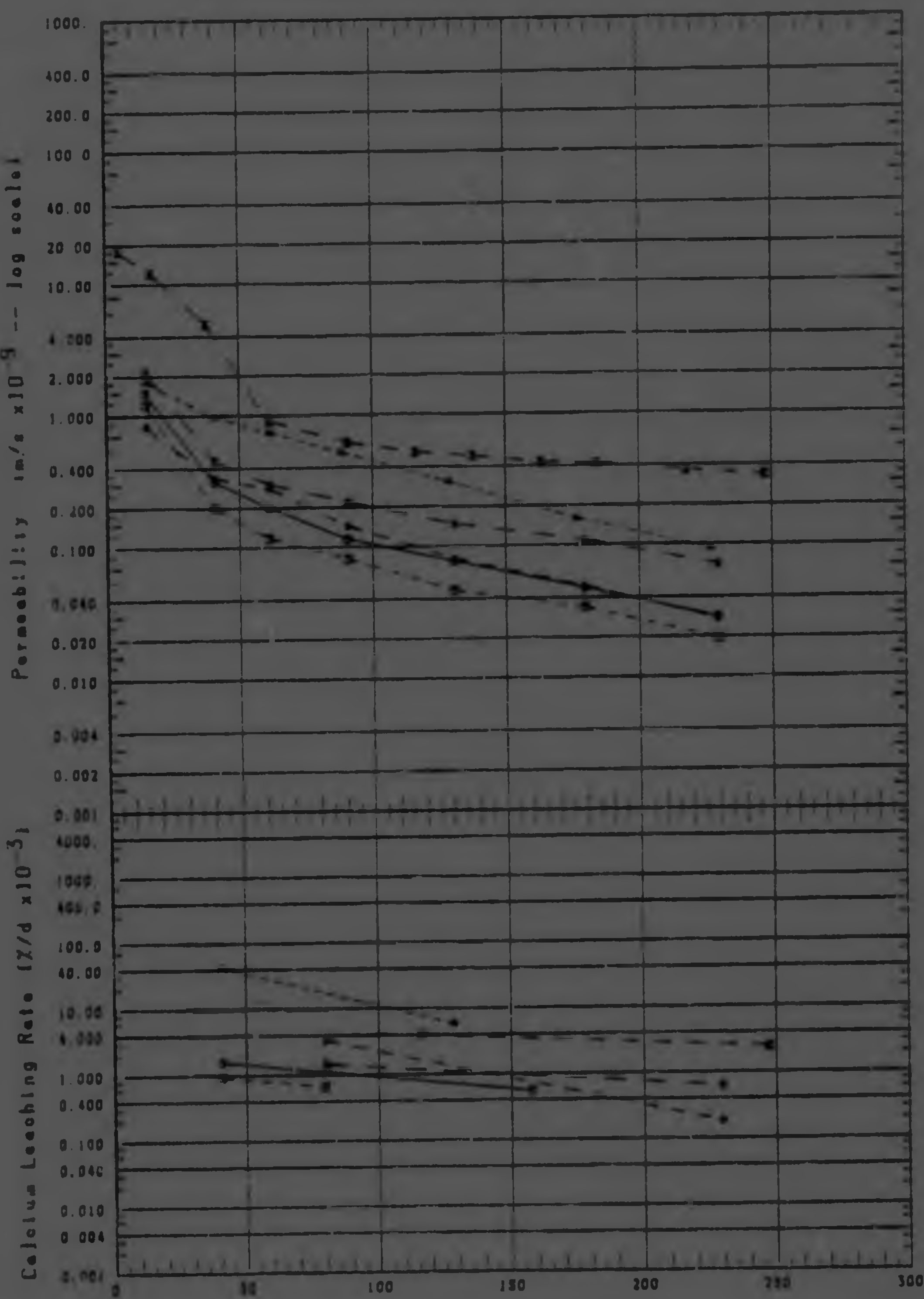


	Specimen #	Imc %	Int %/d	Ctl %/d	Pen mm	Flow l
	13=10/08	13.184	.00221	-.02102	1.78	5.672
	2=10/08	12.994	-.0127	-.03424	4.00	8.988
	9=10/08	12.646	-.0013	-.02406	1.51	5.331
	16=10/08	13.040	.00947	-.01584	1.79	3.696
	5=10/08	13.391	.01118	-.01325	1.09	2.994
	12=10/08	13.032	.00765	-.01614	1.75	4.259
	Average	13.133	.0042	-.01925	1.99	5.157

948-10.PLT -- 18/03/86-8-1202-v 1.0 (LCM-CONF DAT. 2 1

GRAPH D.14

HIGH HYDRAULIC GRADIENT (136 m/m), 10% CEMENT

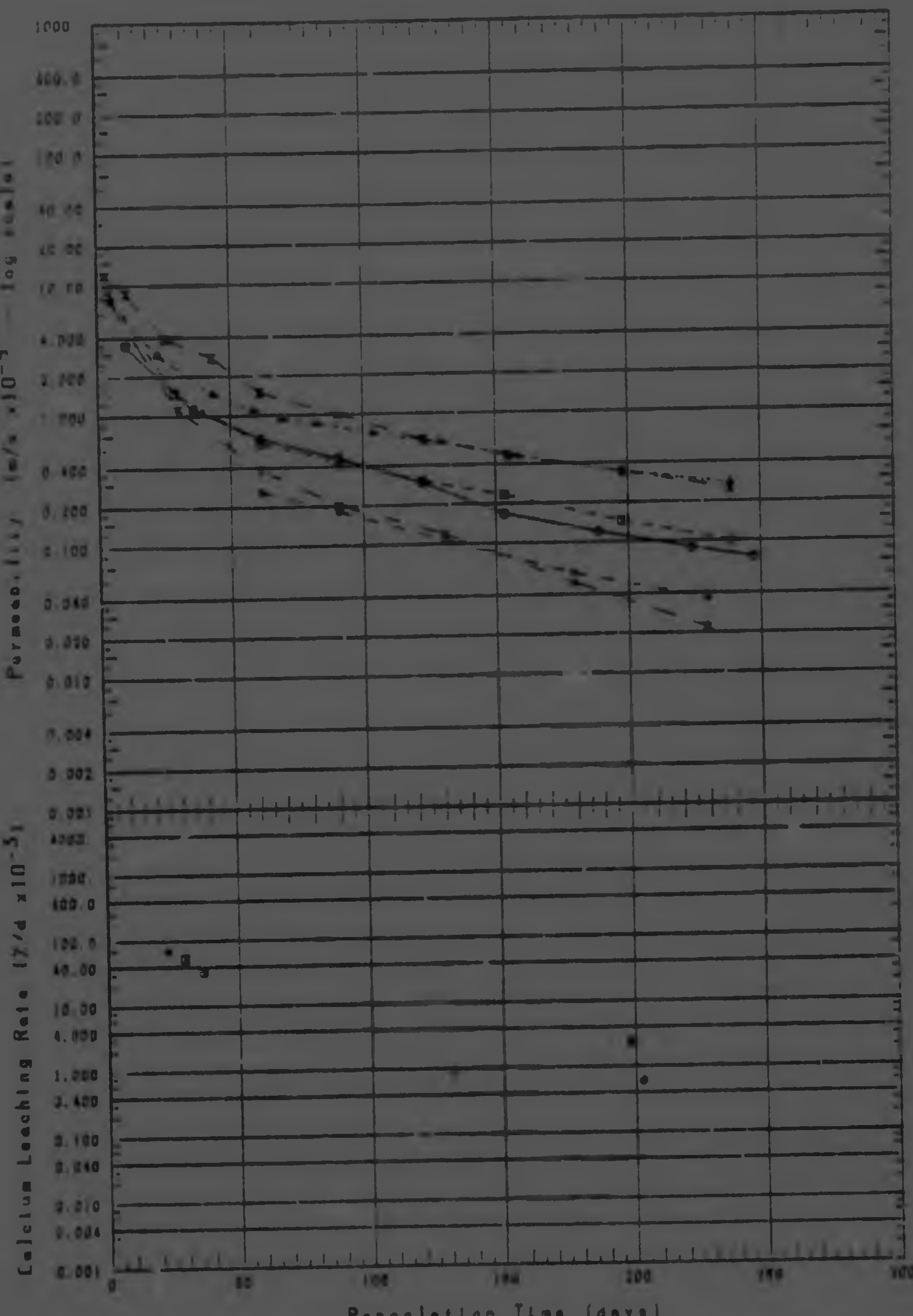


	Specimen #	Imc %	Int %/d	Ctl %/a	Pen mm	Flow l
	1=10/51	13.234	.03531	-.01591	.20	.863
	8=10/51	13.110	.01170	-.03672	1.43	1.752
	15=10/51	13.123	.02865	-.02176	.00	.653
	22=10/51	13.247	.03556	-.01568	.00	.654
	5=10/51	12.904	-.0732	-.11153	4.79	7.575
	12=10/51	12.876	.00952	.03864	1.41	1.350
	Average	13.083	.0079	-.04004	1.30	2.141

HGR-10.PLT — 21/03/88-8=1204-v 1_0 (LCH-CONF DAT. 2)

GRAPH D.15

HIGH HYDRAULIC GRADIENT, 10% CEMENT (SECOND SET)

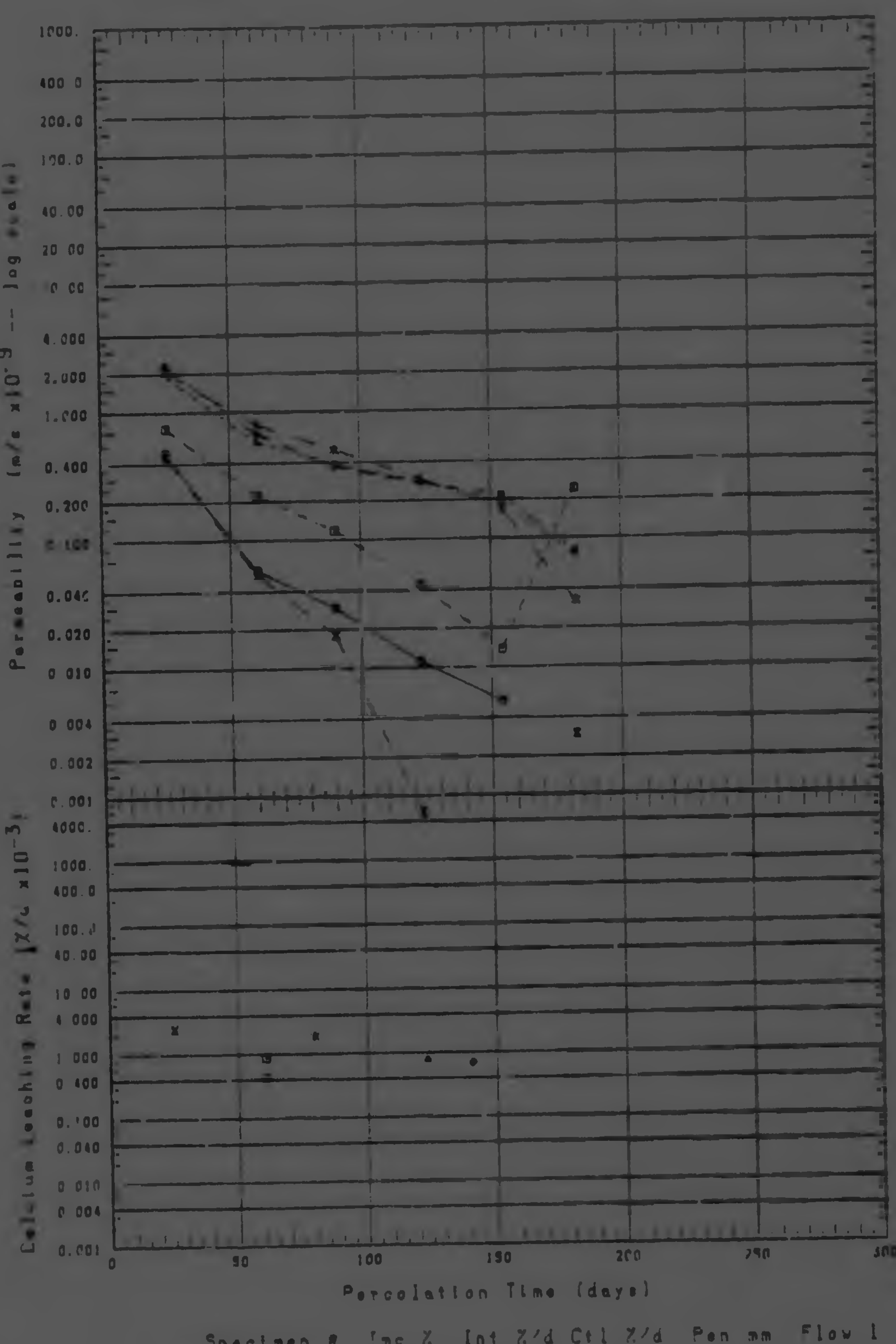


Specimen #	Imc %	Int %/d	Ctl %/d	Per m	Flow l
7=10/51	13.209	.00242	-.04489	1. [illegible]	.085
14=10/51	13.137	-.0146	-.05996	3.	.215
21=10/51	13.269	.00740	-.04061	2.00	.506
4=10/51	13.114	.02622	-.02403	.30	.282
11=10/51	-2.255	-.6153	-.58911	4.59	5.252
18=10/51	13.187	.00384	-.04392	.70	1.901
Average	13.164	-.0008	-.04670	2.17	2.624

AJ8-15.PLT — 11/03/88-4=1208-+ 1.0 (LCH-CONF DAT. 2)

GRAPH D.16

STANDARD GRADIENT, 10% CEMENT, AIR PRESSURISED CONTROL

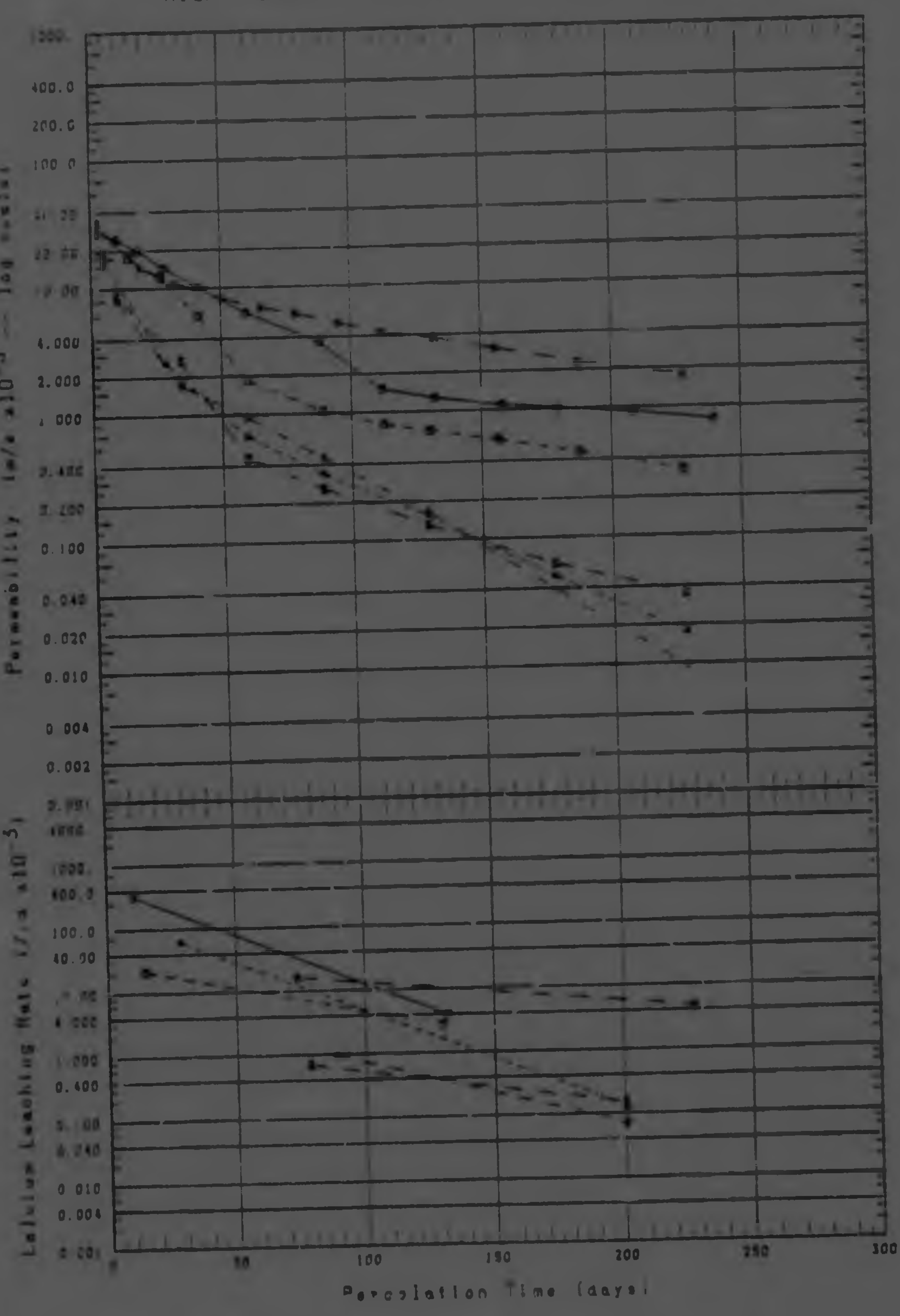


Specimen #	Imc %	Int %/d	Ctl %/d	Pen mm	Flow l
13=10/51	13.248	.06623	-.00329	05	.128
20=10/51	13.291	.04692	-.01854	.00	.677
3=10/51	13.182	.03920	-.02710	.00	.254
10=10/51	13.101	.03936	-.02695	.00	.774
17=10/51	13.126	.07048	.000463	.00	.117
24=10/51	13.387	.05246	-.01542	00	.742
Average	13.222	.0429	-.00920	0.00	.449

BEW-10.PLT -- [illegible] 1.0 (LCM-CBN: DAT. 2)

GRAPH D.17

HIGH HYDRAULIC GRADIENT (136 m/m), 20% CEMENT

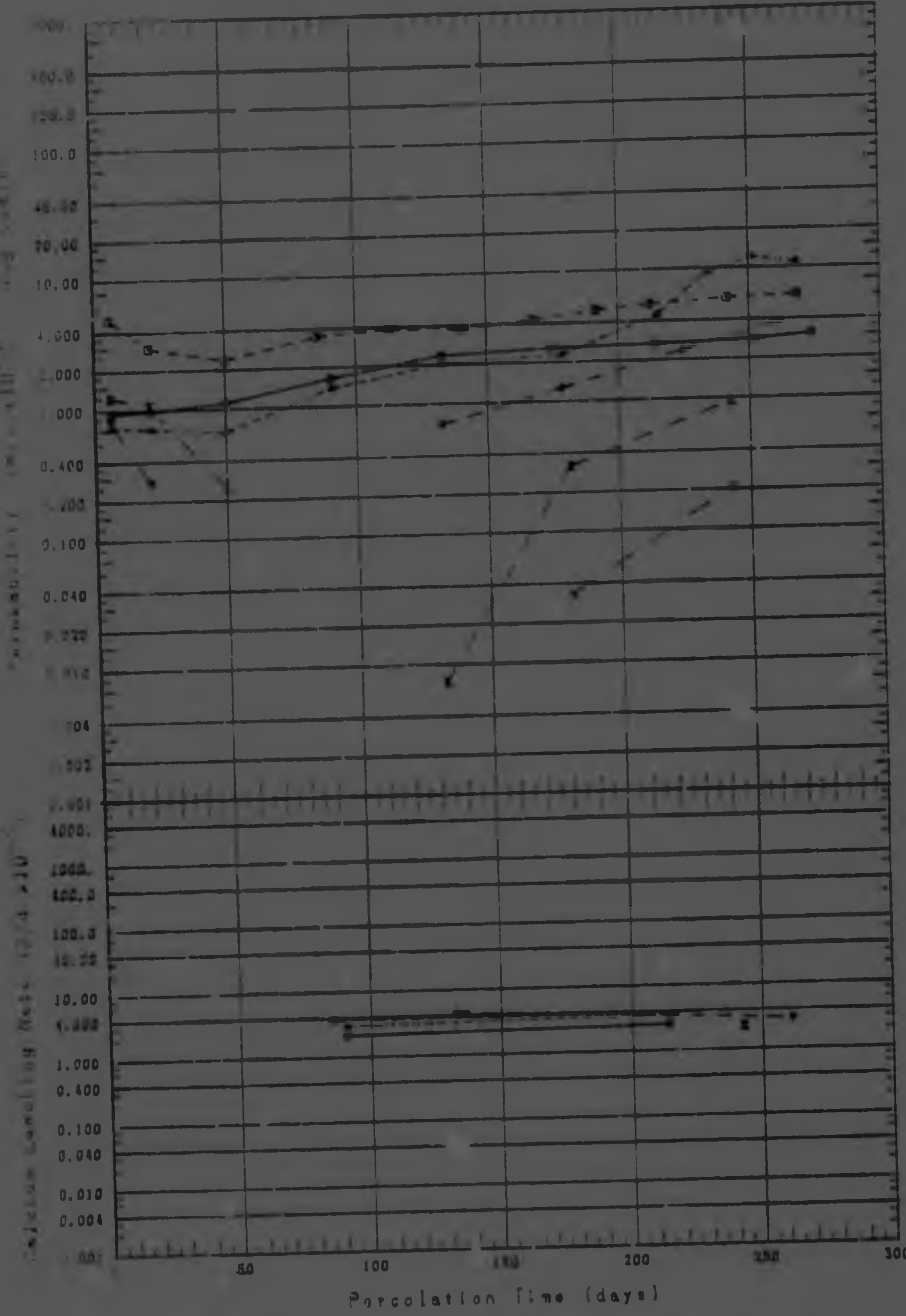


Specimen #	Inc %	Int %/a	Ctl %/a	Pen mm	Flow l
1=20/53	13.136	-.1636	-.16357	4.5'	16.359
8=20/53	13.022	-.0059	-.05662	2.09	3 794
15= 53	12.639	-.0394	-.08578	2.61	9.827
22=20/53	12.942	-.0007	-.05206	1.40	5.334
5=20/53	12.916	-.0024	-.05359	.47	5.670
12=20/53	13.192	-.1781	-.20662	4.55	19.694
Average	'2.975	-.0650	-.10806	2.58	9.446

HGR-20.PLT -- 21/03/84-[illegible]-- 1.0 (LCH-COMP DAT, 2)

GRAPH D.18

OW HYDRAULIC GRADIENT (10 m/m), 5% CEMENT

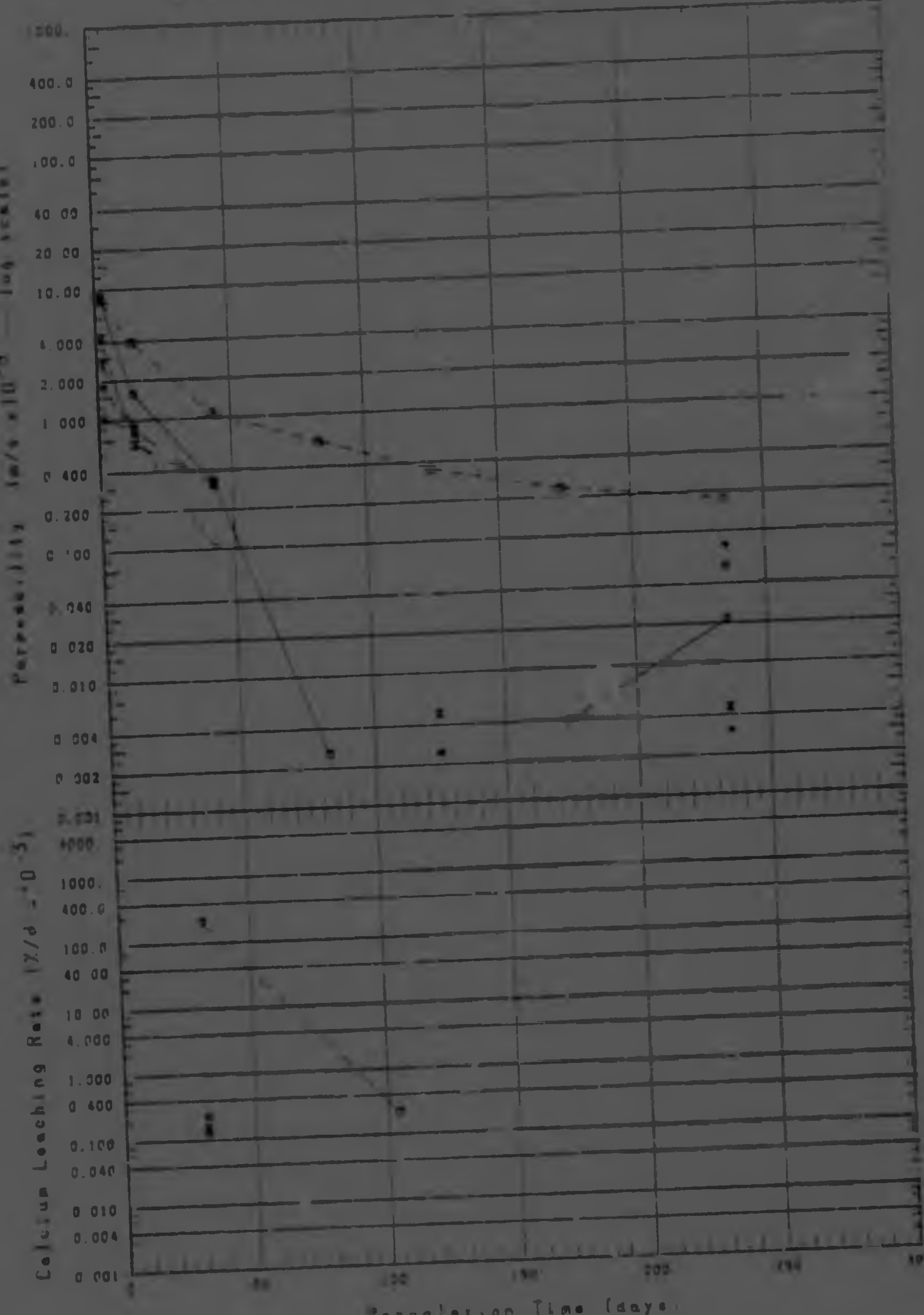


Specimen #	Inc %	Int %/d	Ctl %/d	Pen mm	Flow l
7=05/19	13.326	.00702	-.03516	.60	.547
14=05/49	13.320	-.0083	-.04695	.85	.910
3=05/49	13.345	-.0083	-.04870	1.60	1.118
10=05/49	13.406	.02417	-.01997	.60	.298
17=05/49	13.300	.03643	-.00911	.20	.121
6=05/49	13.459	.03634	-.00919	.45	.023
Average	13.360	.0148	-.02606	.72	.503

*W-08.PLT — 21/03/... L.O FLCH-CONF DAT. 2

GRAPH D.19

HYDRAULIC GRADIENT (10 m/m), 10% CEMENT

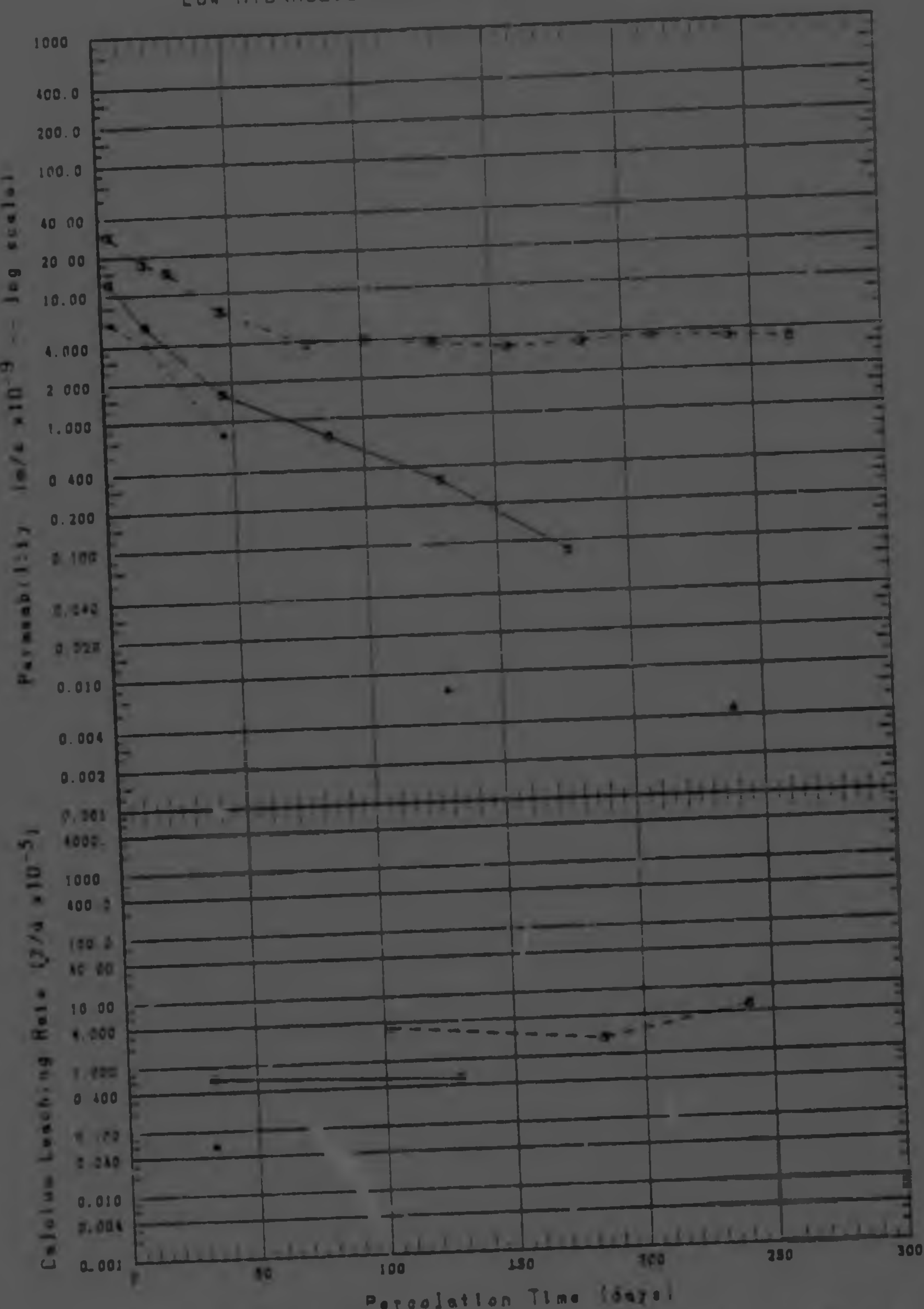


	Specimen #	Imc %	Ist %/d	Ctl %/d	Pen mm	Flow l
●——●	1=10/56	13.302	.04266	-.00082	.20	.077
●- - -●	8=10/56	13.273	.00849	.03706	.35	.025
●- - -●	15=10/56	13.303	.05675	.005423	.20	.206
●— —●	4=10/56	13.121	.05364	.002682	.35	N/A
■— —x	11=10/56	13.252	.06134	.009461	.30	.041
●— —●	18-10/56	13.361	.06380	.011632	.15	.034
	Average	13.268	.0568	.00608	0.26	0.077

-- 14/03/84- ... (LCH-COMP.DAT. 2)

GRAPH D.20

LOW HYDRAULIC GRADIENT (10 m/m), 20% CEMENT



	Specimen #	Imc %	Int %/d	Ctl %/d	Pen mm	Flow l
————	7-20/53	13.252	.18582	-.00360	.00	.297
————	21-20/53	12.949	.05414	-.00045	.05	.145
•- - -•	18-20/53	12.878	.02755	-.02359	.15	1.496
	Average	13.026	.0439	-.00848	.13	.646

LOW-20.PLT — [illegible] 1.0 (LCH-COMP.DAT, 2)