# TRANSPORTATION RESEARCH BOARD
## National Research Council

## ERRATA
## TRB Publications
*(through March 1989)*

### *Special Report 218, Volume 1*

In the keys to Figures 2-4 (page 30) and 3-1 (page 38), two of the definitions were transposed. The middle curve in Figure 2-4 is for male and female drivers, and the bottom curve is for female drivers. In Figure 3-1 the curve represents fatalities per 10,000 population, and the shaded bars represent thousands of fatalities. The correct figures are shown below:

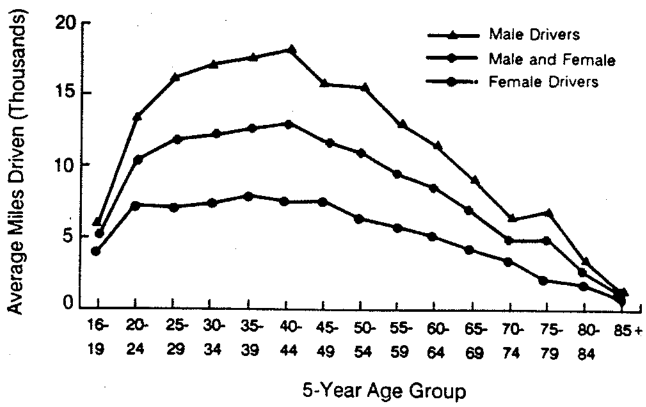


FIGURE 2-4

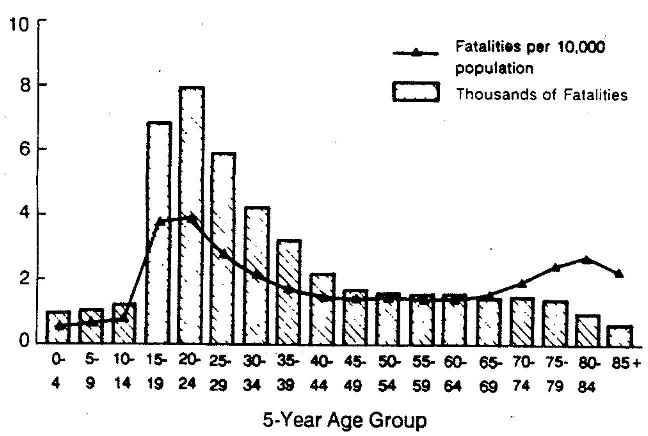


FIGURE 3-1

### *Transportation Research Record 1095*

page 42

Equation 5 should read as follows:

$$\log_{10}(\varepsilon_{yy})_{ov} = -0.689 + 0.793 \log_{10}(\varepsilon_{yy}) - 0.041(H_{ov} + H_1)^{1/2} - 0.057(H_{ov})$$

### *Transportation Research Record 1106*

*Volume 2*

page 225

The second sentence in the abstract should read as follows:

> In the past, a uniform set of geometric design standards for these types of roads was not available in Canada.

page 227

The last item in the bulleted list in column 1 should read as follows:

> One-lane, two-way resource development roads for ADTs up to 100 vpd.

pages 229–232

The figure captions should read as follows:

FIGURE 1 Cross-section elements for two-lane, low-volume earth and gravel roads.
FIGURE 2 Cross-section elements for two-lane, low-volume surfaced roads.
FIGURE 3 Cross-section elements for one-lane, two-way low-volume roads.
FIGURE 4 Cross-section elements for one-lane, one-way low-volume roads.
FIGURE 5 Roadway width versus design speeds of various road agencies (ADT < 50).
FIGURE 6 Roadway width versus design speeds of various road agencies (ADT 50 to 100).
FIGURE 7 Roadway width versus design speeds of various road agencies (ADT 100 to 150).
FIGURE 8 Roadway width versus design speeds of various road agencies (ADT 150 to 200).

*Supplement*

page 50

In the paper by Faiz and Fossberg, references 3 and 4 were transposed. The last two references in the paper should read as follows:


3. *Road Deterioration in Developing Countries.* Report 6968. Infrastructure Department, World Bank, Washington, D.C., Oct. 15, 1987.
4. M. Mason. *Axle Loading Study.* Internal report. Transportation, Water and Telecommunications Department, World Bank, Washington, D.C., 1981.


### *Transportation Research Record 1122*

page 30

The last two columns in Table 2 are incorrect. The correct Table 2 is shown at the top of the next page.

TABLE 2 SITES WITH MERGING-RELATED ACCIDENTS

| Site No. | Climbing Lane Length and AADT | Vertical Alignment | Horizontal Alignment | Sight Distance | Passing Ahead | Accidents[a] Total and Rate | Merging-Related |
|---|---|---|---|---|---|---|---|
| 1 | 0.13 mi<br>3,400 | Up 8.5%<br>No crest | Tight curve | Restricted | Very Restricted | 20<br>1,612[b] | 1<br>#1[c] |
| 2 | 0.73 mi<br>9,725 | Up 6.0%<br>No crest | No curves | Excellent | Restricted | 10<br>282[b] | 1<br>#2[c] |
| 3 | 0.81 mi<br>9,725 | Up 5.0%<br>Crest | After a curve | Good | Average | 12<br>364[b] | 1<br>#3[c] |
| 4 | 0.16 mi<br>11,000 | Up 5.9%<br>No crest | Slight curve | Good | Restricted | 17<br>466[b] | 5<br>#4–8[c] |
| 5 | 0.22 mi<br>2,200 | Up >5%<br>No crest | In middle of tight curve | Restricted | Restricted | 6<br>747[b] | 1<br>#9[c] |
| 6 | 0.28 mi<br>2,200 | Up >5%<br>No crest | In curve | Restricted | Very Restricted | 4<br>498[b] | 2<br>#10–11[c] |

a - Accidents within ±0.10 mile of end of merging taper, 1980-84.
b - Accidents per $10^8$ vehicle miles.
c - Numbers refer to accidents described in text.
Source: California Department of Transportation photolog, site plans, correspondence, TASAS, and (6).

page 34
The first sentence in the last paragraph in column 1 should read as follows:

For car speeds of 38 mph, truck speeds of 22 mph, and speeds of other slow vehicles of 26 mph, the following results are obtained.

### *Transportation Research Record 1131*

The paper by Lacy and Pannee (pp. 99–106) was sponsored by the Committee on Engineering Fabrics.

### *Transportation Research Circular 330*

Portions of the original publication were printed in an incorrect sequence. A revised photocopy of the circular is available on request from the Business Office, Transportation Research Board, 2101 Constitution Avenue, N.W., Washington, D.C. 20418 (telephone 202-334-3218).

### *NCHRP Synthesis of Highway Practice 138*

page 4
The data in Table 2 are incorrect. The following table should be used in place of Table 2:

| Speed (mph) | Speed (km/h) | Stopping Sight Distance (ft) | Stopping Sight Distance (m) |
|---|---|---|---|
| 30 | 48 | 200 | 61 |
| 40 | 64 | 325 | 99 |
| 50 | 80 | 475 | 145 |
| 60 | 97 | 650 | 198 |
| 70 | 113 | 850 | 259 |

### *NCHRP Synthesis of Highway Practice 139*

page 61
In reference 1, the date for "Accident Facts," published by the National Safety Council, should be 1986.

# 1131

TRANSPORTATION RESEARCH RECORD

# *Performance of Aggregates in Railroads and Other Track Performance Issues*

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1987

**Transportation Research Record 1131**
Price $15.50
Editor: Elizabeth W. Kaplan

mode
3 rail transportation

subject areas
21 facilities design
35 mineral aggregates
40 maintenance

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# Transportation Research Record 1131

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# Contents

# Foreword

The first six papers and associated appendix in this record deal with topics related to the performance of unbound rock aggregates used as foundation materials for transportation systems. These papers emphasize field and laboratory testing methods, along with their interpretations, used to identify and predict the behavior of these materials under service conditions.

Selig and Roner present information on the stress-strain-strength properties of railroad ballast. The experimental results of a triaxial test series using various crushed granular materials are given. The effects of particle size, shape, gradation, and angularity are reviewed.

Raymond and Bathurst have investigated the permanent deformation characteristics of ballast under cyclic loading using large-scale laboratory simulations of a single tie-ballast system. Comparisons are made between tests with and without a reinforcing grid in the ballast layer.

Boucher and Selig discuss the utility of petrographic analysis for evaluating ballast. Comparisons are made between Los Angeles and mill abrasion values and cementing indexes and interpreted petrographic data. They indicate that petrographic analyses are useful to supplement commonly used index tests.

The papers by Clifton et al., Klassen et al., and Watters et al. represent a coordinated effort to develop a methodology for specifying railroad ballast. Clifton et al. describe a method for the selection of rock sources used to produce ballast. The approaches to source selection, site investigation, quarry design, and quality control are based on geologic characteristics. Watters et al. present a variety of petrographic characteristics of major ballast types currently in use. The characteristics are discussed in light of material performance, as determined from field test data. Klassen et al. give a history of ballast maintenance and replacement on the Canadian Pacific Railroad. An extensive field sampling and analysis program is detailed, the results of which led to the development of a new ballast specification. Criteria used to develop the specification include abrasion resistance, weathering characteristics, and rock stability. The final specifications reflect a long-term evaluation process, which involved the work reported by Clifton et al. and Watters et al.

The objective of these papers is to describe criteria used to identify or predict the performance of unbound rock aggregates, specifically railroad ballast. These papers should serve as a significant reference source to those charged with the selection, specification, and evaluation of these materials.

The papers by Raymond, Buekett et al., and Harrison and Ahlbeck deal with track structure performance under 125-ton cars and high dynamic forces. Raymond considers the effect of increasing axle loads, associated with 125-ton cars, on subgrade and ballast performance. Preparation of the subgrade for both new track construction and upgrading existing track to withstand the stresses caused by increased axle loads is discussed.

Buekett et al. considered the effects of 125-ton cars on individual track components. Because ties and fasteners provide the greatest opportunity for designing cost-effective track, emphasis is placed on the dynamic behavior of ties and how it is influenced by impact loading from 125-ton cars and other equipment.

Harrison and Ahlbeck discuss results of recent studies of concrete-tie track structural dynamics in the context of wheel-rail impact loading. The use of concrete ties by heavy-haul railroads has increased with the growing use of 125-ton cars.

The paper by Uzarski et al. deals with the development of a maintenance management system to be used in planning maintenance and rehabilitation of the U.S. Army's 3,000 track miles at 81 installations. The basic first steps of component identification and inventory are discussed along with a method for field data collection.

Lacy and Pannee describe an innovative approach for track structure support that accommodates low-strength organic clay and high groundwater beneath the tracks. A combination of geomembranes and geotextiles was used to maintain groundwater at or near the prevailing level to avoid settlement of adjoining structures and to provide a dry, stable foundation for the tracks.

Harry E. Stewart and
Elaine King

# Acknowledgments

Special recognition is given to Harry E. Stewart, a member of the Committee on Railway Maintenance, for his work in assembling the papers related to the performance of aggregates as foundation materials in railroad track beds. Mr. Stewart oversaw the peer review of these papers and collected the final manuscripts for publication.

Recognition is also given to James W. Winger, Chairman, Committee on Railroad Track Structure System Design, for his efforts in organizing the papers related to track structure performance under 125-ton cars and high dynamic forces.

# Effects of Particle Characteristics on Behavior of Granular Material

ERNEST T. SELIG AND CARL J. RONER

**The stress-strain and strength characteristics of railroad ballast are known to be functions of particle shape, size, and gradation. However, the nature and extent of these effects are only partly understood, and opinions are sometimes contradictory. For this reason a review was undertaken of available literature on granular materials relevant to this subject. To supplement this information, triaxial tests were conducted on specimens of crushed granular material with varying sizes and shapes of particles to observe the effects on the strength of the assembly particles. Any amount of flaky particles added to a specimen or crushed granular material, either randomly placed or oriented, increased the shearing resistance except when the particles were oriented generally parallel to the shear plane. However, the presence of flaky materials resulted in greater particle abrasion and breakage, larger permanent strain under repeated loading, and lower stiffness. Increasing particle angularity increased shearing resistance, but particle breakage also increased. Increasing particle surface roughness increased shear strength. Broadening the gradation of the specimen increased shear strength. However, this effect may be primarily because the void ratio typically decreases as the range of particle sizes increases. For the gradations tested, shear strength was independent of gradation for common void ratios. Similarly, particle size had no significant effect on shear strength within the range tested for common void ratios. Broadening the gradation also decreased permanent strain under repeated loading and decreased particle degradation.**

Industry specifications for railroad ballast contain some criteria related to particle size, shape, angularity, and grading. However, the functions of ballast impose conflicting requirements on these parameters. Furthermore, the influence of these parameters is only partly understood. This leads to uncertainty about what the most appropriate ballast particle specifications might be. Results of a study to provide a better understanding of some of the related issues are presented.

A literature review was first conducted to assemble available information on effects of particle size, shape, angularity, and gradation on the mechanical behavior of crushed angular material. Emphasis was on shear strength characteristics of assemblies of particles and associated particle breakage. In most cases only a single loading or a few load cycles were involved so that wear and abrasion often were not a significant consideration.

To supplement information in the literature, triaxial tests to failure were performed on ballast specimens with varying particle sizes, shapes, and gradings. Information was obtained on the relationship of these factors to shear strength and volume change behavior.

E. T. Selig, Department of Civil Engineering, University of Massachusetts, Amherst, Mass. 01003. C. J. Roner, International Technology Corp., Martinez, Calif. 94553.

## PAST WORK

### Particle Shape

Parameters that have been used to define particle shape include flakiness or flatness (*1,2*), elongation, (*1,2*), sphericity (*3,4*), and roundness or angularity (*3,4*). Flakiness or flatness refers to the ratio of particle thickness to width (intermediate dimension), and elongation refers to the ratio of length to width. Sphericity is a measure of how much the shape of a particle deviates from a sphere. Finally, roundness or its inverse, angularity, is a measure of the sharpness of the edges or corners of an individual particle. Some information was found in the literature on the effects of each of these factors except elongation.

Dunn and Bora (*5*) tested a hard, crushed limestone aggregate in a special triaxial device. The particle size ranged from 3/16 in. (4.8 mm) to 1 1/2 in. (38 mm) and the percentage of flaky particles (*1*) was varied from zero to 100 percent of the specimen. Any amount of flaky particles increased the shear strength, but the results suggest that the range of 25 to 75 percent flaky particles is better than 100 percent.

Gur et al. (*6*) used several different tests to evaluate the effect of flakiness on crushed material (type not indicated) ranging in size from 1/4 in. (6.3 mm) to 3/4 in. (19 mm). The aggregate crushing value (*1*) increased 2 1/2 times as the amount of flaky particles increased from zero to 100 percent. Over the same range the Los Angeles abrasion resistance value increased four times. Increasing the percentage of flaky particles also increased the amount of breakage during compaction. Shear strength from triaxial tests was greater with flaky material than with nonflaky material. As a final test the aggregate was compacted in a box and then subjected on the surface to 9,000 coverages with a rubber wheel. Rutting of the flaky material was roughly twice that of the nonflaky material. The increased rutting was attributed to particle alignment of the flaky material.

Siller (*7*) found that the apparent cohesion intercept from triaxial tests on railroad ballast increased with increasing flakiness. He attributed this to increased particle interlock.

Eerola and Ylosjoki (*8*) found that the triaxial shear strength of aggregate specimens increased in proportion to the ratio of particle length to thickness.

In a European study (*9*) modulus of elasticity was calculated from loading tests on ballast specimens confined by steel rings. Ballast with flat particles generally gave a lower modulus of elasticity than did ballast with equidimensional particles.

Chen (*10*) conducted triaxial tests on various sands and gravels with density ranging from loose to compact. Modulus was determined from the 25th cycle with loading to 30 percent of estimated strength. Strength was subsequently measured by loading to failure. The modulus decreased with increasing angularity, but the strength increased.

Holtz and Gibbs (*11*) conducted triaxial tests on subangular to subrounded gravel and on sharp, angular, crushed quartz rock with similar grading curves. As expected, the angular material had a higher strength.

Vallerga et al. (*12*) conducted triaxial tests on both subrounded and angular sandstone aggregate at various void ratios. The strength of the angular aggregate was significantly higher than that of the subrounded aggregate. At the same void ratios the difference in angle of internal friction was 4 to 8 degrees. However, at the same compaction effort the difference was reduced to about 1 degree. These authors also conducted triaxial tests on glass beads etched to provide a range of surface roughness. With increasing surface roughness, strength, as measured by angle of internal friction, increased by about 8 degrees.

Pike (*13*) conducted shear-box tests on 17 aggregates ranging in particle size from fine sand to coarse gravel. Increased angularity or surface roughness generally resulted in reduced dry density for the same compactive effort, but strength still increased.

Holubec and D'Appolonia (*14*) showed that increasing angularity of sands significantly increased shear strength.

Edil and Luh (*15*) showed that the shear modulus of sands, determined in the resonant column test, increased with increasing roundness (decreasing angularity).

Norris (*16*) showed that the shear strength of sand decreased significantly with increasing roundness. The same trend was shown by Frossard (*17*).

Koerner (*18*) conducted triaxial tests on sands of equal grading but different particle angularity and sphericity. He separated the measured effective angle of internal friction into frictional and dilational components. The dilational component was approximately independent of angularity whereas the frictional component increased significantly with increasing angularity or decreasing sphericity for common relative densities. The same trends had been previously shown by Kolbuszewski and Frederick (*19*).

George and Shah (*20*) compared triaxial test results on rounded gravel with the results on the same gravel after waxing the particles. The waxed particles produced a substantially lower strength. These researchers also compared the triaxial results for platy crushed limestone with results for chunky crushed limestone. Although the same peak strength was obtained, the stiffness was less for the platy specimens. The authors did not indicate whether these comparisons were made at common void ratios.

**Particle Gradation and Size**

Roenfeld (*21*) conducted repeated load triaxial tests on limestone ballast. One set of specimens had a narrow range of particle sizes [coefficient of uniformity (CU) = 1.14]; the other set had a broader grading (CU = 4.1) but a slightly smaller mean size. The cumulative plastic strain for the uniform ballast was almost double that for the more broadly graded ballast. Furthermore, the particle degradation for the uniform ballast was four to five times greater than that for the more broadly graded ballast.

Klugar (*22*) reported that replacing a small amount (<15 percent) of large particles in a ballast specimen by smaller particles increased the friction angle in shear-box tests and that replacing a greater amount (>20 percent) caused a significant reduction in the friction angle mobilized at a given displacement.

Rico et al. (*23*) tested three different gradations of crushed basalt in a Texas triaxial cell. The comparisons were made at equal compactive effort. Thus void ratio and unit weight were not identical. A relatively uniform gravel size specimen gave a much higher strength that did a relatively uniform sand size. For the same top particle size (1.5 in. or 38 mm), a more broadly graded specimen gave a higher strength than did a narrowly graded specimen.

Marsal (*24*) tested rockfill dam materials with a maximum particle size of about 6 in. using a large, high-pressure triaxial cell. He showed that shear strength increased as the gradation became broader for the same top size, even though the mean size decreased. Marachi et al. (*25*) came to the same conclusion after making tests with common compactive efforts, although mineral composition and void ratio were not identical for all specimens.

Chen (*10*), in tests previously described, found that increasing the coefficient of uniformity increased shear strength but did not affect stiffness.

Kirkpatrick (*26*) conducted triaxial tests on a sand with three different gradings in which the top and bottom sizes were kept constant while the mean size was varied. In general, shear strength increased as mean size decreased.

Leslie (*27*) conducted triaxial tests on gravelly soils in which the minimum particle size was kept constant while the maximum size was increased, which increased the mean size and broadened the gradation. For comparable compactive effort, as mean size increased shear strength decreased.

Triaxial tests on sands by Koerner (*18*) showed little effect on shear strength of changes in coefficient of uniformity for common void ratios. In some cases increasing the coefficient of uniformity at common relative densities increased shear strength. However, strength increased as particle size decreased.

Triaxial tests by Marachi et al. (*25*) showed that strength increases as particle size decreases. However, Dunn and Bora (*5*), in triaxial tests previously described, found that shear strength increased with increasing particle size. Finally, triaxial tests by Vallerga et al. (*12*) and by Holtz and Gibbs (*11*) showed little effect of particle size on strength. Thus the effect of particle size on strength is unclear. The same conclusion was indicated by results of direct shear tests summarized by Roner (3).

## DESCRIPTION OF TEST

To supplement results available in the literature, triaxial tests were conducted by Roner (3) on nonflaky specimens of several sizes and gradings and on flaky specimens with both random and parallel particle orientations. The triaxial tests used specimens 8 in. (203 mm) in diameter by 20 in. (508 mm) high. Although a larger size would have been preferable, this was the largest size that could be used with available apparatus.

The specimens were contained in a rubber membrane, and a constant confining pressure was applied through water. The ballast voids were also filled with water under constant back pressure so that volume change could be determined from the amount of water flowing into or out of the specimen during the test. The effective confining pressure for all tests was 5 psi (34 kPa). Loading was provided by a compression machine at a constant rate of deformation of about 0.1 in./min (2.5 mm/min).

The primary ballast used in the tests was a crushed quartzite. Its Los Angeles abrasion resistance was 20 and its mill abrasion value was 1.9. Because the quartzite did not contain sufficient flaky particles, additional flaky particles were obtained from a sample of gneiss that had a Los Angeles abrasion resistance of 23 and a mill abrasion value of 2.0. Four ballast gradings were prepared:

| | *Size Range* | | |
|---|---|---|---|
| *Grading* | *Inches* | *Millimeters* | *Coefficient of Uniformity* |
| 1 | 1 1/2-1/2 | 38-13 | 1.47 |
| 2 | 1 1/2-1 1/8 | 38-29 | 1.03 |
| 3 | 1 1/8-3/4 | 29-19 | 1.03 |
| 4 | 3/4-1/2 | 19-13 | 1.03 |

Gradings 2, 3, and 4 are parallel and connected. Grading 1 consists of equal parts by weight of the other three gradings.

The regular (nonflaky) ballast specimens had 6 percent by weight of flaky particles according to British Standard 812 (1) but none according to the Corps of Engineers (2), which was representative of the composition of the quartzite ballast as received. The first method defines flaky particles as having a width-to-thickness ratio greater than 1.7, and the second method uses a width-to-thickness ratio greater than 3. Thus the second is about twice as flat as the first.

The flaky ballast tested was composed of 96 percent by weight of flaky particles according to British Standard 812 (1) but only 4 percent by weight of flaky particles according to the Corps of Engineers (2). The American Railway Engineering Association, before 1986, combined flat and elongated particles into a single definition that required the length-to-thickness ratio to be greater than 5. The flaky specimen had only about 3 percent by weight of flat and elongated particles by this definition. Only Grading 1 was used for the flaky specimens because of the limited amount of flaky material available.

A range of void ratios was achieved by hand placing the particles and tamping, as needed, using a rubber-tipped falling-weight device (28). The flaky particles were placed either randomly or all parallel. The parallel plane was oriented at zero, 45, and 65 degrees from horizontal, the latter representing the approximate orientation of the failure plane. Little variation in the void ratio could be obtained with the oriented specimens.

The void ratio of the completed specimens was obtained by determining the volume of water required to fill the voids while the specimen was still in the forming mold. The error caused by water adsorption by the ballast particles was sufficiently small to be neglected. However, a large error is introduced because the specimen is formed in a smooth-walled cylinder. This results in the boundary voids being too large and hence an overestimate of the true in situ void ratio. Because no procedure has been developed for boundary voids correction, this error was ignored. However, this is largely a systematic error that should not significantly affect comparisons done with the same sized mold for the ranges of particle sizes considered. The details of the test apparatus and procedures are given elsewhere (3).

## TEST RESULTS

The maximum deviator stress (failure state) and corresponding angle of internal friction are shown as a function of initial void ratio in Figures 1 and 2, respectively. No significant

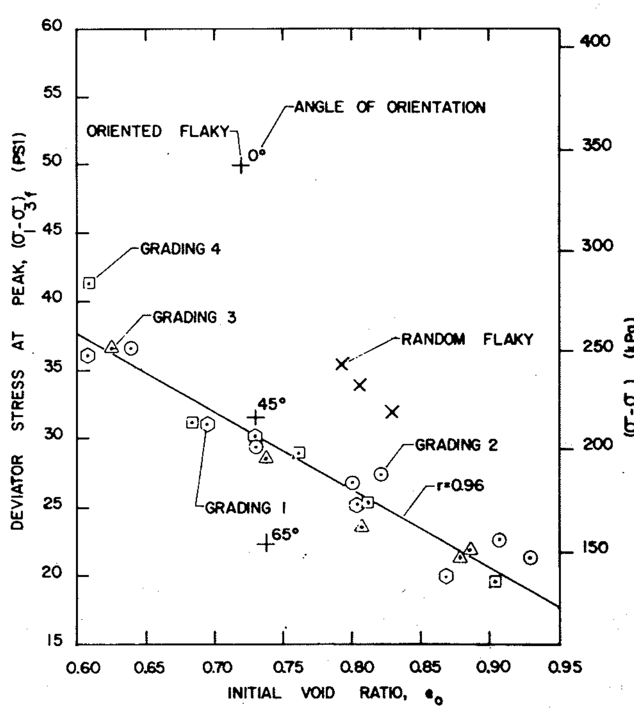


**FIGURE 1 Effect of initial void ratio on deviator stress at failure.**

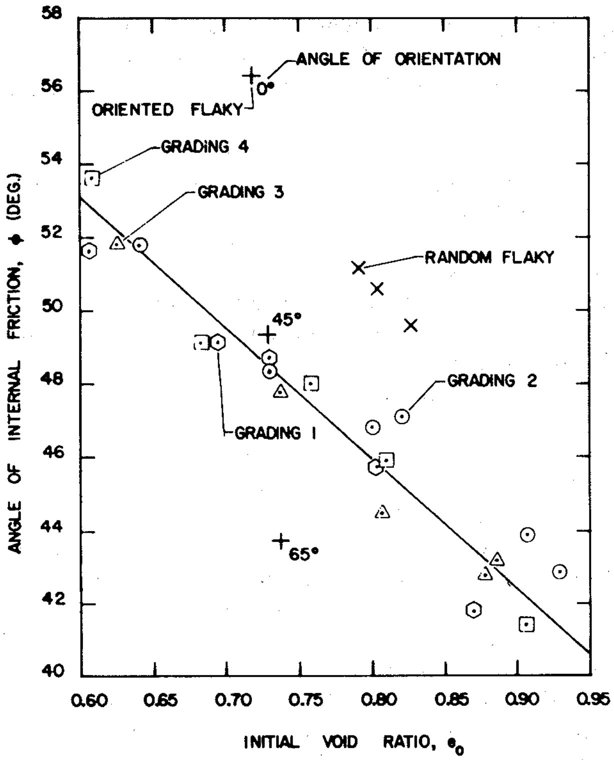


FIGURE 2 Effect of initial void ratio on angle of internal friction.

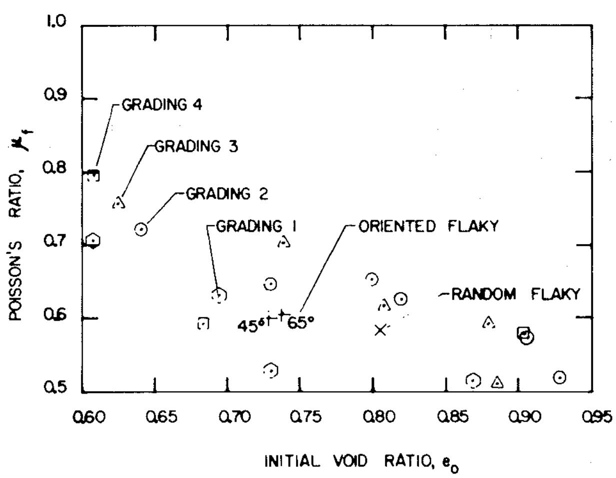


FIGURE 3 Effect of initial void ratio on Poisson's ratio at failure.

influence of particle size or gradation is evident for the nonflaky specimens within the range represented.

Volume change and axial strain at failure were used to calculate the secant Poisson's ratio at failure. The resulting values are shown as a function of initial void ratio in Figure 3. Poisson's ratio decreases with increasing void ratio. All values exceed 0.5, signifying dilation even for the uncompacted specimens (highest void ratios).

The maximum deviator stress and angle of internal friction for the flaky specimens are also shown in Figures 1 and 2. The random flaky specimens had a strength that is significantly greater than the strength at the corresponding void ratios for the nonflaky specimens. The flaky specimen with horizontally oriented particles had by far the greatest strength of all specimens. Only when the particles were oriented parallel to the failure plane did a substantially lower strength occur for the flaky specimens.

The Poisson's ratio values for all of the flaky specimens were consistent with those for the nonflaky specimens at the same void ratio (Figure 3). Three flaky specimens are missing because of problems with volume measurement. It is presumed that these values would also have been consistent.

## SUMMARY AND CONCLUSIONS

The main problem in drawing conclusions about the individual effects of particle size, shape, texture, and gradation on aggregate specimen performance is that tests to determine these trends often involved simultaneous changes in several significant variables. For example, changes in size may also have included changes in particle composition, void ratio, gradation, or angularity. In the following summary the effects of these independent factors are separated as well as possible by considering the conditions associated with all of the reviewed data as well as the supplemental tests conducted in this study. Some of the trends need further verification.

### Shape

Any quantity of flaky particles, either randomly oriented or oriented other than generally parallel to the failure plane, increases the shear strength of the granular specimen. Orientation parallel to the failure plane, when a significant proportion of the particles is flaky, will cause a substantial reduction in strength. The disadvantage of increased flakiness appears to be increased abrasion, increased breakage, increased permanent strain accumulation under repeated load, and decreased stiffness. A further study of these factors is needed.

Increased particle angularity, as is well known, increases shear strength. However, particle breakage increases and specimen stiffness decreases as well. At the same compactive effort, a more angular material will tend to form a higher void ratio that will result in less strength increase than at a common void ratio.

Increasing particle surface roughness increases the shear strength of the assembly of particles. As is the case with angularity, this effect is greater at a common void ratio than at a common compactive effort.

No conclusions about the effects of elongation could be drawn from the literature.

### Gradation

Strength was not affected by the gradation changes investigated by the writers for common void ratios. However, broadening the gradation by increasing the range of particle sizes with the same mean size should increase shear strength as a result of decreased void ratio that would exist with a broader grading.

Additional benefits of broadening gradation are decreased cumulative plastic strain under repeated loading and decreased particle degradation.

### Size

The literature gives contradictory information about the effects of particle size on shear strength. Available results in some cases show an increase, in some cases a decrease, and in some cases no effect with increase in particle size. For the range tested by the writers, mean particle size did not appear to affect strength when compared at common void ratios.

### Volume Change

For all cases tested by the writers, the ballast specimens expanded in volume as a result of shearing to failure. This means that the secant Poisson's ratio at failure exceeded 0.5 even for the uncompacted specimens. No distinctive effect on the relationship between Poisson's ratio and initial void ratio was detected as a result of changes in particle size, shape, or gradation.

### Application to Ballast

On the basis of strength considerations, broader ballast gradations are better than narrow gradations. However, other factors such as durability, fines storage volume, and size segregation must also be considered in selecting gradation. More research on these factors is needed before an optimum grading can be established. Available information does not provide a basis for choosing an optimum ballast size. Factors other than stress-strain and strength characteristics may therefore govern the choice of size. Such factors include particle durability and the movement and storage of fines within voids.

Until 1986 the AREA specifications (*29*) for ballast limited the amount of flat or elongated particles (length-to-thickness ratio greater than 5) to 5 percent by weight. The flaky ballast tested by the writers is acceptable by this standard. However, only extremely distorted shapes would be eliminated by this specification. In 1986 the AREA adopted the Corps of Engineers definitions (*2*) for defining flat (width-to-thickness ratio greater than 3) and elongated (length-to-width ratio greater than 3) particles. The percentage of such particles allowed remains 5 percent by weight. This requirement appears to be more restrictive, and hence better, than the previous AREA requirement, but the flaky ballast tested still is acceptable under this standard.

More research is needed to determine the important reasons for limiting the use of flaky ballast particles in track. Until such research is completed, the best shape criteria and proper limits will not be known. However, if the effects of particle orientation on strength and the effects of shape on stiffness that were indicated in this paper for flaky ballast are to be avoided, an even more restrictive definition of shape will be required. The British standard should then be adopted although perhaps a larger amount of such particles than 5 percent by weight could be allowed.

## ACKNOWLEDGMENTS

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# Performance of Large-Scale Model Single Tie-Ballast Systems

GERALD P. RAYMOND AND RICHARD J. BATHURST

**Large-scale models of a single tie-ballast system were constructed over artificial ballast support that had variable compressibility ranging from rigid to very flexible (California bearing ratio = 10). Test configurations included a 0.45-m depth of crushed limestone ballast conforming to an American Railway Engineering Association grading No. 4. A steel footing 920 mm long by 250 mm wide by 150 mm deep was used to model the bearing area of a typical tie below the rail seat (i.e., one rail). Each rail seat was subjected to a repeated load of between 20 and 150 kN for a (typical) duration equivalent to 12 million gross tonnes in track. The principal objectives of the experimental work were to investigate the influence of load level and ballast support compressibility on the rate of accumulation of permanent deformations and ballast degradation. The test results show that at a given load level the rate of tie settlement is quite sensitive to ballast support compressibility. A competent ballast support resulted in a deformation-log tonnage response that was essentially linear. However, progressively weaker supports gave increasing semilogarithmic rates of settlement with tonnage. For a given support compressibility, a critical load level was identified that, if exceeded, led to a dramatic increase in settlement rate. The critical load level was also identified as a threshold level above which the generation of fines in the ballast directly below the tie was observed to increase markedly.**

Under repeated tie loading, railway ballast undergoes nonrecoverable vertical deformations mostly due to ballast densification, aggregate degradation, and lateral spread of ballast beneath the ties. The current research is part of an ongoing Queen's University and Royal Military College research program directed at correlating aggregate quality, load level, and ballast support compressibility with track performance. The long-term goal of this research is to arrive at a design methodology that includes ballast quality in the forecasting of track performance. A parallel investigation by the authors related to geogrid-reinforced ballast models is reported elsewhere (*1*).

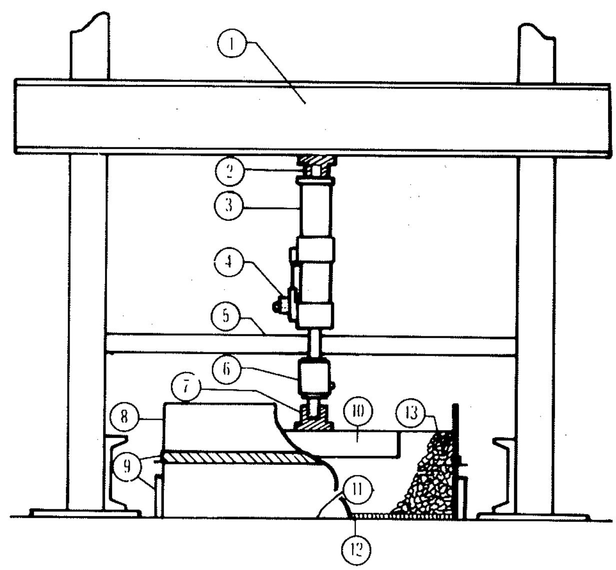
## OBJECTIVES

Large-scale models of a single tie-ballast system over artificial subballast-subgrade support (hereafter referred to as artificial support) were built and subjected to a program of repeated loading. The principal objectives of this study were to

1. Investigate the influence of ballast support compressibility on the load-deformation response of a crushed limestone aggregate,
2. Examine the influence of peak load level on the deformation response of the ballast material, and
3. Determine the influence of both ballast support compressibility and peak load level on the degradation of the limestone ballast under repeated loading.

## GENERAL TEST ARRANGEMENT

The general test arrangement is shown on Figure 1. A 450-mm depth of crushed limestone ballast was confined within a




1 LOADING CROSS BEAMS (2 MC 460 x 63.5)
2 UPPER SWIVEL JOINT
3 MTS HYDRAULIC ACTUATOR / INTERNAL LVDT
4 SERVO CONTROL VALVE
5 ACTUATOR GUIDE & INSTRUMENTATION SUPPORT BEAM
6 LOAD CELL
7 LOWER SWIVEL JOINT
8 PLYWOOD BULKHEAD
9 BULKHEAD SUPPORTS
10 STEEL LOADING TIE (920 mm x 250mm x 150 mm)
11 CONCRETE FLOOR
12 ARTIFICIAL SUPPORT
13 AREA #4 BALLAST

**FIGURE 1 General test arrangement.**

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rigid test box 3 m long by 1.5 m wide. A range of ballast support stiffnesses was incorporated into the test sections by placing ballast over different artificial support materials. A perfectly rigid condition was simulated by placing ballast material directly over a concrete floor. For compressible ballast support models, rubber mats of variable stiffness were placed over the concrete floor.

A steel footing 920 mm long by 250 mm wide by 150 mm deep was used to model the bearing area of a typical tie below the rail seat (i.e., one rail). The tie was placed within a compacted ballast layer to a depth of 150 mm to simulate typical track structure.

The footing was loaded by a computer-controlled closed-loop electrohydraulic actuator that applied a 20- to 150-kN repeated load to the rail seat for a (typical) duration equivalent to 12 million gross tonnes (MGTs) of axle loading in track. The equivalent axle tonnage was calculated by summing the number of load repetitions and multiplying by twice the applied load.

## TEST DETAILS

### Ballast

Crushed limestone aggregate (Sunbury limestone) was used for all test configurations. The aggregate was screened close to an American Railway Engineering Association (AREA) No. 4 grading and washed. The AREA No. 4 grading has a size distribution between about 50 mm (2 in.) and 10 mm (3/8 in.) (*2*). These gradation limits and the mean particle size distribution for the test ballast are shown in Figure 2. The ballast depth below the footing was 300 mm, which corresponds to the minimum recommended depth for new construction according to the AREA. The ballast was placed in 150-mm lifts and compacted using a vibrating plate tamper with a mass per unit area of 105 kg/m$^2$.

To qualify as railway ballast, aggregate must meet other criteria in addition to proper grading. These include specified limits for the Los Angeles abrasion (LAA), elongation factor,

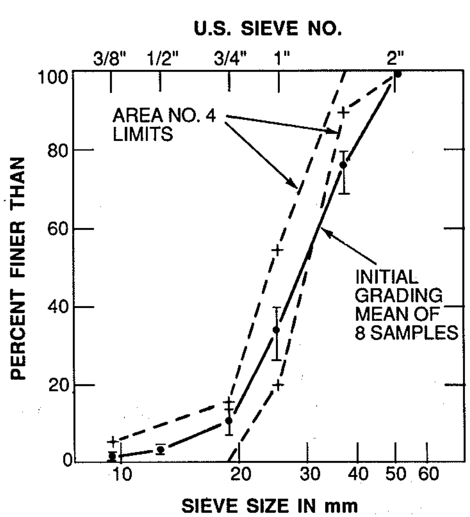


**FIGURE 2 Ballast size distribution.**

and sodium soundness tests. The results of these tests on the Sunbury aggregate showed that this material is acceptable according to AREA specifications. However, in terms of the more detailed ballast quality guidelines recently adopted by Canadian Pacific Railways (CP Rail), the selected ballast was deemed unlikely to be used in main-line track and marginal for use in branch-line track because of its inadequate abrasion resistance (*3*). The senior author (*4,5*) proposed a track class ranking based on a trade-off between the LAA and mill abrasion (MA) values involving limits on both tests and a combination of both tests (i.e., LAA + 5 MA). This quantity was named the aggregate index ($I_a$) in the development of a railroad track degradation model by Bing and Gross (*6*) and abrasion number ($N_a$) by CP Rail in the development of their ballast life model (*3*). Thus

$$I_a = N_a = LAA + 5\ MA \tag{1}$$

The MA test is a nonstandard test that measures the hardness (resistance to abrasion) of a ballast material resulting from the autogeneous grinding of aggregate particles. The results of LAA and MA tests on the Sunbury limestone gave LAA = 27, MA = 8.5, and an abrasion number of 69.5. This abrasion number compares with a maximum value of 65 allowed by CP Rail. Research by CP Rail has related ballast life-cycle times to ballast quality (expressed as the $N_a$ number) and traffic density. According to CP Rail criteria, the ballast used in this investigation would not be recommended for use on main-line track. However, because the same material was used for all tests reported in the current study, the quality of the ballast was not considered a factor that could influence the relative performance of test configurations.

At the time of writing, test sections comprising aggregate with a lower $N_a$-value are planned as part of the long-term goal to equate track performance with ballast quality (subject to obtaining financial support).

### Footing

Footing dimensions were selected to model one-half of the total bearing area of a typical tie (i.e., the bearing area below one rail seat) as outlined in the AREA *Manual for Railway Engineering* (*2*). The footing length (920 mm) using the AREA approach also corresponds to about the tamper influence distance along the tie on either side of each rail. The footing was constructed from a rectangular hollow steel section 3.15 mm thick and closed at the end to prevent aggregate infilling.

### Ballast Support

Test configurations reported in this paper were constructed with artificial subgrades that had three different compressibilities. The purpose of the artificial subgrades was to model ballast support (i.e., subballast-subgrade formation at the subballast-ballast interface) over a range of stiffnesses.

A rigid ballast support condition was simulated by placing ballast directly over the concrete laboratory floor. This condition models a field situation in which track traverses exposed bedrock faces or a chemically stabilized stiff subgrade.

A flexible ballast support condition was modeled using a closed-cell gum rubber mat. A ballast support modulus of 129 $MN/m^3$ was calculated for this material using a 762-mm-diameter plate and a maximum load of 85 kN. A California bearing ratio (CBR) value of 39 was determined for the same material using the test procedure outlined in ASTM D 1883-73. This condition may be considered to simulate ballast support due to a granular subballast over a competent cohesive subgrade.

A very flexible ballast support condition was modeled using a double layer of gum rubber. The ballast support modulus of this configuration was 62 $MN/m^3$ and gave a CBR value of 10. It should be noted that this low value indicates extreme subballast-subgrade formation compressibility, which would generally be avoided in the field although it might be encountered in cases in which poor drainage of subballast-subgrade formation exists. The principal reason for using this weak artificial support was to clearly establish trends in ballast load-deformation behavior and ballast degradation related to ballast support.

### Loading System and Data Acquisition

Footing loads were applied through an MTS closed-loop electrohydraulic actuator controlled by a DEC PDP11/34 computer. A load cell and linear variable displacement transducer (LVDT) located above the actuator base were used to monitor footing load and vertical footing displacements at all test stages. At programmed intervals, the load-deformation response of the footing during a loading cycle was recorded and stored by the computer.

## TEST PROGRAM

Results from 16 tests have been used in the current study to provide data with which to compare the relative performance of tie-ballast-support configurations subject to a range of load levels. A summary of the test program is given in Table 1.

Tests were carried out using the actuator in a load-controlled mode and each footing was subjected to a number of load repetitions equivalent to a typical loading of 12 million and a maximum of about 20 million cumulative axle tonnes in track. European railway experience has shown that, for conventional main-line track, the settlement rate expressed as deformation per log cycle cumulative tonnage is usually constant after about 2 million tones (*7*). In 1980 annual traffic of 10 million to 60 million gross tonnes (MGTs) was recorded for typical heavy branch-line and main-line track sections in Canada.

The applied maximum rail seat loads ranged from 20 to 150 kN. The lower load levels can be considered typical of loadings delivered to the tie rail seat by unloaded trucks or light passenger cars. The 150-kN load may be representative of a small percentage of dynamic impact defects associated with a wheel load delivered by a 100-tonne truck (*8*). Several tests were carried out using an 85-kN load; Figure 3 shows that an 85-kN load (tie bearing pressure = 370 kPa) represents a typical magnitude of dynamic load borne by ballast directly beneath the tie for a track modulus of between 14 and 84 MN/m/m of rail (*9*). Here the dynamic increment is generated by a 51-mm geometrically perfect square wheel flat and is added to the static load caused by two G75 bogies subject to 294-kN axle loads.

The rate of loading varied from 0.5 to 3 Hz depending on the test configuration. The frequency adopted for a particular test was a compromise between a desire to perform the test as quickly as possible and hardware constraints. Nevertheless, it is well documented that the magnitude of permanent deformations generated in track is insensitive to the magnitude of loading frequency when low rates of loading are employed (*10*). A sinusoidal compressive repeated loading waveform was used in the testing program. This waveform is thought to approximate the loading pulse applied to railway ties under actual field conditions (*11*). Finally, it should be noted that

TABLE 1 SUMMARY OF TEST PROGRAM

| Test No. | Load Level (kN) | Subgrade Condition (CBR) |
|---|---|---|
| 1 | 40 | Rigid |
| 2 | 85 | Rigid |
| 3 | 85 (repeat) | Rigid |
| 4 | 85 (flooded) | Rigid |
| 5 | 150 | Rigid |
| 6 | 20 | 39 |
| 7 | 40 | 39 |
| 8 | 60 | 39 |
| 9 | 70 | 39 |
| 10 | 85 | 39 |
| 11 | 20 | 10 |
| 12 | 40 | 10 |
| 13 | 50 | 10 |
| 14 | 60 | 10 |
| 15 | 70 | 10 |
| 16 | 85 | 10 |

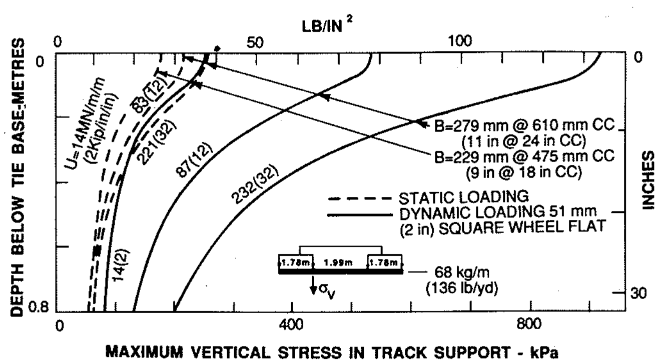


**FIGURE 3 Relationships between maximum vertical track stress and depth below tie base for varying track modulus under static and dynamic loading conditions.**

five initial load repetitions were applied to each test configuration to ensure that the footing was well seated. The seated position then became the reference datum for subsequent footing deformations.

## TEST RESULTS

### Ballast Support Compressibility

Figure 4 shows accumulated permanent deformation recorded for 85-kN tests as a function of equivalent cumulative axle tonnes. Permanent deformations shown in the figure are those measured at the base of the tie. The data illustrate that at a load level considered representative of heavy freight axles in track, the magnitude and rate of permanent deformation accumulation are sensitive to ballast support compressibility. For a rigid ballast support there is an essentially linear relationship between magnitude of permanent deformation and the log number of accumulated tonnage. Similar linear semilogarithmic settlement trends have been observed in full-scale tests in which ballast was placed over a firm subballast-subgrade formation (*12*) and by the European railways who have monitored conventional main-line track constructed over very competent subgrades (*7*). The CBR = 10 test shows that there is a dramatic increase in the semilogarithmic rate of accumulated settlement after about 2 MGT. Qualitatively similar trends have been reported by the European railways for main-line track in need of ballast maintenance (*7*). The 85-kN test with a CBR = 39 support likely represents a transition between a very competent ballast support and a weak subballast-subgrade formation.

### Load Level

Figures 5-7 show permanent deformations recorded from tests with variable peak loads but identical artificial ballast support.

The rigid support tests (Figure 5) show that, over the range of load level and tonnage applied, deformation-log tonnage response is reasonably represented by a straight line the gradient of which increases with load level.

In contrast, the results of compressible ballast support tests (Figures 6 and 7) show that the deformation-log tonnage curves can be classified (as a first simple approximation) into one of two performance categories: Below some critical rail seat load (defined later), the curves are linear on the semilogarithmic plots; above the critical value, the test results show distinct curvatures that indicate progressive deterioration of tie support.

Below the critical load level the rate of settlement-log tonnage on any constant support is approximately proportional to the cycled peak load level. Thus the total settlement recorded after the same tonnage was approximately proportional to the cycled peak load.

Figure 8 is a plot of the equivalent tonnage required to achieve a settlement criterion for all tests with a compressible artificial support. Where necessary, settlements at large

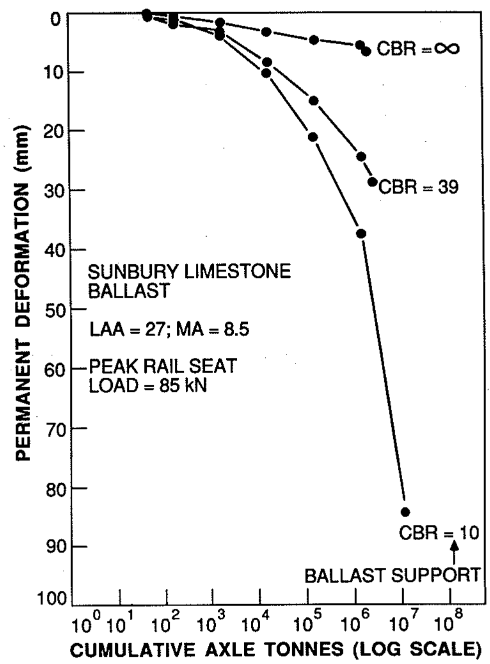


**FIGURE 4 Influence of ballast support compressibility on ballast deformation.**

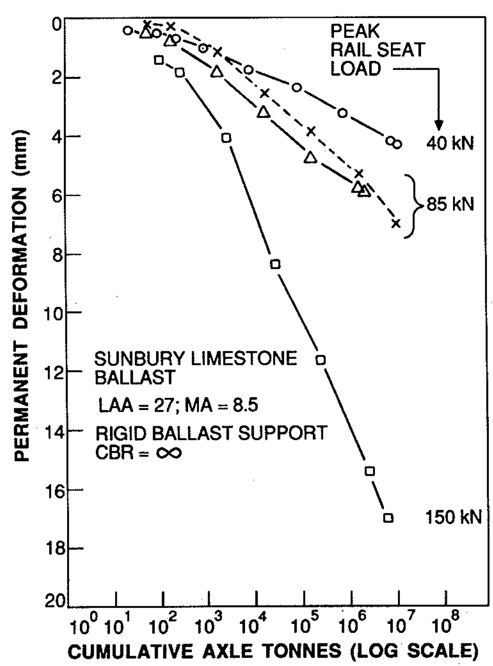


**FIGURE 5 Accumulated permanent deformation (rigid ballast support).**

tonnages have been estimated by linearly extrapolating load-deformation results after 2 MGT. The mean settlement criterion adopted by a given railway may vary, but 40 or 50 mm may be considered a typical upper limit. Clearly, uniform settlement is not detrimental to track performance. However, track quality (expressed as the frequency of cross-level, twist, and alignment defects) will deteriorate in direct proportion to mean settlement recorded at rail seat locations. The figure shows that model tests with compressible artificial support

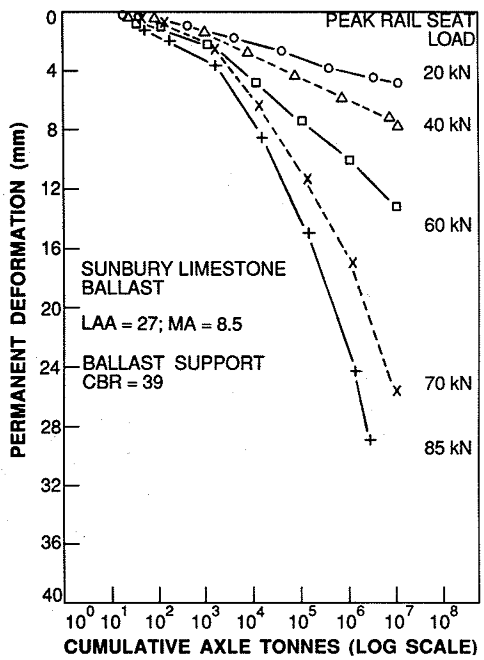


**FIGURE 6 Accumulated permanent deformation (flexible ballast support).**

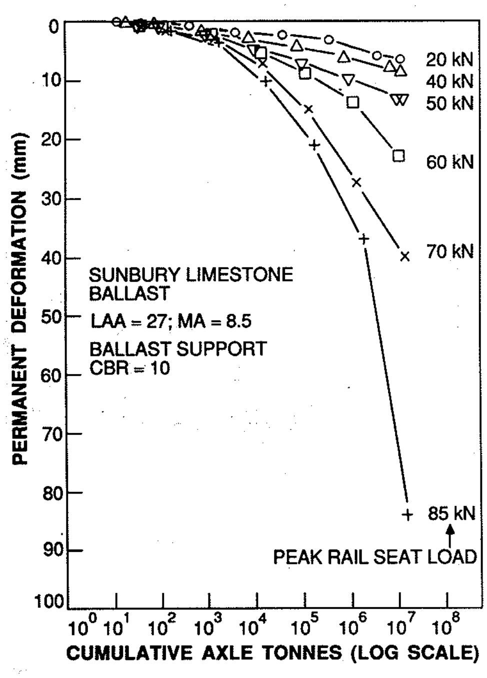


**FIGURE 7 Accumulated permanent deformation (very flexible ballast support).**

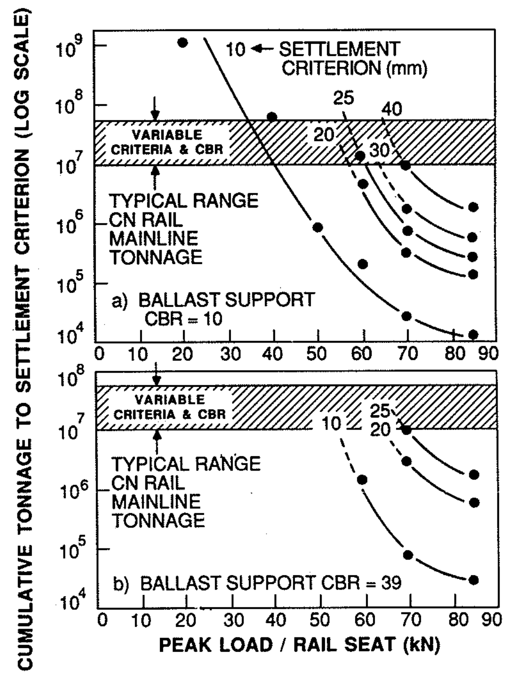


**FIGURE 8 Equivalent tonnage to achieve settlement criterion.**

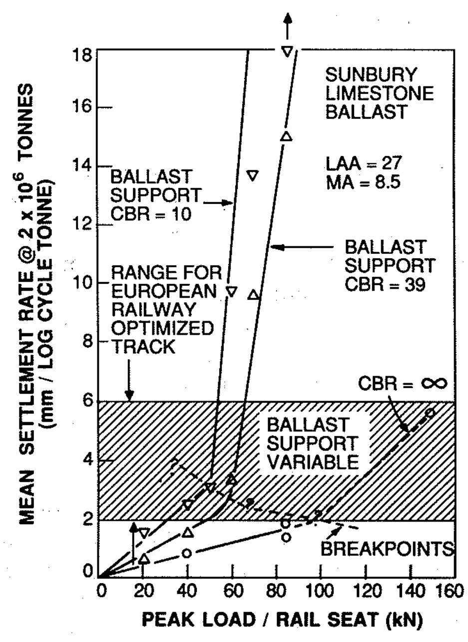


**FIGURE 9 Influence of load level on settlement rates.**

achieved (or would have achieved) an accumulated tonnage that is typical of annual CN Rail heavy branch-line and main-line track (*13*) while recording levels of mean settlement that are probably acceptable in track. Figure 8 also shows that, for the same cumulative tonnage, a heavier wheel load produces greater settlement (and hence more track damage) than the same cumulative tonnage delivered through a lighter axle. This observation is not surprising to many railways that have moved to heavier axle loads in recent years. In many instances they have observed a rapid deterioration of track that had previously performed in a stable manner for many years.

The relative sensitivity of test performance to load level and ballast support stiffness is shown in Figure 9. For each set of tests with a given artificial support compressibility there is a critical peak rail seat load above which there is a rapid increase in the settlement rate per log cycle of cumulative tonnage after 2 MGT. The figure shows that the critical load level is about 50 kN for the CBR = 10 support tests and somewhat higher at about 60 kN for the CBR = 39 support configurations. For the rigid support tests the breakpoint is at a value greater than 85 kN.

It is interesting to note that the breakpoints from the model tests fall within the measured range of settlement rates recorded by the European railways for optimized conventional main-line track (*7*). Above the critical load level the settlement rates increase dramatically and extend to values that would be deemed excessive for both European and North American railways (e.g., greater than 10 mm/log cycle tonne after 2 MGT).

It should be noted that the results of the model tests will be conservative compared with a comparable configuration in the field. In particular, settlement rates are probably higher because the ballast in the single tie-ballast model is less constrained. Nevertheless, the qualitative trends extracted from the model tests are considered by the authors to be valid. An important implication of the current test results is that settlement rates associated with excessive load levels (i.e., greater than critical values) are quite sensitive to the magnitude of wheel loads. For existing track, a modest increase in any wheel loads that are already at about the critical limit will lead to a dramatic increase in settlement rate. For new heavy-haul track, the subballast-subgrade formation should be constructed so that anticipated dynamic wheel loadings are within the critical limits of the ballast support.

### Ballast Degradation

Under repeated loading, ballast aggregate in track can be expected to degrade. To examine this phenomenon, bulk samples of aggregate were taken at the completion of selected tests from ballast located between the tie bottom and the underlying artificial support. Mechanical grain-size analyses were carried out on these samples in an attempt to correlate single tie-ballast model performance with aggregate degradation. Initially, the full grain-size curves corresponding to samples before and after repeated loading were plotted together and compared. The results of this exercise showed a great amount of scatter. For example, full grain-size curves taken from aggregate used in lightly loaded tests often plotted within the scatter band of the initial unloaded samples for particle sizes greater than 10 mm. A more successful approach was adopted wherein only the material passing the No. 4 sieve was examined. In this approach, only fines generated during loading are compared.

It should be noted that a portion of the fines generated in any test must be due to abrasion at the steel footing-ballast interface. A model tie constructed from timber would be expected to generate less fines. Nevertheless, the tie material was kept constant in all tests and hence qualitative comparisons of results are valid.

The particle size distributions for the fines are shown in Figures 10 and 11 for tests having equivalent ballast support. The plotted points represent the average of four samples. A number of observations can be made about these figures: The volume of fines passing the No. 4 sieve is greater for the

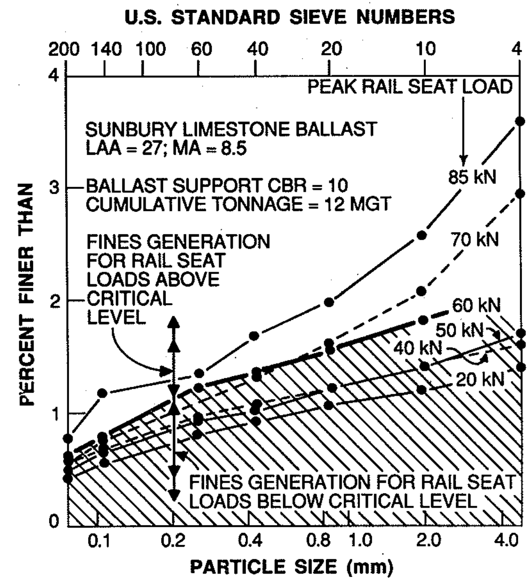


**FIGURE 10 Particle size distribution for fines (ballast support CBR = 10).**

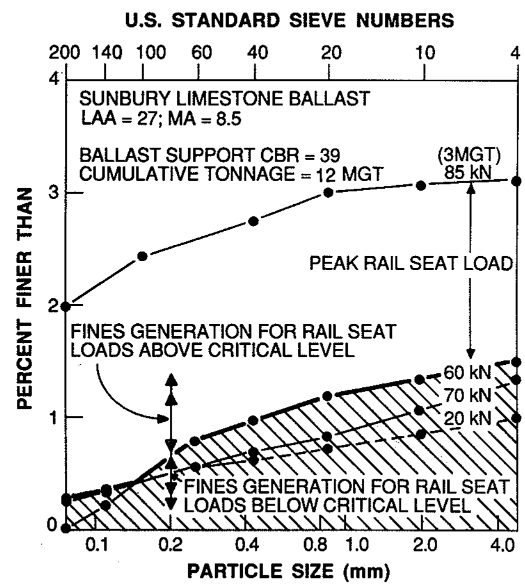


**FIGURE 11 Particle size distribution for fines (ballast support CBR = 39).**

CBR = 10 tests than for the CBR = 39 tests at a given load level and the same cumulative tonnage. This result is expected because the magnitude of repetitive aggregate movements would be expected to increase with decreasing artificial support elastic stiffness. In addition, the amount of fines passing the No. 4 sieve increases markedly after approximately the critical load level identified for each ballast support condition in the previous section. This observation confirms that ballast degradation is a mechanism that is accelerated in overstressed track support. In actual track, the generation of fines contributes to ballast fouling, which in turn inhibits drainage and over time reduces the load-carrying ability of the track support.

**Influence of Flooding on Test Results**

In actual track, precipitation contributes to the deterioration of ballast aggregate. To examine the influence of a wet environment on the performance of single tie-ballast models, a test was carried out in which the ballast was fully saturated to a depth corresponding to the base of the tie for the full duration of loading. The results of this wet test are plotted with those of similar (standard) dry tests in Figure 12. The tests shown were constructed with rigid ballast support. The figure illustrates that, over the range of tonnage applied, both wet and dry tests exhibited a linear semilogarithmic settlement trend but that the rate of settlement for the wet test was almost twice that of the comparable dry tests. The implication of these results for actual track support is that ballast life in the field is reduced when poor drainage exists.

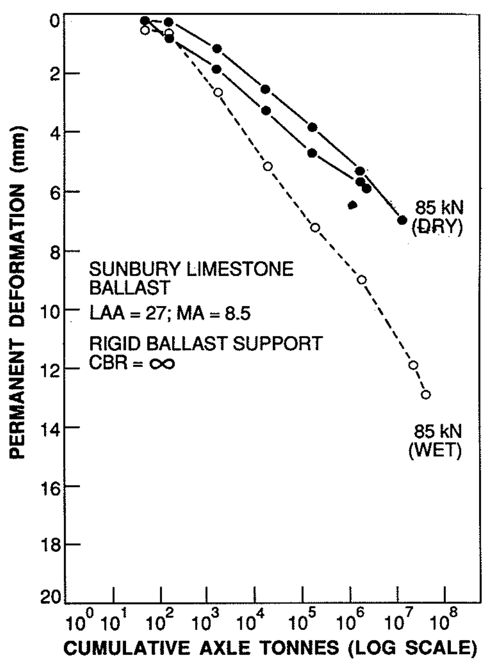


**FIGURE 12 Influence of flooding on settlement rates.**

## SUMMARY OF CONCLUSIONS AND IMPLICATIONS

The major conclusions that can be drawn from the current study and the implications for track can be summarized as follows:

1. The magnitude and rate of permanent deformation accumulation are sensitive to ballast support compressibility. Competent ballast support results in a permanent deformation-log tonnage response that is essentially linear. Progressively weaker ballast supports are characterized by increasing semilogarithmic rates of settlement with tonnage. For any given competent support, the total settlement after a given tonnage was approximately proportional to the cycled peak load.
2. Test results show that, for a given ballast support compressibility and cumulative tonnage, heavier axle loads will do more damage to ballast than lighter axle loads.
3. The current study illustrates that for a given ballast support there is a critical load level that, if exceeded, will cause a dramatic increase in ballast settlement rates after 2 MGT. In contrast, below the critical value, settlement rates are less sensitive to the magnitude of wheel loads.
4. The critical load level for the single tie-ballast models after 2 MGT was observed to increase with increases in ballast support stiffness.
5. The generation of fines in the ballast below the tie after about 10 MGT was observed to increase markedly when the critical wheel load level was exceeded in the model tests.
6. The generation of fines at a given load level and cumulative tonnage was observed to increase with increasing ballast support compressibility.
7. Flooding of the ballast layer in the single tie-ballast model tests increased the rate of settlement to almost twice that recorded for dry configurations.

## ACKNOWLEDGMENTS

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# Application of Petrographic Analysis to Ballast Performance Evaluation

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**In this paper is presented an interpretation of ballast performance test results that uses the information obtained from petrographic analysis of ballast material to help assess the importance of this technique as a means of evaluating ballast. Fourteen ballast materials were used in this study. These were tested in a special ballast box apparatus under differing conditions of equivalent wheel load, material gradation, and number of load cycles. Mill abrasion and ballast cementing tests were also performed on these materials and Los Angeles abrasion values were obtained from the ballast suppliers. Identification of the mineral composition and texture of the ballast material, as well as inherent planes of weakness such as foliation and cleavage, can be accomplished using petrographic analysis. With a few exceptions, petrographic analysis was found to provide a reasonable explanation of the performance of the ballast materials in each of the tests. Performance of ballast in track is a function of field conditions and particle shape and grading as well as of factors identified by petrographic analysis. Thus petrographic analysis is not yet sufficient to predict performance—but neither are commonly used index tests. However, appropriate research involving field observations as well as laboratory testing has the potential to provide a significant improvement in ballast performance prediction using petrographic analysis as an important tool. Such analysis should be performed by a petrographer with experience in railroad applications.**

Performance of railroad ballast as used in this paper is represented by its ability to hold track geometry under repeated wheel loading and existing environmental conditions. Important mechanical properties include (a) strength and resiliency of the assembly of particles and (b) resistance to breakage and abrasion of the individual particles.

Thus, in an evaluation of a material's performance as a ballast, gradation, particle shape, and rock type, including texture and composition, must be considered. However, if all factors but rock type were held constant, there would still be variability of performance because different rock types have different physical properties. A ballast box test was developed at the University of Massachusetts at Amherst (UMass) to investigate the effect of all of these factors. In addition, several index tests were used to measure the effect of rock type. These were mill abrasion tests and ballast cementing tests, which were performed at the university, and Los Angeles abrasion tests conducted by the suppliers of the ballast materials.

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Previous experience with petrographic analysis for aggregate evaluation has suggested that knowledge of the mineral composition and texture of the rock, as well as physical properties such as bedding planes, *foliation* (terms in italics are defined in the Glossary), and porosity, would be useful in understanding the behavior of ballast. Thus identification of rock type and characteristics on the basis of petrographic analyses, including hand-sample and thin-section techniques, has been incorporated into the UMass ballast testing program. The purpose is to assist in the evaluation of results obtained from these ballast performance tests and to help establish the value of using hand-sample and petrographic analysis in predicting ballast performance. The results of this recent research are summarized in this paper.

## CURRENT APPLICATIONS OF PETROGRAPHIC ANALYSIS

The value of petrographic evaluation as a means of assessing or predicting, or both, behavior of an aggregate has been long recognized by the concrete industry. Techniques for evaluating aggregate for use in concrete and for examining hardened concrete have been established by ASTM in standards C 295 (for aggregate) and C 856 (for hardened concrete). The purposes of this petrographic examination are (a) to determine the physical and chemical properties of the material that will have a bearing on the quality of the material for the intended purpose; (b) to describe and classify the constituents of the sample; and (c) to determine the relative amounts of the constituents of the sample, which is essential for the proper evaluation of the sample, especially when the properties of the constituents vary significantly. The value of the petrographic evaluation is said to depend to a large extent on the ability of the petrographer to correlate data provided on the source and proposed use of the material with the findings of the petrographic examination.

A method of numerical evaluation of the quality of aggregates used for concrete on the basis of petrographic analysis was developed by the Materials and Research Section of the Department of Highways (DOH) of Ontario (*1*). This method was based on extensive studies of field performance of aggregate used in concrete and bituminous pavement. With this method, a sample of aggregate is first examined for composition (i.e., rock type or types) and then assigned a number (factor) based on experience with that or similar rock types and their observed field performance. Rock types considered to be excellent are given values close to 1 and those considered to be poor approximately 6. This number is then multiplied by the percentage of each rock type in the aggregate sample, which gives a weighted value. The Petrographic Number (PN) for an aggregate sample is the sum of the weighted values of all components. This number is used as a quality rating for that sample. Limits have been set for acceptable PNs for various uses of aggregate.

After several years of applying this method in the prediction and evaluation of aggregate performance, the DOH of Ontario drew several conclusions (*1*):

1. Petrographic analysis is the most significant of all quality tests. The evaluation is based on actual performance and considers both the physical and chemical properties of the aggregate.
2. Petrographic analysis is the shortest test when time is an important consideration.
3. The results were found to be fairly reliable and to provide a better appraisal of aggregate performance than either abrasion or absorption alone.
4. The technique is adaptable to both the field and the laboratory.
5. The method can be used reliably to maintain the quality of an aggregate from a previously tested and known source.

Raymond (*2*) suggested that petrographic analysis is of major value in the selection of a suitable quarry for ballast and that it can also be useful for prediction of the shape and permeability of the components of future ballast breakdown (i.e., the fines generated by breakage and abrasion of the ballast).

Canadian Pacific Rail (CPR) has included petrographic analysis as a part of their standard ballast specification (*3*). The CPR requirements for the petrographic analysis of potential ballast include

- Description of rock types, including mineralogy, texture, and structure;
- Description of mechanical properties, including *hardness,* shape, and type of fracture;
- Description of chemical properties, including existing and potential chemical weathering;
- Properties of the fine material, including shape and potential for weathering;
- Estimation of index test results; and
- Recommendation for special testing, if necessary.

Acceptance of petrographic analysis for engineering use as indicated in these examples was part of the incentive for the use of the technique as part of the ballast evaluation program at UMass.

## BALLAST IDENTIFICATION

Rocks may consist of only one mineral, but they generally consist of an aggregate of mineral types. A mineral is a "naturally occurring, homogeneous solid with a definite, but generally not fixed, chemical composition and an ordered atomic arrangement" (*4*). For example, the minerals calcite and aragonite are both composed of calcium carbonate, but they have different atomic arrangements, which results in the two minerals having different physical properties of *cleavage,* hardness, and specific gravity.

Rocks are generally classified by texture and mineral composition. The texture of a rock is the relationship between the crystals of minerals that form the rock. A description of a rock's texture generally includes grain (crystal) size, grain shape, degree of crystallinity (ratio of glass to crystals) and the contact relationships of the grains. The presence of voids or fractures is included in the description of the structure of the rock but is generally not specifically considered when naming or classifying a rock. Two rocks with similar mineral compositions but different textures would have different names. Initially, rocks can be grouped into three main types—igneous, sedimentary, and metamorphic—on the basis of the mode of formation. Each category is further divided into classifications of rocks according to their texture and composition.

Petrographic analysis involves the study of a rock in hand specimen as well as in thin section. Analysis of a material in hand specimens gives an indication of the approximate composition and texture of the rock. Thin-section analysis allows a more precise assessment of the rock's constituents and texture.

A thin section is a representative section of a rock that has been cut, mounted on a glass slide, and ground to the standard thickness of 0.03 mm (0.0012 in.). At this thickness most minerals will transmit light. The thin section can then be examined using a transmitting light, polarizing microscope. The rock is identified in part by its mineral composition, which can be determined from a thin section by crystal form or grain shape, grain size, cleavage, color in plain light, *pleochroism, birefringence,* extinction angle, *refractive indices,* and twinning. Also important in the identification of a rock are bedding, banding, foliation, texture, porosity, and the presence of fossils or fossil fragments and *peloids.* A thin section can also be stained to distinguish the members of the carbonate and feldspar mineral groups.

Fourteen ballast materials were identified through hand-specimen analysis and 12 of these were also analyzed as thin sections. The materials were classified using several classification systems (*5-9*). A summary of the results of the analyses is given in Table 1. The materials examined represent a wide range of rock types, including both intrusive and extrusive igneous rocks, chemical sedimentary rocks, and medium- and high-grade metamorphic rocks. This range provides a good foundation for the evaluation and comparison of performance based on material type.

## EVALUATION OF BOX TESTS

### Description of Test

The ballast box test was first developed in 1981 at UMass to study the behavior of ballast under simulated field loading conditions (*10*). The ballast box has an advantage over current standard index tests in that it can simultaneously

TABLE 1 SUMMARY OF HAND SAMPLE AND PETROGRAPHIC ANALYSES

| Material | Rock Type | Rock Name | Description | Predominant Minerals | Approximate Percentage |
|---|---|---|---|---|---|
| Gneiss 3 | Metamorphic | Micaceous quartzo-feldspathic gneiss | A fine-grained, *inequigranular* gneiss with *crystalloblastic* and *lepidoblastic* textures and fair to good foliation | Quartz<br>Feldspar<br>Mica | 40<br>45<br>10 |
| Granite | Intrusive igneous | Granite | A medium-grained, *phaneritic, holocrystalline, inequigranular* granite; the sample was extensively weathered, with sericite occurring as an alteration product of feldspar and no original *mafic* minerals observed | Quartz<br>Sericite<br>Chlorite<br>Feldspar | 35<br>30<br>25<br>10 |
| | | Quartz monzonite | A medium-grained, holocrystalline, inequigranular rock with *subhedral* grains; some plagioclase feldspar grains with euhedral inclusions of quartz or feldspar | Feldspar<br>Biotite<br>Amphibole<br>Quartz | 45<br>25<br>15<br>15 |
| | | Granite | A medium-grained, *equigranular,* holocrystalline rock with subhedral grains; the feldspar frequently had euhedral inclusions and sericite occurred as an alteration product of feldspar | Quartz<br>Feldspar<br>Sericite | 45<br>40<br>12 |
| Gneiss 2 | Metamorphic | Quartzo-feldspathic gneiss | A medium-grained, inequigranular gneiss with subhedral grains, crystalloblastic texture and fair foliation | Quartz<br>Feldspar<br>Biotite<br>Pyroxene | 40<br>30<br>15<br>10 |
| Monzonite | Intrusive igneous | Monzonite | A holocrystalline, medium- to coarse-grained rock with subhedral grains and some alteration of feldspar to sericite and clay | Feldspar<br>Biotite<br>Magnetite | 95<br>3<br>2 |
| Quartzite 2 | Metamorphic | Quartzite | A fine- to medium-grained rock composed of sutured quartz grains with rims of hematite and interstitial muscovite | Quartz<br>Muscovite<br>Hematite | 90<br>8<br>2 |
| Dolomite | Chemical sedimentary | Crystalline dolomite | A compact, fine- to medium-grained rock composed of interlocking crystals with low porosity | Dolomite<br>Calcite | 95<br>5 |
| Micrite | Chemical sedimentary | Detrital micrite | Very fine-grained quartz in a *matrix* of ferroan *micrite* | Calcite<br>Quartz | 80<br>20 |
| Carbonate | Chemical sedimentary | *Sparite* | A medium- to coarse-grained rock composed of interlocking sparry calcite crystals | Calcite | 100 |
| | | Dolomitic micrite | A very fine-grained rock composed of *cryptocrystalline* dolomite with a vein of intrusive dolomite and calcite | Dolomite<br>Calcite | 85<br>15 |
| | | Crystalline dolomite | A medium-grained rock composed of interlocking crystals of dolomite | Dolomite<br>Calcite | 95<br>5 |
| Gneiss 1 | Metamorphic | Quartzo-feldspathic biotite gneiss | A medium- to coarse-grained inequigranular gneiss with crystalloblastic and lepidoblastic texture and fair to strong foliation | Biotite<br>Feldspar<br>Quartz<br>Hornblende | 35<br>25<br>25<br>15 |
| Quartzite 1 | Metamorphic | Quartzite or quartz gneiss | A fine-grained rock with a *groundmass* of sutured and interlocking quartz, with larger crystals of quartz and interstitial muscovite | Quartz<br>Muscovite | 85<br>15 |
| Slag | Artificially produced | Slag | Particle variation from solid and glassy to very vesicular | | |

TABLE 1 *continued*

| Material | Rock Type | Rock Name | Description | Predominant Minerals | Approximate Percentage |
|---|---|---|---|---|---|
| Limestone 2 | Chemical sedimentary | Dolomitic fossiliferous microsparudite | A fine-grained matrix of microsparry calcite and dolomite; fossiliferous, with original material and large molds filled with sparite | Calcite<br>Dolomite | 90<br>10 |
| Limestone 1 | Chemical sedimentary | Bio micrudite | A very fine-grained rock with a matrix of micrite and clay-sized material (unidentified); the rock is fossiliferous, with original material and large molds filled with sparite | Calcite | 80 |
| Basalt | Extrusive igneous | Basalt | A very fine-grained (*aphanitic-phaneritic*) rock composed of phenocrysts of feldspar in a groundmass of mafic minerals; individual particles range from highly to slightly vesicular, with a small (relative) amount of nonvesicular (solid) particles; many of the particles are slightly weathered and coated with clay | Feldspar (phenocrysts)<br>Mafic minerals in groundmass | |

consider the effect of such variables as material size and gradation, material type, wheel load magnitude, number of cycles of load (i.e., traffic), application of maintenance cycles, and type of tie on the settlement and amount of degradation of a ballast, thereby better simulating field conditions. The ballast index tests can only consider the effects of rock type under one set of test conditions.

Briefly, the box represents a section of the track structure that includes a section of a tie directly beneath a rail seat and a portion of the crib on either side of the tie with a ballast depth of 12 in. or 305 mm (Figure 1). To simulate train traffic, a servohydraulic testing machine is used to repeatedly apply vertical load to the tie segment, cycling at 5 Hz between a minimum load of 100 lb (445 N) and the maximum load, which varies depending on the axle load and track conditions. The computer program GEOTRACK (*11*) is used to determine an equivalent wheel load (EWL) to be applied to the test tie segment so that the ballast beneath the segment experiences the same amount of stress as does the track being represented; that is, to simulate a 50-kip (223-kN) wheel load, an equivalent maximum load of 6,000 lb (27 kN) is applied to the tie segment.

To prepare a box test, a ballast specimen of the desired grading is placed in the box in layers, each lightly compacted by rodding. The material in the zone directly beneath the tie segment is dyed so that any breakage and abrasion that occur during the test will be apparent.

For a standard test, settlement readings are taken periodically until completion of the test. At the end of the test the material is carefully removed and inspected to determine ballast degradation. Coarse breakage is defined as a fragment larger than 3/8 in. (9.5 mm) or a particle from which a piece larger than 3/8 in. has been broken. Fine breakage is any material smaller than 3/8 in. that has been generated during the test through breakage or abrasion, or both. Breakage in each of these categories is given as a percentage of the original sample weight.

## 36-Kip EWL Tests

Table 2 gives the relative ranking of five ballast materials according to the amount of breakage (*12*) for a 36-kip (160-kN) EWL. In each case, 500,000 cycles of load were applied in a standard test and the gradation used was American Railway Engineering Association (AREA) 4.

### *Coarse Breakage*

In general, in the category of coarse breakage, the relative behavior of these five ballast materials in the ballast box tests is what could be expected on the basis of the data obtained from the petrographic and hand-sample examinations. The metamorphic Quartzite 1, composed essentially of quartz and without inherent planes of weakness, proved to be the most resistant to coarse breakage (Table 2). The medium-grained, foliated Gneiss 1 had a moderately low amount of breakage as did the medium-grained, softer dolomite. This metamorphic gneiss had an amount of breakage similar to that of the dolomite because of inherent planes of weakness in the gneiss due to the grain size, mineral composition, and foliation (i.e., the presence of minerals such as mica and feldspar that have one or more planes of cleavage). The glassy slag had a moderate amount of coarse breakage; breakage was a function of the vesicularity of the particles comprising

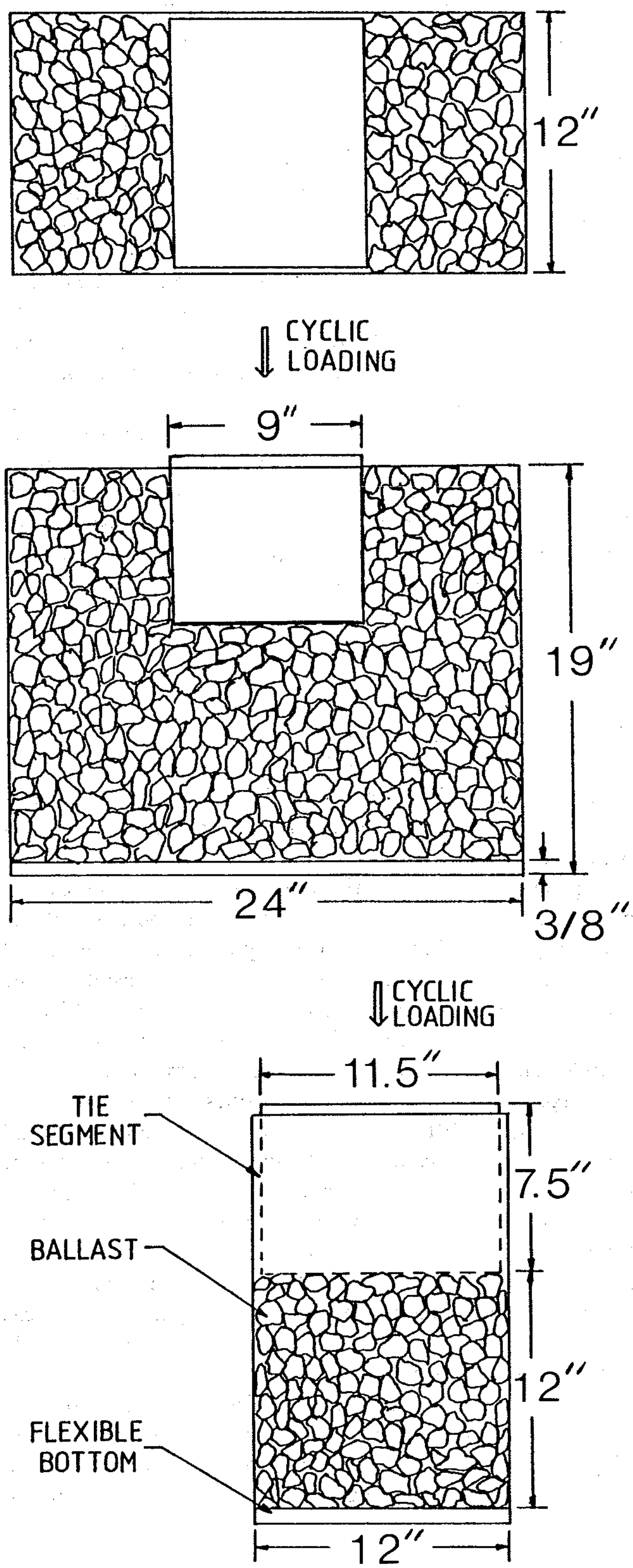


**FIGURE 1 Ballast performance simulator.**

the test sample. The carbonate material had the greatest amount of breakage. Three rock types, all carbonates, that are soft rocks expected to break easily, were identified in this sample.

TABLE 2 RANKING OF THE BALLAST BOX TEST BREAKAGE FOR 36-KIP EWL

| Rank[a] | Breakage >3/8 in. (%) | | Breakage <3/8 in. (%) | |
|---|---|---|---|---|
| | Material | Value | Material | Value |
| 1 | Quartzite 1 | 0.3 | Quartzite 1 | 0.2 |
| 2 | Gneiss 1 | 1.3 | Gneiss 1 | 0.2 |
| 3 | Dolomite 1 | 1.6 | Slag | 0.3 |
| 4 | Slag | 2.3 | Dolomite 1 | 0.4 |
| 5 | Carbonate | 4.4 | Carbonate | 0.7 |

Note: 36 kips = 160 kN; 3/8 in. = 9.5 mm.
[a] 1 = lowest breakage; 5 = highest breakage.

### *Fine Breakage*

When a sample is prepared for the ballast box test, all material passing the 3/8-in. (9.5-mm) sieve is removed. Therefore any material finer than 3/8 in. present in the sample after testing has been generated through breakage and abrasion of the aggregate particles. The amount of breakage and abrasion that will result from a test is a function of particle shape, surface roughness, and size and gradation, as well as material type.

In general, the amount of finer material generated was as would be expected given the properties of the materials. The trend is similar to that of coarse breakage. The two metamorphic materials, the Gneiss 1 and the Quartzite 1, were the most resistant to abrasion and produced the least amount of finer material, a function of the composition and texture of the rocks. The dolomite had a moderate amount of fine breakage because the rock is basically medium grained and composed of a soft mineral, dolomite, that causes the rock to be more easily abraded at particle contacts, which are areas of stress concentration during loading. The slag also had a moderate amount of fine breakage but less than the dolomite because the slag is a harder material. The amount of finer slag material generated by a given test depended on the ratio of vesicular to glassy particles, the sizes of these particles, and their distribution within the sample. The carbonate material, which was the softest material, produced the greatest amount of finer material; the behavior of the material, and the amount of finer material generated, depended on the rock type or types present in the sample.

## 50-Kip EWL Tests

The box tests run using a 50-kip (223-kN) EWL are not easily grouped because a consistent gradation was not used in this series of tests. The variation in gradations complicates evaluation of degradation results based on material type. However, two groups with common conditions are (a) Group A with 500,000 test cycles but gradations ranging from AREA 4 to 24 (*13, 14*) and (b) Group B with an AREA 3 modified gradation for either 1 or 2 million cycles (*14, 15*). The results of the 1 and 2 million cycle tests will be considered together on the assumption that most breakage occurs before

500,000 cycles. The results and ranking of these tests are given in Table 3.

TABLE 3 RANKING OF THE BALLAST BOX TEST BREAKAGE FOR 50-KIP EWL

| Rank[a] | Breakage >3/8 in. (%) | | Breakage <3/8 in. (%) | |
|---|---|---|---|---|
| | Material | Value | Material | Value |
| N = 500,000 Cycles | | | | |
| 1 | Quartzite 2 | 0.4 | Gneiss 2 | 0.08 |
| 2 | Granite | 2.2 | Quartzite 2 | 0.09 |
| 3 | Gneiss 2 | 2.9 | Granite | 0.09 |
| 4 | Monzonite | 3.0 | Monzonite | 0.2 |
| 5 | Gneiss 3 | 4.0 | Gneiss 3 | 0.4 |
| 6 | Basalt | 16.1 | Basalt | 0.8 |
| N = 1 and 2 Million Cycles | | | | |
| 1 | Limestone 1 | 15.0[b] | Limestone 1 | 0.6[b] |
| 2 | Basalt | 16.5 | Limestone 2 | 0.6[b] |
| 3 | Limestone 2 | 19.3[b] | Basalt | 0.9 |

Note: 50 kip = 223 kN; 3/8 in. = 9.5 mm.
[a] 1 = lowest breakage, 5 = highest breakage.
[b] Average value of 1 and 2 million cycle tests.

*Group A*

**Coarse Breakage** With the exception of Quartzite 2 and the basalt, the coarse breakage trends of the ballasts in this group generally were not as expected on the basis of the petrographic analyses (Table 1) and the gradations of the materials. The Quartzite 2, similar to the Quartzite 1 material, had the least amount of breakage, and the vesicular basalt had the greatest, as expected. The basalt had a high breakage value because of the extreme vesicularity of many of the particles, which results in a rock that is easily broken. However, the material had a smaller amount of coarse breakage than was originally expected. This may be because at the particle contacts a very vesicular rock easily powders and fractures, which would reduce the amount of coarse breakage.

A fair amount of breakage was expected for the Gneiss 2 and Gneiss 3, which are fine-grained, weakly foliated metamorphic gneisses, and a greater amount of breakage was expected from the coarser-grained (intrusive) igneous rocks (i.e., the granite and monzonite). The Gneiss 3 was also tested at the finest grading, AREA 4, again suggesting low values of breakage.

The differences between predicted and actual performance of this group of materials may be due to a combination of differences in gradation and shape. Gneisses 2 and 3, which are similar rocks, showed the most deviation from expected performance. Gneiss 3, which had the greatest breakage of the group (4.0 percent), with the exception of the basalt (16.1 percent), was reported to be the flakiest (*13*). The Gneiss 2 had the coarsest gradation (AREA 24), was the most equidimensional sample, and had a moderate amount of breakage. The remaining materials, granite and monzonite, had low to moderate values. Each was tested at gradations that fell between the specified limits of AREA 24 and 4; no information on shape was available for either.

**Fine Breakage** With the exception of the Gneiss 3, the performance of the materials in this category was generally as expected on the basis of the determination of rock type. The finer-grained metamorphic rocks (Gneiss 2 and Quartzite 2) showed less fine breakage than the coarser-grained (intrusive) igneous rocks (granite and monzonite) (Table 3). The Gneiss 3 is ranked fifth with 0.4 percent, twice the amount of fine material generated by the fourth-ranked material. The vesicular basalt had the greatest fine breakage with 0.8 percent for the reasons discussed in the previous section. This trend is consistent with the results for coarse breakage, where the basalt showed significantly more degradation than the other materials and the Gneiss 3 had more breakage than expected. The reason for this trend in behavior of the Gneiss 3 is not clear; particle shape and gradation variations complicate the analysis of performance with respect to material type.

*Group B*

The basalt, Limestone 1, and Limestone 2 were tested in a standard box test at an AREA 3 modified gradation with a 50-kip (223-kN) EWL for 1 or 2 million cycles per test. Table 3 includes a ranking of these materials for each category of breakage.

In general, the comparative breakage behavior of the three ballasts in this group was as expected on the basis of the petrographic and hand-sample analyses of the materials: all three materials had high breakage values. The basalt had the greatest amount of fine breakage, which is explained by the highly vesicular nature of the particles comprising the test sample. The limestone materials had the same values for fine breakage. They are both soft limestones, although they are texturally quite different rocks. The Limestone 1 ballast had the least coarse breakage of the group because of the very fine-grained matrix of micrite and "mud." The coarser-grained Limestone 2 had the most coarse breakage, which is a function of its texture and composition. The Limestone 1 material had less coarse breakage than the Limestone 2 material because it has a fine-grained, muddy matrix, whereas the matrix of Limestone 2 is coarser, composed of slightly altered calcite and dolomite, with many partly filled voids and fractures. The larger crystal size of the Limestone 2 matrix suggests that the cleavage planes of the crystals will play a greater role in fracture initiation and propagation, and the open voids will be regions of stress concentration and fracturing, resulting in a rock that fractures more readily when used as a ballast.

## EVALUATION OF ABRASION TESTS

### Description of Test

Because a ballast material will degrade through both breakage and wear, it is desirable to be able to characterize a material's resistance to these modes of degradation. Currently, three tests are used by various railroads for this purpose. These are the Deval, or British Standard wet attrition, test; the Los Angeles abrasion (LAA) test; and the mill abrasion (MA) test. The latter two tests have been incorporated into the UMass ballast program.

In brief, the LAA test consists of tumbling 10 kg (22 lb) of material using one of three gradings (ASTM C535) with 10 to 12 steel balls weighing a total of 5 kg (11 lb) in a drum 28 in. (711 mm) in diameter for 1,000 revolutions at 33 rpm. The material is removed and the sample is washed on a No. 12 sieve. The LAA-value is the amount of material passing the No. 12 sieve generated during the test, called the loss ratio, as a percentage of the original sample weight.

The MA test involves revolving 3 kg (6.6 lb) of a specified gradation of material about the longitudinal axis of a ceramic jar 9 in. (229 mm) in diameter containing water for 10,000 revolutions at 34 rpm. The mill abrasion value is the amount of material finer than the No. 200 sieve generated during the test, given as a percentage of the original sample weight.

The Deval test (*16*) had originally been used in England for evaluating highway materials. The test was standardized in 1951 and adopted for use by the railroad industry. The Deval wet attrition test is similar to the MA test. It involves revolving 5 kg (11 lb) of a specified gradation of material with 5 kg of water in a cylinder 7 7/8 in. (200 mm) in diameter inclined at 30 degrees to the axis of rotation for 10,000 revolutions at 30 to 33 rpm. The wet attrition value is reported as the amount of material less than the British Standard No. 7 sieve generated, as a percentage of the original sample weight.

Since its adoption as a standard test, British Railway has conducted an investigation of the effect of wet versus dry conditions, particle gradation, particle condition, and presence of slurry on the results of the Deval test. The results show that

1. The coarser the grading, the greater the amount of abrasion.
2. Abrasion without water is less than with water.
3. Previously worn particles had much less attrition than freshly crushed particles.
4. The use of a slurry of fines in water increased the amount of abrasion by only a small amount. This is in contrast with field experience in which significant slurry abrasion has been observed, possibly because of the high velocity of liquid movement in the field, which is not present in the Deval test.

Similar results have been found at UMass using the MA apparatus (*14*).

From studies such as these it is clear that the test conditions must be standardized for comparisons of the effects of rock composition on abrasion resistance. However, correlation with field performance is more complicated because such factors as particle gradation, presence of water, and initial condition of particles are not constant in the field.

## Los Angeles Abrasion Tests

The results of the LAA tests for every material are given in Table 4, ranked from 1 to 14 (1 = lowest loss); these data are also shown graphically in Figure 2. The LAA-values were provided by the suppliers of the ballast materials. It is not known whether the values are based on the same test procedures, so some inconsistency that will complicate the comparisons may exist among materials.

A material's tendency to fracture or abrade during an abrasion test is a function of material type, particle shape, and gradation. In the LAA test, gradation is fixed; therefore the results will reflect material type and particle shape. No information is available on the shapes or the representativeness of the specimens tested because the LAA-values were provided by the suppliers. Thus the LAA results will be discussed primarily with respect to material type as determined from petrographic and hand-sample analyses, assuming that they are representative of the tested materials.

TABLE 4 RANKING OF ABRASION TEST RESULTS

| Material | MA | | LAA | |
|---|---|---|---|---|
| | Rank[a] | Value (%) | Rank[a] | Value (%) |
| Gneiss 3 | 2 | 2.0 | 4 | 23 |
| Granite | 6 | 5.6 | 1 | 17 |
| Gneiss 2 | 5 | 5.4 | 6 | 25 |
| Monzonite | 3 | 2.3 | 3 | 21 |
| Quartzite 2 | 1 | 1.9 | 2 | 20 |
| Limestone 2 | 11 | 11.4 | 8 | 33 |
| Limestone 1 | 9 | 7.0 | 7 | 27 |
| Basalt | 4 | 4.0 | 11 | 48 |
| Gneiss 1 | - | - | 4 | 23 |
| Dolomite | 8 | 6.8 | 5 | 24 |
| Slag | 7 | 6.5 | 9 | 35 |
| Micrite | - | - | 6 | 25 |
| Quartzite | - | - | 1 | 17 |
| Carbonate | 10 | 10.1 | 10 | 36 |

[a] 1 = lowest loss (%).

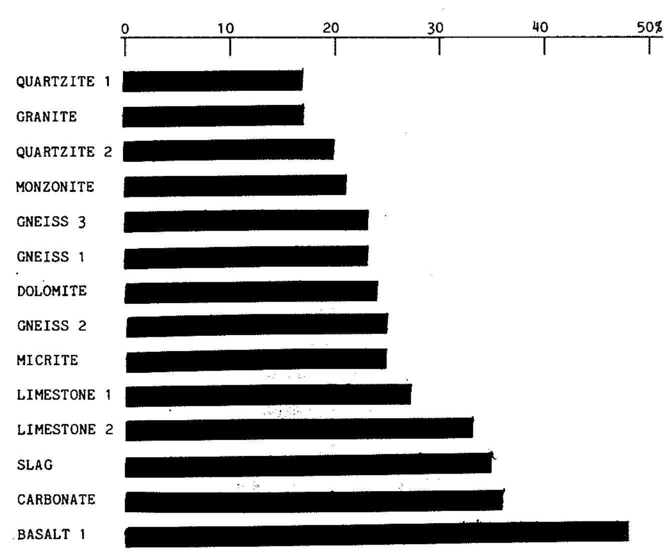


**FIGURE 2 Los Angeles abrasion results for various ballast materials.**

With some exceptions, the LAA results presented in Table 4 are generally what would be expected. The exceptions are the granite, the monzonite, and the dolomite material.

The granite and monzonite were found to be medium- to coarse-grained ((intrusive) igneous rocks composed predominantly of minerals that have at least one plane of cleavage (Table 1). The granite was found to have at least two rock types (both igneous); a complete identification of the rock types and their abundance in the sample has not been achieved. However, a higher loss was expected. The monzonite was a homogeneous sample of a medium- to coarse-grained monzonite. Because of the texture and composition of the rock (large grains of minerals with two cleavage planes), a higher loss was expected.

The dolomite had a lower loss than would be expected for a soft carbonate rock. This material, however, was found to be a compact rock composed of interlocking fine- to medium-sized grains of dolomite, which is a moderately hard mineral. Therefore the reported value may not be misleading.

The remaining materials have reasonable loss values and follow an expected trend. The two quartzites have low values, the three metamorphic gneisses have moderate losses, the four relatively soft carbonates (micrite, carbonate, Limestone 1 and Limestone 2) have moderate to fairly high losses, and the vesicular materials (slag and basalt) have fairly high to very high loss values (Table 4).

### Mill Abrasion Tests

Available standard MA test results are also given in Table 4 and shown graphically in Figure 3 (*12-15*). Because of material limitations, tests could not be conducted using the Gneiss 1, micrite, and Quartzite 1 materials. As with the LAA tests, the results will reflect the rock type and particle shape of the test samples. With the exception of the Gneiss 2, the MA-values are what would be expected on the sole basis of material type.

The metamorphic Quartzite 2 has the lowest value because it is predominantly composed of the hard mineral, quartz. A close second was the Gneiss 3, which contained a large amount of quartz, but also about the same amount of a slightly less hard mineral, feldspar. The monzonite, which was primarily feldspar, was third. The basalt, although it contained feldspar along with other minerals, had more abrasion than the monzonite, probably because of the vesicularity of the particles. Next is the granite with considerable quartz and feldspar. It had a moderate loss but is difficult to evaluate because of the variety of rock types present. The slag had a moderately high loss because of the vesicularity of many of the particles, which created a rougher surface than the glossy particles and hence abraded more readily.

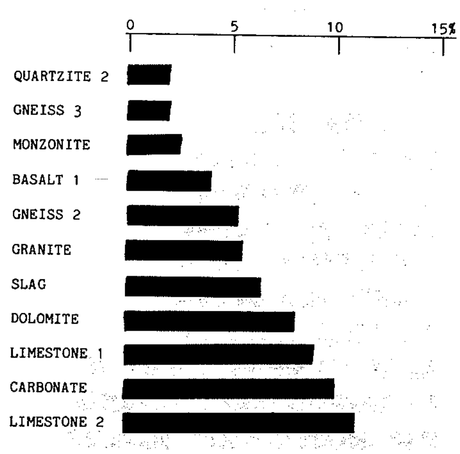


**FIGURE 3 Mill abrasion results for various ballast materials.**

The four carbonate materials had high losses because of the softness of the predominant minerals, calcite and dolomite. The dolomite rock had the lowest loss of the group. The carbonate is a mixture of a variety of rock types and so is hard to evaluate. The two limestones had greater losses than the dolomite. The differences in the two limestones may be a result of their grain fineness.

## EVALUATION OF CEMENTING TESTS

### Description of Test

Ballast cementing tests (BCT) have been incorporated in the UMass ballast testing program to examine the potential for, and mechanisms of, cementing in ballast fine material, as well as to study the relevance of the test to field performance. An initial study was conducted to determine the effects of grain size, compression state, drying sequence, and curing method on the unconfined compressive strength of a cylindrical briquette 1 in. (25 mm) in diameter by 1 in. high (*12*). The test procedure was standardized on the basis of this study. Results with this test were evaluated according to particle characteristics, including shape and composition. These characteristics were determined from microscopic examination of the fine material as well as hand-sample and petrographic analyses of the source (parent) material.

The first step in the BCT is to crush rock particles and sieve the fines to obtain a sufficient quantity of material of the desired particle size. Then a dough is formed using distilled water, with the water content just above the liquid limit, and left to cure overnight. In the standardized test, the briquette is formed by compressing the dough into a cylindrical steel mold 1 in. (25 mm) in diameter by 1.25 in. (32 mm) long using a pressure of 1,875 psi (12.9 MPa) and a *consolidation* time of 10 min. The briquette is then extruded from the mold, trimmed to a length of 1 in., air dried for 20 hr, oven dried for 4 hr, and then placed in a dessicator until it is to be tested. The test is an unconfined compressive test with a constant strain rate to failure of the briquette. The cementing value, or strength, of the material is the average maximum failure stress for five briquettes.

Fourteen materials were tested with all particles less than the No. 200 (0.075-mm) sieve. The strengths for each are given in Table 5 (*12, 14, 15*). Of this group, the carbonate materials generally had the highest strengths, for the reasons of particle shape and composition discussed previously. The exceptions to this are the Quartzite 1, which had a relatively high strength, and the carbonate material, which had a relatively low strength (190 psi or 1300 kPa). The higher strength of the Quartzite 1 may result from the properties and shapes of the quartz grains. This is in contrast with the results for the Quartzite 2, which had a very low strength, as expected. The reason for the relatively low strength of the carbonate material is unknown.

Within the high-strength carbonates, the micrite material had the greatest strength, which could be attributed to the presence of detrital quartz grains and the fine-grained nature of the matrix. The dolomite has a high strength due to particle shape (planar surfaces) and, to a lesser extent, composition (i.e., solubility). The strength of the Limestone 1 material could be due to the finer micritic matrix and the presence of the mud particles that may add a bonding factor that would give a higher cement strength for this material than that of Limestone 2, which has no clay. The degree to which bonding will effect the apparent cement strength of the Limestone 1 material will vary with the amount of clay in the sample.

The slag had a moderate strength, which again is most likely a function of the particle shapes. The igneous and metamorphic materials had moderate to low values. Their values were much lower than those of the carbonate materials.

TABLE 5 BALLAST CEMENTING TESTS RESULTS

| Material | Strength (psi) |
|---|---|
| Micrite | 1,330 |
| Dolomite | 980 |
| Quartzite | 650 |
| Limestone 1 | 560 |
| Limestone 2 | 490 |
| Slag | 320 |
| Carbonate | 190 |
| Granite 2 | 114 |
| Gneiss 3 | 110 |
| Gneiss 1 | 62 |
| Monzonite | 56 |
| Gneiss 2 | 53 |
| Basalt | 38 |
| Quartzite 2 | 23 |

Note: All material passes No. 200 sieve; strength is average for five specimens; kPa = 6.89 × psi.

## CONCLUSIONS AND RECOMMENDATIONS

Ballast is a key element in track performance. Ballast behavior in track is influenced by field conditions including

- Axle load magnitudes and number of repetitions,
- Track characteristics,
- Subgrade properties,
- Environmental factors, and
- Condition of track including drainage.

The remaining factors are the following ballast characteristics:

- Rock type and physical characteristics,
- Particle size and shape,
- Gradation, and
- Void ratio.

Ballast behavior will change for the same ballast characteristics if field conditions change. Even the relative ranking of different rock types can vary with field conditions. Petrographic analysis only provides information on rock type and physical characteristics. Furthermore, rock type is not an equally critical factor in all aspects of ballast performance. Thus petrographic analysis alone cannot be expected to provide sufficient information to predict ballast performance. The same is true of the commonly used index tests such as LAA and MA. Nevertheless, petrographic analysis and index tests do have potentially important roles in performance prediction.

Each ballast material used in the UMass research program was evaluated in hand specimen. Also, thin-section examinations were conducted when material availability permitted. The mineral composition, structure, and texture of each material were determined. The information on the physical properties of the materials obtained from these examinations was then used to evaluate the results of the index and ballast box tests performed with the materials, with the goal of determining how these properties correlated with the performance of the material in the tests.

The ballast characterization tests evaluated include the Los Angeles abrasion test, the mill abrasion test, the ballast cementing test, and the ballast box test. In general, petrographic analysis provided a reasonable explanation of the performance of the materials in each of these tests. In cases in which it could not, consideration of material gradation and particle shape could often account for the discrepancies.

The successful application of petrographic analysis to the evaluation of ballast performance in index and ballast box tests suggests that, with further research, the application of the technique can be extended to include an important role in the prediction of a ballast material's field performance. The research should further development of the ballast box to more completely represent field performance and establish (a) which index tests are most suitable, (b) the correlation between the results of the ballast box tests and field performance, (c) the correlation between the results of the appropriate index tests and field performance, (d) the correlation between petrographic analysis and the other tests, and (e) the potential for developing a numerical means of assessing a material's suitability as a ballast using petrographic analysis. The latter could be similar to the method used by the DOH of Ontario or based on a combination of index tests. The prediction of field performance using petrographic techniques would most likely require the evaluation to be made by a petrographer who is familiar with the determined correlations, as well as the physical and environmental conditions such as amount and type (i.e., acidity or alkalinity) of precipitation and temperature patterns.

Field observations are needed to determine ballast characteristics in response to field conditions. The box test would provide a controlled laboratory approach to studying the correlation among index tests, petrographic analysis, and field performance. The index tests are desired for routine ballast evaluation. Consideration should be given to extending them to represent some of the variables other than material type. Petrographic analysis would provide an efficient and economical means of determining the suitability of a potential quarry or a means of selecting the best material when a variety of materials is being considered. Petrographic analysis can also be used for general evaluations of performance of potential ballast materials, as well as for selection of appropriate index tests, thereby eliminating the need for an entire series of index testing on several materials to determine which is the best material. The evaluation should be made by a petrographer with knowledge of the conditions of the index tests (i.e., what each test is actually testing and what that means with respect to field performance), field conditions, and the effects of gradation and particle shape.

## GLOSSARY

Unless otherwise specified, these definitions were taken from Whitten and Brooks (*4*).

**Aphanitic:** A textural term applied to igneous rocks in which crystals are so fine that individual grains may only be seen under magnification.

**Birefringence:** Color produced when rays are put out of phase by double refraction. Colors vary with the thickness and orientation of the crystal.

**Cleavage:** The tendency of a mineral to break along planes of weak bonding. It is described by the number of planes exhibited and the angles at which they meet.

**Consolidation:** 1. In the geologic sense, the process of conversion of a loose or soft material to a compact, harder material [e.g., sand to sandstone (by cementation)]. 2. In the engineering sense, the gradual reduction in volume of a soil mass resulting from an increase in compressive stress (ASTM D 653-85).

**Cryptocrystalline:** A textural term used to describe a very fine crystalline aggregate in which crystals are so small as to be indistinguishable except under powerful magnification.

**Crystalloblastic:** A texture produced by simultaneous recrystallization of several minerals as a result of metamorphic processes.

**Equigranular:** A textural term applied to a rock in which the constituent grains are all of about the same size.

**Foliation:** The parallel alignment of platy minerals or mineral banding in rocks, especially metamorphic rocks.

**Groundmass:** The finer-grained material constituting the main body of a rock, in which larger units are set.

**Hardness:** A property of minerals that is determined by reference to an empirical scale of standard minerals. Mineral hardness is a "scratch" hardness, as opposed to the engineers' indentation hardness.

**Holocrystalline:** Those rocks that are entirely crystalline.

**Inequigranular:** A textural term applied to a rock in which the constituent grains are of markedly different sizes.

**Lepidoblastic:** A textural term applied to foliations or schistosity (rock cleavage) produced by alignment of planar minerals (e.g., mica).

**Mafic:** A general term used to describe ferromagnesian minerals.

**Matrix:** A term that is similar to groundmass, but used for sedimentary rocks.

**Micrite:** Cryptocrystalline calcite (less than or equal to 0.004 mm). It is used in the description and classification of carbonate rocks.

**Microspar:** Calcite crystals between 0.004 and 0.015 mm. A term used in the description and classification of carbonate rocks.

**Peloid:** A pellet of mud of uncertain origin, generally composed of micrite.

**Phaneritic:** A textural term applied to igneous rocks in which crystals can be separately distinguished by the naked eye.

**Pleochroism:** This occurs when a crystal shows color under nonpolarized transmitted light. The color changes depending on the orientation of the crystal.

**Refractive index:** The ratio of the velocity of light in a vacuum to the velocity of light in some other medium.

**Sparite:** Calcite, with grain size greater than 0.030 mm. A term used in the description and classification of carbonate rocks.

**Subhedral:** A crystalline solid with imperfectly developed faces.

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# Production and Testing of Ballast

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**The traditional method of selecting ballast has been based on physical testing of representative specimens to ensure that materials have adequate wearing resistance, toughness, physical stability, and strength to meet predetermined criteria. Sufficient testing is normally done to ensure that the ballast supply meets these criteria. Most tests are meant to establish that the ballast is durable at the time it is tested. Tests such as magnesium sulphate soundness or abrasion resistance give indirect evidence of how the ballast properties can be expected to change in the track structure. None of the tests currently employed, with the exception of gradation, gives direct information to explain the physical behavior and chemical stability of a ballast source, nor do they give any guidance that the engineer can use in selecting the most economical gradation or processing method for that source. The main objective of this paper is to demonstrate a rational methodology for the selection of natural rock ballast sources. To obtain the most cost-effective investment in ballast, source selection, site investigation, quarry design, quality control, and selection of specifications should be based on the geologic characteristics of the source.**

Ballast is produced from rock, a natural material that was deposited and subsequently modified in accordance with natural laws of physics and chemistry. Traditionally, little attention has been paid to the character of the rock mass when selecting a ballast source. Instead, the behavior of the rock mass has been inferred from a series of physical tests. These tests are not direct measurements of the physical and chemical properties of the rock but empirical measurements that allow interpretation of ballast behavior on the basis of experience with similar materials in similar environments.

Many of the fundamental properties of the rock mass are governed by the origin, alteration, and present condition of the rock. These characteristics, cumulatively known as petrography, give insight into the mechanical properties and allow judgments to be made on how a particular source of ballast can best be exploited. This includes planning quarry operations, designing crushing circuits, and selecting gradations and specifications. All of the properties of ballast are controlled by its geologic origins and the physical environment in which it is placed. Thus a knowledge of geology is essential in locating a ballast source, designing production facilities, setting specifications, and interpreting results of quality control tests.

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## DESIRABLE BALLAST PROPERTIES

A summary of desirable ballast properties and related tests was given by Raymond et al. (*1*). A comprehensive investigation of ballast performance in track structures was undertaken by Canadian Pacific (CP) Rail (see paper by Klassen et al. in this Record) to determine the modes of ballast failure and the factors that affect its performance. This study resulted in adoption of new ballast standards (see Appendix, pp. 59-63 in this Record) based on observed ballast perfomance modified by a combination of physical tests and petrography.

Observations of ballast performance in track confirm the importance of those factors described by Chrismer (*2*), Selig (*3*), and others. However, the CP Rail study was unique in that it considered petrographic factors to a greater degree than had other work to date. The study results are reported elsewhere in this Record (see papers by Klassen et al. and Watters et al.). The study confirmed, by observations of ballast in the track structure, that a desirable ballast must be

1. Hard, to withstand abrasion;
2. Tough, to withstand particle fracture on impact;
3. An intact mass without pores or intercrystalline defects, to withstand the effects of freezing and thawing;
4. Chemically stable;
5. Physically stable, to resist solution, fracture, and other forms of part icle breakdown;
6. Dense and rough, to give adequate stability;
7. Comprised of constant, approximately equidimensional particles with few flat or pointed particles;
8. Resistant to surface polishing that would reduce stability;
9. Permeable, able to drain rapidly;
10. Properly graded, open enough to pass fines and water but sufficiently uniform to ensure density and ease of handling;
11. Consistent, having the same qualities throughout the ballast mass;
12. Nonreactive and with low electrical conductivity, to avoid damage to rail, ties, and buried electrical systems; and
13. Resistant to abrasion but, on breaking down, preferably produces fines that are noncementing.

Klassen et al. (see paper in this Record) reported on a large number of observation trenches excavated through ballast, subballast, and subgrade in a wide range of soil types and water contents. No verifiable cases of subgrade intrusion into the subballast or subballast intrusion into the ballast were observed. The most commonly observed mode of failure was particle abrasion and fracture that plugged the voids with cementitious fines that caused the ballast to degrade into a cemented mass. Thus qualities related to wear resistance, fracture resistance, and the nature of fines were judged to be the most important for ensuring long ballast life.

## EVALUATION BY PHYSICAL TESTING

A summary of the physical tests used by CP Rail to assess ballast quality is given in Table 1. Specifications for standard ballast gradations are given in Table 2. Most of the test methods are standard tests specified by the American Railway Engineering Association (AREA) (*4*). Unique specifications have been developed for determination of fractured particles and abrasion number and the conduct of the mill abrasion test and petrographic analysis.

A rigorous fractured-particle specification was developed to ensure adequate stability, particularly in ballast from gravel sources, and to eliminate consideration of elongated or flaky particles as acceptable fractured particles. Sampling for the fractured-particle determination is done in accordance with ASTM D 75 and C 702. From each coarse fraction representing 5 percent or more of the sample, a representative portion is selected according to the criteria given in Table 3. The samples are separated into fractured and nonfractured particles. A fractured particle is one with three or more fractured faces. Each of the faces must have a freshly exposed rock surface the largest dimension of which is at least one-third of the maximum particle dimension and the smallest dimension of which is at least one-quarter of the maximum particle dimension. The included angle formed by the intersection of the average planes of adjoining fractured faces must be less than 135 degrees for each of the faces to be considered a separate fractured face. These criteria require a fractured face to have a minimum surface area and a minimum intersection angle between fractured faces. This requirement, along with the requirement to have three fractured faces, eliminates consideration of the flaky, shard-like particles (that are common in hard, fine-grained rock) as suitable fractured particles. The weight of fractured particles is calculated as a percentage of the total sample weight.

The mill abrasion test, described by Raymond et al. (*1*), uses a 6.6-lb (3-kg) sample representative of the coarse

TABLE 1 SUMMARY OF TEST METHODS AND STANDARDS FOR CP RAIL MAIN-LINE BALLAST

| Parameter | Test | Test Method | Comments |
|---|---|---|---|
| Permeability | Gradation | ASTM C 136, C 117 | See gradation limits |
| Stability | Bulk specific gravity | ASTM C 127 | Greater than 2.60 |
| Weathering | $MgSO_4$ soundness | ASTM C 88, five cycle on Ballast Grading 3 | Max 1% for primary CWR[a], 1.5% for main-line jointed rail, and 3% elsewhere |
| | Absorption | ASTM C 127 on Ballast Grading 3 | Max 0.5% for primary CWR, 0.75% for main-line jointed rail, and 1% elsewhere |
| Abrasion and wear | Los Angeles abrasion | ASTM 535, Grading 3 | Less than 45% loss |
| | Mill abrasion | CP Rail | Less than 9% loss |
| | Abrasion number | CP Rail | Less than 65 and less than that required for cumulative tons of rail traffic for a 20-year period (from Figure 1) |
| Overall suitability | Petrographic analysis | CP Rail | Professional judgment to identify potential flaws |

[a] CWR = continuously welded rail.
Source: CP Rail, 1983.

TABLE 2 GRADATION STANDARDS, CP RAIL BALLAST

| Ballast Grading | Percentage (by weight) Finer Than | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 2 1/2 in. | 2 in. | 1 1/2 in. | 1 in. | 3/4 in. | 1/2 in. | 3/8 in. | No. 4 | No. 200 |
| 1 | 100 | 100 | 100 | 90-100 | 70-90 | 40-60 | 20-40 | 0-3 | 0-2 |
| 2 | 100 | 100 | 90-100 | 70-90 | 50-70 | 25-45 | 10-25 | 0-3 | 0-2 |
| 3 | 100 | 100 | 90-100 | 70-90 | 30-50 | 0-20 | 0-5 | 0-3 | 0-2 |
| 4 | 100 | 100 | 90-100 | 20-55 | 0-5 | | | 0-3 | 0-2 |
| 5 | 100 | 90-100 | 35-70 | 0-5 | | | | 0-3 | 0-2 |

Source: CP Rail, 1983.

TABLE 3 SAMPLE COMPOSITION, FRACTURED-FACE TEST

| Passing Sieve | Retained on Sieve | Weight (lb. ± 10%) |
|---|---|---|
| 2 in. | 1 1/2 in. | 13 |
| 1 1/2 in. | 1 in. | 6.5 |
| 1 in. | 3/4 in. | 3.5 |
| 3/4 in. | 1/2 in. | 2.25 |
| 1/2 in. | 3/8 in. | 1.0 |
| 3/8 in. | No. 4 | 0.75 |

fraction of the aggregate. Three and one-third pounds (1.5 kg) of the sample are from the fraction passing the 1-in. (25-mm) and retained on the 3/4-in. (19-mm) sieve, and an equal amount is from the fraction passing the 1 1/2-in. (37.5-mm) and retained on the 1-in. (25-mm) sieve. The sample is washed and oven dried in accordance with the Los Angeles abrasion (LAA) test procedure (ASTM C 535). It is then placed in a 1-gal (4.5-L), 9-in. (229-mm) outside diameter porcelain ball mill pot, along with 6.6 lb (3 kg) of distilled water. The pot is rotated at 33 rpm for a total of 10,000 revolutions (about 5 hr), then washed through a No. 200 (71-$\mu$m) sieve and oven dried to determine the percentage loss in weight. The mill abrasion (MA) number equals loss in weight divided by original weight times 100.

The abrasion number (AN) is an index number (see paper by Klassen et al. in this Record) calculated according to the following formula:

$AN = LAA$ loss in % + 5 × $MA$ loss in %
or
$$AN = LAA + 5\ MA \qquad (1)$$

## PETROGRAPHIC EVALUATION

A large amount of physical testing is required to determine the uniformity and consistency of a ballast source. However, such testing gives little insight into the chemical weathering potential of a ballast or whether the physical properties will slowly change with time. Consideration of petrology, and particularly petrography, will give valuable information on the character of the rocks that may make possible a more comprehensive interpretation of the physical test results. Petrography deals with the description of the characteristics of rocks, both in-hand specimens and thin sections. Watters et al. (see paper in this Record) and CP Rail (*5*) (see Appendix, pp. 59-63 in this Record) describe the information that should be provided when an experienced petrologist performs a petrographic analysis. The megascopic features are obtained from inspection of hand specimens, and microscopic features are determined from evaluation of thin sections under a petrographic microscope. A complete petrographic description includes

1. Delineation of rock types;
2. Mineralogy of the rock types including proportions of various minerals;
3. Texture including grain size, shape, orientation, mutual relationships, and matrix materials;
4. Structure, identifying bedding, fracture, and cleavage and foliation planes;
5. Estimation of mechanical properties including hardness, strength, brittleness, and fracture characteristics;
6. Chemical properties that may affect alteration and potential chemical weathering;
7. Properties of fines produced including gradation, permeability, and susceptibility to solution or cementing; and
8. Interpretation or prediction of physical test results and identification of any special tests required.

Petrographic examination is used to verify or modify the AN determined by physical testing. When the AN is selected, the life of various ballast gradations is estimated from Figure 1. Should the anticipated life of the ballast not be adequate, an alternative source must be found.

## SELECTION OF BALLAST SOURCES

A logical sequence for selection of ballast sources, both in bedrock terrain and from gravel deposits, is shown in Figure 2. Watters et al. (see paper in this Record) discuss the role of

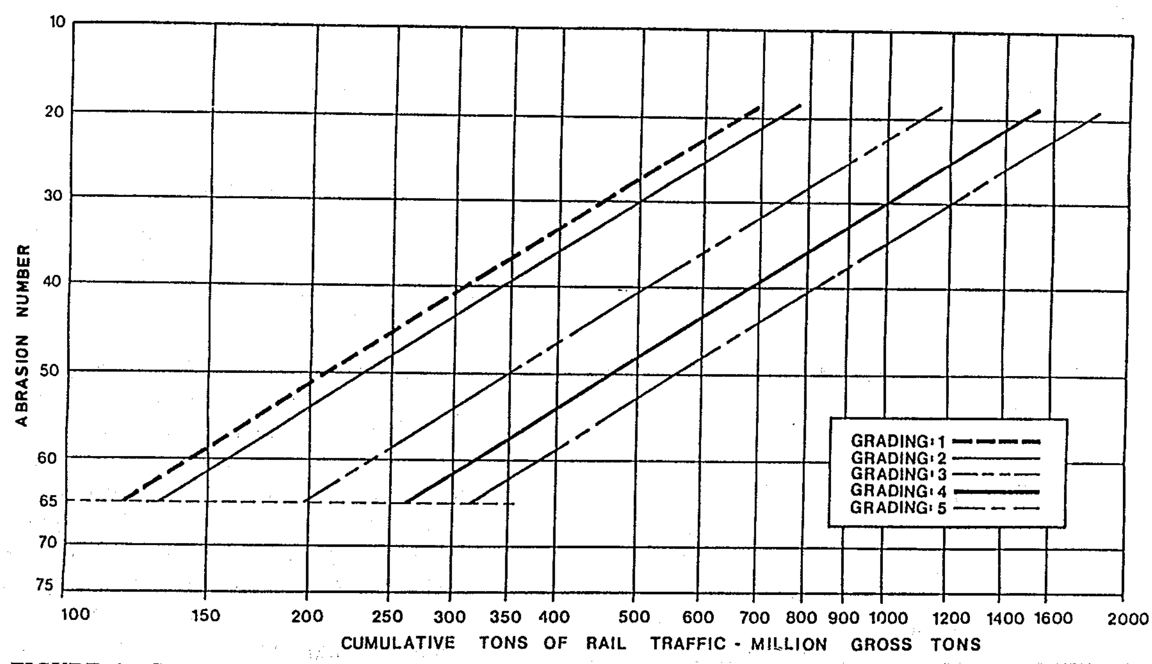


**FIGURE 1 Cumulative tons of traffic versus abrasion number.**

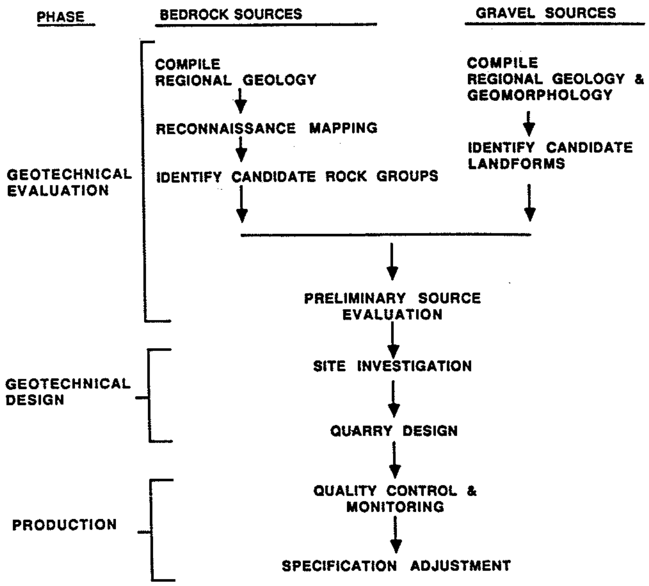


**FIGURE 2 Sequence for selection of ballast sources.**

geology, particularly petrography, in selecting suitable ballast sources. Candidate rock groups can be identified from consideration of geologic properties and from experience with similar materials.

The location of potential sources can be first identified through consideration of regional geology, usually available from small-scale regional geologic maps and reports. The candidate rock groups are further investigated by reconnaissance-level geologic mapping, followed by a preliminary source evaluation that includes initial petrographic screening. If the materials are deemed suitable, a detailed site investigation ensues, followed by qualification testing of the rocks recovered from cores, test trenches, and representative exposures. The geotechnical parameters and geologic structure of the rock mass must be determined so that quarry layout and crushing circuits can be designed. When production begins, quality control testing of the ballast and geologic monitoring of the source rocks is required. If necessary, the specifications may require adjustment to ensure the most economical and effective use of the quarried rock.

A similar procedure is recommended for gravel sources. The geologic evaluation consists of consideration of regional geology and geomorphology. Landforms with suitable depositional environments are identified as potential sources. These usually consist of landforms deposited in a high-energy (fluvial) environment such as terraces, point bars, and deltas on steep, fast-flowing streams and alluvial fans or eroded bedrock surfaces.

In glaciated terrain, outwash plains, eskers, kames, and eroded till plains often provide good-quality gravel sources. Geotechnical design usually includes a site investigation to determine groundwater levels, stripping ratios, and gradations of the source material. The pit design includes designation of storage areas for waste and development of reclamation plans to minimize rehandling of material during pit development. Quality control and specification adjustment procedures are similar to those employed for bedrock sources.

The implementation of this methodology is illustrated by a review of the development of a bedrock quarry near Walhachin, British Columbia, in the Thompson River Valley of Canada.

## DEVELOPMENT OF THE WALHACHIN QUARRY

CP Rail required a high-quality ballast source for their main line in the lower mainland of British Columbia. This track carries in excess of 50 million gross tons per year in an area that receives high rainfall and where the ballast is subjected to freezing and thawing cycles in addition to heavy traffic. Potential quarry locations were selected after consideration of regional geology that identified a band of metamorphosed volcanics in the Thompson River Valley. However, closer examination revealed that most of these rocks had been substantially altered, which reduced their hardness. Reconnaissance identified an area of metamorphosed volcanics that consisted largely of basalt with lesser carbonate rocks. This area was in the vicinity of an abandoned quarry, which made it possible to conduct considerable geologic mapping from outcrops. Preliminary laboratory testing indicated that the unaltered basalt would produce high-quality ballast but that the softer carbonates were unacceptable. If a quarry working plan could be developed to exclude the carbonates, high-quality ballast could be produced.

Detailed mapping was undertaken to delineate the geologic structure of the area. Four test holes were cored to provide additional geologic and geotechnical information and recover representative samples of various rock types. In addition, the joint and fracture patterns in existing rocks were mapped to identify potential zones of weakness that might present a hazard during quarry operations. On the basis of these data, the geology of the site was delineated as shown in plan in Figure 3 and in a typical cross section in Figure 4. The geotechnical evaluation determined that the bench slopes should not be steeper than 65 degrees to the horizontal and that a berm width of 25 ft should be maintained. The overall slope angle on a continuous slope with no ramps should not be greater than 37 degrees. Using these parameters, the quarry plan shown in Figure 5 was developed to recover approximately 1 million tons of ballast from the quarry.

The geologic evaluation indicated that six rock types were predominant in the quarry. Petrographic evaluation indicated that the basic tuffs and breccias and basic intrusives would yield the highest quality ballast suitable for main-line continuously welded rail. The intermediate tuffs and breccias were less desirable but would produce secondary ballast with a slightly lower expected life. The calcareous tuffs and breccias, limestone breccias, and coarse intermediate breccias were judged to be unsuitable for ballast and would be used as riprap or would be wasted. The amounts of primary ballast, secondary ballast, and waste rock to be recovered from the quarry are summarized in Table 4. These data indicate that to recover approximately 1.2 million cubic yards of primary ballast and 0.2 million cubic yards of secondary ballast, approximately 1.6 million cubic yards of rock must be quarried.

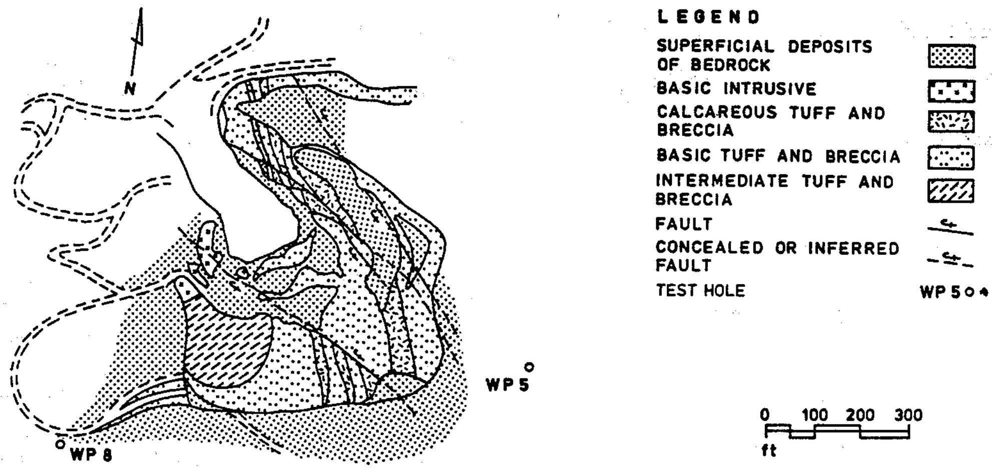


FIGURE 3 Geologic map of the Walhachin Quarry.

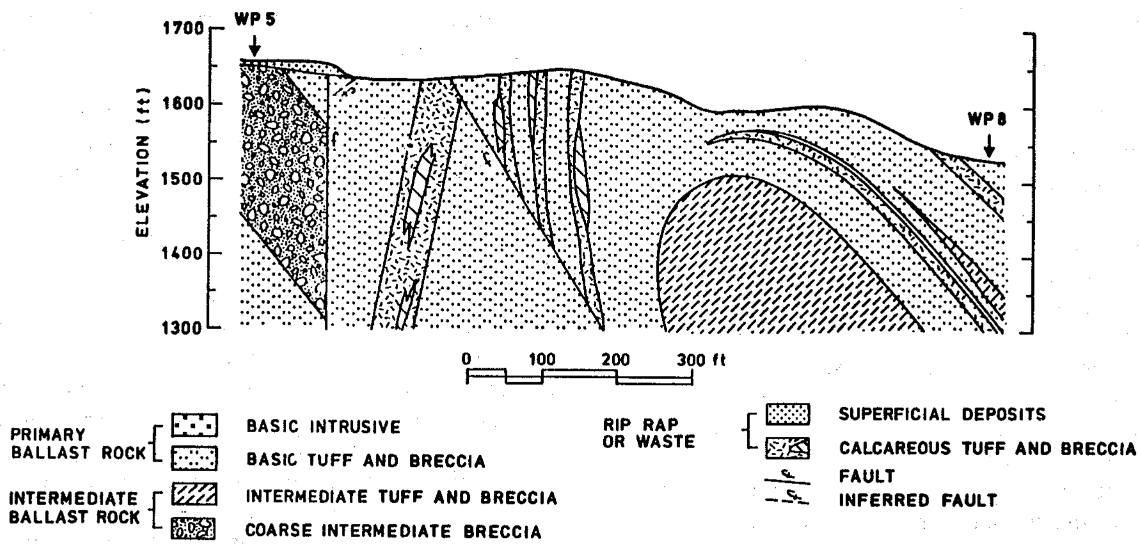


FIGURE 4 Typical cross section of the Walhachin Quarry.

TABLE 4 SUMMARY OF DESIGN QUARRY VOLUMES

| Bench Level (ft above mean sea level) | Volume ($yd^3 \times 1{,}000$) | | | |
|---|---|---|---|---|
| | Primary Ballast | Secondary Ballast | Rock | Total |
| 1,630 | 108 | | 84 | 192 |
| 1,600 | 158 | | 52 | 210 |
| 1,570 | 170 | | 41 | 211 |
| 1,540 | 136 | | 31 | 167 |
| 1,510 | 136 | 2 | 17 | 155 |
| 1,480 | 98 | 21 | 8 | 127 |
| 1,450 | 96 | 32 | 7 | 135 |
| 1,420 | 70 | 41 | 3 | 114 |
| 1,390 | 66 | 41 | | 107 |
| 1,360 | 67 | 17 | | 84 |
| 1,330 | 46 | 13 | | 59 |
| 1,310 | 31 | 7 | | 38 |
| Total | 1,182 | 174 | 243 | 1,599 |

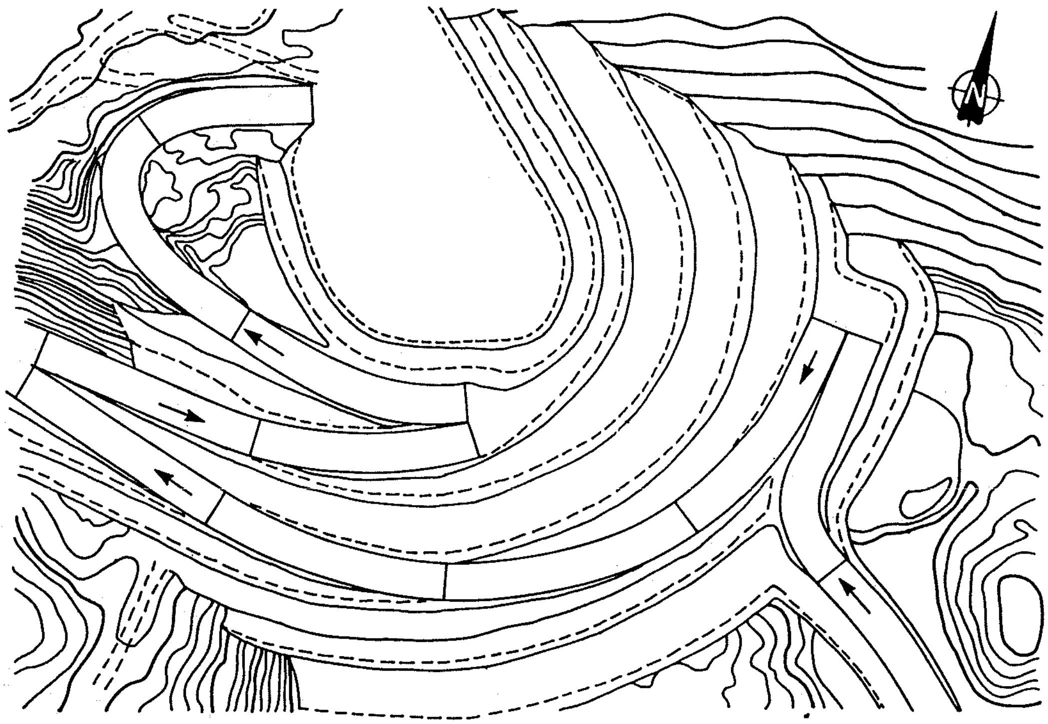


**FIGURE 5 Quarry plan.**

A contract was awarded for development of the quarry and production of CP Grading 4 (similar to AREA 4) ballast. A conventional primary jaw and secondary and finishing cone crushing circuit was envisaged, and it was estimated that approximately 40 percent of the crusher feed would be lost as reject. Quality control testing consisting of unit weight, bulk specific gravity, and gradation testing was conducted continuously on site. Other physical tests were conducted on representative samples of each rock type at various intervals. A comparison of physical test results from preliminary investigation and results obtained during production testing is given in Table 5.

The results indicate that a careful testing program conducted on materials with similar petrographic characteristics can yield reliable and reproducible results. Careful geologic control was the key to obtaining reproducible test results. Geologic control was accomplished through careful mapping of each bench and selection of material in the quarry by a qualified engineering geologist. A great deal of control was required in areas of complex geologic structures and proportionately less in areas of more homogeneous rock types.

TABLE 5 COMPARISON OF PREDEVELOPMENT AND PRODUCTION TEST RESULTS

| Rock Type | LAA | MA | AN | $MgSO_4$ | Absorption | Specific Gravity |
|---|---|---|---|---|---|---|
| Basic tuffs and breccias | | | | | | |
| Predevelopment | 11.3-17.5 | 1.6-3.9 | 19-33 | 1.3-3.3 | .31-.68 | 2.77-2.96 |
| | (14.2)[a] | (3.0) | (29) | (1.6) | (.48) | (2.82) |
| Production | 8.8-14.8 | 2.2-3.9 | 20-34 | 0.6-1.5 | 0.2-0.8 | 2.79-2.86 |
| | (12.6) | (3.1) | (28) | (0.8) | (0.6) | (2.83) |
| Intermediate tuffs and breccias | | | | | | |
| Predevelopment | 16.6-18.9 | 4.9-5.7 | 41-47 | 6.4-6.7 | .48-.60 | 2.9-3.2 |
| Production | 13.0-16.9 | 2.6-5.1 | 26-41 | 0.9-2.8 | 0.25-1.0 | 2.77-2.93 |
| | (16.7) | (4.0) | (36) | (2.1) | (.65) | (2.87) |
| Calcareous tuffs and breccias, limestone breccias, and coarse intermediate breccias | | | | | | |
| Predevelopment | 13.9-33.6 | 5.6-11.9 | 42-89 | 0.93-27 | .28-.86 | 2.53-2.86 |
| Production | 17.9-35.7 | 5.8-7.7 | 47-74 | | | |

[a] Mean values are in parentheses.

## ADJUSTMENT OF SPECIFICATIONS

Gradation changes have a significant impact on void ratio. Fuller and Thompson (*6*) proposed a method of evaluating the relationship between grain size distribution and sample porosity. Their equation is expressed as

$$P = 100\,(d/D)^n \tag{2}$$

where

$d$ = particle size in question,
$D$ = maximum particle size,
$P$ = percentage finer than, and
$n$ = an exponent to adjust the curve.

This relationship is known as Fuller's maximum density curve and is dependent on the exponent $n$. The maximum density results when $n = 0.5$, but $n = 0.45$ is commonly taken as defining the practical maximum density of graded aggregate masses. This is taken as a zero air voids curve. The amount by which a ballast gradation curve deviates from the theoretical maximum density gradation is an indication of the voids in the ballast. The greater the area between the two curves, the greater the volume of voids.

The relationship between gradation and voids in a sample was determined by plotting the appropriate zero voids gradation and the ballast gradation on the same semi-logarithmic plot (Figure 6). The area bounded by the two curves, above the No. 4 sieve size, was determined by planimeter. The area was then plotted against void ratio for each sample. A linear regression analysis was used to establish the relationship between void ratio and area for various gradations (Figure 7).

The importance of voids and the necessary void ratio in a ballast are subject to some conjecture. However, examination of ballast during field studies revealed that, where ballast was poorly drained, cementing generally resulted, causing pumping of ballast fines [that fraction of the sample passing the No. 4 (4.75-mm) sieve] around the tie. Klassen et al. (see paper in this Record) concluded that a high void ratio was desirable for a high-tonnage track to prevent plugging of the voids. Studies showed that ballast with AREA 4 gradation required about 20 to 25 percent fines to plug the voids whereas a Raymond 2 gradation (*1*) required only 10 to 15 percent fines to accomplish the same degree of plugging.

The effect of gradation on void ratio was studied using Fuller's equation to predict the zero voids gradation for 2- and 2 1/2-in. top size. Ballast samples from several pits were graded to the various gradations (Raymond 1 and 2; AREA 3, 4, and 24; and CP 1 and 3) noted in Figure 7 and the void ratio was determined in the laboratory at the maximum density that could be achieved by compacting the sample with an electric "Kango" impact hammer acting on a 1-in. (25-mm) steel plate surface on top of the sample mold. This gave a density higher than that achievable by vibrating table methods (ASTM D 4253-83). The area between the gradation curve and Fuller's curve was determined for $n = 0.45$.

The relationship between void ratio and area under the curve for the various ballast samples is shown in Figure 7.

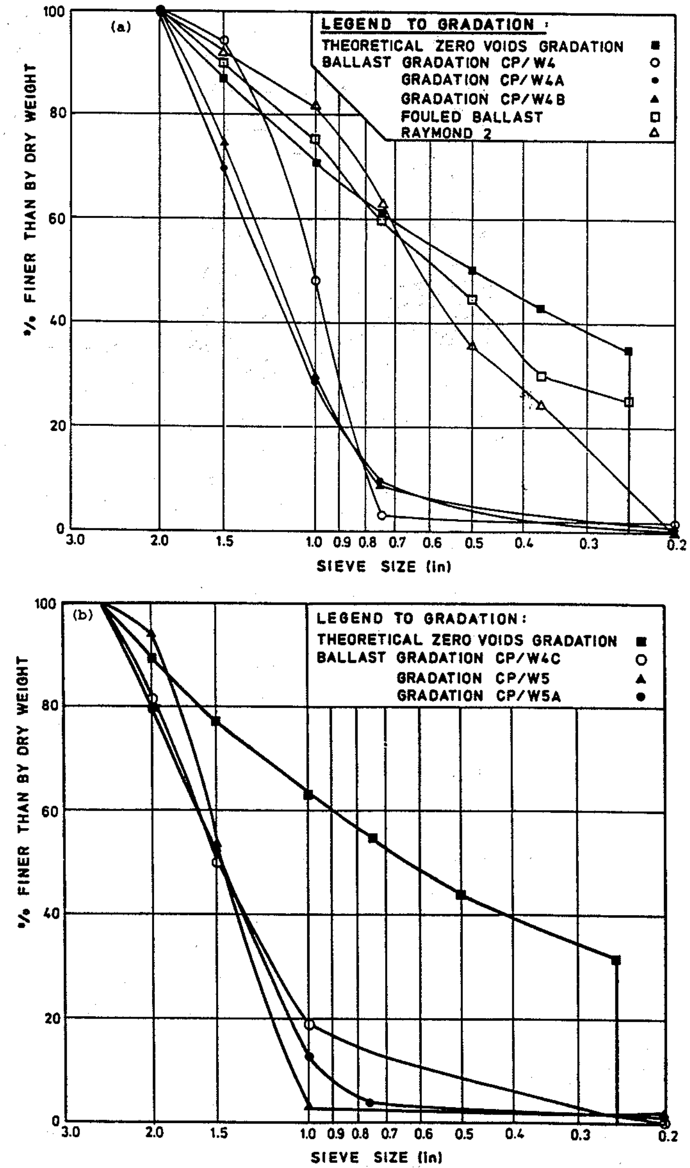


FIGURE 6 Zero voids gradation and ballast gradation.

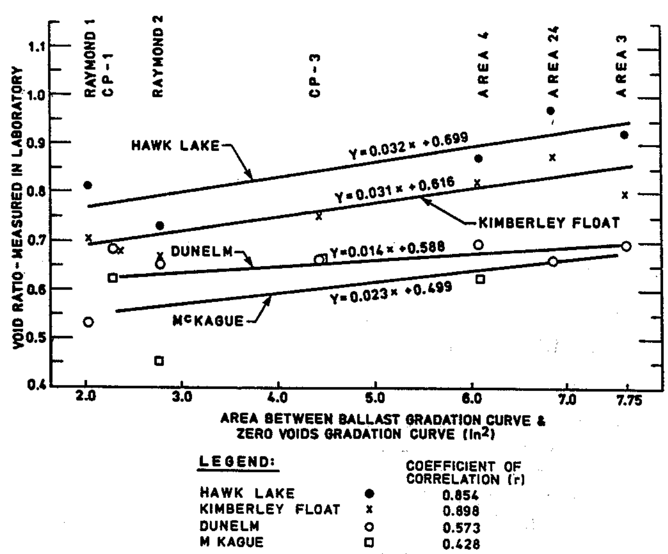


FIGURE 7 Relationship between void ratio and area.

This figure illustrates that the compaction experienced by each ballast is unique and probably dependent on particle roughness and shape. The prairie gravel samples (McKague and Dunelm) compacted more than quarried rock samples (Kimberley and Hawk Lake) and hence had fewer available voids for storage of fines. Although it is probable that ballast may compact more in track than in laboratory samples, these relationships give a subjective evaluation of the impact of gradation on void ratio and allow interpretation of the portions of the grain size curve that contribute to ballast porosity. This is further shown in Figure 6.

A number of crusher tests were undertaken to determine the ballast gradation that could be most economically produced in the Walhachin Quarry. Six trial gradations, summarized in Table 6, were produced in various tests. The amount of fines rejected and ballast produced during each test was carefully monitored using calibrated belt scales. It was found that the amount rejected was quite sensitive to ballast gradation. This sensitivity resulted from a tendency for the very hard basalt to shatter in the cone crushers when crushed to the finer gradations. Table 7 gives a summary of the yield of the quarry (tons of ballast per cubic yard of quarried rock and tons of reject per cubic yard of quarried rock) and the cost index for producing the various gradations, assuming gradation CP W4C as an index of 1.0. It can be seen that Gradation CP W5 was approximately 34 percent more expensive to produce than Gradation CP W4C for this rock type. However, it must be emphasized that these cost ratios are dependent on the rock type and processing circuits used and will vary considerably.

Figure 6a shows the theoretical maximum density gradation for 2-in. top size aggregate and the average gradation curves for the CP W4 (approximately AREA 4), CP W4A, and CP W4B gradations. Also illustrated is the typical gradation for a fouled ballast. The latter curve closely parallels the theoretical maximum density curve, indicating that few usable voids remain in the fouled ballast sample. Figure 6b shows the theoretical maximum gradation curve for 2 1/2-in. top size ballast along with average gradation curves for Walhachin trials CP W4C, CP W5, and CP W5A.

A relationship between void ratio and area under the Fuller curve can be established for any particular ballast. Figure 8 shows such a relationship for the Walhachin ballast. Also shown in Figure 8 are the void ratios corresponding to several of the trial gradations. It can be seen that changing from Gradation CP W4 to Gradation CP W4C increased the void

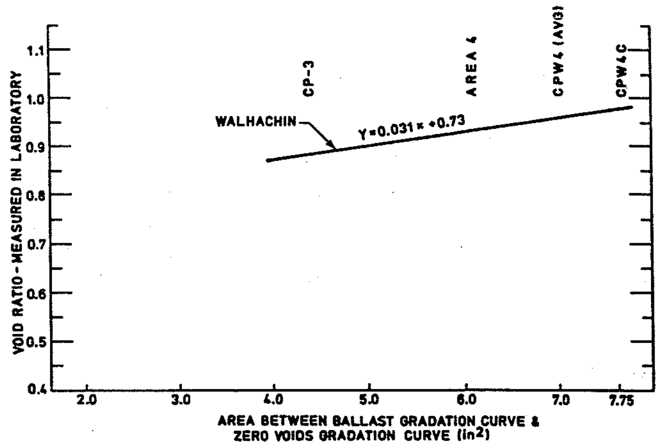


**FIGURE 8 Relationship between void ratio and area for Walhachin ballast.**

TABLE 6 SUMMARY OF CRUSHING TRIALS

| | Percentage Passing Grading | | | | | |
|---|---|---|---|---|---|---|
| | CP 4 | CP W4A | CP W4B | CP W4C | CP 5 | CP W5A |
| Sieve size | | | | | | |
| 2 1/2 in. | | | | 100 | 100 | 100 |
| 2 in. | 100 | 100 | 100 | 82 | 90-100 | 79 |
| 1 1/2 in. | 90-100 | 69 | 75 | 50 | 35-70 | 51 |
| 1 in. | 20-55 | 27 | 28 | 19 | 0-5 | 14 |
| 3/4 in. | 0-5 | 10 | 9 | 6 | | 3 |
| No. 4 | 0-3 | 0.5 | 1 | 0.7 | 0-3 | 1 |
| No. 200 | 0-2 | 0.1 | 0.5 | 0.2 | 0-2 | 0.6 |
| Percentage rejected | 48 | 40 | 42 | 39 | 54 | 45 |

TABLE 7 SUMMARY OF YIELD

| | Grading | | | | | |
|---|---|---|---|---|---|---|
| | CP 4 | CP W4A | CP W4B | CP W4C | CP 5 | CP W5A |
| Ballast/yd$^3$ rock (tons) | 1.139 | 1.313 | 1.270 | 1.335 | 1.008 | 1.205 |
| Reject/yd$^3$ rock (tons) | 1.051 | 0.875 | 0.919 | 0.854 | 1.183 | 0.986 |
| Production cost index | 1.12 | 1.01 | 1.04 | 1.0 | 1.34 | 1.08 |

ratio and reduced processing costs approximately 12 percent. Thus, assuming a similar void size, the ballast should have a life similar to or slightly longer than that of the original Grading CP 4 specification.

## CONCLUSIONS

The geology of the ballast source is a major consideration in determining the suitability and economics of ballast production. The petrographic characteristics should be considered at an early stage in assessing the suitability of a source.

A rational approach incorporating consideration of petrography and physical test parameters has been demonstrated for the location, design, production, and monitoring of ballast production.

Specifications, arbitrarily set, can have a major impact on the cost of ballast. Experience at the Walhachin Quarry yielded at least a 34 percent differential in costs for ballast gradations with relatively minor variations.

Relatively stringent ballast gradations mean that the major impact of a specification change is in the volume of voids available to store fines. A method of evaluating the significance of gradation change that uses Fuller's equations and CP Rail's ballast performance charts has been suggested.

## ACKNOWLEDGMENTS

The data presented in this paper are the result of detailed ballast studies undertaken by CP Rail and of operation of their quarry at Walhachin, British Columbia. Appreciation is expressed to CP Rail staff for making these data available.

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# Track Evaluation and Ballast Performance Specifications

M. J. KLASSEN, A. W. CLIFTON, AND B. R. WATTERS

**A variety of laboratory tests has traditionally been used in the selection of a material and gradation for ballast. The results of the laboratory tests are commonly used to reject or accept material for use as ballast and rarely imply benefits or costs of selecting alternative materials or gradations. In this paper are presented the results of a track sampling and evaluation program to determine the performance of ballast, subballast, and the subgrade on Canadian Pacific (CP) Rail. A simple method was developed to determine if a significant portion of maintenance of the track structure is attributable to the subballast and subgrade. The concept of ballast life is presented with a relationship between ballast quality and grading. CP Rail specifications for the selection of a ballast material and the processing of ballast are also presented.**

The economics of selecting a track ballast is a function of production, transportation, and placement costs and maintenance costs during the period the ballast is in the track. A state-of-the-art review by the Association of American Railroads (AAR) Technical Center (*1*) in 1979 indicated that there were economic incentives for selecting superior quality ballast materials, although there was a variety of opinions on the appropriate standards and tests for ballast.

Field observations at the Facility for Accelerated Service Testing (FAST) (*2*) indicated that track maintenance is required as a result of a combination of deflections of the ballast, subballast, and subgrade. Fouling of ballast is considered a combination of ballast degradation and contamination from outside sources, both above and below the ballast.

A track ballast sampling and evaluation program was undertaken on Canadian Pacific (CP) Rail during 1979 and 1980 to evaluate the performance of ballast, subballast, and subgrade at selected locations. A CP Rail specification for ballast (see Appendix, pp. 59-63 in this Record), which considers the economic factors of ballast production, ballast life, and ballast and subgrade maintenance, was developed.

Research by Raymond et al. (*3*) developed a track-class ballast ranking and recommended a radical change in ballast grading to an extremely broadly graded ballast for heavy tonnage main lines. A secondary consideration of the ballast evaluation program was to determine if the Raymond track-class ballast rankings and ballast gradation should be adopted for the CP Rail main line.

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## MAINTENANCE AND BALLAST REPLACEMENT

Ballast on CP Rail historically was selected on the basis of geologic engineering properties of the rock and its past performance as ballast. Primary consideration was given to materials in proximity or within reasonable train haul of the required location.

Surfacing is planned according to historical maintenance schedules and deterioration of track surface. As the condition of the ballast deteriorates, the track surface becomes difficult to maintain and maintenance costs increase. Reballasting is scheduled to be completed when the old ballast has deteriorated.

A literature review of ballast performance in 1985 (*4*) found general disagreement about properties that are important to ballast life. The consensus of the Ballast and Subgrade Working Group of the AAR (*5*) is that decisions on ballast and subgrade maintenance are subjective in nature and usually made by local personnel.

A proposed model for subgrade performance (*6*) is shown in Figure 1. Permanent settlement decreases with tonnage for a good subgrade but increases with tonnage for a poor subgrade. Track maintenance for a poor subgrade, and possibly the threshold condition, will be controlled by the permanent settlement of the subgrade. Track maintenance on a good subgrade will be controlled by the ballast.

A conceptual ballast performance model on good subgrade (*6*) is shown in Figure 2. The model has been modified to illustrate ballast life and portrays repetitive surfacing cycles with an increasing maintenance frequency as the ballast condition deteriorates. When track roughness increases to an unacceptable level, ballast replacement is required. A similar model for maintenance costs is shown in Figure 3. Curves for a particular ballast indicate the individual performance of the properties of the ballast, which affect the life of the ballast and maintenance frequency.

## TRACK EVALUATION PROGRAM

The purpose of the CP Rail track evaluation program was to

1. Develop an understanding of the performance of ballast, subballast, and subgrade by evaluating existing track structures;
2. Determine the significance of ballast—the importance of ballast material quality, grading, and stability—and the predominant properties of ballast;

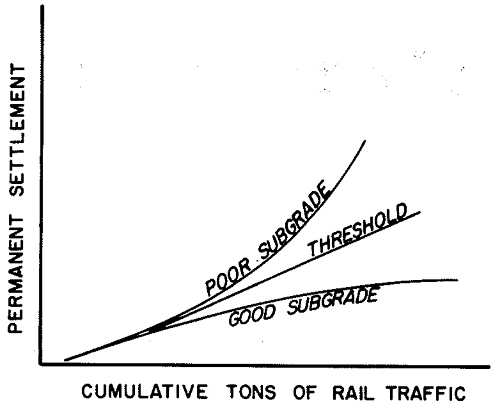


**FIGURE 1 Subgrade performance model.**

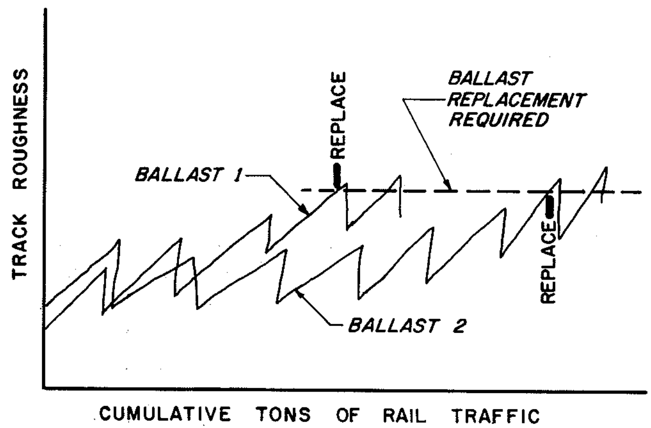


**FIGURE 2 Ballast performance model, track quality.**

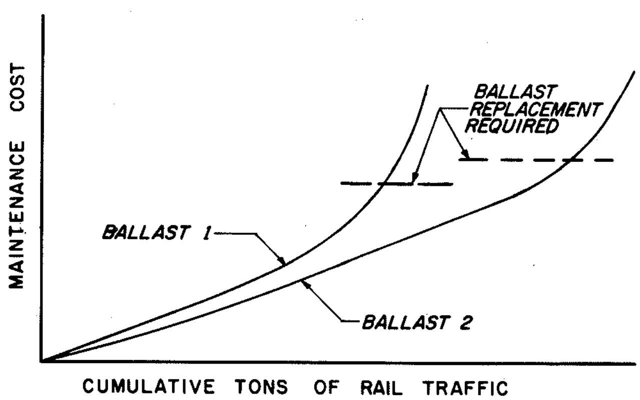


**FIGURE 3 Ballast performance model, maintenance cost.**

3. Develop an understanding of subballast and subgrade conditions and performance; and
4. Develop a CP Rail specification for ballast, evaluation of ballast sources, and processing of ballast.

Track sections were evaluated by hand excavating the ballast between two ties to the subgrade. The rails were raised to allow a tie adjacent to the excavation to be removed. Samples were carefully collected and identified on a ballast sample location diagram similar to that shown in Figure 4. Technical information for each location was cataloged on an inspection form (Figure 5). The samples were subjected to laboratory testing (see paper by Clifton et al. in this Record)

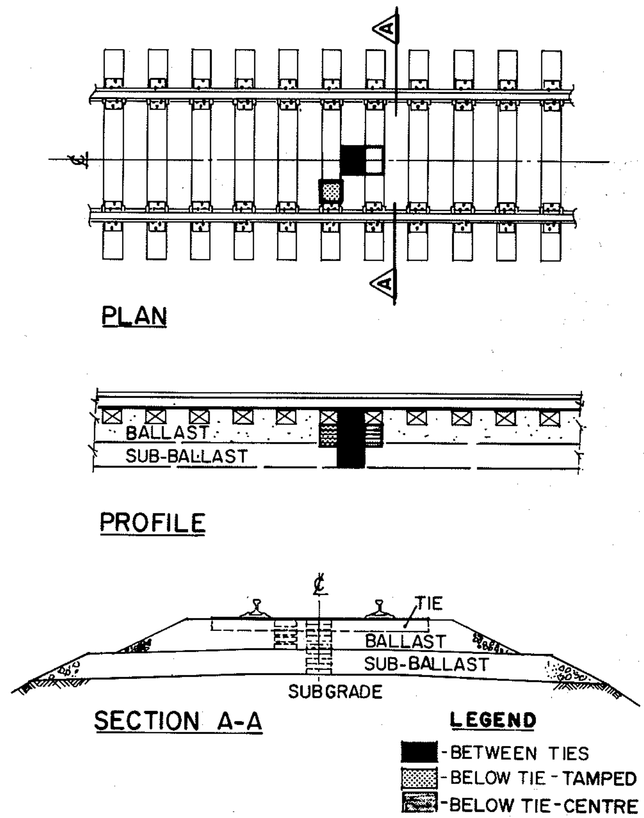


**FIGURE 4 Ballast sample location.**

GENERAL INFORMATION

SUBDIVISION ________ MILEAGE ________ DATE ________

BALLAST SOURCE ________ YEAR BALLASTED ________

BALLAST GRADATION ________ BALLASTING METHOD ________

DESCRIPTION OF BALLAST

MATERIAL ________

FOULING ________

SEGREGATION ________

CEMENTING ________

MOISTURE ________

DESCRIPTION OF SUB-BALLAST

MATERIAL ________

CEMENTING ________

MOISTURE ________

DESCRIPTION OF SUBGRADE

MATERIAL ________

STRENGTH ________

MOISTURE ________

TRACK PERFORMANCE

________

________

**FIGURE 5 Inspection form.**

and petrographic examination (see paper by Watters et al. in this Record) to develop an understanding of performance of ballast in track.

An excavation in broadly graded Kimberley Float ballast at Mile 65.0, Mountain Subdivision, is shown in Figure 6. The ballast had been in service for 5 years with 52 million gross tons (MGT) of rail traffic annually. The tamped ballast on the left side of the photograph was dense with significant fines filling the voids. Below the center of the tie, in the center of the photograph, the ballast was looser with less fines in the voids. Subsequent excavation showed neither mixing of the pit run subballast and the ballast nor permanent deflection of the ballast-subballast contact.

An excavation in broadly graded Walhachin ballast at Mile 37.2, Shuswap Subdivision, is shown in Figure 7. The ballast had been in service for 8 years with 52 MGT of rail traffic annually. The photograph shows dense ballast in the tamped area below the tie with sufficient fines in the voids to hold water below the tie. The voids in the visible ballast between the ties appeared to be filled with fines when compared with the clean ballast below the center of the tie. Subsequent excavation showed neither mixing of the pit run subballast and the ballast nor permanent deflection of the ballast-subballast contact.

An excavation in uniformly graded Kimberley Float ballast at Mile 52.2, Mountain Subdivision, is shown in Figure 8. The ballast had been in service for 17 years with 52 MGT rail traffic annually. The location was adjacent to a siding switch at a sawmill, and the fines in the ballast showed signs of contamination from woodchips. Subsequent excavation showed neither mixing of the pit run subballast and the ballast nor permanent deflection of the ballast-subballast contact.

An excavation in uniformly graded Uhtoff ballast at Mile 13.2, Mactier Subdivision, is shown in Figure 9. The ballast had been in service for 29 years with 16 MGT of rail traffic annually. The fines below the tie had filled the voids and were holding water. The visible ballast between the ties also appeared to have the voids filled with fines. The visible contact between the ballast and the pit run subballast showed neither signs of mixing of the ballast and pit run subballast nor signs of permanent deflections of the ballast-subballast contact.

Old, strong, fouled ballast was examined at Mile 110.4, Ignace Subdivision, as shown in Figure 10. The clear color differentiation between the old subballasts and the darker fouled ballast indicated neither mixing nor permanent deflections of the ballast-subballast contact.


**FIGURE 6 Mile 65.0, Mountain Subdivision.**

**FIGURE 8 Mile 52.2, Mountain Subdivision.**

**FIGURE 7 Mile 37.2, Shuswap Subdivision.**

**FIGURE 9 Mile 13.2, Mactier Subdivision.**

A clear contact between the newer ballast and the older darker fouled ballast was observed at Mile 13.8, Kaministiquia Subdivision, as shown in Figure 11. No mixing of the materials was apparent, although the convex shape of the visible contact below the tie indicated that subgrade deflections were occurring. Subsequent excavation identified a layer of peat moss 2 ft below the base of the tie.

Good performance of crushed gravel ballast is shown in Figure 12. The track ties were stable. The out-of-phase maintenance cycle is 2 to 3 years with annual rail traffic of 10 MGT.

Poor performance of ballast is shown in Figure 13. The track ties were moving in the ballast. Out-of-phase maintenance is required annually with 3 MGT annual rail traffic.

**FIGURE 10 Mile 110.4, Ignace Subdivision.**

**FIGURE 11 Mile 13.8, Kaministiquia Subdivision.**

## DEVELOPMENT OF A GRADATION SPECIFICATION

### Field Studies

The ballast field study consisted of the evaluation of 54 track locations; 259 samples of ballast and subballast were collected. The ballast at Mile 65, Mountain Subdivision, and Mile 37.2, Shuswap Subdivision, has been categorized as broadly graded ballast. The initial gradation at these sites was between the 2-in. and the No. 4 sieve. The ballast at Mile 52.2, Mountain Subdivision, and Mile 13.2, Mactier Subdivision, has been categorized as uniformly graded ballast with the initial gradation between the 2-in. and the 3/4-in. sieve. The ballast from Walhachin and Uhtoff quarries was produced to a tight specification. Ballast from Kimberley met a less stringent specification; processing consisted of removing the oversized and undersized material.

The broadly graded ballast sample gradations are shown in Figures 14-17, and the uniformly graded ballast sample gradations are shown in Figures 18-21.

The fines content (percentage passing the No. 4 sieve) increased with depth between the ties for both the broadly

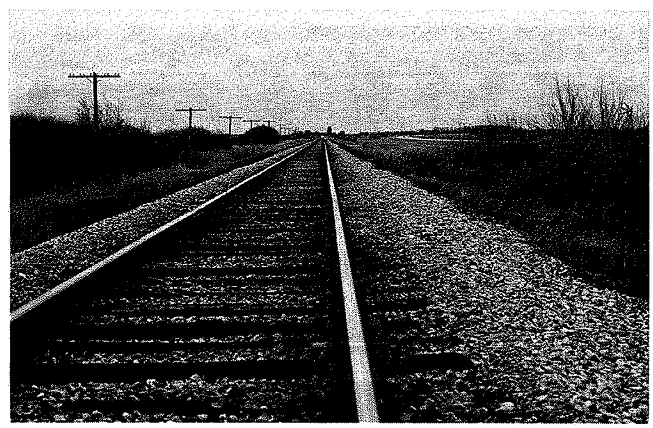

**FIGURE 12 Good performance of ballast.**

**FIGURE 13 Poor performance of ballast.**

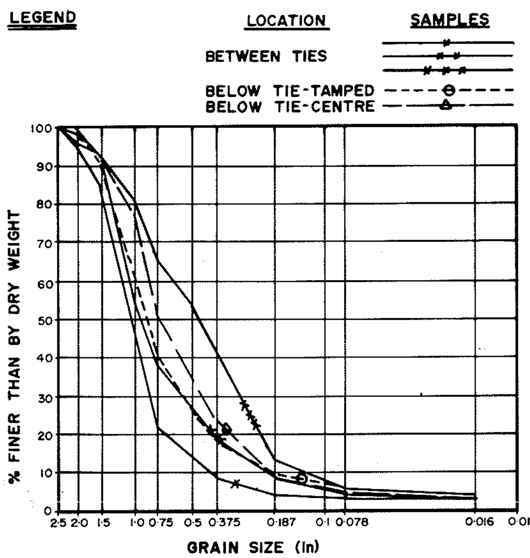


**FIGURE 14 Sample gradations: Mile 65.0, Mountain Subdivision.**

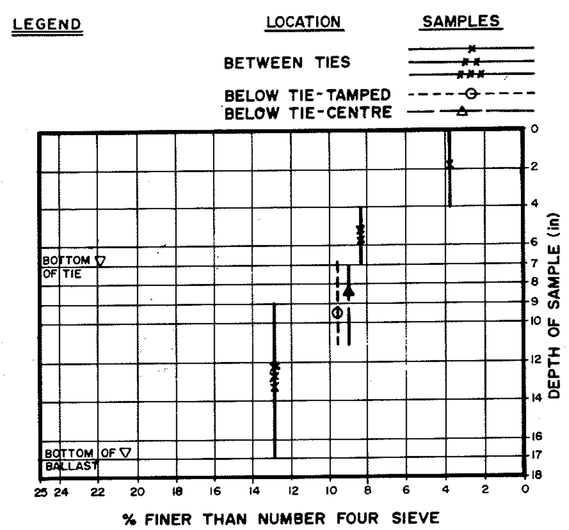


**FIGURE 15 Percentage of fines in samples from Mile 65.0, Mountain Subdivision.**

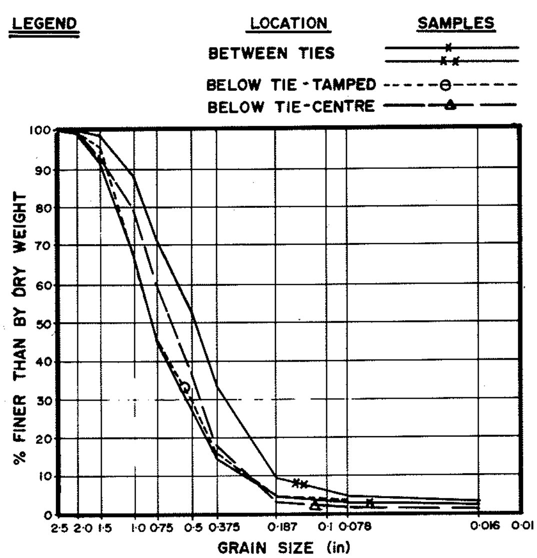


**FIGURE 16 Sample gradations: Mile 37.2, Shuswap Subdivision.**

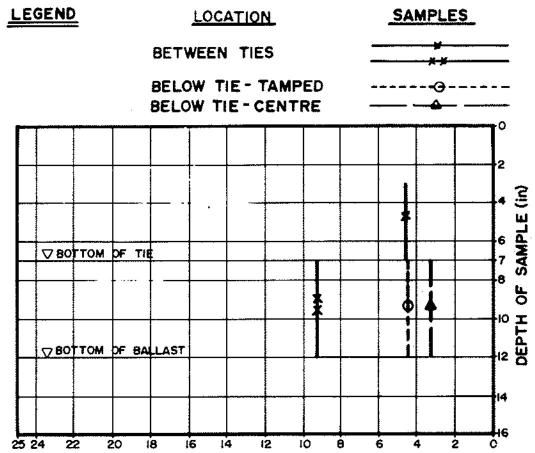


**FIGURE 17 Percentage of fines in samples from Mile 37.2, Shuswap Subdivision.**

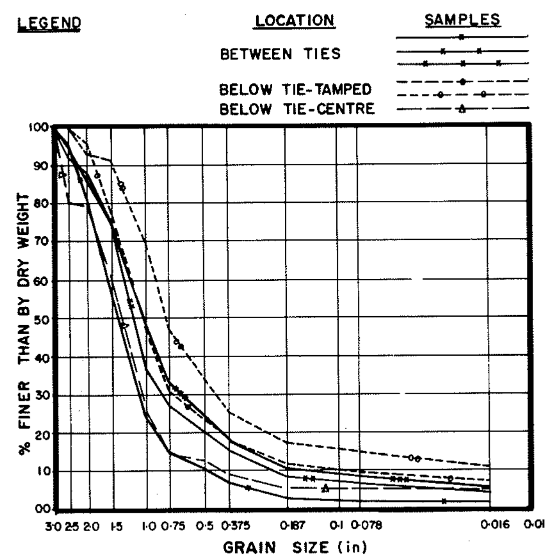


**FIGURE 18 Sample gradations: Mile 52.2, Mountain Subdivision.**

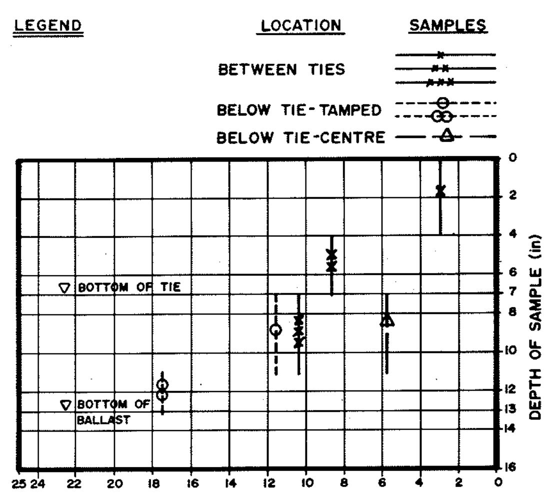


**FIGURE 19 Percentage of fines in samples from Mile 52.2, Mountain Subdivision.**

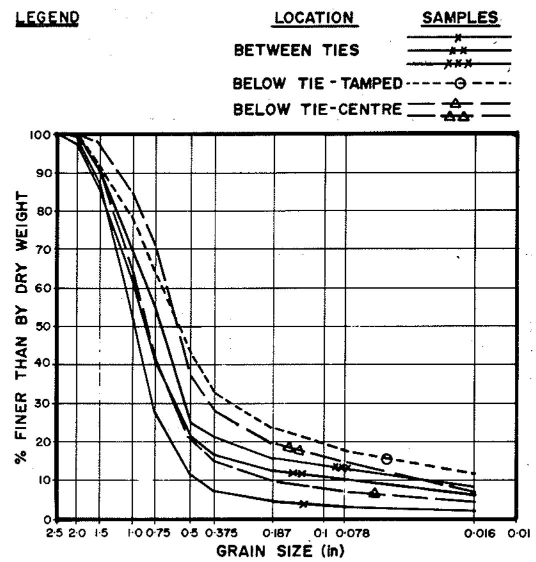


**FIGURE 20 Sample gradations: Mile 13.2, Mactier Subdivision.**

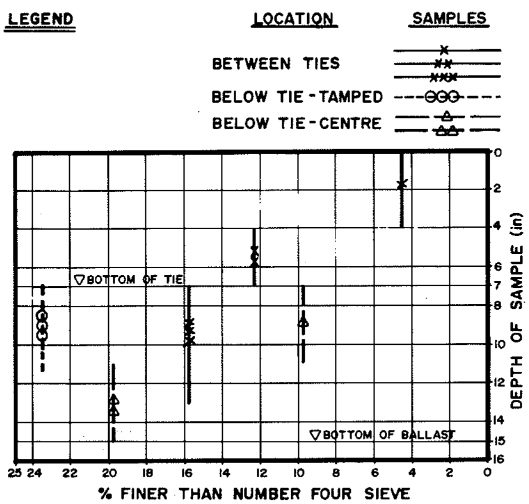


**FIGURE 21 Percentage of fines in samples from Mile 13.2, Mactier Subdivision.**

graded and the uniformly graded ballasts. The fines content below the center of the tie and in the tamped area varied widely, consistent with the scatter inherent in sampling a coarse granular material. In general, a higher fines content represented a higher degree of fouling.

The primary difference between the broadly graded ballast and the uniformly graded ballast is the percentage of fines that can be in the voids without impeding drainage. From 10 to 15 percent fines were required to impede drainage in the broadly graded ballast whereas 20 to 25 percent fines impede the drainage of a uniformly graded ballast.

The remainder of the sites evaluated during the ballast evaluation program were correlated with the performance of the uniformly and broadly graded ballast.

## Laboratory Studies

A laboratory investigation was undertaken (see paper by Clifton et al. in this Record) to develop an understanding of the interrelationship among void ratios, gradation, and index characteristics of various ballast materials. Various uniform and broadly graded specifications were used in the study. The theoretical comparison of voids in each gradation was developed using the theoretical zero voids relationship.

Two quarried rock materials and two crushed gravels were compacted in molds, grouted with epoxy resin, and cut into thick sections. These thick sections allowed a visual evaluation of the interlocking of particles and void characteristics. Figures 22-26 show the thick sections of Kimberley Float ballast.

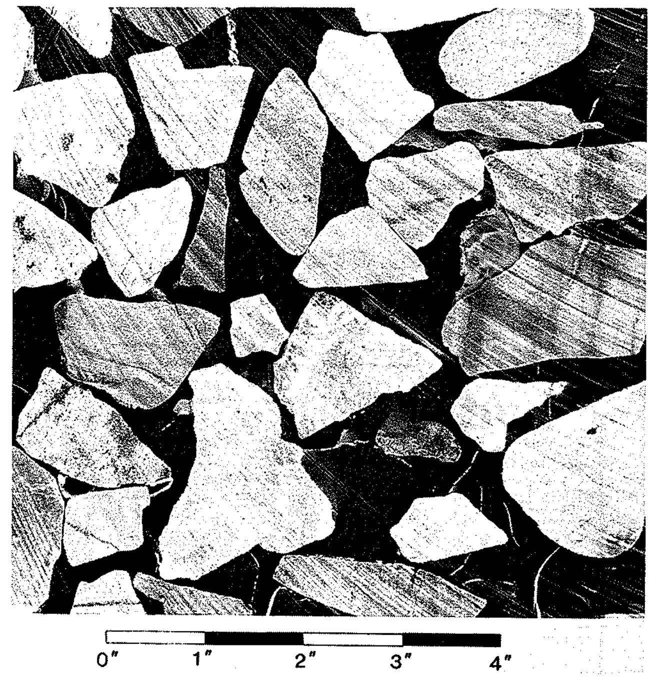


**FIGURE 22 Ballast gradation, AREA 3.**

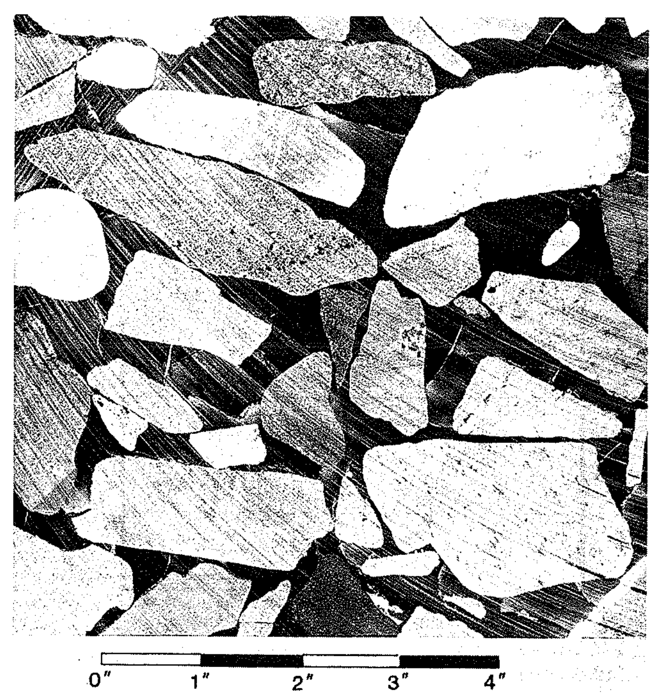


**FIGURE 23 Ballast gradation, AREA 24.**

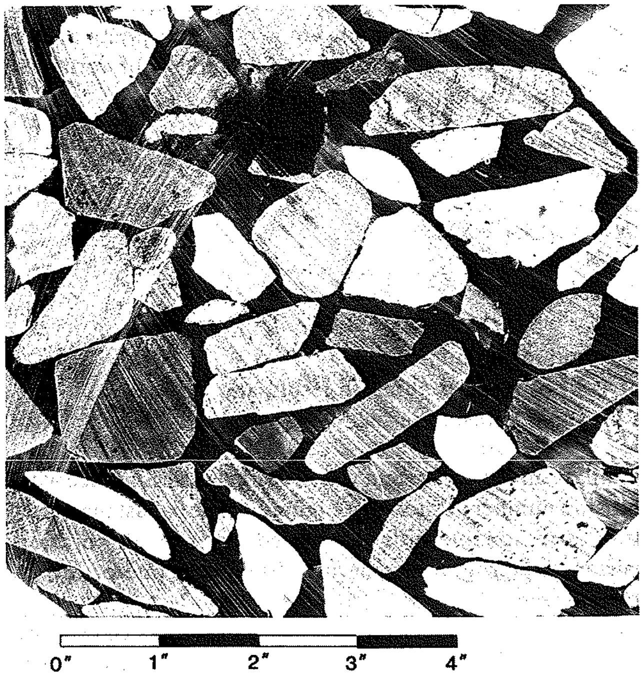


**FIGURE 24 Ballast gradation, AREA 4.**

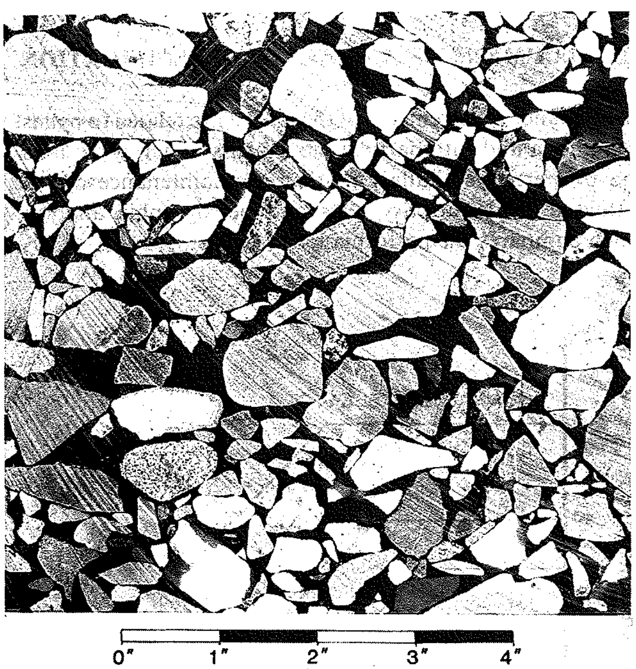


**FIGURE 25 Ballast gradation, CP 2.**

The study indicated that

1. Uniformly graded ballast has larger average void sizes that allow movement of fines;
2. Broadly graded ballast has a lower volume of voids and lower average void size; voids are not continuous and would obstruct movement of fines; and
3. Broadly graded ballast segregates and does not have uniform void size and spacing.

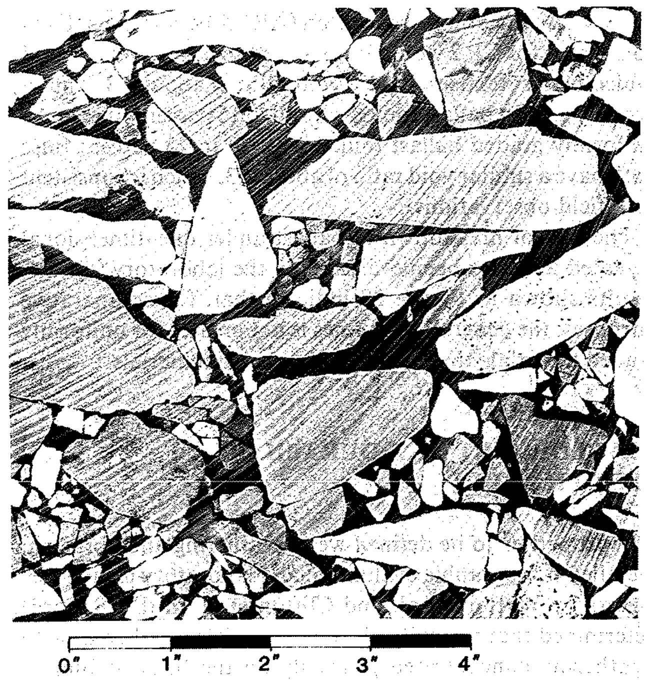


**FIGURE 26 Ballast gradation, Raymond 1.**

Measured void ratios are plotted against the area between the ballast gradation and the zero voids curve in Figure 27. The void ratio curve for each ballast is a function of material type and gradation.

The difference between materials is illustrated by the density attainable in a mold with the same compactive effort. The laboratory tests indicate a difference in interlocking characteristics of the material. This can be related to maintenance and stability of ballast.

Actual density in the field, where track tampers and possibly ballast compactors are used and train loading is experienced, is not the same as that obtainable in the laboratory. The average slope in Figure 27 illustrates the relationship between void ratios and gradation for a given compactive effort.

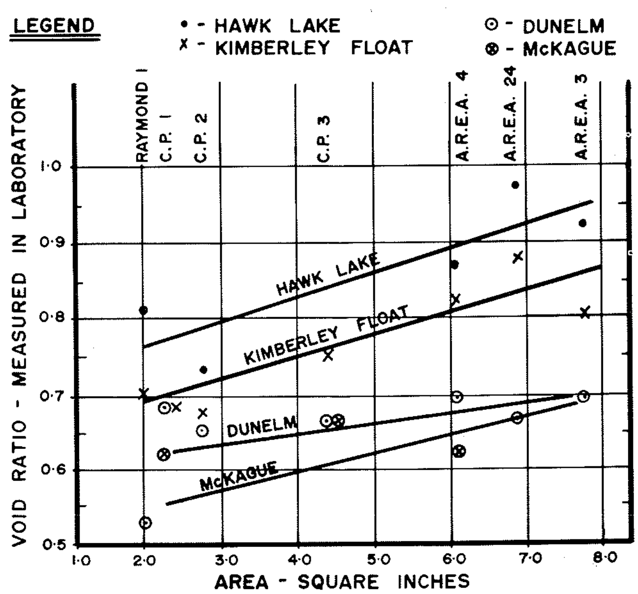


**FIGURE 27 Laboratory void ratio.**

Field void ratios of 0.5 are common (*7*) for American Railway Engineering Association (AREA) gradations. Field void ratios for CP 2 would be expected to be around 0.4, and fouled ballast tends to have a void ratio of about 0.3. Thus a broadly graded ballast fouled with 10 to 15 percent fines and a uniformly graded ballast fouled with 20 to 25 percent fines both have a similar void ratio of about 0.3, which is consistent with field observations.

The rate of breakdown of ballast under one-dimensional repeated load has been evaluated in the laboratory (*8*). The results shown in Figure 28 indicate that, for comparative purposes, the generation of fines is not greater for uniformly graded materials than for broadly graded materials.

## DEVELOPMENT OF A DURABILITY SPECIFICATION

The properties to be defined were weathering and abrasion. Detailed petrographic evaluation and laboratory testing (see papers by Watters et al. and Clifton et al. in this Record) determined that virtually all rock materials weathered, with weathering concentrated primarily on the fines. A limited number of rock types are extremely sensitive to weathering and are not considered suitable for ballast.

The volume of fines generated by abrasion of ballast is a function of the total tonnage of rail traffic. Each of the ballast sections evaluated was extrapolated to a total tonnage in MGT of rail traffic for either a uniform or a broadly graded ballast with 7 in. of ballast below the tie. The extrapolation was based on comparison of the volume of fines present in the ballast with the total storage volume available.

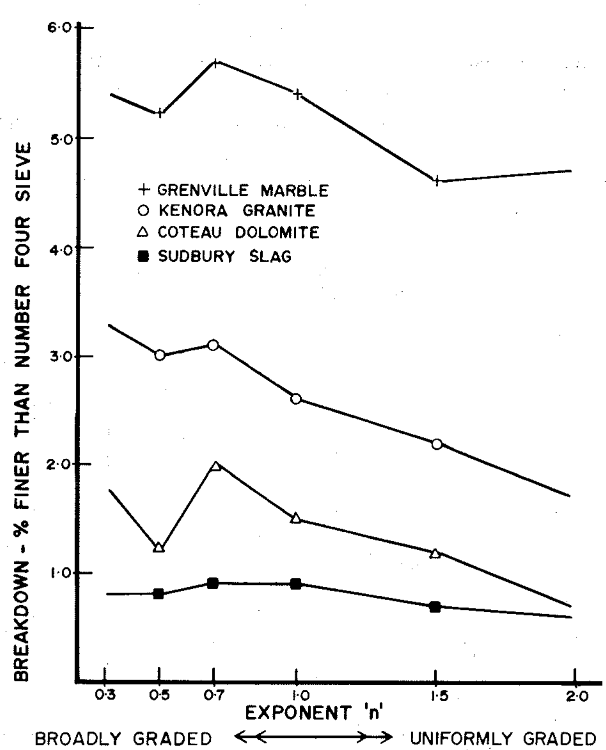


**FIGURE 28 Breakdown variation with gradation (Raymond).**

A complex relationship proposed by Raymond et al. (*3*) is shown in Figure 29 to correlate track classification with ballast rankings. The relationship is not appropriate and is not usable in practice because of the number of variables required to classify track ballast.

CP Rail developed the concept of the abrasion number as follows:

$$AN = LAA + 5MA$$

where

$AN$ = abrasion number,
$LAA$ = Los Angeles abrasion result, and
$MA$ = mill abrasion result.

Ballast on the CP system was classified in accordance with abrasion number as shown in Figure 30. The ballast materials attain a reasonable order that is consistent with abrasion resistance, petrographic characteristics, and rate of breakdown in track.

The abrasion number was plotted against total tonnage for the estimated life of the ballast sections evaluated. The best-fit relationship, plotted to a natural log scale, is shown in Figure 31 for the two ballast gradations considered.

## DEVELOPMENT OF A FRACTURE SPECIFICATION

Maintenance requirements of the ballast are related to ballast breakdown and interparticle movements under cyclic load. The properties of ballast that affect maintenance can be evaluated by laboratory tests and petrographic analysis, which compares the performance of ballast.

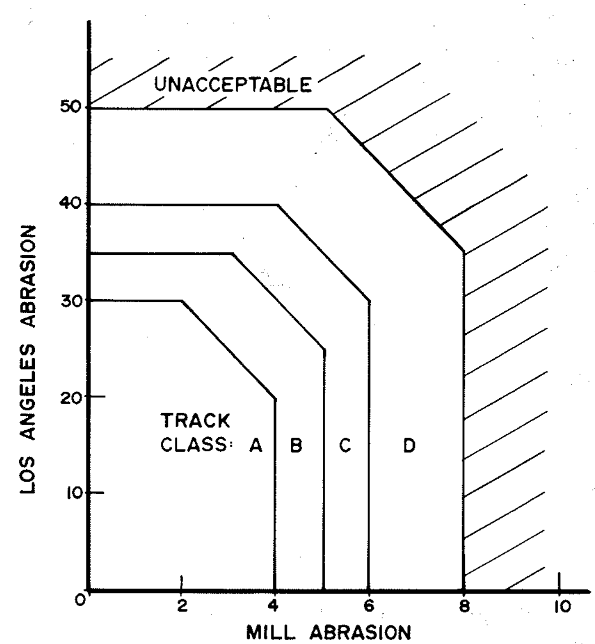


**FIGURE 29 Track ballast rankings.**

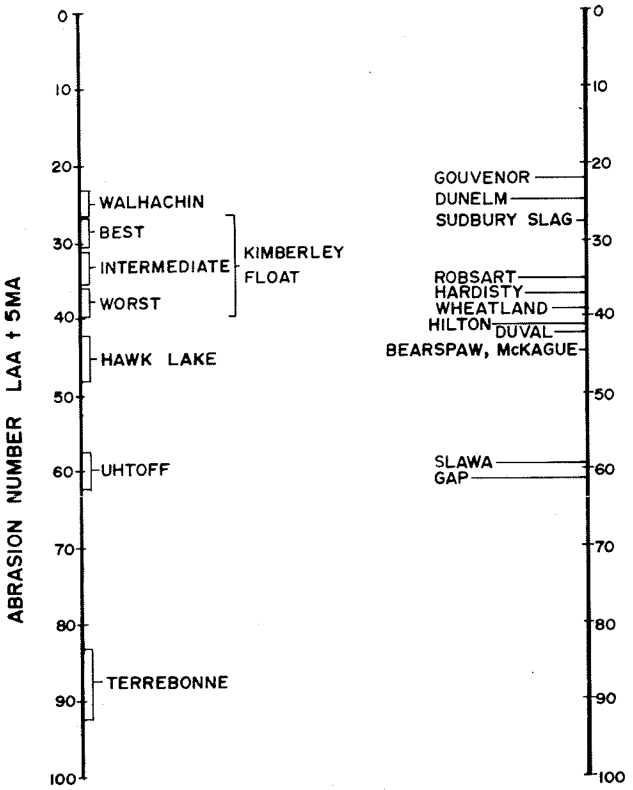


**FIGURE 30 Abrasion number for various CP Rail ballast sources.**

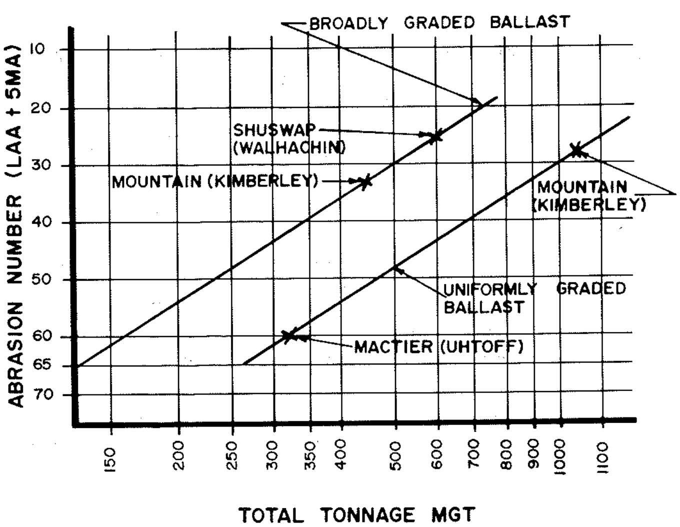


**FIGURE 31 Ballast life.**

Ballast breakdown and related change in gradation occur over the life of the ballast. Ballasts designed for 20 to 30 years have a similar maintenance cycle because of ballast breakdown. The interlocking characteristics of the ballast material are therefore a primary consideration when comparing maintenance of ballast material. One parameter used to evaluate the ballast surface characteristics is the fractured face test (see paper by Clifton et al. in this Record).

CP ballast tracks were classified as either acceptable or not, in accordance with historic maintenance requirements. These ballasts were subjected to laboratory testing procedures to determine the number of fractured faces (see paper by Clifton et al. in this Record). Figures 32-34 show that ballast performance could not be differentiated unless three or more fractured faces were considered.

## CP RAIL SPECIFICATIONS

The three criteria developed to evaluate ballast materials are weathering, abrasion, and stability. These factors are assessed by petrographic examination and laboratory tests, as documented in the CP Rail Specification for Ballast (Appendix, pp. 59-63 in this Record).

The stability of ballast is ascertained by comparison with existing ballasts. Ballast materials that may weather significantly are rejected.

Laboratory tests provide empirical numbers that have been related to ballast performance in track. Petrography provides additional information and a rational basis for interpreting laboratory test results. It allows for a more complete assessment and understanding of the performance of ballast.

## BALLAST MANAGEMENT INFORMATION SYSTEMS

The information presented herein was used to prepare CP Rail Specifications for Ballast (Appendix, pp. 59-63 in this Record). Procedures are now in place to document subgrade, subballast, and ballast conditions; estimate the life of the ballast in the track; and correlate track conditions with maintenance costs and track recording car results.

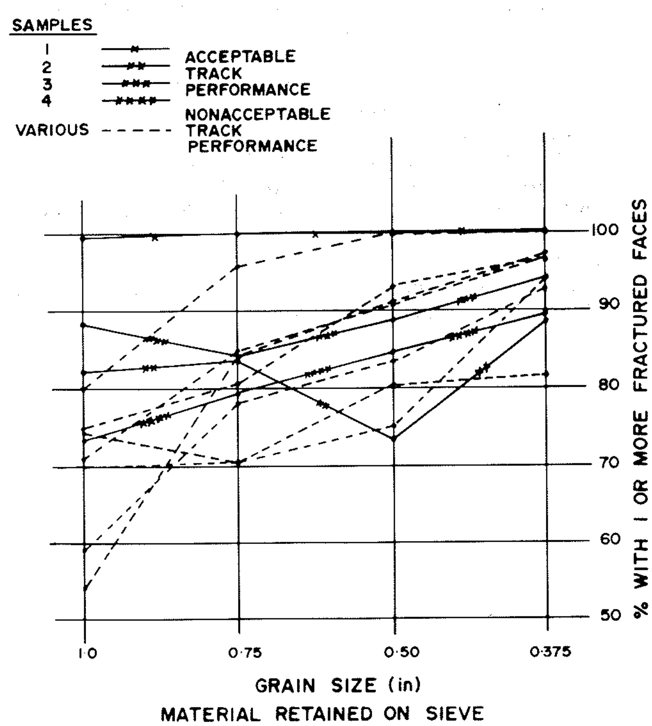


**FIGURE 32 One or more fractured faces.**

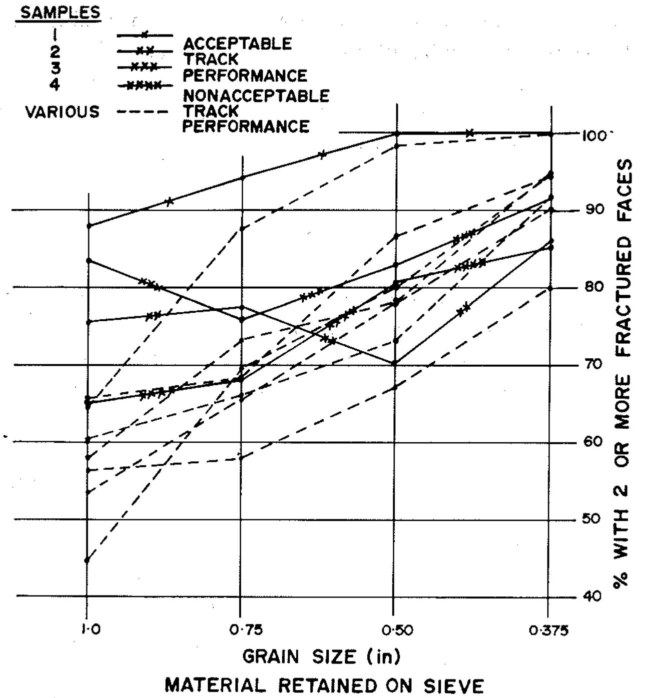


**FIGURE 33** Two or more fractured faces.

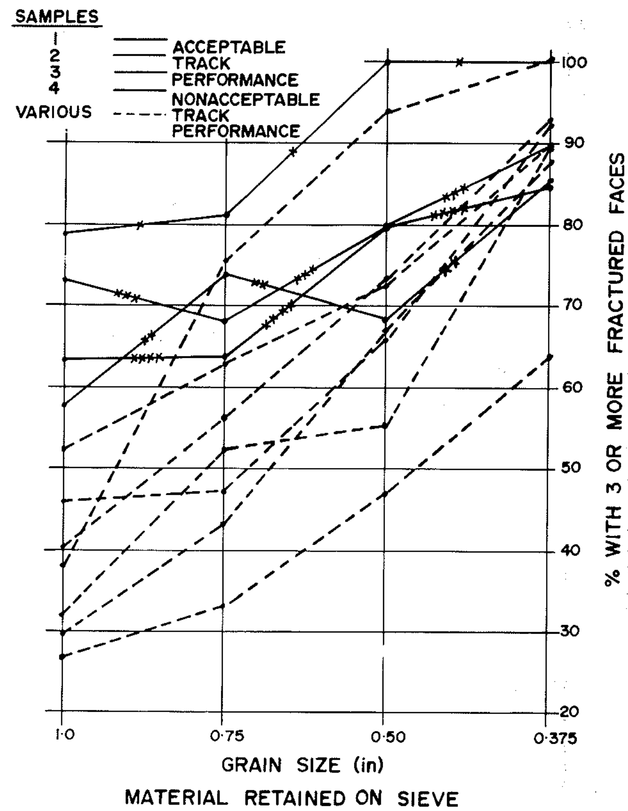


**FIGURE 34** Three or more fractured faces.

The economics of alternative ballast sources and gradations can be considered, which will allow system planning of work programs for as long as a 20- to 30-year period.

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# Evaluation of Ballast Materials Using Petrographic Criteria

B. R. WATTERS, M. J. KLASSEN, AND A. W. CLIFTON

**The performance of rock ballast, when subjected to the physical stresses of loading and to the chemical and physical stresses of a weathering environment, depends to a great extent on the mineralogical, chemical, textural, and structural properties of the material. Because these properties can readily be identified by petrographic analysis, useful predictions of ballast performance can be made by the experienced petrographer. However, because of the difficulty of assessing these properties quantitatively and the large number of variable factors involved, petrographic evaluation of ballast remains, at this stage, largely a subjective process. Mineralogy is a major factor in determining overall rock hardness and physical durability, chemical weathering potential, composition and quantity of derived fines, and degree of susceptibility to wetting and drying. Rock texture also affects hardness and is important in influencing toughness, relative susceptibility to freeze-thaw degradation, and mechanical stability in track. Most of the standard tests commonly applied to ballast materials essentially provide a measure of a combination of petrographic properties; consequently, the results of the tests can be predicted to within certain limits. Numerous techniques, including the use of microscopes and X-ray diffraction equipment, can be applied in the study of the fines fraction of track samples to determine the nature and source of the material. Three ballast types in use by Canadian Pacific (CP) Rail (Kimberley Float, Walhachin, and Prairie gravels) are discussed to demonstrate the influence of petrographic properties on performance.**

In general, petrography can be regarded as that branch of geology that deals with the systematic description of rock materials in hand specimens and in thin sections (a rock slice 0.3 mm thick for microscopic examination). In some instances, a petrographic study may have a broader scope and include more sophisticated techniques of examination such as X-ray diffraction (XRD) and chemical analysis. Petrographic examination is regarded by the authors as one of the most important aspects of the evaluation of ballast materials, both potential ballast and that which has seen service in the track.

The performance of rock ballast, subjected to the physical stresses of loading and the chemical and physical effects of weathering, depends to a great extent on its mineralogical, chemical, textural, and structural properties. Because these properties can readily be determined by petrographic analysis, the experienced petrographer should be capable of providing at least a qualitative assessment of the performance potential of ballast and a reasonable explanation of the failure or durability of ballast from track sections. Other properties that are significant in determining performance are particle size distribution, particle shape, and bulk specific gravity; these can be quantitatively determined and incorporated in a complete performance evaluation (see papers by Clifton et al. and Klassen et al. in this Record).

Many of the physical and chemical tests commonly applied to ballast materials (e.g., mill abrasion, Los Angeles abrasion, magnesium soundness, and absorption) essentially provide a quantitative measure of petrographic properties. Consequently, it is generally possible to predict, to within certain limits, the results of these tests. It follows, therefore, that a complete petrographic analysis should always be the first step in the evaluation of ballast materials and should constitute the basis on which further testing is carried out. Furthermore, petrography should be regarded as a key factor in the selection of a ballast source (see paper by Clifton et al. in this Record).

The relatively low cost of a petrographic analysis can generally be well justified in terms of potential savings in other tests, quite apart from the enormous cost of replacement of unsatisfactory ballast.

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## PETROGRAPHIC EXAMINATION PROCEDURES

During the initial stages of an extensive ballast evaluation program carried out by CP Rail (see paper by Klassen et al. in this Record) the authors developed a standard procedure for petrographic examination of samples and set up a scheme for the description and reporting of results of such analyses.

This scheme is presented in outline form in the following list (see also Appendix, pp. 59-63 of this Record), and some aspects are discussed briefly.

*a.* Rock types: Identification, estimation of relative proportions in various size fractions.
*b.* Mineralogy: Should include estimation of relative proportions of all rock types. Surface encrustations should also be noted.
*c.* Texture: Nature of the constituent particles or crystals of each rock type, their mutual relationships, shape and orientation, nature of matrix and consolidation, porosity.
*d.* Structure: Fractures, joints, foliation, mineralogical banding, bedding, or lamination.

*e.* Mechanical properties
   (1) Hardness,
   (2) Toughness,
   (3) Shape and surface characteristics of particles,
   (4) Freeze-thaw and wetting and drying, and
   (5) Properties of fines and permeability.

*f.* Chemical properties: Existing state of weathering, potential for chemical degradation, weathering potential of derived fines.

*g.* Estimated results of tests: Los Angeles abrasion, mill abrasion, Mg soundness, absorption, bulk specific gravity.

*h.* Specific tests recommended.

*i.* Summary, including recommendations for use as ballast.

The scheme is based on normal petrographic analytical procedures commonly used in the geological sciences (*1-3*) that have been adapted to the requirements of ballast evaluation. For example, one essential aspect of a petrographic report is not only a statement regarding the present condition of the material but also an estimation of the manner in which it will withstand the rigors of use in track.

The equipment necessary to carry out a petrographic analysis can range from very basic to highly sophisticated depending on the specific requirements of the investigation and the nature of the rock material being dealt with. The majority of analyses can be conducted by visual examination of materials in hand specimen and with the aid of a stereoscopic microscope (up to 40× magnification).

Ideally, every petrographic analysis should include examination, under the petrographic microscope (polarizing), of thin sections of all rock types making up the sample. However, time and cost factors do not always permit this, so this type of examination is often reserved for special cases only. Thin-section examination techniques have been described by many authors (*1, 3-6*) and will not be elaborated on here. If thin sections are not available, rock crushes (-80 to +100 mesh size) can be easily prepared and, immersed in a suitable liquid medium (*7*), examined under a petrographic microscope. This is a quick and reasonably good method of identifying the major mineral components of igneous and metamorphic rocks, but it does not provide information regarding textural properties and minor minerals and is also of limited use in dealing with fine-grained rocks.

Examination of the finer fractions (minus No. 4) of a ballast sample can also be conducted to a great extent using a microscope. However, it will not always be possible to positively identify much of the finest material in these fractions by this method, and more sophisticated techniques, such as XRD analysis (*8, 9*) may have to be employed. Only rarely is chemical analysis used in routine ballast investigations, although such methods can be useful when dealing with extremely fine-grained or amorphous materials and may also have considerable potential for solving problems related to the nature and source of fines.

Although a petrographic examination can be carried out by any competent petrographer, the value of the analysis will be vastly enhanced if the petrographer has experience in relating petrographic properties to standard mechanical and chemical tests and to actual performance of ballast rock materials in track.

The identification of rock types in ballast and the determination of the relative proportions present in each size fraction form the basis of any performance evaluation. There are three major rock groups, igneous, metamorphic, and sedimentary, and all yield types suitable for ballast. Classification of rock types can be done according to standard schemes (*2, 10, 11*) which use mainly mineralogical and textural criteria. Mineralogy involves identification of mineral constituents and an estimation of their relative proportions. Texture refers to the size, shape, and mutual relations of the component crystals or particles of a rock (*1, 6*).

Determination of mineralogy and gross textural features can be achieved, in most cases, by examination of hand specimens under a stereoscopic microscope, although it is generally recommended that, whenever possible, this be supplemented by thin-section analysis. The latter will permit more accurate determination of mineralogy, texture, and, in the case of certain igneous rocks, the identification of deuteric (late-magmatic) alteration effects (*2, 11*).

In a petrographic context, structure refers to the mutual relationships among various parts of a rock, which often represent distinct textural units. At the scale of ballast particle size (usually <2.5 in.), many structures of the source rock mass will not be present and are, therefore, of no great importance with regard to ballast evaluation. However, small-scale structures (e.g., finely spaced joints and fractures) are important and there is considerable overlap in what can be regarded as a "textural property" and a "small-scale structure."

The hardness of individual minerals is independent of properties such as texture and cleavage fracture and can be quantitatively determined according to Mohs' scale of mineral hardness (*2*). However, when applied to rocks, the concept of hardness takes on a somewhat different meaning because rocks are usually assemblages of a variety of different minerals. In a strict sense, Mohs' hardness is not directly applicable to rocks, particularly the coarser-grained types. For this reason, the authors prefer the term "effective hardness," which is essentially an estimate, in terms of Mohs' scale, of the overall hardness (or a measure of resistance to abrasion) of the rock material. Such an estimate is based on the hardness characteristics of the constituent minerals, their relative proportions, potential for cleavage fracture, and textural properties. Because of the many variables that influence the effective hardness of a multimineralic material, it is not possible for the petrographer to accurately quantify hardness, especially for the coarse-grained materials. In terms of resistance to abrasion, fine-grained rocks tend to behave more like mineralogically homogeneous materials and their effective hardness characteristics can, consequently, be more accurately assessed. Mill abrasion (MA) tests essentially measure the effective hardness of the ballast.

Estimation of mineral and rock hardness relative to Mohs' scale (*6, 12*) can be achieved using a complete set of Mohs' hardness specimens. However, it is generally more practical (and adequate) to use simple implements such as a pocket knife and copper coin (of relative hardnesses 5 to 5.5 and 3.5, respectively) to bracket the hardness of the materials under examination.

If the ballast is composed of a single rock type, these methods can provide a reasonably accurate hardness value. However, if the ballast is a mixture of materials of differing hardnesses, the relative proportions of the various rock types must be taken into account.

Toughness, a measure of resistance to impact-type stresses, can also be estimated on the basis of mineralogical, textural, and structural properties but cannot be related to a scale in the way hardness can. The Los Angeles abrasion (LAA) test essentially provides a measure of the toughness characteristics of an aggregate.

It is generally not cost-effective or otherwise appropriate for the petrographer to conduct formal tests to determine particle shape. However, the petrographer can determine, qualitatively, any unsuitable shape bias and provide reasonable explanation for such bias. In the assessment of ballast that has seen service in track, an appraisal can be made of current particle shape and the extent of modification by abrasion, impact, and weathering since installation.

The potential for degradation of ballast as a result of freeze-thaw and wetting and drying can be fairly reliably assessed. By considering the mineralogy, hardness and toughness characteristics, and relative proportions of different rock types making up a ballast sample, it is possible to estimate the mineralogical makeup of the derived fine fraction. As will be discussed in a later section, useful predictions of the rate of fouling and permeability loss are extremely difficult to make.

In the evaluation of new or potential ballast, two aspects of chemical weathering have to be taken into account: first, the extent of original weathering and, second, the potential for weathering after the material is placed in the track. The extent to which the rock may have been weathered in the source is easy to establish and constitutes part of a normal petrographic analysis. Rock chosen for ballast is generally free of any extensive chemical weathering because this is an important criterion in the selection of the source. Determining the extent of chemical weathering of ballast that has been in track is also simply a matter of routine petrographic examination. However, the degree to which the derived fines may have been weathered and altered (to clays, for example) is rather more difficult to establish.

Test results can usually be semiquantitatively predicted on the basis of petrographic criteria. However, this should not be regarded as a substitute for all quantitative testing. The petrographer can make recommendations for specific tests if there is any doubt about the suitability of material and, if the petrographic analysis indicates the material to be of unacceptable quality for ballast, further costly testing can be avoided.

## INFLUENCE OF PETROGRAPHIC PROPERTIES AND ASSESSMENT OF BALLAST PERFORMANCE

To assess the performance potential of ballast materials, a large number of factors have to be taken into account. Other than petrographic characteristics, factors such as track loading, environmental conditions, and particle shape and size distribution may all be of importance. There is no simple formula that can be applied in the performance evaluation of ballast using petrographic criteria. Consequently, any such assessment is a subjective process: the relative importance assigned to any single factor, or combination of factors, is dependent on the experience of the petrographer.

In spite of these limitations, petrographic analysis is of great value because it can be used to provide a qualitative assessment of performance in the track, predict qualitatively the results of tests, provide rational explanations of test results, identify potential problems in materials intended for use as ballast, provide reasons for failure (so that the same mistakes are not repeated), and monitor material from sources and from track sections. Specifications for the selection of ballast materials (see Appendix, pp. 59-63 in this Record) are based largely on physical test criteria. Because the results of such tests will be determined to a great extent by petrographic properties, qualitative estimation of test results using petrographic data will permit evaluation of potential ballast materials. Therefore the following discussion will be concerned primarily with the influence of petrographic character on certain properties critical to the selection of ballast (e.g., hardness, toughness, and chemical weathering potential).

### Hardness

Hardness is judged to be the dominant characteristic determining the physical durability of ballast. As discussed earlier, the effective hardness of a rock will depend not only on the hardness of the component minerals but also on mineral cleavage fracture, textural attributes, and, in the case of clastic sedimentary rocks, the degree of cementation or consolidation.

Crystalline rock types composed of soft minerals degrade more rapidly than do those made up dominantly of hard minerals. Furthermore, as is pointed out later, hardness also plays a part in determining the toughness of rock materials, another important criterion commonly used in the selection of ballast (see Appendix, pp. 59-63 in this Record). The hardnesses of common rock-forming minerals are given in Table 1, and it is apparent that most of the anhydrous silicate minerals (e.g., quartz, feldspar) are harder than and will constitute far superior ballast rock components than the hydrous phyllosilicates (clays, chlorite, mica) and carbonates.

The property of cleavage fracture in constituent minerals will lower the effective hardness of a rock. When subjected to abrasion (and impact) stresses, the crystals will tend to fracture along cleavage planes and the influence of this effect on hardness will be far greater for coarse crystalline rocks than for the finer-grained varieties. Friable texture, occasionally present in crystalline igneous and metamorphic rocks, will also have a significant negative influence on effective hardness.

TABLE 1 SOME PROPERTIES OF COMMON MINERALS RELEVANT TO BALLAST ROCK EVALUATION

| Mineral | Mohs' Hardness | Potential for Cleavage Fracture | Chemical Weathering Potential of Ballast Particle | Chemical Weathering Potential of Fines | Specific Gravity | Suitability as a Ballast Rock Component (durability) |
|---|---|---|---|---|---|---|
| Quartz | 7 | Nil | Nil | Nil | 2.65 | High |
| Plagioclase feldspar | 6-6.5 | High | Low | High | 2.60-2.74 | Moderate |
| Potassium feldspar | 6 | High | Low | High | 2.57 | Moderate |
| Hornblende | 5-6 | Moderate | Low | Low | 3.00-3.47 | Moderate |
| Actinolite | 5-6 | Moderate to high | High | Very high | 2.90-3.20 | Low |
| Mica | 2-3 | High | Moderate | Moderate | 2.76-3.10 | Low (soft) |
| Pyroxene | 5-6 | Moderate | Moderate | High | 3.20-3.50 | Low to moderate |
| Chlorite | 1.5-2.5 | High | Low | Low | 2.65-2.94 | Very low (soft) |
| Clay minerals | 1-2.5 | High | Low | Low | 2.60-2.90 | Highly unsuitable |
| Epidote | 6-7 | Low | Very low | Very low | 3.25-3.50 | High |
| Tourmaline | 7-7.5 | Very low | Nil | Nil | 2.98-3.20 | High |
| Garnet | 7-7.5 | Nil | Nil | Nil | 3.50-4.20 | High |
| Olivine | 6-7 | Nil | Very high | Very high | 3.20-4.30 | Low |
| Calcite | 3 | Very high | High | High | 2.71 | Low |
| Dolomite | 3.5-4 | Very high | Moderate | High | 2.80-2.90 | Low |
| Magnetite | 5.5-6.5 | Very low | Very low | Very low | 5.18 | High |
| Pyrite | 6-6.5 | Nil | Moderately high | Very high | 4.80-5.10 | Unsuitable |
| Pyrrhotite | 3.5-4.5 | Low | Very high | Very high | 4.40-4.65 | Highly unsuitable |

The effective hardness of clastic sedimentary rocks (and most volcaniclastic rocks) will be determined primarily by the nature of consolidation, which can be regarded as involving one or more of the following: compaction, cementation, and recrystallization. Compaction will not, on its own, produce a rock sufficiently hard and durable for use as ballast. Because this is the dominant factor involved in the consolidation of fine-grained rocks such as mudstones and shales, these types are unacceptable as ballast. Common cements in sedimentary rocks are silica, carbonates, and iron oxides and hydroxides. Grains well cemented by secondary silica minerals will be strongly consolidated, especially if the grains are quartz and the cement occurs as grain overgrowths. Secondary iron minerals constitute a weak to moderately strong cement depending on abundance. Carbonate cements (mainly calcite) are weak; they are limited in strength by the softness and prominent cleavage that characterize these minerals. Interstitial clay minerals can also constitute a weak cementing agent.

Recrystallization may occur in sedimentary rocks subjected to somewhat elevated temperatures that border on metamorphic conditions. Sedimentary rocks rich in quartz grains (e.g., quartz sandstone) commonly recrystallize to form a very well-indurated rock called orthoquartzite, which has a high hardness ($H = 7$). Carbonate rocks (limestone and dolostone) also commonly recrystallize, but effective hardness will still be low (3 to 3.5) because of the low hardness and prominent cleavage of the constituent minerals, calcite and dolomite.

The effective hardness of a ballast rock type can be estimated using the simple techniques already discussed. A weighted average hardness value (in terms of Mohs' hardness) for the ballast sample can be calculated taking the relative proportions of the constituent rock types into account. Such a value can only be expected to represent an approximation of the hardness characteristics of the ballast as reflected by abrasion in track and by mill abrasion testing because it does not take into account the effects of particle shape, particle surface characteristics, and relative hardness of constituent rock types. Consequently, the petrographer can usually only report ballast hardness in general terms such as "low" ($H < 4$?), "medium" ($H = 4$ to 5.5?), and "high" ($H > 5.5$?), the subdivisions being somewhat arbitrary.

Relating a petrographically estimated hardness value to an equivalent in terms of mill abrasion loss can also only be achieved on a semiquantitative basis, the more so because the steps on Mohs' scale are not equal. Although there is considerable overlap in the ranges, "low," "medium," and "high" hardness ballasts will display high (> 6 percent?), moderate (about 4 to 6 percent?), and low (< 4 percent?) mill abrasion losses, respectively.

As discussed in the section on Case Histories, a more reliable correlation between petrographically estimated hardness and mill abrasion can be achieved if a considerable amount of data is available for a particular ballast type, especially if the material has variable hardness. This allows a plot of petrographic hardness versus abrasion value (MA, or abrasion number, as discussed below) to be drawn.

It has been shown (see paper by Klassen et al. in this Record) that the overall physical durability of ballast can be estimated by determining its abrasion number (AN), which is calculated according to the formula:

$$AN = \text{Los Angeles abrasion} + 5 \times \text{Mill abrasion}$$

Because the mill abrasion test is essentially a measure of the hardness of the material, and toughness will also depend on this property to some extent, this equation is further indication of the importance of hardness in determining the durability of ballast. Abrasion number constitutes an important criterion used by CP Rail in the classification of ballast (see Appendix, pp. 59-63 in this Record).

### Toughness

Major factors that influence the toughness of a rock, its ability to withstand impact-type stress, are hardness and textural and structural features that may lead to fracture. Finely spaced joints or fractures (< 2 in.) constitute an undesirable structural element, and losses due to impact will be far greater for a rock composed of very soft minerals (e.g., calcite in limestone) than for one made up of very hard minerals (e.g., quartz in quartzite). If the rock is crystalline and coarse grained, cleavage fracture in some minerals may cause relatively high losses (e.g., feldspars in granite and gneiss, and calcite and dolomite in limestone and dolostone, respectively).

Finely crystalline igneous and metamorphic rocks and well-consolidated fine-grained sedimentary rocks tend to be tough, provided they are free from obvious or incipient fractures and planes of weakness. It is frequently the case that coarsely crystalline igneous and metamorphic rocks tend to have somewhat friable textures. Similarly, coarse-grained clastic sedimentary rocks are generally less well consolidated than their finer counterparts. Furthermore, soft, well-bedded or laminated sedimentary rocks, such as shale, will be very weak.

Strongly foliated metamorphic rocks such as schist, slate, and phyllite, all of which are commonly rich in mica (a very soft mineral), are generally not very tough. Foliated rocks poor in mica, such as some amphibolites, tend not to part as easily along foliation planes because their mineral cleavage planes do not have such a consistent parallel alignment. In coarse-grained, mica-poor gneisses, the major mineral constituents of which are more or less equidimensional, foliation is defined more by a concentration of minerals into bands than by a strong preferred orientation of crystals, and the foliation does not constitute a prominent weakness.

The tenacity of a mineral refers to properties such as brittleness, elasticity, and flexibility. In extreme cases, these may influence the toughness of a rock composed dominantly of one mineral type. For example, rocks composed dominantly of quartz, such as quartzite, tend to be brittle and produce shardlike (flaky) fragments on impact.

Voids, such as intergranular spaces in a sedimentary rock (porosity) or vesicles in a volcanic rock, may, if sufficiently abundant, seriously weaken a rock and reduce toughness as a consequence. Veins cutting through a rock may also represent weak zones, depending on the vein-filling mineral. A weak mineral such as calcite is poor, whereas quartz as a vein-filling material is very strong.

Although he will not be able to provide a quantitative assessment of toughness, the petrographer will usually be able to rate a ballast as having low (= LAA > 40?), medium (= LAA 30-40?), or high (= LAA < 30?) toughness and provide an indication of any potential problems. The apparent lack of a consistent correlation between LAA test data and track performance characteristics discussed by Chrismer (*13*) suggests that this type of information, based on petrography, can be of considerable value.

### Particle Shape and Surface Characteristics

For good mechanical stability in track, ballast particles should, ideally, be angular and equidimensional in shape (*14*) with rough surfaces to provide maximum friction between particles. For example, rock types such as granite and gneiss usually have rough surfaces whereas quartzite, although very hard and durable, has very smooth surfaces that offer limited friction between particles and consequent poor stability. Very hard, fine-grained rock types also tend to acquire a surface "polish" in track, which further reduces interparticle friction.

Well-rounded particles, or particles with a high proportion of rounded surfaces, such as those common in some glacial gravel ballasts, will constitute a very unstable track bed, especially if the particle surfaces are smooth.

Strongly foliated metamorphic rocks will be weak in one plane, and pronounced tabular or elongated particle shapes, or both, will result from the crushing of such material as well as from fracture in the track as a result of impact. Particles that have strongly developed elongated or flattened shapes, or both, will also be mechanically unstable and have the added disadvantage of being subject to load fracture. Therefore rock types that are rich in mica and have a strong foliation, such as many schists, phyllite, and slate, are highly unsuitable as ballast components.

Particle shape will also influence the results of abrasion tests and is an important factor in the evaluation of ballast from track sections or the comparison of test results of such material with those from fresh source samples. Angular particles will abrade more rapidly, initially, than rounded particles in both types of abrasion tests (and also in track) because the relatively "sharp" edges of the former will be highly susceptible to removal. Similarly, ballast particles that have already suffered abrasion in track and have undergone some shape modification (less angular) but have remained essentially unweathered will provide lower abrasion values (and appear of higher quality) than fresh source material of the same type. This effect is likely to be significant in the majority of cases and should always be kept in mind when making comparisons between sets of abrasion data. This principle can be well illustrated by conducting repeated abrasion tests on the same ballast samples, as was done for various rock types from the Walhachin Quarry. The test results indicate a 10 to 30 percent and a 35 to 50 percent reduction in LAA and MA results, respectively, and a more consistent 25 to 35 percent reduction in AN. This tendency is, however, not clear-cut and the effect may be converse. If the particles have developed soft weathered mantles in the track, abrasion losses may increase, compared with those for fresh material, in spite of the particles being less angular.

### Freeze-Thaw and Wetting and Drying

Breakdown by freeze-thaw will depend, to a great extent, on the porosity and permeability of the rock materials and can take the form of intergranular voids and bedding plane partings in clastic sedimentary rocks, cavities in crystalline sedimentary rocks such as limestone and dolostone, vesicles in volcanic rocks, intercrystalline and cleavage partings in coarse-grained crystalline rocks, and joints and fractures. These features can readily be identified by petrographic examination, particularly if thin sections are available. Relating these properties quantitatively to some soundness index is not possible except in quite general terms (such as "low," "moderate," and "high"). Certainly, petrography can identify those materials that will produce low soundness test results (e.g., Mg soundness) and those that are likely to give unacceptably high values.

Abrasion and impact in track will produce mantles and spots of physically broken down material on ballast particles, which will be relatively porous, loosely cohesive, and subject to removal by freeze-thaw (as well as by further abrasion). In this way, fresh rock is continually being exposed. The contribution of this process to the overall degradation of the material is judged to be significant, but further research is needed to establish this point.

The absorption characteristics of a ballast will depend on petrographic properties similar to soundness and can also only be qualitatively estimated.

Wetting and drying of clay-bearing materials will result in alternate expansion and contraction of the clays causing a physical deterioration. Relatively minor quantities of clay (a few percent) in some sedimentary rocks may be deleterious if the clays are of the swelling type. This latter property can be determined by XRD analysis (*9*).

### Chemical Weathering

The potential for weathering of both the ballast particles and the derived fines is determined primarily by mineralogy (Table 1) and environmental factors. Texture and structure may also play a minor role in that friable, porous, or fractured rocks provide more surface area at which weathering reactions can occur. Chemical weathering of minerals comprising the ballast rock types will usually not be the primary cause of failure of the material because, under normal environmental conditions, most common major rock-forming minerals are relatively resistant to this type of breakdown. However, even minor proportions of some minerals, such as sulfides and olivine, may be deleterious. The latter will rapidly break down to serpentine or talc (very soft), and sulfide minerals will oxidize and, in doing so, generate an acidic environment in the ballast that will, in turn, accelerate the chemical weathering of both the ballast rock and the derived fines. In general, rock types that contain excessive amounts of sulfides and olivine (greater than about 2 and 5 percent, respectively) should not be considered for ballast.

Fines derived from the physical breakdown of ballast particles will be far more susceptible to chemical weathering than the larger parent particles (Table 1) because of the much greater surface area per unit volume exposed to atmospheric elements. For example, fines derived from feldspars, common major mineral constituents of many rock types, can be expected to break down to clay minerals far more rapidly than the parent feldspar crystals.

Carbonate rocks will weather primarily by solutioning, a process that frequently produces thin, soft, powdery mantles on the particles. Such mantles are easily removed by abrasion and freeze-thaw and thereby contribute to the fines fraction and expose fresh rock for further weathering. Furthermore, if the rock is impure, dissolution of any part of the carbonate component will release insoluble particles, commonly clay. Consequently, although "clean" carbonate rocks can constitute satisfactory low-grade ballasts, impure varieties are highly unsuitable.

Another form of alteration, distinct from chemical weathering (a low-temperature process), is deuteric alteration, which takes place at high temperatures during the late-magmatic stages of the formation of an igneous rock mass. In deuteric alteration the primary minerals alter to secondary minerals that are generally softer and less desirable as ballast rock constituents. Recognition of these effects, not always obvious in hand specimen, is important especially for predicting the constitution of the fines component.

Material taken from track sections will often be weathered to some degree and particles may have mantles of relatively soft alteration products. Furthermore, after being in the track for a few years, ballast particles may acquire coatings of oil and loosely adhering grime, which add to the soft mantle. During abrasion tests this soft material will be easily removed, resulting in higher abrasion values compared with those of fresh material from the same source. Testing of Kimberley Float ballast taken from the source and of that placed in track at various times provided a graphic example of these effects, as described in the section on Case Histories.

### Bulk Specific Gravity

The bulk specific gravity (BSG) of ballast materials will depend primarily on the mineralogy of the constituent rock types and their relative proportions. Another factor that is of significance is the presence of void spaces (porosity, vesicles, etc.) in any of the rock types. All of these petrographic properties can be quantitatively determined. Therefore, because the specific gravities of all minerals are known (Table 1), a simple calculation can provide a fairly accurate estimation of the BSG of the aggregate.

## IDENTIFICATION OF "FATAL FLAWS"

Although it is possible to identify certain petrographic properties that are likely to render a rock material unacceptable as ballast, it should be realized that the presence of undesirable properties, if "diluted" by acceptable qualities, can be tolerated because they may simply limit the application of the material (instead of making it totally unacceptable). For a particular petrographic property to constitute a "fatal flaw" it is generally necessary for it to be present in relative abundance. The principal petrographic properties that may cause a material to be of fatally poor quality as ballast have been summarized in Table 2. Although not strictly a petrographic property, particle shape has been included in this list because it may be influenced by texture or structure, or both.

## EVALUATION OF FOULING

Fouling of ballast and resultant loss of permeability can be attributed to the introduction of fines from various sources. Research suggests that, in most cases, a majority of fines are derived from the ballast itself as a result of abrasion, impact, and physical and chemical weathering.

Depending on the prevailing environmental conditions, a significant fines component may also be introduced into the ballast as windblown dust, sand, and various organic materials. Spill from railcars can introduce a variety of contaminants; the most commonly encountered in Canadian ballasts are fine-grained coal fragments and dust. Abrasion of ties contributes fragments of wood. The use of sand to increase traction during freezing conditions can introduce a considerable amount of fine-grained material.

Most of these contaminants can be easily recognized by microscopic examination. However, identification and determination of the proportions of nonorganic fines that constitute contaminants (from nonballast sources) are problematic but can sometimes be achieved using XRD analytical techniques. An XRD scan of a fines fraction will permit identification of the major minerals present. Therefore such a scan for a sample of fines can be compared with a scan for fines derived from abrasion testing of the ballast (where no contamination would occur). Any differences in minerals between the two samples may then be attributed to contamination. A similar technique can be applied when investigating possible contamination of the ballast by fines from the subgrade material.

Success in using XRD techniques in this manner depends on there being identifiable differences in mineralogy between the ballast and the contaminant. Unfortunately, this is not always the case. A further complicating factor may be weathering of the fines fraction, which would result in the material giving a scan with characteristics different from those of the parent ballast even if no contamination had occurred.

Fines from carbonate rock ballasts can be treated with acid to determine the noncarbonate component, some of which may represent a contaminant. Because many carbonate rocks contain a proportion of noncarbonate material (commonly clays), a sample of the ballast would also have to be treated to establish the extent of the initial noncarbonate fraction before any meaningful comparisons could be made.

Prediction of the fouling potential and rate of permeability loss of a ballast is extremely difficult because of the many variable factors involved. Consideration must be given to track loading, environmental conditions, size distribution and shape of particles, mineralogy, texture and structure of the component rock types, their hardness and toughness, chemical weathering potential, and the composition of the fines fraction. Unfortunately, there is no simple formula that incorporates all of these factors, and any assessment carried out by the petrographer can only be qualitative.

The influence of track loading and particle size distribution

TABLE 2 SUMMARY OF PETROGRAPHIC PROPERTIES THAT MAY, IF PRESENT IN ABUNDANCE, RESULT IN FATALLY POOR BALLAST

| Properties of Ballast Rock | Principal Deleterious Effect |
|---|---|
| Mineralogical | |
| General high content of very soft minerals (e.g., clays, mica, chlorite) | Rapid physical degradation, clay and fine, mica-rich fines |
| Argillaceous sedimentary rocks (e.g., mudstone, shale) | Rapid physical degradation, clay-rich fines |
| Mica-rich metamorphic rocks (e.g., slate, phyllite, schist) | Rapid physical degradation, clay and fine, mica-rich fines |
| Igneous with deuterically altered feldspar | Rapid physical degradation, clay-rich fines |
| Sulfide-rich (> about 2 to 3 percent) (e.g., pyrite, pyrrhotite) | Oxidation of sulfide results in acidic conditions promoting chemical weathering of other mineral components |
| Textural | |
| Poor consolidation (in sedimentary and volcaniclastic rocks) | Rapid physical degradation by abrasion, susceptibility to freeze-thaw |
| High porosity (> about 5 percent) in sedimentary rocks | Degradation by freeze-thaw and abrasion if pores large and abundant |
| Vesicularity (in volcanic rocks) | Degradation by freeze-thaw and abrasion |
| Friable texture in crystalline rocks | Rapid physical degradation by abrasion, susceptibility to freeze-thaw |
| Structural | |
| Closely spaced joints, bedding partings, foliation | Rapid physical degradation by abrasion and freeze-thaw; generation of unsuitable particle shapes |
| Particle shape and surface characteristics | |
| Smooth particle surfaces (often due to rock texture) | Poor mechanical stability |
| Unsuitable particle shape | Poor mechanical stability; load fracture or elongated or tabular particles. |

on fouling potential is highly significant (see paper by Klassen et al. in this Record). Particle shape and particle surface characteristics also influence the rate of fines production in that, all other factors being equal, well-rounded, smooth particles will produce fewer fines than angular, rough particles. However, particles that have the former characteristics lack the interparticle friction necessary to maintain a stable track bed so certain trade-offs have to be accepted.

Mineralogy is the most important single factor influencing the rate of fouling of ballast. Simply, rock types rich in soft minerals will produce fines at a higher rate than rock composed of harder minerals. The composition of the fines fraction itself is also of importance in assessing fouling and permeability loss. Quartz-rich fines can be expected to have a low capacity for cementing and will retain permeability. Fines rich in clays and carbonates will have a higher capacity for compaction and cementing. Clay-rich fines may be derived directly from clay-rich rock types such as certain fine-grained sedimentary rocks, igneous rocks with deuterically altered feldspars, or, indirectly, the chemical breakdown of the feldspar and mica components of the fines fraction itself. Theoretically, carbonate fines could be removed from the ballast by solutioning, particularly if the water percolating through the track bed is slightly acidic. Similarly, precipitation of carbonates taken into solution from the ballast or the derived fines may be a factor contributing to cementing of the ballast. However, the extent to which these processes may occur, if at all, has not been fully assessed.

Relative hardness of different rock components making up a ballast may also be of importance in determining the compositional character of the fines fraction. In a mixture of approximately equal proportions of relatively hard and soft components, the fines will be derived dominantly from the softer rock types.

Texture and structure of ballast rock types will influence fouling indirectly because these properties play an important role in determining the effective hardness and toughness of a rock.

## MONITORING OF BALLAST QUALITY

To ensure a consistently acceptable quality of material from ballast sources, it is essential that samples be examined on a regular basis by a petrographer. Even if detailed geologic mapping of the source area has been carried out, unexpected variations in the geologic setting may occasionally result in the production of poor-quality material. Typical geologic features that may result in a change in rock quality are fracture or fault zones, alteration zones, and changes in lithology. The latter is perhaps most important when dealing with layered sequences of rocks (most commonly sedimentary and metamorphic), although changes in composition can also occur in intrusive rock masses, especially at or close to the contact zones with country rock. Significant changes in material from a source may necessitate a brief field reexamination to assess the changes in geologic setting so that changes can be made in the quarrying operation.

Monitoring of ballast in use is of importance and can be achieved most cost efficiently by periodic petrographic examination. If a petrographer is familiar with the characteristics of a particular material, changes that result from its use in track can easily be identified. Because the ultimate test of any ballast is in a track, monitoring will provide useful information from which projections of the life of the ballast can be made. The data will also be of great value in assessing the performance potential of other similar materials.

## CASE HISTORIES

During the ballast evaluation program carried out by CP Rail all major ballast types currently used by CP Rail were subjected to extensive petrographic examination as well as a variety of physical and chemical tests (see papers by Clifton et al. and Klassen et al. in this Record). Table 3 gives a summary of the main characteristics of these materials, some of which are discussed in greater detail later with a view to demonstrating relationships among petrographic properties, results of physical tests, and actual performance.

### Kimberley Float

This material, which is waste from the Sullivan Mine in British Columbia, has been used extensively by CP Rail as main-line ballast. The principal rock type is fine-grained argillite (a mildly metamorphosed argillaceous sedimentary rock) with minor amounts of albitite (a rock consisting almost entirely of feldspar). The argillite is composed of various micaceous minerals (including clays), quartz, feldspar, tourmaline, chlorite, calcite, minor iron oxides, and accessory sulfide minerals. The latter are usually present as small disseminated crystals or aggregates making up 0.5 to 1 percent of the rock on average but occasionally as much as 5 percent. Phyrrhotite and pyrite are the dominant sulfide minerals with lesser galena and sphalerite. Calcite occurs as an original mineral constituent in chlorite-rich argillite and also as a secondary mineral filling fractures.

The mineralogy of the argillite gives Kimberley Float ballast a relatively high BSG (2.78), and its dense, fine-grained, recrystallized texture indicates a negligible porosity, which is reflected in low absorption values (0.29).

Relative proportions of the major minerals vary considerably and this is reflected directly in the effective hardness and overall durability of the rock. Argillite rich in chlorite tends to be soft ($H < 3$) because of the very low hardness of that mineral ($H = 1.5$ to $2.5$). Tourmaline-rich (chlorite-poor) argillite is very hard ($H > 5$), a property attributable to the high hardness of tourmaline ($H = 7$ to $7.5$). The difference in hardness is clearly apparent when track samples are examined: the softer, chlorite-rich argillite particles very quickly acquire a well-rounded outline due to abrasion and impact, whereas the tourmaline-rich particles show only a mild chipping and largely retain their original angular shapes. Some of the chloritic varieties are so soft that they become

TABLE 3 SUMMARY OF MAJOR CHARACTERISTICS OF CP RAIL BALLASTS

| Material | Major Component Rock Types | AN | LAA | MA | Mg-S | Abs | BSG |
|---|---|---|---|---|---|---|---|
| Walhachin | Aphanitic basalt, basaltic tuff and breccia, andesitic tuff and breccia (some calcareous), marble | 23.6 | 11.6 | 2.4 | 0.83 | 0.57 | 2.63 |
| Kimberley Float | Fine-grained argillite (>90%) and albitite (<10%) | 25.6-36.1 | 12.7-15.8 | 2.2-5.2 | 0.1-0.5 | 0.25-0.4 | 2.71-2.77 |
| Hawk Lake | Medium-grained granodioritic gneiss (>90%), coarse-grained granite (<10%), and schist (<1%) | 41.9 | 29.9 | 2.4 | 0.0 | 0.04 | 2.67 |
| Uhtoff | Fine-grained limestone and dolomite | 60.2 | 30.2 | 6.0 | 6.8 | 0.56 | 2.68 |
| Terrebonne | Coarse-grained limestone (50%) and dolostone (50%) | 90.4 | 33.9 | 11.3 | 7.1 | 0.97 | 2.74 |
| Hilton | Medium-grained granite (70%), biotite gneiss (25%), and serpentine (5%) | 42 | 22 | 4 | 2.69 | 0.56 | 2.71 |
| Gap | Mildly recrystallized oolitic limestone | 61.9 | 29.4 | 6.5 | 1.2 | 0.15 | 2.62 |
| Gouvenor* | Orthoquartzite (90%), quartzite (5%), and arkose (5%) | 20.7 | 13.7 | 1.4 | 0.1 | 0.4 | 2.57 |
| Dunelm* | Orthoquartzite (85%), quartzite (6%), arkose (4%), and granite and gneiss (5%) | 26.2 | 19.2 | 1.4 | 0.1 | 0.3 | 2.58 |
| Robsart* | Orthoquartzite (85%), quartzite (4%), arkose (5%), granite and gneiss (4%), and limestone (2%) | 36.0 | 22.5 | 2.7 | 0.1 | 0.17 | 2.60 |
| Hardisty* | Orthoquartzite (95%), quartzite (3%), and granite and gneiss (2%) | 35.4 | 27.9 | 1.5 | 0.2 | 0.97 | 2.59 |
| Wheatland* | Medium- to coarse-grained gneiss (45%), dolostone (40%), and amphibolite (15%) | 37.8 | 24.8 | 2.6 | 1.2 | 1.02 | 2.62 |
| Duval* | Dolostone (60%), medium- to coarse-grained gneiss (30%), amphibolite (9%), and quartzite (1%) | 42.9 | 24.9 | 3.6 | 1.2 | 1.02 | 2.62 |
| Bearspaw* | Orthoquartzite (57%), lithic sandstone (22%), quartzite (3%), limestone (12%), and siltstone (6%) | 47.2 | 22.7 | 4.9 | 2.34 | 0.96 | 2.53 |
| McKague* | Dolostone (50%), gneiss (40%), schist (2%), quartz sandstone (2%), amphibolite (5%), and limonite concretion (1%) | 44.6 | 28.1 | 3.3 | 0.06 | 0.73 | 2.69 |
| Slawa* | Medium- to coarse-grained gneiss (55%), calcareous sandstone (8%), quartz sandstone (15%), limonite concretions (4%), amphibolite (4%), and dolostone (3%) | 60 | 37.5 | 4.5 | 0.5 | 0.4 | 2.54 |

Note: AN = abrasion number, LAA = Los Angeles abrasion, MA = mill abrasion, Mg-S = magnesium soundness, Abs = absorption, BSG = bulk specific gravity. Asterisk denotes crushed pebble or cobble ballast; all others are crushed quarried rock.

subrounded merely as a result of crushing and transportation. Mill abrasion losses for the ballast samples typically range from 2.2 to 5.2 percent.

The argillite is a moderately tough rock, a quality that does not vary greatly with changes in mineralogy (LAA losses range from 12.7 to 15.8 percent for fresh material). The rock has a weakly developed foliation that results in some elongation of particles on crushing, but this is not a prominent feature and apparently has no significant negative influence on toughness or stability. The most important factor limiting toughness is the overall medium hardness of the material.

The importance of effective hardness in influencing the performance potential of a ballast such as this, composed of fine-grained rock, is well illustrated by the relationships between abrasion test results and hardness as estimated by petrographic examination. During the course of examining Kimberley Float ballast from various track sections and from the source stockpile, a semiquantitative estimate of hardness was made simply by determining the proportion of particles with hardness values of less than 5 on Mohs' scale. Graphic plotting of the percentage of soft components ($H < 5$) against abrasion number (LA + 5 MA) shows a fairly crude but quite definite correlation (Figure 1). By drawing on data on the durability of the materials as indicated by their track life and degradation characteristics, tentative subdivisions (based on AN) that indicate suitability as ballast have been indicated on the plot. Although there is a fair amount of scatter in the distribution of data points, it is apparent that, when a relationship such as this has once been established for a particular ballast material, the petrographer should be able to make useful predictions of abrasion test results and performance in track, even without having abrasion test results in hand. This example also demonstrates that the results of abrasion tests and durability in track are determined to a great extent by petrographic characteristics.

This exercise also provided insight into another important aspect of ballast testing. Misleading abrasion results can be obtained from materials that have been in service in track. These data may not be directly comparable with abrasion data derived from fresh materials from the source deposit. Petrographic examination of Kimberley Float ballast from Mile 52.2 of the Mountain Subdivision (CP Rail) indicated a very low content of soft components, an observation that is consistent with the exceptional durability of this material in track. However, initial abrasion test results indicated an anomalously high AN (Sample KF-1 in Figure 1), which was at odds with the petrographic data. Because they had been in the track for 17 years, the ballast particles had developed alteration mantles as well as acquired coatings of oil, grease, and adhering fines. Before further material from the same track section was tested, the samples were treated with a solvent to remove the oil and grease and the particles were scrubbed with a wire brush to remove at least the looser weathered material. The results of samples treated in this manner are shown in Figure 1 (Samples KF-2, KF-3, and KF-4), and it is apparent that they fall within the limits of the general distribution of points, whereas if they had been tested in an untreated state they could have been expected to plot close to Sample KF-1. It is not likely that the treatment restored the Mile-52.2 samples completely to their original condition, so by using the plot it can be estimated that a reasonable absolute minimum AN for the original material placed in this track section would be about 26.5. This study illustrates clearly the danger of comparing abrasion test results obtained from fresh material with those obtained from material that has been used in track. If such comparisons are to be made, due consideration must be given to the already degraded state of the used material.

The study of Kimberley Float ballast also provided clear evidence for the deleterious effects of sulfide minerals, which create an acidic environment in the ballast on oxidation. Evidence for this could be found in iron oxide spots and coatings, thin mantles of clay-rich alteration products, and

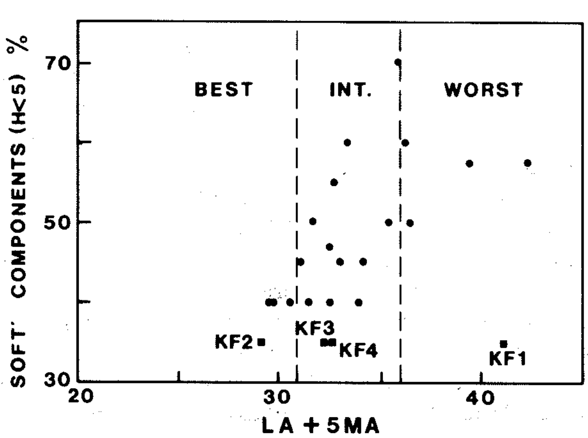


**FIGURE 1 Plot of soft components versus AN.**

sulfate deposits on fragment surfaces. It should also be emphasized that chemical weathering and physical breakdown by abrasion and impact operate hand in hand; the weathered mantles will be soft, allowing easy removal by physical processes and thus exposing further fresh argillite to chemical attack. It was observed that ballast with a low sulfide content (< 0.5 percent) showed little or no sign of chemical weathering whereas material with high sulfide contents (2 to 3 percent and higher) clearly displayed significant chemical breakdown.

The durability of Kimberley Float ballast has been extremely variable, and, in general, that placed in track at an early stage in the use of this material (early 1960s) has performed far better than that used during the 1970s. The reason for this difference becomes clear when the major petrographic characteristics are considered. The earlier material has a relatively high overall hardness and is low in sulfides (< 0.5 percent) whereas the later material, although variable, is always softer and has a higher average sulfide content (2 to 3 percent). Possible reasons for this change in quality of the material over the years can only be guessed at, but certainly it reflects a change in some aspect of the mining operation or of the rock being quarried. Whatever the reason, such changes in material quality, which may necessitate unexpected expenditure in track maintenance, should not go unnoticed. This emphasizes the importance of monitoring ballast sources by regular petrographic examination.

During petrographic examination of Kimberley Float ballast, it was noted that samples from immediately above the subballast in certain track sections contained a relatively high proportion of fines. A study was conducted in an attempt to determine if these had been derived entirely from the ballast or if the fines-rich subballast had also contributed fines by upward migration. Normal petrographic procedures (using a microscope) were not capable of making a satisfactory discrimination so the minus No. 200 fractions were subjected to XRD analysis. The XRD scans indicated that the mineral assemblages in the ballast and subballast fine fractions were quite similar and no clear-cut distinction could be made on this basis alone. However, careful analysis of the XRD traces indicated a definitely greater abundance of mica in the subballast. Mica was present in the overlying ballast fines but in lesser proportion. These proportions were found to be constant throughout all levels of the ballast, which indicates that there had been no significant mixing or contamination of fines in the lower ballast levels (which would have been indicated by an enrichment of mica). Furthermore, the mica in the subballast produced a well-defined, sharp peak on the XRD trace whereas the mica peaks in the ballast fines were broad and diffuse. These differences are not entirely due to the greater abundance of fines in the subballast; they probably also indicate a difference in crystalline character.

On the basis of this evidence, it is suggested that the high proportion of fines concentrated immediately above the subballast can be attributed to a downward migration of material derived mainly by abrasion in high levels of the ballast. Although the results obtained were not entirely conclusive, the study was of value in that it pointed out the feasibility of XRD methods for elucidating problems related to the source of fines.

**Walhachin**

Ballast from the Walhachin Quarry in central British Columbia is made up of a variety of rock types, primarily volcanogenic affinities. The quarry is located within the Nicola Group (Upper Triassic age). The volcanic rock units have been metamorphosed and strongly deformed with steeply dipping and folded beds and considerable faulting and fracturing. Recent redevelopment of the Walhachin Quarry involved a program of geologic mapping, drilling, petrographic analysis, and physical testing of most representative rock types and geotechnical evaluation and quarry design (see paper by Clifton et al. in this Record).

Six major groups of rock types can be distinguished in the quarry and from drilling operations in the immediate vicinity. All of these types have appeared as ballast components at some time or another, and the relative proportions have been dependent on which particular part of the quarry the material was taken from. Because these rock types vary considerably in their durability characteristics, it follows that variation in their relative proportions has resulted in corresponding variation in the overall quality of the material (which has, nevertheless, remained within acceptable limits). Such variations emphasize the importance of having detailed geologic information on the source and monitoring ballast source material, particularly if the geology of the source is complex and compositionally heterogeneous. Differences in durability of the various rock types can generally be accounted for by consideration of their petrographic characteristics.

*Basaltic Tuff and Breccia*

These rock types consist of angular fragments of aphanitic basaltic rock set in a matrix of identical, but finer, material. The rocks are extremely well indurated and generally very hard; they are also somewhat brittle and often break with a conchoidal fracture. Plagioclase feldspar, a relatively hard mineral (H = 6), is an important primary constituent and metamorphism has also resulted in the development of secondary minerals of which epidote (as much as 20 percent) and magnetite (as much as 25 percent) are prominent. Both of the latter are relatively hard minerals (H = 6 to 7 and 5.5 to 6.5, respectively) and are primarily responsible for the excellent abrasion characteristics of these rock types (Table 4). These minerals have a high resistance to chemical breakdown, which further enhances their overall durability. One negative aspect of these rock types is the common presence of irregular fracturing (due to brittleness) that, although fairly widely spaced ($>$ 0.5 in.), does sometimes result in relatively high LAA- and Mg-S-values compared with relatively unfractured samples of the same type (Table 4).

*Aphanitic Basalt*

This extremely fine-grained rock has mineralogy similar to that of the basaltic tuff and breccia and as a result is very hard and tough.

*Calcareous Tuff and Breccia*

These rock types are composed of angular fragments of andesite, set in a matrix of finer but similar material, plus variable amounts of calcite (as much as 30 percent). Epidote and garnet are prominent secondary minerals (20 to 70 percent). The durability of this rock, as indicated by abrasion test data, is variable depending on the proportions of the constituent minerals. It is limited primarily by the calcite, a relatively soft mineral (H = 3) that is prone to cleavage fracture. These properties result in rocks of only medium hardness and AN (Table 4). The significant effect of weathering on the performance of ballast is well displayed by a weathered sample of this rock type that returns a very high AN (89.07) and an extremely high Mg-S (27.01).

*Limestone and Limestone Breccia*

These are relatively uncommon rock types that occur as discontinuous lenses within the volcanic sequence. The

TABLE 4 ABRASION AND Mg-SOUNDNESS CHARACTERISTICS OF SOME TYPICAL MAJOR ROCK TYPES FROM THE WALHACHIN QUARRY (CP RAIL)

| Rock Type | LAA | MA | AN | Mg-S |
|---|---|---|---|---|
| Basaltic tuff | 11.33 | 1.63 | 19.48 | 1.42 |
| Basaltic tuff and breccia | 15.40 | 2.35 | 27.15 | 1.97 |
| Basaltic tuff and breccia (highly fractured) | 20.59 | 3.93 | 40.24 | 4.00 |
| Limestone breccia (recrystallized) | 33.56 | 7.88 | 72.96 | 0.96 |
| Calcareous tuff and breccia | 13.90 | 5.57 | 41.75 | 4.67 |
| Calcareous tuff and breccia (strongly weathered) | 29.52 | 11.91 | 89.07 | 27.01 |
| Intermediate (andesitic) tuff and breccia | 18.90 | 5.71 | 47.45 | 6.70 |
| Coarse intermediate tuff and breccia | 16.06 | 5.52 | 43.66 | 5.91 |

Note: LAA = Los Angles abrasion, MA = mill abrasion, AN = abrasion number, Mg-S = magnesium soundness limestone is a fine- to medium-grained crystalline rock and the breccia consists of fragments of this material set in a matrix that ranges from nearly pure carbonate to fine volcanic rock fragments. Both types have been recrystallized. As ballast components, these are undesirable materials because they have high LAA- and MA-values (Table 4) that can be attributed to the softness of the calcite, its prominent mineral cleavage, and its weakness as a binding agent in the breccia. Notably, the Mg-S-value is very low due to the dense, recrystallized, and unfractured state of the rocks.

### *Intermediate (andesitic) Tuff and Breccia*

These are highly variable rock types composed of fragments of volcanic rock set in a matrix of finer volcanic rock fragments, feldspar crystals, epidote, calcite, magnetite, and minor garnet. Generally these materials are of medium to high hardness and toughness (estimated), but some of the finer tuffaceous varieties have a relatively high calcite content and display bedding; both of these properties result in moderately high abrasion losses (Table 4).

### *Coarse Andesitic Breccia*

This rock type differs from that described previously in that it is coarser grained and has a carbonate fragment component. Moderate MA losses that characterize this material (Table 4) are due to the overall medium hardness (mineralogy and a partly altered condition), and the low LAA-values can be explained in terms of its well-consolidated and relatively unfractured nature. High Mg-S-values can probably be attributed to the presence of small solution cavities (removal of calcite).

## Prairie Gravels

Pleistocene glacial gravel deposits are frequently used as ballast sources in the Canadian prairies region. These gravels are usually composed of a mixture of rock types, and common major components are quartz sandstone (or orthoquartzite), granitic-granodioritic gneiss, dolostone, limestone, granite, and amphibolite. Relative proportions of these vary depending on the source, but invariably one or more of the first three rock types mentioned are dominant, and the latter types are subordinate constituents (Table 3).

Minor components (usually $< 10$ percent) that may be present are mudstone, shale, siltstone, limonitic concretions, calcareous sandstone, schist, arkose, quartzite, and various meta-volcanic rocks. These minor constituents are important in that all except the last three types mentioned are of very poor quality and their presence may be critical in evaluating the material for ballast purposes. Only a few percent of a very soft, clay-rich rock, such as mudstone, will render the material unacceptable as ballast because there will be a rapid initial buildup of fines and loss of permeability.

In addition to mineralogical criteria, particle shape and particle surface characteristics are important considerations when evaluating glacial gravel ballasts. Before crushing, a vast majority of particles are well rounded with generally smooth surfaces (i.e., water worn before deposition). Crushing fractures many of the pebbles and cobbles, thereby reducing the proportion of rounded surfaces and, on the average, increases the angularity of the particles. However, depending on the original particle size distribution of the material, the crushed product will always contain a certain proportion of particles that retain their original well-rounded outlines (small enough to be unaffected by crushing) and particles that have only one or two new fracture faces with a dominance of well-rounded and smooth surfaces. Even a moderate content of particles with these shape characteristics will render the material mechanically unstable. Consequently, shape factor testing is strongly recommended when evaluating this type of ballast.

As a result of their initially well-rounded and approximately spheroidal shape characteristics, the pebbles in these materials display a strong tendency to fracture (during crushing) in such a manner as to produce particles that have marked elongated or flattened shapes, or both; these are particularly abundant in the smaller size fractions ($< 3/4$ in.). Such particles are highly susceptible to load fracture in track, which further limits their applicability. Although largely independent of rock type, the tendency to produce elongated or flattened particle shapes is more prominent in the more brittle rocks such as orthoquartzite.

The nature of the major components of prairie gravels indicates that these materials will not constitute high-grade ballast. The quartz sandstone (arenite) is a clean, "mature" variety composed almost entirely of quartz grains that are well cemented by secondary silica occurring as grain overgrowths. This is a very hard rock type and ballasts composed dominantly of it produce, as a consequence, very low MA losses (1.4 percent). LAA losses are low to moderate (13.7 to 27.9 percent). It is extremely resistant to chemical weathering and its high durability in track is indicated by only a mild chipping of particle edges. The rock is usually dense, but rarely a microporosity is present; this is indicated by high absorption values and otherwise detectable only by thin-section examination. However, in spite of these excellent qualities, fragments and pebbles of this rock type possess very smooth surfaces that result in a high degree of mechanical instability.

The gneiss component ranges from fine to coarse in grain (crystal) size and is composed of feldspar and quartz as major minerals with lesser ferromagnesian constituents (mainly biotite mica or amphibole, or both). The gneisses are moderately tough and moderately hard rocks with a coarse foliation that does not significantly influence the fracture pattern on crushing. Provided that the proportion of original well-rounded surfaces on the particles is low, ballasts composed dominantly of this rock type will have acceptable mechanical stability because of the generally rough fracture surfaces. Cleavage fracture in the constituent feldspar crystals is a major factor limiting toughness and, to a lesser extent, hardness. In track the gneiss particles become rounded but are not greatly modified in shape when subjected to light loading.

Ballasts composed dominantly of dolostone tend to have moderate toughness (Table 3) and low hardness; the primary limiting factor is the softness of the constituent mineral dolomite. Cleavage fracture in the dolomite crystals may also be an important factor, particularly in the more coarsely crystalline varieties. Much of the dolostone has a porosity that may constitute up to 10 percent of the rock volume making it susceptible to degradation by freeze-thaw. The low resistance of carbonate rock types to abrasion is indicated by the considerable rounding of particles that frequently occurs by attrition during crushing, and in track they quickly acquire subrounded to rounded shapes.

Of the other major components of prairie gravels, granite displays behavior that is similar to that of the gneiss. Limestone is similar to, but even softer than, dolostone. Amphibolite is a rock type composed mainly of the mineral amphibole and is a tough, moderately hard material. Of all of the major constituents, gneiss (and granite) is the best because it offers the highest degree of mechanical stability combined with moderate toughness and hardness (Table 3).

Absorption values for prairie ballasts are generally fairly low (< 1.0) and acceptable for the rating assigned on the basis of other characteristics. Occasional high absorption is usually due to one or more of the following: intergranular porosity in the sandstone, porosity in the dolostone, microfractures in the dolostone, and intercrystalline and cleavage partings in the gneiss.

BSG is variable, depending on the relative proportions of the major components. Materials rich in quartz sandstone or gneiss have low BSG on the order of 2.6 whereas those rich in dolostone may range up to 2.7 or higher.

## CONCLUSIONS

Petrographic evaluation of rock ballast, in spite of being a largely subjective process, can provide useful assessments of performance in track, qualitatively predict the results of physical tests, provide rational explanations of test results, identify potential problems in materials intended for use as ballast, and provide reasons for failure of degraded material from track sections.

The performance characteristics of a rock ballast will be determined essentially by mineralogical, textural, and structural petrographic properties, usually in that order of importance.

Effective hardness is judged to be the single most critical factor in determining the durability of ballast. It depends primarily on mineralogy and texture, and to a lesser extent on structure, of the ballast components. These properties can be determined and effective hardness estimated or semiquantitatively measured, which permits useful assessments of ballast performance potential to be made.

Toughness of a ballast will also depend on structural, textural, and mineralogical properties, the relative importance of which is variable. Toughness can be qualitatively determined but cannot be related to a scale as hardness can.

Particle shape and surface characteristics, both critical factors influencing the mechanical stability of ballast, will depend to a great extent on rock structure and texture, respectively.

Chemical weathering of ballast particles will not usually be a prime cause of failure, although it may be a contributing factor. The potential for chemical breakdown of the derived fines component is, however, much higher and represents a significant factor in ballast evaluation.

Petrographic properties that may cause a ballast to be of fatally poor quality fall into three principal categories: mineralogical properties, such as alteration, high content of clay, mica, and sulfides; textural properties, such as poor consolidation, high porosity, and friability; and structural weaknesses, such as finely spaced foliation, joints, and bedding planes.

Petrographic analysis can provide quite specific information on the nature and source of derived fines, especially if the more sophisticated analytical techniques can be applied. X-ray diffraction and chemical analytical methods have great potential in this respect.

Petrographic analysis constitutes the most appropriate method for monitoring the quality of ballast from sources and the degradation characteristics of ballast from track.

## ACKNOWLEDGMENTS

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# Appendix: CP Rail Specification for Ballast

*Revised January 1, 1984.*

## SECTION I: EVALUATING PROCESSED ROCK, SLAG AND GRAVEL BALLAST SOURCES

### 1. Scope

This specification covers the requirements for evaluating properties of rock, slag and gravel for processed ballast.

### 2. Purpose

This specification is intended to identify suitable supply sources and to compare alternative sources for processed ballast. This specification shall also be used in conjunction with *Specification for Ballast — Section II, Processing Rock, Slag and Gravel Ballast,* to monitor the properties of material processed as ballast.

### 3. General Requirements

Processed ballast shall be crushed rock, nickel slag or crushed gravel, composed of hard, strong and durable particles, free from injurious amounts of deleterious substances and conforming to the requirements of this specification.

### 4. Ballast Material Tests and Track Ballast Standards

(a) The ballast material tests described in this specification shall be performed on representative samples.

(b) The petrographic analysis under Appendix A shall be used in conjunction with other ballast material tests listed in this section.

(c) Where a discrepancy arises between the estimated test results from the petrographic analysis and the results from the other ballast material tests, the results from the petrographic analysis shall have precedence; provided the petrologist reviews all test results and identifies the reasons for the discrepancy.

(d) The results of the physical property tests for stability listed below must meet the applicable Track Ballast Standards given in Table 1.

| | |
|---|---|
| Bulk Specific Gravity | ASTM C127 |
| Fractured Particles | Appendix A |

(e) The results of the material quality tests for resistance to weathering listed below should meet the applicable track ballast standards given in Table 1.

| | |
|---|---|
| Magnesium Soundness | ASTM C88 Five cycle on Ballast Grading 3 |
| Absorption | ASTM C127 on Ballast Grading 3 |

(f) The results of the material quality tests for abrasion listed below must meet the applicable Track Ballast Standards given in Table 1.

| | |
|---|---|
| Los Angeles Abrasion | ASTM C535 Grading 3 |
| Mill Abrasion | Appendix A |
| Abrasion Number | Appendix A |

(g) The material will be sampled and sized according to the following methods of test.

| | |
|---|---|
| Sampling Aggregates | ASTM D75 |
| Wire Cloth Sieves | ASTM E11 |
| Sieve or Screen Analysis of Fine and Coarse Aggregates | ASTM C136 |
| Materials Finer than No. 200 Sieve in Mineral Aggregates by washing | ASTM C117 |

(h) The material shall be processed to meet the appropriate grading of the Track Ballast Standards given in Table 1.

Alternative specifications and gradings may be used for special conditions, but only where specially authorized by the Chief Engineer.

### 5. Selection of Ballast Material

Materials meeting the applicable standards for the ballast material tests contained in this specification shall be compared. The ballast source selected shall be the most economical material based on the cumulative tons of rail traffic.

## TABLE 1 TRACK BALLAST STANDARDS

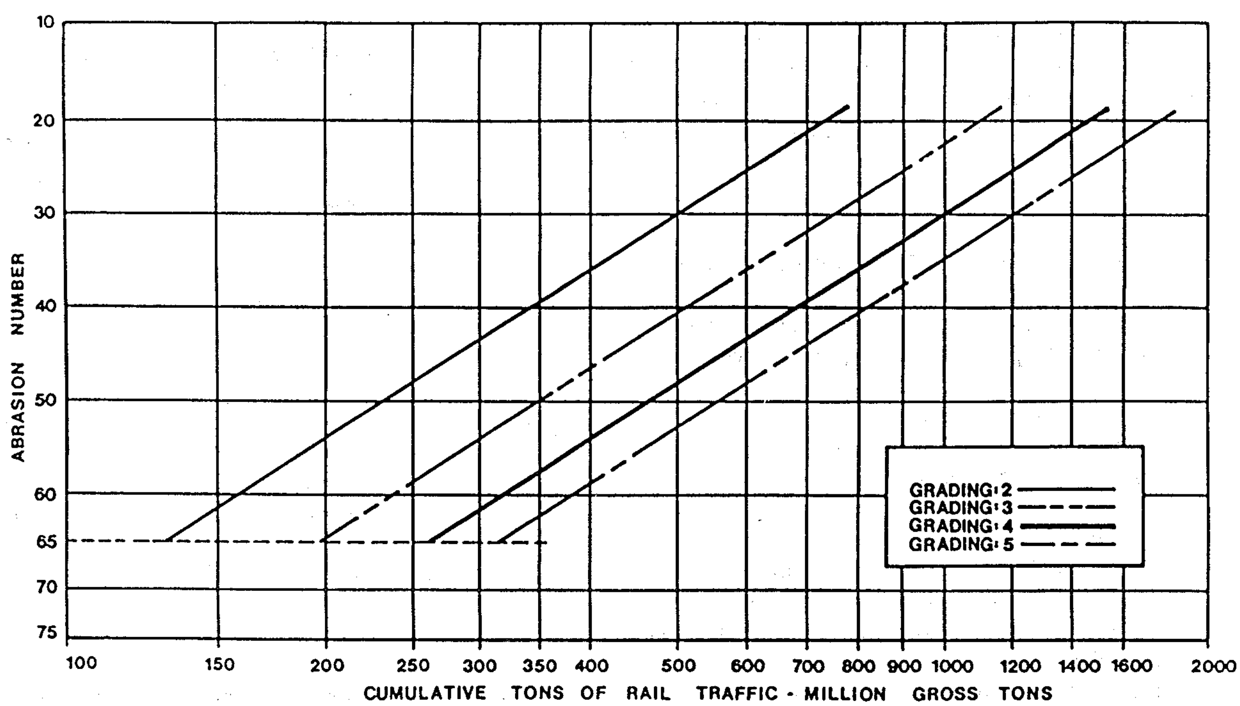
| Track Classification | Primary Main Line CWR | Primary Main Line Jointed | Secondary Main Line | Branch Line Important | Branch Line Minor |
|---|---|---|---|---|---|
| **Stability** - minimum allowable values given | | | | | |
| Bulk Specific Gravity | 2.60 | 2.60 | 2.60 | 2.60 | 2.60 |
| Fractured Particles - percent | | | | | |
| Ballast Grading | | | | | |
| 2 | - | 70 | 60 | 60 | 60 |
| 3 | 80 | 75 | 65 | 60 | 60 |
| 4 | 90 | 85 | 75 | 65 | 65 |
| 5 | 100 | - | - | - | - |
| **Weathering** - maximum allowable values given | | | | | |
| Magnesium Soundness | 1 | 1.5 | 3 | 3 | 3 |
| Absorption | 0.50 | 0.75 | 1 | 1 | 1 |

**Abrasion**

Los Angeles Abrasion shall be less than 45
Mill Abrasion shall be less than 9
Abrasion Number shall be less than 65; also shall be less than Abrasion Number for the cumulative tons of rail traffic for a 20-year period from Plan X-10-16-233, and should be less than Abrasion Number for the cumulative tons of rail traffic for a 30-year period from Plan X-10-16-233.

| Ballast Grading | Maximum Size Inches | 2 1/2" | 2" | 1 1/2" | 1" | 3/4" | 1/2" | 3/8" | No.4 | No.200 |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Percent By Weight Finer Than Specified Sieve | | | | | | | | |
| 2 | 2 | | 100 | 90-100 | 70-90 | 50-70 | 25-45 | 10-25 | 0-3 | 0-2 |
| 3 | 2 | | 100 | 90-100 | 70-90 | 30-50 | 0-20 | 0-5 | 0-3 | 0-2 |
| 4 | 2 | | 100 | 90-100 | 20-55 | 0-5 | | | 0-3 | 0-2 |
| 5 | 2 1/2 | 100 | 90-100 | 35-70 | 0-5 | | | | 0-3 | 0-2 |

Ballast gradings 2 and 3 shall be used for crushed gravel.
Ballast grading 4 shall be used for crushed gravel, crushed rock or slag.
Ballast grading 5 shall be used for crushed rock or slag.




NOTE: **DEVELOPED FOR TANGENT TRACK WITH 7 IN. OF BALLAST BELOW TIE. A SMALL AMOUNT OF CONTAMINATION FROM OUTSIDE SOURCES HAS BEEN TAKEN INTO CONSIDERATION.**

CAUTION: **EXCESSIVE HANDLING OF BALLAST MATERIALS IN THE UPPER RANGE OF ABRASION NUMBERS MAY GENERATE FINES WHICH WILL REDUCE THE APPLICABLE CUMULATIVE TONS OF RAIL TRAFFIC.**

**CP Rail**

BALLAST ABRASION NUMBER VS CUMULATIVE TONS OF RAIL TRAFFIC FOR VARIOUS BALLAST GRADINGS

X-10-16-233

OFFICE OF THE CHIEF ENGINEER.
MONTREAL, ~~FEB. 15, 1982~~ JAN. 1, 1984 GRADING: 1 DELETED

**FIGURE 1 Plan X-10-16-233.**

6. **Acceptance of Supply Source**

   (a) All evaluation results shall be forwarded to the Chief Engineer.

   (b) A summary sheet will be forwarded to the Chief Engineer summarising the economics of the sources considered.

   (c) The Chief Engineer must approve the source prior to its acceptance.

7. **Monitoring of Processed Ballast**

   (a) Representative samples of processed ballast shall be obtained at regular intervals during processing, and tested in accordance with this specification.

   (b) A summary of the test results shall be forwarded to the Chief Engineer at the close of each years processing of ballast.

Signed
Engineer of Track
and
Chief Engineer

This specification supersedes specification dated February 15, 1982.

## APPENDIX A TO EVALUATING PROCESSED ROCK, SLAG AND GRAVEL BALLAST SOURCES

### Petrographic Analysis

Petrography is the systematic description of rocks in hand specimen and thin section.

An experienced petrologist shall perform a petrographic analysis on the material. The megascopic features of the material shall be obtained from visual inspection aided by standard material identification and classification techniques. The microscopic features shall be obtained from inspection of thin sections of the material under a petrographic microscope.

The information obtained from the petrographic analysis on representative block samples and/or representative samples crushed to the various ballast sizes shall be documented under the following headings.

(a) Rock types retained on individual sieves including a description for layman where necessary.

(b) Mineralogy of rock types including proportions.

(c) Texture including grain size, shape, orientation, mutual relationships and matrix material.

(d) Structure identifying bedding planes, fracture planes and cleavage planes and foliation planes.

(e) Mechanical properties including hardness, strength and brittleness, type of fracture, shape and roundness.

(f) Chemical properties defining existing chemical weathering and potential chemical weathering.

(g) Properties of fines including shape, permeability, and susceptibility to solution and precipitation.

(h) Estimated test results including explanations.

(i) Special tests required including explanations.

(j) Summary of remarks including recommendations.

### Fractured Particles Test

The Fractured Particles Test is as follows:

A representative sample is obtained and sized using current ASTM methods of test. From each coarse aggregate fraction representing five percent or more of the submitted sample, split a representative portion into samples, of within 10 percent of the weight specified below.

| *Sieve Passing* | *Sieve Retained* | *Weight in Pounds ±10 Percent* |
|---|---|---|
| 2" | 1 1/2" | 13.0 |
| 1 1/2" | 1" | 6.5 |
| 1" | 3/4" | 3.5 |
| 3/4" | 1/2" | 2.25 |
| 1/2" | 3/8" | 1.0 |
| 3/8" | No. 4 | 0.75 |

Each sample shall then be separated into fractured and non fractured particles according to the following criteria.

A fractured particle shall be a particle with three or more fractured faces. Each of the fractured faces on the fractured particle must have a freshly exposed rock surface with a maximum dimension of at least one third the maximum particle dimension and a minimum dimension of at least one quarter of the maximum particle dimension. The included angle formed by the intersection of the average planes of adjoining fractured faces must be less than 135 degrees for each of the faces to be considered as separate fractured faces.

Particles which do not meet the above criterion will be classified as non fractured particles.

The fractured particles for each sample will be calculated as a percentage by the following formula.

Fractured Particles = Weight of fractured particles/Original Weight $\times$ 100

### Mill Abrasion Test

The Mill Abrasion Test procedure is as follows:

A representative sample is obtained and sized using current ASTM Methods of Test. From the coarse aggregate, split a representative portion into a sample consisting of 3.3 lb. passing the 1 1/2 in. sieve and retained on the 1 in. sieve plus 3.2 lb. passing the 1 in. sieve and retained on the 3/4 in. sieve. The sample shall be washed and oven dried in accordance with the Los Angeles Abrasion procedure. The sample will then be placed in a 1 gallon, 9 in. external diameter porcelain ball mill pot, along with 6.6 lb. of distilled water. The mill pot shall be sealed and rotated at 33 RPM for a total of 10,000 revolutions (five hours). The sample shall then be wash-sieved through a No. 200 sieve and oven-dried before weighing. Mill Abrasion shall be calculated as a percentage loss in weight by the following formula.

Mill Abrasion = Loss in weight/Original weight × 100

### Abrasion Number

The Abrasion Number is a number calculated with the results of the Los Angeles Abrasion Test and Mill Abrasion Test given in this specification. The Abrasion number shall be calculated by the following formula.

Abrasion Number = Los Angeles Abrasion + 5 × Mill Abrasion

## SECTION II: PROCESSING ROCK, SLAG AND GRAVEL BALLAST

### 1. Scope

This specification covers the requirements for material, grading, handling, stockpiling, inspection, measurement and payment for processed ballast.

### 2. Material

The material identified by location, description and/or properties consistent with *Specification for Ballast — Section I, Evaluating Processed Rock, Slag and Gravel Ballast, Sources* shall be processed as ballast.

### 3. General Requirements

Processed ballast shall be crushed rock, nickel slag or crushed gravel, composed of hard, strong and durable particles, free from injurious amounts of deleterious substances and conforming to the requirements of this specification.

### 4. Processing Requirements

(a) Grading

(1) The processed ballast shall be sampled, sized and tested in accordance with the following current ASTM Methods of Test.

| | |
|---|---|
| Sampling Aggregates | ASTM D 75 |
| Wire Cloth Sieves | ASTM E 11 |
| Sieve or Screen Analysis of Fine and Coarse Aggregates | ASTM C 136 |
| Materials Finer than No. 200 Sieve in Mineral Aggregates by Washing | ASTM C 117 |
| Unit Weight of Aggregate | ASTM C 29 Jigging procedure |

(2) The processed ballast shall conform to one of the following Ballast Gradings as specified by the Engineer.

| Ballast Grading | Maximum Size Inches | Percent By Weight Finer Than Specified Sieve 2 1/2" | 2" | 1 1/2" | 1" | 3/4" | 1/2" | 3/8" | No.4 | No.200 |
|---|---|---|---|---|---|---|---|---|---|---|
| 2 | 2 | | 100 | 90-100 | 70-90 | 50-70 | 25-45 | 10-25 | 0-3 | 0-2 |
| 3 | 2 | | 100 | 90-100 | 70-90 | 30-50 | 0-20 | 0-5 | 0-3 | 0-2 |
| 4 | 2 | | 100 | 90-100 | 20-55 | 0-5 | | | 0-3 | 0-2 |
| 5 | 2 1/2 | 100 | 90-100 | 35-70 | 0-5 | | | | 0-3 | 0-2 |

(b) Fractured Particles

(1) The processed ballast shall be tested in accordance with the Fractured Particles Test given in Appendix A.

(2) The processed ballast shall conform to the percent fractured particles specified by the Engineer, consistent with the following, for each size of coarse aggregate tested.

| Ballast Grading | Primary Main Line CWR | Jointed | Secondary Main Line | Branch Line Important | Minor |
|---|---|---|---|---|---|
| 2 | - | 70 | 60 | 60 | 60 |
| 3 | 80 | 75 | 65 | 60 | 60 |
| 4 | 90 | 85 | 75 | 65 | 65 |
| 5 | 100 | - | - | - | - |

### 5. Handling

Processed ballast shall be handled in such a manner that it is kept clean and free from segregation.

### 6. Stockpiling

Stockpiling of processed ballast will only be permitted at a designated site with an adequately constructed base. The processed ballast shall be stockpiled in layers, dumping over the sides of the pile will not be permitted. Stockpiling of processed ballast by pushing or dozing will not be permitted. Crawler type equipment should not be used on the processed ballast stockpile. Travel distances of rubber tired equipment on the processed ballast stockpile should be kept to a minimum, and shall not exceed 300 yards.

### 7. Inspection

The production of processed ballast shall be monitored to ensure the conditions of this specification are met. Samples shall be tested at least once per processing shift or every 1000 tons of production.

**8. Measurement**

The quantity of processed ballast shall be measured by weight on a platform scale equipped with an automatic printout. The scale shall be tested and sealed by the Standards Division, Weights and Measures Branch, Department of Trade and Commerce, Government of Canada.

When less than 100,000 tons of material are to be processed as ballast, an alternative method of measurement may be specified by the Engineer.

The quantity of prepared ballast shall be monitored continuously and the quantity recorded.

**9. Payment**

Prepared ballast will be paid for according to the actual measurement by weight or volume.

Signed
Engineer of Track
and
Chief Engineer

This specification supersedes Specification dated February 15, 1982.

## APPENDIX A TO PROCESSING ROCK, SLAG AND GRAVEL BALLAST

### Fractured Particles Test

The Fractured Particle Test is as follows:

A representative sample is obtained and sized using current ASTM Methods of Test. From each coarse aggregate fraction representing five percent or more of the submitted sample, split a representative portion into samples, of within 10 percent of the weight specified below.

| *Sieve Passing* | *Sieve Retained* | *Weight in Pounds* ±10 Percent |
|---|---|---|
| 2" | 1 1/2" | 13.0 |
| 1 1/2" | 1" | 6.5 |
| 1" | 3/4" | 3.5 |
| 3/4" | 1/2" | 2.25 |
| 1/2" | 3/8" | 1.0 |
| 3/8" | No. 4 | 0.75 |

Each sample shall then be separated into fractured and non fractured particles according to the following criteria.

A fractured particle shall be a particle with three or more fractured faces. Each of the fractured faces on the fractured particle must have a freshly exposed rock surface with a maximum dimension of at least one third the maximum particle dimension and a minimum dimension of at least one quarter of the maximum particle dimension. The included angle formed by the intersection of the average planes of adjoining fractured faces must be less than 135 degrees for each of the faces to be considered as separate fractured faces.

Particles which do not meet the above criterion will be classified as non fractured particles.

The fractured particles for each sample will be calculated as a percentage by the following formula.

Fractured Particles = Weight of fractured particles/Original Weight × 100

# Subgrade and Ballast Requirements for 125-Ton Cars

GERALD P. RAYMOND

**The main points relating to the performance of railway track subgrades are outlined. Particular consideration is given to the effect of increasing axle loads from 30 tonnes, standard on 100-ton cars, to 36 tonnes, estimated as probable for 125-ton cars. Two aspects are discussed, new track construction and upgrading existing track. On existing construction, where subgrade stability is borderline, an appropriate ballast lift to maintain or increase the existing factor of safety may be determined. For new construction, various aspects of subgrade preparation are briefly reviewed. These include the importance of adequate compaction, the treatment of subgrade soils with cement, and the treatment of subgrade soils with lime. Special aspects of dealing with swelling soils or high-sulfate soils, where avoidance is uneconomic, are also mentioned. Drainage and subballast use and requirements are also stressed. For existing track on chronically unstable clay subgrades or unstable embankments, consideration should be given to cement or lime slurry pressure injection. Slurry techniques are best performed by a specialist contractor who has had extensive experience with the technique. New conclusions are stated for many of the aspects reviewed, and often neglected aspects of subgrade and subballast requirements are highlighted.**

The two main requirements of a stable subgrade are (*a*) the provision of sufficient granular or modified soil cover to ensure that overstressing does not occur and (*b*) the provision of a granular filter blanket to prevent piping and thus loss of subgrade fines from below the track load-bearing area. To ensure that overstressing does not occur, track stresses need to be calculated. The writer (*1*) has previously recommended a means of calculating these stresses. The method is used in an example of upgrading of tracks that carry 100-ton cars so they can carry 125-ton cars. When fully loaded, a 100-ton car typically has 30-tonne axles and a 125-ton car has 36-tonne axles. These axle loadings will be assumed herein.

By using the approach previously outlined by the writer, the stresses in any direction at any point may be calculated for any load configuration. As an example of solutions obtainable with the method, Figure 1 shows the calculated vertical stresses below an interior axle of a configuration of two stationary coupled G-75 trucks for 30-tonne and 36-tonne axle loading and two postulated conditions: (*a*) on track that has a modulus of 14 MN/m/m of rail constructed with 229-mm crossties at 459 mm center to center and 68-kg/m rail representing the most conservative North American main-line wood crosstie track and (*b*) on similar track that has a modulus of 224 MN/m/m of rail constructed with 279-mm crossties at 610 mm center to center representing North American concrete crosstie track with stiff tie pads.

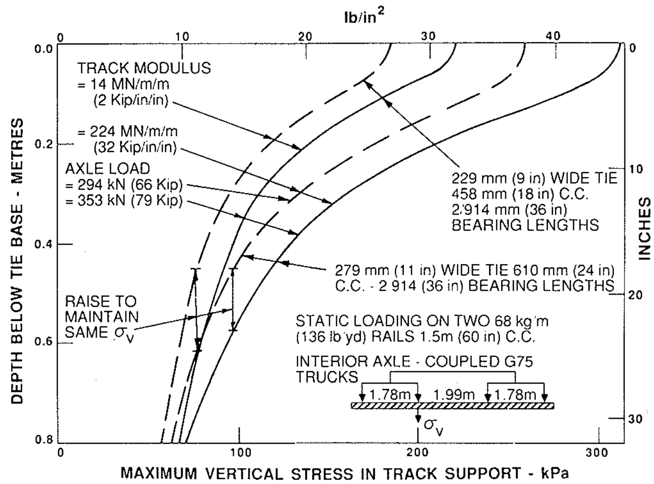



**FIGURE 1 Variation of maximum vertical stress through the soil profile below two extreme examples of typical North American track from the inboard axle of two coupled G-75 trucks with static 30- or 36-tonne axle loadings showing the increased ballast cover to maintain subgrade stresses.**

If only static axle loads are considered, the results in Figure 1 may be compared to obtain the increased ballast depth required to maintain at least the same subgrade factor of safety when axle loads are increased from 30 tonnes to 36 tonnes. Such a comparison is shown for an assumed granular cover of 0.45 m in a track that takes 30-tonne axles. Dimensioned is the increase in cover required to maintain the same vertical stress should the axle loads be increased to 36 tonnes.

Similar calculations have been performed for the same two track configurations but for different track moduli. Figure 2 shows the results obtained, using different moduli between 3.5 and 224 MN/m/m of rail, for initial ballast covers above the subgrade of 300, 500, and 700 mm. It is apparent from Figure 2 that the initial track modulus has less influence than does the initial cover on the increase in cover required to maintain the same vertical stress at the subgrade elevation. Indeed, a large variation in the assumed track modulus has little effect on the results unless it is initially quite stiff. These results confirm the general practice of assuming values of track moduli rather than using accurate measurements. In

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addition, the track modulus is likely to vary with the seasons and along any length of track. Of course, Figure 2 applies only to the upgrading of existing track. In any application in which the track modulus is physically changed, such as when wood ties are replaced by concrete ties, the subgrade stresses will also be changed. This is clearly seen in Figure 1 by comparing the vertical stress profiles for the two different constructions. It also shows why most railways upgrade and place new and more ballast below the ties when replacing wood ties with concrete ties.

It should be clearly understood that Figure 2 deals with the increase in cover required on subgrades stable under 30-tonne axles that are to be maintained at the same factor of safety or better under 36-tonne axles. Of course, many subgrades may have high factors of safety under 30-tonne axles, and these may still be at an acceptable level under 36-tonne axles even without any ballast raise. The major point is that, based on static loading, Figure 2 shows the increases of ballast that may be considered the maximum requirement when safety factors against subgrade failure are considered at their desired limiting values.

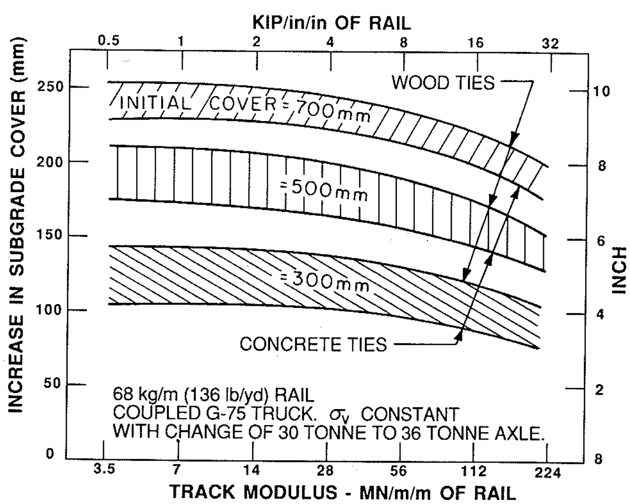



**FIGURE 2 Values of increased ballast cover required to maintain subgrade stresses from 36-tonne axle loadings at the same values as those from 30-tonne axle loadings, obtained from plots similar to those shown in Figure 1.**

## DYNAMIC LOADING

The writer (*1*) also presented information indicating that track irregularities or wheel irregularities, or both, are the principal cause of major dynamic track forces at present-day (1987) speeds. An approximation for the dynamic increment was given (*1*).

Subsequent calibration of this expression for speeds greater than 75 km/hr has resulted in the empirical solution

$$\Delta P_d = 4.3 s M^{0.5} U^{0.375} V \quad (1)$$

where

$\Delta P_d$ = dynamic increment in N,
$s$ = depth of the irregularity in mm,
$M$ = unsprung mass in kg,
$U$ = track modulus in MN/m/m of rail, and
$V$ = vehicle speed in km/hr above 75 km/hr.

Because Equation 1 is an empirical equation, the units of measurement of each parameter must be strictly maintained.

Unless the unsprung mass or track quality is changed, Equation 1 states that the dynamic loading increment would be the same for 36-tonne axles as for 30-tonne axles. This is particularly true for those railways that overload 100-ton cars to produce 20 percent additional axle loading. Typical solutions for a 100-ton car, obtained for the two conditions cited for Figure 1 plus an intermediate condition, are shown in Figure 3. These results assume the rather severe case of a 51-mm$^2$ flat on an inboard axle and thus emphasize that vertical stress is greatest near the tie base level. Thus the values given in Figure 2 will be conservative when static plus dynamic loading is evaluated. The increased static loading from the heavier axles could, of course, be sufficient to cause track or wheel defects to occur quite rapidly, which would result in uneconomical maintenance, particularly if track and wheel quality were to remain unchanged.

Acceptance of Equation 1 as valid means that the conclusion related to the depth increment already expressed for static loading is valid for static plus dynamic loading unless track or wheel defects are permitted to increase. That is, the increased cover shown in Figure 2 should maintain the subgrade stresses below 36-tonne axles at values less than or equal to those existing below 30-tonne axles. Equation 1 also states that dynamic loading of the subgrade may be reduced by decreasing the track modulus. This means using greater depths of clean granular ballast and subballast and softer tie pads with concrete ties.

## BALLAST DEPTH DESIGN

Clearly evident from Figure 3 and Equation 1 is the dramatic effect a higher track modulus has on the dynamic increment.

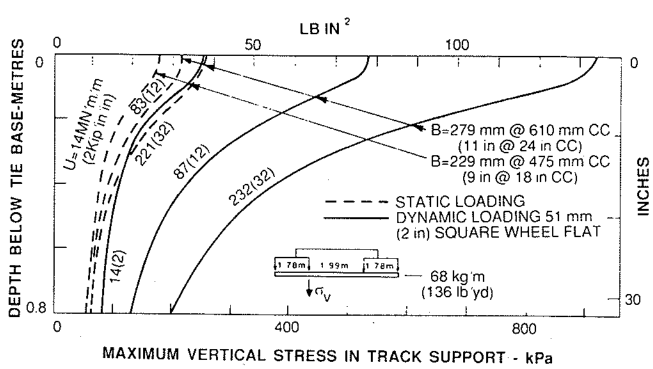
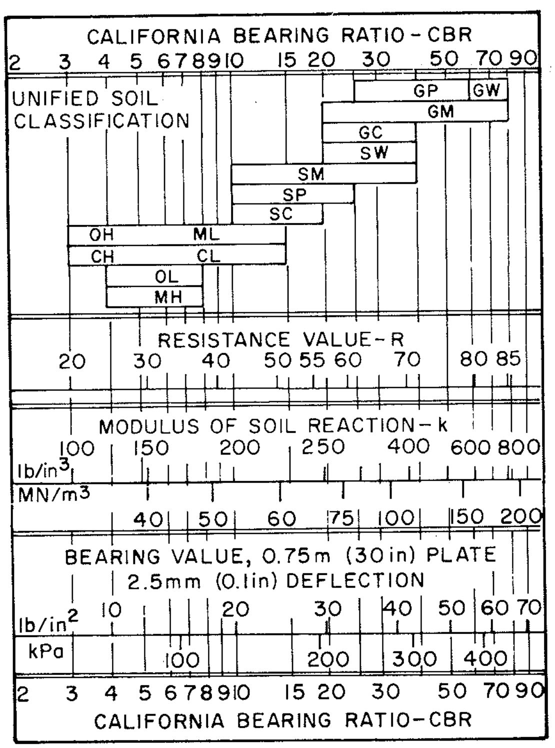



**FIGURE 3 Variation of maximum vertical stress through the soil profile below three examples of typical North American track from an inboard axle of two coupled G-75 trucks with 30-tonne static axle loadings and static loading plus dynamic loading from a 51-mm$^2$ wheel flat on the axle wheels of the inboard axle traveling at 80 km/hr.**

This has recently been confirmed by Scott (*2*) in research designed to establish loading on concrete turnout ties. As pointed out by the writer (*1*), it is fortunate that such moduli are generally associated with firmer, and thus generally stronger, subgrades. The stiffer concrete tie track results in more resistance to the formation of large track irregularities leaving wheel irregularities as the predominant concern related to dynamic increment on continuously welded rail. Fortunately, wheels do not generally have carefully made square flats associated with "worst conditions," and flats are associated with a small percentage of wheels. Typical foundation design is based on the dead load plus some percentage (often 50 percent) of the maximum live load. This is acceptable because established safe soil load-bearing stresses generally include some measure of safety factor that may be exceeded on a limited basis. They are also generally based on repetitive loading under soaked subgrade conditions.

When a design live load has been selected, the calculated stresses may be obtained for any loading including 125-ton cars. These stresses are then related to safe soil load-bearing stress (*3*), such as those shown in Figure 4, or other available soils data to obtain the required granular (subballast plus ballast) design depth. As pointed out elsewhere (*1*), Figure 4 was developed for use in highway and airport design, and the limits are considered conservative for use in track support subgrade design. A 50 percent increase was suggested (*1*).

Where climatic conditions result in freezing temperatures, a minimum granular cover equal to at least one-half the depth of frost penetration is generally specified.




APPROXIMATE INTERRELATIONSHIPS AT MODIFIED MAXIMUM DENSITY

**FIGURE 4 Safe soil load-bearing stresses developed for highway and airport design (*3*).**

## EQUIVALENT DAILY LOADING AND EQUIVALENT GRANULAR COVER

In highway design it is normal practice to account for the variety of axle loadings that traverses any given point. In railway operations it is most economical to load axles to maximum permissible values. Thus equivalent axle counts are not a consideration in railway design. Furthermore, axle loadings are not decreased seasonally. As demonstrated by Heath et al. (*4*), fatigue failure in subgrades occurs fairly rapidly when threshold stresses are exceeded. They also showed that loadings near but below the threshold stress caused little damage. Thus failure is quite sensitive to small increases when stresses are near the threshold but insensitive to large changes as long as the threshold stress is not exceeded. Such findings are in agreement with those of Raymond et al. (*5*) and Gaskin et al. (*6*). Equivalent daily loading considerations are thus not generally required for track design.

Another factor that is sometimes considered in highway engineering is the granular equivalency factor. This is shown in Figure 5, which has been taken from research by Herner (*7*). Because ballast has a very open structure and, even when properly protected from subgrade fouling, is subject to fouling by windblown and other debris, it is doubtful that equivalency factors should be assigned to aggregates used to protect the subgrade.

## SUBGRADE TREATMENT

On new construction designed to minimize future maintenance, whether for 125-ton cars or lighter vehicles, it is normal practice to remove topsoil for further use and proof-roll the surface of the subgrade excavation to locate unacceptable weak zones, particularly in cuts and low fills. Weak zones are generally subject to further excavation

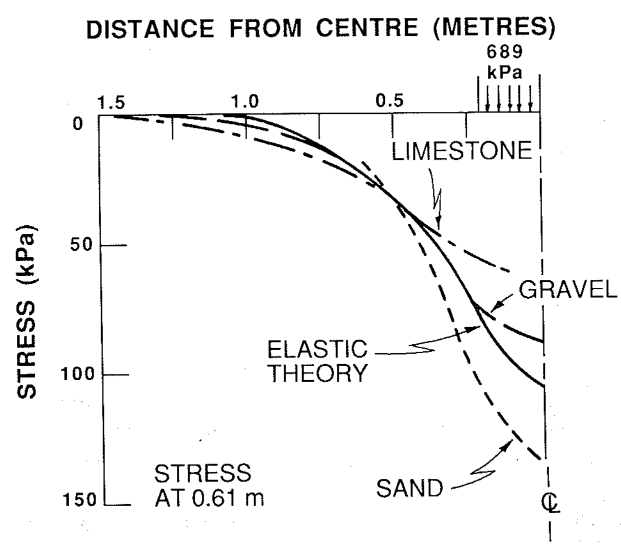



**FIGURE 5 Vertical stresses measured by Herner (*7*) at a depth of 610 mm below loaded aircraft wheel on different types of granular soil support.**

followed by compaction of any replaced soil. Acceptable areas are often scarified, after the addition of any water necessary to achieve optimum, to a depth of about 0.15 m and recompacted to 90 percent of maximum density as defined by ASTM D 1557 or 95 percent of maximum density as defined by ASTM D 698. Even when these precautions have been taken, the compacted, soaked subgrade strength may be sufficiently weak that granular cover requirements exceed the minimum depth on new construction of 305 mm (12 in.) of ballast plus 150 mm (6 in.) of subballast recommended in the *Manual for Railway Engineering* (*8*, Article 1.2.5.4). Where granular materials are scarce, such as in midwestern North America, a makeup depth of the subgrade may be treated with lime, flyash, cement, or other suitable additives to produce a stronger upper crust of treated subgrade to replace part of the granular cover requirement.

The treated subgrade is generally regarded as equivalent to unbound granular material. This is because the modified soil layer is generally quite brittle and would crack under loading. For example, the maximum calculated tensile stress using theoretical solutions developed by Burmister (*9*) at the bottom of the second layer of a three-layer elastic solid is shown in Figure 6. It is therefore essential that such modified layers be covered with a noncementing and thus a nonplastic granular material. Such granular material must be graded to act as a filter to the subgrade and be located immediately above the modified soil.

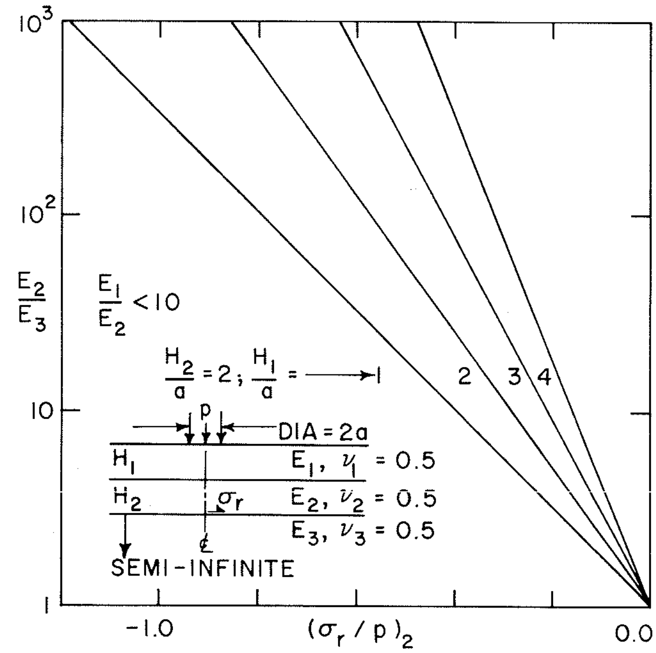



**FIGURE 6 Ratio of maximum tensile stress to surface pressure at the base of the second layer of a three-layer semi-infinite elastic medium subject to a uniform circular surface load.**

## EARTH COMPACTION

Early earthwork practice permitted the construction of embankments and subgrades without compaction. Transportation support systems were not placed on completed subgrades for many months to allow natural consolidation. Unfortunately, detailed observations have shown that consolidation of most soils by nature alone is inadequate and results in many failures due to the lack of adequate compaction. Indeed, failures often occur on clay embankments where compaction has been undertaken (*10*). Such observations become clearer under heavier loads such as those associated with heavy-axle cars. Adequate compaction assures three main factors that are desirable for good embankment, subgrade, and track performance: (*a*) a decrease in the susceptibility of the soil to settlement, (*b*) a decrease in the permeability of the compacted subgrade soil, and (*c*) an increase in the soil's supporting power [e.g., California bearing ratio (CBR)].

Figure 7 shows how these three criteria are satisfied. Starting at the base of the figure and working upward are shown (*a*) a typical density curve, (*b*) a zero air voids or complete saturation curve, and (*c*) a penetration resistance curve.

Settlement is mainly caused by reduction in volume, which is caused by compression of the voids (unless failure is evident). The least value obtainable on the complete saturation line corresponds to the highest value of dry density obtainable for the constant compactive effort curve considered.

Because water must permeate the void space, obtaining a minimum void volume generally means obtaining a minimum permeability. Capillary water movements are also minimized—again indicating the importance of maximum density.

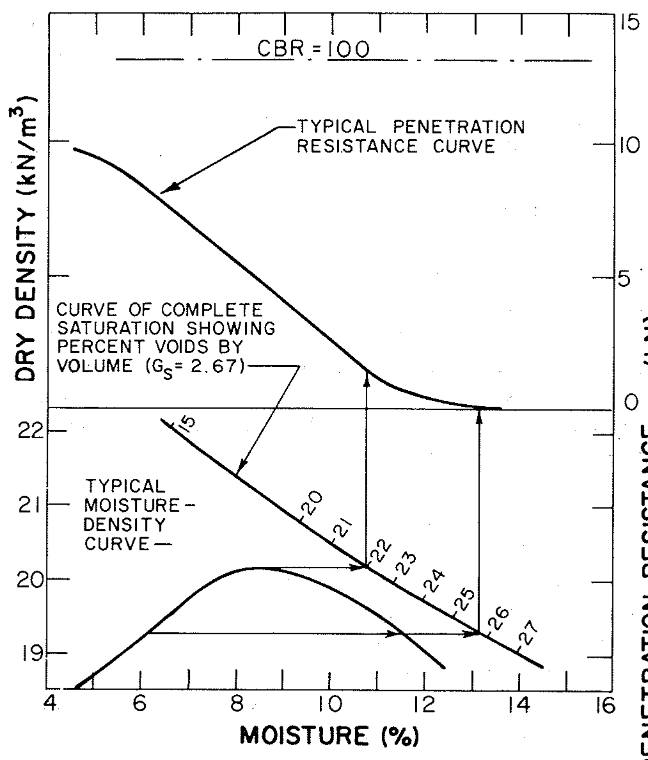



**FIGURE 7 Typical soil compaction data showing density-moisture relationship and zero voids curve below typical penetration resistance-moisture relationship.**

Bearing capacity related to the compaction curve is also shown in Figure 7. Clearly, when the soil is not saturated its bearing capacity can be large. On soaking and saturation, however, the soil's bearing capacity for a given density drops to that corresponding to the zero air voids or complete saturation curve. For a given density-moisture curve, maximum soaked or saturated penetration resistance occurs at maximum density.

## CEMENT STABILIZATION

Cement stabilization involves breaking up (pulverizing) the soil to be treated, adding cement, mixing the cement with the soil, and then watering and compacting in the usual manner. It is generally about 30 percent cheaper to combine the cement and soil in place than to batch mix. Thus in-place mixing is the most common method of treatment. When the cement hydrates, the soil is stabilized and will be stronger and more resistant to penetration by water and other materials. Care must be taken to avoid soils that have more than 1 to 2 percent organic matter and those with sulfate contents greater than 5,000 ppm $SO_4$ (*11, 12*). Even 2,000 ppm $SO_4$ or less may be cause for concern. Where soils with high sulfate contents are used, seal coats of waterproofing should be applied. Seal coats should also be used on so-called swelling soils. Sulfate action causes disintegration and softening of the cement-stabilized soil when the moisture content exceeds that necessary for compaction. This may occur at any time, even after several years of satisfactory performance. In the opinion of the writer, seal coating and good drainage are the only reasonable defense where sulfate soils must be used. They are also considered necessary with so-called swelling soils.

The addition of small quantities (1 or 2 percent) of cement will modify the properties of soil; larger amounts (4 percent or more) cause major property alterations; and very high percentages cause concretelike materials. The effects of the two processes—cement modification and stabilization—differ principally by degree. Recent practice for subgrade stabilization work has tended away from the practice of producing concretelike materials (i.e., very high percentages of cement). More common is stabilization of the soil to maintain mechanical stability. (An exception is the stabilization of water-retaining structures such as open canals.)

The Portland Cement Association in its *Soil-Cement Laboratory Handbook* (*13*) outlines a method of obtaining the amount of cement required to stabilize a soil. The first estimate is given in Table 1. Laboratory moisture-density tests may then be performed using cement values in 2 percent increments of those given in Table 1. A 7-day soaked unconfined strength of about 1.75 MPa is generally acceptable for railway subgrades.

Ordinary portland cement is the most commonly used cement stabilization material. Additives are sometimes used; however, their use should be treated with caution. Incorrectly proportioned quantities may be detrimental to stabilization. The main exceptions are the use of small quantities (2 percent) of lime to aid pulverization in high-plasticity clayey soils (*14*) and pulverized fuel ash (fly ash) or natural materials that act as a pozzolan (ASTM C 618).

TABLE 1 INITIAL CEMENT ESTIMATE (*13*)

| AASHO Soil Group | Percentage Cement by Weight |
|---|---|
| A-1-a | 5 |
| A-1-b | 6 |
| A-2 | 7 |
| A-3 | 9 |
| A-4 | 10 |
| A-5 | 10 |
| A-6 | 12 |
| A-7 | 13 |

Cement-stabilized soil should be compacted as soon as possible after mixing; otherwise hydration begins (*14*) and the soil's strength builds up in clods, which results in an overall strength reduction due to the difficulty of compacting the soil. If correctly compacted, the soil cures to a stiff material with a deformation modulus in the range of 0.14 to 20.0 GPa (*14*). On the basis of Figure 6 it is apparent that cracking is highly likely. Indeed, many consider that the correct treatment of cement-stabilized layers is to allow traffic access as soon as possible after construction (*15*). Such a procedure recognizes that cracking is likely to occur and permits fully cracked interlocking to develop early.

Accepting the probability of cracking suggests a cement content sufficient to result in a 7-day soaked unconfined strength of about 1.75 MPa (*14*). To limit shrinkage cracking, a compaction moisture content 1 to 2 percent dry of optimum and never greater than optimum is generally specified (*14, 15*). (Note that this is different from untreated soil specifications in which 1 to 2 percent wet of optimum is used to enhance workability.)

Where moisture must be kept from the subgrade, the surface is sealed with an asphaltic or mastic seal (*16*). Such an asphaltic seal might consist of a mixture of (*a*) 65 to 80 percent by weight 120 to 150 penetration grade asphalt heated to from 175°C to 205°C, (*b*) 20 to 35 percent ground tire rubber of sieve size No. 16 to No. 25, and (*c*) 5 to 7.5 percent kerosene to reduce viscosity for spraying (*16*). After application, some 10-mm-sized chip screenings are added. Several layers may be applied over swelling subgrades. Good drainage is, of course, essential. This is true for all good subgrade performance.

A series of articles and design guides is available from the Portland Cement Association (Skokie, Illinois 60076).

## LIME STABILIZATION

Stabilization of soils with hydrated lime is similar to cement stabilization. It is generally used on high-liquid-limit clays and is rarely used with granular soils unless they contain plastic fines. Lime or lime slurry may be added as a soil conditioner. Lime is thus a versatile stabilizer of clay subgrades (*14*).

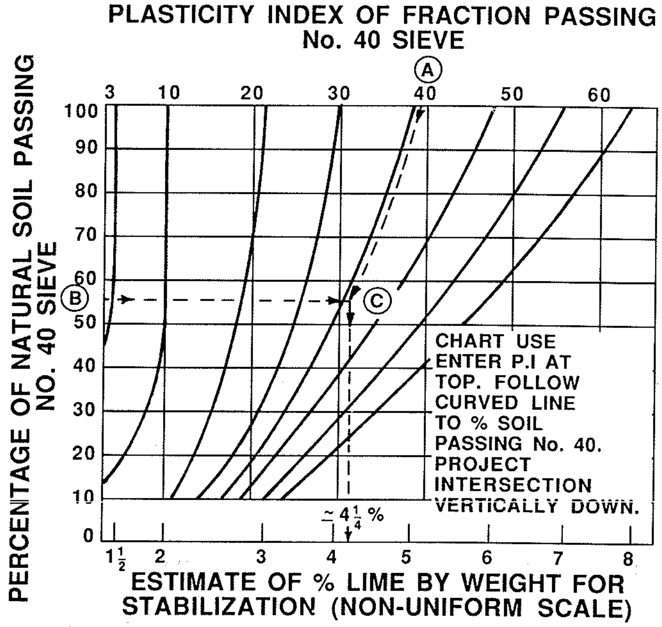



**FIGURE 8 Guide for obtaining percentage of lime for stabilizing clay and silt soil fractions within natural soils (*18*).**

Lime stabilization is achieved with calcium hydroxide (hydrated lime) or calcium oxide (quicklime). Because of environmental considerations, the former chemical is more commonly used. Both have a high percentage (about 90 percent) of calcium and magnesium oxide present. The stabilizing effect results from chemical action and the formation of cementitious compounds (calcium silicates). Calcium carbonate (agricultural lime) does not form silicates and is unacceptable as a stabilizing material (*14*).

Organic soils and sulfate soils need the same type of special attention that cement-stabilized soils require. Similarly, most additives except fly ash are of questionable value.

Unlike cement-stabilized soils, which need compaction as soon as possible after mixing, lime-stabilized soils require a 72 hr rot period after mixing and compaction (*14*). Lime-stabilized soils generally increase in strength with the addition of lime up to a given limit above which additional lime has little value (*14*). In frost penetration areas, a strength of 1.4 MPa is generally considered effective against frost action (*17*). For some soils several months of cure time will be required to obtain this strength (*17*).

Again it is important to perform preconstruction tests to determine the suitability of lime stabilization and obtain the cure times required. An approximate estimation of the lime requirement may be obtained from Figure 8 (*18*). An example of the use of this chart is given for a soil that has a plasticity index of 39 and a percentage passing the No. 40 sieve of 55. These data are entered on the chart as Positions A and B, respectively. Following the curved lines downward from A to the intersection of a horizontal line from B gives Point C. Point C is projected vertically downward to give the estimate of the required percentage of lime by weight.

An extensive series of articles and design guides is available from the National Lime Association (Arlington, Virginia 22201).

## GROUT AND LIME SLURRY PRESSURE INJECTION

On chronically unstable clay subgrades under existing track it will be essential to undertake some form of subgrade or embankment strengthening before heavier axle-loaded cars are introduced. Lime slurry pressure injection (LSPI) roadbed stabilization is an accepted procedure (*19*) used throughout the United States by most of the major railroads to treat high-maintenance, chronic track problems associated with high-plasticity soils. Cement and cement grout injection are also used in deep ballast pockets (*10*). The slurry is injected into the ground under pressure forcing free-water pockets to be drained and forming a network of strong stabilization seams. The slurry is distributed in vertical and horizontal sheets throughout the soil mass as it flows through available voids, fissures, dessication cracks, failure planes, and coarse geologic formations. Certain physicochemical changes occur at the interface of the grout seams, where direct contact is made with the soil, as well as in the soil between the seams because of supernatant impregnation. The stabilized grout-impregnated seams form impervious moisture barriers that impede the movement of moisture and add considerable strength and reinforcement along the former zones of weakness and dislocations. In certain instances multiple injections are used to achieve increased stabilization by creating a more extensive network of seams. The larger fissures and voids are usually filled by the initial injections. The second injection allows the slurry to penetrate small cracks and thus achieve a denser distribution of slurry in the soil mass.

The purpose of grout and lime injection is to improve track stability by (*a*) reducing the water-holing capacity of the ballast pockets; (*b*) sealing, where possible, the ballast-subgrade interface (the writer doubts that this is possible on a long-term basis from grout injection only); (*c*) impeding the movement of moisture in the subgrade by forming relatively impervious seams throughout the treated mass; (*d*) increasing the strength of the treated mass through physicochemical reactions between the soil and the lime and fly ash; and (*e*) reducing the volume change potential of the subgrade clays. The techniques involved are best performed by a specialist contractor who has had extensive experience with injection processes. After treatment, undercutting and ballast cleaning and replacement are advisable along with the possible installation of a subballast or geotextile.

Some articles and design guides on lime slurry pressure injection are available from the National Lime Association (Arlington, Virginia 22201).

## CAUSES OF UNSTABLE SUBGRADES—SEPARATION REQUIREMENTS

Irrespective of the amount of compaction a subgrade receives during construction, some degree of permanent deformation occurs because of traffic loading. This loading is greatest below the rail, which causes the formation of a depression in which water may collect. On good subgrades, this depression may be only millimeters deep, but this depth can be sufficient to cause water ponding and softening of impermeable subgrades. To prevent silt-sized fines from penetrating upward from such subgrades, nonplastic sand-sized materials grading continuously and gradually down to the 74-micron (No. 200) sieve should be included in the subballast, or a nonplastic sand-sized granular capping material should be provided on the fill or subgrade. This capping material should act as a filter to prevent fines from being vibrated or pumped upward. Similarly, subballast should be continuously and gradually graded to prevent it from vibrating upward into the ballast. A nonwoven geotextile may be used to meet this latter require ent. A geotextile, however, only acts as a partial filter to silts or clay. Typical gradings recommended by the writer for use by Canadian National Railway and Canadian Pacific Railway are given in Table 2 (*20*).

It should be clearly understood that the dynamic loading experienced within a railway track support fill is much greater than that generally experienced by any highway support system. As has been shown (*1, 2*), impact loading from wheel flats or out-of-round wheels can impose dynamic loads several times those due to static loading. Modified-clay subgrades, which form a brittle, hard subgrade surface, are generally subject to cracking and must be covered by a nonplastic granular filter material. As wheel loading is increased from 100-ton cars to 125-ton cars, these suggestions will clearly be even more relevant.

Even where subgrades or fills adjoining the subballast are well compacted, dry summer weather may be expected to cause drying of the surfaces in contact with the subballast. Wetting after such dry weather may then be expected to cause collapse of the soil structure of the subgrade surface, which accelerates the erosion of fines upward into any unprotected ballast or substandard subballast. Over time such erosion could be expected not only to foul any substandard subballast but also to foul the ballast. Once fouled, both materials may be expected to heave during freezing weather. The writer cannot overstress the importance of using suitably graded subballast that will remain unfouled by eroding fines from any underlying material. The subballast should also be suitably graded so as not to itself be vibrated upward and foul the ballast. Interface drainage is also of great importance. The subballast material must be nonplastic so as to deform (or collapse) easily by flowing and not cement. Cementing would permit vertical fractures.

## TRACK SECTION

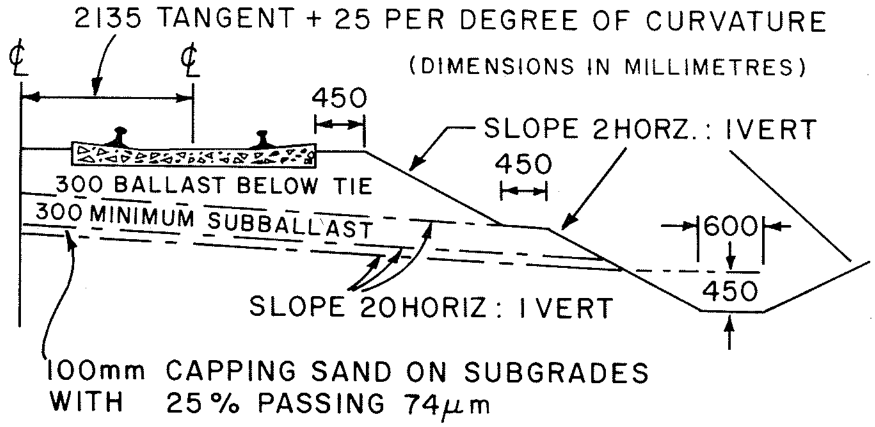
The exact construction of a given track section is clearly a designer's responsibility and should be based on site-specific data and costs. Figure 9 shows a summary cross section indicating the probable main requirements for major two-track installations for tracks carrying 125-ton cars. Figure 9 incorporates the following recommendations from the *Manual for Railway Engineering* (*8*): (*a*) 3.6 m center to center between track centerlines, (*b*) a 0.45-m berm at the upper level of the subballast to allow future track raising without ditch filling, (*c*) a ditch 450 mm below the discharge level of the prepared (compacted) subgrade, (*d*) a 300-mm minimum ballast depth, and (*e*) a 300-mm minimum subballast depth. In addition, Figure 9 shows recommendations, considered justified for quality track with low maintenance, that are more demanding than those specified in the AREA manual. These are (*a*) a 5 percent gradient to the prepared subgrade, (*b*) 450 mm of ballast shoulder as opposed to 150 mm, and (*c*) a 600-mm-wide ditch invert as opposed to a V-shaped base. Other factors shown, which are nonrigid recommendations of the AREA manual, include (*a*) a 100-mm sublayer indirectly specified by AREA as needed where pumping is likely to occur (i.e., a suitably graded subballast layer is needed on pumpable subgrades) and (*b*) a suitable method of partial frost cover protection in cold climates.

Some railroads use less ballast and subballast than recommended by the AREA manual (*8*, Article 1.2.5.4). As tampers are made more powerful to achieve greater productivity, their tamping tines become larger. Their movement back and forth moves ballast to a depth of 200 mm. Assuming a settlement between tamping of 50 mm, it is apparent that a 300-mm

TABLE 2 SUBBALLAST GRADING LIMITS (*20*)

| Sieve Size | Capping Sand[a] | Filter Sand[b] | Open-Graded Subballast[c] | Broadly Graded Subballast |
|---|---|---|---|---|
| 1 in. | | 100 | 100 | 100 |
| 1/2 in. | | | 51-100 | 51-100 |
| 3/8 in. | | 80-100 | 30-80 | 48-80 |
| No. 4 | 100 | 65-100 | 0-15 | 40-65 |
| No. 8 | | 55-85 | 0-3 | |
| No. 16 | 70-100 | 45-70 | | 25-45 |
| No. 50 | 25-70 | 25-45 | | 10-25 |
| No. 100 | 14-40 | 14-30 | | 5-14 |
| No. 200 | 5-15 | 5-15 | | 3-5 |

Note: Ballast shall not be placed directly on capping sand.
[a]Capping sand shall be covered by 280-g/m$^2$ nonwoven needlepunched geotextile.
[b]Filter sand should, preferably, be covered by open-graded subballast or 280-g/m$^2$ nonwoven needlepunched geotextile on heavy-tonnage lines.
[c]Open-graded subballast to be used only on filter sand or on nonwoven needlepunched geotextile.




- CAPPING SAND TO MEET ASTM D1241 AND BE CONTINUOUSLY GRADED WITHIN GRADING LIMITS D, E OR F OF D1241.
- SUBBALLAST IF USED WITHOUT CAPPING SAND SHALL MEET A.R.E.A. Ch. 1, ART. 2.10. - WHERE CAPPING SAND COVERED BY A GEOTEXTILE IS USED, SUBBALLAST GRADING SHOULD BE VOID OF SIZES LESS THAN 3mm.
- IN HIGH FREEZING INDEX LOCATIONS REQUIRED GRANULAR COVER MAY BE REDUCED 15 THICKNESSES FOR EACH THICKNESS OF STYROFOAM HI-60 OR SIMILAR. STYROFOAM TO EXTEND 1300mm BEYOND TIE ENDS AND TO BE PLACED ON GEOTEXTILE THAT IS ON 100mm SUB-LAYER. STYROFOAM MAY NOT BE USED TO REDUCE GRANULAR COVER REQUIRED DUE TO WEAK SUBGRADE.
- MINIMUM FROST PROTECTION GRANULAR COVER TO BE 50% OF FROST DEPTH GIVEN BY AVERAGE YEARLY FREEZING INDEX.
- BALLAST - HARD MINERAL - TOUGH - LOW AGGREGATE INDEX. GRADING A.R.E.A. No. 4 OR 24

**FIGURE 9 Typical track cross section for new double-track construction (*20*).**

ballast depth is minimal for high-quality track subject to high-production tampers. Similarly, as dynamic loading increases with vehicle speed, minimum subballast depths, on new construction, of 300 mm are recommended.

## GEOTEXTILES FOR RAILWAY TRACK APPLICATION

Extensive research and findings have previously been reported on the use of geotextiles in railway track applications (*20-22*). Details of recommendations for use of geotextiles and a user specification can be found elsewhere (*20*). By far the most important aspect of geotextile use is good drainage because subgrade pumping requires a water source.

## BALLAST SPECIFICATION

The detailed requirements of ballast for 125-ton cars are likely to change little from those for 100-ton cars. Heavier loading may, however, tempt the track engineer to consider the use of concrete crossties, which require careful ballast selection. Because concrete is made from silica sands, the majority of minerals of which are harder than Mohs' hardness scale of 5, the predominant minerals in the aggregate source used for the ballast with concrete ties should have Mohs' hardness values of 5 or greater. Otherwise, the silica sand minerals making up the surface of the concrete tie are likely to powder the ballast by abrasion and scratching. Further details on ballast selection are given elsewhere (*23*).

## TRACK DRAINAGE

As stated earlier, good track drainage is essential to subgrade longevity, particularly in wet climates. Both subgrade strength and the minimizing of particle migration are facilitated by good drainage practices. Such practices include (*a*) adequate side-ditch drainage to deal with surface water, (*b*) lowering the groundwater to increase subgrade strength, and (*c*) internal drainage or cross-fall sloping of subgrade and subballast surfaces to prevent water seeping into the subgrade load-bearing area. Internal track drainage is by far the most difficult improvement to ensure on rehabilitation work; however, on new construction, both the subgrade and subballast layers should be constructed with a 5 percent slope as illustrated in the typical cross section shown in Figure 9 for a two-track line.

In any corrective work involving the use of subballast or geotextiles, proper and adequate drainage must be included in the planned maintenance. The elevation of the discharge level must be lower than that of the track-bearing area of the undercut surface. Although granular material for track support may be required to reduce subgrade stresses to an acceptable level, geotextiles within the track structure should be selected only on the basis of their handling strength requirements and their ability to separate, filter, and otherwise facilitate drainage. Of particular importance is the geotextile's ability to facilitate drainage by transmitting water within its plane. Incorrectly installed geotextiles will facilitate drainage into the load-bearing area of the track. Their discharge elevations must be lower than the undercut elevation directly below the track.

In rehabilitation work using undercutters, attention to drainage is unfortunately sometimes neglected (*22*). Special attention should be given to the undercutting of long lengths of track where a "canal effect" is often produced. If, after or before undercutting, the shoulder ballast is not removed and cleaned, drainage cannot occur from the load-bearing area to the side ditches. Where possible, the shoulder undercut should be deeper than the track undercut. On flat and marshy land, French drains may be required. Similarly, short lengths of undercut track such as grade crossings that are not subsequently, or during undercutting, provided with drainage will result in a "bathtub effect." Grade crossings also suffer from lack of drainage along the width of the highway. They should be provided with French drains through the crossing that discharge into side ditches beyond the crossing's limits. Water that is trapped within the load-bearing areas of the track can be expected to provoke or stimulate pumping along with the possibility of frost heave during cold weather.

Where French drains include pipes, the pipes should be laid at gradients that are self-cleaning. Table 3 gives the values of minimum gradients for various pipe sizes so that they self-clean when flowing half full and transmit water at a velocity of 600 mm/sec when full.

TABLE 3 MINIMUM GRADIENTS FOR SEWERS TO BE SELF-CLEANING

| Diameter of Pipe (mm) | Gradient (in.) |
|---|---|
| 150 | 240 |
| 175 | 295 |
| 225 | 415 |
| 300 | 610 |
| 375 | 820 |
| 450 | 1,050 |
| 525 | 1,300 |
| 600 | 1,500 |

## CONCLUSIONS

The main points concerning the performance of railway track subgrades have been outlined and the following conclusions reached:

1. On the basis of theoretical suggestions previously made by the writer (*1*), it is concluded that the ballast raises shown in Figure 2 are the maximum necessary to maintain subgrade stresses below 36-tonne static axles at values less than those below 30-tonne static axles.

2. Conclusion 1 also applies to dynamic considerations provided the maintenance limits associated with wheel or track irregularities are maintained.

3. Where factors of safety are presently safer than necessary for 30-tonne axles, the theory will allow an assessment of the necessity of using a ballast raise. Such an assessment must be done on a case-by-case basis.

4. On new construction, adequate subgrade compaction assures three main factors that are desirable in good subgrade and track performance: (*a*) a decrease in the susceptibility of the soil to settlement, (*b*) a decrease in the permeability of the compacted subgrade soil, and (*c*) an increase in the soil's supporting power (e.g., CBR).

5. On new construction where granular cover exceeds 305 mm ballast plus 150 mm subballast, a makeup depth of stabilized-soil material may be used. This depth should be considered equivalent to a similar depth of unbound material because of the extensive cracking that Figure 6 predicts.

6. Great care must be taken when using soil stabilization if the treated soil is high in sulfates (*11, 12*). Both good drainage practice and the use of a seal coat are recommended.

7. A graded granular subballast that will act as a filter to the subgrade is an essential requirement above treated-soil subgrades and other fine-grained subgrades. Alternative subballast possibilities are given in Table 2.

8. Geotextiles and lime slurry pressure injection have been shown to be viable subgrade improvement techniques on existing track.

9. Good drainage practice is essential for subgrade longevity. Good drainage practice also minimizes the probability of fines separating and pumping from the subballast.

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# Track Components for 125-Ton Cars

JOHN BUEKETT, DEREK FIRTH, AND JOHN R. SURTEES

**Each track component is considered in turn and its engineering development is reviewed. Maintenance equipment is similarly reviewed. Ties and fasteners are identified as items to which an engineered approach could produce significant advances in cost-effective track. A detailed description is then given of the dynamic behavior of ties and how it is influenced by impact loading from equipment. Development of tie design from in-track measurements and the benefits of engineered tie pads are also discussed. Finally, suggestions are given for track for 125-ton cars, and the need to control damage caused by defective wheels is identified.**

Since the first transcontinental railroad was constructed 120 years ago, developments in mechanical equipment and signaling have been dramatic, and today's equipment would be virtually unrecognizable in scale, power, and complexity to a mechanical engineer of the 1860s. Track, however, although stronger, looks quite similar to that installed by the civil engineer of 120 years ago. Competition is driving railroads to maximize the tonnage carried on each track, and part of the increase in tonnage carried has been achieved by a steady increase in car gross weights. Compared with the 70-ton car, the 100-ton car has been shown to reduce rail life by 40 percent in tangent track (*1*). Now that the 125-ton car with 39-ton axles is becoming a reality, the need for more thoroughly engineered track is greater than ever before.

## TRACK MATERIALS

### Rail

Modern rail sections have ample strength to span between ties at 19-in. spacing (wood) or 24-in. spacing (concrete), but increasing contact stresses (Table 1) have led to a situation in which railhead fatigue is the factor that controls rail life in heavy-haul tangent track (*2*). Wear and rail stability, however, remain the dominant problems in curved track.

Developments in lubrication, especially on-board lubricators, should reduce side cut in tight curves, but rail will always wear and its rapid change out or transposition, without damage to the ties, is an important requirement that must influence the design and selection of fasteners.

Rail wear and the risk of derailment are influenced by rail stability under dynamic loading, which is dependent on the ties as well as the fasteners.

Understanding of the mechanisms of the development of railhead defects is still inadequate despite major research efforts on this subject. The scale of research on rail defects reflects the place of rail as the most costly track component (Table 2).

TABLE 1 WHEEL-TO-RAIL CONTACT STRESSES (*1*)

| Vehicle Type | Wheel Diameter (in.) | Wheel Load (1,000 lb) | Avg Contact Stress (ksi) |
|---|---|---|---|
| 125-ton car | 38 | 39 | 148 |
| 90-ton car | 36 | 31.9 | 140 |
| 70-ton car | 33 | 26.3 | 136 |
| 4-axle locomotive | 40 | 32.5 | 138 |
| 6-axle locomotive | 40 | 32.5 | 138 |

Progress in reducing the incidence of railhead defects is relatively slow, perhaps because the significant contribution of ballast, ties, and fasteners to rail performance is neglected. Study of the total system can lead to some interesting findings; for example, fully compacted ballast improves the damping of ties and this can significantly reduce railhead contact stresses (*3*).

### Ties

Until quite recently, manufactured ties were used only in response to specific problems such as timber shortages (especially in wartime), termite attack, and desiccation in desert conditions. The ties that proved successful in overcoming these problems were, in chronological order, steel, twin-block concrete, and monoblock prestressed concrete. Of these, only monoblock prestressed concrete has a record of successful large-scale use in heavy-haul track.

Monoblock prestressed concrete ties were fully described by White in 1984 (*4*). At that time, the main North American users were Florida East Coast Railway, Amtrak, and Canadian National (CN) Rail. Since then, CN Rail has continued its program and now has more than 2.5 million concrete ties in track. The most recent user is Burlington Northern Railroad who plan to install 3.5 million concrete ties by 1992.

In the last few years there have been major advances in understanding the dynamic behavior of concrete ties. The important attributes of concrete ties in this context must be a record of successful use in arduous conditions and the potential for engineered design. This is considered in more detail in a later section of this paper.

Concrete-slab track received considerable attention in the

CXT Systems, 13873 Park Center Road, Suite 327, Herndon, Va. 22071.

TABLE 2 TYPICAL COSTS FOR TRACK MATERIALS

| Item | Concrete-Tie Track | Wood-Tie Track |
|---|---|---|
| Rail (132-lb carbon) (%) | 42.3 | 43.4 |
| Fasteners (%) | 11.2 | 16.9 |
| Ties (%) | 32.0 | 25.6 |
| Ballast (%) | 14.5 | 14.1 |
| Total ($000/mi) | 247.5 | 240.9 |

1970s and now has a limited but important role in specialized applications such as tunnels and elevated high-speed track.

**Fasteners**

The cut spike helps to destroy wood ties but, despite this, is still the predominant means of securing rails to ties. In the last 150 years the only major development of the cut spike fastener has been the introduction of tie plates.

All of the modern elastic fasteners can be used on tie plates, but they are then only as good as the method of securing the tie plate to the tie (usually cut spikes) and the ability of the tie to resist plate cutting.

With concrete ties, there is an opportunity to match an engineered tie to an engineered fastener with only a tie pad to separate the rail from the tie. Tie pads have a vital role in the satisfactory performance of the system, and this is considered in more detail later.

The favored elastic fasteners for heavy-haul use in North America are shown in Figure 1; they all avoid screwed components, which are labor intensive during installation and maintenance. When used with concrete ties, the clip housing (shoulder) is permanently cast into the tie and must therefore be of a design that has a life expectancy similar to that of the tie. In particular, it must survive dragging equipment and rails dropped onto it during installation.

Fasteners are generally specified by means of static or low-frequency parameters (*5*), which can be readily measured in the laboratory. However, because fasteners have to work in a high-frequency dynamic environment, experience with their performance in use is most important.

Development of design is largely empirical, but the introduction of measuring devices such as the Dichroic Displacement Measuring System (*6*) means that behavior in track can be quantified. Thus laboratory testing can be made to more closely reproduce service conditions; this has already been done with the Battelle impact test rig (*7*), which can be used to measure the relative efficacy of tie pads in attenuating impact loads resulting from wheel defects.

Rail anchors are a fastener component, but they should not be required with proven concrete tie fasteners.

An important consideration in fastener design is machine insertion, and progress is being made with clip insertion. At present, insulator placing remains a hand operation, but initial pad placing can be avoided by having tie pads glued to the ties before delivery, using an adhesive that will permit them to be peeled off later.

The tie pad and insulator are wearing items, but their

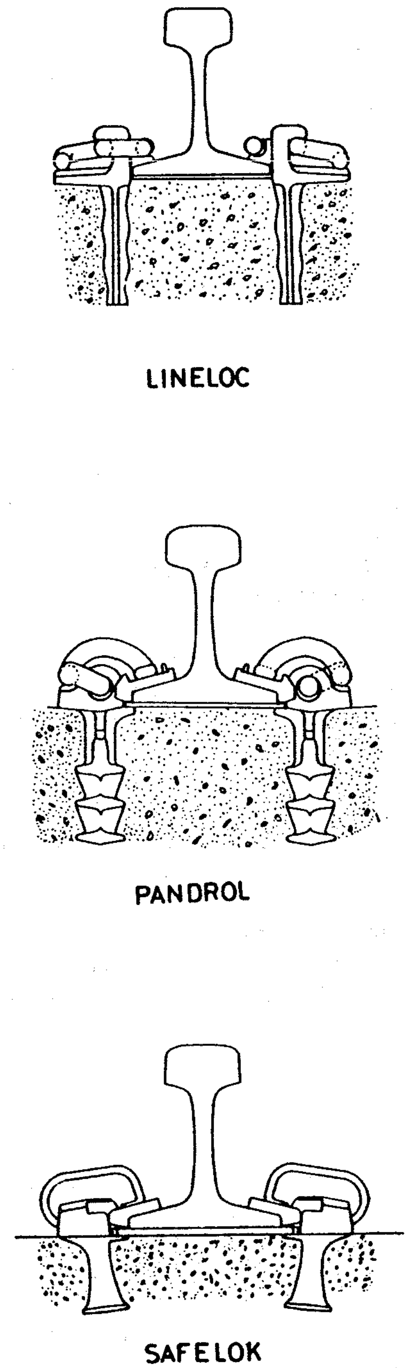


**FIGURE 1 Rail fasteners for concrete ties.**

integrity should be maintained for one rail life or a whole-number multiple of rail life.

### Ballast

The functions of ballast in 125-ton car territory are similar to those in all track. That is, ballast distributes over the subgrade the loads from the ties, especially dynamic loads; holds the ties and prevents lateral, longitudinal, and vertical movement; drains away water; and facilitates lining and leveling of the track.

Only the better types of ballast will continue to perform these functions under the arduous conditions created by intensive high axle-load traffic. Important considerations are particle shape, grading, and individual particle integrity when subject to loading in wet, dry, and freezing conditions.

It is well known that concrete ties impose greater stresses in the ballast. This necessitates particularly careful selection of ballast type and grading for use with concrete ties. Typically, crushed igneous or metamorphic rock such as granite, traprock, and quartzite is suitable.

### Subgrade

Two more recent engineering techniques are the use of geotextiles and cement stabilization, which increases the wet and dry strength of the subgrade. These are particularly useful at the surface of clay subgrades where water does not drain readily and mud pumping may develop.

Geotextiles are used over a blanket of free-draining fine material such as sand or selected crusher run fines (*8*). It is important that the profile of the sand blanket permit water to run away and that the sand itself be free draining.

Mud pumping can occur because of breakdown of ballast particles especially when limestone ballast is used, but the more insidious occurrences are due to ties punching into the subgrade. Under these conditions, the best quality ballast can quickly be rendered almost useless. Concrete ties are more likely to give rise to mud pumping than are wood ties. Where the risk is present, insertion of a geotextile and sand blanket can easily be carried out on discrete lengths of track during reasonable work blocks. Cement stabilization is a technique more suited to new construction or reconstruction.

### Turnouts

With the advent of continuously welded rail (CWR), the fixed frog is now the major discontinuity in the track. Because of the high impact loading at this discontinuity, the frog itself and the supporting ties and ballast require a disproportionate amount of maintenance.

Swing nose frogs have the potential to greatly reduce this discontinuity. They have been developed in Britain, France, Germany, and Japan in a variety of forms ranging from fully fabricated to fully cast (*9*).

Prestressed concrete switch ties, now widely used in Europe and Australia, are also finding acceptance with installations on CN Rail, Florida East Coast Railway, Amtrak, Burlington Northern Railroad, and Conrail. They offer three basic advantages:

- Reduced rail wear arising from higher track modulus and better fasteners that reduce rail movement;
- Supplied as a kit with fasteners cast in at calculated positions, thus eliminating the need for preassembly; and
- Consistent quality and extended life compared with wood.

Design of switch ties is based on plain line ties with constant cross section. Tests in track by CN Rail have confirmed that in their installation the concrete stresses are within acceptable levels (*10*).

## DYNAMIC BEHAVIOR OF CONCRETE TIES

Early work was done by the Talbot Committee (*11*), which studied the behavior of rail, wood ties, ballast, and road bed. A considerable study of in-track loads was also undertaken when prestressed concrete ties were developed in Britain in the 1940s (*12, 13*). The findings were used as a basis for establishing ballast pressure distribution and tie design. Axle loads were complemented by an impact factor to arrive at measured loads.

In the past decade, a greater understanding of the behavior of concrete ties has been achieved. Because loading arises from moving vehicles it is cyclic; even with a perfect wheel on a perfect rail, the duration of load is so short and varies so rapidly that it has a considerable dynamic component. Superimposed on this are the real-life imperfections such as shelled, flat, and out-of-round wheels; rail corrugations; dipped welds; engine burns; and shells. All of these cause dynamic loading of varying frequency and magnitude, which excites the ties at their natural frequencies.

In the first mode, which occurs at about 125 Hz, the ties tend to bounce on the ballast and there is little dynamic bending (*14*). Also, at frequencies below about 200 Hz, well-compacted ballast provides significant attenuation of tie vibration (*3*). The first mode is therefore not particularly significant when the ties are properly supported and there is sufficient quasi-static load to keep the ties in contact with the ballast. In the second mode, the rail seat is close to a node and the dynamic bending is not critical, but in the third mode, which occurs at about 650 Hz, the dynamic bending alternately augments and counteracts the rail seat quasi-static bending (Figure 2). It is this third mode that is generally considered most critical for dynamic strains in the rail seat of ties although significant effects are observed up to 2000 Hz.

When the quasi-static strain is augmented by the dynamic strain, the tie behaves as though the quasi-static load has been increased, hence, the concept of an impact factor applied to the quasi-static load. When the wheel load is relatively small and the speed is high (e.g., empty cars), tie vibration can result in reverse bending that will create a tensile strain in the top of the rail seat of the tie (*15*).

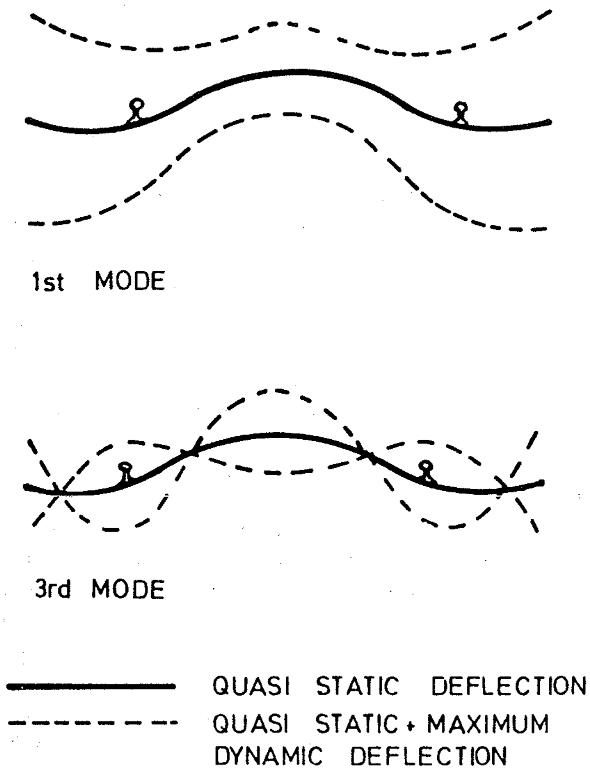


**FIGURE 2 Interaction of tie resonance and quasistatic bending (*14*).**

The amplitude of vibration will depend on the magnitude of the impact, the decoupling provided by the rail pad, and the damping provided by the ballast. Under a high quasi-static load the tie is more likely to be well damped by the ballast than when lightly loaded. Hence, tie strains are not necessarily related to wheel load, and defective lightly loaded wheels, which tend to travel faster, can be quite damaging.

Although the strain induced in the rail by wheel defects is not directly related to tie strains, some data from rail web strain measurements are shown in Figure 3 to demonstrate the relation between speed and dynamic loading. The speed at which the effect of wheel defects becomes significant is 20 to 30 mph, and it is clear that some wheel defects have an increasing effect with speed whereas others are less speed dependent.

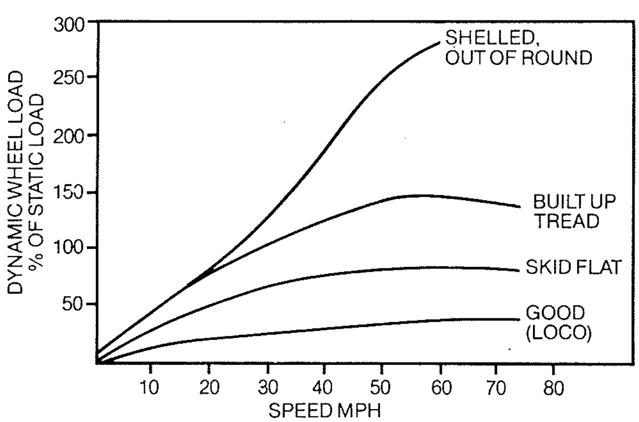


**FIGURE 3 Effect of speed on dynamic load from imperfect wheels (*10*).**

Resonance of the ties will also have an effect on the positive and negative strain at the tie center. When the tie is first installed with no center support, there is a positive bending moment, but as the tie beds into the ballast, this changes to a center negative moment. However, empirical observation is that distress at the tie center is rare, and, when it does occur, it can invariably be related to ties being center bound at some time.

As discussed previously, well-compacted ballast contributes significantly to tie damping. Tamping is effective in restoring track line and level, but it disturbs the ballast and hence reduces damping. An alternative method of restoring track level by pneumatic stone injection (*16*) is therefore attractive because it causes much less disturbance of the ballast once it has been consolidated.

## TIE PADS

Tie pads have the vital role of separating the rail from the concrete tie; apart from its contributions to load distribution and insulation, the pad is vital to the prevention of abrasion of the tie by the rail foot. Tie pads are typically 0.20 in. thick, selected on the basis of their ability to remain in place and not disintegrate or wear out before rail change out, and initial cost.

The importance of tie pads in protecting ties from severe impact loads has been recognized for many years, but consideration of how the contribution of the pad could be assessed was limited to comparison of static spring rates. As part of their investigations of cracked ties in the Northeast Corridor, Dean et al. (*17*) developed the Battelle impact test that has since been refined to incorporate a quasi-static load and more realistic support conditions (*18*). This test is useful in comparing pad attenuation properties, but it must be complemented by other investigations. These include general durability, especially resistance to abrasion; resistance to degradation; and the ability to undergo high-frequency repeated loading without change in elastic properties. It is difficult to measure the dynamic spring rate of pads in the laboratory and, in the absence of suitable tests for the other properties, recourse has to be made to in-track testing.

The effect of temperature on pad performance can be significant because many pad materials are thermoplastic. High-density polyethylene (HDPE) pads, which are satisfactory in South Africa, become brittle under North American winter conditions, but ethyl vinyl acetate (EVA) is satisfactory for North American temperatures.

It is recognized that pads thicker than 0.2 in. made from materials with a low dynamic spring rate, such as natural rubber, are most effective in attenuating impact loads. A profiled surface also contributes significantly to performance (*18*). An optimum thickness of about 3/8 in. has been adopted by Japanese Railways, SNCF (France), and British Rail. This is a compromise among maximum impact attenuation, retention of rail stability, and cost. Not only is the cost increased by the thickness, but the materials are also inherently more costly than EVA. The increased cost can, however, be justified because EVA does not provide any significant impact attenuation.

HDPE and EVA pads predominate on heavy-haul railways and, in general, their durability is good. They provide good electrical insulation and they prevent the rail from abrading the tie. Consideration has therefore been given to increasing their thickness and using a profiled surface, but, because of the inherently high dynamic stiffness of such materials, there is no benefit.

It is notable that the railroads that use high-attenuation pads tend to be those that operate high-speed passenger networks. This is because impact loading from certain wheel and rail defects is speed dependent rather than proportional to static wheel load. However, there are potential benefits to heavy-haul railroads at speeds in excess of about 30 mph. CN Rail is carrying out tests with high-attenuation pads on tangent track where speeds are fairly high and may increase. As yet, there are no reports on in-track tests of such pads in heavy-haul curved territory.

The use of thicker pads raises the logistical problem of how to ensure that the correct pad thickness is used with each fastener assembly. One approach would be for each railroad to specify only one pad thickness and to vary the material depending on speed, curvature, and type of traffic.

## DESIGN AND TESTING OF PRESTRESSED CONCRETE TIES

The AREA specification for concrete ties, which is probably the most comprehensive available worldwide, gives resistance moment requirements and tests to confirm compliance with these requirements. It is based on static calculations augmented by impact and distribution factors. Currently, the impact factor is 150 percent, which is also fairly typical for offshore railroads (*19*).

Specified bending moments have been increased several times as a result of observed in-track performance. More recently, the resistance moments specified for ties at 30-in. spacing have been adopted for ties at 24-in. spacing. The result is that ties designed to these requirements are generally larger in cross section than are ties used anywhere else in the world (Table 3).

The AREA approach to design assumes that the effect of impact loading is proportional to the quasi-static load and hence an impact factor is applied. Although this is conventional and convenient, it does not represent the behavior described in the section on dynamic behavior of concrete ties. Converting the dynamic strain in a tie to an equivalent static load or bending moment does produce figures that the civil engineer can use. However, it does not take into consideration that, within the range of values for stiffness normally encountered, resonance of ties is a strain-inducing phenomenon that is not resisted by increase in the resistance moment of the tie. Tie strain capacity is more important—a point not addressed by the specification except for a requirement for a minimum prestress of 150 psi at any point in the rail seat cross section.

Another assumption that may not be valid is the direct relation between tie spacing and load carried by a single tie. Within the limited range of likely tie spacings, say 20 to 30 in., it is unlikely that direct proportionality applies. Certainly dynamic loading is unlikely to be sensitive to tie spacing.

The present state of the art is that ties somewhat smaller and with lower resistance moments than designs based on the AREA specification are performing well in heavy-haul railroads in Australia, Canada, and South Africa. However, these designs do have a common feature of relatively high extreme fiber prestress at the point of maximum strain (Table 4).

Fatigue of railroad bridges is a well-investigated subject; ties on bridges are probably exposed to an even more demanding fatigue environment but have received less attention. What consideration has been given relates mainly to prestressing tendons for which theoretical studies are well established. In practice, fatigue of tendons is not a problem in countries such as Britain and South Africa with a long experience with pretensioned concrete ties.

Concrete fatigue must be considered because the concrete tie is subject to significant stress changes up to about 2000 Hz. This is much higher than occurs in other prestressed concrete units, and there are no published data on the fatigue behavior of concrete under such conditions. One approach is to design

TABLE 3 COMPARISON OF DIMENSIONS OF TIES DESIGNED TO AREA REQUIREMENTS WITH THOSE USED ELSEWHERE

| Tie Type | Length (in.) | Base Width (in.) | Height at Rail Seat (in.) | Approximate Weight (lb) |
|---|---|---|---|---|
| RT7SS2 (NE Corridor) | 102 | 11 | 9 1/2 | 780 |
| CC365 (design to AREA Chapter 10) | 102 | 11 | 9 3/8 | 780 |
| CN60B (CN Rail) | | | | |
| BN100 (BN Rail) | 99 | 10 3/8 | 8 | 630 |
| Hammersley iron | 102 | 10 3/8 | 8 1/8 | 650 |
| F27AS (British Rail) | 99 | 10 3/8 | 8 | 640 |
| B70 (Deutsche Bundesbahn) | 102 | 11 3/4 | 8 | 600 |

TABLE 4 COMPARISON OF PROPERTIES OF TIES

| Tie Type | Prestress at Unloaded Rail Seat (psi) | | Resistance Moments (in.-kips) | | |
|---|---|---|---|---|---|
| | | | Rail Seat | | |
| | Top | Bottom | Positive | Negative | Center Negative |
| RT7SS2 | 550 | 1,180 | 310 | 200 | 210 |
| CC365 | 400 | 1,630 | 360 | 170 | 200 |
| CN60B | 630 | 1,750 | 260 | 140 | 160 |
| Hammersley Iron | 130 | 2,190 | 320 | 90 | 120 |
| F27AS | 750 | 1,760 | 270 | 150 | 170 |

on the basis that the characteristic loading shall induce no tension in the concrete and that known abnormal loads do not exceed an assumed fatigue limit for tensile stress of, say, 50 percent of ultimate.

The AREA specification includes a rail seat repeated-loading test on ties, but this test requires the tie to first be cracked by a static load so that it is really a check on the fatigue behavior of the prestressing tendons and their bond to the concrete. These properties can be more simply examined in other ways.

Scott (*20*) has applied the test to uncracked ties and obtained useful data on the incidence and propagation of microcracks. The Battelle impact test provides the basis of a test to measure the response of a tie to impact loading. By combining impact and static loading and measuring the strain in the tie, useful data can be obtained.

For design development work, in-track testing, despite the difficulties involved, is really unavoidable. In the past, visual observation of the ability to survive has been the main criterion, but with modern electronic data logging and strain measurement and computer analysis equipment, significant information can be obtained from quite short periods in track.

The prestressed concrete tie is a unique concrete unit in that it has the maximum load applied close to the end. This demands much more rigorous process quality control and finished product testing than are normally applied to pre-tensioned concrete products.

## TRACK MAINTENANCE

Improved material components may have higher initial cost, but the cost of materials is typically only about 25 percent of total maintenance costs. There is great potential for using more durable, albeit more expensive, components and installing them more efficiently. Accurate cost data and analysis can help to justify this development (*21*).

Concrete ties will not perform satisfactorily if interspersed in wood-tie track because their higher modulus and more secure fasteners mean that they receive a disproportionate part of the load. Therefore they must be installed out of face.

The modern track renewal machines are an extremely efficient means of tie and rail replacement. When combined with thorough undercutting, the renewed track is much higher quality than can be achieved by traditional spot replacement.

## EFFECT OF EQUIPMENT

In previous sections, emphasis has been placed on the damage or, more correctly, rapid wear caused by impact loading from equipment.

Unevenness (wheel burns, shelling, corrugations, etc.) and discontinuities in the railhead will result in accelerated wear. The rail joint, which is fortunately becoming a thing of the past, is the classic example of the wear and tear resulting from impact loading. However, such rail-induced impacts are discrete whereas the impact loading from equipment (e.g., wheel irregularities) is continuous over the track traversed.

Railhead defects are partly induced by high contact stresses as well as wheel irregularities, and the effect of wheel irregularities can be exacerbated by suspension design (*22*). It has been observed that the highest stresses in concrete ties are caused by out-of-round wheels that are not readily detected by conventional inspection of wheels. High tie stresses will also be accompanied by high rail stresses, and there must be a reaction in the equipment wheel bearings. The emphasis that has been placed on impact effects must not completely overshadow the effect of equipment on quasi-static loads. Scott (*23*) found that, in a specific case of rapid track deterioration, locomotive wheel configuration, car wheel bearing design, and wheel profile all had significant effects. There is thus a good case for improvement in equipment design and maintenance for the sake of decreasing wear and tear on the equipment as well as the track.

To find the cars that have a potentially destructive effect on track, a wheel flat detector has been developed (*17*) that measures strain in the web of the rail. Scott (*10*) has estimated that 0.75 to 1.0 percent of wheels have defects potentially damaging to track, so the incidence of this problem is enormous.

## CONCLUSIONS

In this selective discussion of track components, those items for which advances in design can readily be made have been identified. A particular case is the use of concrete ties and elastic fasteners to replace the cut spike and rail anchor. Such a change would also require out-of-face track renewal, which would justify the use of more sophisticated and efficient track maintenance equipment.

These potential changes emphasize the need to consider changing from track that has relatively low initial cost and high maintenance to track that has slightly higher initial investment, which pays off in reduced maintenance and less downtime. This change was effectively applied to locomotives 40 years ago, but the equipment designer needs to take more account of the effect of locomotives and cars on track deterioration.

Improved train control methods may well permit more trains on each track with the resulting need for more track maintenance and less time to do it. Better-engineered track is necessary to enable the roadmaster to live with this development.

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# Railroad Track Structure Performance Under Wheel Impact Loading

HAROLD D. HARRISON AND DONALD R. AHLBECK

**Increased use of high-productivity railcars of 100- and 125-ton capacity has pushed current track structure designs to the limit. A number of heavy-haul railroads have turned to concrete-tie track to resist the extreme load environment. Two important factors must be addressed in the use of concrete-tie track. First, the higher axle loads result in a tighter "load tolerance" that requires closer control of wheel- or rail-induced impact loading. Second, the dynamic response of concrete ties and fastener systems to impact loading is quite different from that of traditional wood-tie track. In this paper, results of recent studies of concrete-tie track structural dynamics are discussed in the context of wheel and rail impact loading. These studies include a correlation between the experiments at the Facility for Accelerated Service Testing and revenue traffic load environments, an investigation of Northeast Corridor concrete-tie cracking problems, and recent work on extreme-value wheel loads due to freight car wheel profile geometries. The need for dynamic systems analysis in the design of track structures is emphasized by examples of tie, fastener, and insert response to impact loading.**

The introduction over the past two decades of high-productivity freight cars of 100- and 125-ton capacity in unit train operations has pushed current wood-tie track structure designs to the limit. Some heavy-haul railroads have resorted to concrete-tie track to help resist the severe load environment. Concrete-tie track has now been installed on major portions of the high-speed Northeast Corridor (NEC) trackage; and a few North American freight railroads have installed substantial mileage in concrete ties. The merits of concrete-tie track are slowly gaining recognition and acceptance.

One important aspect of the design and use of concrete-tie track is the structural response to high dynamic loading due to irregularities in wheel and rail geometry. Damage due to rail joints and irregular weld surfaces has long been recognized by the heavy-haul railroads. Similar damage due to wheel profile irregularities (wheel flats, tread buildup, spalls, runout) has now been identified and quantified, both by computer prediction and actual measurement.

In this paper, the effects of vertical wheel impact loads on track structural response, particularly of concrete-tie track, are examined in the context of track component failure mechanisms.

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## BACKGROUND

Recent studies (*1,2*) sponsored by the U. S. Department of Transportation's Federal Railroad Administration (FRA) have shown that damage, to both vehicle and track structures, can be caused by wheel tread irregularities that cannot be identified easily by traditional visual inspection techniques. These studies began in 1980 as an investigation of the causes of rail seat cracks in concrete crossties recently installed on the NEC high-speed track. At the same time, Amtrak noted problems with bearing cage and "hotbox" (hot bearing) failures on its older passenger cars. Wheel load data from instrumented track were used to trace these problems to longer-wavelength wheel profile "runout" geometry errors, primarily on these older cars (*3*).

A Wheel Impact Load Detector (WILD) was designed, fabricated, and installed on the NEC track (*4*) to measure peak wheel loads under each passing wheelset and to identify wheels causing dynamic loads in excess of preset limits. A number of these wheelsets were identified and removed from service by Amtrak and assembled in a test train. A series of test runs was made over an instrumented track section to gather impact load data on these specific wheels. Circumferential profiles were then measured and used as inputs to a vehicle-track interaction model to provide a direct comparison between predicted and measured load time-histories (*5*). An example of this comparison is given in Figure 1.

In addition to the wheel load and geometry measurements, extensive dynamic measurements of track component response to impulse loading were made, both in the laboratory and in the NEC track (*3,6*), for a wide range of loading conditions and for different component combinations. The results of these tests were used to evaluate the relationship between track component dynamic behavior and the observed problems of tie cracking, rail clip movement, and shoulder insert loosening. The results showed that improved track performance can be achieved by designing and combining track components in the context of their dynamic interactions—and, of course, by proper maintenance of the wheel and rail running surface conditions, which are the primary sources of high-impact loads.

## LOAD ENVIRONMENT

The structural integrity of a vehicle or track component can be influenced strongly by the dynamic response to wheel-rail

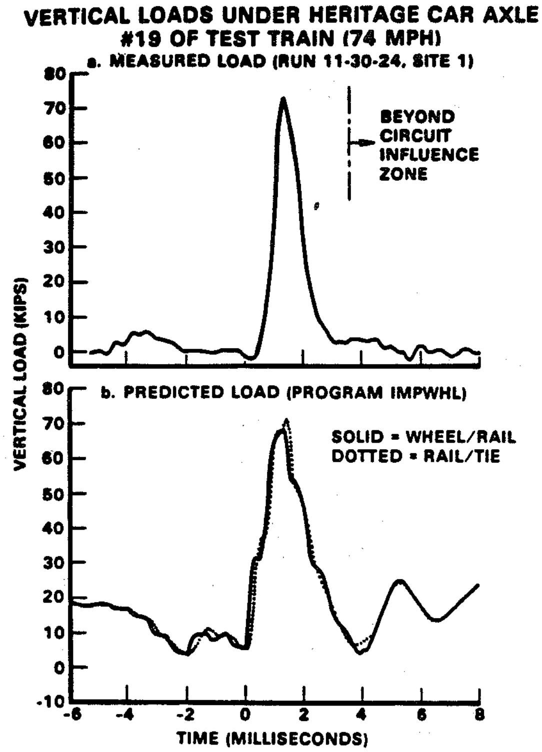


**FIGURE 1 Comparison of predicted and measured load time-histories for Heritage car wheel tread anomaly.**

impact loads. Consequently, a static evaluation of a component in general is incomplete and in many cases misleading. The first step in a dynamic evaluation is therefore the definition of the service load environment. This definition must include not only a statistical description of the load levels but also a descriptor of the frequency content of the load pulse.

Data from several major field measurement programs conducted over the past decade have provided statistical descriptions of the vertical and lateral wheel-rail load environment. In the early measurement programs on wood-tie track (*7*), data were analyzed within a 300-Hz bandwidth, which was deemed sufficient at that time to handle flat-wheel impact load events. Results from the concrete-tie cracking investigation, however, showed the importance of the higher-frequency (short-duration) impulse loads to which these ties and fasteners could respond. Data from this investigation clearly showed low-probability, high-amplitude vertical load events when the data analysis bandwidth was increased beyond 1000 Hz. For example, the "nominal" and dynamic vertical wheel loads under the mixed traffic on NEC track are compared in Figure 2, a typical "percent-level exceeded" statistical amplitude plot.

In one of the measurement programs (*8*), a major objective was the comparison of the wheel load environment at the Facility for Accelerated Service Testing (FAST) at the Transportation Test Center, Pueblo, Colorado, with the revenue service load environment of similar "real world" concrete-tie track structures. Four revenue service sites were chosen to be instrumented and monitored over a period of time. As expected, the vertical load environments at FAST and at the four revenue service sites were all significantly different because of a combination of factors including type of traffic, train speeds, and wheel tread conditions. The comparison is shown in the cumulative probability of exceedance plots of Figure 3. Note that the FAST load

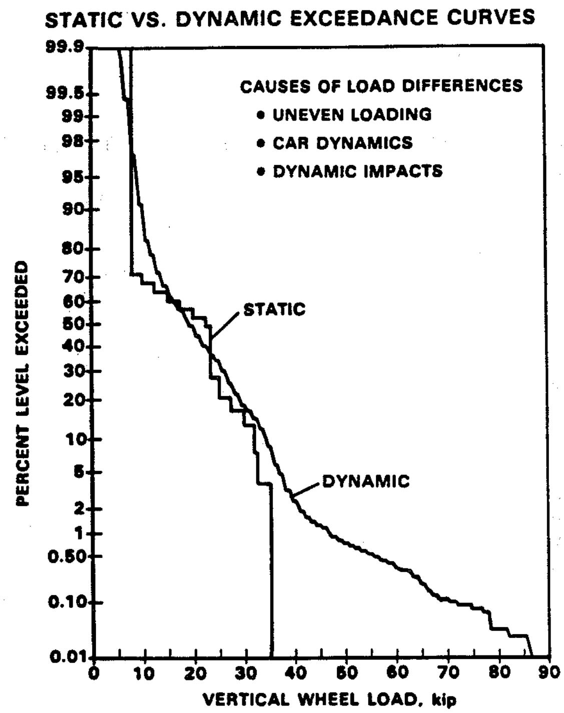


**FIGURE 2 Cumulative probability curves of static and dynamic vertical wheel loads on Northeast Corridor concrete-tie track (all traffic).**

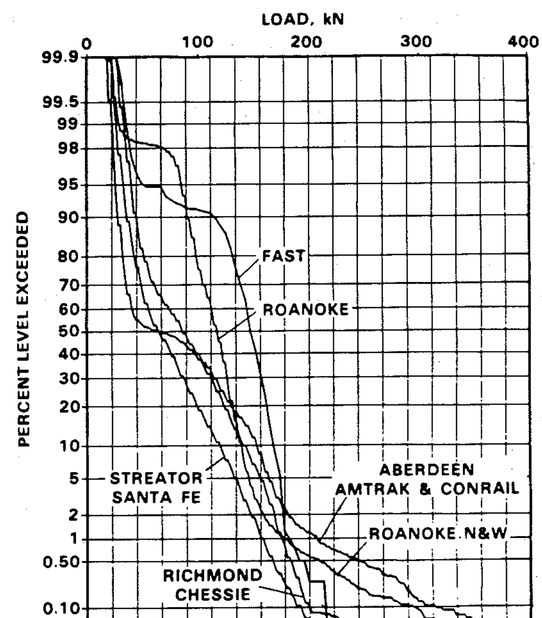


**FIGURE 3 Statistical presentation of vertical wheel-rail loads at five correlation study measurement sites.**

environment has the highest nominal wheel load, the 50 percent (median) level, because the trains consist primarily of loaded 100-ton cars. However, the plots are truncated at about 50,000 lb (222 kN) reflecting excellent (perhaps unrealistically good) wheel maintenance.

The development of the WILD has greatly enhanced the collection and monitoring of load environment data, particularly the low-probability, high-amplitude events most damaging to track structure. Examples of load statistics from the WILD are shown in Figure 4. Note that these extreme-value load events fall into a logarithmic distribution, which is similar to complex system failure statistics.

In addition to the amplitude statistics, the frequency content of the dynamic load is important in the definition of the load environment. The dynamic load is the vehicle-track interactive response to errors in geometry at the wheel-rail interface (neglecting for the moment other vehicle- and train-induced loads). Frequency content will therefore depend on shape of geometric error, wavelength, and passing speed. Response will also depend on the dynamic complexities of the vehicle (particularly the unsprung mass) and track structural components. Therefore the frequency description of the load environment tends to be quite site specific, in terms of the particular vehicle and particular track, and can require detailed analysis of each given case.

The importance of the frequency (as well as load amplitude) aspects of track dynamic response can be illustrated by recent track degradation experiments on the FAST track. Simulated "engine burn" geometry errors were ground into the rail surface in a section of NEC concrete ties. The largest of these divots produced impact loads under the FAST freight train greater than 90,000 lb (400 kN), sufficient (it was thought) to initiate tie cracking—determined from laboratory tests to be in the 75,000- to 90,000-lb (334- to 400-kN) range (*2*). However, track degradation was rather modest at 20 million gross tons (MGTs) of accumulated "traffic" — a few rail clips had fallen out and some ballast degradation had occurred, but no tie cracks or fastener insert failures were found. Why had this experiment "failed"? There are several possibilities:

- The frequency content of impact loads under the 40-mph (65-km/hr) FAST train was significantly different from that under 80- to 125-mph (130- to 200-km/hr) passenger trains, which was more closely matched to NEC track component frequencies;
- Symmetric (both-rail) geometry errors may have reduced the important asymmetric tie-bending response modes;
- An evolution in tie manufacturing techniques (different material mixes, cure times, etc.) may have increased tie strength; and
- Although "microcracks" were generated in the laboratory, substantially higher impact loads may be required under service conditions to create (and enlarge) "field" cracks.

Track dynamic response has long been assumed to be a primary factor in the generation of rail corrugations. Corrugation shapes artificially introduced on FAST rail surfaces have been observed to decrease in depth under accumulated tonnage, rather than grow, because of off-resonance frequency content.

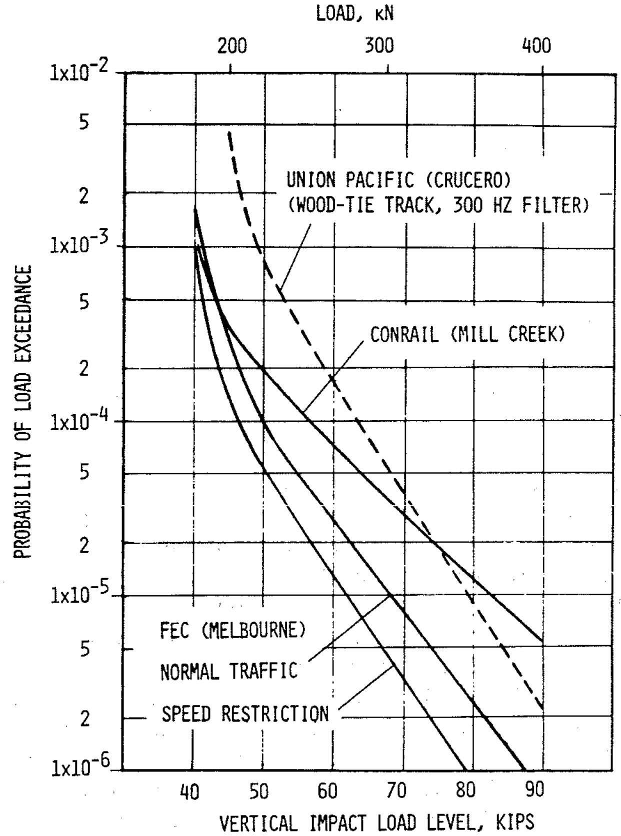


**FIGURE 4 Probability of occurrence of wheel impact loads under freight traffic on concrete-tie, continuously welded rail track.**

## WHEEL AND RAIL SURFACE GEOMETRY

Studies on the NEC proved the existence of long-wavelength (10- to 16-in.) runout geometry errors on older passenger cars. At that time, similar profiles had not been measured on freight equipment. Since then, however, these profiles have been found to be rather typical of flat wheels on freight cars. During 1984 three WILD systems were installed on the concrete-tie track of the Florida East Coast (FEC) Railway. In the first few months of operation, load exceedance reports from the WILD allowed the FEC to "flag" a number of wheelsets that were producing impact loads of more than 70,000 lb (311 kN). Several were generating loads in excess of 100,000 lb (445 kN). Few of these wheels could be rejected from interchange service under the current Association of American Railroad (AAR) Interchange Rules (Rule 41), although some showed signs of distress such as chain spalling, rim spreading, and thrown bearing grease.

With the assistance of FEC personnel, profiles were measured on both wheels of six wheelsets taken from 100-ton freight cars. A special profilometer was used to measure the effective change in rolling radius with distance around the circumference. The wheelset was rotated cradled in its own bearings while these measurements were made. Five of these tread profiles are shown in Figure 5. Profile 1 appears to be a typical wheel flat; similar profiles were measured on two of the other wheels. Although this flat was 0.037 in. (almost 1 mm) in depth, the 6-in. (152-mm) wavelength made visual detection difficult. Profiles 2 and 4 have the well-developed long-wavelength geometry errors typical of older passenger equipment. Profile 5 was measured on the wheel opposite Profile 4. The only other case in which significant error was measured on the opposite wheel was a flat 0.027 in. (0.7 mm) deep directly across from the 0.062-in. (1.6-mm) error of Profile 3.

These five tread profiles were used as direct vertical geometry inputs to the computer model, IMPWHL, to predict peak loads on both concrete- and wood-tie track structures. Some results are shown in Figures 6 and 7. Peak loads versus speed on concrete-tie track are plotted in Figure 6, where a strong speed dependence can be seen for these long-wavelength shapes. In Figure 7 peak loads for concrete- and wood-tie track are compared for three of these profiles. The more resilient wood-tie structure in general produces somewhat lower peak loads, although at some speeds Profile 4 generates higher peak loads on wood-tie track. Note that Profile 5 with its longer (20- to 25-in., 500- to 600-mm), smoother shape produces relatively low dynamic loads in spite of its 0.070-in. (1.8-mm) depth.

In addition to wheel tread imperfections, load-producing geometry errors are also found on the rail running surface—rail joints, battered welds, engine burns, and so forth. These cause repetitive dynamic loads at a fixed point in the track and can produce rapid degradation of track components, ballast, and subgrade. Several engine-burn geometries were

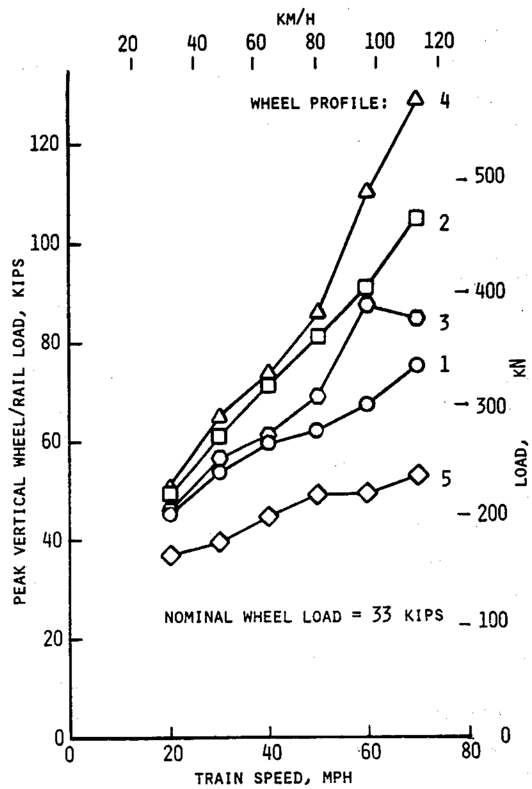


**FIGURE 6 Peak vertical impact load versus speed for five wheel profiles on concrete-tie track (loaded 100-ton freight car).**

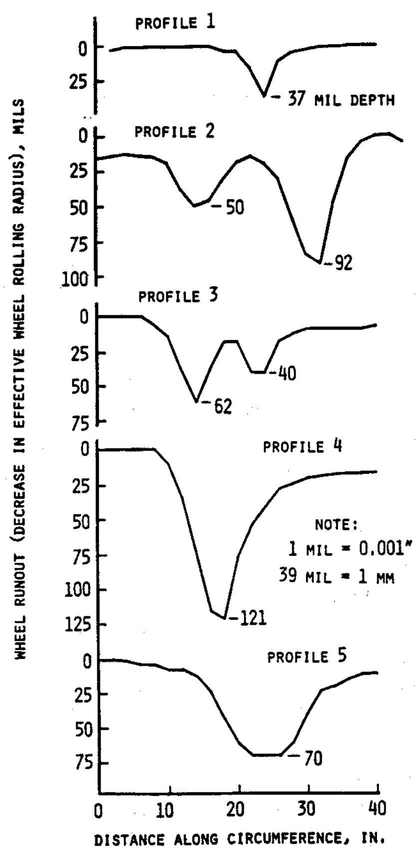


**FIGURE 5 Five representative measured wheel circumferential tread profiles.**

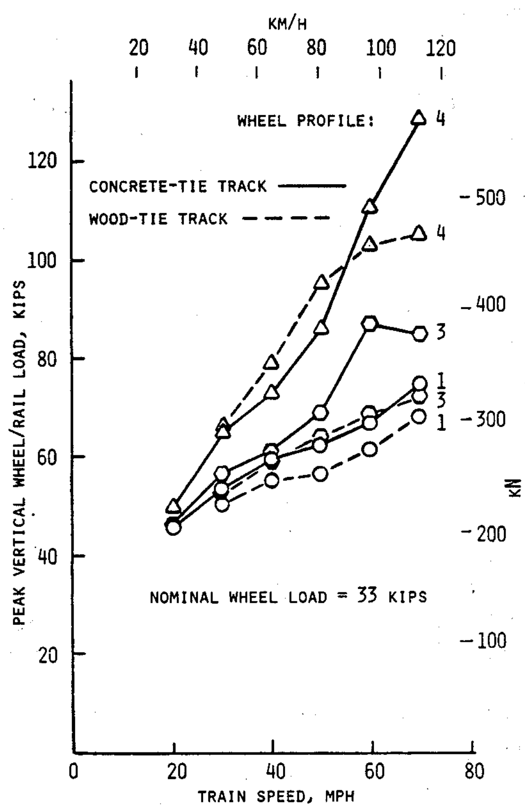


**FIGURE 7 Peak vertical load on concrete-tie track versus good wood-tie track.**

measured on the NEC using a precision-wheel profilometer to measure the change in vertical position of the wheel axis of rotation with longitudinal motion along the rail. With time, the rail surface profile in the vicinity of the engine burn assumes that of a low joint, as shown in Figure 8. This profile had a 0.090-in. (2.3-mm) depth over a 60-in. (1.5-m) span, with a pronounced dip at the engine burn itself. Rail clip fallout and loosened inserts were noted on the tie nearest the burn. A vertical dynamic load of 66,000 lb or 294 kN, (a 16,000-lb, 71-kN, static load) with a 3-msec pulse time duration was predicted under a passenger car wheel at 120 mph (194 km/hr) by using this profile in the computer model.

## STRUCTURAL RESPONSE OF TRACK

Experimental studies were conducted both in the laboratory and on revenue track to investigate the dynamic response of the track to impact loading. Field tests were performed on the NEC concrete-tie track in Maryland in the fall of 1983. In addition to vertical wheel loads, the tie-bending strains; rail, tie, and ballast accelerations; and rail fastener displacements and strains were measured. Track response was measured under revenue traffic loading and under impact loads from an automated drop hammer designed to simulate a moderate flat-wheel impulse. Parameter variations in these tests included impact energy, fastener type, tie pad type, and different tie locations. Similar tests were performed in the laboratory on single ties and on a five-tie NEC track replica. Dynamic data were evaluated by fast fourier transform analyzers, which computed frequency spectra and transfer functions for selected signals (*3,6*).

### Concrete-Tie Dynamics

The performance history of concrete ties in North America (*9*) has included tie failures that generally start with small cracks at the rail seat or the insert, as shown in Figure 9. The rail seat cracks progress upward to the top of the prestress strands and branch out, resulting in shear failure of the tie. This usually causes the insert to fail, thus reducing the gage-holding capacity of the tie. Rail seat cracks, and loss of adhesion around the fastener inserts, are related strongly to wheel-rail impact loads. Other cracks, such as the tie center flexural cracks in Figure 9, are less critical to the structural capacity and function of the tie.

Dynamic analyses of concrete ties have shown that these ties have several modes of vibration, which are strongly excited by wheel-rail impacts. The first three transverse beam-bending modes are shown in Figure 10a. The second

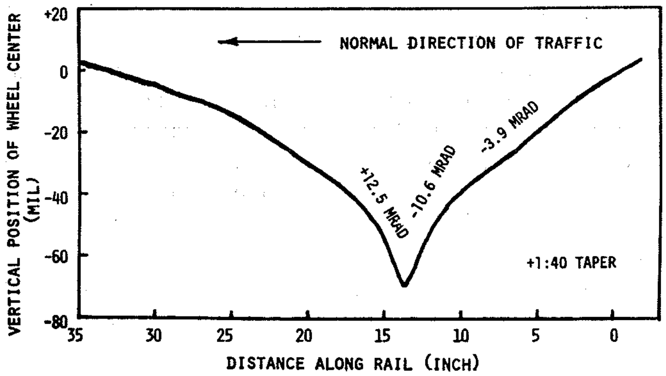


**FIGURE 8 Measured rail running-surface profile near battered engine burn.**

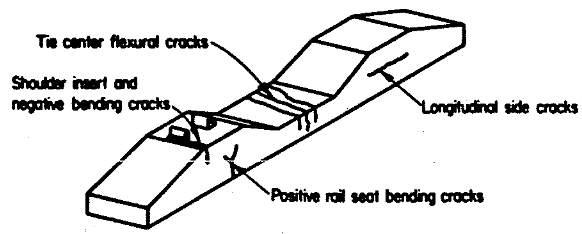


**FIGURE 9 Typical flexural cracks in concrete ties.**

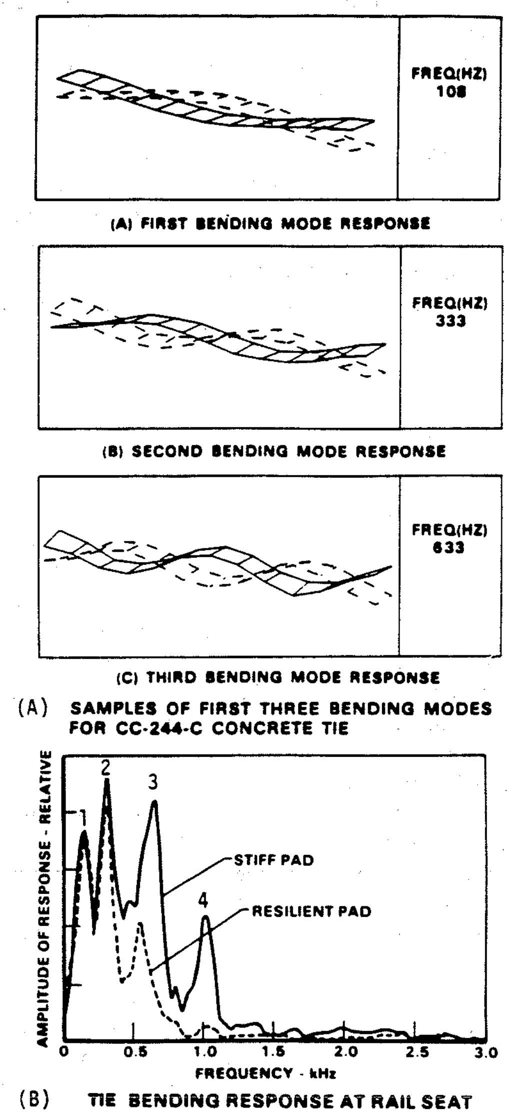


**FIGURE 10 Attenuation of tie-bending response to impact loading with resilient rail-seat tie pad.**

and third modes (333 and 633 Hz, respectively), in particular, generate large tensile strains at the bottom of the tie in the rail seat region. Because the second mode is asymmetric, large dynamic strains can occur by superposition under the rail seat opposite the struck rail. In the field, it has often been noted that rail seat cracks will occur opposite a joint, weld, or engine burn.

The primary load path into the tie for wheel-rail vertical impact loads is through the tie pads. Once excited, the tie — a very lightly damped element (typically less than 1 percent of critical damping in a given mode) — literally rings like a bell. Consequently, the dynamic characteristics of the tie pad are critical to tie performance. The pad must act as a low-pass mechanical filter with a "break frequency" low enough to attenuate most of the energy from the impact at the second and third tie-bending frequencies, reducing the effects of this excitation source. The original NEC ethyl vinyl acetate (EVA) pads proved far too stiff to perform this function. A more resilient pad with a dynamic stiffness of about 1,000 kips/in. (180 MN/m) was tested in the laboratory and in the NEC track. A comparison of tie bending moment response is shown in Figure 10b for this pad versus the standard EVA pad (a dynamic stiffness of 5,000 kips/in., 880 MN/m) under a typical impact load. A substantial reduction in response, particularly at the third bending mode, can be seen.

Field tests on the NEC were undertaken to confirm the results of laboratory tests. Two resilient synthetic rubber tie pads were tested sequentially in the same location under equivalent traffic conditions. Statistical plots of rail-seat bending moment in Figure 11 show a substantial reduction with the resilient pads. The 6.5-mm pad was chosen by Amtrak for new concrete-tie installations and for some retrofit installations on previously installed track.

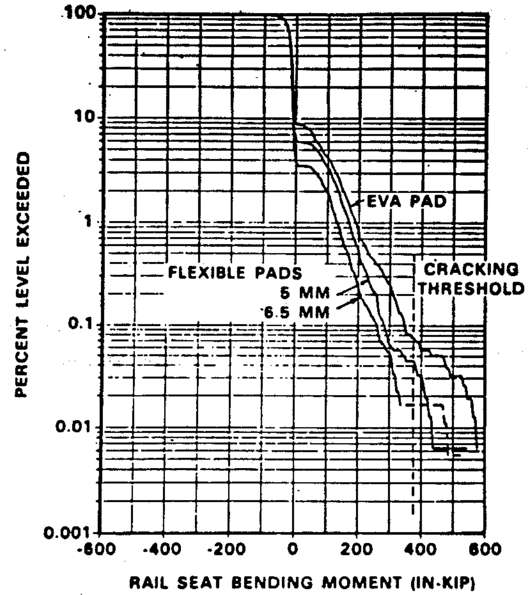


**FIGURE 11 Statistical comparison of rail-seat bending moment under three different pads on NEC track.**

The NEC track had experienced a small but significant number of clip fallouts. Three test sections were set up for an extensive track-walker survey over a 2-year period. Results from this survey were coded and stored in a computer library system called BASIS (*1*). Statistical analysis showed that on one section with older, rougher rail (mechanical joints, engine burns, etc.) 32 percent of fastener fallouts occurred within five ties of a known rail surface anomaly. Other sections had a more random pattern of clip fallouts.

Laboratory and field experiments were performed to investigate the clip fallout phenomenon. Tests included modal vibration analysis of free clips and repeated impact loading on the five-tie track section. Field tests on the NEC included drop hammer impact loading as well as measurements under revenue trains. Comparison of laboratory and field data showed that under impact loading the clips respond strongly to tie dynamics, particularly at frequencies near the third and fourth tie bending modes (633 and 1025 Hz, respectively). Further, clip resonant conditions in the 800- to 2000-Hz range are also excited and influence response in the longitudinal axis, the clip fallout direction. Typical longitudinal displacements of two different clip designs are shown in Figure 12: Type A, the current NEC design, and Type B, a somewhat stiffer design providing about 14 percent more preload. The dynamic behavior of the two in the longitudinal (fallout) direction is strikingly different: Type A exhibits a strong oscillation at 1050 Hz, which is near the fourth tie bending mode. In contrast, the response of Type B is more broadband and well damped. A number of the Type B clips have been installed on the NEC with no reported fallout problems.

The tie pad influences clip deflection response under traffic by its effect on the relative vertical and rocking motions between the rail and tie. Measurements of clip deflection response under traffic (including 100-ton cars) showed that rail-rocking displacements were actually smaller with the

## Rail Fastener Dynamics

In concrete-tie track, the rail fastener system performs the tasks of both cut spike and rail anchor in wood-tie track: maintain track gage, prevent rail rollover, and provide longitudinal rail restraint under thermal and train traction and braking loads. Unlike the cut spike, concrete-tie fasteners provide positive restraint of rail uplift. On the NEC track, the fastener system consists of a Pandrol 601A rail clip and a cast-in clip insert with insulator. Several other types of fastener systems are currently in use or under test at FAST and on freight railroads.

For the current NEC clip design, progressive clip movement out of the insert is resisted only by the frictional forces at the clip-rail and clip-insert interfaces. Clip relative movement will occur when the net static and dynamic forces on the clip exceed the frictional "breakout" levels. Fluctuations in vertical preload occur in response to wheel-rail loads. For impact loads, the dynamic motions of the clip and tie may be sufficient to cause large momentary reductions in preload, which in turn may cause incremental movement of the clip out of the insert. Clip fallout therefore could result from repeated impact loads or from high vibration levels of rail and tie under traffic.

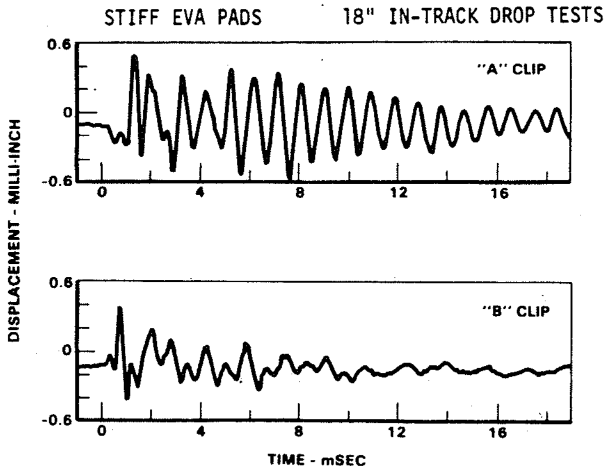


**FIGURE 12 Comparison of clip longitudinal displacements at center leg for two clip designs.**

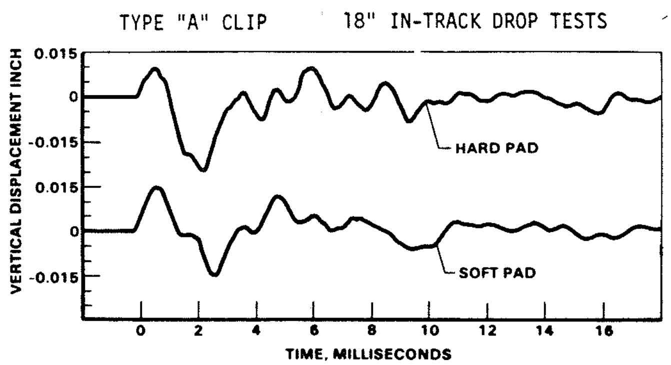


**FIGURE 13 Comparison of clip (toe) displacements with stiff EVA and resilient tie pads.**

resilient pads than with the very stiff pads (*3*). Similar measurements under impact loading are shown in Figure 13. Although the initial peak-to-peak clip deflections are comparable (0.030 to 0.035 in., 0.8 to 0.9 mm), the maximum clip-spreading deflection due to rail uplift relative to the tie is about 40 percent less with the resilient pad than with the stiff EVA pad. The reduction in preload corresponding to the maximum initial depression of the pad is, however, about 50 percent greater with the resilient pad. This is an important area to evaluate for 125-ton traffic because of the significantly higher nominal deflections under the heavier axle loads.

Tests were performed to explore the cause of bond loss between the cast-in fastener insert and the tie shoulder. Although the tests did not fully identify the failure mechanisms, observations and test results indicate that insert loosening can be caused by vertical impact loads smaller than necessary to crack a tie. Perhaps 100,000 impacts of moderate amplitude can loosen the insert. Vibration analyses of insert response to rail impact loading have revealed several modes of axial vibration (vertically oriented to the rail) in the 4- to 8-kHz range. When partly or fully constrained, the insert vibrates at 5 to 7 kHz, and, when unconstrained, the insert responds at 11 kHz. These complex modes are associated with compression ("standing") wave action. Although the amplitudes of motion at the steel-concrete interface are quite small, a microscopic pulverizing action may take place over time that results in gross loosening of the insert. Again, the more resilient tie pad will reduce the excitation energy at the tie, which reduces this vibrational response at the insert.

## CONCLUSIONS

The results of these studies have shown that concrete-tie track, like any other complex mechanical system, requires an understanding of dynamic interactions between components to achieve an optimum design. On the basis of the results of these field and laboratory experiments, the following conclusions can be drawn about the dynamic performance of track components:

1. Concrete-tie track performance is strongly associated with the ability to withstand impact loads. Proper track design should therefore include dynamic analyses to optimize performance.
2. Laboratory evaluation of track components can be accomplished with the aid of a drop hammer that simulates the magnitude and frequency content of the impact loads found in revenue service.
3. Dynamic behavior of components should be evaluated to assure that no undesirable matching of responses takes place between components. For example, decreasing pad stiffness may increase dynamic wheel-rail contact forces, thus increasing the tendency toward initiation of rail corrugations.
4. A dynamically optimized track structure must still withstand the abuse of the largest impact loads actually found in service. An economic trade-off exists between building stronger (more expensive) track and maintaining more perfect wheel and rail running-surface geometries.
5. Impact load response of other vehicle and track components (e.g., wheel bearings and ballast) is not as well defined as concrete-tie-fastener response, but there are ample economic reasons for the dynamic analysis of these components as well. From the limited sample of wheels and bearings examined, there is a strong correlation between high load-producing wheel geometries and bearing damage (loss of grease, cage disintegration, race and roller distress). Wheel "thumpers" are at least a subset of the hotbox problem.

Design and evaluation techniques have been developed that can be applied to track that carries 125-ton cars. Because the 125-ton equipment is ever closer to the design limits of current rail, ties, and fasteners, wheel-rail impact loads caused by imperfections in either wheel or rail geometry become even more significant. Higher productivity is the purpose of using 125-ton cars. A matched higher productivity in track structures may require a greater investment in track design, in construction capital, and in both track and equipment maintenance techniques to meet the closer load tolerances of 125-ton operations.

## ACKNOWLEDGMENTS

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# Component Identification and Inventory of U.S. Army Railroad Trackage

D. R. UZARSKI, D. E. PLOTKIN, AND S. K. WAGERS

**Recognizing the need to effectively plan track maintenance and rehabilitation of the U.S. Army's 3,000 track miles at 81 installations, the U.S. Army, through the Construction Engineering Research Laboratory, has developed a preliminary maintenance management system called RAILER I and is developing an improved system called RAILER II. Both systems define what needs to be managed through component identification and inventory and determine track condition through inspection. Network- and project-level management activities can then be accomplished. Component identification and inventory are the basic first steps in the management process. In this paper, the track component identification procedures for both systems are explained and the RAILER I system track inventory elements are defined along with a method for field data collection. The fundamental component is a track segment, a relatively uniform portion of track that constitutes the basic management unit. Other components identified are track networks, tracks, turnouts, and curves. Surveyor 100-ft stations are used for locating key component and inventory elements. A summary of field test and implementation results is given. The procedures proved simple to implement by personnel with limited experience and training and were well received by those personnel tasked with railroad maintenance management.**

Because of the need for a railroad maintenance management system (*1*), the U.S. Army Construction Engineering Research Laboratory is in the process of developing an overall system called RAILER (*2*) that consists of a preliminary and interim railroad maintenance management system called RAILER I and a more fully capable system called RAILER II. Portions of RAILER are currently undergoing field testing, and the system will be ready for widespread implementation in approximately 2 years. RAILER I is currently being implemented at selected installations where it will be used in the decision-making process for developing critical railroad repair projects during the next 2 or 3 years.

Patterned after the highly successful PAVER pavement maintenance management system (*3*), RAILER (both I and II) is a decision support system designed as a tool to help installation personnel perform network- and project-level analyses of their track. Network-level analysis includes inspection (scheduling and accomplishment), evaluation, budgeting, project development, arranging projects in priority order, and budget and project justification. Major project-level tasks include detailed inspection, evaluation, and selection of the best rehabilitation alternative for selected portions of the track network.

The basic subsystems for RAILER are network definition, data collection, data storage and retrieval, network-level data analysis, and project-level data analysis (*2*). This paper is focused on network definition concepts that are inventory elements for the RAILER I system. The inventory elements of the RAILER II system have yet to be put in final form.

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## BACKGROUND

On paper, the U.S. Army has more than 3,000 mi of track located at 81 installations, including National Guard installations, within the United States. In reality, the actual amount of track is unknown because some installations have abandoned or removed portions or all of their track because of disrepair and lack of need. Rather than being managed as a single commercial railroad of 3,000 track miles, the U.S. Army trackage is analogous to having 81 short-line or industrial railroads, each with its own "president" (the installation commander) who controls the engineering and maintenance forces (in-house or contract) that plan and perform the work. These short lines vary in size from approximately 1 mi to just over 200 mi, with most in the range of 10 to 30 mi.

Daily traffic is generally light (less than one car per day, average) by commercial standards although selected installations do receive considerably more (more than 10 cars per day, average). However, much of the U.S. Army trackage would be subject to large volumes of traffic in the event of a national emergency. It is to meet such emergencies, in addition to the daily traffic, that the trackage must be retained and economically maintained. It is for this purpose that RAILER is needed.

U.S. Army railroad trackage is generally old and, because of low daily traffic levels and low funding priority, in many instances in need of major maintenance and repair. Many of the networks were built to meet World War II traffic needs and, because of the nonavailability of new materials at the time, the tracks were constructed with secondhand materials and light rail weights (average <90 lb). Also, because numerous material sources were used, rail weights, manufacturers, and so forth vary widely even at a given installation.

## NETWORK COMPONENTS

Fundamental to the maintenance management process is knowing just what and how much needs to be managed. Accordingly, the railroad network at each installation must be inventoried and the various components of the network need to be uniquely identified. Five network components have been identified. These are defined in the following list and discussed in the next section of this paper.

- Track network is all of the government-owned railroad track or track constructed to support military operations that is maintained as part of a U.S. Army installation,
- Track is an identifiable portion of the track network serving a distinct purpose,
- Track segment is a division of a track representing the basic unit for railroad maintenance management,
- Turnout is an arrangement of a switch and frog with closure rails used to divert trains from one track to another, and
- Curves are horizontal bends in the track designed to change the direction of travel.

## COMPONENT IDENTIFICATION

### Dividing Track Network into Tracks

The trackage enters the base at one or more connections with one or more commercial railroads. Within the installation, the trackage branches out into a loose tree structure. Each branch of the tree constitutes a track.

#### *Track Identification*

Individual tracks are labeled with unique track numbers so they can be easily identified. Where numbers have been previously assigned to Army tracks, those numbers are retained as part of the identification. Should tracks not be numbered, a logical numbering scheme is assigned during system implementation. A consecutive sequence is used, based on network layout, geography, and train operations. An example of a track numbering sequence is shown in Figure 1.

#### *Stationing*

When tracks have been identified, a location reference system, using surveyor 100-ft stations, is established to assist in locating inventory items and track deficiencies. The use of mileposts was considered impractical because of the short lengths of the majority of U.S. Army tracks. As defined, a track originates at the point of switch of the turnout leading to the track. This point of origin, by definition, is station 0 + 00. Consecutive stations are designated and permanently marked at 200-ft intervals with the aid of a measuring wheel. Tracks terminate and, therefore, the last station is at the point of switch of the turnout where the track joins another track or at a stub end. Figure 2 shows this stationing sequence.

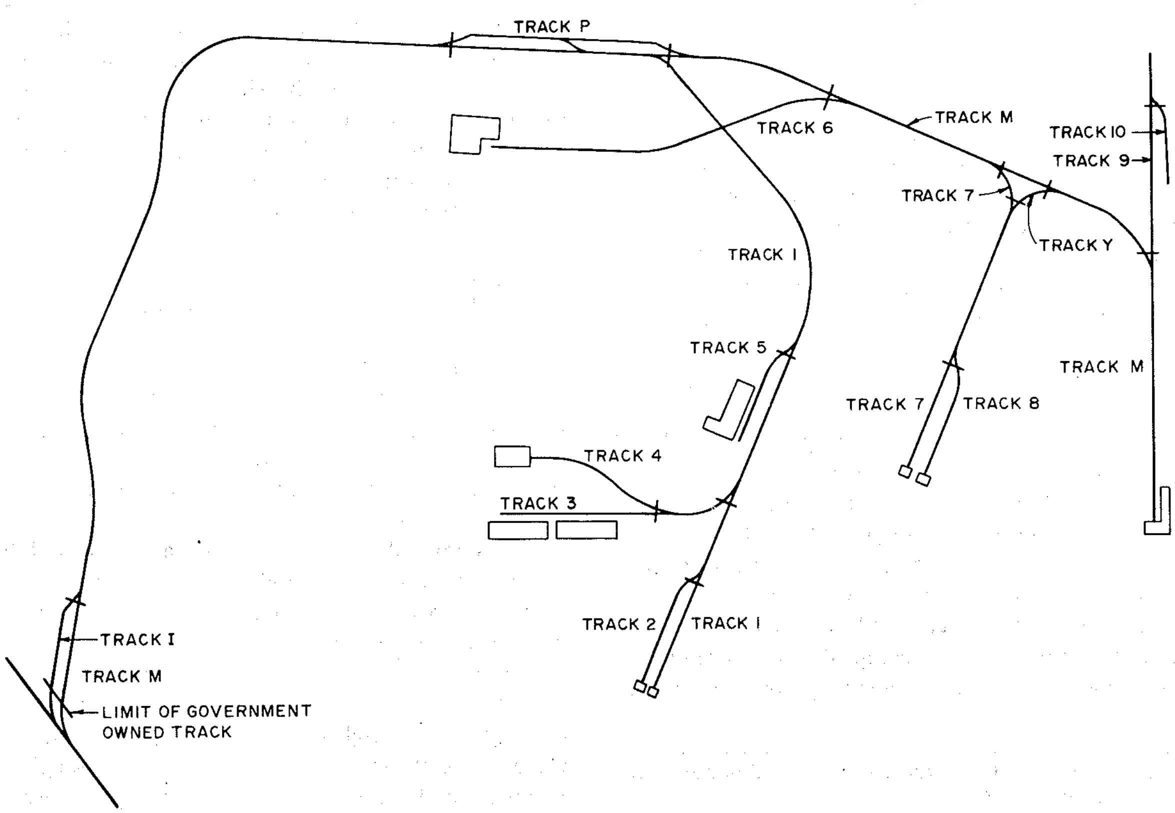


**FIGURE 1 Track numbering sequence.**

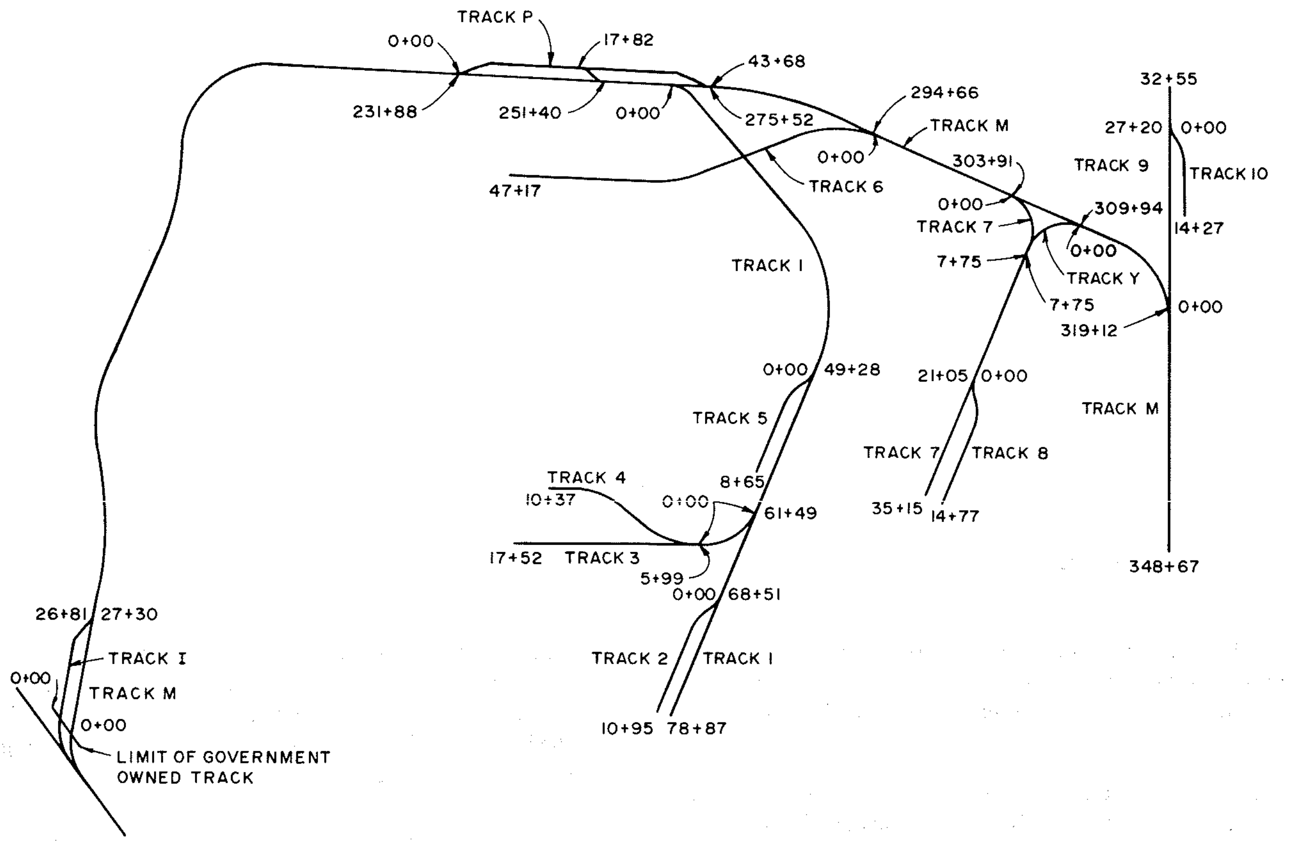


**FIGURE 2 Track stationing sequence.**

Exceptions to this stationing sequence include crossover tracks, back-to-back turnouts, and connections linking installation trackage with commercial railroads. In the latter instance, the point where government ownership begins is designated station 0 + 00. The other exceptions are discussed under "turnouts" later in this paper.

### Dividing Tracks into Track Segments

For effective management and network identification, tracks are divided into units called track segments. Each track must have at least one track segment.

#### *Criteria*

Two required and two optional criteria are used for determining track segment differentiation: train operations, track use, rail weight (optional), and bridges (optional).

**Train Operations** This criterion requires that segments begin or end at virtually every turnout because a turnout allows a choice of routes. In addition, tracks may exist that do not commonly have train operations over their entire length. In such cases, the active and inactive portions of the track may be designated as separate segments.

**Track Use** Although many specific track functions may exist within a network, five general categories are used for management purposes:

1. Loading: tracks used for loading and unloading equipment and supplies;
2. Storage: tracks used for long- or short-term storage of freight cars, including classification yard tracks and interchange tracks;
3. Service: tracks used for servicing either general installation operations or railroad equipment including tracks leading to a power plant, engine house, or car shop;
4. Auxiliary: tracks used to aid train operations including passing sidings, wye tracks, and run-around tracks; and
5. Access: tracks that provide connections between the other types of tracks, as well as those that link the installation and a commercial route.

Figure 3 shows a network with track uses assigned.

**Rail Weight** Sometimes when tracks are divided into track segments on the basis of the foregoing criteria, portions of the segment have rail of a weight that is significantly different from that of other portions. Because track performance is partly a function of rail weight, it may be advantageous for work-planning purposes, at times, to use rail weight as a segmenting criterion. Because rail weight can be quite

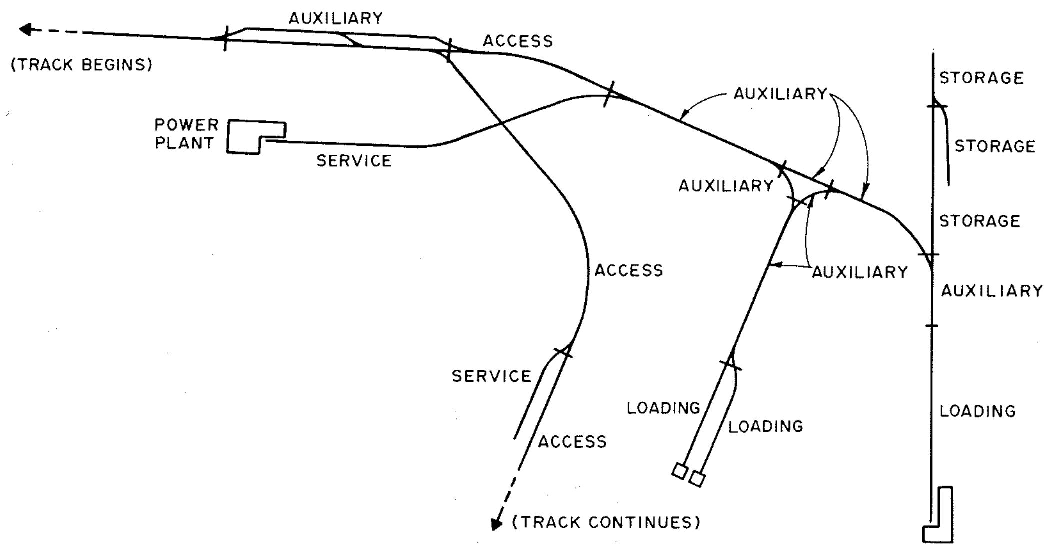


**FIGURE 3 Track uses.**

variable and may change every two or three rail lengths, using rail weight as a segmenting criterion should not normally be considered. It should be reserved for special cases.

**Bridges** Because of the unique maintenance requirements of track on bridges, as well as of the bridge itself, the track on a bridge may be considered a separate segment. The limits of a track segment on a bridge are shown in Figure 4.

**Other Factors** Ideally, the track in any segment should have uniform traffic and physical characteristics over its entire length. Where significant changes in these characteristics occur, new segments should be created. In addition to the four parameters discussed, segments may also be differentiated on the basis of ballast type, subgrade soil type, tie spacing, or overall track condition.

### *Track Segment Identification*

When tracks have been divided into appropriate track segments they must be numbered. The track segment number is created by adding a two-digit suffix to the track number. Numbers are generally consecutive for all track segments

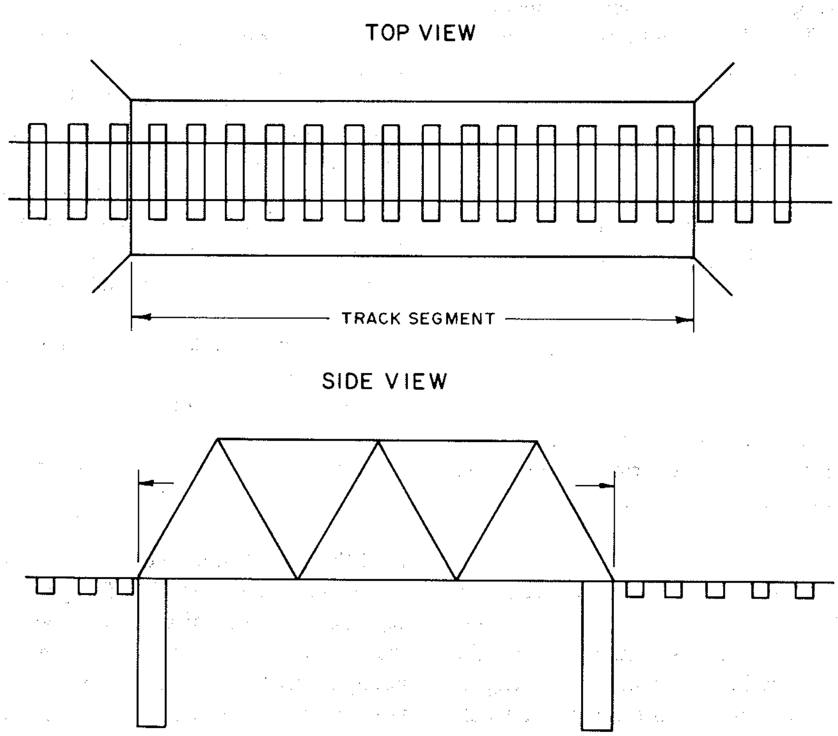


**FIGURE 4 Track segment on a bridge.**

within a given track. For example, M03 indicates the third segment of track M. Examples of track segment numbering are shown in Figure 5.

## Turnouts

Each turnout is located in one and only one track segment, and the location of the switch points determines the track segment in which the turnout resides (Figure 6).

Crossover tracks are divided at the center of the connecting tangent if there is less than 50 ft of track between the last switch ties of the turnouts leading to the crossover. Each half of the crossover is included in the track segment containing the turnout. If there are more than 50 ft, the track is treated like any other track. The same logic is used when turnouts are located back to back (Figure 7).

### *Turnout Identification*

Turnouts are numbered individually within track networks. Where an existing numbering system has been established, it is retained. Otherwise, all turnouts are numbered as follows:

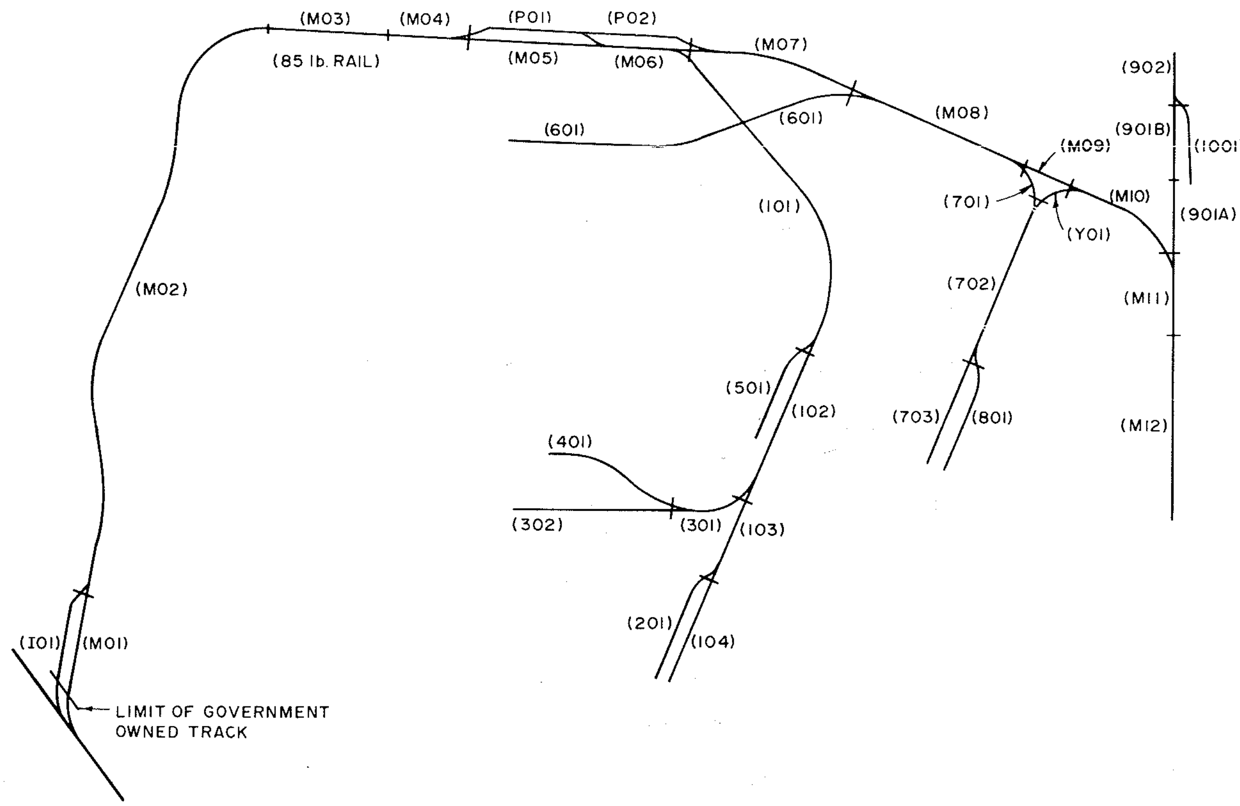


**FIGURE 5 Track segment numbering sequence.**

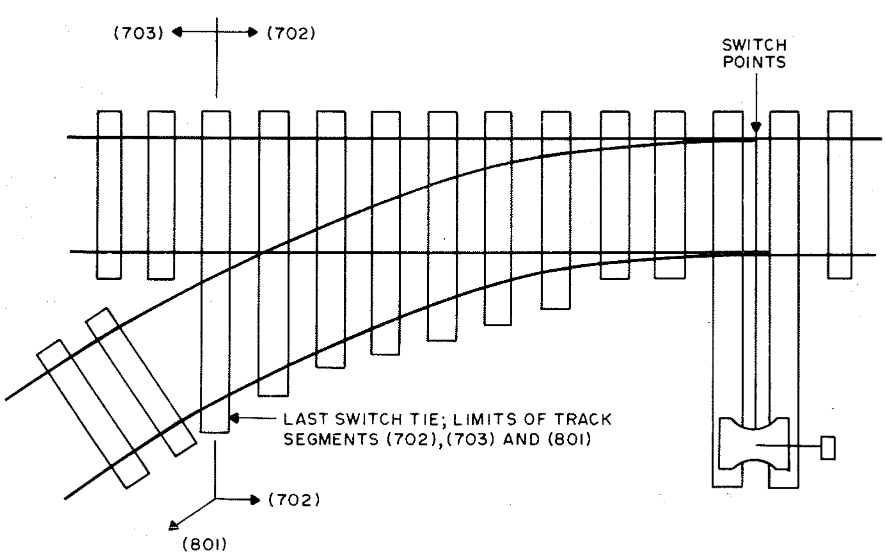


**FIGURE 6 Segmenting a turnout.**

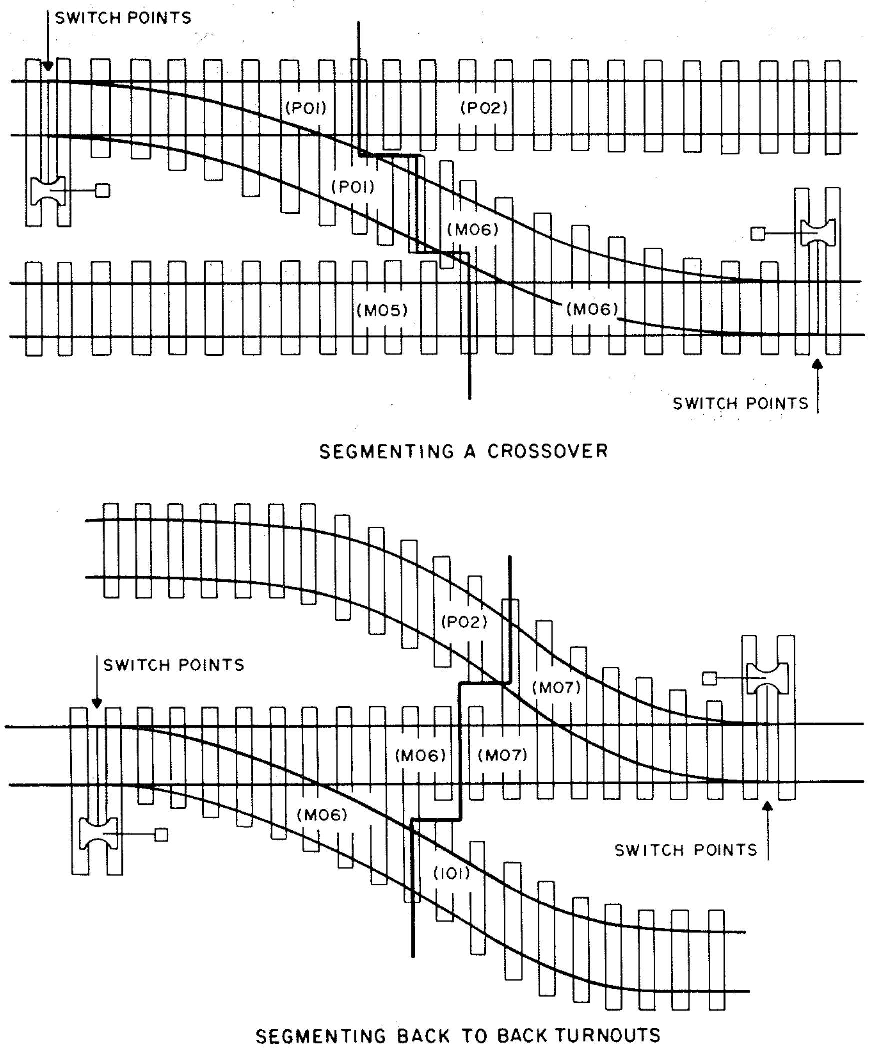


**FIGURE 7 Track segmenting at crossovers and back-to-back turnouts.**

(Integer) T (Diverging track number). This three-part number is established as follows.

1. Integer: 1 is reserved for the turnout where the diverging track begins. The point of switch location for the diverging track is usually 0 + 00. All other turnouts leading to the same track are designated consecutively (2, 3, . . . ) in an order corresponding with increasing station location.
2. T: used to indicate that a turnout (rather than a track, track segment, or curve) is being identified.
3. Diverging track number: the track that the turnout diverges or leads into.

For example, 1TP indicates the first turnout leading into track P. Examples of turnout numbering are shown in Figure 8.

### Curves

A curve may reside in one or more track segments, depending on where the division between track segments occurs. Connecting curves (those that allow a track to run parallel to its originating track) are usually not designated as curves and are not included in the curve numbering sequence or curve inventory. Curves are numbered individually within track networks. Because there is no existing numbering system, all curves are numbered (Integer) C (Track number). This three-part number is established as follows.

1. Integer: the curves in each track are numbered beginning with 1 for the first curve encountered and continuing consecutively to the end of the track.
2. C: used to indicate that a curve (rather than a track, track segment, or turnout) is being identified.
3. Track number: the track that contains the curve.

For example, 1C1 indicates the first curve in Track 1. Examples of curve numbering are shown in Figure 9.

## INVENTORY

### Network Inventory

Inventory items include installation number, installation name, name of commercial railroad serving the installation,

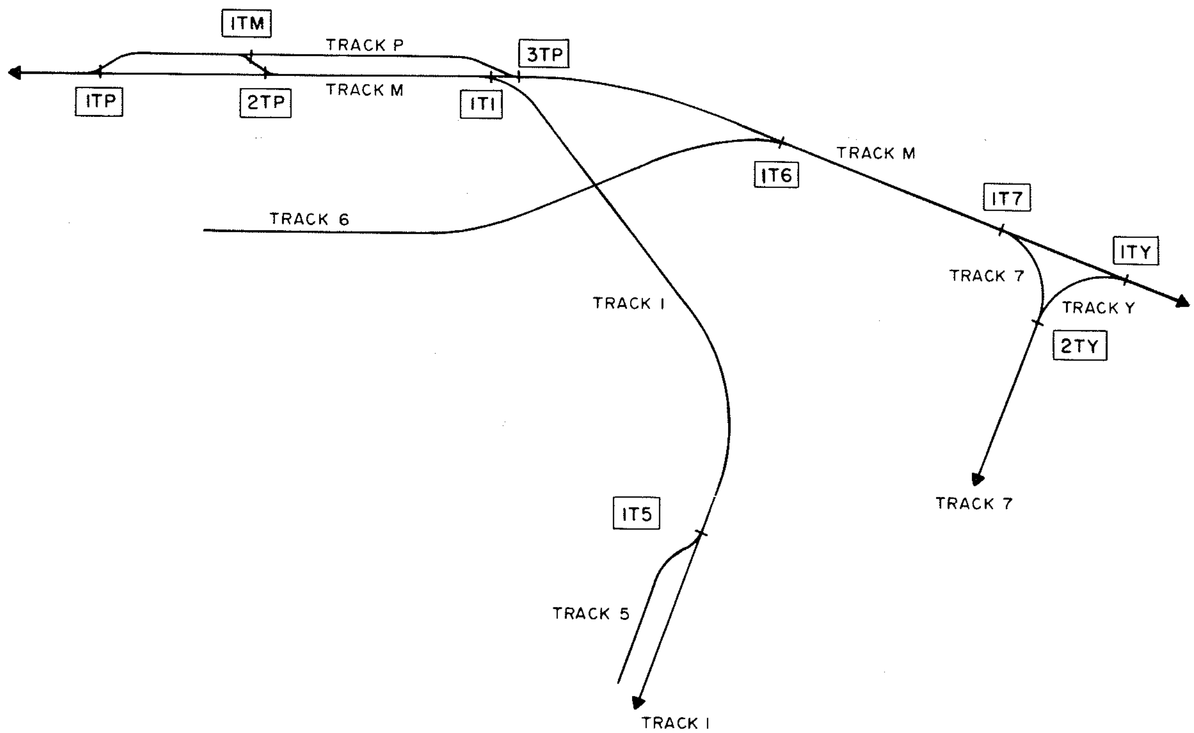


**FIGURE 8 Turnout numbering.**

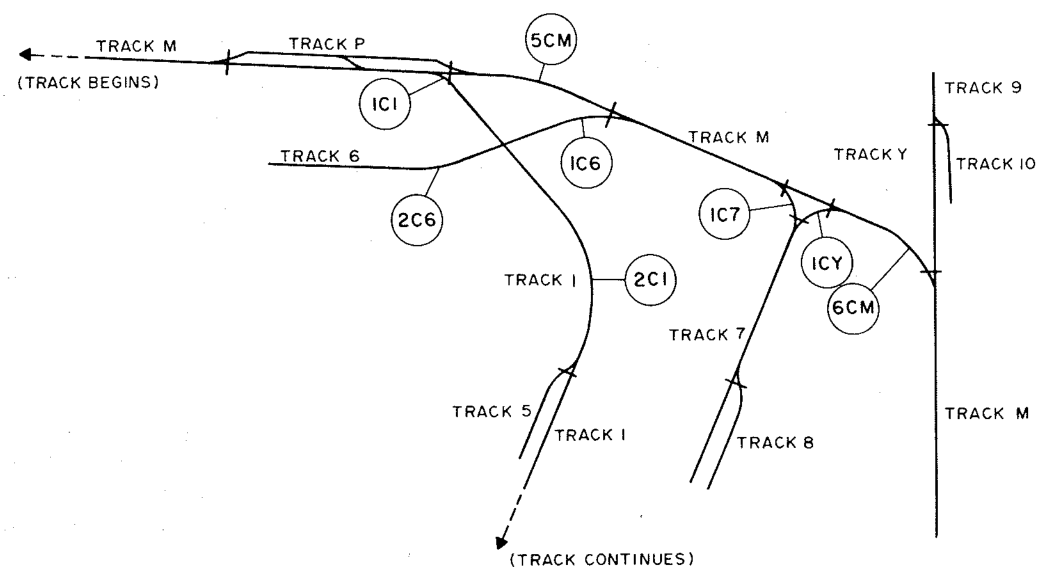


**FIGURE 9 Curve numbering.**

length of each track, and number of track segments for each track. This information uniquely identifies a given installation track network.

## Segment Inventory

The track segment inventory provides an identification and description of track and roadway components, as well as structures and facilities directly associated with a track segment.

Specific data elements for the following items are collected: segment identification, ballast, bridges, culverts, curves, plates and fastenings, rail, rail crossings, road crossings, and turnouts. Table 1 is a list of the data elements that represent minimal requirements for proper maintenance management. Of these, most are self-explanatory or have been previously discussed with the exception of track category, track rank, and preceding track segment number. These have been developed or modified specifically for U.S. Army usage and are discussed in the following list.

TABLE 1 RAILER I TRACK SEGMENT INVENTORY ELEMENTS

| Item No. | Description | Item No. | Description |
|---|---|---|---|
| Segment Identification | | 22. | Rail anchors (No./200 track ft) |
| 1. | Track segement number | 23. | Gage rods (no, yes) |
| 2. | Begin location (station) | 24. | Comments |
| 3. | End location (station) | Rail | |
| 4. | Track category | 25. | Weight (lb/yd) |
| 5. | Track use | 26. | Section |
| 6. | Track rank | 27. | Begin location (station) |
| 7. | Preceding track segment number | 28. | Comments |
| 8. | Comments | Rail Crossings | |
| Ballast | | 29. | Centerline location (station) |
| 9. | Depth (in.) | 30. | Crossing segment number |
| 10. | Comments | 31. | Rail weight (lb/yd) |
| Bridges | | 32. | Frog type |
| 11. | Facility number | 33. | Crossing angle (degrees) |
| 12. | Construction type | Road Crossings | |
| 13. | Deck type | 34. | Road name |
| 14. | Comments | 35. | Centerline location (station) |
| Culverts | | 36. | Crossing length (ft) |
| 15. | Centerline location (station) | 37. | Crossing type |
| 16. | Comments | 38. | Bolted joints (no, yes) |
| Curves | | Turnouts | |
| 17. | Curve identification number | 39. | Turnout identification number |
| 18. | Curvature (degrees) | 40. | Switch point location (station) |
| 19. | Maximum desired speed (mph) | 41. | Direction (left hand, equilateral, right hand) |
| 20. | Comments | 42. | Point length (linear ft) |
| Plates and Fastenings | | 43. | Rail weight (lb/yd) |
| 21. | Tie plates (no, yes) | 44. | Frog type |
| | | 45. | Frog size |
| | | 46. | Guard rail length (linear ft) |
| | | 47. | Comments |

- Track category: A or B. A for active track or track required to support mobilization. B for all other trackage.
- Track rank: A numerical relative value ranking derived analytically from other inventory data, used for priority ordering of work. (Its development, calculation, and use are beyond the scope of this paper.)
- Preceding track segment number: The track segment that a train must pass in order to travel on the current track segment as the train travels into the installation. This may be multiple when passing track or wyes precede current track segments. This information is used for priority ordering of work. (A description of its use is beyond the scope of this paper.)

## IMPLEMENTATION PROCEDURES

The following steps are used in identifying components and inventorying the trackage at specific installation networks as part of a RAILER implementation.

1. Determine initial track, turnout, and curve numbers. With the help of a map or track diagram, and information from someone familiar with the network, determine track, turnout, and curve numbers. This is done in the office.
2. Divide tracks into track segments. Divide each track into logical track segments, assign consecutive track segment numbers, and indicate track, track segment, turnout, and curve numbers on existing network maps or diagrams. This marked-up map will serve as a guide for the field work. This work is also performed in the office.
3. Verify track network and track use. When track, track segment, turnout, and curve numbers have been established, verify this information in the field, along with track uses, with the help of someone familiar with the network. This will ensure that all identification numbers and track uses are clear, logical, and accurate. Obtain the concurrence of the installation railroad maintenance manager.
4. Station the track. With the aid of an appropriate measuring device, such as a measuring wheel, station and

permanently mark each track at 200-ft intervals. Temporarily mark station locations at the track segment origins, turnouts, culverts, rail and road crossings, and bridge ends. As an alternative to temporary marks, the station locations can be immediately written on the track segment inventory forms.

5. Complete the inventory. Use the track segment inventory form (Figure 10) to complete the inventory for each segment. It is especially helpful when conducting the inventory to have the marked-up map present. As each segment is inventoried, it should be checked off on the map, thus ensuring that no omissions or duplications of segments occur. The map will also aid in identifying segments, turnouts, curves, and other items in the field.

6. Final acceptance. Check the inventory sheets for errors, consistency, and omissions, and correct as required. Load data into the RAILER data base. Prepare new station maps on which all segments and components are clearly identified. The inventory sheets should be checked daily and the maps should be redrawn when all inventory is complete.

## FIELD TESTS AND IMPLEMENTATION EXPERIENCE

The component identification and inventory procedures described have been field tested at Fort Belvoir, Virginia; Fort Devens, Massachusetts; and the Consolidated Rail Corporation (Conrail) yard in Urbana, Illinois. Full implementation has occurred at Fort Campbell, Kentucky; Fort Wingate, New Mexico; Fort Carson, Colorado; Camp

TRACK SEGMENT #: 1Ø1　　RAILER I　　DATE: 4 FEB 87
INSTALLATION NAME: FT EXAMPLE　　TRACK SEGMENT INVENTORY INFORMATION

| SEGMENT IDENTIFICATION | | | | | | BALLAST |
|---|---|---|---|---|---|---|
| Begin Location (station) | End Location (station) | Track Category | Track Use | Track Rank | Preceding Track Segment Number (s) | Depth (inches) |
| 1+11 | 50+16 | A B | Acc Aux L Se St | | MØ8 | 14 |
| Comments: | | | | | | Comments: 6" LIFT IN 1973 |

| BRIDGES | | | CULVERTS | CURVES | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Facility Number | Construction Type | Deck Type | Centerline Location (Station) | Curve ID Number | Curvature (Degrees) 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | Avg | Max Desired Speed (m.p.h.) |
| | | Open Ballast<br>Open Ballast | 47+41 | 1C1<br>2C1 | | | | | | | | | 8<br>4.5 | 15<br>20 |
| Comments: | | | Comments: TWIN 36" CMP | Comments: CURVATURE DATA FROM "AS BUILT" DRAWINGS | | | | | | | | | | |

| PLATES/FASTENINGS | | | RAIL | | | RAIL CROSSINGS | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Tie Plates | Rail Anchors (#/200 TF) | Gage Rods | Weight (lbs/yd) | Section | Begin Location (Station) | Centerline Location (Station) | Crossing Segment Number | Rail Weight (lbs/yd) | Frog Type | Crossing Angle (degree) |
| N Y<br>N Y<br>N Y<br>N Y | 80 | N Y<br>N Y<br>N Y<br>N Y | 90 | AS | 1+11 | 12+29 | 6Ø1 | 90 | B MI SM<br>B MI SM<br>B MI SM<br>B MI SM | 60 |
| Comments: | | | Comments: | | | Comments: | | | | |

| ROAD CROSSINGS | | | | |
|---|---|---|---|---|
| Road Name | Centerline Location (Station) | Crossing Length (feet) | Crossing Type | Bolted Joints |
| BRADLEY BLVD | 36 + 48 | 24 | TIMBER/ASPHALT | N Y<br>N Y<br>N Y |
| Comments: | | | | |

| TURNOUTS | | | | | | | |
|---|---|---|---|---|---|---|---|
| Turnout ID Number | Switch Point Location (Station) | Direction | Point Length (LF) | Rail Weight (lbs/yd) | Frog Type | Frog Size | Guard Rail Length (LF) |
| 1T5 | 49+28 | LH EQ RH<br>LH EQ RH | 13 | 90 | B SG RBM SP<br>B SG RBM SP | 7 | 11 |
| Comments: | | | | | | | |

**FIGURE 10 Track segment inventory form.**

Roberts, California; Camp Edwards, Massachusetts; and the Tooele Army Depot, Utah. Each site has served to improve the procedures and the data-gathering process to ensure efficient use of time and usefulness of data collected. The current version has been described in this paper.

The office work of identification numbering and segmenting was easily accomplished by one person. Accurate maps made the defining process quite simple. Inaccurate maps led to many revisions during the validation step.

For the field work it was found that a two-person crew works most efficiently and that two passes over the track are necessary to complete the stationing and inventory process. The first pass should serve to validate the segments and all assigned identification numbers. Track stationing can be easily accomplished concurrently. The inventory itself is best done during the second pass.

The stationing procedure must be accomplished with great care. To date, this has been accomplished with a measuring wheel. Different procedures were used including walking with the wheel on the rail, the wheel mounted to a track cart with the wheel on the rail, and the wheel mounted to a track cart with the wheel riding on the wheel of the cart. In some cases the cart was pulled or pushed manually, with a motor car, or with a locomotive. In general, all worked well, except walking with the wheel on the rail is slow. Accordingly, this should only be used for short tracks where it is inconvenient or impossible to use the other methods. When the measuring wheel is mounted on a cart, productivity can be 9 to 10 mi/day.

A problem encountered with measuring wheels is the inherent inaccuracy associated with them. The wheels used were checked against a steel tape and the error determined. Errors within 1 percent were accepted or the wheel was not used. Some had error as high as 5 percent. All station markings were taken to the nearest foot. This is a reasonable degree of accuracy for maintenance management purposes.

Different methods were employed for marking the stations in the field. Paint, lumber crayons, and metal plates nailed to ties were all used. The stamped metal plates (nailed to the nearest tie) worked best as far as permanency and ease of installation were concerned, but they were also the most expensive because of their manufacture. When paint or crayons were used, the station was written on the rail base. Paint could, at best, be considered semipermanent and crayon markings are considered temporary. The use of permanent markers is considered essential and saves time when locating track deficiencies later in the management process. Also, the marked stations made it possible to easily post flags or other temporary markers trackside every 1,000 ft to aid in location referencing when performing automated track geometry or rail flaw testing, or both.

The inventory itself was quick and easy to accomplish. Between 2 and 5 mi/day were covered depending on number of segments, variability of data elements, and whether the crew was riding or walking. The most time-consuming data element to collect was the determination of rail weight. U.S. Army rail is generally quite old (most was manufactured between 1880 and 1945), and rail brands can be difficult to read. Also, weights vary greatly and each time a pair of compromise joints is encountered the location (average station for staggered joints) must be recorded and the new weight recorded. This procedure, although a bit slow, is necessary to get an accurate compilation of the amounts of different weights present. Ballast depths were estimated on the basis of the knowledge of local personnel.

## CONCLUSIONS

Under field test and implementation conditions, the component identification and inventory procedures described worked well with few problems. The data required were not unreasonably difficult to collect and did not require extensive knowledge of railroad track. In general, the procedures, identification numbering, and inventory list were well received by the installation personnel tasked with the actual maintenance management of the track network.

## ACKNOWLEDGMENTS

The cooperation of the Consolidated Rail Corporation for the ongoing use of their Urbana, Illinois, yard is greatly appreciated. The assistance of the many Directorate of Engineering and Housing personnel as well as the Installation Transportation Office personnel from all of the test and implementation sites is also greatly appreciated. A warm appreciation is extended to the personnel from the Navy Civil Engineering Laboratory and the Transportation Systems Center who participated in the field data collection process.

Special acknowledgments are given to the many full- and part-time U.S. Army Construction Engineering Research Laboratory personnel, especially Debra Piland, who have significantly contributed to the railroad research, and to M.Y. Shahin for initiating the RAILER system research. In addition, special acknowledgments are given to Bill Taylor from U.S. Army Forces Command for his drive, initiative, and funding of the RAILER I system; Robert Williams from Department of the Army, Engineers, for his continued RAILER support; and Jerry Elton from the Tooele Army Depot for his outstanding field support and cooperation.

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# Use of Geosynthetics in the Design of Railroad Tracks

HUGH S. LACY AND JAMIL PANNEE

**An innovative approach was used for an Amtrak Northeast Corridor Improvement Project (NECIP) in Boston for track structure support to accommodate low-strength organic clay and high groundwater beneath the railroad tracks. The new track structure uses a combination of geomembranes and geotextiles to maintain groundwater at or near prevailing level to avoid settlement of adjoining structures and provide a dry, stable foundation support for the tracks. This innovative track support system cost only about 40 percent of a pile-supported concrete slab similar to that used in the section of track west of this project. Considerations that led to the combination of a conventional railroad track structure and geosynthetics are described.**

Design of a 0.4-mi section of Northeast Corridor railroad tracks located in Boston, Massachusetts, required construction of three tracks on weak organic clays at about the same level as to 6 in. below the previous tracks. Special provision was made in the design to accommodate the effect of a high water table, which is generally between 1 and 3 ft below the ground surface. The project is located between Boston South Station and the Massachusetts Bay Transit Authority (MBTA) Southwest Corridor project. This segment of the Northeast Corridor will be heavily traveled, and the level of service per track projected for 1990 is 42 to 47 trains per day with a total tonnage range of from 13 to 18 million gross tons (MGTs). The project includes rebuilding track support for two adjacent Boston and Albany Railroad tracks. Construction of the track support system took place during 1985-1987.

The project site is shown in Figure 1. It is bounded on the north by the Massachusetts Turnpike and on the south by Herald Street. The Massachusetts Turnpike is founded on a reinforced concrete slab slightly above the level of the adjacent tracks extending the full length of the project. To limit settlement of the turnpike slab, the existing groundwater table is being maintained by limiting drainage beneath the turnpike concrete slab into the lower track area. Drainage of the track area is regulated by an overflow weir structure located at the outfall of the storm water drainage system. The areas north of the Massachusetts Turnpike and south of the project site are 15 to 20 ft above track level and are occupied by commercial buildings and streets.

On the south, Herald Street borders the project above a granite-faced gravity wall. The retaining wall is between 15 and 19 ft high. The gravity walls are supported on both timber piles and spread foundations on hard clay and are not particularly sensitive to groundwater level fluctuations.

The MBTA in joint funding with the Federal Railroad Administration (FRA) is rebuilding about 5 mi of depressed rail and transit tracks known as the Southwest Corridor Project (SWCP), which interfaces with the west end of this project. The SWCP tracks in the areas overlying the soft organic clay are supported on reinforced concrete slab and steel H-piles driven to bedrock.

## GEOLOGY OF THE AREA

The project lies within the Boston Basin, which contains a thick layer of fine-grained sediments deposited in late- to postglacial times. The basin was scoured in rock by glacial ice advancing from the northwest. During ice retreat, the sea level rose, inundating the basin, and Boston blue clay was laid down. Subsequently, during a late glacial period, the sea level dropped 50 to 70 ft and exposed the clay surface, producing weathering, desiccation, and erosion of the upper part of the clay. After the low-level stage, the water again rose to submerge low-lying areas of the basin within which soft marsh deposits formed.

## HISTORY OF THE SITE

This project lies within an area that was once mostly covered by water, known as the Back Bay. At the time of the earliest settlement of Boston, a thin strip of land called the Boston Neck bridged the Back Bay between Beacon Hill and Roxbury, running at about the center of the project site (*1*). Nearly all of the project area was a salt marsh and was mostly under water at high tide. The colonial shoreline crossed the site in the vicinity of Washington Street. By 1814 filling had widened this area at the railroad alignment to between Shawmut and Harrison Avenues. By 1836 fill had been placed to slighly west of Tremont Street, and filling was completed at Albany Street by 1871. The Boston and Worcester Railroad was, in 1834, on an embankment through the site along the alignment of the existing Boston and Albany Railroad tracks. About 15 ft of fill was placed over the railroad alignment area for a number of years before the Boston and Worcester Railroad was constructed at the present lower grade. Test pits (*2*) have revealed that at least

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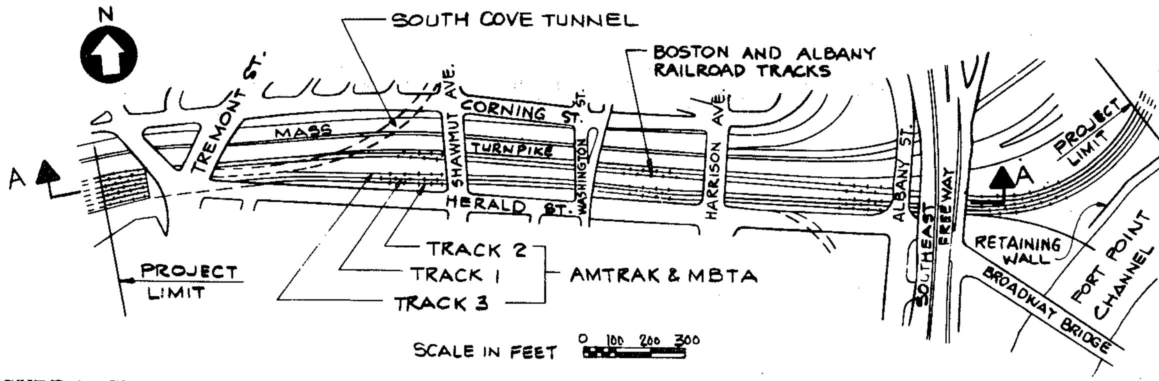


**FIGURE 1 Site plan.**

one of the NECIP tracks was once supported on timber piles. The track has been supported for the last 80 or more years on ballast over highly variable thicknesses and types of granular fill. In recent years, the tracks have experienced increasing misalignment and maintenance costs.

## TOPOGRAPHY

Elevations used in this paper are based on the National Oceanic and Atmospheric Administration (NOAA) mean sea level datum. The existing track grade rises from Elevation -1.5 at the west interface with the SWCP, near Tremont Avenue, to Elevation +9.0 at the eastern end. Grades along the turnpike vary from Elevation -0.8 to +5.5. The area to the south of the project has grades of about Elevation +15.

## EXISTING BRIDGES

There are six bridges within the limits of the project (*3*) that serve as roadway overpasses:

| *Bridge* | *No.* |
|---|---|
| Tremont-Arlington Street | 228.24 |
| Shawmut Avenue | 228.37 |
| Washington Street | 228.34 |
| Harrison Avenue | 228.41 |
| Albany Street | 228.51 |
| Southeast Expressway | 228.65 |

Pavement grade of the overpasses at the centerline of the tracks ranges between Elevations +19 and +25, except for the Southeast Expressway, which is about Elevation +45. In addition, two utility bridges are also located along the east side of the Shawmut and Harrison Avenue bridges.

## SUBSURFACE CONDITIONS

The three distinct soil strata encountered in the project site (*1, 2*) are shown in Figure 2. The three principal soil types are described from the surface downward in the following subsections.

### Stratum F—Fill

A shallow layer of fill, ranging in thickness from 2 to 7 ft, covers the track area. The fill is made up of stone track ballast over a subballast of loose to compact coarse to fine sand and gravel with varying amounts of silts, ashes, brick, and wood.

### Stratum O—Organic Clay

This stratum is up to 19 ft thick west of Tremont Avenue and east of Albany Street, but it pinches out east of Shawmut Avenue and east of Washington Street. The absence of organic soils in the middle of the project reflects the Boston Neck described earlier. This stratum consists of medium gray organic silty clay, trace shells, and fine sand. Natural moisture content varies between 28 and 59 percent, generally higher at Elevation -20, which indicates a thin seam of peat.

### Stratum C—Boston Blue Clay

This layer is made up of stiff to medium green-gray silty clay. The surface of this deposit was generally stiffened by drying and oxidized to a green-brown or yellow color during a depressed sea level. The natural moisture content varies from 15 to 49 percent with desiccated clay having water contents of 30 percent or less.

## SOIL PROPERTIES

The engineering properties of the various soil strata used in the design and analysis of the various alternatives are given in Table 1.

## GROUNDWATER LEVEL

Groundwater levels in the project site are slightly affected by the open waters of the Charles River to the north and Fort Point Channel to the east. In general, the groundwater level within the trackbed sloped from Elevation -2.0 at the west end to about Elevation -1.0 at Washington Street, rising to Elevation +1.5 near the Southeast Expressway. The city of

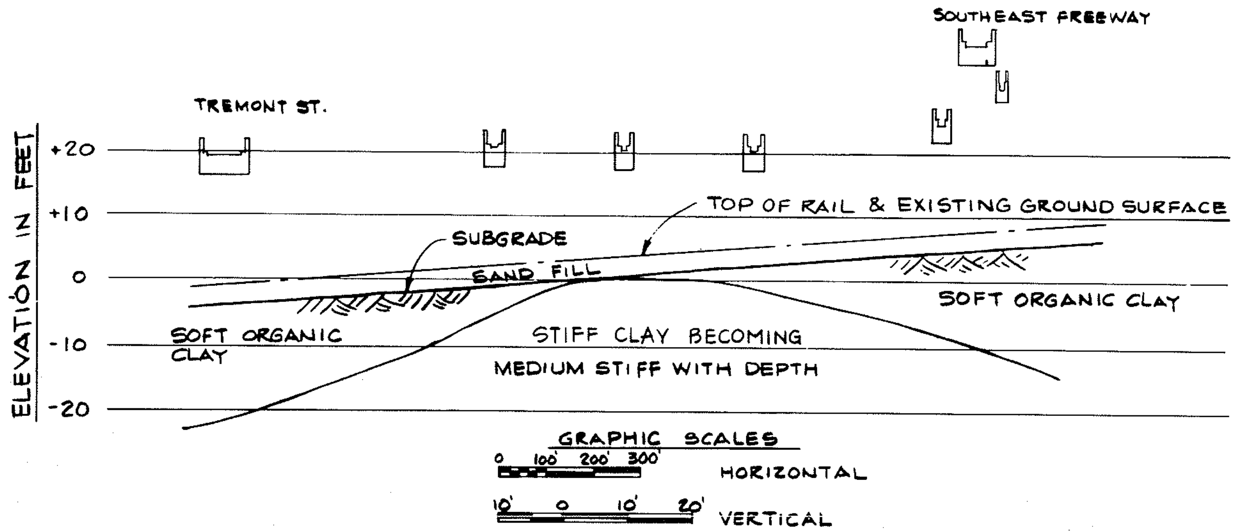


**FIGURE 2 Geologic profile.**

TABLE 1 ENGINEERING PROPERTIES OF SOILS

| | Stratum | | |
|---|---|---|---|
| Soil Property | F | C | O |
| Total weight (pcf) | 115 | — | — |
| Submerged weight (pcf) | 55 | 60 | 40 |
| Friction angle (degrees) | 30 | — | — |
| Shear strength (ksf) | — | 2.0-3.0 | 0.6 |
| Base friction factor (F) | 0.45 | — | — |
| Adhesion strength (psf) | — | 750-1200 | 500 |
| Bearing capacity (tsf) | 2.5 | 2.0-3.0 | 0.6 |
| Recompression index | — | .02 | .06 |
| Subgrade modulus (kcf) | — | 120 | 50 |

Boston code requires that prevailing groundwater levels be unaffected by future developments to avoid settlement and damage to existing untreated wood piling caused by groundwater lowering.

Construction of the Massachusetts Turnpike just north of the site between 1962 and 1965 was done in a manner that maintained existing groundwater levels. The highway structural system was designed as a watertight section with a reinforced concrete base slab 2 to 3 ft thick founded on an impervious foundation with cantilever retaining walls along the north side. The subgrade was covered by polyvinyl plastic sheet, a 4-in. working mat, and a concrete structural slab with water stops. Before construction of the turnpike, the water level in the trackbed was allowed to rise to Elevation -1.0 before the drains became effective. This minimum water level is presently being regulated by a weir overflow structure located at the eastern limits of the turnpike.

## DESIGN ALTERNATIVES

To select the most cost-effective type of trackbed, various conventional support structures were investigated. The designs for all alternatives were based on Cooper E-80 loads. Maximum deflection of the track under train loading was not to exceed 0.375 in. In addition, analysis of the tracks for E-60 loading was done to evaluate the response of a conventional ballasted track support system to presently planned loading. In the preliminary design analysis, use of a ballastless track support system was eliminated from further consideration. Unusually large differential settlement, due to the variable thickness of soft organic clay, would have made it difficult to maintain track gage tolerances.

The various types of track support systems investigated for this project are briefly described next.

### Pile-Supported Concrete Slab

This scheme consisted of a 24-in. reinforced slab supported on steel or concrete piles more than 100 ft long driven to bedrock. Although this type of construction would have resulted in the least deformable track support and negligible differential settlement, the cost of this alternative was high compared with that of other options (Table 2). To reduce the cost of this alternative, low-capacity timber piles extending to underlying Boston blue clay were also considered. The cost for this option was less than for deep piles, but it was also found to be excessive.

### Concrete Slab Supported on Shallow Soils

As a second step in the design analysis, track support by a concrete slab over the thick layers of organic clay and Boston blue clay was considered. This approach is similar to that used for the adjacent Massachusetts Turnpike. Trackbed response was analyzed by the ILLI-TRACK and ILLI-SLAB computer programs developed at the University of Illinois. The finite-element program for ILLI-SLAB employs a Winkler-type subgrade and can be used to study variable subgrade support and multiwheel loading at any position on the loaded surface. This model has been validated and

TABLE 2 COST COMPARISON OF DESIGN ALTERNATIVES

| Alternative | Estimated Cost[a] ($ millions) | Cost/ft of Track ($) |
|---|---|---|
| Concrete slab supported on piles | 12.5 | 800 |
| Concrete slab on grade | 8.5 | 550 |
| Ballast section with geotextiles | 4.8 | 310 |

[a]Costs include all drainage improvements and special treatment at ends of section and at some bridge abutments.

extensively used in various studies (*4-7*). Concrete slab thicknesses of 18, 24, and 30 in. were analyzed. The maximum deflections in all three slab thicknesses under E-60 and E-80 loading were less than the maximum track deflections of 0.25 and 0.375 in., respectively, recommended by the American Railway Engineering Association (AREA). The estimated maximum deflection and extreme tension and compression stress from bending moments for a 24-in.-thick unreinforced concrete slab over organic clay, for two tracks loaded simultaneously, were

| | *E-60* | *E-80* |
|---|---|---|
| Maximum deflection (in.) | 0.19 | 0.36 |
| Maximum bending stress (psi) | 165 | 194 |

Although bending stress and deflection of an 18-in.-thick concrete slab were less than the AREA allowable values, the subsoil information indicated that there may be localized pockets of organic clay that are more deformable than assumed. It was assumed that a 24-in.-thick concrete slab would have the necessary capacity to bridge the softer areas.

A fundamental assumption in the ILLI-SLAB analysis is that the slab cross section is uncracked and flexural stresses are resisted exclusively by concrete. This requirement is satisfied by maintaining flexural tension stress less than the modulus of rupture of the concrete. This condition was met by the 24-in.-thick concrete slab under E-80 loadings for a 4,000-psi concrete with a modulus of rupture of 474 psi. The computed flexural tension stress of 194 psi, for E-80 loadings, yielded a safety factor of 2.4. Because the flexural stress was within the maximum allowable, steel reinforcement was not required. However, longitudinal steel equal to about 0.2 percent of the concrete area was included for crack control. Transverse steel equal to about one-fourth of the longitudinal steel was provided.

As shown in Figure 2, the groundwater level through parts of the project site is near the surface of the trackbed and is required to be maintained during and after construction of the project. It was proposed to do this by adding low walls at the sides of the concrete slab and water stops in both longitudinal and transverse joints so that a watertight "boat" section would be formed.

## Transition from Pile-Supported Construction

Tracks in the adjoining SWCP section are supported on a continuously reinforced concrete slab founded on steel H-piles driven to bedrock. The SWCP section extends further below the groundwater table; water stops are used between joints in the slab and abutting concrete retaining walls to form a long boat section to limit drawdown. Within this project, 132-lb rail supported on concrete ties and a 24-in. layer of ballast over the concrete slab are being provided. Because the concrete boat section will be supported on piles, track deflection through this section will be negligible. Direct connection of the track slab-on-grade with the nonyielding boat slab would have resulted in high rail stresses and fatigue at the interface.

To provide a transition between the two projects, a 50-ft discrete segment of reinforced concrete slab was introduced. The west end of the slab is supported on six 100-ton steel piles and the east end is founded directly on the subgrade. This transition concept is similar to the one used in approach slabs for highway bridges. The length of the transition was chosen so that the leading trucks of the shortest train cars will have crossed the transition slab before the trailing trucks begin to ride on this segment. This would accomplish partial loading of the transition slab, resulting in a further reduction of track deflection between the SWCP piled structure and the slab-on-grade. The piled west end was provided with elastomeric material with keeper bars between the pile cap and the concrete slab so that rotational movement of the transition slab is possible.

As shown on Figure 2, the 1,130-ft section between Shawmut Avenue and Harrison Avenue Bridge is directly underlain by stiff yellow clay. The ILLI-TRACK analysis showed that 36-in. conventionally ballasted track construction on stiff clay would be comparable to slab-on-grade on soft organic materials. Because the groundwater level in this area was high, cut-off walls on both sides of the ballasted track section were specified to protect the track support structure from inflowing groundwater. Steel sheet pile or slurry cut-off walls were proposed to penetrate at least 15 ft below the bottom of the excavations.

## Geosynthetic Ballast Mat over Shallow Soils

The cost of the concrete slab-on-grade track support was estimated at $8.5 million for a 3,095-ft length containing five tracks. It was therefore decided to explore the use of conventional ballasted track support with suitable geosynthetics for added stability, improved drainage, and groundwater level control. A typical cross section is shown in Figures 3 and 4. Because the tracks through the project have

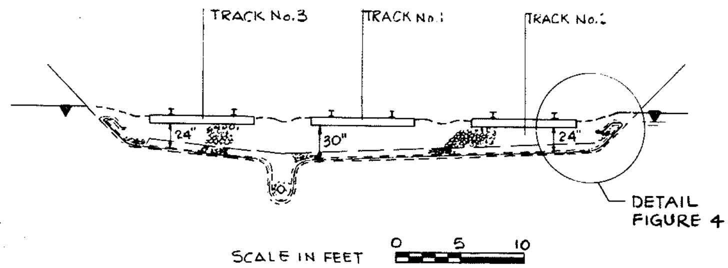


**FIGURE 3 Typical cross section.**

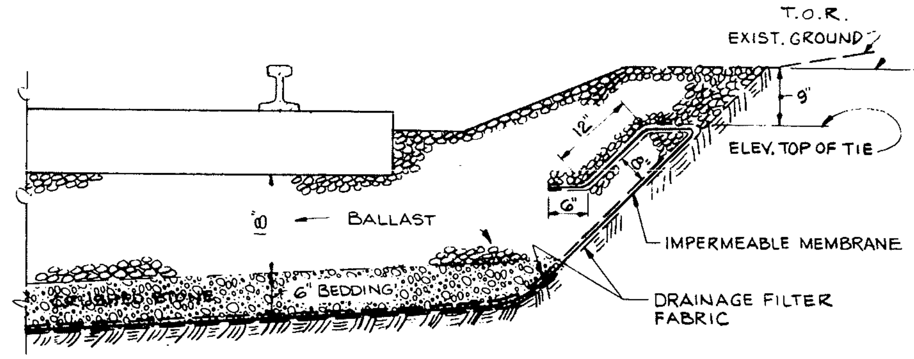


**FIGURE 4 Detail.**

been directly supported on the existing subsoils in the past, some improvement of the soil strength and stiffness over the years should have resulted. In the design and construction of the new track structure an attempt was made to preserve the in situ properties of subsoils that had been subjected to train live loads in the past. Excavation for installation of new ballast was planned to minimize disturbance of the existing subgrades.

### *Analysis of Deflection Under Train Loading*

The analysis of rail deflection was based on a minimum depth of 24 in. of ballast below the bottom of ties. Estimated live loads were based on AREA criteria for E-80 and E-60 train loads on embankment fills. The following vertical stress and corresponding deflections were estimated in the organic clay for a 24-in.-thick ballast:

| | *E-80* | *E-60* |
|---|---|---|
| Live and dead load (ksf) | 1.90 | 1.20 |
| Estimated deflection (in.) | 0.455 | 0.300 |

These track deflections are about 15 percent higher for the E-60 loading and 18 percent higher for E-80 loading than recommended for conventional ballasted tracks. The actual ballast thickness specified is generally greater than the 24 in. assumed in the estimate, ranging between 30 in. in the middle of the center track to a minimum of 24 in. under the outer rails of the outside tracks. The increased ballast thickness will further reduce the estimated deflection.

This scheme has a large cost advantage over the other two schemes (Table 2). Although a ballast-supported track will result in somewhat higher track deflection than other alternatives, that scheme will meet the functional requirements at much lower cost. Ballast strain hardening due to repeated dynamic train loadings in conjunction with periodic track resurfacing during the initial phases of service should further improve track performance.

### *Transition Conditions*

The critical transition in this alternative is also at the interface with the SWCP section on the west end. The concrete slab transition used with this scheme is similar to that previously described for the slab-on-grade alternative. It is shown in Figure 5. However, the first 80 ft of the ballast section east of the transition will be reinforced with a single layer of Tensar geogrid to minimize localized higher track deflections due to the change in track support. A second layer spans the transition for 30 ft. The location of the geogrid is shown in Figure 6. The use of geogrid reinforcement is an experimental attempt to provide a stiffer subgrade modulus in this transition under dynamic train loading.

The Tensar grid structures are produced using a manufacturing technique that orients the long chain molecules within the polymers and increases the tensile strength of the polymer. The geogrid used was Tensar SS2 that has the physical and mechanical properties (*8*) given in Table 3.

The geogrid is specified to be installed in the ballast at a depth of 18 in. below the ties so that it is below the 12- to 14-in. undercutting depth of ballast cleaning machines. In essence, the transition from the SWCP will be accomplished in two stages: initially by a 50-ft section of concrete slab supported at the west end on 100-ton piles and then by an additional 80-ft segment of ballast section reinforced with geogrids.

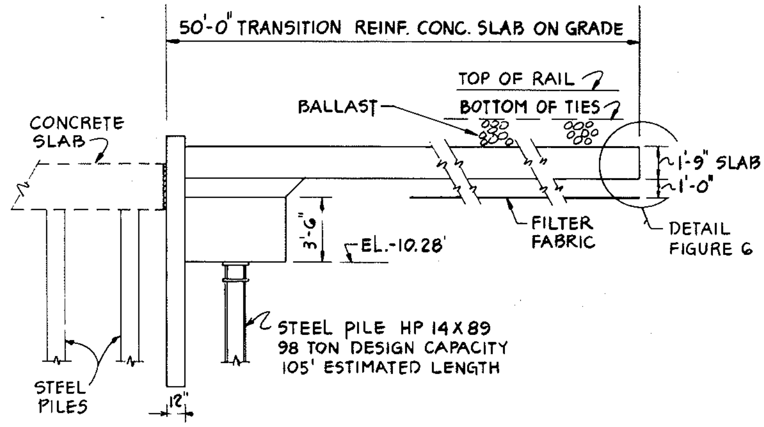


**FIGURE 5** **Concrete slab transition.**

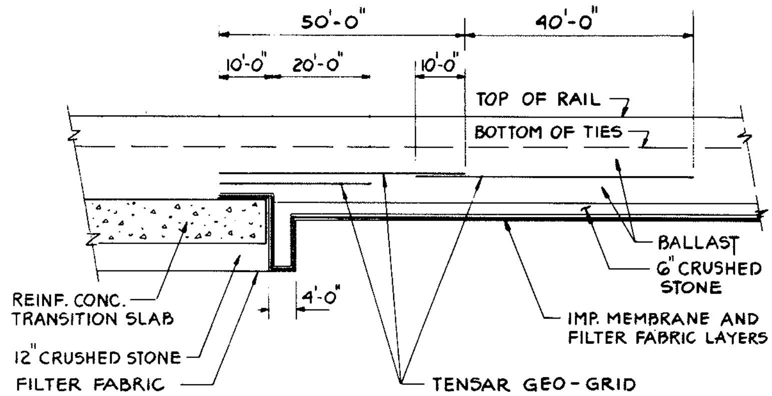


**FIGURE 6** **Concrete slab and ballast transition.**

TABLE 3 PHYSICAL AND MECHANICAL PROPERTIES OF TENSAR SS2 GEOGRID

| | |
|---|---|
| Roll width (ft) | 9.8 |
| Roll length (ft) | 164 |
| Weight (lb) | 102 |
| Aperture size (in.) | 1.0[a]/1.3[b] |
| Thickness of rib (in.) | 0.04 |
| Thickness of junction (in.) | 0.15 |
| Tensile strength (lb/ft) at | |
| 2% strain | 590/1015 |
| 5% strain | 950/1690 |
| Ultimate | 1170/2100 |
| Initial tangent modulus (kip/ft) | 30.6/68.1 |

[a]Machine direction.
[b]Cross machine direction.

### *Dissipation of Static and Dynamic Pore Pressure*

The proposed bottom of ballast of this on-grade track support will be below the existing groundwater level between Elevations -6 and 0 over most of the 2,425-ft length. The city of Boston code requires that the groundwater level not be lowered below Elevation -0.65. To keep the ballast dry and maintain the surrounding groundwater at prevailing levels, an impermeable geomembrane will be installed beneath the full length of the track structure. The ends of the impermeable geomembrane are extended above the water table on both sides to form a flexible boat section shown in Figures 3 and 4.

Because the dynamic load applications will tend to compress the water-saturated subsoils, the associated excess water pressures need to be rapidly dissipated to maintain the effective stresses. The underlying organic soil and Boston blue clay have an estimated permeability of $2 \times 10^{-6}$ ft/min. Assuming an excess water head of 2 ft and a flow path of one-half of trackbed width or 25 ft, the required in-plane transmissivity and cross-plane permittivity are estimated as follows:

1. Estimate maximum flow into the geotextile under train loading using a 20-ft-thick clay layer and Darcy's formula:

$$q = kiA \tag{1}$$

where

$q$ = flow rate ($ft^3$/min/ft of track),
$k$ = permeability coefficient (ft/min),
$i$ = hydraulic gradient (ft/ft), and
$A$ = area of fabric.

Average excess hydraulic head at middepth of clay layer from E-80 train live load = $(1{,}900 - 2 \times 150)/(2 \times 62.4) = 12.8$ ft and

$$q = 2 \times 10^{-6} \times (12.8/20) \times 25 \times 1 = 3.2 \times 10^{-5}\ ft^3/min/ft$$

2. Calculate required in-plane fabric transmissivity (*9*) for static loading:

$$q = K_p i_p W t \quad (2)$$

$$\theta = K_p t \quad (3)$$

where

$K_p$ = permeability coefficient in the plane of the fabric (ft/min),
$i_p$ = hydraulic gradient in the plane of the fabric (ft/ft),
$W$ = width of fabric (ft),
$t$ = thickness of fabric (ft), and
$\theta$ = transmissivity ($ft^2$/min).
$i_p$ = 2 ft (maximum)/25 = 0.08

and

$$\theta = q/i_p W$$
$$= 3.2 \times 10^{-5}/0.08 \times 1 = 4 \times 10^{-4} \text{ ft}^3/\text{min/ft} \quad (4)$$

3. Factor of safety = $\theta$ (fabric)/$\theta$ (required)

$= 8 \times 10^{-3}/4 \times 10^{-4} = 20$

The required in-plane transmissivity for dynamic train loading was estimated to be 10 times the static value due to fabric compression and an average 12.8-ft increase in excess water head within the soil. This is equivalent to the full AREA dynamic E-80 engine loading distributed over the trackbed as a short-term pore pressure increase.

4. Calculate required cross-plane fabric permittivity (*10*):

$$\psi = k/t = q/(h \times A)$$
$$q = KiA = 2 \times 10^{-6} \times 12.8/20 = 1.28 \times 10^{-6} \text{ ft}^3/\text{min}/\text{ft}^2 \times 60 \text{ sec/in.} = 7.7 \times 10^{-5} \text{ ft}^3/\text{sec}/\text{ft}^2$$

and

$$\psi = 7.7 \times 1^{-5}/0.2 \times 1 \times 1) = 3.8 \times 10^{-4} \text{ min}^{-1}/\text{ft}^2$$

5. Check against actual permittivity of the available geotextiles:

$k$ = 6 in./sec
$t$ = 0.21 in.
$\psi$ = $k/t \times 28.6 \text{ sec}^{-1}/\text{ft}^2$

Factor of safety = $28.6/3.8 \times 10^{-4} = 7.5 \times 10^4$

The fabric selected is estimated to have a gradient ratio of less than 3.0 for these subgrade soils to limit clogging and has an initial permittivity well above that required. Hoechst Trevira 1155 spunbound polyester was used. It has the following properties (*11*):

| *Property* | *Value* |
|---|---|
| Thickness unloaded (in.) | 0.21 |
| Weight (oz/$yd^2$) | 16 |
| Puncture strength (lb) | 225 |
| Burst strength (psi) | 750 |
| Effective opening size (microns) | 100-140 |
| In-plane transmissivity ($ft^2$/min) at normal loading of 1,900 psf | $8 \times 10^{-3}$ $ft^3$/min/ft |
| Cross-plane permittivity ($min^{-1}$) | 28.6 $sec^{-1}/ft^2$ |

## DESIGN DETAILS

### Track Support System

The track support system is made up of 133-lb RE rails supported on 7-in. × 9-in. × 8-ft 6-in. timber crossties spaced at 19-in. centers. The crossties are supported on ballast ranging from 24 in. under the outer tracks to a maximum of 30 in. between Tracks 1 and 3. An impervious membrane of nylon-reinforced rubber, 36 mils in thickness, is being used over the subgrade soils to prevent seepage of groundwater directly into the ballast. The impervious geomembrane consists of a minimum of three plies of black Hypalon and two plies of nylon reinforcement. The number of seams to be field installed is being minimized by planning to expose sections of the subgrade to match the factory-available lengths of the impervious liners. The nylon reinforcing fabric (scrim) is 10 by 10, 1000-denier, industrial grade.

The track ballast being used on the NECIP projects is in accordance with AREA Specification 24, which is generally coarse graded with a maximum size of 3-1/2 in. The impermeable membrane was protected from damage by covering it with a layer of 16-oz/$yd^2$ filter fabric before a 6-in. layer of crushed stone with a maximum size of 1/2 in. was placed. The 16-oz/$yd^2$ drainage filter fabric layer discussed in the previous section is being provided beneath the geomembrane to permit drainage of groundwater higher than the top of ties by spillage into the ballast drainage system as shown on Figure 4. The drainage fabric will also dissipate excess hydrostatic pressure, which may be generated from dynamic train loadings. The in-plane transmissivity of the drainage filter fabric under dead and live loading of 1,900 psf is sufficient to provide effective dissipation of the pore pressures (*12*). Because the subsoils are cohesive clays, it was not necessary to provide a bedding sand layer between the filter fabric and the subsoils. Disturbance of subsoils will be minimized by not permitting the operation of excavation equipment on the subgrade. The extent of excavation each day is limited to the distance that can be covered with new construction within 24 hr.

### Site Drainage

The site was previously drained by a system of surface inlets and a closed drain line located near the centerline of Track 3.

The drain line ranged between 24 and 52 in. in size. The old drainage lines terminated in a reservoir from which water was pumped into the tidal basin. The old drainage system operated poorly and much of it will be affected by the new scheme.

Drainage of the ballast section from within the envelope of the impervious geomembrane will be through 12-in. perforated collector drains in a depressed part of the ballast section. The collector drains are fed at intervals into a main storm drain, which is located below the impervious geomembrane. The storm drain will direct all of the rainfall runoff from within the impermeable ballasted boat section to a new pump station near Washington Avenue. Drainage water is pumped to Fort Point Channel via an 18-in. force main.

Drainage pipe penetrations through the impermeable membrane are heat sealed using a boot of Hypalon between the hole and the pipe to prevent leakage into the envelope. The impermeable membrane extends up the side of the boat section to the level of the top of the railroad ties as shown in Figure 4. Examination of water levels and quantities of flow before and during construction indicates that groundwater will flow into the ballasted section only occasionally when water levels rise above this level.

## MONITORING AND INSTRUMENTATION

Five piezometers have been installed to monitor static water levels in the subgrade soils during and after construction. Three vibrating-wire piezometers (Irad gauge model PWS-25) were installed directly beneath the impermeable membrane to measure the rate of dissipation of pore water pressure beneath the fabric during dynamic train loading. Two additional vibrating-wire piezometers were installed 4 ft deeper to provide control readings for comparison with those of the shallow piezometers.

Control points were established on the track structure to permit measurement of permanent track deformation. Deformation of track under train loading will also be observed by photographing scales mounted on the track structure with an intermediate target as a control elevation. Rapid-sequence photography will permit measurement of deflection while a train is passing.

Track monitoring during 1986 and 1987 will provide information to confirm design assumptions.

## CONCLUSIONS

Use of newly developed methods for estimating track deflections under train loading and use of layers of geosynthetics in a relatively new way have permitted construction of track structures over low-strength organic clays at a much lower cost than the more conventional pile-supported and concrete slab-on-grade alternatives. This system will maintain the present groundwater level. Special provisions were made for a smooth transition between the adjacent pile-supported slab and the geosynthetic envelope section. The transition included experimental use of a geogrid reinforcement within the ballast section. The performance of this section under train loading has been observed since construction. Preliminary results indicate satisfactory performance.

## ACKNOWLEDGMENTS

DeLeuw, Cather/Parsons were the NECIP general designers. Mueser Rutledge Consulting Engineers were the general foundation consultants. The section designer for this project was H.W. Lochner, Inc., with their subconsultant Teng & Associates. Marshall Thompson of the University of Illinois performed the ILLI-SLAB and ILLI-TRACK analyses, and Robert Koerner of Drexel University assisted in the design of the geosynthetics.

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