THE USE OF STABILIZED SOIL AS A MATERIAL IN DAM CONSTRUCTION

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VOLUME ONE

A Thesis Submitted to the Faculty of Engineering
University of the Witwatersrand, Johannesburg
for the Degree of Doctor of Philosophy

Johannesburg 1984

DECLARATION

I declare that this thesis is my own, unaided work. It is being submitted for the degree of Doctor of Philosophy in the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University.

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JAMES ALEXANDER ROBERTSON

Twenty ninth dav of October, 1984.

ABSTRACT

The research was undertaken to evaluate the viability of stabilized soil as an alternative material for the construction of large dams. An extensive literature review was undertaken and the optimum section was analysed. Those areas requiring additional investigation were identified. The scope of investigation necessitated limiting further research to soil-cement. Soil was obtained from a site for which designs for earthfill and concrete dams had been prepared and at which a buttress dam was under construction. An extensive laboratory program comprising wetting and drying, permeability, leaching, unconfined compression, split tension, triaxial, oedometer and heat of hydration tests was undertaken. Parameters and proposals for design are derived and the optimum section further analysed. Proposals for further research are made and it is suggested that the construction of a trial embankment should be undertaken. The results are compared with recent developments in roller compacted concrete and a cost comparison is undertaken. It is concluded that soil-cement is a viable alternative material for dam construction, well suited to South African conditions, and that its use should be actively investigated in view of the current South African drought and recession.

SUMMARY

The objective of this research was to determine the viability, or otherwise, of stabilized soil as a material in dam construction.

An extensive literature survey was undertaken including wide ranging correspondence with individuals and institutions who had conducted work in this field. An extensive bibliography is included in the thesis.

An analysis of the optimum section for the stabilized soil dam was

undertaken. On the basis of this analysis and the literature review, those areas requiring further investigation were identified and a laboratory program was formulated. Due to the large number of variables requiring investigation, the scope of this laboratory program was limited to soil stabilized with cement.

The site of a concrete buttress dam, then under construction, was identified as being suitable for the construction of a soil-cement embankment. Design data for an alternative earthfill embankment dam was also available. Two soils were obtained from this site and used in a preliminary test program.

One of these soils was found to be better suited to use in soil-cement and also occurred closer to the dam site. This material was used in the main test program. The soil was residual felsite and classified as A-2-4 (SC-SM).

Cement contents investigated ranged from 5 to 20% cement by dry mass of soil in the majority of tests with 10% adopted as standard. Additional cement contents were investigated where necessary.

Wetting and drying, leaching, unconfined compression, split cylinder tension, triaxial compression, direct shear, oedometer and heat of hydration tests were conducted with test periods of up to 900 days.

Standard ASTM wetting and drying tests were conducted and extended to a second series of 12 cycles. Small cylindrical specimens were also subject to the wetting and drying cycles and then tested in other parts of the program. It was found that the wetting and drying cure led to an increase in compressive strength and resistance to leaching. The wet-dry test was found to provide a reasonable correlation with the results of the leaching test program. It is suggested that the wet-dry test is essentially a surface leaching test. Suggestions are made as to methods of accelerating and simplifying the standard test procedure.

Long term leaching tests were performed using a form of constant head permeameter. A total of 192 specimens were subject to leaching, generally for a period of approximately 260 days. Variables investigated included hydraulic gradient, cement content, degree of compaction, the use of de-aired distilled, natural and carbonated waters and various curing conditions and periods. Concrete mortar specimens were also tested. Surface leaching was examined using a leaching bath. It is found that while leaching occurs under high gradients, it is unlikely to present a significant threat to the type of structure envisioned in this research.

Unconfined compression tests were conducted to provide data for relating the results of the test program to existing work in this field and to provide a simple basis for comparing the effects of variations in curing conditions. The effects of moisture content at time of test and degree of hydration are examined. The unconfined compressive strengths are compared with those reported by other workers and for other materials such as roller compacted concrete. The results are found to compare favourably. Unconfined compressive strength is found to be a linear function of cement content and to be proportional to the logarithm of the age at test.

Cylinder splitting tests were conducted as a measure of tensile strength. Split tensile strength is found to be of the order of 10% of the unconfined compressive strength.

Triaxial compression tests were conducted to further investigate the properties of the material in compression and to provide values for comparison with the results of the direct shear and oedometer tests. Consolidated undrained tests with pore pressure measurement were conducted on saturated specimens. Pore pressure at failure was found to be insignificant and it is recommended that unconsolidated undrained tests on unsaturated specimens can be used for routine triaxial testing. The angle of internal friction is found to increase with increases of cement content up

to 5% and to remain constant thereafter. Cohesion is found to be a linear function of unconfined compressive strength.

Direct shear tests were conducted on specimens 380 mm long, 160 mm wide and 260 mm deep with the shear plane at mid-height. Soil with a maximum particle size of 19 mm was used. Tests were conducted on intact specimens and specimens with construction joints subject to a two day construction delay and various joint treatments. The full material strength was not developed on the construction joint and suggestions for changes to the method of compaction of test specimens are made. Recommendations are made for further tests to be conducted in the field or laboratory. The apparatus used permitted controlled failure and determination of the residual strength of the material. It was found that despite the reduced ultimate strength on construction joints the residual strength was unaffected. A design philosophy based on these observations is proposed. The results are compared with those of the triaxial test program and suggestions for adjusting for the coarse aggregate content of the material are made.

Oedometer tests were conducted using standard soil mechanics apparatus with specimens subject to a loading program designed to simulate embankment construction and reservoir operation. Long term tests were conducted using gradual loading over several months with repeated load cycling to simulate reservoir operation. Corresponding tests were conducted with a two week test period and specimens tested at various ages. Additional tests were conducted on specimens subject to varying degrees of lateral restraint. A standard test technique is suggested. Three moduli for use in design are proposed and related to the unconfined compressive strength of the material.

Heat of hydration tests were conducted from which an estimate of the coefficient of thermal conductivity is obtained which is found to agree well with reported values for compacted soil. The heat gain characteristics of the soil-cement are found to agree well with those reported for cement. However, it is concluded that, due

to the relatively thin layers in which the material is to be placed, no thermal problems are anticipated.

Certain apparatus developed for use in this research represented a departure from that normally used, with alterations being made to reduce costs and increase efficiency. Details of this apparatus are presented in an appendix.

A review of recent developments in the field of roller compacted concrete is presented and the results of this research related to them. The optimum soil-cement embankment profile is determined and proposals for construction are made. The need to avoid unnecessarily complex designs in order to obtain maximum cost efficiency is stressed. A brief cost analysis is conducted to compare the soil-cement dam with alternative designs. This comparison is favourable and it is suggested that considerable savings could have been effected had a soil-cement dam been erected in place of the concrete dam constructed on the site from which the soil used in this research was collected.

Proposals for further research resulting from this program are summarised and it is concluded that the most effective form of further research would take the form of construction of a trial embankment as the first stage in obtaining tenders for the construction of a soil-cement alternative to a conventional structure.

It is concluded that the viability of soil-cement as a material for dam construction has been adequately established and that in view of the current drought and recession in South Africa the use of soil-cement should be actively investigated with a view to making optimum use of available financial resources.

To my parents, Angus and Thelma

and my wife, Fiona.

PREFACE

The objective of this research was to determine the viability, or otherwise, of stabilized soil as a material in dam construction.

The basis of the research was as follows:-

1. The stabilization of soil by the addition of lime or cement in suitable quantities, increases the shear strength.

   The shear strength of soil is the factor limiting the steepness of slopes in an earth dam.

   By increasing the shear strength of the material, the slopes of a dam can be increased until overturning stability becomes the limiting factor in design.

   This should reduce the volume of material in a stabilized soil dam by as much as 80% compared to a conventional earth dam (Rocha, Folque and Esteves 1961; Nash, Jardine and Humphreys 1965; Raphael 1970 and Blight 1973).

   In view of the increase in the unit cost of material, an overall cost saving of up to 40% over a conventional earth dam has been predicted (Raphael 1970).

2. The placement of concrete materials or stabilized soil using earth moving techniques can improve efficiency and reduce costs over a conventional concrete gravity dam (Raphael 1970 and Moffat 1973).

3. The residual soils encountered in many parts of Southern Africa are well suited to stabilization and are frequently not present in sufficient volume to justify the construction of a conventional earth embankment dam.

This direction was refined after completion, with Prof G.E. Blight, of the paper "Stabilized Earthfill Dams" for the Symposium on Soil Reinforcing and Stabilizing Techniques in Engineering Practice, held in Sydney, Australia, from October 16 to 19, 1978. This paper forms the substance of the first three chapters of the thesis.

The sequence of chapters in the thesis discussing the results of the experimental program, has been chosen to limit the amount of forward referencing. Where results for a particular group of tests contribute to the results of tests discussed in a later chapter, discussion of these results is contained in that chapter.

Details of the computer programs used in this research are supplied in appendix A. The information in small print in the bottom left hand corner of all graphs refers to the data files from which the graphs were plotted. Copies of the computer diskettes containing this data and master data files can be supplied to interested parties wishing to process the data further. Information on disk formats is contained in the appendix.

Additional information on the apparatus built for this research program is contained in appendix C.

ACKNOWLEDGEMENTS

I would like to express my appreciation for the support and assistance of the following organisations and individuals.

The Council for Scientific and Industrial Research, the Senior Bursaries Committee of the University of The Witwatersrand, the Freda Lawenski Scholarship Fund and the Hennen Jennings Scholarship Fund for post-graduate scholarships and bursaries without which this research could not have been undertaken.

The Cement Industries Fund, the Department of Water Affairs, Palment Ltd, the Portland Cement Institute, Slagment Ltd and the Water Research Commission for grants for the purchase of equipment.

I would particularly like to acknowledge the contributions of the following:-

My wife, Fiona, without whose love, encouragement and support, both moral and financial, I would never have completed this research.

My parents, Angus and Thelma, for their encouragement and support, who purchased the computer used in processing the data generated in this research and used for compiling this thesis and provided me with office accommodation during the last three years, while the processing of data and writing-up were being undertaken. Also, my father for employing me during this period.

Prof G.E. Blight [illegible] suggested this research topic, supported [illegible] many applications and requests for funds and assistance and guided me through the completion of this research and thesis. [illegible] Department of Civil Engineering for the use [illegible] facilities.

Mr V. Newey, [illegible] work that went into building the equipment used in this research, particularly for his suggestions for improving and simplifying equipment and reducing costs and for his patience with the many changes and additional items [illegible].

Mrs M.[illegible] Thomas [illegible] ndence, applications, requisitions and orders, and her patience with all the [illegible]onal adm[illegible] from this research.

Mr [illegible]. Braddock, [illegible] sharing his [illegible]rries and [illegible].

Mr F. Wild for assi[illegible] calibration of [illegible] cells and proving rings as well as assistance with many other [illegible] workshop.

Mr R. van der Merwe [illegible] help with [illegible]ment and facilities.

Mr [illegible] equipment.

[illegible]ry Bentel [illegible] stages of the long-term [illegible]ching test program and for conducting triaxial and other tests [illegible] completion of the long-term tests. Gary has extended this [illegible]rch into [illegible].

[illegible]ow post[illegible]grad[illegible] office and my [illegible] different [illegible] Macdonald, George Anandale, [illegible]rvin [illegible] and John [illegible].

All the other members of the Department's technical, academic [illegible] secretarial staff who assisted me in one way or another whether [illegible] way of sharing facilities, offering advice [illegible]

Mrs C. Blumenthal of the Engineering Library [illegible] cheerful, assistance in tracking down obscure references.

Prof R.J. Keddy of the Nuclear Physics Research Unit of the University, for advice and assistance with preliminary leachate analysis. Also members of staff of [illegible] advice.

Mike Braum who has kept my computer [illegible] the liquidation of the South African agents and the original [illegible] manufacturers.

Mr P. Nel, the University Accountant, for his advice and assistance in producing financial statements for the Department of Water Affairs and Water Research Commission.

Dr D. Stephenson and Mr E. Manson of Stewart, [illegible] Oliver, consulting engineers, for assisting in the choice of a suitable site, supplying drawings and arranging access to the site. Mr Manson also supplied final cost data for the Rondebosch Dam. Dr Stephenson is now Professor of Hydraulic Engineering in the Department of Civil Engineering.

Mr B.G. Lund of B.G. Lund and Partner, consulting engineers, for supplying drawings, specifications, etc for the alternative earth-fill embankment designed for the Rondebosch Dam site.

Pretoria Portland Cement for donating the cement used, [illegible] for providing detailed chemical analyses of the cement and advice on storage and deterioration. The assistance of [illegible] and Mr S. Makovy is particularly acknowledged.

The **Portland Cement Institute** for designing, casting and performing control tests on concrete mixes for comparative use in the leaching test program. The interest and assistance of Mr J. Lane and Mr L.R. Marais is particularly acknowledged.

The **Hydrological Research Institute** of the Department of Water Affairs for analysing more than 600 samples of leachate for calcium and other elements and **Mr C. Grobbelaar** of the Department of Water Affairs who arranged for this to be done.

The **Department of Chemistry** at the University for supplying the distilled water used in the leaching test program.

The **Department of Mining Engineering** at the University for the use of their core drilling apparatus and for advice on producing cores from the concrete beams produced by the PCI.

Latex Products Limited, in particular Mr Carlin, for their assistance in producing the latex membranes used in the leaching and triaxial test programs.

All the members of staff of other Departments in the University and elsewhere who assisted with advice, materials, facilities and equipment.

My **sister Bridget** Quan who assisted with the data capture of the results of the direct shear tests.

My **mother** and my **sister, Angela Kerslake**, who between them undertook the proof reading of this thesis.

All those throughout the world who responded to my correspondence with advice, references and encouragement and those who made copies of theses and confidential reports available to me. Their names appear under the heading **Correspondence** at the end of the thesis.

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I would particularly like to pay tribute to the late Professor J.K.T.L. Nash who was responsible for much of the early research in this field. His help and encouragement in response to a number of requests for assistance was a source of inspiration.

ABBREVIATIONS

The following abbreviations are used in this thesis:-

HDPE	High density polyethylene
ID	Internal diameter
LDPE	Low density polyethylene
LVDT	Linearly variable displacement transducer
MDD	Maximum dry density (standard proctor compaction)
OD	Outside diameter
OMC	Optimum moisture content (standard proctor compaction)
PVC	Polyvinyl chloride
TMS	Table mountain sandstone (South African geological formation)
UCS	Unconfined compressive strength
UPVC	Unplasticized PVC

The introduction to the list of References contains a list of abbreviations of organisations and institutions which are used in the **References**, **Correspondence** and **Bibliography**. Certain of these abbreviations, which are generally used for the institutions concerned, are also used in the text.

UNITS

nm/s Nanometres per second, m/s x 10^{-9} -- alternative unit of permeability.

Abbreviations of SI units conform to SI practice.

Imperial units are used for certain data extracted from references, in these cases, the customary abbreviations are used. These abbreviations are generally expressed in words where they are first used.

VARIABLES

Sc Factor of safety against compression failure of a gravity dam.

Q Factor of safety against sliding failure of a gravity dam.

Other variables are defined in the sections in which they are used and are not used elsewhere.

LOGARITHMS

All logarithms (log) are to base ten ($\log_{10}$).

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3. THEORETICAL ANALYSIS OF THE STABILIZED EARTHFILL EMBANKMENT

Reproduced from paper (Robertson and Blight):-

CHAPTER 1

COMPARISON WITH CURRENT SOLUTIONS

Initial work to determine the scope of this research included a comparison of the proposed stabilized earthfill dam with current solutions. This was summarised in section 2 of the paper, referred to in the preface, prepared by the author and Prof G.E. Blight (Robertson and Blight 1978):-

"The stabilized earthfill dam is an embankment dam constructed with modern road building techniques. Soil, premixed with a predetermined moisture and stabilizer content, is placed in layers and compacted to a satisfactory density. The increased strength resulting from stabilization permits the construction of an embankment with much steeper slopes than can be constructed with unstabilized material. Slopes as steep as 0.5 (horizontal) to 1 (vertical) may be possible. The resulting structure compares favourably with the accepted solutions represented by concrete gravity and earth embankment dams.

1.1 COMPARISON WITH EARTH EMBANKMENT DAMS

The fill used in a stabilized soil dam is subject to essentially the same handling and processing as soil used for an earth embankment. The material is excavated, stockpiled, transported to the dam site, transported onto the embankment, placed in layers of suitable thickness and compacted to the required density. Water may be added to the material to condition it to the correct moisture content for compaction. Stabilized soil requires only the additional introduction of the stabilizer before the material is placed in the embankment. Blending of the necessary water and stabilizer can be effected by using a mixing plant whose capacity is matched to that of the spreading and mixing equipment as cement stabilized soil (soil-cement) should preferably be compacted within one hour of mixing. Lime stabilized soil (soil-lime) can however tolerate greater delays. The rate of placing for stabilized soil may therefore be identical to that for unstabilized material. While the unit cost of material in-place is increased by the cost of the stabilizer and plant mixing costs, the embankment volume may be reduced by as much as 80% and a consequent reduction in embankment cost of up to 40% has been predicted (Raphael 1970). Because the volume required is considerably less, a significant reduction in construction time can be expected with a corresponding reduction in interest and inflation costs. Reservoir filling may also be commenced sooner, with the result that the project will generate revenue within a shorter period.

The use of soil-cement as slope protection for embankment dams is well established (Holtz and Hansen 1976 and others) and the upstream face of a stabilized soil dam does not require additional slope protection, although it may be necessary to increase the stabilizer content of the outer shell of the dam. The need for downstream slope protection is also eliminated although some aesthetic treatment may be required.

The reduced base width of the stabilized soil dam would considerably reduce the length of diversion works, outlet works, penstocks etc which would result in economies both from the reduced length as well as the consequent reduction in hydraulic losses which may permit reduced diameters of penstocks etc. In addition, the increased erosion resistance of the stabilized material may allow a reduction in capacity of the diversion works by al[illegible]ing the acceptance of a greater risk of having the partia[illegible]pleted embankment overtopped, The increased erosion resist[illegible]e and strength of stabilized soil may well permit the cons[illegible]ion of emergency overflow structures on the embankment itself.

Foundation preparation for a stabilized soil dam would be similar to that for an earth embankment on the same foundation. The width of foundation requiring preparation for the stabilized embankment would be up to 80% less than that for the earth dam. However, the depth to which the site will require stripping to provide a suitable foundation for the stabilized embankment might be greater than that required for an earth dam.

Stabilized soil is less flexible than unstabilized material and consequently a stabilized dam would not be suitable on sites where flexibility is a requirement. In addition, the reduction in base width results in a considerable increase in pressure at the toe of the dam with the result that stronger foundation material is required. Triaxial testing has revealed that a silty or clayey soil stabilized with cement behaves plastically under high confining pressures, whereas granular soils stabilized with cement remain brittle (Rocha, Folque and Esteves 1961, Mitchell 1976a). The stabilized dam can therefore be expected to be more flexible than a concrete or lean concrete dam.

Significant construction pore pressures are unlikely to develop in a stabilized embankment as compaction water will be consumed by the stabilizer. The low compressibility of a stabilized soil will also reduce construction pore pressures. This applies particularly to cement stabilization where the initial hydration reaction occurs rapidly. The initial strength gain with lime stabilization is not as rapid, but lime has a marked drying action, a property often exploited in road construction. The slower strength gain of lime stabilized soils has the advantage that the material remains more flexible during construction. The tendency for autogenous healing of cracks in lime-stabilized materials (McDowell 1966 and Thompson and Dempsey 1969) suggests the possibility of using a core of lime stabilized material on sites where settlement is expected. It should be noted that the final strength developed in a lime stabilized clayey or silty soil may equal or exceed the strength developed with cement stabilization.

The permeability of stabilized soil is frequently less than the permeability of the unstabilized material. However, soil-cement has a tendency to develop shrinkage cracks when used in thin layers. (This tendency may be reduced when using the material in mass.) Seepage along compaction planes may also occur although this may be prevented by suitable construction procedures, The majority of seepage through a soil-cement facing occurs through cracks in the soil-cement. This has been demonstrated by the USBR on the Lubbock Reservoir seepage test section, where the soil-cement used in the facing exhibited a permeability of 1×10^{-9} m/s while the facing as a whole exhibited a maximum permeability of 140×10^{-9} m/s in winter with average values of 17×10^{-9} m/s in summer. The variation with seasons was attributed to thermal movement (De Groot 1971a). Material having permeability of 140×10^{-9} m/s would normally be classified as semi-pervious, this permeability corresponds to approximately the upper limit acceptable for the impervious zone of an earth dam. A stabilized soil core could be expected to exhibit better permeability characteristics than this as it is subjected to three dimensional confining stresses. It is also possible that a soil-cement facing could provide the impermeable element in a stabilized soil dam. This would imply the inclusion of a drain immediately downstream of the facing which would reduce the uplift pressure on horizontal planes and permit a reduction in the section required for gravity stability.

A further aspect of seepage through stabilized material is the possibility of leaching of stabilizing agents with a consequent progressive reduction in strength. Moffat an Price (1978) consider leaching to be a problem with dry lean concrete and have suggested the construction of an upstream membrane of rich concrete backed with no-fines drains to overcome it. Little information is available to indicate whether leaching presents a problem with stabilized soil, although the results at Lubbock Reservoir (see above) seem to suggest it does not. However, considerable testing of soil-cement subject to seepage will be required before the question can be resolved.

On balance, the above discussion shows that a stabilized soil dam has many potential advantages over the earth embankment, all of which can be expected to lead to reduced costs. On certain sites, particularly where foundation soils are deep and flexibility is required, the earth dam remains the only solution. Equally, for small dams, the increased cost of plant and processing for a stabilized soil dam may not be justified.

1.2 COMPARISON WITH CONCRETE GRAVITY DAMS

The rate of placement of stabilized soil can be similar to that of unstabilized material which in turn is as much as an order of magnitude greater than that of mass concrete in a gravity dam. Raphael (1970 and 1976) quotes maximum rates of placement of 77 000 m^3/day for earthfill as opposed to 7 700 m^3/day for mass concrete. The discrepancy results from the continuous and plant intensive nature of earth moving operations as opposed to the batch and labour intensive nature of mass concrete placement. In addition, mass concrete requires shuttering and may require complex cooling systems, whereas stabilized soil requires no shuttering. (With the steep slopes being considered it may prove necessary to place a slip formed stabilized soil, or concrete, kerb at the faces of the embankment.) Heat generation in stabilized soil is not expected to be a problem in view of the thin layers in which the material is placed (Gentile 1970) and the relatively low binder contents, although the effects of heat generation due to pozzolanic reactions later in the life of the structure require further investigation (Raphael 1976).

It should be noted that there is a tendency towards the construction of concrete gravity dams using earth moving techniques. Moffat and Price (1978) have proposed a rolled 'dry lean concrete' dam and Gentile (1970) has described the construction of a dam using a low slump concrete placed in 0.70 m thick lifts over the entire dam. Nevertheless, in these cases the concrete aggregate still requires processing. The handling sequence will generally require mining of the material from a quarry, crushing, screening and blending to give the required grading. This may be compared with the construction sequence for the stabilized soil dam discussed under 1.1 above. In addition to the increased handling and processing requirements, quarry operation and the handling of crushed aggregate generally leads to more wear-intensive operating conditions as regards plant than borrow pit operation with earth materials. It can therefore be expected that the unit cost of stabilized soil in place will be less than that for low slump or dry lean concrete. However the lower density of stabilized soil will require a section with a greater base width and consequently the relative economics of the solutions may well be dictated by the site and materials available.

Many of the advantages of a stabilized embankment over an earth dam do not apply to the comparison with a gravity dam. Penstocks and diversion works for a stabilized embankment will be slightly longer and the width of foundation preparation will be greater. Furthermore a concrete dam will generally be resistant to damage by overtopping and diversion works can be smaller. The doubts regarding the possibly deleterious effects of seepage do not arise for the cement rich concrete of a gravity dam. The numerous horizontal compaction planes in a stabilized embankment present more opportunities for horizontal sliding failure of the dam. The USBR has conducted research into interlayer bonding of soil-cement for slope protection (Davis, Gray and Jones 1973) and a report is expected to be available shortly (Warren 1978a).

The major advantage of a stabilized soil embankment over a concrete gravity dam is the absence of shuttering and cooling systems and the plant intensive nature of operation leading to shorter construction times. In 1976, the PCA quoted an average cost for soil-cement slope protection of US$12.70 per cubic yard for quantities in excess of 100 000 cubic yards (PCA 1976) while in 1973 the average rate for mass concrete including shuttering but excluding cooling was approximately US$33 per cubic yard (Hansen 1973). This gives some indication of the cost difference of the two materials. It should be noted that the cost comparison with concrete buttress and arch buttress dams, which are generally cheaper to construct than gravity dams, will not be as favourable. The somewhat greater flexibility of a stabilized embankment may prove to be an advantage on certain sites."

CHAPTER 2

LITERATURE REVIEW

An extensive literature review was undertaken to determine the state-of-the-art and to avoid duplication of effort. Local sources, in particular the Engineering Library of the University, the Portland Cement Institute and the Department of Water Affairs, were consulted first. From this, it became apparent that many of the tests originally proposed had already been conducted on a wide range of soils and stabilizers. There was therefore no point in duplicating this work.

The review was then expanded to include correspondence with those individuals and organisations that had generated the items obtained locally. The more important of these are listed in the section **Correspondence**, at the end of this thesis.

As a result of this correspondence, a large number of additional papers, reports and theses, many of relevance, were obtained. Input in the form of replies to specific queries was also obtained. Together, these inputs were significant in formulating the final direction taken in the laboratory program described herein.

In addition to the list of **References**, all other documents obtained are listed in the **Bibliography**. This also contains documents of relevance to proposed areas of the research which were eliminated in order to reduce the scope of the test program. These are included as they contribute to the broad background of knowledge on which this research is based and in the hope that they will be of assistance to future workers in this field.

2.1 STATE-OF-THE-ART IN 1978

Robertson and Blight (1978), section 3, summarised the State-of-the-Art as indicated by the literature obtained by June 1978:-

"The controlled stabilization of base and sub-base layers in road and airfield pavements using cement as stabilizing agent dates back to at least 1935 when a project using soil-cement was undertaken in the USA (Mitchell and Freitag 1959). The use of lime in similar applications dates from about 1945 (Herrin and Mitchell 1961). Subsequently, the use of cement and lime stabilization of soils to increase their bearing capacity and improve their durability has become an internationally accepted technique in road and airfield construction.

The first significant use of soil-cement in dam construction occurred in 1951 when the USBR constructed a trial embankment in the reservoir area of Bonny Dam and faced it with soil-cement and asphalt concrete as trial substitutes for rip-rap slope protection (Powers, Benson and Meissner 1952). This trial embankment was closely observed for a period of 10 years after which the USBR reported the results of the test (USBR 1961a). With the exception of minor problems relating to the existence of lenses of unstabilized material between layers, resulting from the in situ mixing procedure, the performance of the soil-cement was entirely satisfactory. As a result of the success of the Bonny Test Section, the USBR, in conjunction with the Portland Cement Association, produced detailed specifications for a soil-cement facing to Merritt Dam, Nebraska as an alternative to rip-rap. The project went to tender in 1961 and the lowest prices bid per square yard of slope were \$5.58 for soil-cement and \$11.97 for rip-rap. The dam was duly constructed with soil-cement slope protection on the upstream face. In December 1976, the PCA reported some 150 projects in which soil-cement had been used for slope protection for earth dams or other embankments. Of these, 28 projects were exposed to flowing water (PCA 1976b). In the interim the PCA had published a number of technical notes and guide-lines for the use of soil-cement as slope protection and together with the USBR and other agencies had investigated more rational means of design. Particularly significant, was a PCA report on the results of erosion and wave run-up tests (Nussbaum and Colley 1971). Also in 1976 Holtz and Hansen produced a state-of-the-art report on "The Use of Compacted Soil-Cement in Water Control Structures". Consequently, the durability of soil-cement as slope protection for dams is a well established fact and the technology of placement has advanced to a stage where interlayer bond and compaction of the layer edge are no longer problems.

In 1962, Rocha, Folque and Esteves suggested the use of cement stabilization to increase the strength of the material in an earth embankment. It was proposed that the cement content should be limited to between 2 and 5% in order "to exploit the improved mechanical properties of soil-cement, retaining those features which will ensure that the soils used for the construction of earth dams will follow the conventional pattern". The features mentioned were "the deformability and plasticity which enable the material to follow the slope of a foundation, and the designer to employ failure criteria based on plastic theories". This approach permitted an increase of embankment slope to 1 horizontal on 1 vertical with a corresponding reduction of volume of between 50 and 60% as compared with an embankment with slopes of 2.5 to 1 on both upstream and downstream slopes, However, it was found that even at these relatively low cement contents, a considerable reduction in flexibility occurred while at the same time the low cement content made uniform mixing of soil and stabilizer extremely difficult. This research has continued and a small experimental dam is to be constructed (Borges 1978).

In 1965 Nash, Jardine and Humphreys discussed "the economic and physical feasibility of soil-cement dams" in the light of preliminary tests for an embankment in Nigeria and concluded:- "At present it appears that progress towards the development of a safe, durable, and economically advantageous dam consisting primarily of soil-cement on a rational (as opposed to empirical, trial and error) basis would involve a good deal more research, together with field experiments." Subsequently, little further research into the use of soil-cement in dams has been conducted by Nash and his colleagues (Nash 1977).

In 1970 Raphael proposed a soil-cement dam on the basis of observed similarities between the Mohr envelope of the mass concrete of Ross Dam and the rockfill of Oroville Dam. Using assumed average material properties, he showed that while there was not a continuous spectrum of materials of which concrete and rockfill were the upper and lower bounds, the cement stabilized earthfill embankment could be expected to be more economical to construct than either the conventional concrete gravity or earth embankment dams. Raphael concluded that a number of unknowns required further research but that the investment in such research would be fully justified by the savings realised. In 1976, on the basis of further research and analysis conducted at the University of California, Raphael demonstrated that an optimum dam existed with slopes of 0.8 horizontal to 1 vertical for an embankment 61 m high having a crest width of 9.15 m and no freeboard. This analysis did not take account of "secondary cost savings associated with the soil-cement dam, chiefly due to the opportunity for making design changes". These factors, discussed above in the comparison with existing techniques, would tend to favour an embankment close to the limiting gravity-stable section of approximately 0.5 to 1."

Section 3 of the paper (Robertson and Blight) concluded: "The basic economic and structural feasibility of the soil-cement embankment dam is thus theoretically established, as is the need for further investigations of material properties together with more detailed analysis of typical sections and application of results to actual cases". The paper went on to examine the latter points in more detail based on the work conducted up to that time. This section of the paper is reproduced in chapter 3.

2.1 ADDITIONAL LITERATURE

While a considerable number of references were obtained after completion of the paper, they served to complement the information already obtained, rather than indicating any major new work in this direction. The most significant of these references are discussed below.

Copies of theses by Abboud (1969), Akky (1974), El-Rawi (1967), Jackson (1965), Jardine (1963) and Price (1977) were obtained and proved particularly useful as references in designing test conditions, procedures and equipment.

Jackson (1965) examined "The Influence of Leaching on the Permeability and Shear Strength of Soil-Cement" in work following on from that performed by Jardine. He concluded that "in a soil-cement dam where micro-cracking may be extensive, localised leaching may result in weakening in the areas immediately surrounding the micro-cracks leading to a reduction in the overall stability subsequent to impounding" (Jackson 1974). Correspondence with Nash (1978) indicated that in view of certain problems experienced by Jackson with his equipment, his results should be treated "as preliminary".

Abboud, El-Rawi and Jardine all investigated various properties of soil-cement with the triaxial test used as the primary test technique. Akky examined "The Erodibility of Cement Stabilized Soils" with a view to determining a more rational basis for the design of soil-cement slope protection.

Additional references applicable to soil-cement include a Portland Cement Association publication with design information for the use of "Soil-Cement Water Barriers for Earth Dams" (PCA 1970) which indicates that no problems were anticipated as a result of leaching. Fetz (1973) suggested a technique for preventing cracking of soil-cement by compacting the material at moisture contents corresponding to a degree of saturation of 70%. He cites an example where soil-cement placed using this technique showed only two cracks over an area of 35 000 m^2. Unfortunately this paper was obtained too late to investigate this proposal further. Hansen and Avera (1978) reported the use of soil-cement for large cooling water reservoir dykes, and Hansen (1979a) indicated that no problems had been experienced with permeability or leaching of these embankments.

Work on the use of roller compacted concrete in various forms has proceeded rapidly over the last few years, and Schrader (1982) reported on the Willow Creek Dam, the first dam to be constructed entirely with roller compacted concrete (rollcrete) and which shows substantial cost savings over an alternative rockfill design. Concrete made with aggregate consisting of crushed rock with up to 30% of overburden is used. Other work in this field includes that by Price (1977), Dunstan (1981) and that reported by The American Concrete Institute (ACI 1978).

It is considered that the two fields are converging steadily and that once rollcrete becomes a proven alternative to conventional concrete, it can only be a matter of time before a major soil-cement dam is attempted. It should be noted that rollcrete containing 30% overburden and up to 7% of fines passing the 200 mesh sieve, as used at Willow Creek, is unlikely to differ substantially in properties from soil-cement produced using a sandy soil supplemented with 30% of crusher-run.

CHAPTER 3

THEORETICAL ANALYSIS OF THE STABILIZED EARTHFILL EMBANKMENT

A detailed theoretical analysis of the proposed stabilized embankment dam was undertaken. The principles applicable to both concrete gravity and earth embankment structures were applied with a view to determining the critical requirements for the stabilized soil dam. The results were then consolidated and material properties assessed to determine whether they were achievable.

3.1 ANALYSIS FROM ROBERTSON AND BLIGHT (1978)

Sections 4 and 5, of the paper, referred to previously, summarised the results of this analysis and are reproduced below:-

4. ANALYSIS OF A STABILIZED EMBANKMENT

Design of a stabilized soil dam for a specified site where reservoir depth, freeboard and crest width are predetermined, is an optimization process in which the strength of the material and hence its unit cost can be varied in relation to variations in the upstream and downstream slopes. Stabilized soil has a substantially enhanced cohesion and there is uncertainty as to whether conventional methods of slope stability analysis will indicate the most severe design requirement. Equally, the embankment sections proposed are much closer to those of gravity sections than those of earth embankments. A gravity analysis based on that used for gravity dams thus appears to be indicated. Either of the above analyses may prove to be critical for the stabilized soil dam. The most economic section for a stabilized embankment would probably be close to the limiting gravity-stable section, and the analysis was confined to this section with further analyses being conducted to determine the effect of varying the assumed dimensions of the structure. The limiting gravity-stable section was then subjected to a slope stability analysis and the material strength required was derived by considering the results of both analyses.

4.1 Analysis Of Gravity Section

The section analysed is shown in Figure 1 and represents an embankment with maximum reservoir depth h, constructed of a material of unit weight γ and with an upstream slope of x_u:1 and a downstream slope of x_d:1. The crest width and freeboard are taken as fractions, 0,1 and 0,03 respectively, of the reservoir depth. It was decided to ignore the effect of earthquake forces. The triangular seepage pressure distribution assumed beneath the dam is that suggested by Justin et al (1945). ζ is a factor between 0 and 1 which represents the proportion of the total head acting on the base. The value of ζ depends on the nature of the foundation and the efficiency of the grout curtain and foundation drains. In view of the fact that the analysis was initially of a homogeneous section and because of the large number of horizontal construction joints that would exist in a stabilized embankment, a value of 1,0 was assumed for ζ as the standard case.

Unit weights considered, range from 16 to 24 kN/m³ with the majority of the analyses based on unit weights of 18 and 22 kN/m³. The sections were assumed to be of uniform density and no account was taken of increased densities below the phreatic surface (see section 4.4). The unit weights were chosen on the basis of a statistical analysis of the densities of 30 soil-cement mixtures examined by the USBR for projects reported by De Groot (1971b). The range of 18 to 22 kN/m³ corresponds to approximately 80 per cent of the mixtures examined and was assumed to be reasonably representative of most stabilized soils. The graphs permit some extrapolation of the results to materials with greater or lower unit weights.

The basic embankment section had equal upstream and downstream slopes. Structurally this is not the most efficient section as can be seen from conventional concrete gravity dams, however, it is simpler to construct than a gravity section having its upstream face nearly vertical. The relative economy of reducing the embankment volume by steepening the upstream face must be compared with the cost of providing an upstream face along which earth moving plant can operate. Horizontal slip-formed soil-cement kerbs which should provide adequate retention on a face of 0,5 to 1 would probably have to be replaced by a vertical slip-formed concrete face.

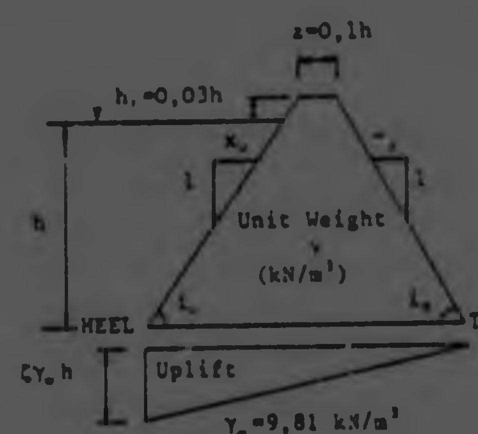


Figure 1

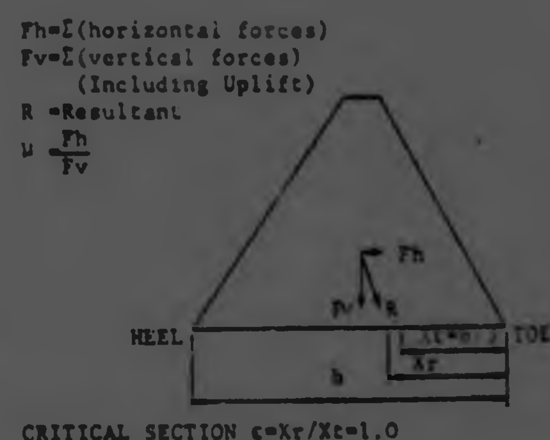


Figure 2

General Gravity Section - Definition of Terms

The assumed section was first analysed to determine the slopes required to give a so-called 'critical gravity section'. This section is defined as one having zero vertical stress at the heel i.e. the resultant of all forces passes through the downstream third point (Fig.2). The factor of safety against overturning is the ratio of stabilizing to overturning moments about the toe. As a result of the definition of crest width and freeboard height as fractions of reservoir depth h, the factor of safety S, position of resultant and ratio of horizontal to vertical forces are all independent of height. It is assumed that the foundation material is at least equivalent to the embankment material and that there are no zones of low strength in the foundation. The range of heights considered in the analysis was from 20 to 100 metres. It was considered that this was representative of a large proportion of all 'large dams' constructed. The analysis was based on a typical section having the properties shown in Table I. The ratio of horizontal to vertical forces μ is a measure of the limiting coefficient of friction required to resist sliding on the base if the material were cohesionless. As such, it is a measure of the shear strength required to prevent sliding. In addition, for a constant reservoir depth and therefore a constant water force, the variation of the ratio is a measure of the variation of the inverse of the vertical force.

The gravity analysis used is the simple rigid-body analysis described by Justin et al (1945) for application to concrete dams. As the side slopes flatten with decreasing material strength, the analysis becomes increasingly inappropriate. However, for a section close to the critical gravity section, the error in the analysis was not likely to be large. The effect of the plastic behaviour of stabilized soil under high confining pressures, as discussed above, on the relevance of this analysis requires further investigation. However, it seems probable that plastic deformation will lead to a redistribution of stresses in highly stressed zones and that consequently the rigid body analysis may be conservative. The section considered was also analysed to determine the material properties required to resist sliding on the base. The assumptions and method of analysis are discussed in section 4.3. The analysis was restricted to the 'reservoir full' condition. With the exception of the case where the upstream slope is vertical, overstressing of the heel in the 'reservoir empty' condition would not be expected to occur. Where an upstream slope approaching verticality is concerned it may be necessary to consider this case.

TABLE I
PROPERTIES OF TYPICAL SECTION

PARAMETER	SYMBOL		VALUE
Maximum reservoir depth	h		60 m
Freeboard height = 0,03h	h_f		1,8 m
Crest width = 0,10h	z		6,0 m
Uplift coefficient	ζ		1,0
Vertical stress at heel	σ_h		0 kPa
Line of action of resultant through downstream third (⅓) point	ε		1,0
Upstream slope (x_u) = Downstream slope (x_d) =	x		
Unit weight	γ	18kN/m³	22kN/m³
Embankment slope	x	0,525	0,449
Factor of safety against overturning	S	1,550	1,668
Maximum principal stress at toe	P_1'	1122kPa	1396kPa
Ratio of horizontal to vertical force	μ	0,556	0,494
Volume per unit length of embankment	V	2376m³/m	2086m³/m
Vertical stress at toe	σ_t	880kPa	1161kPa
Average shearing stress on the base	σ_s	249kPa	287kPa

4.2 Results Of Gravity Analysis

The results are presented graphically in Figures 3 to 9. The following discussion is intended to emphasize salient features of the graphs.

4.2.1 Variation of critical slope with unit weight γ (Figure 3)

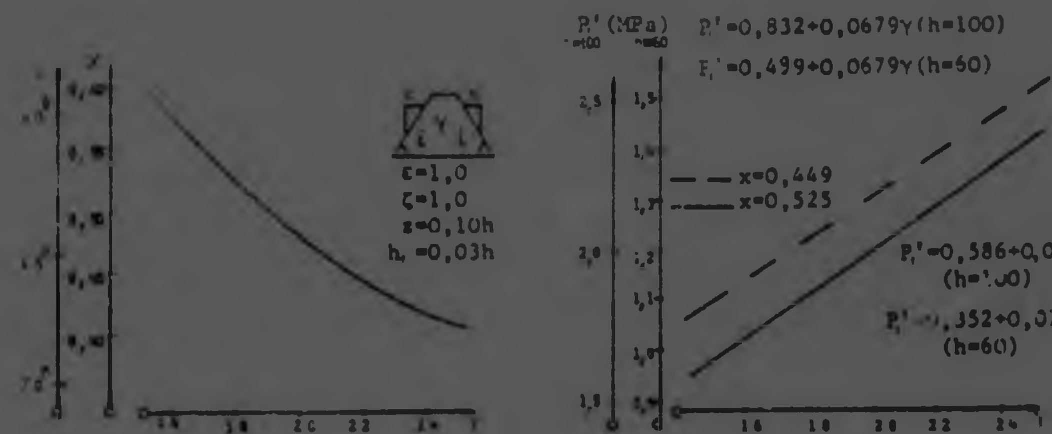
The 'critical' slope of an embankment type section conforming to the general properties of the typical section can be readily determined from this graph. If any of the assumed section properties, for example the crest width, are changed, an estimate can be made of the slope required from Figures 3 to 9.

4.2.2 Variation of parameters with reservoir depth h (Figure 4)

The geometric properties are constant with reservoir depth (h) and constant section (Fig.4a) while all other parameters are simple functions of height, that is, a 'constant' multiplied by h, h² or h³. The 'constants' are functions of factors such as slope, unit weight and uplift. e.g. Fig 3b.

4.2.3 Variation of parameters with slope x (Figure 5)

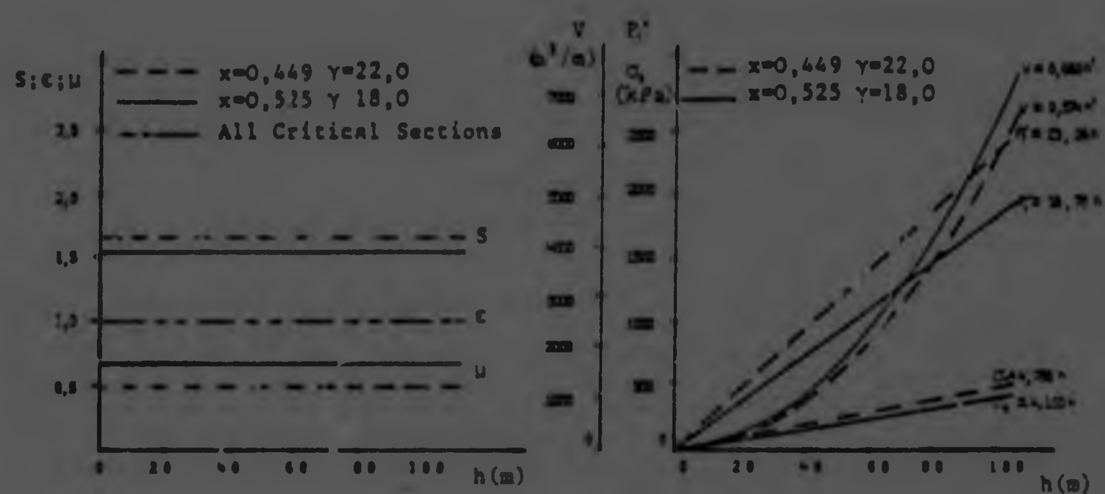
For a dam having upstream and downstream slopes equal, it is interesting to note that as the slopes approach those of a normal embankment dam, the parameters become increasingly independent of slope. It should be noted that, as the slope



(a) Figure 3 (b)

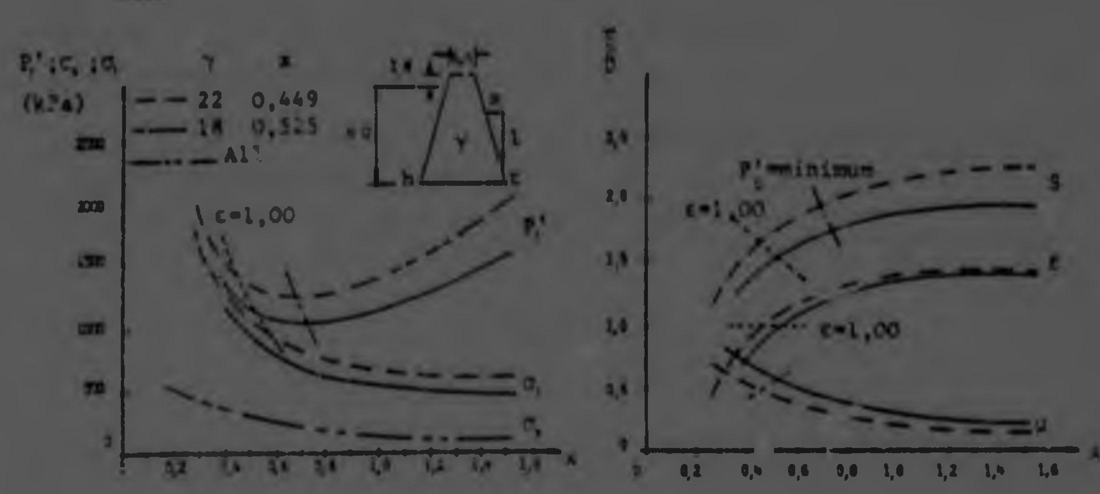
Variation of critical slope x with unit weight γ

Variation of maximum principal stress at toe P_1' with unit weight γ



(a) Figure 4 (b)

Variation of Parameters with Maximum Reservoir Depth h



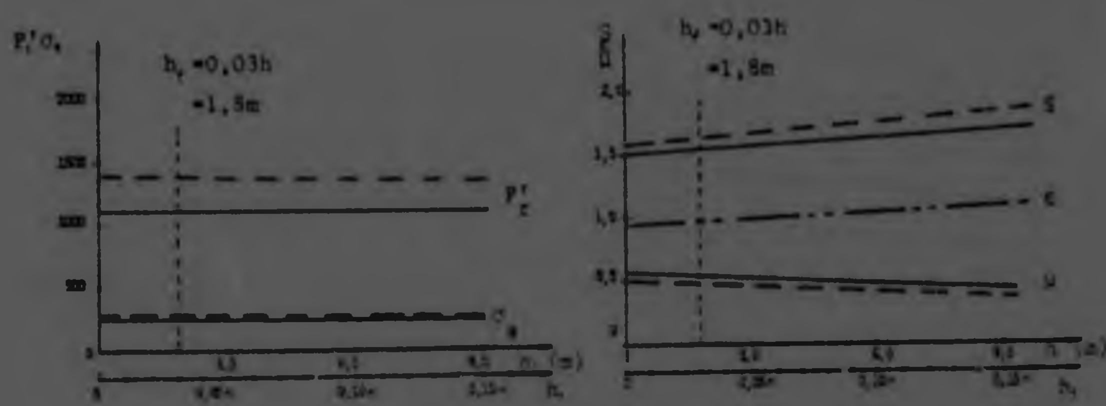
(a) Figure 5 (b)

Variation of parameters with slope x

flattens (x increases) the rigid body method of analysis used becomes inappropriate. This is demonstrated by the curve depicting maximum principal stress at the toe (P_t') in Figure 5a. The upward trend with flattening slope is a geometrical effect associated with the assumption of rigidity. It should be noted that the theoretical vertical stress at the toe σ_n continues to decrease.

4.2.4 Variation of parameters with freeboard height h_f (Figure 6)

As shown in Figure 6, stresses are hardly affected by change in freeboard while a significant change in the dimensionless parameters S, ξ and U occurs. This can be explained by the fact that increasing freeboard causes an increase in embankment volume which results in an increase in both the stabilizing force and the moment arm of this force, while the horizontal force component is unchanged.

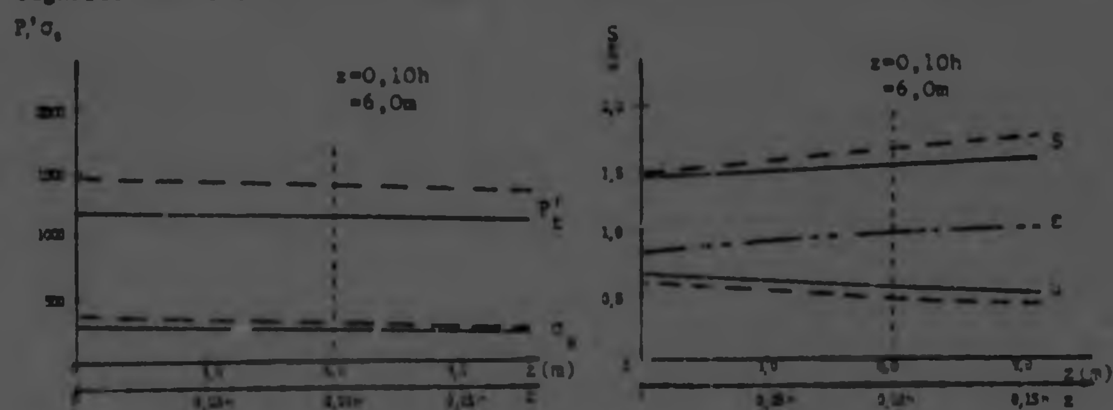


(a) Figure 6 (b)

Variation of Parameters with Freeboard Height h_f

4.2.5 Variation of parameters with crest width z (Figure 7)

The effect of varying the crest width on the parameters considered is much the same as that due to variation of freeboard height (Fig. 6). An increase in freeboard with constant crest width has the effect of increasing the width at a constant 'freeboard' height and imposing a small crest surcharge. The relative significance of these two effects is dependent on the slopes of the embankment.

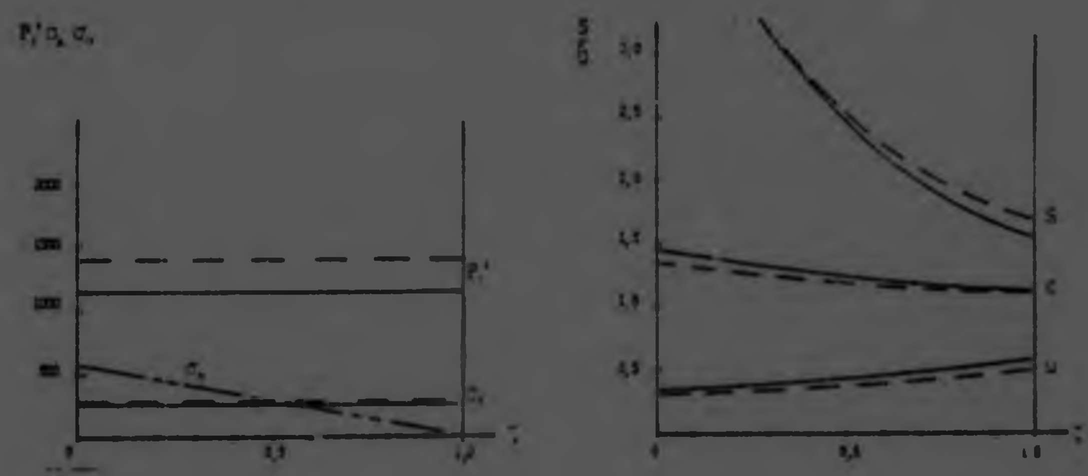


(a) Figure 7 (b)

Variation of Parameters with Crest Width z

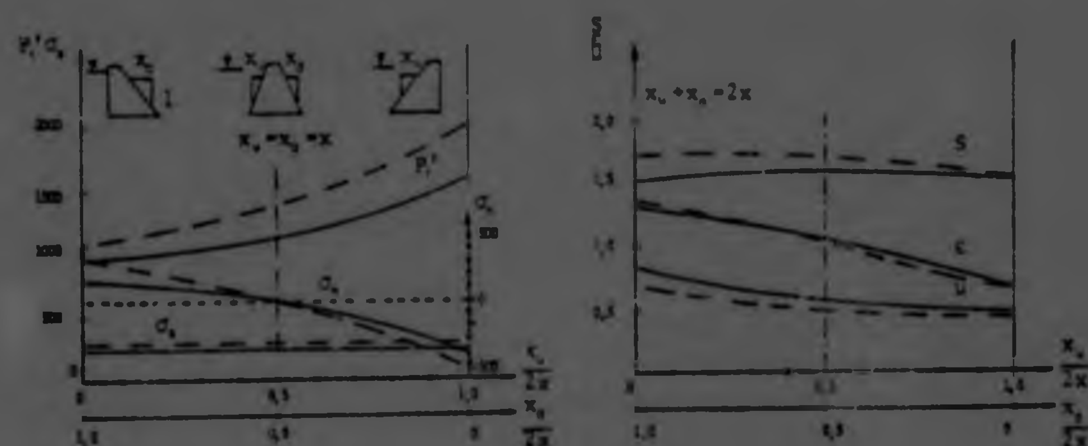
4.2.6 Variation of parameters with uplift coefficient ξ (Figure 8)

The effect of reducing the uplift coefficient ξ from the maximum possible value of 1,0 assumed in most of the analysis is considerable. The line of action of the resultant moves towards the center of the section and consequently the factor of safety increases considerably. The stress at the toe is unchanged by virtue of the fact that the assumed uplift pressure distribution has a zero value at the toe, however the vertical stress at the heel increases by the amount by which the uplift decreases. There is thus a case for the introduction of a cut-off grout curtain, foundation drains and an upstream impervious facing to the embankment backed with an inclined blanket drain. This would substantially increase the factor of safety against overturning and would also permit a reduction in embankment volume. It is however questionable as to whether it would prove practical to reduce the section by a significant amount.



(a) Figure 8 (b)

Variation of Parameters with Uplift Coefficient ζ



(a) Figure 9 (b)

Variation of Parameters with Upstream slope x_u

4.2.7 Variation of parameters with upstream slope x_u (Figure 9)

The section considered thus far is one with equal upstream and downstream slopes, however, the lowest volume conventional gravity section is one with a vertical upstream face. The results depicted in Figure 9 are those of an analysis in which the embankment volume and base width were kept constant by maintaining the average of the upstream and downstream slopes equal to the slope x considered in the previous analysis. It is interesting to note that although increasing the upstream slope to vertical causes the line of action of the resultant to move towards the midpoint of the base, the factor of safety is not significantly affected. The increase in upstream slope causes a decrease in the weight of water over the upstream face and hence an increase in the ratio of horizontal to vertical forces which indicates an increase in required shear strength. At the same time however, the change in slope causes a reduction in toe stress and an increase in heel stress giving a nearly rectangular pressure distribution at the base and thus a more uniform development of frictional shear strength over the base. There are thus a number of factors favouring an increase in the upstream slope of the dam. It is unlikely that a significant reduction in volume could be effected however, as this would require a steepening of the downstream slope and consequently a reduction in safety factor.

4.3 Determination Of Strength Of Stabilized Soil Required for Gravity Section

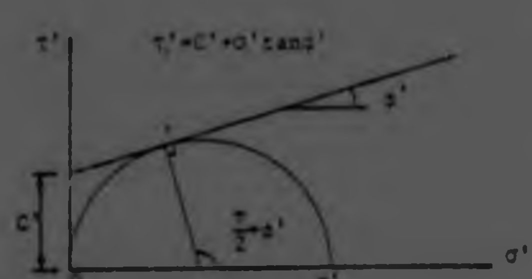
For use in this analysis, the strength of stabilized soil has been defined in terms of a Mohr-Coulomb failure envelope, as shown in Figure 10, from which it can be shown that:

$$C' = 0{,}5\ \sigma_u' \left[\cos \phi' - \tan \phi' (1 - \sin \phi')\right] \tag{1}$$

Since most published results on stabilized soil to date have been expressed in terms of the unconfined compressive strength σ_u', the assumed failure envelope and hence equation 1 can be used to estimate the cohesion of the material. The stabilization of soil with lime or cement is generally accepted as resulting in an increase in cohesion with little change in angle of friction except at low confining pressures (Rocha et al 1961, Nash et al 1965; Raphael 1970). The results of Nussbaum & Colley (1971) however, reflect an increase in angle of friction with increasing cement content but no information is given concerning confining pressures. There is a definite need for further research in this field. For the purpose of this paper it has been assumed that no significant change in angle of friction occurs and that 30° is a reasonably representative value thereof. With $\phi' = 30°$, equation 1 simplifies to:

$$C' = 0{,}289\ \sigma_u' \tag{2a}$$

$$\text{or}\quad \sigma_u' = 3{,}46\ C' \tag{2b}$$



σ_u' = unconfined compressive strength

τ_f' = shear strength at failure

ϕ' = angle of friction

C' = cohesion

Figure 10

Assumed Mohr-Coulomb Failure Envelope

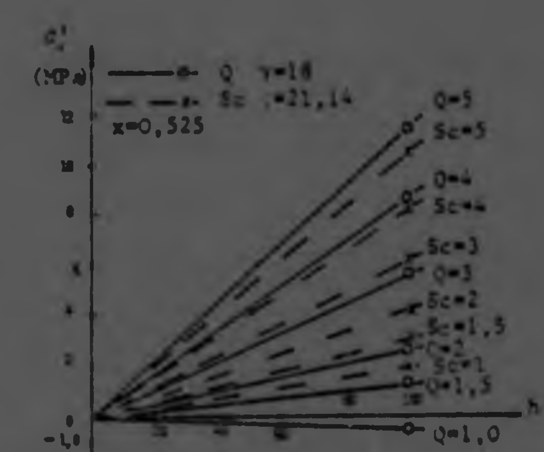


Figure 11

Variation of Unconfined Compressive Strength with Height and Safety Factor-Gravity Analysis

4.3.1 Strength required for gravity stability

The normally specified strength requirement for a concrete gravity dam is that the maximum compressive stress in the concrete should not exceed 25 per cent of the 28 day unconfined compressive strength i.e. the factor of safety against overstressing Sc = 4. Whether such a stringent requirement is necessary with stabilized soil where considerable strength gains are experienced from 28 to 90 days and beyond, is open to question. It appears that the strength requirements for concrete dams have been developed largely on an empirical basis related to past practice. Figure 11 shows the unconfined compressive strengths required for various embankment heights for the typical sections considered and for various factors of safety (Sc) applied to the calculated stresses. The values plotted have been determined from a consideration of the unit weight of the material as determined for the slope stability analysis (section 4.4 below).

4.3.2 Strength required to resist sliding on base

Two approaches are normally adopted to assess the resistance of a gravity dam to sliding. The simplest approach is to consider the 'sliding factor' which is the ratio of horizontal to vertical forces, that is, the parameter μ considered previously. This ratio is generally required to lie in the range of 0.65 to 0.75 for concrete on rock (Thomas 1976; Justin et al 1945). Justin, et al (p 295) specify that this coefficient should be less than or equal to the coefficient of friction "indicated by experiments on well dressed specimens of the same materials". It will be seen that with the exception of those embankments with steep upstream slopes (Fig. 9b) virtually all sections considered have required values of μ below the above limits. However, the value of $\mu = 0{,}65$ corresponds to a value of ϕ of 33° which is greater than the value of 30° assumed for stabilized soil. The simplified analysis can therefore not be applied and it is necessary for this exercise to consider cohesion as well. The alternative approach is to consider the shear friction factor of safety Q which is defined as:

$$Q = (Fv \tan \phi' + r\, b\, C')/Fh \qquad (3)$$

where Fv, Fh and b are as defined in Figure 2, C' and ϕ' are defined previously and r is a factor to take account of the non-uniform pressure on the base and is commonly taken as 0,5 (Thomas 1976; Justin et al 1945).

The above equation is commonly applied to concrete dams with a value of Q not less than 5. However, the same problem may be approached from a soil mechanics view-point using the Mohr-Coulomb failure criterion: Considering the average normal stress on the plane $\sigma' = Fv/b$ and applying a factor of 0,5 to C' as discussed above, the average shear stress at failure is:

$$\tau' = 0{,}5\, C' + Fv/b \tan \phi' \qquad (4)$$

The average shear stress on the plane is Fh/b and applying a factor of safety of 2 to give the allowable shear stress gives:

$$2\, Fh/b = 0{,}5\, C' + Fv/b \tan \phi' \qquad (5)$$

rearranging

$$2 = (Fv \tan \phi' + 0{,}5\, C'b)\ Fh \qquad (6)$$

The right-hand side of equation 6 is identical to that of equation 3 with $r = 0{,}5$ and gives a value of Q equal to 2. It is the writers' opinion that a value of 5 is unduly conservative particularly when based on the 28 day strength of a stabilized soil. Equation 3 was rearranged and combined with equation 2b to permit direct calculation of σ_u' for different sections and different values of Q. The results are plotted in Figure 11 for an analysis using full uplift which gives a conservative assessment.

It will be noted that the above analysis implicitly assumes that the shear strength on construction planes is equal to that of the intact material. Inter-layer bond in stabilized soil is not a completely known quantity. The report on bonding of soil-cement by De Groot will hopefully clarify the matter to some extent. It appears, however, that USBR methods of placing soil-cement slope protection have largely overcome problems of interlayer bond. Layers of soil-cement are brushed with a mechanical steel broom immediately after the 'initial set' has occurred and again immediately before placing the next layer of soil-cement. Brooming "leaves a striated surface with narrow grooves approximately 1/4 inch (0,64 cm) deep and approximately 1/2 inch (1,27 cm) apart" report Davis et al 1973. It seems likely that a surface such as that described above would provide excellent resistance to horizontal shear especially if the grooving runs along the length of the embankment. Because of the uncertainty regarding interlayer bond, initial recommendations regarding strength have been based on a value of Q equal to 5. It is however felt that once more information is available on bond a less conservative approach will be possible. The use in design of values obtained from direct shear tests on construction joints is obviously desirable.

4.4 Slope Stability Analysis

The initial investigation of slope stability was limited to confirming Raphael's (1976) finding that for slopes steeper than 0,8:1 the gravity analysis is critical. This was necessary in order to determine the maximum material strengths required for the 'critical gravity section' discussed previously. This investigation was restricted to the typical section presented in Table I, considering only material with a dry unit weight of 18 kN/m³. Sections varying in height from 20 to 100 m were analysed.

As noted in section 2.1, the development of excess pore pressure in the embankment during construction is not anticipated. Consequently the 'end of construction' stability analysis will not be critical. As the section proposed has equal upstream and downstream slopes, it is to be expected that the 'rapid drawdown' analysis of the upstream slope will be more severe than the 'steady seepage' analysis of the downstream slope. The 'steady seepage reservoir full' flow-net was used to determine the pore-pressures on the slip circle. For programming purposes, this was approximated by assuming the phreatic surface to be a plane joining the intersection of the water surface and the upstream face to the intersection of the Kozeny parabola with the downstream face. The foundation was assumed to be impermeable and equipotential lines were approximated by circles centred on the intersection of the assumed phreatic plane and the base. This satisfies the requirements for orthogonality at the boundaries while the 'equipotentials' have less curvature than those of a true flow net, resulting in an overestimate of pore pressures.

In estimating the mass of the sliding material, the saturated unit weight was used for all material below the seepage surface and the bulk or 'as placed' unit weight was used for material above this surface. The unit weights were estimated on the basis of a particle specific gravity of 2,7 and 5 per cent air voids. The value of 5 per cent air voids corresponds to a degree of compaction close to the maximum for most soils subject to standard proctor compactive effort (Scott 1974).

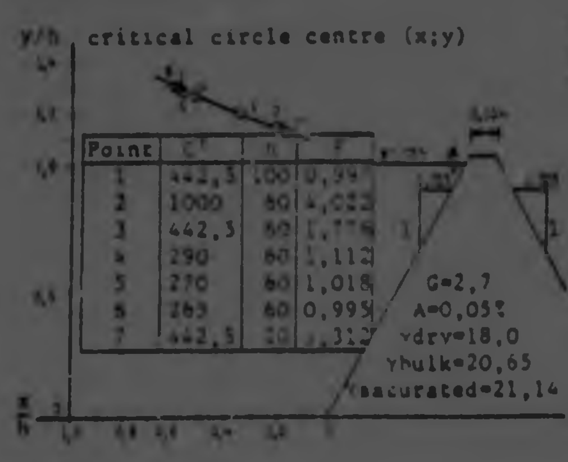


Figure 12

Locus of centres of Critical Circles For Typical Section (Table I x=0,325)

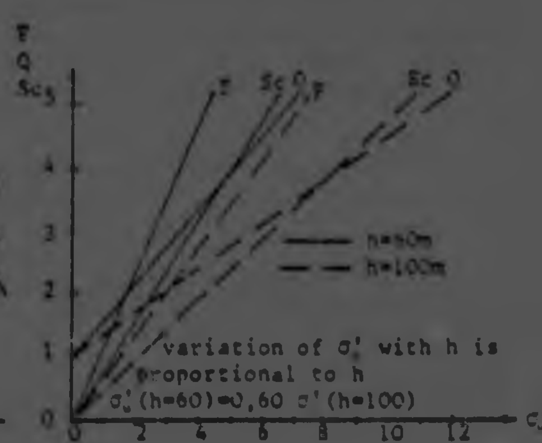


Figure 13

Variation of safety factors with c_u' and height

In view of the requirement of a relatively strong foundation for the stabilized soil dam, it was decided to assume that the foundation strength was considerably greater than that of the embankment material. With these assumptions a number of trial circles confirmed that the critical case for analysis is 'complete instantaneous drawdown' with the critical slip circle passing through the heel of the embankment. The analysis used was the simplified slip circle analysis of Bishop (1955). In contrast to results for flatter slopes, it was found that the simplified analysis gave slightly unconservative results when compared with more rigorous methods of analysis. To date, the analysis has been restricted to determining the critical circle for various combinations of height and cohesion. It appears that the locus of the centre of the critical circle for constant height with varying cohesion is the same as that for constant cohesion with varying height (Figure 12). In addition, the factor of safety for a specific circle appears to be independent of height and cohesion. Assuming this to be the case, a tentative procedure has been developed which enables the required cohesion to be determined for a specified reservoir depth and factor of safety on the critical circle. Further research is, however, required to determine whether these results are of general applicability.

The values of cohesion and hence, unconfined compressive strength, required to provide a specified factor of safety F against failure on the critical slip circle for embankments of heights 60 and 100 m were determined for comparison with the results of the gravity analysis (Figure 13).

5. COMPARISON OF RESULTS

Figure 13 illustrates the variation of required unconfined compressive strength with the factors of safety with respect to overstressing (Sc), sliding (Q) and circular arc failure (F). If these results are examined in the light of the values customarily assigned to the factors (Q = 5, Sc = 4 and F = 3) it is seen that the requirement for sliding stability is by far the most onerous condition for the section considered. If the results are compared on the basis of equal numerical values for safety factors, it is seen that for the section considered sliding stability (Q) is the more severe criterion for factors in excess of 3,75 and that overstressing (Sc) controls for factors less than this. Slope stability is not critical in this case (in agreement with Raphael's conclusion). The intercept of the sliding stability curve of Q = 1 at zero cohesion corresponds to the frictional component of shear strength and agrees with a value of $\mu = 0.57$ which corresponds to $\phi' = 30°$. The zero intercept of the slope stability curve at zero cohesion is in agreement with the method of analysis used which is essentially a "$\phi = 0$" analysis.

5.1 Application Of Results

To examine the viability of the stabilized soil dam, and particularly the soil-cement dam, in the light of the above results, it was decided to examine the material used by the USBR in the projects discussed by De Groot (1971b). Using the 28 day strengths actually obtained in construction, with Q = 5 and referring to Figure 11, the maximum permissible height for the material density concerned was estimated and, where possible, compared with the height of the actual structure. The results are shown in Table II.

It should be noted that the foundation conditions at these sites were not necessarily suitable for a soil-cement embankment.

TABLE II
HEIGHTS OF SOME EQUIVALENT SOIL-CEMENT EMBANKMENTS

	Cement Content	Dry Unit Weight	Average 28 day Strength	Maximum Soil-Cement Height	Actual Height
	% of Dry Mass	kN/m³	MPa	m	m
Merritt Dam	14	17,0-18,9	12,51	110	38
Cheney Dam	12	18,6-19,6	10,31	90	
Lubbock Regulating Reservoir	12	18,1-18,5	7,82	68	
Glen Elder Dam	12	18,4	8,28	72	27,4
Downs Dike	12	19,3-20,0	7,38	64	35
Cawker City Dike	12	19,1	8,87	78	
Starvation Dam	12	18,6	6,24	55	

circle appears to be independent of height and cohesion. Assuming this to be the case, a tentative procedure has been developed which enables the required cohesion to be determined for a specified reservoir depth and factor of safety on the critical circle. Further research is, however, required to determine whether these results are of general applicability.

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Figure 13 illustrates the variation of required unconfined compressive strength with the factors of safety with respect to overstressing (Sc), sliding (Q) and circular arc failure (F). If these results are examined in the light of the values customarily assigned to the factors ($Q = 3$, $Sc = 4$ and $F = 3$) it is seen that the requirement for sliding stability is by far the most onerous condition for the section considered. If the results are compared on the basis of equal numerical values for safety factors, it is seen that for the section considered sliding stability (Q) is the more severe criterion for factors in excess of 3,75 and that overstressing (Sc) controls for factors less than this. Slope stability is not critical in this case (in agreement with Raphael's conclusion). The intercept of the sliding stability curve of $Q = 1$ at zero cohesion corresponds to the frictional component of shear strength and agrees with a value of $\mu = 0.57$ which corresponds to $\phi' = 30°$. The zero intercept of the slope stability curve at zero cohesion is in agreement with the method of analysis used which is essentially a "$\phi = 0$" analysis.

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Glen Elder Dam	12	1[illegible],4	8,28	72	27,4
Downs Dike	12	19,[illegible] 20,0	7,38	64	35
Cawker City Dike	12	19 1	8,87	78	
Starvation Dam	12	18,6	6,24	55	

5.2 Cost Comparison For Merritt Dam

To examine the potential cost saving, an estimate for Merritt Dam was prepared.

It should be noted that the foundation conditions were not suitable for a soil-cement dam and hence the cost comparison is hypothetical. The assumed cement content is the same as that used for slope protection, which provides strength far in excess of that required. The details of Merritt Dam in Table III were obtained from De Groot (1971b), the ICOLD World Register of Dams (to 31 December 1968), Holtz & Walker (1962), Holtz & Hansen (1976) and the Portland Cement Association report "Soil-Cement for Water Control Structures" (1976a). Cost data for soil-cement were obtained from the PCA "Guide for Estimating Cost of Soil-Cement Slope Protection (for Construction in 1978)". Processing and placing costs would probably be lower for an embankment.

TABLE III
DATA FOR MERRITT DAM COST COMPARISON

Height above foundation	38 m		
Crest length	982 m		
Embankment volume	1 184 500 m³		
Slopes	4:1 & 3,5:1		
Slope protection volume	39 000 m³		
Crest width assumed	10 m		
Cost of cement		US $5.71/100 kg	1978 prices
Processing cost of soil-cement	<76 000 m³	US $9.48/m³	
	>76 000 m³	US $8.83/m³	

CALCULATION FOR COST COMPARISON

Dry density of 1800kg/m³ for soil cement gives slopes of 0,525:1 for a soil-cement dam.
Reduction in cross-section* gives volume of soil-cement embankment as 20% of that of earth embankment = 232400 m³
Cost of cement in soil-cement = 1800x0,14x0,01x5.71 = US$14.39/m³
Total cost of soil-cement embankment = US$23.22/m³
Total cost of soil-cement in slope protection = US$23.87/m³
Assume addition of cement increases processing cost of soil by 50%, this indicates processing cost of unstabilized soil = 2/3 x 8.83 = US$5.89/m³
Cost of soil-cement embankment = US$4.80 x 10⁶
Cost of earth embankment = US$6.97 x 10⁶
Cost of soil-cement slope protection = US$0.93 x 10⁶
Total cost of unstabilized embankment = US$7.90 x 10⁶
Saving with soil-cement embankment = US$2.50 x 10⁶
= 32% of earth embankment cost

* Assumed uniform cross-section throughout - only height varies

The above estimate indicates that, at todays prices, a considerable saving could be effected on a dam such as Merritt. It is recognized that the cost of processing unstabilized material could be considerably less than the value of $5.89/m³ estimated but this compares favourably with the range of $3.27 to $6.54/m³ estimated by Raphael (1976) as the range of earthmoving costs indexed to January 1976. The escalation in PCA's estimated processing costs 1976 to 1978 is 22.7 per cent. Applying this to Raphael's costs gives a range of $4.01 to $8.03/m³ for 1978 costs. Using the minimum value of $4.01/m³ gives an unstabilized embankment cost of US$5.68 x 10^6 making the soil-cement embankment 15% cheaper. It should be noted that the above figures do not include the secondary benefits discussed previously. Also, the cement content within the embankment could probably be reduced to that determined from strength requirements. Noting the 22.7 per cent escalation mentioned above, it is apparent that the reduction in construction time could also bring about a significant cost saving.

3.2 ELABORATION ON CONTENTS OF PAPER

A number of points relevant to the above discussion are presented below.

a. The gravity analysis is based on a uniform unit weight throughout the embankment. In design, it will be necessary to assume one unit weight, possibly the dry unit weight, for determination of overturning stability and another, possibly the saturated unit weight, for determination of the maximum principal stress at the toe. These unit weights might be those applicable at an age of 28 or 90 days to take account of hydration.

b. Both analyses assume the development of seepage pressure (uplift) across the width of the embankment. In practice, it might prove desirable to reduce these pressures by provision of filter drains, possibly of no-fines concrete, or drainage galleries. However, it is felt that these refinements will complicate construction and that the savings from reduced volume may well be completely offset by the increased cost of installing them.

   The decision to construct Willow Creek dam with only a gallery for inspection and relief drilling if required (Schrader 1982), supports this view Note that the Willow Creek, downstream slope of 0.8 to 1 with vertical upstream face agrees well with the range of values examined above when allowance is made for the higher unit weight of the material that will result from the high proportion of crushed rock in the mix.

c. Schrader (1982) found that the material as placed rested at a repose slope of 0.73:1 and that the upstream facing could be constructed by means of panels held in place using reinforced-earth techniques. He also found that the panels used could be lighter than normal and that the required anchorage length was considerably reduced. These findings, taken in conjunction with the discussion in section 4.2.7 above, suggest that further consideration should be given to the use of a vertical upstream slope for the stabilized soil dam.

d. After completion of the above paper, a copy of the United States Bureau of Reclamation manual "Design of Gravity Dams" (USBR 1976a) was obtained. This recommends somewhat less conservative values for factors of safety with regard to compressive strength and sliding stability.

The recommended safety factor with regard to compressive stress, Sc above, is specified as 3.0 for "Usual Loading Combinations", 2.0 for "Unusual Loading Combinations" and 1.0 for "Extreme Loading Combinations", this compares with the value of 4 referred to in section 4.3.1 above. The maximum permissible compressive stress is limited to 1500 pounds per square inch (10.3 MPa) under usual loading with corresponding maxima under other loading conditions. This indicates a maximum required compressive strength of material for any gravity structure of 31 MPa.

The shear friction factor of safety Q is computed by a formula which, when converted to the same variables as used above, is identical to equation 3 with a value of r = 1.0 as opposed to the value of 0.5 discussed previously. This takes account of the full cohesive component of the shear strength of the material and has the effect of reducing the required unconfined compressive strength for sliding stability. This reduces the slope of the Q lines in figure 11 to a point where they lie below the Sc lines for the range of values considered in that figure. Likewise, the slope of the Q lines in figure 13 increases, bringing them above the Sc lines for all values. It should be noted that the USBR analysis is based on gravity sections with a vertical upstream face. As shown in figure 9, this leads to a considerably more uniform vertical stress distribution than occurs in the embankment with equal up- and downstream slopes.

In addition, the values of Q specified are 3.0, 2.0 and 1.0 respectively for the usual, unusual and extreme loading conditions. This compares with the value of 5 referred to in section 4.3.2 above and supports the contention in that section that Q = 5 is excessive. On this basis, the most severe strength requirement for the embankment with equal slopes is always the compressive strength on the downstream face of the embankment. Sliding stability may control in the body of the embankment, and under certain circumstances it is possible that stability of the upstream slope may be critical. Other factors, particularly durability and permeability considerations, may however indicate greater cement contents than those required to satisfy structural requirements.

CHAPTER 4

SCOPE OF LABORATORY PROGRAM

[illegible] and Blight (1978) concluded:-

> "In view of the theoretical cost saving shown for Merritt Dam despite a conservative evaluation, it is concluded that there is a strong case to be made for stabilized soil dams and in particular soil-cement dams. While accepting the need for further investigation of certain properties to permit a less conservative and hence more economical design, it is the writers' opinion that there is sufficient information to justify the construction of a relatively large trial structure which will permit a full examination of the performance of the stabilized embankment. Construction of such a structure would revolutionize dam design and construction in much the same way that the Bonny Reservoir Test Section revolutionized slope protection. Investment in such a structure would be amply repaid."

The paper went on to summarize those areas requiring further research as follows:-

> "...... there is a need for further investigation of certain of the properties of stabilized soil for use in dam construction. It is to be expected that initially, most attention would be focused on soil-cement as its durability is particularly well established. Once the concept of the soil-cement dam is put into practice, considerable effort should be expended in investigating the use of soil stabilized with lime and other stabilizing agents such as lime-flyash-gypsum. The following topics are in particular need of investigation:-
>
> The effects of long term seepage, including seepage

through intact material and along compaction planes and shrinkage cracks.

- Shrinkage and swelling of material during hydration and pozzolanic reaction and due to change of moisture content.
- Heat generation related to hydration and pozzolanic reactions during the early life of the structure.
- Flexibility, strength and deformation characteristics of stabilized material -- particularly under high confining pressures."

On the basis of the above statements, it was decided to limit this research to soils stabilized with Ordinary Portland Cement. The laboratory program described in this thesis was based on the results of the analysis presented in chapter 3 and the conclusions presented above. The bulk of the laboratory tests were conducted during 1980. The following investigations were conducted:-

1. Wetting and drying durability tests to determine the cement requirements for use in slope protection in accordance with the PCA (1976c) criteria. For preliminary design of experiments and for use in design. In the second stage of this program the standard test sequence of twelve wet-dry cycles was extended to an additional series of twelve cycles with the objective of providing durability data for comparison with the results of the leaching tests.

2. An investigation into the behaviour of soil-cement subject to percolation with a view to determining whether leaching could be a problem. It was felt that while experience in the United States (De Groot 1971a, PCA 1970, Hansen 1979a and others) indicated that leaching was not a problem, the results of Jackson (1965), although preliminary (Nash 1978), could not be ignored. In addition, leaching was a concern frequently voiced during informal discussion with practising engineers during the early stages of the research.

   In order to examine as large a range of variables as possible, particular attention was given to designing a testing facility that could be built at minimum cost in order to permit the simultaneous testing of the greatest number of specimens for the longest possible period.

3. Unconfined compression tests to supply initial data for design of experiments and to provide data for comparison with the results of other research, most of which is related to unconfined compressive strength.

4. A limited program of cylinder splitting (brazilian) tests in conjunction with the unconfined compression tests to provide comparative data.

5. Consolidated undrained triaxial compression tests on leached specimens as well as on unleached control specimens subject to various curing conditions. To obtain maximum information from the leaching tests and to supply strength and elastic data.

6. In conjunction with the above test programs, measurements of specimen dimensions at various stages of curing and testing to provide information on volumetric change of the material under various conditions. A more detailed investigation using purpose-made specimens with suitable measuring targets was not undertaken due to delays in other test programs.

7. Observations of specimen mass at all stages of curing and testing to monitor control of the various procedures and to examine the effect of variations in density and moisture content on the results in other parts of the test program. This information was for use in assessing the susceptibility of the design data to variations in compaction control.

8. Large scale direct shear tests with particular reference to the behaviour on construction joints with a view to ensuring integrity of the joints. The equipment used permitted the testing of specimens up to 358 mm square and 260 mm deep thus

permitting the testing of material containing the coarse fraction normally excluded in laboratory tests.

9. The settlement characteristics of the material when subject to various rates of loading and when subject to repeated loading and unloading cycles analogous to changes in reservoir level.

10. A limited investigation into the heat generated during the initial hydration of the cement and subsequent secondary hydration or pozzolanic reaction, particularly on the penetration of a wetting front into the material.

CHAPTER 5

CHOICE OF SITE FOR INVESTIGATION

In view of the extent of the proposed laboratory program, the number of possible variables to be examined and the available equipment and facilities, it was decided to restrict the laboratory program to soil from one location. It was expected that the results of this test program would identify the test parameters of importance so that subsequent research could concentrate on applying these parameters to a representative sample of soils and stabilizers.

Since a primary objective of the research was to compare the soil-cement dam with conventional solutions, it was considered desirable to obtain material from a site at which concrete, earth and soil-cement embankments could be constructed. A site at which a dam had been, or was being, constructed was obviously preferable.

With the assistance of Dr. D. Stephenson of Stewart, Sviridov and Oliver, Consulting Engineers, and after consultation with Prof Blight, it was decided to use the **Rondebosch Dam**, then under construction outside Middelburg, Transvaal, as the case for comparison. The dam was constructed as a concrete buttress structure but an alternative tender was submitted on the basis of an earth embankment designed by B.G.A. Lund and Partner, Consulting Engineers (1974).

This site was chosen for the following reasons:-

a. In view of the alternative designs considered, detailed information was available concerning the availability of the

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This site was chosen for the following reasons:-

a. In view of the alternative designs considered, detailed information was available concerning the availability of the basic materials for all three types of construction viz concrete aggregate and soils. Information regarding the soil types available included basic engineering properties, estimated quantities and haul distances.

b. As the foundation conditions were suitable for a concrete dam, they would also be suitable for a soil-cement embankment.

c. Construction would be completed during the research period and it was therefore expected that up to date information on costs would be available if required for comparison.

d. Reservoir filling had not commenced at the time the decision was taken and as the contractors were still on site there would be no difficulty in obtaining representative soil samples.

5.1 SOIL ON SITE

The site is located on Felsite rock of the Rooiberg Group of the Bushveld Complex, east of Middelburg, Transvaal.

Felsite is a fine-grained, cryptocrystalline, igneous rock with composition similar to that of granite (Truswell 1970, Brink 1979). It is extremely resistant to chemical decomposition and residual soils are generally less than 1.5 to 2 m in thickness. Initial excavation of the residual soil to a depth of 1.8 m on the site of the buttress dam was by means of a front-end loader (Brink 1979). While the occurrence of Felsite is fairly limited, it was felt that its mineralogical similarity to granite would give the results wider application in general terms.

Lund (1974) estimated available soil resources for the alternative earthfill embankment at various distances from the site as follows:-

Less than 1525 m	150 000 m^3
1525 to 1830 m	190 000 m^3
1830 to 2135 m	340 000 m^3
2135 to 2440 m	450 000 m^3

The soil closer to the site was relatively shallow and sandy whereas that further away was deeper and more plastic.

Crushed Felsite rock was used as aggregate for the concrete dam constructed on the site. A characteristic of felsite aggregate is its extreme abrasiveness. Brink (1979) notes that "all plant in direct contact with the crusher-run, particularly concrete mixing plant, suffers wear at an unusually rapid rate". This is of relevance in the context of the obser[illegible]ions in section 1.2.

5.2 DETAILS OF DAM

The buttress dam as constructed was 34 m high with a crest length of 620 m (Brink 1979).

The alternative earthfill dam proposed by Lund (1974) was designed for a full supply depth of 35.4 m with freeboard 5.5 m. Freeboard at design flood of 2830 cumecs, was 1.4 m. Crest length including side spillway of 183 m was 720 m. Slopes of 2.5:1 and 2:1 up- and downstream were proposed giving an embankment volume of 890 000 m^3 with 84 000 m^3 of rock rip-rap, mostly produced from the spillway excavation, being required. The volume of earthfill required was such that in view of the haul distances indicated above, scraper operation was not considered to be economic.

5.3 WATER ON SITE

The Klein Olifants River is impounded by the Rondebosch Dam and collects in a catchment of approximately 1600 km^2, located largely on Felsite. The water is extremely pure, total solids content being less than 150 parts per million and pH less than 7.1 (Bond 1946). This water can therefore be considered to be relatively corrosive to concrete in terms of its ability to dissolve solids. Observation of the spillway on the original dam at the site revealed that corrosion of the surface had in fact taken place to a noticeable extent although not sufficiently to seriously impair operation.

5.4 COLLECTION OF SOIL SAMPLES

From an examination of the drawings for the earthfill dam (Lund 1974), the original trial holes from which samples were taken were identified during a site visit. On the basis of Lund's data, two holes were identified which were considered to represent the two soils on the site. Approximate haul distances from the holes were 2000 and 600 m respectively. Permission was obtained from the contractor and from the farmer, on whose land certain of the holes were located, to excavate material and a total of 750 kg of material was obtained for use in the preliminary tests.

CHAPTER 6

PRELIMINARY LABORATORY TESTS

6.1 SOIL CLASSIFICATION

As noted in chapter five, the soil in the available borrow areas could be visually separated into two groups related to the degree of weathering and hence depth of soil. The red soil furthest from the dam site was collected first and was thus labelled as "Soil 1", the yellow soil was labelled as "Soil 2".

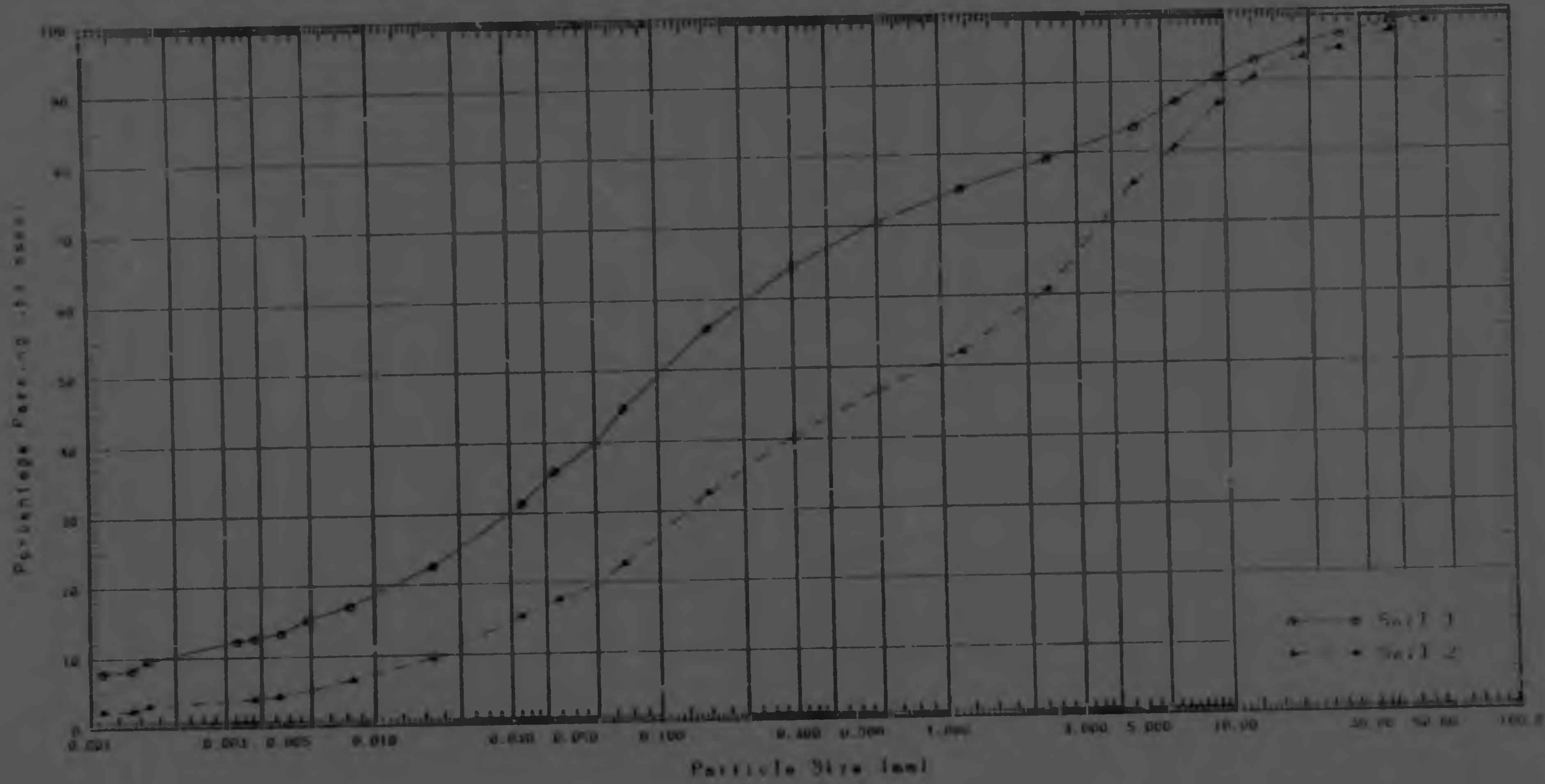
The soil samples were air dried, thoroughly mixed and sieved on a 4.75 mm (No 4) sieve. The coarse fraction was bagged and the fine fraction divided by riffling to approximately 2 kg batches which were stored in sealed polythene bags after moisture content samples had been taken.

The standard indicator tests were performed in accordance with BS 1377 (1975) with results as follows:-

	Soil 1	Soil 2
Liquid Limit	28.7%	23.8%
Plasticity Index	10.0%	5.3%
Linear Shrinkage	6.5%	3.7%
Specific Gravity	2.7	2.7

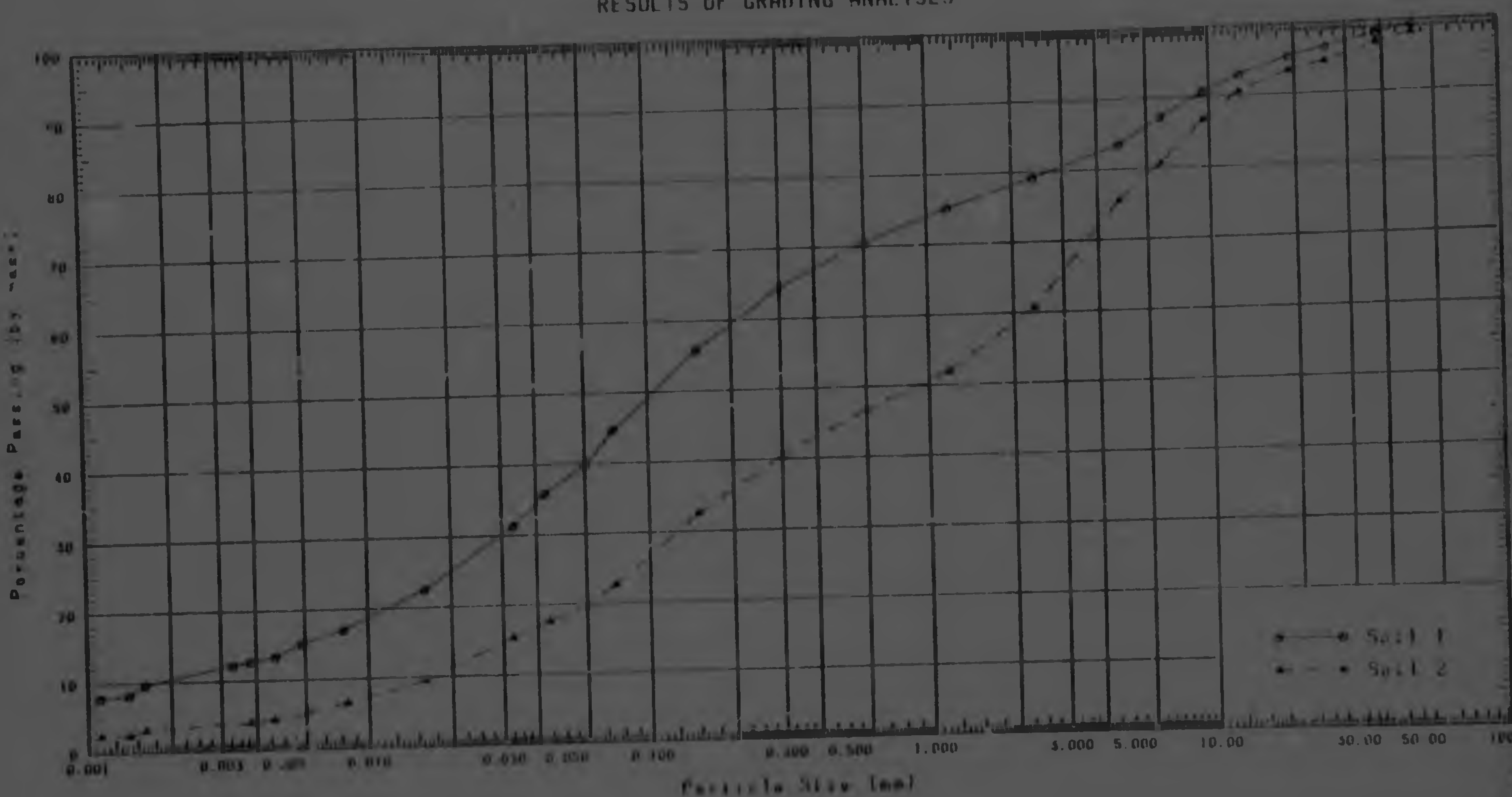
Wet grading and hydrometer analyses were performed and the results are reflected in graph 6.1. These results were all in good agreement with those reported by Lund (1974).

GRAPH 6.1
RESULTS OF GRADING ANALYSES



GRAPH 6.1

RESULTS OF GRADING ANALYSES



GRADING1 PLT 27/12/85 [illegible]

On the basis of these results, the soils were classified as A-4 and A-2-4 according to the ASTM classification system (SC and SC-SM according to AASHO).

Soil colours according to the Burland colour disk were:

Soil 1 -- dark reddish orange to dark red when dry; dusky red in paste.

Soil 2 -- dark yellowish orange when dry; dark brown to dark yellow in paste.

6.2 EFFECTS OF REPEATED COMPACTION OF SOIL

In order to assess the resistance of the material to degradation on handling and to evaluate the proposed sample preparation techniques, a series of compaction and grading analyses were undertaken.

As greater quantities of soil 1 had been obtained and as it was the more weathered material, these tests were limited to this soil. Samples were compacted at "Standard Proctor" compactive effort in accordance with ASTM D698-70. In addition to compaction at various moisture contents, the material was recompacted up to seven times and samples taken for wet grading.

Maximum dry density (MDD) was determined as 1794 kg/m^3 with optimum moisture content (OMC) of 16.2%. Air void ratio was 5.1% with degree of saturation 85.0%. No significant change in moisture-density characteristics occurred with repeated compaction and all results were located in the scatter band indicated by a detailed error analysis in accordance with the techniques suggested by Brinkworth (1968).

A detailed statistical analysis of the grading results indicated that no particle degradation occurred on compaction. A greater spread of results was obtained for the compacted samples. Other conclusions from this analysis were as follows:-

a. The technique used for mixing soil and water involved the use of a standard domestic food mixer with dough hook. This hook was angled at the bottom, thus forcing material to the bottom of the mixing bowl. This formed a cake of soil at the bottom of the bowl and led to significant degradation of the material by the resulting grinding action. The mixing hook produced to overcome this is shown in plate 6.1.

PLATE 6.1

MIXING HOOK USED WITH DOMESTIC FOOD MIXER

b. Material was dispensed into the scale pan for initial batching and for allocating material for compaction by pouring from the polythene bags in which it was stored. This led to significant segregation with the first specimens prepared from a particular bag containing significantly more coarse material. Scoops were obtained and used for all subsequent material handling.

A detailed statistical analysis of the observed variations in specimen moisture content was also conducted. This indicated that significant variations in the air dry moisture content of the material was occurring and that loss of moisture on handling prior to compaction was of the order of 0.13%. The low moisture loss on handling was achieved by keeping mixed material in closed polythene bags and covering all containers when mixing or batching was not taking place. These precautions were adopted in all elements of the test program.

6.3 COMPARISON OF COMPACTION TECHNIQUES

After consideration of the proposed tests to be conducted, it was decided to use cylindrical specimens 75 mm long and 37.5 mm in diameter as the standard. These dimensions were determined by the available triaxial cells, but the specimens were also suitable for use in the majority of proposed test programs. A mould was fabricated in accordance with BS 1924 (1975) with the dimensions adjusted accordingly.

A series of tests were conducted to evaluate different methods of specimen compaction. Compaction methods evaluated included static compaction in a compression machine in accordance with BS 1924, static compaction in a number of layers and dynamic compaction using a small impact rammer (Dietert type) as shown in plate 6.2.

Calibration of the impact rammer indicated a compactive effort of five blows per layer with placement in three layers as giving the closest approximation to the standard proctor moisture density curve. However the curve peaked at an air void ratio of approximately 4% as opposed to 5.1% for proctor compaction, leading to a somewhat higher indicated optimum moisture content. It was also found that the shape of the moisture density curve produced with this apparatus was sensitive to rate of application of compactive effort, a much flatter curve being obtained with a rapid application.

Density variation tests were performed by dividing the specimens longitudinally into six equal portions by means of polythene disks. Three specimens were prepared for each compaction method examined and specimens without disks were moulded for control purposes. Volumetric measurements were made using a custom-built mercury displacement cell. Volumetric shrinkage, moisture content and dry densi y for each specimen were compared.

PLATE 6.2

IMPACT RAMMER AND MOULD USED IN PRELIMINARY TESTS

Compaction in accordance with BS 1924 produced the narrowest scatter of results and was also the quickest and most convenient method of compaction. It was however found necessary to reduce the height of the spacer collar in order to ensure equal plug projection before compaction. This technique was used for the compaction of all standard cylindrical specimens and was adapted for use in the majority of other test programs. The only specimens in the main test program not prepared using static compaction were those for the wetting and drying tests, where the standard requires compaction using the standard proctor apparatus, and the heat of hydration tests where the number of specimens did not warrant the fabrication of special apparatus and where standard AASHO compactive effort could be used.

The moisture content observations for these tests also indicated a significant variation in air-dry moisture content for the stored material. In order to reduce these variations, a discarded oven was rebuilt and converted by the installation of a fan for air circulation to permit drying of all soil at 50°C. All soil used in the main test program, with the exception of the large quantities required for the direct shear tests, was dried in this oven to constant mass and then stored in sealed polythene bags placed in sealed drums. A record was kept of bag masses and it was found that no significant absorption of moisture occurred in the material. This, together with the previously mentioned precautions during handling of the material led to close control of moisture content throughout the main test program.

6.4 WETTING AND DRYING DURABILITY TESTS

Wetting and drying durability tests were conducted on soils 1 and 2 to determine their suitability for slope protection and as an indication of which soil should be used in the main test program.

The tests were performed in accordance with ASTM D559 as described in the PCA booklet "Soil-Cement for Water Control:

Laboratory Tests" (PCA 1976c). Since the leaching and other tests were to be performed on samples containing only the soil fraction passing the 4.75 mm sieve, it was decided to conduct wet-dry tests both on specimens containing the fraction retained on this sieve and those excluding this fraction.

Standard Proctor compaction tests were conducted on all four soil types containing the cement contents indicated by the PCA booklet. For soil 1, tests were conducted at cement contents of 9% and 12% for the material passing the 4.75 mm sieve. The moisture - density curves for both cases were identical giving a maximum dry density of 1770 kg/m^3 and optimum moisture content of 17.2%. For soil 1 including the coarse fraction, tests were conducted at 7% and 9% cement giving MDD of 1828 and 1815 kg/m^3 and OMC of 15.3 and 16% respectively. In the case of soil 2, tests were conducted at cement contents of 3% and 7% for the fine material, again producing effectively identical moisture - density curves with MDD of [illegible] kg/m^3 and OMC of 14.8%. Tests were only conducted at a cement content of 6% for soil 2 including the coarse fraction, with MDD of 1886 kg/m^3 and OMC of 13.7%.

On the basis of these results, all specimens which included only the fraction passing the 4.75 mm sieve were moulded at the same nominal MDD and OMC for a particular soil type.

Cement contents for the wet-dry tests were estimated on the basis of the techniques presented in the PCA booklet and a total of 18 test and 10 control specimens were processed. The resulting mass loss data is reflected in graph 6.2.

Correction for cement water of hydration was performed on the basis of 1/4 of the cement content expressed as a percentage as indicated by the PCA. However, as seen in the graph, the control specimens reflected a mass loss with increasing cement content. A further correction was therefore applied by performing a linear regression on the combined data for the control specimens to reduce the original correction. It will be noted from the graph

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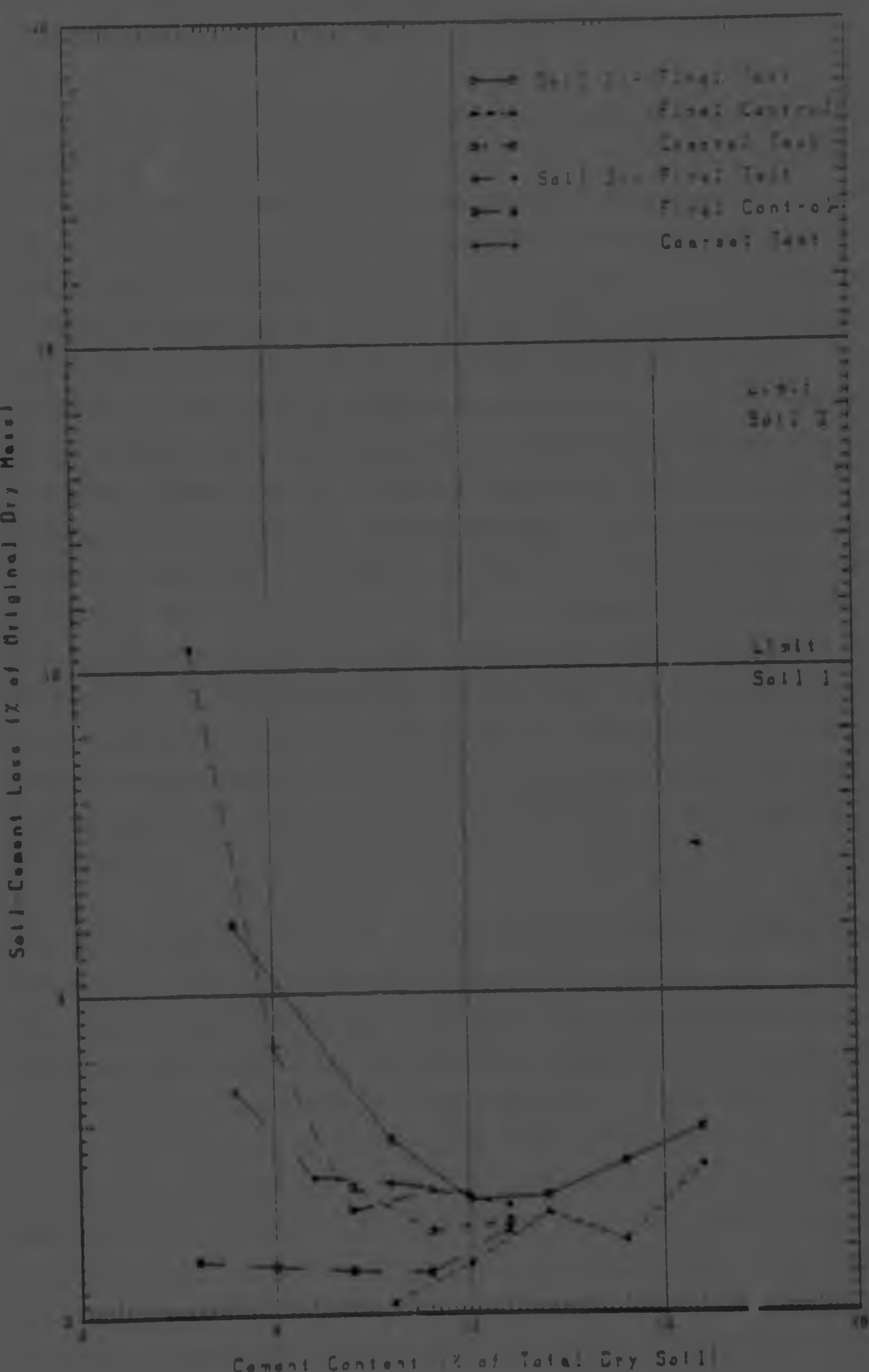
GRAPH 6.[illegible]

RESULTS OF PRELIMINARY WET-DRY TESTS

HYDRATION CORRECTION OF [illegible] CEMENT CONTENT APPLIED



[illegible].PLT — [illegible]

that the mass losses observed were considerably less than the maxima of 10 and 14% prescribed for the two soil types. This was interpreted as indicating that both soil types were well suited to stabilization with cement.

Observing that the form of the data was approximately hyperbolic and that the characteristics of a hyperbolic curve were consistent with the likely behaviour of soil-cement subject to the wet-dry test, hyperbolic regressions were performed on each group of data. Regression coefficients (r^2) in the range 0.89 to 0.98 were obtained together with a good visual fit. This was considered satisfactory in view of the limited number of results. These regressions were used to perform an extrapolation to estimate the minimum cement contents required to satisfy the maximum mass loss criteria. These cement contents were used in determining the range to be considered in subsequent tests. The estimated cement contents were 1.7% and 2.3% for soil 1 with and without coarse fraction and 1.0% and 2.3% for soil 2. It should be noted that the PCA recommend increasing the cement content determined from these tests by two percentage points where the material is to be used in slope protection within 1.5m of the water line. It should also be noted that 2 to 3% is generally regarded as the minimum cement content that can be consistently mixed under site conditions.

The presence of the coarse fraction provides a significant increase in the resistance of the material to abrasion in the wet-dry test. The results for the specimens containing the coarse fraction were adjusted to express the cement content in terms of the fraction passing the 4.75 mm sieve and it was found that the mass loss for these specimens was still lower than that for the specimens containing only the fine fraction. While the number of specimens was not sufficient to be able to attach a great degree of statistical significance to the observed difference, it is suggested that the greater abrasion resistance of the coarse particles provides a degree of protection to the matrix of the soil-cement.

On the basis of these preliminary tests, the following observations were made:-

a. Both soils, which represented the extremes of the material available on the Rondebosch Dam site, are suitable for stabilization with cement and could safely and economically be used for slope protection. Soil 2 provided a greater density and better durability.

b. For soils from this site, all specimens could reasonably be compacted at the same nominal optimum moisture content and maximum dry density, irrespective of cement content.

c. The hyperbolic regression gives a useful fit to the data, enabling the complete set of observations to be used in estimating the required cement content rather than merely taking account of the two observations bracketing the specified mass loss. Note however that this technique should only be used for interpolation in practice.

d. Correction of cement content for specimens containing only the coarse fraction by expressing the cement content as a percentage of the minus 4.75 mm fraction still shows a lower cement loss than that obtained where only this fraction is used in forming the specimens. Tests performed on specimens containing only the minus 4.75 mm fraction are therefore conservative in terms of the soil to be used in the field. These tests indicate a difference in cement content of the order of 1%. It was therefore decided that the main test series could be restricted to tests using the minus 4.75 mm fraction, as a primary objective was to provide a correlation with the leaching test program.

e. The cement contents used in these tests resulted in mass losses substantially less than the maxima specified, consequently, the specimens tested in the final program should include cement contents from 1% upwards in order to ensure bracketing of the required mass loss. It was accepted that the resulting indicated cement content would probably be less than the minimum practical cement content and that higher contents would probably be dictated by other considerations.

6.5 FINAL SELECTION OF SOIL

The results of the wet-dry tests indicated that soil 2 was better suited to use in soil-cement than soil 1. The soil 2 borrow area as identified by Lund (1974) extends between haul distances of 400 and 1130 m and constitutes the 150 000 m^3 of available material indicated as being within a haul distance of 1525 m. Because of the relatively sandy nature of this soil which was unsuited for use in the impermeable zone of an earth embankment, further trial holes had not been excavated between distances of 1130 and 1525 m. However, from an examination of the site, it was apparent that the potential to establish a further borrow area of similar extent within a haul distance of 1220 m existed. All this material was located within the reservoir area of the completed dam so that no additional expropriation was required. The proposed borrow areas for soil 1 all lay outside the reservoir area.

Sufficient material similar to soil 2 was therefore available at relatively short haul distances for the construction of a complete soil-cement embankment. It was accordingly decided to restrict the laboratory program to the use of a single soil blend representative of the proposed borrow area. This decision was reinforced by the large number of variables to be examined in the test program and the limitations imposed by available laboratory equipment.

A further three visits were made to the site to collect material and a total of approximately 2300 kg of soil was collected from six additional holes spread over the proposed borrow area. The majority of these samples were yellow in colour and generally similar to the original soil 2, one sample, from a more deeply weathered zone, was more similar to soil 1. Each sample was mixed, the coarse grading determined, and samples of the minus 4.75 mm fraction stored for reference purposes. The remaining soil was blended and riffled into sub-samples for storage. A portion was sieved to remove the plus 4.75 mm fraction and these fines were further divided into approximately 3 kg lots to permit oven drying for use in the various programs requiring only fine material.

Comparison of the indicator test results for soils 1 and 2 showed good agreement with Lund's results over the area in question and accordingly it was decided that the results for soil 2 could be considered to be representative of the blend and no further indicator tests were performed.

6.6 FINAL MOISTURE-DENSITY TESTS

Moisture-density tests were conducted on the fine fraction of the blend soil containing 2, 7 and 12% cement. In all cases the peak and wet of optimum portions of the moisture-density curves were similar while the curve dry of optimum became flatter with increasing cement content. Maximum dry density was 1905 kg/m^3 with optimum moisture content of 13.75% in all cases and these figures were used throughout the experimental program.

On the basis of this result, and applying the gravity analysis discussed in chapter 3, the limiting slope for gravity stability for the Kondebosch Dam is 0.43 horizontal to 1 vertical. This indicates a 78% reduction in embankment volume to 194 000 m^3 which is readily available from the proposed borrow area.

6.7 PRELIMINARY UNCONFINED COMPRESSION TESTS

Unconfined compression tests were conducted on the blend soil stabilized with cement contents of 2, 5, 8 and 17%. Six specimens were compacted at each cement content and cured for 28 days in a humid chamber at 25°C. From each set of six specimens, three were tested after immersion in water for approximately four hours in accordance with the PCA recommendations (PCA 1976c) and the remainder were tested unsoaked.

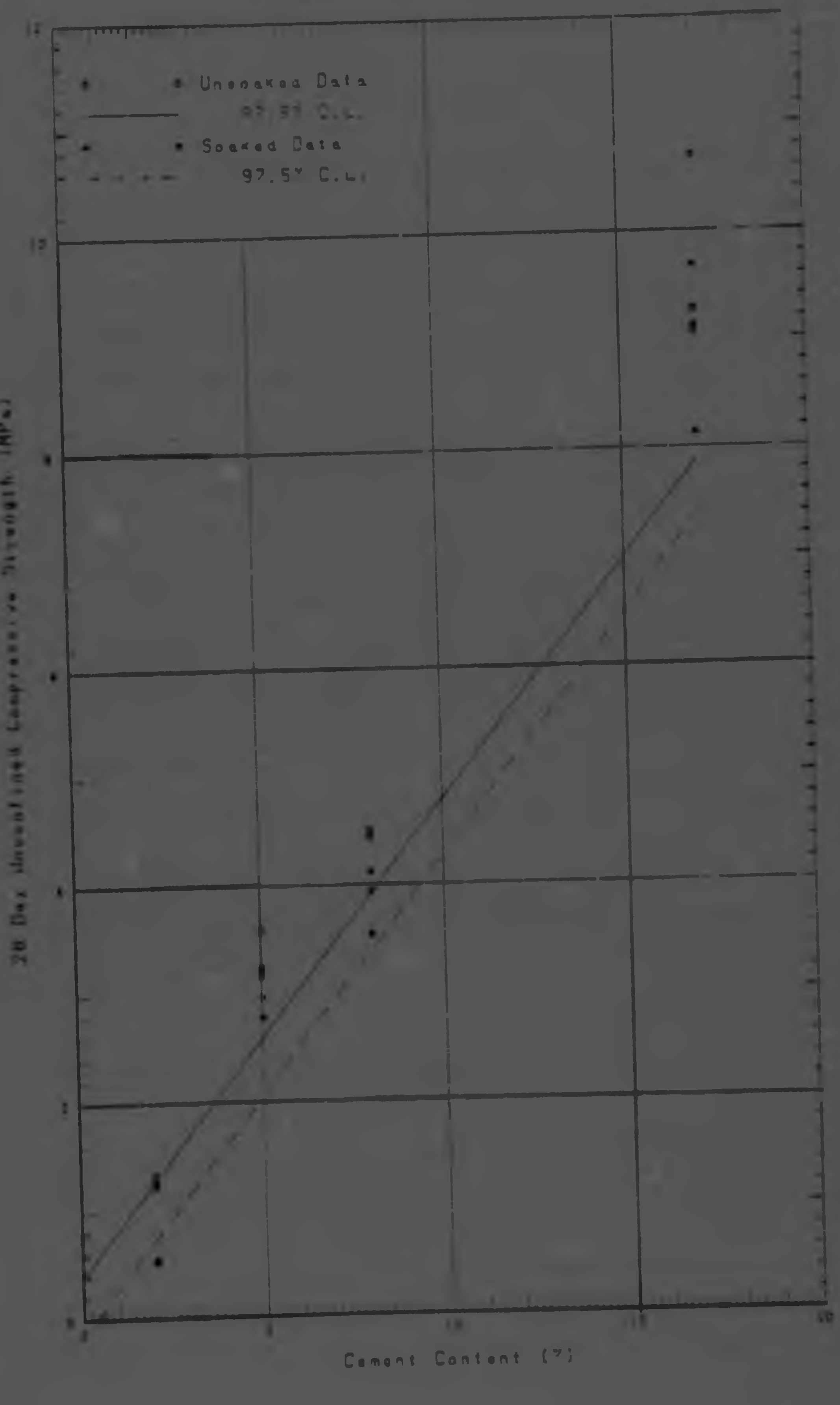
In all cases, except the specimens stabilized with 2% cement, specimens exhibited a sudden brittle failure. The failure plane was generally at an angle of between 55 and 65° to the base. Mass gain on soaking decreased progressively with increasing cement content.

Unconfined compressive strength tended to incr se within a set of specimens with specimen number, suggesting that a short delay between mixing and compacting the material could be beneficial. It is however possible that other fact rs such as location in curing chamber could have had a bearing on this result.

Averages and standard deviations were calculated for each cement content and used to estimate the 97.5% confidence limit (average minus two standard deviations). Linear regressions performed on these data points returned regression coefficients of 0.98 and 0.99 respectively for specimens soaked and unsoaked. Regression lines were effectively parallel with slopes of 0.443 and 0.440 and intercept offset of 0.542 MPa. Linear regressions on the individual data points returned coefficients of 0.98 and slopes of 0.551 in each case with offset 0.658. The individual data points and confidence limit regression lines are shown in graph 6.1.

The parallel slopes referred to above suggest a constant loss of strength due to soaking, irrespective of cement content.

GRAPH 6.3
UNCONFINED COMPRESSIVE STRENGTH



CHAPTER 7

DESIGN OF LABORATORY PROGRAM

The preliminary tests provided the necessary data for design of the laboratory test program. Th ign of individual elements of the program is discussed in chapt 8 to 14.

Each of these chapters concentrates on one test type. The chapter sequence has been chosen in an attempt to minimise forward referencing. Where forward referencing has been necessary, this has been limited to a note at the appropriate point with discussion placed in the last relevant chapter. For example, certain specimens were subject to wetting and drying cycles as a form of curing, these specimens were then tested in unconfined or triaxial compression or subject to leaching, the discussion of the effects of wetting and drying on these properties are discussed in the relevant chapters and not in the chapter on the wetting and drying tests.

This chapter discusses those items which have a bearing on the entire test program.

7.1 CHOICE OF CEMENT CONTENTS

The analysis presented in chapter 3, with assumption of equal slopes, was used in determining the stresses for use in the design of the laboratory program. As indicated in section 6.6, the limiting gravity stable slope for Rondebosch Dam constructed with the blend soil used in the test program was estimated as 0.43 to 1. In terms of this analysis, and applying the principles discussed in chapter 3 to the 97.5% confidence limit estimate of soaked unconfined compressive strength presented in chapter 6, the following cement contents are indicated by structural considerations:-

a. Using the formula for sliding stability referred to in chapter 3, equation 3, with Q = 5, indicates a cement content of 11.2% for sliding stability on the base.

b. Using the USBR formula (USBR 1976a) discussed in section 3.2.d with Q=3 indicates a cement content of 5.6% for sliding stability.

c. From a consideration of the maximum principal stress at the toe of the embankment with Sc=4, a cement content of 8.9% is indicated.

d. From the results of the preliminary wet-dry test results and applying the PCA criteria (PCA 1976c), a cement content of the order of 5% is indicated for slope protection.

From the above, it can be seen that a cement content of 5% could be considered acceptable for most of the Rondebosch Dam if the less conservative analyses were to be applied.

A review of a selection of theses and papers then on hand indicated that cement contents commonly used in soil-cement in practice generally ranged between 5 and 15% with occasional instances of the use of cement contents as high as 30% when dealing with clay soils.

The cost comparison with Merritt Dam presented in chapter 3 indicated a cost advantage for soil-cement although based on a uniform cement content of 14%.

From consideration of the above, and since it was felt that a reasonably conservative analysis was in order for the soil-cement in chapter 3 to the 97.5% confidence limit estimate of soaked unconfined compressive strength presented in chapter 6, the following cement contents are indicated by structural considerations:-

a. Using the formula for sliding stability referred to in chapter 3, equation 3, with Q = 5, indicates a cement content of 11.2% for sliding stability on the base.

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The cost comparison with Merritt Dam presented in chapter 3 indicated a cost advantage for soil-cement although based on a uniform cement content of 14%.

From consideration of the above, and since it was felt that a reasonably conservative analysis was in order for the soil-cement embankment, a cement content of 10% was selected as the standard case for all test programs. As with the analysis presented in chapter 3, a standard condition was selected in order to provide a uniform reference for comparison of all test results. "Branching" of test conditions about this standard was used to investigate as many conditions as possible. Secondary standard cement contents were selected as 0, 5, 15 and 20%, intermediate values being applied where necessary.

During selection of the standard cement contents, it was apparent that zoning of the embankment to satisfy the various design criteria in the most economical way should be given serious consideration at the design stage.

7.2 CURING CONDITIONS

The standard curing condition for the majority of tests was chosen as being seven days in a humid tank, with relative humidity of about 95%, followed by sealing of the specimens for a further three weeks to give a standard age at test of twenty eight days. This condition was chosen to allow some loss of moisture from the specimen in the first few days after compaction, as would be expected to occur on site. A reproducible environment was however required. In practice, a layer of material in an embankment should be covered by several layers of material after seven days, thus reducing the likelihood of further moisture loss.

Other curing conditions were applied in certain tests and a variety of other conditions were examined in those tests which used the above as standard.

7.3 SPECIMEN NUMBERING

In view of the large number of variables considered for many of the test programs, a system of specimen coding was adopted which uniquely identified individual specimens and sets of specimens.

This code contained the specimen number in sequence for the date prepared, the date as a three digit julian number adjusted to the start of the test program, the cement content and codes indicating curing conditions, etc. A system of default codes was established. This system permitted reference to the twenty fifth 37.5 mm diameter cylindrical specimen, containing 10% cement and prepared on Monday 8 December 1980 (day number 343), subject to seven days humid cure followed by twenty one days sealed cure, leached for thirty two weeks at a gradient of 50:1 with de-aired distilled water, the standard test conditions, as specimen 25=10/343L. This uniquely distinguished the specimen from all others in the test program and identified all aspects of the specimens history. The need for continual reference to detailed instructions as to the future of the specimen was thus eliminated. The code could be easily placed on labels on leaching cells etc, where space was limited, and could be entered on data sheets without difficulty. Because of the system of defaults, the length of the code including specimen number and date, seldom exceeded 16 characters.

7.4 SPECIMEN DISTRIBUTION

For the standard specimens used in many of the tests, six specimens were prepared from each batch of material and up to 42 nominally identical specimens were compacted on any one day. For the majority of tests utilising these specimens, six specimens were used. On the basis of previous experience (McLaren, Reid and Robertson 1976) it was decided that a set of six nominally identical specimens was the minimum that would allow for the occasional specimen loss due to experimental problems and at the same time permit statistical significance to be assessed with some confidence. In view of the observation in chapter 6.7 that an increase in strength apparently occurred between specimens compacted with increasing delay after mixing, a system of allocating specimens to sets was developed.

This system ensured that all nominally identical specimens compacted on a particular day were uniformly distributed between the sets for testing and that the average delay between mixing and compaction for each set was similar. This system was used in all test programs where more than one specimen was prepared from one batch of material.

In order to maintain the average delay between mixing and compaction, the batching, mixing and compaction procedures for all specimen types were specified in detail with time allocations for each stage. These operations were then conducted against a stop watch and time to completion recorded for each batch.

7.5 DETERIORATION OF CEMENT

The cement used in the test program was ordinary portland cement. One batch was used for the preliminary tests and a new batch was obtained immediately prior to commencement of the main test program.

After discussion with Prof Blight, Dr J. Rousseau of Pretoria Portland Cement and Mr J. Lane of the Portland Cement Institute, it was concluded that, provided the cement was stored in sealed drums, no significant deterioration would occur during the approximately one and a half year period of the main test program. Less variation in results was expected from the same batch, properly stored, than would occur between batches manufactured at intervals of six months or longer.

The cement used was obtained in standard paper sacks. These were placed in polythene bags and stored in sealed drums. Bags of desiccant were placed in the drums and cement for use in the test program was drawn as required and stored in a smaller drum between batches.

On completion of the test program, a sample of the cement was submitted to the Technical Service and Development Department of Pretoria Portland Cement for testing. The results of these tests were compared with the data from routine testing at the plant concerned over the period when the cement was produced and conducted by the same laboratory. An increase of standard consistency from an average of 25.0 (maximum 26.0) to 27.6 cm^2/g and increases of initial and final setting times from 158 (maximum 180) to 260 and 205 (maximum 235) to 305 minutes were the only values not within the range for the original material. Vibrated mortar cube strengths were close to the maxima for the original material, these results were consistent with those reported by Makovy (1975). It was therefore considered that no significant deterioration of the cement had occurred.

CHAPTER 8

WETTING AND DRYING DURABILITY TESTS

8.1 INTRODUCTION

The objectives of the wet-dry tests were as follows:-

a. To confirm the cement contents required for soil-cement to be used as slope protection, determined in the preliminary tests.

b. To provide data for correlation with the results of the leaching tests with a view to providing a shorter term laboratory test to be used in determining cement contents for leaching protection should this be found to be a critical design consideration. The major advantage of the wet-dry test was considered to be the simplicity of the apparatus and the fact that it is a test in widespread use in materials laboratories.

c. To examine the effect of wetting and drying cycles on the properties of the material to be examined in other parts of the test program i.e. leaching, unconfined compression, brazilian tension and triaxial tests.

d. To examine the potential for use of the standard cylinders (37.5 mm diameter by 75 mm long) used in other components of the test program, as substitutes f[illegible] standard ASTM specimens with a view to reducing the t[illegible]quired to conduct the wet-dry tests. Owing to delays in o[illegible] parts of the test program, it proved necessary to curtail this aspect of the investigation.

Based on the results of the preliminary tests, this program was conducted using only specimens moulded with the soil fraction passing the 4.75 mm sieve. Cement contents examined were 5, 10, 15 and 20% cement to correspond to other elements of the test program and 1, 2, 3 and 7.5% to provide additional points for interpolation as indicated by the preliminary tests. Additional specimens were moulded to examine the effect of small deviations in compaction parameters, these specimens were compacted at moisture contents 1% above and below optimum moisture content and at approximately 2% above standard proctor dry density. In each case, two test specimens were produced and in the case of the four standard cement contents, an additional specimen was prepared for control purposes. The total number of specimens tested was limited by the available oven capacity.

In terms of the objective of providing results to compare with those of the leaching tests, it was decided that more severe test conditions could be produced by extending the duration of the test. The proposal was to subject the test specimens to three complete series of twelve wetting and drying cycles each i.e. three times the duration of the standard test. After each series of twelve cycles, the specimens were dried at 110 ^{o}C to provide data for comparison with the standard test as represented by the first twelve cycles. Unfortunately, a thermostat failed in the drying oven during the drying phase of the 25th cycle resulting in temperatures in excess of 180 ^{o}C before the oven was turned off. The results are therefore restricted to 24 cycles (two series).

In compacting the ASTM standard specimens for this program, the same mould and hammer used in conducting the proctor compaction tests at the start of the program were used. However, it was observed that while the moisture contents of the specimens, as compacted, were generally within one standard deviation (0.1 percentage point) of the specified optimum moisture content of 13.75%, the observed specimen masses were all in the range of 0.6 to 1.2% below the nominal dry density of 1905 kg/m^3. This consistent deviation was not considered serious and was attributed to the likelihood that despite all efforts to ensure uniformity of the soil used, some segregation may have occurred. It is also possible that the increased setting times for the cement noted in section 7.5 led to a slight change in compaction characteristics.

In order to provide specimens for use in other parts of the test program and for use in an initial examination of the behaviour of smaller specimens subject to the standard wet-dry test conditions without brushing, twelve sets, each consisting of six of the standard 37.5 mm diameter cylinders, were prepared. All specimens were compacted at the standard 10% cement content and corresponding moisture content and dry density used in the rest of the program. Four sets were subject to the standard wet-dry test and then transferred to the leaching program as test and control specimens. Five sets were subject to varying wet-dry conditions and then tested as part of the program of unconfined compression tests. Three sets were retained as part of the wet-dry test program with detailed data being recorded and were finally tested as part of the unconfined compression program on completion of the wet-dry tests.

8.2 APPARATUS

The apparatus used complied with the ASTM standard as detailed in the PCA (1976c) booklet "Soil-Cement for Water Control: Laboratory Tests". For convenience of handling, specimen carriers were fabricated to replace the racks in the existing laboratory oven. An ASTM standard wire brush, imported from the USA, was used.

8.3 TEST PROCEDURES

The test procedures detailed in the above mentioned PCA booklet were followed in conducting these tests. In the absence of a humid room, humid curing was effected by placing the specimens in racks above the water surface of a curing bath. A tent was formed from overlapping polythene bags which covered the racks without touching the specimens and with the edges located below the water line. This system produced an atmosphere of near 100% humidity in accordance with the standard.

On completion of the standard twelve cycle test, the specimens were dried at 110 °C in accordance with the standard. Certain of the sets of 37.5 mm diameter cylinders were not subject to this drying stage as a control measure. The second series of twelve cycles followed exactly the same procedure as the first. As noted above, oven failure terminated the tests prematurely at the start of the third series of twelve cycles.

Masses of all specimens were recorded after every wetting and drying stage and after brushing of test specimens. Dimensions of control specimens and the three sets of 37.5 mm diameter cylinders were also recorded. These observations were taken with a view to examining any cyclical trends that might be evident in the data.

8.4 TEST RESULTS

Examination of the cyclical data for the test and control specimens and small cylinders, indicated no particular trends other than a possible tendency for the mass to decrease slightly in the first few cycles and increase slightly thereafter in those specimens where mass loss due to brushing was not great. As the data at the start and end of a twelve cycle series effectively summarised the overall trend, no further processing of the cyclic observations was undertaken.

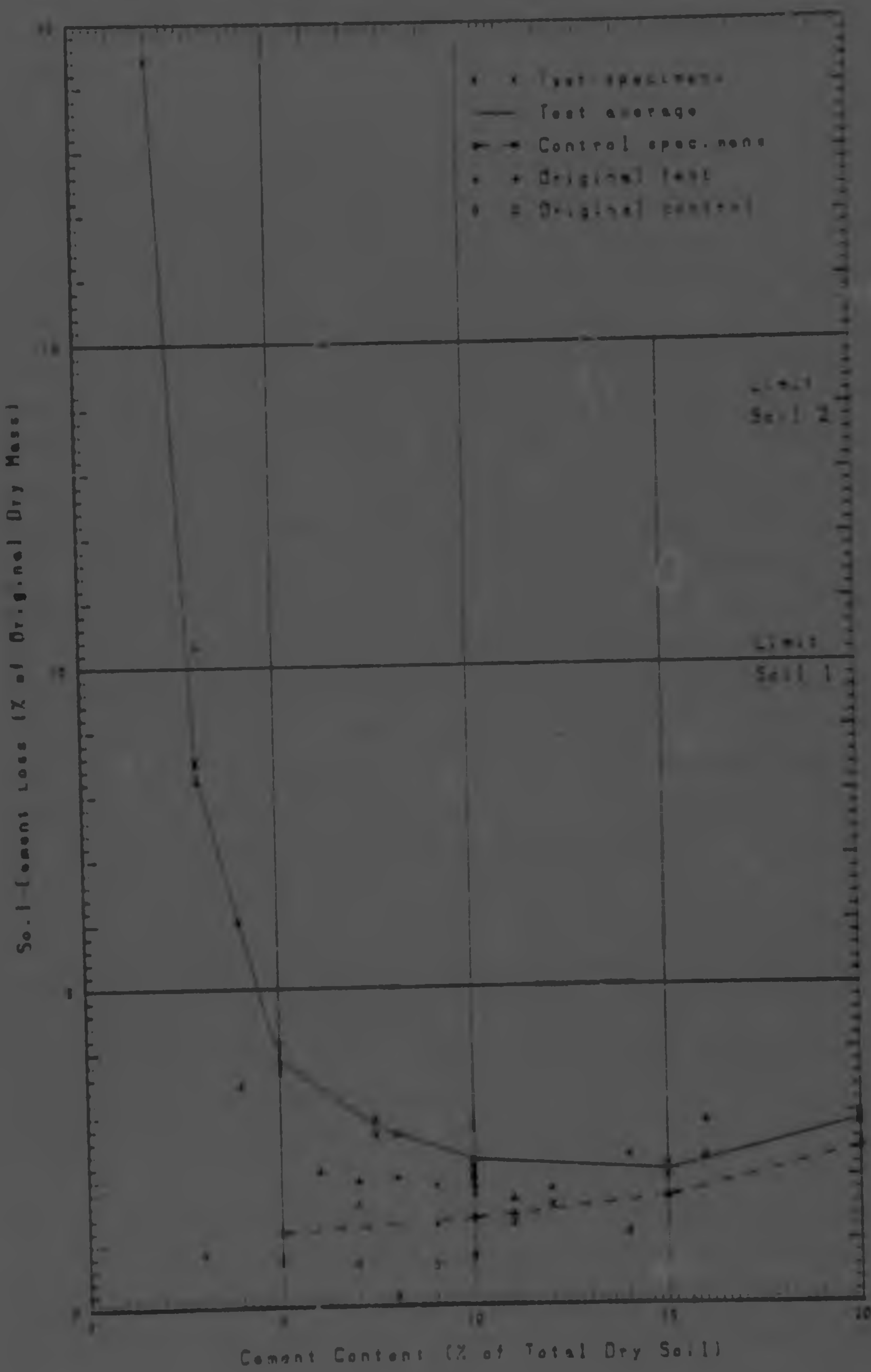
8.4.1 Results for ASTM Standard Test Specimens

The test results for the ASTM standard specimens are shown in graph 8.1 which reflects the soil-cement loss at the end of the

GRAPH 8.1

WET-DRY TEST RESULTS FOR STANDARD SPECIMENS

HYDRATION CORRECTION OF 0.25 x CEMENT CONTENT APPLIED



standard test procedure. This graph also shows the data points for the preliminary tests shown in graph 6.2 for comparison. All points shown include the standard hydration moisture correction of 0.25 x cement content in percent, suggested in the PCA booklet as a rule of thumb correction where control specimens are not tested.

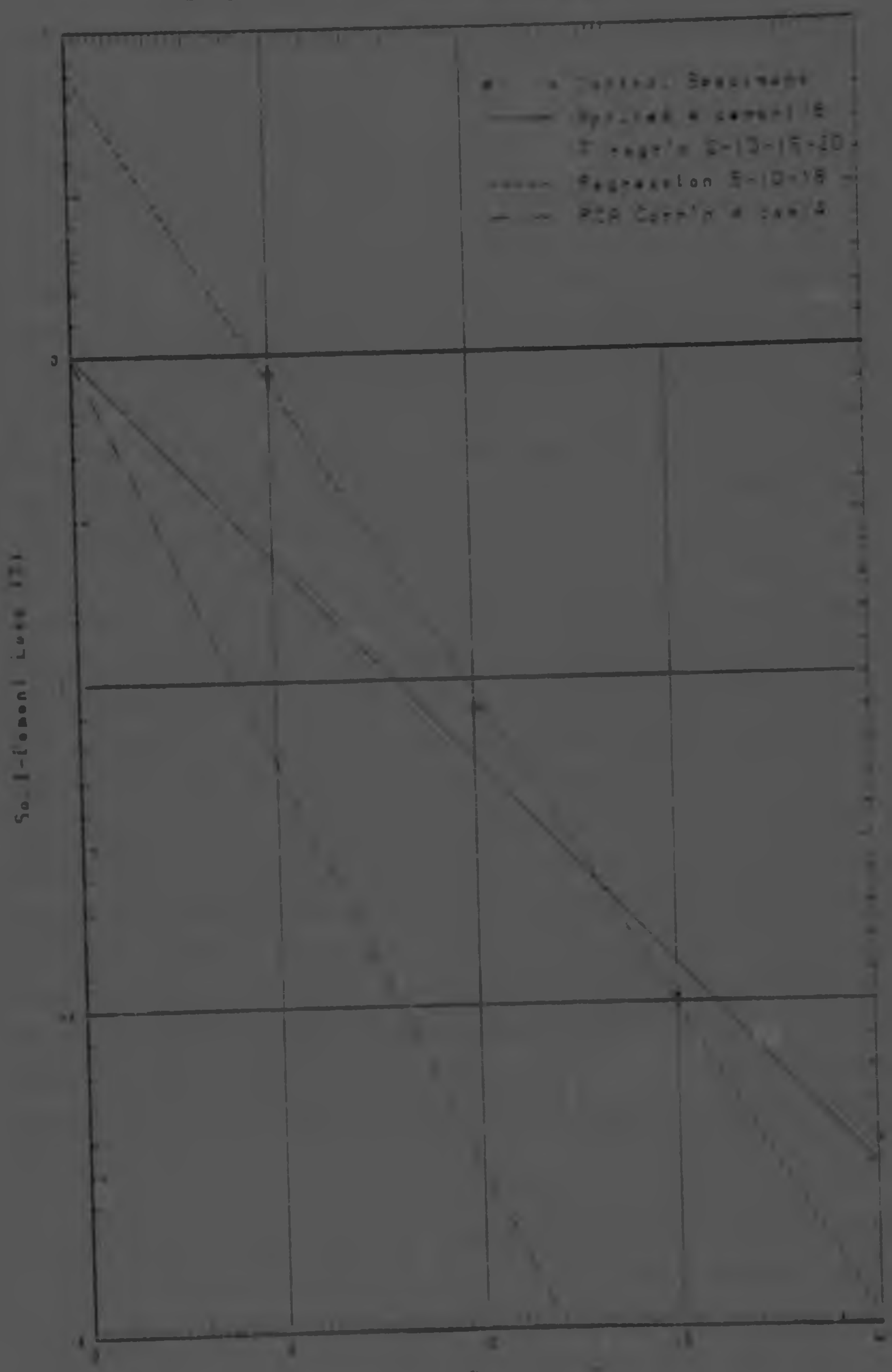
As observed in the discussion of the preliminary tests, the PCA correction appears to be excessive and this is borne out by the increase in indicated loss between specimens with 15 and 20% cement. The substantial indicated loss for the control specimens also supports this observation. The adjusted preliminary test results reflect somewhat lower losses than those for the main test series, both for the test and control specimens. This is explained on the basis that the tests were conducted with an interval of approximately eighteen months between the preliminary and final tests and that a similar interval between date of manufacture of the cement used in each test applied. Small differences in cement properties were therefore to be expected. Allowing for this difference in indicated hydration moisture content, and the difference between the soils used, the results are considered to be directly comparable. Allowing for the upturn at high cement content caused by the high hydration correction, the hyperbolic nature of the results is evident. It should be noted that the use of the PCA correction would again produce conservative results.

In view of the distortion resulting from the excessive hydration correction, the results for the control specimens were analysed to determine a more appropriate correction. Graph 8.2 shows the uncorrected soil-cement "loss" for the control specimens. The data reflects a negative loss i.e. a gain in mass. The PCA correction is also shown.

The first three data points, corresponding to cement contents of 5, 10 and 15% reflect a linear trend, the regression line for this trend is shown and has a coefficient (r^2) of 0.998. The data for the specimen with 20% cement shows a significant deviation from

GRAPH 8.2

WET-DRY TEST RESULTS FOR CONTROL SPECIMENS



this trend. The line indicates a physical loss of material as represented by the positive loss intercept. This is explained on the basis that the control specimens were handled after both wetting and drying stages of the cycle when they were weighed and measured. Measurements were performed using dial calipers with six measurements of diameter and four of length being taken. The specimens were also dried with an absorbent cloth after soaking so that they could be weighed in a surface dry condition. These various factors probably contributed to a real loss of material together with the loss of poorly held surface material during the initial stages of the test. It is also possible that a certain amount of surface deterioration of the specimens with low cement content could have contributed to a loss of material. This loss of material due to handling is a loss that would apply to the test specimens as well and should therefore be excluded from a calibration for hydration moisture gain.

Since there is no reason to expect the hydration moisture content to vary non-linearly with cement content, a straight line through the origin, as used by the PCA, is indicated. As the surface of the specimens with 10% cement and above appeared unaffected by soaking in that they did not tend to become muddy or uneven, it was decided that the deviation of the specimen with 20% cement represented a real deviation. A straight line drawn by eye through the origin and the points for specimens with 10, 15 and 20% cement, yielded a slope of -0.125. A linear regression on the same four points yielded a regression coefficient of 0.962, slope of -0.125 and intercept of 0.03 which is not considered to differ significantly from zero. A correction for hydration moisture content of 0.125 x cement content was thus applied and is shown on the graph.

This correction is half that recommended by the PCA. Possible explanations for this discrepancy include differences in cement properties between US and South African cements, differences in experimental procedure and the fact that the effect of the difference is small at low cement contents and in terms of the critical loss criteria. It should be noted that in proposing a generally applicable "rule of thumb" such as the PCA's, it is necessary to be conservative and that the PCA booklet provides for the hydration correction to be applied on the basis of control specimens as an alternative technique. Consequently, since it is considered that the test procedures used were in agreement with those laid down, it is felt that the above difference in hydration adjustments is not significant.

The cumulative mass loss data for the two series are reflected in graph 8.3 on which cement content is plotted on a reciprocal scale in order to linearize the data and provide separation between the curves. Each plotted point represents the average of two observations. This plot illustrates the semi-hyperbolic nature of the data. The curves converge at a cement content of 15% cement, indicating that no significant loss occurs after twelve cycles for specimens with cement contents above this. The slight upturn from 15 to 20% possibly indicates that the hydration correction is still slightly high but the difference of 0.08 is within the bounds of experimental error.

The indicated critical mass loss for this soil is 14%, as for the original soil 2, and from graph 8.3 it can be seen that the indicated cement content is approximately 2.4%. This agrees with the 2.3% indicated for soil 2 without coarse fraction in the preliminary tests.

The results for the two series of twelve wet-dry cycles are summarised in table 8.1.

Graph 8.4 shows the ratio of the soil cement loss in the second series to that in the first. The graph as plotted has two apparent portions, but probably has a hyperbolic characteristic. Above a ratio of one, the rate of material loss is increasing, thus indicating an unstable situation, the specimens with 2% cement failed during the second series. Below a ratio of one, the rate of material loss decreases, indicating convergence on a stable, no

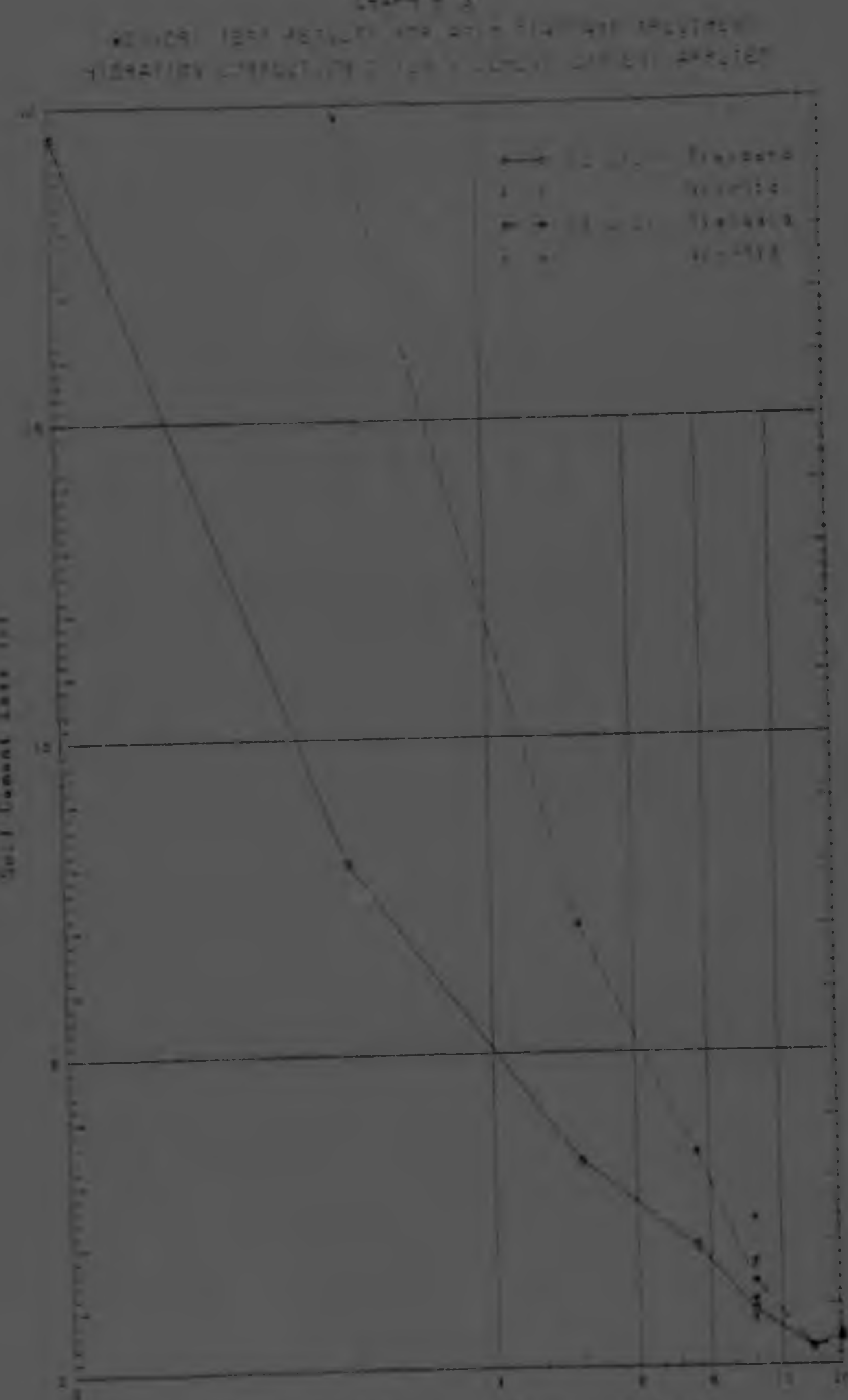


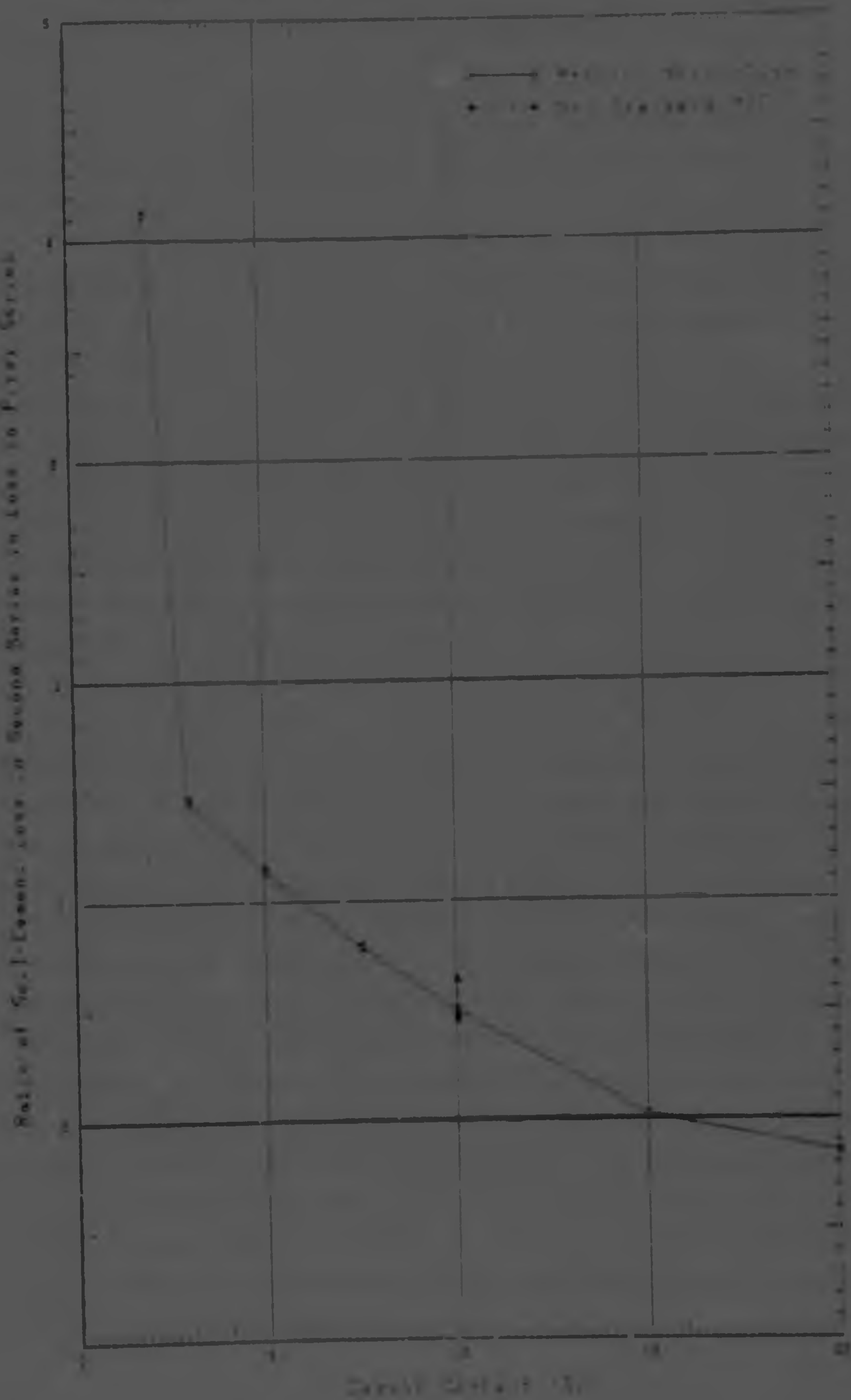
TABLE 8.1
WET-DRY TEST RESULTS FOR ASTM STANDARD SPECIMENS
WITH HYDRATION CORRECTION OF 0.125 × CEMENT CONTENT

Spec No	NOTES	Cement Content %	Initial Dry Mass (g)	Hydrat'n moisture corr'n %	FIRST SERIES s-c loss %	FIRST SERIES Avg loss %	SECOND SERIES s-c loss increm't %	SECOND SERIES Avg loss %	Ratio loss (2) to loss (1)	Avg ratio %	Cummul-ative s-c loss	Avg loss %
2	Control	5.0	1744.3	0.63	0.56	0.17	0.13	-0.14	0.24	-0.62	0.69	0.03
8		10.0	1764.3	1.25	0.16		-0.21		-1.33		-0.05	
5		15.0	1762.8	1.88	-0.12		-0.24		2.05		-0.36	
11		20.0	1757.0	2.50	0.07		-0.24		-3.45		-0.17	
13	Scrapped end	1.0	1750.1	0.13	100.0	100.0	0.00	0.0	0.00	0.0	100.0	100.0
14	of cycle 6		1756.2		100.0		0.00		0.00		100.0	
15	Scrapped end	2.0	1745.7	0.25	19.83	19.52	80.17	80.48	4.04	4.12	100.0	100.0
16	of cycle 13		1751.0		19.21		80.79		4.21		100.0	
17	Standard	3.0	1748.5	0.38	7.91	8.04	11.87	11.75	1.50	1.46	19.78	19.79
18			1745.7		8.18		11.63		1.42		19.81	
1		5.0	1763.7	0.63	3.12	3.29	3.64	3.74	1.17	1.14	6.76	7.04
3			1763.2		3.46		3.85		1.11		7.31	
19		7.5	1757.7	0.94	2.03	1.92	1.48	1.51	0.73	0.79	3.51	3.42
20			1765.8		1.80		1.53		0.85		3.34	
7		10.0	1771.3	1.25	0.85	0.91	0.46	0.45	0.54	0.50	1.31	1.37
9			1768.4		0.98		0.45		0.46		1.43	
4		15.0	1769.3	1.88	0.22	0.30	-0.01	0.01	-0.03	-0.02	0.21	0.31
6			1765.7		0.38		0.03		0.07		0.40	
10		20.0	1762.6	2.50	0.52	0.51	-0.10	-0.08	-0.19	-0.15	0.42	0.43
12			1751.8		0.49		-0.06		-0. 1		0.43	
21	1% dry of	10.0	1753.1	1.25	1.63	1.62	0.78	0.77	0.48	0.48	2.42	2.38
22	optimum m c		1741.5		1.60		0.75		0.47		2.35	
24	1% moist-omc		1738.6		1.05	1.05	0.67	0.67	0.64	0.64	1.72	1.72
25	nom 2% above		1813.3		0.72	0.76	0.31	0.35	0.43	0.46	1.03	1.11
26	proct dens		1806.5		0.80		0.39		0.49		1.19	

TABLE 8.1
WET-DRY TEST RESULTS FOR ASTM STANDARD SPECIMENS
WITH HYDRATION CORRECTION OF 0.125 x CEMENT CONTENT

Spec No	NOTES	Cement Content %	Initial Dry Mass (g)	Hydrat'n moisture corr'n %	FIRST SERIES s-c loss %	FIRST SERIES Avg loss %	SECOND SERIES s-c loss increm't %	SECOND SERIES Avg loss %	Ratio loss (2) to loss (1)	Avg ratio %	Cummul-ative s-c loss	Avg loss %
2	Control	5.0	1744.3	0.63	0.56	0.17	0.13	-0.14	0.24	-0.62	0.69	0.03
8		10.0	1764.3	1.25	0.16		-0.21		-1.33		-0.05	
5		15.0	1762.8	1.88	-0.12		-0.24		2.05		-0.36	
11		20.0	1759.0	2.50	0.07		-0.24		-3.45		-0.17	
13	Scrapped end	1.0	1750.1	0.13	100.0	100.0	0.00	0.0	0.00	0.0	100.0	100.0
14	of cycle 6		1756.2		100.0		0.00		0.00		100.0	
15	Scrapped end	2.0	1745.7	0.25	19.83	19.52	80.17	80.48	4.04	4.12	100.0	100.0
16	of cycle 13		1751.0		19.21		80.79		4.21		100.0	
17	Standard	3.0	1748.5	0.38	7.91	8.04	11.87	11.75	1.50	1.46	19.78	19.79
18			1745.7		8.18		11.63		1.42		19.81	
1		5.0	1763.7	0.63	3.12	3.29	3.64	3.74	1.17	1.14	6.76	7.04
3			1763.[illegible]		3.46		3.85		1.11		7.31	
19		7.5	1757.7	0.94	2.03	1.92	1.48	1.51	0.73	0.79	3.51	3.42
20			17[illegible]5.8		1.80		1.53		0.85		3.34	
7		10.0	1771.3	1.25	0.85	0.91	0.46	0.45	0.54	0.50	1.31	1.37
9			1768.4		0.98		0.45		0.46		1.43	
4		15.0	1769.3	1.88	0.22	0.30	-0.01	0.01	-0.03	-0.02	0.21	0.31
6			1765.7		0.38		0.03		0.07		0.40	
10		20.0	1762.6	2.50	0.52	0.51	-0.10	-0.08	-0.19	-0.15	0.42	0.43
12			1751.8		0.49		-0.06		-0.11		0.43	
21	1% dry of	10.0	1753.1	1.25	1.63	1.62	0.78	0.77	0.48	0.48	2.42	2.38
22	optimum m c		1741.5		1.60		0.75		0.47		2.35	
24	1% moist-omc		1738.6		1.05	1.05	0.67	0.67	0.64	0.64	1.72	1.72
25	nom 2% above		1813.3		0.72	0.76	0.31	0.35	0.43	0.46	1.03	1.11
26	proct dens		1806.5		0.80		0.39		0.49		1.19	

GRAPH 6.4



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loss condition. At a cement content of about 16%, the ratio becomes negative indicating a gain of material in the second cycle. Since the hydration correction is applied as a factor of cement content only, the ratio is unaffected by a change in the correction and a negative ratio indicates that hydration is occurring during every wetting cycle, presumably leading to a continuing increase in strength and resistance to wetting, drying and abrasion.

The tendency of the material to continue hydrating with every wetting cycle is confirmed by the mass loss increments for the control specimens in the second series as shown in table 8.1. The specimens with cement contents of 10, 15 and 20% show mass gains of 0.21, 0.24 and 0.24% respectively while those with 5% cement show a loss of 0.13%. The consistency of the results for the specimens with higher cement contents indicates that further hydration is related to surface area rather than cement content, this is borne out by the results for the 37.5 mm diameter cylinders discussed in section 8.4.2, below.

The non-standard test specimens are also plotted on the graphs discussed above and show a distribution about the results for the standard specimens. Only one specimen compacted wet of optimum is included as the other broke during handling, shortly after compaction. The specimens compacted dry of optimum show the greatest loss, that wet of optimum shows a loss slightly greater than for the standard specimens and those prepared at a higher density show a loss slightly below that for the standard specimens. While these results appear to agree reasonably with what might be intuitively expected, it would be necessary to conduct further tests to confirm these relationships. The variations in behaviour between these specimens are not large in terms of the specified mass loss criteria and would have very little effect on the results obtained in the failure zone unless the results diverge significantly with decreasing cement content, which is considered unlikely.

8.4.2 Results for 37.5 mm Diameter Test Specimens

The test results for the small cylinders are summarised in table 8.2 without a hydration correction.

The specimens subject to drying at 110 °C show mass gains of 1.77 and 1.94% for the two sets in the first series whereas the gains in the second series differ by only 0.04%. These specimens were prepared seven days apart and so were cured separately, the difference in behaviour in the first series thus indicates a variation in curing conditions.

The specimens not subject to drying at 110 °C show a greater gain, reflecting moisture not driven off at 72 °C. The loss increment for the second series is slightly less than for the other specimens but the difference of 0.03% cannot be considered significant. The mass gains in the first series should be compared with that of 1.09% for the standard control specimens with 10% cement, this indicates a substantial increase in hydration moisture content for the small specimens. Likewise, the gains of 0.43 to 0.47% in the second series should be compared with the 0.21% shown in table 8.1. Since sets P and S of the small cylinders were cured and tested together with the control specimens, it would appear that the relatively greater surface area of the smaller specimens permitted a greater intake of moisture during curing and testing.

As the small cylinders show a mass gain for both first and second series, the ratio is positive, this suggests that the curve shown in graph 8.4 may return above the zero ratio line at cement contents in excess of 30% for the standard wet-dry specimens.

8.4.3 Correlation with Leaching Test Results

In view of the stated objective of providing some correlation between the results of the wet-dry and leaching tests, the convergence shown in graph 8.3 and the corresponding levelling

TABLE 8.2
WET-DRY TEST RESULTS FOR
SMALL CYLINDERS WITHOUT HYDRATION CORRECTION

Specimen No	Initial Dry Mass (g)	FIRST SERIES s-c loss %	FIRST SERIES Avg loss %	SECOND SERIES s-c loss increm't %	SECOND SERIES Avg loss %	Ratio loss (2) to loss (1)	Avg ratio %	Cummulative s-c loss	Avg loss %
SET P=10/100									
01=10/100	157.891	-1.36		-0.43		0.31		-1.79	
08=10/100	157.273	-2.12		-0.43		0.20		-2.55	
15=10/100	157.774	-1.73		-0.44		0.25		-2.17	
22=10/100	157.424	-2.08		-0.43		0.21		-2.52	
29=10/100	157.773	-1.54		-0.40		0.26		-1.93	
36=10/100	158.035	-1.78	-1.77	-0.47	-0.43	0.26	0.25	-2.24	-2.20
SET U=10/108									
07=10/108	157.826	-1.85		-0.48		0.26		-2.34	
14=10/108	157.452	-2.16		-0.48		0.22		-2.64	
21=10/108	157.763	-1.73		-0.43		0.25		-2.15	
28=10/108	157.308	-2.14		-0.49		0.23		-2.62	
35=10/108	157.910	-1.78		-0.46		0.26		-2.24	
06=10/108	157.655	-2.01	-1.94	-0.50	-0.47	0.25	0.24	-2.51	-2.42
SET S=10/100 (not subject to drying at 110 °C, max 72 °C)									
19=10/100	157.311	-2.19		-0.46		0.21		-2.64	
26=10/100	157.672	-2.10		-0.41		0.20		-2.51	
33=10/100	157.906	-1.99		-0.45		0.23		-2.44	
11=10/100	157.270	-2.17		-0.47		0.21		-2.63	
18=10/100	157.848	-2.09	-2.11	-0.38	-0.43	0.18	0.21	-2.47	-2.54

Cement content of all specimens 10% of dry mass of soil.

off shown in graph 8.4 indicate possibly characteristic behaviour of the extended wet-dry tests which might provide a basis for such a correlation.

8.4.4 Alternative Test Procedures

The scale effect discussed above indicates just how empirical the wet-dry test is in terms of specified critical mass loss criteria. This suggests that any attempt to develop an alternative laboratory test that can be conducted in less time, with less effort will require a substantial program of investigation to provide correlation. In view of the widespread acceptance of the standard test procedure and the vast amount of supporting experience, any alternative test will have to provide substantial savings of time and effort in order to gain acceptance.

The observation that mass gain with progressive wetting cycles appears to be related to relative surface area, suggests a possible basis for an alternative test. Assuming that the rate of deterioration of the standard test specimen is largely a function of the degree of surface absorption and drying, it is possible that where specimens with relatively high cement contents show accelerated mass gains, those with low cement contents may show accelerated deterioration. This suggests the possibility of reducing the soaking period for small specimens subject to otherwise standard conditions, a doubtful benefit. Alternatively, the number of wetting and drying cycles might be reduced thus reducing the overall test period.

If the deterioration is largely a surface effect, it is possible that the removal of weakened material from the surface of the specimen by brushing, is not essential to the conduct of the test as the weakened material is almost certainly relatively permeable. This suggests that standard specimens might be subject to the standard test procedure without brushing, thus reducing the time required on a daily basis. Specimens thus treated might then be analysed purely on mass change during the test or subject to a modified brushing procedure at the conclusion of the wet-dry cycles.

Extending this proposal to the small cylinders used here, the scale effect could be exploited to allow shorter wetting and drying periods leading to a 24 hour cycle instead of the present 48 hours. Mass change data could then again be considered without abrasion or abrasion could be applied at the end of the wet-dry cycles. The small size of the test specimen would then lend itself to the conduct of an abrasion test using the type of vibrating tumbler used by Blight in the design of stabilized briquettes of coal and ores for metallurgical use and described in appendix B. This would eliminate the potential for error inherent in the application of the manual brushing technique while at the same time further reducing the time required to conduct the test. A test based on the above proposals could be conducted in 19 days instead of 31 and would permit considerably more specimens to be tested with existing facilities and manpower. It is considered that investigation of this technique could be warranted in terms of the potential savings.

The use of unconfined compressive strength as a measure of deterioration might also be considered.

8.5 CONCLUSIONS

The suitability of the soil from the Rondebosch Dam site for use as slope protection is confirmed. A cement content of 2.4% is indicated to satisfy the standard mass loss criteria for the material without fines. On the basis of the preliminary test results, a cement content of 2% for the complete soil with coarse fraction is considered conservative. In accordance with the PCA recommendations, a cement content of 4% is indicated for soil-cement slope protection to be used over the expected range of reservoir levels.

Detailed mass and dimensional observations for every cycle do not provide data of any significance and the conventional mass observations at the start and end of the standard test procedure are sufficient.

The extension of the test procedure to include a second series of twelve wetting and drying cycles provides a useful insight into the long term behaviour of soil-cement subject to wetting and drying cycles. For the soil in question, no significant further loss of material occurs after the first series of twelve cycles for soil-cement containing more than 15% cement. Above a cement content of 16% specimens show an increase in mass due to continuing hydration during the wetting stage of the test. Specimens with a cement content above 6% show a diminishing rate of material loss which suggests convergence on a stable no loss, situation. It is considered that these trends have potential for correlation with the results of the leaching tests.

The effect of wetting and drying cycles on other properties of the material are discussed in the relevant sections.

The standard 37.5 mm diameter cylinders exhibit pronounced scale effects which confirm certain trends observed with the standard ASTM test specimens. A tentative procedure which could significantly shorten the time required to conduct wet-dry tests, as well as increasing the number of specimens that could be tested at one time, is proposed. This makes use of the observed scale effects to shorten the time required for one cycle and suggests mechanical abrasion on completion of the wetting and drying cycles. Considerable further research is required to confirm these observations and to develop a simple and reproducible laboratory test which must provide a simple correlation with the existing technique.

CHAPTER 9

LEACHING TESTS

9.1 INTRODUCTION

The objectives of the leaching tests were:-

a. To examine the effect on permeability of prolonged seepage.

b. To determine to what extent cement was leached from the soil-cement by percolating water and thus whether leaching was likely to be a critical factor in design over the intended life of the soil-cement dam and if so, what provision should be made for leaching in design.

c. To determine to what extent permeability and leaching were influenced by factors such as different compaction and curing conditions and differences in percolating water.

d. To investigate whether a simpler test in the form of a "leaching bath" could be used to obtain comparable results at lower cost.

e. To establish whether the ASTM standard wetting and drying test described in chapter 8 could be used as an indicator of the resistance of soil-cement to leaching.

As noted in chapter 4, experience with soil-cement in slope protection and other applications in the United States indicated that leaching was not a significant problem. Hansen (19[illegible]9a) notes that "in the range of cement contents usually used for soil-cement slope protection (>10% by weight of dry soil) we have noticed no evidence of leaching of cementitious material". However, the results of Jackson (1965) together with the views expressed by a number of practising engineers indicated that concern about the possible effects of leaching was likely to be a major obstacle to the acceptance of the concept of the soil-cement dam. It was accordingly decided that a major portion of the research effort should be devoted to investigating this aspect.

As with other elements of the test program, a set of standard conditions were defined and the effect of varying these conditions was then investigated.

The 75 x 37.5 mm specimen was used as standard with twelve disk specimens from the long-term consolidation program, described in chapter 13, used to investigate the effect of a different length to diameter ratio for similar volume.

Standard conditions of compaction at optimum moisture content and maximum dry density with 10% cement were adopted. Additional specimens were compacted 1% wet and dry of optimum moisture content and 2% above standard proctor maximum dry density, others were compacted at cement contents of 0, 5, 15 and 20%.

Two batches of concrete mortar (microconcrete) were designed by Mr C. Thompson of the Portland Cement Institute to simulate the mortar that might be expected in a low quality hearting concrete. The mixes were designed to give cement contents, permeability and strength values of similar magnitude to that of soil-cement. This was done to permit a reasonable comparison with one of the materials that this research proposes to replace with soil-cement. The microconcrete was cast in beam moulds by the PCI and test specimens were prepared by taking cores from the beams.

A number of specimens were split axially to examine the behaviour of tension cracks under flow.

The standard cure of 7 days humid and 21 days sealed was adopted with further specimens subject to variations. These included 28 day humid cure, 7 day humid cure followed by 21 days in the leaching bath and specimens subject to the standard wetting and drying test as described in chapter 8 but without the final drying at 110^{o}C. Other specimens were subject to extended periods of sealed cure of 77 and 343 days.

In order to attempt to simulate the effect of leaching on a structure with a design life of at least fifty years in a reasonable time in the laboratory, it was decided to attempt to accelerate the leaching process by increasing the hydraulic gradient and thus the flow of water through the specimen in unit time. This is the same approach adopted by Jackson (1965) who used a gradient of 187. It was however felt that in view of the time dependent nature of the curing of cement, the gradient should be limited to one that could be related to a practical situation.

The structure analysed in chapter 3 is a homogeneous embankment without drains and the gradient through the embankment would accordingly be approximately 1 m/m. The limiting case for soil-cement as a water retaining membrane was considered to be that of conventional soil-cement slope protection backed by drains. In the extreme case, this might have a normal thickness of 1 m at a depth of 50 m below the water surface. It was therefore decided that a gradient of 50 was the upper limit of conceivable situations and this gradient was adopted as the standard. This is equivalent to only half the width of one placement strip across the full width of a 50 m soil-cement embankment retaining the full reservoir head.

In order to come somewhat closer to the proposed unit gradient, twelve specimens were tested at a gradient of 10. A further eighteen specimens were tested at a gradient of 136 in order to come closer to Jacksons test conditions as well as to examine the effect of increasing gradient on the test results. The assumption in Jacksons research is that if the gradient in the field is unity, then the use of a gradient of 50 in a laboratory test run for one year will simulate the degree of deterioration that would be expected in fifty years in the field. The spread of gradients was chosen with a view to examining this assumption.

Standard leaching medium was de-aired distilled water which was selected as it was readily available and was internationally reproducible. The de-airing and storage apparatus was designed to keep re-absorption of air by water in storage to a minimum. Since many reservoirs contain acid water as a result of dissolved carbon dioxide, 18 specimens were tested with carbonated water produced by bubbling CO_2 through de-aired distilled water. A further 15 specimens were tested with water drawn from the reservoir formed by the Rondebosch Dam, Middelburg, on the site from which the soil samples were taken. This water was filtered through a 5 micron filter to remove spores which might contaminate the apparatus but was otherwise untreated in order to simulate as closely as possible the water that would percolate through the real structure.

Standard leaching period was 266 days (38 weeks) with additional specimens subject to periods of 587 and 904 days. It was proposed to test specimens at shorter leaching periods of 28, 56 and 112 days. The sets for testing at 112 days were placed in the leaching cells but owing to problems with the triaxial equipment, it was not possible to test them at the specified age and they were allowed to run full term, the tests at 28 and 52 days were therefore not possible. Problems with the triaxial equipment also resulted in the extension of the originally proposed 224 day leaching period to the 266 day period finally used. The standard period was a compromise between the need to use as long a period as possible in order to obtain meaningful results and the need to complete the laboratory program in a reasonable time.

Standard set size was six specimens with sets of three specimens used in certain instances for purposes of interpolation. Two specimen sets were used for disk specimens where apparatus for final testing was limited.

A total of 192 specimens were subject to long term percolation and an additional 90 specimens were tested in the leaching bath. The control specimens used in interpreting this data were prepared together with the test specimens and at the appropriate age were subject to triaxial or unconfined compression tests, a total of 59 sets of control specimens were tested. The results of these tests are discussed in chapters 10 and 11.

9.1.1 Consideration of TMS Water

The Department of Water Affairs requested that the research program be expanded to include a very pure water (low dissolved solids) rich in humic acid from the Table Mountain Sandstone (TMS) region of the Cape Province, an area in which problems with corrosion of concrete in water retaining structures had been experienced. A detailed review of literature was undertaken and the following information obtained:-

a. Bond (1946) classifies water from both the TMS and Middelburg regions as pure, having dissolved solids less than 150 parts per million and pH less than 7.1. Data in Bond indicates that although the "dark brown coloured acid waters from TMS in Cape Province" have very low pH values when compared with other waters occurring in South Africa, the free CO_2 in TMS water is no higher than that of other less acid waters. Bond states that "the high acidity is mainly due to organic acids, and only in small part, due to free CO_2."

b. It is widely accepted (Muller 1978, Fulton 1977, and others) that the principal mechanism for the corrosion of concrete through attack on the cement paste is by the conversion of calcium hydroxide into calcium bicarbonate through the action of aggressive CO_2. The process is summarized by Muller (1978) as follows:-

"In the first instance, calcium hydroxide is removed, being dissolved in the water. If the water contains carbon dioxide as well, its aggressiveness is increased and both calcium hydroxide and calcium carbonate go into solution as calcium bicarbonate. Furthermore, the hydrated calcium silicates are stable only in highly alkaline conditions; if calcium hydroxide is removed, the pH is lowered and decomposition of some more hydrated calcium silicates takes place with the liberation of more calcium hydroxide which in turn is removed by leaching or converted to $CaCO_3$ which can remain or also be removed by flowing soft water as calcium bicarbonate if the water contains CO_2 in excess of that required to react with all the $Ca(OH)_2$ to form $CaCO_3$. Under favourable conditions these reactions can continue until most of the hydrated calcium silicates are decomposed. This leaching action can proceed until finally no cementitious material remains to bind the aggregates together."

c. It is therefore apparent that the water from the TMS is not necessarily more aggressive than the water from the Rondebosch Reservoir. This is supported by Fulton (1977) who states:-

> "many pure mountain waters such as those of the Western Cape coastal belt, contain weak acids in solution. Due to the low solubility of humic acid in water, the actual quantity of acid contained in them is small. In spite of their low pH they have therefore only a slight action on concrete, even over many years. In general they are far less aggressive than are very pure waters containing aggressive carbon dioxide."

Fulton supports this statement by reference to a research report entitled "Corrosion of Sewers" by J.L. Barnard, (CSIR research report 250, 1967).

On the basis of the above, it was concluded that the de-aired distilled water constituted a reasonably aggressive water by virtue of its high purity and that the water from Middelburg was likely to be as aggressive as that from the TMS in terms of both purity and dissolved CO_2.

Bond further indicates that the free CO_2 content of naturally occurring waters in South Africa is generally significantly less than 55 ppm whereas the maximum solubility of CO_2 in water at 25°C is approximately 1450 ppm (Lange 1967). The de-aired distilled water saturated with CO_2 was therefore considered likely to be as aggressive as any naturally occurring water as well as having the advantage of being internationally reproducible.

It was thus concluded that the alteration of the test program and apparatus to incorporate water from the TMS would not greatly improve the value of the results.

9.2 APPARATUS

9.2.1 Leaching Cells

In order to investigate the large number of variables adequately, it was necessary to design the apparatus to permit as many tests as possible to run simultaneously and the design was therefore optimised for minimum cost and space requirement. The final apparatus permitted the simultaneous testing of 192 specimens with continuous monitoring of permeability and collection of leachate.

In arriving at the final leaching cell design, the permeameter cells used by the PCA (1978b) and the leaching cells used by Jackson (1965) were considered together with a variety of other permeameter cells, it was however concluded that these cells were too complex to be mass produced cheaply. The design finally adopted is shown in plate 9.1 and a partially assembled cell in plate 9.8, the barrel was manufactured from 50 mm nominal bore UPVC (unplasticized polyvinyl chloride) pressure pipe (rated at 700 kPa) with the ends faced to provide a sealing surface with a radius to prevent damage to gaskets. The end plates were manufactured from squares of 10 mm UPVC sheet cut to size.

The cells for the disk specimens were manufactured from 75 mm phosphor-bronze tubing with brass end plates but were otherwise similar to the cells for cylindrical specimens. These materials were used for prototype cells of both types but were found to be too heavy and expensive for mass production. Accelerated deterioration of the latex membranes in contact with the phosphor-bronze was also observed.

PLATE 9.1
LEACHING CELL

Gaskets were produced from 3 mm "gum rubber" sheeting (supplied by the Transvaal Rubber Co), cut into squares and punched as required. The assembly was held together by three equal length bolts inserted so that they provided a three point base on which the cell rested on the supporting racks thus ensuring a constant height for the top of the cell above the racks. The outlet elbow on top of the cell was screwed into a tapped hole to give a tolerance of +/- 0.5 mm on the height of the invert above the top plate.

The basic cell design of plastic cylinder, end plates with gum rubber gaskets and clamping bolts was used for many items of ancillary equipment including hydraulic accumulators, de-airing [illegible] flow indicators, etc as the components did not require exacting tolerances, were interchangeable between different items and could be quickly manufactured.

The system of sealing to prevent flow around the sides of the specimen involved the use of two latex membranes of the type used in triaxial tests together with a sealing cell pressure applied using de-aired distilled water as the cell fluid. This was adopted in preference to the system of bentonite used by the PCA (1978b) as it was felt that confining pressure would be unknown and could effect permeability and that leaching of the bentonite could occur.

In order to reduce the cost of conducting the tests at high gradient, it was however decided to use bentonite, separated from the specimen by a latex membrane and flooded with the standard cell water. A set of control specimens at standard gradient was also scheduled. However, trial assembly with sodium bentonite prior to conducting these tests revealed that it was difficult to ensure thorough filling of the annulus with bentonite and to wet the bentonite. About twenty four hours was required before a seal was obtained. It was then found that the confining pressure generated was such that the specimen could not be removed from the cell without damage to the cell and specimen unless the assembly was dried for several days. It was thus not possible to test the specimen further on completion of leaching and the technique was abandoned.

Owing to time constraints, compressed air was used in place of bentonite for the cell fluid in the high gradient cells and their controls at standard gradient. The pressure used was selected to give the same effective stress at the base of the high gradient specimens as applied in the standard case.

The membranes were custom manufactured by the Latex Products division of Adcock Ingram Ltd. Six mandrels were produced from aluminium tubing closed at each end with aluminium plugs drilled and tapped for mounting studs. The membranes were produced by repeated end-over-end dipping of the mandrels in latex solution with intervals for drying. Membranes were 38 mm nominal ID and 0.30 mm nominal thickness in accordance with the recommendations of Bishop and Henkel (1962). Membranes were 600 mm long and were produced at a rate of six per week. These were then cut into three or four lengths to give membranes of 200 and 150 mm for use in the leaching program. One membrane of each length was used in each cell, the shorter membrane in contact with the specimen where damage during assembly and testing was considered more likely. The double thickness was adopted against damage during handling, flaws in the material and deterioration over the duration of the tests. This system proved effective with only one membrane failure, on initial pressurisation.

To ensure even flow through the specimen, spacing disks were made from Filtram Geodrain (produced by ICI). This consists of a layer of plastic mesh (netlon) covered on both sides with geofabric (Terram). The mesh is designed to permit flow within the plane of the drain with the fabric used as a filter. For this application, the geofabric was stripped from one face of the Filtram and disks punched from the sheet. In use, a disk of geofabric was placed against the end cap of the cell, a disk of Filtram was placed with the fabric side adjacent to the end cap and the open mesh against the specimen. This arrangement located the specimen axially in the cell, provided a collection zone at either end of the specimen to permit uniform flow and, at the same time, allowed material loss and loosening to occur if deterioration was pronounced. The geofabric prevented particles from the specimens moving into the adjacent pipes and manifolds while permitting free flow of water.

9.2.2 Leachate Collection

Leaching water was supplied to the bottom of the leaching cell, percolated through the specimen and flowed out through the top plate of the cell. As mentioned above, the height of the invert above the supporting racks was controlled to minimise errors in the gradient applied to the specimens. The outlet elbow was connected to a Y piece connected so that the the overflow passed through the lower arm of the Y and out of the leg of the Y while the upper arm acted as a siphon breaker vent. A length of 6 mm OD tubing was connected to the siphon breaker arm in order to minimise loss through evaporation. This arrangement introduced a greater variation in invert level than intended but was necessary as suitable tee pieces for use on the cell outlet were not available at the time the cells were fabricated. Observed error in invert level did not exceed minus 10 mm plus zero. This error was considered acceptable as it represented a maximum error of 0.27% on the head of 3.75 m used for the standard gradient.

The outflow from the cells drained through clear plastic tubing and was collected in polythene bottles (plate 9.2). The tubes were sealed to the necks of the bottles using "Parafilm M" a self sealing mouldable laboratory film with good air permeability but low moisture permeability. These properties allowed the necks of the collection bottles to be sealed against loss by evaporation while permitting air to diffuse out as the bottle filled. Back pressure development was not observed. The effectiveness of the seal was demonstrated in a number of low flow cases where the film was not properly applied or was damaged in use and measurable evaporation occurred. Where the film was undamaged, and no flow was recorded over several months, no significant change in the mass of water already in the collection bottle was recorded.

9.2.3 Laboratory, Racks and Manifolds

To accommodate the apparatus, two store rooms were converted for use in this research program. The upper room was converted to operate at a constant temperature of 30 °C and was used to accommodate the de-airing apparatus, water reservoirs and constant head tanks to provide the necessary static heads.

PLATE 9.2

LEACHATE COLLECTION

The lower room was converted to a constant temperature laboratory maintained at 25 +/- 1 °C with fans to ensure uniform temperature throughout and recorders to monitor temperature. Insulated compartments were installed for use in the investigation into heat of hydration discussed in chapter 14 and oedometers were installed for use in the long term settlement tests discussed in chapter 13. A mezzanine floor was install , and panelling fastened to the upper walls to facilitate the fixing of the brackets, piping, etc for the leaching cells.

The brackets to support the leaching cells consisted of angle sections fastened to shelf brackets to produce a channel in which the cells could stand on the ends of the three clamping bolts with the constant head supply passing through the gap between the angles.

Three tiers of leaching cells were accommodated above the mezzanine (plate 9.3) with three tiers of shelves to support the collection bottles below (plate 9.2).

Leaching cells for cylindrical specimens were grouped in threes with disk cells in twos, to correspond to the minimum number of specimens in a set. The cell fluid and leaching fluid connections of each group were connected together with clear thick walled plastic tubing and push-fit fittings before the cells were placed on the racks. Clear tubing was used to facilitate the de-airing procedure during assembly.

The major cell fluid and leaching fluid manifolds were located below the racks with stop cocks to which each group of cells was connected. Each manifold incorporated a perspex air trap before the cells as an indicator of malfunction and to facilitate de-airing during initial assembly. During operation, the manifolds were flushed periodically and samples taken for analysis as a precaution against diffusion of leaching products into the supply.

Further manifolds above the top and bottom racks were installed to provide vacuum connections for de-airing of cells on commencement of testing. Plate 9.3 gives a general view of the leaching cells, racks and manifolds on the wall of the laboratory. A gutter was installed around the wall below the cells to collect leakage during cell installation and removal and for running flow to waste in those cases where flows were so great that permeability readings were not taken continuously.

PLATE 9.3
LEACHING CELLS AND MANIFOLDS

9.2.4 Cell Pressure Supply

As noted above, cell pressure for the high gradient cells was supplied by means of compressed air drawn from the central compressor applied directly to the membranes. The cell pressure for all other cells was also supplied using compressed air but in this case de-aired distilled water was used for the cell fluid with the interface provided by two two stage hydraulic accumulators (plate 9.4). An overpressure relief valve was incorporated on the cell side of the accumulators to provide protection in the event of regulator failure. Two stage regulation was adopted as a further safety measure. The de-aired distilled water used as cell fluid for the standard cells was also used as leaching fluid in the high gradient tests.

The two stage hydraulic accumulators consisted of a primary stage air-water interface cell in which the air was applied directly to the water surface together with a secondary stage in which the pressurised water transferred the pressure to the de-aired cell fluid through a membrane formed using a 250 micron polythene bag to minimise transfer of dissolved air. The accumulators were connected in parallel so that one could be disconnected from the system for recharging or maintenance. The water in both stages was regularly flushed out and replaced with fresh de-aired distilled water as an added precaution against contamination of the cell fluid with air.

PLATE 9.4
ACCUMULATORS FOR CELL PRESSURE

The cylinders of the primary stage accumulators were fabricated from 150 mm UPVC pressure pipe while those of the secondary stage accumulators were fabricated from transparent perspex so that the state of the membrane could be monitored and any air accidentally introduced could be detected. End plates were fabricated from 6mm mild steel plate lined with 3 mm UPVC, gum rubber gaskets were used in conjunction with threaded clamping bars to provide the seal. The polythene membrane in the secondary accumulators was folded over the cylinder wall before assembly and cushioned by gaskets on either side. A light coating of silicone rubber adhesive on the membrane, where it was folded over the cylinder, was used to complete the seal by preventing flow through creases in the membrane.

A reserve of compressed air against compressor or electricity failure was provided by means of a reservoir of approximately 30 litres capacity isolated from the compressed air supply by a sensitive check valve. Further back-up of the standard cell pressure supply was provided by means of regulated mains water pressure feeding into a single accumulator of similar design to the second stage accumulators discussed above. All accumulators were connected to the cell fluid system through sensitive check valves to prevent pressure loss in the event of accumulator failure.

Total cell pressure head on the middle tier cells and driving head for the high gradient specimens derived from the accumulators was 10.25 m giving an effective stress at the base of the middle tier standard specimens of 64 kPa and a gradient of 136 m/m on the high gradient specimens. Tests indicated that a head of approximately 3 m was sufficient to seal the membranes on a brass blank the same size as the specimens. The design cell pressure was selected to provide a reserve of additional pressure while keeping the effective stress on the specimen as low as possible and it was found that this head was of similar magnitude to that required for the gradient for the high gradient specimens. The same pressure supply was thus used for both applications.

9.2.5 Vacuum Supply

Vacuum was required for de-airing of leaching cells and specimens, de-airing of water for use in the test program, draining and de-airing of triaxial cells and de-airing of the triaxial specimens. A supply system was thus established using three rotary vacuum pumps connected to a system of manifolds and remote switches serving all the relevant points in the laboratories. In view of the large volumes of water being handled, float valves produced from squash balls located under conical brass seats were installed in appropriate locations to prevent flooding of the pumps. A perspex cell with float valve was installed adjacent to the pumps to provide a reservoir during de-airing operations.

9.2.6 Constant Head Supply

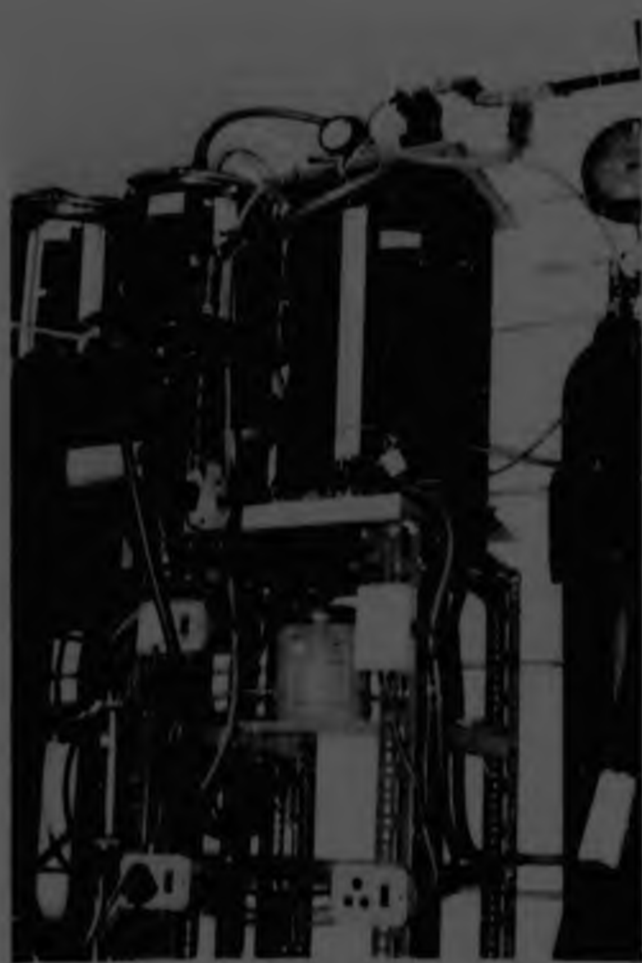
The various levels and heads required, together with the limited funds available, precluded the use of a large number of individual hydraulic accumulators for the leaching supply. Constant head tanks with level controlled by an overflow weir were selected as the most suitable means of pressure control.

Depending on the number of cells to be served, constant head tanks were fabricated from plastic drums of either 5 or 25 litre capacity. Water was gravity fed through needle valves from the reservoirs to the base of the tank close to the tapping supplying the leaching cells. Excess inflow overflowed over a weir formed by an upright length of 20 mm UPVC pipe. This in turn drained into a flow indicator formed from a 200 mm length of 50 mm perspex tubing with UPVC end plates.

A stop cock at the base of the indicator retained the overflow for monitoring and an overflow from the top cap was provided. These indicators were used to monitor the rate at which overflow from the constant head tanks was occurring. The inflow to the constant head tank was adjusted as necessary using the needle valve so that the indicator filled in from one to three days. The 5 litre constant head tanks were translucent while the 25 litre tanks were equipped with sight tubes to enable the level to be monitored. Levels were checked and recorded daily and the needle valves adjusted as necessary. A 5 litre constant head tank and indicator are illustrated in the lower portion of plate 9.5.

This system was used for the leaching supply for all elements of the program other than the high gradient supply. Overflow from the carbonated water system was run to waste while that from the Middelburg water system was collected for re-use. The de-aired distilled water overflow was collected and periodically used to flush the leaching bath described in section 9.2.10, below.

PLATE 9.5
DE-AIRING AND CARBONATING VESSEL, WATER RESERVOIRS, CONSTANT HEAD TANK AND FLOW INDICATOR



Once the constant head tanks were commissioned and the leaching cell racks complete, final adjustment of the applied head was made by insertion of packing under the constant head tanks. An air pressurised manometer in which the applied pressure was monitored by means of either a water of mercury manometer, against atmospheric pressure, depending on head being measured, was used to permit final calibration to a close tolerance. This calibration also allowed an adjustment to be made for the Y piece error referred to in section 9.2.2. This calibration revealed an error in the pressure gauge used to control the cell pressure and high gradient head. As completion of the manometer system had been delayed beyond the start of testing, it was decided not to make an adjustment of this magnitude and the gradient in the high gradient tests was thus 136 as opposed to the 150 intended.

9.2.7 Minimising Gas Absorption in Leaching Supply

Large volumes of de-aired distilled water were required to provide for peak demand during cell installation as well as to provide a reserve over weekends and allow for possible still failure, this necessitated the use of large reservoirs with a free water surface. In addition, a free surface was also present in the constant head tanks.

In all phases of the long term percolation tests considerable care was taken to ensure that neither the percolating water nor the cell fluid absorbed air after being de-aired. The precautions applied to the cell fluid have been described above. Exceptions to the above were the high gradient cells and controls where compressed air was used for cell fluid, the carbonated water system where de-aired distilled water was saturated with CO_2 and the Middelburg water system where the water from the reservoir was used without de-airing.

In view of the free surfaces present, a number of steps were taken to minimise the gas absorbed by water in the tanks:-

a. All connections to reservoirs and constant head tanks through which de-aired water was introduced or tapped for use were placed at the base of the container. These tappings were located to minimise mixing and in the case of the constant head tanks to attempt to ensure that the outflow drawn for the cells was drawn from the inflowing water so that there was always an upflow of surplus water towards the free surface causing the water with the highest gas content to be continually skimmed off the surface. The constant head tanks were periodically flushed by opening a valve bypassing the needle valves in order to remove surface water.

b. The capacity of water to dissolve gases increases with decreasing temperature, accordingly, the water store was maintained at a nominal temperature of 30°C while that of the leaching laboratory was maintained at 25°C. Thus, as the water cooled as it passed into the specimens, any tendency for gas to be released in the specimen was inhibited.

c. The solubility of different gases in water varies considerably, the oxygen content of dissolved air is 33.8% as opposed to 21% in atmospheric air (Lange 1967). This suggested that the use of a relatively insoluble gas for the atmosphere in the reservoirs and constant head tanks would reduce the amount of gas that could be dissolved.

   The solubilities of the major components of air in water were estimated from figures provided in Hodgman, Weast and Selby (1955) as follows:-

Gas	He	Ne	N_2	H_2	O_2	Ar	CO_2
ml/l	8.20	9.69	14.4	16.8	27.7	32.0	730

   Helium and neon are both rare gases and relatively expensive and hydrogen presents the risk of explosion whereas nitrogen comprises 78% of atmospheric air and is readily available at low cost. High purity nitrogen was thus used to provide the atmosphere for the constant head tanks and water reservoirs.

A reserve vessel was connected in the line from the regulator on the nitrogen cylinder for use when the cylinder was being changed. The output from this reserve vessel was then further regulated to provide the input for a precision low pressure regulator which produced a final pressure of 150 mm of water in the constant head tanks and reservoirs. The output from the regulator was passed through water in a flow indicator and was routed to the various reservoirs and constant head tanks in turn. The 150 mm head was the smallest head that could be consistently maintained while at the same time allowing for rapid response as the system drained and was refilled.

The pressure was finally controlled by relief vents, containing a head of 150 mm of water above the outlet, through which excess pressure was vented during reservoir filling or in the event of the regulator adjustment varying. Under normal operation, the regulator was adjusted so that very slight venting was occurring at all times in order to obtain maximum regulator sensitivity. Pressure in the system was further monitored by a manometer. The levels of the constant head tanks allowed for the additional atmospheric head.

The nitrogen atmosphere was not employed on the Middelburg water system which remained open to the atmosphere. The nitrogen atmosphere was also used for the carbonated water system but was completely isolated to eliminate the risk of CO_2 reaching the rest of the system.

9.2.8 Distilled Water Supply

Distilled water was obtained from the Department of Chemistry in bulk and lifted to the top of the building using a compressed air pumping vessel. It was stored in drums and filtered through a 5 micron filter to remove spores and suspended matter. The distilled water was de-aired under vacuum using the de-airing and carbonating vessel shown in plate 9.5. After de-airing, the water was pumped to the storage reservoirs, some of which are shown at the top left of the plate. Details of the operation of this equipment are provided in appendix C.

9.2.9 Carbonated Water Supply

Carbonated water was produced by saturating de-aired water with carbon dioxide in the de-airing and carbonating vessel. The gas was slowly introduced through 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 several hours to ensure saturation and then reduced to atmospheric. The water was allowed to stand overnight to allow excess CO_2 to dissipate and then pumped to the storage reservoir.

9.2.10 Middelburg Water Supply

As previously noted, the water from Middelburg was not de-aired. It was stored in the heavy plastic containers in which it was originally collected and was pumped through a 5 micron water filter and into a reservoir on the same level as the reservoirs for other waters as required. Filtration was undertaken to remove algal spores and suspended solids as it was felt that while these might help to reduce the permeability of the specimens, they represented an unknown quantity which was best omitted. No de-airing or other treatment of the water was employed as it was intended to use the water in its natural state as nearly as possible.

As only 150 litres of water was collected, provision was made to recycle the overflow from the constant head tanks, however, very little reuse was required.

9.2.11 Leaching Bath and Humid Curing Chamber

As previously mentioned, the standard cure employed in the test program involved curing for 7 days in an atmosphere with a relative humidity of about 95%. Specimens cured for other periods in this environment were also tested. A leaching bath was also required to investigate a simpler test procedure.

The leaching bath required to be regularly flushed with distilled water to simulate the continuous removal of dissolved products that would occur in an open reservoir. This was achieved, as described in section 9.2.6 by collecting the overflow from the constant head tanks in a reservoir and using it to flush the leaching bath. This was done at least weekly but for much of the test program occurred much more frequently as adjustments were made to compensate for addition and removal of leaching cells.

An asbestos cement trough 0.9 x 0.3 x 0.3 m lined with polythene sheeting was used for the leaching bath. The specimens stood vertically on a plastic grid in the bottom of the bath so that water could circulate freely around them with a minimum depth of 20 mm above the specimens. Water was introduced through a perforated manifold running the length of the leaching bath and was drained off through an overflow weir. No significant scum formation occurred at any stage and the water remained clear at all times.

A second trough was placed adjacent to the first and connected to it. Plastic racks were placed above the water surface for the humid curing of specimens. A hinged lid was placed over both tanks to maintain the humidity. This lid did not seal completely, thus permitting a limited degree of air circulation since, as previously discussed, a controlled amount of evaporation during curing was required. The humidity was monitored daily and generally varied between 93 and 96%. This humid chamber was used for curing all specimens for the leaching, triaxial, unconfined and settlement programs. Plate 9.6 illustrates the leaching bath at the left with the humid chamber to the right.

PLATE 9.6
LEACHING BATH AND CURING CHAMBER

9.2.12 Leachate Storage and Analysis

Light proof racks were used to store the bottles of leachate prior to analysis. All bottles of leachate were sealed on completion of the sampling period and stored for future analysis.

A number of analysis techniques were evaluated with the help of Prof R.J. Keddy of the Nuclear Physics Research Unit of the

University, these included Neutron Activation in a nuclear reactor and Particle Induced X-Ray Excitation in an accelerator. Neutron activation was examined in some detail and a number of trials of the procedure were conducted.

The procedure adopted involved freeze drying of the leachate in high density polythene bags which were then placed in cartridges and passed through the research reactor at Pelindaba for activation. The magnitude of emissions corresponding to the radioactive isotope, calcium 47 ($_{20}Ca^{47}$) were then recorded at decay periods of 5 and 10 minutes and 7 days and processed by computer to determine calcium concentrations relative to standard samples. This technique, although accurate proved to be very time consuming and expensive. The results of analysis of the first trial leachate samples indicated that the quantities of calcium in the solutions were greater than initially expected and other techniques were accordingly investigated.

Six hundred samples were ultimately analysed by the Hydrological Research Institute of the Department of Water Affairs at Roodeplaat. A model 34000 Applied Research Laboratories, Inductively Coupled Plasma Emission Spectrometer operating on the 315.89 nm wavelength of Calcium was used. This apparatus conducts continuous multiple sampling of 1 ml samples of aqueous solutions under computer control to give a direct print-out in mg/l (parts per million) together with standard deviation of the result.

pH readings of corresponding leachate samples for comparison were taken with a standard digital pH meter.

9.2.13 Compaction and Curing of Specimens

All cylindrical specimens were compacted in the mould referred to in chapter 6. Disk specimens were compacted in a similar mould described in chapter 13. Specimens were transported to the humid chamber described in a closed container in which the specimens stood on a plastic rack above a pool of water to maintain the humidity.

The paraffin wax used to seal the specimens on completion of the humid cure, was maintained at the correct temperature in a domestic electric frying pan.

9.2.14 Penetrometer

PLATE 9.7
BITUMEN PENETROMETER

A standard Bitumen Penetrometer with 200 g needle mass was used to obtain an estimate of depth of deterioration of specimens on completion of leaching (plate 9.7).

9.3 TEST PROCEDURES

9.3.1 Specimen Preparation

The procedures used for the batching, compaction and curing of the standard 75 x 37.5 mm cylinders are summarised below, these procedures were used in preparing all the test specimens of this type used throughout the research program. The procedures used in preparing the 20 x 71.4 mm disk specimens used in the leaching and oedometer tests, differ only to the extent that the mould described in chapter 13 was used in place of the cylindrical mould to BS 1924 (1975).

Soil, oven dried at 50 °C, cement and distilled water were weight batched to the nearest 0.1 g in sufficient quantity to prepare six specimens and six moisture content samples. Mixing proceeded in accordance with a standard timed routine to ensure consistent results. The material was mixed in a domestic food mixer using the mixing hook illustrated in plate 6.1 with alternate periods of hand mixing. The dry soil and cement were mixed until uniform and the water added. The material was allowed to stand in a closed polythene bag for a period of five minutes after completion of mixing, mixing time was four and a half minutes.

On completion of the standing period, the required mass of soil-cement was weighed out and poured into the mould, tamping continuously with the tamping bar until it "walked out" of the material. The funnel was removed, the top plug inserted and the spacing collar removed. The mould was placed in a compression testing machine and compressed until flange closure which was maintained while a sample of the mixture was placed in a sealed moisture content tin. The load was then released and the specimen extruded, weighed to 1 mg, marked with its identifying code and placed in the humid drum. The mould was wiped clean, lightly oiled and reassembled. This procedure was repeated for each of the six specimens. Time to completion was recorded and was generally from 25 to 30 minutes after start of mixing.

On completion of compaction, the moisture content tins were weighed, opened and placed in an oven at 110 °C. The specimens were transferred to the humid chamber in the constant temperature laboratory.

Generally, between 24 and 36 specimens were prepared on one day and these were distributed between sets using the system referred to in section 7.4. In preparing specimens for the leaching test program where short sets due to damage to specimens could not be accommodated and where it was difficult to reschedule testing, an additional "redundant" set was compacted as a reserve in case of specimen damage.

Moisture content samples were weighed the following day and the batch average moisture content calculated as a weighted average of the six results. This figure was used in calculating the original oven dry specimen masses. The deviation from the nominal moisture content of all batches prepared on a particular day was used to adjust the allowance for hygroscopic moisture content in subsequent batches if necessary. Adjustment was however seldom required.

Specimens subject to the standard cure were stored in the humid chamber for a period of 7 days. They were then weighed and length and diameter measurements taken. A rectangle of aluminium foil and a strip of plastic covered wire, to aid stripping, were wrapped around the circumference of the specimen and held in place with a self-adhesive label with specimen and set identification. Disks of foil were placed on the ends of the specimen and the edges of the cylindrical foil folded over. The specimens were dipped in paraffin wax to seal them. Alternate ends were dipped with an overlap, an interval being allowed for cooling. Each end was dipped two times to achieve the desired thickness of wax.

The specimens were then regrouped into sets for testing and returned to the constant temperature laboratory where they were placed on racks for further curing. The standard cure period for sealed specimens was 21 days.

Standard control specimens remained sealed until tested at the same time as the corresponding test specimens. As far as possible, control specimens were compacted at the same time as the corresponding test specimens.

In a number of cases, corrosion of the aluminium foil as a result of the high alkalinity of the stronger soil-cement mixtures was observed. There was no indication of any detrimental effect on the specimens, but it is suggested that the use of alternative seals, such as plastic kitchen wrap, should be investigated.

9.3.2 Preparation for Testing

When ready for testing, the foil was removed and the specimen weighed and measured. Specimens for testing in the leaching bath were placed in the leaching bath and their grid location noted. Specimens were placed with the top, as compacted, alternately up and down.

Specimens for testing in the leaching cells were placed in the humid drum while all specimens in a set were stripped and were then assembled in the leaching cells. Plate 9.8 shows a specimen and the components of the leaching cell ready for assembly, an identifying label is fixed to the cell barrel. The 200 mm membrane was inserted in the cell and folded over the ends and the 150 mm membrane was placed on a conventional membrane stretcher. The top cap with gasket, filtram disk and clamping bolts was placed on a stand, the specimen was placed within the membrane on the stretcher and the cell used as a stretcher to enable the specimen plus short membrane to be placed in the cell. The short membrane was folded over the ends of the cell and the assembly placed on the top cap. The bottom disks, gasket and cover plate were assembled and the clamping nuts tightened.

PLATE 9.8

SPECIMEN AND COMPONENTS OF LEACHING CELL

In assembling the cells the top of the specimen, as compacted, was placed alternately at the top and bottom of the cell to allow for any non-uniform density that might arise from the method of compaction.

9.3.3 Leaching Cell Operation

The connecting manifolds for cell pressure and leaching water supply were attached to groups of three cells so that the specimens were in the correct sequence. The cells were placed on the racks, the leaching water connected to the base and the outlets connected to the vacuum manifold.

The cell pressure supply was connected to the manifold and the valve opened. The clamping nuts on the cells were slackened, if necessary, to allow air to escape from the cells. Once flushed, the cell pressure supply valve was closed and the clamping nuts tightened.

With the vacuum pumps off, water was run past the specimens to remove air from the connecting tubing and cell. When this water was free of visible air, the supply valve was closed and the vacuum pumps turned on. Vacuum was applied for approximately half an hour, the pumps were shut off and the cells flushed. The procedure of alternate flushing and application of vacuum was repeated several times until no further air was drawn from the cell under vacuum.

While de-airing was occurring, the drainage tubes were labelled and placed in the supporting clips and the leachate collection bottles were positioned and connected to the drainage tubes. Once the cells were de-aired, the vacuum lines were disconnected, cell pressure applied, the leaching supply connected, the drainage tubes connected and the starting time recorded.

Because of the large range of flows and the large number of tests being conducted simultaneously, it was not practical to use a constant sampling interval. Collecting bottle sizes were adjusted to give an estimated sampling period of 14 days. Bottle sizes ranged from 100 ml to 5 litre capacity. Readings of leachate mass were taken as sample bottles filled and were replaced. Where very low flows occurred, such that bottles took several months to fill or no change occurred at all, an attempt was made to take readings every four to six weeks. No attempt was made to improve the accuracy of readings for very low flow specimens as calculations indicated that they could be regarded as impervious as far as this research was concerned.

The supply manifolds were flushed at intervals during testing and samples taken for analysis to determine whether diffusion of leaching products into the supply was occurring.

In the case of the split specimens and certain disk specimens, initial flows were so high that a constant head could not be maintained. On the basis that the full hydraulic head would never be applied to the facing of a major dam overnight, particularly on first filling, the water supply to certain of these specimens was shut off for periods of several days between readings until the flow reached an acceptable level. Results for these specimens are thus not directly comparable to those for the remaining specimens but are nevertheless considered to be of value.

Two sets of long-term specimens were left in the leaching cells when the rest of the program was completed. Monitoring of these cells was undertaken by G.M. Bentel who was extending this research into different soils and stabilizers.

9.3.4 Leachate Analysis

Comprehensive sets of leachate samples were kept until the conclusion of the test program and are still available for further analysis should this be required. As mentioned previously, 600 leachate samples and reference solutions were analysed for calcium by the Hydrological Research Institute (HRI). Due to the cost and time required, no further analyses were conducted. A further 24 samples were subject to a more complete analysis by the HRI. The samples for calcium analysis were selected to give four data points per specimen, evenly distributed over the test period, for the majority of primary test specimens with two data points per specimen in the case of duplicate sets and certain secondary sets.

A number of reference solutions of calcium hydroxide, calcium carbonate and the supernatant liquid resulting from agitating fresh portland cement with water, were prepared in order to obtain an estimate of the maximum calcium concentrations obtainable. These were analysed together with the leachate samples.

Where leachate samples exceeded 500 ml in volume, 100 ml subsamples were decanted for submission to the HRI. Measurements of pH were performed on the samples prior to submission to the HRI with a view to investigating the correlation between Calcium concentration and pH suggested by Jackson (1965).

Due to time constraints, it was not possible to conduct further pH analyses nor to investigate the use of freeze drying or other evaporative techniques to determine total dissolved solids. It is considered that the use of an evaporative technique such as oven drying in suitable vessels, if correlated with a program of chemical analysis, could present a relatively quick, low cost technique to determine the extent of leaching as well as its relationship to leaching period.

9.3.5 Completion of Testing: Penetration Tests, etc

On completion of the specified period of leaching, the flow and cell pressure were shut off, the drain tubes disconnected and the final leachate sample mass determined. The state of the collection and supply tubes was noted. Groups of cells were removed from the racks and the connecting manifolds removed.

Each cell was stripped using the cell as a membrane stretcher to remove both membranes together. Care was taken to avoid material loss and the state of the inlet and outlet ends of the specimen and the filter fabric disks were noted. The specimen was weighed to the nearest 1 mg.

In many cases, significant softening of the inlet end of the specimen was observed. It was considered desirable to attempt to measure the depth of this deterioration as well as to provide some basis for determining whether the specimens were sufficiently intact for triaxial testing. It was felt that strength testing of badly deteriorated specimens would give meaningless results and lead to loss of material. The final oven dry mass was considered an important value for determining the extent of leaching in terms of loss of mass. A bitumen penetrometer provided the necessary data. Plate 9.7 illustrates the penetrometer with a specimen in position.

Penetration tests were conducted on the inlet end of all specimens from the leaching cells and on the outlet end of those specimens where the inlet was sufficiently intact to prevent particle loss while the outlet was tested. Both ends of all specimens from the leaching bath were tested.

The test was performed by visually aligning the tip of the penetrometer needle with the surface of the specimen and releasing the needle stem and weights which had a total mass of 200 g. Penetration was read directly to 0.1 mm. Four penetration readings were taken on each end of each specimen tested.

After consideration of the factors affecting the penetration reading, in particular the maximum 4.75 mm particle size, the average of the two greatest readings for one end was selected as the representative numerical value of penetration. A number of trial unconfined compression tests were conducted to evaluate the behaviour of leached specimens in compression and it was concluded that specimens with a penetration value greater than 1 mm on either end were too soft to warrant triaxial testing. All specimens for which penetration exceeded 1 mm were placed on a tray and oven dried at 110 °C to constant mass.

Specimens with penetration within the 1 mm limit were tested in triaxial compression as discussed in chapter 11, care being taken to avoid material loss during assembly and stripping of the triaxial cells. The oven dry mass of these specimens was then determined.

Where leach bath sets corresponded to sets tested in the leaching cells, the leach bath specimens wer[illegible]sted triaxially on the day after the leach cell specimens. C[illegible]l specimens were tested in triaxial compression on the day following the corresponding leach bath specimens or following the leach cell specimens if there was no corresponding leach bath set. In cases where control specimens were not compacted at the same time as leached specimens, they were tested at the corresponding age. Redundant sets were generally utilized in the unconfined compression program described in chapter 10.

9.3.6 Cracked Specimens

Cracked specimens were split at age 28 days using the apparatus described in chapter 10. After splitting, any loose material was blown off the crack surface and the two halves of the specimen carefully placed together. Two O-rings of a diameter which just held the specimen together without applying pressure, were placed around the specimen.

Sealed specimens were sealed in the normal manner and leach bath specimens were placed in the bath. Specimens for testing in the leaching cells had plasticine worked into the crack on the surface of the specimen. A short membrane, the length of the specimen was placed over the specimen with the O-rings over it. The specimen was then placed in the cell in the normal manner.

9.4 TEST RESULTS

The results of the leaching test program fall into a number of categories as listed below, and are discussed accordingly:-

a. Analysis of reference solutions and manifold water for calcium.

b. Detailed chemical analysis of selected leachate and reference solutions, results summarised in table 9.1.

c. Time dependent parameters for specimens tested in the leaching cells, for which detailed records were kept. This data comprises the pH data and the permeability and calcium leaching rate data. The latter data is presented in detail in appendix D in graphs D.1 to D.38 and compared in graphs 9.1 to 9.21.

d. Mass change data from control specimens to determine the hydration correction to be applied to leached specimens. This data is presented in graphs 9.22 and 9.23.

e. Mass change data for specimens tested in the leaching bath and presented in graphs 9.25 and 9.26.

f. Summary data for the specimens tested in the leaching cells, presented in graphs 9.27 to 9.30 and table 9.3. The symbols used in these graphs correspond to those indicated in the legends of graphs 9.1 to 9.21 and listed in table 9.3. To facilitate reference, a fold-out legend is attached to table 9.3, after graph 9.30.

g. Comparison between the results of the wetting and drying tests, leaching bath and leaching cell results, presented in graphs 9.31 to 9.34.

9.4.1 Calcium and pH Analysis of Distilled Water

Calcium concentrations in reservoir water and manifold water collected during the test program did not differ significantly from zero, indicating that no backward diffusion of calcium into the supply had occurred.

pH readings for these samples ranged from 6.4 to 7.7 for distilled water as received from the Department of Chemistry, 5.8 to 7.7 for de-aired distilled water from the reservoirs and 6.9 to 8.1 for de-aired distilled water from the leaching supply manifolds. No trend is apparent from this data. While some contamination of the water in the manifolds by leaching of the piping may have occurred, the spread of results suggests that this is unlikely.

Because of the balance of hydrogen ions in pure water, it is possible that the pH meter was susceptible to errors due to incorrect adjustment or incomplete cleaning of the probe. It should be noted that these samples were randomly distributed amongst samples of leachate and the prevalent sample type was alkaline, this could have biased the readings for the water samples.

9.4.2 Calcium and pH Analysis of Carbonated Water

Calcium content of carbonated water and carbonated manifold water was also zero.

pH ranged from 5.0 for freshly carbonated water, to 6.9 for water that had stood in the reservoir for several months and 7.5 for manifold samples taken a year or more before pH testing. The first two pH readings referred to were taken independently of the other readings while those of the manifold water were interspersed with leachate samples so that the remarks in the previous section are relevant. It would however appear that a significant reduction in CO_2 content of the carbonated water occurred over a period. It should be noted that the carbonated water was supersaturated immediately after processing and that a reduction in CO_2 content was to be expected.

Neutralisation by way of leaching of the piping and fittings may have occurred but no data is available to evaluate this. It is unlikely that any other factor could have caused the CO_2 content of the water to be reduced below the equilibrium level that would apply in the case of water in an open reservoir and it was not established whether this had occurred.

It should be noted that very aggressive waters, rich in CO_2, are generally encountered in areas where considerable decay of organic matter is occurring in the reservoir. This leads to continuous evolution of CO_2 and hence a CO_2 content greater than the equilibrium level. In order to simulate this condition, it would have been necessary to either pass CO_2 through the reservoir water continuously or to recarbonate frequently.

It is therefore unlikely that the carbonated water used in this program was as aggressive in terms of dissolved CO_2 as might occur in the field. Further research in this direction is required.

9.4.3 Calcium and pH Analysis of Middelburg Water

Calcium content of Middelburg water was consistently +/- 23 mg/l (ppm) for samples drawn from both the reservoir and manifold during the test period.

Average pH of reservoir and manifold samples was 8.2 with a range of 8.0 to 8.4 confirming the presence of dissolved substances in the water.

9.4.4 Calcium and pH Analysis of Reference Solutions

The reference solutions for calcium hydroxide ($Ca(OH)_2$) in distilled water provided good agreement between pH and calcium content but indicated a very narrow band of sensitivity. Observations ranged from pH 6.9 at 0 mg/l Ca, through pH 12.1 at 11, 12.5 at 180 and 12.6 at 310 to pH 12.8 at 500 to 600 mg/l Ca. Saturated solutions of calcium hydroxide in Middelburg water and new and old carbonated water all indicated a pH in the range 12.73 to 12.85 with calcium concentrations ranging from 540 to 690 mg/l. This lack of sensitivity in the calcium analysis technique at high concentrations was anticipated as the testing machine was calibrated for maximum accuracy at lower concentrations in the expectation that high leaching rates would be readily evident from the mass loss data for the test specimens.

Observations for the saturated reference solutions of calcium carbonate ($CaCO_3$) in distilled water covered pH from 8.28 to 8.32 and calcium level from 17.8 to 18.6 mg/l. It is notable that this level is comparable with that for the Middelburg water.

Observations for the supernatant liquid produced by mixing unhydrated portland cement with distilled water indicated pH from 12.01 to 12.06 and calcium level from 69 to 81 mg/l.

These results confirm a considerable variation in the solubilities of various forms of calcium. It is also notable that there is no particular relationship between pH and calcium level for the various solutions. In particular, in the case of calcium hydroxide, the pH is considerably more sensitive to the presence of hydroxide ions than to the presence of calcium.

9.4.5 Detailed Chemical Analyses

The results of the detailed chemical analyses performed by the HRI are presented in table 9.1. Only 24 samples were analysed due to

TABLE 9.1
DETAILED ANALYSES OF LEACHATE, ETC

Serial	Description	Soil-Cement Specimen No	Sample No	Leach Time (days)	Na	K	Mg	Al	Sr	Si	Ca	pH
					Concentration of Elements mg/l (ppm)							
1	Top Manifold	–	12		0.00	0.00	0.31	0.03	0.00	0.00	0.00	7.10
2	Bottom Manifold	–	1	–	0.00	0.00	0.03	0.00	0.01	0.00	1.60	7.88
3	Cement Solution	–	1	–	4.41	37.88	0.00	1.04	1.00	0.83	81.20	12.01
4	Cement	–	3	–	4.09	34.67	0.00	1.33	0.89	0.57	78.50	12.06
	LEACHATE											
5	0% Cement	16-00/58	2	54	3.14	7.59	7.81	0.02	0.07	3.84	11.10	8.52
6			10	270	0.00	2.14	4.02	0.11	0.04	3.09	5.30	8.06
7	5% Cement	08-05/84	2	23	0.00	5.50	0.00	6.88	0.66	0.35	24.10	11.17
8			15	257	0.00	0.50	0.00	0.75	0.02	10.05	22.60	7.96
9	10% Cement	29-10/63	2	49	0.00	15.14	0.00	1.01	0.94	0.94	17.40	9.33
10			4	92	0.80	10.52	0.00	7.25	0.49	0.06	103.90	11.89
11			11	262	0.00	2.17	0.00	0.77	0.10	0.00	33.00	8.05
12		01-10/63	5	78	0.00	5.65	0.00	1.51	0.24	0.00	27.40	11.98
13		08-10/63	9	256	1.45	9.85	0.12	1.07	0.21	0.32	35.80	10.75
14		34-10/63	11	254	0.00	1.35	0.08	0.79	0.06	3.17	26.90	7.93
15	15% Cement	15-15/67	2	20	11.35	18.01	0.00	1.41	1.01	0.00	562.10	12.64
16		08-15/67	15	251	0.00	0.00	0.15	1.06	0.03	7.05	25.60	8.71
17	20% Cement	15-20/88	2	13	9.70	16.04	0.00	1.11	1.27	0.00	495.50	12.66
18			19	250	0.00	0.31	0.00	1.73	0.03	7.18	23.90	10.26
19	Middelburg Mfld	–	1	–	23.22	5.79	12.82	0.16	0.31	2.62	23.60	8.38
20		–	12	–	22.88	5.24	12.25	0.13	0.30	2.54	23.20	8.28
21	10% Middelburg	02-10/42	1	31	32.49	59.18	0.00	1.71	4.12	0.00	384.20	12.43
22			4	141	24.15	58.19	0.29	0.31	0.97	0.25	20.70	8.30
23	10% Carbonated	04-10/42	2	71	8.37	58.73	0.00	3.60	2.64	0.43	89.50	11.91
24			9	266			sample too small				24.10	8.12

cost and time constraints. The elements for which the analysis were performed were selected on the basis of those normally examined by the HRI in water samples and is not comprehensive. Analysis for zinc and copper, to check for corrosion of brass fittings, and an analysis for iron, as the soil used is susceptible to leaching of ferric salts, would be desirable extensions.

The following observations are based on a very limited number of analyses for which the sampling period varied considerably. Since in most cases, a changing concentration is noted, the variable sampling period prevents any quantitative assessment of the data. However, the purpose of this analysis was primarily to establish whether leaching of elements other than calcium could have a bearing on the results. A detailed analysis of the cement used in preparing the test specimens is presented in table 9.2 for reference.

A number of trends are evident in table 9.1:-

a. The manifold water samples (1 and 2) do not contain significant quantities of any of the elements examined.

b. The supernatant liquid of cement (3 and 4) contains appreciable amounts of sodium (4 mg/l) and potassium (35 to 38 mg/l) and small amounts of aluminium, strontium and silicon. Very small percentages of sodium and potassium are present in the cement (table 9.2), and in both cases take the form of readily soluble oxides. It would appear that most, if not all, of these elements present in the cement dissolved immediately in the form of the relevant hydroxide. This, in addition to the presence of calcium hydroxide, is the probable cause of the high recorded pH referred to in section 9.4.4.

c. The water from Middelburg (19 and 20) contains appreciable quantities of sodium, potassium, magnesium and silicon in addition to the 23 mg/l of calcium noted in section 9.4.3. Traces of aluminium and strontium are also present. However,

TABLE 9.7
CHEMICAL ANALYSIS OF CEMENT USED

Compound	Percentages of:- Compound in Cement	Element in Compound	Element in Cement
SiO_2	21.93	46.74	10.25
Al_2O_3	3.00	52.93	1.59
Fe_2O_3	3.70	69.94	2.59
Mn_2O_3	.20	69.60	.14
TiO_2	.40	59.95	.24
CaO	64.75	71.47	46.28
MgO	2.40	60.30	1.45
P_2O_5	.06	43.64	.03
SO_3	1.98	40.05	.79
K_2O	.35	83.02	.29
Na_2O	.09	74.19	.07
Loss @ 950°C for 1/2 hr	1.00		
TOTAL	99.86		63.71

Analysis performed by Pretoria Portland Cement despite the presence of sodium, potassium and calcium, the pH is relatively low (8.3) indicating that other factors in addition to those noted in the previous paragraph have a bearing on the pH.

d. The early leachate from the unstabilized specimens leached with de aired distilled water (5) indicates the presence of the same elements present in the Middelburg water, albeit in somewhat different proportions, indicating that the reservoir water is approximately in chemical equilibrium with the soil. Significant reductions in the amounts of all these elements is apparent in the sample obtained after 270 days of leaching (6), indicating that limited amounts of these elements are present in readily soluble form.

e. The analysis of leachate from specimens containing 10% cement and leached with Middelburg water (21 and 22) indicates that sodium is not deposited and that leaching of sodium may increase in the early stages when compared with specimens leached with de-aired distilled water.

With prolonged leaching, the levels of calcium and sodium reduce to those present in the leaching water. In the case of sodium, this is consistent with the observed negligible level of leaching from stabilized specimens with less than 15% cement (7 to 14) or long leaching periods (16 and 18). In the case of calcium, the level at 141 days of 21 mg/l is comparable to the level of 23 mg/l present in the water suggesting that no further leaching of calcium occurred with Middelburg water. The early calcium leaching rate of 380 mg/l (21) is comparable with that of specimens leached with de-aired distilled water.

f. It is particularly notable that the rel. .ively high level of magnesium in the Middelburg water is completely absent in the leachate, suggesting that absorption of magnesium by the hydrating cement occurred. This is supported by the observation that while leaching of magnesium from the unstabilized soil by distilled water is apparent (5 and 6), no significant amounts of magnesium are indicated in the leachate from any of the stabilized specimens. Further research is required to determine whether this indicates additional hydration or pozzolanic reaction and whether the reaction is beneficial.

g. Leaching of sodium and potassium is significant in the early stages of percolation, particularly at higher cement contents (15 and 17), but after prolonged percolation, no further leaching is evident (16 and 18). This is consistent with the observations for the cement solution in paragraph b. Leaching of aluminium and strontium is generally at low levels initially with a gradual reduction in level with time.

h. Leaching of silicon is generally more pronounced after prolonged percolation, but no definite trend is apparent.

i. Leaching of calcium generally starts at a high level, decreasing rapidly to a level of between 20 to 30 mg/l. This aspect is discussed in more detail in the sections that follow.

9.4.6 Correlation between pH and Calcium Content

In general, the data in table 9.1 indicates a decline in pH with increasing period of percolation within a particular category of specimen and test condition. This is consistent with the generally observed decrease in concentration of the elements presented in the table.

Examination of the results of the pH and calcium analysis for the 600 samples analysed by the HRI indicated that while a high pH frequently occurred with a high calcium concentration and vice versa, considerable variation in pH for similar calcium levels was observed. This is particularly evident in the comparison between the data for calcium hydroxide and cement solutions discussed in section 9.4.4. It was concluded that the presence of other ions, soil by distilled water is apparent (5 and 6), no significant amounts of magnesium are indicated in the leachate from any of the stabilized specimens. Further research is required to determine whether this indicates additional hydration or pozzolanic reaction and whether the reaction is beneficial.

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It was therefore concluded that pH could not be considered to be an accurate measure of the extent of calcium concentration in solution.

9.4.7 Detailed Results of Tests in Leaching Cells

The permeability and calcium rate data is presented in graphs 9.1 to 9.21, grouped according to test parameters in order to facilitate comparison. The data for individual sets, with each specimen distinguished, is presented in appendix D for reference in clarifying any trends which are not clear in the grouped plots. Summary data for each specimen, together with set averages, is tablulated below each graph in appendix D and gives an indication of the spread in the various summary parameters discussed in this chapter.

To facilitate comparison, all permeability and calcium rate data is plotted to the same scale. In certain instances where the leaching period exceeds the 300 day standard scale limit, a plot over the range 0 to 1000 days is presented with a second plot of the data in the 0 to 300 day range for comparison.

Leachate masses were converted to volumes for calculation of permeability using specific gravity corrected to 25 ^{o}C (Lambe 1951).

The permeability axis in all plots is a log scale, consistent with the general practice of referring to permeability in terms of powers of ten in a continuous spectrum with no absolute zero. The range is centered on a value of $1x10^{-9}$ m/s which is approximately the median value for the results and is the value referred to in chapter 1. The scale range of $0.001x10^{-9}$ to $1000x10^{-9}$ covers the range $1x10^{-12}$ to $1x10^{-6}$ m/s recommended for the impervious sections of earth dams and dykes by Casagrande and Fadum (Justin, Creager and Hinds 1945). A permeability of less than 10^{-9} m/s (10^{-7} cm/s) is considered "practically impermeable" by Terzaghi and Peck (Lambe and Whitman 1969).

In physical terms, with unit gradient, the permeability can be considered to be equivalent to the rate of advance of a wetting front into the material. In these terms, a permeability of 10^{-9} m/s corresponds to a rate of advance of the order of 31.5 mm/year.

Low flows were obtained from certain specimens as a result of very low permeabilities or low gradients. In some cases, cumulative leachate volume was less than 100 ml over the 266 day test period and in certain cases no flow occurred for several months. In a few cases, evaporative losses of up to 1 ml per month were recorded where the parafilm seal was damaged. These occurrences are reflected by discontinuities on the plots and occurred particularly with the specimens subject to the lower hydraulic gradient (10 m/m) and the microconcrete specimens.

Although very few instances of reduction in leachate mass during a collection interval were observed, it is reasonable to assume that at low flow rates, evaporation from the anti-siphon vents on the collection pipes may have reduced the measured permeability. A permeability of 10^{-10} m/s at gradient 50 is thus considered to constitute the lower limit for which repeatable measurements can be obtained with the apparatus used in this research. Above this level the leachate volume increases rapidly and evaporation becomes insignificant.

The units of %/d were chosen for presenting the calcium leaching data after various alternatives were examined. These units have some physical meaning and provide a basis for comparison with the summary results considered in subsequent sections. The percentage is expressed in terms of the original calcium content of the specimen, based on the chemical analysis of the cement performed by the manufacturers and presented in table 9.2 The limits of calcium concentration were well defined, ranging from a lower detection limit of 1 mg/l to an upper saturation limit of about 600 mg/l. Converted into a leaching rate the range of possible values and the need for separation at low rates indicated the use of a log scale.

As discussed in section 9.1.1, the principal mechanism of corrosion of concrete is by removal of calcium. Since hydration does not affect the calcium content of the cement, the measurement of degree of leaching of calcium expressed as a percentage of initial calcium content is approximately equivalent to the degree of leaching of cement expressed as a percentage of initial cement content adjusted for hydration.

As there was very little loss of soil in either the leaching cells or the leaching bath, the percentage loss of calcium is also comparable with the percentage change in mass of the specimens at the end of testing although a discrepancy due to leaching of the soil may occur.

Most of the graphs indicate a decreasing rate, for the leaching rate - time relationship and the hydration - time relationship is similar. Application of a hydration correction to the summary results is therefore difficult. The limited number of calcium analyses does not permit a detailed investigation of the leaching - time relationship although graph 9.24 does give an indication of the general form. The true loss will lie somewhere between that indicated by a comparison of the final mass with the initial specimen mass and that indicated by comparison with the specimen mass adjusted for the hydration gain indicated by control specimens. Consideration of graph 9.24 indicates that the true value will probably lie closer to that determined in terms of the initial cement content. However, the value obtained with reference to the control specimens will overestimate the degree of leaching, and is thus more conservative. The loss adjusted in terms of the control value was thus used for the summary data presented in graphs 9.22 to 9.32.

The limiting values for specimen mass change, expressed as %/d are tabulated below the graphs in appendix D for comparison with the plotted data. The column headed "Int %/d" is the total mass change (loss negative) relative to the initial specimen mass and that headed "Ctl %/d" is the same mass change relative to the control specimen mass. It should be noted that the units of the calcium rate axis are %/d x 10^{-3}, loss positive (leaching rate), while the mass change figures below the graphs are in %/d (loss negative), these figures must therefore be multiplied by 1000 before comparison with the plotted data. The moisture content at the start of leaching (Imc %), penetration at end of leaching (Pen mm) and total flow through the specimen (Flow l) are also summarised for each specimen below the graphs in the appendix.

It should be noted that the averages of the data presented in appendix D do not always agree exactly with the figures presented in table 9.3. The summary data was calculated independently and certain points for which there were notes indicating likely errors due to material loss on handling, etc were disregarded whereas these points are included in the appendix.

The average mass change rates are generally of similar magnitude to the plotted data but tend to be slightly higher. This is consistent with the inclusion of non-calcium losses in the average data and the high initial leaching rate indicated in graph 9.24. Generally, the first leachate sample for each specimen was not analysed as it was felt that they were more likely to have been contaminated during setting up. There was also a greater error potential in the first permeability value. Data for these samples are thus not included on the calcium rate plots. It is probable that the highest leaching rate for many specimens occurred during the first interval, consequently, the exclusion of this interval has the effect of depressing the peak leaching rate shown on the graphs.

As discussed in section 9.1, the leaching test program was designed on the basis of a set of standard conditions which were each varied separately in order to examine their influence on the results. The convention adopted in titling the graphs in this chapter and appendix D is that only those parameters that are non-standard are indicated, the remainder are regarded as default parameters. The standard conditions are summarised in table D.1 in the appendix.

The combined graphs are discussed in the sections that follow. In most cases, data for one or more standard cases is plotted to provide a frame of reference. The legend entries include the numbers of the graphs in the appendix to facilitate reference if individual trends are not clear.

9.4.7.a Standard Specimens and Test Conditions

The data for the standard test specimens compacted at various cement contents are plotted on graphs 9.1 to 9.5 and summarised on graph 9.6. The corresponding data for the disk specimens is also plotted on graphs 9.1 to 9.5 and is summarised on graph 9.7.

Calcium leaching rate was not calculated for the unstabilized specimens as they contained no cement and therefore represent the base level of zero loss. The calcium content of the soil was not determined. A level of calcium leaching was however observed as indicated in table 9.1. Initial concentration was generally of the order of 10 mg/l with a final level of the order of 5 mg/l, generally about 20% of the final concentration observed for stabilized specimens. This indicates an appreciable amount of calcium in the soil. This is also indicated by the base level of calcium observed in the Middelburg water.

Permeability for the unstabilized specimens increased steadily to about 80 days and then decreased slightly before levelling off. The initial increase is considered to result from the dissolving of pore air while the subsequent decrease could result from movement of fines or consolidation of the specimen. On stripping the cells, constriction of the inlet end was observed, however, similar reductions in permeability were observed for the stabilized specimens so it is considered more likely that this reduction resulted from a redistribution of fines. The disk specimens exhibited similar permeability but with a very gradual increase to meet the asymptote for the cylindrical specimens.

The cylinders with 5% cement exhibit a steeper initial increase in permeability than the unstabilized specimens followed by a similar levelling off but with a greater spread. It is possible that the increased steepness resulted from the leaching of unhydrated cement in the early stages. The initial downward trend exhibited by certain specimens may have resulted from inadequate de-airing. The early calcium rate data was not obtained but the remaining data generally exhibits a slight downward trend with time.

As previously noted, the disk specimens initially exhibited a very high permeability, presumably due to their limited thickness (20 mm). However this converges rapidly on the curves for the cylindrical specimens and at 30 days is virtually indistinguishable from them. Thereafter the data lies on the upper edge of the envelope of results for the cylinders. The early calcium rate data indicates an initial high leaching rate but also converges on that for the cylinders. It should be noted that the supply to all stabilized disk specimens was shut off between readings in the first weeks and that this could have had a significant effect on the long term behaviour.

Graph 9.3 depicts the data for the specimens leached for 904 days together with the standard specimens and confirms that the long term set behaved the same as the other sets over the common test period. Graph 9.3a shows the same data on a scale to 1000 days covering the full period of the long term tests. A significant drop in permeability occurred starting at about 400 days. This suggests that some secondary hydration or pozzolanic effect may have reduced the permeability at this stage. Additional research is required to confirm this observation. The leachate samples were analysed at this point and no leachate analysis is available for the last 500 days of the test, however graph 9.24 indicates a reduction in rate of leaching over this period.

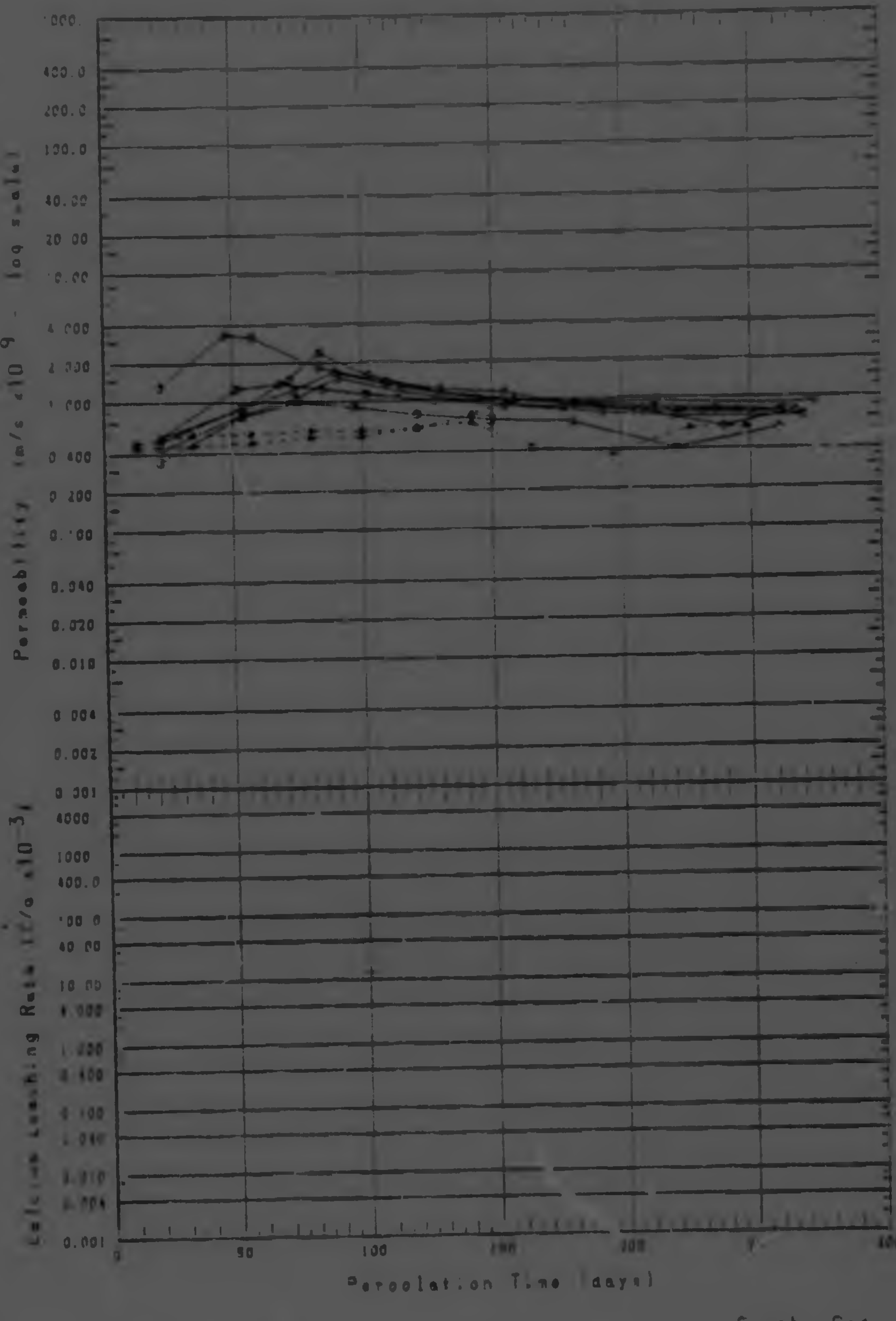
The specimens with 10, 15 and 20% cement all exhibit similar trends to those with 5% cement with a progressive decrease in the extent of the initial upturn, presumably as a result of hydration products reducing the pore volume more rapidly. They all exhibit a similar subsequent downturn. The calcium rate data is coincident for all cement contents.

The results are summarised on graphs 9.6 for cylinders and 9.7 for disks. In both cases there is an order of magnitude increase in permeability from unstabilized to stabilized specimens. All specimens were compacted at the same nominal dry density, thus, since the specific gravity of cement powder is approximately 3.15 (USBR 1975) while that of the soil was 2.7, an increase in void ratio occurs with increasing cement content. However, there is no appreciable difference in permeability over the range of specimens with cement contents from 5 to 20% indicating that the increase in nominal void content is compensated for by the expansion of the hydration products. This suggests that the effective specific gravity of the hydrated cement is very similar to that of the soil particles.

The calcium leaching rate spans the same range for all cement contents which is consistent with the results for permeability. It is notable that the final calcium rate lies largely in the range 0.01 to 0.1 %/day with a very gradual linear downward trend. This indicates complete removal of calcium in 1000 to 10 000 days. While graph 9.24 suggests a substantial decrease in leaching rate by 300 days, it is possible that complete removal of cement may occur. It must however be

GRAPH 9.1

SUMMARY OF RESULTS — UNSTABILIZED SPECIMENS



Graph Code

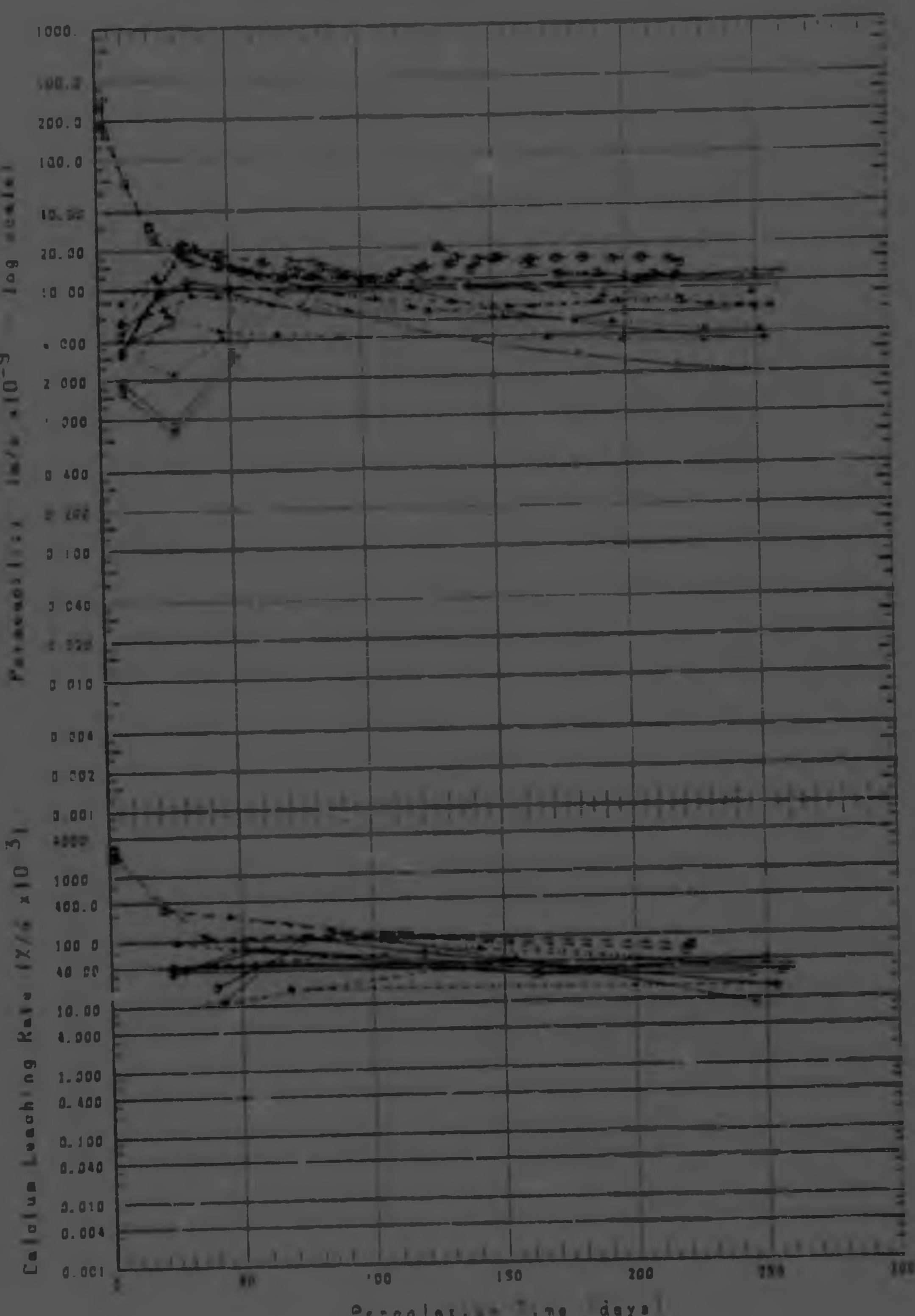
Unstabilized Cylinders (0% Cement . . . . 0.1 .. 1

Unstabilized Disks .... 0.35 .. 2

CONS-00 PLT -- 28/03/84 ...

GRAPH 3.2

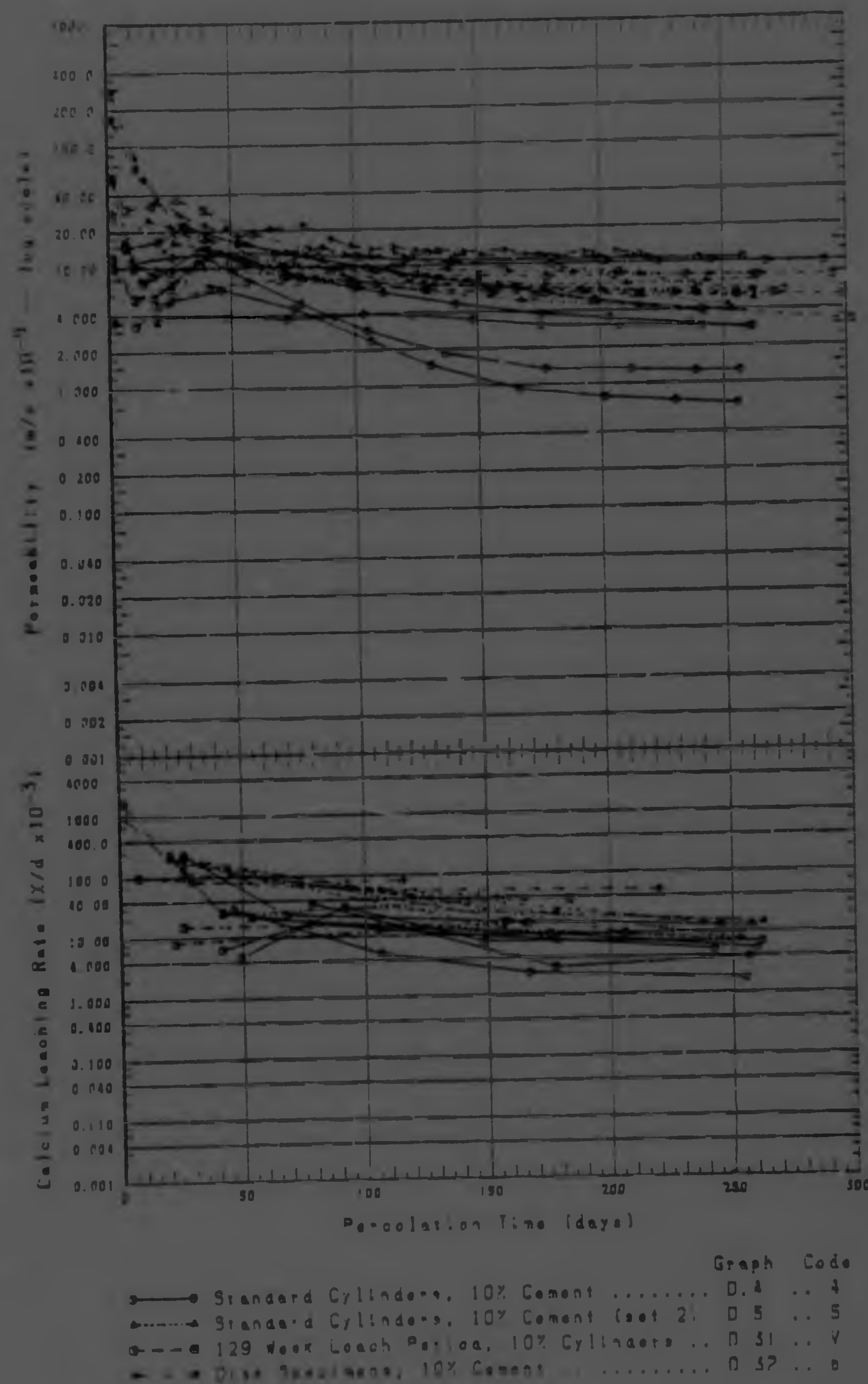
SPECIMENS STABILIZED WITH 5% CEMENT



	Graph	Code
Standard Cylinders, 5% Cement	D.2	.. 2
Standard Cylinders, 5% Cement (set 2) .	D.5	[illegible]
Disk Specimens, 5% Cement	D.56	[illegible]

GRAPH 9.5

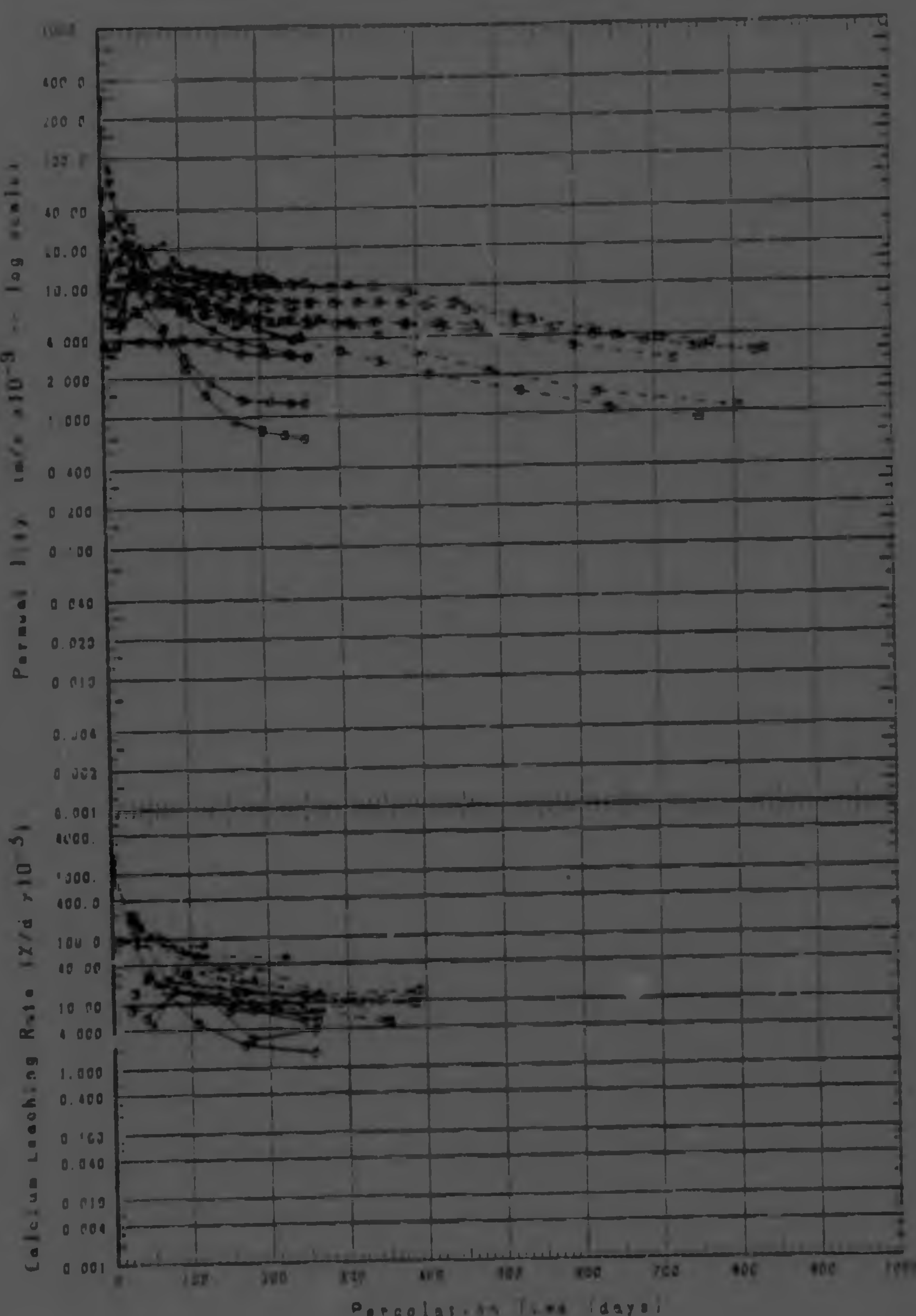
SPECIMENS STABILIZED WITH 10% CEMENT



COMB--O PLT -- 27/03/84-0e1216-v 1.0 (LCH-CONF ORT 0 1

GRAPH 9.5a

SPECIMENS STABILIZED WITH 10% CEMENT (129 WEEK LEACH)

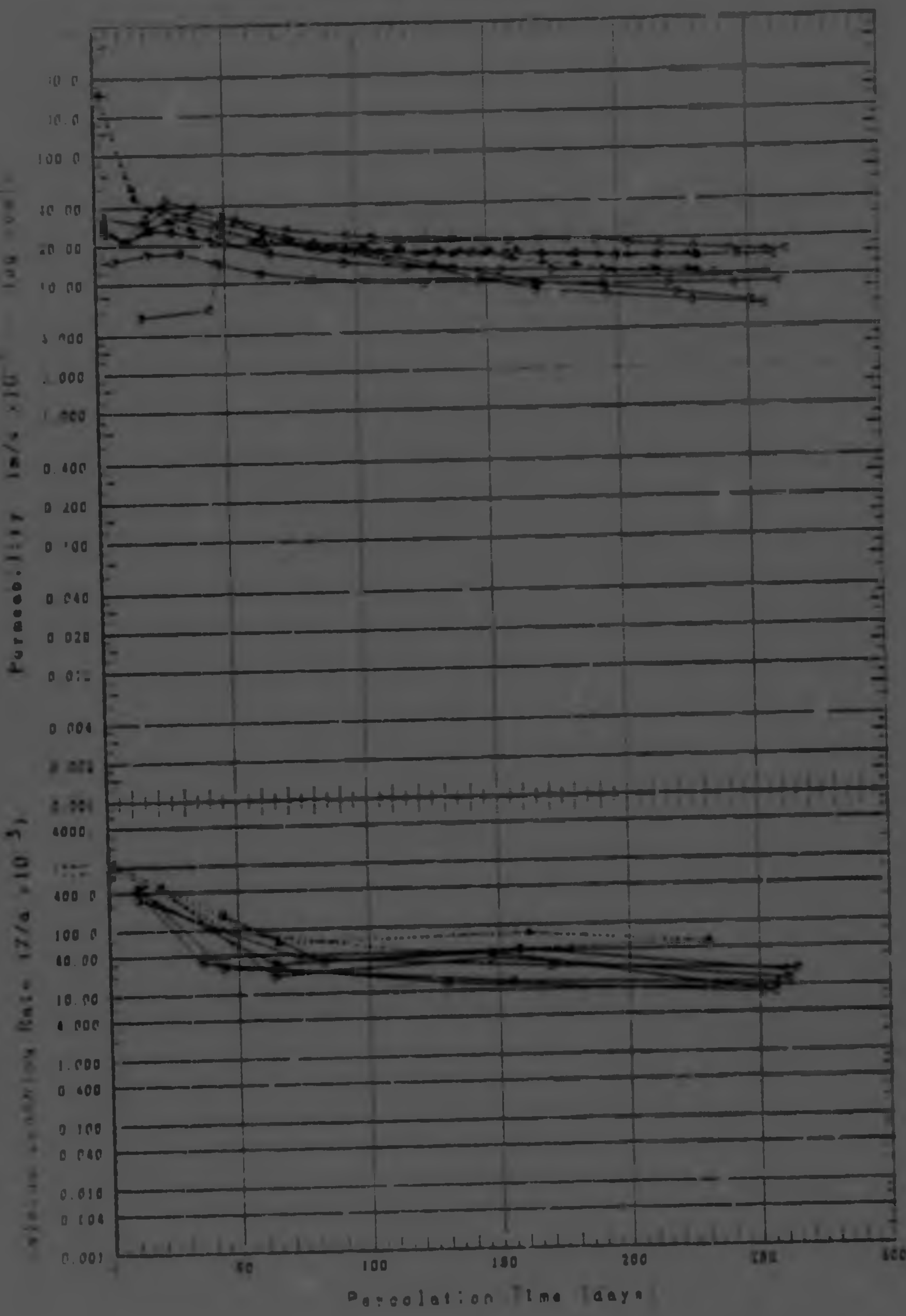


		Graph	Code
———	Standard Cylinders, 10% Cement	D.4	.. 4
- - -	Standard Cylinders, 10% Cement (set 2)	D.5	.. 5
- - -	129 Week Leach Period, 10% Cylinders ..	D.51	.. √
— —	Disk Specimens, 10% Cement	D.57	.. o

CONS-IDR PLT 30/03/84-9= 310-- 1.0 LH-CONF DAT 10

GRAPH 9.4

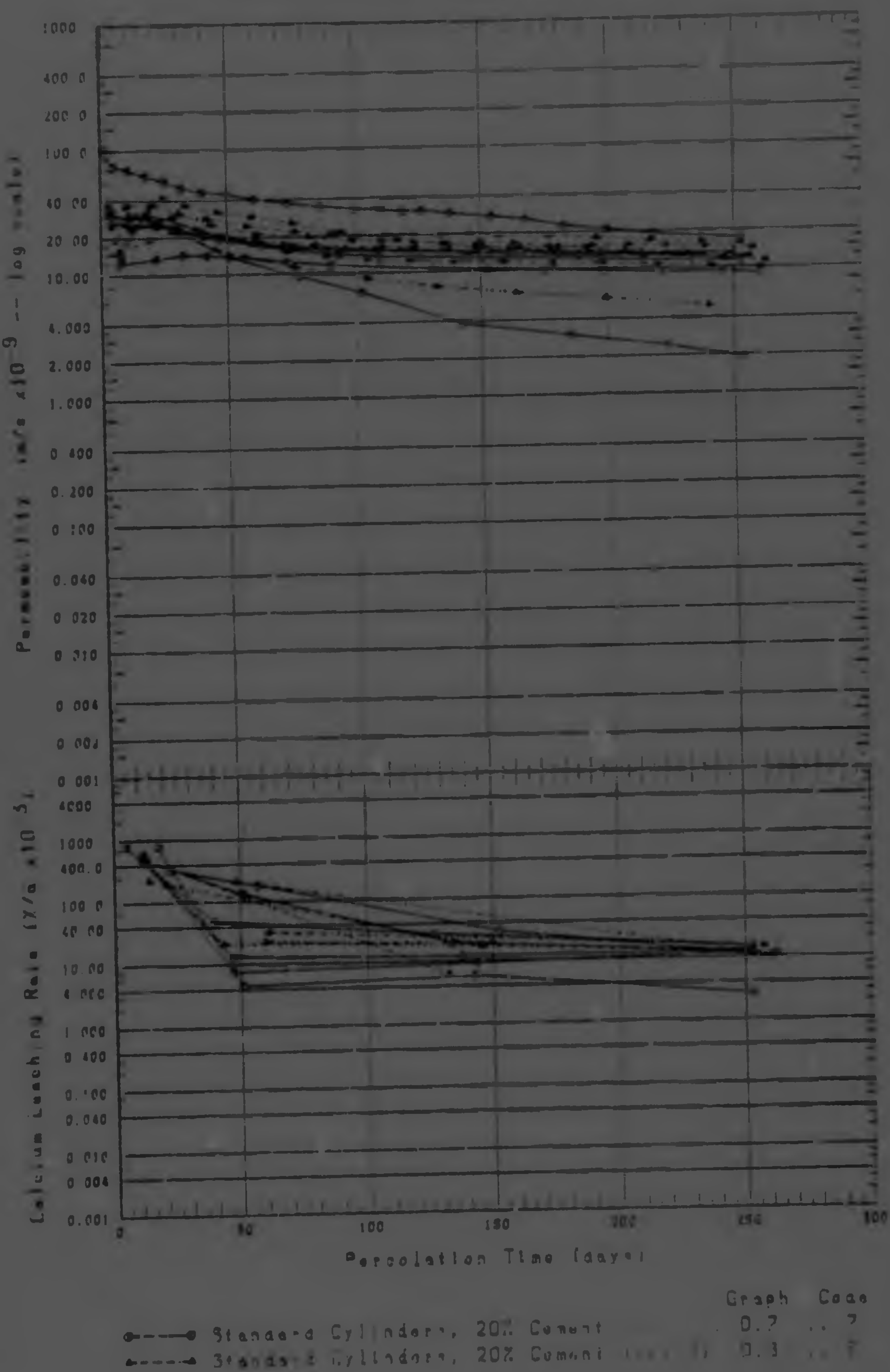
SPECIMENS STABILIZED WITH 15% CEMENT



	Graph	Code
Standard Cylinders, 15% Cement	D.5	b
Disk Specimens, 15% Cement	D.58	a

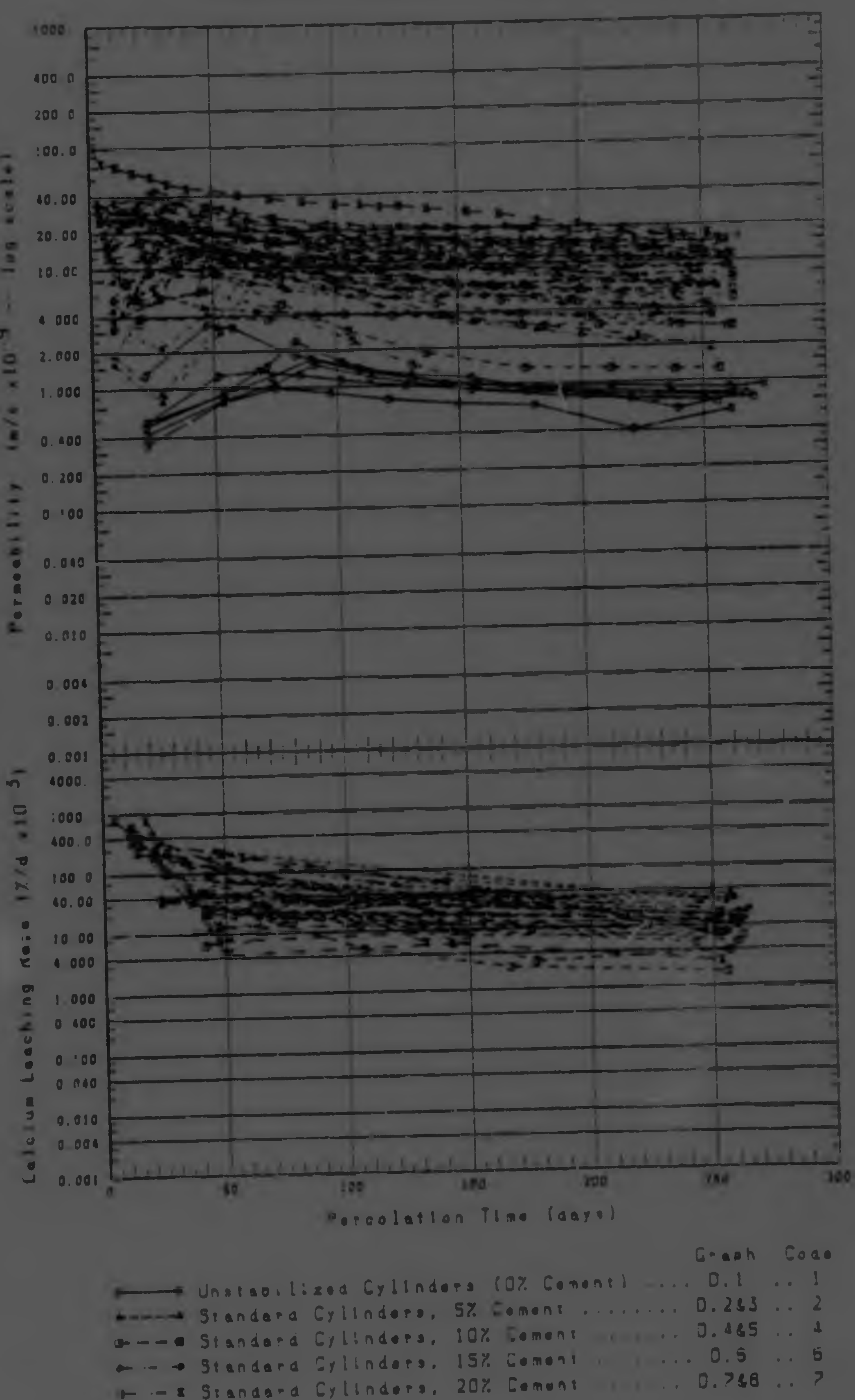
GRAPH 9.5

SPECIMENS STABILIZED WITH 20% CEMENT



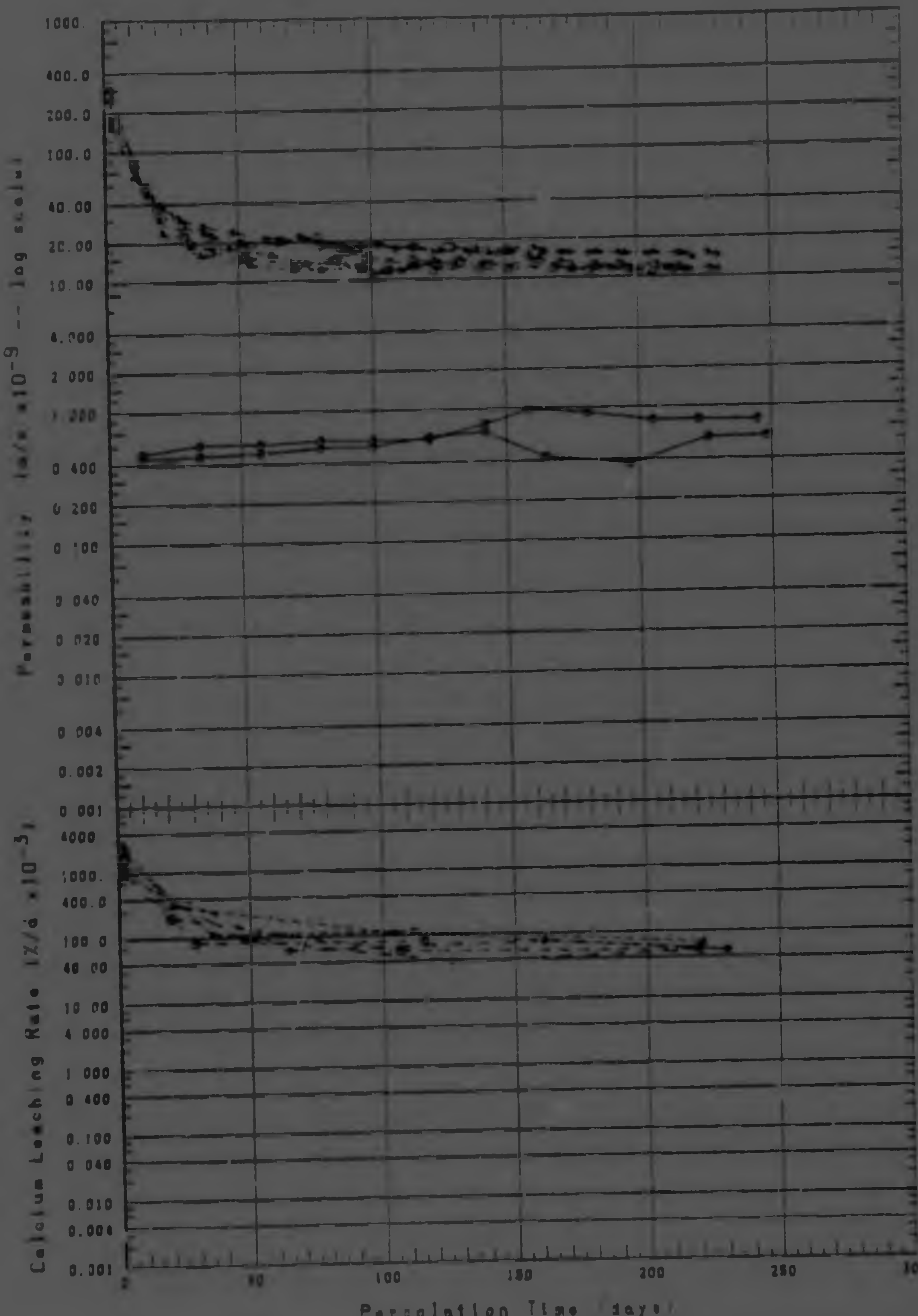
GRAPH 3.5

SUMMARY OF RESULTS -- STANDARD CYLINDERS



GRAPH 9.7

SUMMARY OF RESULTS -- DISK SPECIMENS



	Graph	Code
Unstabilised Disks	D 35	.. £
Disk Specimens, 5% Cement	D.36	.. a
Disk Specimens, 10% Cement	D.37	.. o
Disk Specimens, 15% Cement	D.38	.. a

CBWS-OSK PLT -- 26/03/84 ... (LCH-COMP ..., 8) recognised that this will represent complete removal of cement from a 75 mm deep skin with free drainage under a gradient of 50 and **not** a complete embankment. As previously noted, the maximum calcium capacity of distilled water is of the order of 600 mg/l and the permeability of the material is such that the rate of advance of a wetting front is of the order of 30 mm/year with unit gradient. Once saturated with calcium, percolating water cannot cause further deterioration, it is therefore considered that the observed leaching represents a surface effect.

9.4.7.b Non-Standard Compaction Conditions

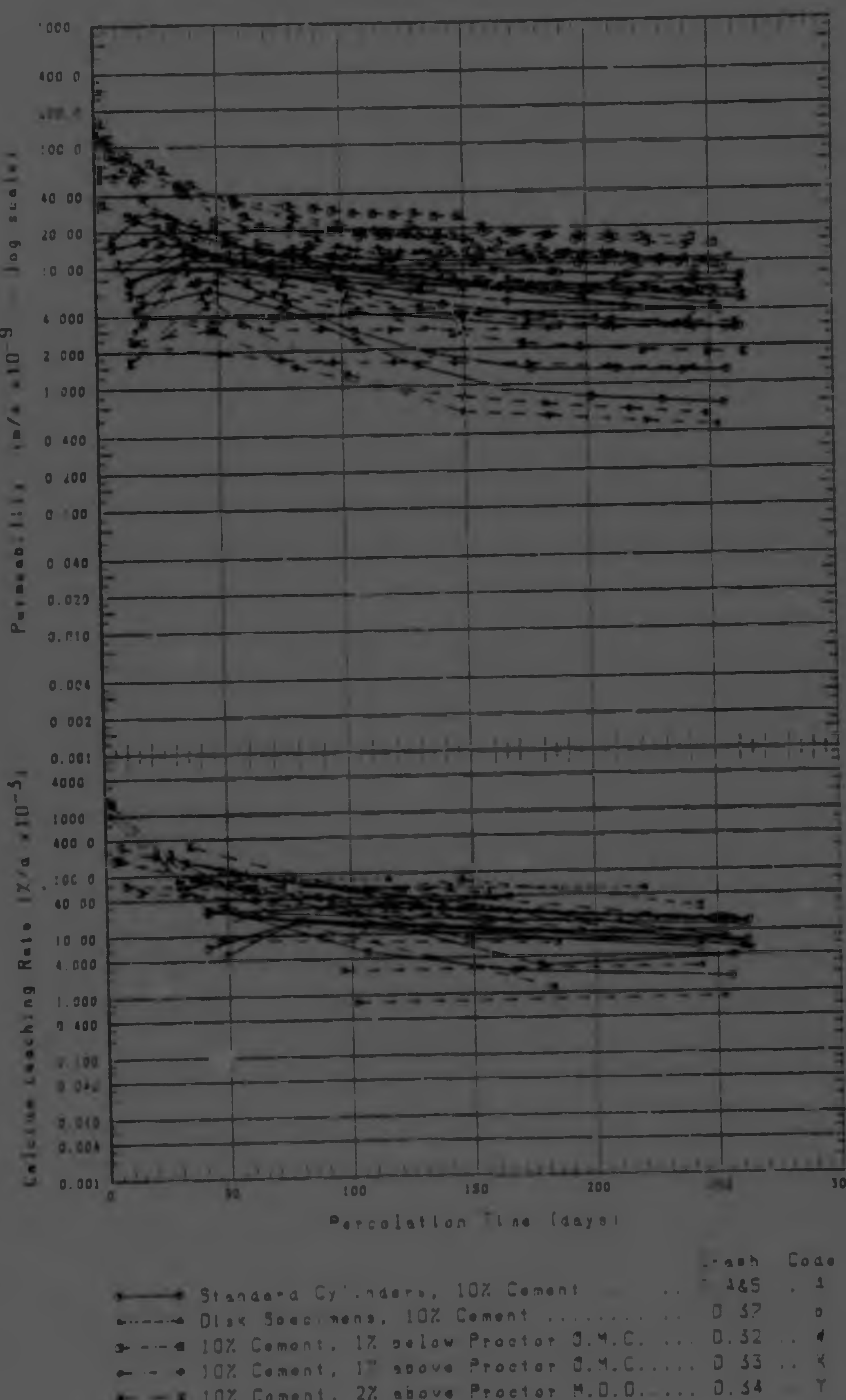
Graph 9.8 summarises the data for the specimens compacted at 1% above and below optimum moisture content (OMC) and 2% above maximum dry density (MDD). The standard 10% specimens are plotted for reference.

The moisture content and density used for the specimens compacted wet and dry of OMC were selected to lie on the standard proctor moisture density curve. These specimens were compacted at moisture contents and dry densities of 14.75% and 1885 kg/m^3 and 12.75% and 1898 kg/m^3 respectively. The moisture content and dry density for the specimens compacted above proctor MDD were obtained by moving along the 5% air voids line to the required density of 1943 kg/m^3, thus obtaining the required moisture content of 13.00%.

The specimens compacted dry of OMC exhibited high initial permeabilities which decayed to just above the upper limit for the standard specimens. The decay curve is similar to that observed for the disk specimens but the initial values are not as great and the rate of decay is slower. It should be noted that leaching supply to these specimens was not cut off at any stage as was the case for the disk specimens. The calcium leaching rate curve exhibits a similar comparison with the standard specimens. Compaction dry of optimum thus has a detrimental effect on permeability.

GRAPH 9.8

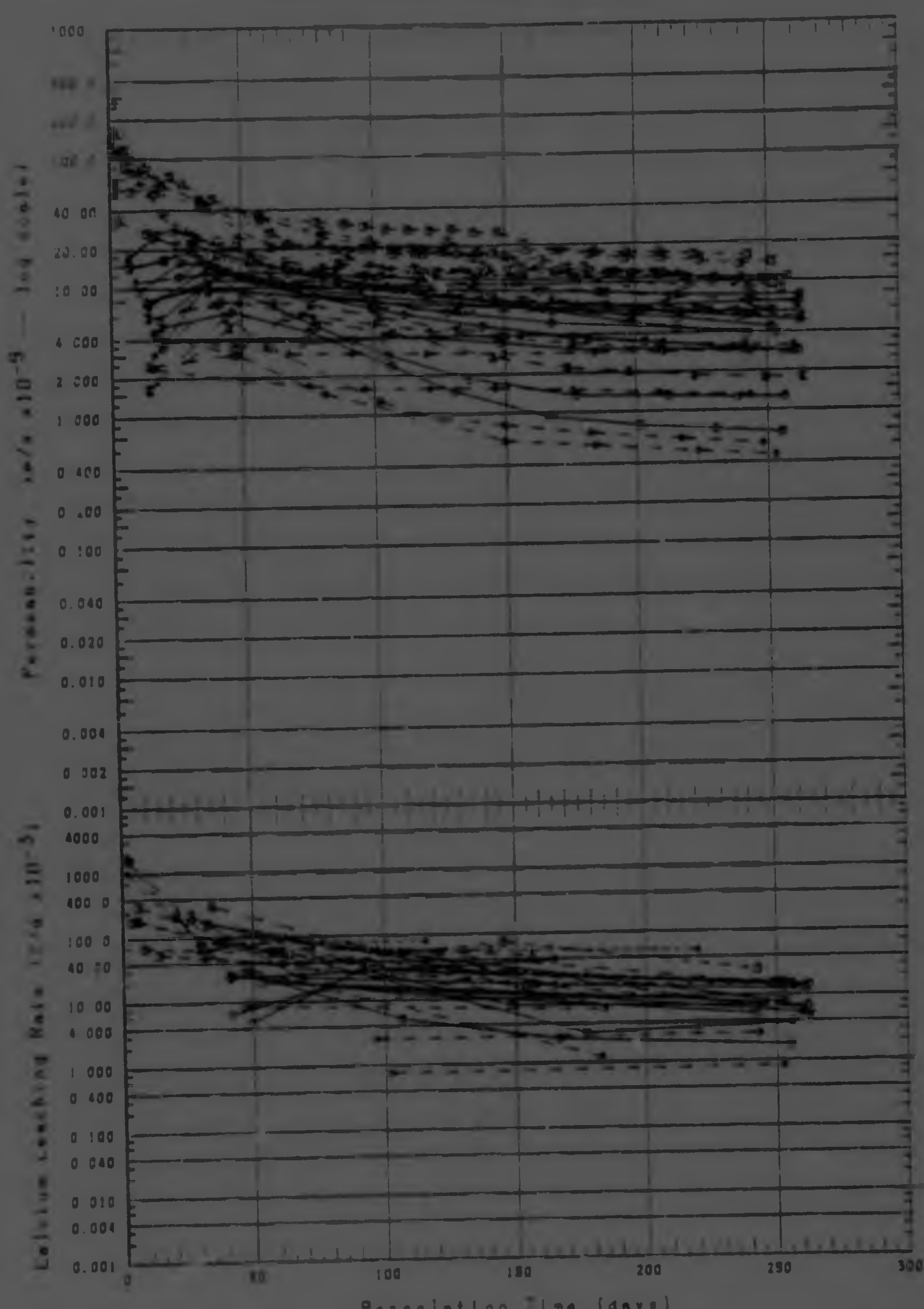
NON-STANDARD COMPACTION CONDITIONS



CBMS-MAT PLT — 28/03/84-0=1218-v 1.0 LCH-CMP.DAT.

GRAPH 9.8

NON-STANDARD COMPACTION CONDITIONS



	Graph	Code
Standard Cylinders, 10% Cement	D.4&5	1
Disk Specimens, 10% Cement	D.37	b
10% Cement, 1% below Proctor O.M.C.	D.32	4
10% Cement, 1% above Proctor O.M.C.	D.33	X
10% Cement, 2% above Proctor M.D.D.	D.34	Y

CONS-MAT PLT — 29/03/84-0:12:6

The specimens compacted wet of OMC exhibit a similarly shaped permeability - time curve to that for the standard specimens but it is generally lower at all stages, straddling the lower limit for the standard specimens. The calcium leaching rate is also somewhat lower. The lower permeability is consistent with that normally associated with clays compacted wet of OMC, but it should be noted that these specimens were difficult to handle immediately after compaction due to the excess moisture content. The slightly lower permeability does not justify the increased difficulty in processing the material.

The specimens compacted at 2% above MDD lie within the range of results for the standard specimens, the development of full flow may be slightly delayed but no significant benefit is apparent.

These results indicate a general tendency for lower permeabilities, and hence flows, to be associated with lower calcium leaching rates.

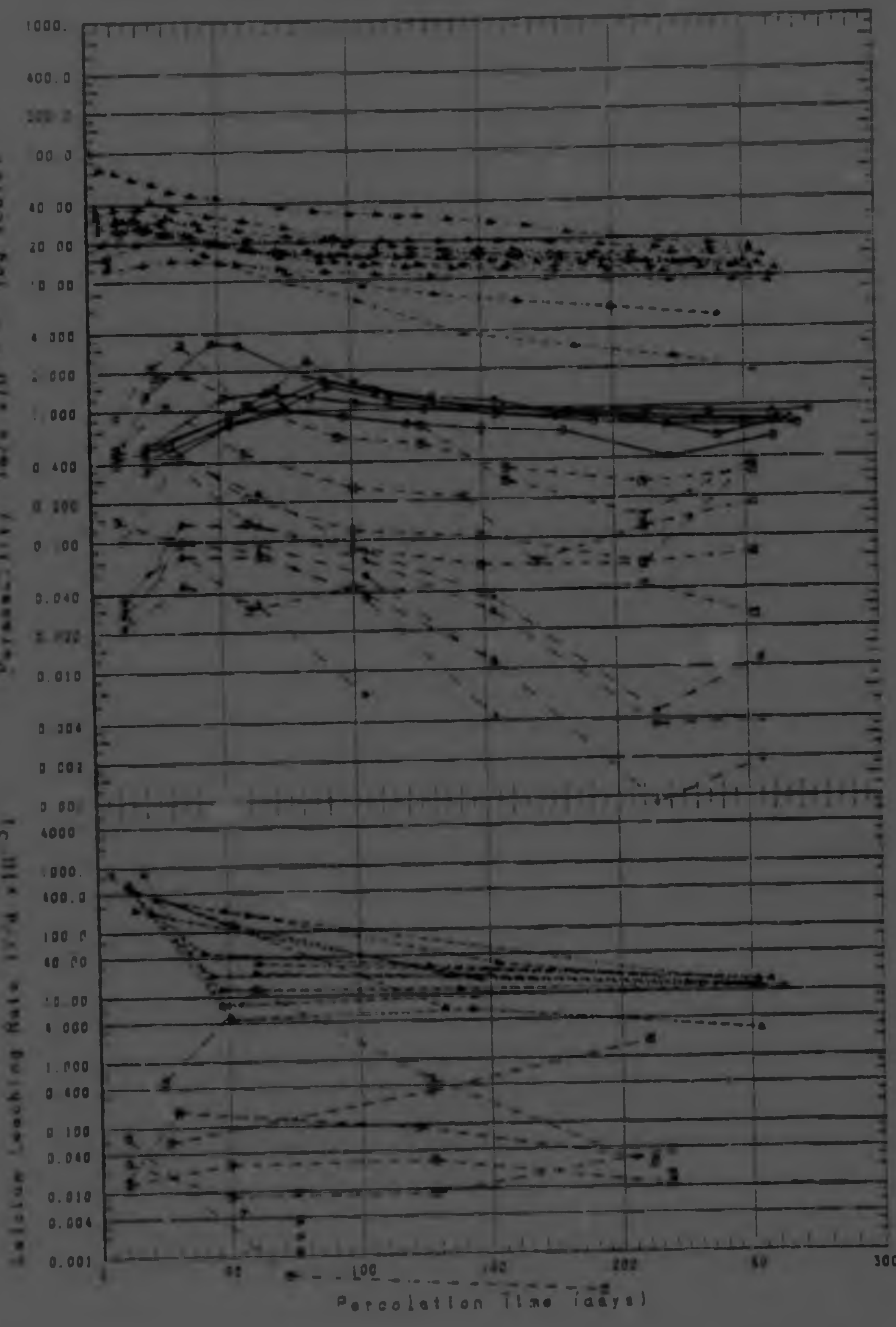
9.4.7.c Microconcrete

Graph 9.9 compares the results for the microconcrete mixes with thôse for the standard 20% specimens and unstabilized specimens. The upper limit of permeability for the specimens with cement - water ratio (c/w) of 1.0 (18.2% cement) corresponds with that for the unstabilized specimens. The lower limit, for the specimens with c/w 1.2 (21% cement), reflects the discontinuities referred to above, indicating permeabilities below those that could be registered with the apparatus used.

The relationship of increasing permeability with decreasing cement - water ratio, is in agreement with the normal behaviour for concrete mixes (Neville 19[illegible], and [illegible]alton 1977). Extrapolation of the data for water retaining quality concrete

GRAPH 9.9

MICROCONCRETE CYLINDERS



Graph Code

Unstabilized Cylinders (0% Cement) .... 0.1 .. 1

Standard Cylinders, 20% Cement .... 0.?58 .. ?

Microconcrete, 18.2% Cement, ?/? 1.0 .. 0.29 .. ?

Microconcrete, 22.0% Cement, ?/? 1.2 .. 0.30 .. ?

CONS-LOW PLT -- 28/03/84-8-1218-v ? ? LCH-CEM? OBT. ?

in these references on a semi-log plot, gives good agreement with the results obtained here.

The pronounced effect of a change in cement content for constant moisture content on the permeability of the micro-concrete is in marked contrast to the negligible effect of increasing the cement content of soil-cement above 5%.

As with the soil-cement, a decrease in permeability is associated with a decrease in leaching rate.

9.4.7.d Non-Standard Curing Conditions

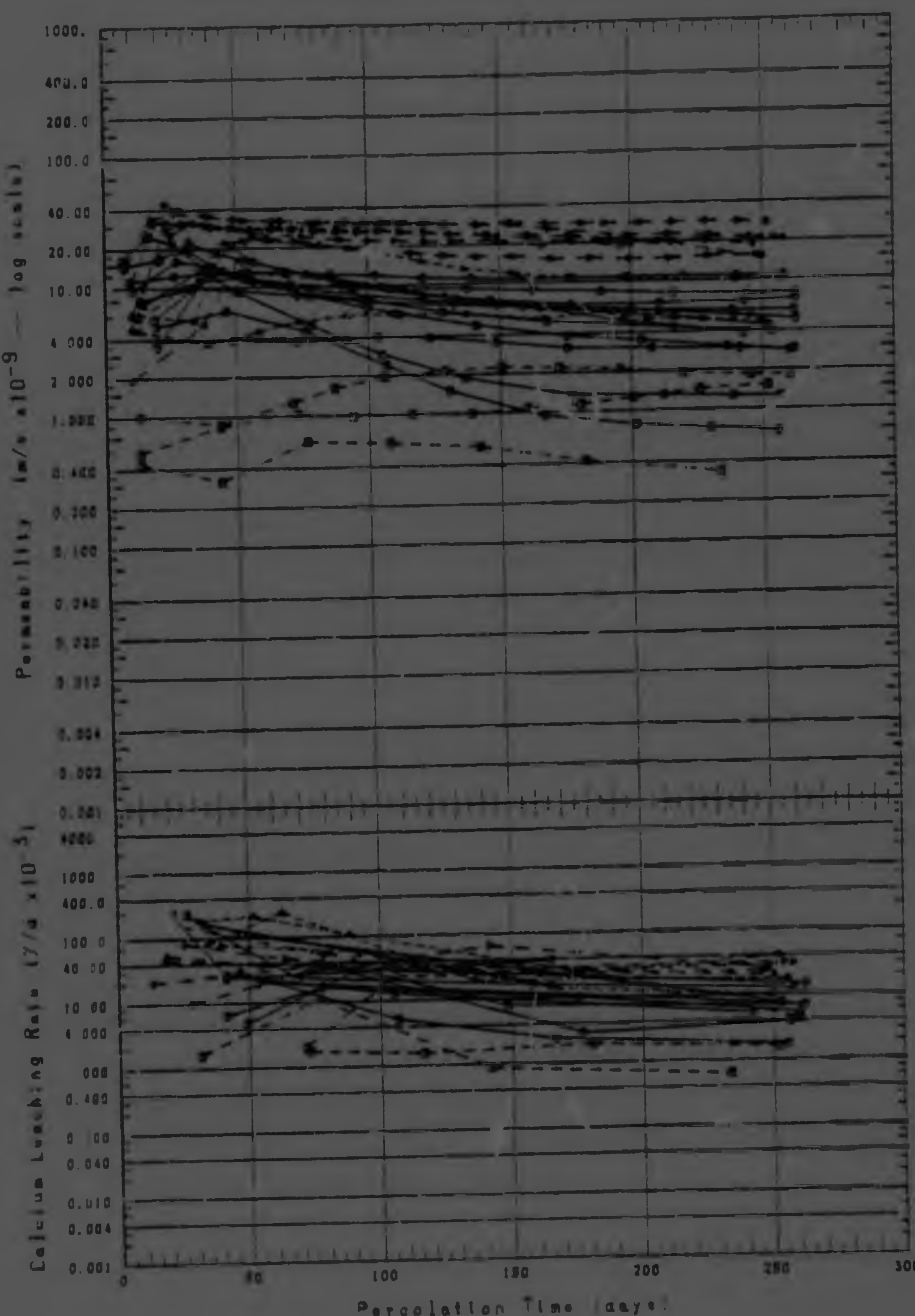
The data for standard 10% specimens subject to various non-standard cures is compared with that for the standard specimens in graph 9.10. The tendency for lower permeability to be associated with lower calcium leaching rate is again apparent.

The specimens left in the humid chamber for 28 days instead of being sealed at 7 days exhibit a similar spread of results to the standard specimens. The permeability and leaching rate curves may be slightly higher than those for the standard case, but it is unlikely that they are significantly different despite a significantly lower moisture content at the commencement of leaching (12.2% cf 13.1%).

The specimens placed in the leaching bath for 21 days at age 7 exhibit substantially lower initial permeabilities followed by an upward trend. The average final permeability trend is considerably lower than that for the standard specimens as is the leaching rate. It is apparent that the presence of additional moisture during the 7 to 28 day curing period has a significant influence in reducing permeability. The application of water during the first days after placing, before subsequent layers are placed, can be expected to aid in reducing permeability.

GRAPH 9.10

CYLINDERS, 10% CEMENT, NON-STANDARD CURING CONDITIONS



	Graph		Code
Standard Cylinders, 10% Cement	D.4&5	..	A
10% Cement, 28 Day Humid Cure	D.9	..	a
7 Day Humid, 21 Leach Bath Cure, 10% ..	D.10	..	A
7 Days Humid, 12 Cycles Wet-Dry, 10% ..	D.11	..	B

CBNS-CTP.PLT -- 26/03/84 ... LCH-COMP DAT.

As the microconcrete was wet cured from 24 hours in beams, these results suggest that the difference in measured permeability may reflect a difference in curing conditions as much as a difference in material properties.

The specimens subject to 12 cycles of wetting and drying, as described in chapter 8, exhibit an initial rapid increase in permeability with little subsequent decline. The final trend is significantly above that for the standard specimens indicating that alternate wetting and drying has a significant influence in increasing permeability. The final permeability is still, however, in the range of impermeable soils.

9.4.7.e Non-Standard Curing Period

The results for specimens subject to extended sealed cures of 77 and 343 days, after sealing at 7 days, are depicted in graph 9.11.

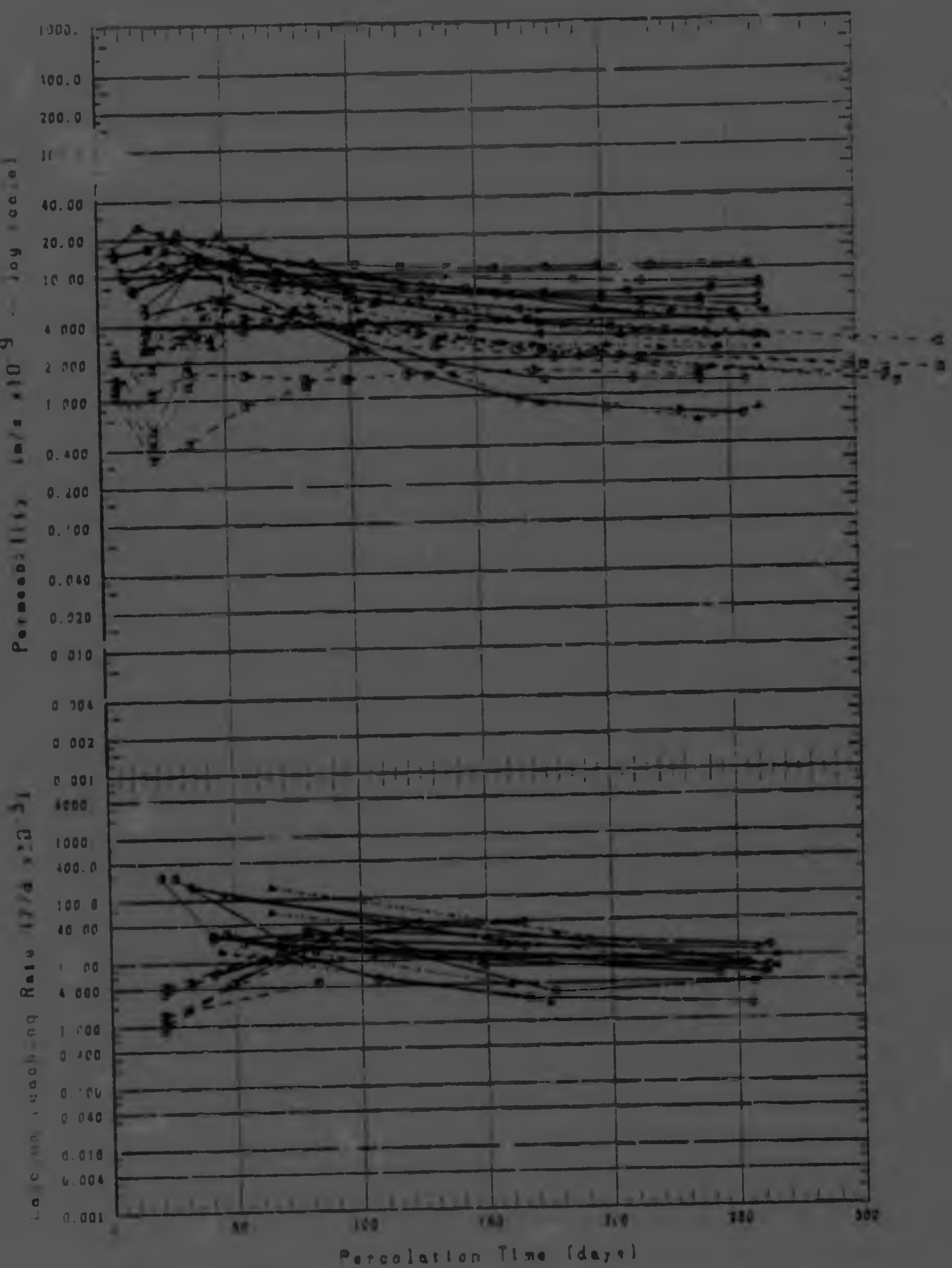
The specimens sealed for 77 days (11 weeks) exhibit a much lower initial permeability than the standard with a more gradual increase and less pronounced peaking. The final permeability and calcium rate trends are in a similar range to the standard specimens, slightly lower on average.

The specimens sealed for 343 days (31.2 weeks) exhibit a greater drop in initial permeability than the 77 day specimens, followed by a further decrease. Permeabilities then increase fairly rapidly until reaching a final trend which is similar to that for the 77 day specimens. Initial leaching rate is also considerably reduced.

The most pronounced effect of prolonged curing is in the reduction of the initial permeability and leaching rate and consequent overall reduction in leaching loss. This is clearly evident in graph 9.26.

GRAPH 9.11

CYLINDERS, 10% CEMENT, NON-STANDARD CURING PERIOD



Graph Code

Standard Cylinders, 10% Cement .... D.4&5 .. 4

7 Days Humid, 77 Days Sealed Cure, 10% D.12 .. C

7 Days Humid, 343 Days Sealed Cure, 10% D.13 .. D

The 343 day specimens were tested for a period of 587 days and the full period data is plotted in the appendix (graph D.13). The envelope for these specimens lies below that for the long term standard specimens depicted in graph 9.3a with a similar trend to the bottom envelope of the standard specimens thus supporting the observations regarding the increased reduction in permeability with sustained percolation.

9.4.7.f Non-Standard Leaching Waters, Gradients, etc

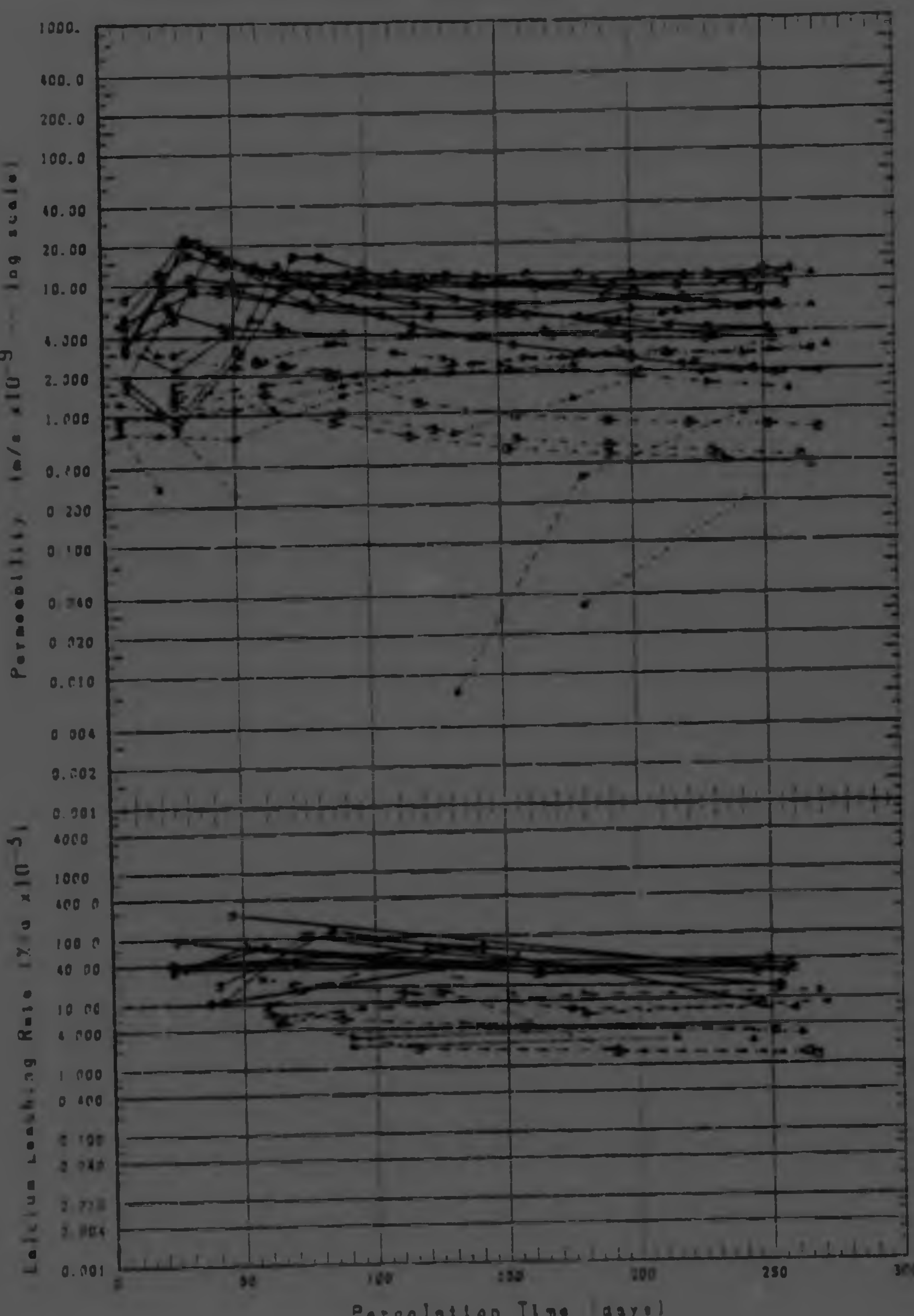
The permeability and calcium rate data for standard specimens leached with Middelburg and carbonated water and for the specimens subject to gradients of 10 and 136 m/m with air cell fluid are compared according to cement content and test condition in graphs 9.12 to 9.16 and summarised according to test condition in graphs 9.17 to 9.20.

Specimens containing 5% cement leached with carbonated water generally have lower permeability and leaching rate than the standard specimens (graph 9.12). A further decrease is evident for those specimens leached with water from the Middelburg reservoir. Those specimens subject to the low hydraulic gradient exhibit a broad scatter with some discontinuities, but generally below the level of the standard specimens.

Specimens containing 10% cement (graph 9.13), exhibit a similar progressive decrease in permeability and leaching rate for percolation with carbonated and Middelburg water to those with 5%. Those subject to the high gradient with air cell fluid (graph 9.14) exhibit permeabilities considerably below those for the standard specimens with de-aired distilled water cell fluid and standard gradient and have a more pronounced downward trend. Calcium rates are also generally lower. The standard gradient specimens with air cell fluid exhibit similar initial permeabilities but thereafter a considerable degree of scatter is evident with the bands for the high and standard gradient specimens with air cell fluid overlapping. The calcium rate data is similar. The results for the low

GRAPH 8.12

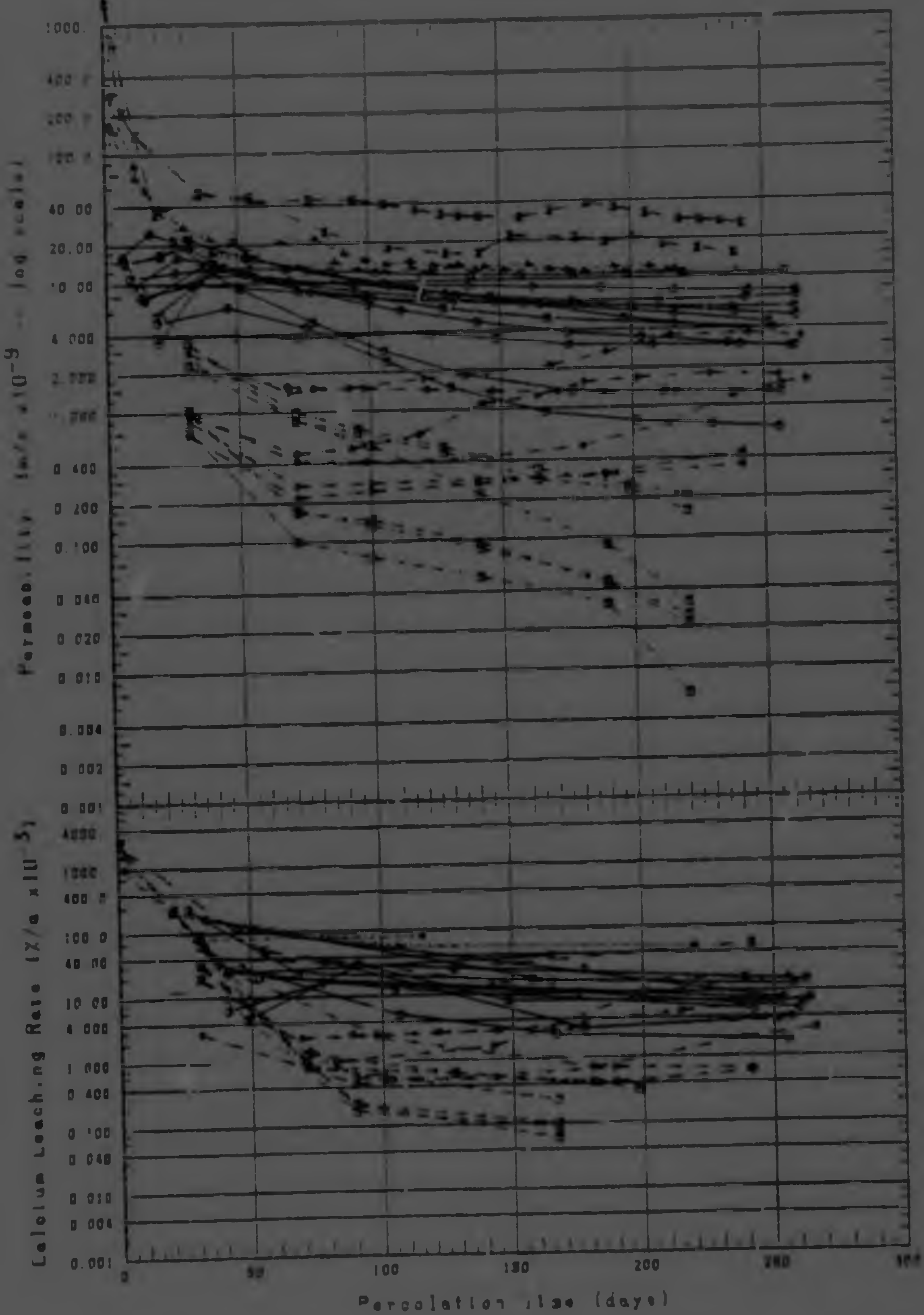
CYLINDERS, 5% CEMENT, NON-STANDARD LEACHING CONDITIONS



	Graph	Code
Standard Cylinders, 5% Cement	D.2&3 ..	2
Low Hydraulic Gradient (10 m/m), 5% ...	D.18 ..	I
Leached with Middelburg Water, 5%	D.21 ..	L
Leached with Carbonated Water, 5%	D.24 ..	O

GRAPH 9.13

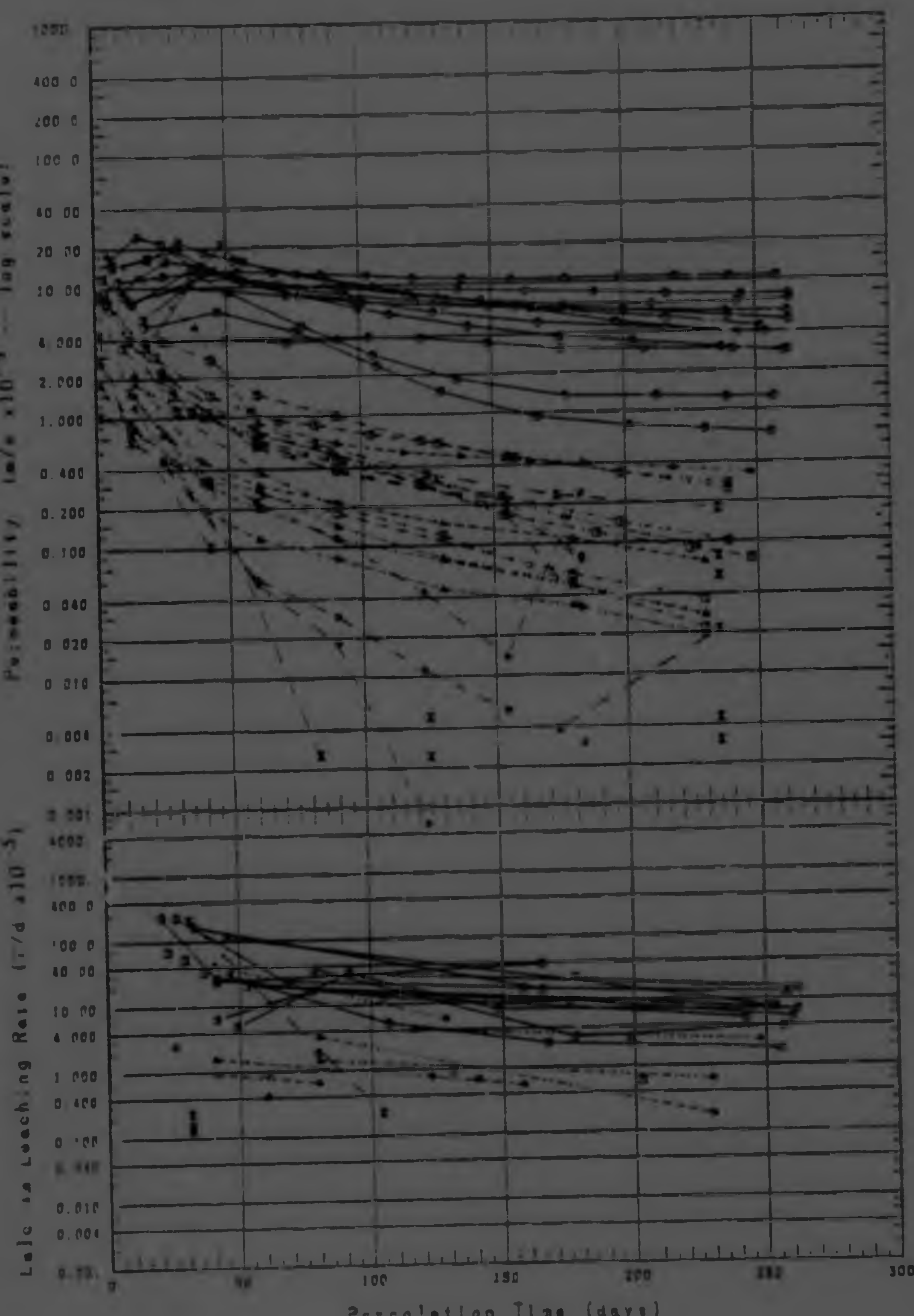
CYLINDERS, 10% CEMENT, NON-STANDARD LEACHING CONDITIONS



	Graph	Code
Standard Cylinders, 10% Cement	0.4&5 ..	A
Disk Specimens, 10% Cement	0.37 ..	D
Leached with Middelburg Water, 10%	0.22 ..	M
Leached with Carbonated Water, 10%	0.25 ..	P
Cracked Cylinders, 10% Cement	0.27 ..	R

GRAPH 9.14

10% CEMENT, NON-STANDARD GRADIENT AND CELL PRESSURE



	Graph	Code
Standard Cylinders, 10% Cement	D.4&5 ..	A
High Hydraulic Gradient (136 m/m), 10%	D.14 ..	E
High Hydraulic Gradient, 10% (set 2) ..	D.15 ..	F
Standard 10%, Air Pressurised Control .	D.18 ..	G
Low Hydraulic Gradient (10 m/m), 10% ..	D.19 ..	J

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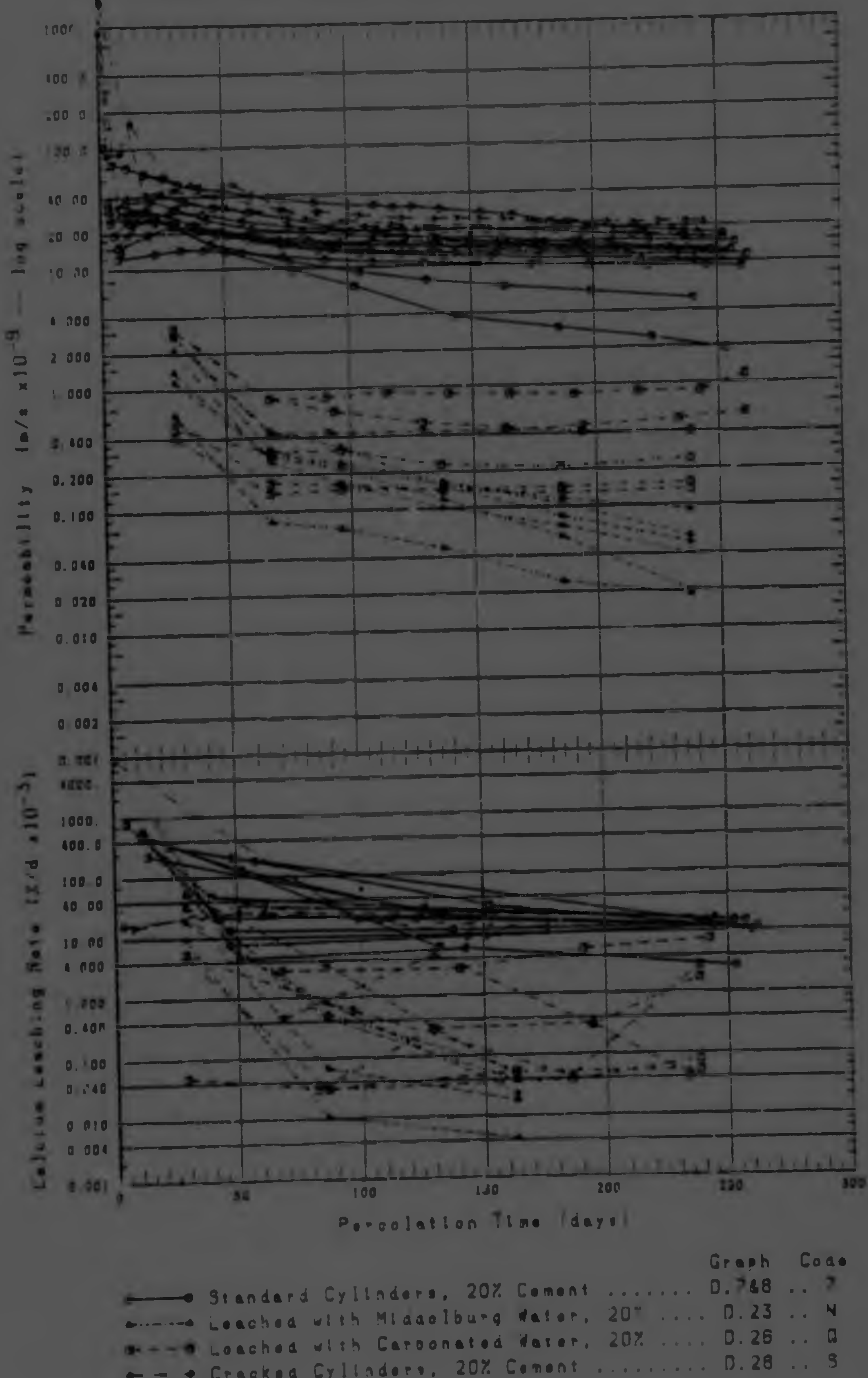
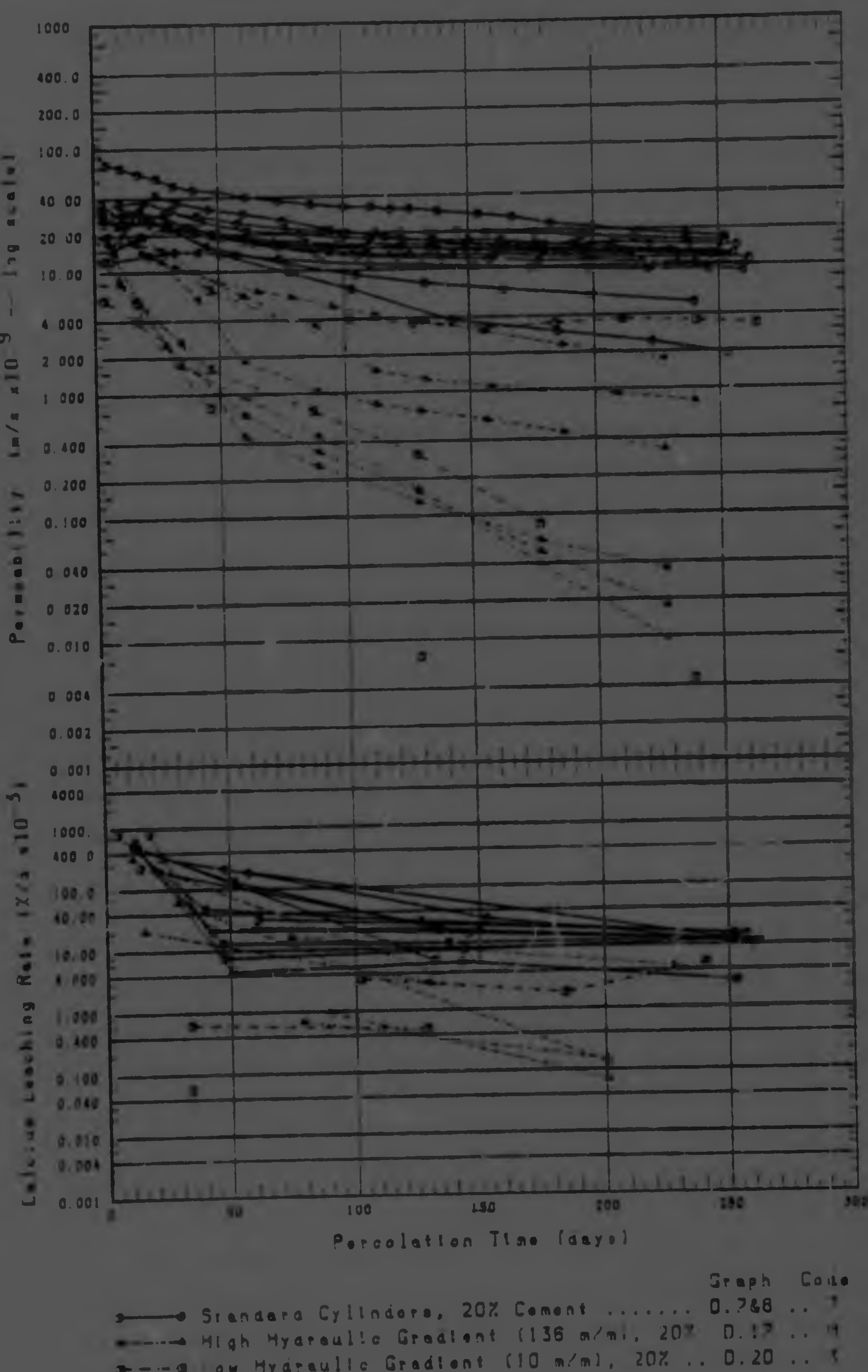
GRAPH 9.15

CYLINDERS, 20% CEMENT, NON-STANDARD LEACHING CONDITIONS



GRAPH 9.16

CYLINDERS, 20% CEMENT, NON-STANDARD GRADIENTS



CONS-200 PLT -- 26/03/84-0=1217-- 1.0 FLCH-CONF.DST. 0 1 gradient specimens again show considerable scatter and discontinuities and are generally below the envelopes for the other test conditions.

Specimens containing 20% cement (graph 9.15) show similar trends with carbonated and Middelburg water to those discussed above as do the high gradient specimens with air cell fluid (graph 9.16). The low gradient data generally overlaps that for the high gradient with some discontinuities.

9.4.7.g High Gradient and Air Cell Fluid

The results for specimens subject to high and standard gradient with air cell fluid are summarised in graph 9.17. All specimens exhibit an initial steep decrease in permeability which levels off to a trend which is still considerably steeper than that for the standard test conditions. The standard gradient specimens exhibit a number of discontinuities due to the very low flows. It is apparent that diffusion of air through the membranes occurred and that this had a considerable influence on the permeability of the specimens. The steps taken to keep the cell fluid for the rest of the test program free of air are therefore justified.

Permeability for high gradient (air cell fluid) specimens with 20% cement is highest with that for high gradient specimens with 10% cement in a broad band below. Standard gradient 10% control specimens, with air cell fluid, overlap the lower portion of the band for high gradient with a number of discontinuities resulting from the low flows.

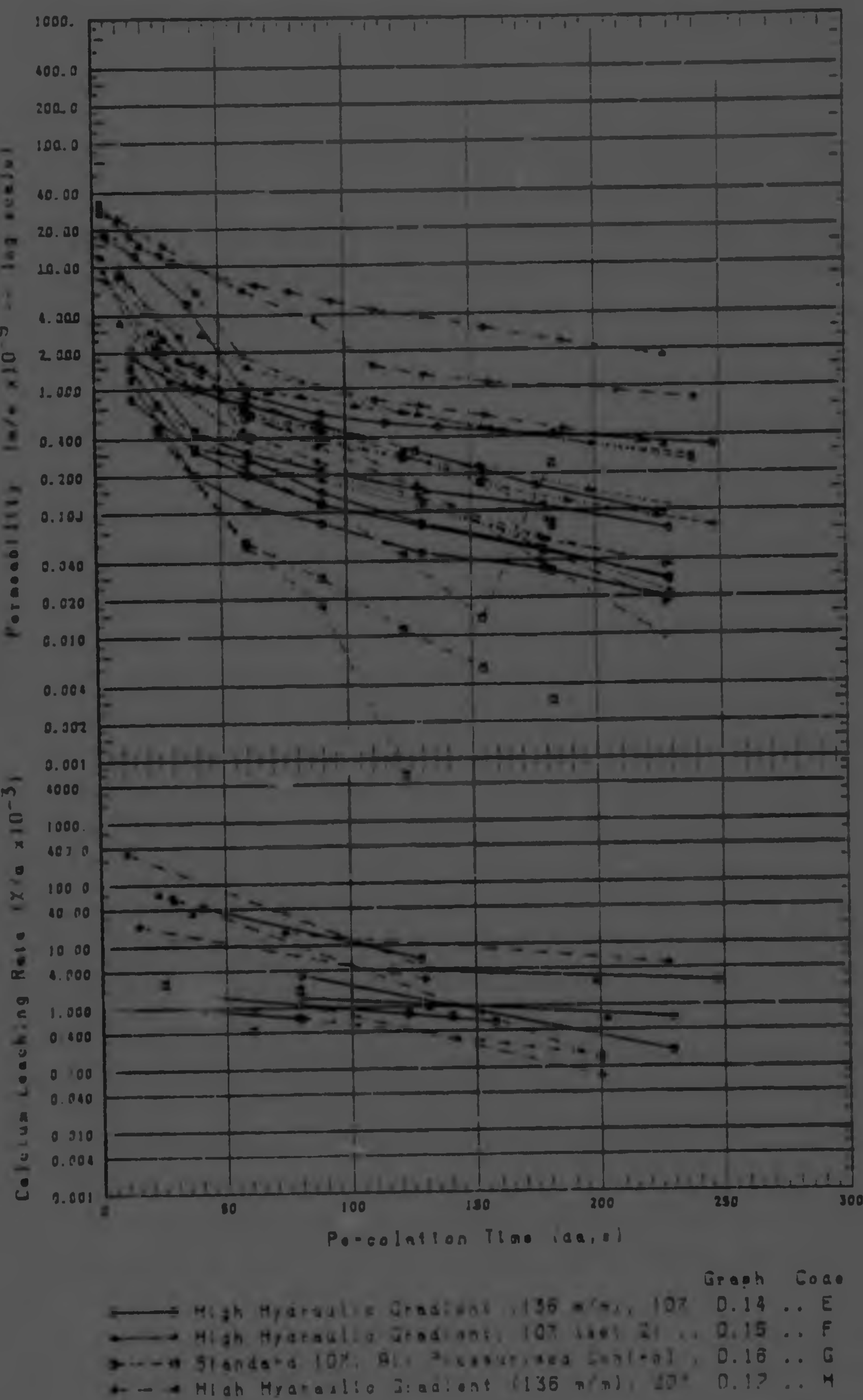
Calcium leaching rate behaves similarly to permeability.

9.4.7.h Low Hydraulic Gradient

The results for the specimens subject to the low hydraulic gradient of 10 m/m are summarised in graph 9.18. The bands of permeability data for 5% and 20% specimens subject to the low gradient are similar. Specimens with 10% cement exhibit

GRAPH 9.17

SUMMARY OF RESULTS -- HIGH GRADIENT, AIR CELL PRESSURE



CONS-HGB.PLT -- 28/03/91

generally lower permeability with a number of discontinuities. Calcium rate is generally fairly level with 5 and 20% generally above 10%.

All permeability data shows a pronounced initial down trend followed by levelling off. In some cases an increase in permeability with time at about 150 to 200 days occurs, particularly at the lower permeabilities. This however remains below the top limit of the envelope for the corresponding standard specimens.

The low flows resulting from the lower gradient were more affected by evaporation and this could account for the lower indicated permeabilities. However, considerable difficulty was experienced in establishing flow in many of the low gradient specimens and it is considered that a real decrease in permeability existed. The lower initial flow rate reduces the rate of removal of calcium and other soluble elements and allows further hydration to occur. This can be expected to reduce the permeability of the material as well as its susceptibility to leaching. This effect would be even more pronounced under unit gradient.

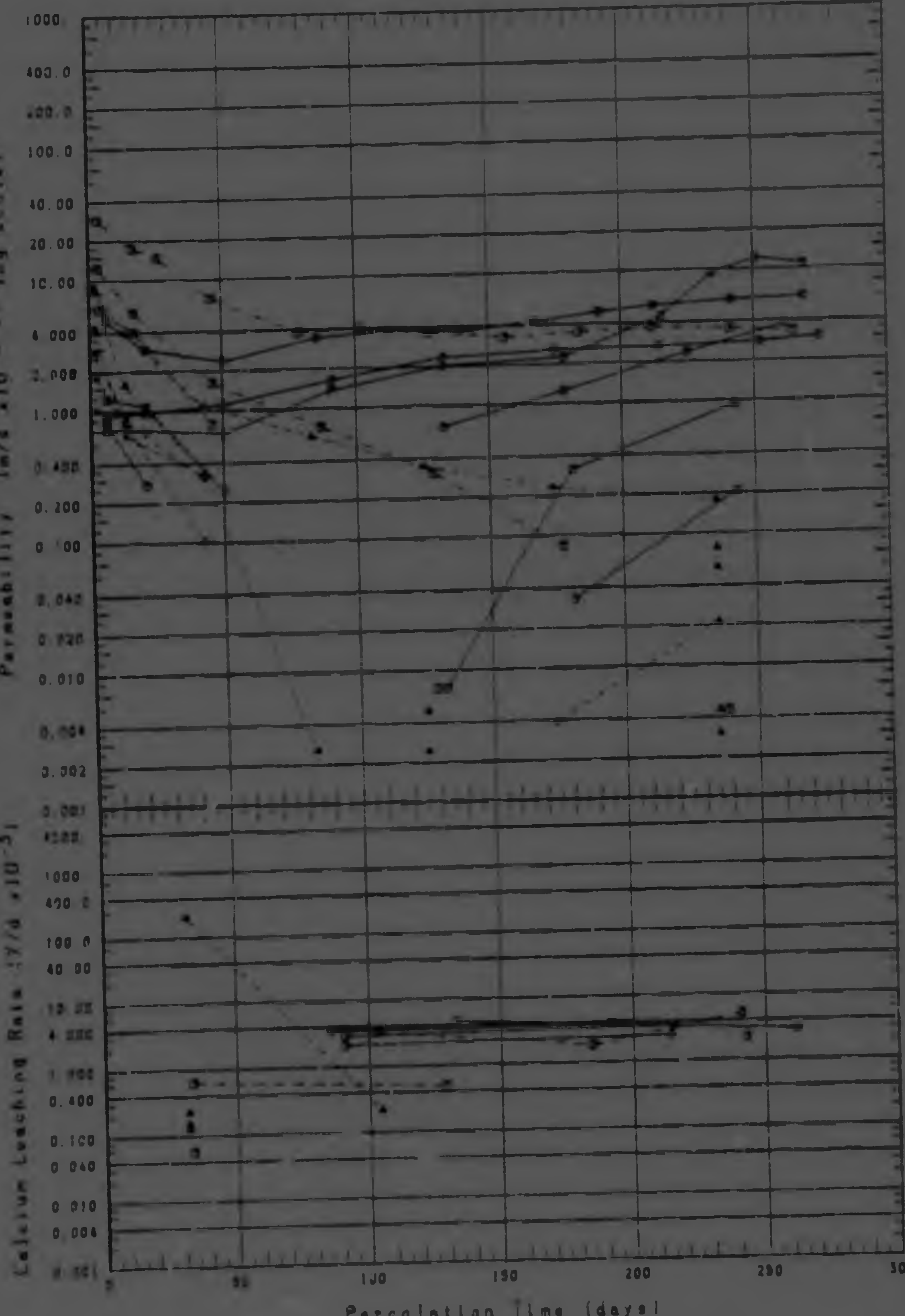
9.4.7.i Natural (Middelburg) Leaching Water

The results for the specimens leached with the water from the Rondebosch Dam at Middelburg are depicted in graph 9.19. The data for the 10 and 20% specimens exhibits a consistent downward trend in both permeability and leaching rate. That for the 5% specimens tends to level off or rise slightly. Both permeability and leaching rate data start in a similar range, considerably lower than that for the standard conditions, but then diverge with progressively lower levels being reached for specimens with 5, 10 and 20% cement respectively.

The Middelburg water was not de-aired and it is considered that this caused the general downward trend of the results. The reduced permeability, together with the other elements

GRAPH 9.18

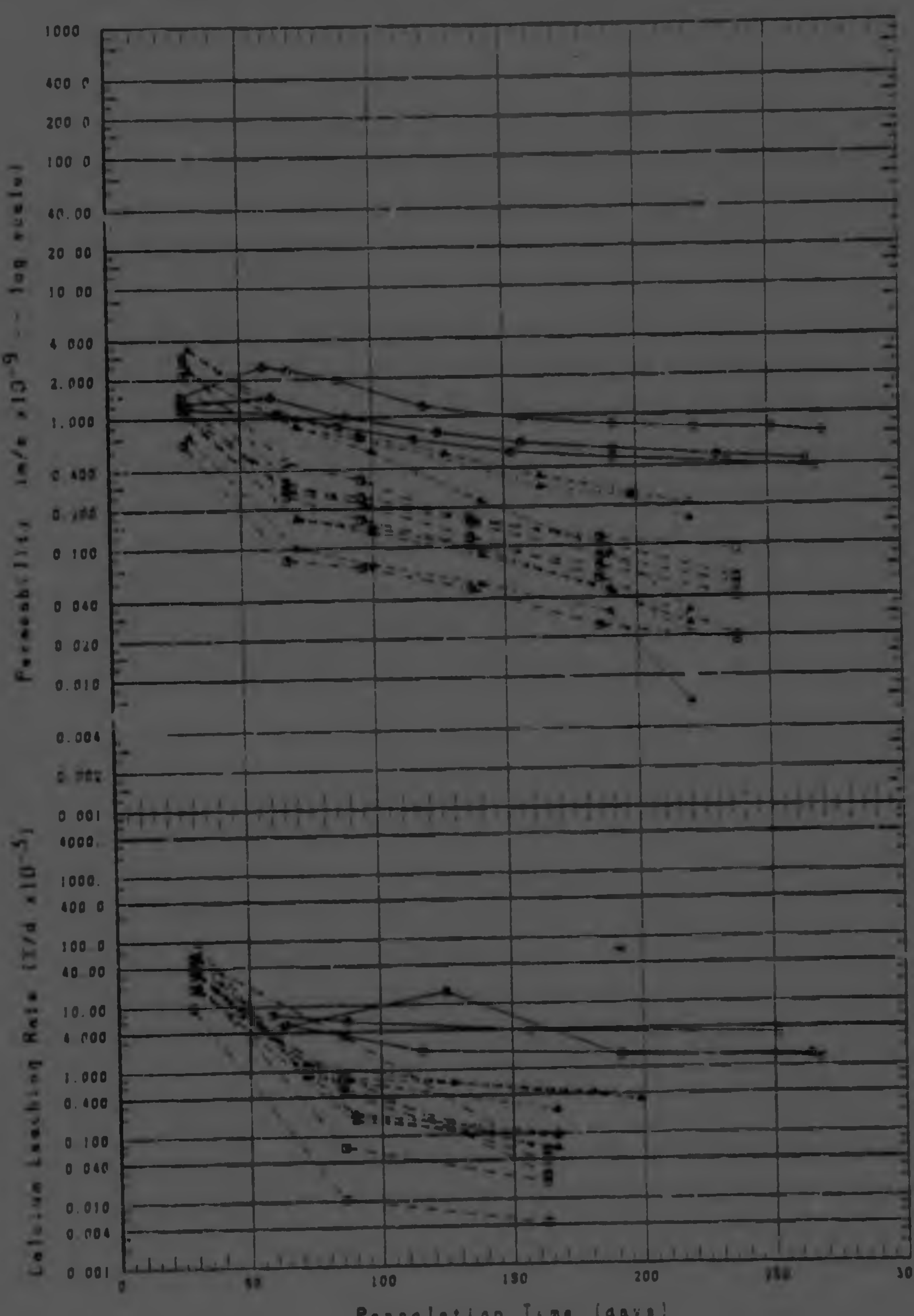
SUMMARY OF RESULTS -- LOW HYDRAULIC GRADIENT



	Graph	Code
Low Hydraulic Gradient (10 m/m), 5% ...	0.18 ..	I
Low Hydraulic Gradient (10 m/m), 10%	0.19 ..	J
Low Hydraulic Gradient (10 m/m), 20% ..	0.20 ..	K

GRAPH 9.19

SUMMARY OF RESULTS -- LEACHED WITH MIDDLEBURG WATER



Graph Code

Leached with Middelburg Water, 5% .... D.21 .. L

Leached with Middelburg Water, 10% .... D.22 .. M

Leached with Middelburg Water, 20% .... D.23 .. N

[illegible]-MID.PLT — 28/03/84-[illegible] 1 0 (LCH-COMP DAT. 8 1 already dissolved in the water, explains the low leaching rates observed.

These results, together with those for specimens subject to the low gradient, indicate a considerable reduction in leaching attack of the soil-cement embankment subject to unit gradient and natural water.

9.4.7.j Carbonated Leaching Water

The results for the specimens leached with carbonated water are presented in graph 9.20. Initial permeability and leaching rate results are in the same range as those produced using the Middelburg water confirming the effect of dissolved gases on permeability. Specimens with 10% and 20% cement again exhibit an initial reduction while those with 5% show a slight upward trend. The sequence is again 5, 10 and 20% in order of decreasing permeability and leaching rate.

The declines in both permeability and leaching rate are not as pronounced as observed with the Middelburg water and the 10% specimens show an increase in both quantities towards the end of the test period. It is considered that this levelling off and upward trend relative to the Middelburg water is as a result of the more aggressive nature of the carbonated water either as a result of acidity, or the absence of dissolved minerals, or both.

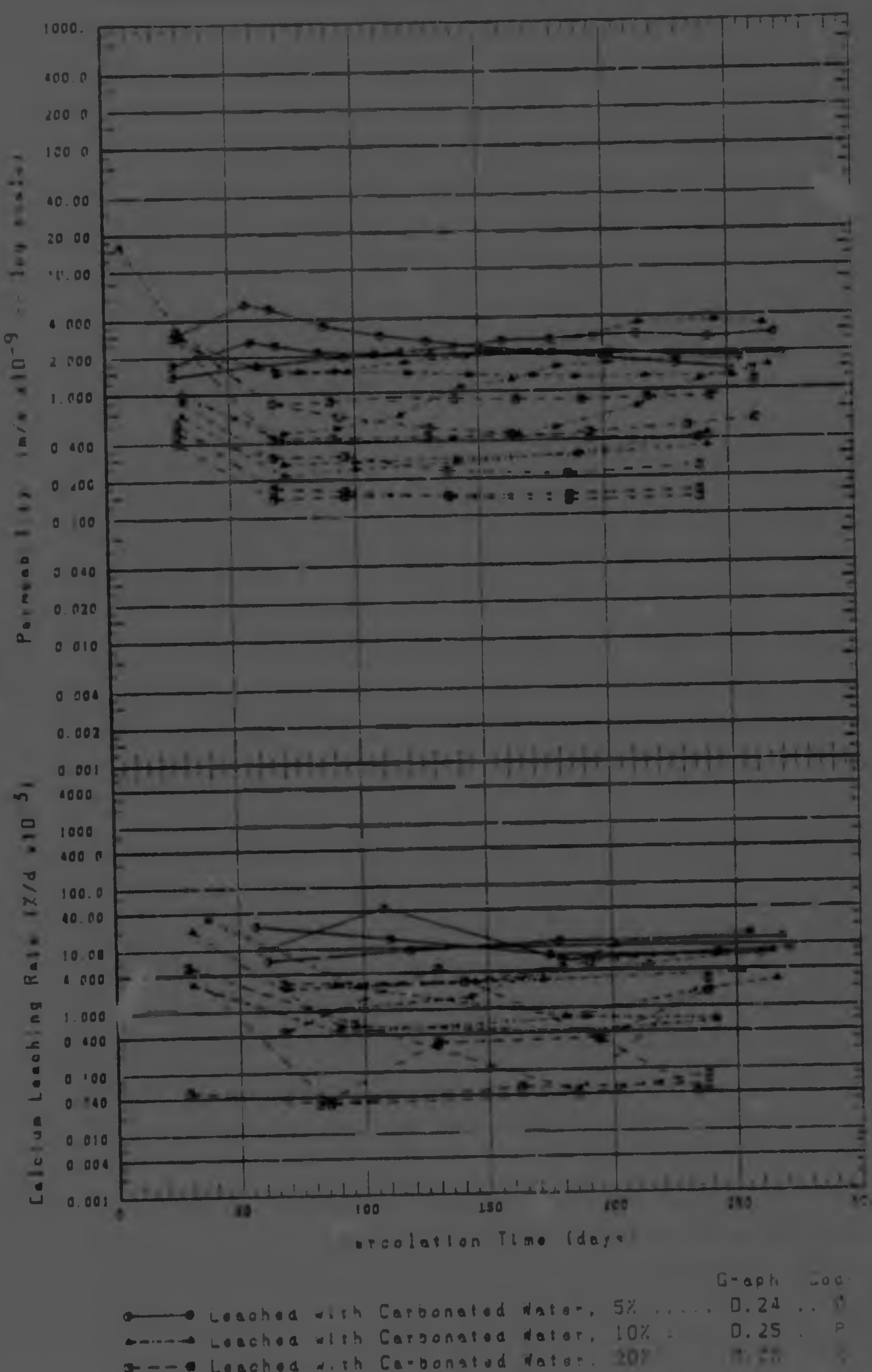
9.4.7.k Cracked Specimens

The results for the two sets of specimens with longitudinal tension cracks are presented in graph 9.21 and plotted for comparison on graphs 9.13 and 9.15.

The specimens with 10% cement (graph 9.13) exhibit very high initial permeabilities, of the order of 30×10^{-5} m/s but converge rapidly on the range for the standard specimens. The final envelope lies slightly above that for the standard disk specimens. The leaching rate is similar to that for the disks

GRAPH 9.20

SUMMARY OF RESULTS -- LEACHED WITH CARBONATED WATER



CONS-CO2.PLT — 26/03/86-9=1217-- 1 0 LCH-CONF.DAT. 6

throughout. Owing to the high initial permeabilities, leaching supply to these specimens was initially intermittent and this could have a bearing on the results. However, since a gradient of 50 would not be applied to any water retaining structure instantaneously, particularly at first filling, it is considered that the test conditions were not entirely unrealistic. It should also be noted that the method of test applied an external pressure to the specimen which would have helped to close the crack, a situation that would not necessarily occur in practice although it might be expected in highly stressed zones.

The specimens with 20% cement (graph 9.15) exhibit similar initial permeabilities and behaviour to those with 10% cement, merging with the upper region of the envelope for the standard specimens.

The results for both sets follow almost exactly the same paths (graph 9.21). This is consistent with the similar permeability and loss results for standard specimens with 10 and 20% cement and with the physical similarity of the crack tested.

On completion of the leaching tests, the specimens were removed and the cracks reopened. Only slight force was required for separation and softening was observed over the entire crack surface indicating that flow along the crack had occurred. The leach bath specimens showed no appreciable deterioration on the crack surface and required slightly more force to separate, it was considered possible that slight bonding had occurred but this is by no means certain.

The sealed specimens were soaked for four hours before separating, some force was required to separate them and in general little wetting of the crack surface was observed. Whether this is an indication of bonding on the surface is uncertain but in view of the relatively dry state of the specimens on sealing, it would appear equally likely that this behaviour resulted from pore suction.

While the above results provide very little information on the behaviour of cracks under flow, they are consistent with the experience of the USBR at the Lubbock Reservoir seepage test section referred to in chapter 1 (De Groot 1971a). Nussbaum and Colley (1971) also found that shrinkage cracks through soil-cement facings did not increase seepage appreciably and that treatment of horizontal joints with a thin neat cement grout reduced permeability to that of the intact material.

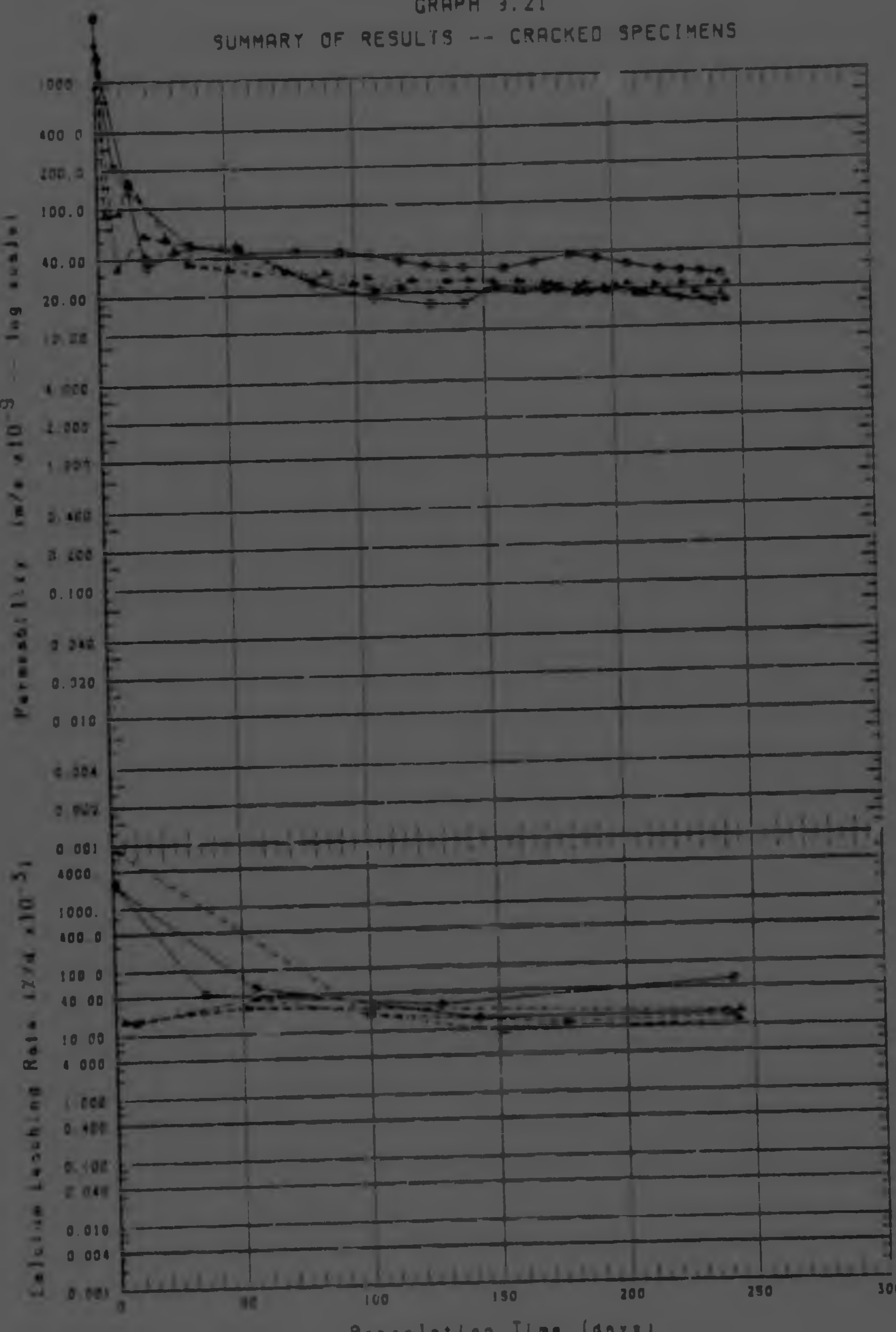
It is considered that, provided reasonable construction procedures to limit crack development are adopted, the alignment of cracks between layers is unlikely to occur because of their disparate ages. In particular, the alignment of cracks of any size through the entire body of a homogeneous embankment is considered highly unlikely. It should be noted that one two meter wide strip of intact soil-cement on any seepage path will provide an adequate water barrier and that the type of crack normally associated with soil-cement facings could be adequately sealed by suitable surface treatment applied below the water surface.

Bofinger and Sullivan (1971) found that cracking of properly cured soil-cement bases occurred within the first 30 minutes after compaction and that further cracking, normally associated with shrinkage, in fact resulted from wheel loads. They also reported a shrinkage crack spacing of up to 5 m. These findings further support the view that cracking of a massive soil-cement embankment will be of no significance.

Provided suitable monitoring equipment is installed in the soil-cement embankment, the installation of special impermeable membranes or drains is considered unnecessary. It should be noted that if a precast upstream face, as used at Willow Creek Dam (Schrader 1982), is employed, the precast concrete

GRAPH 3.21

SUMMARY OF RESULTS -- CRACKED SPECIMENS



Graph Code

Cracked Cylinders, 10% Cement .... 0.27 .. R

Cracked Cylinders, 20% Cement .... 0.28 .. S

CBWS-CBR.PLT -- 26/03/84-0=1212-v I 0 (LCH-CBMP DAT. 6) facing panels will provide an upstream water barrier that will bridge most cracks.

9.4.8 Hydration Correction from Sealed Controls

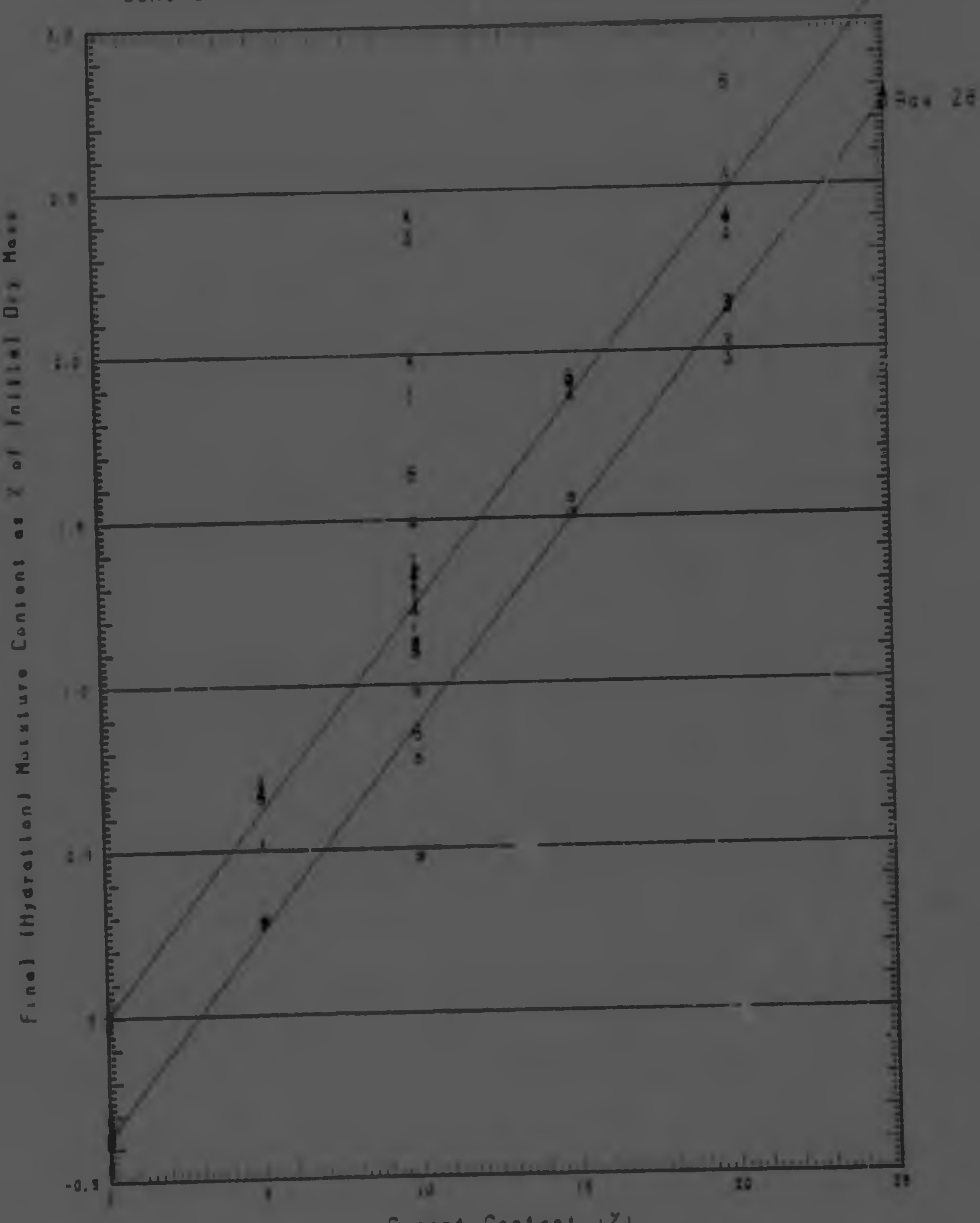
The hydration correction was determined as the final moisture content of the sealed control specimens, this is plotted against cement content in graph 9.22.

Regression on 25 data points for specimens at age +/- 300 days yields a regression line with intercept 0.01% and slope 0.125, regression coefficient (r^2) was 0.95 This regression is identical to the hydration correction of 0.125 x cement content (%) for the standard ASTM wet-dry control specimens subject to twelve cycles, derived in section 8.4.1 and depicted on graph 8.2. This is coincidental as [illegible] specimen shape and size and the curing conditions are entirely different. It will also be noted that the points for the wet dry controls at 10% cement on graph 9.22 (+ x * $) are all well above the regression line indicating a different slope and / or intercept. This coincidence is considered significant in that it reduces the differences between the data for the wet-dry and leaching tests thus improving the comparison between the results.

Regression on the points for specimens at age 28 days also yielded a slope of 0.125 x cement content with a negative intercept of 0.37%, regression coefficient (r^2) was 0.96. Consideration of the range of data indicated that exclusion of certain extreme points would yield a flatter slope in the range 0.119 to 0.125 with similar intercept and r^2. Since there was no particular basis for omission of any of these points, the 0.125 factor was adopted. However, in deriving the relationship with cement content and age, depicted on graph 9.23, it is apparent that a flatter slope at 28 days or a steeper slope at 300 days could be appropriate. For the purpose of analysing the experimental data, the regressions referred to above are considered adequate.

GRAPH 9.22

FINAL MOISTURE CONTENT OF CONTROL SPECIMENS
AT VARIOUS AGES vs CEMENT CONTENT; FOR VARIOUS
CURING AND COMPACTION CONDITIONS AND SPECIMEN TYPES



a Std Cyls, Std Cure 28d
4 +/- 300d
1 Cyls 1% Dry of OMC 290d
| 1% Wet of OMC 290d
1 1% Above MDD 290d
3 Cyls Std->28d, Wet->35d
7 Cyls Humid to 28d, 289d
8 Cyls Leach Bath 7-28, 290d
4 Cyls Wet-Dry 12 cycles
1 ditto Sealed to 292d
4 ditto Humid to 290d
3 Cyls Wet-Dry 24 cycles
6 Disks, Std Cure +/-280d

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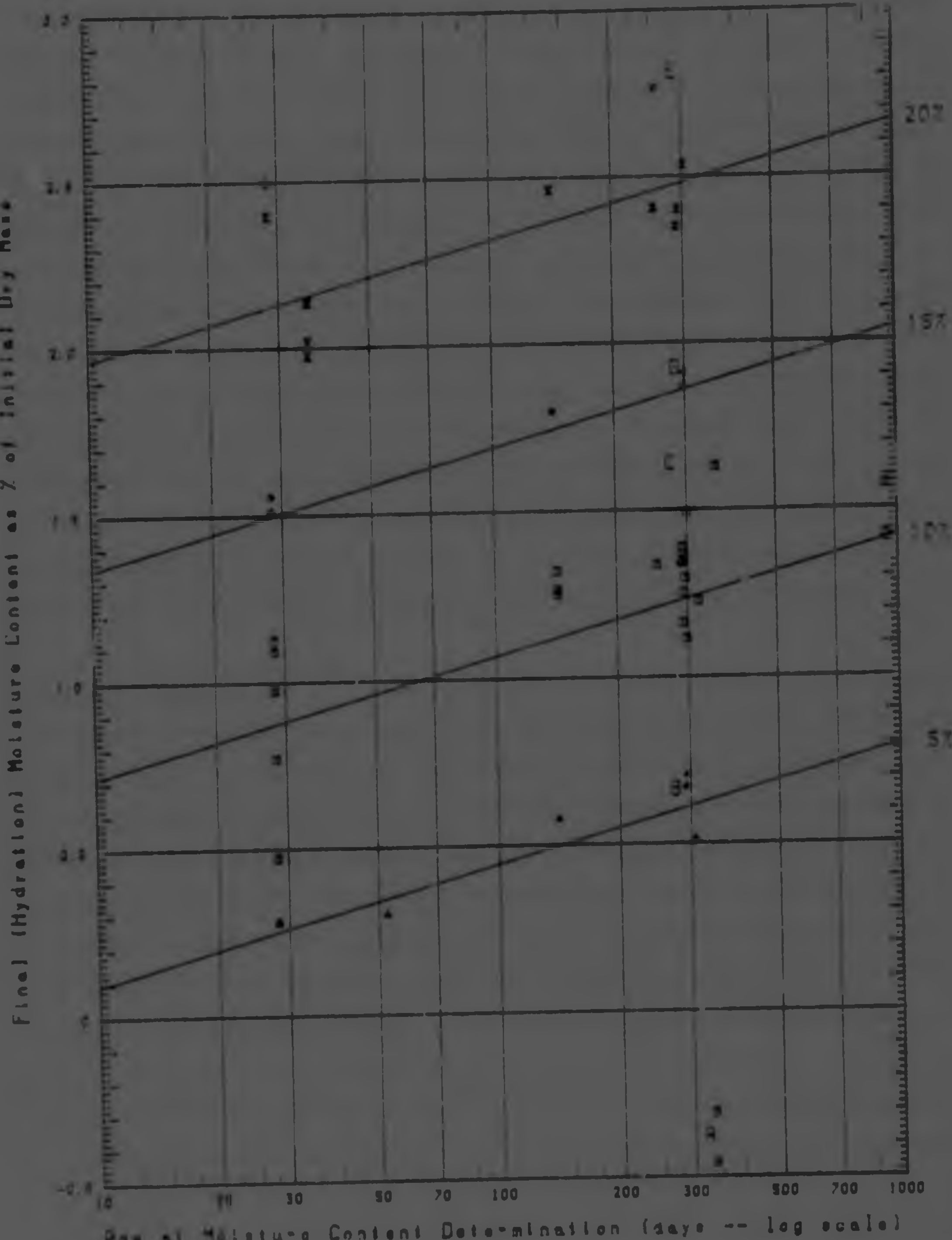
GRAPH 9.23

FINAL MOISTURE CONTENT OF CONTROL SPECIMENS vs AGE



- Standard Cylinders 0%
- Standard Cylinders 5%
- Standard Cylinders 10%
- Standard Cylinders 15%
- Standard Cylinders 20%
- Standard Cylinders 25%

A Disk Specimens 0% Cement
B Disk Specimens 5% Cement
C Disk Specimens 10% Cement
D Disk Specimens 15% Cement
E Disk Specimens 20% Cement

FMC-AGE PLT -- 10/3/84-8=1220-v 1.3 (LCM-CONF DAT. 19)

The hydration moisture content - cement content points at ages 28 and 300 days derived from the regressions were used in plotting the cement content contours on graph 9.23. Regressions on these points were used in calculating the hydration correction at the appropriate age to be applied to test specimens. Because the 28 and 300 day lines in graph 9.22 are parallel, the contours in graph 9.23 are parallel, a degree of convergence might however be reasonably expected but because of the semi-log nature of the plot and the minimum value of 10 days, it was considered that the lack of convergence would not have a significant effect on the results. As data was only available at 28 and 300 days and since this type of data on a semi-log plot generally yields a straight line, straight lines were used for the contours on graph 9.23. This is supported by the points at 900 days at 10% cement. On a linear plot, a family of curves originating at the origin and approaching a time asymptote would be expected. The linear regressions of final moisture content vs log(age) yielded a slope of 0.35.

Control specimens for the unstabilized condition were only examined at +/- 300 days and these specimens indicated a negative final moisture content of the order of 0.4% which coincides with the intercept for the moisture content - cement content line at 28 days on graph 9.22. Since there is no basis on which to expect a change of mass of unstabilized specimens over a period, it is considered that this negative value represents a constant offset for unstabilized specimens over the interval. This offset, which is apparently a zero error, could result from a number of factors:-

a. It is probable that the soil contains a small amount of organic matter, decay of this material in the warm, moist environment of the unstabilized specimens could account for the loss. A similar decay in the stabilized specimens could also have occurred thus leading to the negative intercept for stabilized specimens at 28 days. The increase in alkalinity and decrease in available moisture with increasing cement content, could reduce the extent of this decay thus accounting for the parallel calibration in graph 9.22 where lines diverging from the origin with increasing slope might reasonably be expected.

b. It is possible that an error in moisture content determination during compaction occurred. This would have the effect of a vertical translation of the data in graph 9.22. As noted in section 9.3.1, moisture content samples were placed in sealed tins after the compaction of each specimen but were only weighed after the completion of one or more batches. While it is considered unlikely that moisture loss from the sealed containers could have occurred, evaporation of moisture into the atmosphere within the container would be expected. This might lead to a reduction in the measured mass of the container plus sample. In this case, mixtures with higher cement contents would be expected to have less free water than those with less cement, this could also give rise to a rotation of the lines in graph 9.22.

   This suggestion is supported by a discrepancy between measured hygroscopic moisture contents for soil and cement and batch moisture contents noted at the commencement of the test program. This amounted to a moisture loss equivalent to a moisture content of 0.54%. As the majority of references, including standard methods of test, allow an addition of 1% excess moisture to allow for evaporation during batching and mixing, it was concluded that the discrepancy of 0.54% resulted from this form of evaporation and that the 1% allowance was excessive. It is possible that the evaporation in fact occurred from the moisture content specimens themselves as suggested above. In this case, the error of 0.54% is comparable to that of 0.4% noted in graph 9.22.

The above explanations both have merit but require additional investigation to establish whether they in fact apply or whether the anomalies result from some other factor. In either of the above cases, the correction would involve the same translation and rotation to both control and test data and would be small relative to the observed scatter in the experimental results depicted in graphs 9.22 and 9.23. It was therefore decided that no further investigation or correction was justified.

A number of trends were noted during the processing of the control data:-

c. Hydration gains for individual specimens within a batch were generally consistent, agreeing to the second decimal place in most cases.

d. Variations between batches, particularly over a range of compaction dates were relatively substantial. This is particularly evident in the spread of results for nominally identical specimens containing 10% cement, the triangles and circles on graph 9.22. This could contribute to the spread of permeability results within a set in graphs 9.1 to 9.21 where in most cases, each specimen was from a different batch.

e. A progressive increase in hydration gain from 7 to 28 to 140 to 300 days is discernible for sets compacted on the same day. This gain is masked by scatter for sets compacted on different days.

The effect of various compaction and curing conditions on the hydration gain observed in the control specimens and plotted on graph 9.22, are discussed below, letters in brackets refer to the symbols on the graph:-

f. Specimens compacted wet (m) and dry (d) of OMC appear to show slightly higher and lower gains respectively but are within the range of standard specimens.

g. Curing in the leaching bath from 7 to 28 days followed by sealed curing (") produces a marked drop in final moisture content due to leaching while in the bath.

h. Specimens subject to wetting and drying cycles exhibit a m-rked increase in hydration gain relative to specimens subject to the standard cure. Those subject to 12 cycles and then sealed (x) show no greater gain than those tested after 12 cycles (+). Those s bject to a humid cure after 12 cycles (*) show a marked gain, comparable to that for specimens subject to 24 cycles of wetting and drying ($). The data for the leached specimens subject to initial cure of 12 cycles of wetting a d drying is compared with the humid controls in the remainder of this chapter as this more closely approximates the moisture condition during leaching, the data relative to the sealed control is presented for comparison.

i. Humid curing of standard specimens from 7 to 28 days (H), instead of sealing, has no significant e fect on the observed hydration gain.

j. Hydration gains for disk specimens are g nerally comparable to those for cylinder specimens.

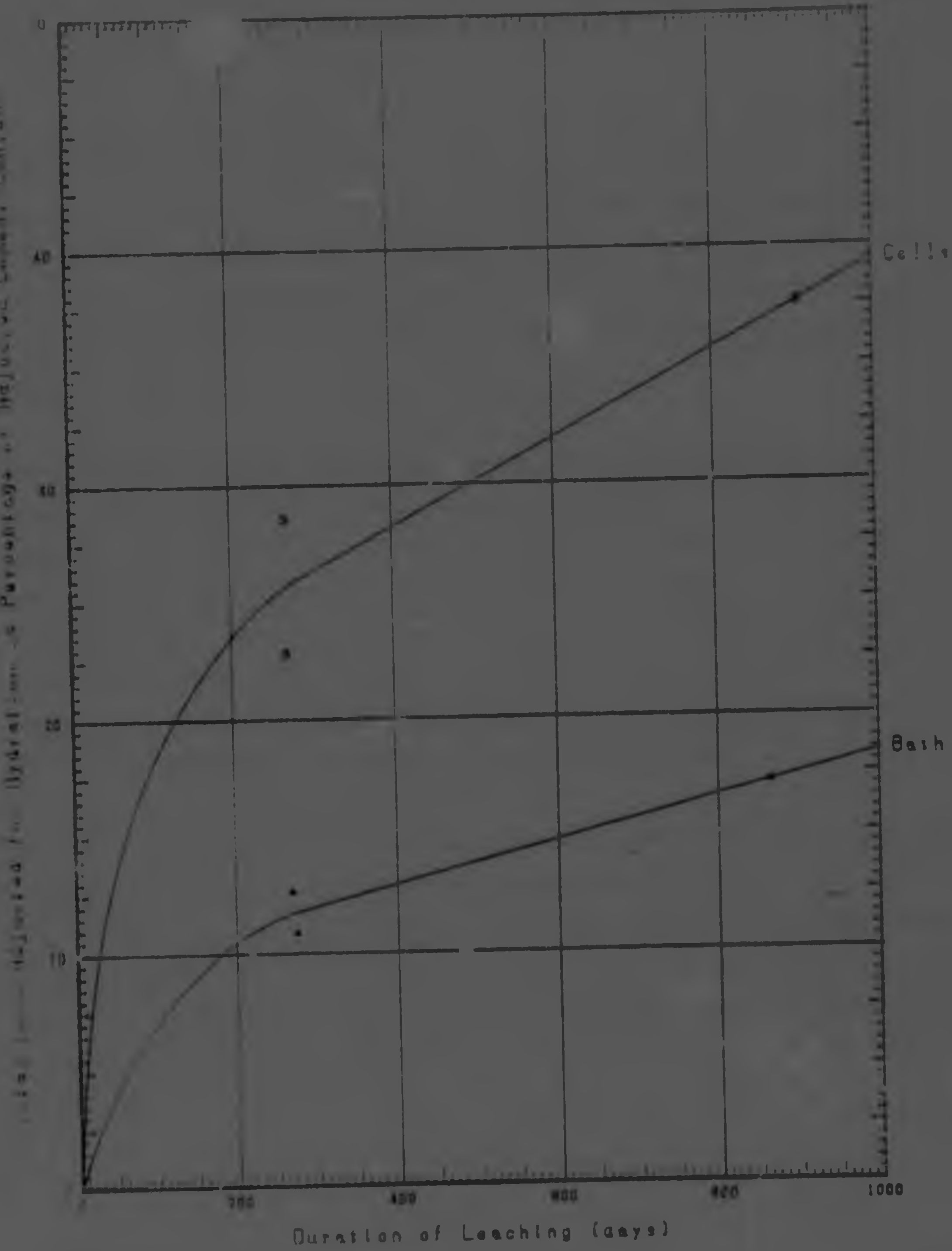
9.4.9 Loss with Period of Leaching

The loss with period of leaching, depicted in graph 9.24, has been referred to in a number of previous sections. As mentioned in section 9.1, the tests at inter iiate ages of 28, 56 and 112 days were abandoned owing to problems with the triaxial apparatus. The only data avail e was thus for the standard specimens subject to 266 and 904 day leaching. The origin provided a third known point.

The specime s cured for 350 days before leaching were leached for a period of 587 days instead of the standard 266 day period thus introducing a second unknown into the results. In order to es.imate the loss in the 266 day period, the data in graph 9.24 was used. Because of the limited number of data points, a linear

GRAPH 9.24

LOSS WITH PERIOD OF LEACHING



Standard Cylinders 10%. Leached in Leaching Cells

Leached in Leaching Bath

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interpolation between the average of the points at 266 and [illegible] days was adopted as the best approximation. It is however likely that a curved trend would apply in which case, curves sketched by eye indicate asymptotes of less than 20% and 40% loss for leach bath and cells respectively.

The linear interpolation yielded a ratio of loss at 58[illegible] to loss at 266 days of 1.218 for both the leach bath and leach cell specimens. It was therefore concluded that the same factor could be applied to the specimens cured for 350 days without excessive error. The results for these specimens were adjusted accordingly and both the corrected ($) and uncorrected (D) results are used in subsequent sections.

The general form of the curves in graph 9.24 is considered to be a good indication that a progressive reduction in leaching rate occurs. The limiting value of 40% loss suggested above is supported by the observation that the maximum loss for cylindrical specimens does not exceed this value for any of the specimens and test conditions investigated. Leaching tests over a period of five years or more are however required to substantiate this. A different asymptote will apply to disk specimens.

It is interesting to note that if the mass loss data for the standard ASTM wet-dry test specimens with 10% cement, discussed in chapter 8, are plotted on graph 9.24 using the cumulative soaking period as the duration of leaching, they plot between the curves for the leaching cells and leaching bath. The estimated equivalent leaching losses at the wet-dry periods of 2.5 and [illegible] days are:-

	Loss at	
	2.5 days	5 days
Leaching Bath	0.25%	0.50%
Standard Wet-Dry	0.91%	1.36%
Leaching Cells	1.25%	2.50%

This suggests that deterioration in the wet-dry test may be primarily as a result of surface leaching.

9.4.10 Summary of Leaching Bath Results

The adjusted loss results for standard disk and cylindrical specimens leached for the standard period after standard cure are plotted against cement con[illegible] in percent on a reciprocal scale (as for the wet-dry results, [illegible]aph 9.25. Linear regressions on the transformed data yield regression coefficients of 0.99 and 0.94 for disks and cylinders respectively indicating hyperbolic characteristics.

The average ratio of losses of disk and cylindrical specimens is 1.14. This compares favourably with the surface area ratio of 1.13 confirming that leaching in the leaching bath is a surface effect.

The leaching bath loss data is plotted against age at start of leaching on a log scale in graph 9.26. A linear regression on the transformed data for the standard cylindrical specimens containing 10% cement and subject to the standard test conditions (A, 4, 5, C and $) yields a regression coefficient of 0.97.

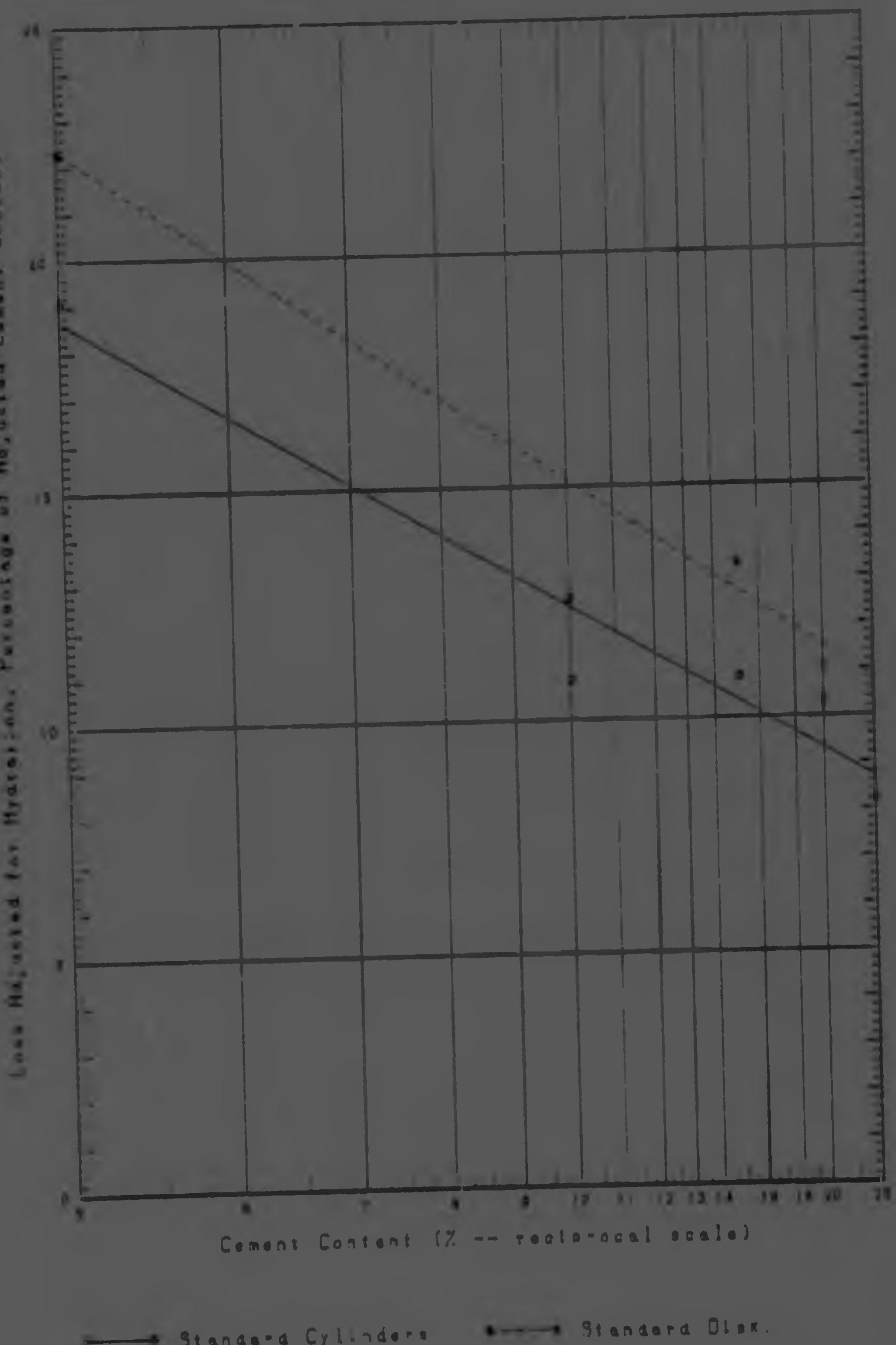
Losses for specimens compacted dry of OMC (W) and above MDD (Y) are slightly above the regression line but within the range of the standard specimens. The loss for specimens compacted wet of OMC (X) lies below the regression line, indicating an increased resistance to leaching.

The loss in terms of humid control for specimens subject to the wet-dry cure (B) lies well below the regression line and the loss for these specimens calculated in terms of the sealed controls (*) lies very close to the axis. The use of the humid control as reference is therefore justified. A significant increase in resistance to leaching for specimens subject to wetting and drying cycles is indicated.

GRAPH 3.25

LOSS IN LEACHING BATH vs CEMENT CONTENT

STANDARD DISK AND CYLINDER SPECIMENS WITH STANDARD CURE

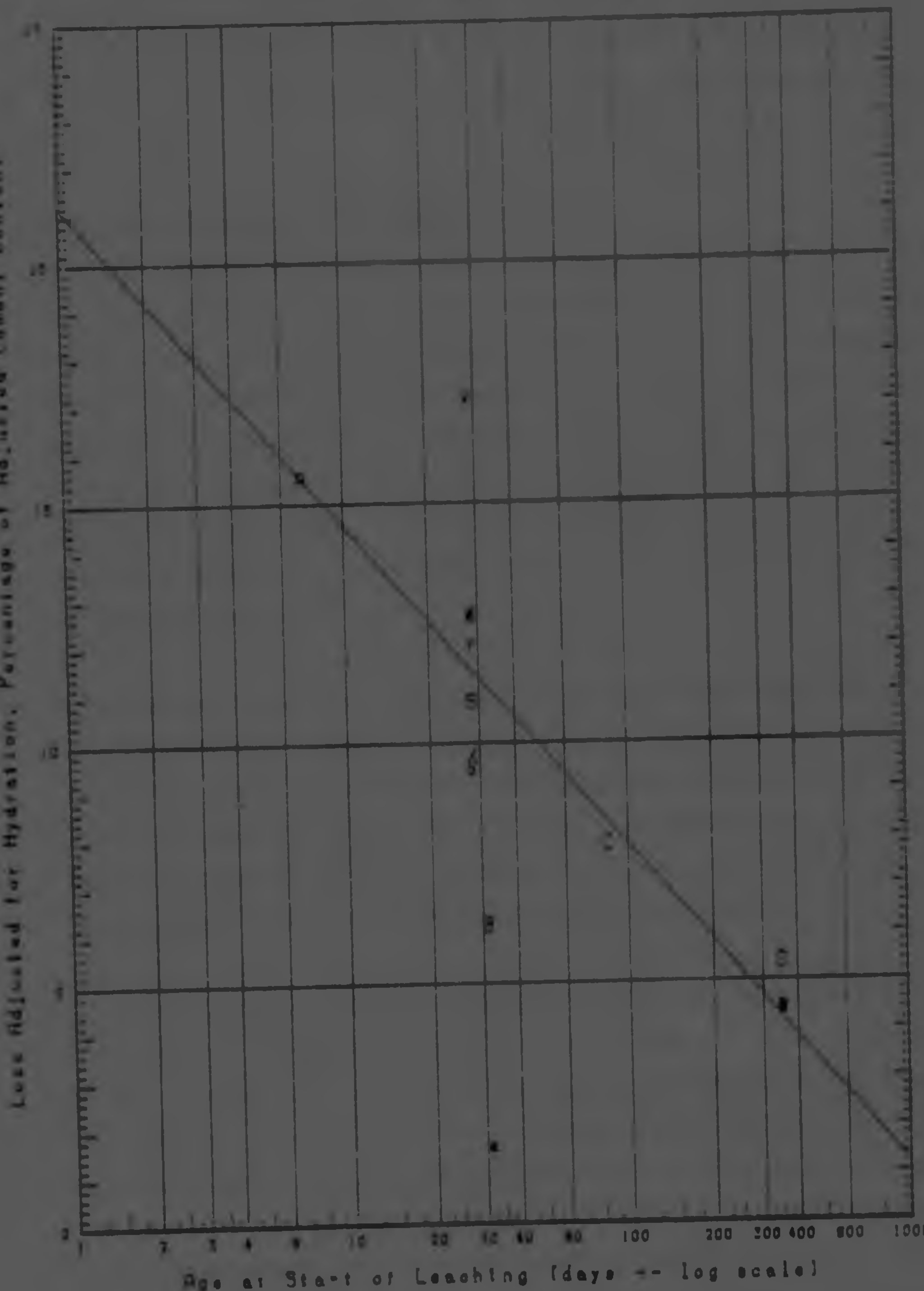


[illegible] -- 10/04/84-[illegible] DAT. [illegible]

GRAPH 9.26

LOSS vs AGE AT START OF LEACHING

IN LEACHING BATH, SPECIMENS CONTAINING 10% CEMENT



A Leach Bath from 7 days
4 Standard Cyl, Std Cure
5 ditto -- second set
C 7d Humid 77d Sealed Cure
D 7 Hmd, 343 Seal, 550 Lch
# D adjusted to 270d leach
Line = regr'n A-4-5-C-#

W 1% Below Proctor O.M.C.
X 1% Above Proctor O.M.C.
Y 2% Above Proctor M.D.D.
V Cylinders 129 week leach
12 Cyc W/D ref Hum.d Ctl
ditto ref Sealed Control
Standard Disks, Std Cure

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9.4.11 Summary of Leaching Cell Results

The results for specimens tested in the leaching cells are summarised in graphs 9.27 to 9.30 and table 9.3 and discussed below, a fold out legend is attached to table 9.3, following graph 9.30.

9.4.11.a Relationship with Cement Content

Graphs 9.27a to d summarise the leaching ce results in terms of cement content on the horizontal axis to facilitate comparison. Graphs 9.27a and b summarise the loss results in terms of cement content. Graph 9.27a depicts the detailed data for the cylindrical specimens while graph 9.27b reflects the trend lines for the cylinders together with the data for the disk specimens. Graph 9.27c relates the total flow to cement content and graph 9.27d compares the final permeability of the specimens.

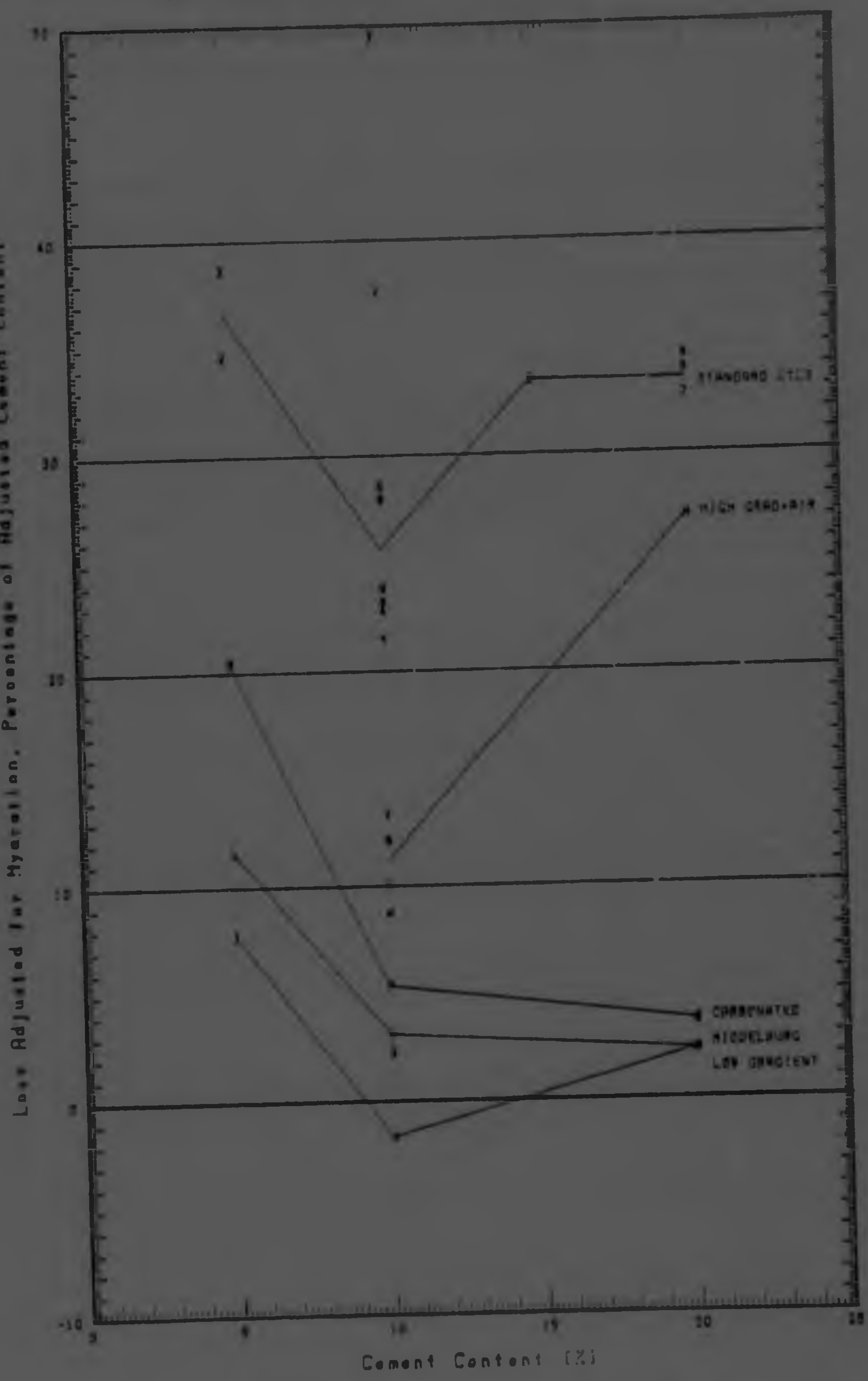
The general shape and relationship between test results of the loss and total flow plots vs cement content is similar, confirming the loss - flow relationship referred to earlier. While the general shape of the final permeability vs cement content plots is similar to those for loss and total flow, the relationship between them differs significantly.

Flow and permeability for standard cylinders and disks show a substantial increase with increase in cement content from 0 to 5%. Thereafte-, they show a slight decrease to 10% cement and this is associated with a substantial decrease in loss. This decrease in flow and permeability from 5 to 10% cement and the associated decrease in loss occurs for all the test conditions examined. The standard cylinders, low gradient cylinders and high gradient cylinders all show a pronounced increase in flow, permeability and loss from 10 to 20% cement. It would appear that where de-aired distilled water is used as the leaching fluid, about 10% cement provides the optimum reduction in pore volume for the cylindrical specimens.

GRAPH 9.27a

ADJUSTED LOSS vs CEMENT CONTENT

FOR CYLINDRICAL SPECIMENS IN LEACHING CELLS

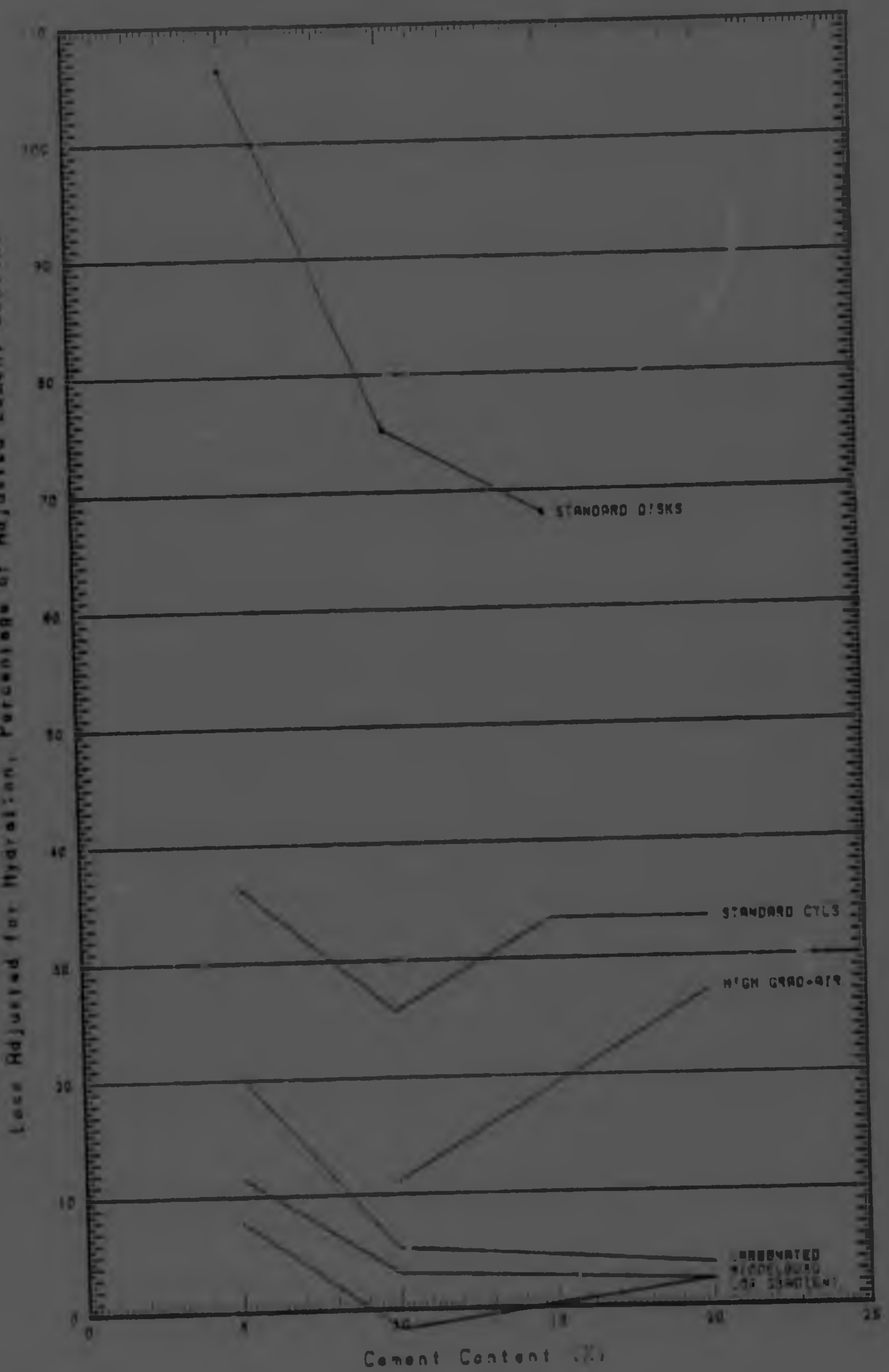


For individual codes see detailed legend.

GRAPH 9.27b

ADJUSTED LOSS vs CEMENT CONTENT

FOR DISK AND CYLINDRICAL SPECIMENS IN LEACHING CELLS

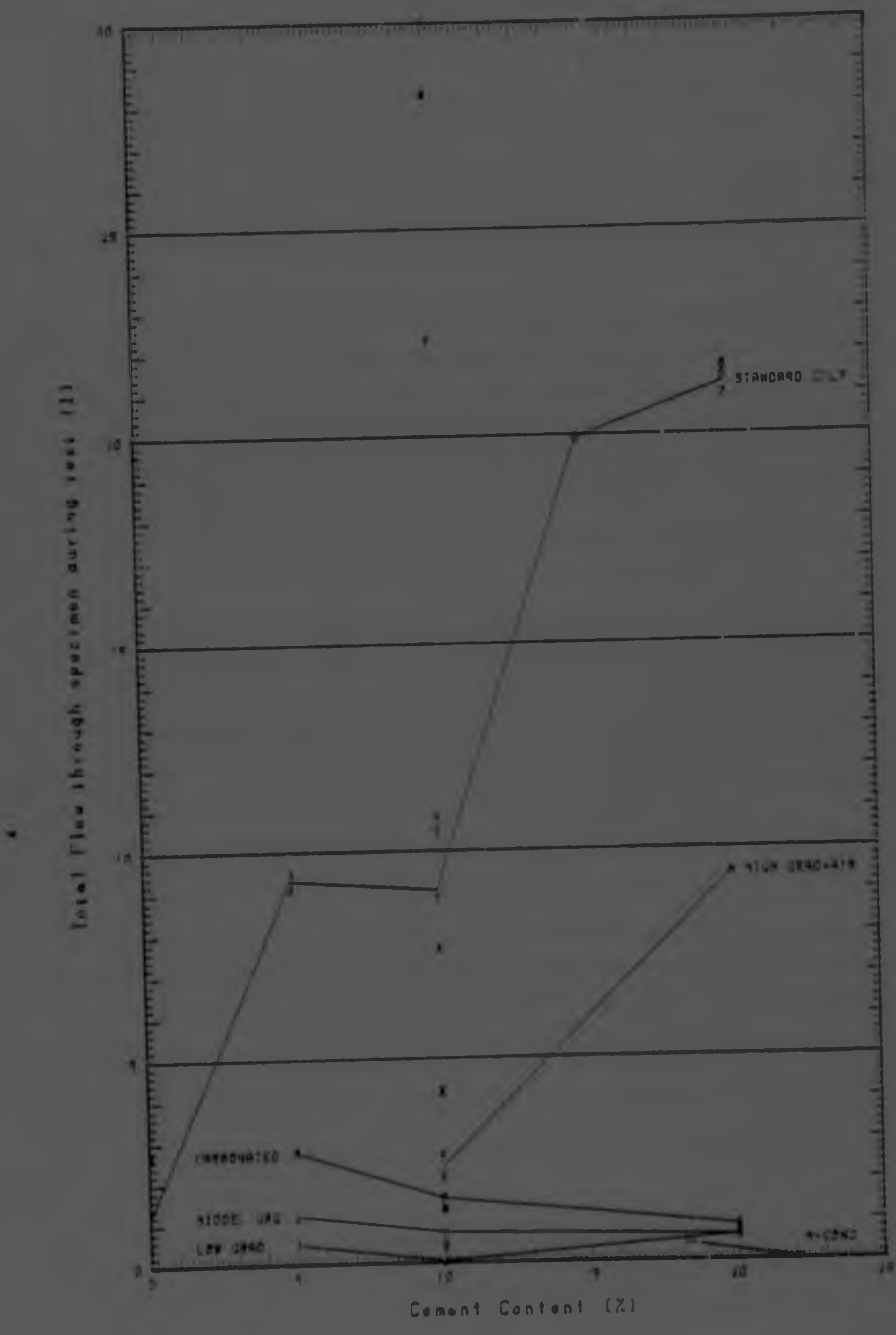


For individual codes see detailed legend.

GRAPH 9.27c

TOTAL FLOW vs CEMENT CONTENT

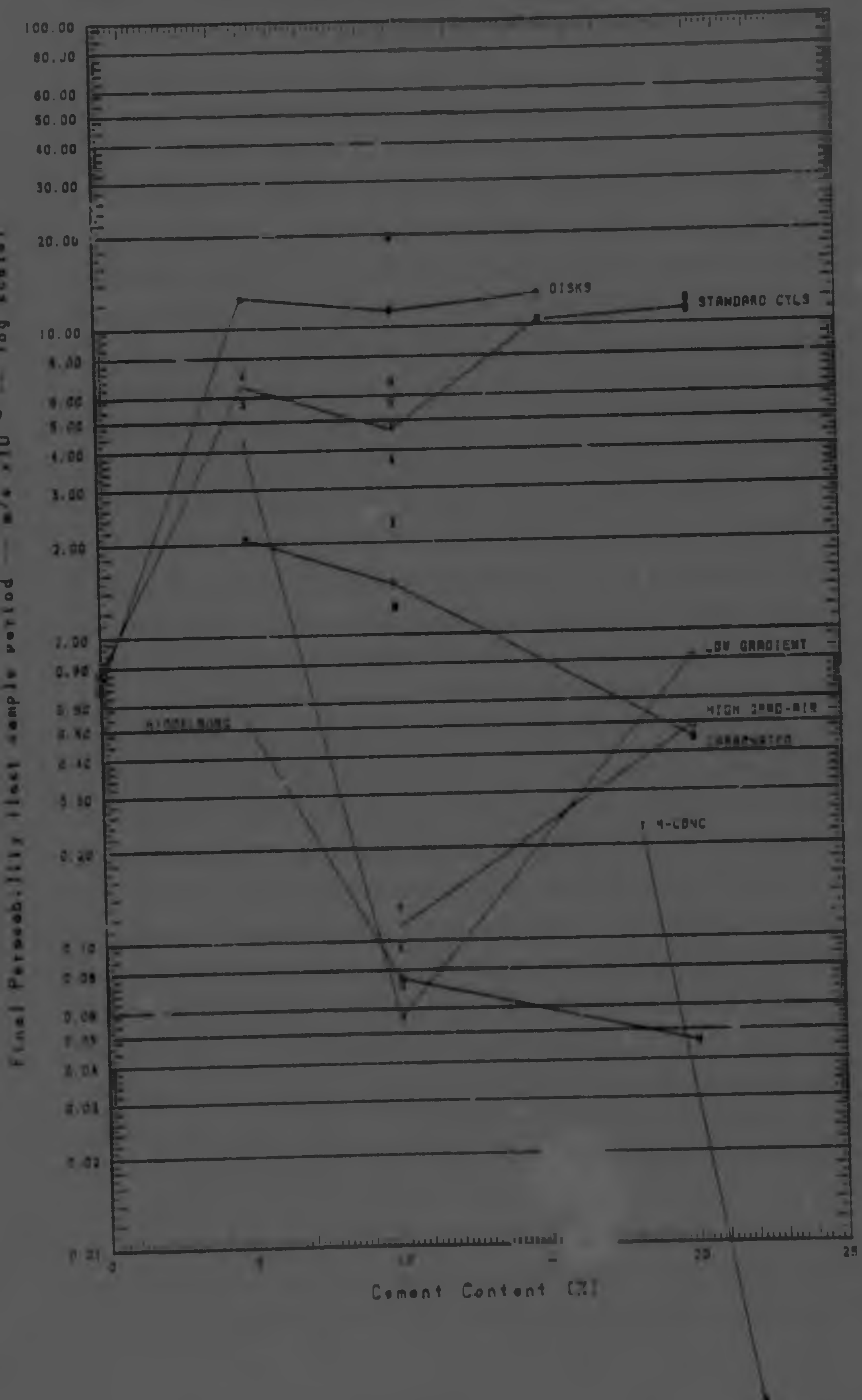
FOR SPECIMENS IN LEACHING CELLS



GRAPH 9.27a

FINAL PERMEABILITY vs CEMENT CONTENT

FOR SPECIMENS IN LEACHING CELLS



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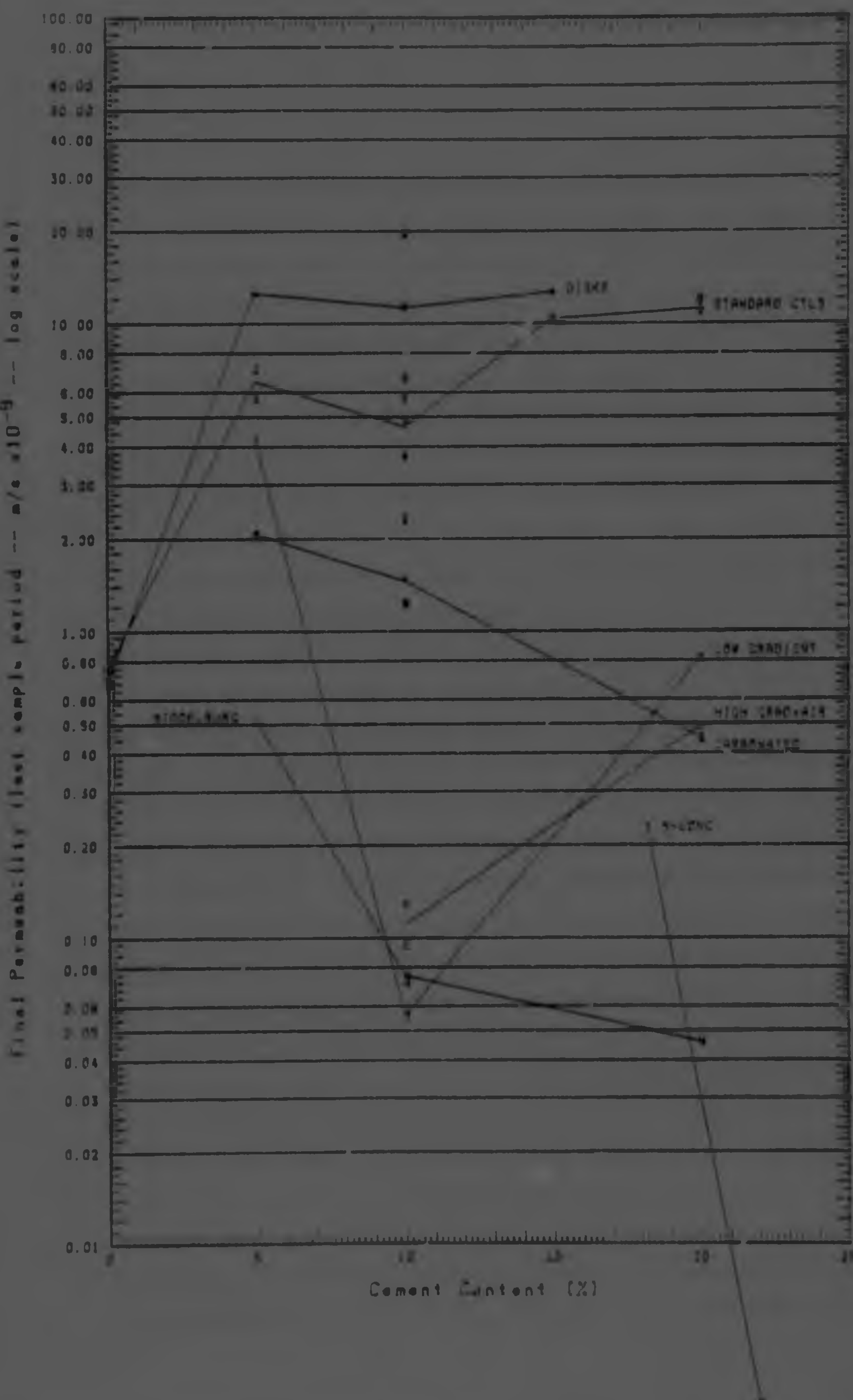
GRAPH 9.27d
FINAL PERMEABILITY vs CEMENT CONTENT
FOR SPECIMENS IN LEACHING CELLS



LC-C-PRM PLT -- 20/10/94-0=1330-v 1.4 (LCM-CONF.DAT, 20 1

Where water containing dissolved gases is used, a further reduction occurs with an increase in cement content from 10 to 20% and is also observed with the disk specimens despite a slight increase in permeability. Total flow data was not calculated for the disk specimens because of intermittent leaching during the first weeks of testing.

It is notable that the low gradient specimens with 10% cement show a negative loss indicating a gain in mass. This indicates that in a low flow situation where additional water is available and leachate is only slowly removed from around the specimen, additional hydration occurs which assists in further reducing permeability. This suggests that in a unit gradient situation, no significant leaching into the body of the structure will occur and that leaching will be limited to surface leaching as observed in the leaching bath.

Losses for the disk specimens are considerably higher than for the standard cylinders, reaching 106% at 5% cement content, indicating complete removal of stabilizer, while permeabilities are similar. It is considered that this behaviour is consistent with a leaching model in which percolating water rapidly dissolves calcium as it enters the specimen and thereafter continues in an equilibrium condition with the soil-cement. A minimum depth is required to develop this equilibrium and thereafter only limited deterioration will occur in the remainder of the specimen. The zone of deterioration will advance into the specimen as leaching progresses. This implies that the loss can be associated with a depth of deterioration and consequently, for the same depth of deterioration indicated by a similar total flow or average permeability, the percentage loss from a 20 mm thick disk specimen will be considerably greater than that from a 75 mm long cylinder.

The presence of air in the cell fluid has a considerable effect on the permeability and thus total flow and loss from the high gradient specimens and their corresponding controls. The final permeability is reduced to a similar level to that for the low gradient specimens, lying between the carbonated and Middelburg results. The total flow and loss data lies between that for the low gradient and standard gradient specimens. The data for the 10% control specimens with standard gradient and air cell pressure is effectively coincident with that for the specimens leached with Middelburg water, confirming the substantial reduction in permeability associated with the presence of air.

The specimens leached with carbonated water show slightly higher loss and total flow results than those leached with Middelburg water but this data is of similar magnitude, well below that for the standard specimens. This behaviour is attributed to the presence of dissolved gases as noted above. The final permeability of the specimens leached with carbonated water is however substantially higher than for the Middelburg water. This is consistent with the results discussed in section 9.4.7.j and indicates that the presence of carbon dioxide inhibited hydration and hence the reduction in pore volume that leads to the reduction of permeability in other specimens. If the difference between the final permeability of the carbonated, Middelburg and standard gradient cases in graph 9.27d is taken into account, it is possible that if the leaching medium in the carbonated specimens had been changed to de-aired distilled water at 266 days and the specimens de-aired, the resulting permeability would have been substantially above that for the standard specimens.

Compaction at 2% above MDD (Y) has no significant effect on loss, permeability, or flow and cannot be considered worthwhile in terms of improved water retaining properties. The data points for the specimens compacted dry of OMC (W) lie significantly above those for the standard specimens confirming the observation in section 9.4.7.b that compaction dry of OMC is detrimental to the leaching resistance of the material.

These results indicate that particular care should be exercised in construction to avoid compaction unnecessarily dry of OMC, particularly in the upstream zones. Compaction wet of OMC (X) leads to an equally significant reduction in permeability, flow and loss indicating that the tendency in construction should be to err on the wet side of OMC as far as practical. Control of moisture content might also be utilised to develop an increase in permeability downstream from the zone considered to comprise the water barrier in a nominally homogeneous embankment in order to avoid development of excess pore pressure towards the downstream face of the dam. This factor could also be considered in specifying construction moisture content tolerances for different zones in the embankment, a tighter tolerance being applicable to the upstream zone.

Losses for specimens subject to 12 cycles of wetting and drying followed by standard test conditions (B), lie in the same range as those for standard specimens despite substantially higher total flows (28 litres cf +/- 10 litres) and somewhat higher final permeability. This indicates a considerable acceleration of hydration or fixing of hydration products as a result of the application of heat.

Specimens subject to 21 day cure in the leaching bath also show a considerable reduction in loss relative to both the standard control specimens (A) and the control specimen subject to 21 day cure in the leaching bath and subsequently sealed (#). Thus, despite the initial loss resulting from curing in the leaching bath, an overall reduction in loss, total flow and permeability is achieved. This indicates the advantages of the application of additional water in the early stages after compaction.

The data for the microconcrete specimens reflects the reduction in permeability with increasing cement content discussed in section 9.4.7.2.

9.4.11.b **Relationship between Loss and Flow**

Graph 9.28 depicts the relationship between adjusted loss and total flow for cylindrical specimens. The general trend is indicated by a curve drawn by eye through the body of the data.

The data for specimens with 5% cement (2, 3, I, L and O) constitutes most of the points significantly above the trend line. The point representing specimens leached for 904 days (V) constitutes the only other major upward deviation suggesting a shift with increased period of leaching. In view of the 340% increase in leaching period, the 12% increase in loss is however relatively minor, as discussed in section 9.4.9.

The data for specimens subject to the wet-dry cure (B and *) and that for specimens subject to extended sealed cure (C and D) constitute the only points significantly below the trend line.

The curve, as drawn, is therefore a good indication of the loss - flow relationship for standard cylindrical specimens containing 10% cement and above and subject to standard cure and leaching period. A family of curves can be expected for various curing periods, conditions and leaching periods. This curve again suggests a 40% loss asymptote for the standard cylinders and leaching period.

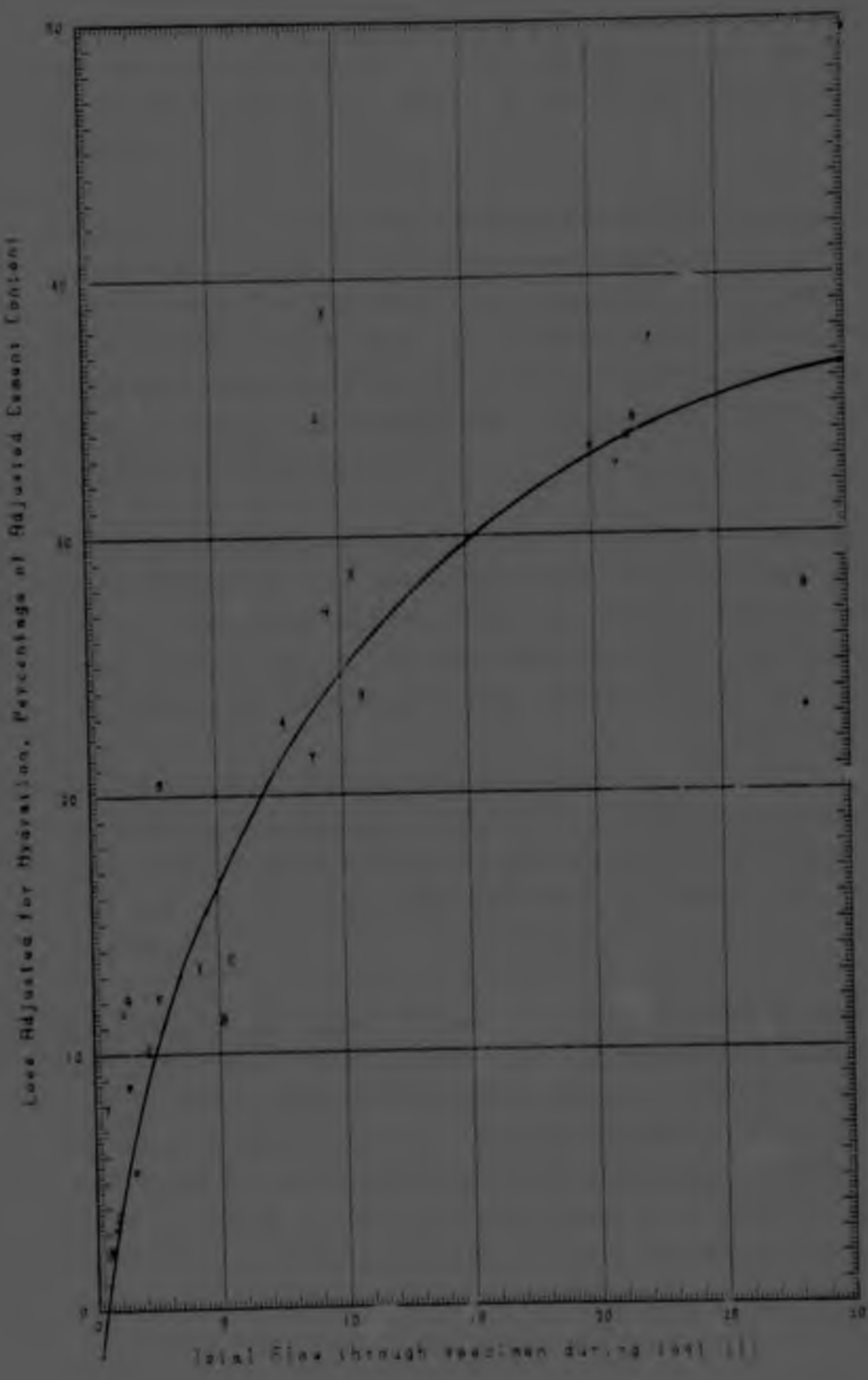
9.4.11.c **Relationship with Age at Start of Leaching**

Graphs 9.29a, b and c summarise the leaching cell results in terms of age at start of leaching. Loss, total flow and final permeability are plotted against age at start of leaching on a log scale.

The trend line for standard cylindrical specimens with 10% cement subject to 7 days humid cure and 21, 77 and 343 days sealed cure and leached for 266 days are plotted on each graph. These lines are drawn by eye through points 4, 5, C and S. Data points for other standard specimens (2, 3, 6, 7 and 8),

GRAPH 8.28

RELATIONSHIP BETWEEN ADJUSTED LOSS AND TOTAL FLOW



For individual codes see detailed legend.

together with that for the standard 10% specimens subject to 904 day leach (V), are plotted as an indication of how the trend would be influenced by changes in the test conditions. The data point for specimens leached from age 350 for 587 days (D) gives a further indication of the effect of prolonged leaching.

In all cases, a pronounced downward trend with increase in curing time is apparent with indicated asymptotes of 7% loss, 2 litre total flow and 2×10^{-9} m/s permeability at a starting age in excess of 1000 days. It is particularly notable that while an increase in flow and loss occurs with an increase in leaching period, the permeability continues to reduce with increased leaching period.

It is reasonable to assume that the full gradient is unlikely to be applied to any soil-cement dam of significant size within a period of less than one year, and the observed data for a starting age of 350 days confirms that a significant improvement in material properties can be expected.

9.4.11.d Relationship between Loss and Gradient

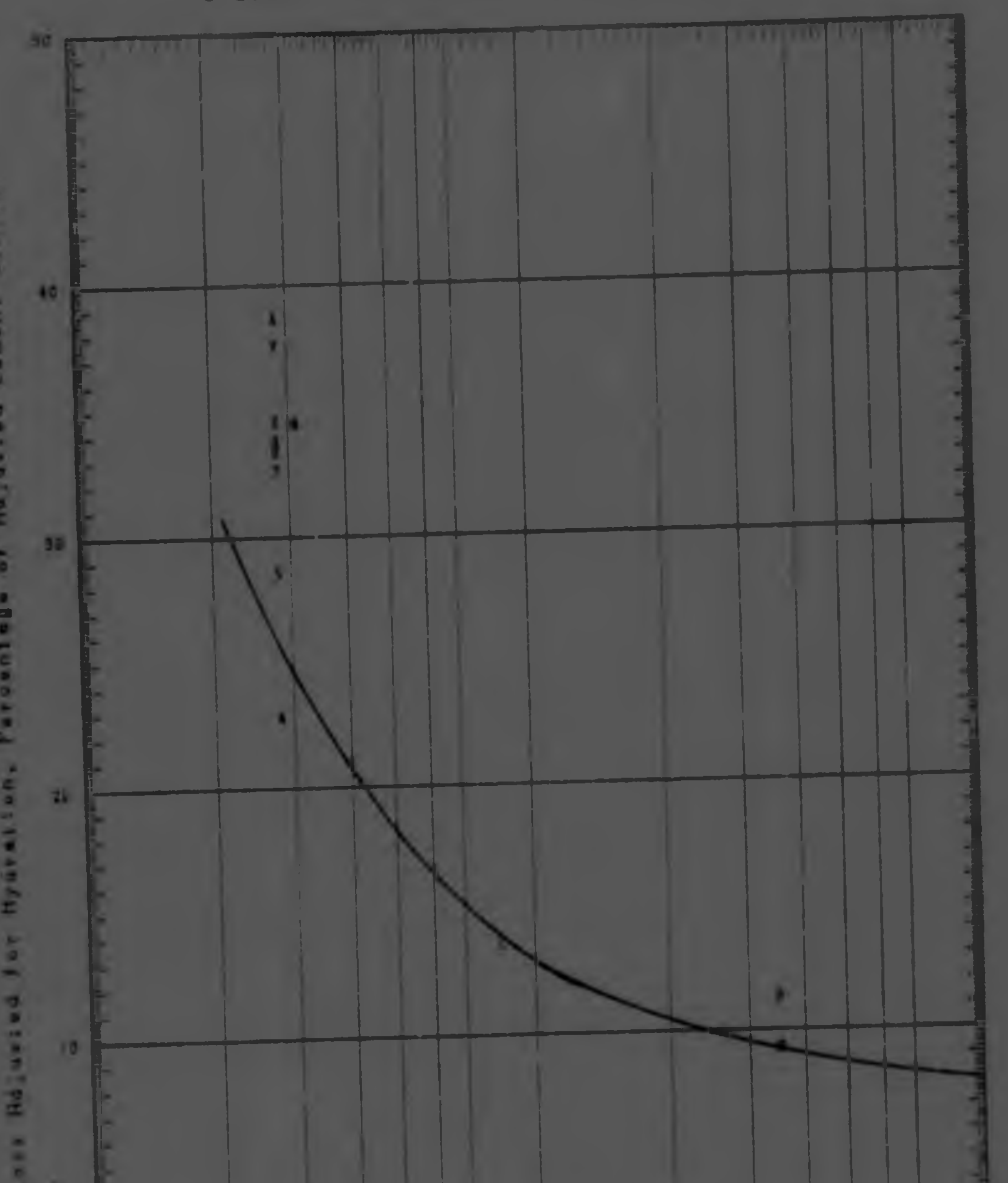
The relationship between adjusted loss and hydraulic gradient is depicted in graph 9.30 for the standard specimens subject to the low (10 m/m), standard (50 m/m) and high (136 m/m) gradients.

The reversal in degree of loss between 5, 10 and 20% cement depicted on graph 9.27a is evident as is a substantial increase in loss with increase in gradient from 10 to 50. A lesser increase is evident with an increase in gradient from 50 to 136. This is to be expected as the percentage increase in gradient from 50 to 136 is 272% as opposed to the 500% increase from 10 to 50, however, it is likely that the presence of air as cell fluid contributed to this reduction.

GRAPH 9.29a

LOSS vs AGE AT START OF LEACHING

CYLINDRICAL SPECIMENS IN LEACHING CELLS

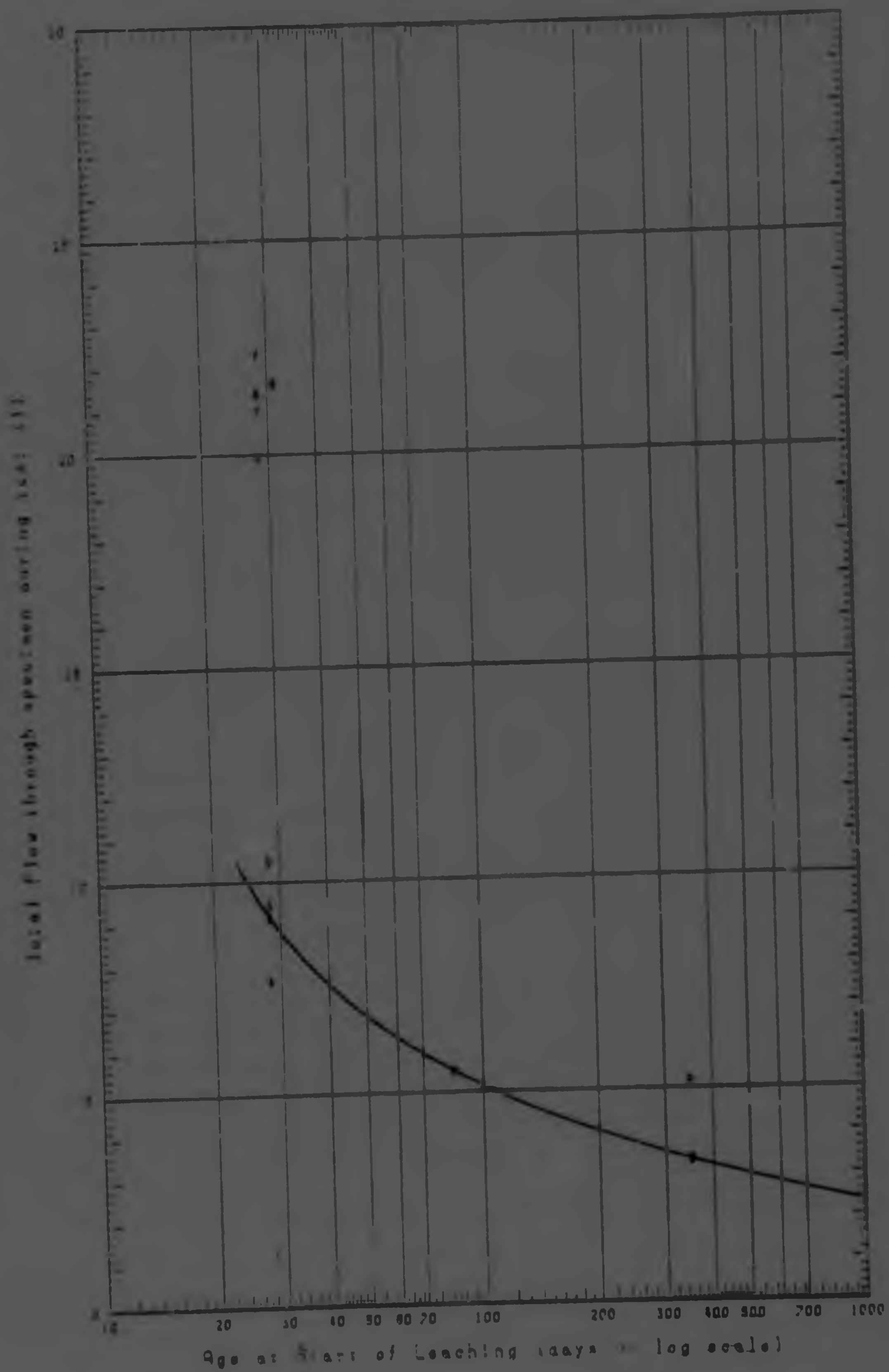


——— Standard cylindrical specimens with 10% cement.
For individual codes see detailed legend.

GRAPH 9.29b

TOTAL FLOW vs AGE AT START OF LEACHING

CYLINDRICAL SPECIMENS IN LEACHING CELLS



——— Standard cylindrical specimens with 10% cement.

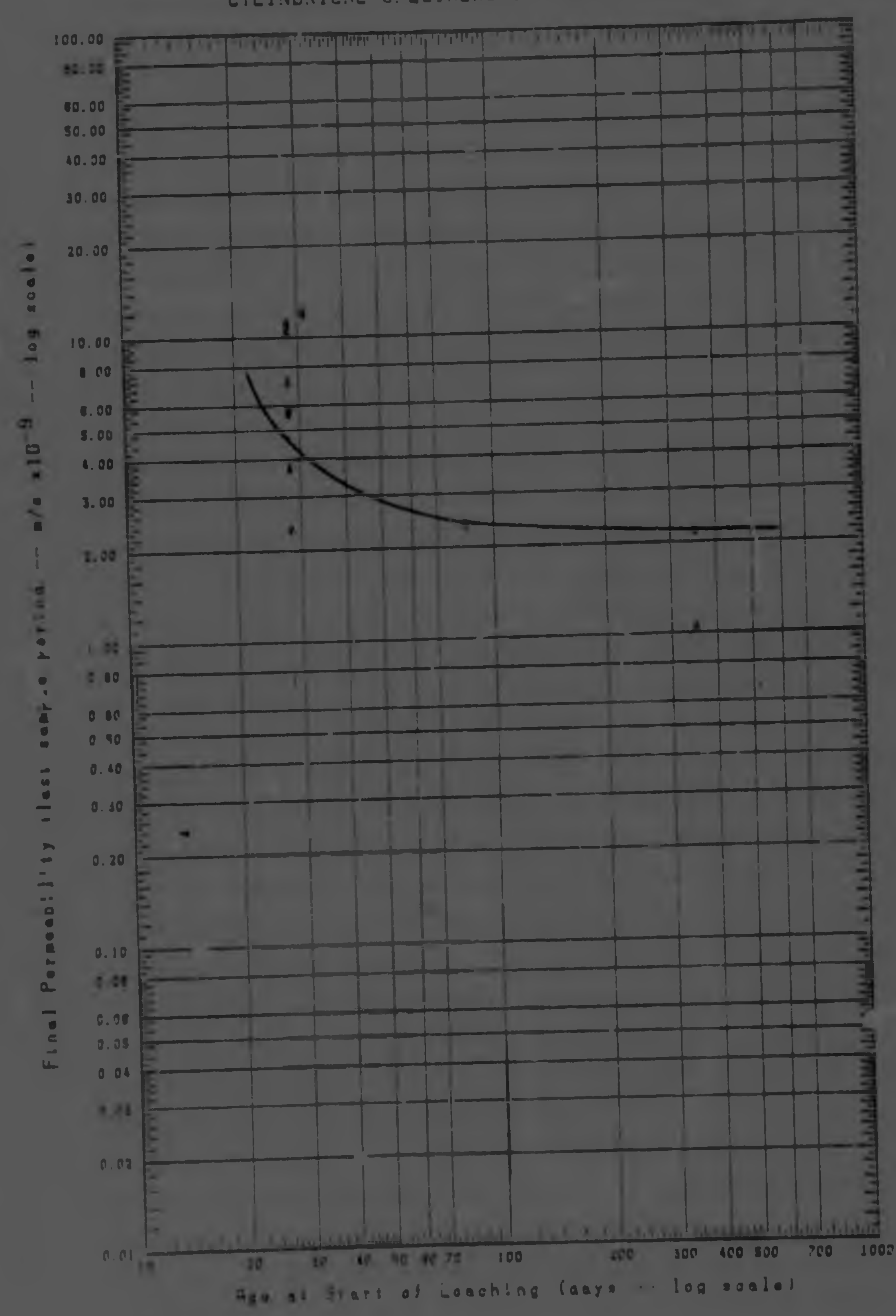
For individual codes see detailed legend.

LCAGEFLW.PLT — [illegible] (LCH-CONF DAT. 35)

GRAPH 9.29c

FINAL PERMEABILITY vs AGE AT START OF LEACHING

CYLINDRICAL SPECIMENS IN LEACHING CELLS



LCBGEPRM.PLT -- 12/4/84-0e1239-- I D LCH-CPMF DBT 37 I

The increase in loss with gradient is accounted for by the increased flow resulting from the increase in gradient with constant leach period. The object of using a higher gradient than will occur in practice was to accelerate the test, the usual assumption being that a test conducted with a tenfold increase in gradient will produce the same results in a tenth of the time. This assumption is implicit in most references on permeability tests on concrete and soil-cement consulted. The same assumption was implicitly made by Jackson (1965) in his research into soil-cement and by Price (1977) in his comments on the potential leaching hazard with dry lean concrete which were based on observations during permeability tests conducted at high gradient.

The observed negative loss for standard specimens at low gradient, referred to in section 9.4.11a indicates that the assumption of proportionality is probably erroneous with regard to soil-cement and other materials where hydration and other long-term reactions have a significant time dependent influence on material properties. The effect of extended curing discussed in the previous section supports this view.

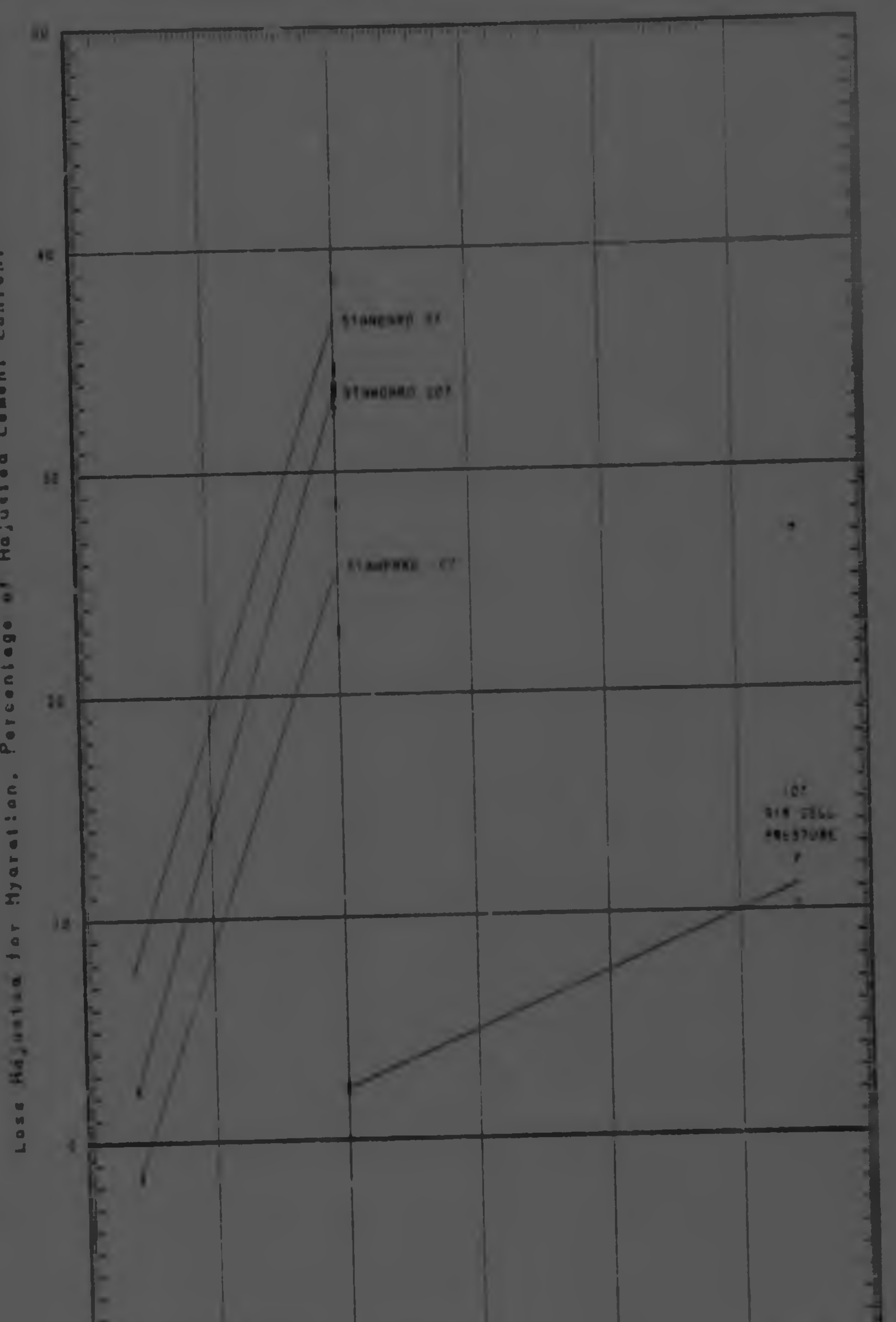
In order to examine the scale effect achieved in the test program, the ratios for the loss, flow and permeability data with change in gradient were calculated. These ratios were then compared with the appropriate gradient ratio. The loss ranged from 90 to 500% of the gradient ratio, the flow from 200 to 2400%, and the permeability from 30 to 1700%. In general, the increase in the measured value with an increase in gradient was substantially greater than the increase in gradient. Average increases were; loss 270%, flow 910% and permeability 540% of gradient ratio.

It is therefore concluded that the use of increased gradients to accelerate permeability and leaching tests will generally lead to a significant overestimate of the values that will be encountered in practice. While the results obtained using

GRAPH 9.30

ADJUSTED LOSS vs HYDRAULIC GRADIENT

SPECIMENS LEACHED FROM 28 DAYS IN LEACHING CELLS



Fo- individual codes see detailed legend.

TABLE 9.3

SUMMARY OF RESULTS FOR LEACHING AND PERMEABILITY TESTS IN LEACHING CELLS

Code	Test Conditions	Cement (%)	Age Start (d)	Lch Time (d)	Start m.c. (%)	Final m.c. (%)	Change of cntl (%)	Adjust Loss % adj cem	Rate ref Init mass ((%/d)/1000)	Rate ref Cntl mass ((%/d)/1000)	Av End Ca Rate ((%/d)/1000)	Linear Deter'n (mm)	Pent'n Inlet (mm)	Ratio Pen to Detr'n	Total Flow (l)	Av End Perm (nm/s)
	STANDARD CYLINDERS SUBJECT TO STANDARD CURE AND TESTED UNDER STANDARD CONDITIONS															
1	Standard:-	0.0	28	276	13.320	-0.538	-0.530	-	-	-	-	-	11.300	-	1.227	0.756
2	Cylinders (soil-cement)	5.0		262	13.309	-1.253	-1.861	34.66	-100.46	-132.30	24.66	26.00	5.775	0.222	9.165	7.190
3	Cure (7 day humid,				13.214	-1.469	-2.077	38.68	-1[illegible]7.77	-147.65	22.13	29.01	6.350	0.219	9.467	5.765
4	21 day sealed)	10.0		266	13.162	-1.121	-2.355	22.81	46.36	-85.75	5.87	17.11	4.900	0.286	7.667	3.716
5	Gradient (50 m/m)				13.111	-1.714	-2.948	28.55	-70.88	-107.34	9.85	21.41	4.420	0.206	10.476	5.643
6	Leach period (+/- 266d)	15.0			12.995	-3.119	-4.977	33.40	-89.90	-125.56	12.65	25.05	5.570	0.220	1[illegible].950	10.367
7	Leach water (de-aired	20.0		262	12.904	-3.772	-6.251	32.6[illegible]	-86.39	-124.62	10.84	24.49	4.670	0.191	20.954	10.776
8	distilled)				12.840	-3.996	-6.475	33.[illegible]2	-91.52	-129.09	N/A	25.37	3.980	0.157	21.418	11.046
8	Cell fluid (water)		31	270	12.926	-4.124	-6.609	34.51	-91.64	-127.81	12.33	25.88	5.250	0.203	21.632	11.994
V	Longterm (904d) leach	10.0	28	904	13.392	-2.537	-3.948	37.59	-30.87	-41.58	N/A	28.19	6.440	0.228	22.335	2.338
	NON-STANDARD CURING CONDITIONS AND/OR CURING PERIODS															
9	28d Humid cure	10.0	28	261	12.189	-1.232	-2.463	23.86	-51.92	-91.42	17.64	17.90	5.680	0.317	10.775	6.633
A	7d Humid, 21d leach bath			262	15.502	-0.016	-1.248	12.09	-0.66	-46.14	2.21	9.07	1.370	0.151	1.338	1.231
#	-- specific control				15.502	-0.016	-0.869	8.74	-0.66	-33.35	2.21	6.55	1.370	0.209	1.338	1.231
B	1 Series W/D, humid cntl				2.472	-0.798	-3.219	27.97	-33.52	-106.74	28.37	20.97	5.370	0.256	28.293	19.652
*	-- sealed control				2.472	-0.798	-2.678	23.27	-33.52	-88.80	28.37	17.45	5.370	0.308	28.293	19.652
C	7d Humid, 77d sealed cure		84	268	13.371	-0.153	-1.414	13.66	-6.27	-50.96	5.53	10.24	3.233	0.316	5.470	2.363
D	7d Humid, 343d sealed		350	587	13.045	0.224	-1.187	11.30	2.73	-19.25	N/A	8.47	1.990	0.235	5.157	1.048
S	-- adjust to 266d leach			266	13.085	0.184	-0.975	9.28	2.24	-15.81	N/A	6.96	N/A	N/A	3.317	2.180
	NON-STANDARD GRADIENT AND/OR CELL FLUID															
E	136 m/m, Air cell fluid	10.0	28	253	13.083	0.182	-1.045	10.13	7.90	-40.04	1.72	7.60	1.300	0.171	2.141	0.095
F				259	13.164	-0.018	-1.248	12.10	-0.78	-46.70	1.29	9.07	2.170	0.239	2.624	0.1[illegible]7
H		20.0		251	12.975	-2.718	-5.191	27.12	-64.98	-108.06	1.96	20.34	2.580	0.127	9.446	0.489
G	50 m/m, Air cell fluid	10.0		253	13.222	[illegible].987	-0.240	2.33	42.91	-9.20	N/A	1.75	0.000	0.000	0.449	0.072
I	10 m/m (low gradient)	5.[illegible]		278	13.360	[illegible].197	-0.419	7.80	14.84	-28.06	3.01	5.85	0.717	0.123	0.503	4.192
J		10.0		273	13.268	1.410	[illegible].172	-1.66	56.80	6.08	0.18	0.00	0.258	N/A	0.077	0.056
K		[illegible]0.0			13.026	2.042	-0.[illegible]43	2.32	44.87	-8.48	1.94	1.74	0.130	0.075	0.646	0.818
	NON-STANDARD LEACHING WATER															
L	Leached with water from	5.0	28	276	13.355	-0.010	-0.625	11.63	-0.77	-42.12	2.05	8.72	1.317	[illegible]	1.210	0.514
M	Rondebosch Dam Reservoir	10.0		268	12.918	0.911	-0.324	3.13	37.41	-11.69	0.21	2.35	[illegible]	[illegible]	0.780	0.075
N	Middelburg (not deaired)	20.[illegible]		264	12.922	2.044	[illegible]	2.28	46.46	-8.62	0.03	1.71	0.020	0.012	0.626	0.046
	Leached with water	5.0		276	13.312	-[illegible].484	-[illegible].099	20.44	-36.85	-7[illegible].08	9.32	15.33	3.580	0.233	2.755	2.068
P	saturated with CO2 at	10.0		268	13.128	0.684	-0.551	5.34	28.06	-19.93	5.18	4.01	1.079	0.269	1.598	1.461
Q	atmospheric pressure	20.0		265	12.936	1.781	-0.599	3.65	40.32	-13.78	2.04	2.74	0.760	0.277	0.884	0.448
	SPECIMENS WITH AXIAL CRACKS AND MICROCONCRETE SPECIMENS															
[illegible]	Cylinders split axially	10.0	29	246	13.125	[illegible]	-4.71[illegible]	50.7[illegible]	-165.16	-187.20	37.91	N/A	N/A	N/A	N/A	20.426
[illegible]		20.0	32		13.110	[illegible]	[illegible]	[illegible]	-54.50	[illegible]	[illegible]	N/A	N/A	N/A	N/A	18.381
T	Microconcrete c/w 1.0	18.2	28	261	N/A	[illegible]	N/A	N/A	N/A	N/A	0.32	N/A	0.000	N/A	0.430	0.227
[illegible]	c/w 1.2	22.0		262	N/A	N/A	N/A	N/A	N/A	N/A	0.00	N/A	0.000	N/A	0.054	0.003
	CYLINDERS COMPACTED AT OTHER THAN STANDARD PROCTOR OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY															
W	Compacted 1% dry of OMC	10.0	28	262	11.853	-3.878	-3.110	49.50	-162.81	-188.93	16.62	37.13	10.490	0.283	30.074	11.375
X	1% wet of OMC			261	13.700	-0.194	-1.375	13.32	-6.07	-51.04	3.59	9.99	2.730	0.273	4.212	2.320
Y	2% above MDD			264	12.394	-0.981	-2.214	21.45	-40.80	-81.25	8.92	16.09	4.430	0.275	8.854	4.739
	STANDARD DISK SPECIMENS															
Z	Disk specimens (same	0.0	28	259	13.223	-2.625	-2.642	-	-	-	-	-	N/A	N/A	2.688	0.687
a	test conditions as	5.0		[illegible]24	13.261	-5.[illegible]98	-5.684	106.3	-477.89	-474.55	61.66	21.26	N/A	N/A	N/A	12.550
b	standard cylinders)	10.0			13.[illegible]77	6.547	-7.757	75.30	-321.49	[illegible]	[illegible]	15.06	N/A	N/A	N/A	11.258
d		15.0		[illegible]31	13.129	-8.287	-10.13	68.04	-275.04	-294.55	50.82	13.61	7.980	0.586	N/A	12.707

TABLE 9.3

SUMMARY OF RESULTS FOR LEACHING AND PERMEABILITY TESTS IN LEA... NG CELLS

Ser No	Test Conditions	Cem cont (%)	Age Start (d)	Lch Time (d)	Start m.c. (%)	Final m.c. (%)	Change of Cntl (%)	Adjust Loss % adj cem	Rate ref Init mass	Rate ref Cntl mass ((%/d)/1000)	Av End Ca Rate	Linear Deter (mm)	Pent'n nlet (mm)	Ratio Pen to Detr n	Total Flow (l)	Av End Perm (mm/s)
	STANDARD CYLINDERS SUBJECT TO STANDARD CURE AND TESTED UNDER STANDARD CONDITIONS															
1	Standard:-	0.0	28	276	13.320	-0.538	-0.530	-	-	-	-	-	11.300	-	1.227	0.756
2	Cylinders (soil-cement)	5.0		262	13.308	-1.253	-1.861	34.66	-150.46	-132.30	24.66	26.00	5.775	0.222	9.165	7.190
3	Cure 7 day humid,				13.214	-1.669	-2.077	38.68	-117.7?	-147.65	22.13	29.01	6.350	0.219	9.467	5.765
4	21 day sealed)	10.0		266	13.162	-1.121	-2.355	22.81	-46.36	-85.75	5.87	17.11	4.900	0.286	7.667	3.716
5	Gradient (50 m/m)				13.111	-1.714	-2.848	28.55	-70.88	-107.34	9.85	21.41	4.420	0.206	15.476	5.643
6	Leach period (+/- 28d)	15.0			12.995	-3.119	-4.877	33.40	-89.90	-125.56	12.65	25.05	5.520	0.22	19.950	10.367
7	Leach water (de-aired	20.0		262	12.904	-3.772	-6.251	32.65	-86.39	-124.62	10.84	24.49	4.670	0.191	20.954	10.776
8	distilled)				12.840	-3.998	-6.475	33.82	-91.32	-129.09	N/A	25.37	3.980	0.157	21.418	11.046
8	Cell fluid (water)		31	279	12.928	-4.124	-6.609	34.51	-91.64	-127.81	12.23	25.88	5.250	0.203	21.652	11.994
X	Longterm (904d) leach	10.0	28	904	13.392	-2.537	-3.948	37.59	-30.87	-41.5?	N/A	28.19	6.440	0.228	22.335	2.338
	NON-STANDARD CURING CONDITIONS AND/OR CURING PERIODS															
9	28d humid cure	10.0	28	261	12.189	-1.232	-2.463	23.86	-51.92	-91.42	17.64	17.90	5.680	0.317	10.775	6.633
A	7d humid, 21d leach bath			262	15.502	-0.016	-1.248	12.09	-0.66	-46.14	2.21	9.07	1.370	0.151	1.338	1.231
*	-- specific control				15.502	-0.016	-0.869	8.74	-0.66	-33.35	2.21	6.55	1.370	0.209	1.338	1.731
B	(1 Series W.D. humid cell				2.472	-0.798	-3.219	27.97	-33.52	-106.74	28.37	20.97	5.370	0.256	28.293	19.652
*	-- sealed control				2.472	-0.798	-2.678	23.27	-33.52	-88.80	28.37	17.45	5.370	0.308	28.293	19.652
C	7d Humid, 77d sealed cure		84	268	13.371	-0.153	-1.414	13.66	-6.27	-50.96	5.53	10.24	3.233	0.316	5.470	2.363
D	7d Humid, 343d sealed		350	587	13.045	0.224	-1.187	11.30	2.73	-19.25	N/A	8.47	1.990	0.235	5.157	1.048
:	-- adjust to 266d leach			266	13.045	0.184	-0.975	9.28	2.24	-15.81	N/A	6.96	N/A	N/A	3.317	2.180
	NON-STANDARD GRADIENT AND/OR CELL FLUID															
[illegible]	[illegible] m/m, air cell fluid	10.0	28	133	13.083	0.182	-1.065	10.43	7.90	-40.04	1.72	7.60	1.300	0.171	2.141	0.095
[illegible]				279	13.464	-0.018	-1.148	12.10	-0.78	-46.70	1.29	9.07	2.170	0.239	2.624	0.123
[illegible]		20.0		231	12.975	-2.718	-5.196	27.11	-64.98	-108.06	1.96	20.34	2.580	0.127	9.646	0.489
[illegible]	[illegible] m/m, air cell fluid	17.0		218	13.312	0.887	-0.240	2.33	42.91	-9.70	N/A	1.75	0.000	0.000	0.449	0.071
[illegible]	[illegible] m/m (low gradient)	5.0		278	13.060	0.192	-0.419	7.80	14.86	-28.06	5.01	5.85	0.717	0.123	0.503	4.192
[illegible]		20.0		271	13.268	1.470	0.132	-1.88	56.80	-6.08	0.18	0.00	0.258	N/A	0.077	0.056
[illegible]		20.0			13.038	2.082	-0.443	2.32	44.87	-8.88	1.94	1.74	0.130	0.075	0.646	0.818
	NON-STANDARD LEACHING WATER															
[illegible]	Leached with water from	5.0	28	276	13.153	-0.010	-0.423	12.61	-0.77	-42.12	2.05	8.72	1.317	0.151	1.210	0.514
[illegible]	[illegible]	10.0		269	12.979	2.811	-0.724	5.13	37.41	-11.69	0.21	2.35	N/A	N/A	0.786	0.075
[illegible]	[illegible]	20.0		264	12.822	2.946	-0.436	2.28	46.46	-8.62	0.03	1.71	0.020	0.012	0.626	0.046
[illegible]	Leached with water	5.0		272	13.112	-0.484	-1.099	25.94	-36.85	-74.08	9.32	15.33	3.580	0.233	2.755	2.068
[illegible]	saturated with CO2 at	10.0		268	13.128	1.684	-0.531	5.74	28.06	-19.93	1.18	4.01	1.079	0.269	1.598	1.461
[illegible]	atmospheric pressure	20.0		263	12.958	1.780	-0.699	3.65	40.32	-13.78	2.06	2.74	0.760	0.277	0.884	0.448
	SPECIMENS WITH AXIAL CRACKS AND MICROCONCRETE SPECIMENS															
[illegible]	Cylinders split axially	15.0	29	266	13.321	-5.450	-6.378	50.78	-165.16	-197.70	37.91	N/A	N/A	N/A	N/A	20.426
[illegible]		20.0	32		13.116	-2.193	-6.353	50.08	-53.50	-99.11	14.42	N/A	N/A	N/A	N/A	18.383
[illegible]	Microconcrete (w 1:1)	14.3	29	252	N/A	N/A	N/A	N/A	N/A	N/A	0.32	N/A	0.000	N/A	0.430	0.227
[illegible]	(w 1:2)	22.0		262	N/A	N/A	N/A	N/A	N/A	N/A	0.00	N/A	0.000	N/A	0.054	0.003
	CYLINDERS COMPACTED AT OTHER THAN STANDARD PROCTOR OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY															
[illegible]	Compacted 1% dry of OMC	10.0	28	262	11.855	-1.474	-5.110	49.30	-162.81	-188.93	16.62	37.13	10.490	0.283	30.074	11.375
[illegible]	1% wet of OMC			261	13.700	-0.144	-1.375	13.32	-6.07	-51.04	3.58	9.99	2.730	0.273	4.212	2.320
[illegible]	1% above MDD			264	12.994	-0.981	-2.214	21.45	-40.89	-81.25	8.92	16.09	4.430	0.275	8.854	4.739
	STANDARD DISK SPECIMEN.															
Z	[illegible] specimens (same	0.0	28	259	13.223	-2.675	-2.642	-	-	-	-	-	N/A	N/A	2.688	0.687
[illegible]	test conditions as	5.0		324	13.263	-5.098	-3.884	106.3	-477.89	-474.45	61.66	21.26	N/A	N/A	N/A	12.550
[illegible]	standard cylinders)	10.0			13.077	-6.547	-7.757	75.30	-321.49	-336.17	66.44	15.06	N/A	N/A	N/A	11.250
[illegible]		15.0		231	13.129	-8.287	-10.13	68.04	-275.06	-294.55	50.82	13.61	7.980	0.386	N/A	12.707

LEGEND FOR GRAPHS 9.27/28/29/30

Code	Test Conditions	Cement (%)	Age Start (d)	Lch Time (d)	Graph (D.)
	STANDARD CYLINDERS, STANDARD CURE AND TEST				
1	Standard:-	0.0	28	276	1
2	Cylinders (soil-cement)	5.0		262	2
3	Cure (7 day humid,				3
4	21 day sealed)	10.0		266	4
5	Gradient (50 m/m)				5
6	Leach period (+/- 266d)	15.0		266	6
7	Leach water (de-aired	20.0		262	7
8	distilled)				8
8	Cell fluid (water)		31	279	8
	NON-STANDARD CURING CONDITIONS &/or PERIODS				
9	28d Humid cure	10.0	28	261	9
A	7d Humid, 21d leach bath			262	10
#	ditto -- specific control				
B	1 Series W/D, humid cntl				11
*	ditto -- sealed control				
C	7d Humid, 77d sealed cure		84	268	12
D	7d Humid, 343d sealed		350	587	13
$	- adjust to 266d leach			266	
	NON-STANDARD GRADIENT AND/OR CELL FLUID				
E	136 m/m, Air cell fluid	10.0	28	253	14
F	ditto			259	15
H	ditto	20.0		251	17
G	50 m/m, Air cell fluid	10.0		253	16
I	10 m/m (low gradient)	5.0		278	18
J		10.0		273	19
K		20.0			20
	NON-STANDARD LEACHING WATER				
L	Leached with water from	5.0	28	276	21
M	Rondebosch Dam Reservoir	10.0		268	22
N	Middelburg (not deaired)	20.0		264	23
O	Leached with water	5.0		276	24
P	saturated with CO2 at	10.0		268	25
Q	atmospheric pressure	20.0		265	26
	CRACKED AND MICROCONCRETE SPECIMENS				
R	Cylinders split axially	10.0	29	246	27
S		20.0	32		28
T	Microconcrete c/w 1.0	18.2	28	261	29
U	c/w 1.2	22.0		262	30
V	Longterm (904d) leach	10.0	28	904	31
	NON-STANDARD COMPACTION CONDITIONS				
W	Compacted 1% dry of OMC	10.0	28	262	32
X	1% wet of OMC			261	33
Y	2% above MDD			264	34
	STANDARD DISK SPECIMENS				
Z	Disk specimens (same	0.0	28	259	35
a	test conditions as	5.0		224	36
b	standard cylinders)	10.0			37
d		15.0		231	38

such techniques can be considered conservative, this limitation must be taken into account in interpreting them. If this is not done, material which is suitable for use in the application under consideration may be rejected or an unnecessarily conservative design adopted. This point is of particular importance when the effect of increased gradient is combined with the use of de-aired distilled water. The results of this research indicate that the use of natural water without de-airing leads to a further substantial reduction in permeability and loss.

A further factor which must be taken into account in comparing results is that permeability tests on concrete frequently use compressed air in direct contact with the percolating water to supply the driving head (Neville 1971, Price 1977 and others). Tests on soil-cement generally adopt the soils laboratory approach of air bladders in accumulators (PCA 1978b, Jackson 1965 and others). The effect of dissolved air discussed previously can be expected to have a significant effect on the results of tests conducted over several weeks if precautions are not taken to limit the diffusion of air into the specimen. This would also be expected to lead to results that would show soil-cement in a less favourable light if comparisons were made with values produced under different conditions.

9.4.12 Theoretical Depth of Deterioration

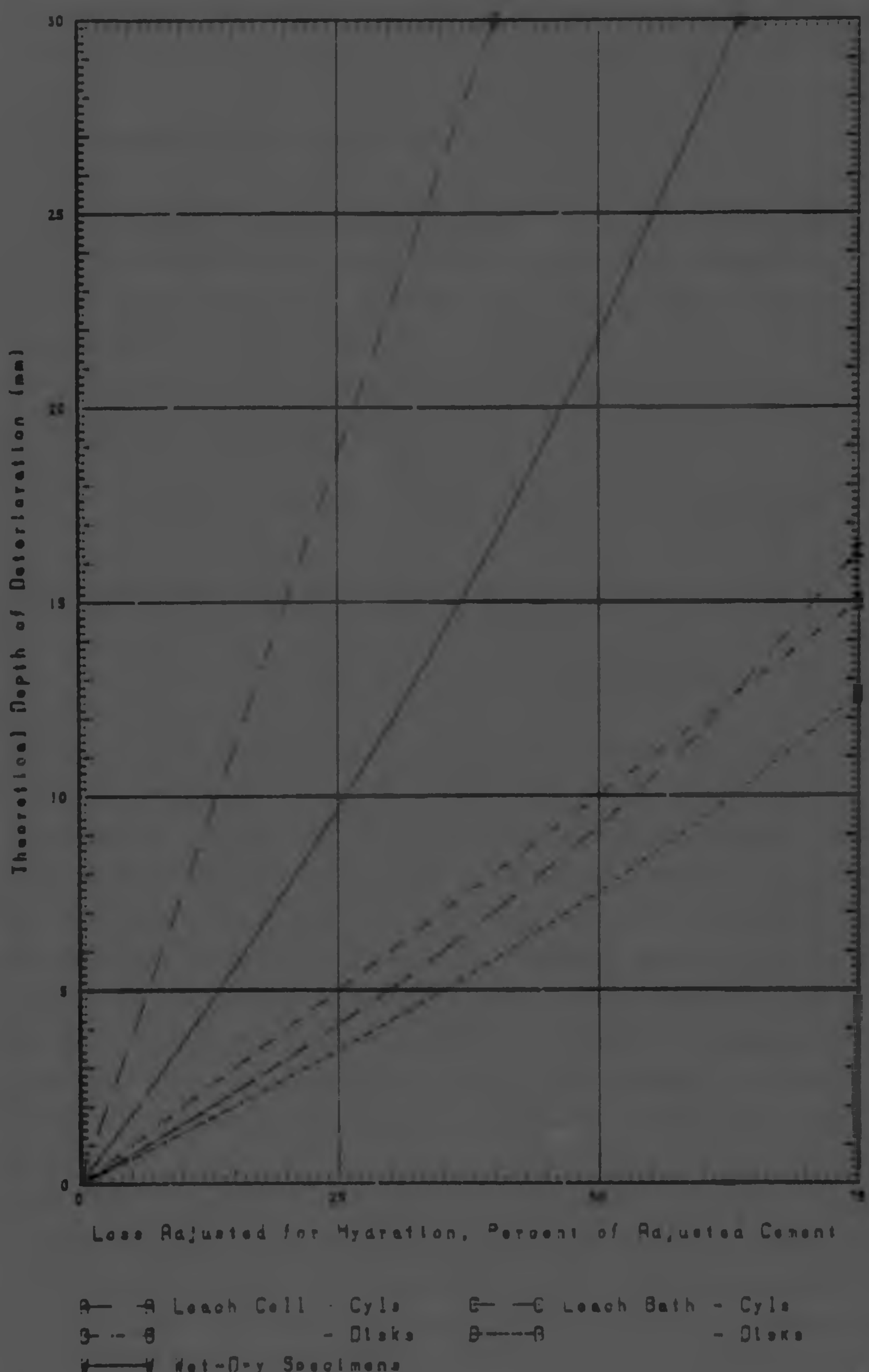
Consideration of the ratio of losses in the leaching bath in section 9.4.10 indicated that leaching occurred uniformly over the surface of the specimen. In section 9.4.11.a a model for leaching in the leaching cells was proposed. This model indicated that a zone of deterioration in which most of the cement had been leached out would develop at the inlet and advance into the specimen as leaching progressed. These proposals are consistent with the form of leaching attack that would be expected for the respective test conditions.

Consideration of the wet-dry tests indicates that an essentially uniform loss of material occurs with successive cycles and abrasion, decreasing with successive cycles. Leaching of a large proportion of the cement from the surface of the specimen will be required to reduce its abrasion resistance to a level at which it can be removed with the wire brush. This is supported by the results of the unconfined compression tests discussed in chapter 10. The loss distribution in the wetting and drying test can therefore be compared with that in the leaching bath. The loss of soil due to abrasion is accommodated by the calculation in terms of initial soil-cement mass adjusted for hydration as opposed to the loss in terms of initial cement content adjusted for hydration that is used for the leaching results.

If it is assumed that most of the cement is removed from the zone of deterioration, then the depth of this zone of deterioration should provide a measure that is independent of the shape and size of the test specimen and which will permit comparison between the different methods of test, and with the structure in the field. The theoretical depth of deterioration was therefore defined as the depth calculated on the basis of complete removal of cement in the deteriorated zone with no loss of cement in the remainder of the specimen.

The relationship between mass loss and theoretical depth of deterioration for the different specimen types and test conditions is plotted in graph 9.31. The relationships for disks and cylinders in the leaching bath are similar, reflecting the 1.13 surface area ratio referred to previously. The relationship for the disk specimens in the leaching cells lies in a similar range reflecting their large cross sectional area. The curve for the cylinders in the leaching cells is much steeper because of their relatively small cross sectional area and that for the wet-dry specimens lies halfway between those for the other specimens, reflecting the smaller area to volume ratio.

GRAPH 9.31
THEORETICAL DEPTH OF DETERIORATION vs LOSS
FOR WET-DRY TEST, LEACHING BATH AND LEACHING CELLS



DEPTHDET.PLT — 10/04/86

9.4.13 Penetration vs Depth of Deterioration

In order to determine the validity and significance of the penetration test results, the penetration values for each set of specimens were compared with the depth of deterioration computed from the measured loss according to type of test.

The results for the cylindrical specimens tested in the leaching bath and leaching cells are plotted on graph 9.32 together with the corresponding loss as a second vertical axis. While a reasonable amount of scatter is evident, a clear trend is apparent for both the leaching bath and cell results. The linear regression on the leaching cell data indicates:-

penetration = -0.12 + 0.22 x depth of deterioration

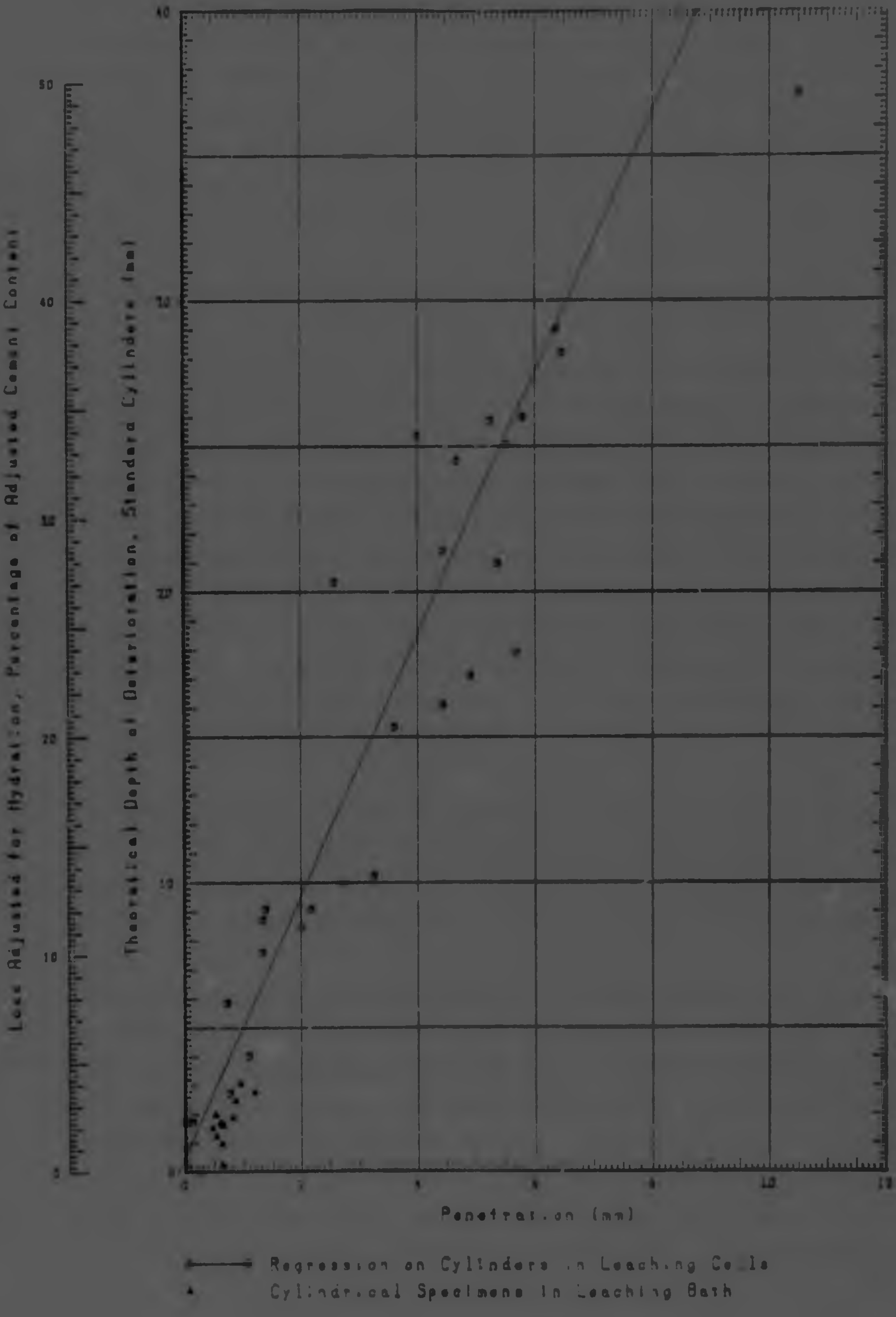
or, if the leach bath data is taken into account:-

penetration = 0.25 x depth of deterioration.

The point in the top right corner of graph 9.32 exhibits the greatest deviation from the linear trend and corresponds to the specimens compacted dry of OMC for which very high flows, losses and permeabilities were recorded. The penetration value of 10.5 mm for this point compares with the value of 11.3 mm for the unstabilized specimens from the leaching cells. The plotted linear regression yields an estimated depth of deterioration of 50 mm for the unstabilized specimens for the 11.3 mm penetration, compared with an effective 75 mm. The straight line can be expected to deviate to form an asymptote with this limiting penetration value. It should be noted that the dry of OMC point lies away from the direction of curvature of this asymptote indicating that a separate calibration may apply in cases where significantly higher flows are experienced. This is supported by the only data available for disk specimens (code d in table 9.3) which would also plot well below the regression line and for which high flows were also experienced.

GRAPH 9.32

RELATIONSHIP BETWEEN DEPTH OF DETERIORATION, LOSS AND PENETRATION -- FOR CYLINDRICAL SPECIMENS



The validity of the penetration test as a measure of depth of deterioration is therefore established. The results indicate that the true deterioration profile differs from the simple model proposed in section 9.4.12 but the validity of the theoretical depth of deterioration as a means of adjusting for different specimen and test geometries is supported by the results plotted on graph 9.32.

9.4.14 Comparison of Leaching Test Results

The results of the tests in the leaching cells and leaching bath together with those for the wetting and drying tests are combined on graph 9.33. The measured loss expressed as a percentage of cement content corrected for hydration, as used throughout this chapter, is used for the leaching results and the soil-cement loss as a percentage of original mass adjusted for hydration, as used in chapter 8, is used for the wet-dry test results. A reciprocal cement content axis, as used in graphs 8.3 and 9.25 is used to linearize the wet-dry and leach bath data and helps to linearize the leaching cell data, particularly in those cases where relatively small losses were recorded.

The same data is plotted on graph 9.34 in terms of depth of deterioration thus removing the geometrical effects as noted in section 9.4.12. The effect of this adjustment is particularly evident with regard to the disk specimens and leach bath results.

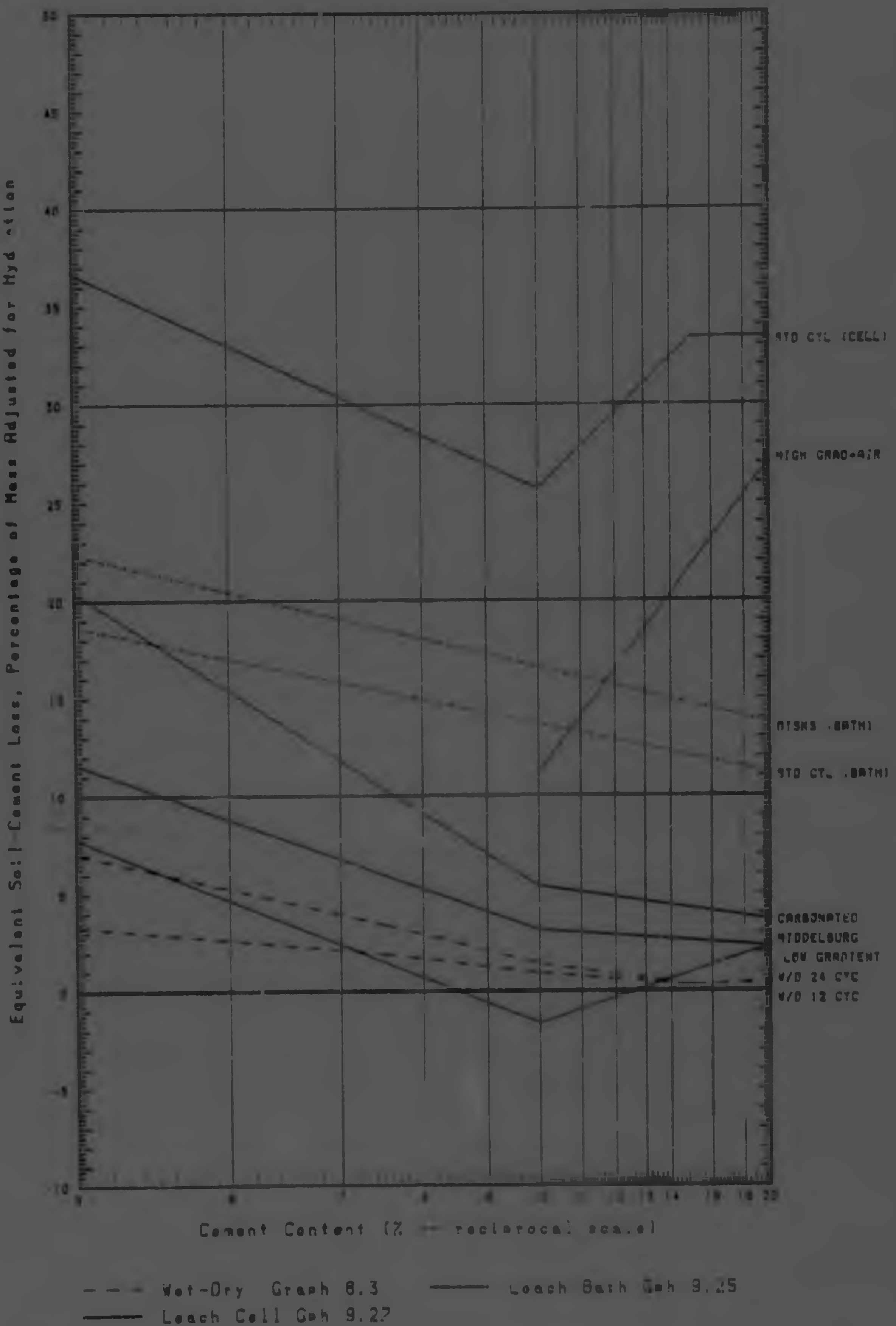
The general shape of the plot of the wet-dry test results, as shown in graph 8.3, is similar to that of the results for the low gradient, Middelburg and carbonated test conditions as well as the disk specimens. On the scale used in graph 9.34, the shape of the leach bath results is also similar.

It should be noted that, in plotting the data against cement content in this chapter, straight lines were used to connect data

GRAPH 9.33

SUMMARY OF RESULTS - LOSS vs CEMENT CONTENT

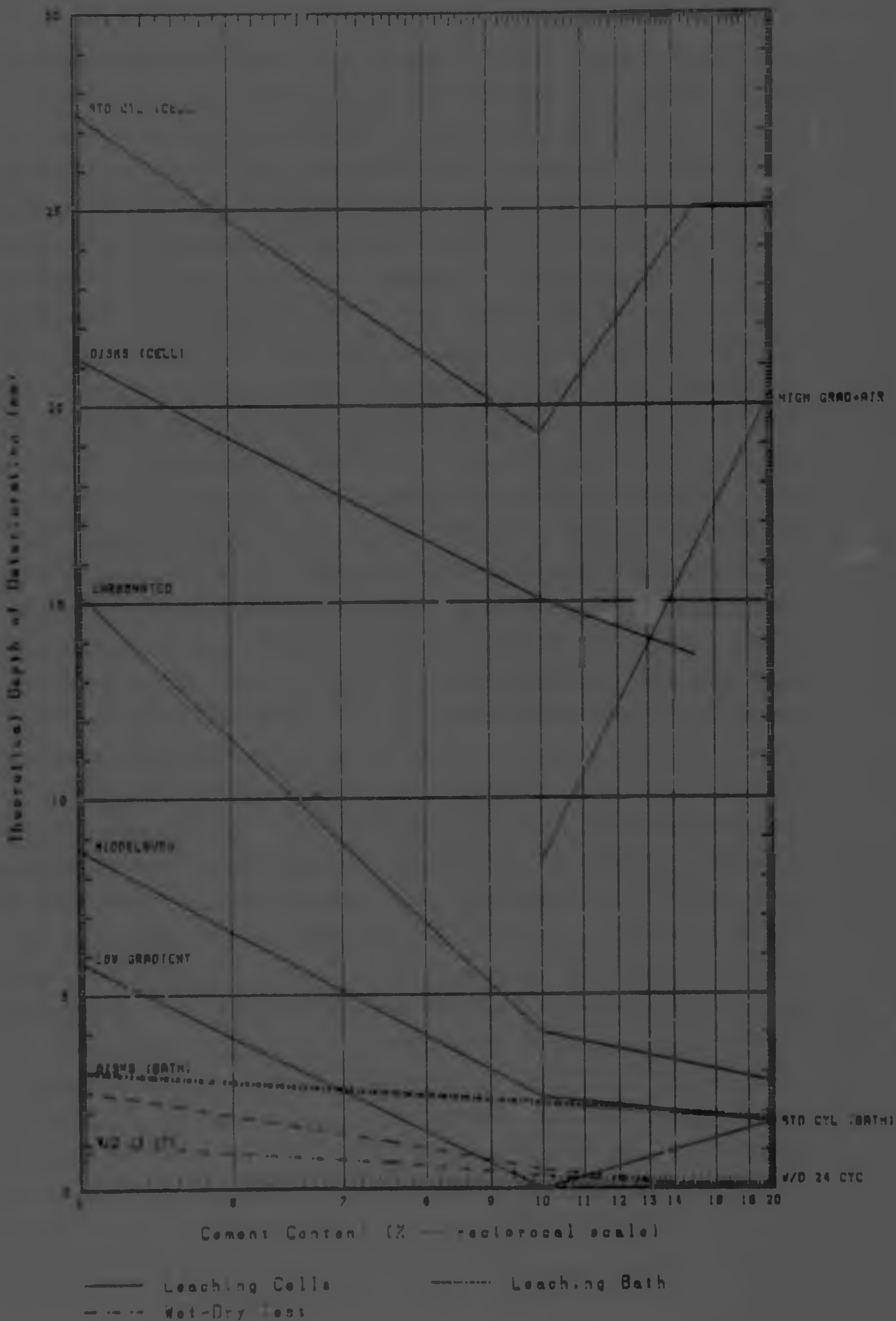
FOR WET-DRY TESTS, LEACHING BATH AND LEACHING CELLS



GRAPH 9.54

DEPTH OF DETERIORATION vs CEMENT CONTENT

FOR WET-DRY TESTS, LEACHING BATH AND LEACHING CELLS



DETN-CEM PLT 30/4/84-8=1338- 1 3 LCH-CONF ORT 43 points as the limited number of cement contents did not provide sufficient points to draw smooth curves with confidence.

The increased resistance of specimens subject to the wet-dry cure to leaching in the leaching bath, was referred to in section 9.4.10 and the observation that the resistance to leaching in the leaching cells of these specimens was comparable to that of the standard specimens was made in section 9.4.11.a. The increased compressive strength of spe imens subject to the wet-dry cure is discussed in the following chapter. It appears that the high temperature (72 ^{o}C) associated with the drying stage of the wet-dry test leads to an accelerated cure or fixing of hydration products which improves the leaching resistance and strength of the 37.5 x 75 mm cylindrical wet-dry specimens. It is likely that a similar improvement occurs in the standard ASTM wet dry specimens, at least on the surface, as noted in section 8.4.1. These factors appear to indicate that the standard wetting and drying test would be made considerably more severe if the temperature during the drying phase was reduced, thus reducing the accelerated curing effect. Alternatively, the test could be simplified by eliminating the drying stage completely and adopting a period of leaching in the leaching bath followed by drying and abrasion as suggested in section 8.4.4.

If the drying stage of the wet-dry test actually improves the durability of the material, as suggested above, then the deterioration observed in the test must result from leaching of cement from the surface of the specimen during the five hour soaking period. This was implied in section 9.4.9 where it was shown that the percentage loss at equivalent age lay between the estimated losses for specimens tested in the leaching cells and leaching bath, a similar relationship to that shown in graph 9.31. It is therefore concluded that the wetting and drying test is essentially a surface leaching test in which the curing of the material is accelerated by heating and the effect of leaching accelerated by abrasion.

Consideration of the reduction in loss indicated for the reduction in gradient from 50 to 10, indicates that, in the case of unit gradient in the leaching cells, it is likely that a negligible or negative loss will occur for all cement contents. If the effect of the use of natural water is taken into account, it is apparent that leaching of cement by percolating water will not be of significance in a soil-cement dam constructed using the material tested. Under these conditions, surface leaching can be expected to be of greater magnitude although a reduction in this effect when natural water is used, is also possible. If it is accepted that surface leaching is likely to be the predominant effect in practice, then the results of this research can be extended to other soil types using the leaching bath and the costly and complex apparatus and test procedures used in this program are not required for routine testing.

Taking account of the effect of extended curing and the addition of coarse aggregate, it is apparent that the loss registered in the wet-dry test is of similar magnitude to that which might be expected to occur in a twelve month period from an actual facing in a reservoir filled one year after placement. The principal method of leaching attack in massive water retaining structures has been shown to be surface leaching. The mode of deterioration in the wet-dry test has also been shown to be surface leaching. The wet-dry test is therefore regarded as a reasonable indicator of the losses that may occur in practice. The vast body of experience relating the in-service behaviour of soil-cement facings to the results of the wetting and drying test provides an adequate basis for extending the findings of this research to other soil types.

While the results of this research program have not provided any grounds to suggest that the cement content indicated on the basis of the PCA recommendations on Soil-Cement for Water Control (PCA 1976c) are inadequate, it is noted that, for the soil used, very low cement contents were indicated on the basis of the wet-dry tests and that higher contents were indicated by structural considerations. An optimum cement content of the order of 10% cement is indicated by the results for the standard leaching cell results for minimum permeability and leaching loss, however, there is no basis for applying this in design if it is accepted that surface leaching is predominant. This cement content might be adopted as a minimum in cases where higher gradients were envisaged, i.e. where the facing was to be used as the water barrier, backed by drains.

9.5 CONCLUSIONS

It is concluded that, for a homogeneous embankment subject to a low hydraulic gradient, retaining natural water and where the elapsed time between start of construction and first filling is likely to exceed twelve months, leaching by percolating water does not present a significant problem. Surface leaching and erosion is considered to be the most significant form of leaching attack. It is therefore concluded that the standard ASTM wetting and drying test provides a reasonable measure of the durability of soil-cement in massive water retaining structures and that the existing body of knowledge with regard to this test provides a basis for extending these results to other soil types.

The leaching bath is shown to provide a simple alternative indication of leaching resistance and is recommended where extension of the findings of this research is considered desirable. This is particularly appropriate where investigations into the effects of different compaction and curing conditions are required.

It is suggested that the standard wet-dry test is essentially a measure of surface leaching of soil-cement subject to an accelerated cure and can be made more severe by reducing the temperature during the drying stage. It is further suggested that the test could be simplified and accelerated by the use of a leaching bath without drying intervals followed by drying and abrasion on completion of leaching.

While the results of tests in the leaching cell indicate an optimum cement content of the order of 10%, there does not appear to be any specific basis for selection of cement content. The cement content determined on the basis of the PCA recommendations in chapter 8 is 4%, but it is felt that a cement content above the 6% [illegible] indicated for a decreasing loss level between series of 12 wetting and drying cycles is more appropriate. The 10% cement content referred to above is felt to be intuitively acceptable and agrees with the low end of the range of cement contents reported by the PCA (1976b) while the 15% "no loss" level referred to in chapter 8 could be adopted as a conservative upper limit for durability.

The carbonated water used did not contain as much CO_2 as would occur in a reservoir in which anaerobic decay of organic matter was occurring leading to a gas content in excess of the equilibrium level. Permeability of soil-cement to this water was significantly reduced by the presence of dissolved gases but a possible increase in final permeability is indicated. It is however considered that under normal circumstances the effect of excess CO_2 would be limited to accelerated surface leaching.

The permeability of soil-cement leached with the water from the Rondebosch Reservoir at Middelburg was also considerably reduced by the presence of dissolved air and the degree of leaching was further reduced by the presence of dissolved solids in the water. Magnesium dissolved in the Middelburg water was absorbed by stabilized specimens and magnesium leached from the unstabilized specimens was not present in the leachate from the stabilized specimens indicating a significant demand for magnesium by the hydrating cement.

The solubility of various calcium salts was found to vary considerably and this was related to the reduction in leaching rate with increased specimen age and accelerated curing in terms of the chemical changes associated with hydration. It was found that permeability and degree of leaching decreased significantly with increase in curing period. Accelerated curing by the application of heat in the wet-dry test and improved curing by the application of additional water had a similar effect. The limited calcium capacity of percolating water associated with the low permeability of the soil-cement and the size of the proposed embankment indicated that leaching by percolating water would not present a problem.

pH does not provide a reliable measure of calcium content in leachate but evaporative methods of determining total dissolved solids are felt to have potential, further investigation is required.

The use of increased hydraulic gradient to accelerate leaching tests leads to a considerable increase in leaching rate which is not in simple proportion to the increase in gradient. Care must therefore be exercised in interpreting the results of tests conducted at elevated gradients as both permeability and leaching loss may be considerably overestimated.

An increase in permeability of stabilized specimens over unstabilized from about 1×10^{-10} m/s to about 1×10^{-9} m/s was observed with scatter of about one order of magnitude for any test category. A progressive reduction in permeability with extended percolation was generally observed and rate of leaching decreased progressively towards an asymptote with increased period of leaching.

Permeability of the microconcrete specimens was considerably lower than that for most of the standard soil-cement specimens and decreased with increasing cement content. The mix design requirements for permeability within a range permitting comparison with soil-cement were, however, satisfied. It was observed that soil cement specimens wet cured form seven days exhibited considerably lower permeability than the standard soil-cement specimens. It is suggested that the lower microconcrete permeability may largely reflect the effect of wet curing from 24 hours.

Very low flows and losses were also associated with the specimens subject to a gradient of 10 m/m, a gain in mass being observed for the specimens containing 10% cement. Extrapolation of this trend supports the expectation that leaching by percolating water is not a major problem in a massive structure.

The use of air as the cell fluid for the high gradient leaching tests lead to a substantial reduction in measured permeability, confirming the observations made with regard to the carbonated and Middelburg water. These observations confirmed the need for the steps taken to exclude dissolved gases from the leaching water and cell fluid in the remaining tests.

The disk specimens subject to percolation exhibited high initial permeabilities but these decayed to similar levels to the equivalent cylindrical specimens, similar behaviour was observed for the specimens with longitudinal tension cracks. Comparison of the losses observed in the leaching bath for disk and cylindrical specimens indicated a ratio equal to the area ratio of the specimens, confirming that the observed loss was a surface effect.

Specimens compacted 1% dry of OMC exhibited much higher permeabilities and leaching losses than normal while those compacted 1% wet of OMC exhibited lower permeabilities. Compaction 2% above MDD had no significant effect on the leaching resistance and permeability of the material. It is recommended that compaction specifications should call for tighter tolerances on the upstream face of the embankment, possibly with a tendency to favour compaction wet of optimum. Tolerances towards the downstream face might be relaxed and a bias to compaction dry of optimum introduced in order to limit crack formation and provide a relatively more permeable backing to the upstream facing. Note, however, that the leaching bath results indicate very little difference in losses for the different compaction conditions when subject to surface leaching so that variations in compaction control are not as serious as they appear at first.

A reasonably close relationship between total flow through the specimen and total mass loss was found to exist, supporting the observations on limiting capacity of the leaching water. A tendency for the loss to approach an asymptote with increasing flow was also observed, suggesting that a minimum contact time was required for leaching to occur.

A model for leaching in the leaching cells was proposed whereby a zone of deterioration at the inlet end of the specimen advanced into the specimen as leaching progressed. This model was suggested by the observed deterioration of the specimens and suggested that the observed losses could be interpreted in terms of a theoretical depth of deterioration determined as the depth to which all cement would be removed if a deterioration gradient did not apply. The depth of deterioration was compared with the results of the penetration tests with good correlation for specimens in both the leaching cells and leaching bath, confirming its value as a means of comparing the different test results and the value of the penetration test as an indicator of extent of deterioration. A relationship of penetration equal to approximately 25% of depth of deterioration was indicated.

The depth of deterioration was used to compare the mass loss data for the leaching tests with the wet-dry tests, serving to eliminate the geometrical effects of the different specimen and test types. It is considered that the depth of deterioration is a valuable tool for comparing the results of leaching tests using different specimen geometries and test conditions. The depth of deterioration could provide a useful means of comparing the results of simplified wetting and drying tests with those of the standard technique.

CHAPTER 10

UNCONFINED COMPRESSION AND CYLINDER SPLITTING

10.1 INTRODUCTION

The objectives of the unconfined compression tests were:-

a. To provide data for comparison with the results of other research. Most work in the field of stabilized soils has been related to the unconfined compressive strength.

b. To examine the effects of various curing and testing conditions on the physical properties of the material. The unconfined compression test can be conducted in much less time than the triaxial test, permitting the examination of a larger number of variables in the available time.

c. To make use of the surplus specimens prepared as part of the leaching test program.

The objectives of the cylinder splitting tests were:-

a. To provide some measure of the tensile strength of the material.

b. To provide cracked specimens for use in the leaching test program.

c. To make use of the surplus specimens prepared as part of the leaching test program.

Apart from specimens prepared specifically for testing at ages of 28, 140 and 290 days at the standard cement contents, those specimens resulting from the wetting and drying test program and the microconcrete specimens, the remainder of this program was determined on an ad hoc basis to investigate specific conditions and to make use of specimens which were surplus to other parts of the program. A number of specimens prepared for preliminary triaxial tests to establish test techniques and not tested due to problems with the triaxial apparatus were tested in unconfined compression.

The cylinder splitting test was chosen for determining the tensile strength of the material after consideration of a number of references. Raad, Monismith and Mitchell (1977a) compared the various methods of test and concluded:-

> "The split tension test would seem best for use in practice for evaluation of tensile strength because of its simplicity. The true tensile strength can be estimated to be 1.11 times the split tensile strength. On this basis the error in the estimated value would not exceed +/- 13% according to the results of this analysis."

10.2 APPARATUS

Initial unconfined compression tests were conducted on a 4000 kg Losenhausen hydraulic testing machine which was found to be too soft, large amounts of strain energy being released on failure. As the final oven dry specimen mass was required for analysis of the leaching test results, the remaining tests were conducted on a 600 kN Tinius Olsen hydraulic testing machine which produced a controlled failure with no specimen disintegration.

Cylinder splitting tests were conducted on the Losenhausen testing machine. Consideration of the available literature indicated that there was no standard method of conducting split tensile tests on soil-cement. Neville (1973) indicates that packing strips for concrete generally have a width of 1/12th the specimen diameter and indicates that platens should permit rotation along the axis of the specimen while preventing rotation at right angles to the specimen. Bofinger (1970) quotes research which indicates that packing strips are not required for soil-cement and presents results for specimens tested with rubber packing strips and without packing. The specimens tested with rubber packing indicated higher tensile strengths.

As the use of rubber strips introduces a further unknown to the test procedure and since the inclusion of any form of loose packing introduces problems with regard to alignment, it was decided to conduct the tests without any form of packing. In order to achieve the required platen rotation, it was necessary to fabricate suitable platens for the test program. The platens were designed to comply with the width ratio suggested by Neville and were manufactured from steel tee section with the upstanding leg of the tee providing a 3 mm wide loading strip.

The loading apparatus is shown in plate 10.1. The wooden blocks on either side of the testing rig provide a slot approximately 2.5 mm wider than the specimen in which it can be optically centered parallel to its axis as well as providing a gauge for centering along the length of the specimen. The blocks also serve to prevent the specimen halves from rolling off the platen after failure.

To produce the cracked specimens for the leaching test program, the platens were replaced with knife edges as it was found that the standard test produced local crushing which it was not possible to seal in the leaching cells. The knife edge platens were machined from the same tee section used for the standard platens shaped to give a crest width of approximately 0.25 mm with an included angle of approximately 60°.

PLATE 10.1

APPARATUS FOR CYLINDER SPLITTING TESTS

10.3 TEST PROCEDURES

All specimens tested were the standard 37.5 mm diameter by 75 mm specimens used in the leaching program. Specimens were compacted as described in chapter 9 and the standard cure of 7 days in the humid chamber followed by sealing in foil and wax was adopted. Some specimens were subject to humid curing as recommended by the PCA (1976c), some were subject to wetting and drying cycles as described in chapter 8 and others were placed in the leaching bath, described in chapter 9, at ages of 1, 7 and 28 days. Only specimens subject to the standard cure and those placed in the leaching bath at age 7 days were used in the cylinder splitting tests.

Specimens were tested "as cured" and after soaking for four hours in de-aired distilled water. Testing without soaking is stipulated in BS 1924 (1975) for measurements of unconfined compressive strength while the PCA recommends soaking for periods of one or four hours according to specimen size (PCA 1976c). The soaked test was adopted as standard since the PCA specifications had been adopted in other parts of the test program. The longer soaking period was adopted as it allowed greater flexibility in scheduling. It was also considered that variations in soaking time would have less influence on the results and that the longer soaking period would, if anything, yield more conservative results. The same conditions were employed in the cylinder splitting tests. In addition, some specimens tested in unconfined compression were subject to prolonged soaking and others were subject to the vacuum saturation procedure used in preparing specimens for the triaxial tests and described in chapter 11.

As with the leaching test program, specimen masses and dimensions were noted at all changes in cure, before and after soaking, as applicable, and after oven drying on completion of the test.

10.3.1 Test Procedure for Unconfined Compression Test

Specimens were aligned on the center of the testing machine platen by means of a circular steel plate with concentric circle to suit the specimen diameter.

The PCA specifies testing at a loading rate of 20 psi/sec (+/- 140 kPa/sec). An approximately uniform rate of loading was achieved by continually adjusting the load control to maintain a constant rate of change of the digital read-out on the testing machine. Tests with extreme rates of loading indicated that over the range used in the test program, no significant variation in indicated strength occurred while at substantially greater rates indicated strength increased.

Specimens were loaded past failure, until a substantial reduction in load occurred, and then oven dried. Due to the stiffness of the machine, no significant surface disruption of the specimen occurred during the test.

10.3.2 Test Procedure for Cylinder Splitting Test

The specimens were aligned on the bottom platen of the loading apparatus by means of the blocks described above and the top platen brought into contact under manual control. Load was applied at a rate of approximately 15 kPa/sec giving a similar time to failure as that obtained for the unconfined compression specimens.

10.4 TEST RESULTS

Stresses were calculated on the basis of the nominal specimen dimensions of diameter 37.5 mm and length 75 mm. Owing to time constraints, length and diameter measurements were not processed as the repeatability and precision of the observations was not great and it was felt that the results were unlikely to be particularly significant.

Mass change data in the form of moisture content at time of test and final oven dry moisture content were only processed for those unconfined compression specimens where the strength results indicated a possible trend. It was felt that while some further useful information regarding the effects of variations in curing conditions might be derived from a full scale analysis of these results, the time required for this analysis could not be justified.

In general, the influence of variations in test and curing conditions for the unconfined compression and the cylinder splitting tests are similar. These results are presented in graphs 10.1 to 10.6, odd numbered graphs containing unconfined compression results and even numbered graphs containing the corresponding cylinder splitting results with a 10x vertical scale magnification. The results of the investigation into the relationship between unconfined compressive strength and moisture content at time of test and final (hydration) moisture content are presented in graphs 10.7 and 10.8.

In all cases, the data points on these graphs are the averages of sets, most of which contained six nominally identical specimens. A total of sixty sets were tested in compression, of which seven contained less than four specimens. Twenty sets were split, of which two contained less than five specimens.

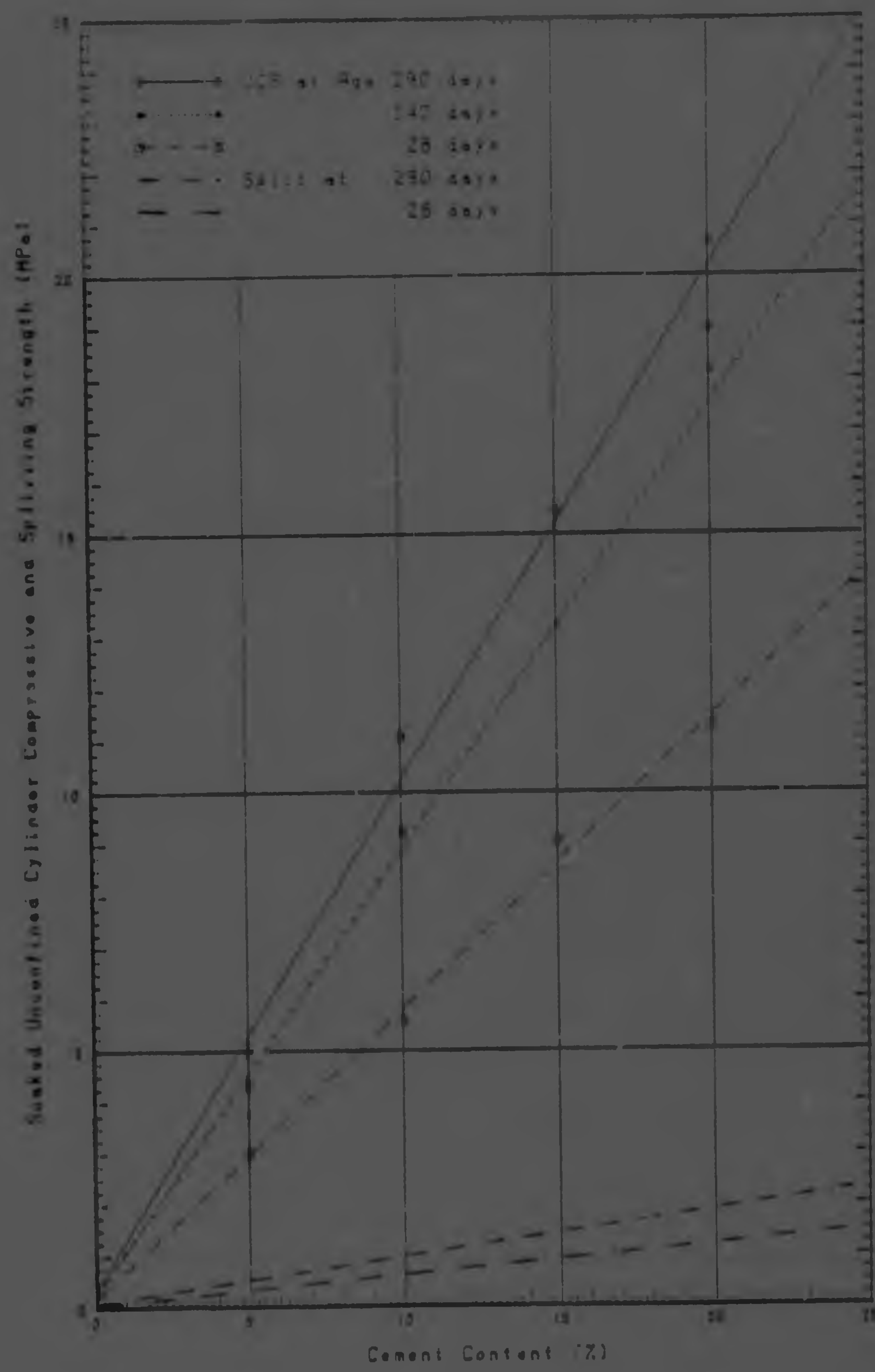
10.4.1 Variation with Cement Content and Age

Graph 10.1 shows the variation of compressive strength with cement content at ages of 28, 140 and 290 days for specimens subject to the standard cure and tested after soaking for four hours. Linear regressions performed on the data indicate regression coefficients generally better than 0.99 confirming the normal linear relationship. The slope of the regression line at an age of 28 days is 0.56 MPa/% (81 psi/%) which corresponds with the boundary between granular and fine grained soils suggested by Abboud (1969) and reported by Mitchell (1976d). The regression lines for the corresponding splitting strength at ages of 28 and 290 days are shown for comparison. Graph 10.2 shows the split tension test results for standard cure and four hour soak in greater detail.

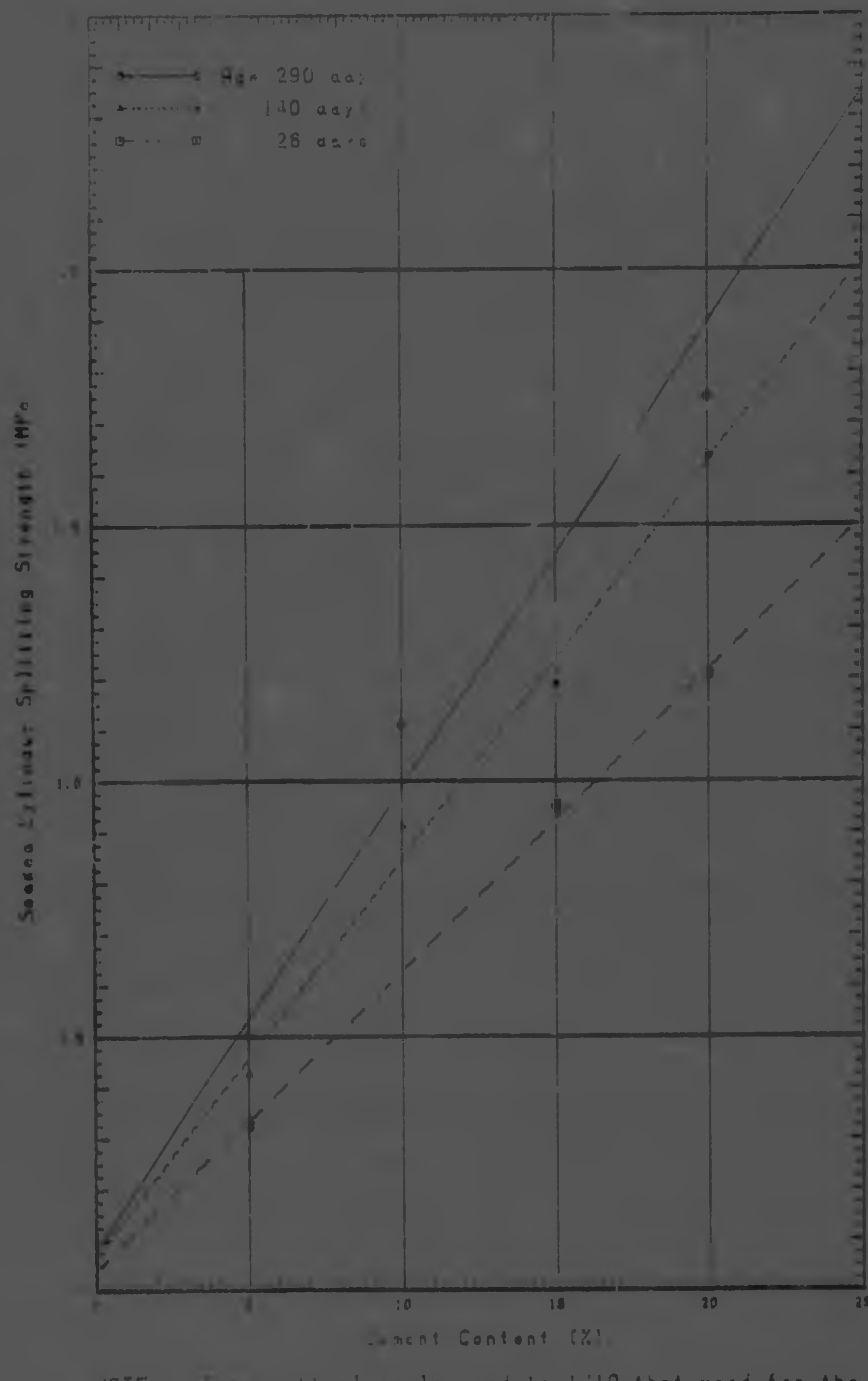
Graph 10.3 presents the variation of compressive strength with age for cement contents of 5, 10, 15, 20 and 25%. The family of curves for these cement contents was drawn by eye through the regression points determined for the ages reflected in graph 10.1. This data

GRAPH 10.1

SOAKED CYLINDER UNCONFINED COMPRESSIVE AND SPLITTING STRENGTH vs CEMENT CONTENT AT VARIOUS AGES

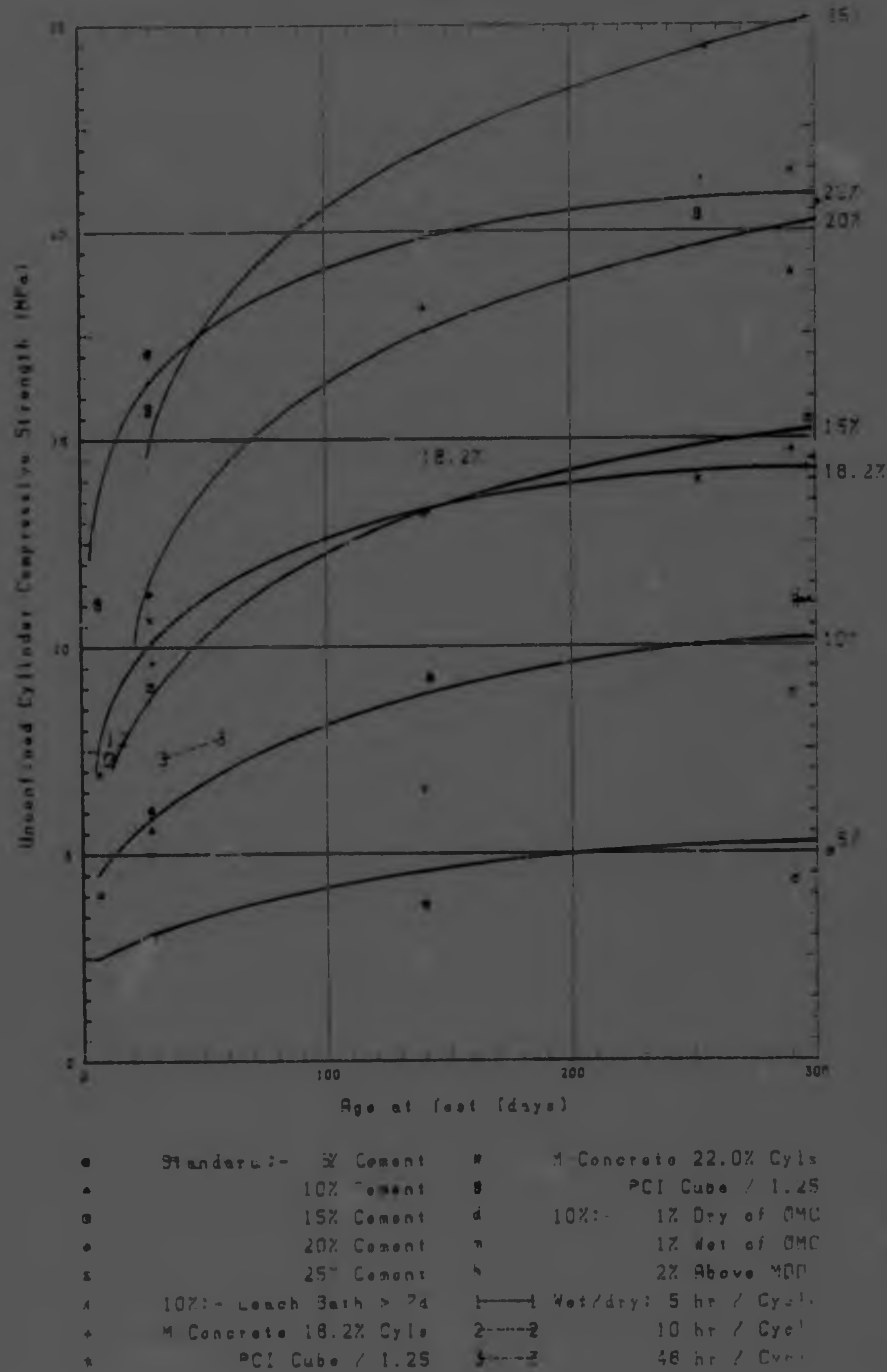


GRAPH 10.2
SOAKED CYLINDER SPLITTING STRENGTH
vs CEMENT CONTENT AT VARIOUS AGES



NOTE The vertical scale used is 1/10 that used for the results of the unconfined compression tests in graph 10.1.

GRAPH 10.3
UNCONFINED CYLINDER COMPRESSIVE STRENGTH
vs AGE FOR VARIOUS CEMENT CONTENTS AND CURES



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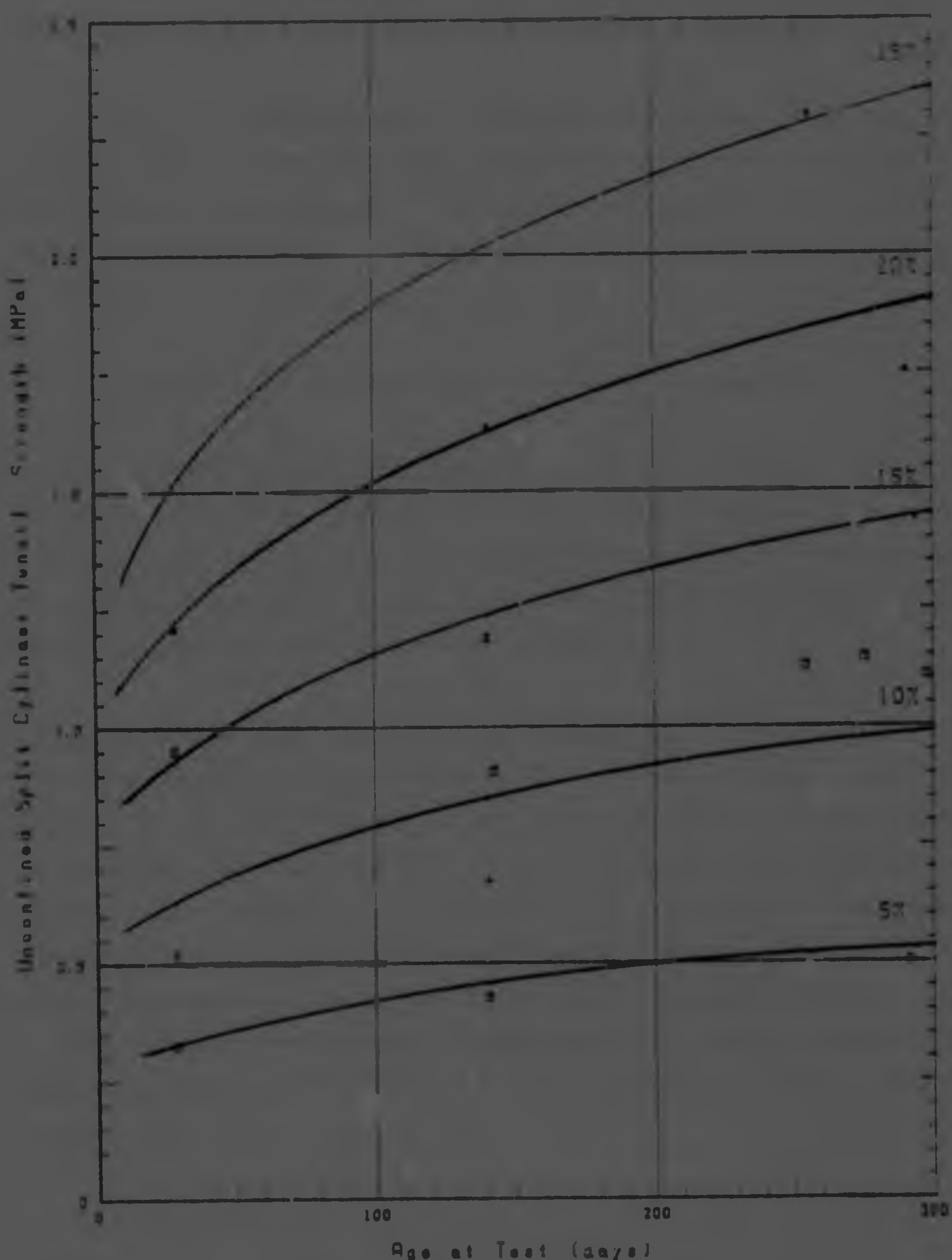
GRAPH 10.4

CYLINDER SPLIT TENSILE STRENGTH

vs AGE FOR VARIOUS CEMENT CONTENTS AND CURES



NOTE:- The vertical scale used is 1/10 that used for the results of the unconfined compression tests in graph 10.3.

	Standard:- 5% Cement		Standard:- 20% Cement
	10% Cement		25% Cement
	15% Cement	+	10%:- Leach Bath > 7d

TENS-AGE.PLT -- 13/01/88-8=1087-- 1 3

is plotted against log(age) in graph 10.9 where a family of straight lines is indicated, again confirming the results suggested by Abbot (1969) and reported by Mitchell (1976d). Graph 10.4 shows the corresponding results for splitting strength.

Graph 10.3 also shows the data points for specimens subject to various other curing and testing conditions as well as those for the microconcrete produced for the leaching program. The significance of these results is discussed in the following sections.

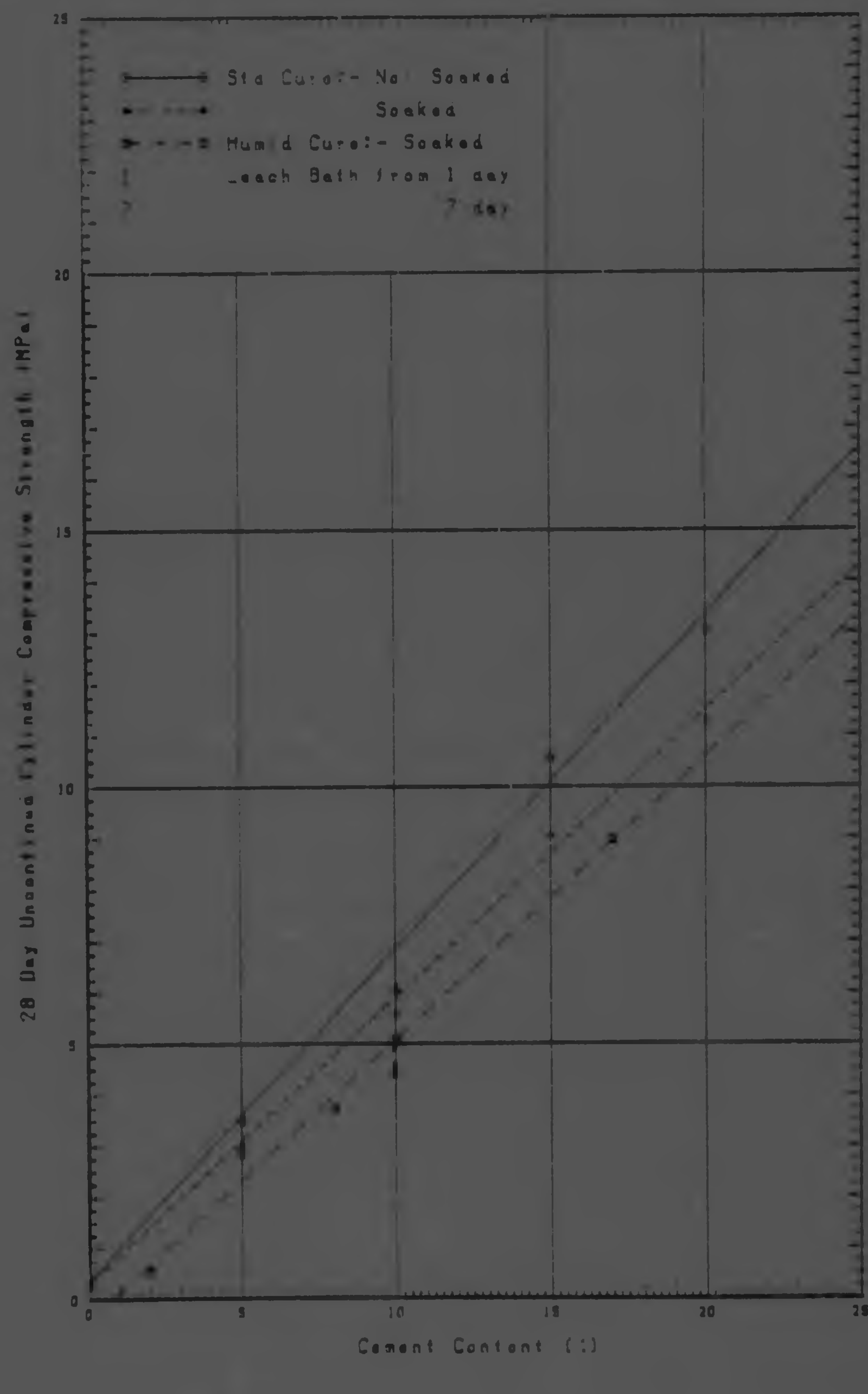
10.4.2 Effect of Variations in Test Conditions

Graphs 10.5 and 10.6 show the effect on 28 day strength of soaking the specimens before testing. As found in the preliminary tests, a significant reduction in unconfined compressive strength occurs in specimens soaked for four hours. The effect of soaking on the split tensile strength is not as pronounced.

It is notable that the parallel alignment of the soaked and unsoaked lines noted in section 6.7 is not evident here. The results for the preliminary test specimens soaked before testing are also plotted on graph 10.5, these specimens were subject to a humid cure. Further investigation is required to determine whether the parallel or divergent characteristics for soaked and unsoaked specimens apply, the fact that an unstabilized specimen soaked for four hours can be expected to collapse suggests that there is some justification for the parallel behaviour noted in section 6.7

It is noteworthy that the line for the specimens subject to the standard cure and tested soaked, is approximately coincident with that for the preliminary specimens which were not soaked before testing, this line, which is shown on graph 6.3 is not plotted on graph 10.5. The lower strength of specimens subject to 28 days humid, as opposed to 7 days humid and 21 days sealed cure, confirms

GRAPH 10.5
28 DAY CYLINDER UNCONFINED COMPRESSIVE STRENGTH
vs CEMENT CONTENT FOR VARIOUS TEST AND CURE CONDITIONS



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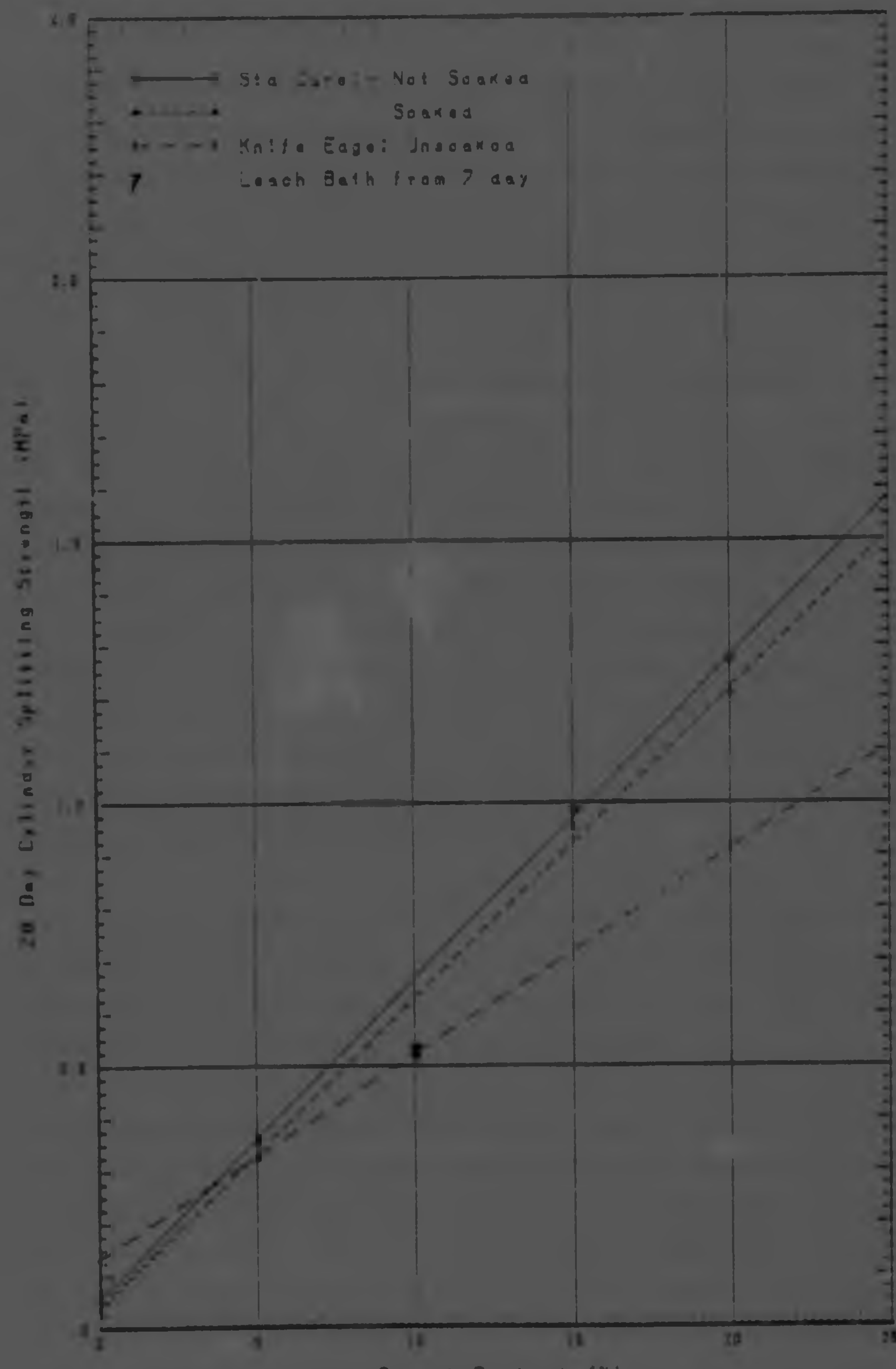
GRAPH 10.6

28 DAY CYLINDER SPLITTING STRENGTH

vs CEMENT CONTENT FOR VARIOUS TEST AND CURE CONDITIONS



NOTE:- The vertical scale used is 1/10 that used for the results of the unconfined compression tests in graph 10.5.

the observation that moisture loss occurred at the relative humidity of approximately 95% which was maintained in the curing chamber.

Graph 10.6 also indicates the split tensile strength of the 10% and 20% cement content specimens used in the leaching test program and split using the knife edges. A significant reduction in splitting strength is evident due to the very narrow platen width and consequent high stress concentrations.

10.4.3 Ratio of Tensile to Compressive Strength

The ratio of split tensile strength to unconfined compressive strength for comparable conditions was examined. In all cases, sets of six specimens compacted on the same day and subject to the same curing conditions were compared. The sets were thus nominally identical with variations between batches and dates of compaction effectively eliminated. For specimens subject to the standard cure and soaked for four hours before testing at ages ranging from 28 to 290 days and cement contents of 10, 15, 20 and 25%, the ratio varied between 0.094 and 0.099 with standard deviation of between 0.002 and 0.011.

For specimens subject to the standard cure and tested unsoaked at an age of 28 days and cement contents of 5, 15 and 20%, the average ratio was 0.098 with standard deviation of 0.004, effectively identical to the results for soaked specimens.

For the specimens containing 10% cement and cured in the leaching bath from seven days, the ratio was 0.104 with standard deviation of 0.001. A significance test performed on this data indicated that these results were not significantly different from the specimens subject to standard cure at the 95% level of confidence. It was further observed that the data for these specimens lay within the scatter zone of the data for the standard specimens.

For specimens subject to the standard cure, soaked before testing and containing 5% cement, the ratio was found to be 0.113 with a standard deviation of 0.003 as compared to the average of 0.097 with a standard deviation of 0.007 for the remaining specimens. These results were found to be significantly different at a level of confidence of 99.5%. Applying the regression values of compressive strength derived from graph 10.1 to plot the curves shown in graph 10.3, the ratio at 5% cement reduces to 0.098 with a standard deviation of 0.006 which agrees with the results for the remaining specimens.

Examination of graph 10.3 confirms that the relevant data points for compressive strength at 5% cement lie below the smooth curve indicated from graph 10.1. This is particularly evident with the specimens at ages of 140 and 290 days and it should be noted that the second set of specimens indicated at age 290 days does not show an appreciable deviation from the regression line. It is therefore considered that this result represents an extreme accumulation of experimental errors and that there is no significant difference in the ratio.

Split tensile strength for the test conditions described above is therefore slightly less than 10% of the unconfined cylinder compressive strength. On the basis of the findings of Raad, Monismith and Mitchell (1977a) quoted in section 10.1, the true tensile strength of the material is of the order of 10.8% of the unconfined compressive strength.

10.4.4 Influence of Moisture Content at Time of Test

In processing the results of the unconfined compression tests, it was observed that fairly considerable scatter of results for nominally identical specimens was evident. Considering the case of soaked and unsoaked specimens discussed in section 10.4.2, it was apparent that the moisture content of the specimen at the time of test had a bearing on the compressive strength. Compressive strength is plotted against moisture content at test in graph 10.7. Since hydration leads to an irreversible gain in mass as discussed in chapter 8, the moisture content was calculated in terms of the dry mass of the specimen determined immediately after testing by drying at 110°C.

Lines 0-1, E-F and G-H show the same data reflected in figure 10.5 for specimens not subject to soaking and soaked for four hours respectively, at cement contents of 5, 15 and 20%. A similar trend is evident, particularly at the higher cement contents. Line B-C reflects the effect of soaking the specimens subject to one series of twelve wetting and drying cycles.

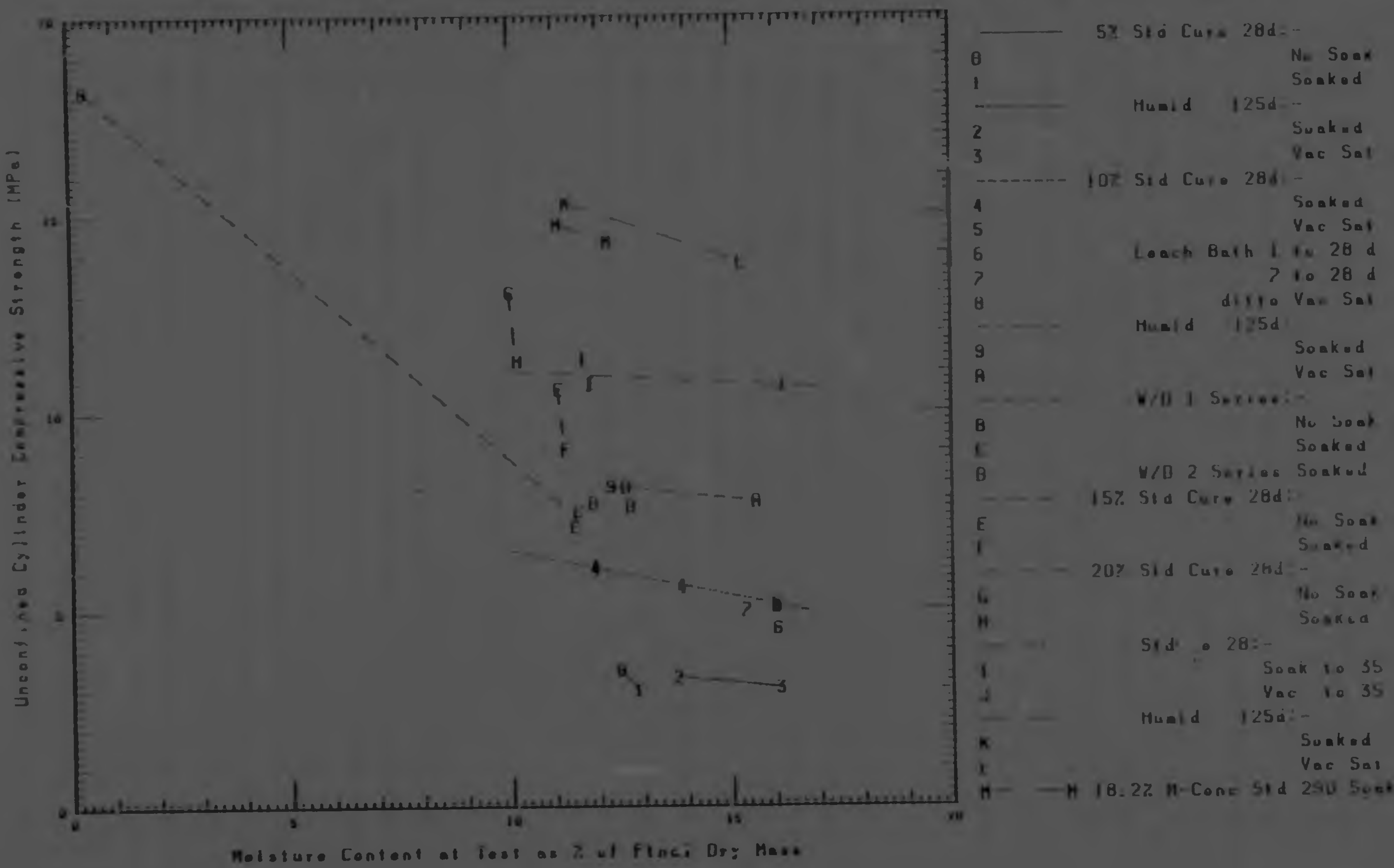
Lines 2-3, 4-4-5, 9-A, I-J and K-L show the effect on specimens of various cement contents, subject to a variety of curing conditions and tested at various ages, of saturating in a desiccator under vacuum, the procedure used for saturating control specimens for the triaxial tests. It will be noted that the slopes of these lines are generally similar and considerably flatter than those reflecting the effect of soaking.

On the basis of the 5.1% air voids referred to in section 6.2 and measured linear expansion of about 1%, the theoretical saturated final moisture content of the material is of the order of 16%. This agrees well with the points for vacuum saturated specimens plotted on graph 10.7 and indicates that this technique was effective in saturating test specimens. Moisture contents at test for the specimens cured in the leach bath were similar, those specimens subject to longer periods of leaching showing moisture contents approximately 0.5% higher. This is consistent with the reduction in final moisture content.

It is apparent that the initial period of soaking has a far more pronounced effect on the unconfined compressive strength than subsequent prolonged soaking or saturating under vacuum. It

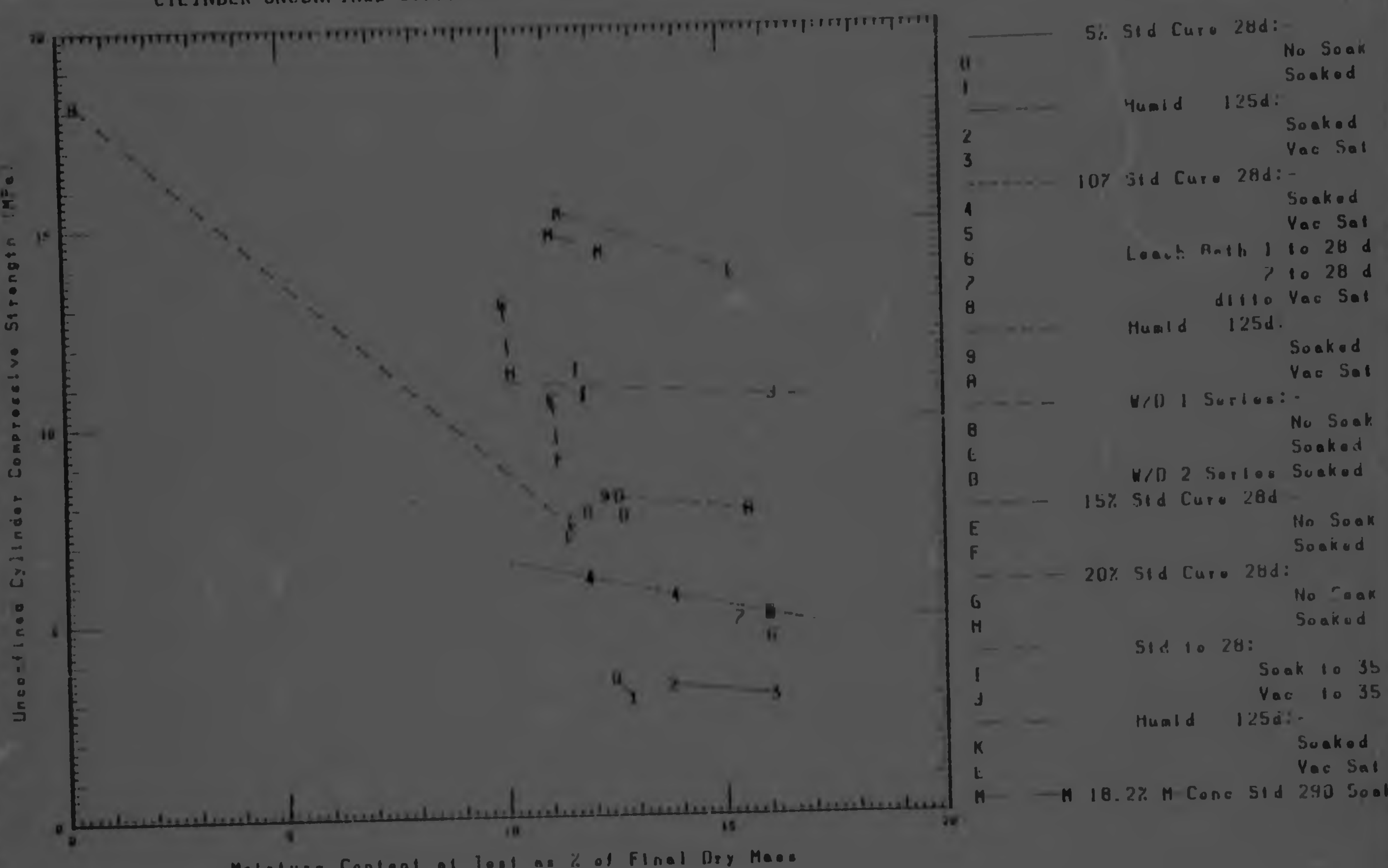
GRAPH 10.7

CYLINDER UNCONFINED COMPRESSIVE STRENGTH vs MOISTURE CONTENT AT TEST FOR VARIOUS CONDITIONS



GRAPH 10.7

CYLINDER UNCONFINED COMPRESSIVE STRENGTH vs MOISTURE CONTENT AT TEST FOR VARIOUS CONDITIONS



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would appear that most of the reduction in strength due to soaking occurs in the four hour period adopted. It should be noted that the points I are equivalent to the point H but with a soaking period of seven days as opposed to four hours and that an increase in strength of the material resulting from the additional cure might be expected. In view of the scatter of the points I, and the observations in section 10.4.6 below, it is likely that the line G-H-I-J gives an indication of the probable relationship that applies over a full range of moisture contents.

The significance of the remaining points reflected on this graph is discussed in the sections which follow.

It should be noted that, while straight lines have been used to indicate the trends in graph 10.7, it is possible that a more detailed investigation would reveal a hyperbolic or logarithmic relationship between strength and moisture content at time of test. Such an investigation would need to eliminate the variability between batches and date of compaction and the effects of variations in curing conditions in order to establish an accurate picture of the overall significance of this effect.

10.4.5 Influence of Final Moisture Content

Point 6 on graph 10.7 reflects specimens cured in the leaching bath from age one day and tested at 28 days and indicates a probable loss in strength when compared with points 5, 7 and 8 immediately above it (points and 8 are coincident). Results for specimens placed in the leaching bath and tested at ages of 140 and 290 days showed substantially greater reductions in strength. While comparative vacuum saturated results were not available, the trend from soaked to leach bath cured results was generally steeper than that from soaked to vacuum saturated results although moisture content at time of test was similar. As a reduction in hydration gain for specimens cured in the leaching bath had been observed, the unconfined compression results for selected specimens were plotted against final oven dry (hydration) moisture content. These results are reflected in graph 10.8.

The alphabetic and numeric codes used in graph 10.7 are employed for the same ·s in graph 10.8 for ease of comparison. In the case of the soaked and unsoaked specimens, the symbol for the unsoaked specimen has been used for both specimens for convenience. Non-alphanumeric symbols are used for specimens not reflected in graph 10.7.

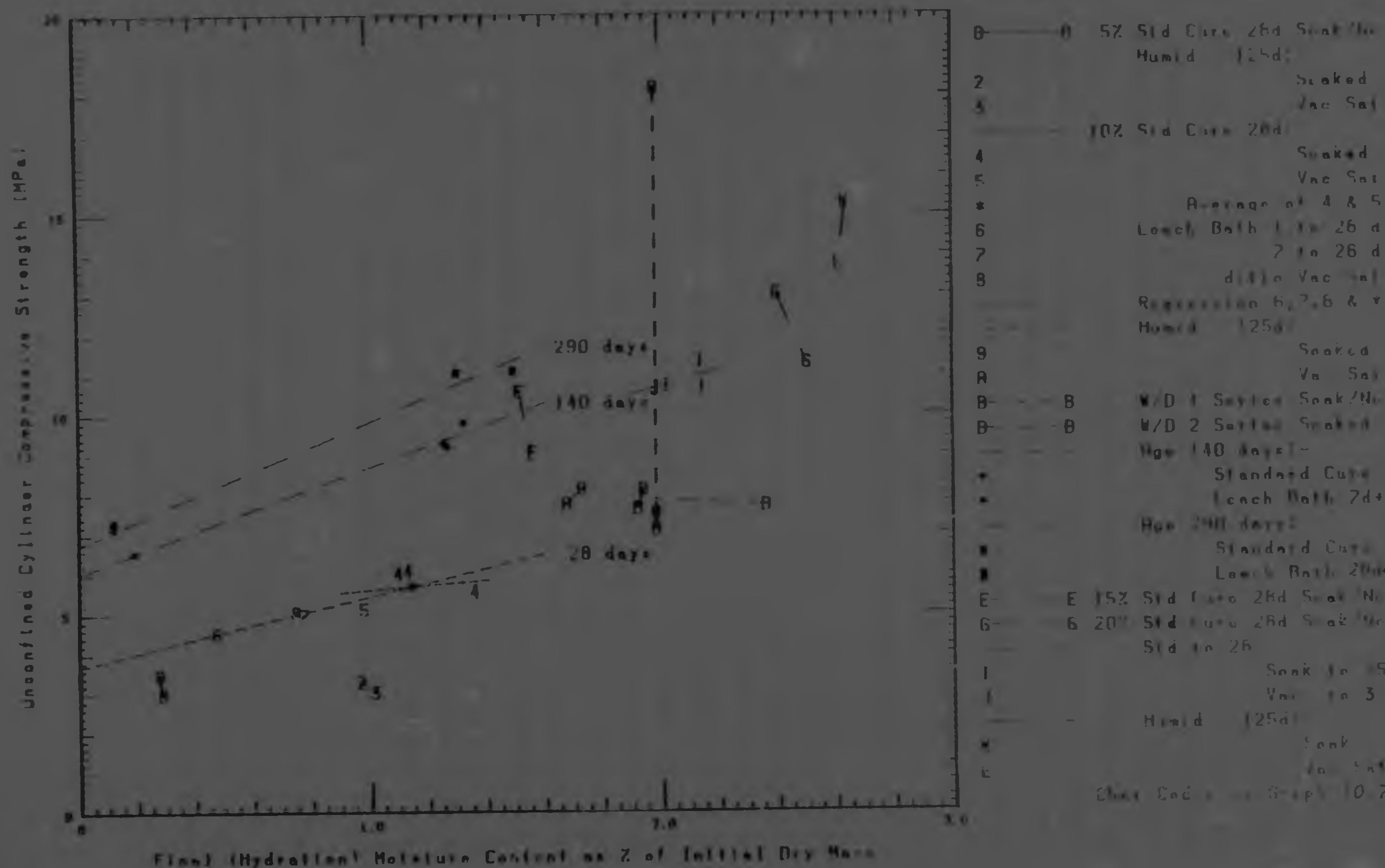
The trend at ages of 28, 140 and 290 days is immediately apparent, particularly at 28 days where specimens placed in the leach bath at different ages are reflected.

The effect of soaking on specimens at cement contents of 5, 15 and 20% subject to standard cure as reflected by lines 0-0, E-E and G-G is not pronounced, a slight increase in hydration being indicated but no distinguishable effect on compressive strength. This supports the observation of a small but measurable increase in mass with every soaking cycle in the wet-dry tests, referred to in chapter 8. However, effectively no mass gain occurs for the specimens subject to one series of twelve wet-dry cycles (line B-B). It should be noted that these specimens were held over in the oven for some time after completion of the wet-dry tests, and the observation during the second series wet-dry tests was that very little mass gain occurred during the first few cycles after prolonged storage in the oven at 72^{o}C but that thereafter mass gains returned to normal.

There is no consistent trend for specimens subject to vacuum saturation and indicated by lines 2-3, 4-5, 7-8, 9-A, I-J and K-L. There is no basis on which a trend might be expected and the completely random orientation of these lines confirms this. This is in marked contrast with the corresponding lines on graph 10.7.

GRAPH 10.8

CYLINDER UNCONFINED COMPRESSIVE STRENGTH vs FINAL OVEN DRY MOISTURE CONTENT FOR VARIOUS CONDITIONS



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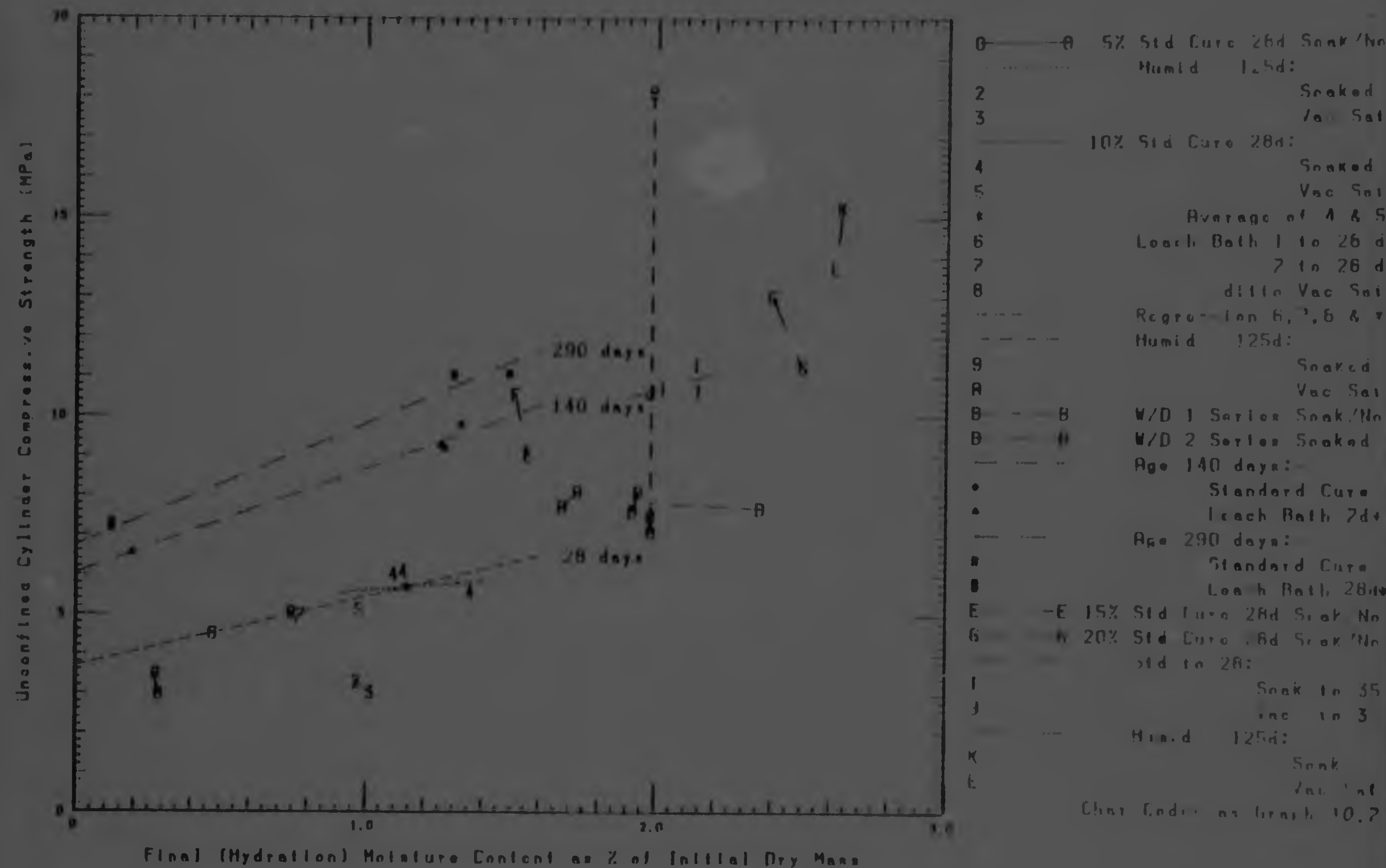
GRAPH 10.8

CYLINDER UNCONFINED COMPRESSIVE STRENGTH vs FINAL OVEN DRY MOISTURE CONTENT FOR VARIOUS CONDITIONS



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With regard to the line G-H-I-J in graph 10.7, referred to in section 10.4.4 above, it is apparent that a loss of strength of the specimens I and J due to leaching probably occurred as the specimens were soaked in a relatively large volume of de-aired distilled water, this is confirmed in graph 10.8 where a significant reduction in final moisture content is evident. It is likely that this compensated for the expected increase in strength resulting from the extra seven day cure. This supports the trend, referred to in section 10.4.4, which could be hyperbolic in form.

10.4.6 Variations in Strength for Identical Sets

The sets indicated by the points I on graphs 10.7 and 10.8, were prepared together, subject to exactly the same curing conditions and tested consecutively. Two of these sets had effectively identical moisture contents at test, plotting together on graph 10.7 while the final moisture content of one of these sets was very similar to that of the third set as seen in graph 10.8. Unconfined compressive strength for these sets was 11.31, 10.71 and 10.68 MPa with standard deviations of 0.43, 0.86 and 0.82 MPa. Significance tests indicated that the first set was significantly different from the other two at the 90% level of confidence but not at the 95% level. It was therefore concluded that difference at a level of significance of at least 95% was required when comparing any other unconfined compression results.

The spread of these results gives an indication of the potential range of variation in the other points plotted on these graphs. It should be noted that a detailed analysis of the mass history of the specimens could well indicate differences in curing conditions which would account for this variation.

10.4.7 Influence of Ageing of Cement

As mentioned in section 7.5, tests on the cement on completion of the test program indicated that no significant deterioration had occurred. At the same time that the tests discussed in section 7.5 were being conducted, two sets of specimens containing 10% cement were compacted and tested in soaked unconfined compression at 28 days.

The data for these two sets is represented by the two closely grouped points 4 on graphs 10.7 and 10.8. Compressive strength for these sets was 6.02 and 6.10 MPa respectively. The third point 4 shown on the graphs was for a set compacted 108 days after the start of the test program, 371 days before the final specimens were prepared, and also tested at an age of 28 days. Strength for this set was 5.57 MPa and it was found that this was significantly weaker than the two later sets at the 97.5% level of confidence thus suggesting an improvement in the stabilizing properties of the cement.

However, consideration of graph 10.7 reveals a very close relationship between these three points and the moisture content at test. This relationship is supported by the further point 5 for a corresponding set subject to vacuum saturation. Linear regression on these four points yields a regression coefficient of 0.99. No significant correlation with final moisture content exists (r^2=0.04).

These results therefore confirm that no significant change in the properties of the cement as a stabilizing agent occurred during the test program.

10.4.8 Variation in Strength with Compaction Delay

It was observed in section 6.7 that the results for the preliminary unconfined compression tests indicated an increase in strength with delay between mixing of the material and compaction of the specimen. This phenomenon was not observed in the main test program and it is considered that the behaviour observed in the preliminary tests was probably due to the relatively long time taken in compacting the first sets before the procedure was finalised.

10.4.9 Influence of Rate of Loading

As mentioned in section 10.3.1, the required constant rate of load application for the compression tests was achieved by manual adjustment of the load control valve on the testing machine. The final valve setting was generally between 5 and 5.3. To examine the effect of rate of loading on indicated compressive strength, three nominally identical sets of six specimens were tested at age 140 days after standard cure and soaking. These sets were tested by setting a scale value on the control valve and allowing the specimen to fail at the resulting rate.

Settings used were 4, 5 and 8 with resulting average loading rates of 41, 66 and 245 kPa/sec and compressive strengths of 9.19, 9.28 and 9.79 MPa respectively. In all cases, the load increased rapidly initially with the rate becoming progressively slower with increasing load. Setting 3 was found to be insufficient to fail the specimens.

A plot of compressive strength vs load setting yields a straight line with regression coefficient 0.99. The data is represented by the three diamond shaped points on graph 10.8. Range in moisture content at time of test was 0.21 percentage points and showed no correlation with the observed variation in strength. On the basis of the observed rate of loading under manual control, and the limited range of final settings, it is concluded that, while variation in rate of loading produces a measurable variation in compressive strength, the rate was sufficiently controlled not to have an appreciable effect on the reported results.

10.4.10 Effect of Wetting and Drying Cycles

The results for the specimens subject to wetting and drying cycles have been referred to above insofar as the effect of soaking is concerned. It is notable that the compressive strength after twelve cycles of wetting and drying, which corresponds to a nominal age of 33 days, is substantially greater than that for specimens subject to the standard cure and tested at an age of 28 days. In addition, the final dry moisture content is substantially greater than that for specimens with 10 and 15% cement subject to standard cure at ages up to 290 days. As mentioned in section 9.4.11.a, it appears that heating of the specimens at 72°C has the effect of accelerating the cure while there is moisture available. There is no significant difference between the set subject to drying at 110°C and that held at 72°.

It should be noted that the specimens were held over in the oven at 72°C for a considerable period before testing after both the first and second wet-dry series due to delays in other parts of the program. In view of the fact that no mass gain occurred during this period, it is considered unlikely that any strength gain occurred. It should also be noted that all of the second series sets were heated to temperatures in excess of 180°C due to thermostat failure.

The strength of the second series wet-dry specimens, represented by the points D on graph 10.8 which correspond to the sets presented in table 8.2, appears to be independent of final moisture content suggesting that this factor is of little significance in specimens subject to wetting and drying cycles. The first series results confirm this.

An increase in strength is apparent for the specimens subject to two series of wetting and drying cycles relative to those subject to one series. The point D to the left in graph 10.7 and to the right in graph 10.8 represents the set removed from the oven after drying on completion of the second series for compression tests.

The remaining two points represent the two sets that were subject to one further wetting cycle and then subject to overheating. A significant loss of hydration moisture and increase in permeability therefore resulted from the overheating. No loss of strength is evident however. The gain in final mass referred to in chapter 8 is confirmed for specimens not subject to overheating.

As mentioned above, it is assumed that no strength gain occurred in specimens stored at 72° in a drying oven with no available moisture, it is however accepted that further research could disprove this assumption. Assuming this to be valid and since a gain in mass was observed after most cycles of wetting and drying, an attempt was made to assess the equivalent age of the specimens. Since a period of soaking of five hours is used in the wet-dry test, it was considered that this was probably the shortest period over which hydration might occur, the results depicted on the basis of an increase in age of five hours per cycle is shown by the line 1-1 on graph 10.3. Line 2-2 on the same graph assumes that hydration occurs for ten hours per cycle, i.e. from the time the specimen is placed in the water to five hours after removal, this assumes that the specimen is effectively dry after five hours in the oven. Line 3-3 reflects the relationship if the nominal ages, including drying at $110^{\circ}C$, at the end of 12 and 24 cycles of 33 and 49 days, are applied.

Any of these assumptions indicates an increase in strength for specimens subject to wetting and drying cycles when compared with standard specimens of equivalent age. This is consistent with the accelerated curing achieved with concrete by the application of heat. This is confirmed by Clare and Pollard (1954) and Metcalf (1964) who report that curing at high temperatures increases the strength gain of soil-cement and soil-lime mixtures.

In many areas, ground surface temperatures may exceed $43^{\circ}C$ (Blight 1962) and it is likely that under such conditions, strength gains for slope protection and other exposed soil-cement surfaces will be substantially greater than those indicated by laboratory tests.

These results suggest the possibility of accelerating strength gain of soil-cement in critical areas, such as those that might be subject to erosion in the event of overtopping by floods during construction, by using sheets of black plastic to provide a moisture seal and elevated temperatures for curing during the first few days after placing. A similar technique might be employed in zones such as spillway linings, where high quality soil-cement is required. This technique would have the added advantage of increasing the resistance of the material to leaching, as discussed in chapter 9.

10.4.11 Effect of Non-Standard Compaction Conditions

One set each of the non-standard specimens prepared for the leaching program were tested in unconfined compression. These were compacted at 1% above and below proctor optimum moisture content and at a density 2% above proctor maximum dry density. All were compacted at a cement content of 10%, subject to the standard cure and soaked before testing at age 290. These points are plotted on graph 10.3 and are designated by the symbols m, d, and h for moist, dry and high density respectively.

The strength of the sets compacted dry of optimum and at the higher density do not differ from the strength for the corresponding standard specimens, while the set compacted wet of optimum was substantially weaker. Compaction dry of optimum is thus preferable to compaction on the wet side. This supports the recommendations of Fetz (1973) with regard to crack control, referred to previously. The strength results are in contrast to the leaching results which indicate a significant increase in permeability for specimens compacted dry of optimum. The proposal in chapter 9 to vary construction tolerance from wet of optimum in the upstream face to dry of optimum downstream is relevant as the downstream zone of the embankment will be more highly stressed if the equal slope section is adopted.

Moisture content at test and final moisture content for the specimens compacted dry and at higher density were very similar while the specimens compacted wet of optimum exhibited somewhat higher moisture contents in both cases. The observed strength difference was therefore not related to these factors.

10.?.12 Concrete Mortar (Microconcrete)

Graph 10.3 also reflects the results for the microconcrete prepared by the Portland Cement Institute. Results for cored cylinders as used in the leaching program are shown together with the results of the cube tests performed by PCI on 100 mm cubes (Thompson 1980a & b). The cube results have been divided by 1.25 to give the equivalent cylinder compressive strength (Fulton 1977). Reasonably good agreement between the two sets of data is evident with scatter comparable to that for the soil-cement and no consistent tendency for adjusted cube strength to exceed cylinder strength or vice versa.

Strength for cement content is comparable with that for the soil-cement with greater early strengths for the microconcrete but a reduced rate of strength gain with age. Fulton (1977) indicates that the relationship between compressive strength and log(age) for concrete is typically curved, approaching a horizontal asymptote with increasing age and this is confirmed by other authorities (Blake 1975, Neville 1973, USBR 1975). Abboud (1969) reported by Mitchell (1976d) indicates a linear relationship for soil-cement.

The data for the standard soil-cement specimens shown in graph 10.3 is depicted against log(age) in gr[illegible] 10.9 and confirms the linear nature of the strength vs log([illegible]lationship for soil-cement. Regression coefficients range [illegible]m 0.87 to 1.00 for different cement contents. The lines plotted for the soil-cement correspond to the curves in figure 10.3. Note that an upward trend is possibly indicated by the data points for 10 and 20% cement.

Graph 10.9 also shows the data points for the microconcrete specimens and confirms the curved relationship referred to above. It is notable that within 290 days, the soil-cement used in this research is as strong as the microconcrete for equivalent cement content and that thereafter relative strength of the soil-cement becomes progressively greater. A possible long-term advantage of soil-cement over concrete is therefore suggested.

It should be noted that, in both cases, the material tested contained only the fraction passing the 4.75 mm sieve and that the microconcrete was deliberately produced to simulate the mortar phase in a relatively low quality hearting concrete. This was done with a view to producing material that could be compared directly with soil-cement and these results confirm that this mix design requirement was fulfilled with regard to strength.

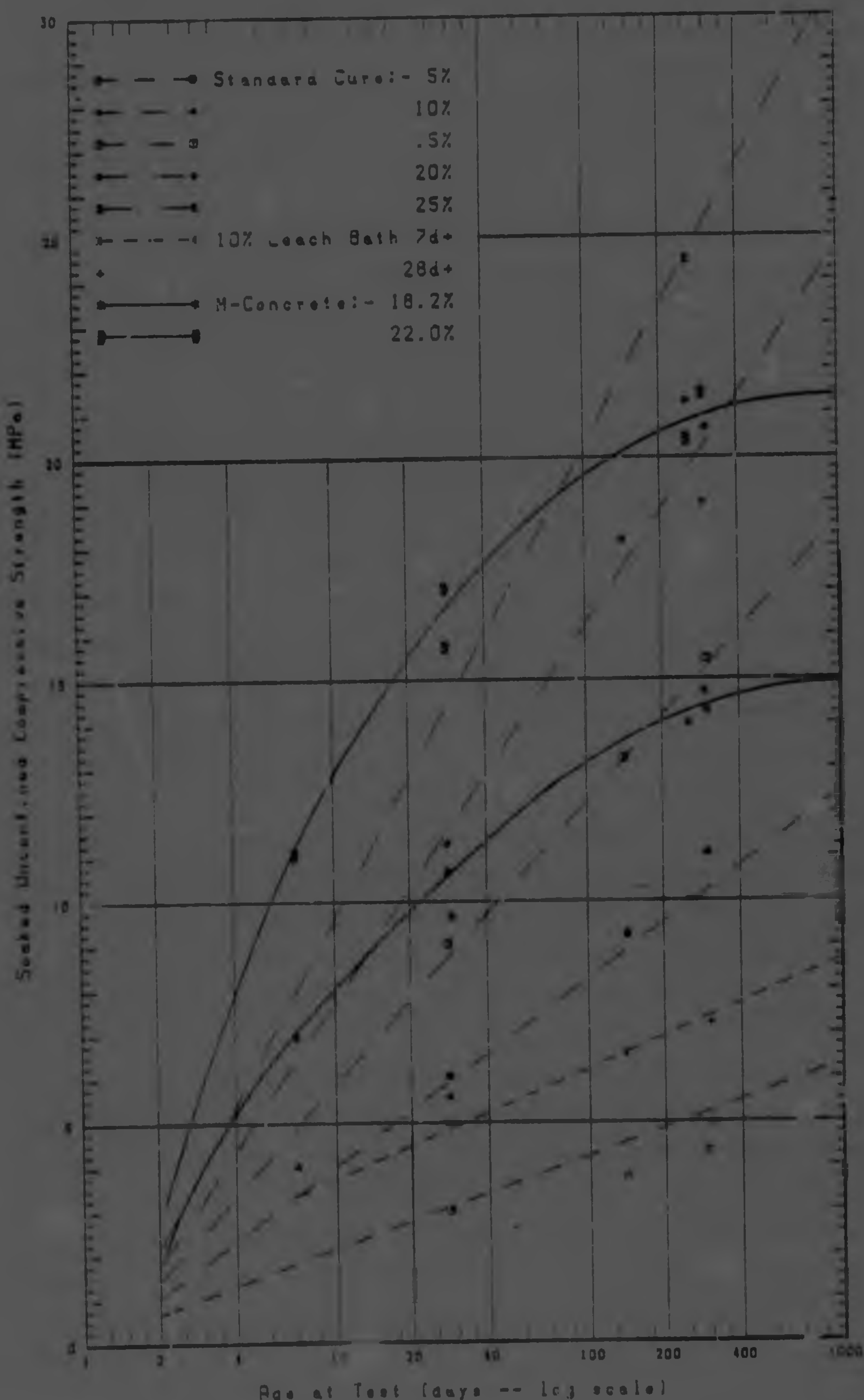
The microconcrete shows similar behaviour to the soil-cement in its relationship to moisture content at test, both in terms of observed values and trend as indicated by line M-M in graph 10.7. It was not possible to determine the final moisture content of the specimens as the initial dry mass was unknown.

10.4.13 Effect of Leaching Bath Cure

The effect on compressive strength of curing specimens in the leaching bath was referred to in section 10.4.5 and is depicted on graph 10.8. These results conflict with the findings of Majumdar (1968) who found that submerged curing led to an increase in strength, but are consistent with the results of the leaching tests reported in chapter 9. This supports the view that the leaching bath provided a relatively aggressive curing environment when compared to a conventional concrete curing bath filled with tap water changed at irregular intervals and containing a large number of closely packed specimens.

GRAPH 10.9

UNCONFINED COMPRESSIVE STRENGTH vs LOG(AGE) FOR STANDARD SOIL-CEMENT, MICROCONCRETE AND LEACH BATH CURE



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The data points for specimens with 10% cement placed in the leaching bath at age seven days and tested at ages 28 and 140 are indicated by the points x on graph 10.3 the corresponding points (+) are plotted on graph 10.4 indicating the same effect on split tensile strength. As noted in section 10.4.3, the ratio of split to compressive strength was effectively unchanged for these specimens.

The data points for the specimens leached from 7 to 28 and 7 to 140 days together with that for those leached from 28 to 301 days are plotted on graph 10.9 from which it can be seen that a progressive **relative** weakening of the material is indicated. The available data suggests a linear relationship with log(age) diverging from the standard 10% line at an age of 7 days, the age at which leaching commenced. Linear regression on the two date points for specimens leached from 7 days, together with the point of diversion, yield a regression coefficient of 1.00 and slope 2.45. This compares with the slope of 2.10 for the line for 5% cement. Further confirmation of the linear nature of this trend is required. The flatter slope, similar to that for specimens with 5% cement, suggests that the effect is equivalent to a reduction in cement content.

Provided this trend is linear, the loss in long term strength gain can be compensated for by an increase in cement content of perhaps two to three percentage points above that indicated by other considerations. This compares favourably with the arbitrary 2% increase recommended by the PCA for slope protection.

It should also be noted that provided the strength log(age) line is above the horizontal it merely indicates a reduction in long-term strength gain. On the expectation that under normal circumstances any layer of soil-cement facing in a major structure is unlikely to be submerged in under one year, this effect is unlikely to be of great significance, particularly when it is noted that design is generally based on the 28 day or possibly 90 day strength.

The above discussion assumes that the observed reduction in strength represents a uniform deterioration throughout the specimen. However, as with observations discussed in previous chapters, it is considered likely that this is in fact a surface effect. The penetration tests discussed in chapter 9 indicate that the strength of the surface of the specimen is reduced to a level which indicates almost complete removal of cement.

If the mode of deterioration is a surface effect, then the observed reduction in unconfined compressive strength calculated in terms of nominal cross sectional area reflects a reduction in the effective area of the specimen and not a reduction in strength. Penetration readings were only taken for two of the sets tested in unconfined compression and since these sets were not directly comparable, no examination of trends was possible. Interpretation of these results was therefore restricted to a comparison with equivalent standard soaked test specimens.

The data in graph 10.7 was used to estimate the expected compressive strength for the standard specimens at the moisture content at test of the specimens cured in the leaching bath. The failure loads for the leached specimens were used to estimate the equivalent diameter of unleached material and hence the depth of deterioration. This was compared with the depth of penetration for the set determined as the average of the two greatest penetration values for each specimen, the same value used as an acceptance criterion for testing leached specimens prior to conducting the triaxial tests.

For the set cured in the leaching bath from 7 to 140 days, the depth of penetration was 1.2 mm and the estimated depth of deterioration was 1.92 mm, a ratio of 62%. For the set cured in the leaching bath from 28 to 301 days the depth of penetration was 0.8 mm and the depth of deterioration 2.72 mm, a ratio of 29% which compares favourably with the 25% referred to in section 9.4.13 and indicated on graph 9.32 for specimens subject to similar initial cure and leaching period. The suggestion in chapter 9 that leaching in the leaching bath is a surface effect is confirmed by this observation based on a completely different analysis. It is considered likely that the reduction in penetration with increased curing reflects an irreversible, or more leaching resistant, strength gain. This is consistent with the substantial reduction in leaching loss associated with the increase in age at start of leaching from 7 to 28 days as depicted in graph 9.26.

10.4.14 Comparison with Other Data

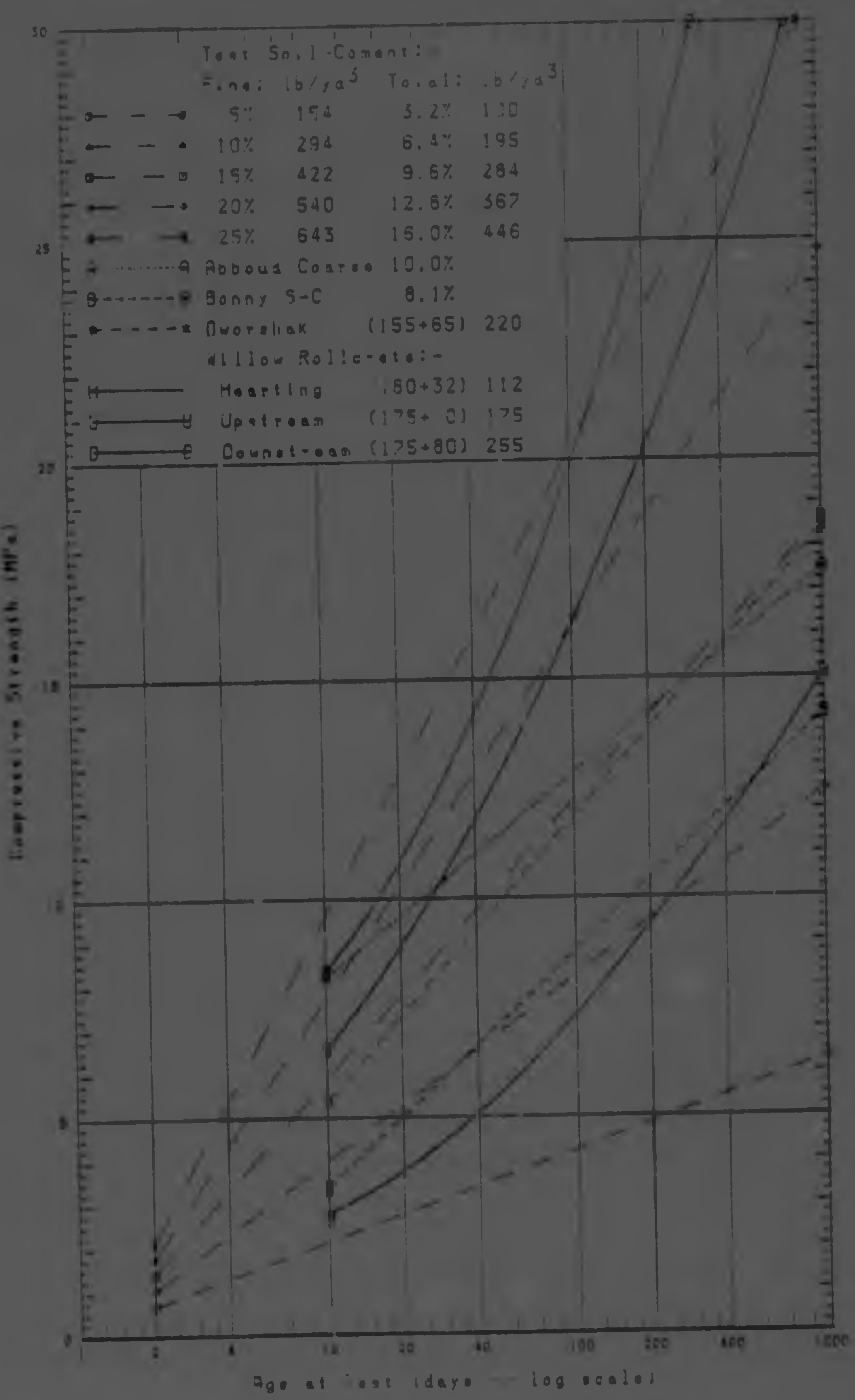
The strength vs log(age) lines for the standard soil-cement specimens are duplicated on graph 10.10 for comparison with a number of other materials. Corresponding symbols are applied to the end points of the lines for ease of identification and do not reflect observed data. It should be noted that the cement contents used in this research are determined as a percentage of the soil fraction passing the 4.75 mm sieve as this is the fraction used in the majority of the test specimens. Expressed in terms of total soil, the percentages are 3.2, 6.4, 9.6, 12.8 and 16% respectively for the cement contents referred to as 5, 10, 15, 20 and 25% throughout this thesis. The basis of this equivalence is detailed in section 12.1. Note that as no compressive strength tests were conducted on the material containing the coarse fraction, the direct conversion of the cement content - strength - age curves is not necessarily entirely correct. However, it is considered unlikely tha a substantial reduction in strength will result from the addition o the coarse fraction and the two limits of cement content applied to each line give an indication of probable range for comparison with the data discussed below.

As a number of metrication errors were evident in one of the references from which data was extracted for comparison, it was considered prudent to compare the results of this research on the basis of the units of lb/yd^3 in which the data for the reference

GRAPH 10.10

COMPRESSIVE STRENGTH vs

LOG(AGE) -- COMPARISON WITH OTHER DATA



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was generated. In these units, the cement contents for the fraction passing the 4.75 mm sieve are 154, 294, 422, 540 and 647 lb/yd^3 and in terms of the total soil 100, 195, 284, 367 and 446 lb/yd^3 respectively.

Abboud (1969) reported by Mitchell (1976d) suggested an upper bound for soil-cement containing 10% cement and produced with coarse grained soils, this line (A-A) is reproduced on the graph, from which it will be seen that the strength values for the material considered in this research are comparable. It should be noted that this data suggests that the test results should be compared on the basis of total cement content rather than as a percentage of the fraction passing the 4.75 mm sieve.

The PCA reported the results of compressive strength tests on cores taken from the Bonny Reservoir Test Section in 1961 which indicated a linear relationship with log(age) (PCA 1961). Cement content of this material was 10% by volume, 8.1% by mass (USBR 1961a). Line B-B on graph 10.10 reflects this data and compares well with the results of this research.

Schrader (1982), reporting on the proposed Willow Creek Dam to be constructed from roller compacted concrete (rollcrete), plotted strength vs log(age) for the conventional mass concrete used in Dworshak Dam as well as the various mixes to be used in Willow Creek Dam. The data for Dworshak (*-*) and the Willow hearting (H-H) and facing (U-U and D-D) mixes is also plotted on graph 10.10. It is notable that the data for Dworshak is linear, reflecting the influence of the flyash component and that the Willow Creek mixes show an upward curve with increasing age which is attributed to the effects of roller compaction. The presence of flyash and up to 7% of fines passing the No 200 sieve possibly contributes to this behaviour.

The Dworshak mix depicted by Schrader exhibits almost identical characteristics to the 15% (9.6%) soil-cement used in this program. On the basis of the fine fraction alone, the ratio of cementitious content in the soil-cement to that of the Dworshak concrete is of the order of 1.9 and if reckoned in terms of the total soil, the ratio reduces to 1.3.

The Willow Creek hearting mix shows similar strength characteristics to the Bonny Test Section soil-cement and to the 10% (6.4%) soil-cement adopted as standard in this program. This supports the decision to adopt this cement content as standard and confirms that soil-cement of the type used in this research is suitable for use in a major embankment dam (crest height of Willow Creek is 66m). The Willow hearting mix contains 80 lb/yd^3 cement and 32 lb/yd^3 flyash (80+32), representing a total cementitious content of 112 lb/yd^3. At 28 days this is directly comparable with the soil-cement used in this program.

The Willow facing mixes show greater strengths than the soil-cement for equivalent cementitious content. It is however notable that the strengths obtained from these mixes can be readily obtained from the soil-cement tested, albeit at comparatively high cement contents.

Comparison of cementitious contents for equivalent strength to the Willow mixes at age 1000 days indicates cement contents of between 2.7 and 3.1 times the total Willow cementitious content for this soil-cement for the proven strengths of the fraction passing the 4.75 mm sieve. When reckoned in terms of total soil, the ratio decreases to between 1.8 and 2.1 and at age 28 days, the ratio reduces further.

It is thus apparent that while the range of strengths applied at Willow Creek Dam can be achieved using the soil-cement in question, significantly higher cement contents will be required to achieve them. This does not necessarily indicate that the soil-cement alternative will not be economic, 70% of the aggregate for Willow Creek was required to be quarried, so that it is reasonable to assume that unit cost of aggregate will be significantly higher than cost of soil from suitable borrow areas.

Referring to the Merritt Dam cost comparison in chapter 3, an approximate assessment of equivalent costs for a soil-cement dam for Willow Creek was undertaken. It was assumed that the cost of producing the aggregate at Willow Creek was half the remaining cost per unit volume after allowing for the cost of cement and that the cost of producing the soil for soil-cement was half the cost of producing the concrete aggregate. It was further assumed that the soil-cement alternative would contain twice as much cement. This comparison indicated that the soil-cement alternative would cost approximately the same as the rollcrete dam. While the above estimate is entirely arbitrary, it is felt that it serves to confirm that soil-cement can be expected to be competitive on suitable sites.

It is notable that a similar cementitious content is indicated for the hearting mix whether rollcrete or soil-cement is used. In addition, the strength of the Willow hearting mix is similar to that of the Bonny Test Section slope protection with a relatively small addition of cementitious material being indicated to exceed the strength of the Bonny facing at all ages. In view of the excellent behaviour of the Bonny facing and other slope protection with similar properties, graph 10.10 again serves to highlight the anomalies between the principles used in the design of concrete and earthfill dams referred to in chapter 3. In this case, soil-cement of the type used at Bonny is deemed adequate to protect an earth embankment from wave action when placed with an effective thickness of less than one meter, but the material used in facing a concrete dam is specified to be several times stronger at early ages with greater divergence at greater ages despite the fact that the upstream facing is to be protected by precast cladding panels and the underlying material has substantial strength. In addition, these mixes, while containing less cementitious material, are appreciably stronger than that used at Dworshak Dam. It should also be noted that the maximum unconfined compressive strength indicated by the analysis in chapter 3, as reflected in figure 11, is approximately 12 MPa for a 100 m embankment.

Allowing for the fact that Willow Creek is essentially an experimental structure and that there may be significant differences between the respective sites, it would appear that the high rollcrete strengths are not actually necessary. In this case, the cement contents required for a soil-cement dam can be expected to be similar to those used for Willow Creek.

10.5 CONCLUSIONS

The unconfined compressive strength and split tensile strength of the soil-cement is found to increase linearly with cement content and logarithm of age. Soaking of the specimens prior to testing produces a significant reduction in compressive and tensile strength but the uniform reduction indicated by the preliminary tests is not confirmed, further investigation is required to establish this relationship. Humid curing in the chamber used in this program led to lower strengths than for specimens sealed after seven days.

Reduction of the platen width in the split tensile test from the 3 mm (1/12th diameter) width used to a knife edge of 0.25 mm produces a significant reduction in indicated strength.

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 compressive strength.

The moisture content of the material at the time of test is found to have a significant effect on the compressive strength and is used to account for a number of apparent anomalies in the test results. No measurable effect due to ageing of the cement was observed. Final (hydration) moisture content is also found to influence compressive strength.

A level of confidence of at least 95% is indicated as the minimum required to indicate a real difference between sets of test results. Compaction delay, within the half hour batching - mixing - compaction cycle used, had no discernible influence on the results and, while variations in rate of loading from 41 to 245 kPa/sec produced a significant variation in strength, it is considered that the control applied during the test program effectively eliminated this factor.

Specimens subject to wetting and drying cycles, as described in chapter 8, show a greater compressive strength than those subject to the standard cure and it is suggested that this is probably due to the high temperatures used in the wet-dry test. Further tests are required to establish the exact effect, but it is possible that higher ground surface temperatures than those used in the laboratory could account for the higher strength of soil-cement as determined from cores taken in the field. It is suggested that accelerated curing of critical areas of soil-cement could be accomplished by the use of black plastic sheeting to apply a moisture seal and raise curing temperature.

Compaction of specimens at a moisture content of 1% below optimum and at a density of 2% above standard proctor maximum produces no change in compressive strength, however, compaction 1% wet of optimum produces a significant reduction in strength. In the light of the findings of Fetz (1973) with regard to crack control, compaction dry of optimum would appear to be indicated as the standard construction condition in zones where strength is the primary design consideration. This is consistent with the graded moisture control proposed in chapter 9 which indicates that compaction tending to wet of optimum is to be preferred near the water retaining face provided that adequate crack control and compressive strength can be achieved.

The concrete mortar (microconcrete) prepared by the Portland Cement Institute is comparable to the soil-cement in terms of the relationship between compressive strength and cement content, confirming the validity of the mix design. The ratio of cube to cylinder strength of 1.25 is confirmed for these mixes and results. Compaction delay, within the half hour batching - mixing - compaction cycle used, had no discernible influence on the results and, while variations in rate of loading from 41 to 245 kPa/sec produced a significant variation in strength, it is considered that the control applied during the test program effectively eliminated this factor.

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The concrete mortar (microconcrete) prepared by the Portland Cement Institute is comparable to the soil-cement in terms of the relationship between compressive strength and cement content, confirming the validity of the mix design. The ratio of cube to cylinder strength of 1.25 is confirmed for these mixes and specimen sizes. The strength gain with log(age) occurs at a reducing rate as is normal for concrete but which contrasts with the linear behaviour of soil-cement, indicating a possible long-term advantage for soil-cement. It is however noted that the mix was deliberately designed as a low quality hearting concrete.

Curing of specimens in the leaching bath with continual flushing with distilled water leads to a reduction in both indicated tensile and compressive strength with the strength ratio remaining effectively constant. Tentative correlation with the results of penetration tests on a few of the specimens confirms the observation in chapter 9 that the deterioration is a surface effect and leads to a reduction in effective cross section of the specimen. Further investigation of this phenomenon is required.

On the basis of unconfined compressive strength, the soil-cement used in this investigation compares well with the data for other investigations, and is also similar to the material used in the Bonny Dam test section. The strengths obtained are comparable to those for the conventional mass concrete used at Dworshak Dam and the rollcrete used at Willow Creek Dam although larger amounts of cementitious material are required. Increased stabilizer contents of the order of 20 to 60% would be indicated for designs based on the 28 day strength of the material. The extent of this difference is such that on sites where suitable soil is available in convenient borrow areas, and quarrying of rollcrete aggregate is required, soil-cement can be expected to be competitive as regards overall cost of material. This is particularly applicable in the South African context where large deposits of alluvial gravels are relatively uncommon.

It is observed that there appears to be a discrepancy between the strength of soil-cement considered adequate to produce durable slope protection and the strength of concrete used for the facing of Willow Creek Dam. It would appear that if strength requirements are limited to those required for stability and proved adequate for slope protection, the extra cementitious content for soil-cement referred to above will prove largely unnecessary. The higher cement content of soil-cement for equivalent strength will, however, still apply.

THE USE OF STABILIZED SOIL
AS A MATERIAL IN DAM CONSTRUCTION

James Alexander Robertson

VOLUME TWO

[illegible] submitted to the Faculty of Engineering
[illegible] of the Witwatersrand, Johannesburg
[illegible] of Doctor of Philosophy

Johannesburg 1984

To my parents, Angus and [illegible]

and my wife, [illegible].

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