

BRICK AGGREGATES FOR ROAD BASE

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DECLARATION

I hereby declare that this thesis has been composed by me and that the work described in it was performed solely by me, unless otherwise stated.

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ABSTRACT

In Bangladesh crushed brick aggregates are used as a road base material due to shortage of stone. Brick aggregates are much weaker compared with stone aggregates and undergo degradation during rolling and subsequent trafficking. The strength of the aggregates depends on the crushing strength of the original bricks. Possible ways to improve the strength of bricks have been suggested. Bricks and brick aggregates of five different strength groups have been tested for engineering properties and the intercorrelations between the properties have been discussed. Degradation of aggregates along with a review of degradation studies have been discussed. California Bearing Ratio tests with a modified mould have been performed. Experimental full scale road bases were constructed with two groups of aggregates. The aggregates were hand crushed to the required size to represent field conditions in Bangladesh. The rolling was done by a roller especially constructed to simulate an eight ton smooth wheel roller. The pavements were constructed as per Bangladesh practice. Each pavement was subjected to repeat loading at two locations. The load was the simulation of a wheel load of a typical truck in Bangladesh. At least 100,000 load repetitions were applied at each location and the deformation characteristics of the pavements were recorded. In situ C.B.R. tests with a modified plunger, approximately representing the contact area of a commercial tyre, have been performed. The same plate was used for the repeat loading tests. Multistage triaxial tests with samples to represent field conditions have been performed.

Based on the test results and a consideration of the British and the Indian specifications for water bound macadam, a specification for road base suitable for conditions prevailing in Bangladesh has been suggested.

CHAPTER 1

INTRODUCTION

Bangladesh is a developing country having an area of about 142,000 square kilometres (55,000 square miles) and a population of 75 million. The total paved road would not be more than 4,000 kilometres (2,500 miles). Three major rivers and many of their tributaries and distributaries flow through the country. All the three rivers are in their tertiary stage and hence their flow is a minimum. Their beds are silted up. Almost the entire area of Bangladesh is little above the sea level. The land is composed of alluvial soil which consists mostly of silt and clay. The land is inundated by flood almost every year. Roads are constructed almost always on embankments, the formation level of which is kept about one metre (3 feet) above the highest flood level. For these reasons the height of embankment is considerable. The proximity of water level to the formation level, the nature of the soil type and the inadequacy of compacting machinery, etc. sometimes leads to the failure of pavements. From the past experience of more than twenty five years, it has been found that construction of flexible pavement, when compared with that of rigid pavement, is better suited to conditions prevailing in Bangladesh.

Bangladesh is not rich in her natural resources. She has very little stone of her own and has to import the same from abroad. As a result brick aggregates are almost always used as a road base material to save scarce foreign exchange required for importing stones. Sometimes brick aggregates are used even in the base course (bituminous grouting). The bricks to be crushed for aggregates are manufactured locally. The crushing strengths of bricks are much lower than those

of stones. The crushing strength of first class and selected slightly over-burnt bricks (local classification of bricks, the latter is mostly used for making aggregates), are around 13818 kN/m^2 (2000 p.s.i.) and 20727 kN/m^2 (3000 p.s.i.) respectively.

Brick aggregates when used as road base undergo considerable degradation during compaction by rollers as well as during subsequent trafficking.

The aim of the present work has been to investigate:

- (1) The engineering properties of the brick aggregates and the inter-correlation between the properties.
- (2) The degradation characteristics of the aggregates under compacting efforts and repeat loading
- (3) The deformation characteristics of the brick aggregates as a road base under static and repeat loading.
- (4) The bearing capacity of the aggregates as road base.
- (5) Possible improvement to the specification for the usage of brick aggregates as a road base material.

CHAPTER 2

MANUFACTURE OF BRICKS IN BANGLADESH

2.1 INTRODUCTION

Since the present work deals with brick aggregates, it will be relevant to discuss the manufacturing process of bricks in Bangladesh. The crushing strength of the bricks and hence the strength of the aggregates depends on the manufacturing process.

Good brick earth is needed to make quality bricks. The earth is well mixed, moulded and dried before being burnt.

2.2 PROPERTIES OF GOOD BRICK EARTH

Good brick earth contains 20 to 35 percent Alumina and 50 to 70 percent Silica. Lime content of the same should not exceed 5 percent - which should be in a finely divided state and not in lumps. Alkalies may remain in traces. Excess of alkalies make the clay unsuitable for burning bricks. Presence of excess alkalies disfigures the bricks. Oxides of iron may remain up to 5 percent. The colour of bricks very much depends on the amount of iron and ranges from light yellow to orange and reddish. The colour increases in intensity according to the proportion of iron contained in the clay. Pebbles of stone and gravel, Alkalies, Limestone and Vegetation and organic matters are the four harmful constituents of brick clays.

2.3 PREPARATION OF BRICK EARTH

2.3.1 TOP SOIL STRIPPING. The surface of the site selected for obtaining clay is cleared of the vegetation and loose earth.

2.3.2 CLAY DIGGING AND WEATHERING. Generally this operation is carried out manually, but mechanical excavators can easily be used for this purpose for major brick works. The earth which is dug up in lumps is broken and pebbles, if any, removed from it. Then the earth is spread in heaps two to four feet high and exposed to the atmosphere for about two weeks or more. The process is called weathering and it dries up the clay. The clay should be kept damp during the period it is stored. In case the clay is hard or clods cannot be easily broken, the earth should be ground in a pugmill.

About 2.832 cubic metres (100 cu. ft.) of clay is required for making 1,000 Bangladesh size bricks (247mm x 121mm x 70mm). Clay should be improved by the addition of sand, ashes or lime, if so desired, before it is thrown into heaps. This process is called blending.

2.3.3 TEMPERING. Brick earth should be kneaded to the required consistency before moulding by adding water. The earth is cut, slashed and well worked with a spade and is trodden which makes it soft. Water is gradually applied when the treading is in progress. No free lime should be allowed to remain and the clay should be turned into a uniformly soft and yielding mass fit for moulding. About 25 to 30 percent water is needed to make a soft consistent clay. Generally good bricks can be moulded within a few days after digging the soil. During the operation of tempering, the clay should be cleaned from stones and pebbles, etc., which are generally picked up by hand and thrown away. In major works the mixing and tempering of clay is always done in a pugmill.

2.4 MOULDING OF BRICKS.

When the clay has been thoroughly mixed, it should be moulded either by hand or by machine moulding.

2.4.1 METHODS OF HAND MOULDING. The methods employed are:

(i) Slop moulding and (ii) Sand moulding. In slop moulding the mould is dipped in water after moulding every individual brick to prevent the edges of the brick from sticking to it. This is not a good method for moulding high class bricks. In sand moulding the mould is sprinkled with, or dipped in, fine dry sand or ashes every time the piece of clay for moulding a brick is placed and pressed into the mould. This method prevents the adhesion of the clay to the mould and the bricks produced by this method have better surfaces. The streaks of sand left on the brick surfaces vitrify on burning which gives a stronger surface to the brick. Hand moulding is further sub-divided into two methods, namely: (i) Ground moulding, and (ii) Block, or Table moulding. In the method of ground moulding the bricks are made on the ground usually in wooden moulds, whereas in the case of table moulding, they are made on a block or table, usually with metal moulds. More regular shapes and smoother surfaces can be obtained by adopting the process of table moulding. Brick moulds are $\frac{1}{10}$ th larger than the required size of the bricks to allow for shrinkage.

2.5. DRYING OF BRICKS

Before the bricks are burnt they should be dried to make them sufficiently strong. The moulded bricks are dried by being first placed on their edges for some time and then piled in open order in long rows and stacks. The bricks may be dried artificially also,

but it should be done with great care. Rapid drying may develop cracks in the bricks which may also warp.

2.6 BRICK BURNING

The temperature required for the purpose is about 1100°C to 1200°C (2000°F to 2200°F). In a brick kiln, the first two or three days of firing the kiln is meant to drive away the moisture and to heat the bricks. Thereafter, the fire is made vigorous and maintained for three or four days when the bricks are burnt to the required standard. It takes eight to ten days for the kiln to cool down, whereafter it is emptied from the top.

2.7 STRENGTH OF BRICKS

The crushing strength of burnt bricks varies from 4299 kN/m^2 to 19346 kN/m^2 (40 to 180 tons/ft²). It is usually about 13819 kN/m^2 (2000 p.s.i.) for first class bricks and about 20728 kN/m^2 (3000 p.s.i.) for 'picked jhama' (selected slightly over-burnt) bricks in Bangladesh. (Bricks are classified as picked jhama, 1st class, 2nd class and 3rd class in Bangladesh and India). The crushing strength of brick masonry is only about $\frac{1}{3}$ to $\frac{1}{5}$ of the crushing strength of a single brick.

The relative density of a brick should not be less than 1.8.

2.8 BRICKS FOR MAKING AGGREGATES FOR TESTING

Bricks could be made in the laboratory only on a very small scale (twelve at a time) using the only oven which could develop the temperature required. But bricks needed for the full scale tests were a few thousand. Hence it was not feasible to make them.

It would have been ideal if bricks similar to those manufactured in Bangladesh could be obtained. British bricks are stronger, better made and often manufactured in modern factories (engineering bricks). But some manufacturers do produce bricks which are made in kilns and which vary quite a lot in strength depending on the amount of burning (or heat) they get in the kiln. Although these bricks have higher crushing strengths than those in Bangladesh, they did have a wide range of strength so as to cover the range of strengths of bricks in Bangladesh. These types of bricks were obtained from the Scottish Brick Company at Niddrie in Edinburgh.

2.9 ENGINEERING PROPERTIES OF BRICKS

Six groups of bricks were tested for crushing strength (Test No. 43) and water absorption (Test No. 44) in accordance with B.S. 3921/1965. (Appendix A and Tables 2.1.1 to 2.2.B).

Of the six groups, Group I, II, A and B were properly burnt bricks but Group III and Group IV were under-burnt bricks which would normally be recycled into the burning process.

Statistical analyses were performed on the data obtained from the above mentioned tests to find mean and standard deviation.

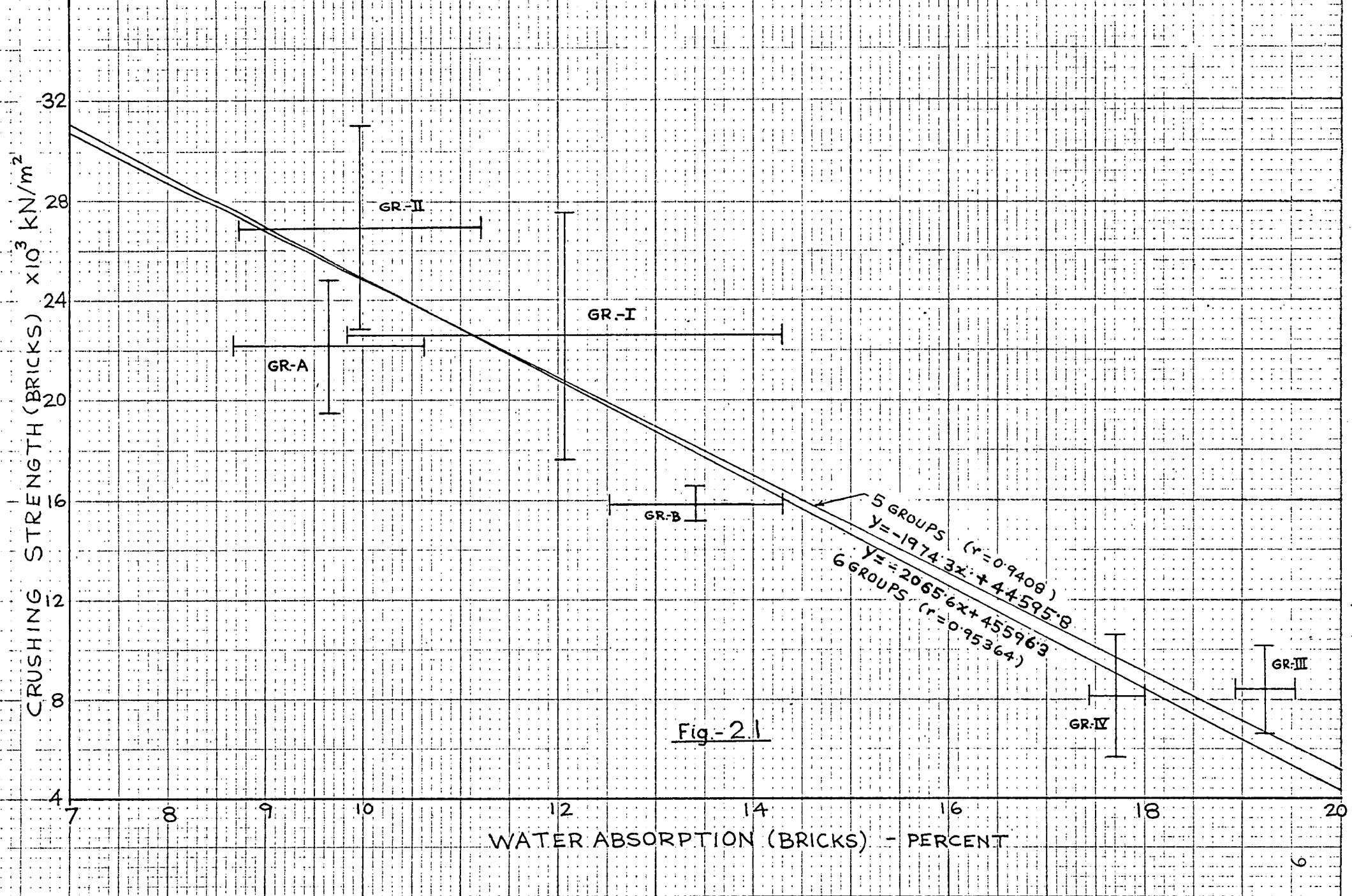
Crushing strength (mean of ten values in each group) was plotted against water absorption (mean of ten values in each group) with (a) five groups & (b) six groups (Fig. 2.1). (The reason for plotting graphs with five and six groups is that all the six groups were tested for the properties of bricks, but Group IV was not tested for properties of crushed brick aggregates and engineering behaviour of aggregates as road base material). The equation of the best fit straight line for both (a) and (b) have been found out statistically and have been drawn (Fig. 2.1).

The line for (a) has a correlation coefficient of 0.94080, and that for (b) a correlation coefficient of 0.95364. Plus and minus one standard deviation of both the variables of each point have also been shown on the graphs.

The arithmetic mean plus and minus one standard deviation contains 68.27 percent of the data, plus and minus two standard deviations contains 95.45 percent, and plus or minus three standard deviations contains 99.73 percent. The standard deviations show a large scatter for all values of crushing strength in all the groups, except group B and water absorption in the first two groups only.

The correlation coefficients of the graphs indicate a strong correlation between crushing strength and water absorption. The higher the crushing strength, the lower is the water absorption.

A photograph of bricks from different groups before and after crushing is shown in Plate 2.1.





Key to group



PLATE 2.1

BRICKS FROM DIFFERENT GROUPS BEFORE AND AFTER CRUSHING
GROUP II, THE STRONGEST, IS THE DARKEST IN COLOUR AND
GROUP III, THE WEAKEST, IS THE LIGHTEST IN COLOUR.

CHAPTER 3

PROPERTIES OF CRUSHED BRICK AGGREGATES

3.1 INTRODUCTION

Due to shortage of stone aggregates in Bangladesh, crushed brick aggregates are used as a substitute for them in the road base. Crushed brick aggregates are obtained by crushing whole bricks and these aggregates are weaker than stone aggregates. The properties of whole bricks have been discussed in Section 2.9 of the previous Chapter. The properties of brick aggregates derived from these bricks are discussed in this Chapter.

3.2 GROUPING OF BRICKS

Bricks were grouped according to their strength. Initially they were grouped into four groups, two groups in the higher and two in the lower range of strengths. The strength of the bricks depends on the amount of heat they get in the kiln. Even in the same kiln different bricks get different amounts of heat due to their location within the kiln. The more heat they get the stronger they become, but after a certain stage they start to become mis-shapen. With further heat they start to melt. The optimum strength of the bricks is obtained just before they start to become mis-shapen. But it is possible to get a little more strength at the cost of a little mis-shapening which is desirable when the bricks are used to make aggregates by breaking them. This type of brick is classified as "picked jhama" brick in Bangladesh and the term means "selected slightly overburnt" brick. The colour of the brick changes with temperature. So from the same kiln bricks of a wide range of strengths will be obtained. Their colour will also vary. When two bricks are struck together, a

metallic sound is obtained which varies with their strength - the stronger the brick, the sharper is the sound. People working in the kiln can, by experience, group bricks according to their strength by observing their colour, by the sound when two bricks are struck and also from their position in the kiln. The bricks were grouped initially on this basis - Group I and Group II in the higher region and Group III and Group IV bricks in the lower range. In fact Group III and Group IV bricks were underfired and would normally be recycled into the kiln to develop more strength. The four groups of bricks have been tested for crushing strength and water absorption. The crushing strengths of Group I and Group II overlapped, as did those of Group III and Group IV. They also had a wide standard deviation. In practice this would also happen. But for controlled experiments it is desirable to have groups in which the strengths did not overlap and the standard deviation was not as wide as for those obtained in the above mentioned groups (i.e. the difference between the crushing strengths of the strongest and the weakest brick is not very much). For this purpose it was attempted to estimate the crushing strength of bricks by a non-destructive method. An ultrasonic pulse was sent from a PUNDIT machine (Plate 3.1) through a brick and the distance travelled by the pulse being known the velocity of the pulse could be determined. The ultrasonic pulse velocities through thirty bricks (of three different groups) were determined at three different moisture conditions, (i) oven dry, (ii) air dry at room temperature and (iii) saturated (and surface dried). These bricks were then crushed to determine their crushing strength. Their crushing strengths were then related to the ultrasonic pulse velocities at different moisture contents by obtaining the best fit straight line

through the points and the correlation coefficients found out. The graph crushing strength of bricks vs. U.P.V. (ultrasonic pulse velocity) through oven dry bricks has a correlation coefficient of 0.9286. The graph crushing strength of bricks vs. U.P.V. through air dry (room temperature) bricks has a correlation coefficient of 0.9348. The correlation coefficient of the graph crushing strength of bricks vs. U.P.V. through saturated and surface dried bricks is 0.9482 (Appendix F). The coefficients indicate a very strong correlation. The bricks to be tested for U.P.V. had been kept for at least two days inside the laboratory which is kept at normal room temperature of around 20°C. Two groups of bricks were selected whose strengths had been estimated from the U.P.V. (through air dry bricks). To differentiate these groups from the previous groups these have been denoted as Group A and Group B. Bricks having estimated crushing strengths between 22109 kN/m² and 26254 kN/m² (3200 - 3800 p.s.i.) were selected for Group A and bricks whose crushing strength ranged between 15200 kN/m² to 19345 kN/m² (2200 - 2800 p.s.i.) were selected for Group B. Of 630 bricks tested, 454 bricks were selected for Group A. Out of 300 bricks, 174 bricks were selected for Group B (Appendix F).

3.3 PREPARATION OF THE AGGREGATES

In Bangladesh aggregates are always prepared by hand crushing (putting a brick on top of a boulder, striking it with a hammer and breaking it down to the required sizes). To save a little time Group I and Group II bricks were first broken to a few pieces by hand and then crushed in the brick crusher in the laboratory. Group III bricks had to be crushed carefully by hand only because these bricks are underfired and their strengths are very low. If these bricks are

fed into the crusher too many fines are produced. As Group A and Group B bricks are meant for full scale tests and would represent actual aggregates in the field which are hand broken, it was decided to crush them by hand.

3.4 TESTS

All tests were performed in accordance with B.S. 812, Parts 1, 2, 3/1975.

The aggregates could not be tested for the determination of aggregate crushing value (Test No. 3/7) as the percentage of fines produced for the Group I aggregates has been found to be more than 46%. It is stated in B.S. 812, Part 3, Test No. 7, that if the percentage of fines produced in the above test is more than 30%, the test is not valid. Hence, the determination of the ten percent fines value (Test No. 3/8) has been carried out instead. The other tests which have been carried out are Aggregate Impact Value (Test No. 3/6) Relative Densities (oven dried basis, saturated and surface dried basis, apparent), and Water Absorption (Test No. 2/5), Angularity Number (Test No. 1/7.5), Elongation Index (Test No. 1/7.4), Flakiness Index (Test No. 1/7.3) and Sieve Analysis (Test No. 1/7.1).

3.5 RESULTS

3.5.1 TEN PERCENT FINES VALUE

The Ten Percent Fines Values have been found to be 59.51, 43.93, 14.19, 57.53 and 31.54 kN for Group I, Group II, Group III, Group A and Group B aggregates respectively.

Group B aggregates are weak, but the Group III aggregates are very weak. Degradation in the form of rounding off the angular aggregates and smoothing of their rough surfaces occurred during sieving

after application of the load to the aggregates. Hence these aggregates were sieved for only 5 minutes in a mechanical shaker to obtain the fines passing No. 7 B.S. sieve. When these materials (especially Group IV) were sieved for a further period, more and more fines were obtained as the sieving process continued and the particles became rounded.

3.5.2 AGGREGATE IMPACT VALUE

The Aggregate Impact Values of Group I, Group II, Group III, Group A and Group B aggregates have been found to be 49, 57, 73, 45 and 55 respectively.

During the tests some fines were lost which flew from the cup due to the thrust of the hammer when there were enough fines on top of the cup after a few blows of the hammer. The B.S. states that if the loss of material is more than one gramme the test is to be repeated. But there were losses more than the above mentioned amount in all the cases. The losses were around two grammes for Group I, Group II and Group A materials and more than three grammes for the Group III and Group B aggregates. It was considered most reasonable to add the loss to the finer fraction as the losses were powders and would definitely pass through 2.40 mm (No. 7 B.S.) sieve. Repeating the tests so that losses were less than one gramme would not have been normally achieved unless some special arrangements were made to prevent the fines from flying or recovering the fines from the surroundings by special arrangements, which was not considered worthwhile.

3.5.3 RELATIVE DENSITIES AND WATER ABSORPTION

Relative densities and water absorptions have been determined by Test No. 2/5.4 of the B.S. since Test No. 2/5.3 is not suitable for

friable aggregates which may break down during the test. Relative densities on an oven-dried basis, saturated and surface dried basis, apparent relative density and water absorption have been determined. Relative density on an oven dried basis has been found to be 1.84, 1.88, 1.66, 1.85 and 1.77 for Groups I, II, III, A and B aggregates respectively. Relative density on a saturated and surface dried basis for Groups I, II, III, A and B are 2.07, 2.09, 2.02, 2.05 and 2.05 respectively. The apparent relative density of Group I, II, III, A and B aggregates are 2.39, 2.39, 2.59, 2.33 and 2.45 respectively. Group I, II, III, A and B aggregates have a water absorption of 12.07, 9.79, 19.23, 11.22 and 15.64 (percent of dry weight) respectively. The above tests were carried out with the 19.05 - 12.70 mm ($\frac{3}{4}$ " - $\frac{1}{2}$ ") size aggregates. The same size was used for determination of Angularity Number. It is the predominant size retained between the pair of sieves ($\frac{3}{4}$ " - $\frac{1}{2}$ ", $\frac{1}{2}$ " - $\frac{3}{8}$ ", $\frac{3}{8}$ " - $\frac{1}{4}$ " and $\frac{1}{4}$ " - $\frac{3}{16}$ "). Only one test (instead of two) in Group I, II and III was carried out.

3.5.4. ANGULARITY NUMBER

Angularity numbers of the aggregate size passing 19.05 mm ($\frac{3}{4}$ ") and retained on 12.70 mm ($\frac{1}{2}$ ") B.S. sieve, which were the predominant sizes, were determined. They are 11, 13, 10, 13 and 14 for Groups I, II, III, A and B aggregates respectively. The Angularity Number usually ranges from 0 to about 12. The results indicate that all the aggregates are highly angular. The Group III materials are slightly less angular than the other groups but nonetheless they are very angular.

There was difficulty in carrying out the test with the Group III and Group B aggregates as these tended to break down. The B.S. states

that "the angularity number does not apply to an aggregate which breaks down during the test". Hence the validity of the results, especially for the Group III and Group B materials, is questionable. However, they give a relative comparison between the groups and they clearly indicate that all of them are highly angular.

3.5.5 ELONGATION INDEX

Elongation Index is usually determined by taking 200 pieces of the aggregates from each of the sieve size fractions between two consecutive sieves of the standard sieve test which constitutes more than 15%, and 100 pieces from size fractions which constitutes between 5 and 15%, and none from the fractions which constitute less than 5% (by weight) of the total quantity of the aggregates in each group. But 200 or 100 numbers of pieces were not available and a proportionate decrease in the number was made as per availability.

The elongation index for Group I, II, III, A and B aggregates are 27, 18, 22, 14 and 9 respectively. These indicate the percentage by weight of the elongated particles in the whole sample.

3.5.6 FLAKINESS INDEX

The flakiness index for each group of aggregates was determined with exactly the same aggregates which were used to determine elongation index. It was found to be 11, 11, 1, 5 and 2 for Group I, II, III, A and B aggregates respectively. The fact that the Group III, A and B bricks were broken to aggregates entirely by hand might have led to this smaller percentage of flakey materials compared with the other two Groups.

This phenomenon was also observed in Bangladesh. Machine

crushed aggregates always tended to be flakey, whereas hand crushed aggregates tended to be more cubical.

3.5.7 SIEVE ANALYSIS

Sieve analysis is performed to determine the particle size distribution of aggregates, and it is usually done either on air dry or oven dry (110°C) aggregates.

The aggregates were air dry before they were sieved. The room temperature (of the laboratory) was always a little above 20°C ($22 - 23^{\circ}\text{C}$). The Group III aggregates were not sieved more than 5 minutes in the mechanical shaker to prevent degradation of the particles.

The cumulative percentage by weight of the total samples passing each of the sieves has been shown graphically. (Figs. 4.2 - 4.6).

The particles size distribution of all the five Groups show that they are much coarser than the recommended particle size distribution of Granular sub-base material Type 1, Type 2 and Wet-mix Macadam of the DOE Specification for road and bridge works (Clauses 803, 804 and 808 respectively).

3.6. CORRELATION BETWEEN DIFFERENT PROPERTIES OF THE BRICKS AND THE BRICK AGGREGATES

To investigate correlations between different properties of the bricks and the aggregates the following graphs have been plotted, the best fit straight line has been found out by linear regression and drawn. The correlation coefficients have also been calculated.

<u>Fig.</u>	<u>Plot</u>
3.1	CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. WATER ABSORPTION OF BRICKS (AV.)
3.2	CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. AGGREGATE IMPACT VALUE
3.3	CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. TEN PERCENT FINES VALUE
3.4	CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. WATER ABSORPTION (AGGREGATES)
3.5	CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. RELATIVE DENSITY (i) Oven Dried Basis (ii) Saturated and Surface Dried Basis (iii) Apparent
3.6	CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. DRY DENSITY (COMPACTED AGGREGATES)
	CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. BULK DENSITY (COMPACTED AGGREGATES)
3.7	CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. CALIFORNIA BEARING RATIO
3.8	AGGREGATE IMPACT VALUE Vs. WATER ABSORPTION (AGGREGATES)
3.9	AGGREGATE IMPACT VALUE Vs. TEN PERCENT FINES VALUE
3.10	AGGREGATE IMPACT VALUE Vs. RELATIVE DENSITY (i) Oven dried basis (ii) Saturated and surface dried basis (iii) Apparent
3.11	AGGREGATE IMPACT VALUE Vs. DRY DENSITY (COMPACTED AGGREGATES)
	AGGREGATE IMPACT VALUE Vs. BULK DENSITY (COMPACTED AGGREGATES)
3.12	TEN PERCENT FINES VALUE Vs. RELATIVE DENSITY (i) Oven dried basis (ii) Saturated and surface dried basis (iii) Apparent

<u>Fig.</u>	<u>Plot</u>
3.13	TEN PERCENT FINES VALUE Vs. CALIFORNIA BEARING RATIO
3.14	DRY DENSITY (COMPACTED AGGREGATES) Vs. RELATIVE DENSITY (i) Oven dried basis (ii) Saturated and surface dried basis (iii) Apparent
3.15	DRY DENSITY (COMPACTED AGGREGATES) Vs. CALIFORNIA BEARING RATIO
3.16	BULK DENSITY (COMPACTED AGGREGATES) Vs. CALIFORNIA BEARING RATIO.

DISCUSSION ON THE GRAPHS

3.6.1 CRUSHING STRENGTH OF BRICKS Vs. WATER ABSORPTION OF BRICKS

This graph has been drawn with the mean of ten values in each group. There are six groups. But five groups were investigated for the properties and behaviour of their aggregates. So two graphs have been plotted, one with the five groups and the other with all six groups. One standard deviation of both the x and y coordinates of each of the six mean values have been shown and the best fit straight line considering five and six sets of values have been shown. The graph with Groups I, II, III, A and B has a correlation coefficient of 0.9408 and that with all the groups has a correlation coefficient of 0.9536. This correlation coefficient is a measure of the goodness of fit of the graph drawn. The nearer the value to 1.0, the better is the goodness of fit, i.e. the stronger the correlation between the two variables. The above result, therefore, indicates that the higher the crushing strength of the bricks, the lesser is the water absorption.

3.6.2 CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. AGGREGATE IMPACT

VALUE

This graph has a correlation coefficient of 0.7396, which does not indicate a good correlation. This may be due to the fact that the Aggregate Impact Values obtained did not follow a logical order. The weakest aggregates have the highest A.I.V., but the strongest does not have the lowest A.I.V. The crushing strength of bricks has a wide range within the groups I, II and III and the strengths of Group I and Group II bricks are very much similar although the average crushing strength of Group II bricks are higher than those of Group I. So it is possible that sometimes aggregates of Group I materials will have more compressive strength than those of Group II, resulting in the anomaly as above. Since Group A and Group B materials were carefully selected, their results show a logical order.

3.6.3 CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. TEN PERCENT FINES VALUE

A very good correlation is expected in this case as both the properties indicate resistance to crushing. The graph has a correlation coefficient of 0.8353 which is not a very good correlation. Although Group III aggregates, the weakest, have the lowest ten percent fines value, Group II, the strongest, does not have the highest ten percent fines value. This is again possibly due to the wide range of, and overlap over, the values of crushing strengths in the first two groups, as explained in Section 3.6.2.

3.6.4 CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. WATER ABSORPTION (AGGREGATES)

This graph has a correlation coefficient of 0.9665 which is a very good correlation. This indicates that the stronger the brick, the lesser is the water absorption of the aggregates prepared by crushing such bricks, and vice versa.

3.6.5. CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. RELATIVE DENSITY (AGGREGATES)

Average crushing strength of bricks of the five groups have been plotted against (i) Relative Density on an oven-dried basis, (ii) Relative Density on a saturated and surface-dried basis, and (iii) Apparent Relative Density. The graph average crushing strength of bricks Vs. Relative Density on an oven-dried basis has a correlation coefficient of 0.9907, that of average crushing strength of bricks Vs. Relative Density on a saturated and surface dried basis has a correlation coefficient of 0.9322, and that of average crushing strength of bricks Vs. Apparent Relative Density has a correlation coefficient of 0.8889. The first two have a very good and the third a good correlation.

The slope of the first two graphs is positive which indicates an increase in relative density with increasing crushing strength of bricks. But the slope of the third is negative indicating a decrease in apparent specific gravity with the increase in crushing strength. It appears from the second graph that the rate of increase in specific gravity on a saturated and surface dried basis with the increase in crushing strength, is very small, but the same relationship for specific

gravity on an oven dried basis is much more. Relative Density on a saturated and surface dried basis for Groups I, II, III, A and B are 2.07, 2.09, 2.02, 2.05 and 2.05 respectively and specific gravity on an oven dried basis for Groups I, II, III, A and B are 1.84, 1.88, 1.66, 1.85 and 1.77 respectively. This is possibly due to the fact that the aggregates from the strongest bricks have the least water absorption and vice versa.

Relative density was determined with aggregates passing a 19.05 mm sieve and retained on a 12.70 mm ($\frac{3}{4}$ " - $\frac{1}{2}$ " B.S.) sieve, as this size was the predominant size among the size range, 19.05 mm - 12.70 mm, 12.70 mm - 9.52 mm, 9.52 mm - 6.35 mm, 6.35 mm - 5.00 mm ($\frac{1}{4}$ " - $\frac{1}{2}$ ", $\frac{1}{2}$ " - $\frac{3}{8}$ ", $\frac{3}{8}$ " - $\frac{1}{4}$ " and $\frac{1}{4}$ " - $\frac{3}{16}$ ").

3.6.6 CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. DRY DENSITY AND BULK DENSITY OF COMPACTED AGGREGATES

Average crushing strengths of bricks for the three groups have been plotted against dry density and bulk density of compacted aggregates of respective groups and the best fit straight line for each has been found out statistically and drawn. The straight line of crushing strength Vs. dry density has a correlation coefficient, r , of 0.9922 and that of crushing strength Vs. bulk density has an r value of 0.99997. Both the coefficients indicate a very strong correlation between the variables. It shows in both cases that the weaker the brick the higher is the dry density and bulk density of the compacted aggregates.

Under the circumstances where the aggregates of all the groups have the same range of particle size, almost similar particle size distribution and having similar porosities, the reverse should be true.

This is because specific gravity of the particles on an oven-dried basis is the only variable in determining the dry densities, and specific gravity of the particles on a saturated and surface-dried basis is the only variable for determining bulk densities. These observed behaviours happened due to the fact that under the same compactive effort the weakest aggregates had undergone maximum crushing and thus become most dense, i.e. least porous and this has over-ridden the effect of specific gravity in both the cases. It indicates that more compactive effort is needed to reverse the phenomenon. So, if more compactive effort were applied to the stronger aggregates, more fines would be produced which would improve the packing.

3.6.7 CRUSHING STRENGTH OF BRICKS (AVERAGE) Vs. CALIFORNIA BEARING RATIO

This graph has a correlation coefficient of 0.9985 which indicates a strong correlation. It indicates an increasing CBR value with decreasing crushing strength. The CBR tests were done on the compacted aggregates in the 225 mm (9") diameter cylinder in which the aggregates were compacted by the Instron machine. Strictly speaking the test is not valid and has been done to find out the relative behaviour and in the hope that the CBR thus found out may have a relationship with the actual CBR of the aggregates of the three groups. This has been discussed in Section 5.5.3. Group III aggregates (which are the weakest) have the highest CBR value and Group II aggregates, the strongest, have the lowest CBR. But under the circumstances where the aggregates

of all the groups have the same range of particle size, almost similar particle size distribution and having similar porosities, the reverse should have happened. The porosities of the particles of the three groups (and as such densities) were not the same and this has been explained in Section 3.6.6.

3.6.8 AGGREGATE IMPACT VALUE Vs. WATER ABSORPTION (AGGREGATES)

Aggregate Impact Values of the aggregates of the five groups have been plotted against their respective water absorptions and the best fit straight line has been drawn. It has a correlation coefficient of 0.8813 which indicates a good correlation. This indicates that the strongest aggregates which undergo least crushing under impact have the least water absorption and vice versa. But the actual test results for Groups I and II are anomalous to this idea, and the reason for it has already been explained in Section 3.6.2.

3.6.9 TEN PERCENT FINES VALUE Vs. AGGREGATE IMPACT VALUE

Ten percent fines value of the five groups of aggregates have been plotted against their respective aggregate impact value and the best fit straight line has been drawn. It has a correlation coefficient of 0.9287, which indicates a strong correlation between the variables. In fact both the tests are very similar and indicate strength (or resistance to crushing) of the aggregates against static load and impact in the ten percent fines value test and aggregate impact value test respectively. The stronger the aggregates the lesser is the aggregate impact value, i.e. it undergoes lesser crushing, and the higher is the ten percent fines value, i.e. more

load is required to produce ten percent fines, and vice versa. Although the B.S. (812/1975) states that the results may be anomalous if the aggregate impact value is more than 30 which is the case in all the five tests, the results show a logical order for all the groups except Groups I and II. This is, again, possibly due to the wide range of, and overlap over, the values of the crushing strengths of Groups I and II materials and has already been explained in Section 3.6.2.

3.6.10 AGGREGATE IMPACT VALUE Vs. RELATIVE DENSITY - (i) On an oven-dried basis, (ii) on a saturated and surface-dried basis, and (iii) apparent

Aggregate impact values (A.I.V.) of the five groups have been plotted against corresponding relative densities - (i) on an oven-dried basis, (ii) on a saturated and surface-dried basis, and (iii) apparent. The graph A.I.V. Vs. Relative Density on an oven-dried basis has a correlation coefficient, r , of 0.8202, the one with A.I.V. Vs. Relative Density on a saturated and surface-dried basis has an r value of 0.5395 and the last one with A.I.V. Vs. Apparent Relative Density has a correlation coefficient of 0.9506. The third one indicates a good correlation but the first one does not indicate a good correlation. The second one indicates a poor correlation. However, these indicate the higher the relative density on an oven-dried basis and saturated and surface-dried basis of an aggregate, the lower is the aggregate impact value, i.e. the stronger is the aggregate and vice versa. The situation reverses with apparent relative density which means the lower the apparent relative density of an aggregate the lower is the aggregate impact value indicating a stronger aggregate and vice versa.

3.6.11 AGGREGATE IMPACT VALUE Vs. DRY DENSITY AND BULK DENSITY (COMPACTED AGGREGATES)

Aggregate Impact Values (A.I.V.) of the three groups have been plotted against corresponding dry and bulk densities. The best fit straight lines have been drawn. The graph Aggregate Impact Value Vs. Dry density has a correlation coefficient, r , of 0.7744 which is not a good correlation. The graph Aggregate Impact Value Vs. Bulk density has an r value of 0.8428 which is not a very good correlation. These graphs indicate that the lower the A.I.V., i.e. the stronger the aggregates, the lower the dry and bulk density value. But the reverse should be true under the circumstances where all the groups have the same, or similar, range and distribution of particle size and having similar porosities which has already been explained in Section 3.6.6.

3.6.12 TEN PERCENT FINES VALUE Vs. RELATIVE DENSITY - (i) On an oven-dried basis, (ii) on a saturated and surface-dried basis, and (iii) apparent.

The three graphs - (1) Ten percent fines value vs. Relative density on an oven-dried basis, (2) Ten percent fines value vs. Relative density on a saturated and surface-dried basis, and (3) Ten percent fines value vs. Apparent relative density have been drawn with test results of all the five groups of aggregates. The correlation coefficients are 0.8763, 0.6493 and 0.9328 for the 1st, 2nd and 3rd graph respectively. The third indicates a good correlation but the first and second do not, and the situation is very similar to that of Section 3.6.10. The higher the relative density on an oven-dried basis and/or saturated and surface-dried basis, the higher is the ten percent fines value, i.e. the stronger is the aggregate and vice versa.

The situation reverses with apparent relative density. The lower the apparent relative density, the higher is the ten percent fines value, i.e. the stronger is the aggregate and vice versa. This is due to the fact that the stronger aggregates have lesser water absorption.

3.6.13 TEN PERCENT FINES VALUE Vs. CALIFORNIA BEARING RATIO

The best fit straight line through the points obtained by plotting ten percent fines values of the three groups of aggregates against corresponding CBR values has a correlation coefficient of 0.8103 which is a good correlation. The straight line graph obtained indicates that the higher the ten percent fines value, i.e. the stronger the aggregate, the lower is the CBR value indicating a weaker aggregate. The reason for this anomaly has been explained in Section 3.6.7.

3.6.14 DRY DENSITY Vs. RELATIVE DENSITY - (i) On an oven-dried basis, (ii) on a saturated and surface-dried basis and (iii) apparent

The three graphs - Dry Density Vs. Relative Density, (i) on an oven-dried basis (ii) on a saturated and surface-dried basis, and (iii) apparent, have been drawn and have a correlation coefficient of 0.9838, 0.9975 and 0.9388 respectively, which indicate a very good correlation for the first two and a good correlation for the third. The third graph indicates the higher the apparent relative density the higher is the dry density and vice versa. The first and the second indicates that the higher the relative density both on an oven-dried basis and on a saturated and surface-dried basis, the lower is the dry density. But under normal and similar circumstances for all

the graphs, the reverse should be true. The reason for this anomaly has already been explained in Section 3.6.6.

3.6.15 DRY DENSITY (COMPACTED AGGREGATES) Vs. CALIFORNIA BEARING RATIO

Dry density values of the three groups of aggregates have been plotted against corresponding California Bearing Ratio values (CBR) and the best fit straight line through them has been drawn. It has a correlation coefficient of 0.9975, which indicates a very strong correlation between the two variables. The graph demonstrates the fact that the higher the dry density, the higher is the California Bearing Ratio and vice versa.

3.6.16 BULK DENSITY Vs. CALIFORNIA BEARING RATIO

Bulk density values of the compacted aggregates having a moisture content equal to the water absorption of the three groups have been plotted against respective California Bearing Ratios (CBR). The best fit straight line through the above points has a correlation coefficient of 0.9989. This indicates a very strong correlation between the variables.

It is demonstrated from the graph that the higher the bulk density of the compacted aggregates, the higher is the CBR and vice versa.

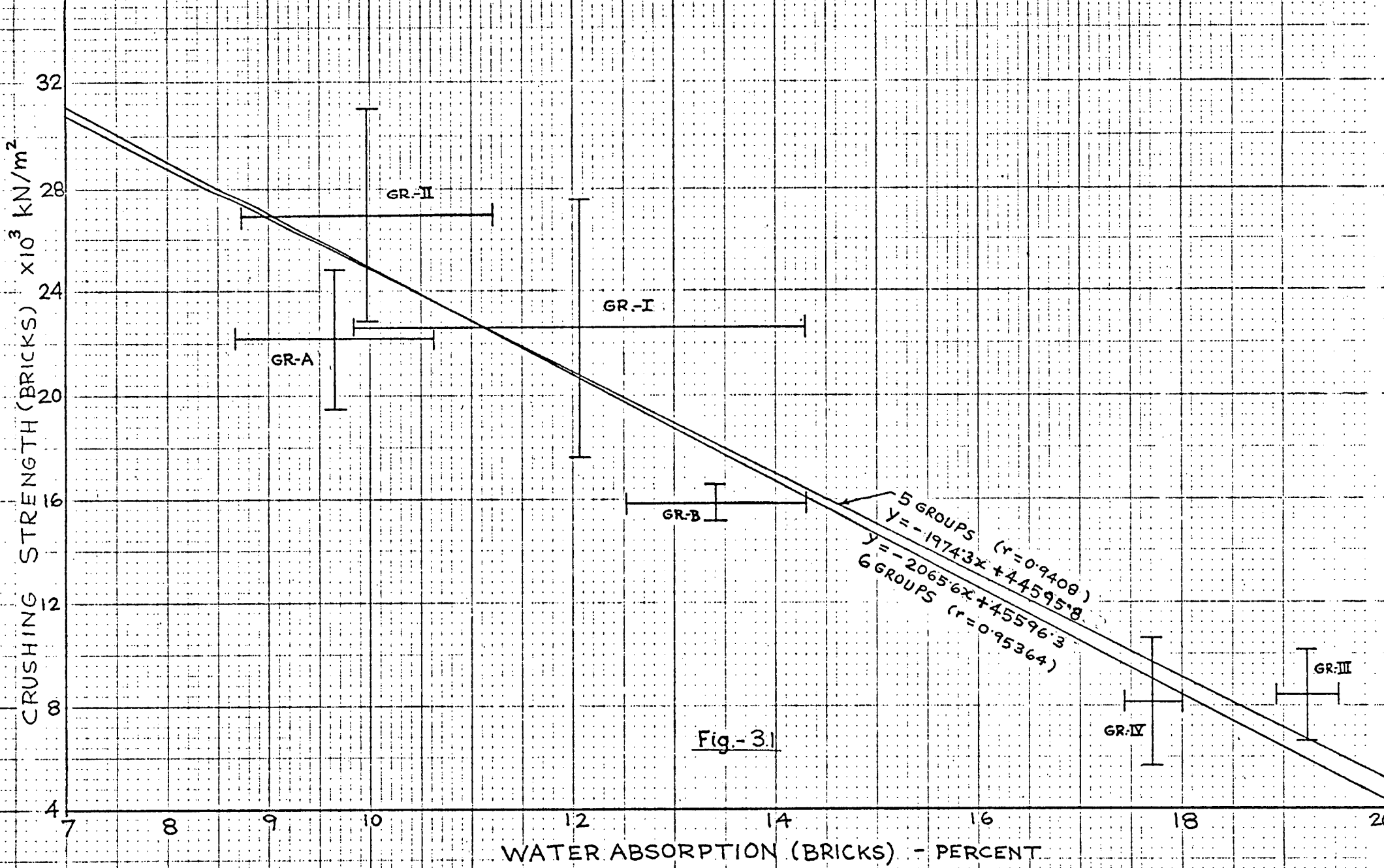
3.6.17 OVERALL DISCUSSION OF GRAPHS

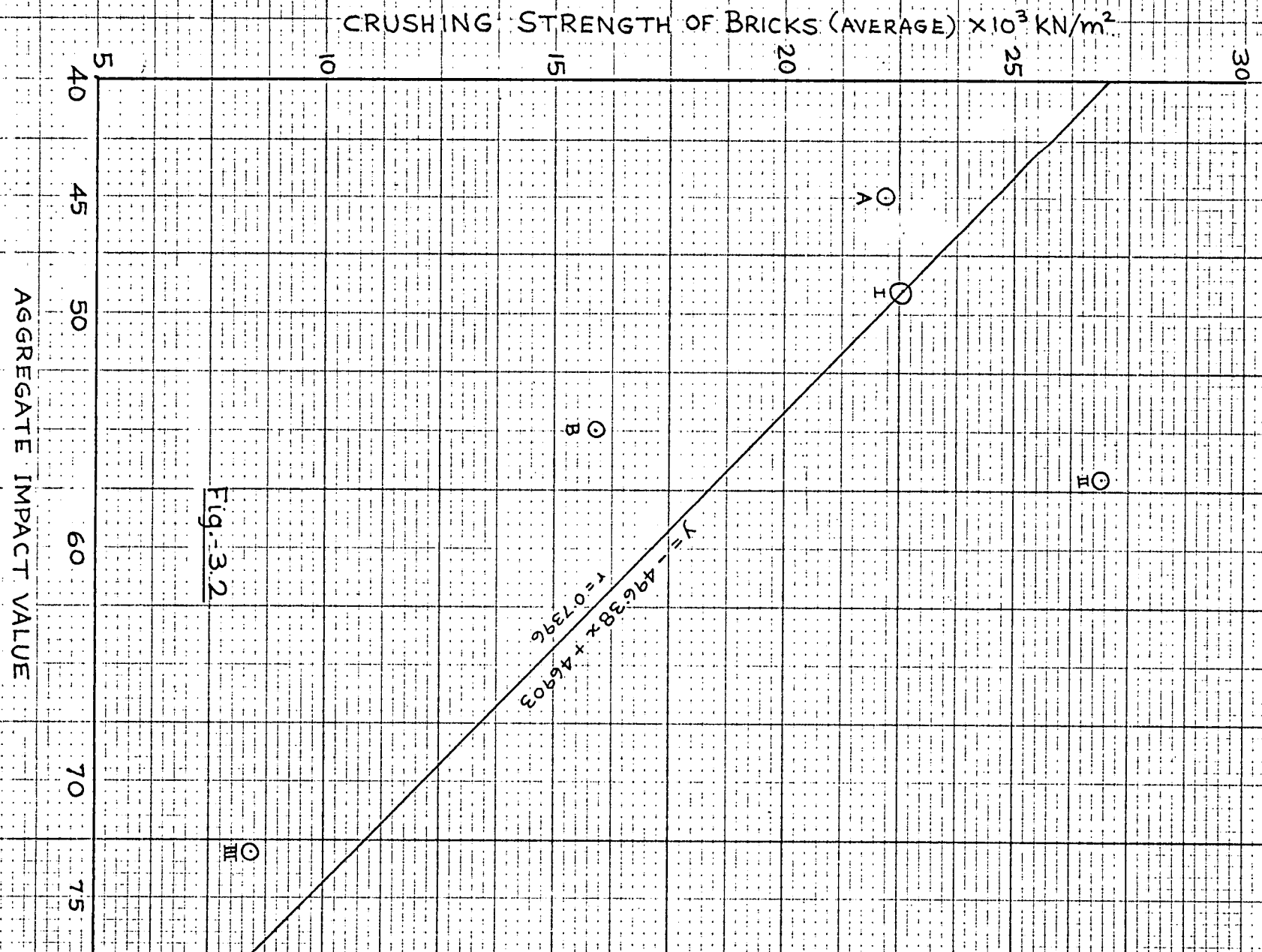
In the previous sections the intercorrelation between the various properties of the bricks and the aggregates of different groups has been discussed. Crushing strength of bricks, ten percent fines values,

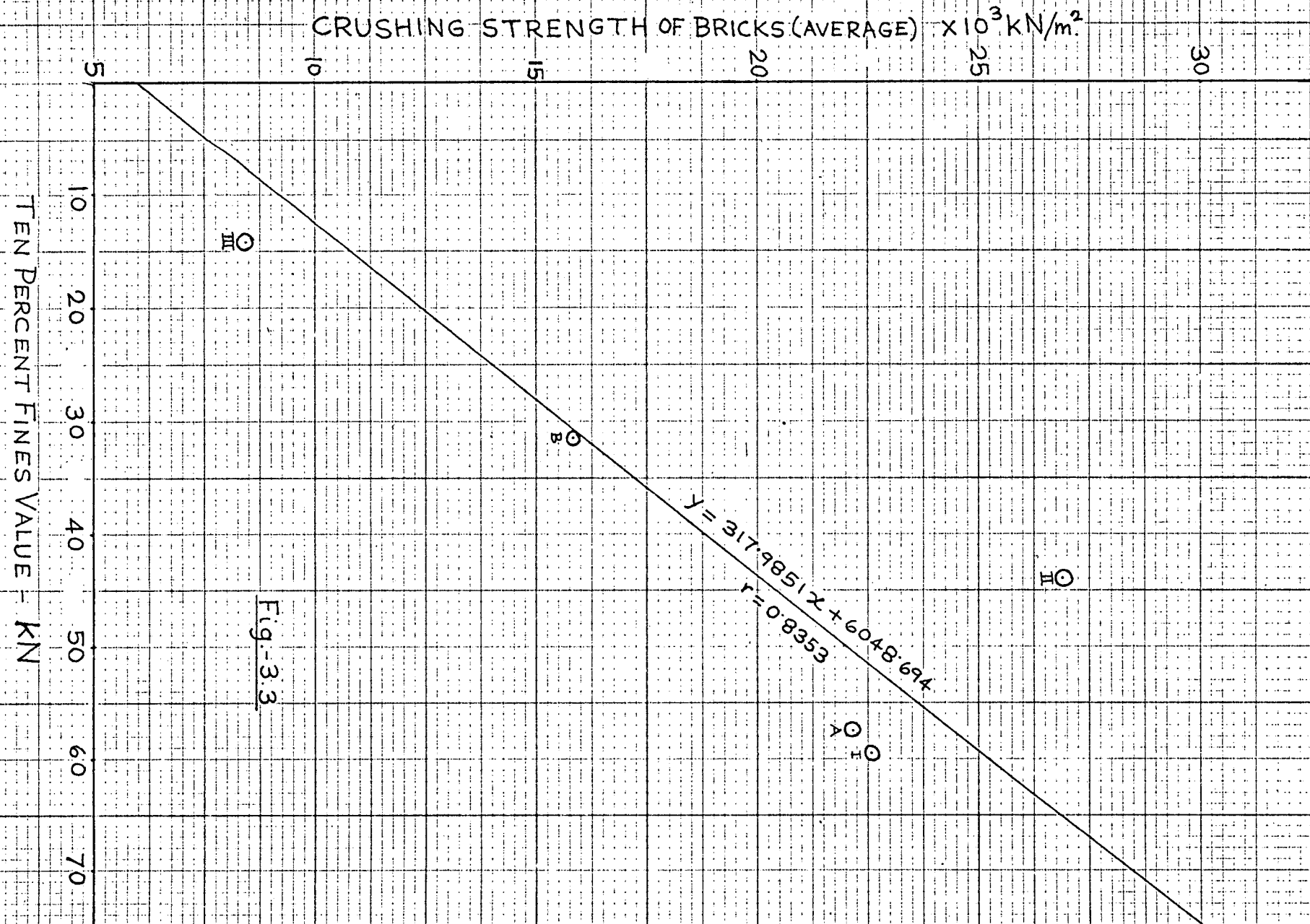
aggregate impact values and C.B.R. values of the aggregates are strength properties which reflect resistance to different types of loading (e.g. static loading, crushing, impact and shear). For the aggregate impact value a lower value shows a stronger aggregate whereas for the other properties a higher value shows a higher strength and vice versa. Relative densities and water absorption are physical properties of the aggregates. The stronger the brick or the aggregate, the lower is the water absorption. Relative densities on an oven-dried basis and saturated and surface dried basis are higher with stronger aggregates but apparent relative density is lower when the aggregate is stronger and vice versa.

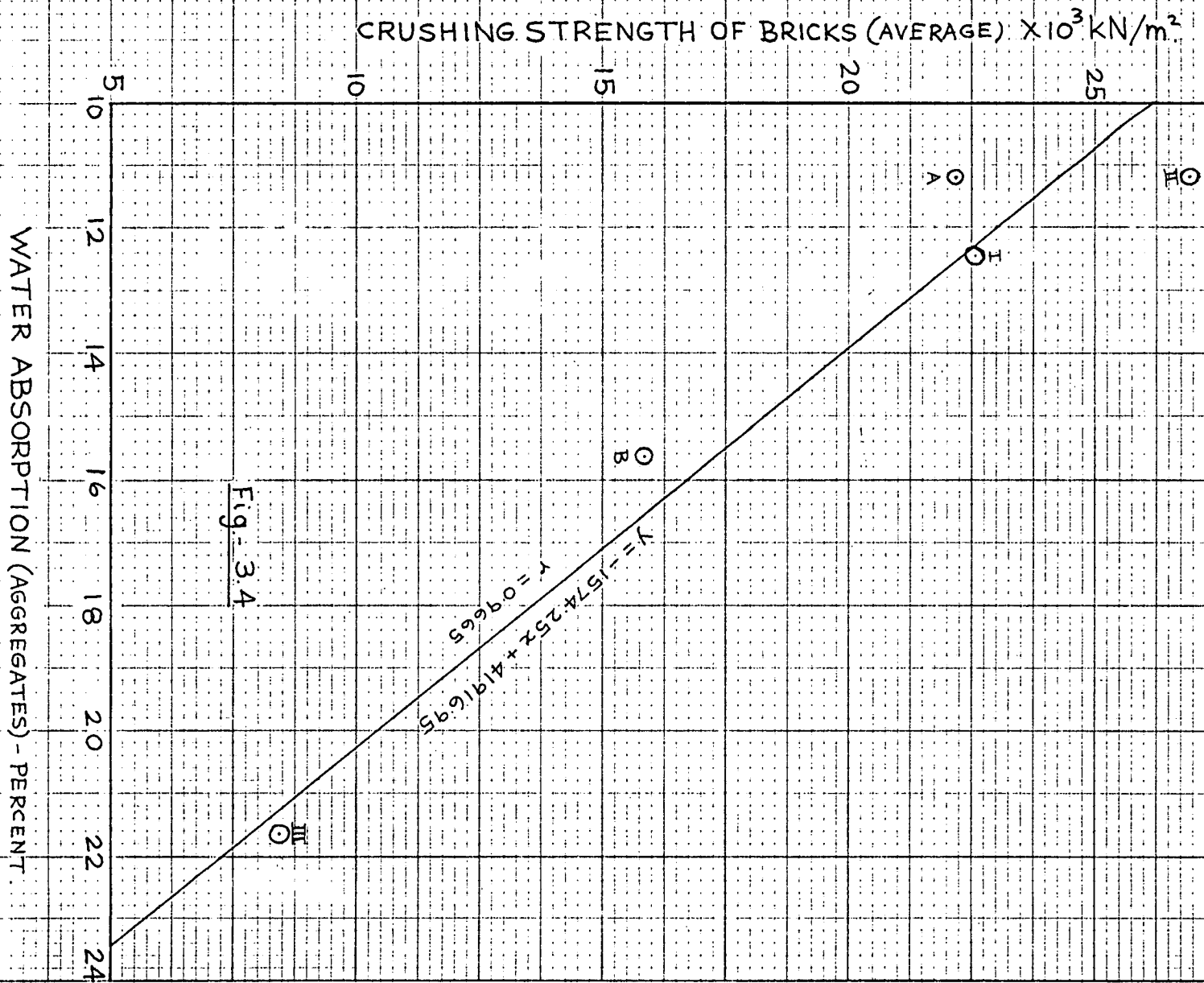
The California Bearing Ratio of the aggregates depend on the strength, compaction and grading of the aggregates. From the results of the C.B.R. tests performed on the aggregates of Groups I, II and III, it appears that the presence of extra fines after crushing due to the compacting effort increases the shear strength, i.e. C.B.R. The gradings of the three groups of aggregates also became denser. In the C.B.R. tests, due to the same compactive effort the weakest aggregates undergo maximum crushing and become most dense. As a result the highest C.B.R. is obtained for the weakest aggregates. Normally the strongest aggregates should have the highest C.B.R. One conclusion that can be drawn from this is that possibly the grading is optimal for the weak aggregates but more fines are required for stronger aggregates. One way of achieving this goal is to increase the compacting effort in order to obtain more fines, or simply include more fines in the stronger aggregates.

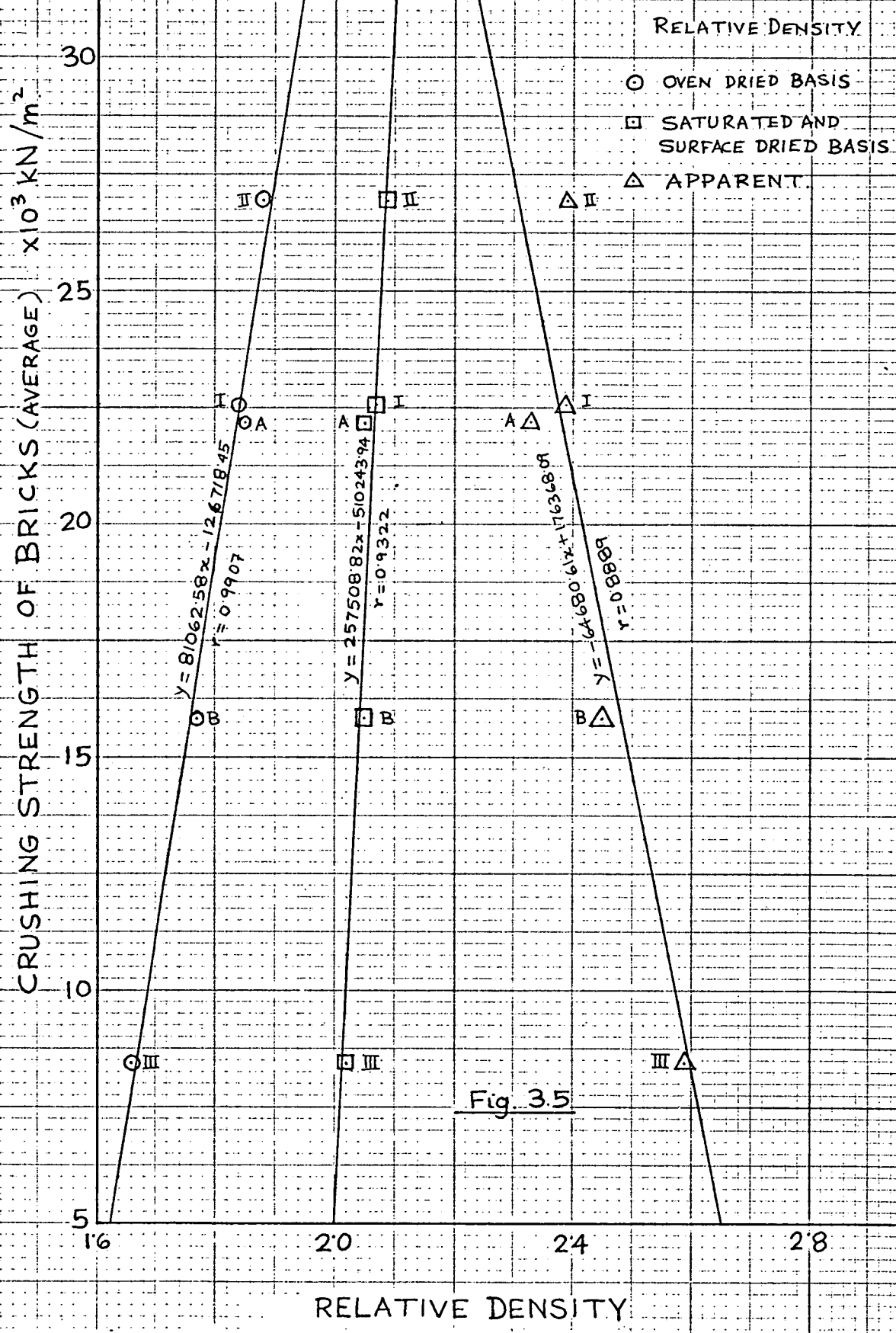
There is a correlation between all the properties but the degree of correlation varies from very good to fair. An exception is the case where relative density on a saturated and surface dried basis is correlated with strength properties, especially with aggregate impact value. This is due to the fact that the relative densities of the aggregates of the various groups change very little with change in strength.

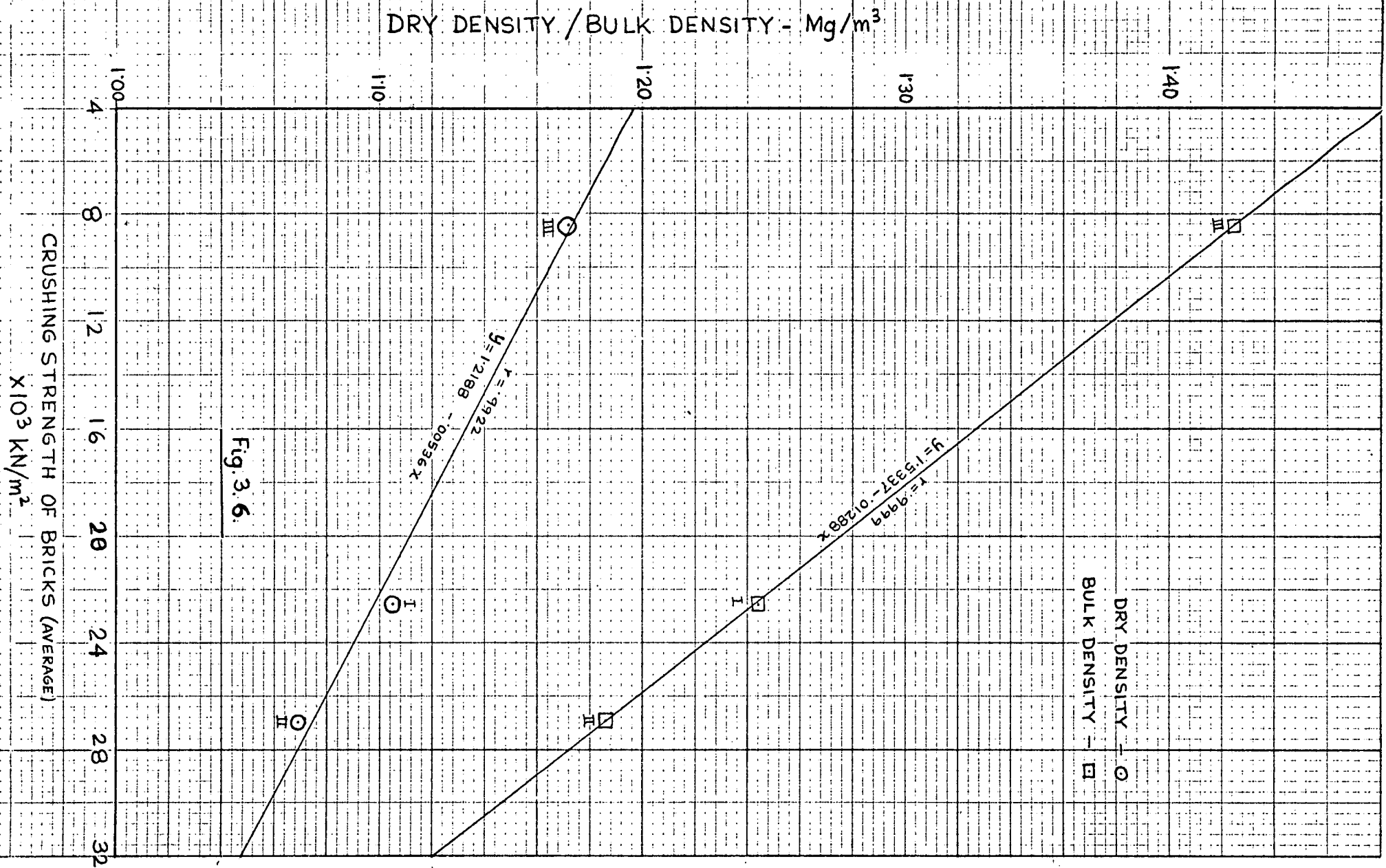


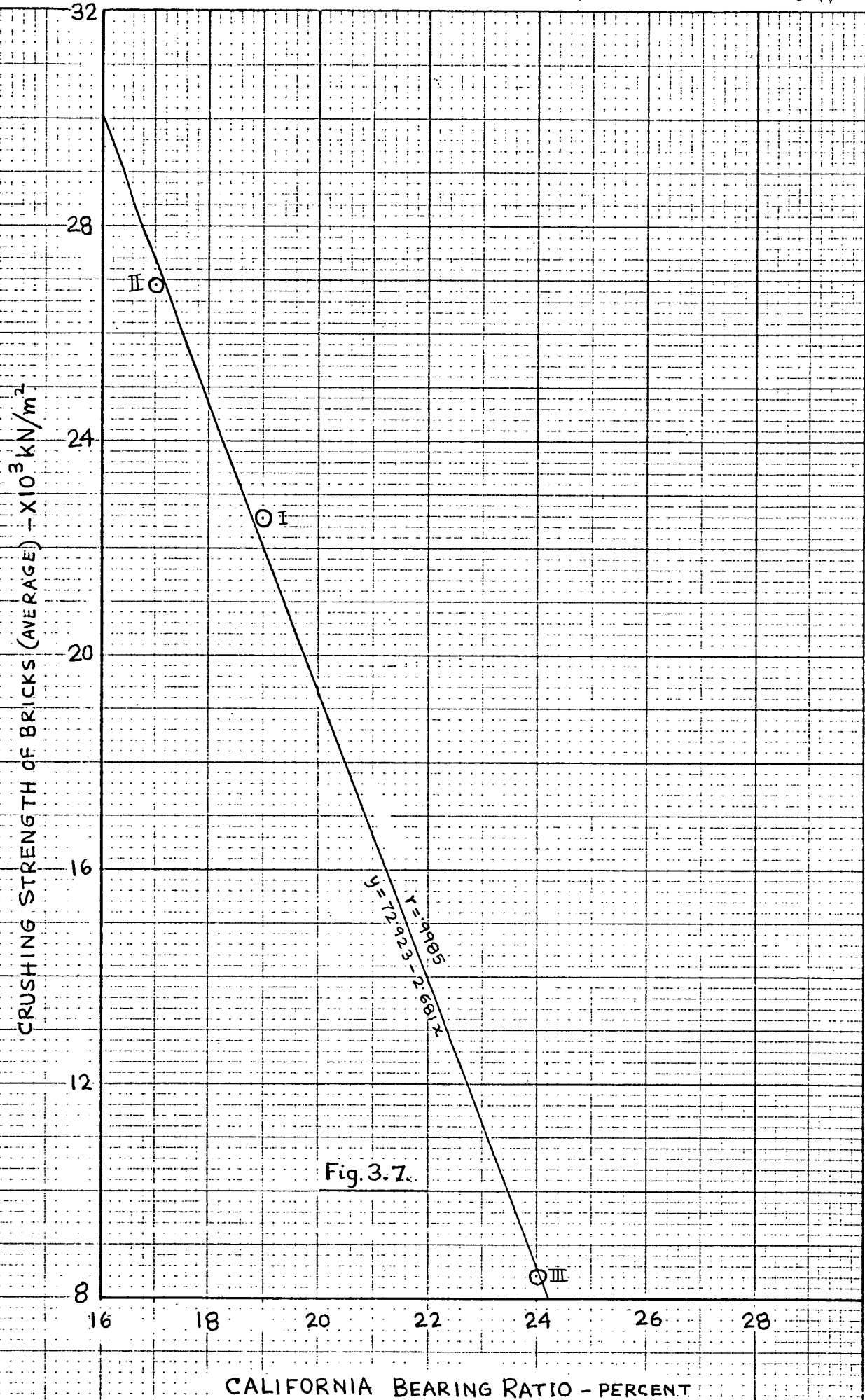












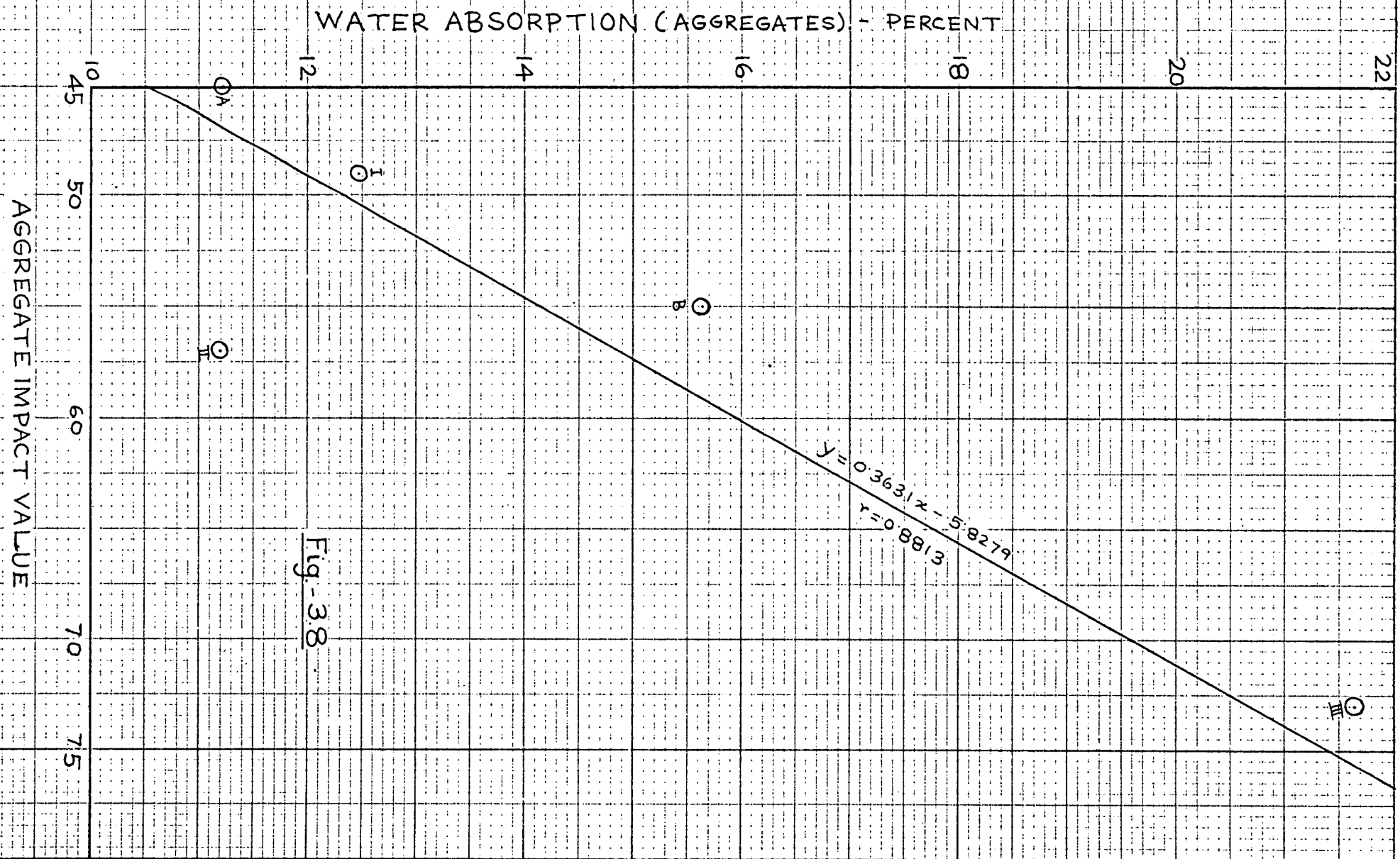


Fig-3.8

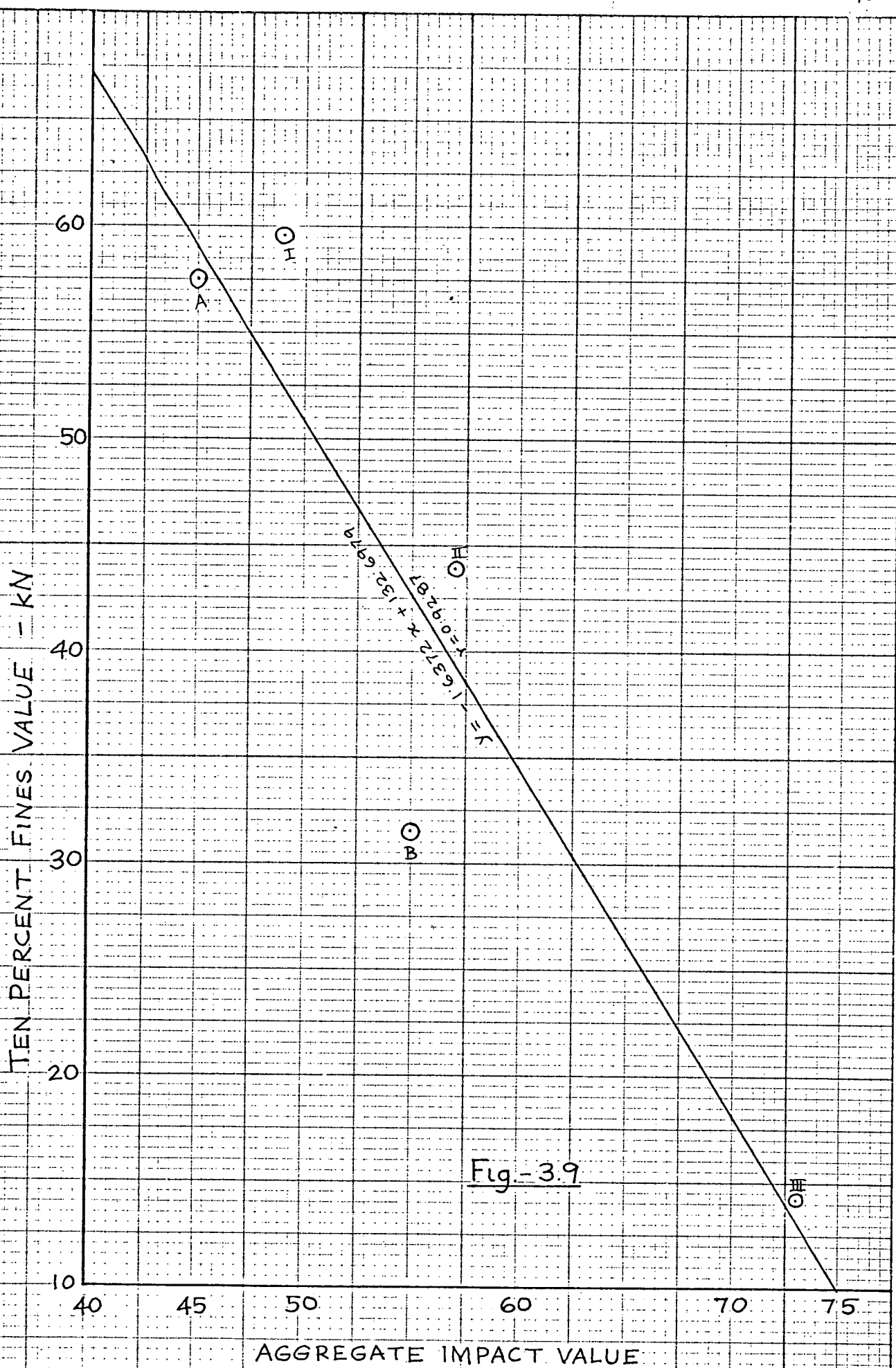
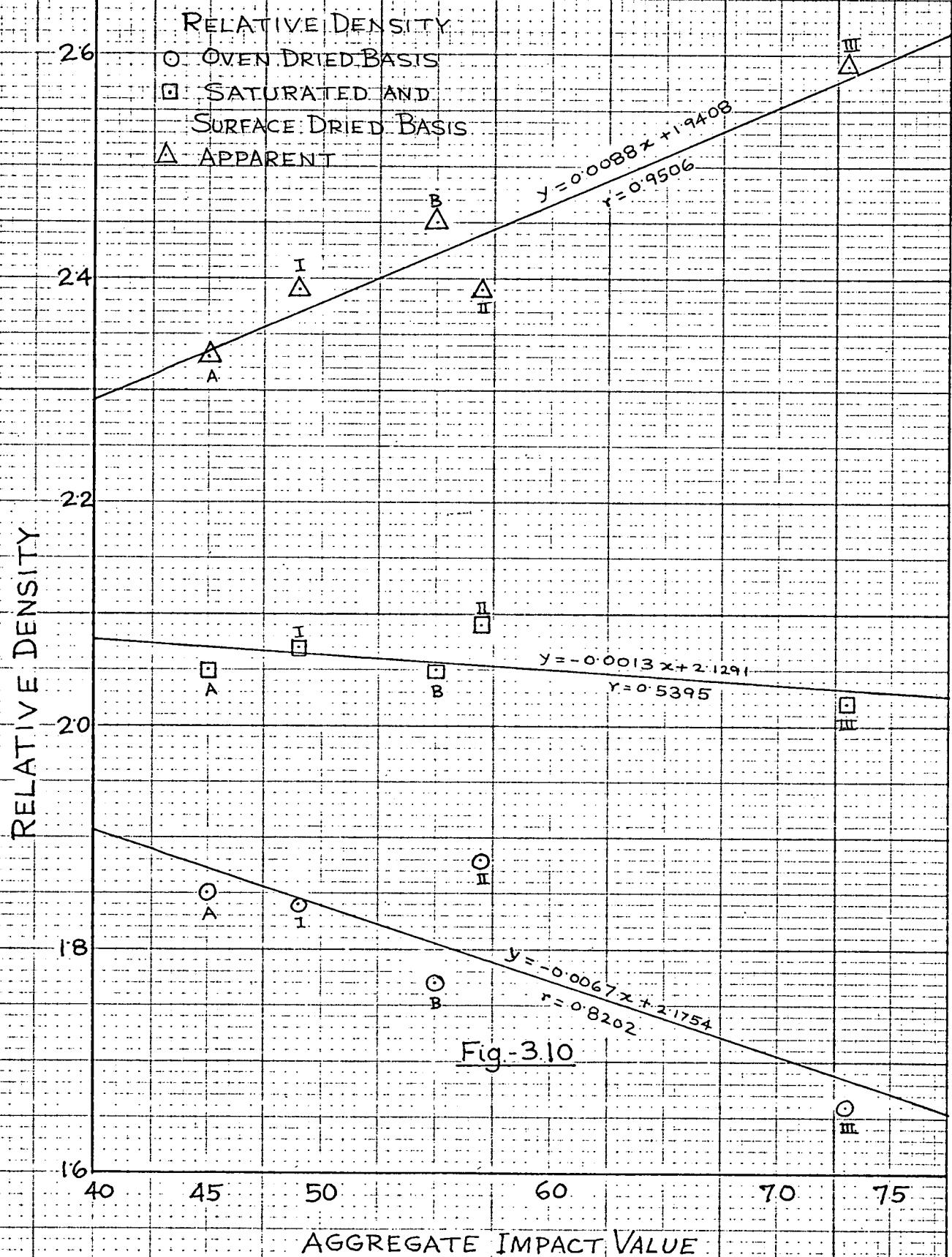


Fig.-3.9



DRY DENSITY / BULK DENSITY - Mg/m³

1.50
1.40
1.30
1.20
1.10
1.00

45 50 60 70 75

AGGREGATE IMPACT VALUE

DRY DENSITY - $\circ$

BULK DENSITY - $\square$

$r = .8428$

$y = .0086x + .77197$

$r = .7744$

$y = .00331x + .61769$

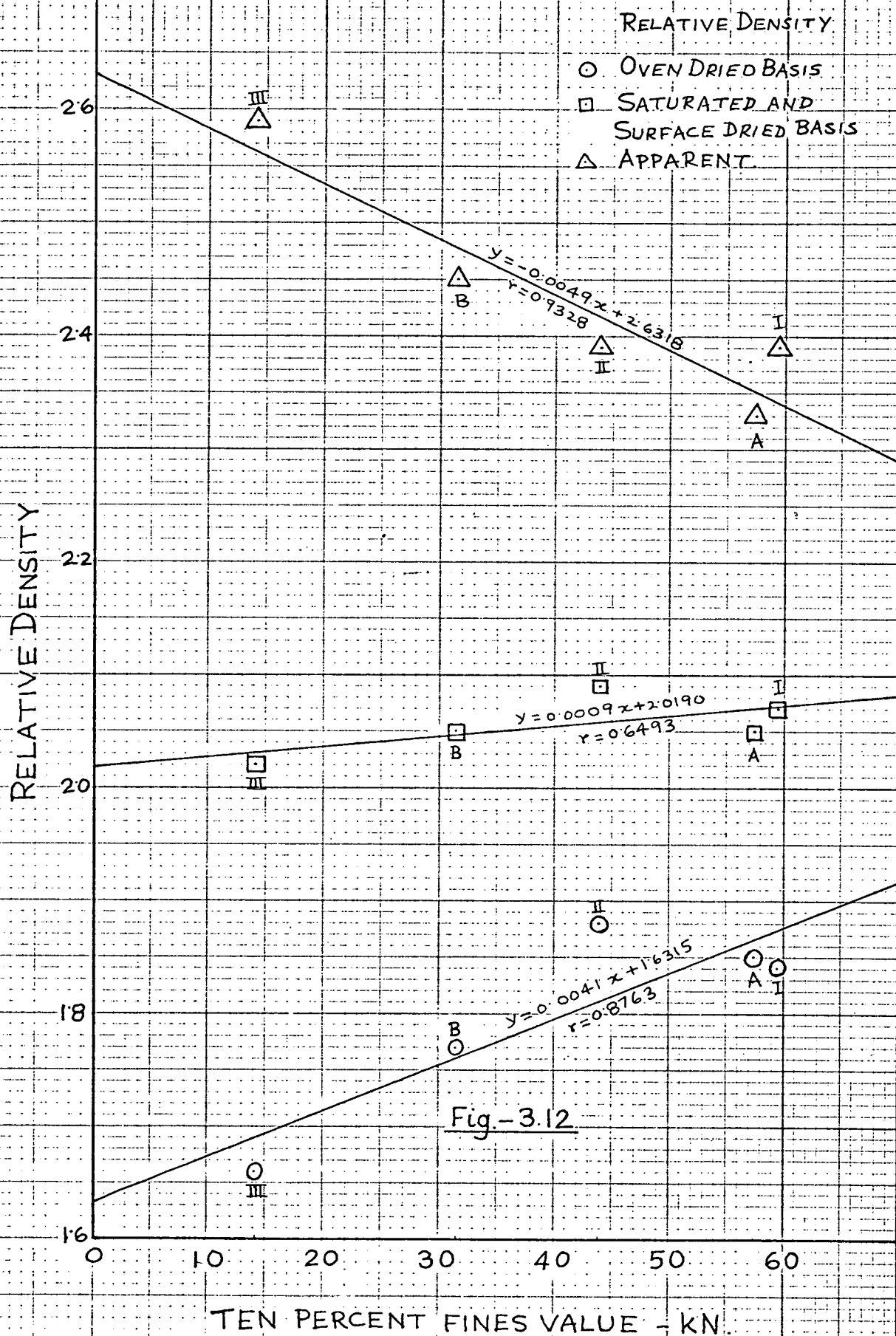
Fig. 3.11

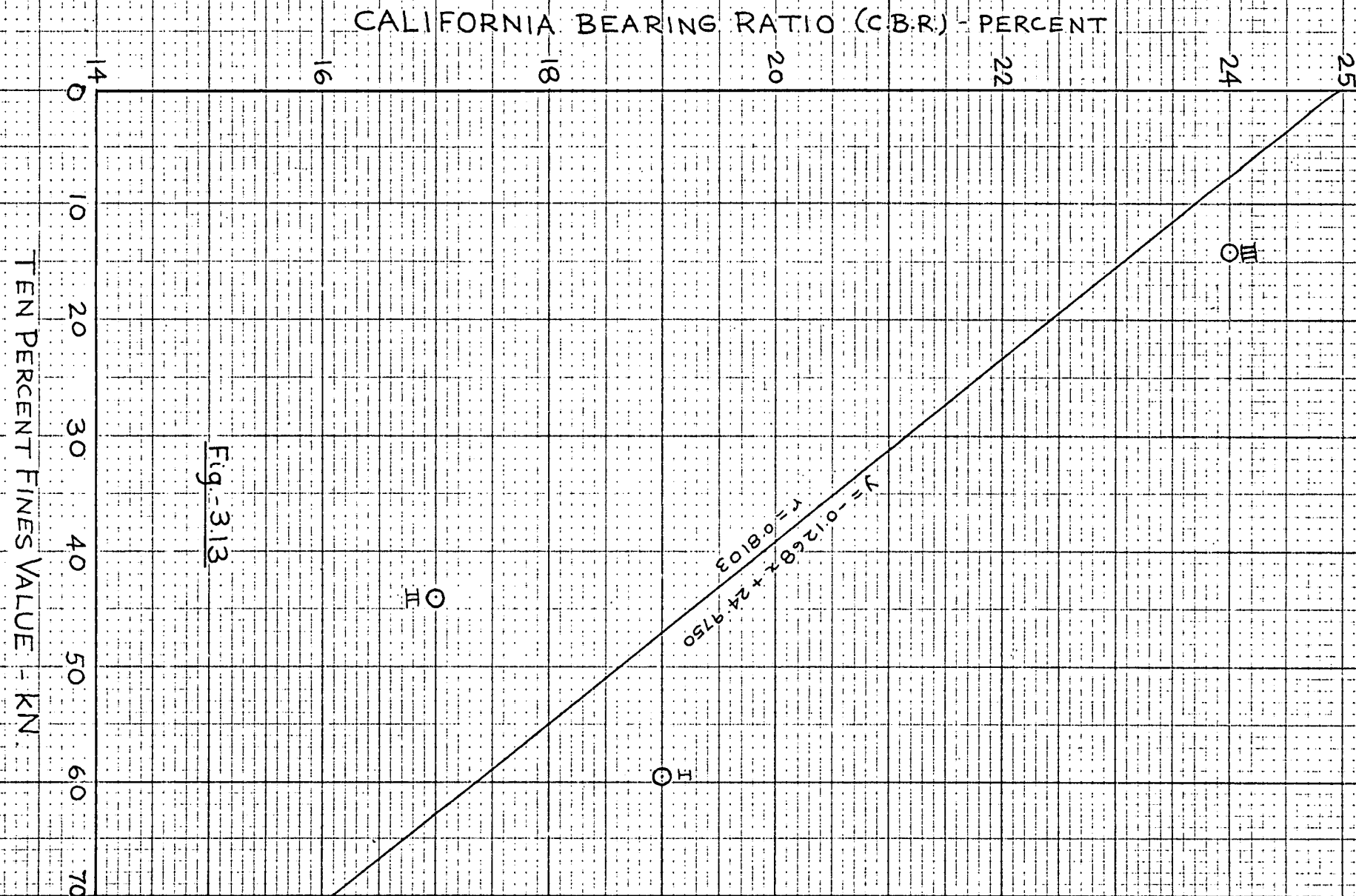
I

II

III

IV





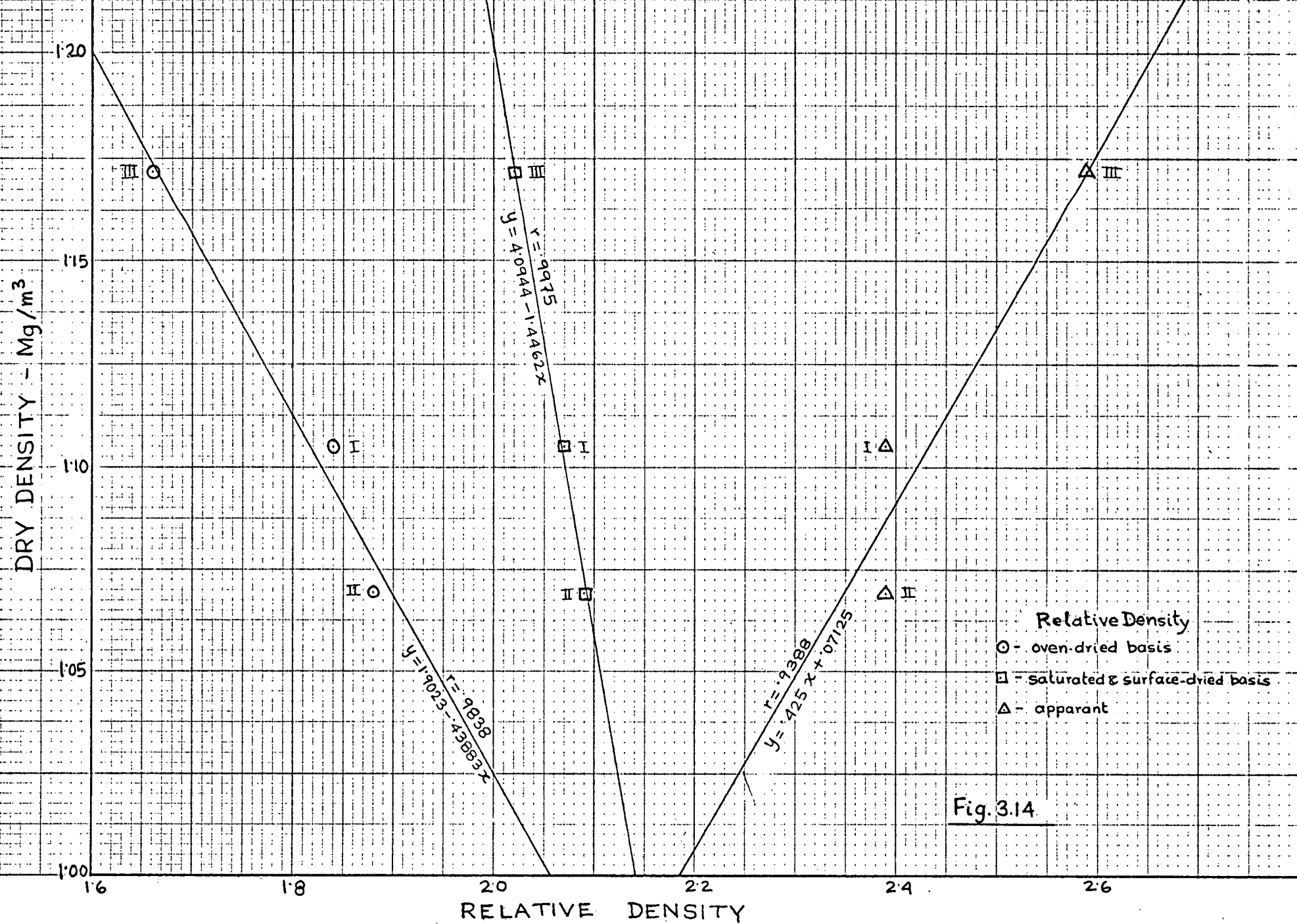
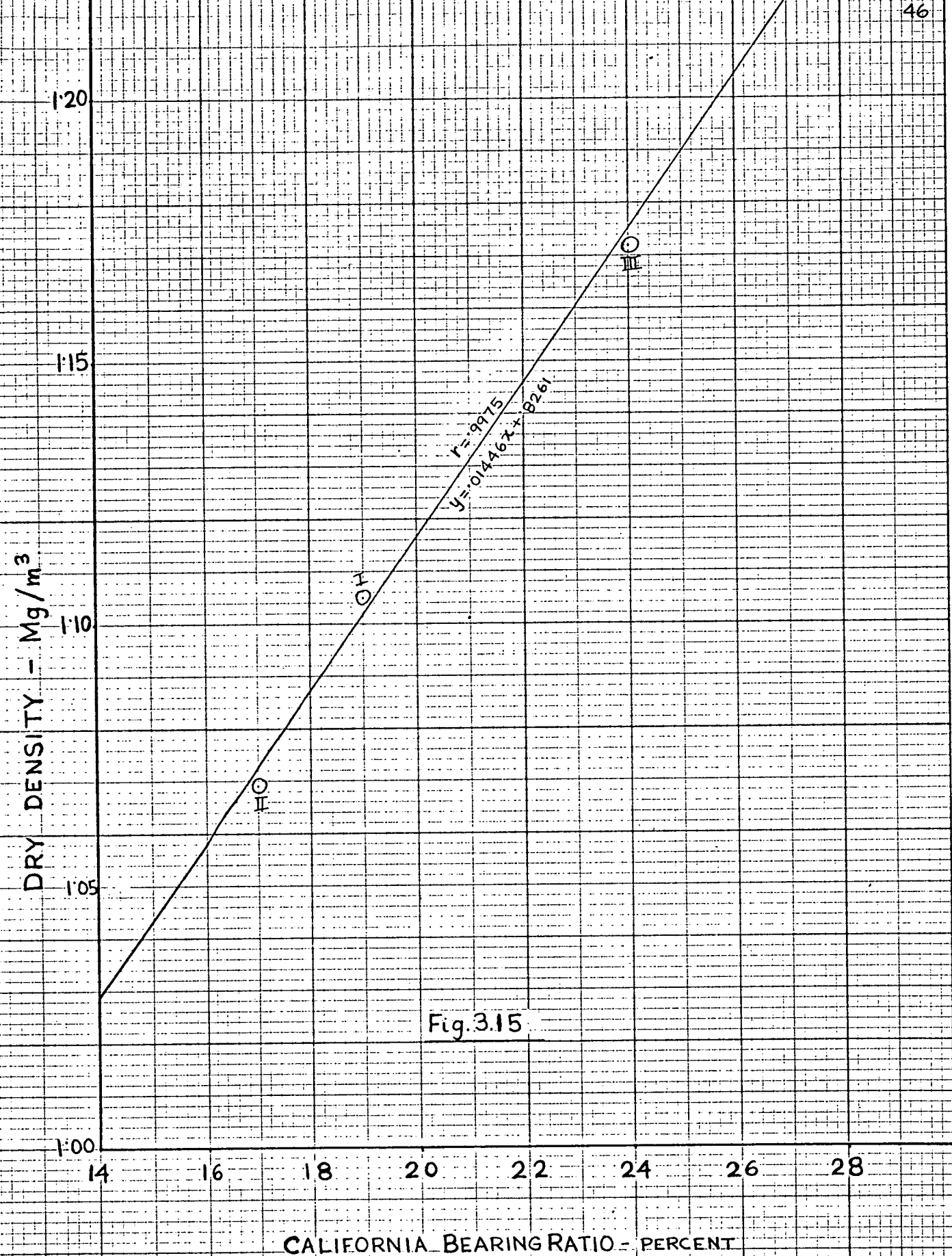


Fig. 3.14



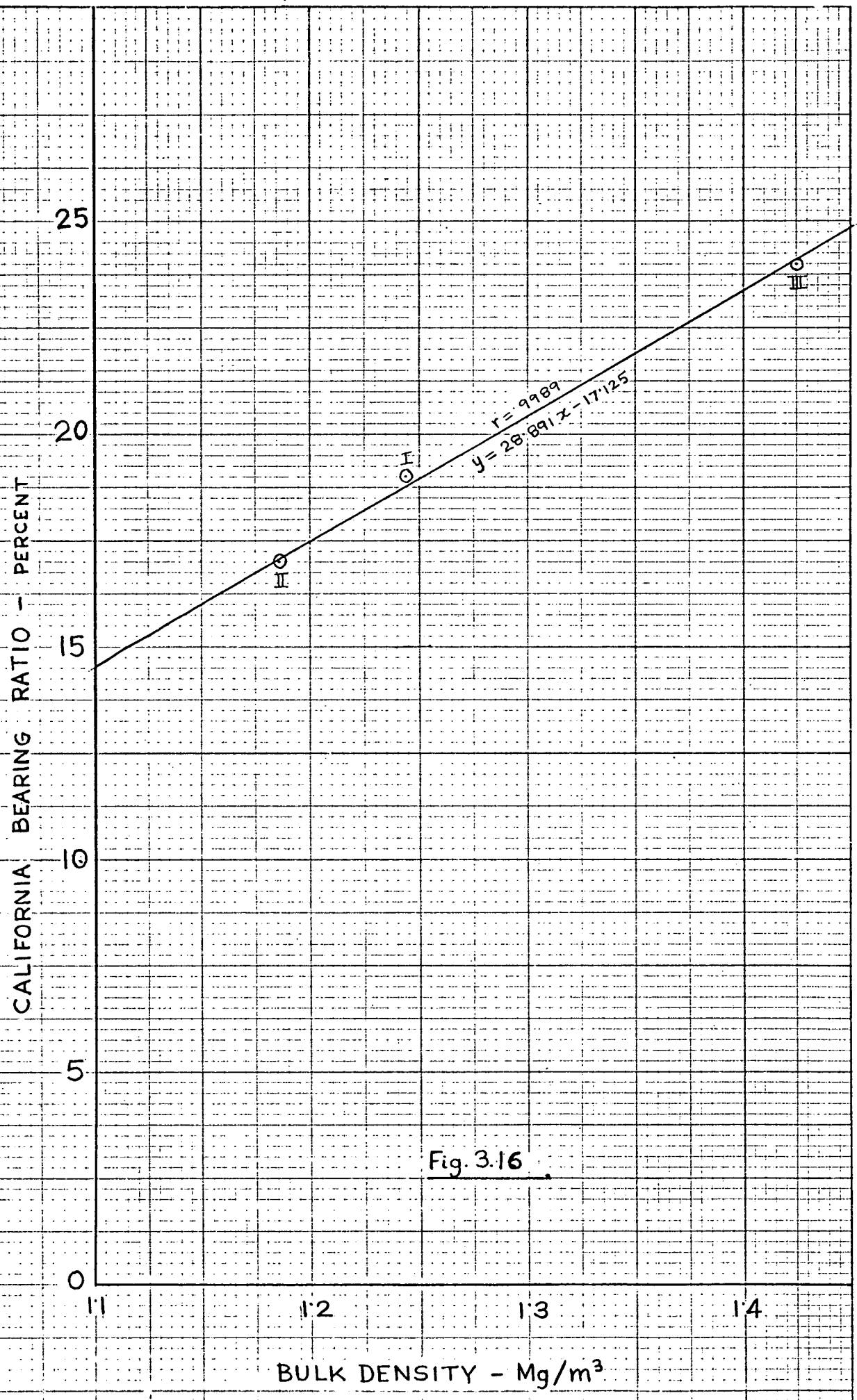


Fig. 3.16



PUNDIT

SOME OF THE TESTED BRICKS



PUNDIT

SOME OF THE TESTED BRICKS

CHAPTER 4

DEGRADATION OF AGGREGATES

4.1 INTRODUCTION

In the previous Chapter the engineering properties of brick aggregates and the intercorrelation between them have been discussed. In this Chapter the degradation of aggregates, a very important aspect, in the behaviour of the aggregates, especially if they are weak, is discussed.

Many engineering materials undergo degradation in their normal engineering use. Degradation of aggregates means the reduction in sizes and/or angularity of the particles and introduction of smooth surfaces in the aggregate mass. The aggregates of a flexible pavement, both bound and unbound, would develop contact stresses under the action of compacting forces or traffic loads. When the stress concentrations on the contact points are higher than the crushing strength of the aggregate particles forming the contact surfaces, cracks develop. This breakdown of the particles is termed crushing, disintegration or degradation. Some aggregates are highly susceptible to degradation and may show noticeably different engineering properties following degradation. Changes of this type in unbound aggregate bases have led to many highway failures.

The characteristic of this type of base failure in flexible pavement is initial longitudinal cracking followed secondly by "allegatoring" and finally by actual pot-holing of the pavement. The failure is not usually accompanied by rutting of any consequence. This type of failure has been reported by many research workers - Melville (1948) in Virginia, Scot (1955) in Oregon, Turner and Wilson (1956) and Minor (1959) in Washington State and Day (1962)

in Idaho in the U.S.A. In the majority of these studies, black basalts and gravels of Eocene age rock type were identified. A number of other rock types including other tertiary sandstones and shales and gravels of Mesozoic age were also identified. A layer of saturated highly plastic fines has practically always been found between the surface layers and the underlying base in the failed areas. In severe cases the paste of the plastic fines may be as much as 12 mm ($\frac{1}{2}$ ") thick. The aggregates containing secondary minerals degrade into plastic fines under the combined action of water and attrition (movement of rock against rock) as the pavement is flexed under moving loads, resulting in the slurry. The plastic fines permeate the entire base course quickly. After this occurrence the stability of the base course is reduced sharply and failure of the pavement cannot be prevented.

Although these failures are all related to the production of plastic fines, variation in stability may be caused by the degradation of aggregate particles without production of harmful fines, but leading to a change in grading and therefore, particle interlock. It is possible that this latter failure mechanism may be more common in brick aggregate pavements. As it is very difficult to sample unbound aggregate layers, the extent of these changes in the road environment is very difficult to estimate.

4.2 THE DEGRADATION PROCESS

There are both physical and chemical action on the aggregate particles in the degradation process. The original composition of the material and the extent of weathering that the material has been subjected to ascertains the extent and the rate of occurrence

of physical breakdown of the particles under certain stress conditions. Disintegration or degradation of the aggregate particles appear to proceed as follows. Firstly, as a crushing load is applied, the load is transmitted to an aggregate particle through its protruding points, if any, resulting in the crushing of these points and the formation of contact areas. With the increase of the load, either the flexural or the crushing strength of the aggregate particle is exceeded and a crack is formed resulting in the breakage of the particle. The degradation process may constitute the following:

- (1) The splitting of particles (into approximately equal parts).
- (2) The breakage of angular projections which may exist on the particles.
- (3) The crushing, shearing or grinding off of small scale asperities on the major faces or planes of the particles.

Among the three cases, the differences are mainly in the ratio of the size of the broken product to that of the original particles. But all are dependent on the same shape and strength characteristics and on the kind of stress to which the particles are subjected to. The relative importance of these may differ in the three cases. For example, parallel sided slab-like particles and non-crushed high sphericity rounded gravels will undergo only type 1 degradation. Angular cubical particles and tapering rod, disc and blade shape particles will undergo type 2 degradation before any type 1 degradation occurs. Type 3 degradation will be experienced by weakly cemented sedimentary rock particles and surface weathered rock



particles - for example mineral aggregates which have been stock piled for long periods or which have suffered weathering over geological time from multiple closely spaced joint planes exploited in the quarrying process. The fatigue process of repeated stressing at small strains is likely to produce this kind of surface degradation. Attrition of this kind may ultimately degrade the particles to their individual crystal or grain sizes. All the three types of degradation are likely to be experienced by natural aggregates like crushed rock as well as artificial aggregates like broken bricks. Brick aggregates being weaker compared with crushed stone aggregates will undergo more degradation. The degradation also depends on the shape and size characteristics of the aggregates. Hand broken brick aggregates are mostly cubical and highly angular, but all other shapes are also found in a large sample. Bricks burnt in kilns have a very wide range of crushing strengths depending on the amount of heat they consume in the burning process. Moreover, when a single brick burnt in the kiln is broken into aggregates, all the aggregates would not have the same strength even if they could be broken into exactly the same shape and size. This again is due to the same reason. This variation in strength from brick to brick and from aggregate particle to aggregate particle of the same brick contributes to the degradation process of brick aggregates.

The first effect of degradation leads to a reduction in (1) the particle interlock and (2) interparticle surface friction among the original particles. These are the two main factors of stability, i.e. resistance to deformation under load. Secondly, the smaller fragments produced as a result of degradation fill in

the voids within the aggregate structure resulting in a denser aggregate mass. This results in producing additional points of interparticle contact within the aggregate structure at which surface friction may be mobilised.

In short, the physical significance of degradation of an aggregate structure depends on the relative importance of these two changes mentioned above. Stability of the aggregate structure decreases if the effect of the additional number of contact points formed by the disintegration is less than the effect of reduced particle interlock, but increases if the situation is reversed.

This observation of Kennedy (1974) has also been found true in the present study as will be observed in Chapter 5, part 1.

In a situation where three groups of aggregates having similar sizes of particles, similar particle size distribution and similar packing under compaction, but different relative densities and crushing strength, then their densities (dry) would depend on the relative densities. Group II aggregates being the strongest and having the highest relative density should have the highest dry density. Group III aggregates being the weakest and having the lowest relative density should have the lowest dry density. But the test results indicate the reverse. This is possibly due to the fact that under the same compactive effort Group III being the weakest has undergone maximum crushing and the effect of additional contact points has been more than the effect of reduced particle interlock, etc. But in Group II the reverse might have happened. Hence the dry density of Group III has been found to be the highest, and that of Group II, the lowest.

4.3 MATHEMATICAL SOLUTION TO THE THEORY OF DEGRADATION

Research workers applied two largely empirical theories, one put forward by Rittenger (1867) and the other by Kick (1885) to the crushing of brittle material to account for the energy required to crush them until fairly recently. The Kick Theory is considered more theoretically sound, whereas the Rittenger theory is favoured by experimental evidence in the coarse particle range obtained by Gaudin (1926), Shelburne (1940) and Yashima (1970).

According to Rittenger the energy required to grind a substance goes largely into supplying the additional energy associated with the new free surface. This means that the energy to grind a given volume varies directly as the change in the square of the mean particle size, $E \sim (D^2)$. But Kick's theory is based on the concept of strain energy to cause rupture. His law requires the energy to vary as the cube of the mean particle size, $E \sim (D^3)$. The failure of Kick theory in the coarse particle range is probably due to the fact that it does not allow for the existence of flaws and expects the material to exhibit full theoretical strength of a perfect structure. A third theory has been put forward by Bond (1952) fairly recently.

His theory is based primarily on the strain energy absorbed by the particles. He acknowledged the existence of failures along cracks by considering the strain as proportional to D^2 prior to failure and D^3 during failure. His theory can be stated as "the total useful work in breakage which has been applied to a stated weight of homogenous broken material is inversely proportional to the square root of a diameter of the product particles, $E \sim (D^{-1/2})$ ".

The following explains the apparent discrepancy between the theories of Rittenger and Bond. The relation between energy and size reduction of a brittle solid may be written in general form as:

$$dE = \frac{-c}{x^n} dx, \text{ where}$$

dE = infinitesimal energy change

c = a constant

dx = infinitesimal size change

x = object size

n = a constant

According to Charles (1957), experimental results suggest values for the exponent n between 1.32 and 2.4, depending on the method of crushing of the material tested. Rittenger's theory has an exponent of 2, whereas that of Bond an exponent of 1.5. Charles concludes that these theories are only applicable to specific cases of crushing, grinding and type of material. Surface area may be a convenient measure of degradation if Rittenger's theory is considered as approximately correct for the work in the coarse particle range.

The mathematical solutions to the theory of degradation discussed above is very difficult to apply in the case of the brick aggregates used in the present study as the aggregate particles range from passing through 50.80 mm sieve and retained in 38.10 mm sieve to passing 75 microns sieve and the Kick's law requires the material to exhibit full theoretical strength of a perfect structure, which may not be the case with brick aggregates. Hence, although

these theories have been included in the discussion for completeness they are not altogether relevant to the brick aggregate used.

4.4 THE MEASUREMENT OF DEGRADATION

A number of investigations have been undertaken into the degradation behaviour of mineral aggregates under the compactive effort and traffic load. Most of the investigators have performed sieve analysis before and after the load application, plotted grading curves and showed that degradation occurs leading to the change in grading. Some investigators have considered the area between the two curves as a measure of degradation. But, so far, very little has been achieved to form an acceptable criteria for the measurement of degradation.

Recent researchers like Kennedy (1974) consider surface area determination to be a precise measure of degradation. There are a variety of techniques for measuring surface area. But once it has been decided to measure degradation by surface area, the next problem is to decide whether the 'bulk' surface area, i.e. neglecting the surface area of small internal cracks and fissures, or the 'absolute' surface area, i.e. including the area of cracks and fissures, is to be determined. Sometimes the appropriate surface area (bulk or absolute) to be determined, is decided intuitively. For example, to determine the amount of binder needed in a bituminous macadam the measurement of 'bulk' surface area of the aggregate is needed provided that no absorption of the binder occurs. This can be decided intuitively. But the problem of considering a measure of degradation

in unbound granular material is more complicated because neither the 'bulk' nor the 'absolute' surface area can be adopted intuitively. The mathematical solution to the degradation process (discussed earlier) states that the net crushing energy applied to a given aggregate structure is a direct function of the new surface area produced. Hence this theoretical argument is in favour of the absolute surface area. Some researchers suggested that experimental results confirmed the same. When a load is applied to the aggregate structure, new surface areas develop as a result of cracks including internal cracks and they represent useful work applied to the aggregate structure. Kennedy (1974) considers it incorrect to neglect the surfaces produced as a result of the internal cracks as less energy will be required for the subsequent splitting of the particles along these cracks.

4.5 DEGRADATION STUDIES

Although a number of studies have been carried out for the measurement of degradation, very few were dedicated to find out correlation between laboratory and field crushing. One of these works is that of Shelburne (1940) who was mainly concerned with the crushing resistance of surface treatment aggregates. He determined the crushing by 5 and 10 ton rollers on both flexible and rigid bases, analysed correlation between the Los Angeles abrasion machine and road rollers and investigated the effect of gradation and bituminous coating on the amount of crushing.

Shelburne observed, "The degradation of aggregates under

conditions of mixing, rolling and traffic as well as in the Los Angeles abrasion machine, approaches a Fuller's maximum density curve as an ultimate. This fact suggests a desirable trend in the design of surface treatment mixtures, towards longer gradings which approach maximum density. From the stand point of degradation, rolling should be minimised and a denser grading selected in the design rather than obtaining of the ultimate grading by crushing under rolling and traffic".

Ghani (1969) is in disagreement with the generalization of the first statement above. Degradation of an aggregate designed with Fuller's grading would have been the least as suggested by the said statement. But Ghani did not find it so while making some comparative degradation studies. However, he suggested "that it would be more appropriate to state that each aggregate under mixing, rolling and traffic, undergoes crushing in such a way so as to approach a maximum density (or minimum porosity) grading as an ultimate, which may be given by Fuller's density curve in some cases, and some other representative curves in others". He, however, agrees that there is a necessity to minimise traffic compaction, with low strength aggregates in particular, and of adopting denser gradings. It was also concluded by Shelburne that the Los Angeles abrasion machine at 100 revolutions gives a good indication of the amount of degradation under road rolling, but it is significant to notice from his results that the field roller affected mainly the large particles without producing much dust, whereas the rattler produced more dust as it acted on all particle sizes. The limitation of his inference is

that he based his conclusions on the study of one open grading. No other tests to indicate the relationship between field crushing and various laboratory methods of compaction were carried out.

Studies conducted by Moavenzadeh and Goetz (1963) later were restricted to laboratory investigations only. They also concluded that the gradation of the aggregate mixture is the most important factor controlling degradation, and the denser the grading, the lesser is the degradation. Other factors controlling the degradation were physical characteristics like Los Angeles value, size of aggregates and compacting effort.

Salehi (1968) investigated the extent of crushing undergone by a conventional B.S. 594 : 35 percent stone content mixture when subjected to various methods of compaction. He based his results on the shape of the particles, viz. flaky and non-flaky and his results are reported in the form of average degradation factors (percentage increase in the surface area of the coarse aggregate particles) and are reproduced below.

METHOD OF COMPACTION					
Field Roller (7 passes of a 6 ton steel wheel roller)		Impact (50 drops of Marshal on each side)		Static (15 tons at 2 Tons/min)	
<u>Flaky</u>	<u>Non-flaky</u>	<u>Flaky</u>	<u>Non-flaky</u>	<u>Flaky</u>	<u>Non-flaky</u>
23.4	15.5	54.1	23.6	87.2	63.1

From the above results, Salehi came to the conclusion that both impact

and static methods of compaction overdegrade the particles resulting in degradation factors considerably higher than those observed with the field roller. The above figures refer to a particular aggregate used with a particular grading and hence may not be generally applicable, but may be used as a first approximation for general comparative purposes assuming that type and gradation of the aggregate do not appreciably affect the relative degradation in the three methods of compaction studied.

Lately Kennedy (1974) studied degradation of unbound base material. He measured specific surface of different sizes of aggregates by a gas adsorption method known as the dynamic nitrogen adsorption method, which was developed by Nelson and Eggertson (1958). He covered sieve size ranges 9.52 mm - 6.35 mm to 112.5 microns - 75 microns ($\frac{3}{8}$ " - $\frac{1}{4}$ " to No. 150 - No. 200). He plotted log of sieve size against the mean value of specific surface per gramme of the aggregates and he extrapolated the curve for the size ranges 19.05 mm - 12.70 mm, 12.70 mm - 9.52 mm ($\frac{3}{4}$ " - $\frac{1}{2}$ ", $\frac{1}{2}$ " - $\frac{3}{8}$ ") and passing 75 microns (No. 200). His results of surface area when compared with those obtained for a crushed granite by Shacklock and Walker (1957) show that his areas are between 15 to 20 times larger than those obtained by them. This, he considers, is due to the fairly crude technique (permeability) employed by them, the smaller effect of particle shape and packing, and inclusion of areas produced by cracks and micro-surface texture by his method which the permeability method does not measure.

In the present work, degradation has been shown by (i) plotting grading curves before and after load application (ii) plotting per-

centage increase or decrease in each sieve fraction after load application, (iii) plotting aggregate sieve size ratio of the original grading vs. that of the final grading. (Figs. 4.1 - 4.9).

4.6 PREDICTION OF CHANGES IN GRADING

Since degradation occurs in the aggregate mass under the action of rollers and traffic and the grading of the same changes, it is useful if the final grading can be predicted as grading is an important factor in determining the behaviour of the compacted aggregates.

Ghani has plotted the aggregate sieve size ratios of each sieve size of both the original grading and the final grading after degradation on a log log graph, and obtained a straight line for each of the three different gradings.

$$\text{Aggregate sieve size ratio} = \frac{\text{percent coarser than sieve size}}{\text{percent finer than sieve size}}$$

The original ratio has been denoted x and the final, y , and he obtained the following equations:

$$(1) \quad y = 0.23x^{1.2}$$

$$(2) \quad y = 0.45x^{1.15}$$

$$(3) \quad y = 0.7x^{1.06}$$

for the three gradings, where the power of x denotes the slope of the straight line on the log log graph and decreases with the amount of coarse aggregate fraction in the mix as well as with the extent of degradation. The constant preceding x increases with increasing fines in the aggregate. These two constants controlled the extent of crushing and would obviously depend upon the aggregate characteristics

including its grading and the method of compaction adopted.

Similar graphs have been plotted with the original and final sieve size ratios of the brick aggregates of the five groups (Fig. 4.1). Straight line graphs have been obtained for all the groups except group III. The graph for Group III aggregates is a straight line except for the finer fraction of the aggregates starting from 1.20 mm sieve (No. 14) which is a curve. At least fourteen sieve sizes in each group have been plotted as compared with Ghani's six and the points are much closer to the graphs drawn. The equations for the straight line graphs of all the groups except group III and the straight line part of the graph of Group III aggregates have been calculated and are given below:

Group I	$y = 0.78x^{1.02}$
Group II	$y = 0.84x^{0.93}$
Group III	$y = 0.47x^{0.99}$
Group A	$y = 0.762x^{0.936}$
Group B	$y = 0.373x^{1.032}$

4.7 DISCUSSION

In this Chapter the meaning of degradation in relation to aggregates for flexible road pavements has been discussed. Then the degradation process and the measurement of degradation have been explained. Some degradation studies have also been reviewed. Finally, a method of prediction of changes in the grading of an aggregate mass due to degradation has been discussed.

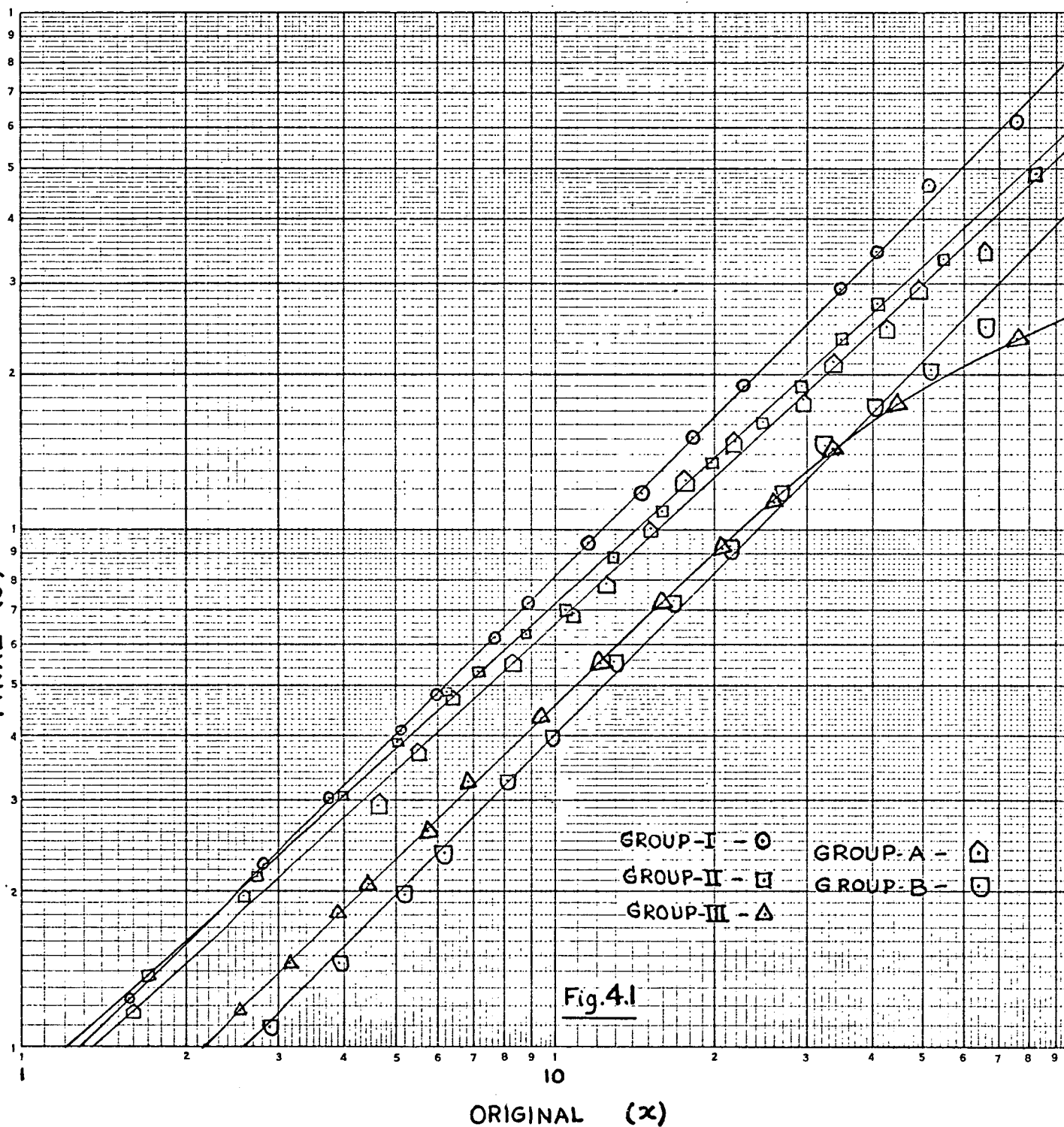
The reduction in sizes and angularity of the particles of an aggregate mass and the introduction of smooth surfaces therein due to application of a load is termed as degradation. Degradation results in a change in the stability of the aggregate structure due to changes in particle interlock and interparticle surface friction. Degradation in unbound bases can lead to pavement failures.

In order to quantify degradation experimentally, a number of investigators have performed sieve analyses and plotted the grading curves before and after degradation. Ghani (1969) plotted the aggregate sieve size ratios of each sieve fraction (sizes) of both the original grading and final grading after degradation on log log paper and obtained a straight line. From this graph the changes in the sieve size ratio and thus changes in grading after degradation could be estimated.

Kennedy (1974) used surface area determination as a measure of degradation and used a gas adsorption method known as dynamic Nitrogen method. But the amount of sample he used in each sieve fraction was in the order of 40 grammes and the maximum size used was passing 9.52 mm and retained in 6.35 mm. The samples used in the present study are much too large compared with those of Kennedy both in size and weight.

In the present study it has been considered most appropriate to perform sieve analysis before and after degradation. The grading curves, the sieve size ratio graphs and the percentage change in sieve fraction graphs have been plotted. (Figs. 4.1 - 4.9).

SIEVE SIZE RATIO



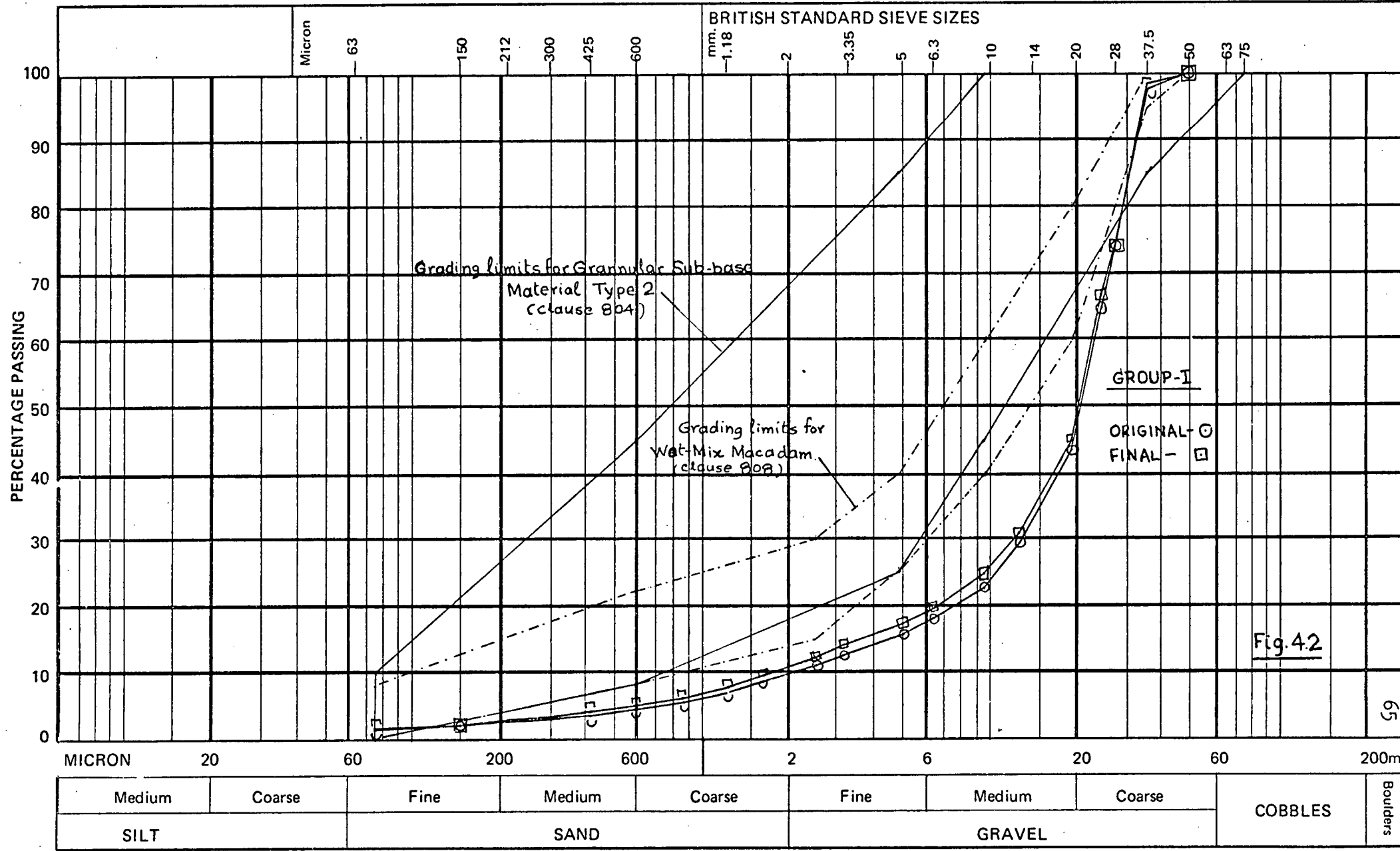
BORE HOLE No.....

SAMPLE No.....

PRETREATMENT DETAILS.....

DESCRIPTION.....

LOSS ON PRETREATMENT.....%



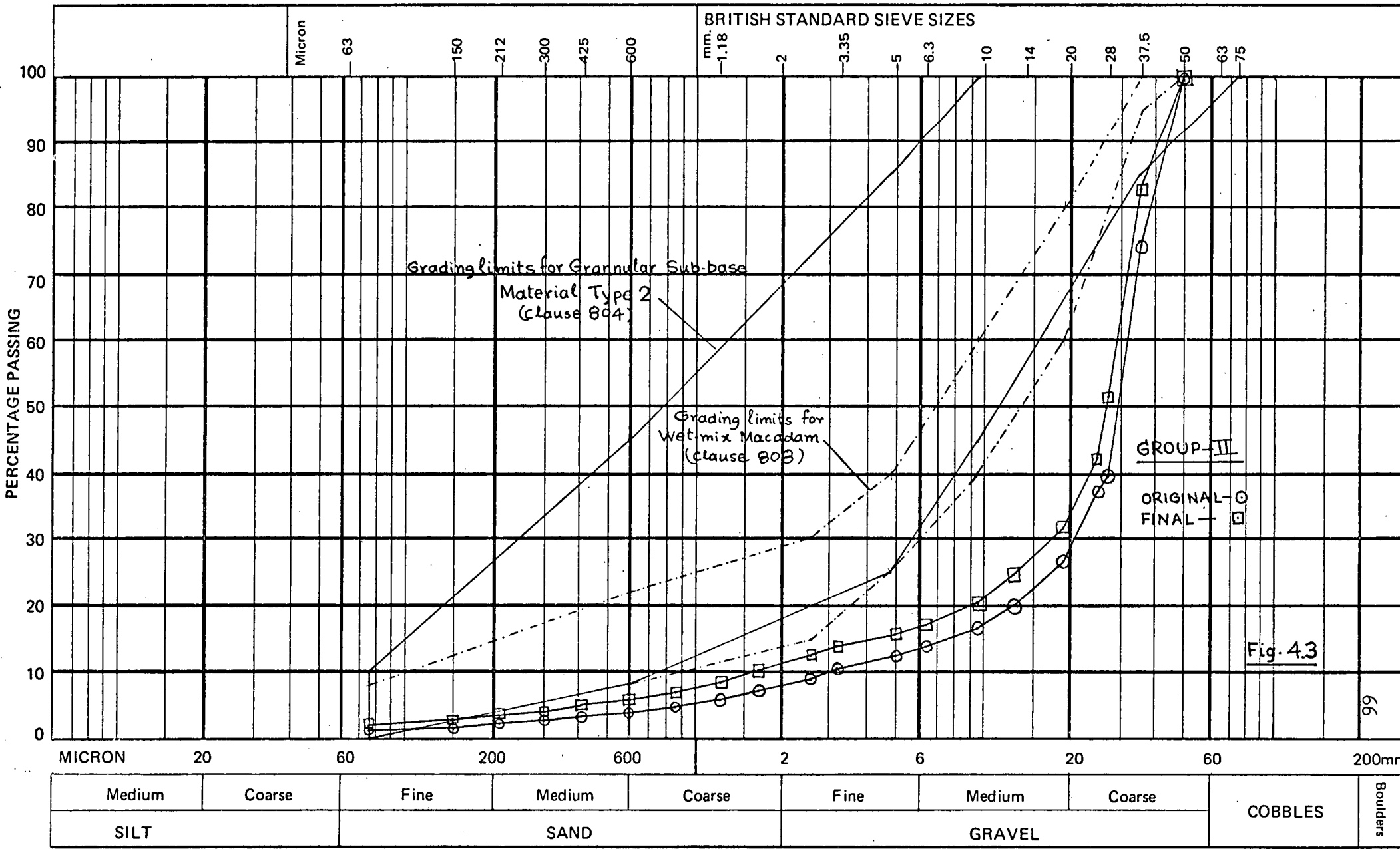
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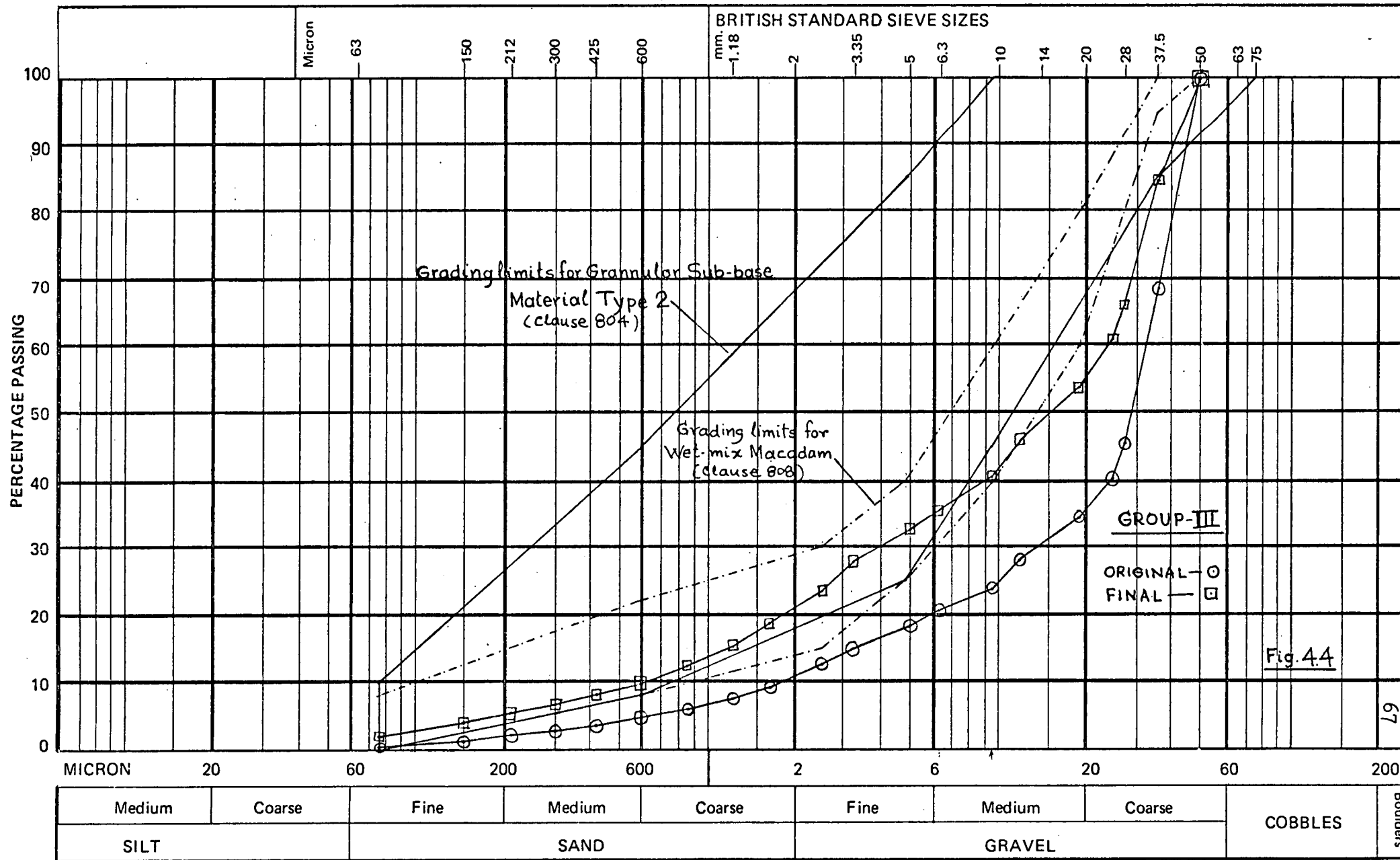
BORE HOLE No.

SAMPLE No.

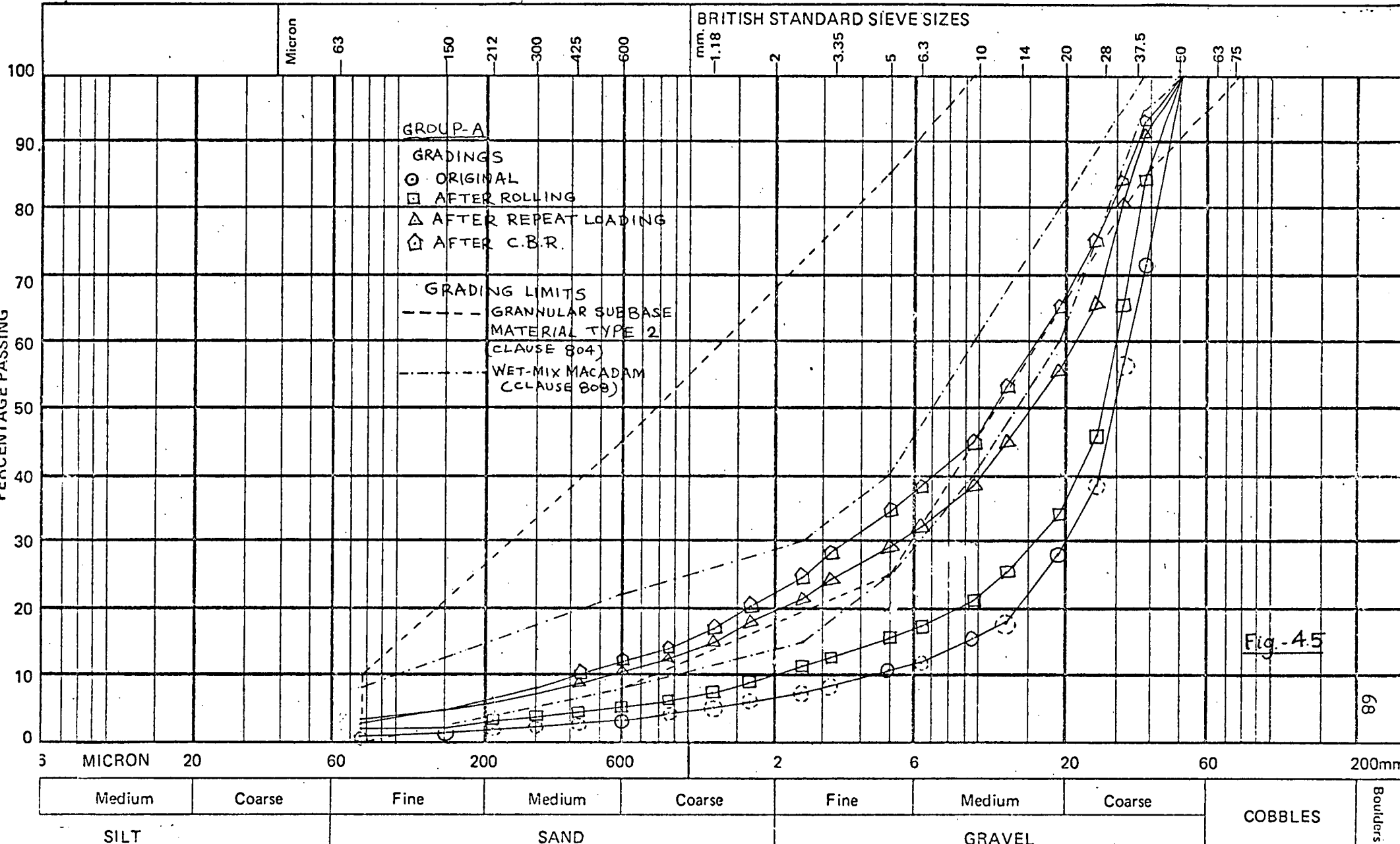
PRETREATMENT DETAILS

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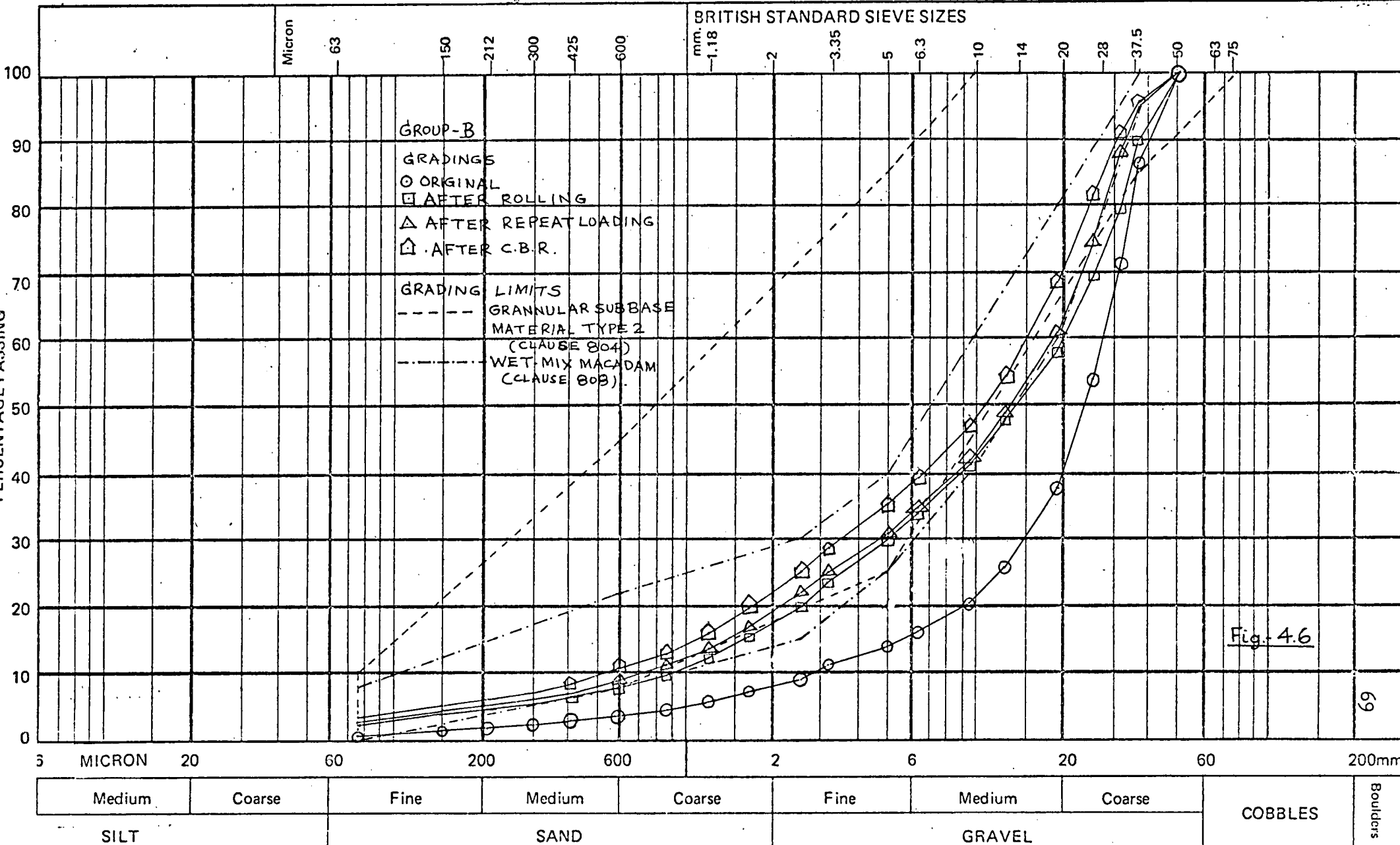


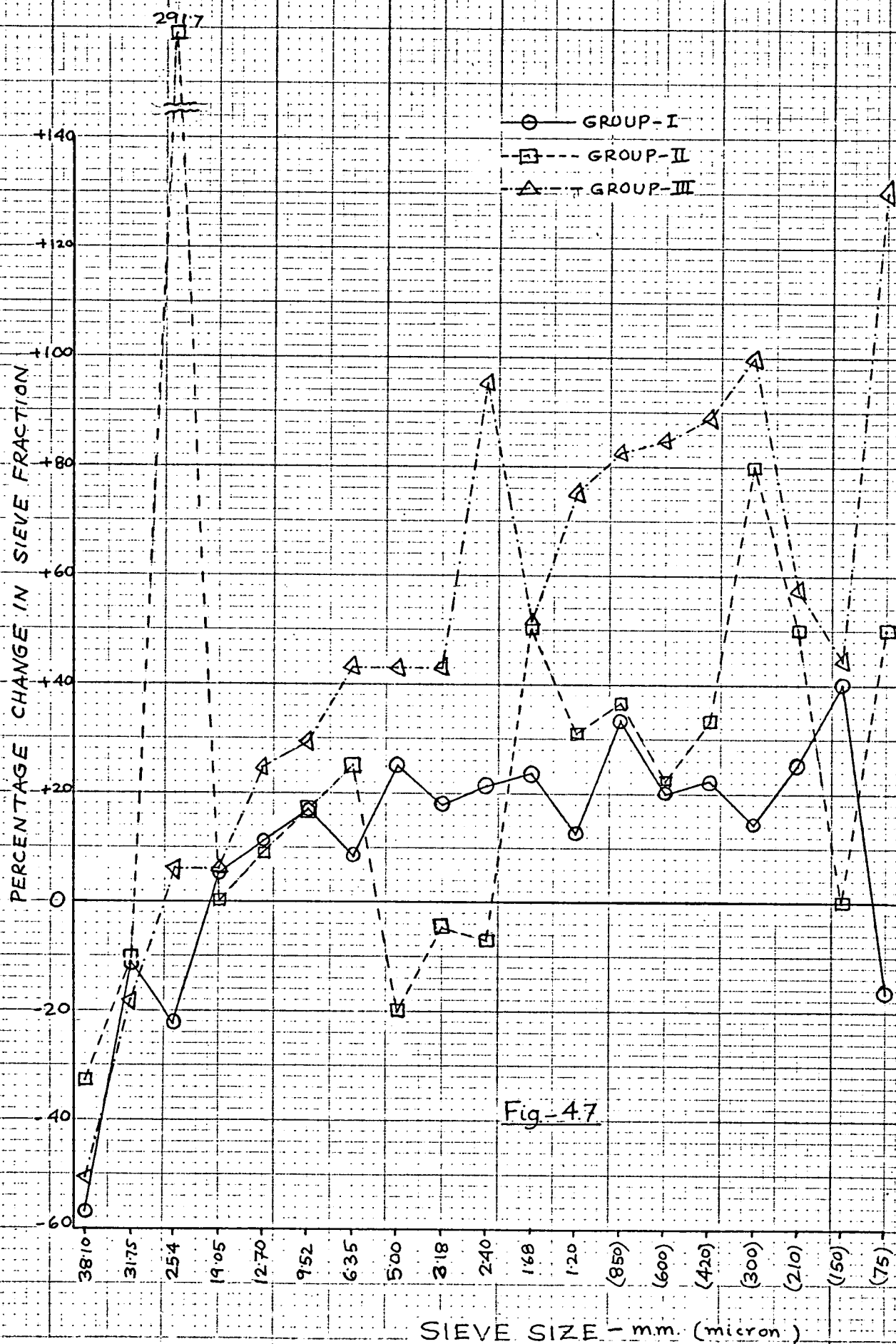
PARTICLE SIZE DISTRIBUTION GROUP - A.



PARTICLE SIZE DISTRIBUTION.

GROUP-B





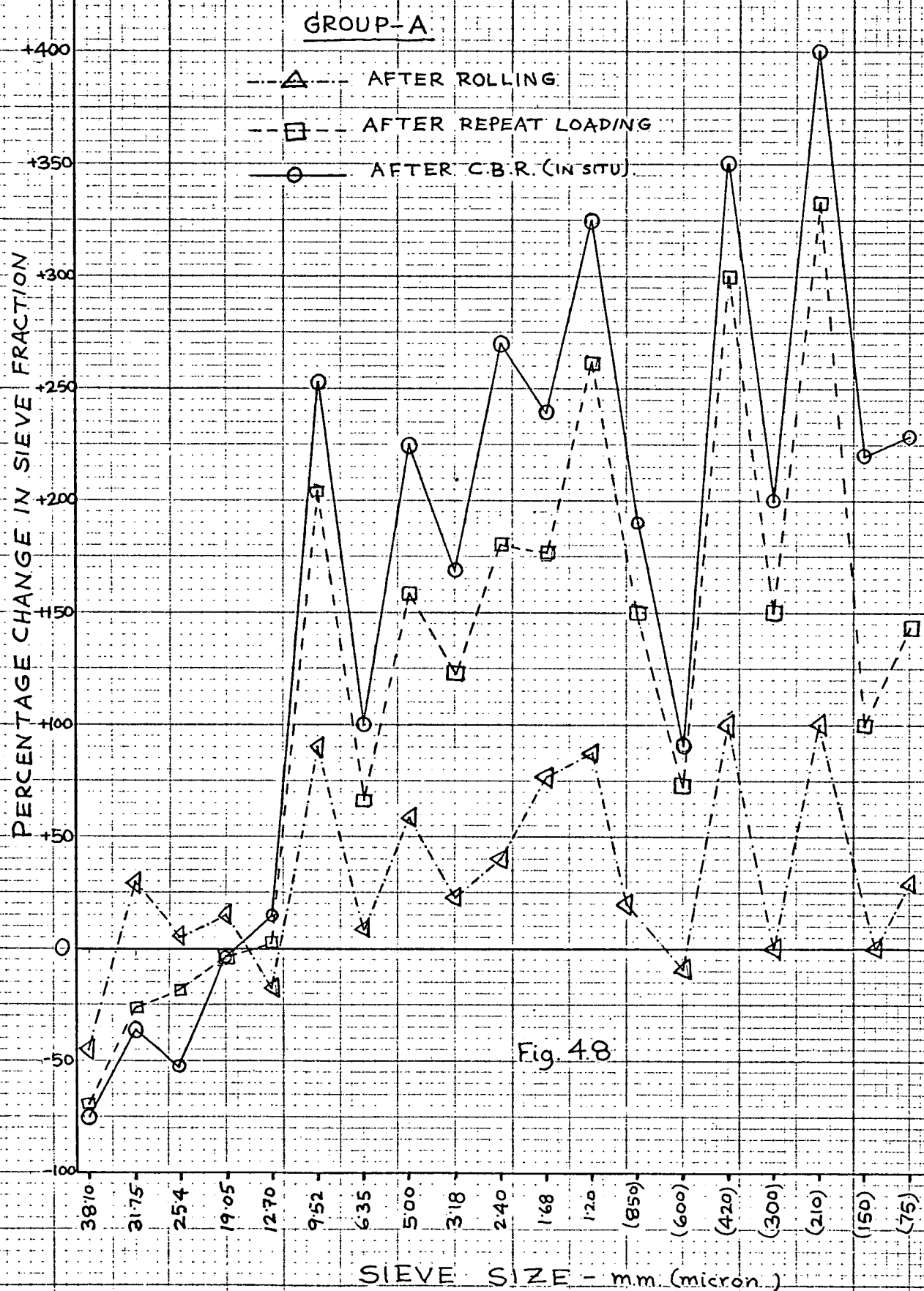
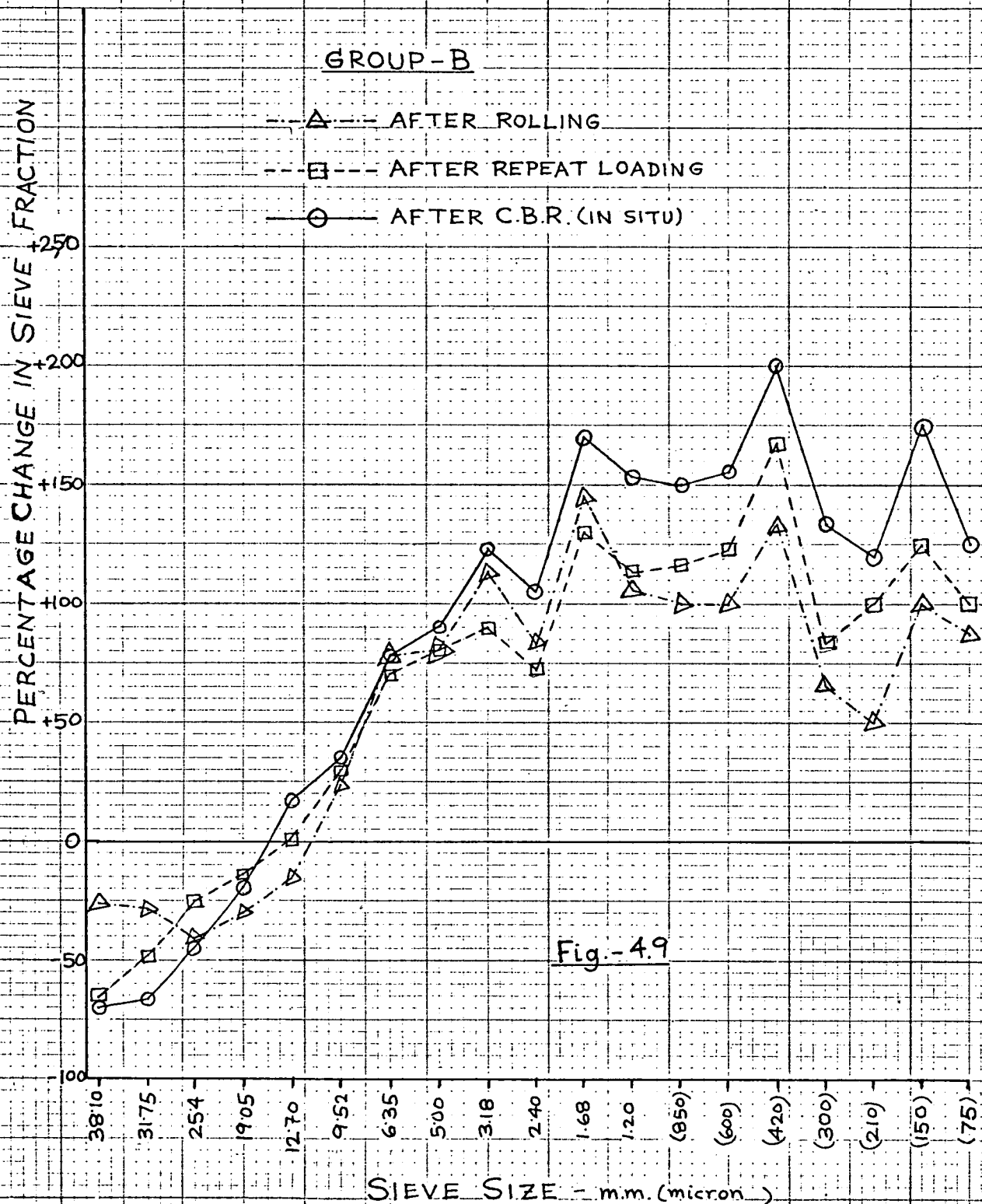


Fig. 48

SIEVE SIZE - mm. (micron.)



CHAPTER 5

PART I. C.B.R. TESTS WITH MODIFIED MOULD

5.1 INTRODUCTION

Due to shortage of stone aggregates, brick aggregates are used as road base and even base course material in Bangladesh. Hence it was intended to investigate experimentally the engineering behaviour of brick aggregates (i) under the compactive action of the road roller during construction and also (ii) under the action of traffic after the road has been opened to traffic.

Efforts were made to simulate the field conditions of rolling brick aggregates of the road base and to find out the effects thereof.

The second part of the experiment will be discussed in Chapter 7.

5.2 DESIGN

5.2.1 THE EQUIPMENT

For containing the aggregates, a cylinder 228 mm (9") in diameter and 304 mm (12") high was considered the maximum possible size which could be handled conveniently in laboratory conditions. It was intended that this cylinder would be half filled with aggregates and compacted so that the thickness of the aggregates would be about 115 mm (4½") after the usual compaction. Then a second filling would be made on top of the first layer and the compacting process repeated so that the total compacted thickness would be about 230 mm (9") to simulate field practice of compacting in two layers when the thickness is more than 150 mm (6").

A cylinder was made with a 228 mm (9") internal diameter mild

steel pipe cut to a length of 304 mm (12"). A base plate was made from a mild steel plate. Arrangements were made to clamp the base plate with the cylinder by means of bolts and wing nuts. Two handles were welded to the body of the cylinder for convenience of handling. A 222 mm (8¾") diameter mild steel top plate was made and other pieces of mild steel discs were also made to apply the load on top of the materials of the first layer inside the cylinder. The plunger of the aggregate crushing value apparatus was used in conjunction with the top plate and other disc or discs to apply the load. Another base plate had to be made for the 'Instron' machine as the normal bases of the same were about 150 mm in diameter. This was done by fixing the 230 mm (9") base plate of the California Bearing Ratio machine with a 150 mm Instron base plate by means of bolts, wing nuts, blocks, etc. The specimens of aggregates of different groups would be compacted in the Instron machine.

5.2.2 THE COMPACTIVE EFFORT

Eight to ten ton smooth wheeled rollers are used to compact the road base course in Bangladesh.

A road construction site (near Juniper Green, Edinburgh) was visited where an 8 - 10 ton Aveling Barford roller was compacting a crushed stone (rock) aggregate road base. The diameter and width of the wheels were measured. The width of the contact area between the road base aggregates and the wheels of the roller were measured when the rolling was half way through. Knowing the distribution of the weight of the rollers between the front and the rear wheels (Soil Mechanics for Road Engineers, H.M.S.O., pp. 181) and the contact area,

the average intensity of pressure under each wheel was calculated. (Appendix C.) These intensities came to about 214 kN/m^2 and 152 kN/m^2 (31 and 22 lbs per square inch) for the front and rear wheels respectively. To create an intensity of 214 kN/m^2 (31 p.s.i.) a load of 890 Kg (0.874 tons) would have to be applied on the model. In a single pass of the roller in the field, both the front and the rear wheels will pass once over the materials - the cumulative effects are not known. Static load was being applied in this experiment instead of dynamic load, a load of 950 Kg (about one ton) was considered appropriate to begin with. It was also assumed that the contact area would reduce to about half at the end, and thus the load would have to be increased to 2000 Kg (about two tons).

The intensity of pressure is least when the rolling starts and as the number of passes of the roller applied is increased the contact area is reduced due to compaction and the intensity of the pressure is increased gradually. In Bangladesh, when the aggregate of the road base is properly laid two passes of the roller are given on the dry aggregates, then water is spread over the aggregates and eight more passes of the roller are given. The roller starts from the edge of the road and gradually comes towards the centre. Considering the above conditions the loading was designed as follows:

First application	-	950 Kg.
Second	"	- 950 Kg.
3rd	"	- 950 Kg.
4th	"	- 1100 Kg.
5th	"	- 1250 Kg.
6th	"	- 1400 Kg.
7th	"	- 1550 Kg.
8th	"	- 1700 Kg.
9th	"	- 1850 Kg.
10th	"	- 2000 Kg.

The weight of the top plate, other discs and the plunger, which was about 30 Kg., for the first layer, and about 10 Kg. for the top layer has been neglected.

Water was sprinkled over the aggregates after the second load application. The amount of water was determined by the water absorption of the aggregates (which has already been determined). Adding more water than that would not probably be of any use. A test showed that about seventy percent of the water absorption occurs within ten seconds of a sample being placed in water. Hence water was sprinkled slowly over the aggregates after the second application of load. After a short while the rest of the load applications (from the 3rd to the 10th) were done.

5.3 TESTS

Three groups of brick aggregates were compacted as per the method described above. CBR tests were performed on the compacted specimens immediately after the compaction when they were transferred from the Instron machine to the CBR machine. A surcharge plate was used while determining the CBR to simulate 50 mm (2") of the bituminous pavement thickness.

Sieve analysis was performed for each of the three specimens after compaction.

The Instron chart recorder was used which indicates the load applied and the corresponding deformation undergone by the specimen.

The height of the specimens in the cylinder were measured after the completion of the compactive efforts. All other dimensions and weight being known, dry density and bulk density of each of the

compacted specimens has been determined. (Appendix A).

5.4 TEST RESULTS

Sieve analysis after compaction shows the particle size distribution after degradation due to compaction. Both the graphs of the particle size distribution of the original grading and the grading after compaction have been shown on the same sheet to show the amount of degradation undergone by the aggregates. The grading limits of Granular Subbase Material Type 2 (Clause 804, Specification for Road and Bridge Works, H.M.S.O., 1969) have been shown for comparison.(Fig.4.2-4.4).

The California Bearing Ratio tests show that the weakest aggregates after being compacted have the highest bearing capacity and the strongest aggregates the lowest. This occurred due to the fact that under the same compacting effort the weakest aggregates had undergone maximum crushing resulting in minimum voids (or porosity) and highest bearing capacity and vice versa. This is also evident from the dry and bulk densities of the three groups. Group III, which is the weakest, and which has the least specific gravity, have the highest dry and bulk densities and Group II, the strongest, have the least dry and bulk densities. (Appendix A).

5.5 LIMITATIONS

5.5.1 THE BOUNDARY EFFECT

Since the aggregates are compacted in the confined space of a cylinder, the boundary effect of the cylinder wall is there.

Experimental workers like Kolbuszewski (1948), Hosking (1961) and Hughes (1962) had observed that the measured porosity of a given

sand or gravel increased as the size of the container in which the determination was made, was reduced. It was also clear that a similar effect was produced by increasing the size of the aggregates in a given container.

From these studies it was apparent that the presence of the wall had a disturbing, i.e. loosening, effect on the packing of particles and that the total porosity was determined by the relative magnitudes of the outer loosely packed zone and the internal more densely packed zone. Lees (1969) in his article on "Influence of boundary effects on the packing and porosity of granular materials" concluded that where it is desired to know porosity values for a given aggregate in a given container, it may under certain circumstances be possible to use correction factors and graphs previously published. These correction factors seem to have greater validity when the size of aggregates is smaller compared to that of the container. This condition represents an $\frac{A}{V} \times d$ value of less than unity, where A is the confining area, V is the enclosed volume and d is the equivalent spherical diameter of the particle. The proposed correction factors, even in such cases, may not be valid for all combinations of particle shape and grading and the shape of the container.

In other cases, especially in those where the $\frac{A}{V} \times d$ value is significantly above one, any attempt to predict the porosity of an aggregate mass in a given container from its known porosity in a container of a different size and/or shape may lead to serious error. Consequently, there may be no alternative to determining the porosity of that aggregate under a given effort directly in a container of the same or nearly the same dimensions.

Most of the above studies were conducted on single size aggregates. Lees (1969), however, experimented on aggregates of all sizes, but the same shape like equidimensionals, rods, discs and blades. But no work has yet been undertaken considering aggregates containing all sizes and shapes as these make the case very complicated.

It is also to be borne in mind that resistance is offered by the side walls in the form of friction when the sample is undergoing deformation and compaction and a fraction of the compacting force is lost. It is extremely difficult to determine these factors.

In field compaction there is practically no boundary effect.

5.5.2 THE COMPACTING EFFORT

In field conditions the action of the road roller is dynamic whereas in the laboratory compacting effort is static.

In the field, the aggregates arrange themselves to interlock and undergo compaction under the action of the wheels of the roller. The aggregates have the advantage of forward, sideward and downward movement whereas the aggregates in the cylinder in the laboratory have much less freedom to do so. Moreover, there is the boundary effect and frictional resistance of the side walls of the container.

5.5.3 THE CALIFORNIA BEARING RATIO TEST

With the plunger having an area of 19.35 cm^2 (3 in^2), the CBR test is not valid when the aggregate size is more than 19.05 mm ($\frac{3}{4}''$). In all the groups the aggregates having a maximum size 50.80 mm - 37.8 mm (passing 2" and retained on $1\frac{1}{2}''$ B.S. Sieve) and a minimum size of passing Number 200 B.S. Sieve were used. Strictly speaking, therefore, the test is not valid. However, the tests were carried

out neglecting the fact that larger particles under the plunger would have distributed the load over a wider area and thus necessitating greater force for penetration of the plunger and resulting in higher CBR value.

The CBR tests were performed on aggregates containing all sizes and shapes and having an open grading made up by breaking whole bricks without any sieving to simulate field conditions. Although the tests on such aggregates are not strictly valid, the results thus obtained at least show the comparative bearing capacities of the three groups of aggregates and the bearing capacities thus found out may have a relationship with their actual bearing capacities had they been tested strictly in accordance with the British Standard.

5.6 DESIGN OF THE EXPERIMENT IN GENERAL

After the tests, it can be observed from the load deformation charts obtained from the Instron machine that deformation is a maximum on the first application of load. The deformation decreases considerably on the second application. Thereafter the deformation is about the same each time the load is applied, although the load is increased upon each application at a constant rate. The number of applications of load has been determined from field practices. But whether the particles should undergo further compaction is to be determined on the basis of whether further compaction would result in more strength of the road base or not.

It is noticeable from the dry and bulk densities of the compacted aggregates that Group III aggregate, which is the weakest and which

has the least specific gravity, has the highest dry and bulk densities.

In a situation where three groups of aggregates have similar sizes of particles, similar particle size distribution and similar packing under compaction, but different relative densities and crushing strengths, then their densities would depend on the relative densities. Group II aggregates being the strongest and having the highest relative density should have the highest density. Group III aggregates are the weakest and having the lowest relative density should have the lowest density, under the above circumstances. However, the test results indicate the reverse. This is possibly due to the fact that, under the same compactive effort, the third group being the weakest has undergone maximum crushing (degradation) and thus is the most dense and this has overridden the effect due to specific gravity as described earlier. The net result of this is that after the designed compactive effort the third group of aggregates has the highest CBR and the second group of aggregates the least. So if more and more compactive effort is put to the aggregates a stage will come when this order will reverse and the second group will have the highest CBR and the third the lowest, which is more logical and desirable. However it may be uneconomical in practice.

CHAPTER 5

PART II

C.B.R. TESTS WITH MODIFIED PLUNGER

5.7 INTRODUCTION

In part 1 of this Chapter the engineering behaviour of brick aggregates under the action of rolling has been discussed. The tests were performed on a sample and the rolling simulated. Later a full scale experimental pavement was constructed and the rolling was done by a specially constructed roller. Hence it has been possible to investigate the behaviour of the aggregates under the action of rolling as if in a practical situation.

5.8 DESIGN OF THE EXPERIMENTS

Design and construction of the experimental pavement

An experimental full scale pavement 1.75 m x 1.68 m has been constructed with a road base of brick aggregates 230 mm (9") thick. The design and construction of the same has been discussed in Chapter 7. The following tests were involved in evaluating the behaviour of the aggregates under the action of rolling.

- (i) Sieve analysis - before and after rolling.
- (ii) Dry density - before and after rolling.
- (iii) Modified C.B.R. tests.

(i) Sieve Analysis

Sieve analyses have been performed on each of the two groups of aggregates (Groups A and B) prepared by hand crushing from bricks selected on the basis of crushing strength as described in Chapter 3. A few samples from each group have been taken and the sieve analysis

performed. The mean gradings have been calculated and the curve drawn. (Figs. 4.5 - 4.6).

Sieve analyses of the aggregates in the road bases after completion of rolling have also been performed. One sample has been taken from each group.

(ii) Dry Density

Dry density of the aggregates before compaction has been measured by placing the aggregates in a 406 mm (16") diameter 203 mm (8") high cylinder by a shovel, filling it up to overflowing and then striking off the aggregates level with the top with a straight edge. The material is then weighed. The test is repeated three times and the mean weight is taken. The volume of the cylinder being known, the dry density is calculated.

The dry density of the compacted aggregates has been found by the sand replacement method (B.S.1377: 1975).

(iii) C.B.R. Test (modified plunger)

C.B.R. tests are usually performed in the laboratory on aggregates passing through a 19.05 mm ($\frac{3}{4}$ ") sieve. The mould for holding the sample is 152 mm (6") in diameter and the plunger has an area of 19.355 cm^2 (3 sq.in). The maximum size of the aggregate particles is passing the 50.8 mm (2") sieve and retained on 38.1 mm ($1\frac{1}{2}$ "). Hence the test is not valid. It was therefore decided that a 152 mm (6") diameter steel plate, instead of the standard plunger, would be more appropriate, as it would overcome the short-coming of the larger particle size. It would also enable comparison to be made with the

results of the repeat load tests where the same plate was used to load the pavement. Also the area of the 152 mm diameter circular plate approximately represents the contact area of a commercial tyre. The load was applied manually by a hydraulic jack (Plate 5.1).

5.9 TEST RESULTS

(i) Sieve Analyses

The sieve analyses before the compaction in all the groups show a much coarser grading than the M.O.T. grading of wet mix macadan, clause 808.

After compaction the gradings become nearer to the M.O.T. grading, especially with the weaker groups.

The grading curves have been drawn to show the gradings of the groups before and after compaction along with the M.O.T. grading.(Figs.4.5-4.6).

(ii) Dry Density

The dry densities of the aggregates of the two groups before and after compaction by rolling give an indication of the extent of reduction of volume under the compactive effort and enable the field engineer to estimate the quantity of aggregates needed to construct a given volume of road base.

The dry densities of the aggregates of the two groups before and after compaction are as given in the table below.

Group	Dry Density(before compaction)	Dry Density (after compaction)
	Mg/m ³	Mg/m ³
A	0.964	1.375
B	0.947	1.337

(iii) Modified C.B.R. tests

The modified C.B.R. tests indicate the bearing capacity of the pavements. Although it is not a standard test, it gives an indication of strength and can be used to compare the two pavements made from group A and group B aggregates.

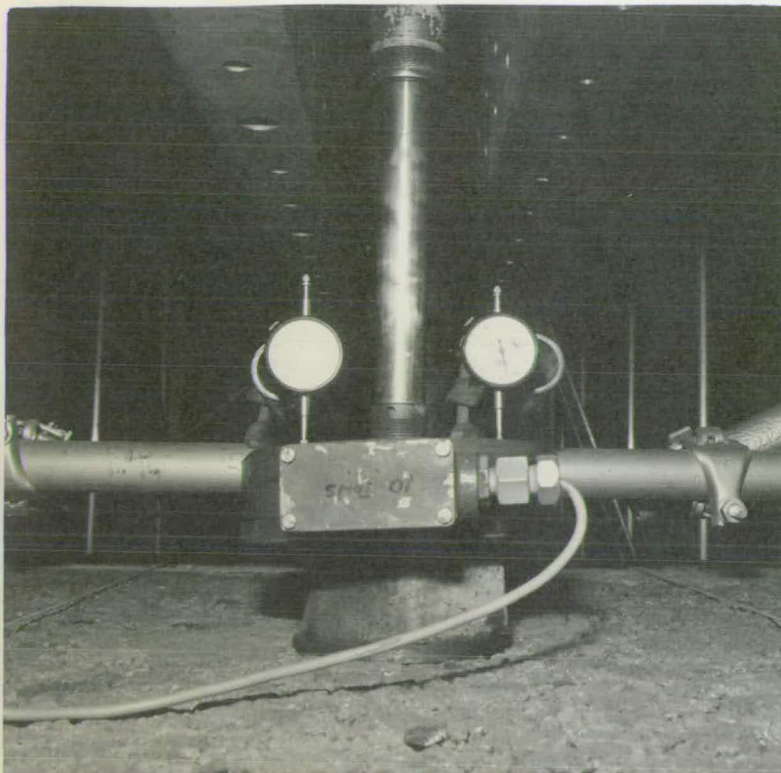
The pavement made with group A aggregates has a C.B.R. of 34 and that made with group B aggregates has a C.B.R. of 21. (Appendix B).

5.10 DISCUSSION

In Part 1 of this Chapter the results of C.B.R. tests with a modified mould on Group I, II and III aggregates which have a maximum size of 50.80 mm have been reported. From the tests it was noted that the weaker aggregates degraded much more than the stronger aggregates under the same compactive effort thus produced more fines resulting in a higher density and hence a higher C.B.R.

In Part 2 of the Chapter the C.B.R. tests with a modified plunger on Group A and Group B aggregates have been discussed. The tests were performed on an experimental full scale road base. This was done to overcome the inadequacy of the standard plunger when testing an aggregate which has a maximum aggregate much too large compared with the standard size. The size of the plunger was chosen to be 152 mm in diameter, the contact area of which approximately represents the contact area of a typical commercial tyre. The C.B.R. values obtained are 34 and 21 percent for Group A and Group B respectively. These values indicate that these two groups of aggregate qualify for use only as a sub-base according to the Specification for Road and

Bridge Works (Clause 801, Table 7). The minimum C.B.R. value of a standard aggregate to be used in the U.K. as a road base is 80% (Ashworth, 1966). But in Bangladesh, the commercial vehicles are lighter and traffic volumes lower compared with the U.K. Due to the shortage of crushed stone aggregates, brick aggregates must be used. The C.B.R. values indicate that a much higher thickness of brick aggregates is needed to substitute stone aggregates.



IN SITU MODIFIED C.B.R. TEST



MARK ON ROAD BASE AFTER C.B.R. TEST

CHAPTER 6

TRIAXIAL TESTS

6.1 INTRODUCTION

The triaxial test gives the laboratory measurement of shear strength under controlled conditions of drainage and deformation characteristics (other than compressibility) (Bishop and Henkel, 1962). Although triaxial tests are normally carried out on soil and sand, investigators like Kennedy (1974) performed special triaxial tests on crushed rock aggregates and Boyd (1976) on crushed brick aggregates - both the aggregates to be used as a road base. Since the present study deals with brick aggregates, a number of triaxial tests have been undertaken to relate the present work with that of Boyd.

6.2 PREPARATION OF THE SAMPLES

The maximum size of the aggregate particles used in the experimental pavement passes 50.8 mm sieve, which is too large for the 105 mm diameter (x 203 mm high) mould used for the preparation of the triaxial samples. Hence the maximum size has been reduced to half. The grading curves of the hand broken brick aggregates have been adopted with this modification. The shape of each curve has been kept the same by reducing the size of each of the sieve fractions by half and keeping the corresponding percentage passing the same as in the original grading curve. Hence it has been a horizontal shift of the grading curve towards the left.

The samples have been prepared with the modified grading as described above. A two-way split mould 105 mm in diameter has

been used. The mould is fitted with a triaxial base lined with a 102 mm (4") diameter rubber membrane used in a triaxial sample. The quantity of aggregates needed for a 50.8 mm (2") thick compacted layer of the sample has been so determined that the compacted aggregates in the mould have the same dry and bulk density as those of the experimental pavement. The quantity of aggregates is then made up by taking the exact quantity of aggregate from each sieve fraction. A thin (3 mm) plastic disc about 100 mm in diameter is placed at the bottom of the mould. The aggregates are then placed in the mould. Water is sprinkled on the aggregates in such a quantity that the moisture content of the aggregates would be the same as that of the experimental pavement during the time of testing. After a few minutes the aggregates were compacted by means of a vibrating hammer (Kango hammer, type L). A steel tamper like the one used in the vibrating hammer compaction test (B.S. 1377: 1975 test 4.3) but having a circular foot 95 mm (3.75 in) in diameter has been used.

The compaction process was continued until the thickness of the layer reduced to exactly 50.8 mm (2"). Three more such layers were provided by repeating the process thrice more, thus making the total height of the sample 203 mm (8"). Another plastic disc was put on top of the sample. Another rubber membrane was fitted over the sample because the first membrane gets punctured during the compaction process.

6.3 TRIAXIAL TESTS

The samples were tested in a 50 kN (5 ton) Wykeham Farrance loading machine operating at a constant rate of strain. The rate of loading was 0.1905 mm (.075") per minute which is approximately 1% per minute. Multistage quick undrained tests on partly saturated samples were performed. Samples were prepared from group A and group B aggregates. The multistage test with each sample was started with a low cell pressure of 69 kN/m^2 (10 p.s.i.) and run in the ordinary way until the ratio of the stresses, σ_1/σ_3 , reached its peak value. The cell pressure was then increased to 138 kN/m^2 (20 p.s.i.) and the test continued until a new peak stress ratio was obtained. In the next stage the cell pressure was increased to 276 kN/m^2 (40 p.s.i.) and in some specimens the cell pressure was raised to 414 kN/m^2 (60 p.s.i.) in the fourth stage. The samples were tested until failure occurred. Occasionally failure occurred due to the puncture of the outer rubber membrane of the sample.

Manual recording of the loads was made from the dial gauge on the proving ring. Readings were taken at certain strain increments.

6.4 TEST RESULTS

6.4.1 THE MEASURED VARIABLES

The following measurements were involved in the tests:

1. Axial load
2. Axial deformation
3. Cell pressure

The axial load was measured by a proving ring. The axial

deformation was measured by means of a dial gauge and indicated the deformation of the loading piston. The cell pressure was provided by water and compressed air supply controlled by an adjustable reducing valve. The cell pressure was observed from a pressure gauge connected with the system.

From the measured variables stress-strain curves, Mohr circles and failure envelopes have been drawn.

6.4.2 TEST RESULTS

The stress-strain curves, and Mohr circles and failure envelopes as obtained from the multistage triaxial tests have been shown graphically in Figures 6.1 to 6.14. Each graph represents an individual test. In Figures 6.8 to 6.14 the estimated values for the undrained cohesion, C_u and the undrained angle of shearing resistance, ϕ_u , for the samples have been shown. The values used to construct the Mohr circles are the maximum values of each stage of a test.

The stress-strain curves drawn do not show an elastic behaviour of the samples as is indicated by the constant variation of the secant modulus throughout the tests. The curves (graphs) start with a concave upward trend but soon change to a concave downward curve and then flatten out very suddenly. At this point the cell pressure is increased and the curve rises steeply in a concave downward trend and then flattens out again when the cell pressure is increased a second time. In some tests a third increase in cell pressure was made leading to a fourth stage. The initial part of the curve being concave upward may be due to the loosening of the

specimen at the ends, especially at the top end due to handling of the sample for fitting rubber membranes, O rings, loading cap, etc.

From the Mohr circles drawn it appears that the failure envelope is slightly curved and hence the values of ϕ_u and C_u vary. To show the variation two failure envelopes in the form of two straight line tangents to the Mohr circles showing the maximum and minimum values of ϕ_u and the corresponding C_u values have been drawn. A comparison of stress ratios for different tests was made at equal axial deformations. Stress ratios and corresponding axial strain for the tests have also been shown in Table 6.3.

Seven samples prepared from two groups of aggregates, group A and group B, have been tested. In all of them the stress ratios reached their peak at axial strains of approximately 1.0 and 1.25% for group A and between 0.75 and 2.25% for group B aggregates. In each of the tests the maximum stress ratios were obtained in the first stage and they varied between 11.90 to 16.88 for group A and between 11.10 to 15.21 for group B aggregates.

Boyd's (1976) samples were prepared from similar bricks (from the same source) as those used in the present work. In the two multistage tests on his type I and type II bricks (compressive strength 20 - 23 MN/m^2 for type I and 10 - 13 MN/m^2 for type II aggregates) the total stress ratio reached its peak at an axial strain of 7.5% and 5.5% and the stress ratios were 7.03 and 7.83 for type I and type II aggregates respectively. In his single stage tests the peak stress ratios were obtained at axial strains varying between 5.0 and 12.5% and the stress ratios vary between 8.70 and 4.89. Two graphs of stress ratio vs. axial strain for the two multistage tests of Boyd (1976) have been shown along with some of those of the present tests for comparison (Fig. 6.15).

Photographs of a triaxial sample before and after failure have been shown in Plate 6.1.

6.4.3 SIEVE ANALYSIS

Sieve analyses of the samples were performed after the triaxial tests. Sieve analysis of one sample from each of the groups A and B was also performed after compaction in the mould (but before any triaxial test) to find out the gradings of the compacted sample. The original grading, the grading after compaction and the grading after the triaxial tests of the samples of group A and B have been shown in Table 6.1 and Table 6.2 respectively. From the tables it can be seen that the compacted triaxial samples fairly represent the gradings of the compacted full scale experimental road base. Degradation of the samples occur during triaxial tests but only to a very small extent as can be seen from the tables. (This was also observed by Boyd (1976)). Degradation is much greater in repeat loading tests as can be observed from Tables 6.1, 6.2 and 7.2.1 to 7.2.4.

6.5 DISCUSSION

From the graphs it can be seen that group A aggregates have much higher stress ratios when compared with those of group B as would be expected because of the higher crushing strength of group A aggregates. They have similar gradings and were compacted with the same effort to represent the field conditions. Although Boyd's type I aggregates were very similar in strength to group A aggregates and his type II aggregates were a little weaker than group B aggregates of the present study, the tests showed that the stress ratios were much smaller when compared with those of the respective groups for the present study. This is possibly due to the difference in the compacting effort. In the present study the compaction process is a simulation of the actual rolling by a field roller. Whereas Boyd's samples were compacted on a vibrating table with a frequency of 10 Hz and the amplitude adjusted to give an acceleration of 1.34g. Each sample was compacted in four layers in a two way split former 105 mm in diameter. Kennedy also used the same method for preparation of 150 mm diameter x 300 mm high triaxial samples of crushed rock. He used a constant frequency of vibration but varied the amplitude of vibration in each of the four layers to give the optimum compaction. Boyd used only one amplitude. After compaction of the first layer to optimum, when the table is again vibrated for the second layer there may be some change in the orientation of the first layer and the compaction may not remain optimum. Kennedy applied an amplitude to give an acceleration so that the particles fall individually within the sample under vibration. As the acceleration is reduced, the level to which this occurs is

progressively raised, and eventually only affects the topmost sample. When the second layer is compacted, the particles of the first layer will not fall individually but may still have movement and may change the compaction away from the optimum.

From the stress ratio vs. axial strain graphs it is evident that Boyd's samples had lower stress ratios compared with those of the present study. The peak stresses for the present study were obtained very much more rapidly when compared with those of Boyd's (0.75 to 2.25% as against 6 to 8 percent) axial deformation. The materials and the gradings being similar in both the studies the only main variable left to account for the differences is the density of the samples. It is, therefore, reasonable to conclude that the compaction of Boyd's samples were not adequate and do not represent the field condition so far as density is concerned. Boyd's aggregate type II which is weaker shows a slightly higher stress ratio when compared with his type I aggregate, which is stronger. This shows that compaction by the vibrating table is not very reliable.

The effect of compaction can also be seen from the stress ratio vs. axial strain graphs of the present tests/ (Fig.6.15). The compaction of samples A_1 and A_2 is 5% more than that of sample A_3 and the compaction of samples B_1 , B_2 and B_3 is 5% more than that of sample B_4 . The results indicate that the stress ratios of samples A_3 and B_4 are lower compared with those of the other samples of their respective groups. (Table 6.3).

The net ultimate bearing capacity of the pavements for the two groups of aggregates have been calculated according to Terzaghi's bearing capacity formula using the various ϕ_u and C_u values obtained from the Mohr circles (drawn for each test) and shown in Table 6.4.

The experimental pavement was subjected to a repeat loading. The pressure exerted by the load initially was 345 kN/m^2 (50 p.s.i.) and in the fourth test raised to 690 kN/m^2 (100 p.s.i.), which is the pressure exerted by the full load of a typical wheel of a truck in Bangladesh. The axial deformations of the triaxial samples corresponding to the 345 kN/m^2 (50 p.s.i.) pressure have been found to be 0.15, 0.17 and 0.31% for samples A_1 , A_2 and A_3 of group A and 0.21, 0.47, 0.37 and 0.33% for B_1 , B_2 , B_3 and B_4 of group B aggregates respectively.

The axial deformations corresponding to 690 kN/m^2 (100 p.s.i.) pressure have been found to be 0.32, 0.33, 0.63, 0.42, 0.92, 0.84 and 0.67% for samples A_1 , A_2 , A_3 , B_1 , B_2 , B_3 and B_4 respectively. The pavement thickness being 230 mm (9") the total deformation corresponding to 345 kN/m^2 pressure is in the order of 0.235 to 0.713 mm for group A and .483 to 1.081 mm for group B pavement. The deformation corresponding to 690 kN/m^2 (100 p.s.i.) is of the order of 0.736 - 1.449 mm for group A and 0.966 - 2.116 mm for group B aggregates. In a flexible pavement the vertical displacement at the surface under the wheel should be limited to 0.2 inches (5 mm) according to Burmister (Ashworth, 1966). The pressure in the triaxial samples rose to more than 690 kN/m^2 during the first stage of the tests in all the samples. The confining stress during the first stage was 69 kN/m^2 (10 p.s.i.). From the tables of 'Stresses in Three-Layer Elastic Systems' by Jones (1962), the

stress factor for the horizontal stress at the top of the road base (second layer) was found to be 0.23. Hence the horizontal stress in the repeat load sample is 159 kN/m^2 (23 p.s.i.) which is much higher than the 69 kN/m^2 (10 p.s.i.) confining stress in the triaxial samples in their first stage. If the triaxial samples were subjected to a cell pressure equal to the horizontal stress in the road base the axial deformations would have been much less.

From the above results it appears that the road base is strong enough to withstand the typical wheel load mentioned earlier on its own (i.e. without the surfacing) even if it is constructed with group B materials. The vertical displacement of 5 mm is the total vertical displacement allowable. In the above discussion, figures of deformation are due only to the road base and the deformation of the brick layer and the sand sub-base is also to be added to those figures. Even so the pavement should be adequate from the design point of view to allow vehicular traffic on the road base so long as the wheel load does not exert a pressure too much in excess of 690 kN/m^2 .

TABLE 6.1

SIEVE ANALYSIS

GROUP A

"PERCENTAGE PASSING"

SIEVE SIZE (Reduced - ½ scale)	Original Grading - Hand- broken Samples	TRI-AXIAL SAMPLES				After compaction by rolling in full scale experiment	
		After Compaction in mould with Kango hammer (105%) ⊕	After Tri-axial test (Compaction - 100%)⊕	After Tri-axial test (Compaction - 105%)⊕	After Tri-axial test (Compaction - 105%)⊕		
mm/ (micron)	Inch /(No)	A ₀	A ₄	A ₃	A ₂	A ₁	A _R
5.00	1	100.0	100.0	100.0	100.0	100.0	100.0
	¾	71.3	84.8	86.6	86.4	85.1	84.2
	½	38.5*	59.0	61.9	62.7	63.5	46.1
	⅜	27.9	44.7	45.0	46.7	49.7	33.9
	¼	17.7*	32.3	34.0	35.3	37.0	25.5
	-	15.4	27.8	28.6	30.1	31.6	21.1
	⅛	11.9*	21.5	21.9	23.3	24.6	17.3
	(7)	10.7	18.8	18.9	20.2	21.5	15.4
	(10)	8.5*	14.6	14.5	15.4	16.8	12.7
	(14)	7.5*	11.7	11.5	12.3	13.6	11.3
	(18)	6.2*	9.5	9.4	10.1	11.0	9.0
	(25)	5.4*	7.7	7.6	8.3	9.0	7.5
	(36)	4.4*	6.2	6.0	6.8	7.3	6.3
	(52)	3.3	4.9	4.9	5.5	6.0	5.3
(72)	2.9*	4.1	4.2	4.7	5.1	4.5	
(100)	2.3*	3.4	3.4	3.9	4.2	3.9	
(200)	1.5	2.4	2.2	2.8	2.9	2.8	

* by interpolation of a smooth graph.

⊕ Compaction compared with that of the full scale experimental bed (road base) after compaction by rolling (Grading - A_R).

TABLE 6.2

SIEVE ANALYSIS

GROUP B

"PERCENTAGE PASSING"

Sieve Size (reduced -½ scale)	mm/ (micron)	Inch/ (No)	Original Grading - Hand- broken samples (average of 3 samples - weighted Mean	After compaction in mould with Kango hammer 105% ⊕	After Tri-axial test (Compaction - 100%) ⊕	After Tri-axial test (Compaction - 105%) ⊕	After Tri-axial test (Compaction - 105%) ⊕	After Tri-axial test (Compaction - 105%) ⊕	After Compaction by rolling in full scale experiment
			B ₀	B ₅	B ₄	B ₃	B ₂	B ₁	B _R
5.0	1		100.0	100.0	100.0	100.0	100.0	100.0	100.0
	¾		86.2	91.0	93.6	91.3	93.1	90.0	89.9
	½		53.9	67.6	72.1	74.8	72.4	70.3	69.1
	⅜		37.7	51.8	57.1	61.6	57.1	57.6	57.8
	¼		25.7	39.3	43.3	48.9	44.1	46.0	47.8
	-		20.3	32.9	36.3	42.7	37.8	39.9	41.1
	⅛		16.2	25.3	28.1	33.5	29.5	31.5	33.8
	(7)		14.0	21.8	24.2	29.2	25.5	27.3	29.8
	(10)		11.0	16.8	18.4	21.3	20.0	21.0	23.4
	(14)		9.2	13.4	14.5	17.6	15.9	16.8	20.1
	(18)		7.2	10.8	11.2	13.9	12.6	13.2	15.2
	(25)		5.7	8.3	9.0	11.3	10.2	10.6	12.1
	(36)		4.5	6.7	7.1	9.4	8.3	8.8	9.7
	(52)		3.6	5.6	5.9	7.7	7.0	7.1	7.9
	(72)		3.0	4.8	4.9	6.6	6.0	6.1	6.5
	(100)		2.4	4.0	4.1	5.6	5.0	5.1	5.5
	(200)		1.5	2.7	2.7	4.0	3.4	3.5	3.9

⊕ Compaction compared with that of the full scale experimental bed (road base) after compaction by rolling (Grading - B_R).

TABLE 6. 3

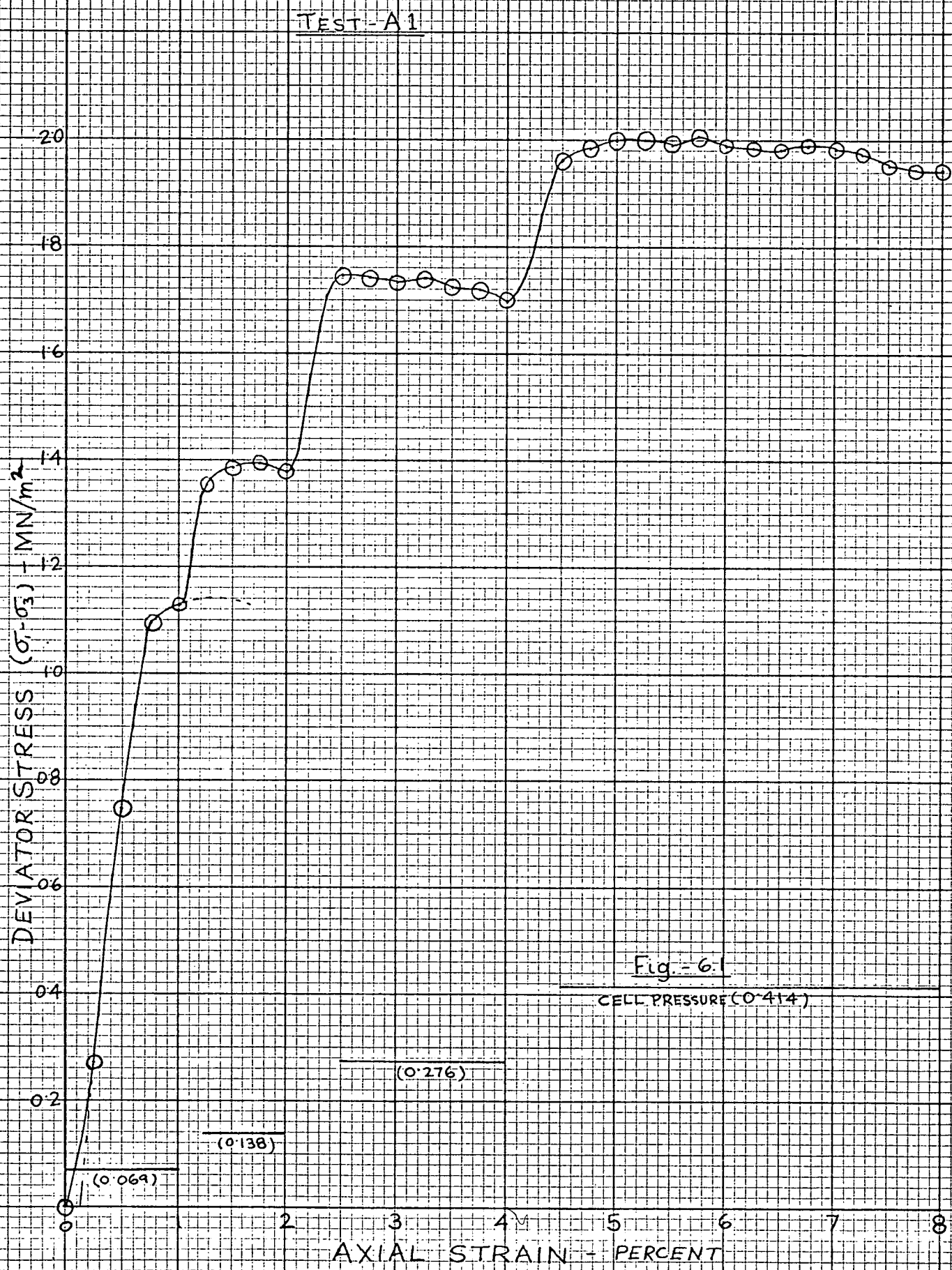
AXIAL STRAIN Vs. STRESS RATIO

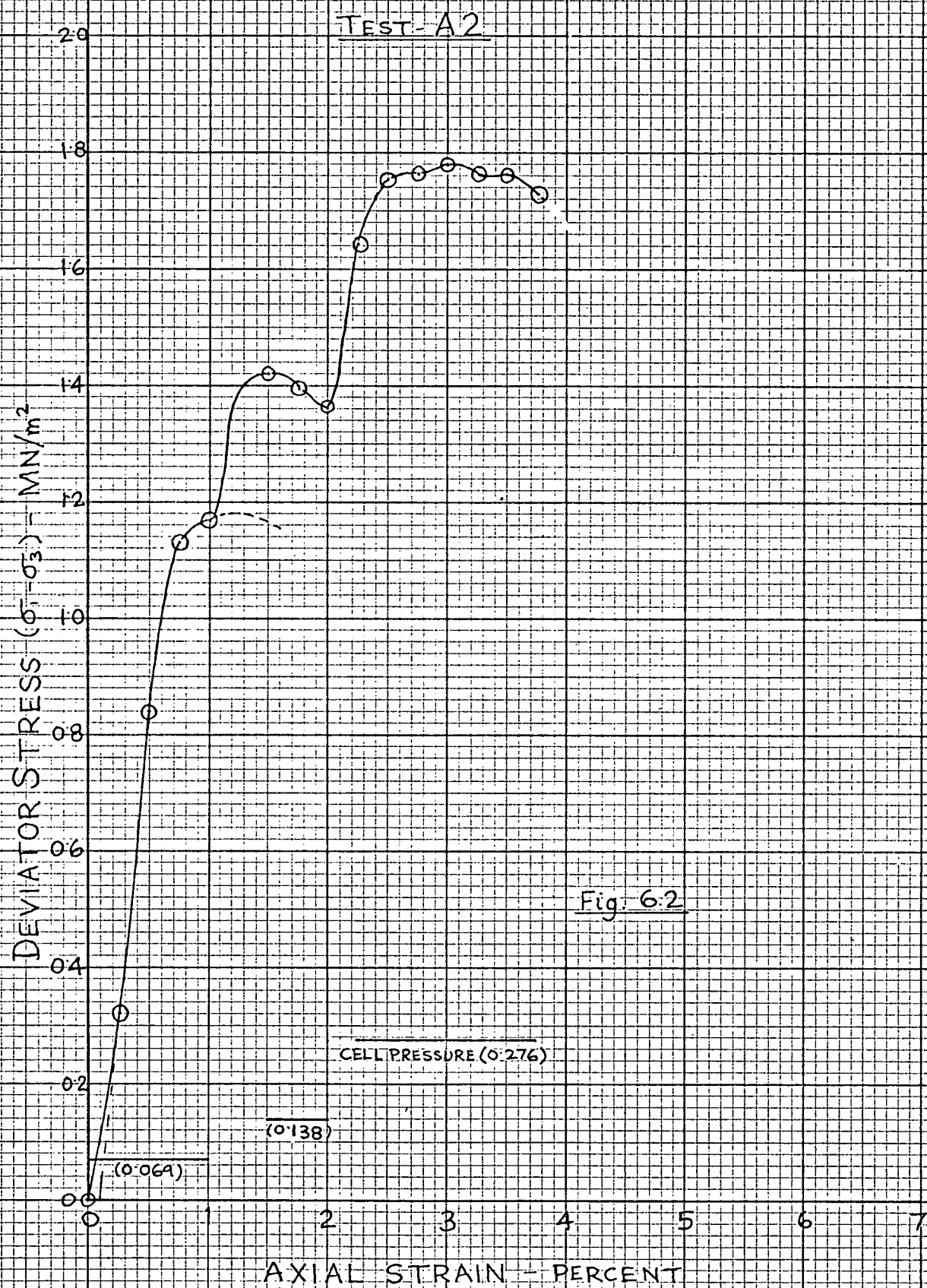
Axial Strain	S T R E S S R A T I O								
	A ₁	A ₂	A ₃	B ₁	B ₂	B ₃	B ₄	P.B.I	P.B.II
0.25	3.94	4.34	2.74	0.52	4.46	2.02	2.75		
0.50	10.81	12.17	6.73	1.55	10.46	5.61	6.94	3.18	2.90
0.75	15.85	16.37	10.29	3.58	<u>15.21</u>	9.57	10.19		
1.00	<u>16.31</u>	<u>16.88</u>	11.74	6.11		12.64	11.26	3.91	4.35
1.25	9.81		<u>11.90</u>	9.18		13.71	<u>11.10</u>		
1.50	10.02	10.27		12.30		<u>14.17</u>	6.46	4.49	5.29
1.75	10.10	10.10	7.49	14.17			6.84		
2.00	<u>9.97</u>	<u>9.85</u>	7.65		9.86	8.76	6.84	4.78	6.09
2.25		5.94	7.75	<u>14.26</u>	8.98	8.94	<u>6.88</u>		
2.50	6.32	6.35	<u>7.70</u>		8.73	8.92		5.07	6.67
2.75	6.30	6.38				<u>8.81</u>	4.63		
3.00	6.27	6.44	4.89			5.59	4.69	5.36	6.96
3.25	6.30	6.38	4.98			5.73	4.76		
3.50	6.24	6.38	4.99			5.78	4.78	5.65	7.39
3.75	6.33	<u>6.27</u>	5.06	8.27	8.14	5.80	4.77		
4.00	<u>6.14</u>		5.12			5.79	4.78	5.94	7.54
4.25			<u>5.14</u>	8.26		5.77	<u>4.72</u>		
4.50	4.73			<u>8.21</u>	7.74	5.73		6.09	7.61
4.75	4.79		4.05			5.78	3.85		
5.00	4.82		4.10	4.79	<u>7.61</u>	5.67	3.90	6.23	7.75
5.25	4.82		4.13	5.33		5.61	3.94		
5.50	4.80		<u>4.18</u>			5.52	3.98	6.52	7.83
5.75	4.84			<u>5.46</u>	4.87	5.50	4.00		
6.00	4.80				4.95	5.43	4.00	6.52	<u>7.83</u>
6.25	4.78				4.98	<u>5.33</u>	4.01		
6.50	4.78				4.94		<u>4.01</u>	6.74	4.57
6.75	4.80								
7.00	4.78				4.93			6.96	5.18
7.25	4.76				<u>4.93</u>	4.19			
7.50	4.72					4.19		7.03	5.36
7.75	4.69					4.17			
8.00	4.69					4.13		<u>7.03</u>	5.43

— end of stage end of test (failure) A = Group A
P.B. = Philip Boyd B = Group B
I = type I, II = type II

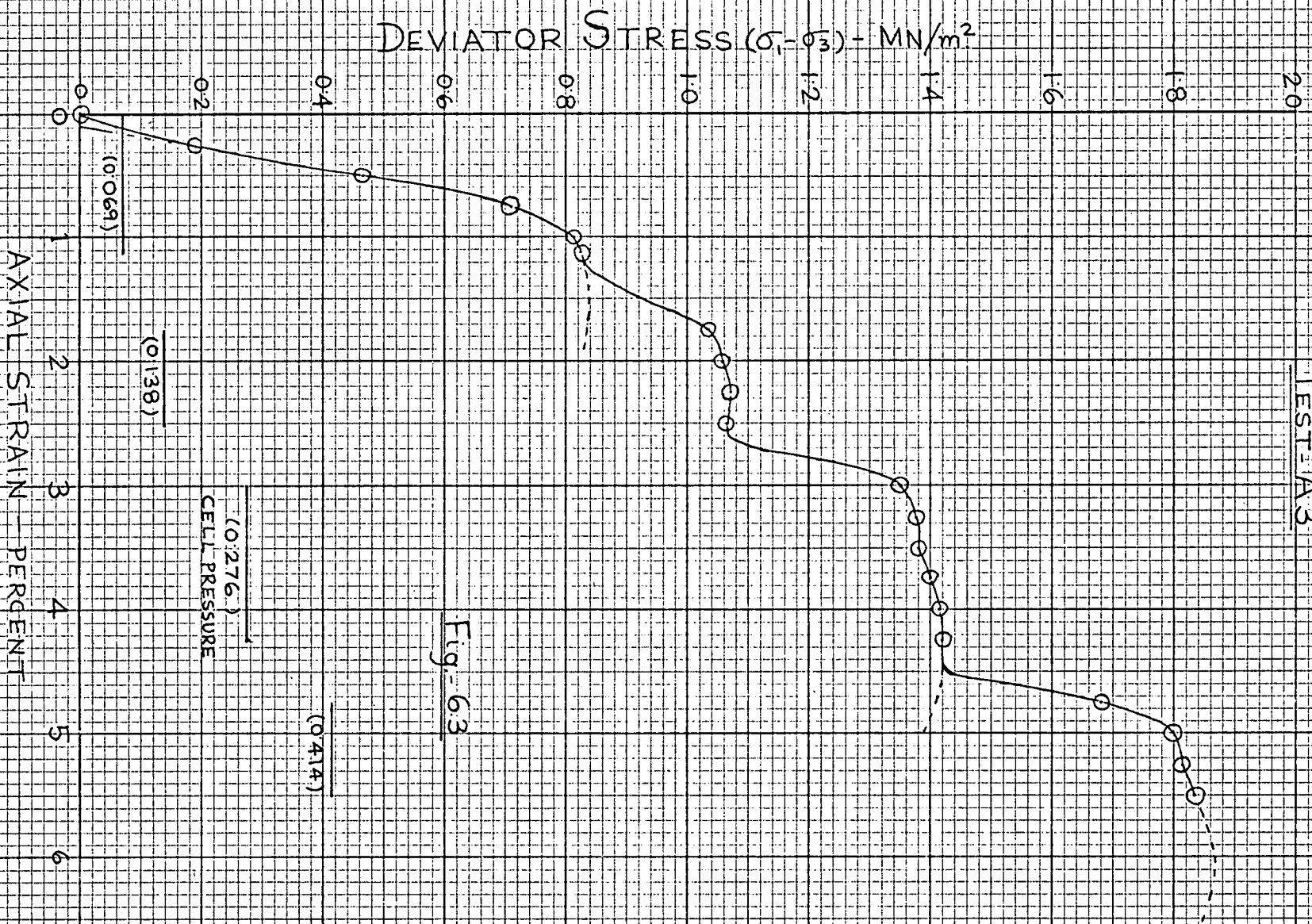
TABLE 6.4 NET ULTIMATE BEARING CAPACITY (q_{nf})

SAMPLE	c_u	ϕ_u	q_{nf}
	MN/m ²	degrees	MN/m ²
A ₁	0.21	40°	27.3
A ₁	0.34	30°	16.4
A ₂	0.195	43°	25.4
A ₂	0.33	31.5°	17.2
A ₃	0.15	38°	15.6
A ₃	0.17	36.5°	14.4
B ₁	0.216	34°	15.4
B ₂	0.15	44.5°	19.5
B ₂	0.295	26.5°	10.0
B ₃	0.18	39°	21.1
B ₃	0.24	33.5°	16.2
B ₃	0.415	23°	12.4
B ₄	0.14	38°	14.6
B ₄	0.18	33°	11.7

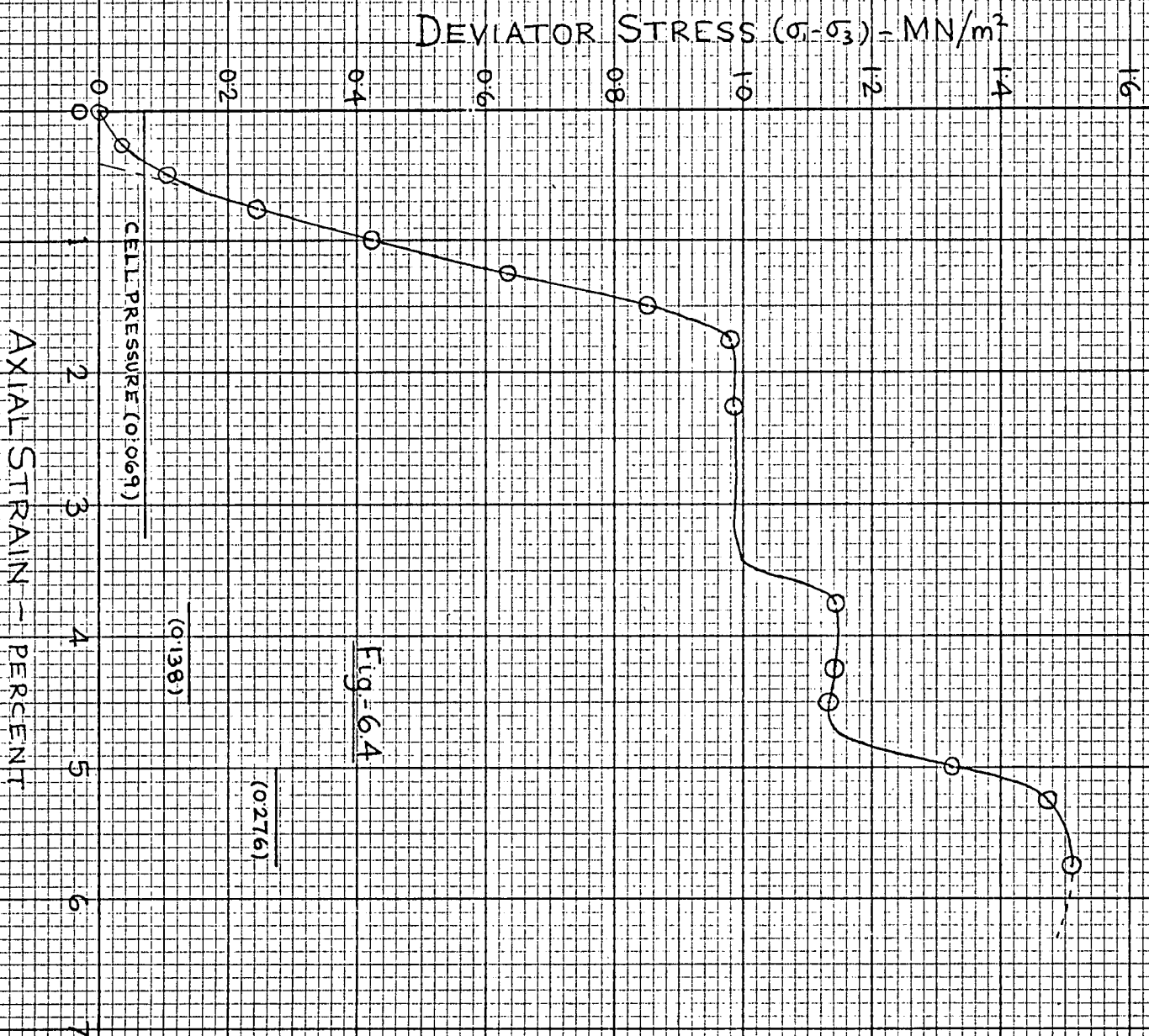


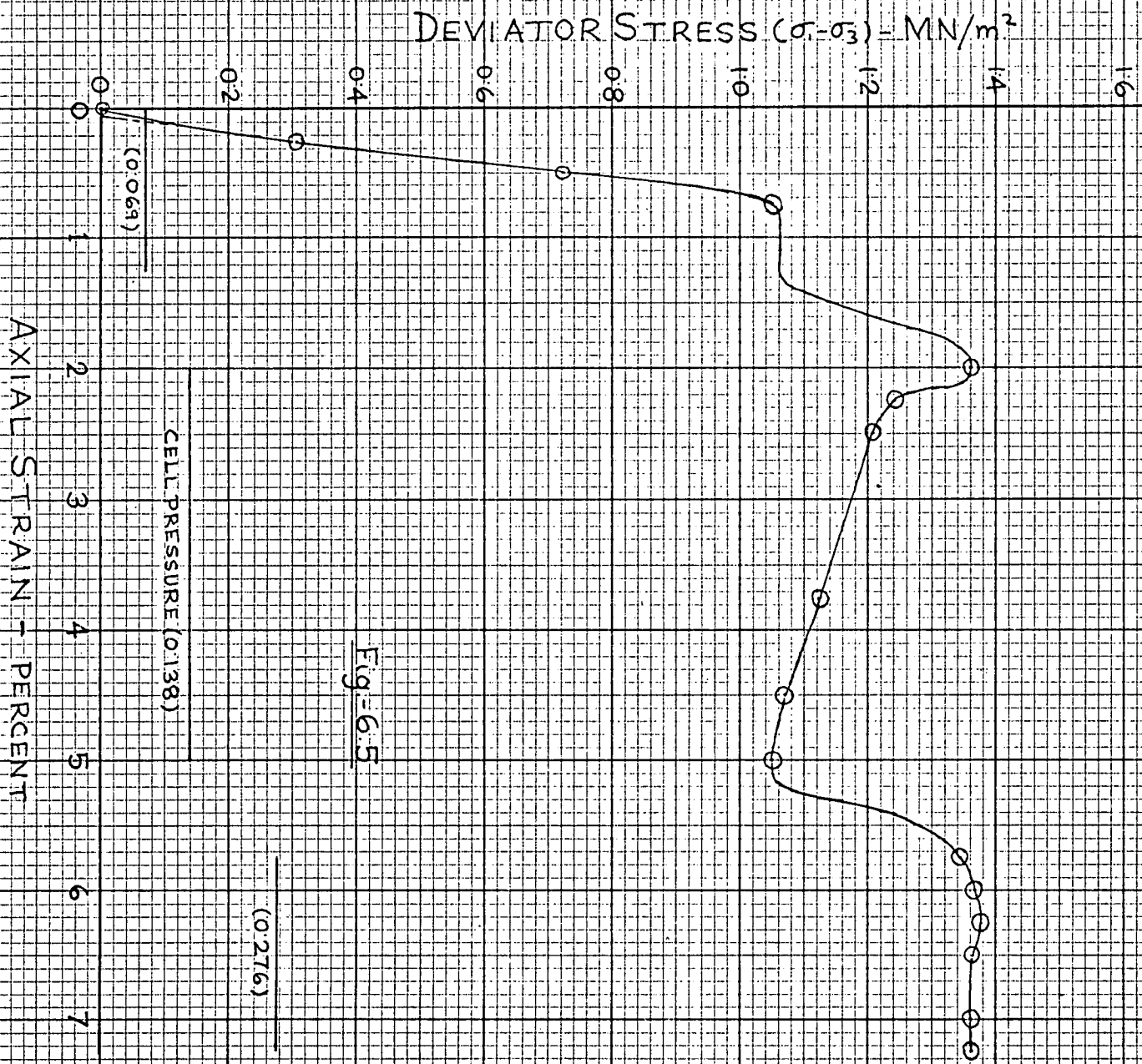


TEST-A3



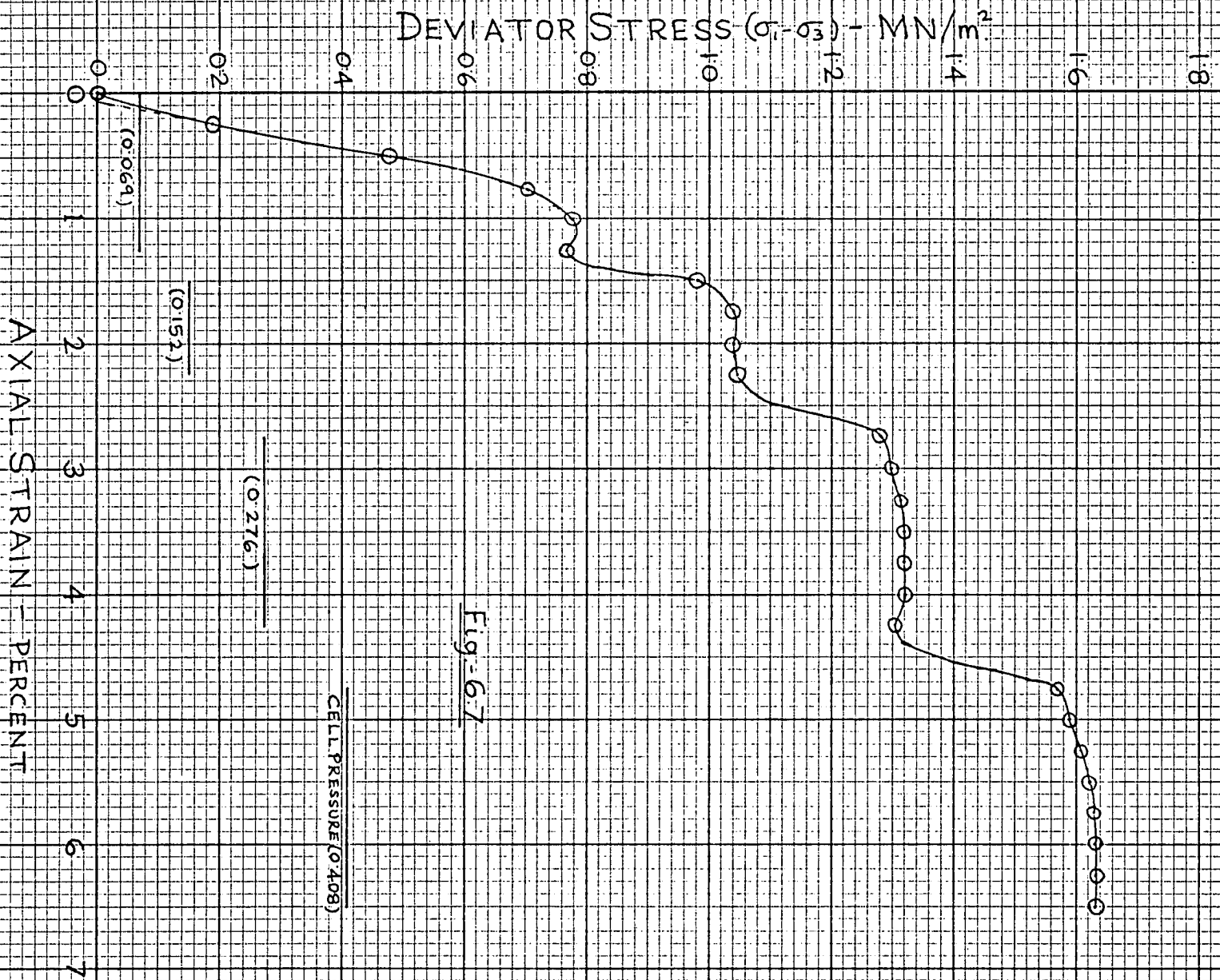
TEST-B1



TEST-B2Fig-6.5



TEST-B4



TEST-A1

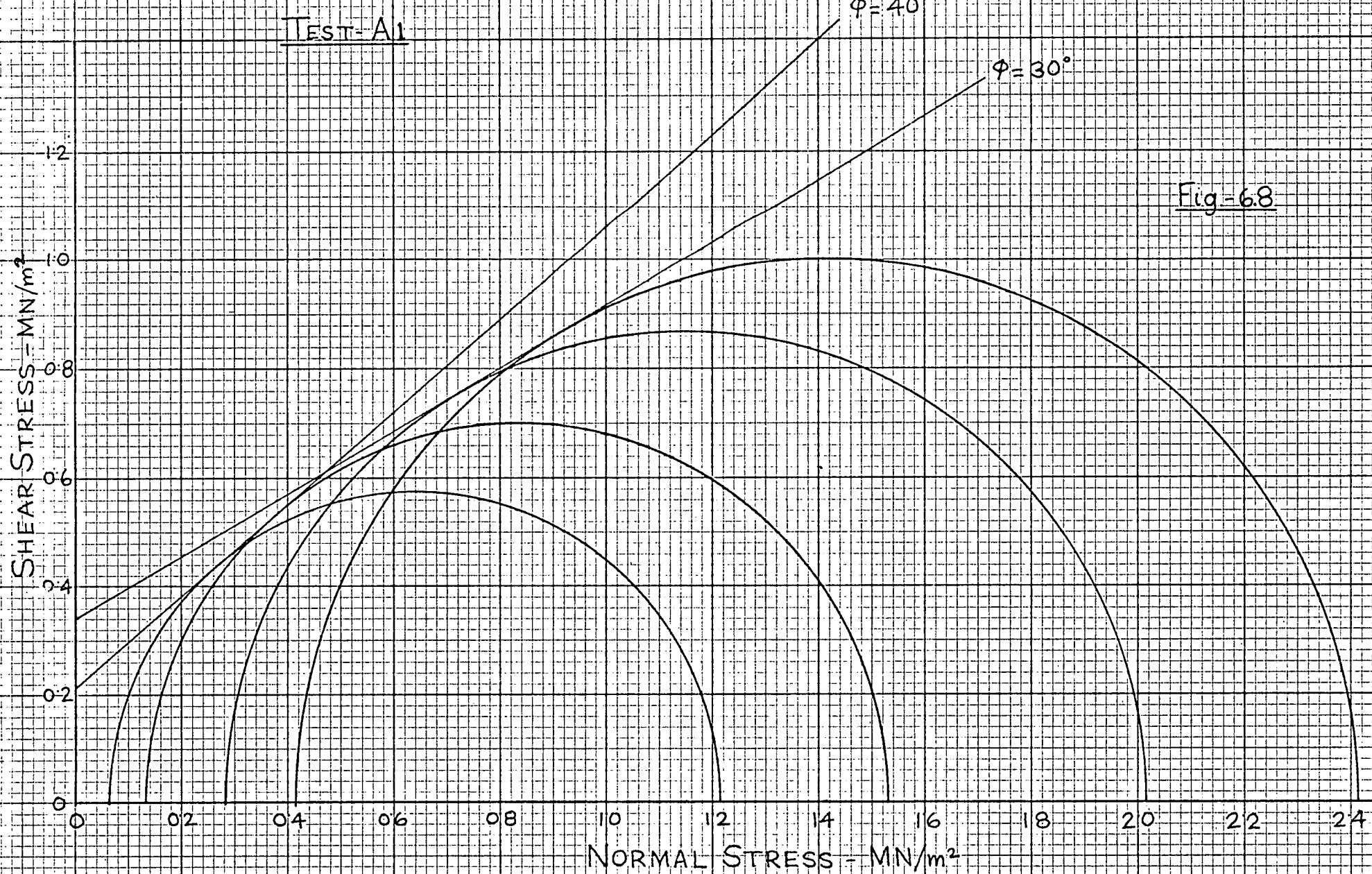


Fig-6.8

TEST-A2

SHEAR STRESS - MN/m^2

$\phi = 43^\circ$

$\phi = 31.5^\circ$

0.2

0.4

0.6

0.8

1.0

1.2

0

0.2

0.4

0.6

0.8

1.0

1.2

1.4

1.6

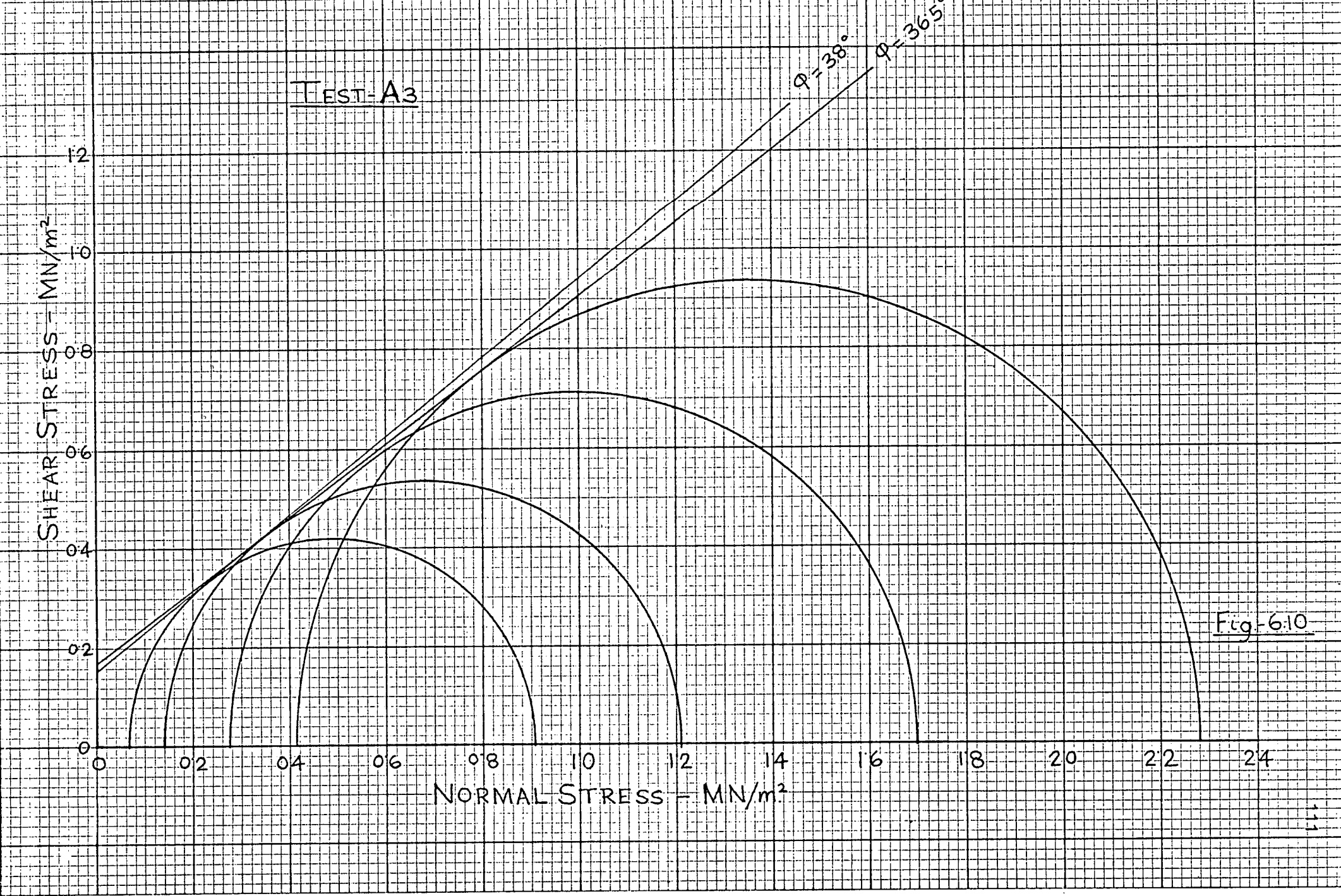
1.8

2.0

2.2

NORMAL STRESS - MN/m^2

Fig-69



TEST-B₁

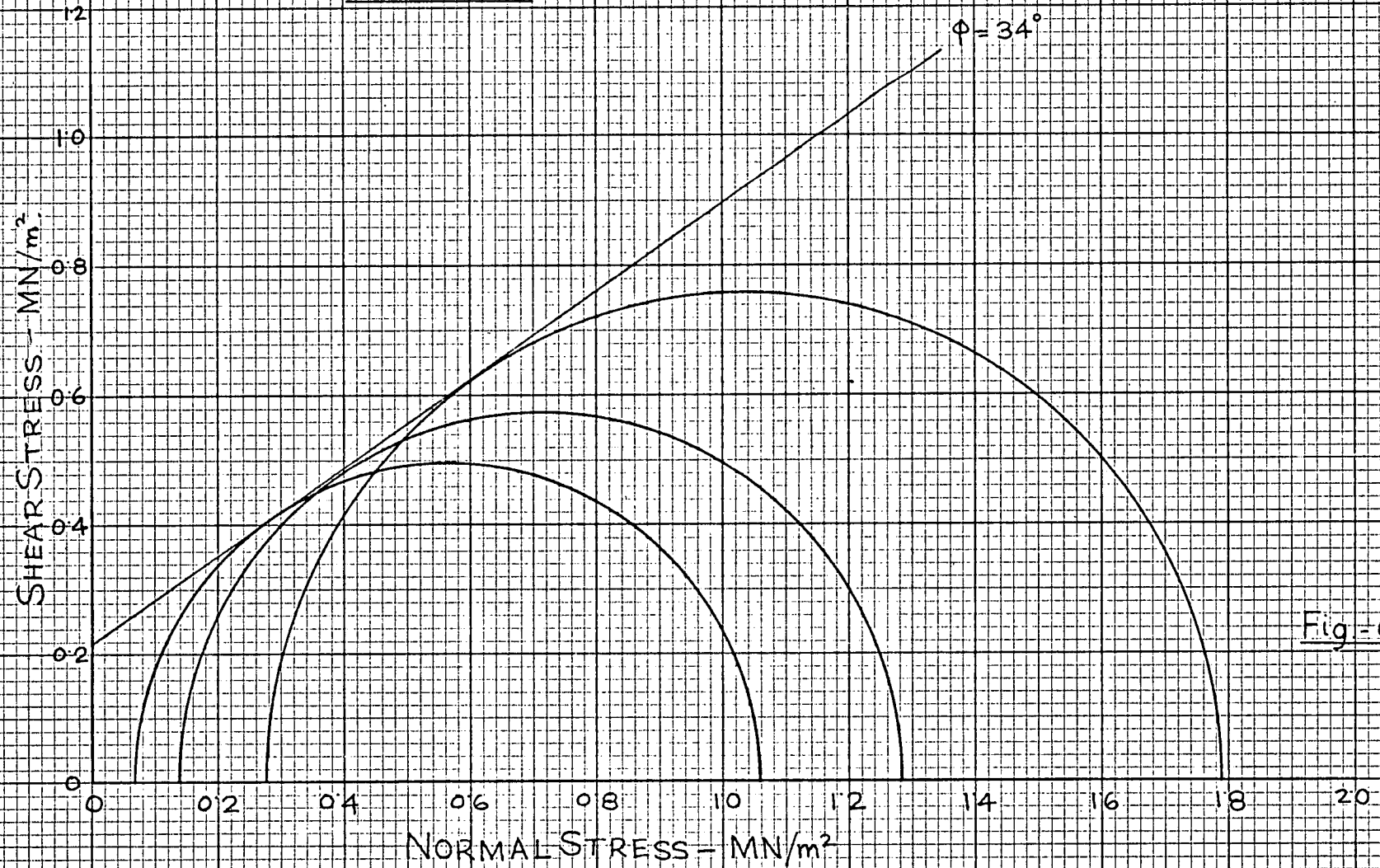


Fig - 6.11

TEST-B₂

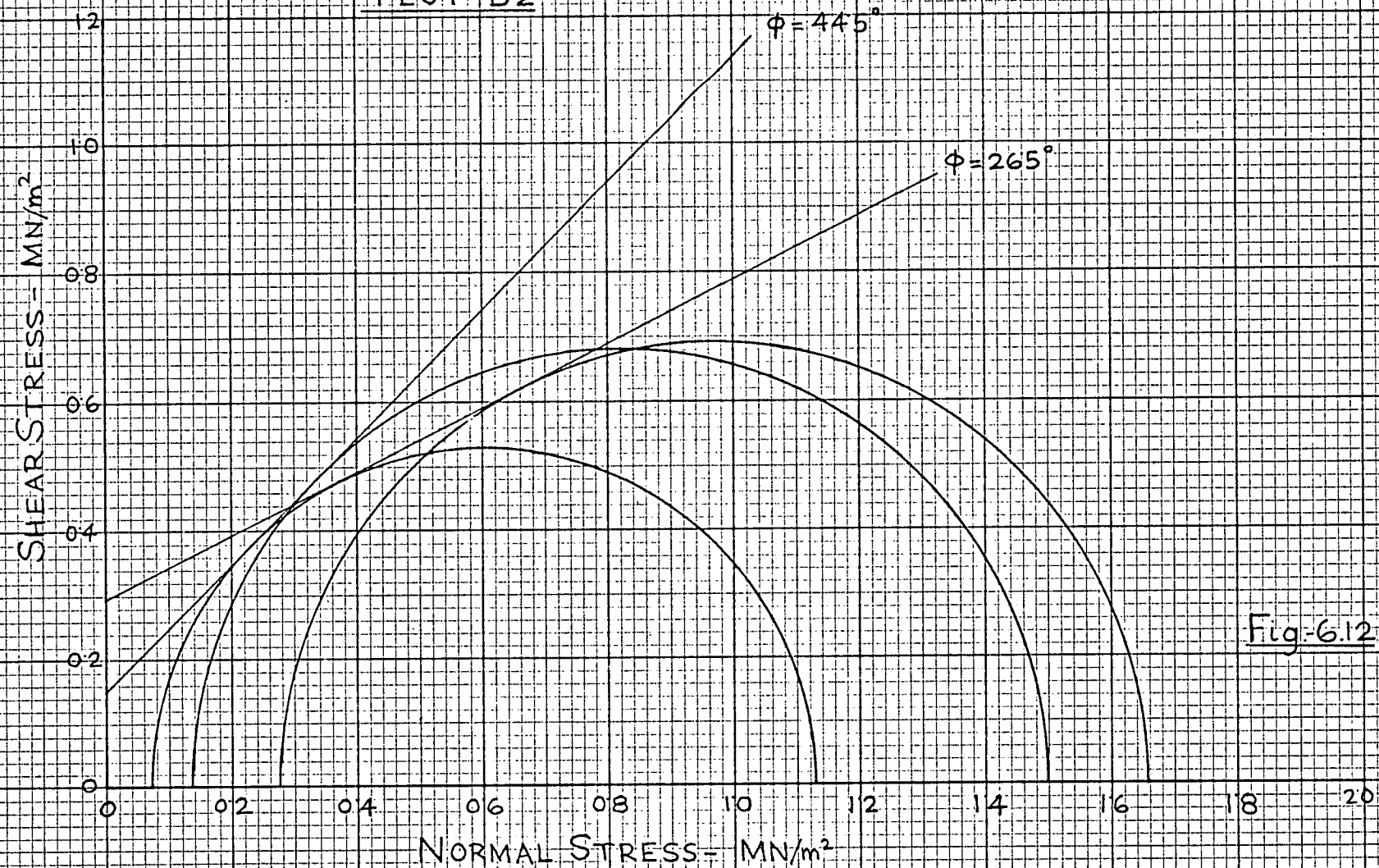


Fig-6.12

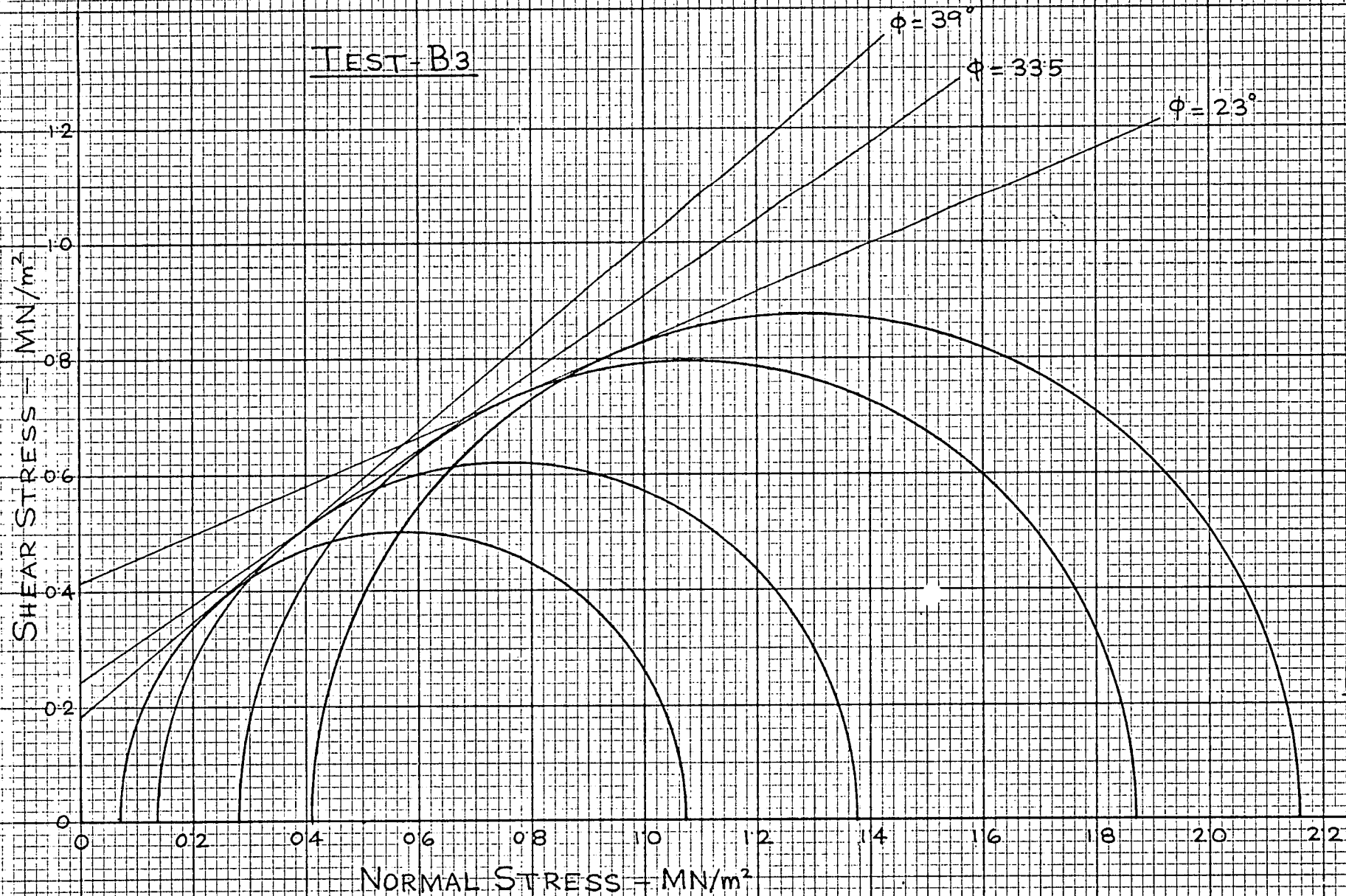
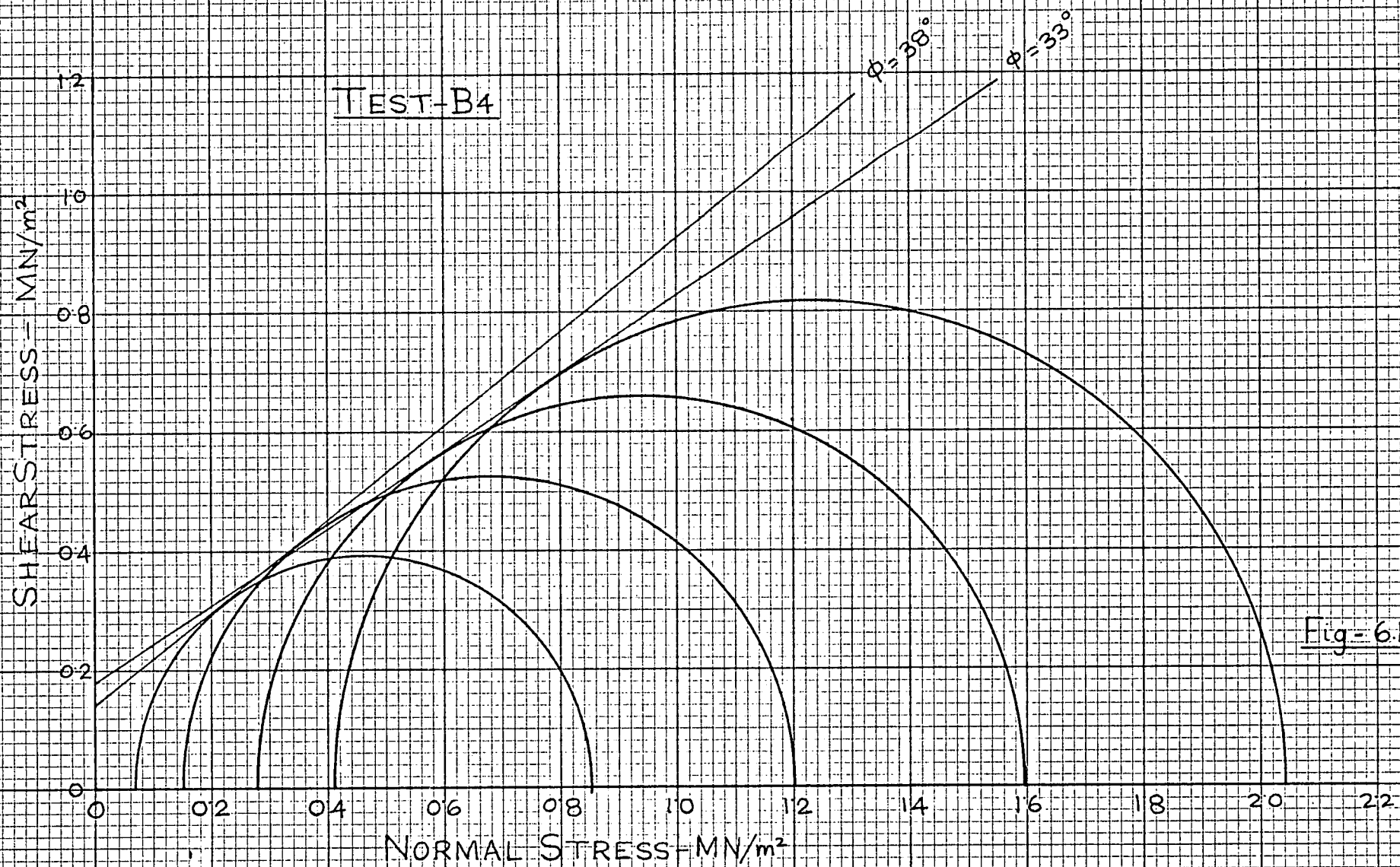


Fig-6.13



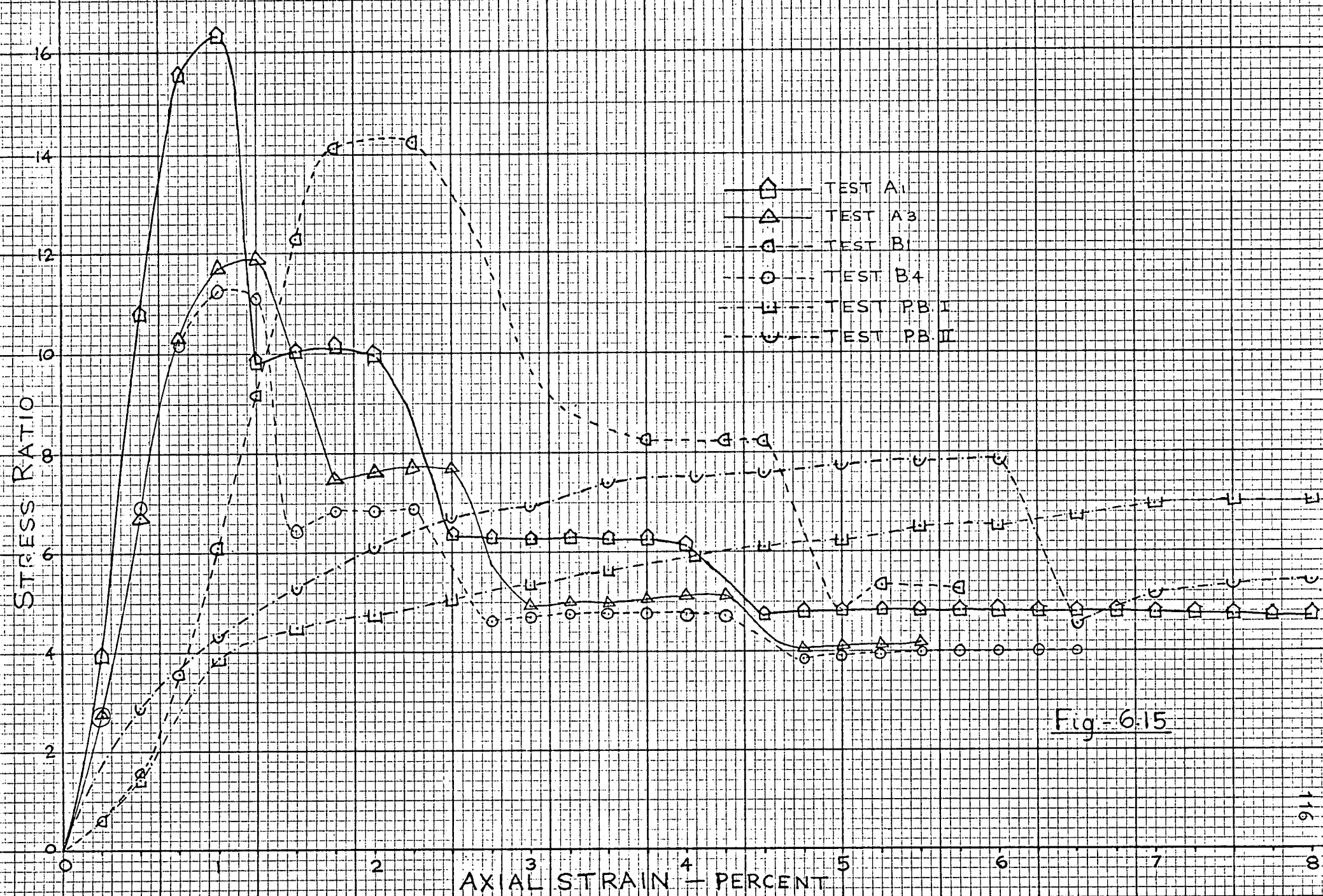
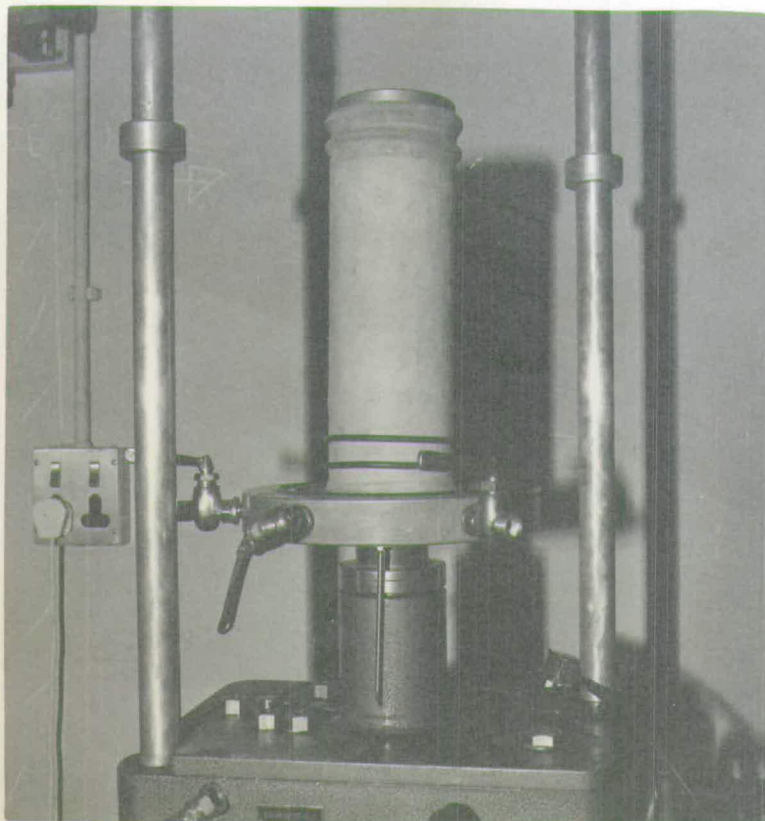
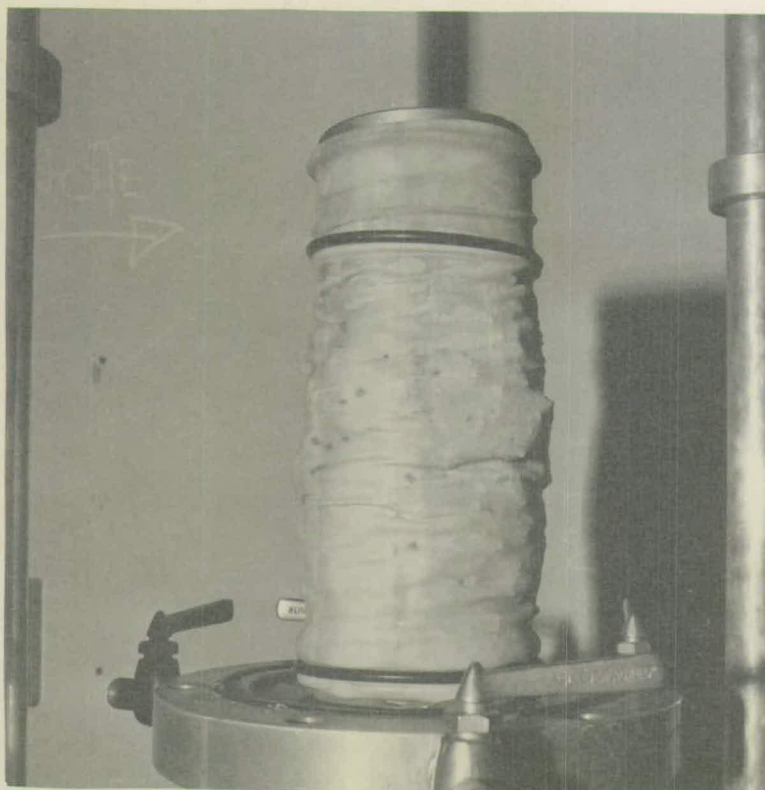


Fig-6.15



TRIAXIAL SAMPLE BEFORE LOADING



TRIAXIAL SAMPLE AFTER FAILURE

CHAPTER 7

REPEAT LOADING OF ROAD PAVEMENTS

7.1 INTRODUCTION

In order to investigate the behaviour of a flexible road pavement, made with brick aggregates, under the action of vehicular traffic, an experiment was designed to simulate the field conditions in the laboratory. A full scale pavement was constructed. The road pavement was then subjected to repeat loading by means of a "Losenhausen" machine which applied the load by a hydraulic jack. The load was transmitted to the road base through a 152 mm (6") diameter circular steel plate placed on the pavement. The response of the plate, and thus that of the pavement surface, to repeated load applications, was recorded by means of two transducers fitted on top of the loading plate. The load was automatically applied by the machine gradually from zero load to full load and the cycle repeated. The load was applied at the rate of approximately 90 cycles per minute. The transducers were connected to a pen chart recorder via an amplifier. More than 100,000 load cycles were applied in each test.

7.2 DESIGN OF THE EXPERIMENT

The design of the experiment involved the following operations:

- (a) Design of the pavement
- (b) design of the road roller
- (c) design of the dimensions of the experimental pavement; and
- (d) design of the wheel load.

7.2.1 DESIGN OF THE PAVEMENT:

A Bangladesh standard design for flexible pavement construction with brick aggregates has been adopted. This consists of providing a compacted layer of fine sand over the prepared subgrade and then providing a layer of brick flat soling over the compacted sand. 50 mm (2") down (nominal size) brick aggregates obtained by hand breaking whole bricks are then spread over the brick flat soling to a height of about 150 mm (6") which is then compacted by an eight ton smooth wheel roller. The aggregates are watered profusely after one or two passes of the roller over the dry aggregates. Then it is rolled starting from one edge and gradually approaching the centre, every time overlapping the rolled strip of the previous rolling. The same procedure is repeated starting from the other edge of the pavement. At least 16 passes are given. The compacted thickness of the layer should be about 114 mm. Another layer of the road base is provided so that the total thickness of the compacted aggregate is about 230 mm (9"). Over this a 50 mm (2") bituminous concrete layer with crushed stone aggregates is provided.

7.2.2 DESIGN OF THE ROAD ROLLER

It was not possible to have an 8 ton smooth wheel roller in the laboratory. Hence a roller much smaller in size but producing the same effect was designed. The design was based on the value of

$$\frac{\text{Load on roll (lb)}}{\text{Width of roll (in) x diameter of roll (in)}}$$

which gives a very good indication of the performance of rollers of different size (Section 9.70, SOIL MECHANICS FOR ROAD ENGINEERS).

Hence this value for an eight ton roller being known, the load on roll for a 304 mm (12") wide and 406 mm (16") diameter roller was calculated. (Appendix G).

7.2.3 DESIGN OF THE DIMENSIONS OF THE PAVEMENT

The road base constructed in the laboratory had to be surrounded by a boundary wall all round. The main object of the design was to make the pavement wide enough so that a 304 mm (12") wide section was obtained in the middle which was free from the boundary effect as would be any spot on an actual pavement. For this purpose the roller is considered as a continuous footing (continuous in the direction of rolling), the width of the roller wheel being the width of the footing. The base of the roller has been considered smooth and the soil ideal. Then the zone of influence of the roller has been found out from "conditions of general shear failure" from Terzaghi (1943). This has been explained in detail in Appendix D. The width of the bed has been determined on this basis. This depends on the value of ϕ . The value of ϕ has been determined and explained in detail in Appendix D.

The width of the roller being 304 mm (12") and the value of ϕ being 44° the width of the bed has been found to be 1.753 metres (69"). The other dimension, i.e. the length of the road pavement has been determined bearing in mind that a 304 mm (12") strip in the middle of the pavement would be tested at three positions using a 152 mm (6") diameter loading plate and was determined to be 1.676 metres (66") as shown in Appendix D.

7.2.4 DESIGN OF THE LOAD

For loading, the wheel load of a typical truck used in Bangladesh has been considered. The tyre pressure of the truck's wheel is 622 kN/m^2 (90 p.s.i.) and the tyre exerts a pressure of approximately $1.1 \times 622 = 684 \text{ kN/m}^2$ (99 p.s.i.) to the pavement. The contact area can approximately be considered as a circular area of 152 mm (6") diameter. Hence a pressure of 684 kN/m^2 (99 p.s.i.) over a 152 mm (6") diameter circular plate simulates field conditions and the load for the pressure has been calculated to be 1272 kg (2798 lbs). This would be the load if it were to be applied on the bituminous surfacing. But the load is being applied on the road base. The load at the road base/surfacing interface has been calculated from the "tables of stresses in a three layer elastic system", after Jones (1962). The reduction factor has been calculated (Appendix E) and has been found to be 0.52. This was rounded off to 0.5 ie. the load was found to be reduced by 50%. Thus the load required is 636 kg.

7.3 CONSTRUCTION OF THE ROAD ROLLER

The roller was made from a 406 mm (16") diameter x 304 mm (12") long steel cylinder filled with concrete and fitted with bearings, etc. A steel frame was made with two I beams and angles. A steel plate was placed on the frame over the roller and lead weights placed on it so that the exact load as calculated in the design could be obtained. The frame of the roller was attached to the forks of a fork lift truck which was used to propel the roller (Plate 7.1). The load exerted by the roller in this position was checked with a load cell to ensure that the roller exerted the correct pressure (on the pavement).

7.4 CONSTRUCTION OF THE EXPERIMENTAL PAVEMENT

In the laboratory a compacted sand layer approximately 304 mm (12") thick was provided in the box 2.621 m x 2.621 m (8'6" x 8'6") to reduce the effect of the strong floor to a practicable minimum. The sand was compacted by a vibrating hammer (Kango hammer). Then a layer of brick flat soling was provided over the compacted sand layer. A guide wall was constructed all round the box over the brick soling to reduce the size of the pavement to 1.753 m x 1.676 m. (6'9" x 6'6") as per design. Then a layer of brick aggregates about 150 mm (6") thick was spread over the brick soling and rolled. Two passes of the roller were given on the dry aggregates and then water was sprayed over the aggregates and at least sixteen passes of the roller were given. Similarly a second layer was provided so that the total compacted thickness was about 230 mm (9"). The amount of water sprayed over the aggregates was the water absorption of the aggregates plus four percent which is intended to remain as free moisture. The rolling was done with the roller specially designed and constructed for the purpose and has been described earlier (in Sections 7.2.2 and 7.3). The compacted road base was kept covered with a polythene sheet to maintain the moisture content constant. A photograph of compaction of road base in progress is shown in Plate 7.2.

7.5 EXPERIMENTAL PROCEDURE

The repeat loading of the experimental pavements was performed by a Losenheim machine. The load was applied by a hydraulic jack which was set up over the test position in the experimental road base by means of portal frames made of steel sections. The

maximum load that could be applied by a small jack was two metric tons, which was adequate for the tests. The lower limit of the repeat load was set at zero and the upper limit at approximately 640 kg. The frequency of the loading could be raised to approximately 90 cycles per minute without jeopardising the smooth functioning of the cyclic loading system. The load and the frequency were gradually increased from zero to the desired level within a few minutes. In each cycle the load gradually increased from zero to 640 kg and then dropped to zero again. The vertical displacement of the loading plate, and hence that of the pavement surface under the load, were recorded by means of two linear displacement transducers. The probes of the transducers were fixed within a few millimetres from the edge of the loading plate and at 180° to each other from the centre of the loading plate. The transducers were supported from a frame which was independent of both the loading frame and the experimental bed. Two other dial gauges were set up in a similar way to check and compare the displacements found by the transducers. The transducers were connected to a pen chart recorder via an amplifier. Recordings of the displacements of the transducers were made at the start of a repeat loading test and then for 50 cycles or approximately half a minute at every 500 or 1000 cycles. At least 100,000 cycles were applied to each test location.

A set of photographs of the loading jack, loading plate, transducers, dial gauges and the recording equipment are shown (Plate 7.4 - 7.5).

7.6 SEQUENCE OF THE REPEAT LOADING TESTS

Four repeat loading tests were performed on two locations for each of the two road bases constructed with group A and group B aggregates.

A road base was constructed with group A aggregates as explained in section 7.4. It was tested at two locations with the designed load of approximately 634 kg. Then the road base materials were taken out from a strip 0.61 metre (24") wide throughout the entire length of the bed at the centre of the pavement. (Plate 7.3).

A second road base was constructed in this space with group B aggregates exactly in the same way as with group A aggregates. This was done to save time, materials and with the assumption that the presence of the group A aggregates outside the strip of group B aggregates would change the boundary effects of the walls around the pavement very little. With this pavement thus constructed with group B aggregates a third repeat loading test was carried on exactly in the same way and with the same load as in the previous two tests. In the fourth test the load was increased to 1200 kg which approximately represents the full wheel load if there were to be no bituminous surfacing over the road base. This increase in load lead to the reduction in the frequency of the loading from approximately 90 to a little over 50 cycles per minute. After 52,500 load applications the load was increased further to the maximum capacity of the 'Losenhausen' machine with the jack fitted. It could be raised to approximately 1667 kg but the rate of loading had to be reduced to 41 cycles per minute. The loading was continued up to a total of 100,000 load applications in that location.

7.7 REPEAT LOAD TEST RESULTS

The mean of the displacements recorded by the two transducers gives the displacement of the pavement surface under the centre of the loading plate. The pen chart recorder recorded the displacements of the two edges of the loading plate where the probes of the transducers were attached. In the recording two peaks in each cycle were obtained. One was for the full load when the displacement was maximum. The other one, the lower peak, was recorded when the load was released, and the pavement surface partially recovered. The pavement surface could not recover fully as the loading was cyclic. By the time it would recover another loading cycle was in progress. After a time there was a gap between the jack and the loading plate due to permanent displacement of the pavement surface under the plate. As a result the loading plate had more room to recover. The jack then was adjusted to avoid impact bouncing which again could alter the situation (especially the lower peak). Hence the graph of the lower peak was not as regular as the higher peak.

Graphs were drawn by plotting the vertical displacements of the pavement surface at the centre of the loading plate against the number of load cycles (i) at full load and (ii) at the maximum recovery of the surface at zero load. (Fig. 7.1-7.4).

7.8 DISCUSSION ON THE GRAPHS

7.8.1. GRAPH A-I (Group A aggregates, test location 1)

The deformation of the pavement surface at full load increases rapidly at the beginning and soon slows down. It increases, flattens out and increases again. Then it falls, rises again, repeats this

behaviour and ultimately remains approximately constant. The deformation at zero load approximately follows the other graph. The recovery of the surface is very low at the very beginning, then it increases gradually but reduces a little at the end when it also becomes approximately constant. The recovery of the pavement surface between the two peaks is approximately 0.254 mm (.010") at the end of the test.

Deformations corresponding to a few particular load cycles are given below:

No. of cycles	1500	5000	17500-25000
Deformation at full load	0.3683mm (.0145")	0.71mm (.028")	1.02mm (.040")
Deformation at zero load	0.20mm (.008")	0.53mm (.021")	.66mm (.026")

No. of cycles	45,000	52,000	78-110,000
Deformation at full load	1.63mm (.064")	1.50mm (.059")	1.9304mm (.076")
Deformation at zero load	1.27mm (.05")	1.09mm (.043")	1.6764mm (0.066")

7.8.2 GRAPH A-2 (Group A aggregates, test location 2)

The vertical deformation or displacement of the pavement surface at full load after only 500 cycles is 45% of the corresponding deformation at the end of the test at 102,500 cycles. At zero load the corresponding percentage is 37.5. The deformation of the pavement surface at full load increases rapidly initially. Then it flattens out, increases slowly again after which it falls a little, rises a

little and then remains fairly constant until the end of the test.

The deformation graph at zero load follows the other graph very well. The recovery of the pavement surface between the two peaks is approximately 0.3048mm (.012") at the end of the test. Deformations corresponding to a few particular load cycles are given below:

(1) No. of load cycles	500	5000	27500	50,000
(2) Deformation at full load	.97mm (.038")	1.35mm (.053")	1.78mm (.070")	1.98mm (.078")
(3) Deformation at zero load	.69mm (.027")	0.99mm (.039")	1.47mm (.058")	1.68mm (.066")

(1) No. of load cycles	60,000	75,000	9500	102,500
(2) Deformation at full load	2.11mm (.083")	2.01mm (.079")	2.13mm (.084")	2.13mm (.084")
(3) Deformation at zero load	1.80mm (.071")	1.73mm (.068")	1.83mm (.072")	1.83mm (.072")

7.8.3 GRAPH B-1 (Group B aggregates, test location 1)

The vertical displacement of the pavement surface under the full load increases sharply initially. Later it slows down and the increase is slow until the end of the experiment. The displacements at zero load follow approximately the other graph and tend to be flatter. The recovery of the pavement surface between the two peaks at the end of the test was about 0.635mm (.025"). Deformations corresponding to a few load cycles are given below

No. of cycles	500	5000	17500
Deformation at full load	0.635mm (.025")	1.8415mm (.0725")	2.718mm (.107")
Deformation at zero load	0.381mm (.015")	1.384mm (.0545")	2.235mm (.088")

No. of cycles	50,000	100,000	
Deformation at full load	3.175mm (.125")	3.708mm (.146")	
Deformation at zero load	2.667mm (.105")	3.023mm (.119")	

7.8.4 GRAPH B-2 (Group B aggregates, test location 2)

This test was performed with an increased load compared with the previous tests. Initially it started with approximately 100% wheel load instead of the 50% of the previous tests. After 52,500 cycles the load was again increased (to approximately 130% of the wheel load). The deformation of the pavement surface rose very sharply at the beginning, then it increased slowly. At about 50,000 cycles there was a sudden increase in deformation of approximately 1.143mm (.045"). This could have been due to the change in the orientation of the particles due to degradation. After the load was increased at 52,500 cycles the deformation of the pavement surface at full load increased a little faster than the situation prior to the increase in the load.

Deformations corresponding to a few load cycles are given below:

(1) No. of cycles	500	2,500	25,000	49,000
(2) Deformation at full load	2.08mm (.082")	2.54mm (.100")	3.63mm (.143")	4.22mm (.166")
(3) Deformation at zero load	1.45mm (.057")	1.78mm (.070")	3.05mm (.120")	3.73mm (.147")

(1) No. of cycles	50,000	75,000	100,000	
(2) Deformation at full load	5.40mm (.213")	6.40mm (.252")	7.11mm (.28")	
(3) Deformation at zero load	4.88mm (.192")	5.99mm (.236")	6.50mm (.256")	

7.9 COMPARISON OF THE GRAPHS

7.9.1 THE EFFECT OF STRENGTH ON THE NUMBER OF LOAD REPETITIONS

The pattern of the graphs are similar in all the cases. Deformation increases very rapidly for a short length of the graphs at the beginning and then it slows down gradually. Ultimately deformation becomes approximately constant. This happened in group A aggregates near the end of the tests (102,500 in A1 and 95,000 in A2). In tests with group B aggregates this did not happen within the 100,000 load cycles but the trends of the graphs show that it would happen after more load repetitions. An extrapolation of the graph B-1 indicates that this may happen at approximately 200,000 cycles. Hence group B aggregates would require approximately double the number of load repetitions compared with group A aggregates whereas the ratio of their crushing strength is 1 to 1.4.

7.9.2 THE EFFECT OF STRENGTH ON THE AMOUNT OF DEFORMATION

From the graphs it can be seen that the road base with group A aggregates undergo much less deformation when compared with group B

aggregates. After 100,000 load cycles the deformation corresponding to zero load for test A2 was 1.83mm (.072") which had been constant since 95,000 cycles, whereas the corresponding deformation for test B-1 was 3.02^{mm} (.119"). The difference in deformation between full load and zero load at the end of the test was 0.3mm (.012") for group A aggregates and 0.7mm (.027") for group B aggregates (Test B-1). The mean crushing strengths of group A and group B aggregates were 22,189 kN/m² and 15,855 kN/m² respectively. In situ modified CBR values obtained were 34 and 21 for group A and group B aggregates respectively. Whereas the ratio of crushing strength and CBR for group A and group B are 1.4 and 1.6 respectively the ratio of total deformation is 0.61 and that for the difference in deformation between full load and zero load at the end of the test is 0.43.

7.9.3 THE EFFECT OF INCREASED LOAD ON DEFORMATION

The load was nearly doubled in test B2 (compared with the first three tests) at the beginning and further increased after 52,500 load applications. The total deformations at full load at 50,000 cycles were 3.18mm (.125") and 5.40mm (.213") for tests B1 and B2 respectively. Total deformations at zero load at 50,000 cycles were 2.54mm(.10") and 4.88mm(.192") for B1 and B2 respectively. However the figures at 49,000 cycles for test B2 was much less e.g. 4.22^{mm} (0.166") at full load and 3.73mm(.147") at zero load. There was a sudden increase in the deformation between 49 and 50 thousand load cycles. Although the load was doubled the deformation at zero load was only 47% more at 49,000 cycles but almost doubled at 50,000 cycles. The load was further increased after 52.5 thousand cycles by another 30%. The total deformation at zero load at 100 thousand load cycles was

6.50mm (.256") whereas the corresponding figure for test B1 was 3.02mm (.119"). Hence it appears that total deformation is proportional to the applied load. The difference in the deformations between full load and zero load at 50 thousand cycles were 0.64mm (.025") and 0.52mm for test B1 and B2 respectively. The corresponding figures at 100,000 cycles were 0.69mm and 0.61mm. These figures for the recovery appear to be much less dependent on the load.

7.9.4 THE EFFECT OF STRENGTH ON MAXIMUM TOTAL DEFORMATION

As stated earlier the deformation both at full load and at zero load became nearly constant after approximately 100,000 load cycles in group A aggregates and 200,000 load cycles in group B aggregates. The deformation at full load was 1.93mm (.076"), 2.13 (.084") and *4.04mm (.159") for tests A1, A2 and B1 respectively. The corresponding figures at zero load were 1.68mm (.066"), 1.83mm (.072") and *3.33mm (.131") for tests A1, A2 and B1 respectively (*estimated by extrapolation of the graph). These figures indicate that the deformations for group B aggregates are approximately twice as much when compared with group A aggregates. The ratios of the crushing strength and in situ modified CBR tests for group A to group B aggregates are 1.4 and 1.6 only. This demonstrates that the deformation of the aggregates under repeat loading decreases more rapidly than the rate of increase of the crushing strength of the aggregates.

GRAPH A-1

VERTICAL DISPLACEMENT OF PAVEMENT SURFACE - mm.

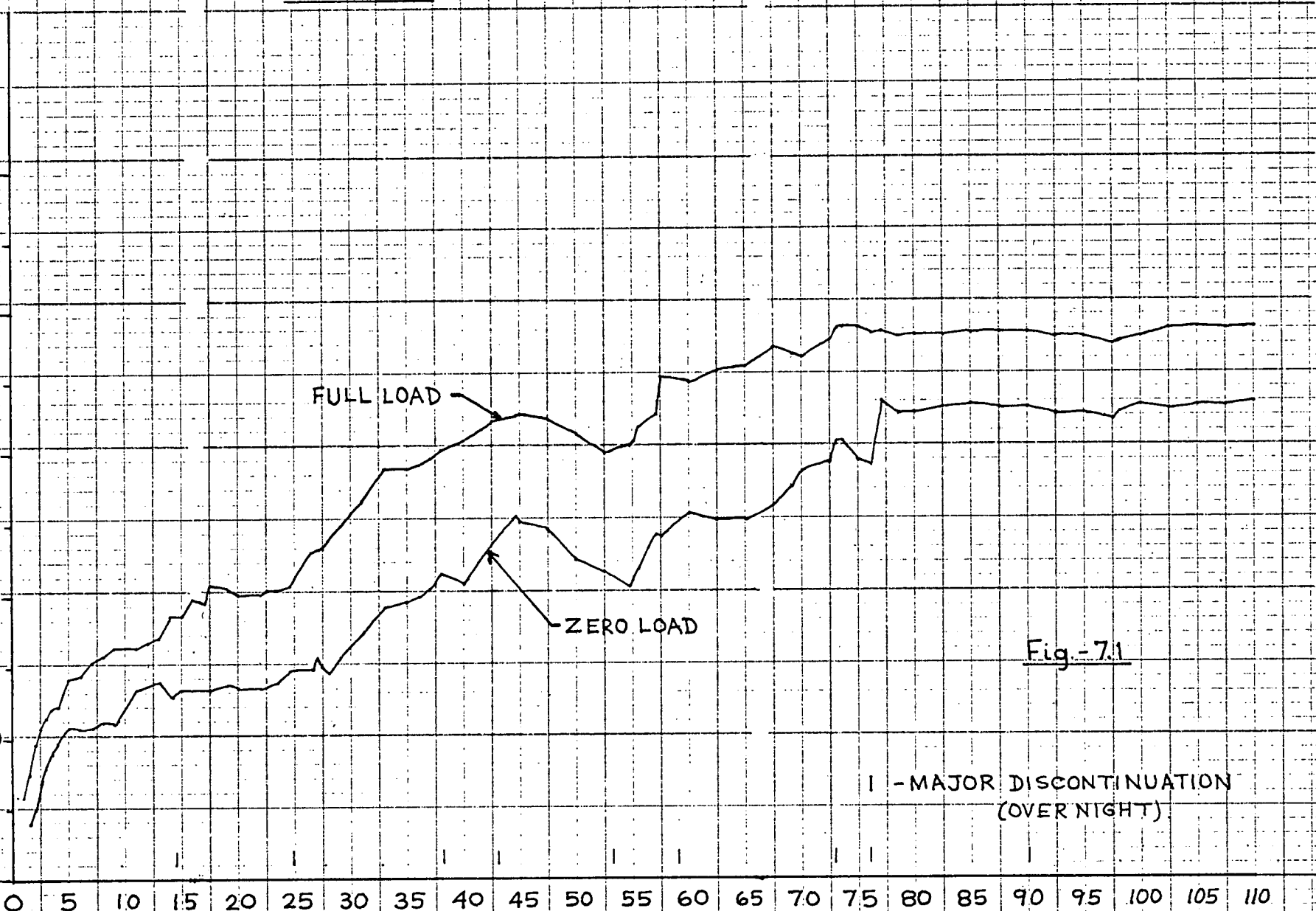
FULL LOAD

ZERO LOAD

Fig.-7.1

I - MAJOR DISCONTINUATION
(OVER NIGHT)

NUMBER OF LOAD CYCLES ($\times 10^3$)



GRAPH A-2

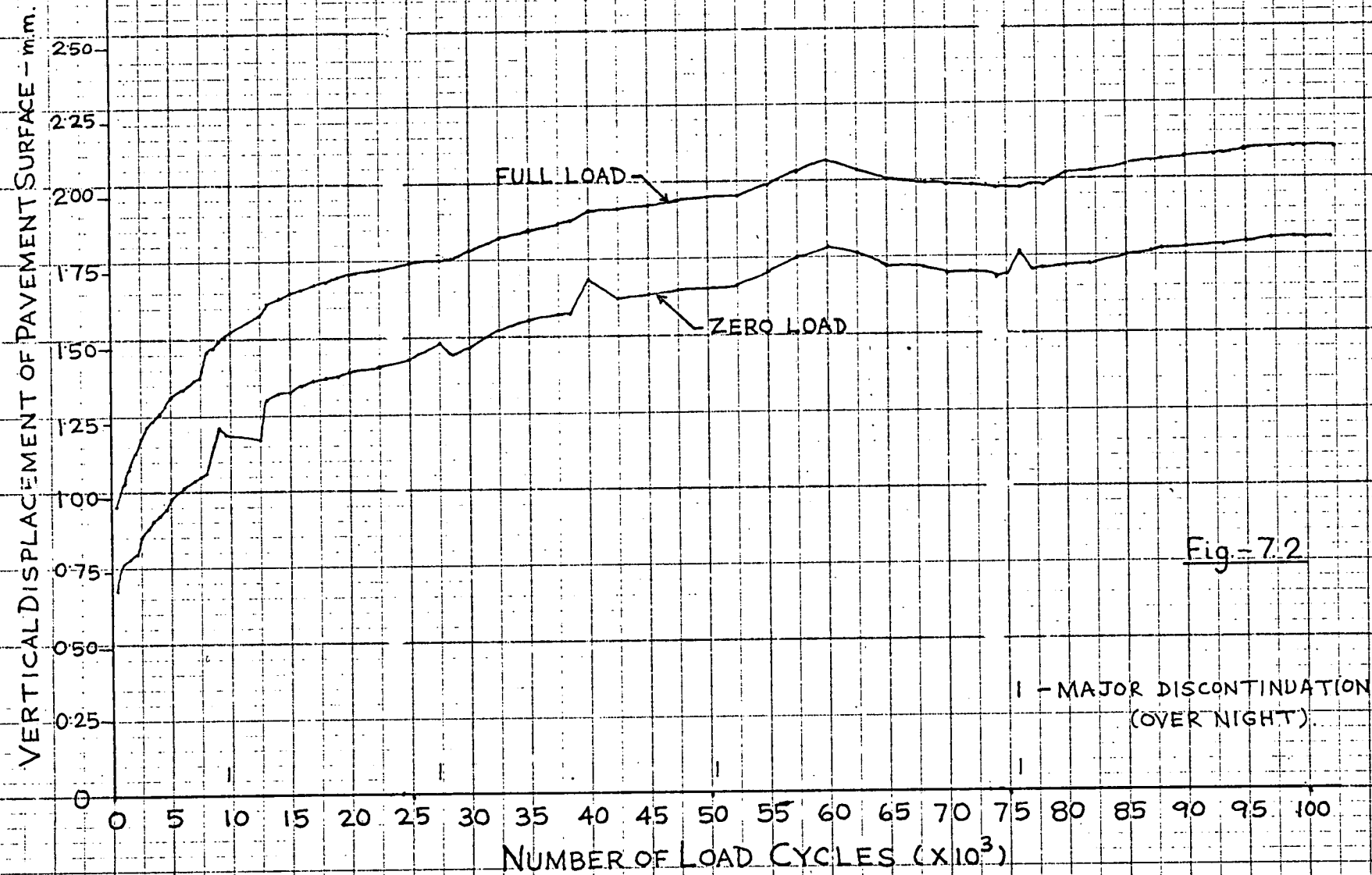


Fig. 7.2

I - MAJOR DISCONTINUATION
(OVER NIGHT)

GRAPH B-1

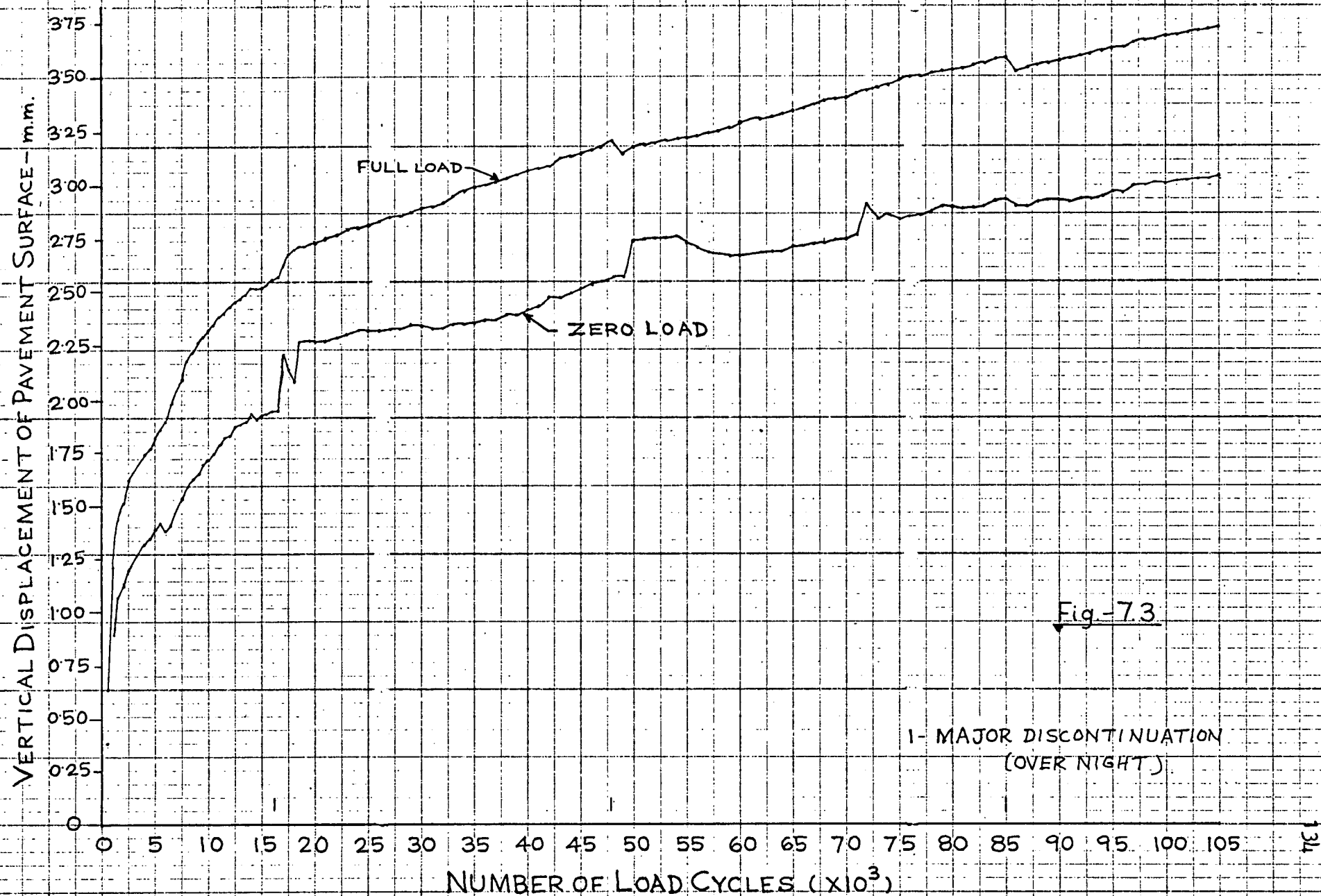
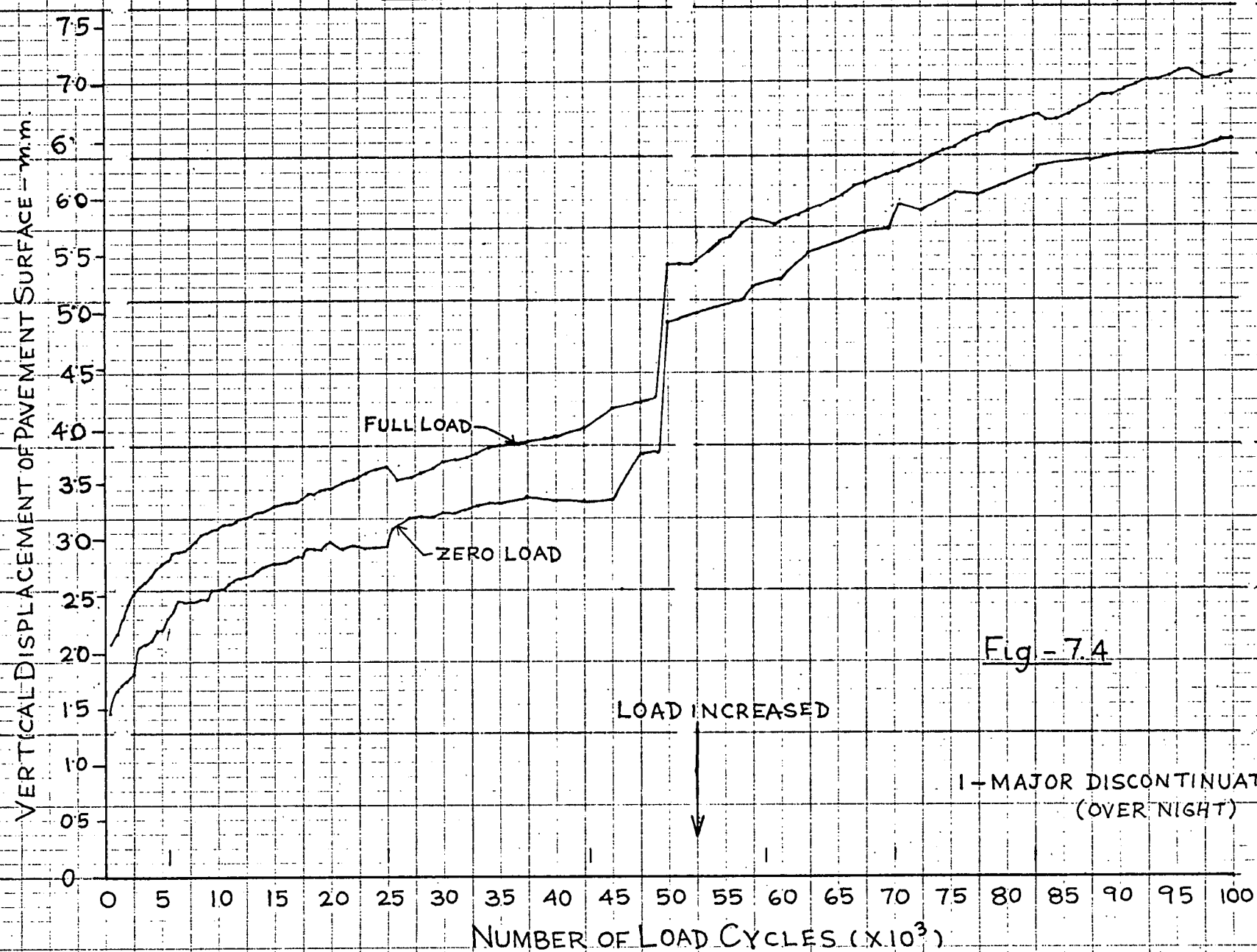
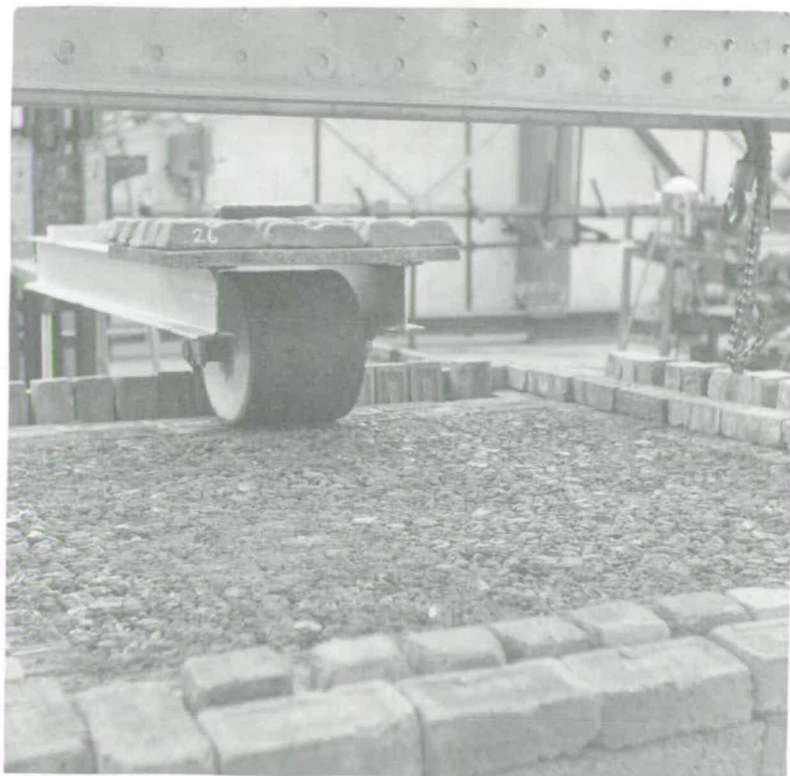


Fig-7.3

1- MAJOR DISCONTINUATION
(OVER NIGHT)

GRAPH B-2

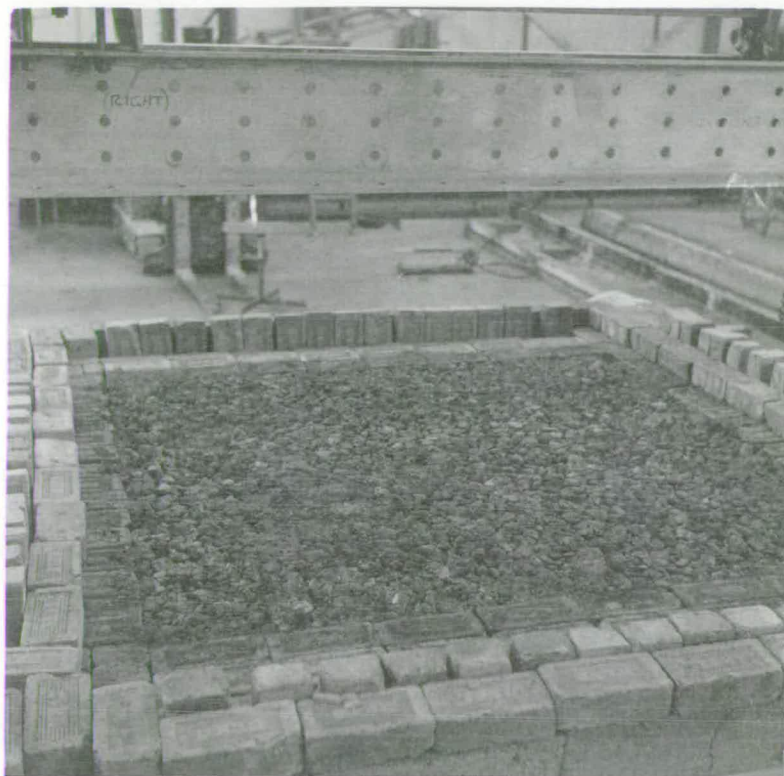




THE ROLLER

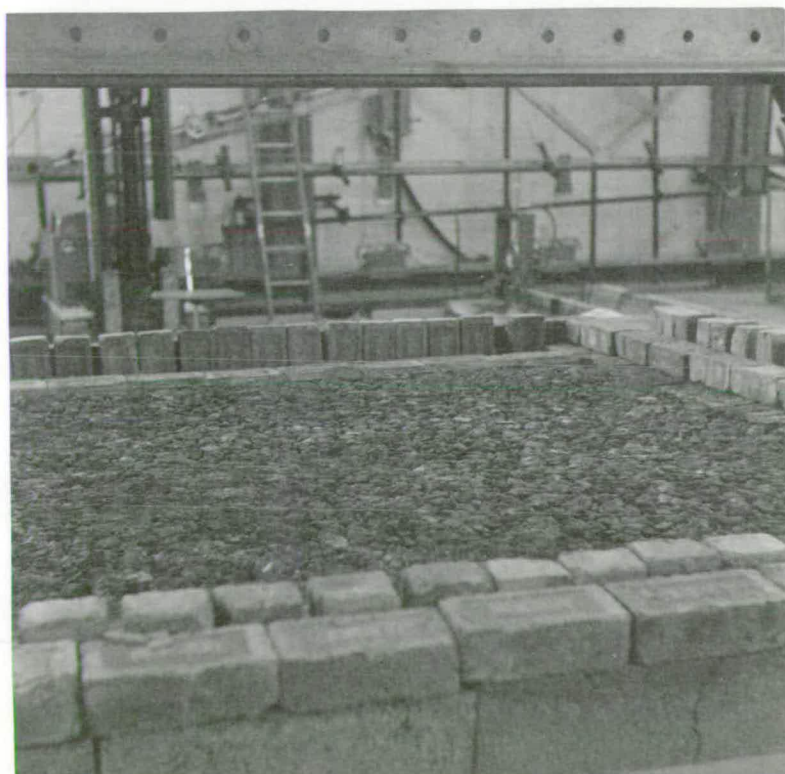


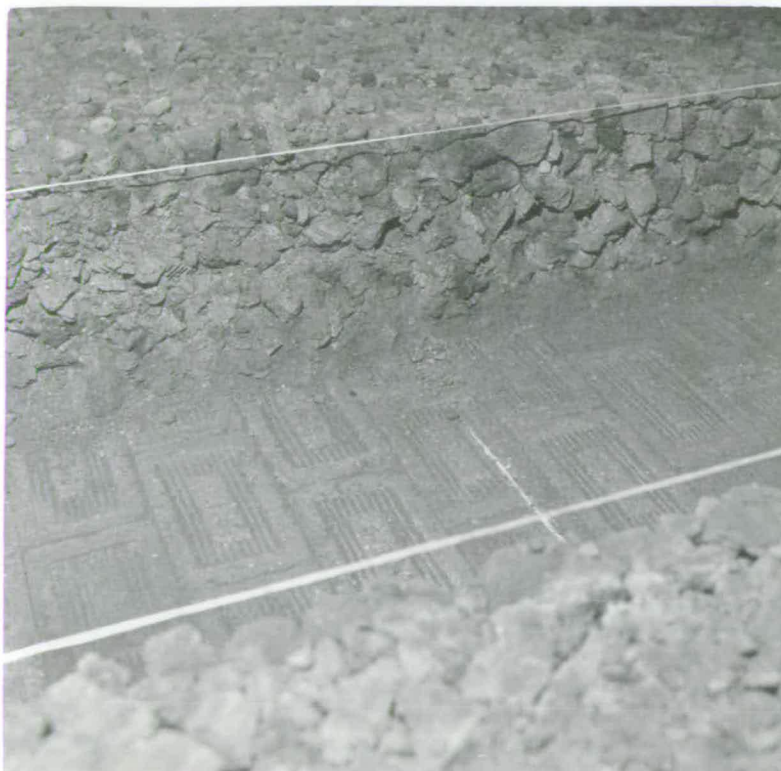
THE DRIVING UNIT OF THE ROLLER



COMPACTION OF ROAD BASE IN PROGRESS

COMPACTION OF ROAD BASE IN PROGRESS

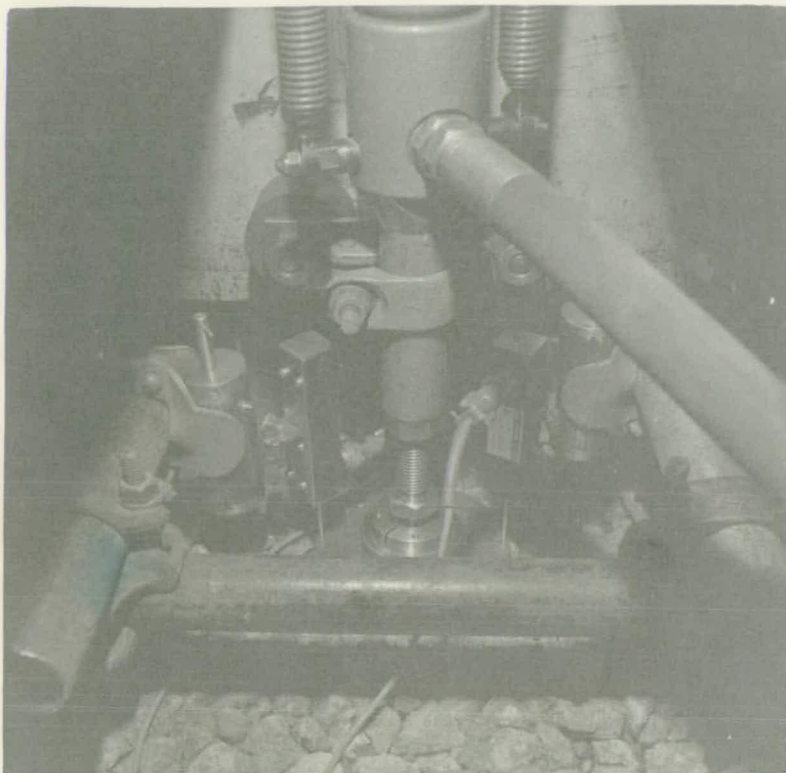




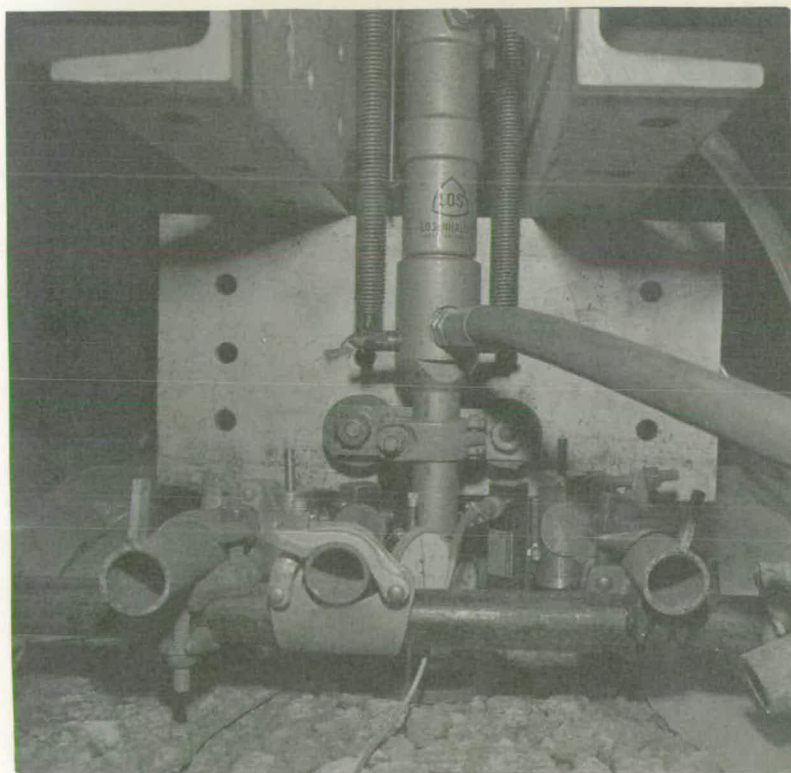
CROSS SECTION OF ROADBASE ON BRICK FLAT
SOLING. VIES ON EITHER SIDE OF TRENCH

CROSS SECTION OF ROADBASE ON BRICK FLAT
SOLING. VIES ON EITHER SIDE OF TRENCH

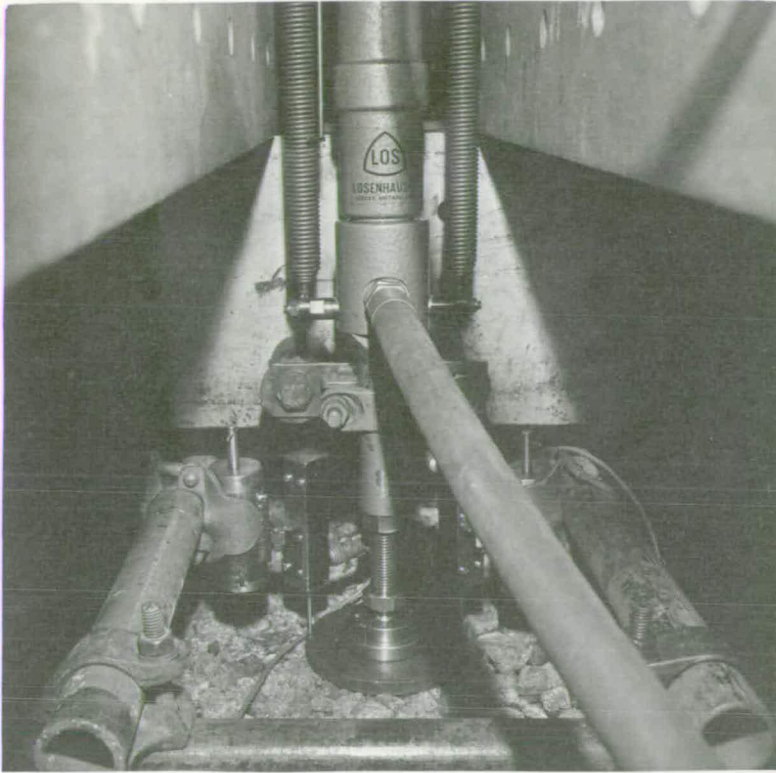




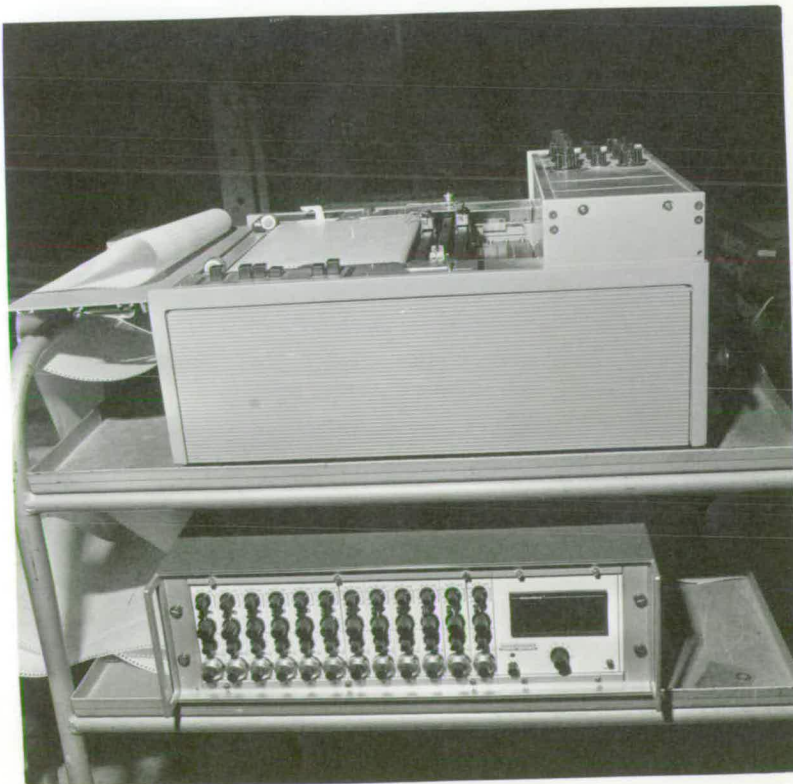
THE LOADING JACK SHOWING THE TRANSDUCERS



THE LOADING JACK SHOWING THE TRANSDUCERS AND
THE DIAL GAUGES



LOADING JACK ON LOADING PLATE



THE RECORDING EQUIPMENT

CHAPTER 8

IMPROVEMENT OF THE STRENGTH OF BRICKS

8.1 INTRODUCTION

The performance of brick aggregates as a road base depends upon many factors. One of the main factors of any aggregate is its crushing strength. Another factor is grading. So far the reason for using brick aggregates and the manufacturing process of bricks in Bangladesh, properties of brick and crushed brick aggregates and the behaviour of brick aggregates as a road base material have been discussed. Since crushing strength is the main factor in an aggregate's behaviour as a road base material, it will be relevant to discuss the possibilities of improving crushing strength of bricks and hence that of the aggregates in this Chapter.

8.2 COMPARISON OF STRENGTH OF BRICKS AGAINST PERFORMANCE

One way of designing a flexible pavement is the California Bearing Ratio (C.B.R.) method of thickness design which is used in Bangladesh and also used in the U.S.A. In the present study two groups of aggregates, group A and group B, having a C.B.R. of 34 and 21 respectively, have been studied. Group A and Group B bricks have an average crushing strength of 3,500 p.s.i. (24182 kN/m^2) and 2,500 p.s.i. (17273 kN/m^2) respectively. (Note: since the C.B.R. method is an American method, the different units in this Chapter will be in Imperial units followed by metric units in brackets). Assuming that a pavement is to be designed for a 9,000 lb (4,082 kg) wheel load, the C.B.R. value for the compacted subgrade is 3, the thickness for the road base required with both aggregate A and aggregate B is calculated from Highway design curves (Woods, 1960) as follows:

<u>Material</u>	<u>Design C.B.R.</u>	<u>Required thickness</u> <u>in (mm.)</u>
Compacted subgrade soil	3	20 (510)
Group A base material	34	5 (125)
Group B base material	21	7 (180)

From the design it can be seen that a compacted thickness of 15 inches (380 mm) of base with group A material should be provided and the thickness of the surfacing needed is 5 inches (130 mm), whereas a compacted base thickness of 13 inches (330 mm) and a surfacing of 7 inches (180 mm) is needed if group B aggregates are used for the base. Two inches (50 mm) of extra thickness of surfacing are needed with group B aggregates and there is a saving of 2 inches (50 mm) of base thickness. The cost of crushed stone aggregates is more than double when compared with that of brick aggregates and in addition the cost of bitumen (/and sand) would make the cost of surfacing even more. Whereas the difference in the cost of group A and group B bricks is not much (no more than 20%). The most important factor to be considered is the shortage of stone and bitumen and the fact that both the materials are imported. It would, therefore, be very helpful for the Bangladesh economy if aggregates with higher strength could be used as this would necessitate use of a lesser quantity of stone and bitumen. With a very high compressing strength, the brick aggregates may also be used in the base course (wearing course) in the form of bituminous grouting which would further reduce the use of stone. There would be extra difficulty of breaking harder bricks which would involve a little more cost of breaking them into aggregates. This

extra cost would be nominal as manual labour is very cheap in Bangladesh. The possible ways of increasing the strength of bricks are discussed in the following section.

8.3 PRODUCTION OF HIGH QUALITY BRICKS

There has been practically no research upon the production of very high quality bricks in Bangladesh. It has already been explained in Chapter 3 that the strength of bricks depends mainly on the temperature and the duration of burning of the moulded bricks. The composition of the brick clay also contributes to the strength.

One way of producing very high strength bricks is to use modern brick manufacturing plants. Bangladesh has very few plants and the quantity of bricks produced in those plants is very negligible compared with the requirement of bricks for the construction of highways in Bangladesh. The establishment of more modern plants especially designed to produce very high strength bricks for road aggregates is a very expensive programme for a country like Bangladesh. Even if there were a plan to establish such plants it would only be possible on a very long term basis.

However, the next possibility is the use of gas which Bangladesh has in abundance. Gas could be used instead of coal which is imported. But the difficulty of using gas is that a very wide network of pipelines would be needed to transport gas to possible brick fields, which is again a very expensive programme. There is a network of pipeline in the country and it is being expanded. The possibility of using gas in areas where there is gas pipelines should be explored. The next problem is designing a device which would enable the use of gas instead of the coal in the kiln.

The immediate improvement that can be made to the strength (and quality) of bricks made in ordinary kilns is simply to increase the burning time and temperature. In Bangladesh coal is used for burning. It is very easy to transport coal to any part of the country. This can be done either by trucks in the dry season or by boat during the rainy season. If more attention is paid to the selection of sites for clay which would be suitable for the production of very high strength bricks, the situation would improve further.

Chaturvedi and Tayal (1958) discussed brick pavements in highway construction with reference to the production of high quality bricks. The main point was careful selection of the clay which included laboratory testing of the same for physical properties such as the particle size distribution, Atterberg limits and shrinkage characteristics. The next point was to raise the temperature of firing to 1000°C and maintain it for a considerable period.

Small size bricks made with clay from certain sites in India which were burnt in an electric muffle furnace at a temperature of 1000°C for seven hours gave a compressive strength of around 10,000 lb per sq. in. ($69,090 \text{ kN/m}^2$). Then full size bricks were moulded from clay from those sites and burnt in Bull's kiln (an ordinary kiln normally used for burning bricks in the region where the tests were carried out). At high temperature vitrification occurred which imparted higher density and strength to the bricks. Beyond a certain limit vitrification resulted in the general softening of the clay and the bricks lost their shape and became porous. Hence it was important to control the temperature and fire the bricks to obtain the optimum amount of vitrification.

In India bricks are normally burnt for 18 hours and coal is fed at the interval of 15 minutes. Maximum strengths of the bricks in the kiln were obtained at 12 hours extra burning when the average strength of 'straight overburnt bricks' was 7,293 lbs per sq. in. ($50,387 \text{ kN/m}^2$). They also reported that bricks possessing compressive strength of 8,400 to 12,600 lb per sq. in. ($58,036 - 87,053 \text{ kN/m}^2$) are produced in Holland by using soil of a particular mechanical composition and by burning them at a higher controlled temperature for a longer duration. However such a control of temperature is not possible in an ordinary Bull's kiln unless some improvements in the design of the kiln and mode of stacking and firing of the bricks is made and tried.

8.4 DISCUSSION

The compressive strength of bricks is a very important factor in the performance of brick aggregates as a road base. Higher compressive strength results in a higher C.B.R. of the aggregates which reduces the thickness of the surfacing required. Bricks of very high compressive strength can be manufactured in ordinary kilns with the use of suitable clay and burning them for an extra period of 6 to 12 hours over the normal period of 18 hours at a high temperature ($1000^\circ\text{C} - 1200^\circ\text{C}$). In India strengths of more than $48,363 \text{ kN/m}^2$ (7,000 p.s.i.) of 'straight overburnt bricks' have been reported by Chaturvedi and Tayal (1958). They also reported that in Holland strengths of bricks between 8,400 and 12,600 lb per sq. in ($58,036$ and $87,053 \text{ kN/m}^2$) are obtained in kilns where the temperature can be controlled accurately. Efforts should be made in Bangladesh to

raise the compressive strength of bricks by raising the temperature to 1000°C - 1200°C and burning them for an extra period.

Suitability of clays for producing high compressive strength bricks should be determined by laboratory testing. Particle size analysis, Atterberg limits and shrinkage characteristics are the physical properties which are tested. Worrall (1968) states that the size distribution of brick clays range from 2.5 mm to less than 2 microns and it is the distribution of particles below 2 microns that is important. It is commonly believed that the fraction below 2 microns consists entirely of clay. According to Worrall(1968) it is not strictly true as some non-clay impurities as small as 2 microns do exist. The working moisture content for a particular process depends on the properties of clay mineral, the surface area, etc. Since the compositions and the size distribution cover a wide range, it is reasonable to expect a wide range of working moisture content. Even with a similar particle size distribution it may vary because of the particle size distribution of the particles below 2 microns. The firing shrinkages are different for different types of clays and so are the firing temperatures because vitrification occurs at different temperatures in different clays. The following table shows the physical properties of some brick clays.

Physical Properties of some brick clays

Clay	Particle size % smaller than 2 μ diameter		Working Moisture content (%)		Linear firing shrinkage (%)		Fired colour for different temperatures
	Range	Average	Range	Average	Temp ^o C	Range	Temp ^o C colour obtained
Etruria marls	24-74	47	15.2-30.2	23.6	1180	0.6-12.1	1000 - Light buff, pink buff, light red, light brown, light chocolate brown, pink cream. 1180 - Dull brown, red brown, purple brown med. red, stone, grayish buff.
Weald Clays	15-85	47	23.8-42.2	30.9	900	0-3.4	850 - Salmon pink, light red-brown, buff.
					1000	2.5-8.6	1200 - Dark red brown
Boulder Clays	30-60	47	17.1-39.6	28.8	900	0.1-4.3	880 - Salmon pink, light brown
					1070	2.5-9.4	1070 - Light chocolate brown, light brown.

(After Worrall, 1968).

The Public Works Department Research Institute of India has drawn up a specification for soils for producing good quality bricks by correlating the physical properties of soils with the quality of the fired bricks based on the results of investigations carried out in the U.P. (a Province in India).

Further research is required to investigate the extent to which the properties and firing behaviour of bricks can be determined from the nature and amount of clay minerals present in the soils. Chaturvedi and Tayal (1958) are in agreement with this view and consider that chemical and mineralogical analysis should be carried out as properties and firing behaviour can be determined to a great extent by the nature and amount of clay minerals present.

Finally it can be stated that there is scope for improving the crushing strength of bricks in Bangladesh and the extent of the improvement can only be found out by carrying out research in this field.

CHAPTER 9

SPECIFICATION FOR ROAD BASE WITH BRICK AGGREGATES

9.1 INTRODUCTION

Brick aggregates are used as a road base material in Bangladesh. The aggregates are made from 'slightly overburnt' bricks by breaking them manually. For making the aggregates only the maximum size is specified. The bricks are broken into aggregates by placing them on a boulder (stone) and then striking them with a hammer. The bricks are first halved, then quartered and the process continues until the aggregates are small enough to comply with the maximum specified size. In the process aggregates ranging from the maximum specified size to less than 75 microns are produced. No particular grading of the aggregates are specified due to the difficulties of sieving such huge quantities of aggregates at the construction site as would be involved in a road construction. The grading is expressed as for example "2" (50 mm) down" which would mean that the aggregates would consist of all particles sizes from passing a 50.8 mm (2") sieve to smaller than 75 microns (No. 200 sieve). There is no specific mention of any particular grading or range of gradings such as laid down in the "Specification for road and bridge works" (HMSO, 1969) which is followed in the U.K.

9.2 COMPARISON OF BRICK AGGREGATES WITH AGGREGATES USED AS ROAD BASE IN THE U.K.

In the present study two groups of aggregates have been studied for their behaviour under repeat loading and tested for modified C.B.R. test, triaxial tests, etc. Group A and Group B aggregates

would approximately represent aggregates made from 'slightly over-burnt' and 'first class' bricks of Bangladesh respectively. Group A and Group B aggregates have C.B.R. values of 34 and 21 respectively. These are values obtained from an in situ C.B.R. test with modified plunger as described in Chapter 5, Part II. There were no surcharge weights applied. The C.B.R. values would be a little higher with a surcharge weight. Group A and Group B aggregates qualify only as sub-base Group B and sub-base Group C respectively of Table 7 of the 'Specification for Road and Bridge Works' (HMSO, 1969). The range of gradings for sub-base groups B and C corresponds to Clause 804. In the following table the range of gradings of the aggregates of Clause 804, 808 (Wet-mix macadam) and group A and group B aggregates are given for comparison.

B.S. Sieve Size		% Passing					
		Clause 804	Clause 808	Group A		Group B	
				original	after comp-action	original	after comp-action
75 mm	3"	100	-				
50 mm	2"	-	100	100	100	100	100
38 (40)	1½"	85-100	95-100	71	84	86	90
19 mm	¾"		60-80	28	34	37	58
10 mm	⅜"	45-100	40-60	15	21	19	41
5.0 mm	3/16"	25-85	25-40	11	15	13	30
2.4 mm	No.7	-	15-30	av.7.5	11	9	20
600μ	No.25	8-45	8-22	3	5	4	8
75μ	No.200	0-10	0-8	av. 1	2	4	2

From the Table it can be seen that the gradings of both Group A and Group B hand broken aggregates are much too coarse compared with those of clause 804 or 808. Brick aggregates being susceptible to crushing under rolling the grading changes and after compaction group B aggregates nearly conform to the grading requirements of clauses 804 and 808. The grading of Group A aggregates becomes finer after compaction yet stays coarser than both clause 804 and 808. The aggregates for clause 808 should consist of crushed rocks and crushed slags in accordance with Clause 2602 which requires the material to have 'ten percent fines value' (B.S. 812) of not less than 5 tons (50 kN) (10 tons concrete of 6000 lbs/in² or more - 28 days). The T.P.F.V. for group A and group B are 60 and 30 respectively, which means that only group A aggregates qualify..

9.3 BRICK AGGREGATES AS ROAD BASE IN INDIA AND ITS COMPARISON WITH THE PRACTICE IN BANGLADESH

India uses brick aggregates and the Indian Road Congress has drawn up specifications and a code of practice for water bound macadam which are given below.

1. Overburnt brick metal - shall be made out of overburnt bricks or brick bats and free from dust and other foreign matter.
2. Physical requirements of coarse aggregates for water bound macadam:

Sl.No.	Type of construction	Test	Test Method	Requirement
1.	Sub-base	Los Angeles Abrasion Value or Aggregate Impact Value	IS: 2386 (Part IV) -do- or IS:5640**	Max. 60% Max. 50%

Sl.No.	Type of construction	Test	Test Method	Requirement
2	Base course with bituminous surfacing	Los Angeles Abrasion Value or Aggregate Impact Value	IS.2386 (Part IV) -do- or IS.5640**	Max. 50% Max. 40%

** Aggregates like brick metal which get softened in presence of water should invariably be tested for impact value under wet conditions in accordance with IS. 5640.

In the present study Group A and Group B have Aggregate Impact Values of 45 and 55 percent respectively when tested according to British Standard.

The size range as laid down in the Indian specification that may be compared with that used in Bangladesh is grading No. 3 which is given below.

Grading No.	Size range	Sieve designation (IS: 460)		% by wt. passing the sieve
3	50mm to 20mm	63mm	(2½")	100
		50mm	(2")	95-100
		40mm	(1½")	35-70
		20mm	(¾")	0-15
		10mm	(¾")	0-5

The standard mentions that for crushable types of aggregates the brick metal, kankar and laterite, the above grading is not so relevant and need not be strictly enforced, but the material should generally be within the size range indicated.

Screenings to fill voids in the coarse aggregates shall generally be of the same material as the coarse aggregates. Screenings of type B shall be used with coarse aggregates of grading 3. It also says that the use of screenings may be dispensed with in the case of

crushable type coarse aggregates like brick metal, etc. However the grading for the screening type B is given below.

Grading Classification	Size of Screenings	Sieve designation (IS. 460)	Per cent by weight passing the sieve
B	10mm	10mm ($\frac{3}{8}$ ")	100
		4.75mm (3/16")	85-100
		150 μ (No.100)	10-30

It also suggests the use of a binding material to prevent ravelling of water bound macadam (WBM) which will consist of a fine grained material possessing Plasticity Index (Ip) value of up to 6 when the WBM is being used as a base with bituminous surfacing.

It further suggests that the application of binding material may not be necessary where the screenings consist of crushable type material. Approximate quantities of coarse aggregates and screenings required for 75 mm compacted thickness of WBM base course for 10 m² are given below.

Coarse Aggregates			Screenings	
Grading	Size range	Loose quantity	Crushable type	
3	50 - 20mm	0.91 to 1.07m ³	Grading Classification and size	WBM Base Course
			Not uniform	.33 to .35m ³

This indicates that the quantity of the screenings is between 32 to 36% of the coarse aggregate by volume, i.e. in every 100 m³ of coarse aggregate, 32 to 36 m³ of screenings are to be provided. The unit weight of the screenings would be much higher than that of

the coarse aggregates and hence the quantity of the screenings could be as high as 50 percent by weight. The resulting grading of the mixture of the coarse aggregates and the screenings if mixed in the above proportion would be as detailed in the following table.

Sieve Size	% passing	fraction of aggregates	% of the total mixture	Overall grading of the mixture
63mm	100	coarse aggregate	67	100
50mm	95-100			97-100
40mm	35-70			57-80
20mm	0-10			33-40
10mm	0-5			33-37
4.75mm	85-100	screenings	37	28-33
150µ	10-30			3-10

This grading is much coarser compared with those of the British sub-base type 2 (Clause 804) and Wet mix macadam (Clause 808) except for the finer fractions from 5 mm down. The grading looks more like the British dry-bound macadam (Clause 809) which uses 50mm (2") or the 40mm (1½") nominal sizes for the coarse aggregate and the fine aggregates all pass the 3/16" (5mm). The coarse aggregate is laid to a thickness of 3 to 4 inches and given two passes by a smooth wheel roller. Then fine aggregate is spread on it to a thickness of approximately 1 inch and vibrated into the voids of the coarse aggregate. In the Indian practice, the coarse aggregates are dry rolled until the aggregates are partially compacted with sufficient

void space in them to permit application of the screenings. The screenings are then applied gradually over the surface uniformly in successive thin layers (3 or more) accompanied by dry rolling and brooming. After this the surface is copiously sprinkled with water, swept with hand brooms and rolled.

In Bangladesh hand broken brick aggregates are first dry rolled, watered profusely and then rolled to full compaction as described in detail earlier (Ch. 7, Sect. 7.2.1).

9.4 SUGGESTIONS FOR IMPROVEMENT OF THE SPECIFICATION OF THE ROAD BASE IN BANGLADESH

From the different tables depicting the various gradings of aggregates used in the U.K., India and Bangladesh, it is evident that group A and group B aggregates (representing Bangladesh aggregates) suffer from a lack of fines in their gradings, especially in the stronger aggregates like group A. It has already been discussed in Chapter 5 (Part I) that the presence of more fines definitely improves the packing of the road base and hence the bearing capacity. Under the compacting effort of the roller weaker aggregates like group B undergo more crushing than those of group A. After rolling the group B represents a grading nearer to the grading of the British wet mix macadam (Clause 808). It is very difficult to comply with an exact grading or even a range of gradings for aggregates which are crushable under rolling. It is because the grading of the aggregates will change after rolling and it is the grading of the compacted aggregates which is important. This change may not be uniform throughout a section of a road base under construction. This change can, however, be predicted to some degree of accuracy as

described in Chapter 4. The Indian Road Congress specification states that the gradings recommended therein are not altogether relevant and need not be strictly followed for brick aggregates as they are crushable.

The situation can be improved in Bangladesh by adding more fines to the aggregates, especially the stronger ones. This can be done in two ways without involving too much effort of sieving the aggregates to different sieve fractions, making up the required grading and mixing them together. Firstly by breaking the bricks to make aggregates as usual and then breaking some more bricks to a very small size say " $\frac{3}{8}$ " or 3/16" down to make the screenings which can be added to the original aggregates to the required quantity. The other way is to sieve the hand broken sample with a " $\frac{3}{8}$ " or 3/16" screen and mix the coarser fraction with the finer in the desired proportion. The second method obviously is better in that it provides the exact amount of the fines whereas there may be a little variation in the percentage of the finer fraction in the first method as it would be determined from samples. The grading of the British wet mix macadam requires 40 - 60 percent passing the 10 mm ($\frac{3}{8}$ ") sieve and 25 - 40% (by weight) passing the 5 mm sieve. The percentage passing 10 mm ($\frac{3}{8}$ ") sieves of group A and group B aggregates even after compaction by rolling are 21 and 41 respectively. The percentages passing the 5 mm (3/16") sieve for group A and group B are 15 and 30 respectively. The Indian standard requires 33 to 37 percent to be passing the 10 mm and 28 - 33% passing the 5 mm sieve. It can be seen that group B aggregates may be acceptable from the grading point of view but group A aggregates need more fines.

If we consider both the British and the Indian grading and compromise 40% passing the 10 mm sieve would reasonably satisfy both the standards. To obtain 40% passing the 10 mm sieve size in the final aggregate after compaction of the group A aggregates, the percentage of the same before compaction can be found out from the sieve size ratio equation of the group given in Chapter 4, which is $y = 0.762 x^{0.936}$ where x and y are the original and final sieve size ratios. In this case $y = \frac{100-40}{40} = 1.5$. From the equation $x = 2.062$, hence the % passing would be $32.7 \approx 33\%$. Originally we had an average of 15% and hence another 18% of the total aggregates by weight of aggregates passing 10 mm should be added to the original sample. This would involve only a single size of sieve which could be made up from wire meshes of the required sizes (10 mm or $\frac{3}{8}$ ") and the whole operation would involve little extra cost for sieves and additional labour. It is to be noted here that the above percentages are by weights and may easily be converted to volume by determining the unit weights of the aggregates of both the whole sample and the screenings. The screening can either be mixed up with the original sample or it can be introduced in the same way as in the Indian practice.

In the present study it has been observed that group B aggregates perform just adequately as road base (as stated in Section 6.5) and hence aggregates much weaker than group B aggregates should not be used. It has also been shown that there is correlation between the different properties of the bricks and the aggregates. Namely the crushing strength of bricks can be estimated from the relative

densities of the aggregates on an oven dried basis. Group B aggregates have a relative density of 1.77*and hence brick aggregates having relative density of less than 1.77*should not be used. In Bangladesh brick supplies are taken for road construction on two classifications - 'picked jhama' (selected slightly overburnt) and 'first' class bricks and their crushing strengths are determined from samples at central laboratories. If their crushing strengths are correlated with their relative densities (*oven dried basis), crushing strengths of the brick supplies can be checked at sites by determining the relative densities of the bricks or the aggregates. This would ensure better control over the materials.

It would be better practice to group the aggregates made from the two classifications mentioned above and use them separately instead of the present practice of breaking all bricks together. This would reduce the thickness of the road base when aggregates made from 'slightly overburnt bricks' are used.

The suggested minimum value of the relative density on an oven dried basis of the brick aggregates relates to aggregates prepared from Scottish bricks and hence this value should be determined for brick aggregates and bricks in Bangladesh.

CHAPTER 10SUMMARY AND CONCLUSIONS10.1 OVERALL CONCLUSIONS

Brick aggregates are crushable aggregates as they undergo degradation during rolling for the construction of a road base. This is due to the very low strength of brick aggregates in Bangladesh. There is a scope for improving the performance of brick aggregates by improving the strength of the bricks from which they are made. Their performance can also be improved by using a dense grading which may not naturally occur by the usual manual breaking of the bricks into aggregates, especially when the bricks tend to be stronger. There is a lack of fines and hence to make the grading a dense one more fines should be added.

10.2 BRICK MANUFACTURING IN BANGLADESH

The strength of brick aggregates depends on the bricks from which they are made. In Chapter 2 the manufacturing process of bricks in Bangladesh has been described. Bricks are moulded from clay soils manually, dried and fired in kilns. The strength depends on the soil, the temperature and the duration of firing. Bricks have their maximum strength when they are fired between 1000° - 1200°C until vitrification occurs and the bricks start to become mis-shapen. These slightly overburnt bricks are suitable for making aggregates. Engineering properties such as crushing strength and water absorption and the correlation between the properties have been reported for six groups of bricks.

10.3 PROPERTIES OF BRICK AGGREGATES

In Chapter 3 the properties of brick aggregates such as ten per cent fines value, aggregate impact value, relative densities (oven dried basis, saturated and surface dried basis, and apparent), water absorption, angularity number, elongation index, flakiness index and sieve analysis have been determined according to relevant British Standards. Then intercorrelation between the various properties of the bricks and the aggregates of different groups have been discussed. It was observed that there is a correlation between all the properties but the degree of correlation varies from very good to fair.

10.4 DEGRADATION OF AGGREGATES

In Chapter 4, the degradation of aggregates used in flexible road construction has been described. The degradation process, mathematical solution to the theory of degradation, measurement of degradation, a review of degradation studies and prediction of changes in grading have been discussed in detail.

The reduction in sizes and angularity of the particles of an aggregate mass and the introduction of smooth surfaces in it due to the application of a load is called degradation (or crushing). Degradation results in a change in the stability of the aggregate structure due to changes in particle interlock and interparticle surface friction. In unbound bases degradation can lead to pavement failures. A method for predicting the changes in the grading of an aggregate mass due to degradation has been discussed.

10.5 MODIFIED C.B.R. TESTS

In Chapter 5 modified C.B.R. tests on brick aggregates have been discussed in two parts. In the first part C.B.R. tests of Groups I, II and III aggregates compacted in a modified mould 228 mm (9") diameter to a thickness of 225 mm by static loading in an Instron machine have been reported. In the second part in situ C.B.R. tests on full scale road bases with Group A and Group B aggregates with a modified plunger 152 mm (6") diameter plate have been discussed. The area of the loading plate represents the contact area of a wheel and thus gives some indication of the bearing capacities of the two road bases under investigation.

10.6 TRIAXIAL TESTS

Multistage triaxial tests on the two groups of aggregates have been reported in Chapter 6. Tests were done on 100 mm (4") diameter x 200 mm (8") high samples prepared by compacting the samples to the same densities and with the same grading as in the full scale pavement but the maximum particles sizes were reduced to half. Stress deformation graphs, Mohr circles and failure envelopes have been drawn for each of the tests. Sieve analyses of the samples after failure were also performed. From the ϕ_u and C_u values obtained corresponding ultimate bearing capacities of the samples were calculated.

The test results indicate that 230 mm (9") thick road bases made from Group A and Group B aggregates are adequate from the design point of view even to allow vehicular traffic without surfacing provided the wheel load does not exceed the design 690 kN/m^2 pressure.

10.7 REPEAT LOADING OF PAVEMENTS

In Chapter 7 the repeat loading of the experimental road bases have been reported. The road base was constructed with brick aggregates of the two groups of aggregates, Group A and Group B. The maximum size, the grading and the compaction of the aggregates of the road bases represent actual field conditions. The load was designed to represent the wheel load of a typical truck as used in Bangladesh. The load was transmitted through a 152 mm (6") diameter plate which is approximately the contact area of a typical wheel. The load was reduced to account for the absence of the surfacing. The analysis relating to this reduction was undertaken using the "tables of three layered elastic systems" of Jones (1962). The load was applied through a hydraulic jack by a Losenheim machine. There at least a 100,000 load applications were made. The deformation of the centre of the pavement surface under the loading plate was recorded. Graphs were plotted showing the deformation against the number of load repetitions. Initially the deformation increased sharply with the number of load repetitions but soon slowed down.

10.8 IMPROVEMENT OF STRENGTH OF BRICKS.

The strength of brick aggregates depends on the bricks from which they are made. The higher the strength of bricks, and therefore brick aggregates, the lesser is the thickness required for road bases. Possible ways of increasing the crushing strength of bricks have been discussed in Chapter 8. The main point stressed is the need for testing the soil, the raising of the temperature of firing to 1000° -

1200°C and maintaining it for periods longer than the usual period of 18 hours.

10.9 PROPOSED ROAD BASE SPECIFICATION

In Chapter 9 an attempt has been made to improve the present specification of water bound road bases of Bangladesh. This has been done from the indications of the present study and considering both the British and the Indian Standard for water bound macadam. It is difficult to specify an exact grading (or range of grading) for crushable aggregates like brick aggregates and more difficult to adhere to it in the actual construction in a developing country like Bangladesh where facilities are inadequate. Hence a simple method of using a single size sieve for adding extra fines to the hand broken samples has been suggested. This additional quantity of fines to be added would be determined from analysing a few samples in the laboratory and then specifying the ratio of the fines to the original aggregates by volume.

The aggregates can be classified in two groups - one made from 'slightly overburnt' bricks and the other from '1st class bricks'. The strength of bricks can be checked at site from the relative density oven dried basis of the aggregates or bricks.

10.10 SPECIFIC CONCLUSIONS

The following conclusions may be drawn from the present study:

- (i) There is direct correlation between all the properties of bricks and brick aggregates but the degree of correlation varies from very good to fair.
- (ii) The road base constructed according to a Bangladesh standard design with group B aggregates (which is weaker of the two groups tested) is strong enough to withstand the typical design wheel load on its own (i.e. without the surfacing) provided the maximum wheel load does not exert a pressure significantly in excess of 690 kN/m^2 .
- (iii) With a given increase in the crushing strength of bricks the performance of a brick aggregate road base improves to an even greater extent.
- (iv) Group A aggregates require approximately half the number of load applications compared with group B aggregates to reach stability.
- (v) In the repeat loading rest, the total deformation on a road base is proportional to the applied load.
- (vi) Research in India indicates that the strength of bricks produced in ordinary kilns can be improved by firing the bricks at the usual temperature ($1000^\circ - 1200^\circ \text{ c}$) for an extra period.
- (vii) To improve the performance of road bases with brick aggregates it is essential to ensure that there are sufficient fines in the aggregate mass, especially with stronger aggregates.

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APPENDIX A

SUMMARY OF TEST RESULTS

APPENDIX A

SUMMARY OF TEST RESULTS

GROUP	BRICKS		AGGREGATES					
	CRUSHING STRENGTH	WATER ABSORPTION	Aggregate Impact Value	Ten percent Fines Value	Elongation Index	Flakiness Index	Angularity Number	Water Absorption
	kN/m ²	%		kN				%
I	Av. 22582 Range 16829 - 29429	Av. 12.07 Range 9.4 - 15.9	49	60 (59.51)	27	11	11	12.47
II	Av. 26928 Range 19237 - 33976	Av. 9.97 Range 8.4 - 12.8	57	40 (43.93)	18	11	13	11.19
III	Av. 8417 Range 5425 - 11154	Av. 19.23 Range 18.7 - 19.6	73	15 (14.19)	22	1	10	21.65
A	Av. 22189 Range 18737 - 26087	Av. 9.65 Range 7.5 - 10.9	45	60 (57.53)	14	5	13	11.22
B	Av. 15855 Range 14977 - 16963	Av. 13.41 Range 12.2 - 14.6	55	30 (31.54)	9	2	14	15.64

APPENDIX A (Continued)

SUMMARY OF TEST RESULTS

GROUP	AGGREGATES			COMPACTED AGGREGATES		
	RELATIVE DENSITY					
	Oven-dried basis	Saturated and Surface dried basis	Apparent	Dry Density	Bulk Density	C.B.R.
				Mg/m ³	Mg/m ³	%
I	1.84	2.07	2.39	1.105*	1.244*	19*
II	1.88	2.09	2.39	1.069*	1.186*	17*
III	1.66	2.02	2.59	1.172*	1.425*	24*
A	1.85	2.05	2.33	1.375**	1.584**	34**
B	1.77	2.05	2.45	1.337**	1.600**	21**

* Aggregates compacted in a 230 mm diameter cylinder by Instron machine

** Aggregates compacted by a roller on the experimental bed.

APPENDIX B

TEST RESULTS

TABLE 2.1.1 CRUSHING STRENGTH OF BRICKSGROUP I

Sl. No.	Length	Breadth	Area(min)	Load	Strength	Mean	Standard Deviation
	mm	mm	mm ²	kN	kN/m ²	kN/m ²	kN/m ²
1	217	102	21935	635	28952	22582	4971
2	216	104	22581	380	16829		
3	217	103	22452	380	16925		
4	216	102	21935	480	21880		
5	216	103	22258	655	29429		
6	217	105	22839	422	18475		
7	217	103	22323	625	27997		
8	216	102	21935	430	19949		
9	216	103	22258	515	23139		
10	216	103	22258	495	22240		

TABLE 2.1.2GROUP II

Sl. No.	Length	Breadth	Area(min)	Load	Strength	Mean	Standard Deviation
	mm	mm	mm ²	kN	kN/m ²	kN/m ²	kN/m ²
1	219	103	22581	620	27471	26928	4089
2	219	106	23161	825	33976		
3	219	103	22581	650	28786		
4	221	104	22968	550	23942		
5	217	102	22000	530	24080		
6	217	106	23097	423	25430		
7	217	104	22581	610	26987		
8	217	103	22452	432	19237		
9	219	103	22581	640	28370		
10	219	103	22581	700	31000		

TABLE 2.1.3 CRUSHING STRENGTH OF BRICKSGROUP III

Sl. No.	Length	Breadth	Area(Min)	Load	Strength	Mean	Standard Deviation
	mm	mm	mm ²	kN	kN/m ²	kN/m ²	kN/m ²
1	226	109	24710	232	9390	8417	1733
2	227	108	24516	194	7916		
3	227	109	24839	235	9459		
4	226	108	24323	175	7196		
5	227	108	24516	182	7425		
6	227	109	24839	277	11154		
7	227	108	24516	258	10525		
8	227	108	24516	133	5425		
9	226	109	24516	177.5	7238		
10	227	108	24516	207	8442		

TABLE 2.1.4GROUP IV

Sl. No.	Length	Breadth	Area(min)	Load	Strength	Mean	Standard Deviation
	mm	mm	mm ²	kN	kN/m ²	kN/m ²	kN/m ²
1	227	108	24516	240	9791	8128	2463
2	227	108	24516	228	9293		
3	226	108	24323	272	11189		
4	227	108	24516	302	12316		
5	227	108	24516	207	8442		
6	227	108	24516	143	5840		
7	227	108	24516	177	7217		
8	226	108	24323	147	6048		
9	227	108	24516	135	5508		
10	227	108	24516	138	5633		

TABLE 2.1. A Crushing Strength of BricksGROUP A

Sl. No.	Length	Breadth	Area	Crushing Load	Crushing Strength	Mean	Standard Deviation
	mm	mm	mm ²	kN	kN/m ²	kN/m ²	kN/m ²
1	213	99	21087	444	21056	22189	2678
2	212	105	22260	458	20575		
3	215	104	22360	421	18828		
4	214	103	22042	575	26087		
5	215	105	22575	423	18737		
6	213	100	21300	515	24178		
7	212	99	20988	540	25729		
8	216	103	22248	476	21395		
9	213	99	21087	510	24186		
10	215	98	21070	445	21120		

TABLE 2.1. BGROUP B

sl. No.	Length	Breadth	Area (min)	Crushing Load	Crushing Strength	Mean	Standard Deviation
	mm	mm	mm ²	kN	kN/m ²	kN/m ²	kN/m ²
1	215	100	21500	322	14977	15855	688
2	215	102	21930	367	16553		
3	215	102	21930	372	16963		
4	216	103	22248	337	15147		
5	215	101	21715	338	15565		
6	219	104	22776	358	15718		
7	218	104	22672	369	16276		
8	218	105	22890	358	15640		

TABLE 2.2.1 WATER ABSORPTION OF BRICKSGROUP I

Sl. No.	Dry Weight	Wt. of Brick + water	Wt. of water	Water Absorption	Mean	Standard Deviation
	gm	gm	gm	%	%	%
1	2662	2912	250	9.4	12.07	2.224
2	2505	2865	360	14.4		
3	2334	2704	370	15.9		
4	2517	2791	274	10.9		
5	2590	2859	269	10.4		
6	2453	2785	332	13.6		
7	2601	2850	249	9.6		
8	2420	2756	336	13.9		
9	2903	3249	346	11.9		
10	2619	2891	272	10.8		

TABLE 2.2.2GROUP II

Sl. No.	Dry weight	Wt. of Brick + Water	Wt. of water	Water Absorption	Mean	Standard Deviation
	gm	gm	gm	%	%	%
1	2608	2839	231	8.9	9.97	1.236
2	2645	2885	240	9.1		
3	2629	2865	236	9.0		
4	2561	2805	244	9.5		
5	2683	2944	261	9.7		
6	2507	2826	319	12.8		
7	2679	2960	281	10.5		
8	2637	2908	271	10.3		
9	2701	2963	262	9.7		
10	2609	2828	219	8.4		

TABLE 2.2.3 WATER ABSORPTION OF BRICKSGROUP III

Sl. No.	Dry Weight	Wt. of Brick + Water	Wt. of water	Water Absorption	Mean	Standard Deviation
	gm	gm	gm	%	%	%
1	2641	3151	510	19.3	19.23	0.295
2	2694	3220	526	19.5		
3	2682	3193	511	19.0		
4	2649	3165	516	19.5		
5	2700	3223	523	19.4		
6	2697	3202	505	18.7		
7	2722	3242	520	19.1		
8	2610	3122	512	19.6		
9	2758	3279	521	18.9		
10	2720	3245	525	19.3		

TABLE 2.2.4GROUP IV

Sl. No.	Dry Weight	Wt. of Brick + water	Wt. of water	Water Absorption	Mean	Standard Deviation
	gm	gm	gm	%	%	%
1	2777	3274	497	17.9	17.72	0.278
2	2768	3272	504	18.2		
3	2811	3315	504	17.9		
4	2872	3378	506	17.6		
5	2837	3345	508	17.9		
6	2887	3385	498	17.2		
7	2814	3305	491	17.5		
8	2822	3320	498	17.6		
9	2847	3348	501	17.6		
10	2811	3312	501	17.8		

TABLE 2.2.AWATER ABSORPTION OF BRICKSGROUP A

sl.no.	Dry Weight gm	Wt. of brick + water gm	Wt. of Water gm	Water Absorption %	Mean %	Standard Deviation %
1	2632	2881	249	9.5	9.65	0.969
2	2799	3057	258	9.2		
3	2532	2783	251	9.9		
4	2595	2790	195	7.5		
5	2747	3046	299	10.9		
6	2751	3002	251	9.1		
7	2667	2923	256	9.6		
8	2437	2698	261	10.7		
9	2472	2712	240	9.7		
10	2665	2943	278	10.4		

TABLE 2.2.BGROUP B

sl. no.	Dry Weight gm	Wt. of brick + water gm	Wt. of Water gm	Water Absorption %	Mean %	Standard Deviation %
1	2482	2830	348	14.0	13.41	.881
2	2813	3201	388	13.8		
3	2766	3128	362	13.1		
4	2770	3163	393	14.2		
5	2634	3019	385	14.6		
6	2871	3224	353	12.3		
7	2821	3166	345	12.2		
8	2869	3246	377	13.1		

TABLE 3.1 DETERMINATION OF TEN PERCENT FINES VALUE

Row		Unit	Symbol	Group I	Group II	Group III	Group A	Group B
1	Load applied	kN	X	80.0	60.0	27.5	60.0	38.0
2	Total weight of sample	gm	W	1822	1812	1612	1850	1803
3	Weight of fines passing No.7 sieve	gm	w	270	274	373	196	232
4	% fines $\frac{\text{Row 3} \times 100}{\text{Row 2}}$		$y = \frac{100, w}{W}$	14.82	15.12	23.14	10.60	12.87
5	(Row 4) + 4		Y + 4	18.82	19.12	27.14	14.60	16.87
6	Ten percent fines value $\frac{(\text{Row 1}) \times 14}{\text{Row 5}}$	kN	$\frac{14X}{Y+4}$	59.51	43.93	14.19	57.53	31.54
7	Ten percent fines values*	kN		60	40	15	60	30

* As per B.S. 812 to nearest 5 kN up to 30 kN and to nearest 10 kN over 30 kN load.

TABLE 3.2 DETERMINATION OF AGGREGATE IMPACT VALUE

	Symbol	Test No.	Group I	Group II	Group III	Group A	Group B
Weight of surface dry sample (gm)	A	1	210.5	194.4	193.8	221.30	208.0
		2	210.5	194.3	193.8	221.42	208.0
Weight of fraction passing No. 7 sieve (gm)	B	1	103.5	112.8	145.5	101.13	114.29
		2	104	110.6	135.6	98.06	113.71
Aggregate Impact Value	$\frac{B}{A} \times 100$	1	49.17	58.02	75.08	45.7	54.95
		2	49.41	56.92	69.97	44.3	54.67
Aggregate Impact Value*		mean	49	57	73	45	55

* To the nearest whole number

TABLE 3.3 DETERMINATION OF RELATIVE DENSITIES AND WATER ABSORPTION

Row		Symbol	Group I	Group II	Group III	Group A	Group B
1	Wt. of vessel + sample + water (gm)	A	4967	4969	5000	1425.5	1400
2	Wt. of vessel + water (gm)	B	4398	4398	4396	1245	1212.5
3	Wt. of saturated surface-dry sample (gm)	C	1100	1093	1197	352	366
4	Wt. of oven-dry sample (gm)	D	978	983	984	316.5	316.5
5	(Row 1 - Row 2)	(A-B)	569	571	604	180.5	187.5
6	(Row 3 - Row 5)	C- (A-B)	531	522	593	171.5	178.5
7	(Row 4 - Row 5)	D- (A-B)	409	412	380	136.0	129.0
8	(Row 3 - Row 4)	(C-D)	122	110	213	35.5	49.5
9	Rel. Den. (o.d.b.) $\frac{\text{Row 4}}{\text{Row 6}}$	$\frac{D}{C- (A-B)}$	1.84	1.88	1.66	1.85	1.77
10	Rel. Den. (s.s.d.b.) $\frac{\text{Row 3}}{\text{Row 6}}$	$\frac{C}{C- (A-B)}$	2.07	2.09	2.02	2.05	2.05
11	App. Rel. Den. $\frac{\text{Row 4}}{\text{Row 7}}$	$\frac{D}{D- (A-B)}$	2.39	2.39	2.59	2.33	2.45
12	Water Absorption* $\frac{\text{Row 8}}{\text{Row 4}} \times 100$	$\frac{100 (C-D)}{D}$	12.47	11.19	21.65	11.22	15.64

* (percent of dry weight)

Rel. Den. (o.d.b.) = Relative Density on an oven-dried basis.

Rel. Den. (s.s.d.b.) = Relative Density on a saturated and surface-dried basis.

App. Rel. Den. = Apparent Relative Density.

TABLE 3.4 DETERMINATION OF ANGULARITY NUMBER

Row		SYMBOL	GROUP I	GROUP II	GROUP III	GROUP A	GROUP B
1	Wt. of aggregate in cylinder (gm)	W	2894	2864	2498	2798	2632
2	Wt. of water re- quired to fill cylinder (gm)	C	2812	2816	2636	2819	2812
3	Relative Density of aggregate (oven-dried basis)	G_A	1.84	1.88	1.66	1.85	1.77
4	Row 2 x Row 3	$c.G_A$	5174	5294	4376	5215	4977
5	$\frac{\text{Row 1} \times 100}{\text{Row 4}}$	$\frac{100.W}{C.G_A}$	56	54	57	54	53
6	Angularity Number = 67 - Row 5	$67 - \frac{100.W}{C.G_A}$	11	13	10	13	14

TABLE 3.5/6.1 DETERMINATION OF ELONGATION AND FLAKINESS INDICESGROUP I

Size of aggregate				No.of part- icles	Weight of the particles (gm)	Weight of particles retained by the length gauge (gm)	Weight of particles passing the thickness gauge (gm)
Passing BS Sieve		Retained BS Sieve					
in	mm	in	mm				
1½	38.10	-	28.00	100	3073	712	340
-	28.00	1	25.40	50	977	-	81
1	25.40	¾	19.05	100	1220	565	135
¾	19.05	½	12.70	50	245	169	28
½	12.70	⅜	9.52	50	75	39	12
		TOTAL WEIGHT			5590	1485	596

$$\text{ELONGATION INDEX} = \frac{1485}{5590} \times 100 = 26.56 \approx 27$$

$$\text{FLAKINESS INDEX} = \frac{596}{5590} \times 100 = 10.66 \approx 11$$

TABLE 3.5/6.2 DETERMINATION OF ELONGATION AND FLAKINESS INDICES

GROUP II

Size of aggregate				No. of part- icles	Weight of the particles (gm)	Weight of particles retained by the length gauge (gm)	Weight of particles passing the thickness gauge (gm)
Passing BS Sieve		Retained BS Sieve					
in	mm	in	mm				
2	50.80	1½	38.10	90	6323	176	833
1½	38.10	-	28.00	90	3030	1047	135
1	25.40	¾	19.05	45	617	419	90
¾	19.05	½	12.70	45	212	156	32
		TOTAL WEIGHT			10182	1798	1090

$$\text{ELONGATION INDEX} = \frac{1798}{10182} \times 100 = 17.66 \approx 18$$

$$\text{FLAKINESS INDEX} = \frac{1090}{10182} \times 100 = 10.71 \approx 11$$

TABLE 3.5/6.3. DETERMINATION OF ELONGATION AND FLAKINESS INDICES

GROUP III

Size of aggregate				No.of part- icles	Weight of the particles (gm)	Weight of particles retained by the length gauge (gm)	Weight of particles passing the thickness gauge (gm)
Passing BS Sieve		Retained BS Sieve					
in	mm	in	mm				
2	50.80	1½	38.10	86	7550	426	105
1½	38.10	-	28.00	86	3914	1986	-
1	25.40	¾	19.05	43	432	178	32
¾	19.05	½	12.70	43	194	99	9
		TOTAL WEIGHT			12090	2689	146

$$\text{ELONGATION INDEX} = \frac{2689}{12090} \times 100 = 22.24 \approx 22$$

$$\text{FLAKINESS INDEX} = \frac{146}{12090} \times 100 = 1.21 \approx 1$$

TABLE 3.5/6.A

DETERMINATION OF ELONGATION AND FLAKINESS INDICES

GROUP A

Size of aggregate				No. of part- icles	Weight of the particles (gm)	Weight of particles retained by the length gauge (gm)	Weight of particles passing the thickness gauge (gm)
Passing BS Sieve		Retained BS Sieve					
in	mm	in	mm				
2	50.80	1½	38.10	100	7833	0	0
1½	38.10	1¼	31.75	100	4414	1741	42
1¼	31.75	1	25.40	100	2506	-	196
1	25.40	¾	19.05	50	646	270	30
¾	19.05	½	12.70	50	268	173	20
				TOTAL WEIGHT	15667	2184	779

$$\text{ELONGATION INDEX} = \frac{2184}{15667} \times 100 = 13.94 \approx 14$$

$$\text{FLAKINESS INDEX} = \frac{779}{15667} \times 100 = 4.97 \approx 5$$

TABLE 3.5/6.B

GROUP B

Size of aggregate				No. of part- icles	Weight of the particles (gm)	Weight of particles retained by the length gauge (gm)	Weight of particles passing the thickness gauge (gm)
Passing BS Sieve		Retained BS Sieve					
in	mm	in	mm				
2	50.80	1½	38.10	50	3562	0	38
1½	38.10	1¼	31.75	50	2167	442	0
1¼	31.75	1	25.40	100	2385	-	84
1	25.40	¾	19.05	100	1047	298	52
¾	19.05	½	12.70	50	232	108	7
½	12.70	⅜	9.52	50	73	26	5
TOTAL WEIGHT				9466	874	186	

$$\text{ELONGATION INDEX} = \frac{874}{9466} \times 100 = 9.23 \approx 9$$

$$\text{FLAKINESS INDEX} = \frac{186}{9466} \times 100 = 1.96 \approx 2$$

TABLE 3.7.1 SIEVE ANALYSISGROUP I

Sieve Size		Weight retained (gm)	% retained	Cumulative % retained	% passing
mm/(microns)	Inch/(No)				
50.80	2	0	0	0	100.00
38.10	1½	435	3.5	3.5	96.5
28.00	-	3473	27.6	31.1	68.9
25.40	1	1172	9.3	40.4	59.6
19.05	¾	2636	20.9	61.3	38.7
12.70	½	1575	12.5	73.8	26.2
9.52	⅜	658	5.2	79.0	21.0
6.35	¼	594	4.7	83.7	16.3
5.00	-	258	2.0	85.7	14.3
3.18	⅜	353	2.8	88.5	11.5
2.40	(7)	182	1.4	89.9	10.1
1.68	(10)	270	2.1	92.0	8.0
1.20	(14)	200	1.6	93.6	6.4
(850)	(18)	157	1.2	94.8	5.2
(600)	(25)	127	1.0	95.8	4.2
(420)	(36)	89	0.7	96.5	3.5
(300)	(52)	84	0.7	97.2	2.8
(210)	(72)	50	0.4	97.6	2.4
(150)	(100)	58	0.5	98.1	1.9
(75)	(200)	73	0.6	98.7	1.3
Passing No. 200		160	1.3	100.0	-

TABLE 3.7.2 SIEVE ANALYSISGROUP II

Sieve Size		Weight retained (gm)	% retained	Cumulative % retained	% passing
mm/(microns)	Inch/(No)				
50.80	2	0	0	0	100.0
38.10	1½	6664	25.8	25.8	74.2
28.00	-	8961	34.7	60.5	39.5
25.40	1	628	2.4	62.9	37.1
19.05	¾	2683	10.4	73.3	26.7
12.70	½	1716	6.6	79.9	20.1
9.52	⅜	899	3.5	83.4	16.6
6.35	¼	714	2.8	86.2	13.8
5.00	-	377	1.5	87.7	12.3
3.18	⅛	549	2.1	89.8	10.2
2.40	(7)	375	1.4	91.2	8.8
1.68	(10)	418	1.6	92.8	7.2
1.20	(14)	338	1.3	94.1	5.9
(850)	(18)	285	1.1	95.2	4.8
(600)	(25)	237	0.9	96.1	3.9
(420)	(36)	155	0.6	96.7	3.3
(300)	(52)	139	0.5	97.2	2.8
(210)	(72)	98	0.4	97.6	2.4
(150)	(100)	148	0.6	98.2	1.8
(75)	(200)	167	0.6	98.8	1.2
Passing No. 200		319	1.2	100.0	-

TABLE 3.7 .3. SIEVE ANALYSISGROUP III

Sieve Size		Weight retained (gm)	% retained	Cumulative % retained	% passing
mm/(microns)	Inch/(No)				
50.80	2	0	0	0	100.0
38.10	1½	7730	31.6	31.6	68.4
28.00	-	5524	22.6	54.2	45.8
25.40	1	1192	4.9	59.1	40.1
19.05	¾	1670	6.8	65.9	34.1
12.70	½	1484	6.1	72.0	28.0
9.52	⅜	1000	4.1	76.1	23.9
6.35	¼	857	3.5	79.6	20.4
5.00	-	518	2.1	81.7	18.3
3.18	⅓	852	3.5	85.2	14.8
2.40	(7)	523	2.1	87.3	12.7
1.68	(10)	761	3.1	90.4	9.6
1.20	(14)	494	2.0	92.4	7.6
(850)	(18)	403	1.7	94.1	5.9
(600)	(25)	314	1.3	95.4	4.6
(420)	(36)	218	0.9	96.3	3.7
(300)	(52)	188	0.8	97.1	2.9
(210)	(72)	154	0.7	97.8	2.2
(150)	(100)	214	0.9	98.7	1.3
(75)	(200)	243	1.0	99.7	0.3
Passing No. 200		67	0.3	100.0	-

TABLE 3.7.A

SIEVE ANALYSISGROUP A

(HAND BROKEN AGGREGATES)

Sieve Size		% Passing			
mm/ (micron)	Inch/ (No)	SAMPLE 1	SAMPLE 2	SAMPLE 3	MEAN
50.80	2	100	100	100	100
38.10	1½	71.7	73.0	69.1	71.3
31.75	1¼				*56.7
25.40	1				*38.5
19.05	¾	26.5	27.6	29.6	27.9
12.70	½				*17.7
9.52	⅜	16.3	17.4	12.4	15.4
6.35	¼				*11.9
5.00	-	11.5	12.5	8.0	10.7
3.18	⅛				*8.5
2.40	(7)				*7.5
1.68	(10)				*6.2
1.20	(14)				*5.4
(850)	(18)				*4.4
(600)	(25)	3.4	4.1	2.5	3.3
(420)	(36)				*2.9
(300)	(52)				*2.3
(210)	(72)				*2.0
(150)	(100)	1.4	1.9	1.3	1.5
(75)	(200)				*0.8

* by interpolation of a smooth grading curve.

Note: Total Weight of Sample 1 = 23314 gms

Total Weight of Sample 2 = 33753 gms

Total Weight of Sample 3 = 28876 gms

TABLE 3.7.B

SIEVE ANALYSIS

GROUP B

(HAND BROKEN AGGREGATES)

Sieve Size		% Passing				
mm/ (micron)	Inch/ (No)	Sample 1	Sample 2	Sample 3	Mean	Weighted Mean
50.80	2	100	100	100	100	100
38.10	1½	83.0	85.2	89.4	85.9	86.2
31.75	1¼	68.8	63.3	79.7	70.6	71.4
25.40	1	51.1	40.9	66.2	52.7	53.9
19.05	¾	35.7	24.4	49.6	36.6	37.7
12.70	½	25.3	13.4	35.6	24.8	25.7
9.52	⅜	19.7	8.9	29.6	19.4	20.3
6.35	¼	16.2	6.0	24.2	15.5	16.2
5.00	-	14.1	4.8	21.2	13.4	14.0
3.18	⅛	11.2	3.5	17.0	10.6	11.0
2.40	(7)	9.4	2.8	14.3	8.8	9.2
1.68	(10)	7.3	2.1	11.2	6.9	7.2
1.20	(14)	5.8	1.7	9.0	5.5	5.7
(850)	(18)	4.5	1.4	7.2	4.4	4.5
(600)	(25)	3.6	1.2	5.8	3.5	3.6
(420)	(36)	3.0	1.0	4.9	3.0	3.0
(300)	(52)	2.4	0.9	3.9	2.4	2.4
(210)	(72)	1.9	0.8	3.1	1.9	1.9
(150)	(100)	1.5	0.7	2.4	1.5	1.5
(75)	(200)	0.9	0.5	0.9	0.8	0.7

Weight of sample 1 = 15,370 gm

Weight of sample 2 = 15,094 gm

Weight of sample 3 = 19,390 gm

TABLE 4.1.1 SIEVE SIZE RATIO

Sieve Size		GROUP I		GROUP II		GROUP III	
mm/ (microns)	Inch/ (No)	Original	Final	Original	Final	Original	Final
38.10	1½	.036	.015	.348	.209	.462	.185
28.00	-	.451	.351	1.53	.939	1.18	.517
25.40	1	.678	.497	1.70	1.37	1.47	.647
19.05	¾	1.58	1.23	2.75	2.14	1.93	.869
12.70	½	2.82	2.24	3.98	3.07	2.57	1.18
9.52	⅜	3.76	3.03	5.02	3.88	3.18	1.46
6.35	¼	5.13	4.08	6.25	4.88	3.90	1.81
5.00	-	5.99	4.81	7.13	5.33	4.46	2.07
3.18	⅛	7.70	6.19	8.80	6.25	5.76	2.62
2.40	(7)	8.90	7.20	10.4	7.00	6.87	3.26
1.68	(10)	11.5	9.42	12.9	8.90	9.42	4.32
1.20	(14)	14.6	11.8	15.9	10.9	12.2	5.54
(850)	(18)	18.2	15.1	19.8	13.5	15.9	7.20
(600)	(25)	22.8	19.0	24.6	16.2	20.7	9.20
(420)	(36)	27.6	23.4	29.3	19.0	26.0	11.3
(300)	(52)	34.7	29.3	34.7	23.4	33.5	14.4
(210)	(72)	40.7	34.7	40.7	27.6	44.5	17.5
(150)	(100)	51.6	46.6	54.6	33.5	75.9	23.4
(75)	(200)	75.9	61.5	82.3	49.0	332	54.6

TABLE 4.1.2 SIEVE SIZE RATIO

Sieve Size		Group A		Group B	
mm/(microns)	Inch/(No)	Original	Final (after com- paction by rolling)	Original	Final (After com- paction by rolling)
38.10	1½	0.403	0.188	0.160	0.112
31.75	1¼	0.764	0.531	0.401	0.261
25.40	1	1.60	1.169	0.855	0.447
19.05	¾	2.58	1.950	1.653	0.730
12.70	½	4.65	2.922	2.891	1.092
9.52	⅜	5.49	3.739	3.926	1.433
6.35	¼	7.40	4.780	5.173	1.959
5.00	-	8.35	5.494	6.143	2.356
3.18	⅛	10.76	6.874	8.091	3.274
2.40	(7)	12.33	7.850	9.870	3.975
1.68	(10)	15.13	10.111	12.889	5.579
1.20	(14)	17.52	12.333	16.544	7.264
(850)	(18)	21.73	14.873	21.222	9.309
(600)	(25)	29.30	17.868	26.778	11.658
(420)	(36)	33.48	21.222	32.333	14.385
(300)	(52)	42.48	24.641	40.667	17.182
(210)	(72)	49.00	29.303	51.632	20.277
(150)	(100)	65.67	34.714	65.667	24.641
(75)	(200)	124.00	51.632	141.857	40.667

TABLE 5.1.1 SIEVE ANALYSIS (AFTER COMPACTION)GROUP I

Sieve Size		Weight retained (gm)	% retained	Cumulative % retained	% passing
mm/(microns)	Inch/(No)				
50.80	2	0	0	0	100.0
38.10	1½	193	1.5	1.5	98.5
28.00	-	3076	24.5	26.0	74.0
25.40	1	907	7.2	33.2	66.8
19.05	¾	2759	22.0	55.2	44.8
12.70	½	1751	13.9	69.1	30.9
9.52	⅜	766	6.1	75.2	24.8
6.35	¼	637	5.1	80.3	19.7
5.00	-	313	2.5	82.8	17.2
3.18	⅛	404	3.3	86.1	13.9
2.40	(7)	206	1.7	87.8	12.2
1.68	(10)	321	2.6	90.4	9.6
1.20	(14)	225	1.8	92.2	7.8
(850)	(18)	191	1.6	93.8	6.2
(600)	(25)	148	1.2	95.0	5.0
(420)	(36)	105	0.9	95.9	4.1
(300)	(52)	93	0.8	96.7	3.3
(210)	(72)	63	0.5	97.2	2.8
(150)	(100)	89	0.7	97.9	2.1
(75)	(200)	67	0.5	98.4	1.6
Passing No. 200		197	1.6	100.0	-

TABLE 5.1.2 SIEVE ANALYSIS (AFTER COMPACTION)GROUP II

Sieve Size		Weight retained (gm)	% retained	Cumulative % retained	% passing
mm/(microns)	Inch/(No)				
50.80	2	0	0	0	100.0
38.10	1½	1817	17.3	17.3	82.7
28.00	-	3268	31.1	48.4	51.6
25.40	1	990	9.4	57.8	42.2
19.05	¾	1093	10.4	68.2	31.8
12.70	½	755	7.2	75.4	24.6
9.52	⅜	427	4.1	79.5	20.5
6.35	¼	367	3.5	83.0	17.0
5.00	-	126	1.2	84.2	15.8
3.18	⅛	209	2.0	86.2	13.8
2.40	(7)	133	1.3	87.5	12.5
1.68	(10)	256	2.4	89.9	10.1
1.20	(14)	179	1.7	91.6	8.4
(850)	(18)	162	1.5	93.1	6.9
(600)	(25)	118	1.1	94.2	5.8
(420)	(36)	84	0.8	95.0	5.0
(300)	(52)	93	0.9	95.9	4.1
(210)	(72)	58	0.6	96.5	3.5
(150)	(100)	65	0.6	97.1	2.9
(75)	(200)	96	0.9	98.0	2.0
Passing No. 200		203	1.9	100.0	-

TABLE 5.1.3. SIEVE ANALYSIS (AFTER COMPACTION)

GROUP III

Sieve Size		Weight retained (gm)	% retained	Cumulative % retained	% passing
mm/(microns)	Inch/(No)				
50.80	2	0	0	0	100.0
38.10	1½	1536	15.6	15.6	84.4
28.00	-	1816	18.5	34.1	65.9
25.40	1	515	5.2	39.3	60.7
19.05	¾	708	7.2	46.5	53.5
12.70	½	748	7.6	54.1	45.9
9.52	⅜	522	5.3	59.4	40.6
6.35	¼	493	5.0	64.4	35.6
5.00	-	283	3.0	67.4	32.6
3.18	⅛	471	5.0	72.4	27.6
2.40	(7)	393	4.1	76.5	23.5
1.68	(10)	442	4.7	81.2	18.8
1.20	(14)	333	3.5	84.7	15.3
(850)	(18)	296	3.1	87.8	12.2
(600)	(25)	225	2.4	90.2	9.8
(420)	(36)	163	1.7	91.9	8.1
(300)	(52)	151	1.6	93.5	6.5
(210)	(72)	108	1.1	94.6	5.4
(150)	(100)	125	1.3	95.9	4.1
(75)	(200)	222	2.3	98.2	1.8
Passing No. 200		158	1.7	100.0	-

TABLE 5.2.A SIEVE ANALYSIS (After compaction and CBR Test)

GROUP A

Sieve Size		Weight retained (gm)	% retained	Cumulative % retained	% passing
mm/ (micron)	Inch/ (No)				
50.80	2	0	0	0	100
38.10	1½	795	7.1	7.1	92.9
31.75	1¼	1033	9.2	16.3	83.7
25.40	1	961	8.6	24.9	75.1
19.05	¾	1140	10.2	35.1	64.9
12.70	½	1311	11.7	46.8	53.2
9.52	⅜	911	8.1	54.9	45.1
6.35	¼	783	7.0	61.9	38.1
5.00	-	433	3.9	65.8	34.2
3.18	⅛	660	5.9	71.7	28.3
2.40	(7)	415	3.7	75.4	24.6
1.68	(10)	497	4.4	79.8	20.2
1.20	(14)	377	3.4	83.2	16.8
(850)	(18)	324	2.9	86.1	13.9
(600)	(25)	238	2.1	88.2	11.8
(420)	(36)	202	1.8	90.0	10.0
(300)	(52)	198	1.8	91.8	8.2
(210)	(72)	170	1.5	93.3	6.7
(150)	(100)	178	1.6	94.9	5.1
(75)	(200)	260	2.3	97.2	2.8
Passing No. 200		312	2.8	100	-

TABLE 5.2.B

SIEVE ANALYSIS (AFTER COMPACTION AND CBR TEST)GROUP B

Sieve Size		Weight retained (gm)	% retained	Cumulative % retained	% passing
mm./ (microns)	Inch/ (No)				
50.80	2	0	0	0	100
38.10	1½	376	4.1	4.1	95.9
31.75	1¼	456	4.9	9.0	91.0
25.40	1	891	9.6	18.6	81.4
19.05	¾	1216	13.1	31.7	68.3
12.70	½	1307	14.1	45.8	54.2
9.52	⅜	681	7.3	53.1	46.9
6.35	¼	677	7.3	60.4	39.6
5.00	-	389	4.2	64.6	35.4
3.18	⅛	619	6.7	71.3	28.7
2.40	(7)	341	3.7	75.0	25.0
1.68	(10)	502	5.4	80.4	19.6
1.20	(14)	349	3.8	84.2	15.8
(850)	(18)	278	3.0	87.2	12.8
(600)	(25)	210	2.3	89.5	10.5
(420)	(36)	165	1.8	91.3	8.7
(300)	(52)	128	1.4	92.7	7.3
(210)	(72)	100	1.1	93.8	6.2
(150)	(100)	105	1.1	94.9	5.1
(75)	(200)	171	1.8	96.7	3.3
Passing No. 200		305	3.3	100	-

TABLE 7.1.A

SIEVE ANALYSIS (AFTER COMPACTION BY ROLLING)GROUP A

Sieve Size		Weight retained (gm)	% retained	Cumulative % retained	% passing
mm./ (microns)	Inch/ (No)				
50.80	2	0	0	0	100
38.10	1½	1464	15.8	15.8	84.2
31.75	1¼	1749	18.9	34.7	65.3
25.40	1	1781	19.2	53.9	46.1
19.05	¾	1128	12.2	66.1	33.9
12.70	½	777	8.4	74.5	25.5
9.52	⅜	415	4.4	78.9	21.1
6.35	¼	355	3.8	82.7	17.3
5.00	-	174	1.9	84.6	15.4
3.18	⅛	250	2.7	87.3	12.7
2.40	(7)	134	1.4	88.7	11.3
1.68	(10)	215	2.3	91.0	9.0
1.20	(14)	137	1.5	92.5	7.5
(850)	(18)	117	1.2	93.7	6.3
(600)	(25)	91	1.0	94.7	5.3
(420)	(36)	74	0.8	95.5	4.5
(300)	(52)	55	0.6	96.1	3.9
(210)	(72)	52	0.6	96.7	3.3
(150)	(100)	50	0.5	97.2	2.8
(75)	(200)	85	0.9	98.1	1.9
Passing No. 200		175	1.9	100	-

TABLE 7.1.B SIEVE ANALYSIS (AFTER COMPACTION BY ROLLING)GROUP B

Sieve Size		Weight retained (gm)	% retained	Cumulative % retained	% passing
mm./ (microns)	Inch/ (No)				
50.80	2	0	0	0	100
38.10	1½	967	10.1	10.1	89.9
31.75	1½	1018	10.6	20.7	79.3
25.40	1	981	10.2	30.9	69.1
19.05	¾	1078	11.3	42.2	57.8
12.70	½	954	10.0	52.2	47.8
9.52	⅜	643	6.7	58.9	41.1
6.35	¼	695	7.3	66.2	33.8
5.00	-	383	4.0	70.2	29.8
3.18	⅛	609	6.4	76.6	23.4
2.40	(7)	317	3.3	79.9	20.1
1.68	(10)	417	4.9	84.8	15.2
1.20	(14)	298	3.1	87.9	12.1
(850)	(18)	230	2.4	90.3	9.7
(600)	(25)	170	1.8	92.1	7.9
(420)	(36)	131	1.4	93.5	6.5
(300)	(52)	95	1.0	94.5	5.5
(210)	(72)	75	0.8	95.3	4.7
(150)	(100)	80	0.8	96.1	3.9
(75)	(200)	140	1.5	97.6	2.4
Passing No. 200		235	2.4	100	-

TABLE 7.2.1 SIEVE ANALYSIS (After compaction and repeat loading)

GROUP A (Test spot 1)

Sieve Size		Weight retained	% retained	cumulative % retained	% passing
mm/ (micron)	Inch/ (No)				
50.80	2	0	0	0	100
38.10	1½	922	8.8	8.8	91.2
31.75	1¼	1122	10.7	19.5	80.5
25.40	1	1557	14.9	34.4	65.6
19.05	¾	1056	10.1	44.5	55.5
12.70	½	1095	10.5	55.0	45.0
9.52	⅜	729	7.0	62.0	38.0
6.35	¼	601	5.8	67.8	32.2
5.00	-	328	3.1	70.9	29.1
3.18	⅛	511	4.9	75.8	24.2
2.40	(7)	292	2.8	78.6	21.4
1.68	(10)	375	3.6	82.2	17.8
1.20	(14)	300	2.9	85.1	14.9
(850)	(18)	267	2.5	87.6	12.4
(600)	(25)	200	1.9	89.5	10.5
(420)	(36)	164	1.6	91.1	8.9
(300)	(52)	152	1.5	92.6	7.4
(210)	(72)	141	1.3	93.9	6.1
(150)	(100)	104	1.0	94.9	5.1
(75)	(200)	182	1.7	96.6	3.4
Passing No. 200		352	3.4	100	-

TABLE 7.2.2. SIEVE ANALYSIS (After compaction and repeat loading)

GROUP A (Test spot 2)

Sieve Size		Weight retained	% retained	cumulative % retained	% passing
mm/ (micron)	Inch/ (No)				
50.80	2	0	0	0	100
38.10	1½	1545	16.1	16.1	83.9
31.75	1¼	1695	17.7	33.8	66.2
25.40	1	1672	17.4	51.2	48.8
19.05	¾	999	10.4	61.6	38.4
12.70	½	791	8.2	69.8	30.2
9.52	⅜	425	4.4	74.2	25.8
6.35	¼	450	4.7	78.9	21.1
5.00	-	243	2.5	81.4	18.6
3.18	⅛	324	3.4	84.8	15.2
2.40	(7)	189	2.0	86.8	13.2
1.68	(10)	228	2.4	89.2	10.8
1.20	(14)	168	1.7	90.9	9.1
(850)	(18)	142	1.5	92.4	7.6
(600)	(25)	107	1.1	93.5	6.5
(420)	(36)	86	0.9	94.4	5.6
(300)	(52)	83	0.9	95.3	4.7
(210)	(72)	74	0.8	96.1	3.9
(150)	(100)	61	0.6	96.7	3.3
(75)	(200)	117	1.2	97.9	2.1
Passing No. 200		198	2.1	100	-

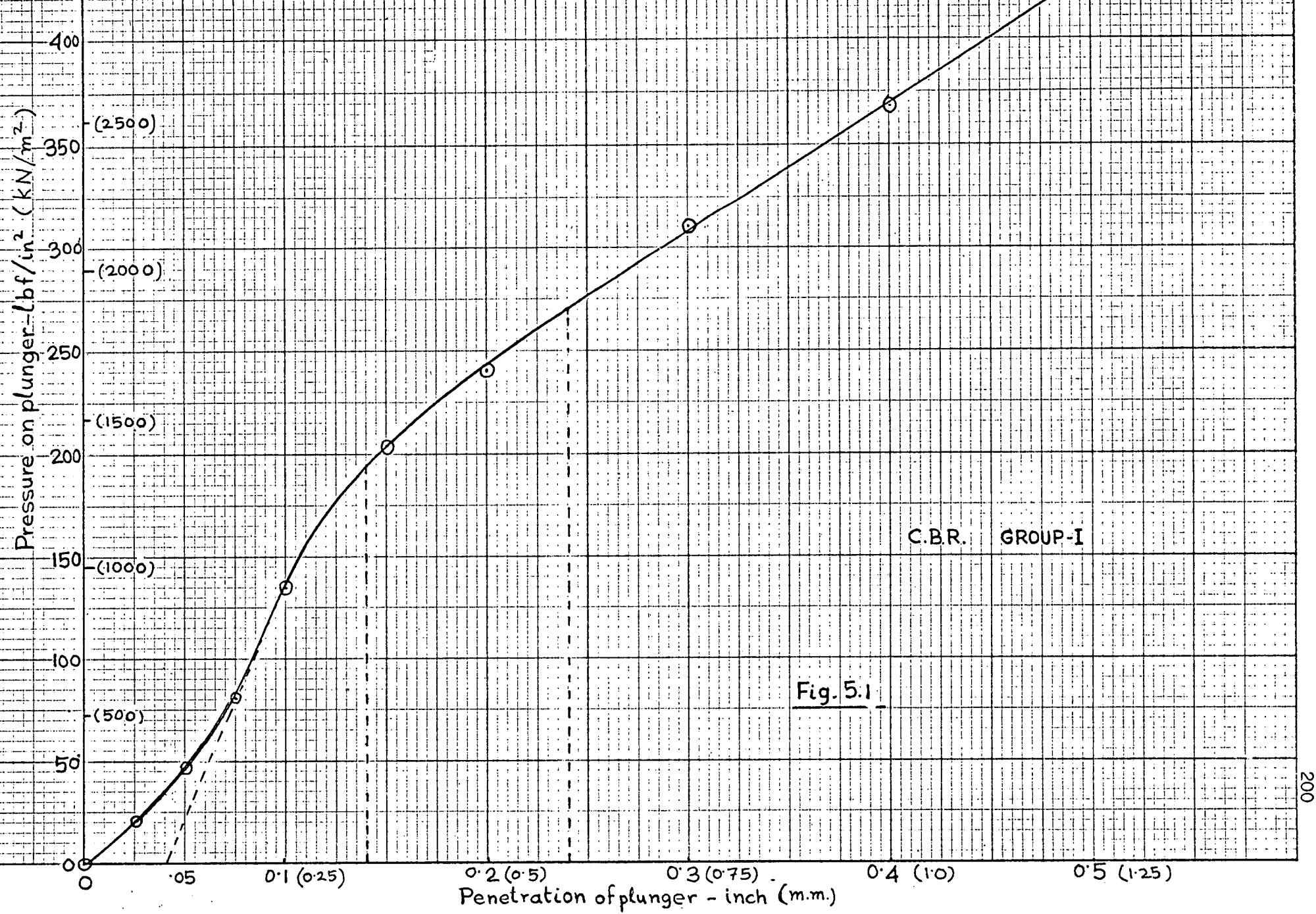
TABLE 7.2.3SIEVE ANALYSIS (AFTER COMPACTION AND REPEAT LOADING)GROUP B (test spot 1)

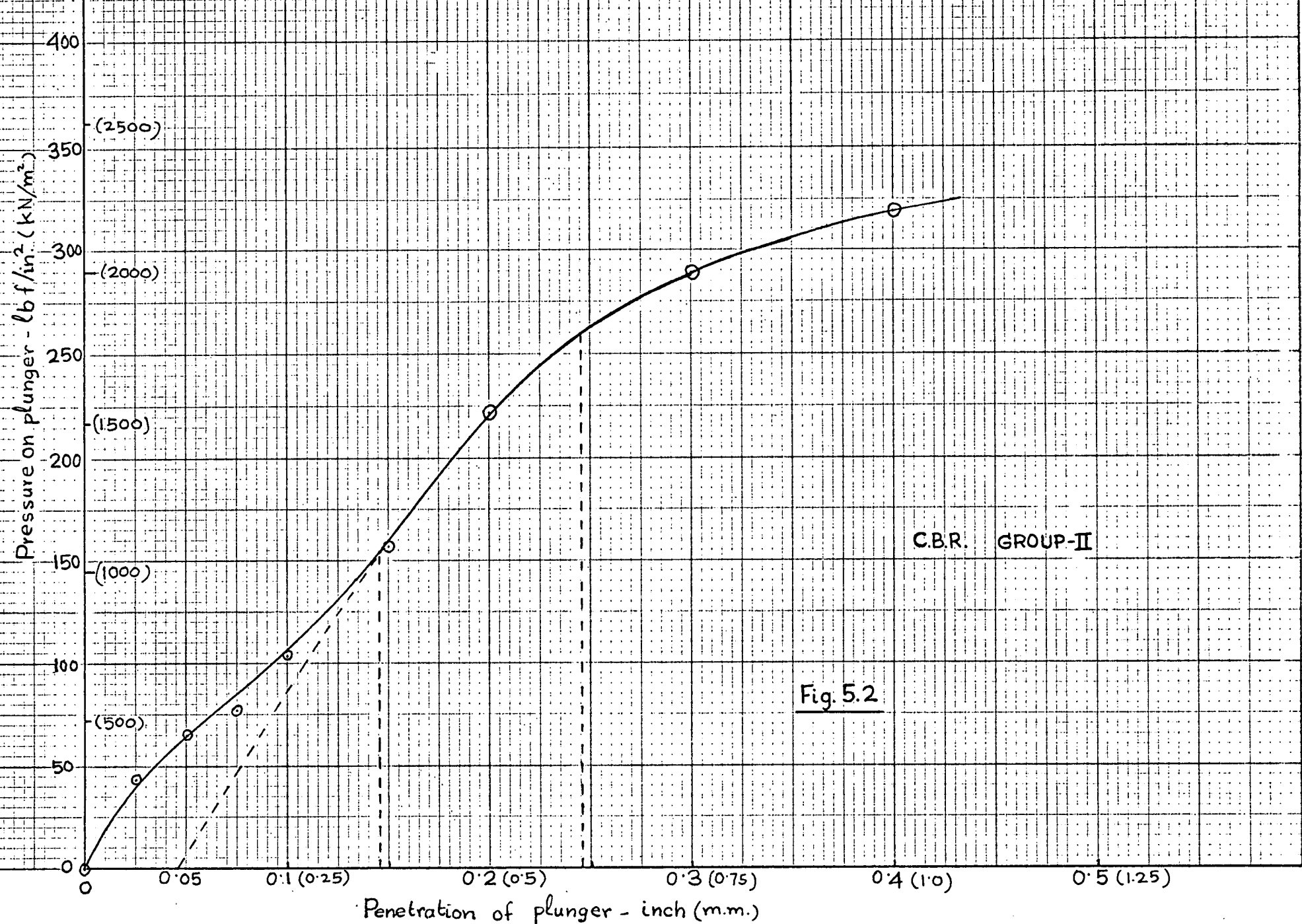
Sieve Size		Weight retained (gm)	% retained	Cumulative % retained	% passing
mm./ (microns)	Inch/ (No)				
50.80	2	0	0	0	100
38.10	1½	436	4.6	4.6	95.4
31.75	1¼	721	7.6	12.2	87.8
25.40	1	1234	13.0	25.2	74.8
19.05	¾	1324	13.9	39.1	60.9
12.70	½	1163	12.2	51.3	48.7
9.52	⅜	662	7.0	58.3	41.7
6.35	¼	663	7.0	65.3	34.7
5.00	—	378	4.0	69.3	30.7
3.18	⅛	544	5.7	75.0	25.0
2.40	(7)	296	3.1	78.1	21.9
1.68	(10)	484	5.1	83.2	16.8
1.20	(14)	306	3.2	86.4	13.6
(850)	(18)	244	2.6	89.0	11.0
(600)	(25)	192	2.0	91.0	9.0
(420)	(36)	147	1.6	92.6	7.4
(300)	(52)	108	1.1	93.7	6.3
(210)	(72)	90	1.0	94.7	5.3
(150)	(100)	87	0.9	95.6	4.4
(75)	(200)	155	1.6	97.2	2.8
Passing No. 200		264	2.8	100	—

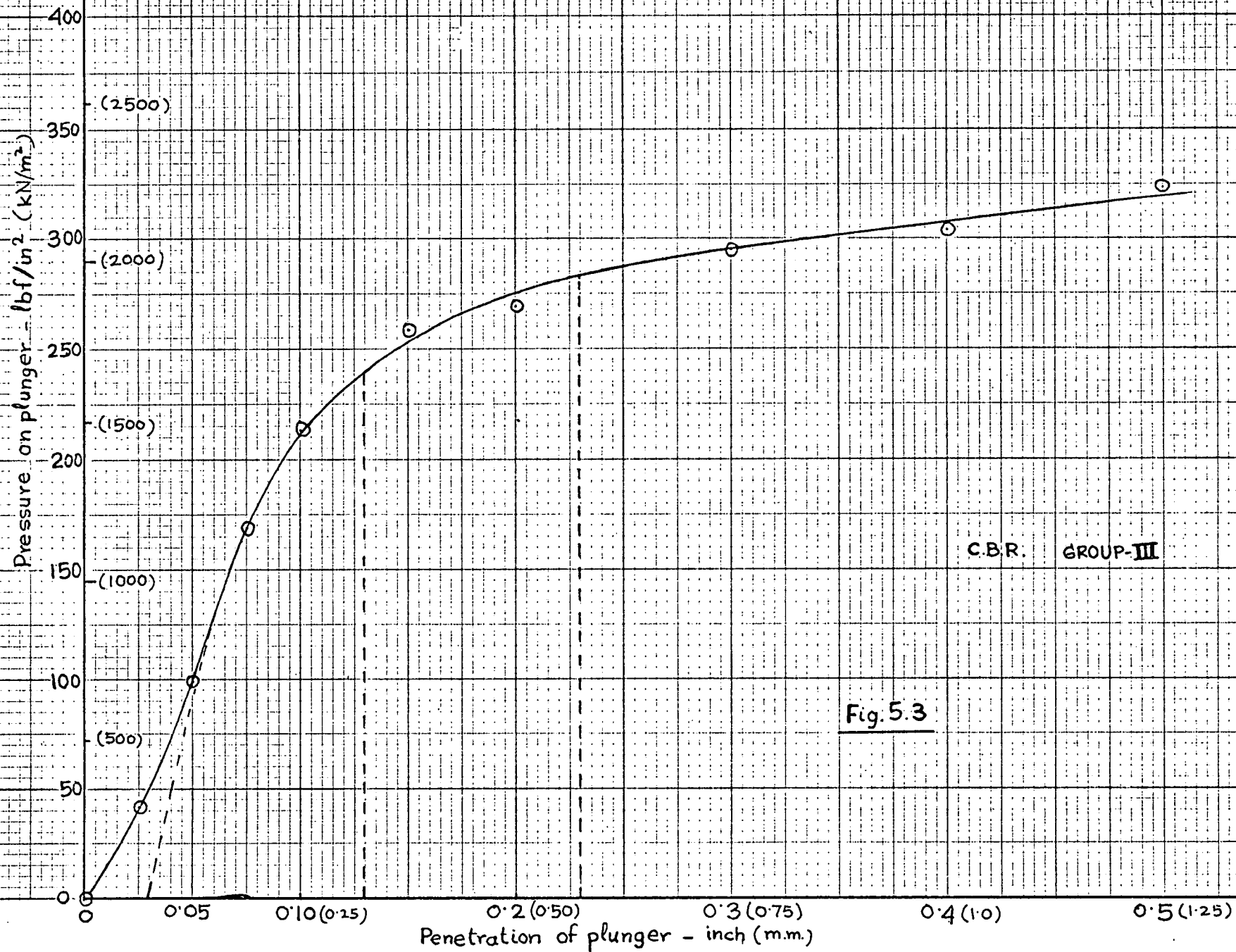
TABLE 7.2.4 SIEVE ANALYSIS (AFTER COMPACTION AND REPEAT LOADING)

GROUP B (test spot 2)

Sieve Size		Weight retained (gm)	% retained	Cumulative % retained	% passing
mm./ (microns)	Inch/ (No)				
50.80	2	0	0	0	100
38.10	1½	624	6.4	6.4	93.6
31.75	1¼	882	9.0	15.4	84.6
25.40	1	1013	10.3	25.7	74.3
19.05	¾	1270	13.0	38.7	61.3
12.70	½	1165	11.9	50.6	49.4
9.52	⅜	726	7.4	58.0	42.0
6.35	¼	642	6.6	64.6	35.4
5.00	-	315	3.2	67.8	32.2
3.18	⅙	538	5.5	73.3	26.7
2.40	(7)	301	3.1	76.4	23.6
1.68	(10)	513	5.2	81.6	18.4
1.20	(14)	324	3.3	84.9	15.1
(850)	(18)	279	2.8	87.7	12.3
(600)	(25)	218	2.2	89.9	10.1
(420)	(36)	175	1.8	91.7	8.3
(300)	(52)	130	1.3	93.0	7.0
(210)	(72)	109	1.1	94.1	5.9
(150)	(100)	109	1.1	95.2	4.8
(75)	(200)	180	1.8	97.0	3.0
Passing No. 200		296	3.0	100	-







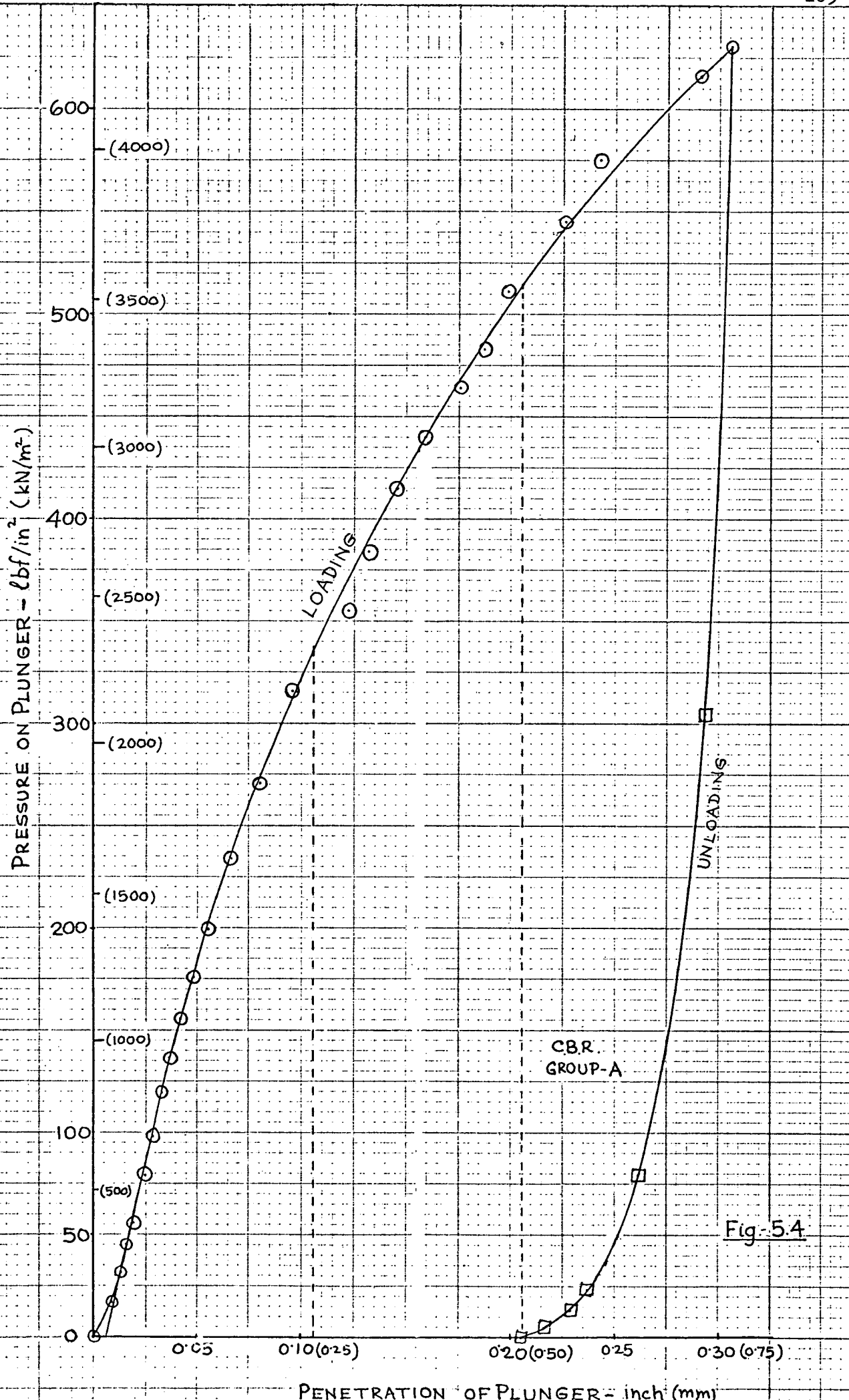


Fig-5.4

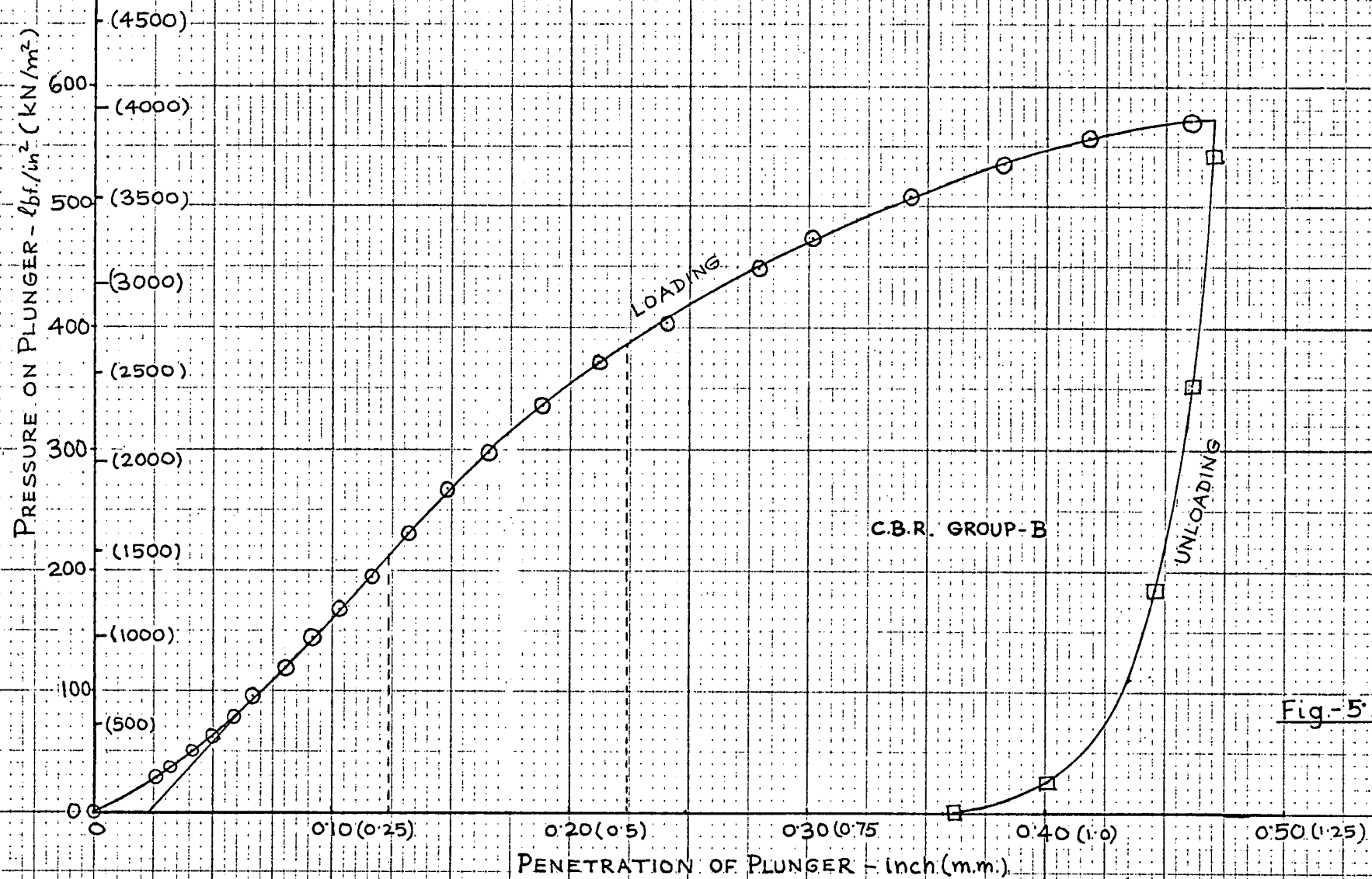


Fig-5.5

APPENDIX C

FIELD TRIAL

APPENDIX CFIELD TRIAL

CONTRACTORS: EDINBURGH ROAD DEVELOPMENT COMPANY

SITE : Near Juniper Green (after old railway bridge but
before Juniper Green).

Construction of new widened road dismantling the
old one with new specifications.

Crushed stone aggregates 2" nominal size.

DATE : 5th August 1975.

ROAD ROLLER

NAME OF ROAD ROLLER: AVELING BARFORD

WEIGHT: 8 - 10 tons (10 tons when ballasted)

Rear Wheel -

Contact area - width = (i) 320mm (ii) 290mm (12")

Width of wheel - 530mm (21")

Diameter of wheel - 1350mm

Front Wheel -

Contact area - (i) 230/210mm (ii) 190mm (8")

Width of wheel - 2 x 400mm

Diameter of wheel - 1,000mm

Intensity of pressure -

$$\text{Front wheel} - \frac{78221\text{b}^* \times 25.4 \times 25.4}{2 \times 400 \times 205} = 30.771\text{b}/\square"$$

$$\text{Rear Wheel} - \frac{11,1901\text{b}^* \times 25.4 \times 25.4}{2 \times 530 \times 305} = 22.331\text{b}/\square"$$

Load on 9" diameter specimen - $30.77 \times 63.623 = .874 \text{ tons} \approx 890\text{Kg}$

(* - H.M.S.O. Soil Mech. for Road Engineers, pp. 181)

APPENDIX D

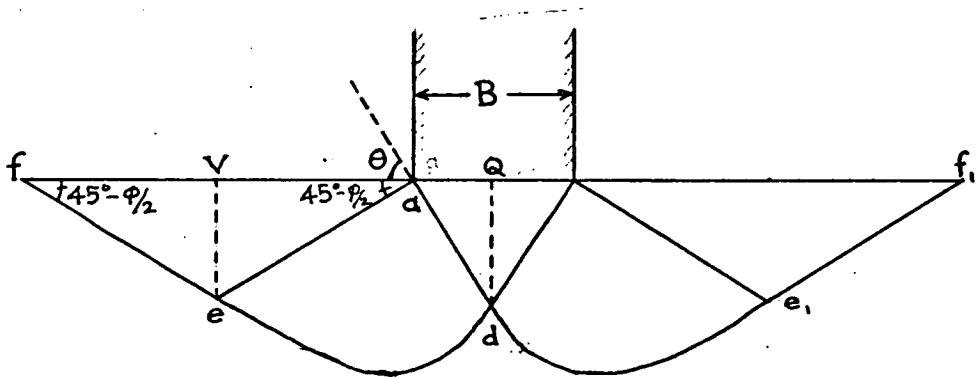
DIMENSIONS OF EXPERIMENTAL ROAD BASE

APPENDIX D

DIMENSIONS OF EXPERIMENTAL ROAD BASE

As stated in Chapter 7 the experimental pavement dimensions were chosen such that a strip 305 mm (12") wide was available at the centre of the bed relatively free from boundary effects due to the edge walls around the pavement. The approach was based upon Terzaghi's (1943) conditions for the general shear failure of soil supporting a shallow, continuous footing, considering the roller as a continuous footing (continuous in the direction of rolling).

In the following figure (from Terzaghi, 1943) B is the width of the footing (roller) and ff_1 is the width of the bed required to fulfil the conditions mentioned above. From the figure $ff_1 = B + 2af$.



The situation was very much simplified by first assuming $ad = ae$. Although this was an over-simplification, this gave an indication of the size of the experimental pavement initially. The value of ϕ was taken to be 44° as indicated by a series of triaxial tests on similar materials by Boyd (1976). With the above assumptions and the ϕ value, the width of the bed was found to be $5.71B$ or $1.73m$ (69") from the relation $af = B \frac{\cos(45^\circ - \phi/2)}{\cos(45^\circ + \phi/2)}$ if the boundary effects were to be avoided.

A second assumption that $dQ = eV$ leads to a width of the bed required to be 12.1B or 3.69m (12.1') from the relation $af =$

$$B \frac{\tan(45^\circ + \phi/2)}{\tan(45^\circ - \phi/2)}.$$

Yet a third assumption that eV (Fig. above) equals the thickness of the brick aggregate road base which is approximately 230 mm (9") gave a width of 2.46 m (8.07') from the relation $af = \frac{2eV}{\tan(45^\circ - \phi/2)}$. The above values were found assuming that the soil was ideal and the base smooth. If the base is considered rough it would mean that there would be friction and adhesion between the soil and the base of the footing. This would result in a change in the angle θ (of zone I) from $45^\circ + \phi/2$ to ϕ , but there would be no change in zone III. Now the width of the bed ff_1 becomes 3.56B or 1.09m (3.56') for the first assumption, 5.55B or 1.69m (5.55') for the second assumption and there is no change for the third assumption as the value for af changes to $B \frac{\cos(45^\circ - \phi/2)}{\cos \phi}$ and $B \frac{\tan \phi}{\tan(45^\circ - \phi/2)}$ for the first and second assumptions respectively.

The other set of calculations were done considering ϕ as 38° which was the lowest value Boyd obtained and are tabulated below.

	<u>Smooth base</u>	<u>Rough base</u>
First assumption	1.55m (5.1')	1.0m (3.28')
Second assumption	2.87m (9.41')	1.28m (4.2')
Third assumption	2.18m (7.15')	2.18m (7.15')

The other dimension of the road base, i.e. the length, was determined keeping in mind that there would be three tests at different places with a loading plate 153mm (6") in diameter and ideally there should be no interference with each other. The approach was based

upon considering the dispersion of the load from the edge of the loading plate at an angle $45^\circ - \phi/2$ with the vertical as is the case in the active Rankine zone. If the passive Rankine zone is considered the angle of dispersion would be $45^\circ + \phi/2$ with the vertical. Three tests were to be performed at three different places along the central 305 mm (12") strip which would be free from boundary effects. The lengths required for the two conditions as stated above and different ϕ values are given below in a tabular form.

ϕ	dispersion of load (< with vertical)		
	$45^\circ - \phi/2$	$45^\circ + \phi/2$	45°
44°	1.04m (41")	3.69m (145")	1.83m (72")
38°	1.13m (44")	3.27m (129")	

The width of the bed varied between 1 metre and 3.69 metres and the length between 1.04 and 3.69 metres with various assumptions.

Additional factors influencing the choice of size of the test area were that each and every brick had to be tested to estimate its compressive strength and then each one had to be broken by hand to make the aggregates. This was very time consuming. The size of the experimental bed and thus the volume of the aggregate required for a test was a very important consideration. The width of the pavement including the boundary walls determined the size of the portal frames which would support the repeat loading jack.

From the above considerations a compromise was made between the

theoretical minimum requirements and the practicable maximum.

It was decided that the net dimensions of the pavement would be 1.75 metres (69") and 1.68 metres (66") for width and length respectively.

In the above calculations the values of ϕ were taken to be 44° and 38° as the maximum and minimum values from Boyd (1976).

Later values of ϕ were obtained from triaxial tests in the present study and were found to vary between 38° and 43° for group A aggregates and 34° to 44.5° for group B aggregates.

APPENDIX E

COMPUTATION OF STRESSES IN THE FIRST INTERFACE OF A THREE
LAYER SYSTEM

APPENDIX E

COMPUTATION OF STRESSES IN THE FIRST INTERFACE OF A THREE LAYER SYSTEM

To compute the stresses in a three layer system the E values of the three layers, the thickness of the first and the second layer and the radius of the loaded area are required. The E values are estimated as follows:

E.1 E for sand

The E value of a material can be found out from its shear modulus, G from the equation $E = 2(1 + \mu)G$ where μ is Poisson's ratio. "For dense soils and solid granular materials such as concrete or sandstone Poisson's ratio increases from small values of the order of magnitude of 0.2 at low stresses to more than 0.5 at very high stresses (Terzaghi, 1943).

$$\therefore E = 2(1 + 0.5) \times G = 3G \text{ at high stresses}$$

$$\text{and } E = 2(1 + 0.2) \times G = 2.4 G \text{ at very low stresses.}$$

The sand layer being the sub-base would be under a low stress and the value of E has been taken to be 2.4G. The average value of G found out approximately from two direct shear tests (secant modulus from the first point of the graph) is 10,844 per sq. in. and hence the value of E equals to 26,025 p.s.i). (The value of E for sand varies between 24,000 p.s.i. to 40,000 p.s.i. For making calculations simpler the value of E has been taken as 25,000 p.s.i.

E.2 E for crushed brick aggregates

The value of E for compacted crushed brick aggregates has been found out by first drawing a tangent to the initial part of the stress-deformation curve of the in situ modified C.B.R. test performed on the

compacted aggregates with a 6" diameter circular plate instead of the standard plunger. Then from the tangent a stress value and the corresponding deformation was taken. Considering the stress dispersed at an angle $(45^\circ - \phi/2)$ from the vertical (for Active Rankine's pressure) from the edge of the loading plate. ϕ is angle of shearing resistance. The value of E was determined by integration from the equation $\Delta l = \int_0^l \frac{P}{A \cdot E} dx$, where P = total load on the loading plate, Δl is the corresponding deformation, A is the area under the load and l is the thickness of the brick aggregate road base as follows:

$$A = \pi(3 + \tan\theta \cdot x)^2$$

$$\sigma = \frac{P}{A} = \frac{P}{\pi(3 + \tan\theta \cdot x)^2}$$

$$\epsilon = \frac{\sigma}{E} = \frac{P}{\pi(3 + \tan\theta \cdot x)^2 E}$$

$$\Delta l = \int_0^l \epsilon \cdot dx$$

$$= \int_0^l \frac{P}{\pi E(3 + \tan\theta \cdot x)^2} \cdot dx$$

$$= \frac{P}{\pi \cdot E} \cdot \int_0^l (3 + \tan\theta \cdot x)^{-2} dx$$

$$= \frac{P}{\pi \cdot E} \cdot \frac{1}{(-1)\tan\theta} \left[(3 + \tan\theta \cdot x)^{-1} \right]_0^l$$

$$= \frac{P}{\pi \cdot E \tan\theta} \left(\frac{1}{3} - \frac{1}{3 + \tan\theta \cdot l} \right)$$

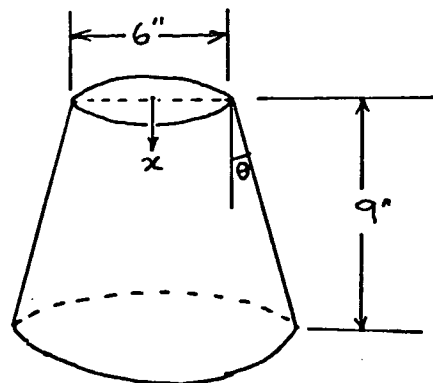
$$\text{or } E = \frac{P}{\Delta l} \times \frac{1}{\tan\theta} \left(.3333 - \frac{1}{(3 + 9 \tan\theta \cdot l)} \right)$$

$$\theta = (45 - \frac{\phi}{2}), \text{ the value of } \phi \text{ was taken as } 44^\circ \text{ from Boyd (1976)}$$

and hence $\theta = 23^\circ$.

$$P = p \cdot A, \quad p = 500 \text{ p.s.i.}, \quad A = \pi \times 3^2 \text{ in}^2 \text{ and the corresponding}$$

$$\Delta l = .097 \text{ in.}$$



$$\therefore E = \frac{500 \times \pi \times 3^2}{.097} \times \frac{1}{\pi \times \tan 23^\circ} \cdot \left(0.333 - \frac{1}{(3 + 9 \tan 23^\circ)}\right)$$

$$= 20,400 \text{ p.s.i.}$$

(Later an average value of ϕ was found to be 40.33° from 3 triaxial tests done on aggregate A and hence the value of E becomes 19,422 p.s.i.) For making calculations simpler the value of E was taken as 20,000 p.s.i.

E.3 E for surfacing

"The actual value for E of the bituminous surface layer will depend on the composition of the mix, the grade of bitumen used, the degree of compaction, the speed of loading and the temperature and will probably lie within the range 1.5×10^5 to 1.5×10^6 lb/sq.in." according to Pell (1965). For conditions prevailing in Bangladesh it would be appropriate to choose the lowest value. Hence the value of E for the top layer was taken as 1.5×10^5 lb. per sq. in.

E.4 Computation of stress factors

a = radius of the loading plate = 3 in.

h_1 = thickness of the surfacing = 2 in.

h_2 = thickness of the second layer = 12 in.*

(* the thickness of the compacted brick aggregate road base is 9 inches but there is a brick flat soling 3" thick beneath the brick aggregate road base and over the sand layer. Hence it would be more appropriate to consider the thickness of the second layer as 12").

With the above values, the vertical stress in the first interface has been calculated from Jones' (1962) graphical representation of the vertical compressive stress factor, $6Z_1$ by interpolation.

$$k_1 = E_1/E_2 = \frac{1.5 \times 10^5}{2.0 \times 10^4} = 7.5$$

$$k_2 = E_2/E_3 = \frac{2.0 \times 10^4}{2.5 \times 10^4} = 0.8$$

$$a_1 = \frac{a}{h_2} = \frac{3}{12} = 0.25$$

$$H = \frac{h_1}{h_2} = \frac{2}{12} = 0.167$$

$$\sigma_{z_1} = 0.52 \approx .50 \quad (50\%)$$

Thus the vertical stress on top of the road base has been taken as 50% of the wheel load at the surface of the bituminous wearing course.

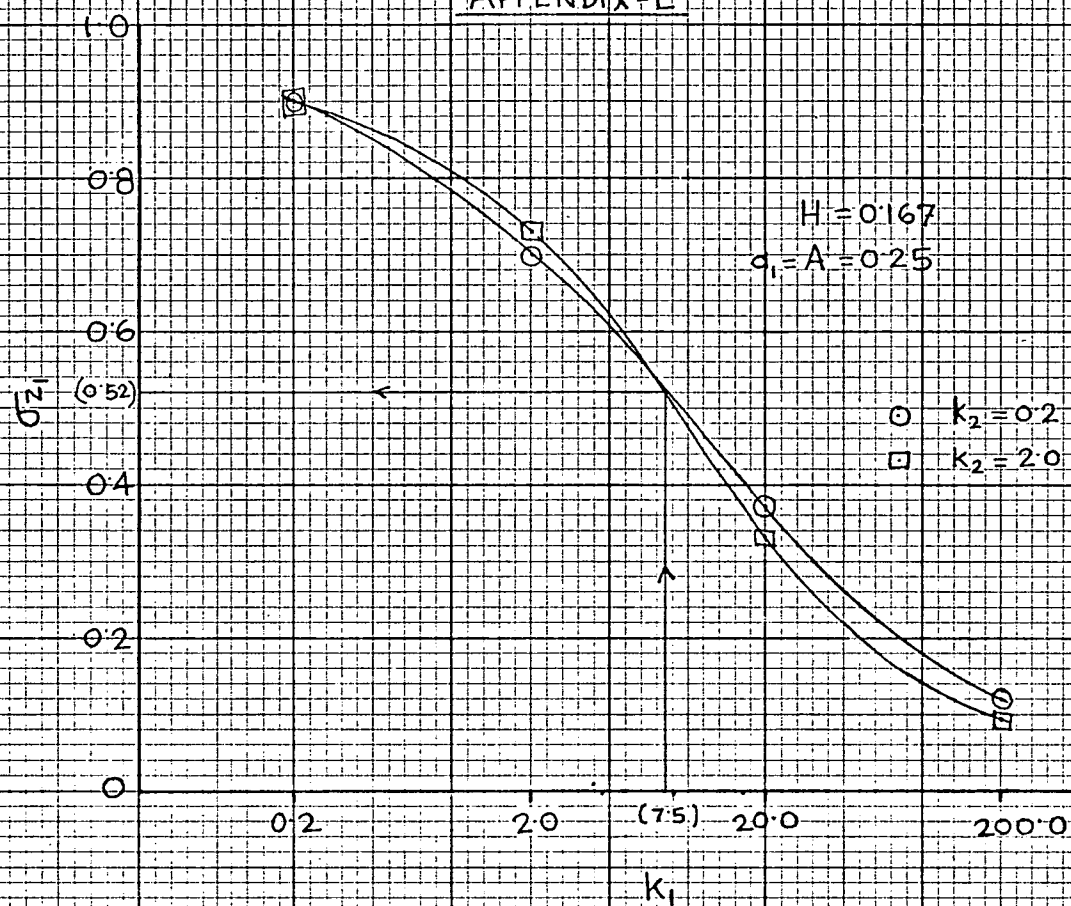
The value of σ_{z_1} was also determined by interpolation of the figures given in Jones' tables and was found to be 0.535. Similarly the horizontal stress factor σ_{r_2} was determined from the vertical and radial stress factor $\sigma_{z_1} - \sigma_{r_2}$ which was found out by interpolation from the tables.

$$\sigma_{z_1} - \sigma_{r_2} = .29$$

$$\sigma_{r_2} = \sigma_{z_1} - .29 = .52 - .29 = 0.23$$

Thus the horizontal stress at the top of the second layer is 23% of the wheel load at the surface of the bituminous wearing course.

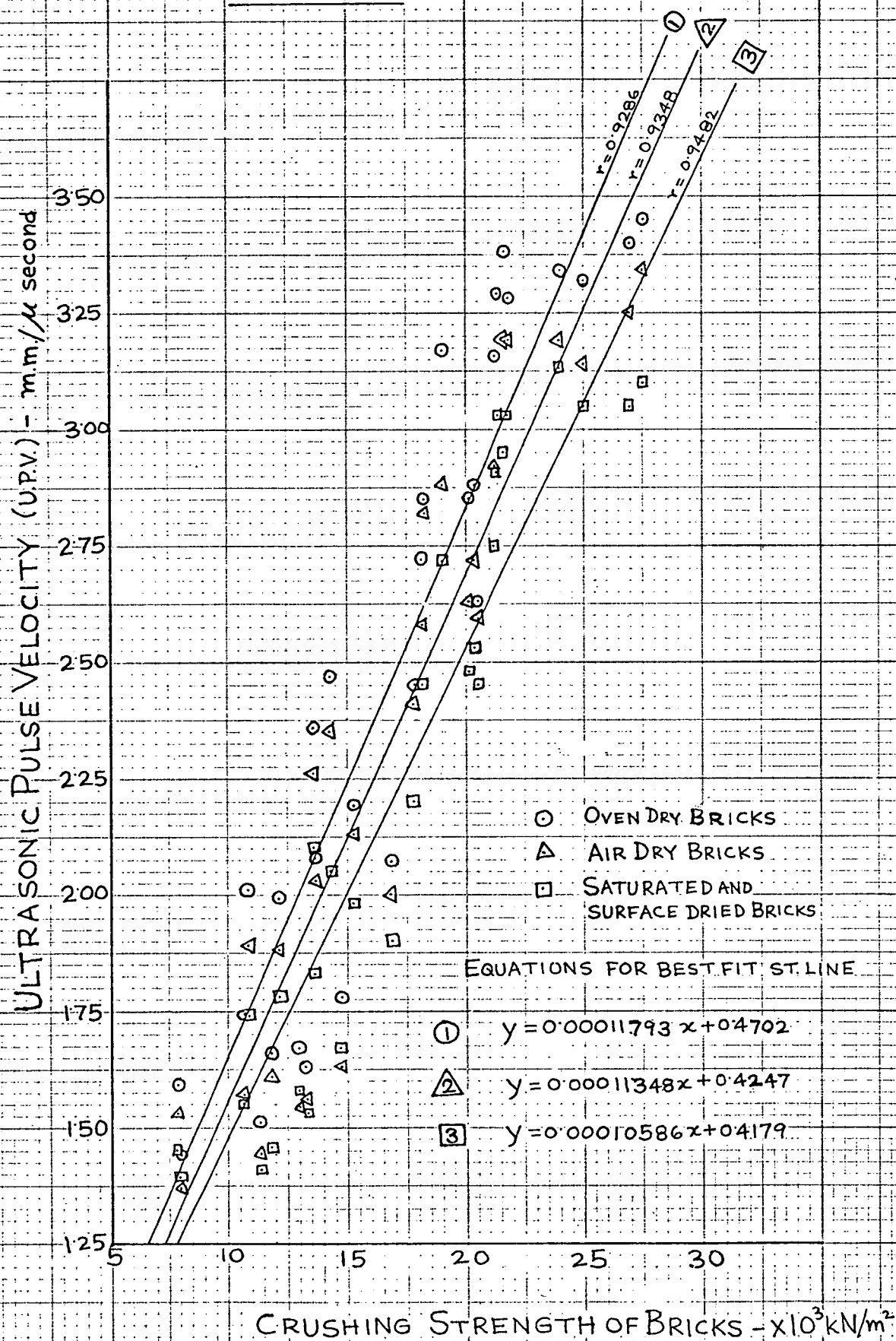
APPENDIX-E



APPENDIX F

SELECTION AND GROUPING OF BRICKS BY PUNDIT

APPENDIX - F



GROUP A

Sl.No.	Length	time	U.P.V.	C.S. R	Sl.No.	Length	time	U.P.V.	C.S. R
	mm	μ sec	mm/ μ sec	kN/m ²		mm	μ sec	mm/ μ sec	kN/m ²
1	215	66.9	3.21	24546	36	215	65.0	3.31	25442
2	214	68.5	3.12	23753	37	216	84.9	2.54	18641 x
3	216	92.9	2.33	16792 x	38	215	68.7	3.13	23836
4	215	89.3	2.41	17496 x	39	215	67.4	3.19	24367
5	216	76.2	2.83	21193 x	40	216	78.5	2.75	20490 x
6	215	67.7	3.18	24277	41	214	67.1	3.19	24367
7	216	76.1	2.84	21283 x	42	219	79.0	2.77	20669 x
8	215	65.9	3.26	24981	43	214	71.2	3.01	22780
9	215	68.7	3.13	23836	44	221	94.6	2.34	16875 x
10	214	64.9	3.30	25333	45	215	81.0	2.65	19607 x
11	214	70.3	3.04	23042	46	214	71.1	3.01	22780
12	214	66.4	3.22	24629	47	221	87.0	2.54	18641 x
13	214	65.8	3.25	24898	48	216	67.4	3.20	24457
14	216	67.5	3.20	24457	49	217	81.5	2.66	19696 x
15	215	69.0	3.12	23753	50	214	67.5	3.17	24188
16	215	76.9	2.80	20931 x	51	215	73.8	2.91	21897 x
17	214	72.8	2.94	22166	52	215	71.1	3.02	22870
18	216	90.0	2.40	17406 x	53	215	74.6	2.88	21635 x
19	216	76.9	2.81	21021 x	54	217	93.8	2.31	16613 x
20	215	68.3	3.15	24015	55	215	67.0	3.21	24546
21	220	89.0	2.47	18020 x	56	215	68.9	3.12	23753
22	215	69.2	3.11	23663	57	215	86.9	2.47	18020 x
23	215	70.1	3.07	23311	58	215	71.3	3.02	22870
24	215	67.9	3.17	24188	59	216	71.9	3.00	22690
25	215	70.7	3.04	23042	60	215	77.5	2.77	20669 x
26	216	72.1	3.00	22690	61	214	72.5	2.95	22249
27	215	65.6	3.28	25160	62	215	78.3	2.75	20490 x
28	215	68.6	3.13	23836	63	214	66.4	3.22	24629
29	214	74.8	2.86	21462 x	64	215	66.0	3.26	24981
30	215	70.2	3.06	23222	65	216	74.4	2.90	21814 x
31	214	68.1	3.14	23925	66	220	79.3	2.77	20669 x
32	215	65.6	3.28	25160	67	217	79.0	2.75	20490 x
33	216	79.9	2.70	20048 x	68	215	69.3	3.10	23574
34	215	72.9	2.95	22249	69	220	91.0	2.42	17585 x
35	215	69.3	3.10	23574	70	215	66.7	3.22	24629

Sl.No.	Length	time	U.P.V.	C.S. R	Sl.No.	Length	time	U.P.V.	C.S. R
	mm	μ sec	mm/ μ sec	kN/m ²		mm	μ sec	mm/ μ sec	kN/m ²
71	214	70.6	3.03	22960	106	214	62.5	3.42	26395 x
72	215	71.6	3.00	22690	107	214	63.8	3.35	25774
73	214	65.6	3.26	24981	108	214	61.6	3.47	26837 x
74	216	67.3	3.21	24546	109	215	68.8	3.13	23836
75	215	72.8	2.95	22249	110	215	73.5	2.93	22076
76	215	64.9	3.31	25422	111	218	80.2	2.72	20228 x
77	216	72.8	2.97	22428	112	214	64.3	3.33	25602
78	214	62.0	3.45	26657 x	113	214	63.0	3.40	26216
79	219	73.6	2.98	22518	114	214	66.8	3.20	24457
80	219	68.4	3.20	24457	115	214	64.0	3.34	25691
81	214	66.8	3.20	24457	116	214	60.8	3.52	27271 x
82	215	70.6	3.05	23132	117	214	61.3	3.49	27009 x
83	219	87.9	2.49	18199 x	118	214	63.9	3.35	25774
84	215	66.5	3.23	24719	119	215	61.4	3.50	27099 x
85	216	64.5	3.35	25774	120	215	65.4	3.29	25250
86	214	64.9	3.30	25333	121	214	63.5	3.40	26216
87	220	82.2	2.68	19876 x	122	215	65.6	3.28	25160
88	215	74.1	2.90	21814 x	123	214	63.0	3.40	26216
89	220	73.0	3.01	22780	124	215	64.7	3.32	25512
90	215	67.8	3.17	24188	125	214	65.3	3.28	25160
91	216	75.0	2.88	21635 x	126	214	61.5	3.48	26919 x
92	215	69.9	3.08	23394	127	215	65.8	3.27	25071
93	213	61.9	3.44	26568 x	128	213	60.3	3.53	27361 x
94	215	69.8	3.08	23394	129	217	86.0	2.52	18461 x
95	215	70.3	3.06	23222	130	214	64.9	3.30	25333
96	216	88.1	2.45	17847 x	131	215	72.0	2.99	22608
97	216	84.4	2.56	18813 x	132	215	70.2	3.06	23222
98	214	66.3	3.23	24719	133	214	67.1	3.19	24367
99	216	81.6	2.65	19607 x	134	224	83.4	2.69	19959 x
100	215	71.7	3.00	22690	135	214	64.3	3.33	25602
101	215	66.3	3.24	24808	136	216	65.4	3.30	25333
102	214	63.4	3.38	26043	137	216	74.7	2.89	21725 x
103	215	70.6	3.05	23132	138	215	68.5	3.14	23925
104	215	67.9	3.17	24188	139	214	62.8	3.41	26305 x
105	214	66.1	3.24	24808	140	218	81.9	2.66	19696 x

Sl.No.	Length	time	U.P.V.	C.S. R	Sl.No.	Length	time	U.P.V.	C.S. R
	mm	μ sec	mm/ μ sec	kN/m ²		mm	μ sec	mm/ μ sec	kN/m ²
141	215	68.2	3.15	24015	176	219.5	84.2	2.61	19255 x
142	214	61.7	3.47	26837 x	177	216	80.3	2.69	19959 x
143	215	69.6	3.09	23484	178	216	76.4	2.83	21193 x
144	215	63.9	3.36	25864	179	216	84.4	2.56	18813 x
145	215	68.1	3.16	24105	180	214	65.4	3.27	25071
146	214	63.9	3.35	25774	181	214	63.6	3.36	25864
147	214	63.8	3.35	25774	182	216	62.3	3.47	26837 x
148	215	67.8	3.17	24188	183	215	68.7	3.13	23836
149	215	68.9	3.12	23753	184	217	89.4	2.43	17668 x
150	214	63.5	3.37	25954	185	216	72.3	2.99	22608
151	216	70.6	3.06	23222	186	215	69.7	3.08	23394
152	216	75.8	2.85	21373 x	187	213.5	65.3	3.27	25071
153	219	78.2	2.80	20931 x	188	215	69.3	3.10	23574
154	217	77.9	2.79	20842 x	189	214	66.6	3.21	24539
155	215	65.7	3.27	25071	190	215.5	84.5	2.55	18731 x
156	220	86.5	2.54	18641 x	191	214	68.3	3.13	23836
157	215	66.7	3.22	24629	192	215	63.4	3.39	26126
158	216	72.0	3.00	22690	193	215	65.6	3.28	25160
159	218	91.9	2.37	17144 x	194	214.5	63.9	3.36	25864
160	214	68.9	3.11	23663	195	214	59.1	3.62	28154 x
161	216	67.4	3.20	24457	196	214	68.6	3.12	23746
162	216	64.0	3.38	26043	197	214	64.4	3.32	25512
163	218	79.4	2.75	20490 x	198	214	64.4	3.32	25512
164	215	67.3	3.19	24367	199	215	62.2	3.46	26747 x
165	215	73.3	2.93	22076	200	215	63.9	3.36	25864
166	215.5	73.8	2.92	21987 x	201	215	65.5	3.28	25160
167	214	62.8	3.41	26305 x	202	214	64.2	3.33	25602
168	216	81.9	2.64	19524 x	203	214	64.4	3.32	25512
169	214	60.6	3.53	27361 x	204	215	67.5	3.19	24367
170	215	70.8	3.04	23042	205	215	66.7	3.22	24629
171	221	76.8	2.88	21635 x	206	214	65.7	3.26	24981
172	215	64.9	3.31	25422	207	216	87.1	2.48	18110 x
173	216	68.4	3.16	24105	208	214	63.0	3.40	26216
174	216	69.9	3.09	23484	209	219	94.7	2.31	16613 x
175	218	83.8	2.60	19172 x	210	215	62.5	3.44	26568 x

Sl.No.	Length	time	U.P.V.	C.S. R	Sl.No.	Length	time	U.P.V.	C.S. R
	mm	μ sec	mm/ μ sec	kN/m ²		mm	μ sec	mm/ μ sec	kN/m ²
211	215	63.8	3.37	25954	246	215	79.8	2.69	19959 x
212	215	73.8	2.91	21897 x	247	215.5	66.4	3.25	24891
213	214.5	70.6	3.04	23042	248	215.5	77.6	2.78	20752 x
214	217	89.1	2.44	17758 x	249	215	66.8	3.22	24629
215	216	64.3	3.36	25864	250	216.5	81.7	2.65	19607 x
216	214	62.4	3.43	26478 x	251	214	66.6	3.21	24539
217	214	63.0	3.40	26216	252	215	70.2	3.06	23222
218	217	81.4	2.67	19786 x	253	217	94.1	2.31	16613 x
219	215	63.6	3.38	26043	254	215.5	71.6	3.01	22780
220	215	68.8	3.13	23836	255	215	70.7	3.04	23042
221	214	65.6	3.26	24981	256	215	67.7	3.18	24277
222	215	64.4	3.34	25691	257	215	77.2	2.78	20752 x
223	215	70.8	3.04	23042	258	214.5	62.5	3.43	26478 x
224	215	63.9	3.36	25864	259	216	64.8	3.33	25602
225	215	75.4	2.85	21373 x	260	214.5	63.2	3.39	26126
226	215	67.8	3.17	24188	261	219	75.9	2.89	21725 x
227	214	64.3	3.33	25602	262	220	79.6	2.76	20579 x
228	214.5	66.3	3.24	24808	263	214.5	60.8	3.53	27361 x
229	214.5	66.8	3.21	24539	264	215	80.8	2.66	19696 x
230	213.5	60.9	3.51	27189 x	265	216	61.6	3.51	27189 x
231	215.5	69.3	3.11	23663	266	214	63.0	3.40	26216
232	215	66.2	3.25	24891	267	215	69.5	3.09	23484
233	215	69.9	3.08	23394	268	215	71.3	3.02	22870
234	215	66.3	3.24	24808	269	215.5	69.4	3.11	23663
235	215	67.0	3.21	24539	270	214.5	64.4	3.33	25602
236	215	68.0	3.16	24105	271	215	63.8	3.37	25954
237	213	63.5	3.35	25774	272	215	69.7	3.08	23394
238	214.5	70.3	3.05	23132	273	215	66.4	3.24	24808
239	214.5	63.0	3.40	26216	274	216	65.9	3.28	25160
240	214	64.0	3.34	25691	275	215.5	70.6	3.05	23132
241	214	61.8	3.46	26747 x	276	215	66.8	3.22	24629
242	214	64.5	3.32	25512	277	215	68.9	3.12	23746
243	214.5	62.8	3.42	26395 x	278	214	65.3	3.28	25160
244	215	69.4	3.10	23574	279	217	77.5	2.80	20931 x
245	214	62.0	3.45	26657 x	280	215	67.4	3.19	24367

Sl.No.	Length	time	U.P.V.	C.S. R	Sl.No.	Length	time	U.P.V.	C.S. R
	mm	μ sec	mm/ μ sec	kN/m ²		mm	μ sec	mm/ μ sec	kN/m ²
281	215.5	67.7	3.18	24277	316	215	64.6	3.33	25602
282	216	73.2	2.95	22249	317	214.5	64.4	3.33	25602
283	217	69.8	3.11	23663	318	215	70.8	3.04	23042
284	216	66.3	3.26	24981	319	214.5	64.6	3.32	25512
285	218.5	84.6	2.58	18993 x	320	215.5	73.3	2.94	22166
286	215	65.4	3.29	25250	321	215.5	62.4	3.45	26657 x
287	215	72.3	2.97	22428	322	214.5	65.8	3.26	24981
288	215.5	64.7	3.33	25602	323	214	63.3	3.38	26043
289	214.5	64.9	3.31	25422	324	215	66.0	3.26	24981
290	215	74.1	2.90	21814 x	325	214	69.6	3.07	23311
291	215	73.2	2.94	22166	326	214.5	66.9	3.21	24539
292	215	65.9	3.26	24981	327	214	71.8	2.98	22518
293	215	68.9	3.12	23746	328	216.5	75.8	2.86	21462 x
294	218	74.6	2.92	21987 x	329	213.5	62.6	3.41	26305 x
295	214.5	63.0	3.40	26216	330	214	64.5	3.32	25512
296	216	69.9	3.09	23484	331	215	72.3	2.97	22428
297	215.5	63.3	3.40	26216	332	214	65.3	3.28	25160
298	215	62.8	3.42	26395 x	333	215	70.0	3.07	23311
299	214	69.4	3.08	23394	334	215	65.3	3.29	25250
300	216	63.3	3.41	26305 x	335	216	65.3	3.31	25422
301	215.5	68.2	3.16	24105	336	216	66.8	3.23	24719
302	214.5	62.9	3.41	26305 x	337	215.5	70.3	3.07	23311
303	215	71.7	3.00	22690	338	217	90.8	2.39	17316 x
304	219	79.6	2.75	20490 x	339	216	74.6	2.90	21814 x
305	219	73.7	2.97	22428	340	214.5	66.6	3.22	24629
306	220	75.3	2.92	21987 x	341	214.5	65.7	3.26	24981
307	219.5	65.8	3.34	25691	342	217	81.6	2.66	19696 x
308	216.5	76.7	2.82	21104 x	343	214	61.2	3.50	27099 x
309	216	71.2	3.03	22960	344	214.5	65.2	3.29	25250
310	215	68.0	3.16	24105	345	214	61.5	3.48	26919 x
311	216	84.5	2.56	18813 x	346	215	64.0	3.36	25864
312	214	66.2	3.23	24719	347	216	76.1	2.84	21283 x
313	214	67.7	3.16	24105	348	214	61.6	3.47	26837 x
314	215	71.3	3.02	22870	349	215	67.7	3.18	24277
315	215.5	68.0	3.17	24188	350	215	71.6	3.00	22690

Sl. No.	Length	time	U.P.V.	C.S. R	Sl.No.	Length	time	U.P.V.	C.S. R
	mm	μ sec	mm/ μ sec	kN/m ²		mm	μ sec	mm/ μ sec	kN/m ²
351	215.5	77.6	2.78	20752 x	386	216	67.2	3.21	24539
352	213	59.8	3.56	27623 x	387	215.5	68.3	3.16	24105
353	215	67.8	3.17	24188	388	215	65.6	3.28	25160
354	215	68.7	3.15	24015	389	215	64.4	3.34	25691
355	215	66.8	3.22	24629	390	215	64.4	3.34	25691
356	215	71.2	3.02	22870	391	215	66.4	3.24	24808
357	216	74.9	2.88	21635 x	392	214	63.8	3.35	25774
358	215	62.4	3.45	26657 x	393	215	65.8	3.27	25071
359	215	63.2	3.40	26216	394	215	69.7	3.08	23394
360	214	66.5	3.22	24629	395	215	71.4	3.01	22780
361	216	70.6	3.06	23222	396	215	64.3	3.34	25691
362	215	64.6	3.33	25602	397	214.5	64.0	3.35	25774
363	215	72.6	2.96	22339	398	215	71.8	2.99	22608
364	214	61.6	3.47	26837 x	399	214	62.3	3.43	26478 x
365	216	73.9	2.92	21987 x	400	214.5	64.6	3.32	25512
366	215	64.3	3.34	25691	401	214	65.9	3.25	24891
367	215	65.8	3.27	25071	402	215	69.4	3.10	23574
368	216	79.5	2.72	20228 x	403	216	81.5	2.65	19607 x
369	214	64.6	3.31	25422	404	214.5	64.1	3.35	25774
370	215	71.2	3.02	22870	405	217	82.6	2.63	19434 x
371	215.5	64.3	3.35	25774	406	216.5	77.0	2.81	21021 x
372	214.0	63.6	3.36	25864	407	215	67.9	3.17	24188
373	215	64.6	3.33	25602	408	214	63.4	3.38	26043
374	215	70.4	3.05	23132	409	214	61.2	3.50	27099 x
375	215	64.3	3.34	25691	410	215	61.7	3.48	26919 x
376	218	78.8	2.77	20669 x	411	214.5	60.0	3.58	27803 x
377	215	67.5	3.19	24367	412	214	61.4	3.49	27009 x
378	215.5	65.6	3.29	25250	413	216	65.9	3.28	25160
379	215	65.8	3.27	25071	414	214.5	63.4	3.38	26043
380	214	63.1	3.39	26126	415	215.5	67.9	3.17	24188
381	215	72.2	2.98	22518	416	214	63.5	3.37	25954
382	215.5	67.1	3.21	24539	417	215.5	67.8	3.18	24277
383	215	68.0	3.16	24105	418	214.5	71.9	2.98	22518
384	216	79.2	2.73	20317 x	419	215	66.3	3.24	24808
385	217	76.2	2.85	21373 x	420	215	72.5	2.97	22428

Sl.No.	Length	time	U.P.V.	C.S. R	Sl.No.	Length	time	U.P.V.	C.S. R
	mm	μ sec	mm/ μ sec	kN/m ²		mm	μ sec	mm/ μ sec	kN/m ²
421	215.5	63.3	3.40	26216	456	214.5	62.1	3.45	26657 x
422	215	66.4	3.24	24808	457	215.5	79.6	2.71	20138 x
423	215.5	69.9	3.08	23394	458	214.5	64.8	3.31	25422
424	215	67.8	3.17	24188	459	216	71.1	3.04	23042
425	215	63.7	3.38	26043	460	214.5	64.0	3.35	25774
426	214.5	63.4	3.38	26043	461	215.5	68.7	3.14	23925
427	215	68.2	3.15	24015	462	214	63.6	3.36	25864
428	215	65.3	3.29	25250	463	214.5	66.0	3.25	24891
429	214.5	62.5	3.43	26478 x	464	216	72.0	3.00	22690
430	215	70.8	3.04	23042	465	215.5	67.1	3.21	24539
431	214.5	63.4	3.38	26043	466	214.5	64.8	3.31	25422
432	215	68.4	3.14	23925	467	214	62.5	3.42	26395 x
433	216	70.3	3.07	23311	468	215.5	69.3	3.11	23663
434	216	72.4	2.98	22518	469	216	84.4	2.56	18813 x
435	215	68.0	3.16	24105	470	215	66.8	3.22	24629
436	215	66.3	3.24	24808	471	214.5	64.8	3.31	25422
437	214	63.1	3.39	26126	472	215.5	79.4	2.71	20138 x
438	214	64.8	3.30	25333	473	215	67.2	3.20	24457
439	215	71.6	3.00	22690	474	215	63.8	3.37	25954
440	214	63.4	3.38	26043	475	214	69.4	3.08	23394
441	215	64.8	3.32	25512	476	217	90.7	2.39	17316 x
442	214.5	67.3	3.19	24367	477	217	92.7	2.34	16875 x
443	215.5	65.1	3.31	25422	478	215	69.6	3.09	23484
444	214.5	74.5	2.88	21635 x	479	218	101.8	2.14	15115 x
445	215.5	77.9	3.31	25422	480	214	63.3	3.38	26043
446	215.5	76.1	2.83	21193 x	481	216	70.8	3.05	23132
447	215.5	71.1	3.03	22960	482	218	89.0	2.45	17847 x
448	215	72.4	2.97	22428	483	216	71.7	3.01	22780
449	214	65.4	3.27	25071	484	215	72.0	2.99	22608
450	214.5	62.6	3.43	26478 x	485	215	70.4	2.97	22428
451	214.5	65.5	3.27	25071	486	216	79.3	2.72	20228 x
452	216	78.8	2.74	20400 x	487	214	66.9	3.20	24457
453	214	64.0	3.34	25691	488	214	64.7	3.31	25422
454	215.5	71.8	3.00	22690	489	215	68.6	3.13	23836
455	215.5	72.6	2.97	22428	490	215	74.2	2.90	21814 x

Sl.No.	Length	time	U.P.V.	C.S.	R	Sl.No.	Length	time	U.P.V.	C.S.	R
	mm	μ sec	mm/ μ sec	kN/m ²			mm	μ sec	mm/ μ sec	kN/m ²	
491	215	77.1	2.79	20842	x	526	215	66.8	3.22	24629	
492	216	71.9	3.00	22690		527	218.5	93.2	2.34	16875	x
493	215	72.0	2.99	22608		528	215	63.9	3.36	25864	
494	216	87.8	2.46	17937	x	529	215	65.8	3.27	25071	
495	216	72.7	2.97	22428		530	213.5	61.5	3.47	26837	x
496	214	62.1	3.45	26657	x	531	215	70.2	3.06	23222	
497	215	69.7	3.08	23394		532	214.5	70.4	3.05	23132	
498	215.5	76.1	2.83	21193	x	533	215	64.3	3.34	25691	
499	214	64.7	3.31	25422		534	214.5	65.8	3.26	24981	
500	214.5	70.2	3.06	23222		535	215	70.5	3.05	23132	
501	215	71.3	3.02	22870		536	215	68.5	3.14	23925	
502	215	75.0	2.87	21545	x	537	215	67.9	3.17	24188	
503	214	67.4	3.18	24277		538	215.5	77.0	2.80	20931	x
504	217	91.7	2.37	17144	x	539	215.5	71.1	3.03	22960	
505	215	70.3	3.06	23222		540	214.5	66.6	3.22	24629	
506	215	73.8	2.91	21897	x	541	215	66.9	3.21	24539	
507	215.5	72.7	2.96	22339		542	215	68.5	3.14	23925	
508	216	67.5	3.20	24457		543	215	65.2	3.30	25333	
509	214.5	70.0	3.06	23222		544	215	65.6	3.28	25160	
510	215.5	67.6	3.19	24367		545	214	68.2	3.14	23925	
511	215	63.9	3.36	25864		546	215	75.7	2.84	21283	x
512	215.5	78.6	2.74	20400	x	547	216	77.8	2.78	20752	x
513	217.5	86.7	2.51	18379	x	548	214	64.8	3.30	25333	
514	214.5	64.0	3.35	25774		549	214.5	70.2	3.06	23222	
515	215	62.4	3.45	26657	x	550	217	70.7	3.07	23311	
516	214	66.5	3.22	24629		551	216	72.2	2.99	22608	
517	215.5	71.0	3.04	23042		552	214	66.5	3.22	24629	
518	216	67.4	3.20	24457		553	215	70.0	3.07	23311	
519	214.5	62.5	3.43	26478	x	554	215	69.8	3.08	23394	
520	214	63.1	3.39	26126		555	215	66.2	3.25	24891	
521	215	74.5	2.89	21725		556	214.5	68.0	3.15	24015	
522	215	74.9	2.87	21545	x	557	215	71.6	3.00	22690	
523	214.5	64.2	3.34	25691		558	214.5	64.6	3.33	25602	
524	216	81.0	2.67	19786	x	559	215	64.0	3.36	25864	
525	216	77.6	2.78	20752	x	560	214	63.7	3.36	25864	

Sl.No.	Length	time	U.P.V.	C.S. R	Sl.No.	Length	time	U.P.V.	C.S. R
	mm	μ sec	mm/ μ sec	kN/m ²		mm	μ sec	mm/ μ sec	kN/m ²
561	215	67.3	3.19	24367	596	215	76.2	2.82	21104 x
562	215.5	69.8	3.09	23484	597	214.5	67.1	3.20	24457
563	215	64.8	3.32	25512	598	214.5	68.2	3.15	24015
564	215	72.8	2.95	22249	599	214.5	67.9	3.16	24105
565	215	70.6	3.05	23132	600	215	63.9	3.36	25864
566	214	64.2	3.33	25602	601	215	71.2	3.02	22870
567	215	74.5	2.89	21725 x	602	214.5	72.5	2.96	22339
568	214.5	71.4	3.00	22690	603	214	60.9	3.51	27189 x
569	215	67.0	3.21	24539	604	215	70.1	3.07	23311
570	215	71.2	3.02	22870	605	215	68.4	3.14	23925
571	215.5	68.9	3.13	23836	606	215	66.1	3.25	24891
572	215.5	72.4	2.98	22518	607	214	64.1	3.34	25691
573	214	65.9	3.25	24891	608	214	66.5	3.22	24629
574	214.5	69.0	3.11	23663	609	214.5	65.3	3.28	25160
575	213	64.0	3.33	25602	610	215	67.7	3.18	24277
576	215.5	70.9	3.04	23042	611	214.5	72.7	2.95	22249
577	214.5	61.8	3.47	26837 x	612	215	68.4	3.14	23925
578	214	66.3	3.23	24719	613	216	72.5	2.98	22518
579	215	70.4	3.05	23132	614	214.5	72.0	2.98	22518
580	214.5	78.6	2.73	20317 x	615	215	68.5	3.14	23925
581	215	69.2	3.11	23663	616	214	66.7	3.21	24539
582	215	67.8	3.17	24188	617	214	68.6	3.12	23746
583	216	69.8	3.09	23484	618	214	67.8	3.16	24105
584	214.5	61.3	3.50	27099 x	619	213.5	66.1	3.23	24719
585	215	72.7	2.96	22339	620	215	68.4	3.14	23925
586	216	65.9	3.28	25160	621	215	66.2	3.25	24891
587	214.5	68.2	3.15	24015	622	214.5	67.1	3.20	24457
588	214.5	67.0	3.20	24457	623	214	67.8	3.16	24105
589	215	76.8	2.80	20931 x	624	214.5	68.4	3.14	23926
590	215.5	72.7	2.96	22339	625	215	71.1	3.02	22870
591	215	67.5	3.19	24367	626	215	69.6	3.09	23484
592	215	69.1	3.11	23663	627	215	68.8	3.13	23836
593	214.5	64.5	3.33	25602	628	215.5	73.3	2.94	22166
594	215	68.4	3.14	23925	629	216	69.3	3.12	23746
595	215	67.8	3.17	24188	630	214	69.1	3.10	23574

GROUP B

Sl.No.	Length	time	U.P.V.	C.S.	R	Sl.No.	Length	time	U.P.V.	C.S.	R
	mm	μ sec	mm/ μ sec	kN/m ²			mm	μ sec	mm/ μ sec	kN/m ²	
1	218	79.9	2.73	20317	x	37	216.5	84.0	2.58	18993	
2	218	101.2	2.16	15295		38	216	79.5	2.72	20228	x
3	217.5	99.5	2.19	15557		39	216	86.0	2.51	18379	
4	216	71.3	3.03	22960	x	40	220	122.0	1.80	12101	x
5	220	117.6	1.87	12735	x	41	219	106.7	2.05	14322	x
6	218	94.6	2.30	16523		42	218.5	83.3	2.62	19345	x
7	219	100.5	2.18	15467		43	219	108.0	2.03	14150	x
8	220	108.6	2.03	14150	x	44	216	73.4	2.94	22166	x
9	221	103.0	2.15	15205		45	218.5	94.4	2.31	16613	
10	215	75.3	2.86	21462	x	46	216	76.9	2.81	21021	x
11	217	81.6	2.66	19696	x	47	221	126.2	1.75	11680	x
12	219.5	111.0	1.98	13708	x	48	218	96.3	2.27	16261	
13	215.5	88.2	2.44	17758		49	217	75.3	2.88	21635	x
14	220	116.7	1.89	12915	x	50	221.5	95.9	2.31	16613	
15	218.5	93.0	2.35	16964		51	219	91.3	2.40	17406	
16	219	112.3	1.95	13439	x	52	218.5	101.5	2.15	15205	
17	218	92.5	2.35	16964		53	221	105.4	2.10	14764	x
18	217	93.6	2.32	16702		54	217.5	92.3	2.36	17054	
19	217	81.5	2.67	19786	x	55	218	88.6	2.46	17937	
20	216.5	88.9	2.44	17758		56	217.5	102.1	2.13	15026	x
21	217.5	95.9	2.27	16261		57	217.5	90.0	2.42	17585	
22	219	105.9	2.07	14501	x	58	217.5	99.1	2.19	15557	
23	218.5	94.6	2.31	16613		59	216.5	83.8	2.58	18993	
24	219	98.2	2.23	15909		60	217	88.9	2.44	17758	
25	216	79.1	2.73	20317	x	61	218.5	96.1	2.27	16261	
26	217	92.9	2.34	16875		62	217	85.8	2.53	18551	
27	217	85.6	2.54	18641		63	217.5	90.4	2.41	17496	
28	220	79.6	2.76	20579	x	64	217	90.1	2.41	17496	
29	218.5	97.6	2.24	15999		65	217	79.0	2.75	20490	x
30	219	103.3	2.12	14943	x	66	218.5	99.6	2.19	15557	
31	219.5	109.5	2.00	13881	x	67	215	72.8	2.95	22249	x
32	216.5	84.4	2.57	18903		68	216	86.4	2.50	18289	
33	216	85.3	2.54	18641		69	218	102.5	2.13	15026	x
34	217	90.4	2.40	17406		70	215	82.9	2.59	19082	
35	220.5	92.5	2.38	17233		71	217	81.4	2.67	19786	x
36	217	80.1	2.71	20138	x	72	220	128.7	1.71	11328	x

Sl.No.	Length	time	U.P.V.	C.S.	R	Sl.No.	Length	time	U.P.V.	C.S.	R
	mm	μ sec	mm/ μ sec	kN/m ²			mm	μ sec	mm/ μ sec	kN/m ²	
73	216	77.7	2.78	20752	x	107	219	96.4	2.27	16261	
74	215.5	84.3	2.55	18731		108	221	128.4	1.72	11418	x
75	215.5	78.2	2.76	20579	x	109	219	97.4	2.25	16088	
76	216	76.7	2.82	21104	x	110	216	71.4	3.03	22960	x
77	216	81.7	2.64	19524	x	111	219.5	111.3	1.97	13618	x
78	217	83.5	2.60	19172		112	219.5	109.6	2.00	13881	x
79	216	84.2	2.57	18903		113	218.5	87.7	2.49	18199	
80	217	81.5	2.66	19696	x	114	217	83.5	2.60	19172	
81	218	96.0	2.27	16261		115	217.5	86.4	2.51	18379	
82	220	106.7	2.06	14412	x	116	217	88.4	2.45	17847	
83	219	97.7	2.24	15999		117	217.5	92.7	2.35	16964	
84	217	79.1	2.74	20400	x	118	216	83.8	2.57	18903	
85	218	89.8	2.43	17668		119	217	88.9	2.44	17758	
86	217	86.4	2.51	18379		120	216.5	83.1	2.61	19255	
87	219	98.8	2.22	15819		121	220	111.7	1.97	13618	x
88	220	105.0	2.10	14764	x	122	217	81.5	2.66	19696	x
89	217.5	85.0	2.56	18813		123	218.5	99.4	2.20	15647	
90	215	73.0	2.95	22249	x	124	218	92.8	2.35	16964	
91	218	90.7	2.40	17406		125	218	107.8	2.02	14060	x
92	216	77.6	2.78	20752	x	126	218	91.3	2.39	17316	
93	217	86.0	2.52	18461		127	218.5	96.4	2.27	16261	
94	219	107.0	2.05	14322	x	128	217	92.0	2.36	17054	
95	218.5	87.8	2.49	18199		129	219	100.8	2.17	15378	
96	216	79.0	2.73	20317	x	130	218.5	101.4	2.15	15205	
97	219	98.4	2.22	15819		131	219	106.0	2.07	14501	x
98	217.5	103.0	2.11	14853	x	132	218	95.82.28		16350	
99	219.5	109.6	2.00	13881	x	133	216.5	78.6	2.76	20579	x
100	218.5	110.0	1.99	13798	x	134	222	115.3	1.99	13798	x
101	218	93.6	2.33	16792		135	217	86.5	2.51	18379	
102	218	91.8	2.38	17233		136	218.5	94.4	2.31	16613	
103	217	92.3	2.35	16964		137	214	73.9	2.90	21814	x
104	216.5	84.0	2.58	18993		138	214.5	70.1	3.06	23222	x
105	218	92.6	2.35	16964		139	217	84.5	2.57	18903	
106	218	106.6	2.05	14322	x	140	216.5	83.6	2.59	19082	

Sl. No.	Length	time	U.P.V.	C.S.	R	Sl.No.	Length	time	U.P.V.	C.S.	R
	mm	μ sec	mm/ μ sec	kN/m^2			mm	μ sec	mm/ μ sec	kN/m^2	
141	216	76.9	2.81	21021	x	175	216.5	86.1	2.51	18379	
142	219	95.5	2.29	16440		176	218	94.2	2.31	16613	
143	220	100.5	2.19	15557		177	217	82.2	2.64	19524	x
144	217	90.9	2.39	17316		178	217	89.0	2.44	17758	
145	217.5	83.4	2.61	19255		179	219.5	102.9	2.13	15012	x
146	218.5	94.9	2.30	16523		180	217.5	88.2	2.47	18020	
147	218	87.7	2.49	18199		181	217	90.1	2.41	17496	
148	218	96.4	2.26	16171		182	218	89.9	2.43	17668	
149	217	85.7	2.54	18641		183	218	90.1	2.42	17585	
150	218.5	102.5	2.13	15026	x	184	217.5	97.2	2.24	15999	
151	218	83.3	2.62	19345	x	185	218	102.0	2.14	15115	x
152	215	76.8	2.80	20931	x	186	218	85.5	2.55	18731	
153	217	84.0	2.58	18993		187	218.5	90.5	2.42	17585	
154	219.5	96.5	2.27	16261		188	218	97.8	2.23	15909	
155	220	106.9	2.06	14412	x	189	216	82.9	2.61	19255	
156	217.5	90.8	2.40	17406		190	217.5	92.6	2.35	16964	
157	220.5	114.5	1.93	13267	x	191	217	90.6	2.40	17406	
158	216.5	87.6	2.47	18020		192	220	135.0	1.63	10624	x
159	219.5	91.8	2.39	17316		193	217	99.2	2.19	15557	
160	217	81.4	2.67	19786	x	194	222	92.7	2.39	17316	
161	219	100.2	2.19	15557		195	216	79.5	2.71	20138	x
162	216.5	89.4	2.42	17585		196	219	101.4	2.16	15295	
163	217	94.4	2.30	16523		197	218.5	77.3	2.83	21193	x
164	217.5	83.5	2.60	19172		198	217	93.9	2.31	16613	
165	218	98.4	2.21	15729		199	219	69.9	3.13	23836	x
166	216	82.9	2.61	19255		200	219	101.0	2.17	15378	
167	218.5	101.2	2.16	15295		201	215	70.4	3.05	23132	x
168	218.5	102.8	2.13	15026	x	202	220.5	100.6	2.19	15557	
169	220	103.6	2.12	14943	x	203	219	115.9	1.89	12915	x
170	218	78.7	2.77	20669	x	204	216.5	73.9	2.93	22076	x
171	218	97.4	2.24	15999		205	222	107.5	2.07	14501	x
172	215.5	79.3	2.71	20138	x	206	217	74.8	2.90	21814	x
173	217	88.6	2.45	17847		207	217	89.7	2.42	17585	
174	217	84.4	2.57	18903		208	217	85.4	2.54	18641	

Sl.No.	Length	time	U.P.V.	C.S.	R	Sl.No.	Length	time	U.P.V.	C.S.	R
	mm	μ sec	mm/ μ sec	kN/m ²			mm	μ sec	mm/ μ sec	kN/m ²	
209	216.5	78.3	2.77	20669	x	243	217	80.9	2.68	19875	x
210	220	87.6	2.51	18379		244	216.5	87.3	2.48	18110	
211	220	79.0	2.78	20752	x	245	223.5	122.4	1.83	12384	x
212	218.5	95.7	2.28	16350		246	219	107.8	2.03	14150	x
213	219	104.8	2.09	14674	x	247	216.5	80.2	2.70	20048	x
214	216.5	86.2	2.51	18379		248	220	108.0	2.03	14150	x
215	217	83.9	2.59	19082		249	217.5	85.0	2.56	18813	
216	217	82.2	2.64	19524	x	250	215.5	78.5	2.75	20490	x
217	217	83.4	2.60	19172		251	220	109.5	2.01	13970	x
218	217	82.2	2.64	19524	x	252	217	75.6	2.87	21545	x
219	220	108.4	2.03	14150	x	253	220	113.4	1.94	13356	x
220	220	115.2	1.91	13087	x	254	216	69.6	3.10	23574	x
221	217	84.8	2.56	18813		255	217	81.0	2.68	19876	x
222	219	77.0	2.84	21283	x	256	218.5	88.2	2.48	18110	
223	220	85.7	2.56	18813		257	218	104.6	2.08	14584	x
224	215	72.9	2.95	22249	x	258	220	99.7	2.21	15729	
225	219	97.2	2.25	16088		259	217	79.2	2.74	20400	x
226	219.5	87.4	2.51	18379		260	221	118.8	1.86	12646	x
227	219.5	97.0	2.27	16261		261	217	82.9	2.62	19345	x
228	216.5	90.7	2.39	17316		262	217	81.4	2.67	19786	x
229	218	96.8	2.25	16088		263	217	75.8	2.86	21462	x
230	219	75.1	2.91	21897	x	264	217	82.3	2.64	19524	x
231	220	117.9	1.86	12646	x	265	218.5	82.2	2.66	19696	x
232	216	79.2	2.73	20317	x	266	219	90.5	2.42	17585	
233	218	79.9	2.73	20317	x	267	217	77.8	2.79	20842	x
234	216.5	80.6	2.69	19959	x	268	216.5	69.0	3.14	23925	x
235	218.5	95.8	2.28	16350		269	217.5	84.6	2.57	18903	
236	221	88.5	2.50	18289		270	217	84.5	2.57	18903	
237	219.5	85.3	2.57	18903		271	219	99.9	2.19	15557	
238	219.5	96.4	2.28	16350		272	218	92.0	2.37	17144	
239	218	81.6	2.67	19786	x	273	219.5	87.9	2.50	18289	
240	218	89.6	2.43	17668		274	218	94.7	2.30	16523	
241	218	86.2	2.53	18551		275	220.5	114.1	1.93	13267	x
242	217	90.9	2.39	17316		276	218	84.5	2.57	18903	

Sl.No.	Length	time	U.P.V.	C.S.	R	Sl.No.	Length	time	U.P.V.	C.S.	R
	mm	μ sec	mm/ μ sec	kN/m^2			mm	μ sec	mm/ μ sec	kN/m^2	
277	220	107.7	2.04	14232	x	289	218	88.7	2.46	17937	
278	219	100.2	2.19	15557		290	217	83.0	2.61	19255	
279	219.5	97.4	2.25	16088		291	217	77.4	2.80	20904	x
280	219	99.6	2.20	15647		292	217.5	90.5	2.40	17406	
281	220	100.4	2.19	15557		293	220	88.7	2.48	18110	
282	218	99.0	2.20	15647		294	218.5	101.3	2.16	15295	
283	218	87.4	2.50	18289		295	217	83.0	2.61	19255	
284	217	84.3	2.58	18993		296	219	100.8	2.18	15467	
285	218.5	83.0	2.63	19434	x	297	219.5	95.0	2.31	16613	
286	218	92.0	2.37	17144		298	219	81.7	2.68	19876	x
287	220.5	115.6	1.91	13087	x	299	220	110.5	1.99	13798	x
288	219	89.1	2.46	17937		300	219	97.9	2.24	15999	

C.S. = Crushing Strength (estimated from U.P.V.)

x = rejected

CRUSHING STRENGTH

GROUP A

Total 630 Nos.

Mean = 23,663.31429 kN/m^2

S.Dev. = 2304.374608 kN/m^2

Selected 454 Nos.

(Range 22076 kN/m^2 to 26,216 (both inclusive))

Mean = 24,315.5859 $\approx$ 24316 kN/m^2

S.Dev. = 1,143.510967 $\approx$ 1144 kN/m^2

GROUP B

Total 300 Nos.

Mean Crushing Strength = 17,493.73 kN/m^2

Standard Deviation = 2,552.01132 kN/m^2

Selected 174 Nos.

(Range 15,178 kN/m^2 to 19,317 kN/m^2)

Mean Crushing Strength = 17,315.3046 kN/m^2

Standard Deviation = 1,232.435323 kN/m^2

APPENDIX G

DESIGN OF THE LOAD ON ROLL OF THE ROAD ROLLER

APPENDIX GDESIGN OF THE LOAD ON ROLL OF THE ROAD ROLLER

Since it was not possible to have an eight ton smooth wheel roller, a roller much smaller in size but producing the same effect was designed. The design was based on the value of

$$\frac{\text{Load on roll (lb.)}}{\text{width of roll (in.)} \times \text{diameter of roll (in.)}}$$

which gives a good indication of the performance of rollers of different sizes. This value for the rear roll of an eight ton roller was taken from Table 9.5 of "Soil Mechanisms for Road Engineers", H.M.S.O. (1952) and the load was calculated as follows:

$$\frac{\text{Load on roll (lb.)}}{\text{width of roll (in.)} \times \text{diameter of roll (in.)}} = 5.8$$

or load on roll (lb.) = 5.8 x width of roll (in.) x diameter of roll (in.)

The width of the roller is 12" and the diameter is 16".

$$\begin{aligned} \therefore \text{Load on roll (lb.)} &= 5.8 \times 12 \times 16 \\ &= 1,114 \text{ lb.} \\ &= 506.4 \text{ kg} \end{aligned}$$
