dation materials research program conducted by the Transportation Research Laboratory of the Department of Civil Engineering, University of Illinois at Urbana-Champaign. The research was sponsored as a subcontract between the Research and Test Department, Association of American Railroads, and the University of Illinois. This subcontract is part of a larger contract that is a cooperative effort between the Federal Railroad Administration and the Association of American Railroads on improved track structures. This paper represents our views and positions and does not necessarily reflect those of the Federal Railroad Administration or the Association of American Railroads.

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**Dr. Tayabji was at the University of Illinois when this work was performed.*

Improvement in Rail Support

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An on-going investigation on rail support material is briefly summarized. Static and repeated-load triaxial compression and extension tests on a dolomite ballast are reported, and their significance to track design is discussed. Model tests using static and repeated loading on a small scale with Ottawa sand as a foundation material and on a large scale with rail track, ballast, subballast, and sandy subgrade were made, and the significance to tie and track design of their results is discussed.

The replacement and upkeep of fills and tracks cost Canadian railways an estimated $100 000 000 annually, of which about 40 percent is spent for the procurement, distribution, and rehabilitation of ballast. The potential savings that would accrue from research and the better use of track-support materials is therefore very large. A complete assessment of the economic importance of ballast in policies and practice, however, should include the costs of derailments and of the restricted speed and other delays caused by deteriorating track support.

The Canadian railways are at present mainly freight carriers, but as high-speed passenger trains are developed and put into service, the length of track traveled per vehicle will increase, and the technical and financial requirements of the track will tend to dominate these costs. Despite this, in comparison with the research effort devoted to such items as control systems, switching, and guidance systems, there has been little research devoted to track design. It is not surprising then that the Canadian Institute of Guided Ground Transport requested a feasibility study on the types of research that are needed in the track-support area in 1969.

The first efforts of this study were concentrated in the area of ballast selection and have been reported elsewhere (1, 2, 3, 4). This paper will discuss certain aspects of the work in relation to track design.

TRIAXIAL ONE-CYCLE TESTS OF RAILROAD BALLAST

Two properties have been found to have a dominant effect on the performance of a ballast under load. The first is the hardness of the ballast, which is sometimes difficult to assess because ballast is composed of different minerals of different hardnesses, and the second is the toughness of the ballast, which is best measured by the crushing-value test. Another property of importance is the ability of the ballast to resist chemical weathering. Four materials were selected for study, (a) a dolomite with a Mohr's hardness of 3.5 to 4, which is relatively soft, but has a crushing value of 16.4, which indicates a relatively tough material; (b) Sudbury slag, which is hard and tough; (c) Kenora granite, which is hard, but weak and brittle; and (d) marble, which is soft and also weak and brittle. Thus far, extensive testing has been restricted to the dolomite.

Immediately after the maintenance cycle, when the ballast material is in its loosest condition, the major concerns for its performance are its strength against bearing failure and its ability to withstand lateral forces. The ability to resist bearing failure can best be assessed in the laboratory by standard triaxial compression tests. The ability to withstand lateral forces, on the other hand, is best assessed by extension tests. Each of these types of tests has been performed on the dolomite ballast for a variety of different densities. Of the most interest to the track engineer is the performance of the ballast in its densest condition because the effect of the in-track compaction is to place the ballast in as dense a packing as possible. For the compression and extension tests, specimens 0.2 m (9 in) in diameter by 0.45 m (18 in) high were prepared at dry densities varying from 1.4 to 1.7 g/cm^3 (87 lb/ft^3 to 106 lb/ft^3). To ensure uniform density throughout the sample, a precalculated weight of ballast [e.g., 25 kg (55 lb) for a density of 1.4 g/cm^3] was prepared, divided into four equal parts and placed in the specimen preparation mold, which was marked in four equal 0.11-m (4.5-in) increments of height for a total of 0.45 m (18 in). The sample with a dry density of 1.7 g/cm^3 was prepared by a vibratory compaction process, again in four equal increments of height. In every test, the dimensions of the specimen were measured to verify the actual densities. The samples were saturated before removal of the molding forms.

The tests performed on the ballast were constant strain rate, saturated, drained tests. The volume changes were continually monitored by using a large burette connected to the bottom of the specimen. The strain rate was slow enough to prevent any buildup of pore pressure. A cell with a rotating bushing was used. Figure 1 compares the results obtained from compression and extension triaxial tests that used the maximum placement density that could be achieved in the laboratory. This figure shows that the material is considerably stiffer in the extension tests than in the compression tests and that the failure is higher in the compression tests. This is to be expected because in the extension tests, the cell pressure is the major principal stress, but in the compression tests, it is the minor principal stress. Thus, a direct comparison of the strength is rather misleading. Far more realistic is a comparison on a Mohr circle (Figure 2). The failure envelope from the compression tests in Figure 2a is imposed on Figure 2b. Somewhat surprisingly, the failure envelope from the extension tests is considerably less than that from the compression tests. Somewhat surprisingly, because Green (5) had found that for rounded dense sand, the angle of internal friction (ϕ) was greater in extension than in compression, but that for loose sand, there was very little difference between the two values of ϕ. In these results, ϕ is greater in compression than in extension. Green attributed the observed difference in his values of ϕ to anisotropy introduced during the setting-up period. However, the difference is more likely caused by ballast particle packing and shape that, of course, may be related to anisotropy, but is more likely due to random nonhomogeneity.

An alternative method of interpreting the Mohr circle diagrams is to consider each sample a different material and, because ballasts are granular in performance, assume them to be noncohesive. Under such circumstances, the Mohr circle would be represented by a friction angle only. Figure 3 shows a comparison of the compression and extension values of ϕ obtained by such an interpretation and an average line developed by Leps from compression tests on rock fill for earth dams (6). Most of the samples tested by Leps consisted of large-size specimens with particle sizes up to 15.2 cm (6 in). Leps tests were performed at high pressures similar to those occurring in earth dams, but ours were performed at low pressures because the height of ballast in track is small and therefore the confining pressures are small. In particular, the resistance mobilized to prevent the phenomenon known as sun kinks will occur at low confining pressures. From Figure 3, it can be seen that at low pressures, a zero cohesion intercept results in friction angles at failure that are greater than 45°. However, a value of ϕ greater than 45° is meaningless, which means that a considerable portion of the strength at low confining pressures is obtained from interlock forces. Also, crushing and the crushed faces of a ballast will be important; it is not surprising that specifications dealing with railroad ballast specify particles having a high percentage of crushed faces.

The importance of good compaction at low confining stress can be seen from Figure 4, which shows the differences in the triaxial stress at failure obtained for the dolomite ballast. At low cell pressures, the density of placement has an immense effect on the ballast failure resistance. This difference is less significant at higher confining pressures, as when the ballast is subject to the weight of a loaded train. Intermediate confining pressures would result from a lightly loaded or an unloaded train. Proper compaction and the achievement of high densities during maintenance cycles would therefore be valuable in resisting sun kinks and in maintaining the stability on curves of unloaded trains traveling at high speeds. Under heavily loaded trains the high frictional resistance will cause considerable ballast resistance, and the ballast density or good compaction will be of less immediate importance. However, this ignores the possibility of large deformations occurring in the ballast due to a large number of repetitive loads. To assess this, it becomes important to look at the performance of a ballast under repeated loading.

TRIAXIAL REPEATED-LOAD TESTS OF DOLOMITE BALLAST

Triaxial repeated-load strain tests on the dolomite ballast were carried out with different fractions of the difference in the axial stresses at static failure. In both

Figure 1. Results of stress versus strain tests.

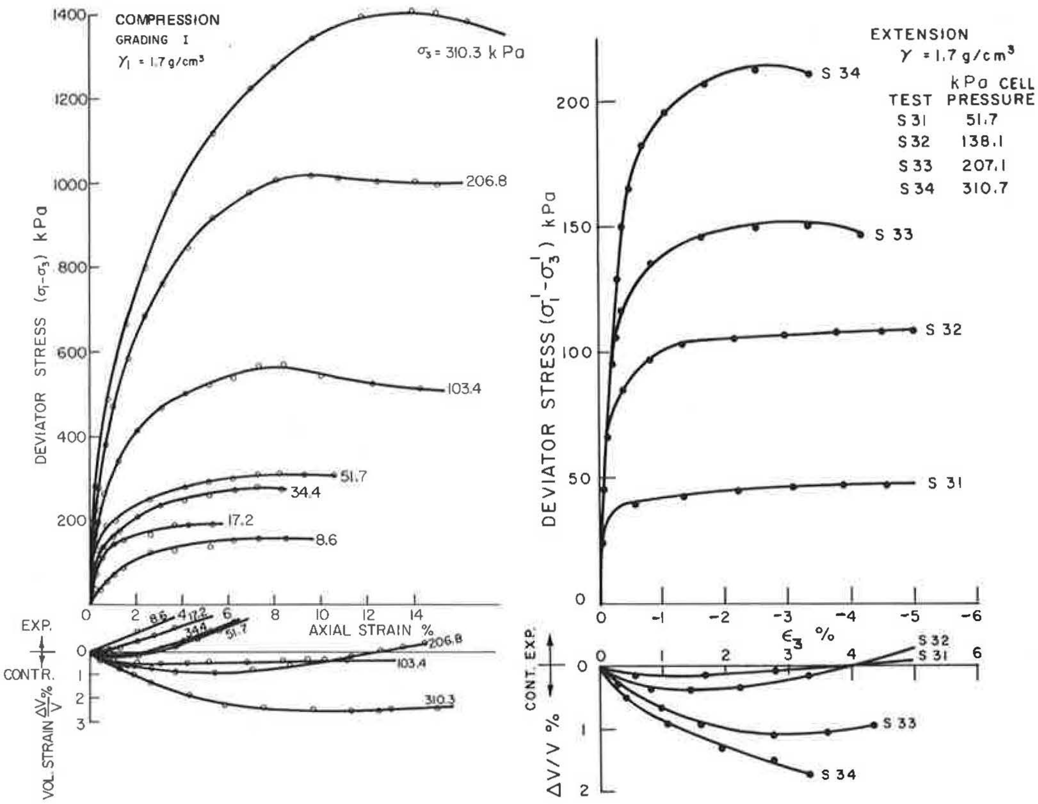


Figure 2. Mohr circles for dense samples.

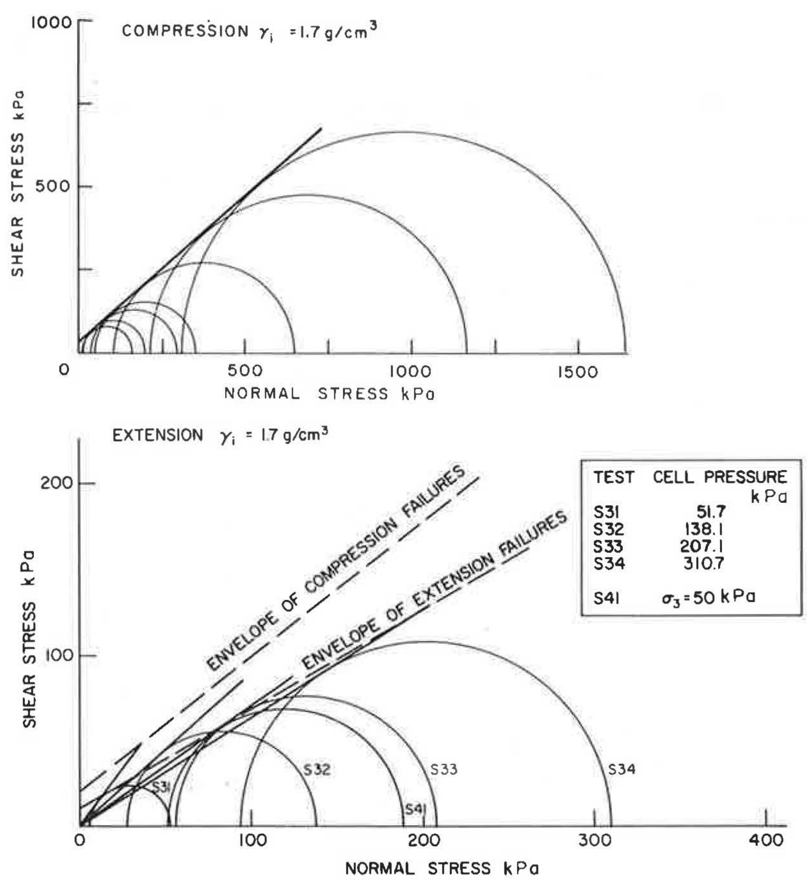
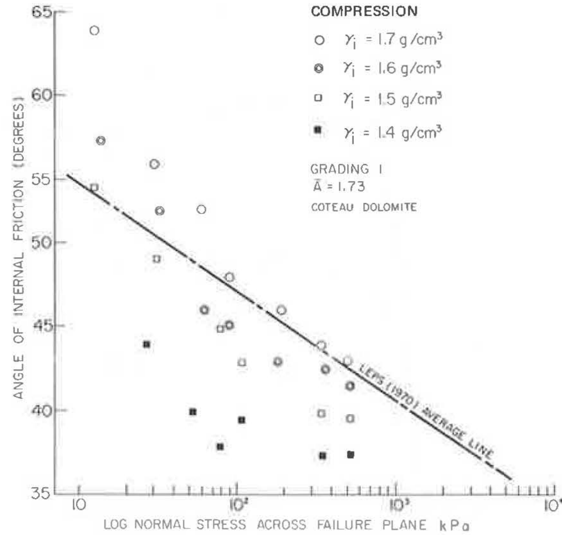


Figure 3. Interpretation of test results representing Mohr circle as friction angle.



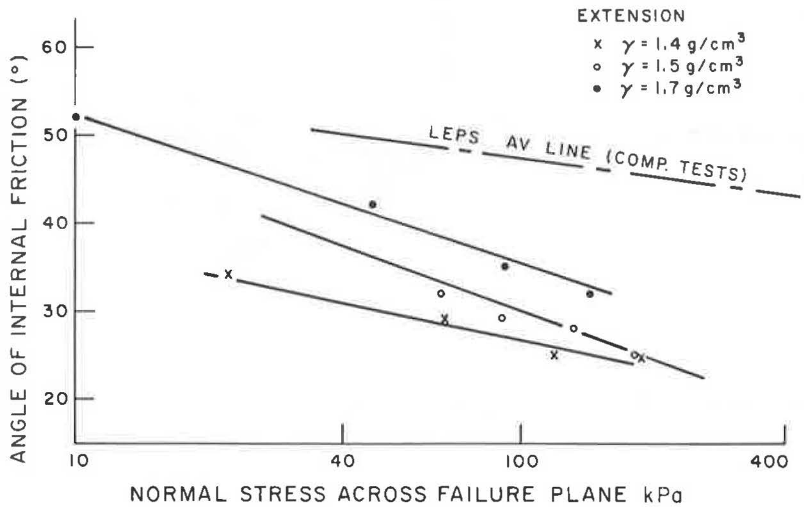
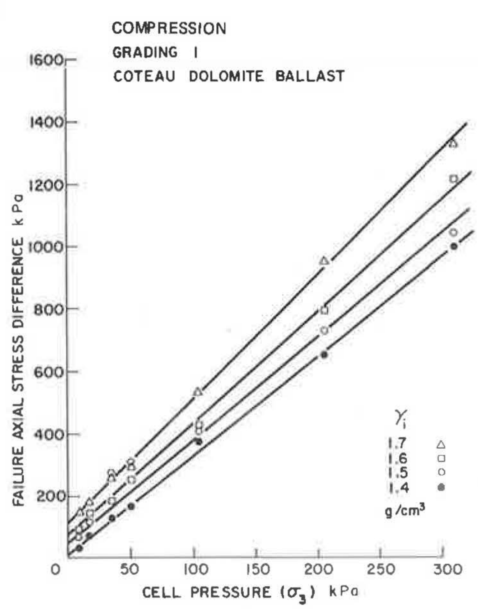
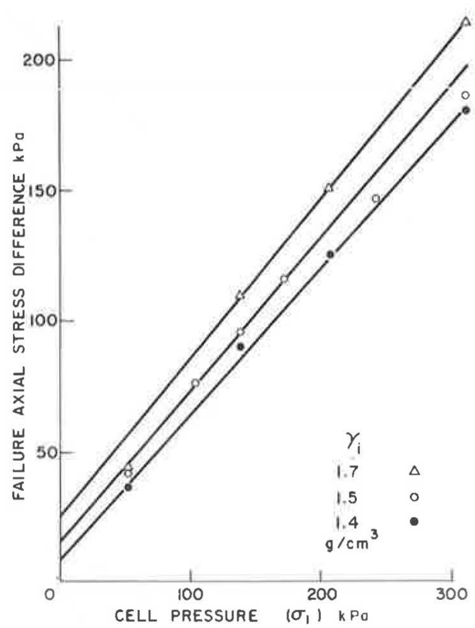


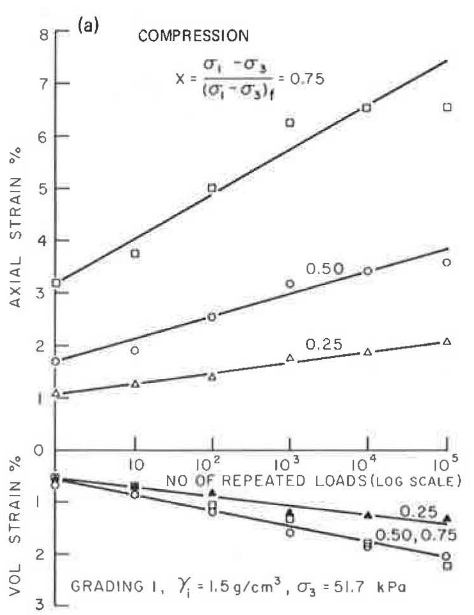
Figure 4. Failure stresses.





the compression and the extension repeated-load tests, the minor principal stress was maintained constant. Figure 5a shows the results of the plastic axial and volume strains plotted against the logarithm of the number of repeated loads in the compression tests, and Figure 5b shows the same type of plots for the extension tests. As in the static tests, the first cycle of strain is considerably less for the extension test than for the compression test. The slope of the line shown in Figure 5 is an indication of the change in strain per logarithm of the number of the repeated loads. These slopes are shown in Figures 6a and 6b for the compression and extension tests respectively. There is considerable difference in performance between the compression and the extension repeated-load tests. In the compression test, the axial strain is approximately proportional to the fraction of the

Figure 5. Plastic axial strains.



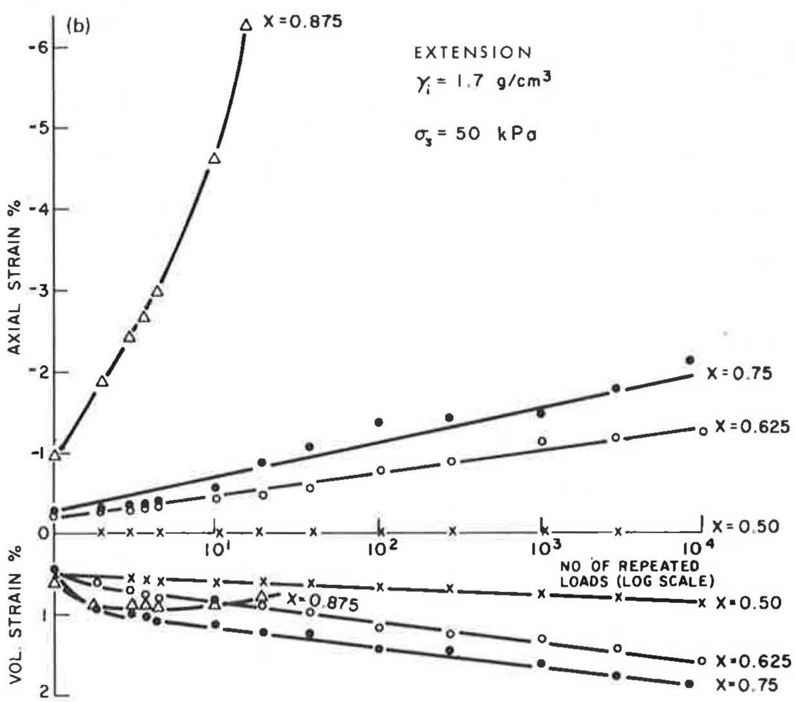
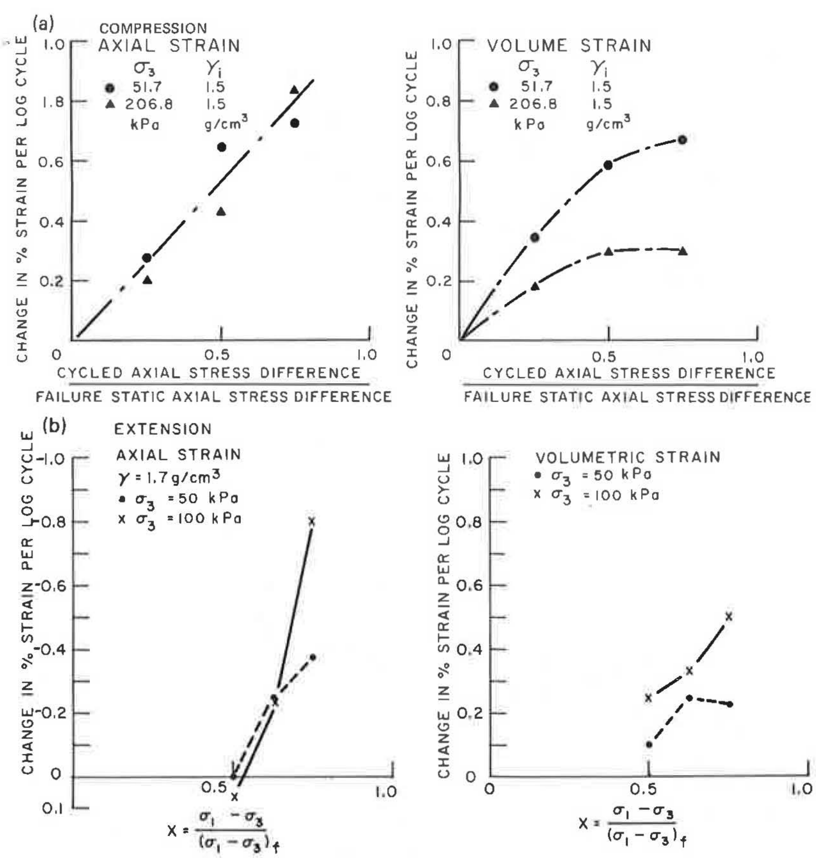


Figure 6. Rates of plastic strain.



axial-stress difference used, whereas in the extension tests, there is little strain up to a value of about ½, but it then increases rapidly as the stress fraction increases. The axial strain at low stress fractions decreases. If the stress fraction used is constant, the change in axial strain per logarithm of the cycle of repeated load is similar; however, the compression and expansion tests will show different behavior with regard to the volumetric strains. At high values of the minor principal stress, the curve of the volumetric strain versus the logarithm of the number of repeated loads in the compression test is less than that at lower values. The behavior in the extension test is directly opposite. Because the axial strain in the extension test is the minor principal strain, the deduced major principal strain must be calculated from the volumetric and axial strains. It becomes obvious that the rate of change of the major principal strain per logarithm of the number of repeated loads in the extension test, is greater for larger values of the minor principal stress and similar deviator-stress fractions.

Repeating the stress difference causes the ballast to become stiffer. The increase in stiffness of the ballast with repeated loading is shown in Figure 7. The static tests showed that the axial secant modulus in the extension test was larger than that in the compression test, however; Figure 7 shows that, after a large number of repeated loads, the axial secant modulus in the compression test becomes larger than that in the extension test. Furthermore, after a large number of repeated loads, the sample failed in the extension test, but there were no failures in the compression test.

Figure 8 shows the number of repeated loads required to cause failure in the extension test as higher values of the stress fraction were used. As the logarithm of the number of loads increases, the fraction of the repeated-load deviator stress required to cause failure decreases approximately linearly, down to a value of about 0.5. At fractions less than 0.5, failure was not observed after 100 000 cycles of load. Whether failure would have occurred with further loading is unknown—more testing is

Figure 7. Secant moduli.

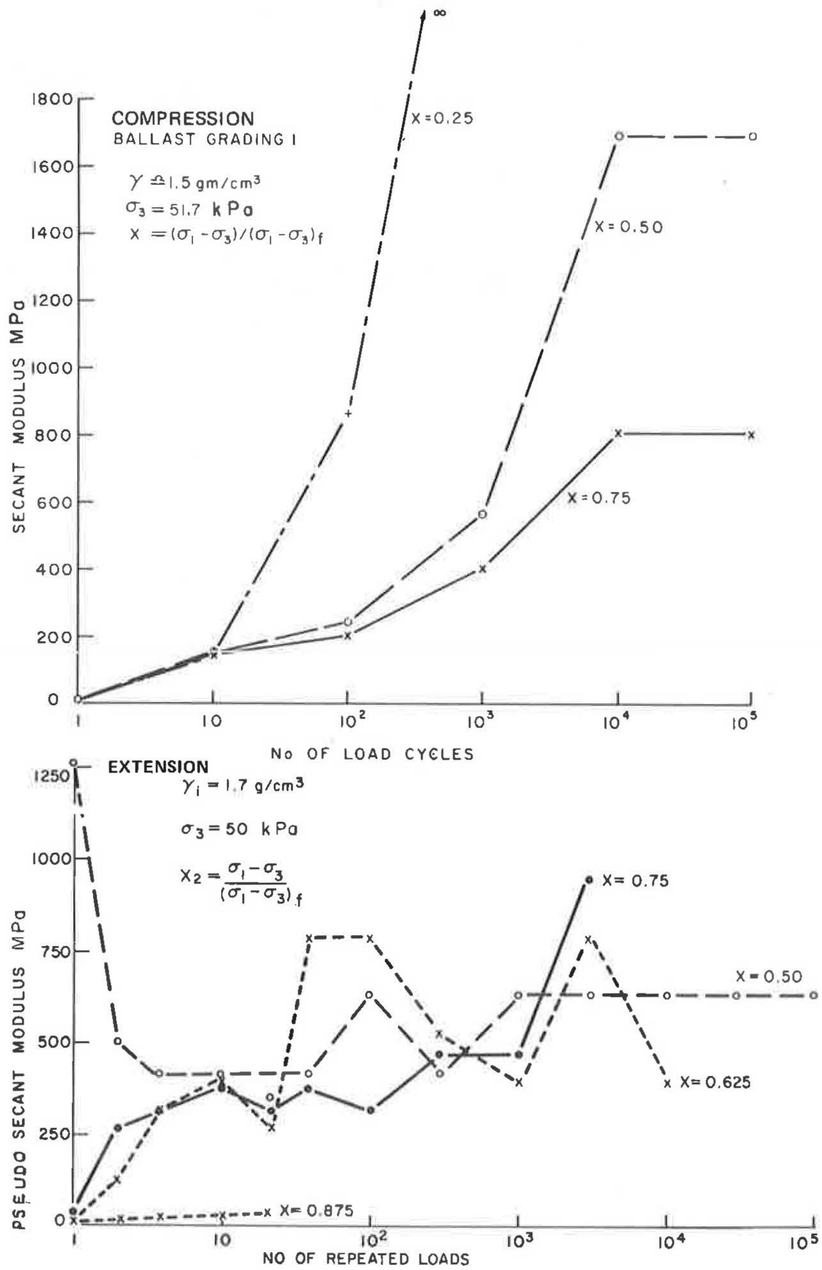
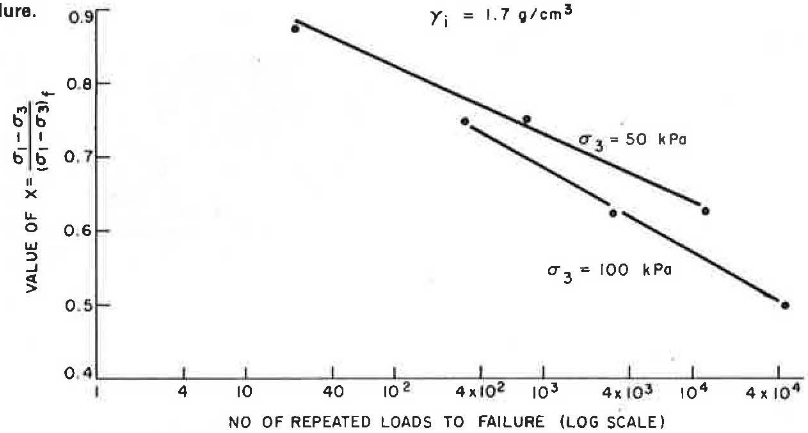


Figure 8. Repeated loads to failure.



required to clarify this point. Because the change in strain per logarithm of the number of repeated loads for the extension test (shown in Figure 6) is negative at low values of deviator-stress fraction, it is speculated that failure would not occur. If so, then there is a fatigue limit of 0.5 for extension repeated loads for the dolomite ballast.

During the compressive static and repeated loading, the ballast breakdown after testing was measured. The breakdown, recorded as the percentage of the material passing a 4.75-mm (no. 4) sieve, is plotted against the cell pressure for the static test (Figure 9) and against the fraction of the deviator stress used for the repeated loading test (Figure 10). Figure 10 shows that the breakdown increases as the deviator-stress fraction used increases. The results for the static test would be expected to fit onto the relation shown in Figure 10 had the deviator stress been used. In the compression repeated-load test, the minor principal stress appears to have little effect on the relation. From this, it is clear that the higher the shear strength (and thus the density) of the ballast, the better it will perform in service, again showing the importance of good initial compaction.

A number of practical factors can be derived from the triaxial repeated-load tests. First, the practice of using train loads to compact ballast in situ after maintenance seems to be a realistic one because the ballast stiffens as the number of loads increases. Second, if the breakdown is independent of the minor principal stress, then any practical application that causes higher confining pressures will result in better performance because, for the same deviator stress, the factor of safety will decrease. This means that broader ties and smaller ratios of tie spacing to tie breadth would decrease the rates of breakdown and of plastic deformation. If the tie breadth is increased, however, the rate of decrease of stress with depth would decrease so that the depth of ballast might have to be increased. While it is possible to calculate these effects theoretically, caution should be exercised until more knowledge is obtained about the validity of the theories in use. For this reason, model studies have been undertaken to attempt to rationalize the theoretical or semiempirical performance of foundations subject to repeated loads on granular material.

MODEL STUDIES

Model studies have been undertaken on a small scale as shown in Figure 11 and on a large scale as shown in Figure 12. These studies confirmed the evidence that the strength and deformation characteristics of a foundation soil are greatly affected by the repeated application of stresses caused by live loads. These characteristics cannot be predicted satisfactorily by conventional static tests. Our studies used a repeated load that returned to zero at the end of each cycle; this is characteristic of a train wheel load passing over a railroad tie. Of equal importance is the case of a repeated load imposed on a dead load; it is hoped to extend our studies to this in future years. Space limitations do not permit a complete presentation of our work.

Four series of tests were conducted by using the small-scale test apparatus. Series A consisted of static load tests incrementally applied to failure. The results of these tests provided a basis for comparison with subsequent repeated-load tests. Series B consisted of applying repeated loads that varied between zero and an upper limit of 90 percent of the static failure load. The upper limit was constant for any given test but was varied among tests. Series C consisted of repeated-load tests that were a continuation of the series B tests. The foundation soil was not renewed between the series B and the series C tests, and the compaction resulting from the series B tests was left untouched. Only the berms caused by the sinking of the footing during the series B tests were removed before the start of the series C tests. Series D was essentially a repetition of series A on the now-compacted foundation soil; however, the berms thrown up during the series C tests were removed before the series D tests were performed.

Typical plastic vertical deformations observed during repeated loading are plotted against the number of loads on a logarithmic scale in Figure 13. This figure shows that the permanent vertical deformation increased as the number of loads and as the percentage of the static failure load increased. At the end of 100 000 loadings, the vertical settlement was 1.8 mm under 13.5 percent of the static failure load, and 18 mm under 50 percent of the static failure load. This difference of 16.2 mm is primarily due to the repeated loading. The difference after the first loading for the same two tests was only 1.5 mm.

The relation of the plastic deformations and the number of loadings is nonlinear and can be fitted to a hyperbolic equation as is commonly done for modeling stress versus strain triaxial-test results. However, Figure 13 is based on the following modified equation:

$$\log N = S/(a + bS) \qquad (1)$$

where

S = deformation after first loading,
N = number of loadings, and
a and b = experimentally determined constants.

The values of a and b were determined for tests conducted on 75- and 228-mm broad footings. Regression analysis was used to fit these values to the percentage of the static failure load used and to the footing breadth. The equations resulting from these fits are

$$a = -0.151 + 0.000\,069\,B^{1.18}(F + 6.09) \quad (2)$$

and

$$b = 0.154 + 0.000\,036\,B^{0.82}(F - 23.1) \quad (3)$$

where F = percentage of failure load used and B = footing width (mm). Equation 3 indicates that the strength in terms of excessive deformation does not vary in proportion to the footing breadth squared, which is the well-known result for static loading of a footing on granular soil. The strength increases, however, at a greater rate than does the breadth of the footing. Thus, under

Figure 9. Breakdown in one-cycle compression tests.

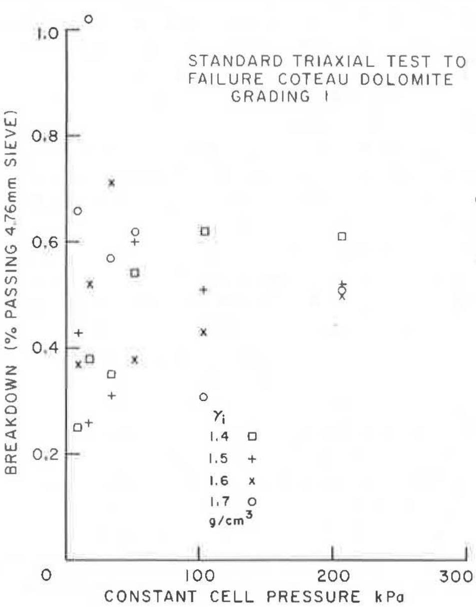


Figure 10. Breakdown in repeated-load compression tests.

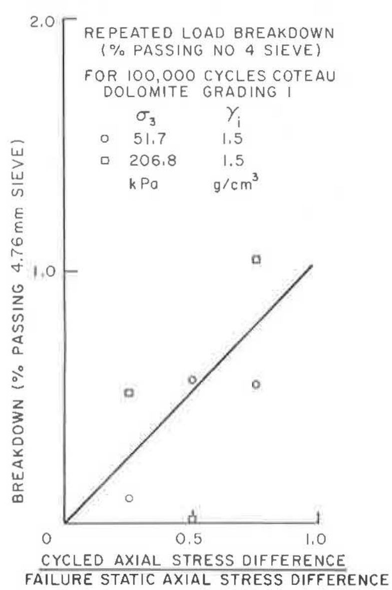


Figure 11. Small-scale planar strain model equipment.

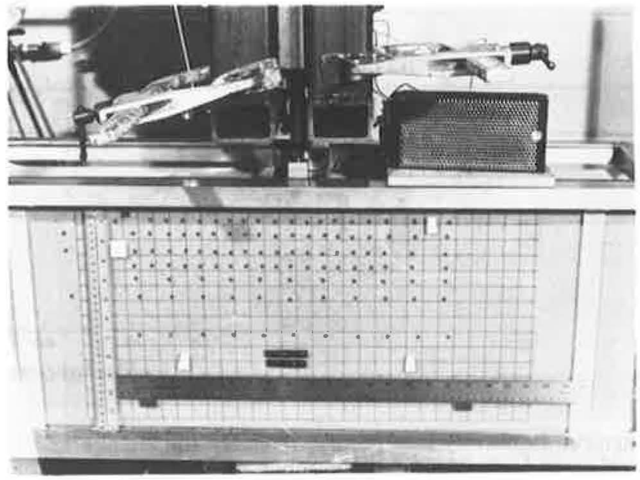


Figure 12. Full-scale model equipment.

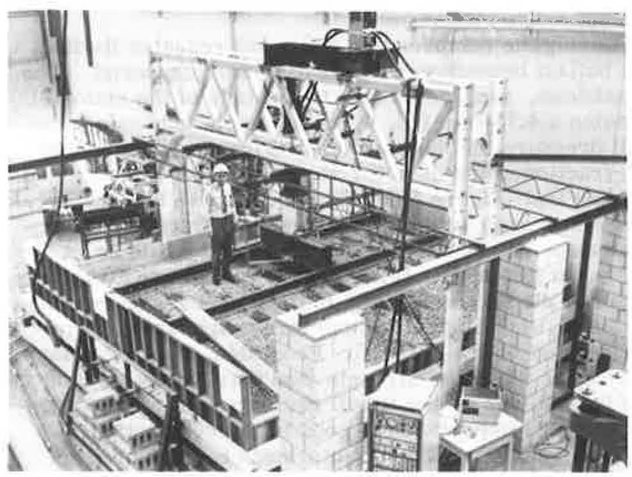


Figure 13. Deformations in small-scale planar strain tests.

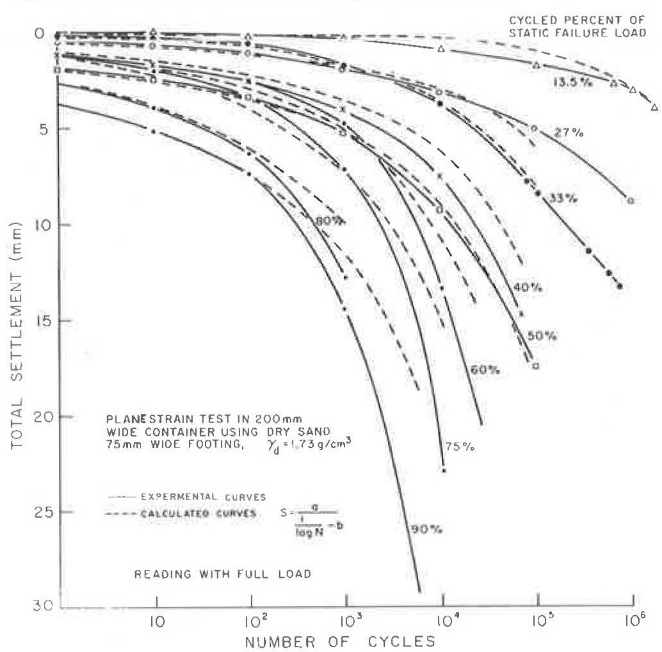


Figure 14. Rebound deformations in small-scale planar strain tests.

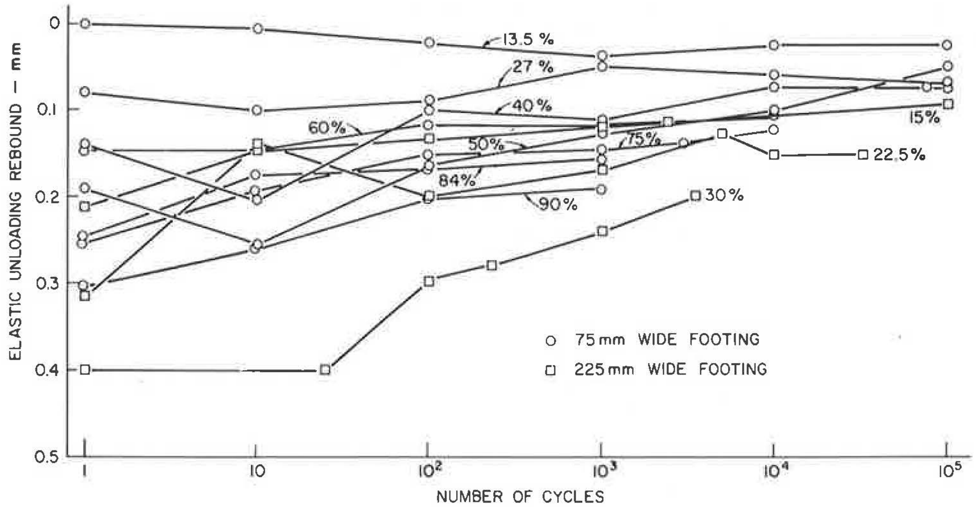
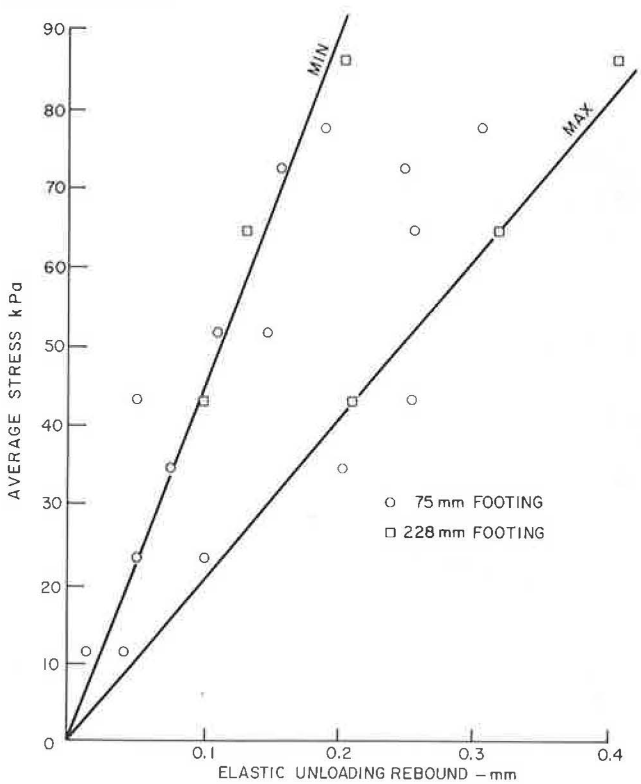


Figure 15. Average contact pressure: rebound results for small-scale planar strain tests.



repeated loading, a broad footing loaded with an equal pressure will deform less than a narrow footing.

Figure 14 shows typical rebound readings recorded under 75 and 228-mm broad footings at different percentages of their failure loads. The failure load of the larger footing would be approximately nine times greater than that of the smaller footing. As the number of loadings increases, the rebound deformation decreases, as is to be expected, because repeated loading causes an increase in the soil density. The rebound from the first loading was neglected, because of the possibility of a rough initial sand surface, and the minimum and maximum deformations were selected. Typical values are plotted against the average footing contact pressure in Figure 15. While the maximum values for the smaller footing show some scatter, the minimum values approximate a straight line. The scatter of the maximum values would seem to occur with the nonuniformity of placement. Only after complete bedding down under many loadings does the sand tend to act in conformity with the Winkler foundation concept of a constant deformation modulus. Even so, Figure 14 indicates that the rebound under the larger footing is decreasing, while that for the smaller footing is almost constant. This means that if the loadings were continued, the foundation rebound for the larger footing would decrease, which would make the foundation rigidity for the larger footing greater than that for the smaller footing. Another advantage of the larger footing is that it has a three times larger average contact stress for the same percentage of failure load.

Confirmation of these conclusions and observations is being sought from the full-scale tests. It is also possible to measure the pressure distribution within the track bed. A set of pressure distributions for a single tie is shown in Figure 16. After relatively few applications of the load, the vertical pressures registered beneath the centerline of the loaded tie began to increase, but the vertical pressures measured beneath the rails decreased. This phenomenon was also observed for an 11-tie test and is associated with center binding.

If the load had been evenly distributed over the contact surface of the interface between the tie and the ballast, the average contact pressure would have been 371 kPa. At 100 loadings, the maximum pressure under the rail was about 60 percent greater than the average value. After 1 000 000 loadings, the pressure under the centerline, however, was about 20 percent greater than the average.

Figure 17 shows, for three of the dial gauges on the tie, the deformations for the single tie and its foundation after various numbers of loadings. In each case, the plastic deformations from previous loadings have been deducted. Thus, each loading curve starts from zero. After the first loading, the deformation curves at the rail and the free end do not fit smoothly through the origin. If a linear relation was fitted to the higher load readings, a considerable gap would exist at zero load. This is consistent with the pressure readings and with the phenomenon of center binding already mentioned.

Figure 16. Pressures below single tie (full-scale test).

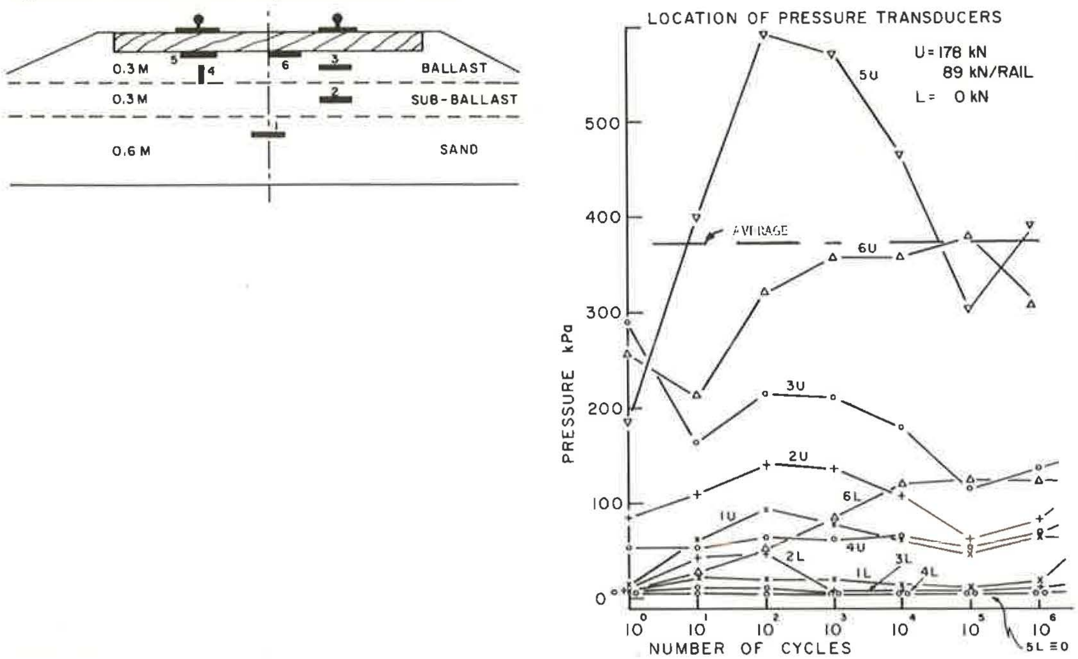
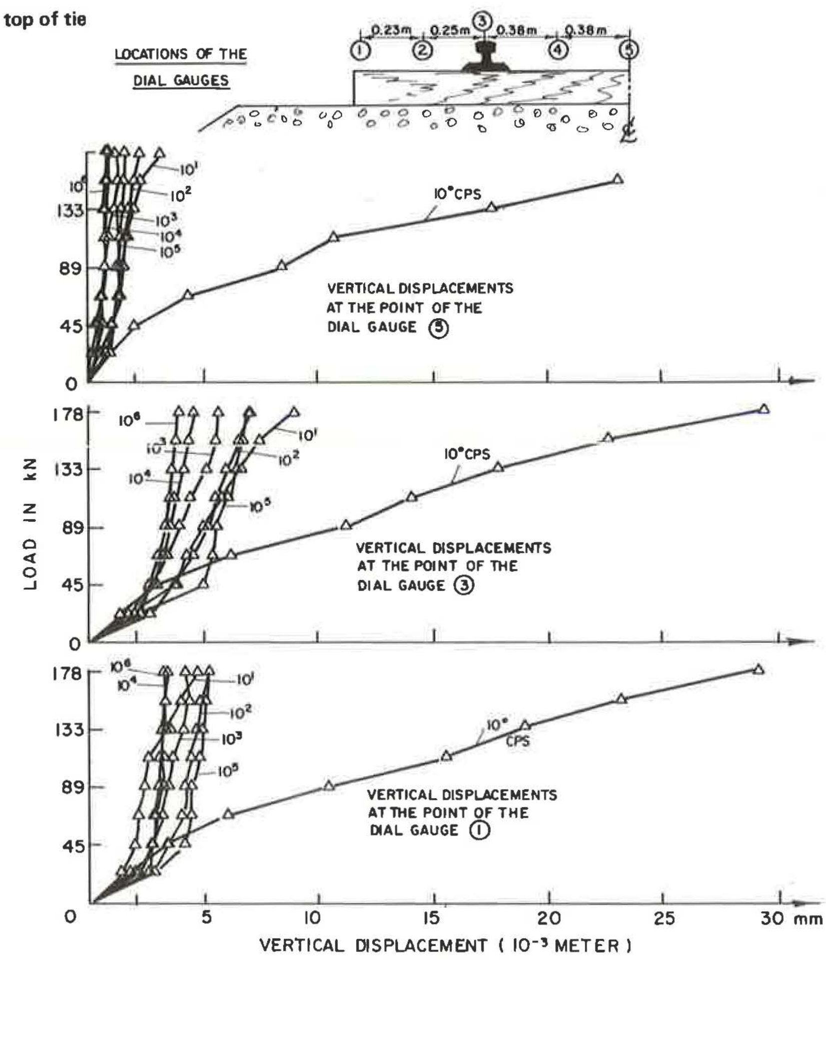


Figure 17. Deformations of top of tie (single-tie test).



CONCLUSIONS

1. In static triaxial compression tests, ballast breakdown is only slightly affected by cell pressure and is closely related to the factor of safety.

2. In repeated-load triaxial compression tests, the breakdown of ballast is related to the fraction of the failure stress used, rather than to the confining stress. Higher shear strength means less breakdown.

3. Factors one and two may be associated with broader ties and a smaller ratio of tie spacing to tie breadth.

4. Cycling the stress causes ballast to become stiffer, which confirms the practice of using train loads to further compact ballast in track.

5. The extension tests tended to confirm the results of the compression tests; however, failure was observed in the repeated-load extension tests. The number of loadings required to cause failure in the extension tests decreased as the fraction of the deviator stress used increased. The repeated-load failure in the extension tests is important with regard to track stability. There appeared to be a fatigue limit of about one-half the static failure for the dolomite tested.

6. The results of the small-scale model tests reinforce the conclusion that broader ties should perform better than narrow ones in minimizing plastic deformations.

7. The footing rebounds measured suggest that, because of repeated loading, a broader footing should produce a marginally higher foundation stiffness.

8. The plastic deformation of a footing on Ottawa sand subjected to repeated loading has been quantified by using a hyperbolic equation.

9. After parameters for the quantification of plastic deformation of different ballast materials have been obtained, then a design methodology for track maintenance (in terms of plastic deformation) can be formulated.

10. The full-scale model tests indicated that equilibrium in terms of zero plastic strain at the interface of the tie and the ballast is never reached. The displacement is not uniform across the tie. The settlements under the rails are greater than those at the tie centerline. This leads to a center binding phenomenon, the ultimate result of which is fatigue failure of the tie at or near its midpoint. To counteract this phenomenon, tie geometry and dimensions could be changed to encourage a more uniform displacement. The advent of synthetic ties that more readily lend themselves to nonuniform shapes may make such changes practical. Further full-scale testing of ties of various geometries would be a potentially fruitful course of action.

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Track Structure Systems

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The railway track-system concept is a way of looking at things that takes into account secondary and tertiary effects in the totality of cause and effect. A track system is not simply a collection of curves, tangents, switches, frogs, turnouts, crossings, and crossovers, but includes the interrelations among the various components—the rails, ties, ballast, fasteners, and subgrade. One of the earliest railway engineers to employ system thinking was Robert L. Stevens of New Jersey, who in 1830 conceived the flat-bottomed-tee rail and the first cut spikes and joint bars. Later, he evolved the idea of wooden crossties. He single-handedly developed the basic system of mutually complimentary components used in railroad trackage today. The next system thinker to have a profound influence on track technology in North America was Arthur N. Talbot of the University of Illinois, who developed the concept of the modulus of elastic track support, first reported in 1918. This was a quantifiable response to load of ties, ballast, fasteners, and subgrade material that can be used to predict track deformation under vertical load. The Stevens' legacy was a system design of railway track, and Talbot's contribution was a system analysis of track structure. Talbot also left a challenge because, while track performance can be predicted when the modulus is known, how to design to a modulus has not yet been learned. The rate of return on incremental investment in individual track components can be determined only by full-scale experiments. The new full-scale laboratory the Association of American Railroads is building in Chicago should bring about validation of mathematical models of track that are being developed.