VERTICAL STRESS DISTRIBUTION IN BALLAST AND SUBGRADE UNDER STATICALLY LOADED WOODEN TIES

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AUGUST, 1999

CERTIFICATION

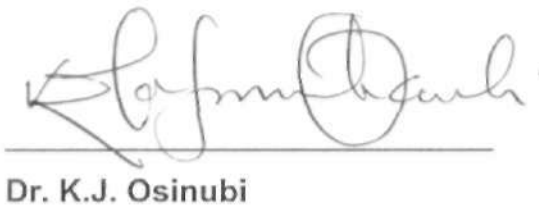
This thesis **"VERTICAL STRESS DISTRIBUTION IN BALLAST AND SUBGRADE UNDER STATICALLY WOODEN-TIES"** by **EDALERE, ADEGOKE ILESANMI** has met the regulations governing the award of the Degree of Master of Science (Geotechnics) of Ahmadu Bello University as is approved for its contribution to knowledge and literary presentation.



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DECLARATION

I hereby declare that this thesis has been conducted, written and recorded solely by me under the guidance of DR. K.J. OSINUBI. It has not been accepted in any previous work for a higher degree.

All sources of information, which are not my original work, are specifically acknowledged by means of references.



EDALERE, A.I.

AUGUST, 1999.

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DEDICATION

This thesis is dedicated to:

My Wife	-	*Mrs V. O. Edalere*
My Daughter	-	Miss Oluwaseun O. Edalere
My Son	-	Master Oluwasegun E. Edalere
My Son	-	Master Oluwasola S. Edalere

AND

My parents, Mr. and Mrs. L. Edalere.

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ABBREVIATION

CBR	-	California Bearing Ratio
LL	-	Liquid Limit
MDD	-	Maximum Dry Density
OMC	-	Optimum Moisture Content
PI	-	Plasticity Index
PL	-	Plastic Limit
XRD	-	X-Ray Diffraction
Z	-	Depth

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CHAPTER ONE

1.1 GENERAL INTRODUCTION

Railway track consists of rails, sleepers, and the ballast. It provides a continuous elastic support for static and dynamic loads and distributes them to the ballast and subgrade below (Gulpta, 1981).

The subgrade provides a smooth platform for laying the track or super- structure and distributes pressure from the track with diminished unit pressure to the sub soil. Due to the passage of wheels of the train over the track, each element of the track is subjected to the deflecting and bending stresses repeatedly. The stresses generated by both the ballast and the subgrade vary down the vertical height of the formation (Maslov,1987).The concept of transmission of pressure through the track is based on the principle of elastic theory which states that the super structure and its support, deflect slightly under load but regain their initial position as soon as the load is removed.

Ballast maintenance is the means by which the deterioration of track geometry is controlled, irrespective of the driving forces behind the geometry changes. Whether the structure deficiency is in the ballast, the subgrade or the track super structure or even if the track deterioration has been caused by an over-loading of normal traffic-carrying capacity of the train, correction is usually affected by reworking the ballast, especially in Nigeria where railroad construction is labour intensive. But, reworking the ballast changes its physical state and makes it prone to increased deformation. In the light of this, a physical model was constructed in form of an embankment and the effect of the stress distribution on the local ballast (Quarry Granite and Natural Gravel) with the subgrade was analysed.

ABU

1.2 HISTORICAL BACKGROUND OF NIGERIAN RAILWAY

The Nigerian Railway was established on the 1st of October, 1955. But, before this date, the Corporation existed as a department of the Federal Government of Nigeria. It is the sixth largest railway in Africa and is owned by the Federal Government, (Nichael, 1964).

The railway performs various functions among which are: the technically competent transportation service in pursuit of the country's socio-economic development. It also provides a dependable all - year low cost mass transit service including the transportation of heavy as well as special equipment/machinery and some other materials necessary for the construction of important projects in the country.

1.3 PROBLEMS ASSOCIATED WITH RAIL ROAD AND SPECIAL QUALITIES OF METHODS ADOPTED TO SOLVE THEM

1.3.1 **FORMATION:** This is the track bed and consists of series of embankments and cuttings and has to stand up to all the loads coming on the track. A suitable width and stable slopes have to be prepared which depend on the gauge, width of ballast and if in a cutting, the width of drains on either side (Nichael, 1964).

During construction, therefore, some cuttings may intercept springs water bearing strata, seams of clay and so on. After a few years, water and clay are found to be rising under the formation bed keeping it continually saturated with water. Such a cutting is then called a 'wet cutting' or soft cutting, thus, ballast sinks in the plastic mass of wet clay. The mud works upward and outward due to the pumping action of the rise and fall of sleepers.

Such formation is then unsuitable and unstable for traffic. Digging in and putting in fresh ballast does not help as the formation turns into pond where the water drains and softens the formation still further. Treatment of soft cutting, therefore, is by dismantling the track and the ballast removed and the wet clay between and under the sleepers is dug out to a depth where reasonable formation can be formed to sustain the weight of traffic. The formation is then dressed to a new slope of 1 in 1.5. The area is then filled with laterite to the sleeper level, this filling is done in layers of 25mm each and each layer is well compacted with a wooden rammer. Sleepers are laid to distribute the weight of the traffic over a greater area. Other problems in the formation are slips in cuttings and soft banks which are treated in the same way as soft cuttings in formation.

1.3.2 **DETERIORATION OF BALLAST:** This is caused by

(i) abrasion, that is, mechanical wear of the ballast due to traffic and packing,

(ii) clay working up into ballast from the formation,

(iii) dirt, ashes, working into the ballast from above,

(iv) ballast sinking into the formation (Nichael, 1964).

This could be solved by a special method called re-ballasting. In this case, all the existing ballast is removed and screened and the quantity of additional ballast which might be required is put in.

1.3.2 **CURVES:** Curved tracks are subjected to lateral forces and therefore do not retain correct alignment. Periodical realignment is, therefore, necessary. This also provides an opportunity to introduce proper transitions at the two ends of the main curve. Curve beacons if correct will help the resolution of the curve. Where curves are too sharp, a complete fresh alignment to the curve may be provided to flatten the curve.

1.4.1 TYPE, DESCRIPTION AND FUNCTION OF BALLAST AND SUBGRADE

Ballast are broken stones, solid or any other suitable material placed under and around the sleepers for distributing the load from sleepers to the formation other types of ballast used in Nigerian rail-roads are earth, ashes, sands, Natural Gravel, laterite, but the most predominant one is Quarry Granite (Nichael, 1964). The stress absorbed by the ballast differ from each other due to physical configuration and characteristics.

Quarry Granite: The solid rock generally used for making broken stone ballast consists of Quarry Granite. It is hard, durable and non-porous. Broken stone ballast is the best because it provides a good drainage medium. It prevents rain from forming pools of water on the road bed and destroying it. It does not interfere with the water which is running off the surface below the track because it is angular in shape.

It gives a firm support to the sleepers and by friction resists their tendency to move out of line. It forms a convenient packing medium, retards the growth of vegetation, does not get pulverized easily, does not get flaked when broken and forms the least dust which would have injured the journal of the vehicles when carefully dressed. It prevents a very neat standard ballast on the Nigerian Railway (Nichael, 1964).

The size of ballast used varies between 37mm to 50mm. Stones of larger sizes are not desirable as they do not interlock each other well.

The depth of ballast is an important factor in the bearing capacity of the track The minimum depth of ballast (below top of sleeper at the rail seat) prescribed by (Nichael, 1964) is 150 mm.

The width of ballast is very important because the lateral strength of the track depends very much on the quantity of ballast on the sleeper ends but there is a limit beyond which no useful purpose is served. The minimum width of ballast at level of foot of rail on the Nigerian railway is 1.152m. On curves of over 2° an additional 150mm ballast layer is allowed at the end of sleepers on the outer rail side.

Natural Gravel: Consists of round stones of various sizes. It is usually mixed with sand and is found in banks of rivers or stream beds. It is not a good ballast because the smooth stones roll down easily due to vibration and packing under the sleepers becomes loose.

1.4.2 Description Of Ballast

DESCRIPTION OF BALLAST BED: The bed is trapezoidal in configuration and consists of ballast that is, broken stones which is used around and under the sleepers and distributes the load from the sleepers to the formation level. Ballast bed consists of the following parts: *(see Fig. 1.1)*

- **CRIB:** This is that part of the ballast which lies between two adjacent sleepers.
- **SHOULDER OF BALLAST:** This is that part of the ballast which lies between the rail and the toe of the ballast.
- **TOE OF THE BALLAST:** This is that part of the ballast which rests or lies on the road bed and forms the slope of the ballast.
- **BOXING THE BALLAST:** This term means the dressing of the ballast to the standard ballast section.
- **SLOPE OF BALLAST:** This is the slope at which the ballast is dressed to true parallel lines on each side of the track with a slope of 1 to 1.
- **WIDTH OF BALLAST:** The maximum width of ballast specified in Nigerian railway is 1.152m.
- **DEPTH OF BALLAST:** This plays an immense role in the load bearing capacity of the track. The minimum depth is 150 mm.

- **SIZE OF BALLAST:** The size of ballast varies, but in Nigerian railways, 37 to 50mm is permissible.

Traditionally therefore, angular crushed hard stones and rocks, uniformly graded to drain freely, free from dust and dirt and not prone to cementing action have been considered good ballast materials. However, availability and economic considerations have often been the prime factors considered in the selection of ballast materials in some parts of the world. Thus, a wide variety of materials - such as crushed quarry granite, basalt, limestone, slag and Natural Gravel have been used in other parts of the world as ballast. But, predominantly, Quarry Granite *(crushed)* is used in Nigeria.

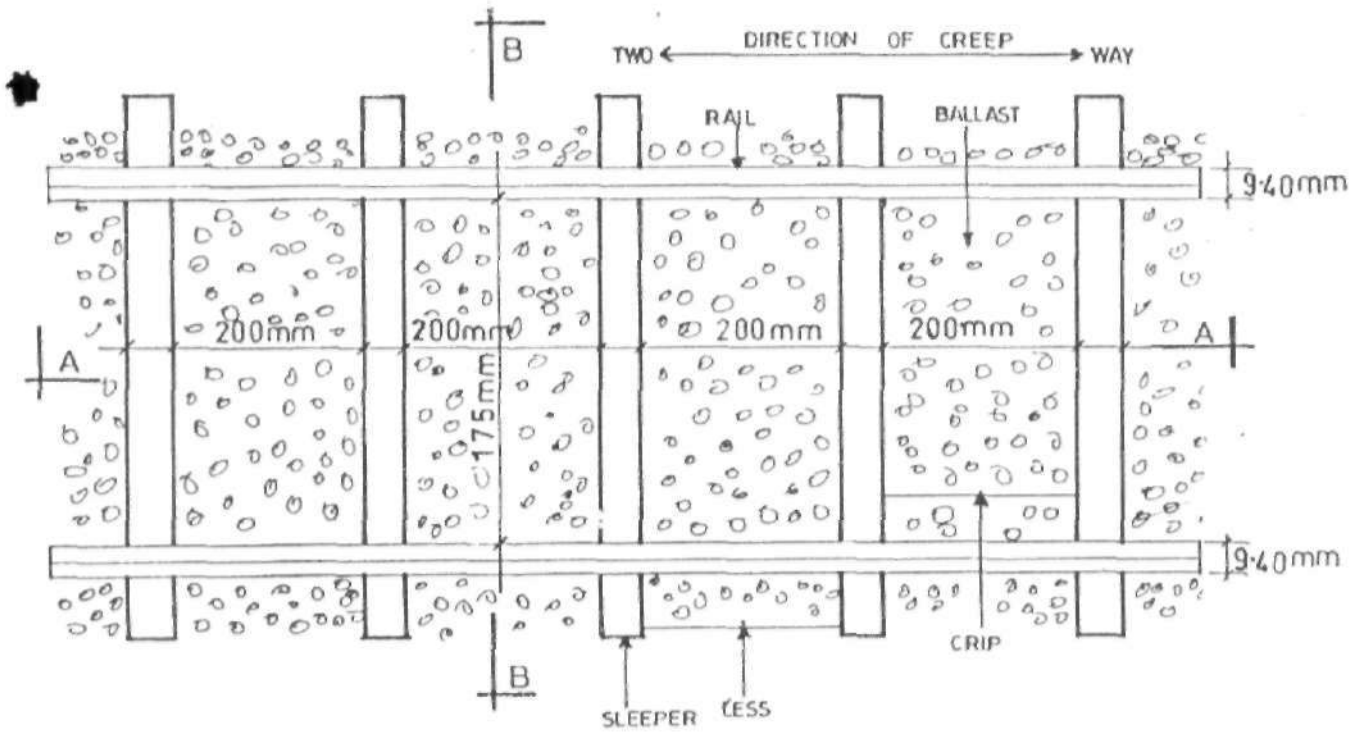


FIG 3·1 TRACK LAYOUT

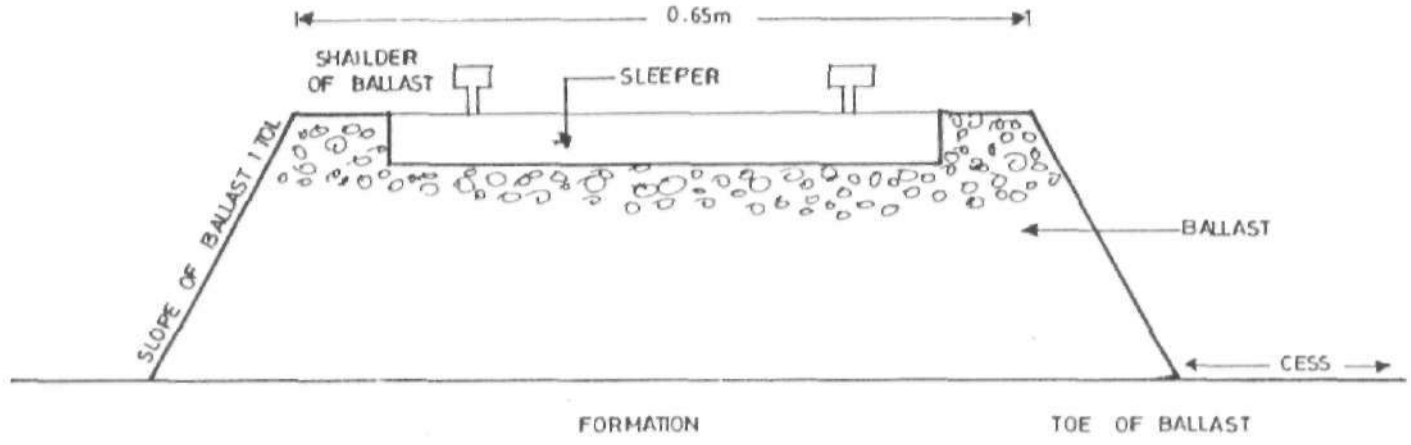


FIG 3·2 SECTION B - B

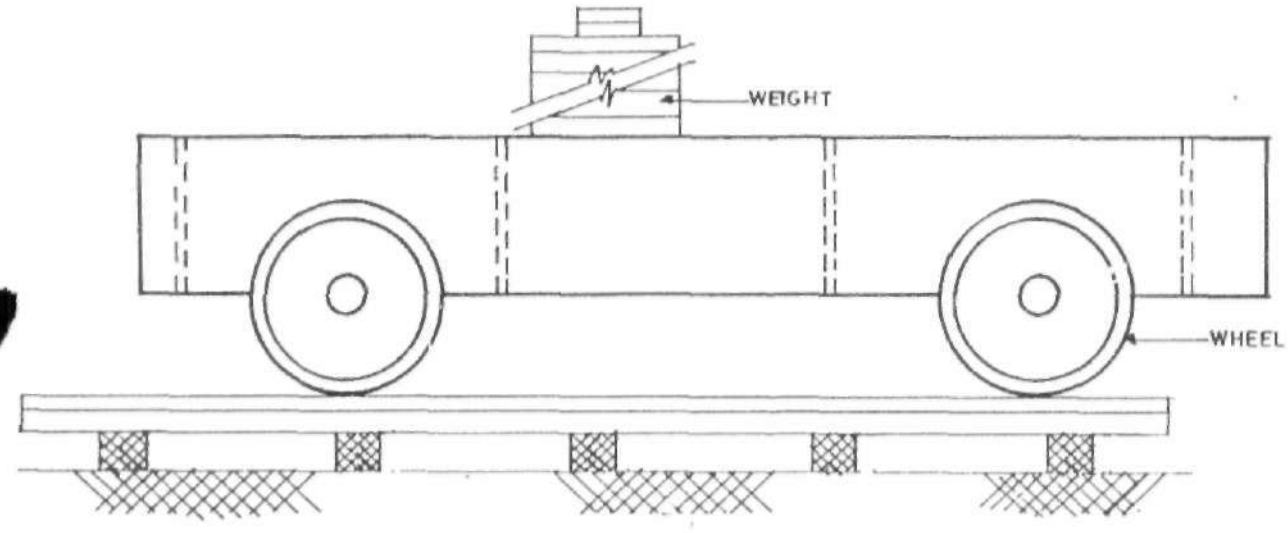


FIG 3·3 SECTION A - A

1.4.3 Functions Of Ballast Bed

The following are the functions of ballast bed in a track geometry (Maduekwe, 1983):

1. *Provision of a resilient but firm and level foundation for the sleepers to rest on.*
For a stiff and rigid sleeper, the ballast must provide an unyielding support. This it does through its high compressive strength which prevents crushing of the stone as the train moves on it. Also, irregular, angular material ensures a high frictional resistance. Hence, a good ballast should be able to provide a firm and level foundation for the super structure on it.

Therefore, the various factors that make up the gauge modulus such as:

(a) ***type of the track***

(b) ***type and spacing of sleepers***

(c) ***type of fastenings***

(d) ***type and section of ballast and***

(e) ***type of sub-grade***

should make their contribution so that the cummulative result is favourable for the satisfactory performance of the sleeper.

The ballast bed should not only develop sufficient compactness after being tamped but also retain it for a long time.

Ballast provides a means whereby track levels, which get disturbed, can be readily restored to their original condition.

2. *Ballast hold the sleeper in position:* The bottom surface of the sleeper should be pierced by the corners and edges of the ballast so that the sleeper is held in a stable position.

3. *Filling the inequalities in the formation*: The formation level is undulating in its surface even without load on it. The amount of settlement on the formation depends on the nature of the material, extent of compaction, moisture content, etc. and the thickness of sleepers and height of rails also vary slightly. The ballast guide against all these inequalities.

4. *The ballast supports the tracks and distributes the pressure on the formation:* Differential settlement sets in if the rail sleeper assembly is placed directly on the laterite formation since, the laterite alone will not be able to bear the load of the moving vehicles. Hence, the aim of the ballast is to distribute the super imposed loads uniformly within its load bearing capacity. Bearing capacity is maximized by good drainage which the ballast provides.

   The maximum pressure on the formation can be reduced by increasing the depth of ballast under the bottom of sleepers, the bearing area of the sleepers on the ballast, the number of sleepers and the stiffness of the rail and by ensuring that the wheel load is distributed by the rail to a large number of sleepers. This distribution is possible if the sleepers and ballast bed underneath are sufficiently elastic to allow the stiffness of the rail to transmit to the neighbouring sleepers a portion of the load.

5. *Provision of efficient drainage*: There is an adage that says "Water is the greatest enemy of a permanent way man (track man)". So, drainage is of very great importance for the retention of the strength and elasticity of the rail road. Thus, ballast must be sufficiently hard, to prevent it from serious pondering due to attriction and abrasion effect. If not, ballast dust gets mixed with the soil that works up from the formation through the interstices and clogs the voids, thus preventing water from draining through freely.

6. *Ballast protects the sleepers from capillary moisture in the formation*: Ballast acts as a separating medium which protects the sleepers from the effects of moisture rising up to the top of formation due to capillary action.

7. *Ballast contributes to the elasticity of the track:* Ballast bed possesses high elasticity which not only produces smooth running of trains but also increases the life of the track components and the rolling stock by providing a cushion against shocks and damping vibrations.

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8. *Ballast provides resistance to lateral displacement of the track:* A track with insufficient or unsuitable ballast goes out of alignment much more quickly than a track which is well ballasted with good material. In the latter, the functional resistance of the ballast under the sleeper and between the sleepers, and shoulder ballast keep the tracks in good alignment. The better the packing of the sleeper the greater the functional resistance there is to relative movement between the sleeper and the ballast.

The coefficient of friction between ballast and steel is very low, especially under vibration and therefore, the shape of the steel through sleeper has been designed to increase the resistance of lateral movement.
This quality is of supreme importance for the preservation of the lateral stability of the track not only on the straight but also on curves. Owing to the digging on of the ballast into the bottom and sides of the sleeper, frictional resistance to movement between the sleeper and ballast develops. In addition to the frictional

resistance, there is also the effect of shoulder ballast which tends to prevent lateral movement of the track.

The area of the surface of the concrete sleeper, consisting of two blocks and a tie, in contact with ballast which is effective in resisting lateral movement is more than in a one piece prestressed concrete sleeper (the dimensions being the same) and is likely to develop a high resistance to lateral deformation of track.

Conversely, ballast resistance to longitudinal movement is less with the former. Under static lateral load shoulder ballast is not of much significance, when a train passes over a track, the rails and sleepers begin to vibrate and the vibrations flicker at the ends of the sleeper; shoulder ballast merely resists lateral slippage of the sleeper.

9. *Resistance to longitudinal displacement of track:* Rails tend to move longitudinally on account of variations of temperature and adhesion between the wheel and the rail, especially when a train is accelerating or decelerating. If the fastening between the rail and the sleeper is not firm, the rail tends to move. On the other hand, if the fastening is firm, this tendency is countered; at the rail end sleeper will tend to move as an assembly. In this case, the friction between the sleepers and ballast and frictional resistance of the (crib) ballast between the sleepers tend to oppose the movement.

10. *Resistance to a good, firm and level of the track*: By providing a good, firm and level foundation, ballast offers resistance to downward movement of the sleeper. If a wooden sleeper rests on earth formation without ballast, a force equal to the weight of the sleeper will be required to raise it, but if the formation is well ballasted, the force required to lift it is increased by one-third on account of the frictional resistance between the ballast and the sleeper (Maduekwe, 1983).

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11. *Protection of the formation in general*: The ballast protects the formation from direct exposure to the sun in hot countries, leading to its drying up and shrinking with consequent cracking, and from direct exposure to rain during heavy rains which would lead to its softening. It also protects the formation from frost in cold countries; clay formation is specially susceptible to these influences.

1.4.4 Function Of Subgrade

Subgrade is the layer of material on which the ballast and sub-ballast layers rest. The subgrade is a very important component in the track structure and has frequently been the cause of track failure and the development of poor track. It has the following functions and requirements:

i. it must support, without appreciable permanent deformation, the maximum dynamic, traffic-induced stresses transmitted through the ballast.

ii. it must resist the cyclic stresses without excessive cummulative volume or strength reductions.

iii. it must be volumetrically stable during cycles of wetting and drying.

iii. it must resist softening that could cause pumping and penetration into the ballast.

1.5 REVIEW OF EXISTING LITERATURE ON STRESS DISTRIBUTION

Talbot [1940] contributed immensely to the understanding of the behaviour of a railway track system with vehicle loadings. He suggested that the tie loads by ballast section do not proceed vertically downward but arise from the frictional

interlocking of the ballast particles. He proposed a model whereby intergranular distribution of ballast occurs at successive depth layers. The pressures at a depth of n layers can be represented by an expansion of the binomial $(\frac{1}{2} + \frac{1}{4})^n$, which closely resembles values of Gaussian probability curve

$$P_x = \frac{K P_a e^{-K^2 x^2}}{\pi^{\frac{1}{2}}}$$

where P_x = Pressure in the ballast at any distance x from the centre line of the tie, P_a = uniform bearing pressure over tie face; K = a factor to be determined by experiment.

These relations were further illustrated by a bell-shaped pressure intensity-curve, where pressure intensities on horizontal planes at depth h_1, h_2 and h_3 were plotted as ordinates for various values of x for a load distributed uniformly over a centimeter strip. The pressure intensity decreases as the depth increases and the area under the bell-shaped pressure intensity curves remain equal to the applied load.

Clarke [1957] summarized Talbot's approach above to represent a basis for track design procedures. However, this theory does not include several important factors that are known to affect the stresses and deflections in railroad track, such as longitudinal loads from thermal stresses, a restoring moment proportional to the rotation of the rail and ties, the eccentricity of the vertical load on the rail head, or any tract-dynamic effects. In addition, a rather significant limitation to the approach is that it does not adequately model the stress-strain behaviour of the ballast and subgrade.

Meacham and Prause [1968] attempted to overcome some of the limitations of the earlier beam-on -elastic foundation approach by assuming that

(1) all layers of material have the same elastic properties that is homogenous

(2) each layer has a finite thickness except for the lower layer and all infinite in the lateral directions

(3) the time dependency of strain under a constant stress level could be viscous or non-viscous

(4) the degree to which the material rebounds or recovers from strain after the removal of applied stress could be plastic or elastic

(5) the relationship between theoretical method for determination of track modulus value.

In this method, each component of the track structure is represented by a series of elastic springs, the springs stiffnesses are computed considering various track parameters (such as rail type, tie-type, ballast depth, ballast type, subgrade type and tie spacing).

The finite- beam- on- elastic foundation approach is basically similar to the above theories except that it considers the tie as a finite beam resting on an elastic (Winkler type) foundation as the representation of the response of a tie resting on the ballast. The finite beam on elastic foundation has been extensively studied by **Hetenyi [1946]** and various methods for the solution have been presented. For example, **Barden [1963]** considered a non-uniform foundation modulus in his own analysis. Later, an approximate analytical method was developed that makes assumptions about the distribution of wheel load over the rail and across ties.

The vertical stress distribution with depth in the ballast and subgrade layers under any given tie is then computed by using the Boussinesq theory.

Finite element methods have also been used for track structure analysis by various authors such as **Winkler [1867], Hoege [1968], Kandanrov et al [1969], Schiffman et al [1970], Harrison [1972], Desai and Abel [1972]. Winkler [1867],** concentrated on the behaviour of the upper stiff layer, which he considered to be the structure and relegated to the soil only a supporting role.

A model was presented by Winkler, where the deflection of the ground at a point, W, was assumed to depend only on the pressure P, acting at that point through a proportionality constant, K_o, so that $P = K_oW$.

The constant, K_o has been given a variety of names. It is the foundation stiffness but it has been called the coefficient of subgrade reaction or subgrade modulus. Its dimensions are FL^{-3}. Winkler's assumption that the deflection of the ground at a point, W, depends only on the pressure P, acting at that point through a proportionality constant, K_o, was first used to analyze the stresses and deflection of rail road tracks.

Salem [1966] studied the vertical stress distributions in ballast and subgrade under statically loaded wooden ties in a series of ballast depths tie spacings and type of ballast. Tests indicated that the depth of ballast needed to obtain a fairly uniform pressure on the subgrade equal to the spacings minus 7.6 cm

Tayabji and Thompson [1976] used another finite-element model to analyze the effect and distribution of stresses in ballast, sub-ballast and subgrade material. The model consisted of a three-dimensional model, one transverse and the other longitudinal and used the output from the longitudinal model as the input to the transverse model. This gives a three dimensional effect at less computer cost than a three dimensional model. Non-linear mechanical properties for the materials were obtained in the laboratory from repeated load technique which was used to effect a solution. However, the model did not prevent tension from being transferred across the rail base, into the plate. The three-dimensional qualities of track structure must be fully accounted for if the behaviour of track system is to be successfully predicted by using finite element models.

Boussinesq [1885] postulated the theory on stress distribution based on the results given by the mathematical theory of elasticity for the simplest case of loading of a solid homogeneous, elastic-isotropic semi-infinite medium; namely

the case of a single vertical point load on horizontal boundary surface. The assumptions made are that:

i. The soil medium is elastic, homogeneous, isotropic, semi-infinitely in all directions from a level surface and which obeys Hooke's law.

ii. The soil is weightless.

iii. Originally, before the application of the single concentrated load, the soil is not subjected to any other stress, that is, the soil is stress-less or unstressed.

iv. The stress distribution from the applied concentrated load is independent of the type of material of which the homogenous, elastic-isotropic body is made. Relative to the soil the change in volume upon the application of the stress to the soil is neglected.

v. In the medium, the law of linear stress distribution is valid.

vi. There exists a continuity of stress.

vii. In such a system the stresses are distributed symmetrically with respect to the Z-axis.

The limitations of the theory based on the above assumptions restrict it to the proportionality between stress and deformation. In Boussinesq theory the vertical, radial and shear stress at a depth Z and a horizontal distance r from the point of application of the load were given.

He gave the formula as:

$$\sigma = \frac{KP}{Z^2}$$

where = stress generated kN/m^2

K = is the influence factor

P = is the load in kN

Z = is the depth (m).

1.5.1 Plastic Deformation And Residual Stresses

In rail-road, track mechanics and technology, because of the high stress levels that accompany the contact of counter formal bodies, it is likely that plastic flow will occur sometimes in the history of the track loading. When external loads are removed from a body that has undergone plastic flow, a certain amount of permanent strain (or set) remains along with an associated self balanced set of residual or locked-in stresses. When a load is reapplied to a body with residual stresses, further plastic flow may or may not occur depending upon the history of the loading and the strain hardening characteristic of the material **[Arnold, 1975].**

Hodge [1959] reported that it is possible that loading state would not cause a massive failure if applied once but can cause a monotonic increment in deformation in each cycle of loading and lead to a so called incremental collapse. This is the model of failure which results in the gross plastic deformation which is often observed at the edge of a rail.

1.6 SIGNIFICANCE OF RESEARCH

The evaluation of track stresses is an important step in the selection and design of rail sections which results in the judicious deduction of the following essential factors:

i. The type of motive power (example steam, electric or diesel) axle load, wheel spacing and speed factors which influence the design of a rail section as a whole.

ii. The intensity of traffic and the physical characteristics of the line, including the

sub-soil conditions in different parts of the country at different times of the year. These factors influence the service life of the rail in the track **[Soboyejo, 1967].**

Serious derailments to passenger trains are fortunately comparatively rare occurrences, but when they occur, they could be very serious. Thus, the defect in the track, especially, formation and the train contribute to the cause of derailment in a track system **[Nichael, 1964].** Thus, when the stresses in the ballast and subgrades are known at any particular time of the year, the load intensity of the train can be controlled, in order to guide against derailment that can cause loss of life.

Prevention of 'Wash-out' of the track during the rainy season cannot be overlooked, which results in the total removal of the track if the formation is not well compacted and the grade is not strong enough to withstand the traffic during the 'wet' season **[Nichael, 1964].**

Lastly, comfortability of the passengers as the train moves on the track is very important because of the cushion effect provided by ballast as trains move over it.

1.7 AIM OF RESEARCH

The main aim of the research was to investigate the stresses in the track system under static loading.

1.8 OBJECTIVE

In order to achieve the above aim, the following objectives were to be met:

i. To know the stresses in two different ballasts commonly used in Nigeria namely, Quarry Granite and Natural Gravel and the subgrade laterite.

ii. To determine the best fit model for the stresses in the track system.

1.9 SCOPE

The research involved the preliminary experimental determination of the characteristic of the two types of ballasts namely: sieve analysis, specific gravity, bulk density, crushing value test, impact value test, flakiness and elongation test.

The laterite soil sample which acted as the subgrade was equally tested for the purpose of identification and classification.

The tests conducted include specific gravity test, plastic limit and liquid limit tests, compaction characteristics, sieve analysis and California bearing ratio test. The stresses generated as the train is statically on the track were measured at optimum moisture content.

1.10 LIMITATION

i. The type of ballast used was restricted to Natural Gravel and Quarry Granite since they are commonly used in Nigeria.

ii. Static loading was used because the modelled train's speed could not be readily determined, most especially, when it was loaded.

iii. Timber sleepers were used because they are commonly used in the field.

CHAPTER TWO

2.0 METHODOLOGY, EXPERIMENTS AND RESULTS

2.1 BALLAST: QUARRY GRANITE AND NATURAL GRAVEL

Various experiments including sieve analysis, crushing value test, flakiness index, elongation test, specific gravity, bulk density and impact were carried out in the laboratory to find out peculiar characteristics of the ballast type.
Natural Gravel are obtained from natural sources like pit, rivers and ocean beds where they have been deposited by natural phenomena. They consist of flints, quartz and igneous rock. It has smooth edges which can be rolled down easily.

In addition to the petrological character of natural gravel its external characteristics are of importance in particular, the particle shape and surface texture. The shape is three dimensional and is difficult to describe and it is, therefore, convenient to define certain geometrical characteristics of such bodies.

Roundness measures the relative shapeness or angularity of the edges and corners of a particle. Roundness is controlled largely by the strength and abrasion resistance of the parent rock. Thus natural gravel has smooth edges which can be rolled down easily by vibration and the packing under the sleepers.

Quarry granite is usually obtained from rock beds either by blasting or by manual means when large quantity is not required. It is hard, durable and non-porous stone. It is white and dark in colour and presents neat appearance. The size ranges between 12 mm and 50 mm.

The Quarry Granite provides a good drainage medium because it prevents rain from forming pools of water on the bed. It has sharp corners and edges and hence forms a convenient packing medium. **[Nichael, 1964].**

2.1.2 Specific Gravity

Specific gravity, according to BS 1377 (1990) test No.6A is defined as the ratio of the weight in air of the aggregate to the weight of an equal volume of distilled water.

It was carried out in an air dried sample placed in a 50ml density bottle with a stopper and weighed (M_2), water was then topped in the bottle and weighed (M_3). The bottle was emptied and washed clean and its weight full of water recorded (M_4). The weight of the empty density bottle being (M_1), the specific gravity was expressed as:

$$G_s = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)}$$

Where M_1, is the mass of density bottle, M_2 is the bottle and dry soil, M_3 is the mass of bottle, soil and water, M_4 is the mass of bottle when fully of water only. The result obtained was 2.60.

2.1.3 Bulk Density

This was carried out according to B.S. 812 (1960) Test No. 25 and is defined as the ratio of the mass of aggregate required to fill container to the volume of the container. Each cylinder was first calibrated to determine its actual volume: the weight of water in kg required to fill the cylinder with no meniscus above the rim this gave the volume in litres. The appropriate dried cylinder was filled to overflowing with the surface - dry aggregate by means of a scoop. The aggregate being discharged from a height of not more than 5cm above the top of the cylinder. The surface was levelled with a straight edge and the net weight of aggregation in the cylinder determined.

$$\rho = \frac{M_g}{V_g}$$

Where M_g = Mass of aggregate, V_g = Volume of cylinder

The results obtained was 0.24kg/cc for quarry granite and 0.33kg/cc for natural gravel.

2.1.4 Flakiness Index

This was carried out according to B.S. 812 (1960) Test No. 16 by sieving a sufficient but known quantity of coarse aggregate to provide a minimum of 200 pieces on each of the above sieves.

The flaky material of each fraction were separated by "placing" on the thickness gauge and determining the total weight in the appropriate slot of metre. The total amount passing should be weighed to an accuracy of at least 0.1 percent of the weight of test sample. The test was repeated, using the length gauge. Flakiness index is calculated as the total weight of material passing the various thickness gauges expressed as a percentage of the total weight of the sample gauged to the nearest whole number.

$$\text{Flakiness Index} = \frac{\text{Weight of material passing}}{\text{Total weight of material gauged}} \times 100$$

The results obtained were in the range of 10.26% to 16.20% for quarry granite and 5% to 62% for natural gravel. This should not be less than 10% (Arnold, 1975) hence, quarry granite is very good as a ballast material.

2.1.5 Elongation Index

This was carried out in accordance to B.S. 812 (1960) Test No. 17 and it is defined as the particle whose length is more than 1.8 times the mean sieve size of the sieve fraction to which the particles belong. The elongation index was carried out in a similar manner as flakiness index and was determined by expressing the total weight of material passing the percentage of the total weight of the sample gauged to the nearest whole number.

Elongation Index = Weight of aggregate passing x100

Total weight of sample gauged

The results obtained were in the range of 12% to 13.5% for quarry ballast, 27% to 47% for natural gravel. The result should not be greater than 10% (Arnold, 1975). Quarry ballast is very good as ballast.

2.1.6 Sieve Analysis

This was carried out in accordance to B.S.812 (1960) Test No. 12 by taking the coarse aggregate approximately 3 kg. With sieve number 75 mm, 63 mm, 50 mm, 37.5 mm. 28 mm, 20 mm, 14 mm, 10 mm, 6.3 mm, 5 mm, 3.35 mm, 2 mm, 1.18 mm, 60 m, 30 0 m, 212 m, 150 m, 63 m respectively were used.

The aggregate were air dry and sieves dried and cleaned. The samples were sieved ensuring that the samples were not forced through the sieves by hand pressure. A table was prepared and grading curves on the chart was drawn. The two ballast were well graded.

2.1.7 Crushing Value Test

This was carried out in accordance to B.S. 812 (1960) Test No. 30 by using the aggregate passing the 12.7mm and retained on the 9.5 B.S. sieve. About 5 kg were used for the test, cleaned and surface dry.

The aggregate was placed in the cylinder on the base plate and filled with the aggregate in three layers giving 27 blows to each layer. The top was levelled with tamping rod and insert plunger so that it rests horizontally on the surface of the aggregate.

The test sample was then put in the compression machine loading at 40kN/min to a load of 400kN for 10 minutes. The sample was then removed from the cylinder and sieved on 2.40 mm B.S. test sieve.

The weight of the fines passing the 2.40 mm sieve was then expressed as a percentage of the total weight of the aggregate used; and this is thus the crushing value. Two tests were carried out and the mean found.
The results obtained were quarry granite 23.2% and natural gravel 37%. The result should not be greater than 25% (Arnold, 1978). Quarry granite is better in this respect than natural gravel.

2.1.8 Impact Value Test

The impact value test gives a relative measure of the resistance of an aggregate to sudden shock or impact of an aggregate to sudden shock or impact which in some aggregates differs from its resistance to a slow compressive load B.S. 812 (1960) Test No.29.

Samples of the two aggregates were prepared which passed through sieve number 12.7, 1.20mm and retained in sieve number 9.52. Sufficient aggregates were sieved and it was ensured that the samples were cleaned and the surface in dry condition.

Cylinders of 7.6 cm in diameter was filled in three layers respectively for the two samples and were given twenty-five strokes of the 22.9 cm metal tamping rod to each layers. The top of the cylinder in each case were levelled off and weighed. The samples were placed in the cup, fixed firmly in position on the base of the impact machine and 25 blows of the tamping rod were applied. The hammer on the machine was allowed to fall freely for 15 times that is 15 hammer blows were impacted on the two types of aggregates. The samples were then poured into 2.40mm sieve and the percentage passing by weight was determined.

The results obtained were 24.15% and 29.55% for quarry granite and natural gravel respectively. The result should not be greater than 25% (Arnold, 1975); hence quarry granite is very good as ballast.

2.1.9 Los Angeles Abrasion Test

This was carried out in accordance to B.S. 812 (1960) Test No. 33. The machine used consists of hollow steel cylinder 50.8 cm long and 70 cm inside diameter. The inside of the cylinder carried a steel plate 8.90 cm wide along the full length of the cylinder. Steel balls from 3 to 12 each of 4.75 cm in diameter were used. The steel cylinder revolved at the rate of 30 revolution per minute.

Sample of both ballast ***(Quarry Granite and Natural Gravel)*** were taken of about 1000 g each dried were put in turn into the cylinder along with the required number of steel balls. In each case, 500 revolutions were performed. The aggregates were taken out and screened through sieved 2.4mm separately.

The results obtained were 49.2% and 70.2% for quarry granite and natural gravel respectively. The result should not be greater than 45% (Arnold, 1978); hence quarry granite is very good as ballast.

A summary of test results is given in Table 2.1.

TABLE 2.1 **TEST RESULTS OF BALLAST MATERIAL**

S/NO	*AGGREGATE TESTS*	*Quarry Granite*	*Natural Gravel*
1.	**Gradation**	Well graded	Well graded
2.	**Bulk density**	0.24kg/cc	0.33kg/cc
3.	**Specific gravity**	2.60	2.60
4.	**Flakiness Index**	Ranges between 10.26% to 16.20%	Ranges between 5% to 62%
5.	**Elongation Index**	12% to 13.5%	27% to 47%
6.	**Crushing value**	23.2%	37%
7.	**Impact Value Test**	24.15%	29.55%
8.	**Los Angeles Abrasion Test**	49.2%	70.2%

2.2 LATERITE AS SUBGRADE

2.2.1 Location of Sample:

The laterite used as subgrade was got from Shika area along Zaria - Sokoto road. The geological and soil maps of Nigeria shown in Figure 2.1 and Figure 2.2 respectively, show that the soil belongs to ferruginous and derived from acid igneous and metamorphic rocks **[Areola ,1982], [Akintola, 1982].**



Fig 2.1 Geological Map of Nigeria (after Akintola 1982)

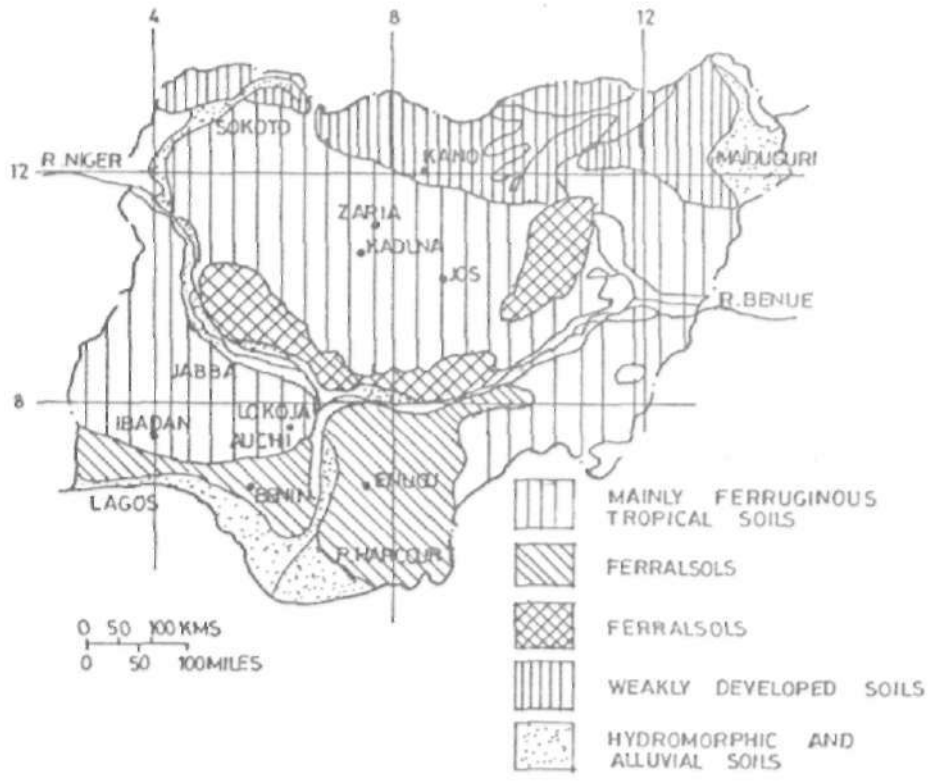


Fig 2.2 Soil Map of Nigeria (after Areola 1982)

2.2.2 Natural Moisture Content

The procedure was in accordance to B.S. 1377 (1990) test (1A). The freshly dug sample was placed in two metal containers of known weight and weighed.

They were then put in a drying oven at a controlled temperature of between 105°c to 110°c and left for 24 hours. The moisture content is expressed as:

$$W = \frac{W_w - W_d}{W_d} \times 100$$

Where W = moisture content (%)

W_c = weight of container (g)

W_d = dry soil (g)

W_w = weight of container and wet soil (g)

The average moisture content is 19.2%

2.2.3 Specific Gravity

The specific gravity of the sample was carried out in accordance with BS 1377 (1990) test 6(B). Air dried sample was put in a 50ml density bottle with a stopper and weighed (M_2), water was then topped in the bottle and weighed (M_3). The bottle was emptied and washed clean and its weight full of water was recorded (M_4). The weight of the empty density bottle being (M_1), the specific gravity was expressed as:

$$G_s = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)}$$

The specific gravity calculated is 2.65

2.2.4 Particle Size Analysis

The particle size analysis was carried out in accordance with BS 1377 (1990) test 2(1). The solution of sodium oxalate was used as a dispersing agent which was prepared by dissolving 50 g of the sodium oxalate in a litre of distilled water, 20 ml of this solution was then used in each determination of the particle size. The cylinder was then removed and the palm placed squarely and agitated for two minutes. It was then placed on the table and the hydrometer inserted into the cylinder and readings of the hydrometer were recorded at 20 secs., 30 secs., 1 min, 2 mins, 5 mins, 10 mins, 1 min 30 mins, 1 hour, 7 hours, 12 hours and 24 hours. The cylinder was completely emptied into the BS No. 200 sieve and the soil washed clean. The clean soil sample was then transferred to a container and dried in the oven.

The dried samples were then passed through a series of sieves with aperture sizes 1.18 mm, 600, 425, 300, 212, 150 and 75 μm. The percentage passing each sieve size was calculated. A graph was plotted consisting of coarse and fine materials which were obtained from the sieving and hydrometer analysis respectively. The particle distribution curve is shown in Fig. 2.3.

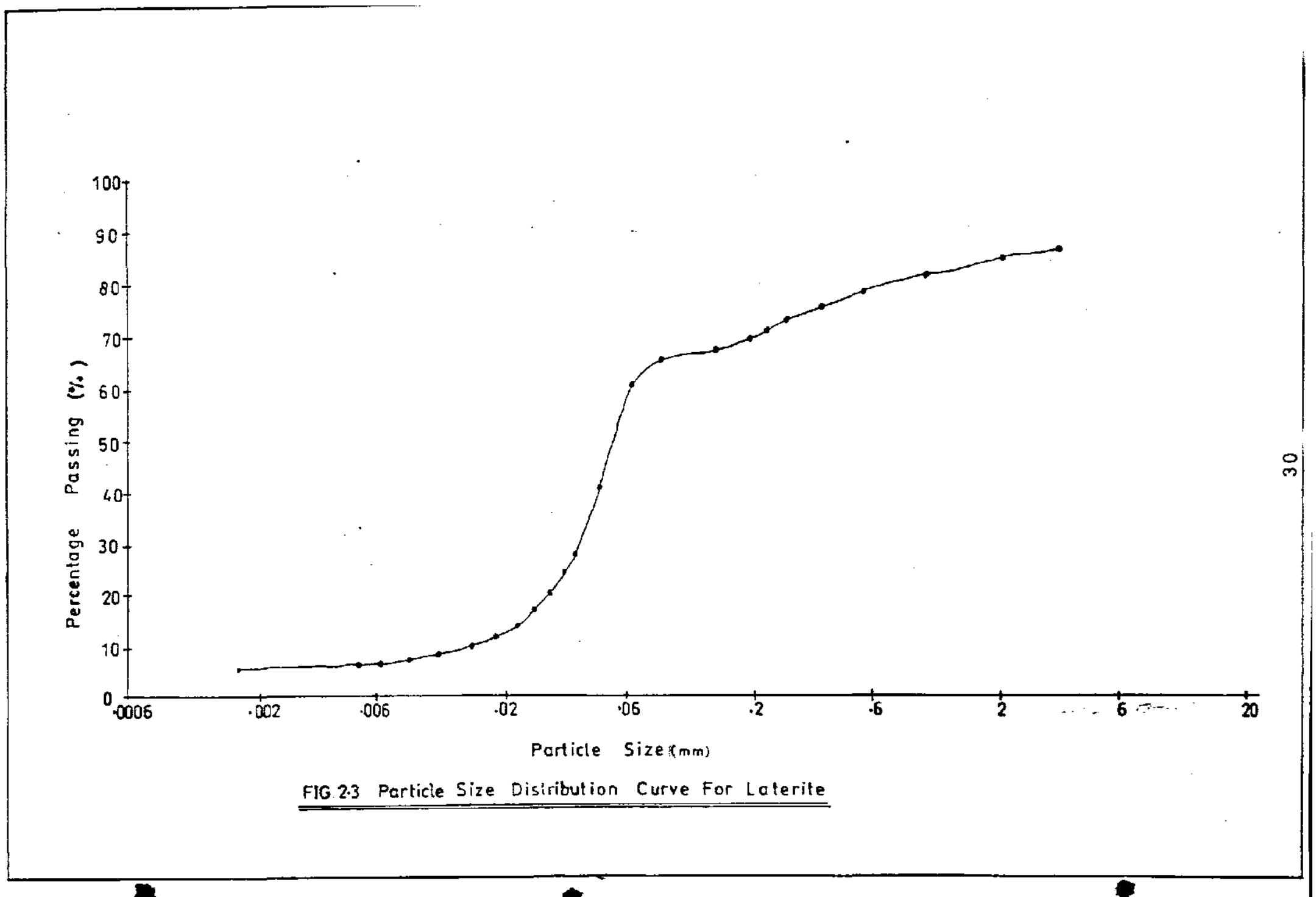


FIG 2·3 Particle Size Distribution Curve For Laterite

2.2.5 Atterberg Limits

The liquid limit test was performed in accordance with BS 1377 (1990) test 1(a) on soil passing through sieve with aperture 425μm. The soil was mixed with water on a flat glass plate and placed in the blows of the case-grade apparatus and pressed from the middle outward to prevent air being tapped. It was then cut with grooving tool into two equal halves, from the hinge of the bowl closed. The number of bowls required to close the groove was recorded and the moisture content determined.

The same procedure was repeated for varying blows such that it was spaced between 10 and 50 and two counts performed on each side of 20.

The results were plotted on a semi-logarithmic chart with moisture content as ordinate and the corresponding number of blows as abscissa. The moisture content at 25 blows was read off and its moisture content value gave the liquid limit of the soil sample or the liquid limit of the soil.

The plastic limit was performed on a little of the soil taken from the bulk used in the liquid limit test. The sample was relatively drier than all subsequent portions used in the liquid limit test. The sample was rolled into balls between the palms and divided into 4 parts; each part was then rolled into threads on the glass plate until threads about 3mm diameter began to crumble. The first crumbling point is the plastic limit. The crumbled threads were placed in a moisture content container and the moisture content determined as explained in section 2.2.3. The value of the moisture content obtained gave the value of the plastic limit.

The plasticity index is the difference between the liquid limit and the plastic limit. That is,

$$PI = LL - PL$$

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The average result of the Atterberg limits are shown in Table 2.2.

TABLE: 2.2 **ATTERBERG LIMIT OF LATERITE SUBGRADE**

SAMPLE	LL (%)	PL (%)	PI (%)
1	36.51	21.74	14.77
2	39.71	22.95	16.76
3	40.88	21.74	19.14

2.2.6 Compaction Test

Soil compaction can be defined as a mechanical soil stabilization process whereby the soil particles are brought closer space of compaction through a reduction in void ratio, reduction in volume and increase in bulk density.

The compaction test was carried out in accordance to B.S. 1377 (1990) Test No. 13 for Proctor method using 2.5 kg rammer falling from standard height of 300 mm.

A sample of air dried soil passing the 20 mm BS test sieve was taken and mixed through with a suitable amount of water. The mould was weighed and moistened soil was compacted into it, in three layers, each layer being given 27 blows from the rammer dropped from 300mm height. The blows were distributed uniformly over the surface of each layer.

The mould and the soil was then weighed (M_2).

The compacted soil sample was taken from the mould and placed on the large metal tray. A representative sample of the specimen was taken and its moisture content, W, was determined. The remainder of the soil specimen was broken up, rubbed through the 20mm BS test sieve and then mixed with the remainder of the original sample. Suitable increments of water was added successively and mixed into the sample and the above procedure repeated for each increment of water added. Five tests were carried out. The density and the bulk density were calculated using the following formulae:

$$\rho_d = \frac{100\rho}{100+w}$$

Where ρ_d = dry density (mg/m^3)

w = moisture content of soil (%)

$$\rho = \frac{M_2 - M_1}{1000}$$

where ρ = bulk density (mg/m^3)

M_1 = the mass of mould and base (g)

M_2 = the mass of mould, base and soil (g).

The maximum dry densities (ρ_d) was obtained by plotting series of calculated dry densities against moisture contents. The result obtained is shown in Table 2.3.

2.2.7 California Bearing Ratio Test

The California Bearing ratio test was carried out in accordance with BS 1377, (1990) test 13 but modified to comply with the Nigerian General Specification (1970).

The moulds were arranged in place and 5 kg of the soil sample was weighed into mixing trays and mixed with the required water thoroughly and at the required OMC. The soil was compacted in 3 layers in the CBR mould with 62 blows

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applied per layer using the 2.5 kg rammer for standard Proctor compaction. The soil specimen in the mould was trimmed off carefully using a straight edge upon removal of the mould extension. The bottom cover of the mould was removed and both faces of the mould were covered with wax. The wax cured samples were left to cure for 6 days in the humidity room at a temperature of 20 ± 2°c and 100% humidity. After 6 days, they were de-waxed and soaked in water for 24 hours.

The California bearing ratio machine was thoroughly cleaned and the mould containing the specimen with the base plate in position was placed on the lower plate. Surcharge rings were placed on the exposed surface and carefully arranged such that the plunger penetrated at a uniform rate of 1.3 mm/min. according to **AASHTO** specification until the specimen failed. Later it was inverted and tested again.

A force-penetration curve was drawn with the values of the force plotted on the ordinate and the penetration on the abscissa. A correction was made where the curve was concave upwards by drawing a target to the curve of the point of greatest slope. The point where this tangent cuts the penetration axis served as the new origin.

The value of the force of penetration of 2.5mm and 5.0mm were read off. The CBR was expressed as:

$$\text{CBR} = \frac{\text{Measured force}}{\text{Standard force}} \times 100\%$$

The results of the CBR is recorded as the mean of the top and bottom results. The results are shown in Table 2.3.

TABLE: 2.3 **INDEX PROPERTIES OF SUBGRADE SOIL**

PROPERTY	QUANTITY
Colour	Reddish Brown
Specific gravity	2.65
Natural moisture content (%)	5.6
Liquid limit, (%)	42
Plastic limit, (%)	22.14
Plasticity index, (%)	19.86
Group index	6
AASHTO Classification	A - 7 - 6
Max. dry density Standard Proctor test (Mg/m^3)	1.72
Optimum moisture content, (%)	19.2
California Bearing Ratio (C.B.R) Proctor Soaked, (%)	4.1
California Bearing Ratio (C.B.R) Proctor unsoaked, (%)	8.0

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CHAPTER THREE

3.1 THE MODELLED TRACK AND TRAIN

The model of railway track embankment and train were used in order to simulate what is obtainable in the field.

3.1.1 Embankment

When the rail level of the track is to be higher than the level of the ground, an embankment is constructed on the ground on which the track is carried.

The embankment is made of wood and trapezoidal in shape with dimension of 1.50 m base length and depth of 0.35 m. It was filled with laterite which acts as a subgrade on which the track was laid. The subgrade was compacted in layers until it was filled to the brim section of the embankment is shown in Figure 3.2.

3.1.2 Track Rails

Two 9.40 mm steel rods 1m in length were placed on the ballast rigidly fixed to move the train. Generally, rail is that part of the track on which the wheels run. This is shown in Figure 3.1.

Rails are used for (1) to provide a means of carrying the rolling stock while offering a hard durable, smooth and unyielding surface so that the resistance due to the moving load will be reduced to the minimum; (2) to direct the course of the vehicle in conjuction with the flanges of the wheels.

3.1.3 Track Gauge

The track gauge is defined as the perpendicular distance between the inside edges of the running rails. The standard gauge used in Nigeria Railway System is 1.152m for the purpose of this project about 175mm gauge was used which is ratio 1:7 of the one obtained on the field.

3.1.4 Sleeper Spacing

The strength of any track is increased by increasing the Sleeper density. Sleeper density is, the number of sleepers in a track length, hence if the sleepers are spaced closely together to increase the sleeper density, then the track strength increases.

In this study, five sleepers were used at a spacing of 200 mm length. A wooden sleeper 0.62 m in length, 0.05 m in width and 0.03m in thickness was used (see Plate1,2,3).

3.1.5 Modelled Train

A 10 kg model metal train was constructed which is similar in design and out look to the 1900 class of the Nigerian Railways locomotive engine with 16.76 tonnes axial load. Thus, the model is at a ratio of 1:1676 of the locomotive engine on the field. The model (see plate 2) is 0.23 m in height and 0.63 m in length. It consists of the following parts.

(i) Engine room (0.32 m x 0.11 m)

(ii) Drivers room (0.21 m x 0.21 m)

(iii) Front Bumper (0.21 m x 0.13 m x 0.05 m)

(iv) Back Bumper (yellow) (0.21 m x 0.13 x 0.05 m)

(v) Guard Rail (0.21 m x 0.07 m)

(vi) Four wheels (0.08 m) diameter and

(vii) Mud guard (red) (0.21 m x 0.08 m).

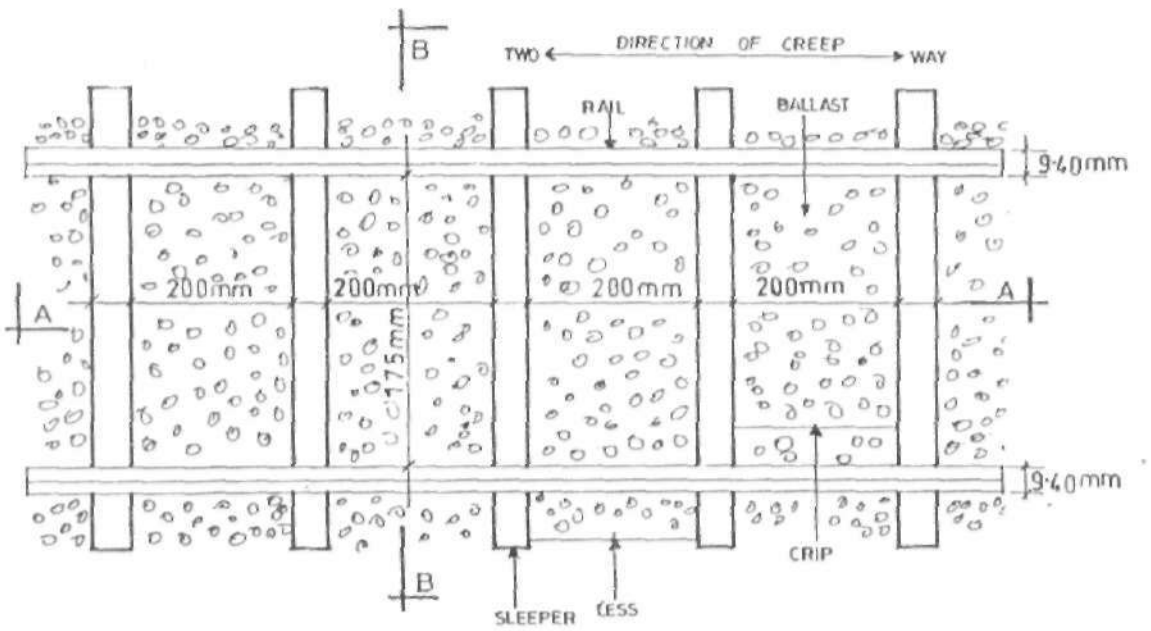


FIG 3·1 TRACK LAYOUT

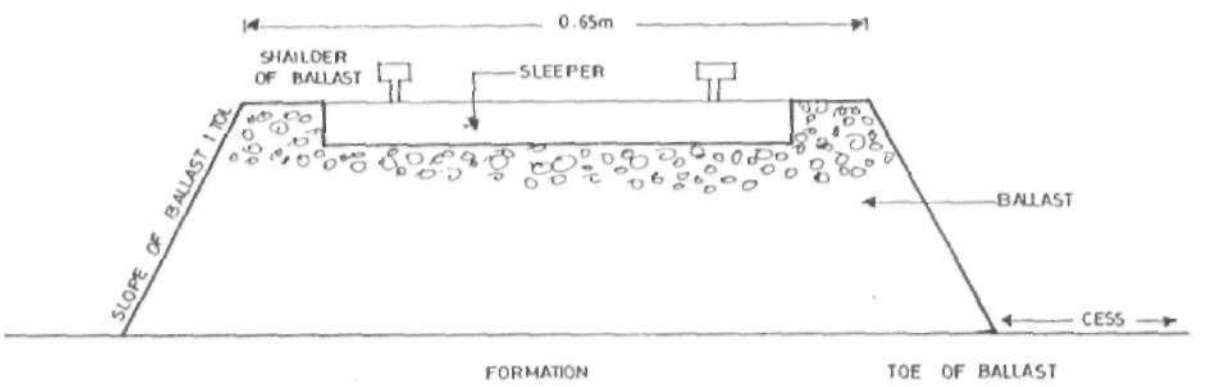


FIG 3·2 SECTION B - B

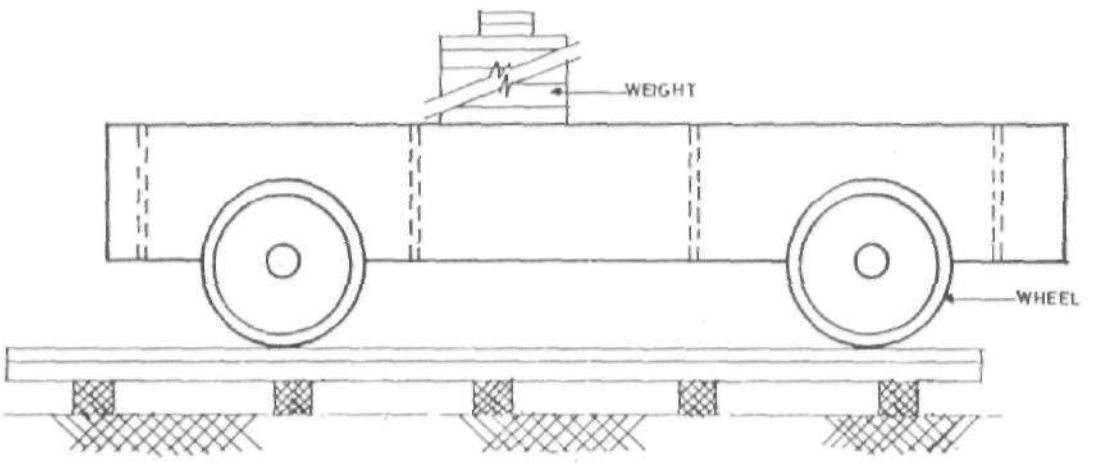


FIG 3·3 SECTION A - A

3.2 STRAIN GAUGE

Strain gauges are resistances that are strained bodily so as to alter their physical cross-section and length. They are often used in stress analysis for the design of safe and reliable products and engineering structures. The techniques of stress analysis may be applied at different stages in the life of a product, from preliminary design concepts to the testing of the finished product and failure analysis of products already in use.

Within the wide field of experimental stress analysis, several techniques are available, including the mechanical extentiometers, optical methods and the electrical resistance strain gauges. One of the three electrical resistance strain gauges was used for the stress analysis in this study. This is because the resistance R of a wire is directly proportional to the length of the wire conductor and inversely proportional to the cross-sectional area. So if we extend a conducting wire of length L, and resistance R, by δL, we should expect a corrreponding change of δR in resistance and hence the change in strain could be obtained by recording variations in change of resistance. Hence, the electrical strain gauges were adopted for this investigation.

3.2.1 Structure Of The Strain Gauge

The strain gauge was constructed by bonding the fine resistance wire or the photographically etched metallic resistance foil on a base of electrical insulation with the appropriate bonding material and the take out leads welded on. The most commonly used strain gauges are produced from Copper Nickel foil on either epoxy resin or polymide backing. The strain gauge used has a gauge length of 20mm, and gauge resistance of 120 ± 0.3 Ohms.

3.2.2 Arrangement Of Strain Gauges

This was done by placing the gauges at a spacing of 300mm from each other vertically from bottom of the subgrade to the bottom of the rail. In order to measure the stress distribution per depth of the subgrade and static loading of

the train already modelled. (see Figs. 3.4, 3.5, 3.6). The loading was carried out in an incremental manner of 10 kg weight to the modelled train until a maximum of 100 kg was reached.

But for ballast, the strain-gauges were put at a spacing of 10mm from top to bottom up to 60mm. Since, the maximum depth of ballast in the field is usually 300mm. For experimental reasons it was extended to 60mm and each ballast was tested and depth varied accordingly.

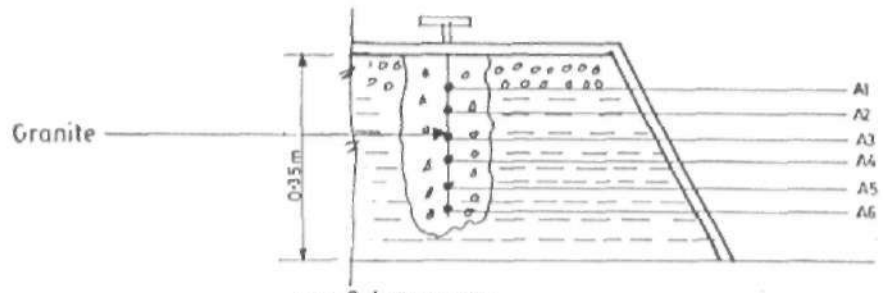


FIG 3·4 GRANITE

MICRO – STRAIN METRE INSIDE THE GRANITE (1)

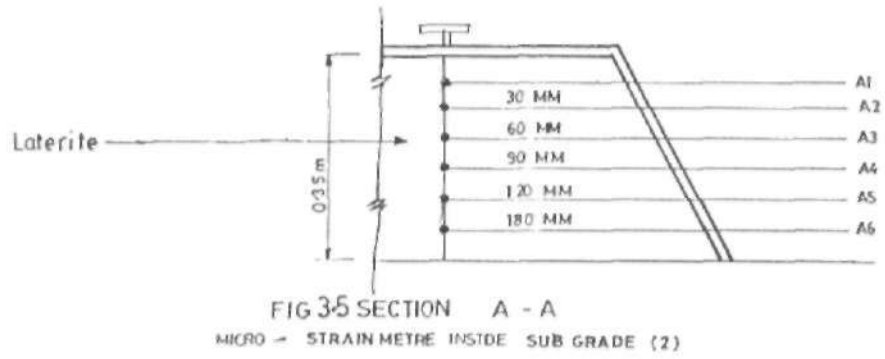


FIG 3·5 SECTION A - A

MICRO – STRAIN METRE INSIDE SUB GRADE (2)

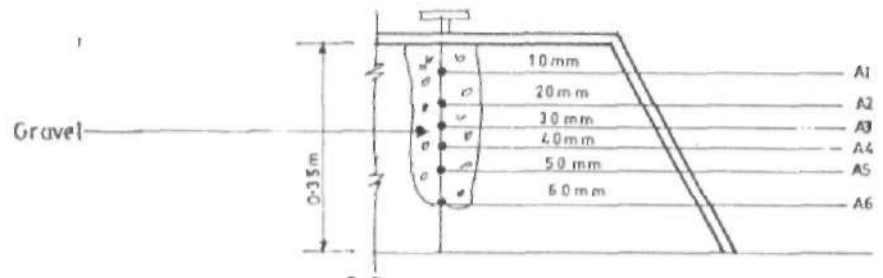


FIG 3·6 GRAVEL

MICRO – STRAINMETRE INSIDE GRAVEL (3)

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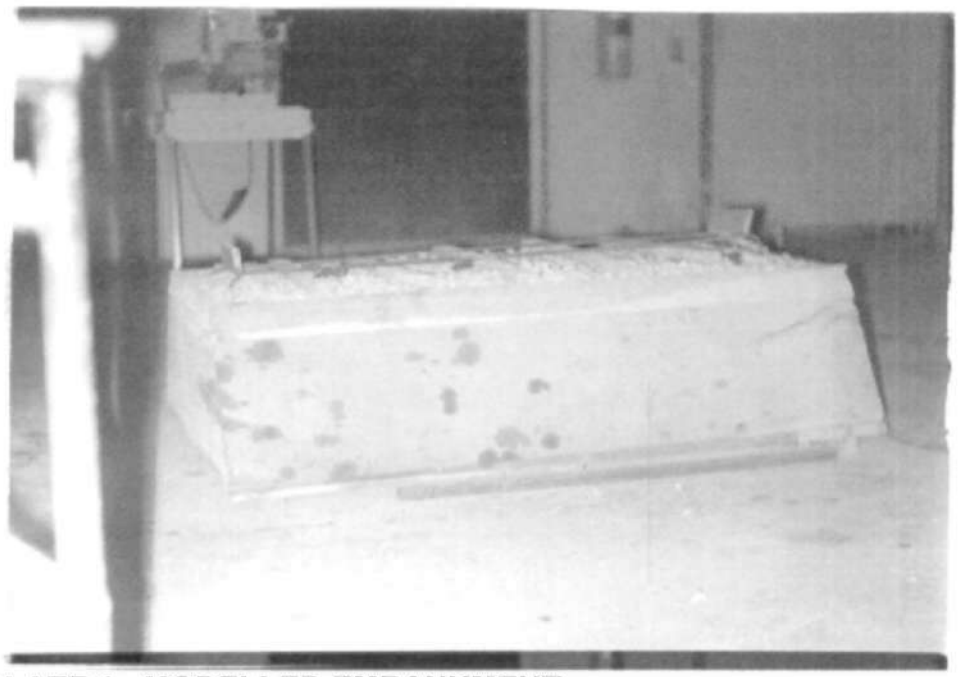


PLATE 1: MODELLED EMBANKMENT

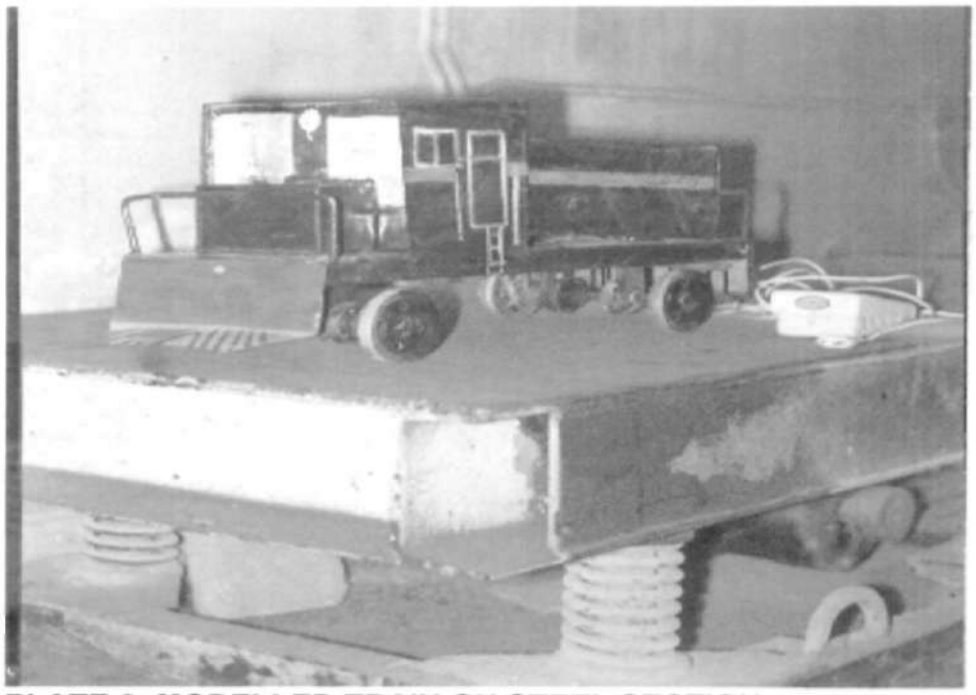
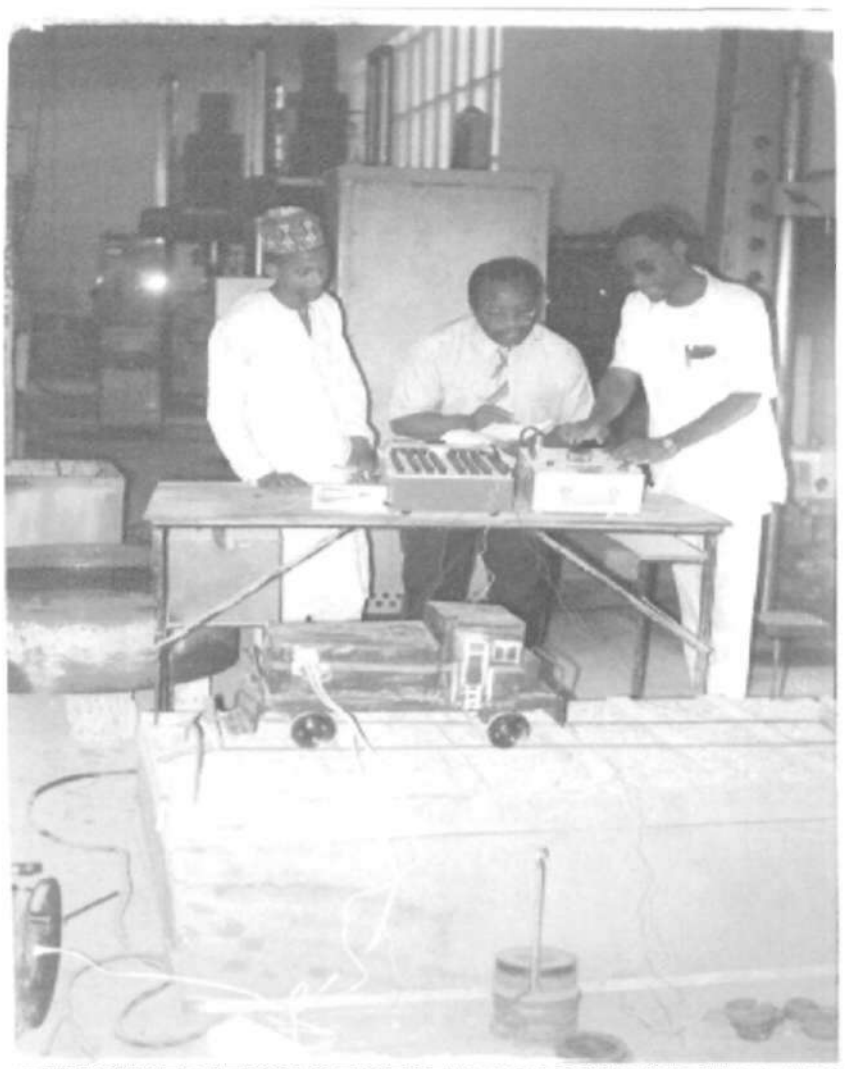


PLATE 2: MODELLED TRAIN ON STEEL SECTION



DEMONSTRATION OF STATIC LOADING IN THE LABORATORY

PLATE: 3.

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3.3 THEORETICAL ANALYSIS

3.3.0 Mathematical Equations

Many Scientists have contributed enormously to the problem of stress distribution. Among the significant ones are Boussinesq (1885), Talbot (1940) and Winkler (1867).

The three equations postulated by them will be used in the analyses of the simple track system modelled.

Boussinesq (1885) equation is given as:

$$\sigma = \frac{KP}{Z^2} \ (kN/m^2) \qquad (3.1)$$

where σ= stress generated, K is the influence factor, Z is the depth.

Talbot's equation is given as:

$$\sigma_T = \frac{16.8Pa}{h^{1.25}} \ (psi) \qquad (3.2)$$

$$= \frac{16.8Pa}{h^{1.25}} \times 6.894 \ (kN/m^2)$$

where σ_T is the stress generated, Pa is the load and h is the depth.

Winkler's (1867) equation is given as:

$$\sigma_w = \frac{3P}{2\lambda h^2} \ (N/mm^2) \qquad (3.3)$$

where σ_w = stress generated, P is the load, $1/\lambda$ is the interaction factor between rail and subgrade or between rail and Ballast, and it is equal to 1.24 for both Ballast and Subgrade; h is the depth of subgrade.

3.3.1 Boussinesq Equation

Boussinesq (1885) postulated equation for the determination of vertical normal stress under the soil which is written as:

$$\sigma = \frac{KP}{Z^2} \ (N/mm^2)$$

Where σ is the stress generated

Where k is the influence factor and it is given as: $\frac{3}{2\pi\{1+(r/z)^2\}^{5/2}}$

Where P is the load applied, z is the depth of subgrade.

Boussinesq stress distribution were based on the results given by the mathematical theory of elasticity for the simplest case of loading of a solid homogeneous, elastic, isotropic, semi-infinite medium namely, the case of a single vertical, point load applied at a point on the horizontal boundary surface.

Applying the equation on the ballast and subgrade in the thesis the following results were obtained:

Boussinesq Equation on ballast:

At depth 10mm, 100N load:

$$\sigma_{10mm} = \frac{KP}{Z^2} \ (N/mm^2)$$

$$= \frac{0.1083 \times 10/2 \times 9.8}{10^2}$$

$$= 5.310 \times 10^{-1} \ N/mm^2$$

Similar calculations were carried out for 200N, 300N, 400N up to 1000N at depth 20 mm, 30 mm to 60 mm and the results are shown on Table 3:2.

Table 3:1 INFLUENCE FACTORS FOR VERTICAL PRESSURE CAUSED BY A POINT LOAD

r/z	K	r/z	K	r/z	k
0.00	0.4775	1.00	0.0844	2.00	0.0085
0.10	0.4657	1.10	0.0658	2.10	0.0070
0.20	0.4329	1.20	0.0513	2.20	0.0058
0.30	0.3849	1.30	0.0402	2.30	0.0048
0.40	0.3294	1.40	0.0317	2.40	0.0040
0.50	0.2733	1.50	0.0251	2.50	0.0034
0.60	0.2214	1.60	0.0200	2.60	0.0029
0.70	0.1762	1.70	0.0160	2.70	0.0024
0.80	0.1386	1.80	0.0129	2.80	0.0021
0.90	0.1083	1.90	0.0105	2.90	0.0018

BOUSSINESQ EQUATION ON SUBGRADE.

At 30mm depth and 100N load

$$\sigma_{30mm} = \frac{KP}{Z^2} \ (N/mm^2)$$

$$= \frac{0.3849 \times 10/2 \times 9.81}{30^2}$$

$$= 0.020975 \ N/mm^2$$

Similar calculations were carried out for 200N, 300N, 400N up to 1000N at depth 30 mm, 60 mm to 180 mm and the results are shown on Table 3:3.

TABLE 3.2 **STRESSES ON QUARRY GRANITE (BOUSSINESQ EQUATION)**

DEPTH Z (mm)	100N	200N	300N	400N	500N	600N	700N	800N	900N	1000N
	STRESS (N/mm^2)									
10	5.310×10^{-1}	1.063×10^{-1}	1.5935×10^{-1}	2.125×10^{-1}	2.656×10^{-1}	3.188×10^{-1}	3.719×10^{-1}	4.250×10^{-1}	4.781×10^{-1}	5.312×10^{-2}
20	4.039×10^{-1}	8.075×10^{-2}	1.212×10^{-1}	1.612×10^{-1}	2.019×10^{-1}	2.424×10^{-1}	2.828×10^{-1}	3.232×10^{-1}	3.636×10^{-1}	4.079×10^{-1}
30	2.098×10^{-1}	4.195×10^{-1}	6.295×10^{-2}	8.309×10^{-2}	1.049×10^{-1}	1.259×10^{-1}	1.469×10^{-1}	1.678×10^{-1}	1.888×10^{-1}	2.098×10^{-1}
40	1.325×10^{-1}	2.654×10^{-1}	3.980×10^{-1}	5.310×10^{-2}	6.635×10^{-1}	7.965×10^{-2}	9.290×10^{-2}	1.062×10^{-1}	1.195×10^{-1}	1.195×10^{-1}
50	9.135×10^{-3}	1.825×10^{-2}	2.740×10^{-2}	3.6545×10^{-2}	4.569×10^{-2}	5.480×10^{-2}	6.395×10^{-2}	7.310×10^{-2}	8.225×10^{-2}	9.12×10^{-2}
60	6.115×10^{-3}	1.223×10^{-2}	1.835×10^{-2}	2.447×10^{-2}	3.058×10^{-2}	3.669×10^{-2}	4.280×10^{-2}	4.893×10^{-2}	5.505×10^{-2}	6.115×10^{-2}

TABLE 3.3 **STRESSES ON LATERITE SUBGRADE (BOUSSINESQ EQUATION)**

DEPTH Z (mm)	100N	200N	300N	400N	500N	600N	700N	800N	900N	1000N
	STRESS (N/mm^2)									
30	2.098×10^{-2}	4.195×10^{-2}	6.295×10^{-1}	8.390×10^{-2}	1.049×10^{-1}	1.259×10^{-1}	1.469×10^{-1}	1.678×10^{-1}	1.888×10^{-1}	2.098×10^{-1}
60	6.115×10^{-3}	1.224×10^{-2}	1.835×10^{-2}	2.447×10^{-2}	3.058×10^{-2}	3.670×10^{-2}	4.280×10^{-2}	4.893×10^{-2}	5.500×10^{-2}	6.115×10^{-2}
90	2.812×10^{-3}	5.625×10^{-3}	8.435×10^{-3}	1.125×10^{-2}	1.405×10^{-2}	1.688×10^{-2}	1.970×10^{-2}	2.250×10^{-2}	2.800×10^{-2}	2.810×10^{-3}
120	1.601×10^{-3}	3.202×10^{-3}	4.803×10^{-3}	6.405×10^{-3}	8.005×10^{-3}	9.605×10^{-3}	1.120×10^{-2}	1.281×10^{-2}	1.440×10^{-2}	1.600×10^{-3}
150	1.031×10^{-3}	2.061×10^{-3}	3.092×10^{-3}	4.122×10^{-3}	5.150×10^{-3}	6.185×10^{-3}	7.215×10^{-3}	8.250×10^{-3}	9.250×10^{-3}	1.030×10^{-3}
180	7.18×10^{-4}	1.436×10^{-3}	2.153×10^{-3}	2.153×10^{-3}	3.589×10^{-3}	4.307×10^{-3}	5.000×10^{-3}	7.400×10^{-3}	6.450×10^{-3}	7.175×10^{-4}

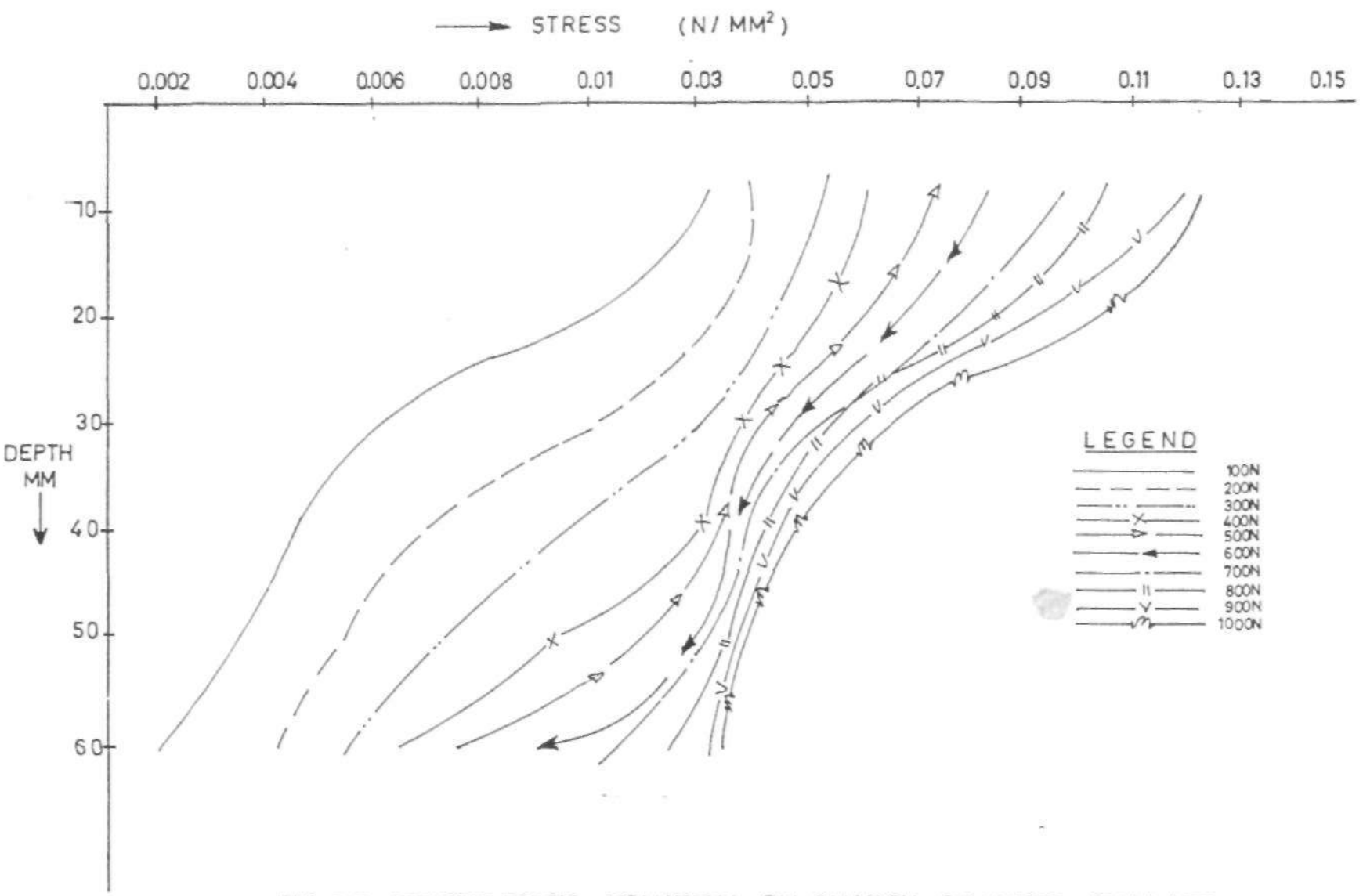


FIG. 3.7 BOUSSINESQ'S EQUATION ON QUARRY GRANITE BALLAST

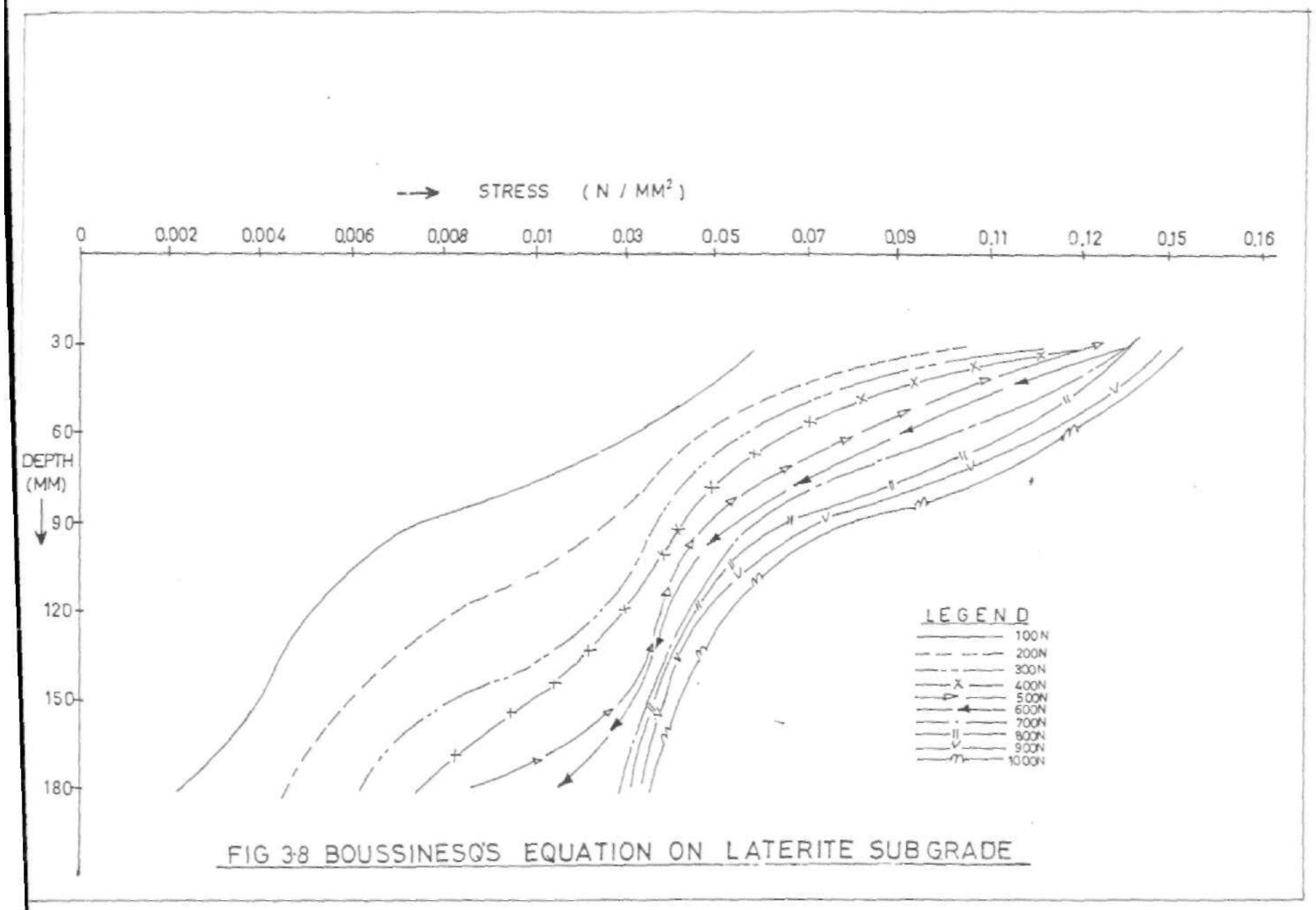


FIG 3·8 BOUSSINESQ'S EQUATION ON LATERITE SUBGRADE

3.3.2 Talbot's Equation

In order to calculate the stresses generated in the ballast and subgrade of rail system, Talbot (1940) used the following expression:

$$_T = \frac{16.8Pa}{h^{1.25}}(Psi)$$

Where $_T$ = pressure in pound per square inch under the tie centre line
Pa = uniformly distributed pressure over the tie face.
h = depth below the face in inches.

Therefore, in metric system the equation becomes:

$$_T = \frac{16.8Pa}{h^{1.25}} \times 6.894\ (kN/m^2)$$

Talbot's equation on Ballast:

At 10 mm depth and 100N load:

$$_{10mm} = \frac{16.8 \times 10/2 \times 2.22}{0.39^{1.25}} \times 6.894(KN/m^2)$$

$= 172.37KN/m^2$

$= 172.37N/mm^2 \times 1/1680$ (scaled to laboratory condition)

$= 1.026x10^{-2}N/mm^2$

Similar calculations were carried out for 200N, 300N 400N up 1000N at depth 20 mm, 30 mm to 60 mm and the results are shown on Table 3:4.

Talbots Equation on Subgrade

At 30mm depth and 100N load:

$$_{30mm} = \frac{16.8Pa \times 6.894(KN/m^2)}{h^{1.25}}$$

$$= \frac{16.8Pa}{1.1h^{1.25}}$$

$$= \frac{16.8 \times 10/2 \times 2.22}{1.17^{1.25}} \times 6.894(KN/m^2)$$

$$= 57.456\ KN/m^2$$

$$= 57.456N/mm^2 \times 1/1680 \text{ (scaled to laboratory condition)}$$

$$= 3.420 \times 10^{-2}N/mm^2$$

Similar calculations were carried out for 200N, 300N to 1000N at depth 30 mm, 60 mm to 180 mm and the results are shown on Table 3:5.

TABLE 3.4 **STRESSES ON QUARRY GRANITE BALLAST (TALBOT'S EQUATION)**

DEPTH Z (mm)	100N	200N	300N	400N	500N	600N	700N	800N	900N	1000N
	STRESS (N/mm^2)									
10	1.026×10^{-1}	1.656×10^{-1}	3.078×10^{-1}	4.104×10^{-1}	5.130×10^{-1}	6.157×10^{-1}	7.183×10^{-1}	8.207×10^{-1}	9.235×10^{-1}	1.0260×10^{-1}
20	1.265×10^{-2}	8.530×10^{-2}	1.279×10^{-1}	1.706×10^{-1}	2.125×10^{-1}	2.559×10^{-1}	2.986×10^{-1}	3.412×10^{-1}	3.839×10^{-1}	4.266×10^{-1}
30	3.420×10^{-2}	6.084×10^{-2}	9.127×10^{-2}	1.098×10^{-1}	1.710×10^{-1}	2.052×10^{-1}	2.394×10^{-1}	2.736×10^{-1}	3.078×10^{-1}	3.420×10^{-1}
40	2.565×10^{-2}	5.132×10^{-2}	7.697×10^{-2}	1.026×10^{-1}	1.283×10^{-1}	1.539×10^{-1}	1.796×10^{-1}	2.055×10^{-1}	2.309×10^{-1}	2.566×10^{-1}
50	2.052×10^{-2}	4.100×10^{-2}	6.157×10^{-2}	8.208×10^{-2}	1.026×10^{-1}	1.231×10^{-1}	1.436×10^{-1}	1.641×10^{-1}	1.847×10^{-1}	2.052×10^{-1}
60	1.71×10^{-2}	3.420×10^{-2}	5.130×10^{-2}	6.841×10^{-2}	8.550×10^{-2}	1.026×10^{-1}	1.197×10^{-1}	1.368×10^{-1}	1.539×10^{-1}	1.710×10^{-1}

TABLE 3.5 **STRESSES ON LATERITE SUBGRADE (TALBOT'S EQUATION)**

DEPTH Z (mm)	100N	200N	300N	400N	500N	600N	700N	800N	900N	1000N
	STRESS (N/mm^2)									
30	3.420×10^{-2}	6.840×10^{-2}	1.026×10^{-1}	1.369×10^{-1}	1.711×10^{-1}	2.053×10^{-1}	2.396×10^{-1}	2.737×10^{-1}	3.078×10^{-1}	3.420×10^{-1}
60	1.71×10^{-2}	3.420×10^{-2}	5.130×10^{-2}	6.840×10^{-2}	8.55×10^{-2}	1.026×10^{-1}	1.197×10^{-1}	1.367×10^{-1}	1.539×10^{-1}	1.710×10^{-1}
90	1.14×10^{-2}	2.280×10^{-2}	3.420×10^{-2}	4.560×10^{-2}	5.700×10^{-2}	6.840×10^{-2}	7.980×10^{-2}	9.118×10^{-2}	1.026×10^{-1}	1.140×10^{-1}
120	8.550×10^{-3}	1.71×10^{-2}	2.565×10^{-2}	3.420×10^{-2}	4.275×10^{-2}	5.130×10^{-2}	5.980×10^{-2}	6.839×10^{-2}	7.696×10^{-2}	8.550×10^{-2}
150	6.840×10^{-3}	1.378×10^{-2}	2.052×10^{-2}	2.736×10^{-2}	3.420×10^{-2}	4.104×10^{-2}	4.788×10^{-2}	5.471×10^{-3}	6.156×10^{-2}	6.840×10^{-2}
180	5.700×10^{-3}	1.140×10^{-2}	1.171×10^{-2}	2.280×10^{-2}	2.850×10^{-2}	3.420×10^{-2}	3.990×10^{-2}	4.559×10^{-2}	5.130×10^{-2}	5.700×10^{-2}

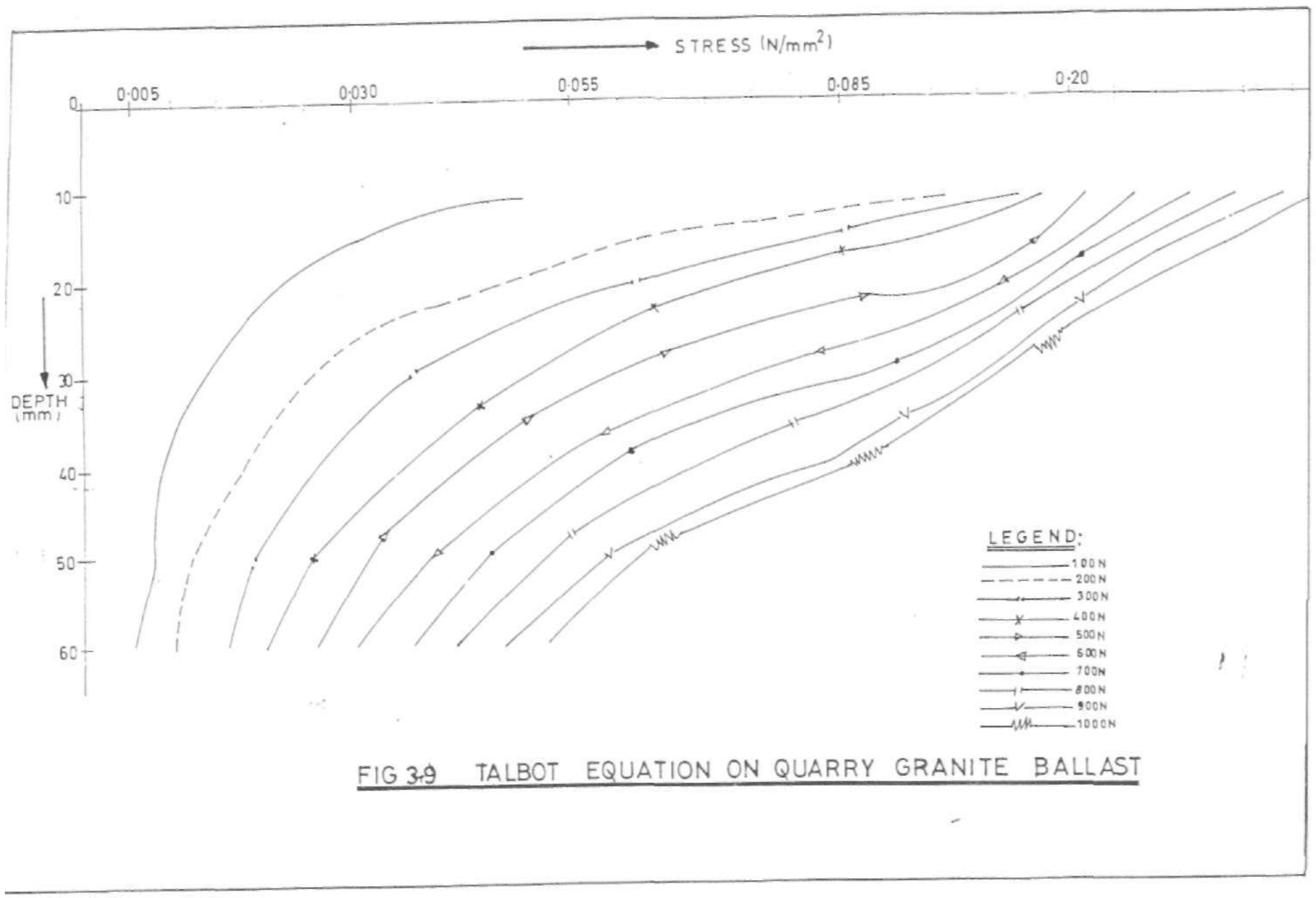


FIG 3.9 TALBOT EQUATION ON QUARRY GRANITE BALLAST

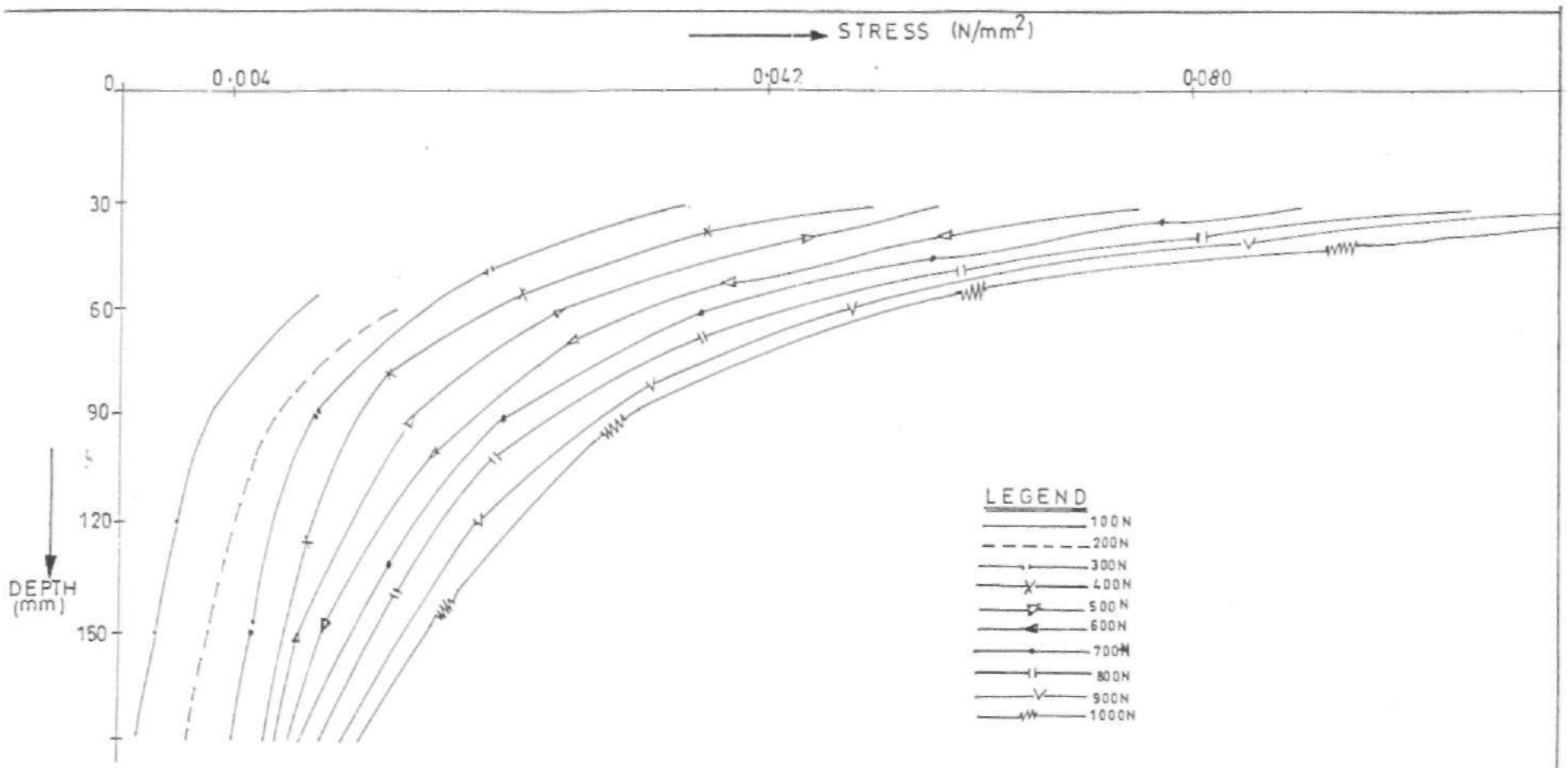


FIG3·10 TALBOT EQUATION ON LATERITE SUBGRADE (STANDARD PROCTOR TEST)

3.3.5 Winklers Equation

Winkler (1867) postulated an equation for the determination of stresses for ballast and subgrade of track. The equation states that:

$$\sigma_w = \frac{3P}{2\lambda h^2} = \frac{1.5P}{\lambda h^2} \quad (N/mm^2)$$

where σ_w is the stress generated, where h is the depth

where P is the load (N/mm^2)

Where $1/\lambda$ is the interaction factor between the rail and subgrade.

FOR BALLAST:

At 10 mm, with 100N load

$$\sigma_{10m} = \frac{3P}{2\lambda h^2}$$

$$= \frac{3 \times 10/2 \times 9.81 \times 1.24 \times 10^2}{2 \times (10)^2}$$

$$= \frac{36493.2}{2 \times (10)^2}$$

= 182.466N/mm^{2}x1/1680 (scaled to laboratory condition)

= 5.430 x 10^{-1}N/mm^2

Similar calculations were carried out for 200N, 300N to 1000N at depth 10 mm, 20 mm to 60 mm and the results are shown on Table 3:6.

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FOR SUBGRADE:

At 30mm, 100N load:

$$\sigma_{30m} = \frac{3P}{2\pi h^2} \text{ (N/mm}^2\text{)}$$

$$= \frac{3 \times 10/2 \times 9.81 \times 1.24 \times 10^2}{2 \times (30)^2}$$

$$= 2.03 \text{N/mm}^2$$

$$= 20.27 \text{N/mm} \times 1/1680 \text{ (scaled to laboratory condition)}$$

$$= 6.030 \times 10^{-1} \text{N/mm}^2$$

Similarly stresses generated when 200N, 300N, 400N up to 1000N at depth 30 mm, 60 mm to 180 mm and the results are shown on Table 3:6. And so on were put on the modelled train were recorded on tables for optimum stage of the laterite subgrade as displaced later in the thesis.

TABLE 3.6 **STRESSES ON QUARRY GRANITE BALLAST (WINKLER'S EQUATION)**

DEPTH Z (mm)	100N	200N	300N	400N	500N	600N	700N	800N	900N	1000N
	STRESS (N/mm^2)									
10	5.430×10^{-2}	1.086×10^{-1}	1.629×10^{-1}	2.173×10^{-1}	2.715×10^{-1}	3.256×10^{-1}	3.800×10^{-1}	4.342×10^{-1}	4.889×10^{-1}	5.430×10^{-1}
20	1.358×10^{-2}	2.718×10^{-2}	4.672×10^{-2}	5.430×10^{-2}	6.785×10^{-2}	8.145×10^{-2}	9.500×10^{-2}	1.086×10^{-1}	1.222×10^{-1}	1.358×10^{-1}
30	6.030×10^{-3}	1.207×10^{-2}	1.840×10^{-2}	2.421×10^{-2}	3.018×10^{-2}	3.619×10^{-2}	4.223×10^{-2}	4.828×10^{-2}	5.430×10^{-2}	6.030×10^{-2}
40	3.393×10^{-3}	7.790×10^{-3}	1.016×10^{-2}	1.358×10^{-2}	1.697×10^{-2}	2.037×10^{-2}	2.360×10^{-2}	2.715×10^{-2}	3.054×10^{-2}	3.394×10^{-2}
50	2.172×10^{-3}	4.345×10^{-3}	6.515×10^{-3}	8.690×10^{-3}	1.086×10^{-2}	1.303×10^{-2}	1.520×10^{-2}	1.589×10^{-2}	1.955×10^{-2}	2.173×10^{-2}
60	1.509×10^{-3}	3.018×10^{-3}	4.527×10^{-3}	6.030×10^{-3}	7.540×10^{-3}	9.050×10^{-3}	1.056×10^{-2}	1.207×10^{-2}	1.207×10^{-2}	1.508×10^{-2}

TABLE 3.7 **STRESSES ON LATERITE SUBGRADE (WINKLER'S EQUATION)**

DEPTH Z (mm)	100N	200N	300N	400N	500N	600N	700N	800N	900N	1000N
	STRESS (N/mm^2)									
30	6.030×10^{-2}	1.207×10^{-1}	1.810×10^{-1}	2.413×10^{-1}	3.015×10^{-1}	3.619×10^{-1}	4.223×10^{-1}	4.824×10^{-1}	5.430×10^{-1}	6.030×10^{-1}
60	$7.280 \times 10^{-1}2$	3.018×10^{-2}	4.518×10^{-2}	6.030×10^{-2}	7.540×10^{-2}	9.050×10^{-2}	1.056×10^{-1}	1.207×10^{-1}	1.358×10^{-1}	1.509×10^{-1}
90	6.695×10^{-2}	1.341×10^{-2}	2.012×10^{-2}	2.682×10^{-2}	3.351×10^{-2}	4.003×10^{-2}	4.694×10^{-2}	5.365×10^{-2}	6.035×10^{-2}	6.705×10^{-2}
120	3.779×10^{-2}	7.540×10^{-3}	1.132×10^{-2}	1.508×10^{-2}	1.887×10^{-2}	2.263×10^{-2}	2.640×10^{-2}	3.018×10^{-2}	3.393×10^{-2}	3.771×10^{-2}
150	2.411×10^{-2}	4.828×10^{-3}	7.240×10^{-2}	9.655×10^{-2}	1.207×10^{-2}	1.448×10^{-2}	1.690×10^{-2}	1.932×10^{-2}	2.173×10^{-2}	2.413×10^{-2}
180	1.676×10^{-2}	$3.351X 10^{-3}$	5.030×10^{-3}	6.705×10^{-3}	8.380×10^{-3}	1.005×10^{-2}	1.173×10^{-2}	1.341×10^{-2}	1.509×10^{-2}	1.676×10^{-2}

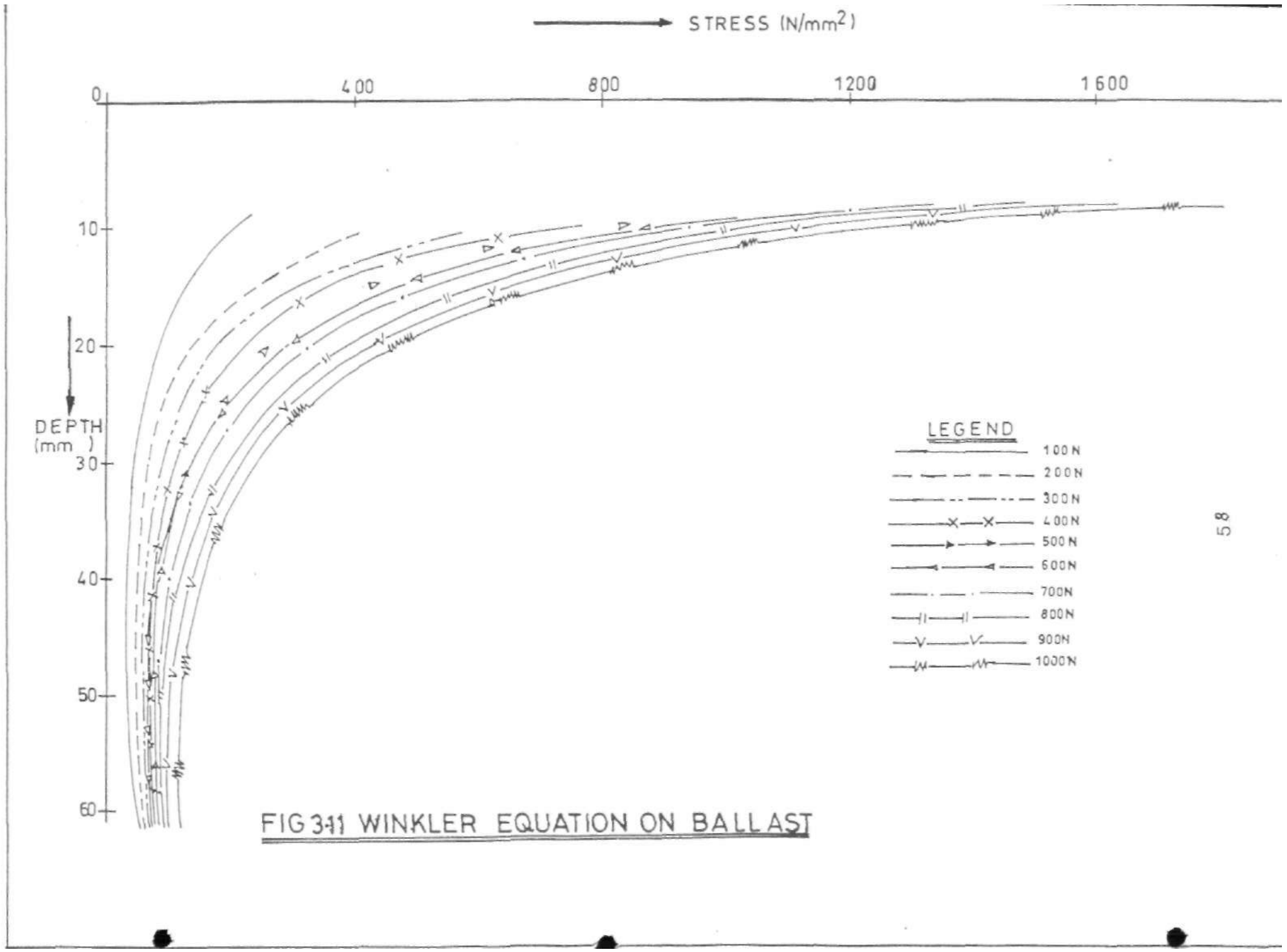


FIG 3.11 WINKLER EQUATION ON BALLAST

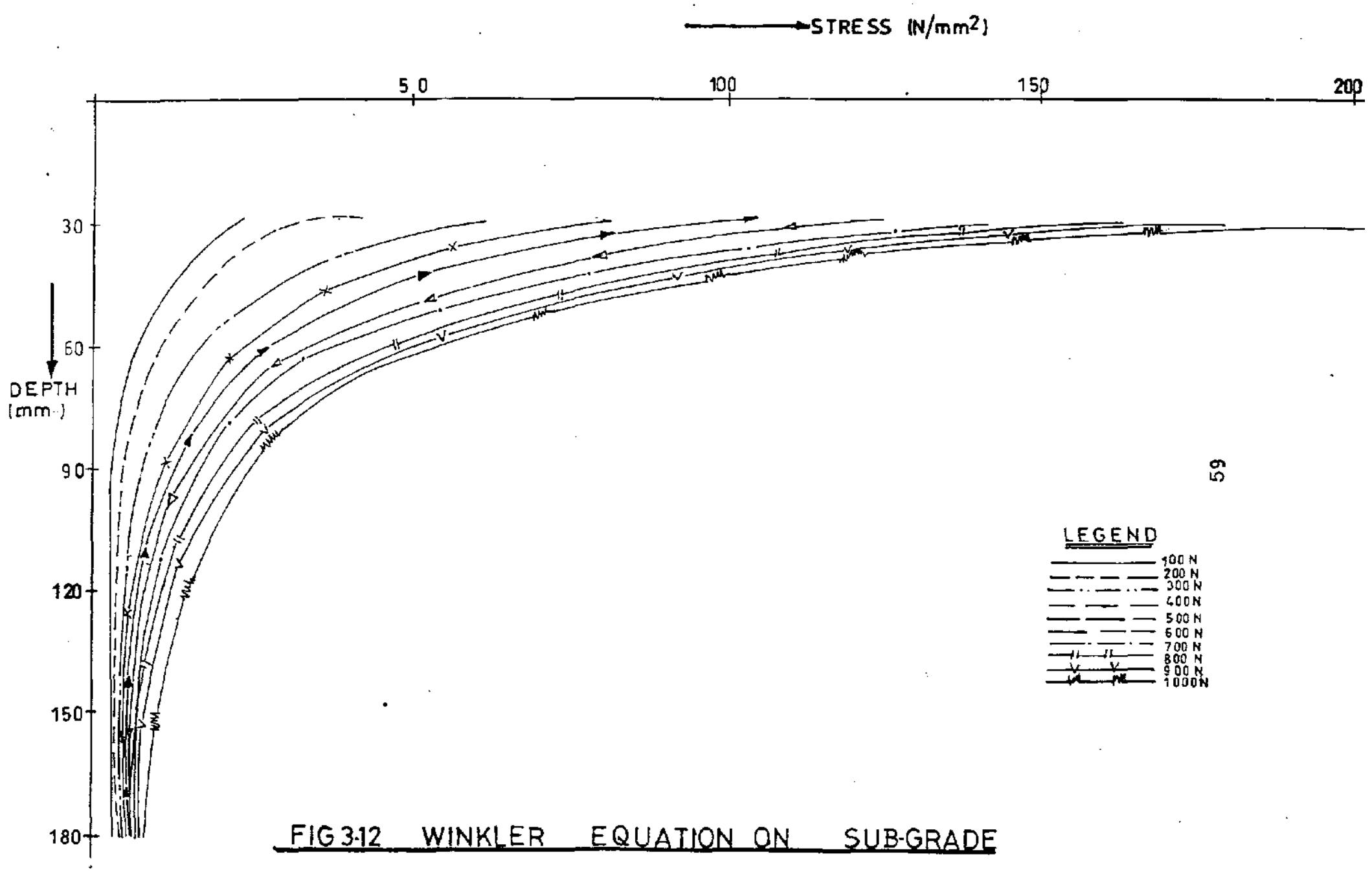


FIG 3.12 WINKLER EQUATION ON SUB-GRADE

3.4.0 Practical Observation

The following practical results were obtained in the laboratory with the modelled track system statically loaded under the loads of 100N, 200N, 300N, 400N, 500N, 600N, 700N, 800N, 900N, and 1000N respectively.

Strain guages were put at the depth of 10mm to 60mm in the case of ballast and 30mm to 180mm in the case of subgrade.

The study was carried out with ballast and subgrade under standard proctor compaction at optimum moisture content.

3.4.1 Static Loading For Ballast And Subgrade (Standard Proctor Test)

The stresses generated in the ballast bed were very small for instance the Quarry Granite ballast were of the order of $2.40x10^{-3}N/mm^2$ minimum to $3.76x10^{-3}N/mm^2$ maximum as shown in Table 3.7. In Natural Gravel the stresses were of the order of $2.16x10^{-3}N/mm^2$ to $3.44x10^{-3}N/mm^2$ as shown in Table 3.7.

This difference in value could be as a result of the good quality of Quarry Granite to Natural Gravel as shown in Table 3.8.

The stresses generated in the subgrade is small of the order of $2.71x10^{-3}N/mm^2$ minimum and $3.79x10^{-3}N/mm^2$ maximum as shown in Table 3.9.

TABLE 3.8 **STATIC LOADING FOR QUARRY GRANITE BALLAST (STANDARD PROCTOR TEST)**

DEPTH z (mm)	100N	200N	300N	400N	500N	600N	700N	800N	900N	1000N
	STRESS $X 10^{-3}$ (N/mm^2)									
10	3.504	3.536	3.596	3.600	3.616	3.656	3.668	3.680	3.696	3.760
20	3.216	3.096	3.162	3.240	3.184	3.224	3.196	3.284	3.304	3.284
30	2.964	2.960	2.968	2.976	2.976	2.980	2.964	2.964	2.958	2.964
40	2.728	2.728	2.728	2.728	2.728	2.800	2.804	2.824	2.860	2.880
50	2.484	2.492	2.484	2.524	2.564	2.576	2.604	2.616	2.640	2.724
60	2.400	2.404	2.404	2.404	2.404	2.406	2.408	2.412	2.424	2.436

TABLE 3.9 **STATIC LOADING FOR NATURAL GRAVEL BALLAST (STANDARD PROCTOR TEST)**

DEPTH z (mm)	100N	200N	300N	400N	500N	600N	700N	800N	900N	1000N
	STRESS $X 10^{-3}$ (N/mm^2)									
10	3.132	3.164	3.156	3.204	3.264	3.276	3.296	3.344	3.376	3.444
20	2.928	2.952	2.962	2.996	2.956	2.964	2.964	2.992	3.036	3.040
30	2.680	2.684	2.682	2.720	2.728	2.728	2.736	2.726	2.752	2.760
40	2.556	2.560	2.576	2.604	2.616	2.612	2.620	2.640	2.648	2.672
50	2.488	2.484	2.480	2.504	2.540	2.516	2.516	2.516	2.516	2.516
60	2.176	2.182	2.212	2.216	2.220	2.224	2.240	2.252	2.252	2.256

TABLE 3.10 **STATIC LOADING FOR SUBGRADE AT OPTIMUM (STANDARD PROCTOR TEST)**

DEPTH z (mm)	100N	200N	300N	400N	500N	600N	700N	800N	900N	1000N
	STRESS X 10^{-3} (N/mm^2)									
30	3.713	3.716	3.720	3.728	3.766	3.766	3.785	3.792	3.794	3.796
60	3.348	3.314	3.55	3.321	3.386	3.386	3.325	3.378	3.380	3.382
90	3.162	3.196	3.198	3.203	3.1996	3.196	3.198	3.196	3.199	3.203
120	2.979	2.903	2.911	2.956	2.922	2.926	3.309	2.956	2.968	2.968
150	2.808	2.808	2.803	2.827	2.835	2.842	2.846	2.835	2.835	2.838
180	2.713	2.734	2.734	2.739	2.742	2.747	2.774	2.774	2.804	2.806

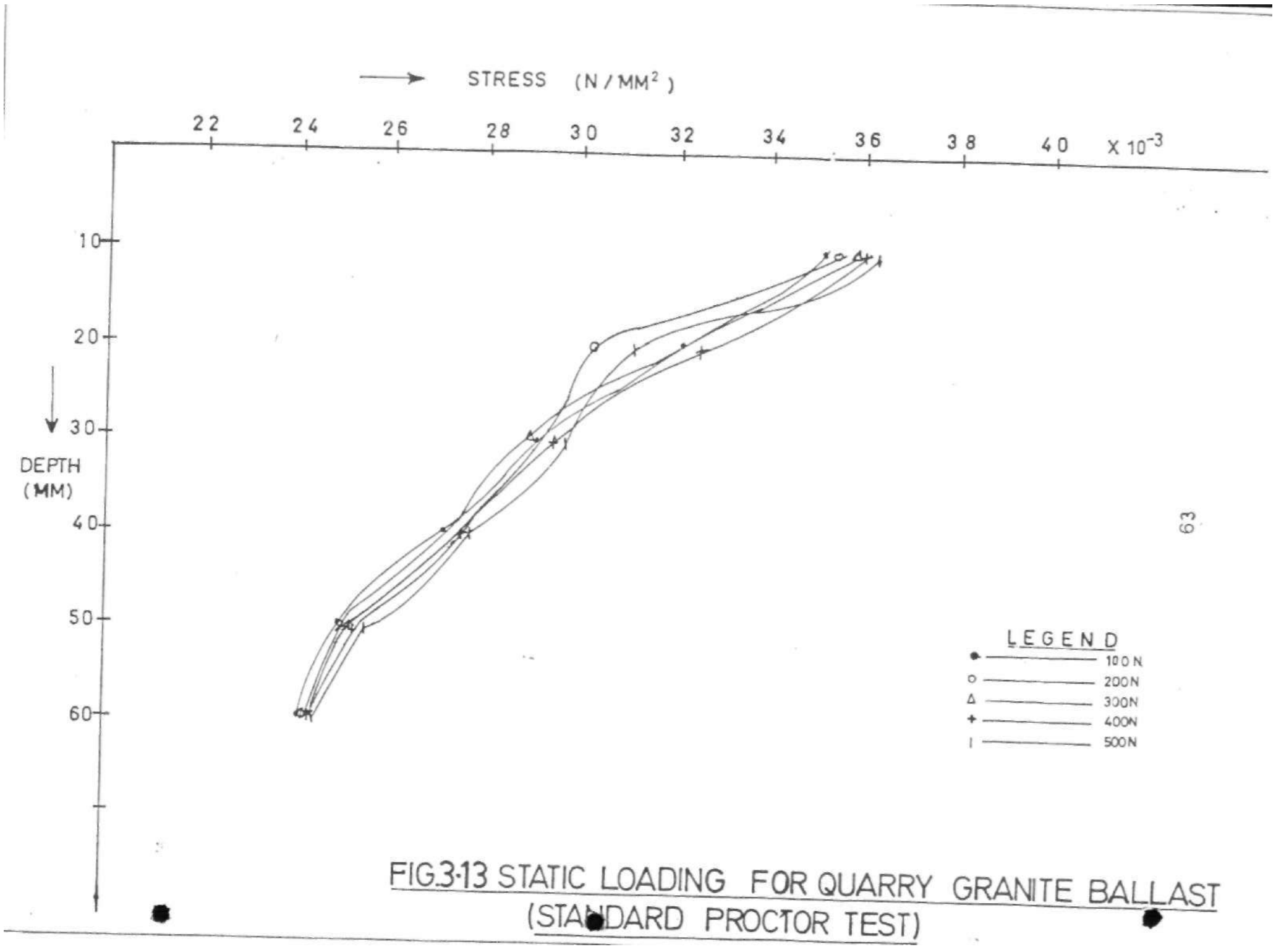


FIG.3·13 STATIC LOADING FOR QUARRY GRANITE BALLAST (STANDARD PROCTOR TEST)

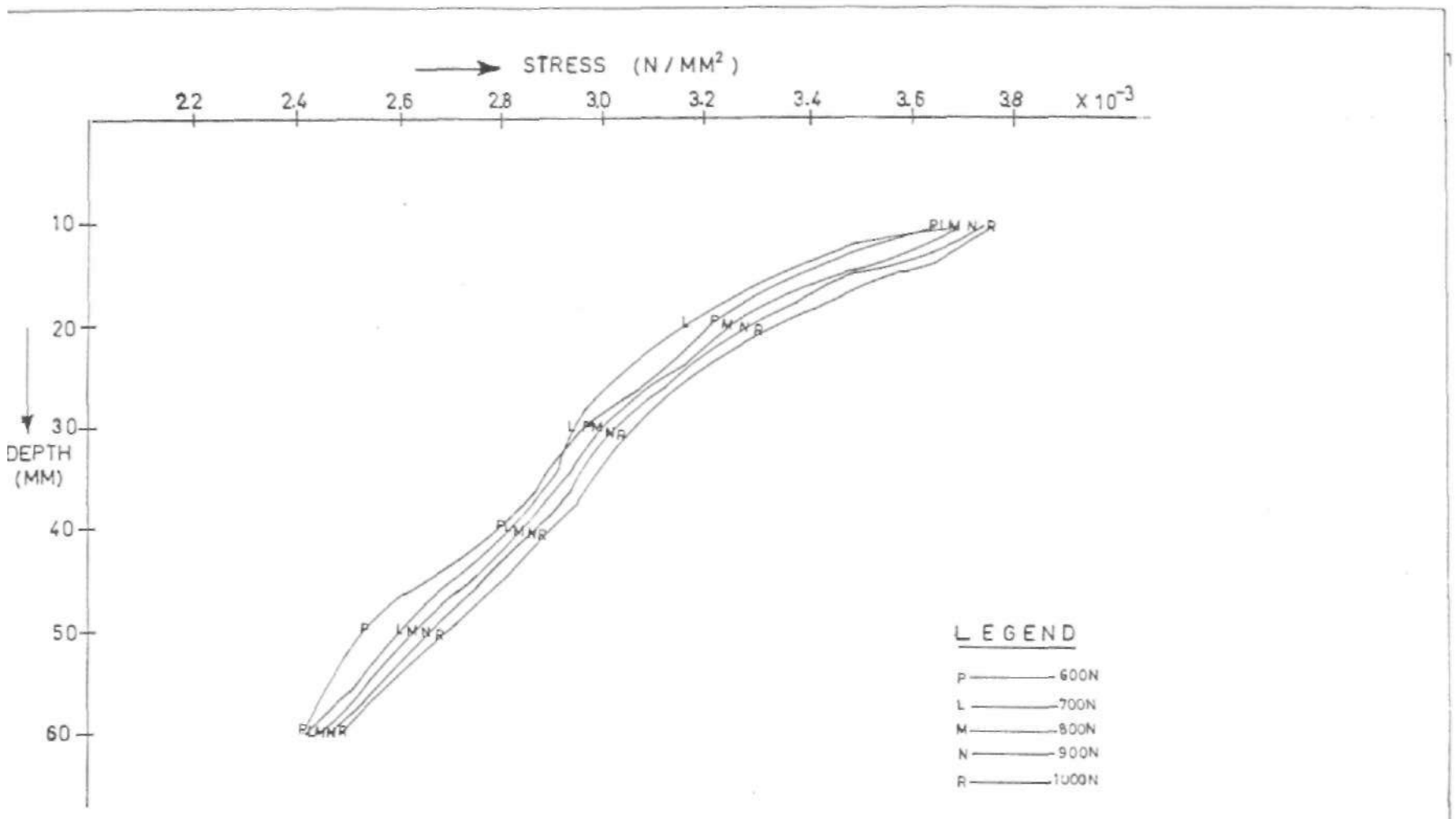


FIG 3.14 STATIC LOADING FOR QUARRY GRANITE (STANDARD PROCTOR TEST)

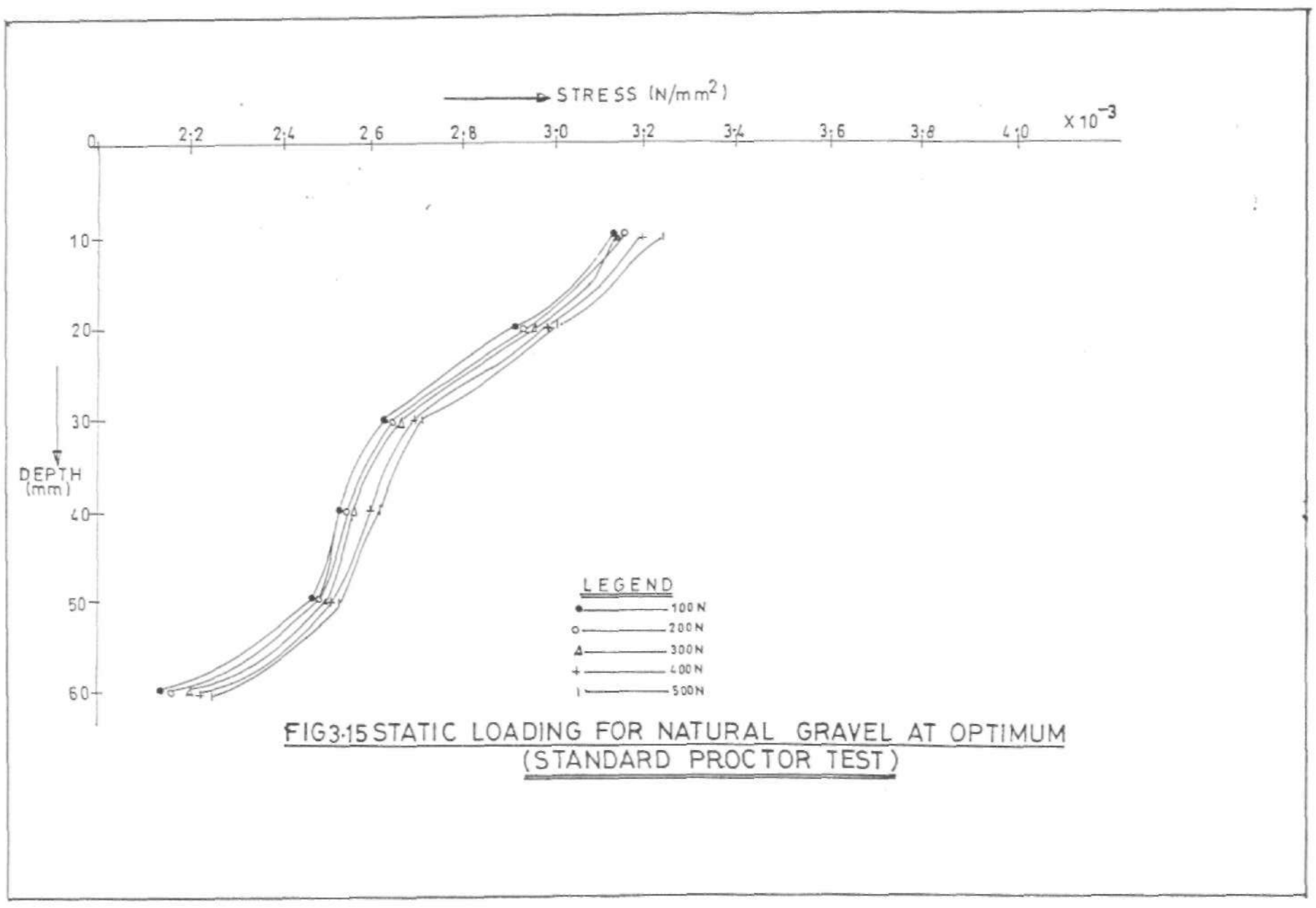


FIG 3·15 STATIC LOADING FOR NATURAL GRAVEL AT OPTIMUM (STANDARD PROCTOR TEST)

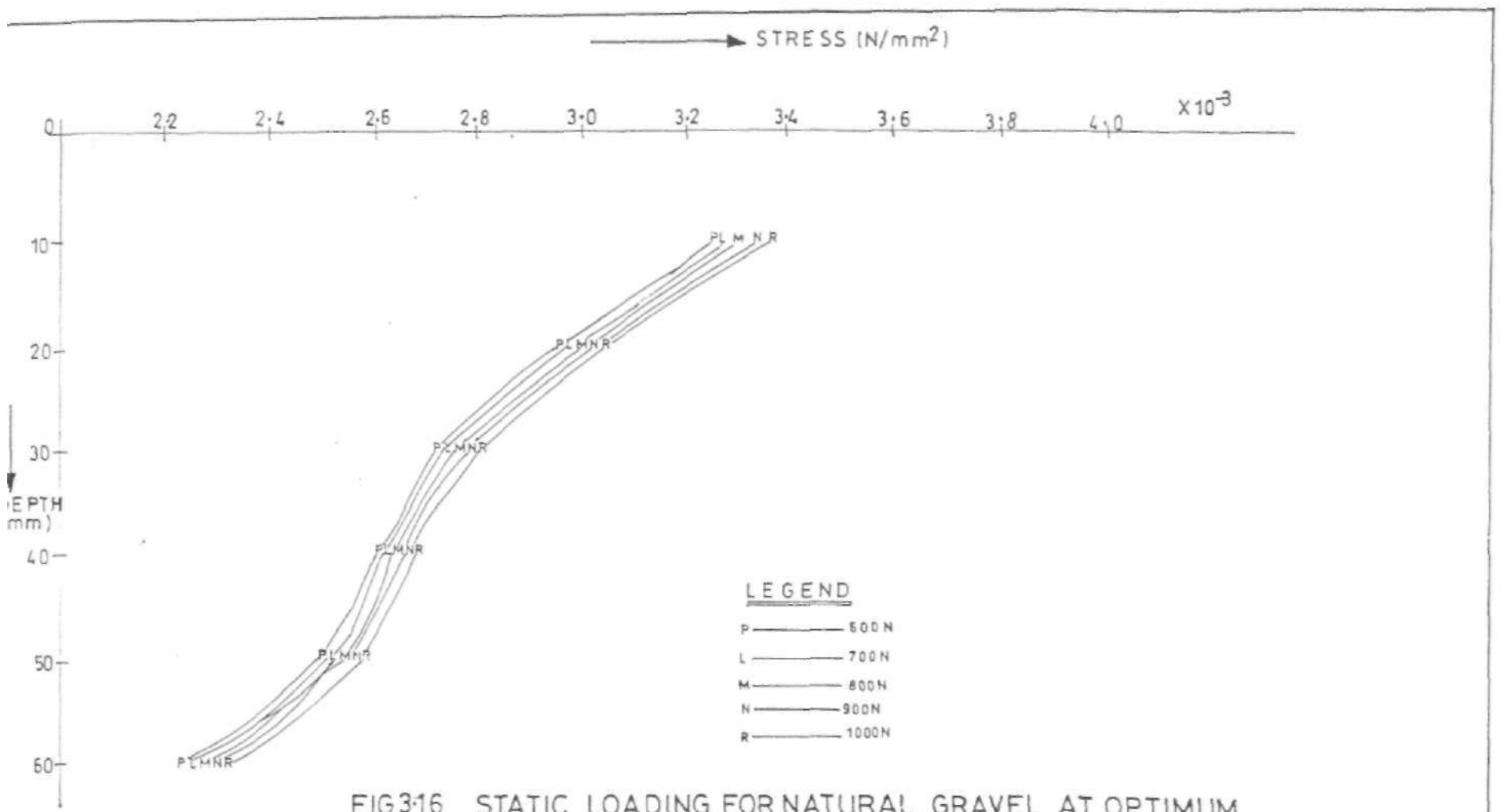


FIG 3·16 STATIC LOADING FOR NATURAL GRAVEL AT OPTIMUM (STANDARD PROCTOR TEST)

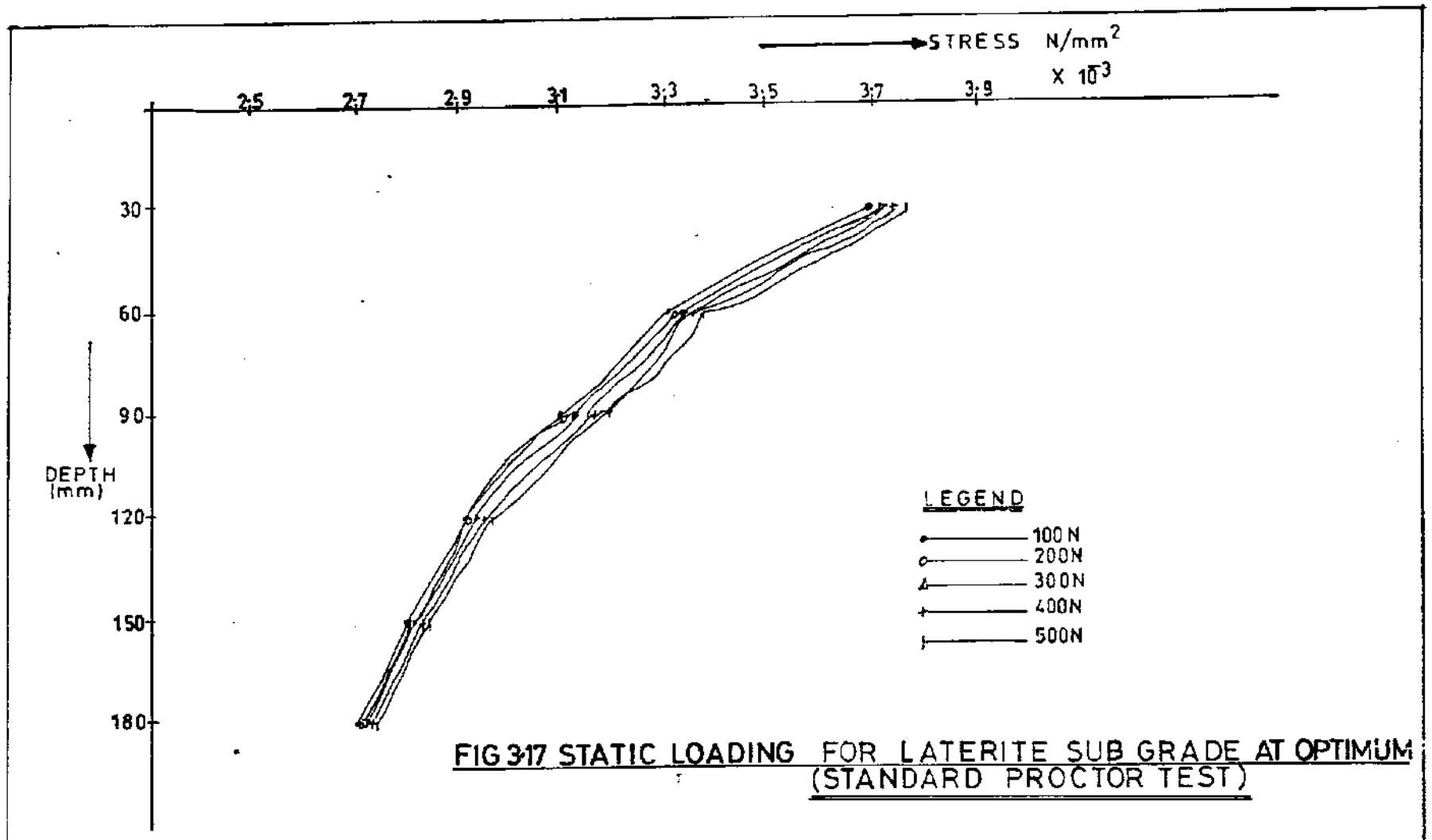


FIG 3·17 STATIC LOADING FOR LATERITE SUB GRADE AT OPTIMUM (STANDARD PROCTOR TEST)

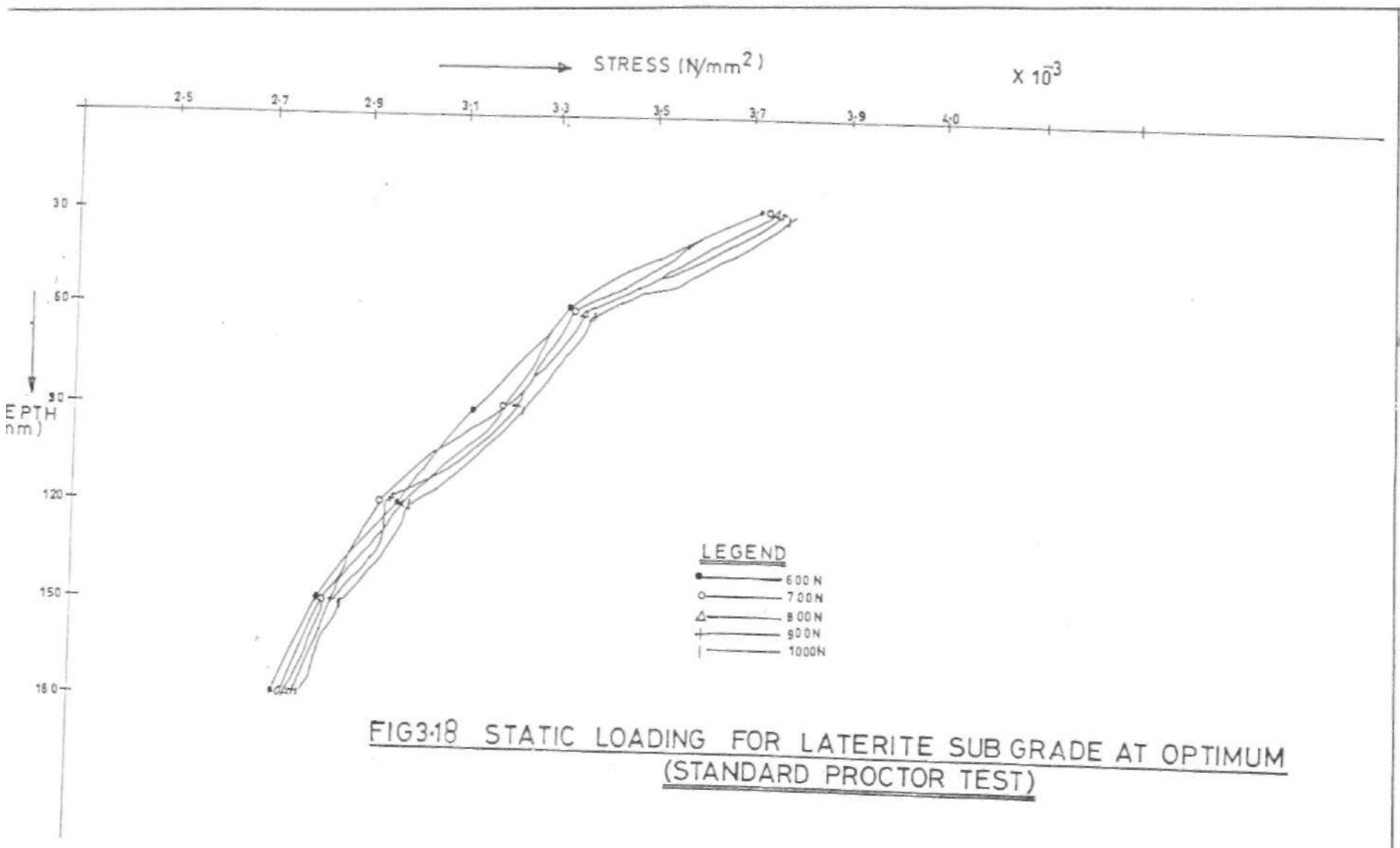


FIG3·18 STATIC LOADING FOR LATERITE SUB GRADE AT OPTIMUM (STANDARD PROCTOR TEST)

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3.4.2. Curves

Two curves were prepared for the purpose of analyses in the thesis namely: the theoretical curves and the experimental curves. The theoretical curves consist of the curves showing the Boussinesq's equation, Talbot's equation and Winkler's equation respectively; as shown in figures 3.7, 3.8, 3.9, 3.10, 3.11 and 3.12. While the experimental work was the practical results displaced graphically in Figure 3.13, 3.14, 3.15, 3.16, 3.17, 3.18. Curves showing the comparison between the three materials, quarry granite, natural gravel and laterite subgrade were displaced in Figures 3.19,3.20 and 3.21 respectively.

(1) Curves showing the Boussinesq's equation showed a curvi-linear displacement of the equation. The curve is curvi-linear because the equation is hypothetical in orientation. The maximum value of the stress was 5.310N/mm^{1} and minimum $6.115x10^{-3}$N/mm^2 for Ballast and maximum and minimum for subgrade $2.098x10^{-2}$ N/mm^2 and $7.180x10^{-4}$N/mm^2 as shown in Tables 3.2 and 3.3 respectively.

(2) Similarly, Talbot's equation produced a curvi-linear curve with gradual fall in gradient but with a slight high value of stresses than the experimental results. The maximum and minimum values for stresses produced with the equation for Ballast were $1.026x10^{-1}$N/mm^2 and $1.710x10^{-2}$N/mm^2 and subgrade $3.420x10^{-2}$N/mm^2 and $5.700x10^{-3}$N/mm^2.

(3) Lastly, on the theoretical equation, Winkler's equation produced a curvi-linear graph with little or no conflict towards the tail end. The stresses generated by the equation were higher compared to that generated by Talbot's equation. The maximum and minimum values for quarry Granite Ballast and laterite subgrade respectively were $5.430x10^{-2}$N/mm^2 and $1.509x10^{-3}$ and $6.030x10^{-2}$N/mm^2 and $2.411x10^{-2}$N/mm^2 respectively.

(4) The practical results show curves with various conflicts due to high resistance require to break the intermolecular particles of the moisture and soil at the optimum stage.

3.4.3 Comparison Between Quarry Granite, Natural Gravel And Laterite Subgrade At 100N, 500N And 1000N Loadings.

The curves of the three materials were shown in Figures 3.19, 3.20, and 3.21 for 100N, 500N and 1000N loads respectively. Considering 100N load in Fig. 3.19, it could be seen that the curves were at a distance apart most especially the curve of laterite, secondly, the stressed absorbed by the materials at 30mm depth for instance, was highest in laterite subgrade about 3.3 x 10^{-3}N/mm^2 followed by quarry granite about 3.1 x 10^{-3}N/mm^2 and then 2.7 x 10^{-3}N/mm^2 by natural gravel.

This showed that:

(I) More stresses were needed to bring the materials together, that is, for the stresses to be transferred to the laterite subgrade, since the curves of laterite subgrade was far apart.

(II) Quarry granite and laterite subgrade were better materials than natural gravel.

At 500N load in figure 3.20, the curves were very close together most especially the curve of laterite subgrade which joined the curve of quarry granite at a depth of 40mm and stress of about 2.62 x 10^{-3}N/mm^2 but the curve of natural gravel was still apart. Secondly, at 30mm depth, the stress absorbed by quarry granite was the highest about 3.0 x 10^{-3}N/mm^2, followed by laterite 2.9 x 10^{-3}N/mm^2.

This showed that:

(I) The stress transfer from the quarry granite to the subgrade has commenced hence the curves were interlocking together at 40mm depth but the curve of natural gravel was still standing apart.

(II) Quarry granite and laterite were better materials for use in railway than natural gravel.

Lastly, at 1000N as in Fig 3.21, the interlocking of the curves of laterite subgrade and quarry granite was pronounced upto a depth of 60mm with decrease in stress down the depth. Secondly, the stresses absorbed by both laterite subgrade and quarry granite were higher at 30mm as before. For instance, 2.85 x 10^{-3}N/mm^2 by laterite subgrade while that of natural gravel was 2.75 x 10^{-3}N/mm^2.

This showed that:

As the load increased from 500N to 1000N there was more transfer of stresses down to the subgrade while the curve of natural gravel was still apart hence there was no transfer of stress as such.

(I) This further showed that quarry granite and laterite subgrade were better material s than natural gravel.

3.4.4 Vertical Stress Distribution Diagrams At 100N, 500N And 1000N For Quarry Granite, Natual Gravel And Laterite Subgrade

The vertical stress distribution diagrams are shown in Figures 3.22, 3.23, 3.24, 3.25, 3.26, 3.27, 3.28, 3.29 and 3.30 for 100N, 500N and 1000N loads for the three materials. Quarry granite, natural gravel, and laterite subgrade respectively.

AT 100N LOAD:

Considering 100N load at 30mm depth, the corresponding stresses for quarry granite, natural gravel and laterite subgrade as shown in Figures 3.22, 3.23, and 3.24 respectively were 3.1 x10^{-3}N/mm^2, 2.7 x 10^{-3}N/mm^2 and 3.3 x 10^{-3}N/mm^2.

This showed that the quarry granite and the laterite subgrade were able to withstand more stresses at that depth that the natural gravel.

Hence quarry granite and laterite subgrade are better materials than the natural gravel. Similarly, at 60mm depth, the corresponding stresses for quarry granite, natural gravel and laterite subgrade were 2.7 $x10^{-3}N/mm^2$, 2.2 $x10^{-3}N/mm^2$and 3.25 $x\ 10^{-3}N/mm^2$ respectively as in Figures 3.22, 3.23, and 3.24 respectively.
This showed that quarry granite and laterite subgrade were better than natural gravel because they could withstand more stresses.

AT 500N LOAD:

At 30mm depth, the corresponding stresses were 3.0 $x10^{-3}N/mm^2$, 2.6 $x10^{-3}N/mm^2$ and 2.9 $x\ 10^{-3}N/mm^2$ for quarry granite, natural gravel and laterite subgrade as shown in Figures 3.25, 3.26, and 3.27 respectively. This showed that quarry granite and laterite are better materials than natural gravel because they could withstand more stresses than the natural gravel. Similarly at 60mm depth the corresponding stresses were 2.4 $x10^{-3}N/mm^2$, 1.8 $x10^{-3}N/mm^2$ and 2.6 $x\ 10^{-3}N/mm^2$ for quarry granite, natural gravel and laterite subgrade respectively as shown in Figures 3.25, 3.26, 3.27. This showed that quarry granite and laterite subgrade could withstand more stresses than natural gravel.

AT 1000N LOAD

Considering figures 3.28, 3.29 and 3.30 for quarry granite, natural gravel and laterite subgrade respectively at 30mm depth, the stresses corresponding to this depth were 2.85 $x\ 10^{-3}N/mm^2$,2.75 $x10^{-3}N/mm^2$ and 2.9 $x10^{-3}N/mm^2$ for quarry granite and laterite subgrade respectively. This showed that quarry granite and laterite subgrade could withstand more stresses than natural gravel. This means that quarry granite and laterite subgrade are better materials than natural gravel.

Similarly, at 60mm depth the stresses corresponding to the depth were 2.45 x10 $^{3}N/mm^{2}$, 2.25 x$10^{-3}N/mm^{2}$ and 2.6510$^{-3}N/mm^{2}$ x for quarry granite, natural gravel and laterite subgrade respectively as shown in Figures 3.28, 3.29, and 3.30 respectively.

TABLE 3.11 TABLE OF COMPARISON FOR QUARRY BALLAST AT 100N WEIGHT

DEPTH Z(mm)	EXPERIMENTAL RESULTS (N/mm^2)	BOUSSINESQ (N/mm^2)	TALBOT (N/mm^2)	WINKLER (N/mm^2)
10	3.504×10^{-3}	5.310×10^{-1}	1.026×10^{-1}	5.430×10^{-2}
20	3.216×10^{-3}	4.039×10^{-1}	1.265×10^{-2}	1.358×10^{-2}
30	2.964×10^{-3}	2.098×10^{-1}	3.420×10^{-2}	6.030×10^{-3}
40	2.738×10^{-3}	1.325×10^{-1}	2.565×10^{-2}	3.393×10^{-3}
50	2.484×10^{-3}	9.135×10^{-3}	2.052×10^{-2}	2.172×10^{-3}
60	2.4×10^{-3}	1.223×10^{-3}	1.710×10^{-2}	1.509×10^{-3}

TABLE 3.12 TABLE OF COMPARISON FOR QUARRY BALLAST AT 500N WEIGHT

DEPTH Z(mm)	EXPERIMENTAL RESULTS (N/mm^2)	BOUSSINESQ (N/mm^2)	TALBOT (N/mm^2)	WINKLER (N/mm^2)
10	3.616×10^{-3}	2.656×10^{-1}	5.130×10^{-1}	2.715×10^{-1}
20	3.184×10^{-3}	2.019×10^{-1}	2.125×10^{-1}	6.785×10^{-2}
30	2.976×10^{-3}	1.049×10^{-1}	1.710×10^{-1}	3.018×10^{-2}
40	2.728×10^{-3}	6.635×10^{-2}	1.283×10^{-1}	1.697×10^{-2}
50	2.564×10^{-3}	4.569×10^{-2}	1.026×10^{-1}	1.086×10^{-2}
60	2.404×10^{-3}	3.058×10^{-2}	8.550×10^{-2}	7.540×10^{-3}

TABLE 3.13 TABLE OF COMPARISON FOR QUARRY BALLAST AT 1000N WEIGHT

DEPTH Z(mm)	EXPERIMENTAL RESULTS (N/mm^2)	BOUSSINESQ (N/mm^2)	TALBOT (N/mm^2)	WINKLER (N/mm^2)
10	3.760×10^{-3}	5.312×10^{-2}	1.0260	5.430×10^{-1}
20	3.284×10^{-3}	4.679×10^{-1}	4.266×10^{-1}	1.358×10^{-1}
30	2.964×10^{-3}	2.098×10^{-1}	3.420×10^{-1}	6.030×10^{-2}
40	2.880×10^{-3}	1.195×10^{-1}	2.566×10^{-1}	3.394×10^{-2}
50	2.724×10^{-3}	9.120×10^{-2}	2.052×10^{-1}	2.173×10^{-2}
60	2.436×10^{-3}	6.115×10^{-2}	1.710×10^{-1}	1.509×10^{-2}

TABLE 3.14 TABLE OF COMPARISON FOR SUBGRADE AT 100N WEIGHT

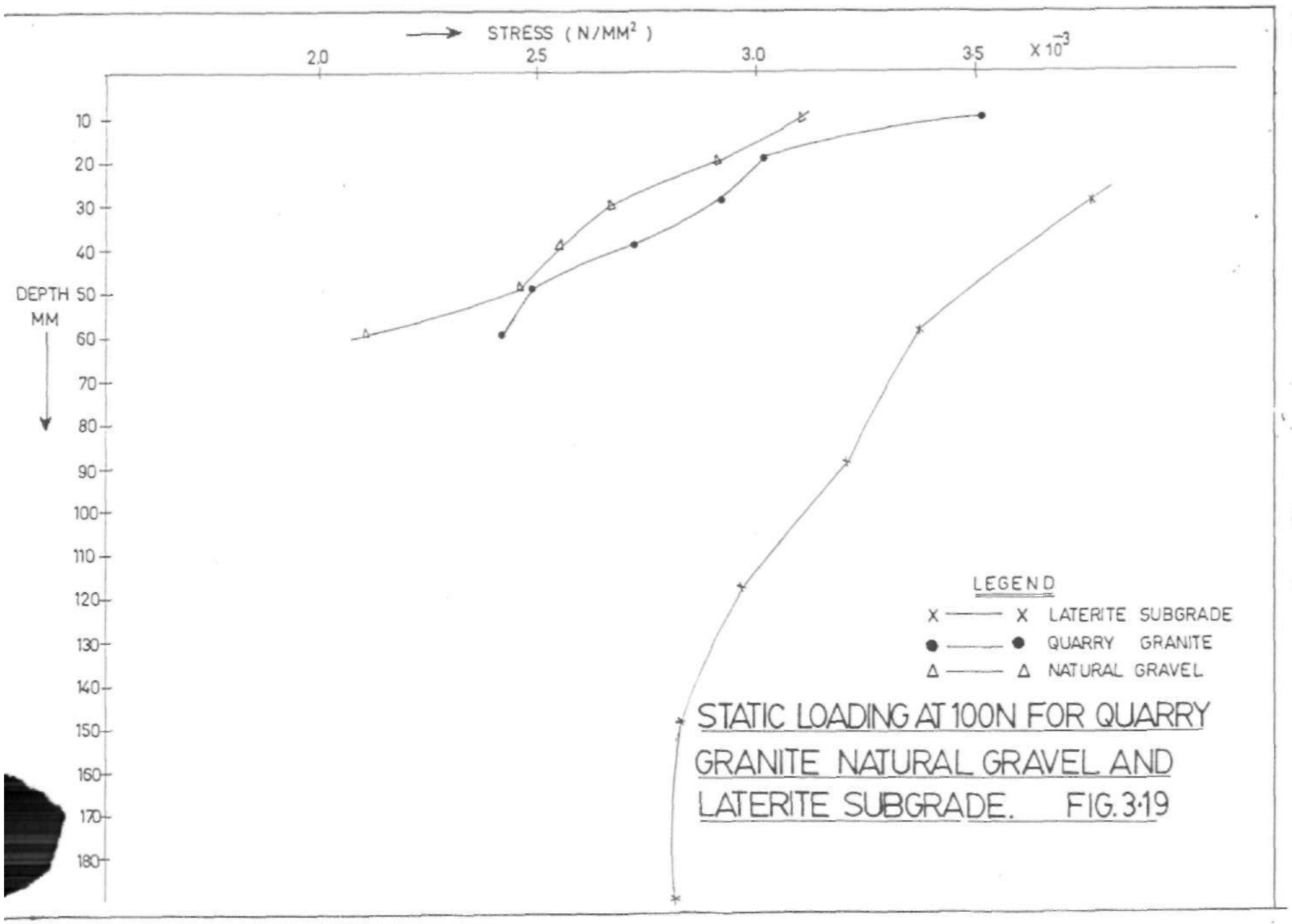
DEPTH Z(mm)	EXPERIMENTAL RESULTS (N/mm^2)	BOUSSINESQ (N/mm^2)	TALBOT (N/mm^2)	WINKLER (N/mm^2)
30	3.796×10^{-3}	2.098×10^{-2}	3.420×10^{-2}	6.030×10^{-2}
60	3.382×10^{-3}	6.115×10^{-3}	1.710×10^{-2}	7.280×10^{-2}
90	3.203×10^{-3}	2.812×10^{-3}	1.140×10^{-2}	6.693×10^{-2}
120	2.968×10^{-3}	1.601×10^{-3}	8.550×10^{-3}	3.779×10^{-2}
150	2.838×10^{-3}	1.031×10^{-3}	6.840×10^{-3}	2.411×10^{-2}
180	2.806×10^{-3}	7.180×10^{-4}	5.700×10^{-3}	1.676×10^{-2}

TABLE 3.15 TABLE OF COMPARISON FOR LATERITE SUBGRADE ON 500N WEIGHT

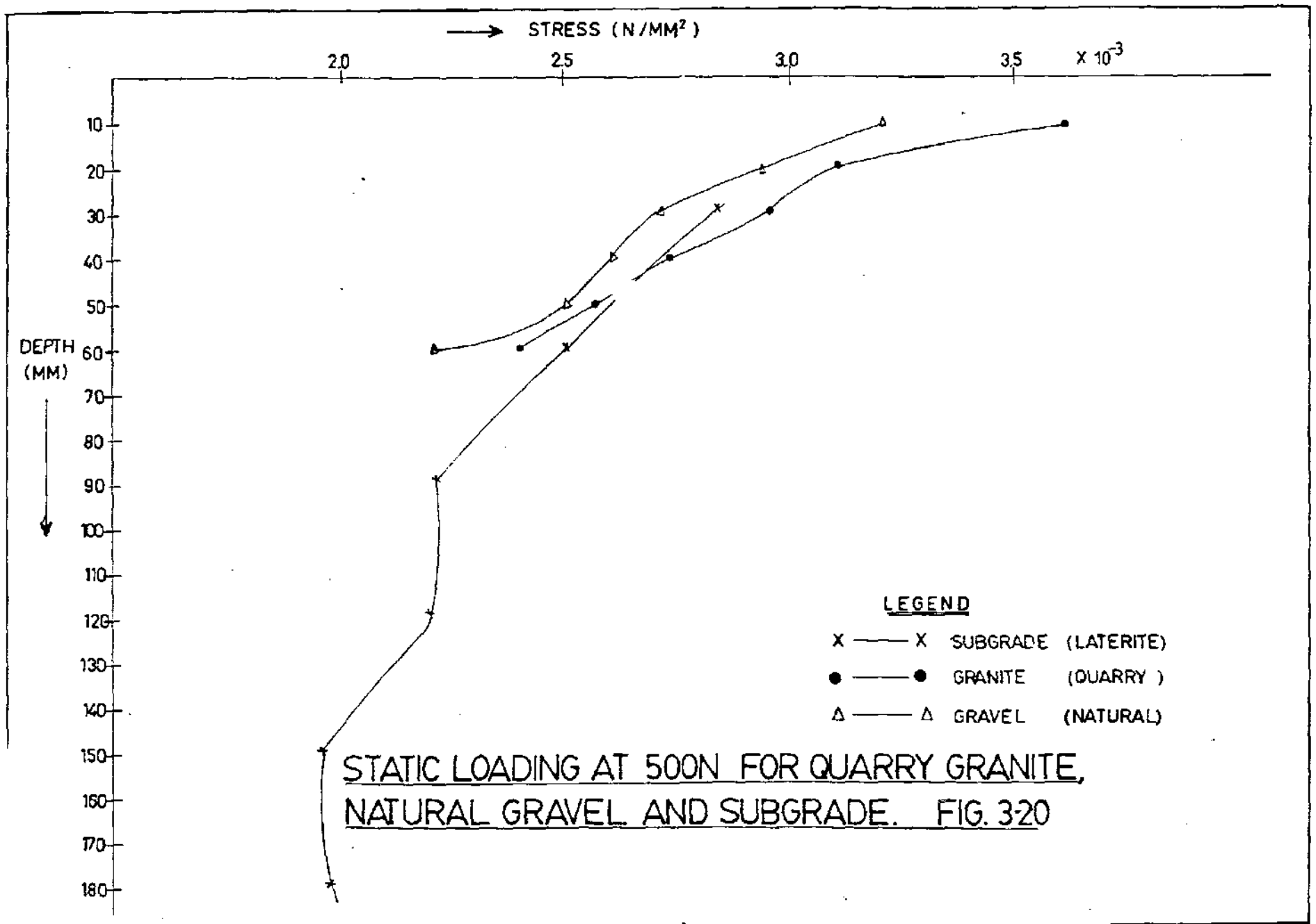
DEPTH Z(mm)	EXPERIMENTAL RESULTS (N/mm^2)	BOUSSINESQ (N/mm^2)	TALBOT (N/mm^2)	WINKLER (N/mm^2)
30	2.845×10^{-3}	1.049×10^{-1}	1.711×10^{-1}	3.015×10^{-1}
60	2.500×10^{-3}	3.058×10^{-2}	8.55×10^{-2}	7.540×10^{-2}
90	2.219×10^{-3}	1.405×10^{-2}	5.700×10^{-3}	3.351×10^{-2}
120	2.014×10^{-3}	8.005×10^{-3}	4.275×10^{-2}	1.887×10^{-2}
150	1.976×10^{-3}	5.150×10^{-3}	3.420×10^{-2}	1.207×10^{-1}
180	1.976×10^{-3}	3.589×10^{-3}	2.850×10^{-2}	8.380×10^{-3}

TABLE 3.16 TABLE OF COMPARISON FOR LATERITE SUBGRADE ON 1000N WEIGHT

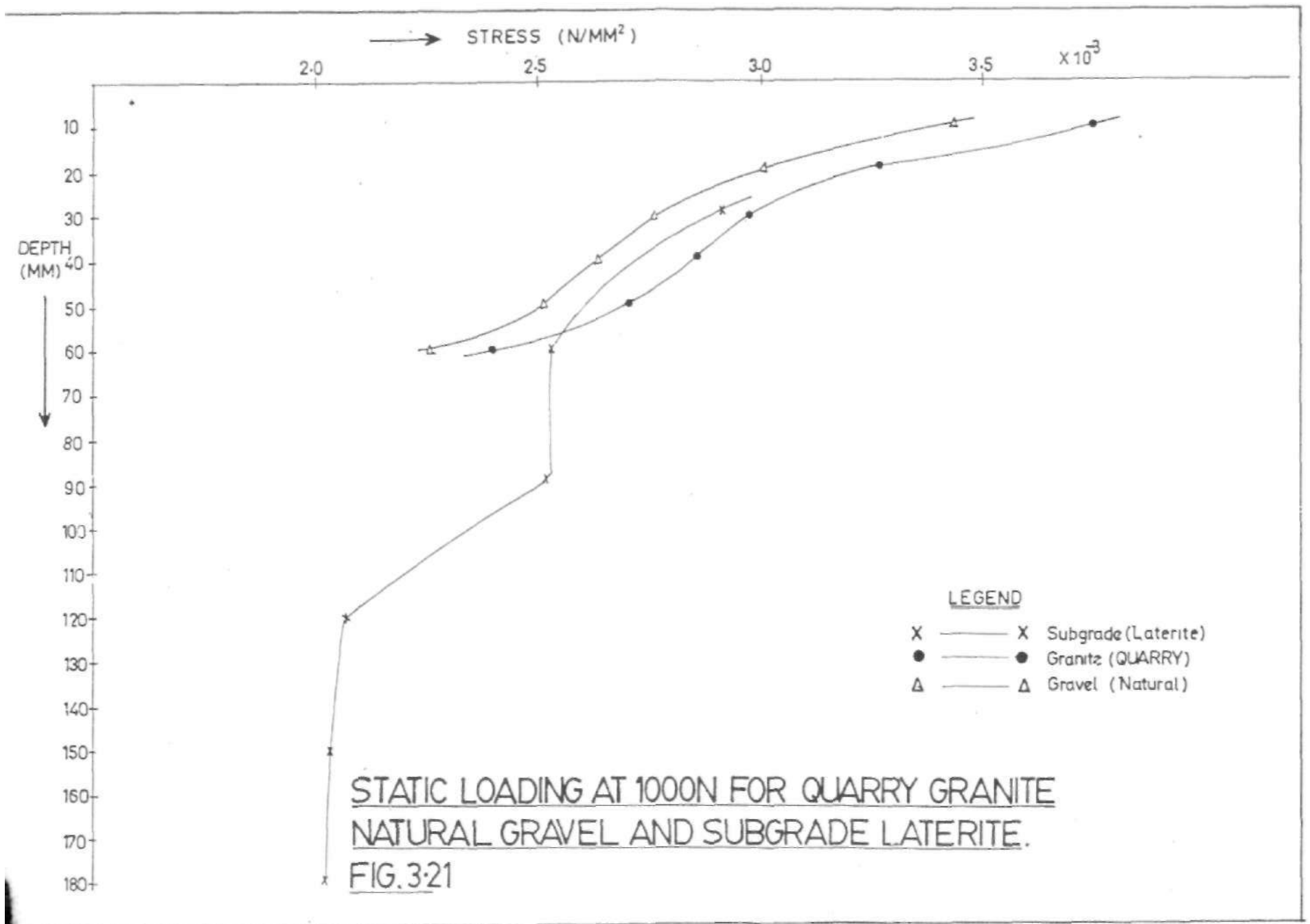
DEPTH Z(mm)	EXPERIMENTAL RESULTS (N/mm^2)	BOUSSINESQ (N/mm^2)	TALBOT (N/mm^2)	WINKLER (N/mm^2)
30	2.918×10^{-3}	2.098×10^{-1}	3.420×10^{-1}	6.030×10^{-1}
60	2.546×10^{-3}	6.115×10^{-2}	1.710×10^{-1}	1.509×10^{-1}
90	2.542×10^{-3}	2.810×10^{-3}	1.140×10^{-1}	6.709×10^{-2}
120	2.06×10^{-3}	1.600×10^{-3}	8.550×10^{-2}	3.771×10^{-2}
150	2.037×10^{-3}	1.030×10^{-3}	6.840×10^{-2}	2.413×10^{-2}
180	2.018×10^{-3}	7.175×10^{-4}	5.700×10^{-2}	1.676×10^{-2}



STATIC LOADING AT 100N FOR QUARRY GRANITE NATURAL GRAVEL AND LATERITE SUBGRADE. FIG. 3·19

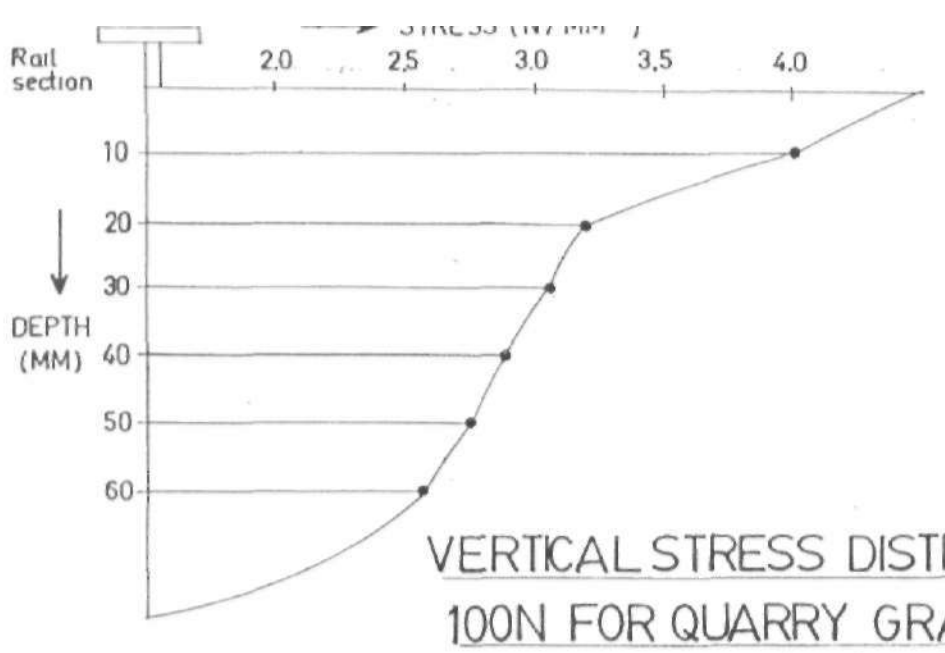


STATIC LOADING AT 500N FOR QUARRY GRANITE, NATURAL GRAVEL AND SUBGRADE. FIG. 3·20

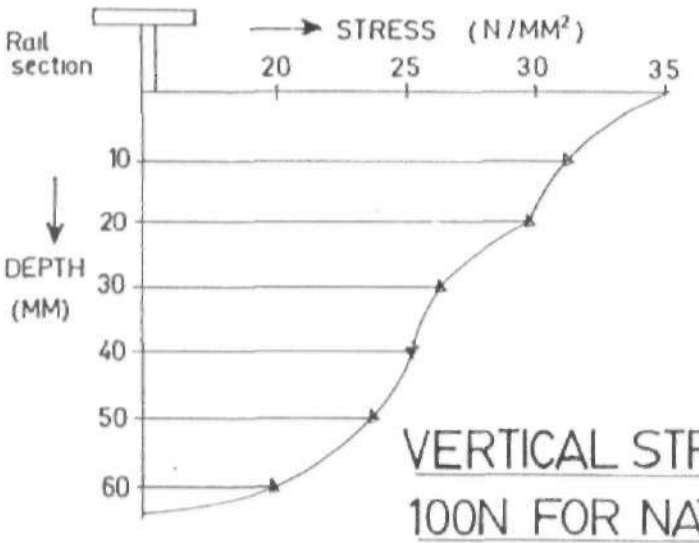


STATIC LOADING AT 1000N FOR QUARRY GRANITE NATURAL GRAVEL AND SUBGRADE LATERITE.

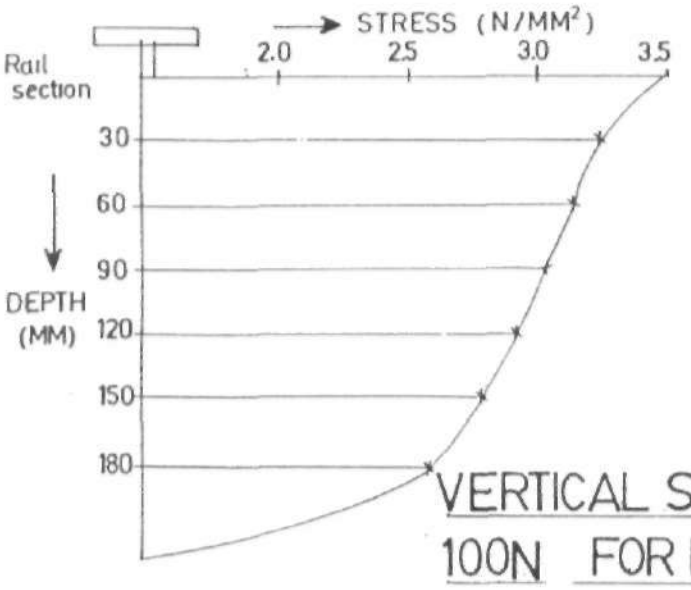
FIG. 3·21



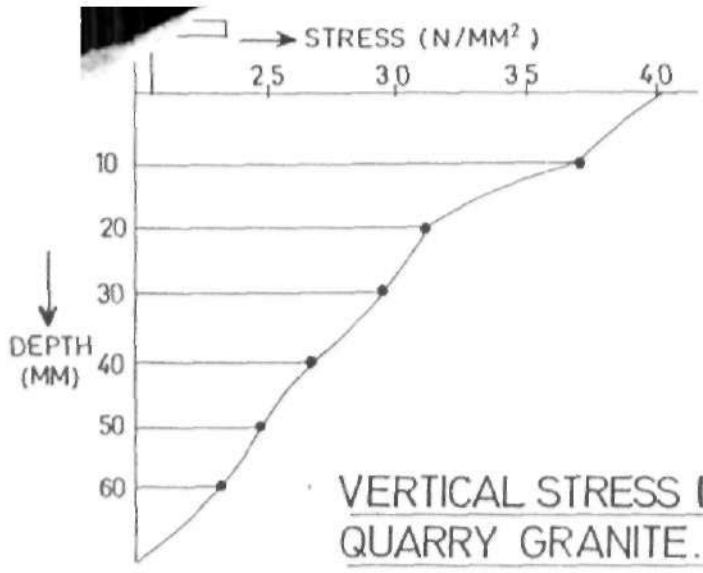
VERTICAL STRESS DISTRIBUTION AT 100N FOR QUARRY GRANITE. FIG. 3·22



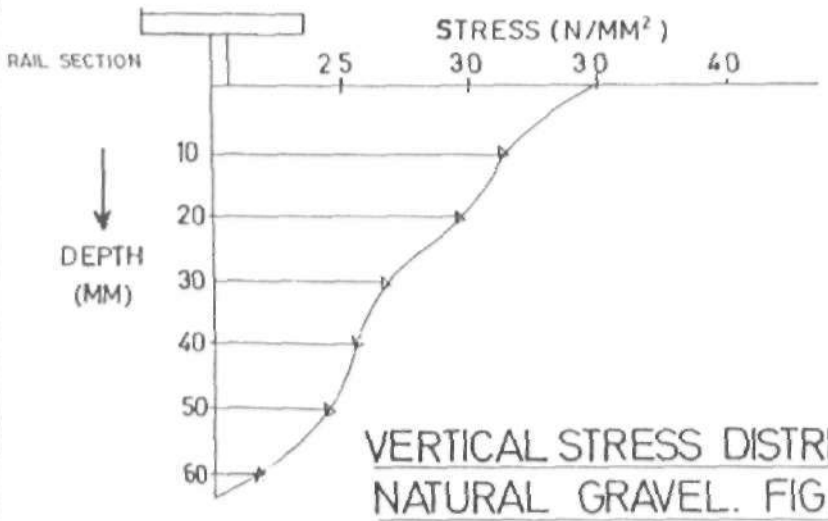
VERTICAL STRESS DISTRIBUTION AT 100N FOR NATURAL GRAVEL. FIG. 3·23



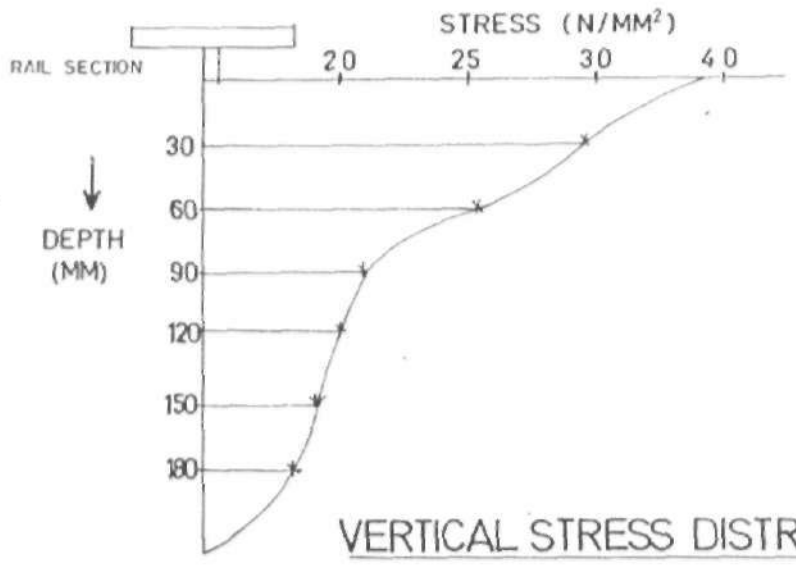
VERTICAL STRESS DISTRIBUTION AT 100N FOR LATERITE SUBGRADE. FIG. 3·24



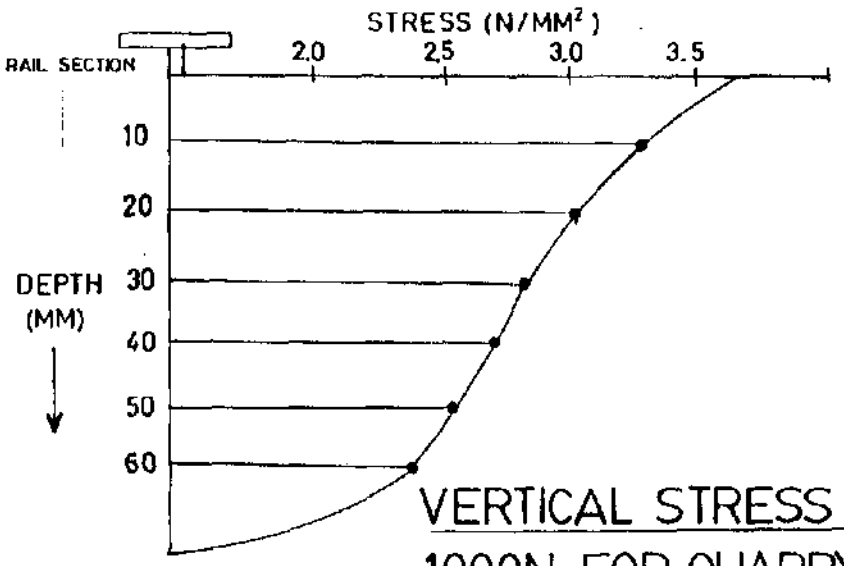
VERTICAL STRESS DISTRIBUTION AT 500N FOR QUARRY GRANITE. FIG.3·25



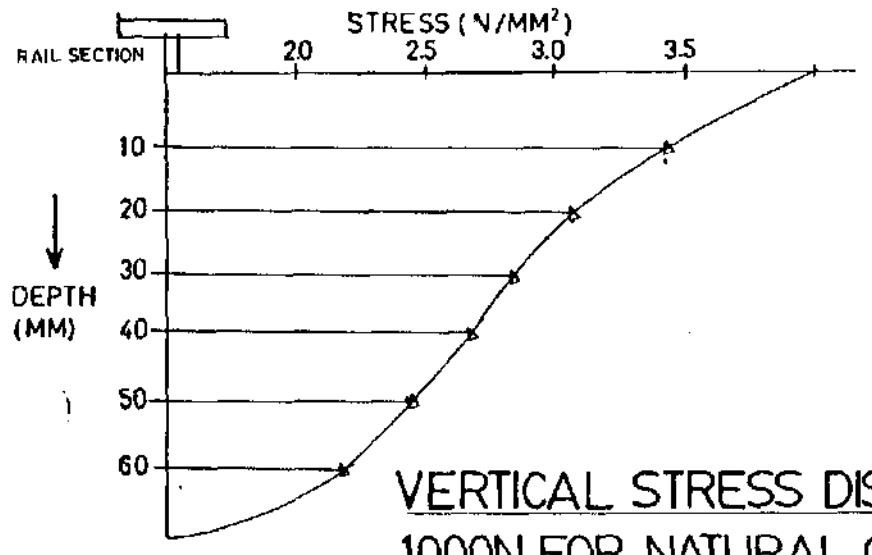
VERTICAL STRESS DISTRIBUTION AT 500N FOR NATURAL GRAVEL. FIG.3·26



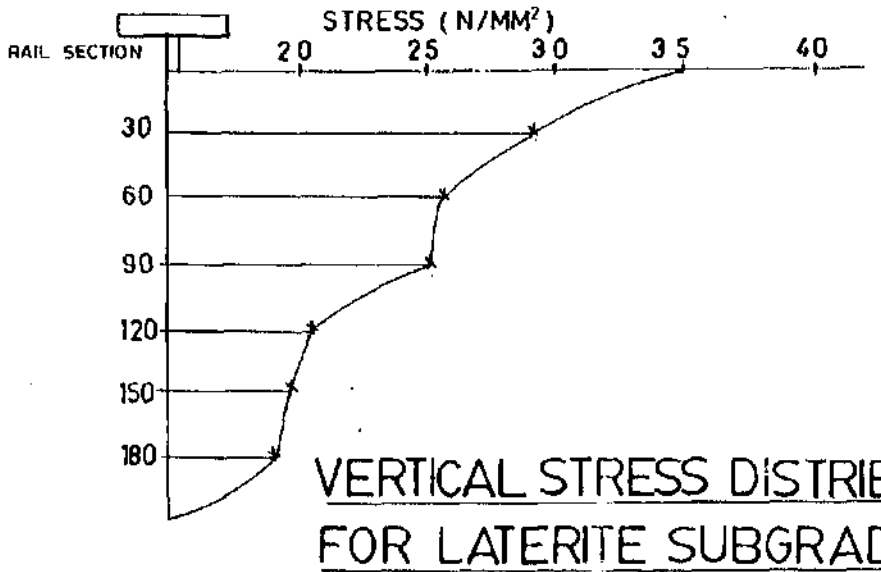
VERTICAL STRESS DISTRIBUTION AT 500N FOR LATERITE SUBGRADE. FIG.3·27



VERTICAL STRESS DISTRIBUTION AT 1000N FOR QUARRY GRANITE. FIG.3·28



VERTICAL STRESS DISTRIBUTION AT 1000N FOR NATURAL GRAVEL. FIG.3·29



VERTICAL STRESS DISTRIBUTION AT 1000N FOR LATERITE SUBGRADE. FIG.3·30

CHAPTER FOUR

4.1 CONCLUSION

The following deductions were made from the study carried out

(I) Quarry Granite was regarded as better ballast to Natural Gravel judging from the characteristic test results summarised in Table 2.1.

(II) The stresses generated in ballast were higher than that of subgrade because the load was felt first on the ballast and later on the Subgrade. This is shown on Tables 3.7 and 3.8, for ballast, while Tables 3.9 to 3.11 show the values of the stresses generated in laterite.

(III) The values of the stresses generated by the theoretical equations were slightly higher than the experimental results because the theoretical equations were only empirical in orientation.

(I) The order of preference of the materials for the use railway is quarry granite, laterite subgrade and natural gravel as shown in Figures 3.20 to 3.30.

4.2 RECOMMENDATION

I hereby recommend that the research be continued under the dynamically loaded wooden ties and the results compared with the statically loaded condition.

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APPENDICES

STATIC LOADING FOR GRANITE BALLAST

(STANDARD PROCTOR TEST)

DEPTH Z(mm)	100N	200N	300N	400N	500N	600N	700N	800N	900N	1OO0N
	(MICRO – STRAIN) x 10^{-6}									
10	8760	8840	8990	9000	9040	9140	9170	9200	9240	9400
20	8040	7740	7910	8100	7960	8060	7990	8210	8260	8210
30	7410	7400	7420	7440	7440	7450	7410	7410	7395	7410
40	6820	6820	6820	6820	6820	7000	7010	7060	7150	7200
50	6210	6230	6260	6310	6410	6440	6510	6540	6600	6810
60	6000	6010	6010	6010	6010	6015	6020	6030	6060	6090

(E = Young's modulus for Granite 4 x 10^{-8}N/mm^2)

STATIC LOADING FOR GRAVEL BALLAST

(STANDARD PROCTOR TEST)

DEPTH Z(mm)	100N	200N	300N	400N	500N	600N	700N	800N	900N	1OO0N
	(MICRO – STRAIN) x 10^{-6}									
10	7830	7910	7890	8010	8160	8190	8240	8360	8440	8610
20	7320	7380	7405	7490	7390	7410	7410	7480	7590	7600
30	6700	6710	6705	6800	6820	6820	6840	6815	6880	6900
40	6390	6400	6440	6510	6540	6530	6550	6600	6620	6680
50	6220	6210	6200	6260	6310	6290	6290	6290	6290	6290
60	5440	6455	5530	5540	5550	5560	5600	5630	5630	5640

(E = Young's modulus for Gravel 4 x 10^{-8}N/mm^2)

STATIC LOADING FOR SUBGRADE AT OPTIMUM

(STANDARD PROCTOR TEST)

DEPTH Z(mm)	100N	200N	300N	400N	500N	600N	700N	800N	900N	1OO0N
	(MICRO – STRAIN) x 10^{-6}									
30	9770	9760	9790	9810	9910	9930	9960	9980	9985	9990
40	8810	8720	8830	8740	8910	8910	8750	8890	8895	8900
90	8320	8410	8415	8430	8420	8410	8415	8410	8420	8430
120	7840	7640	7660	7780	7690	7700	7810	7780	7810	7810
150	7390	7390	7375	7440	7460	7480	7490	7460	7460	7465
180	7140	7195	7195	7210	7215	7230	7300	7310	7380	7385

(E = Young's modulus for Laterite 3.83 x 10^{-8}N/mm^2)