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# Laboratory Performance of Railroad Ballast

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**SUMMARY.** Vibration, one dimensional repeated loading and standard triaxial test results on a railroad ballast are presented. The vibration tests were performed in an oedometer using independently variable vertical amplitudes, frequencies and surcharge pressures. The repeated load tests were also performed in an oedometer to study the influence of initial density, loading intensity and ballast grading. The main properties obtained from the above tests are the vibration characteristics, strength, stress-strain characteristics, density and disintegration of the ballast. A more complete study of the soil layer interaction when placed in the form of a track support has also been started.

## 1 INTRODUCTION

The replacement and upkeep of fills and tracks are estimated to cost Canadian Railroads, which are predominantly freight orientated, about \$100 million per annum. To this must be added the other costs such as derailments caused by deteriorated track support. About 40% of the above sum is spent on ballast. For example the Canadian National Railways Company which owns about half of Canada's rail tracks, spends about \$9 million to purchase and haul new ballast and an additional \$12 million to place it in the track and maintain existing ballast (Peckover, 1973). The general waste on track maintenance due to geotechnical problems is therefore very large.

The most favoured method of placing ballast at present is by mechanical means. When the ballast is placed it is generally compacted mechanically by means of vibration. Little thought appears to have been given to the amplitude or the frequency of these vibrations. Generally ballasts, like most soils used in construction, will not be transported great distances and will therefore tend to vary in gradation every few miles along a railroad. Where ballast is to be transported large distances it is important that the best ballast available is used. Thus it is essential to have some means of measuring the geotechnical characteristics of the ballast and sub-ballast.

Most railroad fills are made up of differently graded layers of soil. As a train travels along the tracks it sets up vibrations which are transmitted into the fill. These vibrations may cause degradation of the ballast resulting in a decrease of the permeability of the ballast and making the ballast susceptible to frost action.

These problems have been the subject of an investigation which is partially described herein.

While the present Canadian Railroads are mainly freight carriers it is worth noting that as high speed passenger trains are developed and put into service the miles of track per vehicle increase and the track's technical and financial requirements tend to dominate. On new high speed passenger systems it may be expected that 60% of the capital costs will go on the track and 75% of

the operating costs will be due to track maintenance. This is very evident from the performance of Japan's 200 kilometers per hour Tokaido line between Tokyo and Osaka. For the presently available trains of over 300 kilometers per hour present methods of track maintenance are inadequate. Despite this little research monies are being spent on track design in comparison with control systems, switching, guidance systems, etc.

## 2 BALLAST CHARACTERISTIC TESTS

Samples of ten different ballasts were obtained from the Canadian National Railways Company. These ballasts have been subjected at Queen's University to the following tests:

Bulk Specific Gravity	(ASTM C-127-68)
Apparent Specific Gravity	(ASTM C-127-68)
Los Angeles Abrasion *	(ASTM C-131-69)
Absorption *	(ASTM C-127-68)
Soundness (a) Magnesium Sulphate	
(b) Sodium Sulphate *	(ASTM C-88-71 a)
Flakiness Index *	(B.S. 812:1967)
Crushing Value	(B.S. 812:1967)
Sphericity ("Sedimentary Rocks" F. Pettijohn)	
Roundness ("Sedimentary Rocks" F. Pettijohn)	
Freeze-Thaw *	(AASHTO T 103-62)
Repeated Loading (Non Standard)	

There was generally good agreement between these results and duplicate results obtained by the Canadian National Railways Company marked by an asterisk. One exception was the results of the freeze-thaw tests where, although the ballasts were rated in the same order, the numerical values were different. Further freeze-thaw tests to investigate the reasons for this difference are under way.

Field performance ratings were assigned to the ballasts based on field tests performed by the Canadian National Railways Company (Dalton, 1973). They ranged from 1 for excellent performance to 6 for very poor performance. Correlation of the laboratory results and field performance ratings was performed using a linear regression analysis. These correlation analyses showed that the field performance rating could be forecast from the results of the following standard tests.

- (i) Soundness ( $\text{Na}_2\text{SO}_4$ ) (SS)
- (ii) Freeze-Thaw (FT)

The relationship is:

$$\text{Field Index} = 0.89 + 1.66 \text{ SS} + 0.26 \text{ FT} \quad (1)$$

The analysis also showed the correlation between the different standard tests as follows:

- (a) A good correlation (99%) was found between
  - (i) Freeze-thaw, Soundness ( $\text{MgSO}_4$ ) and Absorption
  - (ii) Crushing Value and Los Angeles Abrasion
  - (iii) Specific Gravity and Soundness ( $\text{Na}_2\text{SO}_4$ )
  - (iv) Elongation and Flakiness Index
- (b) A poor correlation between the two soundness tests.

The significance of these correlations is that it may only be necessary to perform fewer of the standard tests to obtain the essential information for rail track performance.

### 3 ONE DIMENSIONAL REPEATED LOADING TESTS

A series of large size repeated loading pneumatic oedometer tests were carried out on Coteau Dolomite Ballast. One set was performed on the ballast at grading 1, shown in Fig. 1, the second set at grading 2 and some tests at grading 3. The gradings, shown on Fig. 1, were described by Hudson's gradation modulus  $\bar{A}$  (Hudson and Waller, 1969), which becomes larger as the particles become smaller. The test program included tests performed with initial densities between 1.39 and 1.69  $\text{g}/\text{cm}^3$ , pressure ranges between 0 to 170  $\text{kN}/\text{m}^2$  and 0 to 690  $\text{kN}/\text{m}^2$ , and grading modulus  $\bar{A}$  between 1.73 and 2.76.

The deformation of the ballast is in general agreement with results reported by O.R.E. (1970). A statistical analysis of the results has been completed and the behaviour of the Coteau Dolomite ballast can be summarized as follows:

- (a) The total vertical strain increased with increasing number of load cycles, varying from -1.1% to +21% after 100,000 cycles. An approximately linear relationship was observed between the strain after the first load cycle and the logarithm of the number of load cycles. Two strain paths after the first cycle are shown as an example in Fig. 2.
- (b) The total vertical strain increased as the initial density decreased and the cycled pressure increased but was independent of the grading. The initial density had a much greater effect than the cycled pressure. The relationship between total vertical strain, initial density and cycled pressure is shown in Fig. 3.
- (c) The strain after the first cycle ranged from -3.9% to +13.9%. It increased as the initial density decreased but was independent of the cycled pressure and the grading. The relationship is shown in Fig. 4.
- (d) The total breakdown ranged from 0.09% to 2.23%. It increased as the cycled pressure and gradation modulus  $\bar{A}$  increased but was independent of the initial density. The relationship is shown in Fig. 5.

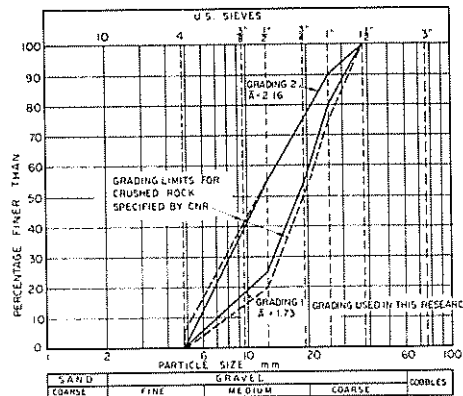


Fig. 1 BALLAST GRADING CURVES

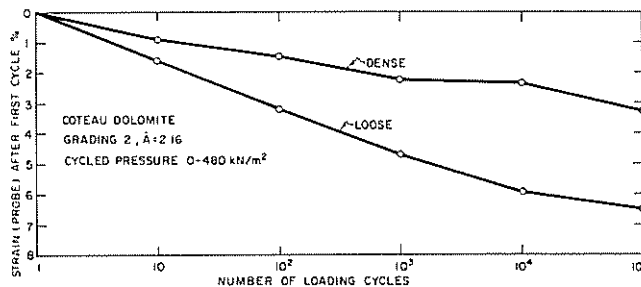


Fig. 2 STRAIN AGAINST NUMBER OF LOADING CYCLES

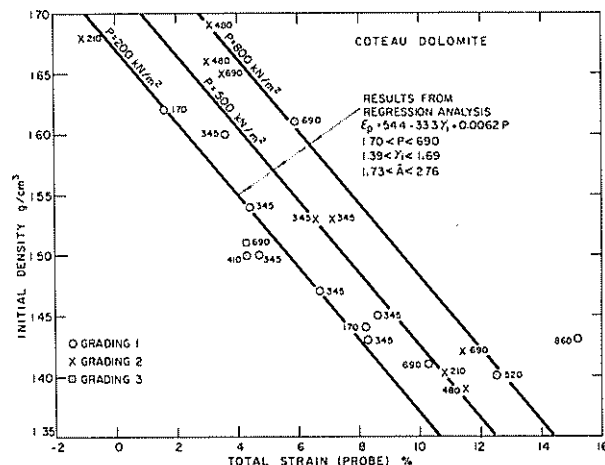


Fig. 3 STRAIN, DENSITY AND CYCLED PRESSURE

From these results it can be concluded that:

- (i) The ballast should be placed at the maximum obtainable density in order to reduce the vertical deformation.
- (ii) The ballast should be as coarse as allowable in order to reduce the amount of breakdown.

A third set of tests was carried out on samples of ten different ballasts made up to grading 1 and tested at their minimum density with a cycled pressure of 0-690  $\text{kN}/\text{m}^2$  for 100,000 cycles. The results have been analysed and compared, using a linear regression, with the results of the Canadian National Railways Company field tests and the results of the standard ballast tests. The behaviour of the different ballasts is summarized below.

- (a) The total vertical strain ranged from 8.2% for the Sudbury Slag to 20.1% for the St. Marc Limestone. There was no correlation between any measure of vertical strain and the standard ballast test results.

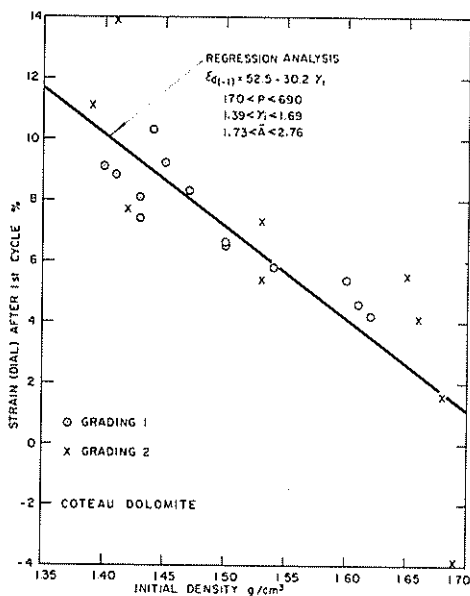


Fig. 4 STRAIN AFTER 1st CYCLE  
IN REPEATED LOADING TEST

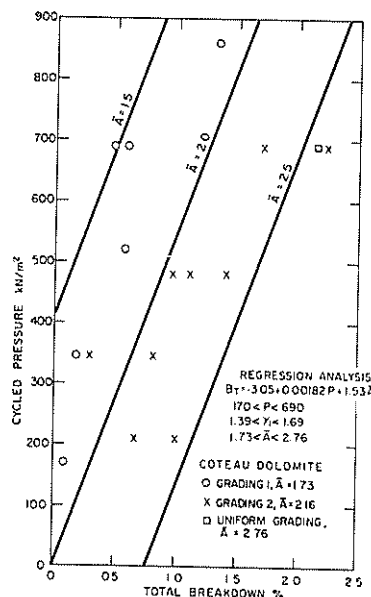


Fig. 5 REPEATED LOADING  
TOTAL BREAKDOWN

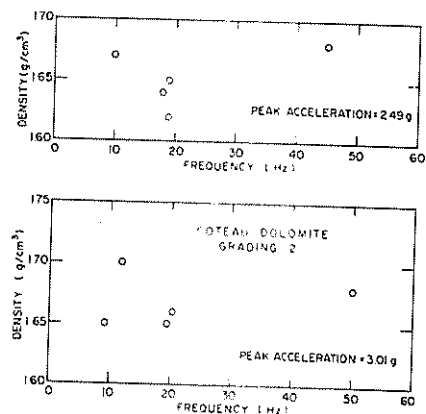


Fig. 6 VIBRATION TEST RESULTS  
DENSITY vs FREQUENCY

- (b) The total breakdown ranged from 0.61% for one sample of Coteau Dolomite to 3.71% for the St. Marc Limestone. There was quite good agreement between the breakdown in the laboratory repeated loading test and that reported in the Canadian National Railways Company field test. Good individual correlation was found between total breakdown and Crushing Value, Los Angeles Abrasion and Roundness. The correlation was best with the Crushing Value. The individual correlations are

$$B_T = -1.55 + 0.151 \text{ Crushing Value} \quad (\text{Corr. Coeff. } 0.786)$$

$$B_T = -2.12 + 0.0771 \text{ Los Angeles Abrasion} \quad (\text{Corr. Coeff. } 0.650)$$

$$B_T = 0.211 + 0.581 \text{ Roundness} \quad (\text{Corr. Coeff. } 0.636)$$

From these results it can be concluded that

- (i) None of the standard classification tests used enabled a prediction of measured vertical strain under repeated loading to be forecast with any degree of reliability.
- (ii) The Crushing Value and Los Angeles Abrasion give a good indication of the breakdown under field loading conditions (but not necessarily under field weathering conditions).

#### 4 ONE DIMENSIONAL VIBRATION TEST

Vibration tests were carried out on Coteau Dolomite, grading 2, to investigate the effect of frequency on density. The tests were performed in a large size pneumatic oedometer mounted on a vibrating table. In the tests the frequency was varied while the peak acceleration was kept constant. Three values of peak acceleration were chosen. The results are plotted in Fig. 6. The plot shows that a change in frequency had no effect on the density.

Another set of vibration tests were carried out on Coteau Dolomite, gradings 1 and 2. In these tests the frequency varied from 7.5 to 63 Hz and the peak acceleration from 0.77 to 4.81 g.

The results are similar to those reported for sand by Selig (1963), Vierig (1965), Prakash and Gupta (1967) D'Appolonia and D'Appolonia (1967), and Whitman et al (1967, 1968).

The behaviour of Coteau Dolomite ballast under vertical vibrations is described below.

- (a) Typical density against peak acceleration results are shown in Fig. 7. In general, the density of the ballast remained unchanged as the peak acceleration increased from zero to about 1.25 g. Above 1.25 g the density increased rapidly and then began to level off at a peak acceleration of about 2 g. Above 2 g the density increased only slightly.
- (b) The increase in density with peak acceleration was unaffected by surcharge pressures of up to 345 kN/m<sup>2</sup>. Higher surcharge pressures of 520 and 690 kN/m<sup>2</sup> caused a decrease in final density, especially for grading 1.
- (c) The shape of the density/peak acceleration

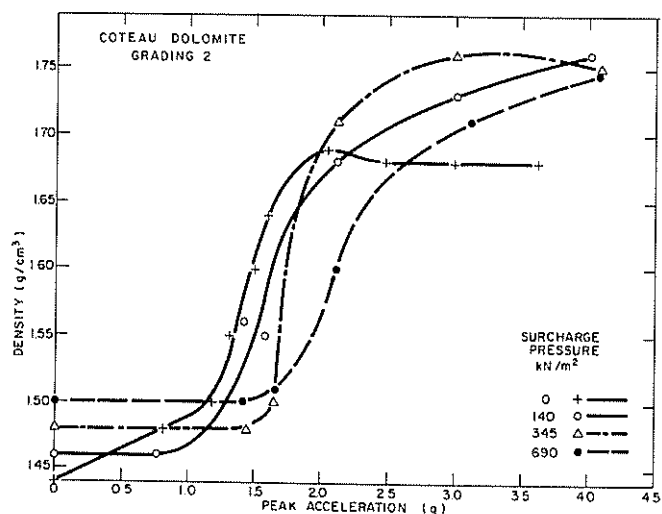


Fig. 7 EFFECT OF PRESSURE ON DENSIFICATION

curve was similar for both gradings although the values of density were slightly different.

- (d) The breakdown measured showed an increase as the peak acceleration and surcharge pressure increased. The breakdown varied from 0.35% at zero surcharge pressure to 1.98% at 520 kN/m<sup>2</sup>.

From these results it can be concluded that

- (i) Acceleration of 2 g must be applied to soils to produce substantial increases in density.
- (ii) A surcharge pressure is of little significance and has an optimum value between 150 to 350 kN/m<sup>2</sup> with a decrease in pressure being more beneficial than an increase in pressure.
- (iii) The frequency of vibration has little effect on the final density.

## 5 STANDARD TRIAXIAL TESTING OF COTEAU DOLOMITE

Standard triaxial tests have been performed on Coteau Dolomite Ballast at gradings 1 and 2 (Fig. 1). Specimens 0.225 m dia. by 0.45 m high were prepared at densities varying from 1.4 g/cm<sup>3</sup> to 1.7 g/cm<sup>3</sup>.

The standard tests performed on the ballast were constant strain rate, saturated, drained tests. Volume change was continually monitored using a large burette connected to the bottom of the specimen, and the strain rate was slow enough to prevent any pore pressure build up. From the results the initial tangent moduli and pseudo Poisson's ratio may be obtained for use in some form of mathematical modelling. These properties are shown in Figs. 8 and 9.

Strength envelopes based on Coulomb's failure criterion were plotted using Mohr's stress circles. Assuming an unique failure envelope for each stress state and zero cohesion for each failure envelope, the angle of internal friction  $\phi'$  at failure, of any triaxial test specimen, is the slope of the tangent through the origin. This gave an angle of internal friction which varied with the stress intensity. A plot of internal friction angle against log normal stress across the failure plane for each of the densities used gave results which were parallel to the average rockfill line suggested by Leps (1970). Thus the friction angle decreases with increasing normal stress as reported by Leps. Results for the dense samples fell above Leps' average line while results for the loose samples fell below his average line.

The triaxial testing is being extended to include the study of stress-strain relationships for samples failed in extension tests and for samples subjected to repeated loading.

## 6 FULL SCALE MODEL TESTS

Two full scale tests have been performed. The first was performed on a six metres length of rail track consisting of eleven standard wood ties 0.2 m deep by 0.225 m wide and 2.4 m long spaced at 0.5 m centre to centre with 78 kg/m rail. The ties were placed on 0.3 m of ballast underlain by 0.3 m of sub-ballast and 0.6 m of sand. The rails above the two outer ties working toward the centre were first preloaded with 268 kN (i.e. 4 point loads of 67 kN each). Then the model rail was subject to a centrally placed stationary repeated

load of 268 kN applied for one million cycles followed by a 150 thousand cycles of 400 kN load. The load was shared equally by each rail above the central wood tie. The maximum settlements were recorded on the centreline of the central tie and are shown in Fig. 10 for various stages of the loading programme. From these results it is obvious that the cause of the major settlements are due to repeated loading conditions and that one cycle of static loading, at the present time at least, gives little indication of the final performance.

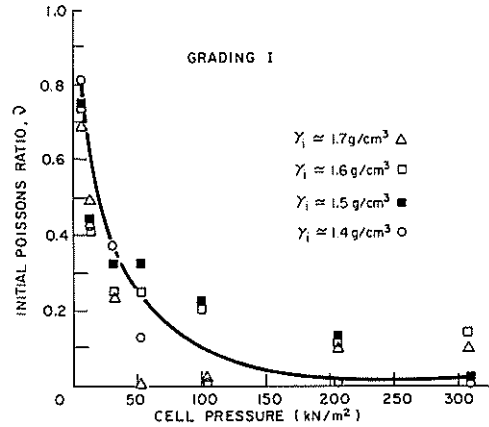


Fig. 8 TRIAXIAL VALUES OF POISSON'S RATIO

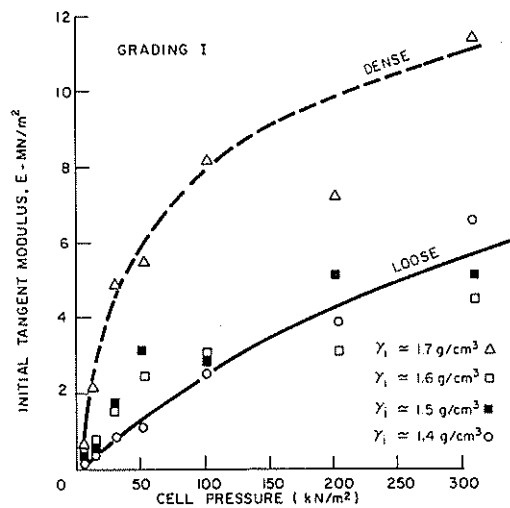


Fig. 9 TRIAXIAL VALUES OF TANGENT MODULUS

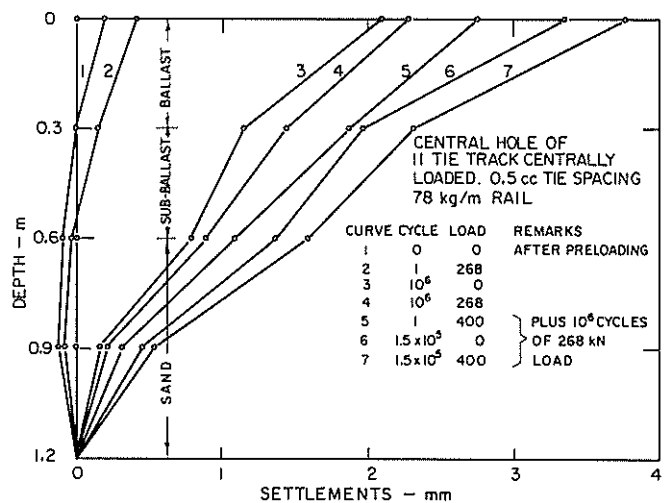


Fig. 10 MODEL TRACK REPEATED LOAD TEST

The second test which has only commenced has been the loading of a single tie on a similarly made up fill. So far only one cycle of load to 178 kN load has been applied. These results for the central location, which gave the major settlement readings, are shown in Fig. 11 and indicate the imminent failure of the single tie. Obviously, during the first test the rail stiffness had a major effect in spreading the load. This point has already been noted by railroad companies since rail weights and consequently stiffness have gradually increased over the years. Tie sizes however appear to have changed very little. Present emphasis is thus to concentrate on the effect of the tie since the failure of a single cycle load will increase as the square of the width and using data from Bond (1961) for any given single cycle intensity of loading the settlement will decrease in proportion to the square of the width. Increasing the tie width may thus be more beneficial than increasing the rail weight per given increase in cost.

## 7 CONCLUSIONS

Results so far indicate the following tentative conclusions.

- (a) A rough ballast performance to weathering and degradation could be predicted from two of the standard laboratory tests. These are -
  - (i) Freeze-Thaw
  - (ii) Soundness using Sodium Sulphate.
- (b) Ballast should be placed at the maximum density obtainable and should be as coarse as allowable within the range studied. The grading had an effect on breakdown.
- (c) None of the standard classification tests used enabled a prediction of measured vertical strain under repeated loading to be forecast with any degree of reliability.
- (d) The Crushing Value and to a less extent the Los Angeles Abrasion Value give a good indication of ballast breakdown under loading but not weathering.
- (e) Accelerations of at least 2 g must be applied to ballast to produce a substantial increase in density. A surcharge pressure and the frequency of vibration has little beneficial effect.
- (f) Full scale results are so far inconclusive but suggest further investigation of increasing tie width as a means of improving rail track performance.

## 8 ACKNOWLEDGEMENT

The work presented forms part of a general study on geotechnical problems of railroad structures and fills being financed under Project 2.22 of the Canadian Institute of Guided Ground Transport at Queen's University. Also acknowledged are discussions, advice and help obtained from the personnel from the Canadian National Railways and Canadian Pacific Railways. In particular, Mr. L. Peckover, Mr. C. Dalton and Dr. N. Caldwell.

## 9 REFERENCES

DALTON, C.J. 1973. "Field Durability Tests on Ballast Samples as a Guide to the Significance of the Specification Requirements". Canadian

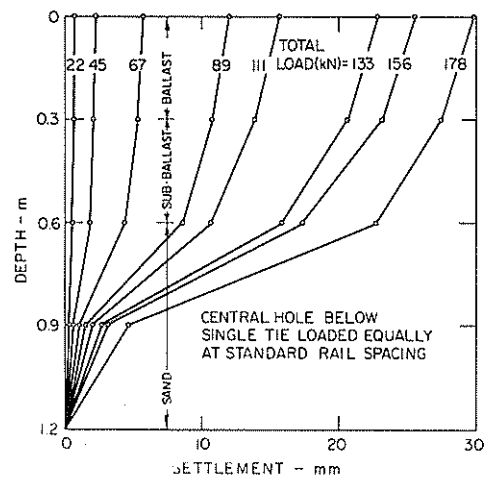


Fig. 11 SINGLE TIE STATIC LOAD TEST

National Railways, Technical Research Centre Publication, p. 40.

- D'APPOLONIA, D.J. and D'APPOLONIA, E. 1967. "Determination of the Maximum Density of Cohesionless Soils". Proc. 3rd Asian Reg. Conf. Soil Mech. Found. Engrg., Haifa, pp. 266-268.
- HUDSON, S.B. and WALLER, H.F. 1969. "Evaluation of Construction Control Procedures - Aggregate Gradation Variations and Effects". National Co-operative Highway Research Program Report 69, Chapter 2.
- LEPS, T.M. 1970. "Review of Shearing Strength of Rockfill". J. Soil Mech. Found. Div., Proc. A.S.C.E. Vol 96, No (SM4), pp. 1159-1170.
- OFFICE FOR RESEARCH AND EXPERIMENTS OF THE INTERNATIONAL UNION OF RAILWAYS (O.R.E.) 1970. "Stresses in the Rails, the Ballast and in the Formation Resulting from Traffic Loads". Question D 71, Report No. 10. (Vol. 1, p. 39, Vol 2, Tables and Fig.).
- PECKOVER, F.L. 1973. "Railway Ballast Material". Canadian National Railways, Office of the Chief Engineer, p. 97.
- PETTILJOHN, F.J. 1949. "Sedimentary Rocks". Harper and Brothers.
- PRAKASH, S. and GUPTA, M.K. 1967. "Compaction of Sand under Vertical and Horizontal Vibration". Proc. S.E. Asian Conf. on Soil Mech. Found. Engrg. pp. 201-209.
- SELIG, E.T. 1963. "The Effect of Vibration on the Density of Sand". Proc. 2nd. Panam. Conf. Soil Mech. Found. Engrg., Sao Paulo, Brazil, Vol. 1, pp. 129-144.
- VIERIG, G. 1965. "The Vibrational Behaviour of Soil in Relation to its Properties". Proc. 5th Int. Conf. Soil Mech. Found. Engrg., Paris, Vol. 1, pp. 547-552.
- WHITMAN, R.J., LUSCHER, H., ORTIGOSA, P., ROCHER, K. 1967. "Repeated Load and Vibration Tests upon Sand". Progress Report No. 1, Dept. Civ. Engrg., M.I.T. Soil Publ. No. 203.
- WHITMAN, R.J. and PABLO, P.O. 1968. "Densification of Sand by Vertical Vibrations". Progress Report No. 4, Dept. Civ. Engrg., M.I.T. Soils Publ. No. 222.