SOILS ENGINEERING IN RAILWAY CONSTRUCTION AND MAINTENANCE

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SYNOPSIS

This paper is mainly a description of the investigation by the railroad industry into soil and ballast problems affecting the stability of roadbeds. The problems are very similar basically to those encountered in highway and airport construction.

The first investigations of record were conducted by the Committee on Stresses in Railroad Tracks, formed in 1914 as a joint committee of the American Society of Civil Engineers and the American Railway Engineering Association and later continued under the sole support of the Association of American Railroads. The early work of this committee led to the publication in 1918 and 1920 of the Modulus of Track Support based on the elastic equations and very similar to the modulus of subgrade reaction. These early tests also included the use of pressure capsules under typical railroad loading.

The more recent research was instituted in 1945 sponsored by the Roadway and Ballast Committee of the AREA and conducted by the research office of the Engineering Division of the AAR. It is this later work that comprises the majority of the discussion in this paper.

Roadway stabilization now is mainly a problem of maintenance. New construction is limited principally to line and grade revisions. On these, control of the soil structures is in accord with modern and standard practice. The results of study, however, indicate that this type of control for some soils is not sufficient to obtain the expected results and sections with abnormally high maintenance may appear after operations begin. Treatment of new grades to restore original stability, of course, is a maintenance problem.

Discussed in the paper are the various methods of treatment now utilized for subgrade stabilization. Two general classifications of conditions leading to high maintenance costs are water carrying ballast pockets and unstable fills. Pressure grouting using a cement sand water slurry has produced good results in overcoming high maintenance costs for pocketed sections. Six year maintenance records show these sections have usually returned the grouting cost within that time. Other methods also giving good results have been poles or cull ties driven vertically at the ends of the cross ties. Briefly described are methods for sand filled spud holes (sand piles) and sand filled blast holes.

Treatment for unstable fills have included buttressing, drainage, piling and pressure grouting.

Other projects for study now in progress are the actions of simulated subgrades under a laboratory oscillator. Considerable data have been obtained on the critical values of moisture and density conducive to the development of pockets. The oscillator was devised by the Engineering Experiment Station of the University of Illinois particularly for this project.

Earth pressure cells have been installed under railroad tracks for the past several years. This cell is the AAR type developed mainly for dynamic loads. Oscillograph records under rail traffic have been obtained and are discussed briefly. In addition, several unstable fills have been studied through sampling and laboratory analyses. To facilitate field work of this nature a portable apparatus for obtaining unconfined compressive strengths has been developed. Ballast tests are also in progress.

Many of these projects are of a continuing nature and as yet have not yielded data from which unqualified conclusions can be drawn. Indicated, however, are the following:

1. With good quality ballast the depth rather than the type largely determines the load on the subgrade.

2. Instability can develop through an appreciable depth of ballast under unit pressures much lower than the indicated strength of the soil material.
3. This instability is largely a failure in bearing of a thin layer at the top of the subgrade subject to water.
4. Instability of roadbed and fills can and often does develop only from precipitation waters falling on the track.
5. Analyses of critical circles do not show cause of most fill failures. Most failures do not follow critical circle. Many fills with low factor of safety show no failing tendency.
6. Control of moisture and density are not sufficient to insure stable roadbeds. Swell characteristics must be considered, also type of clay mineral.
7. Vibrations produced by traffic affect stability in an as yet undetermined manner.
8. Various methods of stabilization will produce good results and are usually self-amortizing within a few years.

Soil engineering for the railroad industry had its inception in 1914 with the formation of the Committee on Stresses in Railroad Tracks—a joint committee of the American Society of Civil Engineers and the American Railway Engineering Association. This committee was later continued under the sole support of the Association of American Railroads. The late Dr. Talbot of the University of Illinois directed the work. In 1918 and 1920 the report of this committee included evaluation of the modulus of track support, based on the elastic equations and supported by considerable experimental work with static railroad loads. This work also included the use of pressure capsules under the ballast and formulated the principle that vertical pressures transmitted to the subgrade were practically independent of the type of ballast used. The modulus of track support is similar in concept and application to the modulus of subgrade reaction used in the design of highway pavements and bases. The distribution of pressures as generally independent of the transmitting medium is used in methods of design of flexible pavements. The information developed by the committee is basic and is used today in design of the track structure.

The organization started in 1914 has carried through to this day and is now the research office of the Engineering Division of the AAR. In 1945, in view of the increasing cost of roadway maintenance and the number of problems directly ascribable to subgrade conditions, active investigations in both laboratory and field were instituted for determining cause and correction of areas requiring high maintenance expenditures. These investigations, continued and expanded from that date, are aimed at specific problems and in that respect are not basic research.

Considerable general information, however, has been obtained and the problems themselves are basic for any soil structure, railroad, foundation or highway. This paper will cover briefly several of the investigations now in progress.

CONSTRUCTION

Railroad construction is now of secondary importance to maintenance in extent. However line and grade revisions are often under construction as well as relocations necessitated by construction of dams, etc. On these the soil practice is in accord with modern concepts and standard practice. In addition, present economical construction methods have rendered largely obsolete the old standard practice of trestle fill construction on which compaction control could not be obtained. In the course of these investigations, however, it has been shown that ordinary moisture and density control is not sufficient to insure acceptable performance of railroad roadbeds in service. On one particular project on which no difficulty was encountered in obtaining specified compaction, the maintenance cost on eight miles of roadway during the first five years in service approximated $300,000 over and above that required to maintain to equal standards on the old line. This action was unexplainable through any of the criteria commonly used in the evaluation of stability. A mineralogical analysis of the clay material disclosed the presence of considerable montmorillonite with its resultant high volume change. On the basis of usual limit, compaction and strength tests the fills on the project were of ample strength and stability. Results of this investigation have indicated the necessity of including more complete analysis in future design information for those areas where soils are questionable.

Other projects entailing new construction have included underground drainage, mechanical modification of the upper subgrade and selection of soils. The latter type of soil engineering is not always feasible with the materials immediately involved. However in highly plastic soils a cover course or grade capping with granular material can usually be justified economically on the basis of reduced maintenance. In the presence of deleterious clay minerals as discussed previously this procedure may not be fully successful unless the weight of the cover is sufficient to contain the swelling pressures produced. Compaction on the wet side of optimum may be beneficial for these soils. The present knowledge of the fundamental characteristics of fills in service is very inadequate and adequate treatment for all conditions can not be specified until more is known about service behavior of controlled roadbed construction.

MAINTENANCE

Ballast Pockets—There are two general classifications of roadbed instability requiring above normal maintenance attention: ballast or water pockets and sliding or slipping fills. The pocketed condition is relatively a surface condition, although pockets up to 10 ft. in depth are not unknown. These pockets are developed on the surface of the subgrade under the ballast by a failure of the soil in bearing or lateral shear. The mechanics of the growth are indicated by Figure 1, sketched from actual conditions encountered in a trench into the track. An accumulation of moisture so weakens the soil that under traffic loads a thin layer of material is displaced laterally and forced upwards beyond the end of the ties. The distance of this heave from the tie end is indicative of the depth of the pocket but this distance will vary with soil characteristics for pockets of the same depth.

When a pocketed condition starts, it is often indicated by soft track and a pumping action under moving loads very similar to joint pumping on rigid pavements. Development of pockets is progressive, and surface

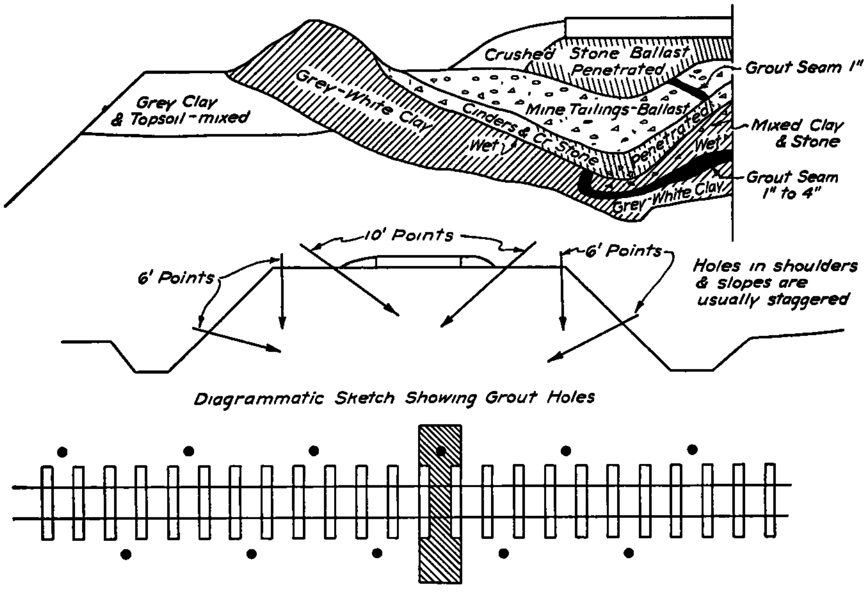


Figure 1. Trench Section and Typical Grouting Layout

pumping is not usually evident for the deeper and older pockets.

Clays of high plasticity are most susceptible to this condition, but under sufficient loading and available free moisture, pockets can be developed in much lighter textured soils. In recent years contemporary with increases in weight and speed of traffic an increase in pocketed areas has been noted. To maintain track line and surface over these areas requires excess labor and ballast.

To reduce these expenditures and to provide stable track, various stabilization processes have been devised. Drainage where feasible is effective but over many pocketed areas there are variations in depths, and full correction by drains is costly and uncertain. For most areas the water in the pockets is that which has fallen on the track. Where underground flow occurs from off track, drainage to intercept is of course the first requirement.

A comparatively recent process that has been used successfully over a considerable aggregate mileage is pressure grouting by which a slurry of sand and portland cement is injected under pressure into the subgrade. For pocketed areas injection pipes are usually driven through the ballast material and a few inches into the subgrade. Average spacing is about 6 ft. on each side of the track. Figure 1 shows schematically the layout of injections on one project. The injections nearest the track are designed to correct the pocket action. Additional injections shown are to correct slope slips, which are discussed later. The grout concentrations after treatment are also shown in Figure 1. In Table 1 the maintenance cost for this section is listed under Project A.

TABLE 1
GROUTING AND MAINTENANCE COST DATA—BALLAST POCKETS

Project	Length of Track	Grout Acceptance	Cost of Grouting			Annual Maintenance Cost Lining and Surfacing		Annual Savings	Age of Project
			per Cu. Ft.	per Tr. Ft.	Total	Before	After		
	ft.	*cu. ft. per track ft.*							*yr.*
A. Nebraska	6,336	7.85	$0.28	$2.20	$13,939	$2,366	$17	$2,349	4
B. Ontario	7,820	5.75	0.483	2.78	21,800	9,400	699	8,701	4
						man hr.	*man hr.*	*man hr.*	
C. Colorado	17,950	1.42	0 378	0 535	9,600	25,100	9,210	15,890	8
D. N. Dak.	10,545	5.36	0.35	1.88	19,824	15,015	6,700	8,315	3

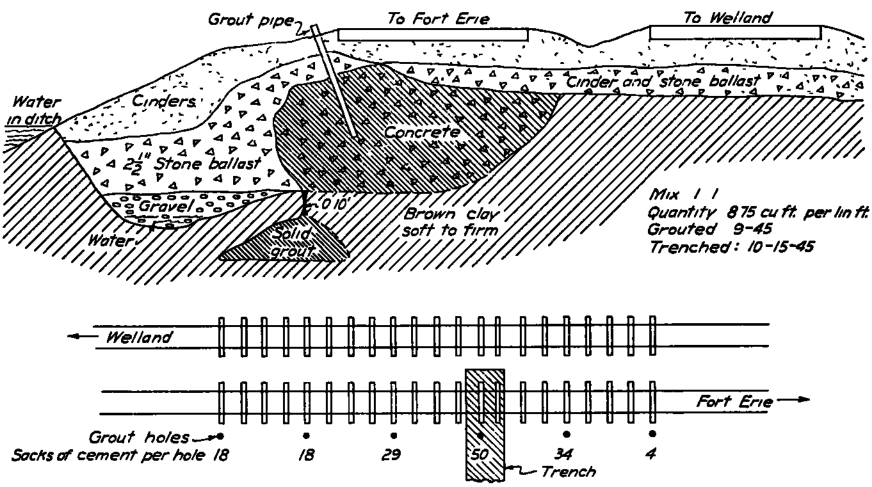


Figure 2. Trench Section Showing Grout Distribution

Figure 2 shows the results obtained by grouting a deep water pocket. Maintenance data for this section, Project B, are also shown in Table 1.

Various mixtures for the slurry have been used by various roads with good results. Sand cement combinations varying between 32 to 1 and 1 to 1 have been used. Sand with 100 percent passing a number 20 sieve and up to 15 percent silt and clay are most practical from an operational standpoint and will permit leaner mixtures of cement. Two general methods of injections are in common use:

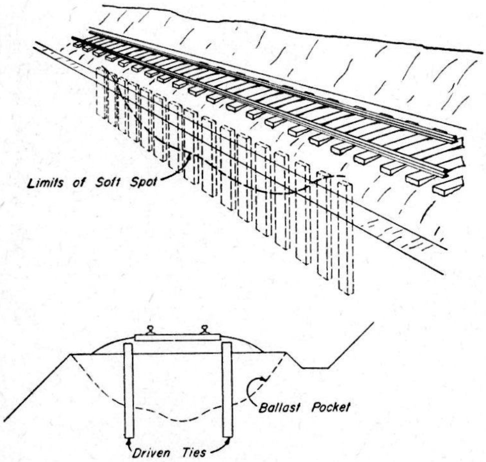


Figure 3. Tie and Pole Driving Typical Treatment for Soft Track

pneumatic pressure and hydraulic pressure provided by piston arrangement as typified by a mud-jack.

An older type of treatment is the driving of culled ties or poles near the ends of the cross ties as indicated in Figure 3. This procedure has been effective in low cuts and fills particularly in wet areas, and has been used extensively in the central southwest. The manner in which such installations restore subgrade stability is still a subject of some conjecture, but it seems probable that the driving increases compaction. The timbers themselves may exert a restraining influence and there is some relief for excess moisture. In addition, recent pile tests have indicated a high skin friction can be developed at least momentarily between soil and pile, and it is possible that under rail traffic considerable load is carried by the installation reducing subgrade soil stresses sufficiently to prevent lateral displacement. Success of the vertical tie or pole installation requires that approximately one half the length is penetrated into undisturbed material. Sand piles and sand filled blast cavities give similar results and appear to perform similarly.

Figure 4. Unstable Fill

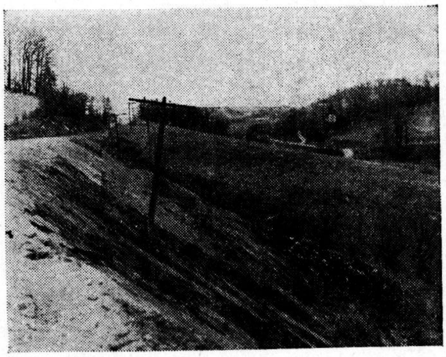


Figure 5. Slope Failure on Fill

Unstable Fills—Sliding and slipping fills make up the second general classification of roadbed maintenance problems. While not as extensive by far as the pocketed conditions, maintenance of even short sections on unstable fills can be costly. Also, speed restrictions may be required.

Fill instability is manifested in several manners but the most prevalent failure is a slope failure with the sliding surface intersecting the slope near the toe. Other slope types are failures through the foundation, and slope failures above the toe, the latter often the result or the culmination of deep pocket development. In addition, failures may occur along an inclined surface on or under the original ground and through swamp subsidence. Figure 4 shows a fill in central Ohio on whch a slope failure has occurred. This fill has been restored but the amount of vertical slippage in 10 months is indicated by the elevation of the outer shoulder containing the crack. This previously was at normal track level. This which the sliding surface intersects the slope about 15 ft. below the top. Apparently this failure is the outgrowth of deep ballast pocket development.

Every year, however, slips and slides de-

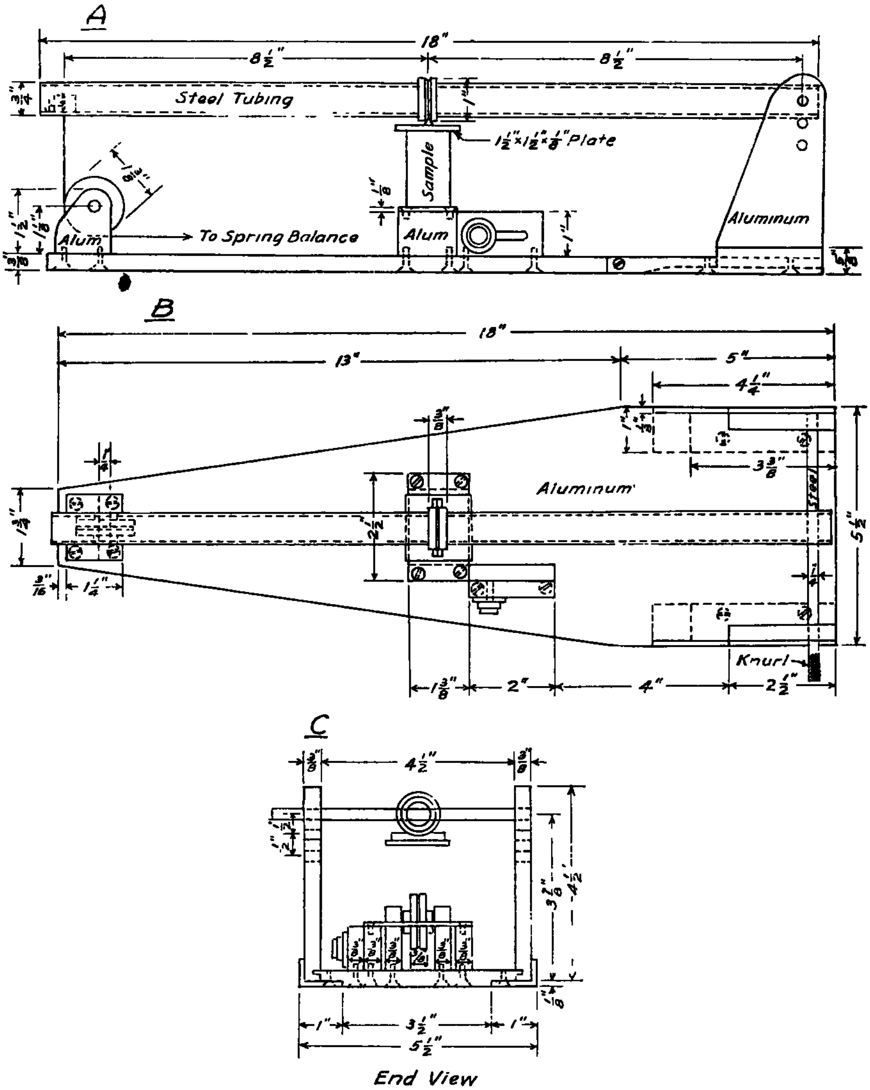


Figure 6. Portable Compression Testing Machine

A—Elevation; B—Plan; C—End View

condition is typical of most of the unstable fills encountered. Movement downward is relatively slow after the initial break and becomes more of a viscous movement or plastic flow. Figure 5 shows a fill in southern Ohio on velop in new locations and investigations have been directed toward the detection of those on which movement is imminent. A number of fills showing slope instability and on which a knowledge is available on method of construction have been investigated by means of borings and tube samples. Generally, however, analyses of these samples fail to indicate the cause of failure. In these cases the apparent factor of safety should have been sufficient to provide stability. Conversely, other fills with an unsatisfactory apparent safety factor remain stable. The data indicate that information obtained from tube samples while yielding valuable information is not definite enough to evaluate the true stability conditions. This is particularly true for the older fills on which no construction record as to soils, moisture and compaction is available.

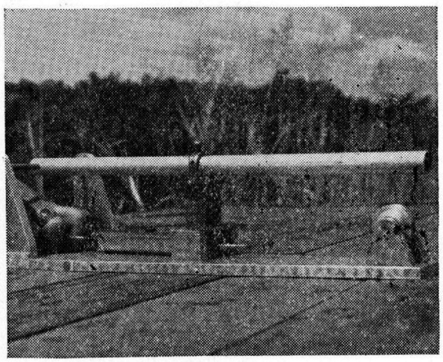


Figure 7. View of Field Compression Apparatus

Examination of open excavations in both stable and unstable fills appears necessary and with a backlog of data obtained from open excavation studies a more complete analysis from auger borings may be possible. This problem is national in scope and includes a wide variety of conditions. Other agencies have initiated similar investigations.

In the course of the AAR investigations into fill stability in general, a number of slides activated by seepage pressures have been noted, particularly along rivers subject to quickly subsiding flood stages. These are special cases for the general slide classification given above.

In connection with the fill investigation a lightweight small unconfined compressive strength apparatus has been developed to permit field determination of this value. Figure 6 is a set of drawings of this apparatus, and Figure 7 is a photograph showing its operation. This apparatus will permit rapid evaluation of soil strengths in the field and reduces greatly the number of samples for shipment to the laboratory.

Full correction of sliding conditions on fills has been an expensive process and often included interruption or delay of traffic. However, in the past five years or so an increasing number of these unstable sections have been treated very successfully by pressure grouting. This, too, can be somewhat expensive but the savings will often amortize the original cost within three years. Fills up to 90 ft. in height have been stabilized. Grout acceptance will often approximate 3 percent of the theoretical volume of the fill and acceptance on one project accepted approached 20 percent of the fill volume. The fill depicted in Figure 4 accepted an average of 38.5 cu. ft. of grout per track foot. Acceptance in the center of the fill was probably double that of the ends.

A diagrammatic plan for fill grouting is shown in Figure 8. Local conditions and type of sliding failure may require modification of this plan. Figure 1 also shows the plan actually used on the project.

In this particular location, however, the slope failures were induced by expansive clays. Figure 9 shows diagrammatically the plan of stabilization used on a specific project in Iowa on which the failure was a combination of water pockets, slope failure and slip on the foundation. Table 2 shows pertinent data on grout acceptance, cost and maintenance before and after treatment on three projects including that represented by Figure 9. Much the same results can and have been obtained by deep injection pipes from near the top of the fill. Hazards in driving injection pipes, however, make the slope injection system usually more economical. Figure 10 pictures typical procedure.

LABORATORY INVESTIGATIONS

Laboratory work has been required and performed in connection with all field investigations. In addition, by means of an oscillator specially constructed for the project, unit loads of intensity comparable to those of rail traffic are applied to a simulated subgrade. These loads between 20 and 25 psi. can be applied at a rate up to 229 per min. This apparatus was devised for the purpose of better understanding of conditions contributing to the development of soft spots and ballast pockets. The loads are applied through a plate 6 to 12 in. square to a simulated sub-

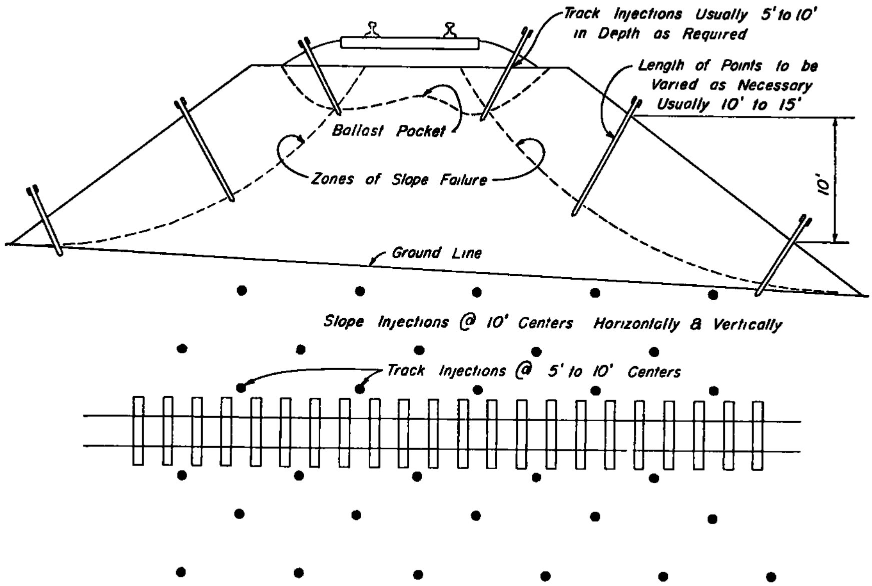


Figure 8. Pressure Grouting—Typical Injection Plan for Fill Slope Failures

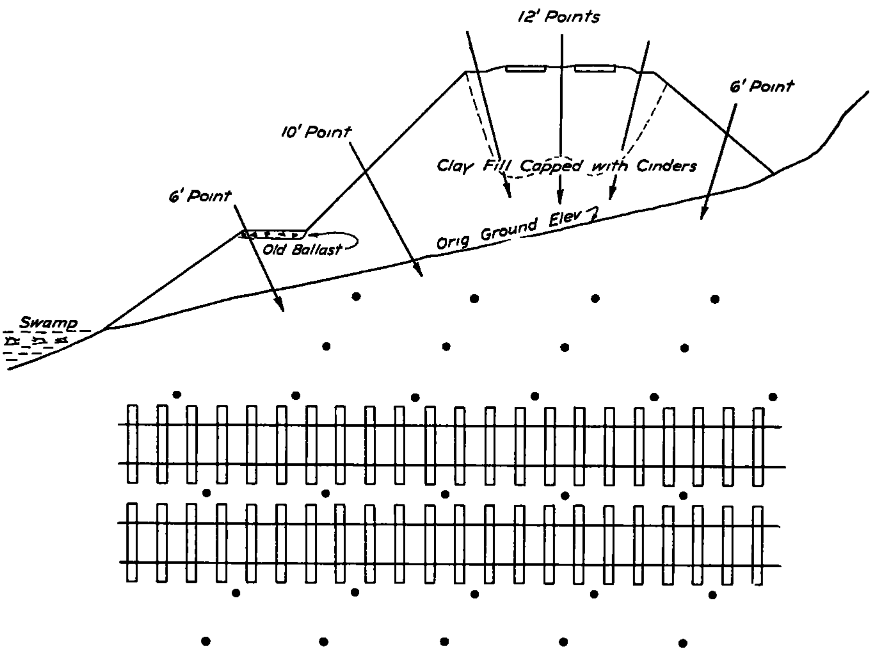


Figure 9. Pressure Grouting—Sliding Fill on Foundation and Typical Injection Plan

grade about 30 in. square and 12 in. deep. In the laboratory this simulated subgrade can be controlled as desired. Tests so far performed include soils compacted at various moisture contents and density and the time-penetration relationship of the loading plate determined. At a stage in some tests the subgrade was flooded and the loads continued. Figures 11 and 12 show results of these tests. Tests with simulated ballast section have also been run. Auxiliary tests for density, unconfined compressive strength and Atterberg limits are required.

The investigation is a long term project. The variables are numerous but indications are favorable for the accumulation of valuable basic data. At the present stage of the tests it is indicated that because of the slight swelling tendency of any clay, water absorbed is driven into the soil mass under load, and this proceeds at a faster rate under intermittent loading than under a static condition. This increase of moisture reduces shearing strength and when this reduced strength becomes less than the shearing stresses lateral movement occurs. The loading plate then penetrates into material relatively unaffected and the process repeats itself. It has been noted that penetration increases sharply when the compressive strength approaches some critical value (See Fig. 12). It has also been found on one material that loading plate penetration after flooding of the simulated subgrade decreased with increased starting water content up to standard Proctor optimum or a few percentage points above.

TABLE 2
GROUTING AND MAINTENANCE COST DATA—UNSTABLE FILLS

Project	Length of Track	Grout Accept-ance	Cost of Grouting			Annual Maintenance Cost Total		Annual Savings	Age of Project
			per Cu. Ft.	per Tr. Ft.	Total	Before	After		
	ft.	*cu. ft. per track ft.*							*yr.*
E. Ohio	600	38.48	$0.586	$22.52	$13,512	$3,000	Normal	$3,000	2
						man hr.	*man hr.*	*man hr.*	
F. Iowa	325	27.9	0.43	16.02	5,206	2,545	247	2,298	4
G. Texas	1,870	35.14	0.252	8.89	15,700	2,210	122	2,088	3
H. Minn.	815	45.88	0.39	17.99	14,662	3,448	Normal	3,488	2

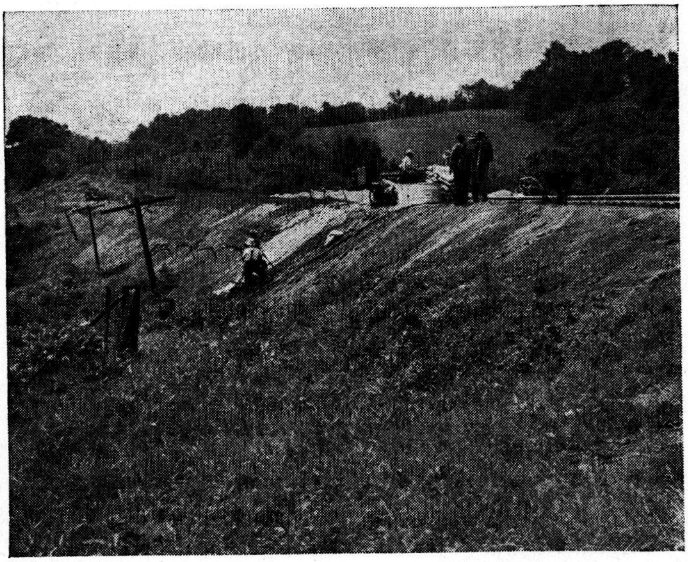


Figure 10. Grouting Operations on Fill

Tests are being continued with clays of very high plasticity and with simulated grout injections and various ballast sections.

PRESSURE CELLS

In 1947 the research staff of the Engineering Division of the AAR made a trial installation of soil pressure cells. This work was of a development nature and led to a design

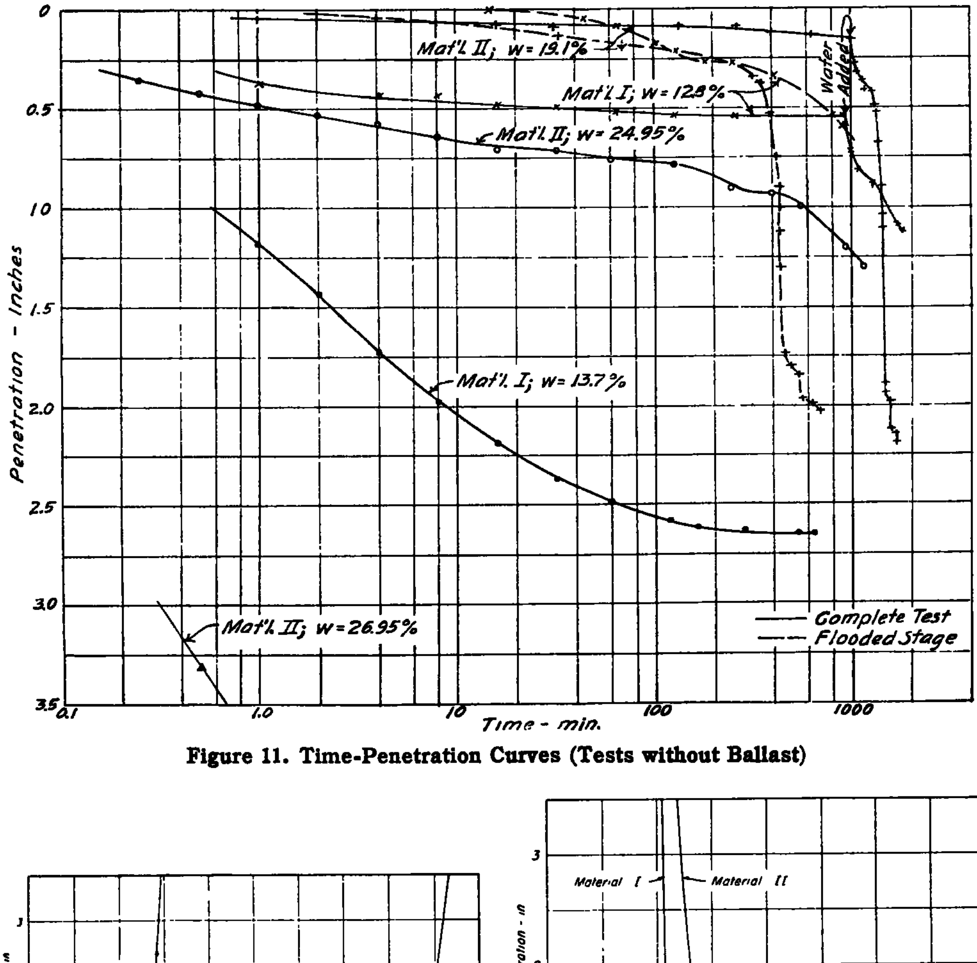


Figure 11. Time-Penetration Curves (Tests without Ballast)

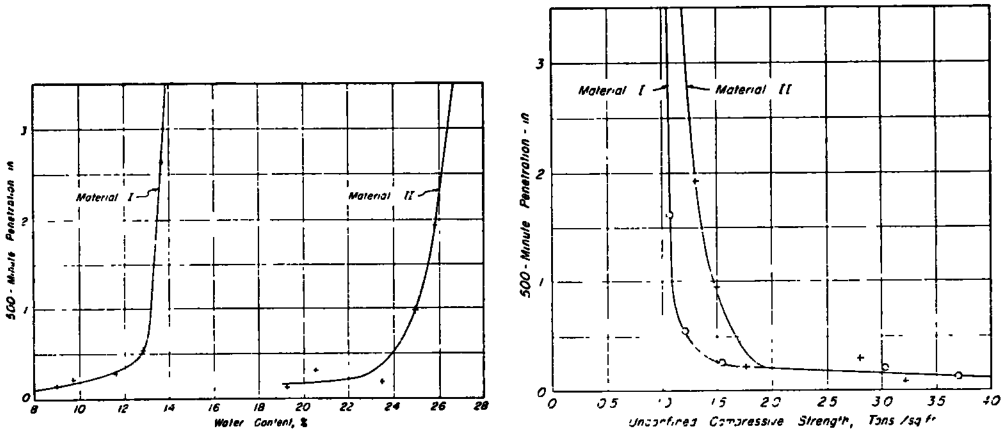


Figure 12. Penetration against Water Content and Compressive Strength (Tests without Ballast)

(Fig. 13) of a cell utilizing four SR-4 strain gages. This cell is designed primarily for the registration of live loads.

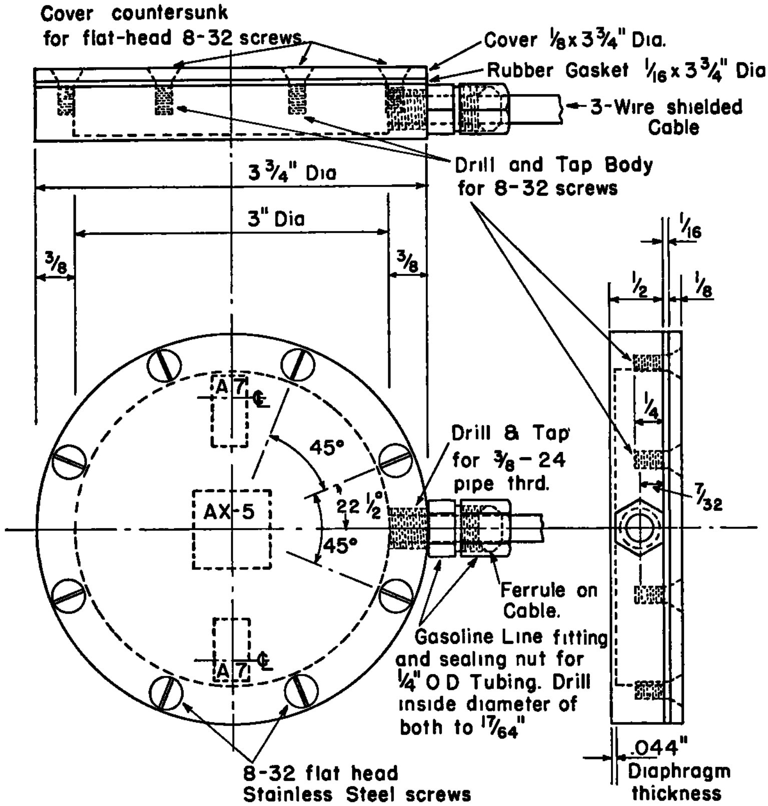
Cells of this type were in place under main line track for 15 months during which time three series of records were obtained from normal rail traffic. These records appeared to register loads adequately except that the intensity averaged 70 percent of theoretical values or less. Upon removal the cells were recalibrated with results closely approximating the original values and it was concluded that arching was still effective over the narrow trench in which the cells had been placed.

For the 1950 installation the trench for cell placement was excavated 9 ft. wide.

Twelve cells were placed 4 ft. below the bottom of the tie, and covered with 6 in. of the lacustrine clay subgrade; five were placed to measure horizontal pressure, and two for lateral pressure 45 deg. downward. A schematic layout of the installation is shown in Figure 14.

To measure tie load simultaneously with soil pressure intensities the rail was shimmed off the tie plates and the tie plates off the tie and two SR-4 strain gages attached to the bottom of the plates. These were so connected as to record the sum of the tie plate stresses for the points at which the gages were attached and thus required only one channel in the oscillograph for each plate. As rail reactions are not equal for each side of the rail and vary from run to run, two gages are required to obtain full tie plate load.

All records were registered on sensitized paper through 22 channels in two oscillographs. Forty-two runs were obtained, 20 before pressure grouting and 22 after grouting. These records were obtained approximately 3½ mo. after installation. Values of pressure intensities and tie loads were obtained by reading the oscillograph records. The values discussed were read at the instant at which the centerline cell registered maximum vertical pressure. Tie loads are simultaneous for thi instant. Full analyses of the data have not been made but a number of relationships



Note:
Body and Cover made of Stainless Steel.
Gages AX-5 and A-7 shown in outline only and are to be cemented to inside face of diaphragm.

Figure 13. Soil Pressure Cell

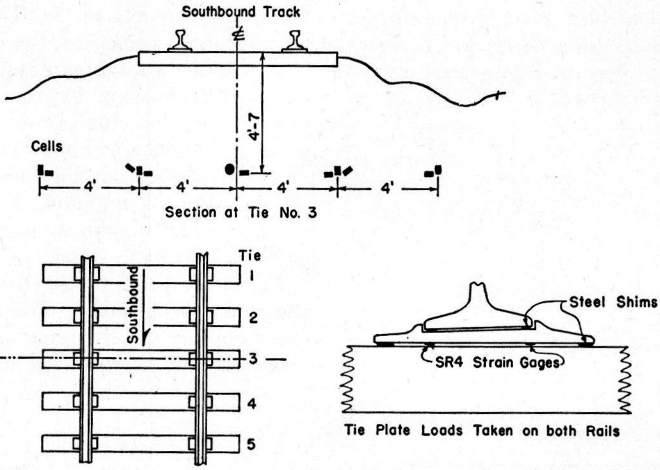


Figure 14. Layout of Pressure Cell System

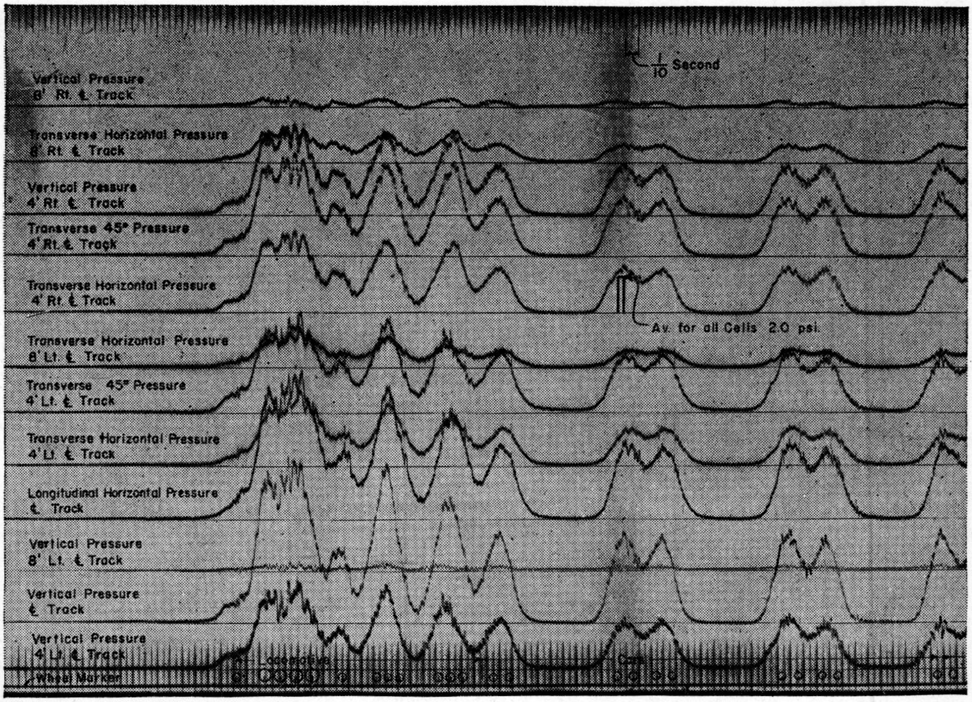


Figure 15. Typical Oscillograph Record for Pressure Cells

have been established as discussed below. Figure 15 shows a typical oscillograph run for the pressure cells.

Through the use of Newmark's Charts (Influence Charts for the Computation of Stresses in Elastic Foundations, Nathan M. Newmark, Bulletin Series 338, University of Illinois Engineering Experiment Station) the theoretical pressures at the position of the cells recording vertical and horizontal stresses were calculated.

In using the charts the total load on each tie was assumed to be transmitted uniformly over the area of its lower surface. To reduce the variations in recorded pressures at any one cell caused by possible speed and track

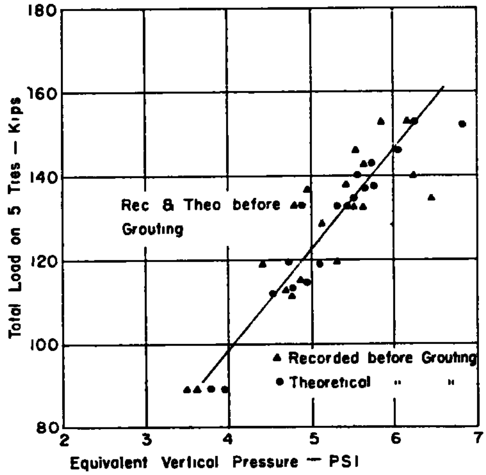


Figure 16. Before Grouting—Relationship of Vertical Pressure to Load

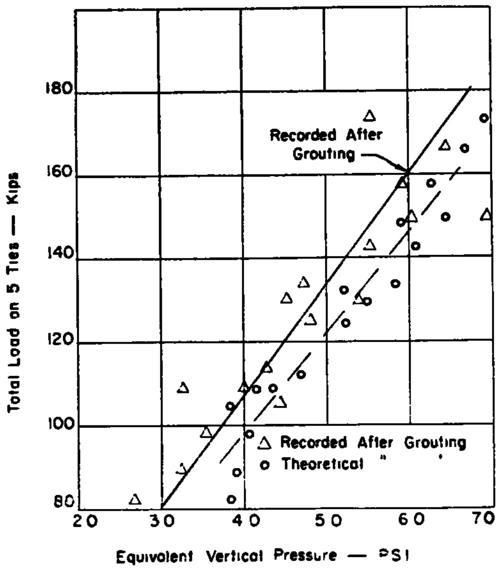


Figure 17. After Grouting—Relationship of Vertical Pressure to Load

effect and also locomotive dynamic augment and to give a more general and valid comparison, the vertical pressures both theoretical and recorded were integrated graphically over the 16-ft. section encompassed by the cells and the average or equivalent pressure plotted as abscissae in Figure 16. The total load on five ties as recorded are the ordinates. There is a considerably greater spread from the average rélationship for the recorded values but the same average proportionality represents both sets of values. Deviation of theoretical values from the curve is caused by the variation of individual ties load from the average. Load on the same tie varied appreciably from run to run.

Figure 17 shows the similar relationship for the runs after grouting. The line for theoretical values naturally remains the same but recorded values decreased about 9 percent.

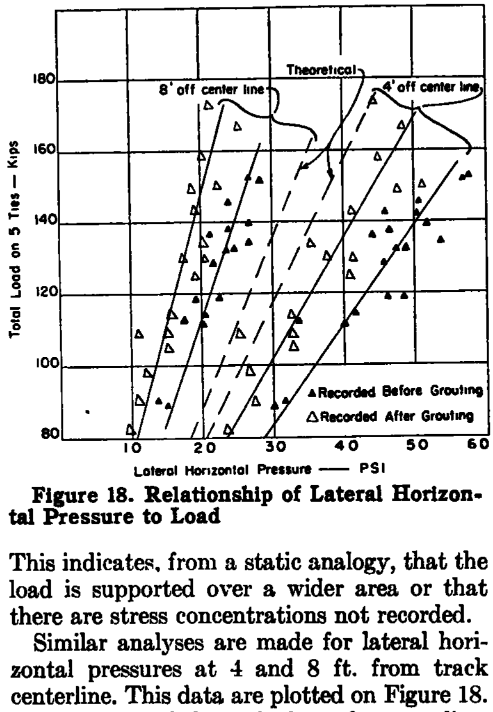


Figure 18. Relationship of Lateral Horizontal Pressure to Load

This indicates, from a static analogy, that the load is supported over a wider area or that there are stress concentrations not recorded.

Similar analyses are made for lateral horizontal pressures at 4 and 8 ft. from track centerline. This data are plotted on Figure 18. It is to be noted that 4 ft. from the centerline, the recorded pressure before grouting, for equal tie loads, is approximately 40 percent greater than the theoretical; 8 ft. from the centerline the recorded pressure is about 20 percent less than the theoretical. The trend of the change after grouting is similar to that developed for vertical pressure but larger proportionally.

This trend is not apparent for longitudinal horizontal pressure at the centerline as plotted in Figure 19.

The spread of recorded pressures before

grouting is great but the average appears to be close to that of recorded pressures after grouting, which conform fairly well to an average relationship as represented by the curve. The theoretical pressures show the

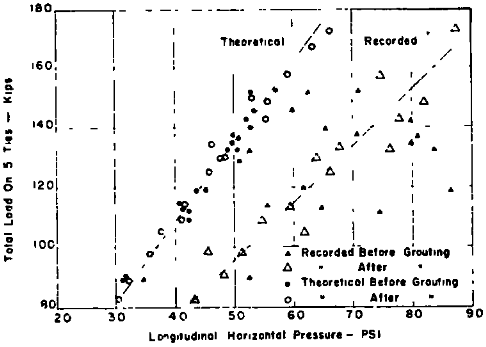


Figure 19. Relationship of Longitudinal Horizontal Pressure to Load

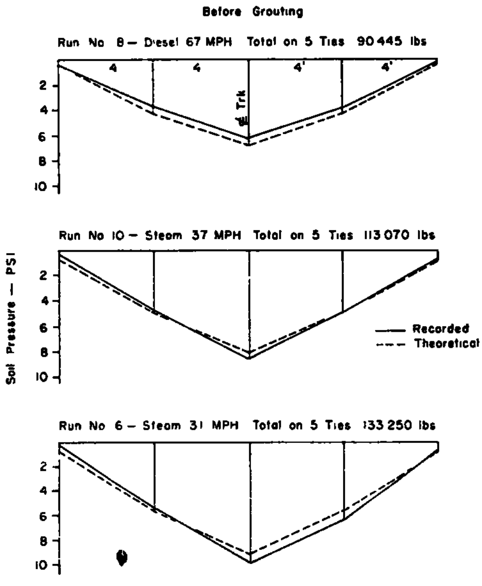


Figure 20. Vertical Pressure Distribution Before Grouting

relationship as indicated by the curve on the left side of Figure 19. This average relationship is approximately 40 percent below the recorded average. To calculate the longitudinal theoretical pressure without serious error it was necessary to compute the influence of eight additional ties on the pressure. As only five tie loads were weighed, the loads on these additional ties were taken as the average for the five loads recorded. For lateral horizontal pressure four additional ties were used similarly. For vertical pressures the five tie loads

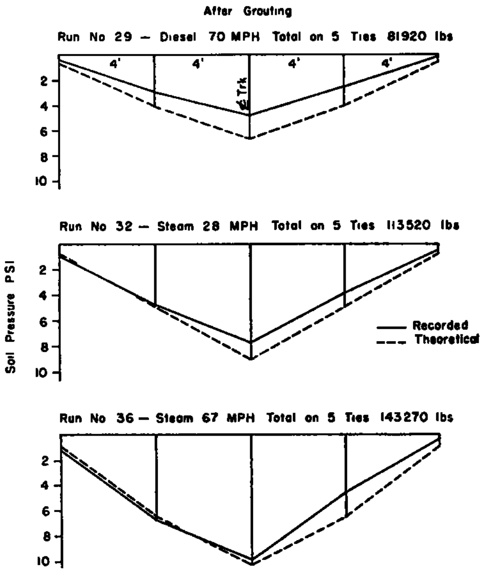


Figure 21. Vertical Pressure Distribution After Grouting

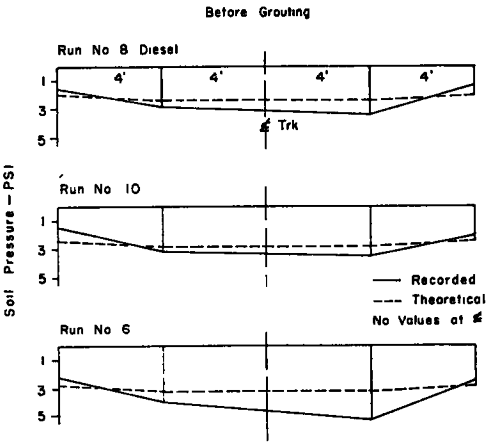


Figure 22. Lateral Horizontal Pressure Distribution Before Grouting

would account for at least 98 percent of the theoretical pressure.

Figures 20 and 21 show the vertical pressure distribution on the section before and after grouting, respectively. Figures 22 and 23 show the lateral horizontal pressure distribution for the same runs. The effect of grouting discussed above can be noted here also.

It is not known if this effect is permanent. Previous experimentation has indicated that it is not. The cells are still in place and additional records will be made in 1951. For the vertical pressures the decrease does not appear sufficient to explain the good results usually obtained. The pressure cells record vertical pressures remarkably close to those computed. With a known load uniformly distributed it appears possible to determine vertical pressure intensities easily and quickly at any depth by use of the charts. If it is assumed that the

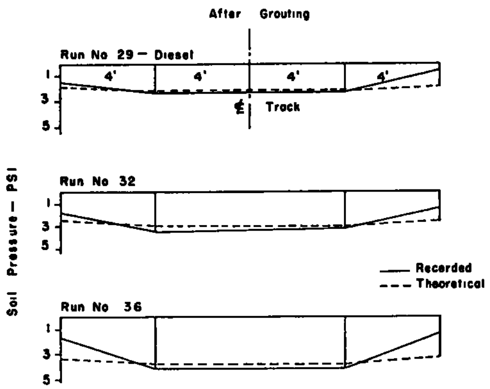


Figure 23. Lateral Horizontal Pressure Distribution After Grouting

pressure cells are recording vertical pressure adequately and these tests so indicate, the horizontal pressures should also be recorded correctly and, if so, the elastic theory may not be fully applicable for this case.

For this particular test the difference between recorded and theoretical pressure may be influenced by the passive resistance of the material which is conceivably much greater at the centerline than at 8 ft. removed. Also, the correct gage installation for horizontal pressure may be more difficult to obtain and hold. The data are too limited for a general conclusion but the deviation of the horizontal pressures is not totally unexpected.

The data obtained include three plane measurements at two points. This will permit analyses of principal stresses and shear stresses. Consideration should also be given to the strains involved.

SUMMARY

These projects are of a continuing nature and as yet have not yielded data from which unqualified conclusions can be drawn. Indicated, however, are the following:

1. With good quality ballast the depth rather than the type largely determines the load on the subgrade.
2. Instability can develop through an appreciable depth of ballast under unit pressures much lower than the indicated strength of the soil material.
3. This instability is largely a failure in bearing of a thin layer at the top of the subgrade subject to water.
4. Instability of roadbed and fills can and often does develop only from precipitation waters falling on the track.
5. Analyses of critical circles do not show cause of most fill failures. Most failures do not follow critical circle. Many fills with low factor of safety show no failing tendency. Borings and tube samples are not sufficient for stability appraisal.
6. Control of moisture and density are not sufficient to insure stable roadbeds. Swell characteristics must be considered.
7. Vibrations produced by traffic affect stability in an as yet undetermined manner.
8. Pressure grouting changes the load distribution of pocketed areas. Character, degree and permanence of the change has not been established. Other factors appear more important in reducing instability. Full explanation of restored fill stability after grouting has not been indicated as yet by the present research. Full scale investigations appear to be required.

ACKNOWLEDGMENT

The investigations discussed in this paper are part of the research work of the Engineering Division of the AAR. They are being conducted in a cooperative agreement with the Engineering Experiment Station of the University of Illinois under the general direction of G. M. Magee, Research Engineer of the Engineering Division and Dr. R. B. Peck, Research Professor of Soil Mechanics of the University. The writer wishes to express his appreciation for the assistance received.

The material in this paper has been drawn from the reports of the Roadway and Ballast Committee published in *Proceedings*, American Railway Engineering Association, Vols. 47 to 52, 1945–1951 incl., and Bulletins of the Engineering Experiment Station.