TRANSPORTATION RESEARCH RECORD 1006

Track Design and Construction

TRB
TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1985

Transportation Research Board 1006
Price $8.60
Editor: Edythe Traylor Crump
Compositor: Harlow Bickford
Layout: Theresa L. Johnson

modes
2 public transit
3 rail transportation

subject area
21 facilities design

Transportation Research Board publications are available by ordering directly from TRB. They may also be obtained on a regular basis through organizational or individual affiliation with TRB; affiliates or library subscribers are eligible for substantial discounts. For further information, write to the Transportation Research Board, National Research Council, 2101 Constitution Avenue, N.W., Washington, D.C. 20418.

Printed in the United States of America

Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.
Track design and construction.

(Transportation research record; 1006)
1. Railroads–Track–Addresses, essays, lectures. I. National Research Council (U.S.) Transportation Research Board. II. Series.
TF7.H5 no. 1006 380.5 s [625.1'4] 85-16327
[TF240]
ISBN 0-309-03816-2 ISSN 0361-1981

Sponsorship of Transportation Research Record 1006

GROUP 2–DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES
Robert C. Deen, University of Kentucky, chairman

Railway Systems Section
George H. Way, Jr., Association of American Railroads, chairman

Committee on Track Structure System Design
Robert E. Kleist, Association of American Railroads, chairman
Ben J. Johnson, Railroad/Transit Consultant, secretary
Gilbert L. Butler, Louis T. Cerny, William R. Hamilton, Amir N. Hanna, John B. Heagler, Jr., J. C. Hobbs, William P. Hofmann, Thomas B. Hutcheson, Arnold D. Kerr, Mohammad S. Longi, W. Scott Lovelace, Howard G. Moody, James F. Pearce, Robert H. Prause, Gerald P. Raymond, Alfred E. Shaw, Jr., Charles L. Stanford, Daniel H. Stone, Marshall R. Thompson, Erland A. Tillman, George H. Way, Jr., J. W. Winger

Edward J. Ward, Transportation Research Board staff

The organizational units, officers, and members are as of December 31, 1984.

Contents

Addresses of Authors

Greif, Robert, Transportation Systems Center, U.S. Department of Transportation, Kendall Square, Cambridge, Mass. 02142
Keffler, Arthur J., De Leuw Cather & Company, 525 West Monroe Street, Chicago, Ill. 60606
Lamson, So'n T., Canadian Institute of Guided Ground Transport, Queen's University, Kingston, Ontario K7L 3N6, Canada
Lee, Harvey S., Transportation Systems Center, U.S. Department of Transportation, Kendall Square, Cambridge, Mass. 02142
Mahon, Vincent P., Bay Area Rapid Transit District, 800 Madison Street, Oakland, Calif. 94607
Pickrell, Don H. John F. Kennedy School of Government, Harvard University, 79 John F. Kennedy Street, Cambridge, Mass. 02138
Raymond, Gerald P., Queen's University, Kingston, Ontario K7L 3N6, Canada
Scott, J. Frank, CN Rail Research Centre, 3950 Hickmore Avenue, St. Laurent, Quebec H4T 1K2, Canada
Smith, Michael E., Association of American Railroads, 1920 L Street, N.W., Washington, D.C. 20036
Weinstock, Herbert, Transportation Systems Center, U.S. Department of Transportation, Kendall Square, Cambridge, Mass. 02142

Research on Railroad Ballast Specification and Evaluation

GERALD P. RAYMOND

ABSTRACT

Research leading to recommended procedures for ballast selection and grading are presented. The ballast selection procedure is also presented and offers a sequential screening process to eliminate undesirable materials. The procedure classifies the surviving ballasts in terms of annual gross tonnage based on 30 tonne (33 ton) axle loading and American Railway Engineering Association grading No. 4. The effect of grading variation and its effect on track performance is also presented.

From 1970 to 1978 Transport Canada Research and Development Centre, Canadian National Railway Company, and Canadian Pacific Limited cosponsored a research program at Queen's University through the Canadian Institute of Guided Ground Transport to investigate the stresses and deformations in the railway track structure and the support under dynamic and static load systems. The findings and recommendations regarding the specification for evaluating processed rock, slag, and gravel railway ballast sources are summarized in this paper. Comments are included about the new Canadian Pacific Rail ballast specification, which was partially based on the findings presented by Raymond et al. (1).

The selection of the top ballast (hereafter referred to as ballast) used for railway track support is of major importance in establishing and maintaining the characteristics of the track response and, consequently, the riding quality. For ballasted track, an elastic, noncemented, stable and weather-resistant ballast bed, well laid and compacted on a stable, compact subballast and subgrade, is the first condition for low maintenance expenditures.

Ballast must be capable of withstanding many forces. Extremely large cyclic loadings, vibrations of varying frequencies and intensities, repeated wetting and drying involving crystallization of rain-dissolved soluble salts, plus other factors caused deterioration of the ballast. Ballast must also be easy to handle during maintenance. These requirements are invariably conflicting and require considerable judgment in aggregate selection for railway ballast. Some of the different requirements that should be clearly understood in making proper economic selection from available aggregate sources are outlined here.

ROCK MATERIAL

Rock consists of an intergrowth of one or more minerals. These minerals are chemical compounds and have both a specific crystal structure (or arrangement of atoms) and a specific chemical composition. Note that two or more different minerals may have the same chemical composition but will have different crystal structures. The way in which the minerals of the rock are intergrown is called the texture of the rock.

Rock names are based on the minerals that constitute the rock and the texture of the rock. Thus, two rocks of identical mineral composition having different textures would have different names. Mineral identification is generally based on simple tests that involve hardness, cleavage, luster, streak color, and chemical composition. From a ballast performance viewpoint, mineral hardness, generally based on Mohs hardness scale, is of considerable importance.

Particular geological processes give rise to three rock types, igneous, sedimentary, and metamorphic. Rock specimens may be used to classify the rock type and also to provide information about the geological history of the area where it was located. This information is valuable to the ballast selection process.

Igneous Rocks

Igneous rocks are formed from a cooling magma (a very hot, molten liquid of silicates and other compounds). The rate at which the magma cools determines the texture of the igneous rock formed. The composition of the magma determines mineralogical constituents of the rocks. These two properties, texture and composition, are the basis for the classification of igneous rocks and provide a basis for their identification.

Extrusive igneous rocks are formed when the magma is poured out onto the earth's surface. Extrusive magma solidifies rapidly to form a glassy rock or an extremely fine-grained rock.

Intrusive igneous rocks are formed when the magma cools within the earth. As such it cools slowly allowing coarser grained rocks to form. In general, the closer the intrusion comes to the earth's surface, or the smaller the size of the intrusion, the more rapidly it will cool, and the finer the grain size of the minerals will be.

Sedimentary Rocks

Under normal weathering processes, all rocks slowly disintegrate to form clay, silt, sand and gravel, plus dissolved materials, which are eventually deposited. Over time, these unconsolidated deposits may become compacted and cemented together to form clastic sedimentary rocks, whereas the dissolved materials may precipitate to form chemical sedimentary rocks.

Metamorphic Rocks

Rocks formed under one set of temperature, pressure, and chemical conditions and then exposed to a different set of conditions may undergo structural and chemical changes without melting that produce rocks with different textures and new minerals. Typically, this process results in the linear orientation of minerals along well-defined planes of weakness. This process is known as metamorphism and results in metamorphic rocks.

Comments

Although not an absolute guide, the rock formation processes outlined in the preceding sections provide a useful rough-screening criteria for ballast applications. In general, fine-grained igneous rocks are preferable to either sedimentary or metamorphic rocks. Medium to coarse-grained igneous rocks and hard, well-cemented sedimentary rocks are still preferable to most metamorphic materials.

AGGREGATE SELECTION

General

The increasing cost of track and roadbed maintenance has made the selection of an appropriate aggregate for each ballast application a matter of considerable financial importance. It is clearly not cost-effective to haul a first class ballast long distances to surface a little-used branch line, and it is equally inappropriate to use an inferior ballast material on a main line track subject to a high density of heavy, fast traffic. The aggregate selection procedure must permit the decision maker to identify the physical characteristics of a ballast so as to assess the differential aspects of the material with respect to other available materials with similar properties and to evaluate, in financial terms, the expected costs and benefits from the use of each ballast.

To perform well in track, the aggregate for ballast must be tough enough to resist breakdown through fracturing under impact, and must be hard enough to resist attrition through wear at the ballast particle contacts. It must be dense enough so that it will have sufficient mass to resist lateral forces and anchor the ties in place. The aggregate must be resistant to weathering so that weakening of the ballast does not occur from crystallization or acidity of impurities dissolved in rainwater or from daily or seasonal fluctuations in temperature or other weathering processes.

It must also be resistant to the chemical degradation resulting from the action of rainwater on foreign source fines. For example, trace elements such as sulphur in coal are highly likely to increase the acidity of any moisture trapped within the ballast. This acidity will cause solution weathering of the aggregate, particularly limestones.

All aggregate material may be expected to degrade to some extent with time. Because of this it is strongly recommended that aggregates be subjected to some form of petrographic examination to assess the long term effect of the degradation and production of ballast origin fines on the ability of the ballast to remain free draining and elastic. Where aggregate is composed of fine-grained minerals whose identification is difficult to ascertain in a hand-held examination, petrographic thin section analysis is strongly recommended. Thin section examination as well as a conclusive determination of rock type, mineralogy, and structure, also establishes whether microfractures exist within the aggregate source and whether former microfractures have been weakly cemented with secondary minerals that might weather and soften quickly.

Origin and Aggregate

Quarried stone ballast should be obtained from competent strata of reasonable thickness. The extent of the rock deposit should be sufficient for economic ballast production. A large variety of rock types are used as ballast. In general, the fine hard mineral-grained unweathered aggregates make the best ballast. These include igneous rock types such as rhyolite, andesite, and basalt. Second best are the coarser grained igneous rocks such as granite, diorite, and gabbro, along with the hard mineral-grained well-cemented sedimentary rock and hard mineral-grained metamorphic (or transformed) rock such as quartzite. Less satisfactory but often more commonly used because of their cheaper production cost and wider availability are the sedimentary rock types such as limestone, dolomite, sandstone, and siltstone.

Rock types such as shale and slate, which result in flaky or elongated particles, should not be permitted because these shaped particles do not result in good interlocking, particularly when subjected to vibrations. Similarly, sedimentary and metamorphic rock types that contain visible quantities of secondary minerals, which weather quickly, should be rejected. A typical example would be small quantities of pyrites, which oxidize to produce a ferric compound and then sulphuric acid, which is highly corrosive to the metallic parts of the track structure.

Where cobble and pebble size gravel is to be used to produce broken stone, it is recommended that cobble size (plus 75 mm) particles should first be sorted by the use of a coarse bar mesh. The material retained is then used as the source for crushing.

Where slag material is permitted, air cooling is generally required. No molten or meltable material should be present, and slags from hematite castings are normally prohibited. Slag should also be free from splintered or glassy components. Before the use of slags, it is generally worth examining their chemistry because their properties as ballast aggregate may often be cheaply improved by the addition of silica sand before cooling.

Petrological and Geological Requirements

A visual petrological analysis, using a hand lens or low-powered (stereo) microscope, of freshly broken rock samples is of major value in the selection of a suitable quarry. In addition, examination of the sand and smaller sized particles produced during physical testing, such as the Los Angeles abrasion test, gives an indication of the angularity and permeability of future ballast breakdown. If the minerals are fine-grained and hard, supplemental examination by means of thin section analysis and possible chemical analysis may be required. In general, the petrographer, if experienced, should be given freedom to decide the extent of the testing required.

The information obtained from the petrological analysis should be documented under those of the following headings that are appropriate to the aggregate under examination:

- Rock type of particles along with percentages where more than one rock type is present.
- Mineralogy of rock types including proportions present in each rock type.
- Texture, which should include comments on grain size, shape, orientation, plus mutual relationships and matrix material between minerals.

• Structure of rock identifying bedding planes, fracture planes, cleavage planes, and foliation planes.

• Mechanical properties from a geological point of view, including Mohs' hardness of the minerals; induration or compactness, including porosity of the rock; possible strength and brittleness, including comments on possible or existing types of fracture weaknesses; and shape and roundness of particles. Estimate of specific gravity of rock.

• Chemical properties defining existing chemical weathering and potential chemical weathering. Where known, this may be related to the environmental pollution of the locale where the aggregate is to be used.

• Properties of sand and smaller sized particles resulting from induced fracture of rock including shape, probable effect on permeability, and their susceptibility to solution and precipitation weathering.

• Suggestions with explanations for any additional engineering test not included in the testing specification.

• Estimation of expected results of engineering test results with explanation in terms of petrological features.

• Recommendations along with summary of important petrological features.

Engineering Assessment of Weathering

A number of tests are available to assess the potential of aggregates to degradation or weakening due to weathering. These include

• Soundness testing where rock particles are alternatively immersed and then dried using a salt solution. The American Railway and Engineering Association (AREA) specification calls for a solution of sodium sulphate although the author has found magnesium sulphate more appropriate, and this has been adopted in the Canadian Pacific Railway specification. The test is considered by many to be only applicable to assessing the expansive pressures generated from freezing water; however, it also provides an indirect assessment of the resistance of the aggregate to crystal growth from dissolved pollutants in rainwater through the use of a standard salt solution (e.g., magnesium sulphate). For example, the soundness test is the main test used in the United Kingdom to assess the resistance of building stone to the crystallization of soluble salts contained in rainwater that penetrates within the pores of the stone (2). In this regard building stones are observed to degrade on their surfaces depending on their degree of exposure to rainwater, the extent of atmospheric pollution, and their climatic exposure (inland or coastal and frost or no frost locations). Note that the test has been found applicable in no frost zones of the United Kingdom subject to different degrees of atmospheric pollution. Clearly the importance of the soundness test will be more significant in regions where both atmospheric pollution and freeze-thaw are greatest.

If weathering occurred independently of other factors, a higher standard for aggregates would need to be specified for branch lines than for main lines where ballast breakdown from loading is clearly less. Unfortunately, these processes are not independent, and any weakening from weathering allows accelerated breakdown from the loading environment. Thus higher standards are generally specified for ballast used in main line tracks than those used in branch lines. Because of the importance of ballast stability, the 1984 Canadian Pacific specification requires a maximum breakdown of 1 percent for primary main line track [using continuous welded rail (CWR)] rising to 3 percent for minor branch lines. This is considerably more demanding than presently required by the AREA specification (i.e., 7 percent for use with both wood and concrete ties).

• Absorption testing where oven dry ballast particles are immersed in water to measure their surface absorption. The test indicates the ability of the particles to retain water that would freeze and cause degradation during daily freeze-thaw cycles. Its importance is related to the rainfall during the time of year that freeze-thaw cycles occur and the extent of such weather. It should be noted that in the United States, many of the northernmost states and the Canadian provinces have less cyclic freeze-thaw weather than many of those U.S. states regarded as climatically warm. Again, because of the importance of ballast stability, the 1984 Canadian Pacific specification requires a maximum increase in weight of dry aggregate particles after submersion in water of 0.50 percent for primary main line track rising to 1 percent for minor branch lines. Again this is considerably more demanding than the AREA specification (unlimited for use with wood ties and 1.5 percent for use with concrete ties). Absorption may also indicate the susceptibility of an aggregate to chemical attack from any detrimental pollutants in rain water.

Stability

The properties of an aggregate that affect the holding power of a ballast are measured by

• The bulk specific gravity of the rock that is related to the unit weight of the processed ballast. The unit weight is a major factor in determining both the vertical and lateral holding capacity of the track; the holding capacity increases as the track mass increases. The 1984 Canadian Pacific specification requires a minimum of 2.60 for all lines.

• The shape and texture of the particles, which, theoretically, is a production factor. But clearly if the aggregate source is pebbles and cobbles, the source particles must be large enough to result in sufficient fractured faces after crushing. The Canadian Pacific Railway specification defines a fractured face as a freshly exposed surface whose maximum length is at least one-third the length of the maximum particle dimension and whose maximum width, measured perpendicular to the maximum fracture length direction, is at least one-quarter the maximum particle dimension. A fractured particle is defined as having not less than three fractured faces whose planes intersect at greater than 45 degrees.

Similarly, aggregate that is used to produce ballast particles that are not elongated or flaky must not exhibit excessive schistocity or slate-like structure. Examination of the laboratory crushed aggregate is a helpful but not a definitive guide to what is likely to be produced.

Load Resistant Characterization

Without exception every ballast specification attempts to assess the quality of the ballast particles under loading. Ideally this quality measure should reflect both the hardness and toughness of the mineral-bonding matrix making up the ballast particles or parent rock. The typical tests that are performed on a mix of the particles by railroads worldwide include impact testing, crushing value testing, Los Angeles abrasion testing, and the like. These tests, unfortunately, measure mainly the toughness of an aggregate and are only slightly affected by the hardness of its minerals. Satisfactory correlation has been obtained from a comparison of the results of any two of the tests (3). To make comparative measures of the toughness of different rocks it is only necessary to use one of these tests. The Los Angeles Abrasion (LAA) test is almost universally accepted in North America. Even so, as observed from the results in Table 1, this test conducted on different sized particles results in different LAA values as does the use of track-used particles instead of freshly crushed aggregate. It is therefore important to compare ballasts on a standard size of freshly crushed particles, for example, 19 to 38 mm (0.75 to 1.5 in.), irrespective of the maximum particle size of the ballast being used. For this reason the Canadian Pacific specification requires the exclusive use of ASTM C535, Grading 3 for the LAA evaluation.

A second factor that needs understanding in relation to toughness evaluation is that for rocks of similar field rating, the impact from the LAA steel ball charge will increase slightly as the hardness of the mineral grains increases, resulting in a higher LAA breakdown. However, the field breakdown from the harder mineral rock is often slower because less powdering occurs at the points of contact between particles, and the broken particles are more angular and coarser resulting in a slower rate of track fouling. Comparisons based on LAA alone that measure primarily rock toughness would, therefore, result in decisions contrary to field performance, where better performance of ballasts with equal LAA values is noted from the ballasts composed of harder mineral grains. This is particularly evident with the use of concrete ties, where the concrete is made from silica sands having minerals of Mohs' hardness of 6 or more and are thus more abrasive than wood. Hard mineral ballasts have now been generally adopted (4). Indeed, the AREA concrete tie specification specifically excludes carbonate ballasts (limestones composed of calcite, Mohs' hardness = 3; and dolomites, Mohs' hardness = 3.5-4) because of their recorded poor performance with concrete ties. Because rocks are composed of minerals, each having different hardness values, it was necessary to develop a method of assessing the rock's overall hardness. This was achieved by the adaption of an autogeneous grinding test known as the Mill Abrasion (MA) that is commonly used in the mining industry to access the grindability of ores (5). Rocks having a predominance of hard minerals were noted to have low MA values. Rocks having similar minerals were also noted to have a variation in values based on their degree of induration or compactness, which added to the significance of the test in terms of assessing rock hardness.

Such observations mean that a proposed ballast performance load environment class system, for financial costing purposes, was assessed from a combination of the LAA value and the MA value. Laboratory load classification research resulted in an aggregate index (I_a) renamed in the Canadian Pacific specification an abrasion number (N_a) given by

$$I_a = N_a = LAA + 5\ MA \quad (1)$$

Research by Canadian Pacific has related this value to the observed life of ballast from the loading environment alone. Ballasts that weathered or were badly fouled from foreign sources were eliminated from the study. For AREA No. 4 graded ballast, the cumulated short tons of 30 tonne (33 ton) axles to result in breakdown to the point where the ballast needed renewal was found to be given by

$$\text{Life} = 10^6 \exp(8.08 - 0.0382\ N_a) \text{ short tons} \quad (2)$$

Test data from this author's work on different aggregates are shown in Figure 1 with a petrological description given in Table 2 that allows an estimate of aggregate life for those organizations not having an MA apparatus. It must be clearly understood that

TABLE 1 Los Angeles Abrasion Result Finding

TEST	METHOD	RESULT	AGGREGATE
C535	GRADING 3	13.9	Andersite freshly crushed
C131	GRADING A	17.2	Andersite freshly crushed
C131	GRADING B	16.6	Andersite freshly crushed
C131	GRADING C	19.6	Andersite freshly crushed
C131	GRADING D	24.6	Andersite freshly crushed
C535	GRADING 3	34-39*	Granite freshly crushed
C535	GRADING 3	25	Granite particles already used in LAA test to obtain above result

* Range of three tests on same source.

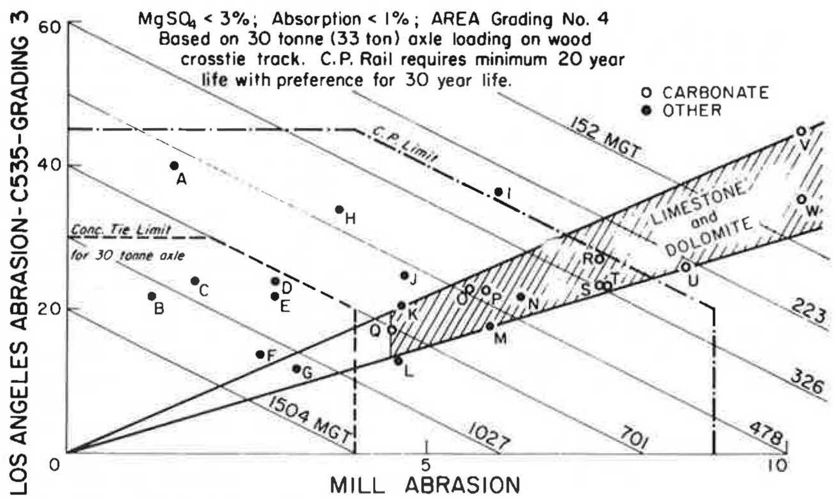


FIGURE 1 Ballast field life predictor model and laboratory data points.

TABLE 2 Hand Specimen-Petrological Examination

Material	Description
Kenora granite	Granodioritic gneiss—mostly plagioclase feldspar, quartz horneblende. Hard but weak to medium toughness. Prone to fracture on foliation.
Sudbury slag	Two-phase material from smelting of nickel ore—mostly silicates; very hard and relatively tough. Fine material is about 90 percent angular fragments; should remain highly permeable.
Noranda slag	Very hard, tough material—a by-product from the smelting of copper-zinc sulphide ores. Fine material is predominantly angular fragments—should remain highly permeable.
Medicine Hat	Gravel containing quartzite, diabase, and granite predominant; tough, medium hard, isotropic material; should retain permeability.
Suicide Creek	Creek gravel, mostly granodirite with some gabbro, granite; of medium hardness and toughness; fractures on grain boundaries; material should be permeable, but could produce clays from weathering of feldspars.
CEM-1	Initially classified as fine- to medium-grained igneous rock then identified by thin section as metamorphosed quartz diorite porphyry. Two sets of joints present with substantial weathering and secondary mineral formation on the joint planes. Fractures along joints would produce 25 to 75 mm (1 to 3 in.) blocky fragments. Mineral matrix appears sound. If main body of rock is sound and unweathered, aggregate should make excellent ballast.
Marmora trap rock	Actually epidote skarn; generally hard, but composition varies considerably. Otherwise quite tough. Mostly calcium-magnesium-iron silicate. Chemical weathering of fines could produce clay minerals. Otherwise should remain permeable.
CP-2	An isotropic, equi-granular grey granite, composed of plagioclase feldspar, potassium feldspar, biotite, horneblende, and quartz. Material is tough and hard and should remain highly permeable.
Steel (OH) slag	About 20 percent of the slag is a massive grey stone material which, although the surface is sometimes coated with a relatively soft scale, is relatively hard, tough, of high density, angular shape, and exhibits no obvious planes of weakness. The other approximately 80 percent consists of pieces exhibiting either one or more of the following features: (a) widespread vesicles, (b) substantial fine-grained crystal growth, (c) inclusions of several materials including a soft carbonacious substance of about the consistency of coke, (d) spherical particles about 0.50 mm in diameter resting within the larger vesicles, and (e) a substantial degree of rust. Although much of the material grains are hard, the aggregate is weak, crumbling on impact and unsuitable on its own for ballast. About 5 percent of the OH material is either brick or other nondescript material presumably some flux derivative. These materials will not make good ballast.
Brandon gravel	River gravel; mostly gneiss of varying types; weak, medium hard, will fracture along gneissosity; permeability will be low.
Steel (BOF) slag	About 60 percent of the slag is a massive grey stone material which, although the surface is sometimes coated with a relatively soft scale, is relatively hard, tough, of high density, angular shape, and exhibits no obvious planes of weakness. The other approximately 40 percent consists of pieces exhibiting either one or more of the following features: (a) widespread vesicles, (b) substantial fine-grained crystal growth, (c) inclusions of several materials including a soft carbonacious substance of about the consistency of coke, (d) spherical particles about 0.50 mm in diameter resting within the larger vesicles, and (e) a substantial degree of rust. Although for much of this 40 percent-portion the material grains are hard, the aggregate is weak, crumbling on impact and unsuitable on its own for quality ballast. About 2 percent of the BOF material is either brick or other nondescript material, presumably some flux derivative. These materials will not make good ballast, but because of their relatively minor occurrence should have little impact on ballast performance.
Kinmberly Float	Metasediments with highly variable mineral assemblages; of medium hardness and toughness, but showing pronounced planar fabrics due to metamorphism. Should be permeable and relatively resistant to chemical weathering.
CP-1	Creek gravel; 60 percent horneblende-biotite skarn, 30 percent feldspar horneblende intrusive, and 10 percent limestone; materials generally tough but soft; long-term permeability could be a problem.
Walachin Pit	Gravel; about 65 percent skarn, 25 percent marble, 10 percent hornfels. Materials are soft, weak—marble especially prone to fracturing on cleavage; should remain permeable.
Alberta North	Limestone, abundant crinoidal fossils; weak and soft; should remain permeable but will be prone to solutional weathering by weakly acidic waters.
PAR-1	Fine to medium-grained rather massive limestone or dolomite. Rather dirty with substantial portion of clay-sized particles among fines. May be expected to abrade easily although aggregate appears tough.
Coteau dolomite	Tough, very fine-grained dolomite; relatively soft, but no preferred directions of fracture. Fines 95 percent powder, should remain relatively permeable.
Saint Isidore limestone	Very dirty, fossiliferous limestone; soft, weak; will evolve considerable clay and sand on breakdown, which will foul ballast.
Joliette limestone	Limestone, fossiliferous; soft, weak; fractures on bedding planes and cleavage of $CaCO_3$ crystals. Fine material about 95 percent powder, 5 percent angular cleavage rhombs. Should remain permeable if track environment is weakly acidic.
Montreal limestone	Coarse-grained, clayey limestone; soft, weak; cleavage in coarse crystals forms planes or weakness encouraging fracturing. Fines about 90 percent powder, initially permeable, but could foul with clays as weathering proceeds.
Megantic limestone	Compact, isotropic limestone, soft and weak; should remain permeable, but will be prone to solution weathering by weakly acidic water.
Grenville marble	Non-ballast used for laboratory test comparisons. Coarsely crystalline calcite. Soft mineraled and weak. Not suitable as a ballast material.
Saint Marc limestone	Coarsely crystalline, fossiliferous limestone; soft, weak; fractures on cleavage planes. Clayey material, some quartz present in fines. Permeability likely to decrease over time in track.

foreign source fouling of ballast dramatically reduces the life times given in Figure 1. For example, excessive fouling from highly plastic clay fines may result in as much as an approximate drop in the observed life.

A similar correlation or aggregate index was noted by Raymond and Diyaljee (6) when load ranking ballast for permanent settlement. This suggests that the same aggregate index may be used to estimate the settlement performance between tamping cycles, provided there is no fouling from foreign source fines. For example, note the data obtained by Hay et al. (7) related to ballast type and maintenance cycle shown in Figure 2. It may be observed that the fine-grained hard mineral ballasts clearly outperform the soft mineral-grained and large harder grained aggregates.

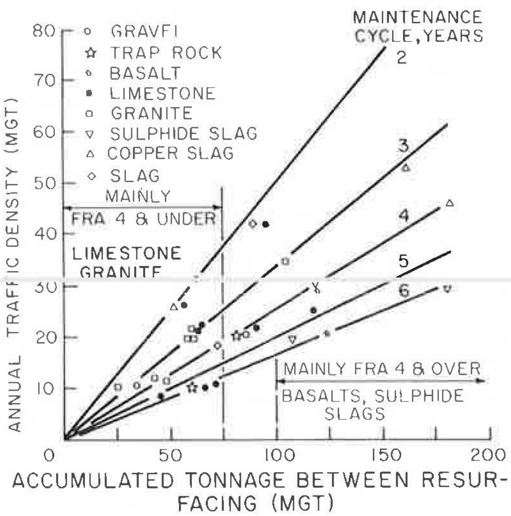


FIGURE 2 North American ballast usage showing track class and resurfacing MGT (7, Table 5.1).

Apart from having limits of the LAA, MA, and N_a, the Canadian Pacific specification suggests a minimum life for ballast of 30 years and requires a life of more than 20 years based on expected line tonnage.

PRODUCTION TESTING

General

After the aggregate source for the ballast has been selected and a contract has been signed for ballast production, it is necessary to monitor its production. Although it is advisable to check the quality of the aggregate from time to time by performing the same tests as used for the aggregate selection, more important is the performance of tests to monitor the ballast properties that are production variable.

Shape and Surface of Particles

As already commented on in relation to (track) stability in the section on Aggregate Selection, the particle shape and its surface is of utmost importance and has long been recognized as having a major effect on track stability. High quality ballasts are normally required to have a high percentage of fractured faces and are required to be cubical. European practice, as given by the International Union of Railways (8), is primarily based on limiting the percentage of particles whose ratio of longest dimension to least dimension (the least dimension is measured by passage through an infinite slot) exceeds 3, with no single particle having a ratio greater than 10. The percentage limit varies from railroad to railroad ranging from 5 to 20 percent for tolerance A ballast and 20 to 33 percent for tolerance B ballast. The desire for cuboid shaped particles is unquestionable. Canadian Pacific Rail, based on their experience with their ballast, have chosen in their specification to have no direct restrictions on particle shape but they do have stringent controls on ballast particle surface.

The Queen's University study (1) pointed out the importance of clarity regarding the definition of a crushed face. This led Canadian Pacific to conduct an extensive study of the maintenance costs associated with poorly crushed gravel ballast compared with well crushed gravel ballast. The results of these studies are reflected in the Canadian Pacific specification, which requires not less than 60 percent crushed particles for branch lines rising to not less than 90 percent for main line track built with continuous welded rail. In addition, the largest particle grading (No. 5) is required to have 100 percent crushed particles. The Canadian Pacific definition of a crushed particle is also very stringent; it requires three crushed faces whose planes must intersect at greater than 45 degrees for a particle to be termed a crushed particle. A crushed face is defined as being a fresh crushed surface having a maximum length not less than one-third the maximum particle dimension and whose maximum width, measured perpendicular to the maximum fracture length direction, is at least one-fourth the maximum particle dimension. Even on quarried rock these requirements will limit elongated and flaky particles.

Purity

In the case of quarried material it is always possible for clay seams or seams of soft rock to be present within the deposit. Similarly, clay or soft particles may be present in gravel sources. During production of ballast it is necessary to periodically check for purity. Three tests are normally required although they do not form part of the Canadian Pacific specification because they would be noted in the petrographic examination. These tests are used in the AREA specifications whose limits are (a) soft and friable pieces < 5 percent (< 3 percent for concrete ties), (b) material finer than No. 200 sieve < 1 percent (< 0.5 percent for concrete ties), and (c) clay lumps < 0.5 percent (< 0.5 percent for concrete ties).

Gradation

The particle gradation of a ballast selected for track use is clearly independent of the aggregate source. Ballast gradings are usually close to uniformly graded with field productions based on the use of two or three sieves. For example, the AREA No. 4 grading, which is one of the most often used gradings, permits as an extreme 100 percent passing the 38-mm (1.50-in.) sieve with 80 percent retained on the 25-mm (1-in.) sieve. The AREA and new Canadian Pacific gradings are given in Table 3.

Single-sized ballasts have larger void volumes than broadly graded ballasts, and thus where ballast fouling from aggregate breakdown is the major source of contamination, they are generally to be preferred. Broader graded ballasts are generally

TABLE 3 AREA and CP Recommended Ballast Gradations

Grading	Nominal Size	Limits of Percent Passing Each Sieve (Square Openings) Percent by Weight								
		3"	2-1/2"	2"	1-1/2"	1"	3/4"	1/2"	3/8"	#4
AREA 24	2-1/2"-3/4"	100	90-100		25-60		0-10			
AREA 3	2" -1"		100	95-100	30-70	0-15				
CP 5	2" -1"		100	90-100	35-70	0-5				0-3
AREA 4	1-1/2"-3/4"			100	90-100	20-55	0-15			
CP 4	1-1/2"-3/4"			100	90-100	20-55	0-5			0-3
CP 3	1-1/2"-1/2"			100	90-100	70-90	30-50	0-20		0-3
CP 2	1-1/2"-3/8"			100	90-100	70-90	50-70	25-45	10-25	0-3
AREA 5	1" -3/8"				100	90-100	40-75	15-35	0-15	
AREA 57	1" -3/16"				100	95-100		25-60		0-10

stronger, and where track stability is a major concern, such as on high curvature track or track with high grades, the broader graded AREA 24 ballast or an even more broadly graded ballast may be beneficial, provided the aggregate quality is high and aggregate degradation is estimated to be minimum. Such a grading has been used by British Columbia Rail on mountain territory that has up to 12-degree curves combined with 2.2 percent grades. The aggregate source was a basalt. Before the adoption of a broad grading, an AREA No. 4 grading was in use and the maintenance cycle in this territory was as low as 3 months on the worst curves. After adoption of the grading shown in Figure 3 the lowest maintenance cycle rose to 2 years.

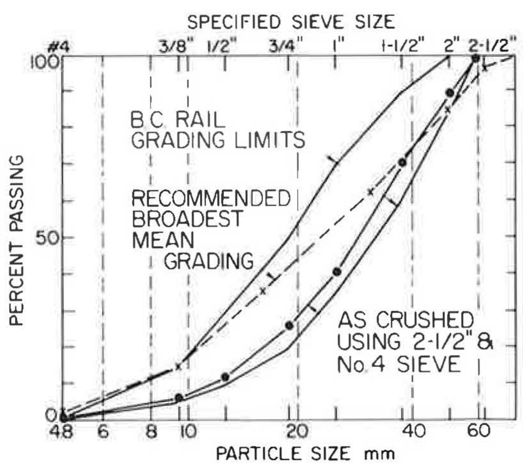


FIGURE 3 British Columbia Rail grading curve used to extend maintenance life cycle on high degree curves and steep grades.

TESTING

A distinction has been made between aggregate selection and its production into ballast. This distinction is recognized in the Canadian Pacific specification although no distinction has been made in relation to the frequency of testing during production. The Canadian Pacific specification requires tests to be performed every 1,000 tons compared with the AREA recommendation of every 200 tons. If the 200-ton requirement is followed it is suggested that only production-related testing as outlined here be conducted every 200 tons, whereas aggregate selection tests should be performed every 1,000 tons. Based on the author's knowledge of railroad practice, many would consider a 1,000-ton test requirement excessive.

CONCLUDING COMMENTS

The evaluation of ballast requires a two-stage process involving first an aggregate selection and then a monitoring of the processed material. The reasoning for the specification of each test and the explanation of what it evaluates is given. Generally not understood is the value that can be obtained from petrological analysis because no mention is made of its use in the ballast specification of the present AREA Manual for Railway Engineering (9). This was a major recommendation of the Queen's University research and has been made a major requirement of the new Canadian Pacific Rail specification that states (Item 4c) (10)

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

Because ballast is generally made from rock and petrology is the study of rocks such a requirement is nothing more than common sense.

ACKNOWLEDGMENTS

The original work formed part of a general study on the geotechnical problems of railroad support being financed by Canadian National Rail, Canadian Pacific Rail, and Transport Canada Research and Development Centre under Project 2.22 of the Canadian Institute of Guided Ground Transport (CIGGT) at Queen's University under the direction of the author. Additional financial support was obtained from a grant from the National Research Council of Canada.

REFERENCES

1. G.P. Raymond, C.J. Boon, and R.W. Lake. Ballast Selection and Grading--A Summary Report. Report 79-4. Canadian Institute of Guided Ground Transport, Queen's University, Kingston, Ontario, April 1979, p. 50.
2. E. Leary. The Building Limestones of the British Isles. Building Research Establishment Report, Her Majesty's Stationery Office, London, England, 1983, p. 91.
3. P.N. Gaskin and G.P. Raymond. Contribution to Selection of Railroad Ballast. Journal of the Transportation Engineering Division, ASCE, Vol. 102, No. TE2, May 1976, pp. 377-394.
4. J.W. Weber. Development of the Prestressed Concrete Tie in the USA. Symposium on Railroad Track Mechanics and Technology, A.D. Kerr, ed. Pergamon Press, Elmsford, N.Y., 1975, pp. 265-282.
5. A. McIntyre and L.R. Plitt. The Inter-Relationship Between Bond and Hardgrove Grindabilities. Canadian Institute of Mining and Metallurgy Bulletin 63, June 1980, pp. 149-180.
6. G.P. Raymond and V.A. Diyaljee. Railroad Ballast Load Ranking Classification. Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT10, Oct. 1979, pp. 1133-1153.
7. W.W. Hay, R. Baugher, and A.J. Reinschmidt. A Study of Railroad Ballast Economics. Report FRA/ORD-77/64. FRA, U.S. Department of Transportation, Sept. 1977.
8. Classification of Ballast Prescriptions. Report 7. Office for Research and Experimentation, International Union of Railways, Question D71, Stresses in the Rails, the Ballast and in the Formation Resulting from Traffic Loads. Utrecht, The Netherlands, June 1967, p. 62.
9. Manual for Railway Engineering. Vols. 1 and 2, American Railway Engineering Association, Wash., D.C., 1984.
10. C.P. Rail Specification for Evaluating Processed Rock, Slag and Gravel Ballasts. Canadian Pacific Ltd., Montreal, Quebec, 1981.

Publication of this paper sponsored by Committee on Track Structure System Design.

A Proposed Track Performance Index for Control of Freight Car Harmonic Roll Response

HERBERT WEINSTOCK, HARVEY S. LEE, and ROBERT GREIF

ABSTRACT

Analytical and experimental study results of high center of gravity freight car response to a range of track cross-level deviations are presented. Based on the criteria of excessive car body roll or excessive wheel lift, boundaries between safe and unsafe track cross-level conditions are established. These studies and industry experience indicate that although isolated low joints producing cross-level deviations as large as 4 in. can be safely traversed, at the critical harmonic roll speed, a continuous series of 0.75-in. low joints will produce an unacceptable roll response for high center of gravity cars resulting in a potential derailment. The results also indicate that a 400-ft length of track is sufficient for the harmonic roll resonance to build to a critical amplitude. The cross-level variation conditions that form the boundaries between safe and unsafe harmonic roll response are reviewed. The results are then combined in a heuristically developed performance index that is intended to identify potentially unsafe track conditions without rejecting an excessive amount of track that does not have the potential for producing harmonic roll derailment. Analyses have been conducted on track geometry measurements of selected Class 2 and Class 3 track to evaluate the statistics of the proposed cross-level index. The results indicate that the index does successfully identify potential harmonic roll situations while permitting occasional large cross-level deviations that may be undesirable but are not unsafe. Illustrations of the response of the index to selected measured track geometry conditions are included.

As noted in a survey conducted by the Government Industry Program on Track Train Dynamics (1), the harmonic roll response of freight cars to periodically recurring cross-level variations on jointed track having one-half staggered rail lengths is of major concern to the railroad industry, in terms of both safety and damage to equipment and lading. This phenomenon of harmonic roll, often referred to as freight car rocking or rock and roll has been known to exist for many years, dating back to the 1920s (2,3). The harmonic roll problem is a highly nonlinear resonance condition typically occurring in the 10 to 25 mph speed range.

Figure 1 illustrates a freight car in the normal and rocking positions. In the normal (nonrocking) position, the car body moves with the bolster. For the more severe rocking case, the car body partially separates from the bolster and rocking occurs about the side bearing. A typical roll amplitude versus speed response characteristic for a freight car responding to periodic cross-level variations is shown in Figure 2. As the car speed is increased, the roll amplitude follows the lower branch (A) of the curve until it reaches a critical speed where a sudden jump in roll amplitude occurs accompanied by violent oscillations. As speed increases further, the oscillation decreases in amplitude. However, if the track section is entered at a decreasing speed, the response follows the upper branch of the curve (B) with an increase in roll amplitude as speed decreases. The roll responses encountered at decreasing speeds are higher than those obtained for increasing speeds.

The new generation of larger freight cars (70- to 100-ton range) with high center of gravity (c.g.) has increased the frequency of the harmonic roll problem. Although significant efforts are currently being made by the equipment supply industry to produce devices that control harmonic roll through the use of new truck and car design, it will be some time before the current fleet is replaced. It is therefore necessary to establish limits on track geometry variations to assure safe operation of the existing fleet. The simulation studies described here, along with industry experience, indicate that although isolated low joints producing cross-level deviations as large as 4 in. can be safely traversed, at the critical harmonic roll speed a continuous series of consecutive 0.75-in. low joints will produce an unacceptable roll response for high c.g. cars, resulting in a potential derailment. The results also indicate that a 400-ft length of track is sufficient for the harmonic roll resonance to build to a critical amplitude.

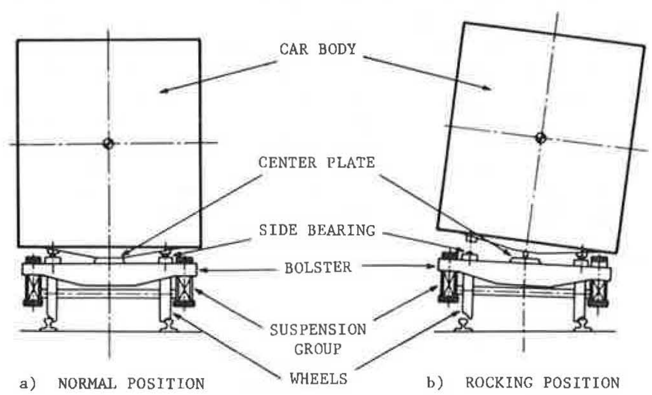


FIGURE 1 A typical freight car and truck illustrated in a transverse plane.

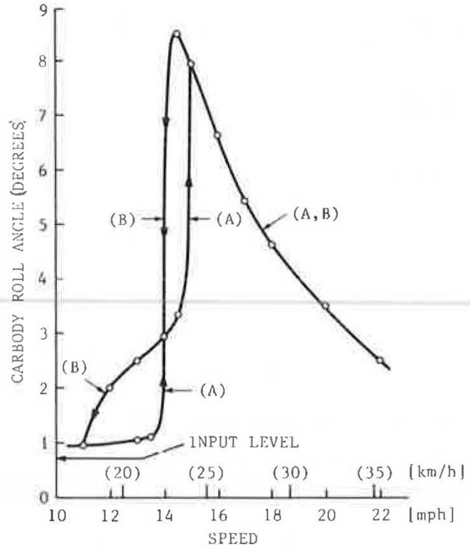


FIGURE 2 Typical roll amplitude versus speed response characteristic for freight car responding to periodic cross-level variations.

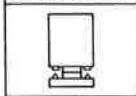
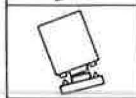
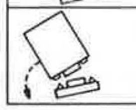
CONFIGURATION	#	DESCRIPTION
	C-0	NO ROLL
	C-1	BOLSTER ROLLING
	C-2	CENTERPLATE ROCKING
	C-3	CENTERPLATE/SIDEBEARING ROCKING
	C-4	SIDEBEARING ROCKING
	C-5	CARBODY ROLLOVER

FIGURE 3 Car body roll configurations.

Based on the criteria of excessive car body roll or excessive wheel lift, boundaries between safe and unsafe track cross-level conditions have been established by the simulation studies described here. The cross-level variation conditions that form these boundaries are combined into a heuristically developed performance index or cross-level index (CLI), which is intended to identify potentially unsafe track conditions without rejecting an excessive amount of track that lacks the potential for harmonic roll derailment. The statistics of this proposed cross-level index are evaluated from analyses of track geometry measurements of Class 2 and Class 3 track. The results indicate that the index does successfully identify potential harmonic roll sections while permitting occasional large cross-level deviations that may be undesirable but are not unsafe. This paper includes illustrations of the response of the index to selected measured track geometry characteristics.

MODELING TECHNIQUES

The analytical model used in this study is an approximation of the flexible vehicle model used by Tse (4) in a study of freight car rocking (5). The model simulates freight car dynamic response based on numerical integration of the equations of motion, including nonlinearities related to springs, side bearing clearances, friction snubbers, and kinematic constraints. The freight car rocking model has been validated by comparison with field test data from tests conducted at the Transportation Test Center (6).

As a rail vehicle undergoes harmonic roll, six car body roll configurations are possible, depending on the degree of rocking as shown in Figure 3. The first configuration (C-0) is the no-roll static condition. The smallest degree of roll will produce configuration C-1 where the centerplate surfaces remain in contact as the car body and truck bolster roll together. At larger car body roll angles, there is partial centerplate separation in which rocking takes place on the centerplate (C-2). Further rotation will result in both centerplate and side bearing rocking together (C-3). In configuration C-4, there is full centerplate separation with rocking taking place about the side bearing. In the final configuration (C-5), the car body rolls off. The analytical model used in this study is a 5 degree of freedom nonlinear rail car model capable of representing all the foregoing roll configurations. The model of the rail car in a transverse plane is shown in Figure 4 along with the bolster and truck suspensions. The orientation of the rail car model traveling over a track with cross-level variation is shown in Figure 5.

The vertical rail profile is based on a five term exponential decaying series of the form

$$z(x) = \sum_{i=-2}^{i=2} A_i e^{-|x-(2i+1)L/2|/a} \qquad (1)$$

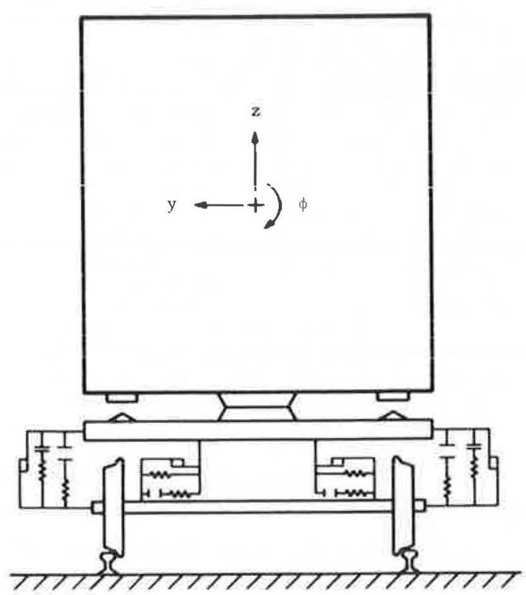


FIGURE 4 Rail car model with observer facing the direction of forward motion.

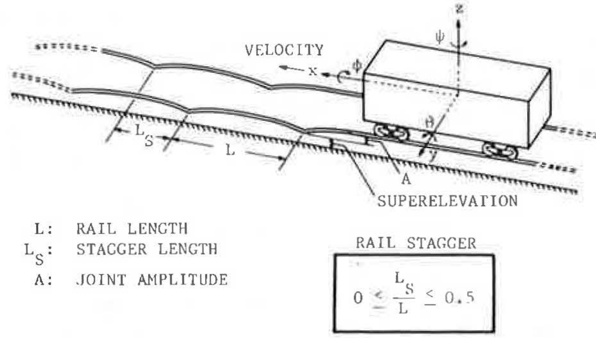


FIGURE 5 Rail car model at initial position (T = 0) traveling over a track with cross-level variation.

where the A_i are selected to produce the appropriate joint amplitude, L is the rail length, and α is the inverse decay rate. This relationship is based on a study of measured track geometry data and characterizes bolted rail with one-half staggered joints (7). Based on this relationship, the shape of the vertical rail profile for a 1-in. joint amplitude is shown in Figure 6. Further discussion of this exponential series, as well as the effect of summing a greater number of terms, is given by Corbin (7). Because the vertical rail amplitudes can be individually specified, it is possible to produce discrete (single, double, etc.) or continuous perturbations. Superelevation is included in the track geometry by adding independent terms to the left and right rail functions, so that either rail can serve as the high rail. To obtain variable rail stagger, the relative longitudinal position of the right and left rails is a variable that can be specified to produce any stagger from 0 (no stagger) to 0.5 (full stagger).

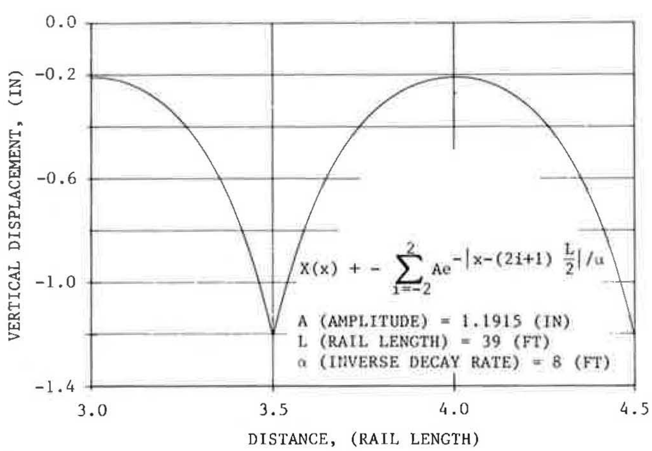


FIGURE 6 Vertical rail profile for 1-in. joint amplitude using five exponential terms.

SIMULATION RESULTS

Three types of track cross-level geometries were studied: cross-level variation on one-half staggered level track (typically encountered in jointed rails on tangent track), cross-level variation on one-half staggered superelevated track (to acquire insight into curving behavior), and cross-level variation at different staggers (situations for a rail stagger other than one-half). The two important safety performance measurements for rail car response are peak car body roll angle and maximum wheel lift. In this paper, the safety criteria chosen were a threshold value of 5 degrees (10 degrees peak-to-peak) for peak car body roll angle and 0.50-in. maximum allowable wheel lift.

Results are presented for a 100-ton loaded hopper car with truck center spacing of 39.5 ft. Response for a 70-ton loaded box car is presented by Lee and Weinstock (8). The results presented here are for typical representative loaded cars having the nominal parameters given by Tse (4). More severe responses result with reduced snubber friction. Variations in truck design parameters, such as side bearing clearance and spacing, have a small effect on response characteristics. More significant improvements are indicated by the use of supplemental devices such as centerplate extension pads and hydraulic snubbers. The rail cars studied here, however, are typical of those in current service. It is expected that these cars will not be replaced or fully retrofitted in the immediate future. One-half staggered track is assumed and the car enters the cross-level variations from level track with zero initial conditions.

Peak car body roll angle and maximum wheel lift are shown in Figures 7 and 8, respectively, as a function of track cross-level amplitude. As the cross-level amplitude increases for a given speed,

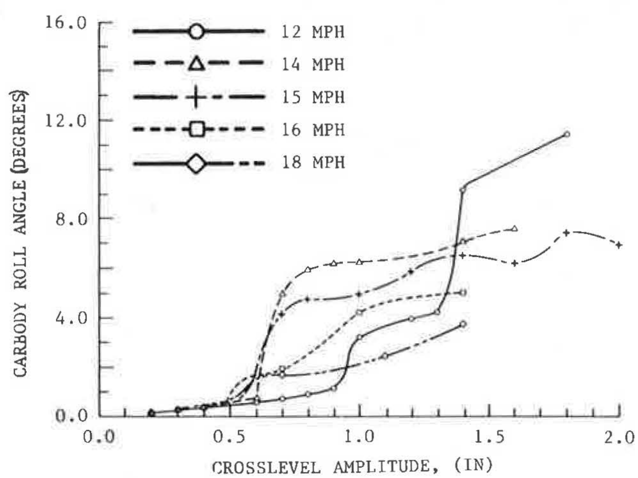


FIGURE 7 Peak car body roll angle response to periodic cross-level variations.

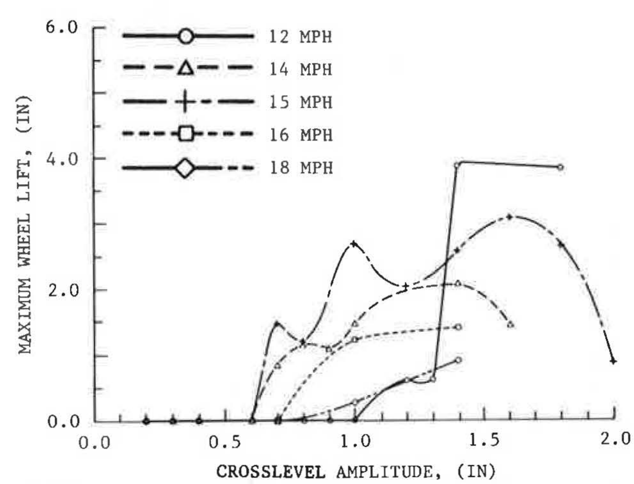


FIGURE 8 Maximum wheel lift response to periodic cross-level variations.

the peak car body roll angle increases almost linearly until a cross-level amplitude is reached, where a dramatic change in response occurs. At this critical amplitude, the peak car body roll angle continues increasing, but at a faster rate with increasing cross-level amplitude. The critical amplitude at which the response characteristic changes is a function of speed. For a 15 mph case, the critical cross-level amplitude is about 0.6 in. The 5-degree threshold value for peak car body roll angle is reached at 1 in. of cross-level amplitude. The trend for the maximum wheel lift is similar to the peak car body roll except that below the critical amplitudes there is no wheel lift, as shown in Figure 8. The 0.50-in. maximum allowable wheel lift is reached at a cross-level amplitude of 0.52 in. for a rail car at 15 mph.

A convenient display for defining the number of repetitions of a track cross-level variation amplitude that can be tolerated is the relation between the cross-level amplitude and the distance a rail car can travel before a derailment threshold is reached. The data points plotted in Figure 9 are the joint amplitudes that will produce a 0.50-in. wheel lift as a function of the length of track having consecutive low joints for a 70-ton boxcar and a 100-ton hopper car while traveling at the critical speed. Similar results have been constructed in terms of the threshold car body roll angle. For safe operation, a given track cross-level amplitude must not cause the vehicle to exceed these threshold values for a specified distance of travel.

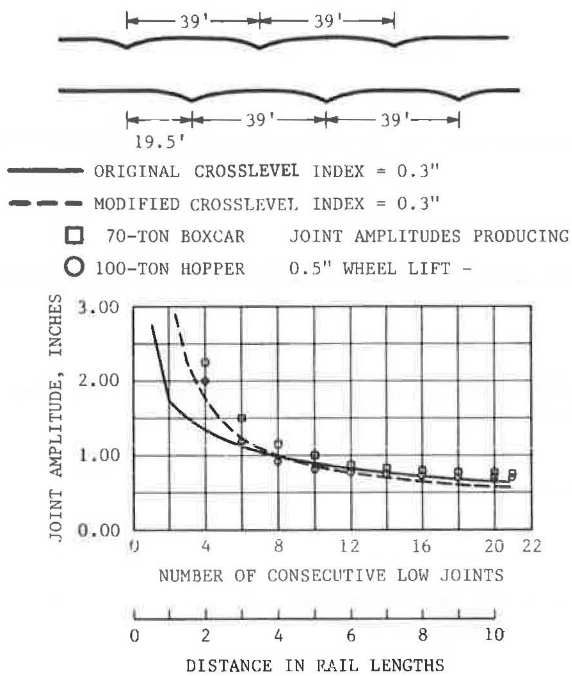


FIGURE 9 Permissible cross-level variations versus number of consecutive low joints.

Although consecutive low joints of 0.75-in. amplitude produce wheel lift and car rocking that would likely lead to derailment, consecutive low joints of 0.50 in. can be sustained indefinitely. It was also found that for track segments with repeated low joints of 0.75-in. amplitude or greater, the results were strongly initial-condition dependent. For example, entering the zone at a decreasing speed would produce a different response than entering at an increasing speed. However, for consecutive low joints of less than 0.50 in., the results were not significantly influenced by initial conditions. If the track perturbations are limited to four consecutive low joints, then a joint amplitude of 1 in. can be sustained. As the number of consecutive low joints is reduced, a larger track cross-level amplitude can be safely sustained.

Response for the 100-ton covered hopper car was also obtained for continuous track cross-level variation on one-half-staggered track while operating at a speed corresponding to a 3-in. unbalance. This situation was simulated by running the computer program by Platin (5) on tangent track with a superelevation of 3 in. The car body roll angles, relative to the initial static car body roll angle (3.54 degrees), are only slightly higher than the corresponding values on level track. For wheel lift on track producing a 3-in. unbalance, the peak values are 23 percent lower than peaks in wheel lift occurring on level track.

The potential effects of track curvature on the roll response and wheel lift were investigated by Blader and Mealy (9). Their studies indicated that the roll response and wheel lift characteristics were not strongly related to track curvature. However, with track curvature, the wheel lift tendency was accompanied by a tendency toward wheel climb on the high rail.

In terms of overall dynamic response, a 3-in. superelevation is no worse than the results obtained on level track (8). The effect of variation in rail stagger is studied by using a track input of one-third and one-fourth rail stagger. As the rail stagger is reduced, there is an increase in the joint amplitude required to maintain a constant cross-level amplitude. Comparison of peak car body roll angle and maximum wheel lift with similar results for one-half rail stagger indicate that varying the rail stagger has little effect on vehicle response at the corresponding cross-level amplitudes (8).

TRACK SAFETY PERFORMANCE INDEX

Initial Formulation of Cross-Level Index

The requirements for a useful safety performance index are to develop a specification for identifying track with a sequence of cross-level variations that may produce unsafe response, without rejecting an excessive amount of track that does not have the potential for harmonic roll derailment. It must agree with simulation results and industry experience pertaining to derailment. For example, although isolated low joints as large as 4 in. can be safely traversed, a continuous series of consecutive 0.75-in. low joints will produce an unacceptable roll response of high c.g. cars traveling at the critical harmonic roll speed and result in a potential derailment. Simulation results also indicate that a 400-ft length of track is sufficient for the harmonic roll resonance to build to a critical amplitude. Conversely, existing transient roll oscillations are usually dissipated entirely in less than 10 rail lengths.

A root mean square (RMS) deviation in cross-level greater than 400 ft of track is used as the basic criterion in the track performance index. To prevent the index from being triggered by normal superelevation in curves or by the variation in superelevation associated with spirals, the calculations are based on the deviations of cross-level, $\delta(x)$, from a 100-ft moving average of cross-level, $Z(x)$, as defined by

$$\delta_1(X) = Z(x) - 1/100 \int_{X-50}^{X+50} Z(v)\,dv \qquad (2)$$

where

$\delta_1(X)$ (in.) = cross-level deviation at location x,
$Z(x)$ (in.) = cross-level at location x,
x (ft) = position on track, and
v (ft) = integration variable representing position on the track in the 100-ft track segment centered at x.

The initial formulation of the cross-level index is then

$$\mathrm{CLI}(x) = \left(1/400 \int_{x-200}^{x+200} \delta_1^2\,(v)\,dv\right)^{1/2} \qquad (3)$$

A property of this RMS cross-level deviation is that it acts to filter out those cross-level variations at wavelengths that are longer than the lengths of the moving averages. The intent of this filtering process is to exclude those cross-level variations that do not contribute to harmonic roll response, such as superelevation in curves. In order to quantify this filtering effect, a study was conducted of the effect of averaging length on the cross-level deviation for a sinusoidal cross-level variation with variable wavelength. Figure 10 is a plot of cross-level deviation against wavelength, with averaging length as a parameter. Wavelengths exceeding the averaging lengths are attenuated and filtered, whereas wavelengths that are shorter than the averaging lengths pass through.

With the cross-level index set at a limiting value of 0.3, Figure 9 shows the amplitude of cross-level variation that would be permitted for a specified number of low joints within a 400-ft length of track. The specification implied by establishing a limit of 0.3 for the CLI is compared with the simulation results for a 100-ton and 70-ton car obtained for the same number of low joints encountered consecutively. This index value of 0.3 will permit consecutive cross-level variations up to 0.625 in. and represents a good fit to the simulation results for eight or more consecutive low joints (four rail lengths). For a smaller number of low joints, the index is somewhat conservative in comparison with the simulation results. However, current practice

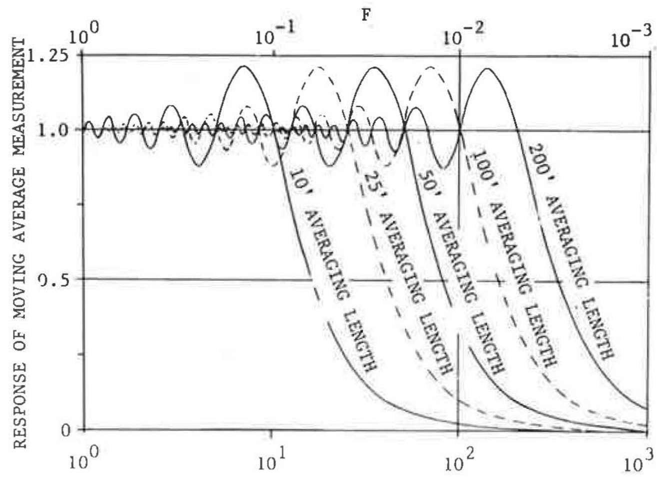


FIGURE 10 Plot of cross-level deviation against wavelength.

does not permit individual cross-level deviations greater than 2 in. for Class 2 track, so that the cross-level index with a level of 0.3 in. appears to be a fair representation of current accepted practice.

Although the cross-level index, as initially defined, has the advantage of simplicity of formulation, further evaluation indicated some drawbacks in selectivity. For example, the index was applied on a pilot basis to records of track geometry accumulated by the Federal Railroad Administration (FRA) to examine its effectiveness as a track safety index. As shown in Figure 11, the index could be triggered by large single events or large track warp. In reviewing the records it was found that two out of three times the cross-level index value of 0.3 was exceeded; there was either a periodic cross-level variation capable of inducing harmonic roll, or some other track situation likely to result in an unsafe condition. However, in one out of three exceedances, a clearly unsafe condition could not be identified although maintenance was definitely desirable.

Although this error rate might be acceptable for a maintenance criterion, it was believed that the error was too large to permit the index, as originally formulated, to be used to define safety. Other evaluations of the applicability of CLI indicated that in some cases, the index was triggered by 78-ft wavelength cross-level variations of a smaller level

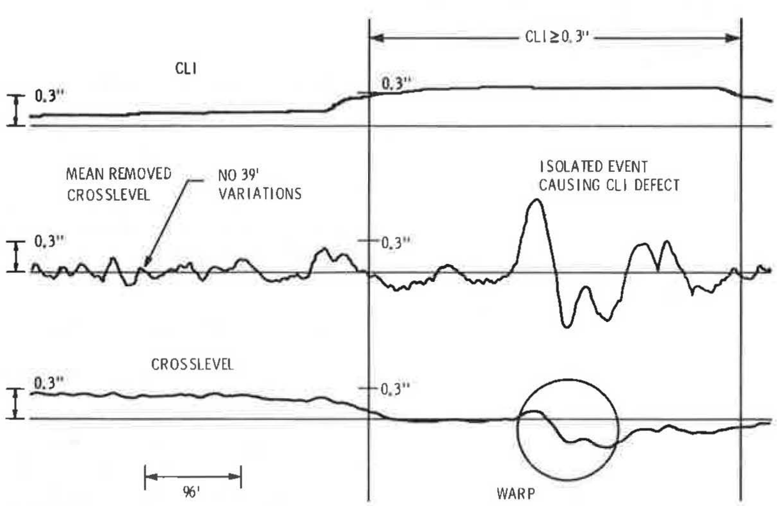


FIGURE 11 Cross-level index (CLI) and warp exception: spiral of curve.

than would be expected to cause harmonic roll problems.

To obtain further insight into wavelength effects, a study was conducted of the variation of CLI with wavelength due to a 1-in. amplitude sinusoidal cross-level variation. As shown in Figure 12, CLI has a fairly flat response for wavelengths up to 100 ft then falls off somewhat gradually at longer wavelengths, making it essentially a cross-level energy measure. It also has peak response at about a 78-ft wavelength, which is responsible for the sensitivity to warp and the longer wavelengths as previously noted.

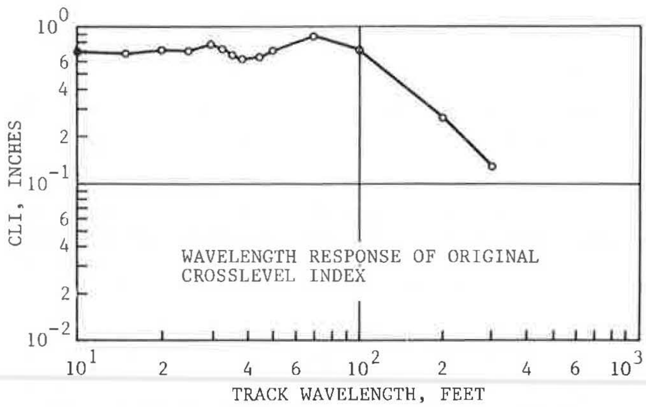


FIGURE 12 RMS deviation for sinusoidal cross-level variations, 1-in. amplitude.

Formulation of Modified Cross-Level Index

Modification to the cross-level index should permit the index to distinguish between randomly located low joints and successive low joints. A succession of six low joints occurring consecutively produces a more severe condition than three pair of low joints distributed with one pair at the start of the 400-ft track segment, one pair in the center, and the final pair at the end of the track. For a given amplitude, the first scenario would permit a significant roll to build up, whereas the second scenario would produce a response that would not be significantly worse than a single low joint pair. Therefore, additional terms were added to the cross-level index to represent the correlation of the cross-level variation with the cross-level at one and two rail lengths away. This produces a heavier weighting to periodic events than to events occurring several rail lengths away. The additional terms Q1, Q2, and Q3 are defined as follows:

$$Q1(x) = 1/400 \int_{x-200}^{x+200} \delta_2^2(v)dv$$

$$Q2(x) = 1/(400 - L) \int_{x-200+(L/2)}^{x+200-(L/2)} \delta_2[v + (L/2)] \times \delta_2[v - (L/2)]dv$$

$$Q3(x) = 1/(400 - 2L) \int_{x-200+L}^{x+200-L} \delta_2(v + L)\ \delta_2(v - L)dv \quad (4)$$

where L is rail length in feet (usually 39 ft). In addition, the averaging length was modified to 40 ft to eliminate some of the undesirable amplification effects of the 100-ft moving average for wavelengths longer than 39 ft:

$$\delta_2(x) = Z(x) - 1/40 \int_{x-20}^{x+20} Z(v)dv \quad (5)$$

The final requirement in the formulation of the modified cross-level index was the proper weighting of the values of Q1, Q2, and Q3 to optimize predictions of derailment for a wide range of track wavelength. Initially, equal weighting of 1/3 was given to each correlation value, Q, producing the modified cross-level index with equal weighting $CLIM_E$:

$$CLIM_E = [1/3\ (Q1 + Q2 + Q3)]^{1/2} \quad (6)$$

As shown in Figure 9, for consecutive low joints on 39-ft rail, $CLIM_E$ produced a much better agreement with the simulation results and was definitely less conservative than the original CLI. It should be noted, however, that with this equal weighting, the modified index requires a knowledge of the rail length in order to be applied. In a real-time inspection of track this information might be ambiguous, because of the use of mixed rail lengths or welded rail. To obtain more insight into the effect of rail length, the behavior of $CLIM_E$ with track cross-level variation wavelength was examined, as shown in Figure 13. The modified index with equal weighting and L set at 39 ft is sharply tuned to

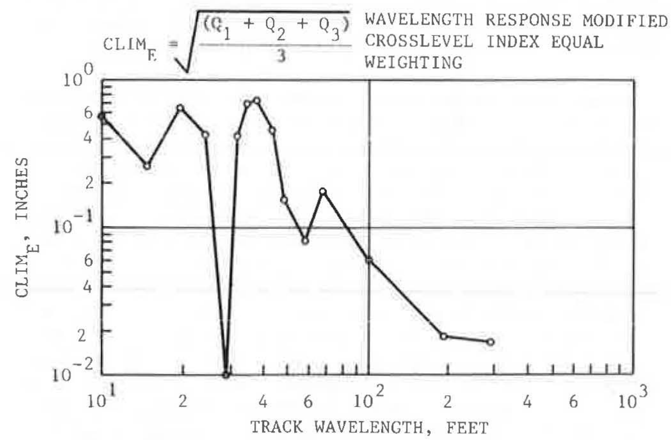


FIGURE 13 Behavior of $CLIM_E$ with track cross-level variations.

39-ft wavelength effects, but sharply attenuates effects at rail lengths other than 39 ft, including some shorter rail lengths that are still in common use. At some of these shorter rail lengths, the index would not be conservative if L was fixed at 39 ft in the index computation, and it might permit the existence of potentially unsafe conditions. Consequently, a modified cross-level index with unequal weighting $CLIM_U$ was formulated,

$$CLIM_U = (0.6Q1 + 0.3Q2 + 0.1Q3)^{1/2} \quad (7)$$

with L set at 39 ft, which produces the wavelength characteristics shown in Figure 14. This weighting produces an emphasis of the 39-ft rail length as well as an adequate response to other rail lengths in common use. Comparison with simulation results indicates that $CLIM_U$ provides good agreement with estimated safe limits.

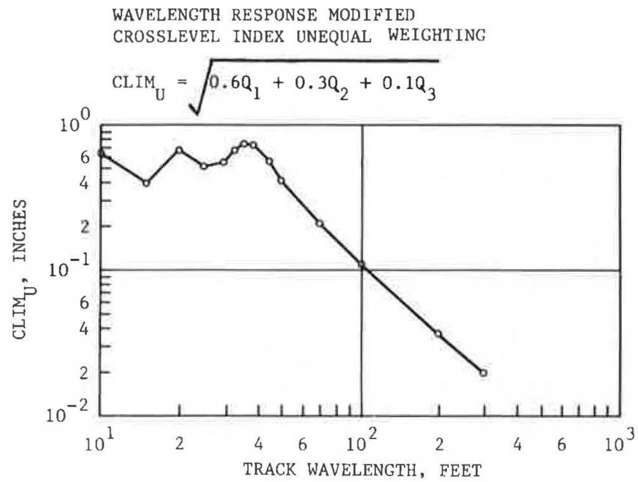


FIGURE 14 Behavior of $CLIM_U$ with track cross-level variations.

STATISTICS OF CROSS-LEVEL INDEX

The behavior of the cross-level indices on selected track geometry records of track classified as Class 2 and Class 3 track has been studied by Ensco Incorporated under contract to the FRA. The study considered 223 miles of Class 2 and 361 miles of Class 3 track geometry data. All three index formulations successfully identify cross-level situations that are capable of producing harmonic roll response. However, the modified indices are more selective and do not respond to the large single exceedances that would trigger the original index. Comparison of the indices with typical field data containing significant harmonic cross-level variations is shown in Figure 15.

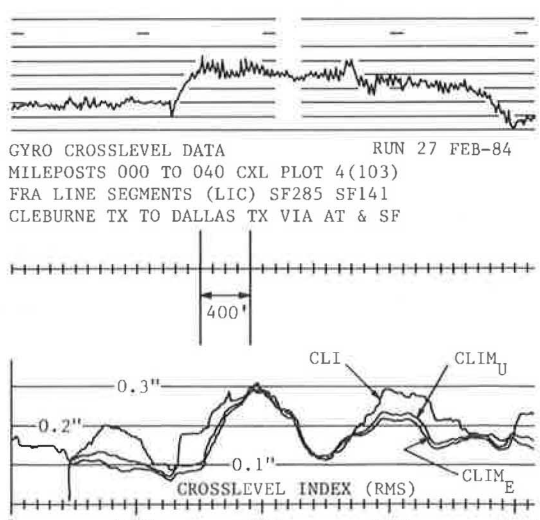


FIGURE 15 Comparison of cross-level index formulations.

For the 223 miles of Class 2 geometry data studied, there were 243 locations at which the CLI index threshold of 0.3 was exceeded, representing 5.7 percent of the length of track. Of these 243 exceedances, 75 could be clearly associated with harmonic cross-level deviations, 62 were associated with exceedances to current FRA track warp standards, about 20 represented single cross-level deviations in excess of 2 in., and the remainder were produced by long wavelength effects.

For the 361 miles of Class 3 geometry data, there were 206 locations at which the CLI index threshold of 0.3 was exceeded, representing 1.2 percent of the length of track. Of these 206 exceedances, 18 were clearly associated with harmonic cross-level activity, 144 represented exceedances to current FRA track warp standards, about 10 represented single cross-level deviations in excess of 2 in., and the remainder were produced by long wavelength effects.

For the 223 miles of Class 2 data, the modified cross-level index $CLIM_U$ threshold of 0.3 was exceeded at about 42 locations, representing about 0.9 percent of the track length. For the Class 3 data, the $CLIM_U$ index of 0.3 was exceeded at 18 locations, representing about 0.13 percent of the track length. Each of the locations identified by exceedances of the $CLIM_U$ threshold of 0.3 could be identified with a high level of harmonic cross-level deviation activity.

The results of the analyses of the statistics of the cross-level index indicate that it is successful at identifying locations of cross-level variations that are capable of producing harmonic roll derailment. It is, however, sensitive to other track cross-level deviations that are not likely to produce harmonic roll and may not require as high a level of maintenance priority. It is therefore recommended that the CLI index be used as a maintenance tool to identify track segments with high cross-level activity that should be given special attention in maintenance planning.

The modified index $CLIM_U$ has been found to be a highly selective identifier of track segments capable of producing harmonic roll derailment. It is hoped that this index will be adopted by the industry as a safety specification for improving control of harmonic roll.

PILOT APPLICATION OF CROSS-LEVEL INDEX

To facilitate measurement of cross-level deviations and computation of the cross-level index, the Transportation Systems Center (TSC) has developed a portable, self-contained cross-level measurement system that is mounted on the end of a locomotive axle. This system includes an environmentally rugged navigation grade rate integrating gyroscope that measures the roll angle of the locomotive axle. The signals from the gyroscope are transmitted to a minicomputer that calculates the cross-level and cross-level indices on a continuous basis. The computed data are displayed on a chart recorder and are recorded on a cassette tape recorder for use in later analysis. This instrumentation package is shown in Figure 16.

This system permits the measurement of cross-level and computation of the cross-level index during normally scheduled runs without requiring special measurement cars or interfering with normal train operating schedules. Use of the locomotive axle results in loaded track geometry measurements with vertical loads that are comparable to those of loaded hopper cars.

The system has been applied to recent surveys of track condition on track owned by the following railroads: Sante Fe, Burlington Northern, Kansas City Southern, Boston and Maine, and the Alaskan railroad.

In addition, several railroads have been augmenting their current track geometry data collection and maintenance and safety programs with computation and

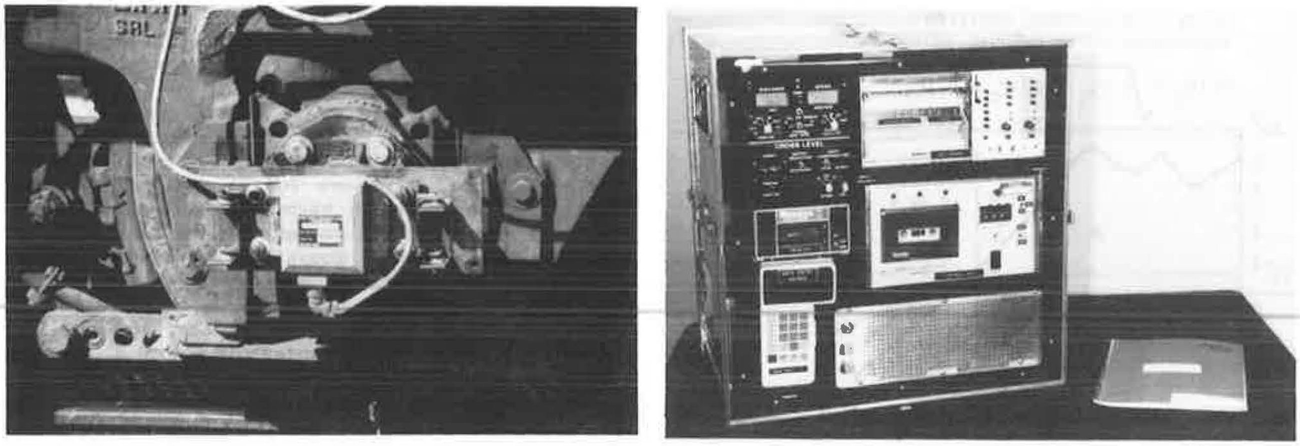


FIGURE 16 Instrumentation package for measurement of cross-level deviations and computation of cross-level index.

evaluation of the cross-level indices. The Chessie System has integrated the CLI index algorithm into its geometry car, and the Norfolk and Southern, Boston and Maine, and Atchison Topeka & Sante Fe railroads have been active in processing and monitoring CLI and $CLIM_U$ data.

These activities are expected to provide the basis for comparing the performance of the cross-level indices with the judgment and operational experience of railroad personnel under actual operating conditions. The results reported at this point tend to provide additional confidence in the use of the CLI as a tool for maintenance surveys of track cross-level and the use of $CLIM_U$ as an indicator of potentially unsafe track.

ACKNOWLEDGMENTS

The work described in this paper was conducted as part of the Improved Track Safety Research Program sponsored by the Federal Railroad Administration, U.S. Department of Transportation. The development of the cross-level indices was strongly influenced by discussions with members of the AREA ad-hoc Committee on Track Performance Standards. The authors would like to thank the members of this committee for their encouragement and constructive criticism. The authors would also like to thank H. David Reed, Transportation Systems Center Program Manager, Improved Track Safety Research Program, for his continuous encouragement and support and his significant role in developing the portable cross-level measurement system described here.

REFERENCES

1. Track Train Dynamics, Harmonic Roll Series. Vol. 1: Problem Definition, Historical Background, Current Industry Practices, Recommended Guidelines and Bibliography. Technical Report. Association of American Railroads, Chicago, Ill., 1975.
2. T.H. Symington. Causes and Prevention of Freight Car Derailments. Railway Age, Vol. 80, Jan. 1926, pp. 234-236.
3. B.R. Leffler. The Relation Between the Swaying of Hopper Cars and the Stagger of Rail Joints in Track. AREA Proceedings, Vol. 27, 1926, pp. 1243-1251.
4. Y.H. Tse. Method of Analysis for the Dynamic Behavior of a Flexible Railroad Freight Car. Technical Documentation of the Vehicle Model Developed for the Track Train Dynamics Research Program, Association of American Railroads, Chicago, Ill., July 1975.
5. B.E. Platin. Dynamic Analysis and Stabilization Control of Freight Car Rocking. Ph.D. thesis. Massachusetts Institute of Technology, Cambridge, Mass., 1978.
6. S.M. Hussain, S.P. Singh, and V.K. Garg. Validation of Non-Linear Flexible Car Body Vehicle (Freight Car) Model. Track Train Dynamics Program-Phase II. Technical Report. Association of American Railroads, Chicago, Ill., March 1979.
7. J.C. Corbin. Statistical Representations of Track Geometry. Report DOT-TSC-1211, FRA/ORD-80/22. National Technical Information Service, Springfield, Va., March 1980.
8. H.S. Lee and H. Weinstock. Rail Car Harmonic Roll Response to Periodic Track Cross-Level Variations. Report WP-743-C-15-075. U.S. Department of Transportation, Transportation Systems Center, Cambridge, Mass., 1979.
9. F.B. Blader and G.L. Mealy. Analytic Studies of the Effects of Track Geometry Variations. Report DOT-TSC-1797. FRA, U.S. Department of Transportation, 1984.

Publication of this paper sponsored by Committee on Track Structure System Design.

Granular Depth Requirement for Railroad Track

J. FRANK SCOTT

ABSTRACT

In late 1962 severe deterioration of the CN Rail Mountain Region Sangudo Subdivision roadbed occurred following introduction of six-axle locomotives and 100-ton sulphur car traffic. Studies concluded that subgrade soils were being overstrained. In 1977 some 25 theoretical roadbed design theories were identified in a report by the Federal Railroad Administration, but none of them had been simplified or validated to the stage where they had been adopted by practicing railroad engineers. In 1963 British Rail Research embarked on an investigation to evolve a comprehensive design method for railroad track foundations, and in 1972 they published granular depth design charts based on soil stress threshold levels and vehicle axle loadings. The design charts did not appear to correspond with CN Rail Research Centre experience, and in 1979, the Centre embarked on a 4 1/2 year investigation of roadbed behavior. In this study, wheel loadings, tie plate loadings, tie deflections, roadbed deflections at depth, rail settlements, and subgrade surface settlements were measured. In addition, McGill University Geotechnical Research Centre was retained to investigate subgrade soil dynamic triaxial test and creep test properties. Although field measurement and laboratory test work has been terminated, not all of the information has been correlated. Some of the findings from the field measurement work are presented in this paper.

Faced with significant increases in projected traffic, CN Rail in the late 1970s embarked on a new track construction double tracking program in Western Canada. Traffic densities were not only increasing, but the percentages of heavy axle loadings from 100-ton cars were also increasing. With rising costs of track construction and track maintenance, it was imperative that proper granular depths be selected for the new track. Remembering the Sangudo Subdivision experience, requirements for upgrading existing lines before introduction of new traffic were also a prime consideration.

At the outset of the roadbed investigation research program, some difficult questions had to be answered. What track measurements should be taken? How should instrumentation be deployed in order to collect information in a short time frame? How was information to be analyzed and correlated? British Rail Research had measured roadbed stresses (with difficulty), and had concluded that stress levels could be predicted with acceptable accuracy through use of the Boussinesq equations and knowledge of tie loadings. It was believed that there was no requirement to repeat the British Rail stress measurement and theoretical validation.

For the initial field measurement work, it was decided to measure tie plate loads, tie deflections, and subgrade surface deflections at 12 Edmonton area sites with variable depths of granular and variable traffic conditions. Subgrade materials were under the general category of Lake Edmonton clays. The instrumentation setup was as shown in Figure 1. Standard 14-in. tie plates, each fitted with four strain-gauged button load cells, were used to measure vertical tie plate loads. To determine the depth of granular at each site, holes were dug outboard of tie ends midway between the two selected ties. A 2-in. diameter hole was then augered into the subgrade soil, and a 6-ft 1.50-in. diameter plastic pipe was inserted. A 9-ft 1-in. diameter steel benchmark rod was then driven into the subgrade. To measure subgrade surface deflections, an encased rod with cone was driven through the ballast and subballast until the cone was embedded in the top surface layer of the subgrade. The rod and cone were then driven an extra 0.50 in. to free them from the casing. Foam rubber compressed between rod and casing minimized lateral vibrations but did not affect vertical movement of the rod. Spring-loaded displacement transducers attached to the benchmark rod were used to measure tie and subgrade surface deflections.

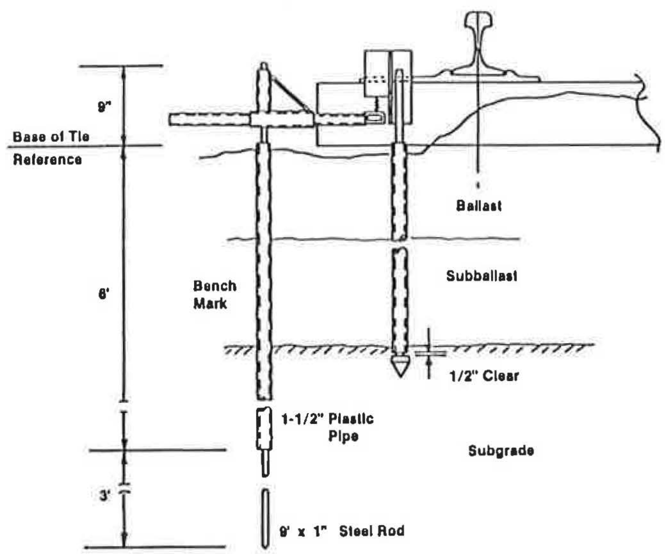


FIGURE 1 Instrumentation setup.

CN Rail experimented with using accelerometers as inertial references to eliminate the benchmark rod, but without success. It is doubtful if the 9-ft benchmark rod provided an absolute reference, but the deflections should have been small relative to those at subgrade surface. Data were recorded on an 8-channel chart, with individual loads and deflections displayed as well as simultaneous average tie loads and tie deflections.

As shown in Figure 2, average tie and subgrade deflections were plotted against tie plate average loads to determine tie and subgrade compliance values. Plots of this type were found necessary to eliminate initial softness and nonlinearity in tie support conditions. The excellent relationship between tie and subgrade deflection is shown in Figure 3, where 56 percent of the roadbed deflection occurred within the subgrade at this site.

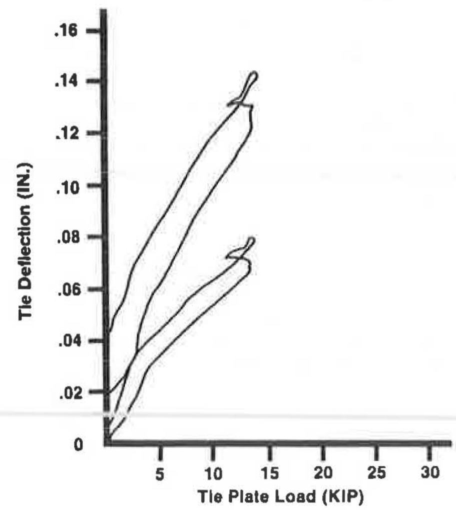


FIGURE 2 Tie and subgrade deflections.

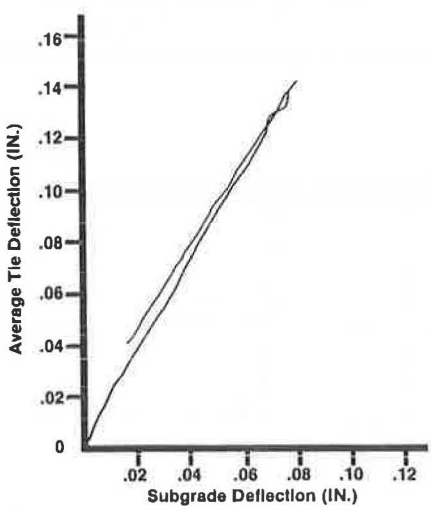


FIGURE 3 Tie versus subgrade deflections.

A substantial amount of chart reading and graph plotting was required to develop this type of information for the 12 measurement sites. The ratio of subgrade to tie deflection for the 12 sites is plotted against depth of granular as measured from base of tie in Figure 4. The data fit is surprisingly good considering the relative crudeness of the measurement system. One of the points off the curve resulted from frozen roadbed (measurements were taken in early May 1979), and the other two points represent data scatter, the bugaboo of track measurement researchers. It should be noted that the curve simply shows a ratio of measured track deflections and is independent of track loading. At 20-in.

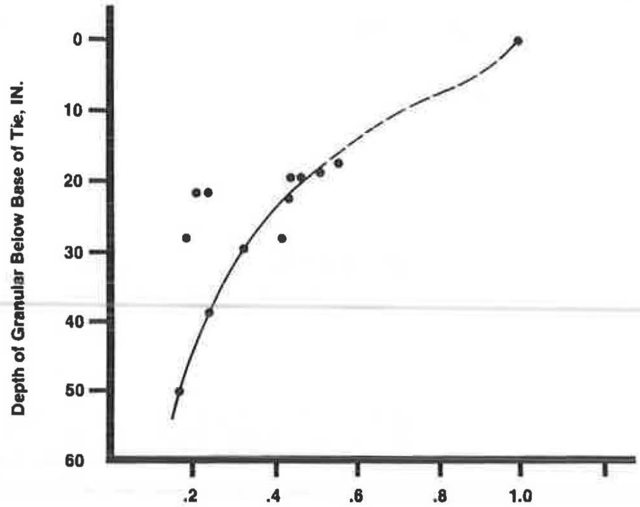


FIGURE 4 Proportion of total deflection within subgrade (C_S/C_T).

granular depth, 50 percent of the track deflection came from the subgrade and 50 percent came from the granular layer, whereas at a depth of 50 in., less than 20 percent of the total deflection came from the subgrade. The relatively good data fit was most encouraging, because it meant that the roadbed behaved in a logical manner, and that a simple means had been found to document that behavior. Once organized, it was possible to measure two sites per day.

The deflections of both tie and subgrade, normalized per kip of tie plate loading, have been plotted in Figure 5. There is reasonably good fit for both curves, but it is apparent that there is less scatter in the subgrade data points. Ties tend to flop around, but subgrade is more sedate and well-behaved. Through the previously mentioned procedure of plotting tie load-deflection curves, an attempt was made to eliminate tie slack and initial spongy deflection from the results, but it is clear that it was not entirely successful. There is one somewhat surprising result in this graph in that the tie and subgrade curves are roughly parallel. The difference between the two curves represents the amount of deflection that occurred within the granular layer, and this deflection appeared to be relatively con-

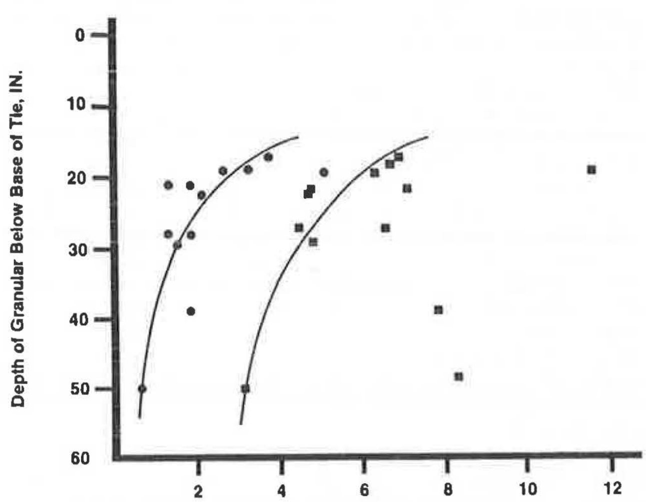


FIGURE 5 Normalized subgrade and tie deflections.

stant for granular depths ranging from 17 to 50 in. It is believed that, with shallower granular depths, a significant portion of the granular deflection comes from shear deformation, whereas at greater depths, the granular layer undergoes compressive deformation with very little shear.

The absolute peak subgrade deflections have been plotted against depth of granular as shown in Figure 6. The curve has been drawn through the lower data points because they represent the higher deflections, and it was desired to adopt the conservative data approach. At 19-in. granular depth, the subgrade deflected eight times more than it did at 15-in. depth. There are three small kick-off curves shown below the main curve that trace the deflections that occurred 6 in. below the top of subgrade. The coned subgrade rods appeared to be giving satisfactory results, and it was believed that if a series of rods could be driven to different depths in the subgrade, then the elastic deflection curve could be plotted and the subgrade soil strains could be determined. These three trial measurement sites indicated that the apparently crude rod system could be used to measure subgrade soil strains.

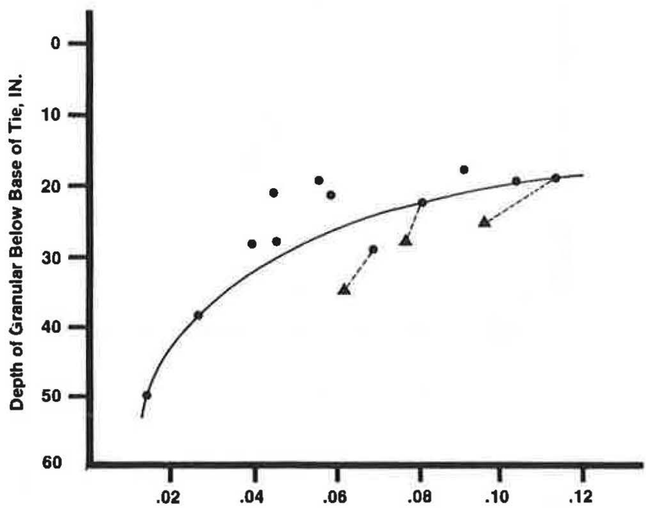


FIGURE 6 Peak subgrade deflections.

Reflecting on the main curve, it will be realized that it could be used as a granular depth design chart provided a level of subgrade deflection could be specified that should not be exceeded. For example, if 0.04 in. was specified as maximum permissible subgrade deflection, then roughly 32 in. of granular depth would be required. Unfortunately, no such subgrade deflection limits have been defined. There are shortcomings in this simplistic approach in any event, because traffic density has not been taken into account. The curve is, however, attractive from another viewpoint. Wheel and tie loadings need not be measured to generate this graph, and the problem of how to deal with load spectra has been largely eliminated. Hundreds of chart readings and plot load-deflection curves need not be taken in order to normalize the subgrade deflections with respect to load. It is necessary only to scan the charts, read the peak deflections, and assume that they represent the worst loading conditions that need to be considered. Data analysis time with this approach was reduced by a factor of 20.

A number of conclusions were drawn as a result of the 1979 Edmonton area measurements.

- The tests demonstrated the feasibility of using simple instrumentation to rapidly obtain useful track and subgrade elastic deflection measurements.
- There was a fairly well-defined relationship between tie and subgrade elastic deflection and depth of granular in track.
- This relationship was established without specific knowledge of granular or subgrade properties.
- The tie and subgrade deflections could be defined as a function of tie loading or as an envelope encompassing maximum deflections from the upper end of the track loading spectrum (without having to measure the actual track loading).
- The curves relating tie and subgrade elastic deflections to depth of granular could be used as design charts if limiting elastic deflection criteria could be specified for the traffic on a territory.
- There was far less variation in subgrade deflection than there was in tie loading or tie deflection, indicating the desirability of working through deformation measurement at depth as opposed to load or stress measurement at the surface.
- Subgrade and tie deflections were directly proportional except at low loading levels where slack and initial sponginess came into play.
- For comparative tie loading levels, deformations within the granular layer were relatively constant for depths ranging from 17 to 50 in.
- In order to define granular depth design criteria, relationships among roadbed elastic deflections and strains, roadbed settlement rates, track loading spectra, and traffic density would have to be established. These would have to be correlated with acceptable track maintenance levels.

Encouraged by the 1979 results, it was recommended that work be continued in 1980. Three additional sets of elastic deflection measurements were taken in the Edmonton area, and subgrade settlement plates were installed at five Edmonton area sites. To combat the logistics of taking measurements 2,000 miles from home base, two test sites were established in the Montreal area. A series of elastic deflection measurements were taken at one site, and rail and subgrade settlement rates were measured at both sites.

The subgrade elastic deflection measurements taken at the three Edmonton area sites in June 1980 correlated well with the May 1979 measurements, indicating again that the subgrade was a reasonably well-behaved component of the track structure. Tie deflections and loads showed more erratic behavior, as they had in 1979, indicating that tie slack and ballast compaction were variables that affected results.

At these 1980 Edmonton sites, three cones were driven into the subgrade: one at subgrade surface, one at 5 in. below subgrade surface, and one at roughly 10 in. below subgrade surface. Plots of elastic deflection with subgrade depth were drawn, and subgrade strains were estimated from the slopes of the curves. The curves were somewhat erratic, and apparent soil strains at subgrade surface ranged from 0.06 to 0.16 percent.

At the Montreal Joliette Subdivision test site, four cones were driven to depths of 12.50, 22, 29.50, and 40 in. The track had 10 in. of ballast and 10 in. of subballast. It should be noted that one of the cones was within the granular layer. Tie plate loads were measured, and, in addition, wheel loads were measured with rail shear strain gauge circuits. By this stage of the study, it had been realized that wheel loads were a better indicator of traffic conditions than were the more variable tie plate loads.

Sample deflections from the four cones have been plotted against depth from tie base in Figure 7. The individual plots are for different wheel loads. The deflection curves were very smooth, indicating that the simple cones can be used to define the railroad track elastic deflection curve.

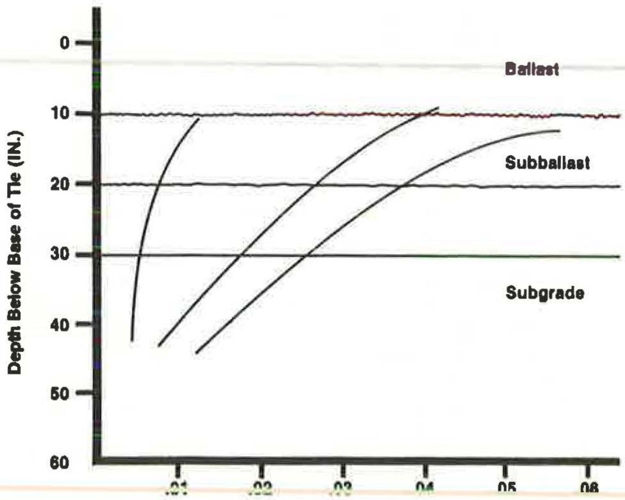


FIGURE 7 Subgrade deflections with depth.

Many of the roadbed theories available today have been presented as two- and three-layer theories. Substantive complications have arisen in trying to define and categorize the properties of the individual layers. As a result, these theories have never been validated and have never been used by railroad engineers. In Figure 7, the deflection curves run smoothly from the subgrade through the interface and into the subballast layer. Although these few measurements cannot be presented as absolutely correct, they do illustrate a significant point. Limited research dollars should be spent taking field measurements, where the true story lies buried in the existing roadbeds.

At the Joliette site, measurements were taken in August 1980, and the cones were left in place over the winter. From March 26, 1981, when the roadbed was frozen solid, to November 26, a series of 10 sets of elastic deflection measurements was taken with the same undisturbed setup. Soil strains were determined from the slopes of the elastic deflection curves, and these have been plotted against wheel load in Figure 8. Best-linear-fits were drawn through the scatter of data points for the 10 sets of measurements. The roadbed behaved as a solidly frozen block on March 26, was partially thawed on April 1 and April 23, and was completely thawed and soggy in May. In October and November, it had dried and stabilized. It will be noted that wheel loads have now been used in place of tie plate loads, and that soil strains at subgrade surface were on the order of 0.20 percent for 40 kip dynamic wheel loads. A tremendous amount of chart reading and data analysis work went into the preparation of this graph.

Unlike highway departments, railroads do not impose load restrictions during spring thaw. Of necessity they sometimes impose temporary slow orders at certain locations. The traffic must go through. Realizing that only the higher track deflections were significant, the envelope of peak strains has been plotted in Figure 9. This graph, requiring only

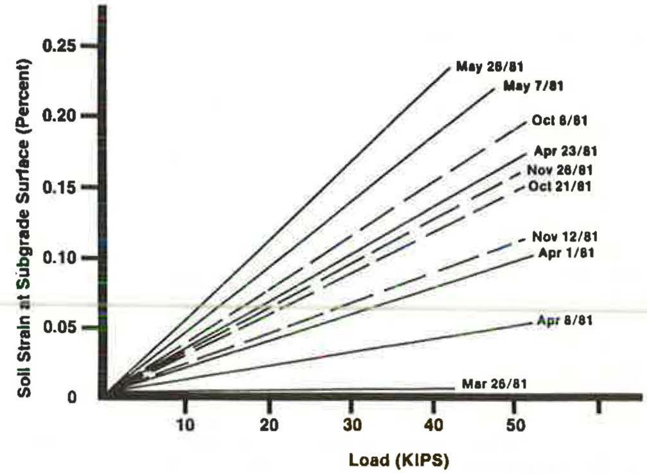


FIGURE 8 Soil strains.

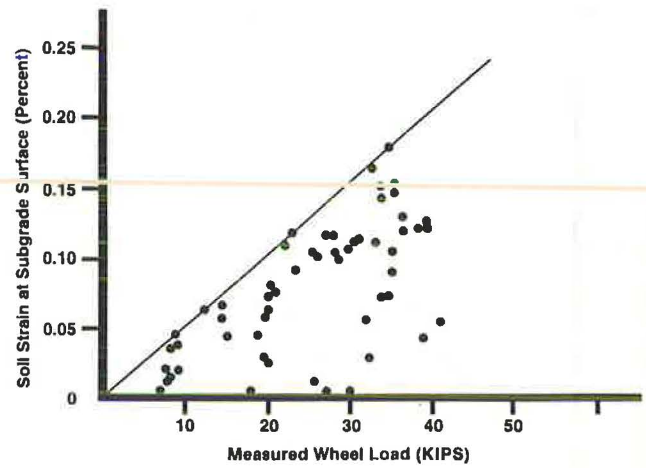


FIGURE 9 Soil peak strains.

a fraction of the work of Figure 8, shows the same peak strain relationship. This is termed simplification of data analysis, an important consideration in any longer term research project.

Having defined to some degree the roadbed elastic deflection behavior, the next problem was to relate this information to track maintenance requirement. Roadbed elastic deflection by itself is not necessarily harmful. Indeed, those railroads that have opted for rigid slab track construction have had to take great pains to reprovide elastic deflection from special pads. The requirement for track surfacing maintenance arises from uneven track settlement.

Track settlements were measured at five Edmonton area sites and two Joliette Subdivision sites. Six-inch square steel plates welded to 0.50-in. diameter rods were dug in to rest on top of the subgrade at a point midway between ties and just clear of the field side of the rail. At Joliette, elevations of rods were read with precise rod and level. For the Edmonton sites, pipes were welded to the settlement plates, and concentric benchmark rods were driven to depths of 9 ft. Settlements were recorded by means of special jig and dial gauge, measuring displacement between pipe and benchmark. At the Joliette sites, rail settlements were also measured with precise level.

A sample graph of rail settlement at one of the Joliette sites is shown in Figure 10. Frost heave of 4 to 5 cm or nearly 2 in. occurred each winter.

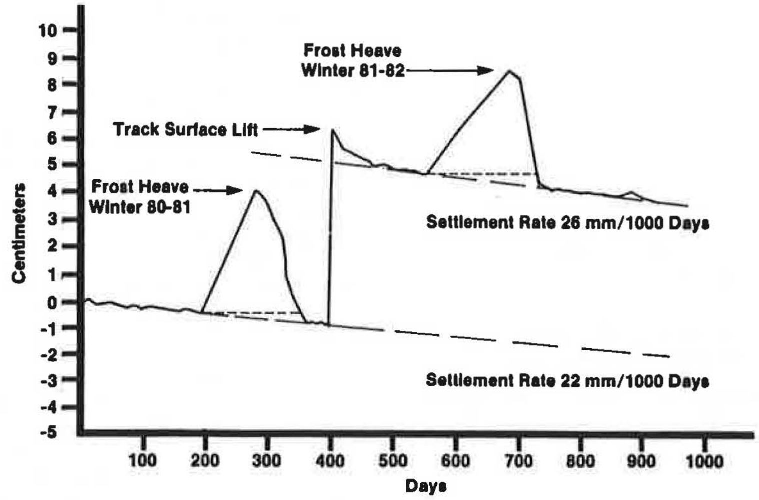


FIGURE 10 Sample rain settlement.

During spring 1981, a track surface lift of roughly 7 cm or nearly 3 in. was carried out. There are several points of interest in this graph. The rail settlement rate increased following the track lift, even after the initial rapid settlement stage. This graph is an example of why the British Rail development of the stone blowing technique, more commonly called troweling in North America, would provide significant benefits in track maintenance. The graph also shows that spring thaw settlements apparently continued at the same rates as those of the previous fall. Perhaps zero settlement occurred during the winter, and the extra little drop in spring brought the settlement rates into line. It is clear from this graph that the rail was settling.

During the course of the Joliette measurements, the track had also been skin-lifted and hand-tamped at the second site. As shown in Figure 11, the average rail settlement rate was 9 mm per year for both sites. Lifts of 6 mm or 0.25 in. were lost almost immediately, and even the 70 to 75 mm machine lift would be dissipated in 8 years. As the rates of subgrade settlement were far lower than the rates of rail settlement, it is speculated that there must be lateral migration within the granular layers.

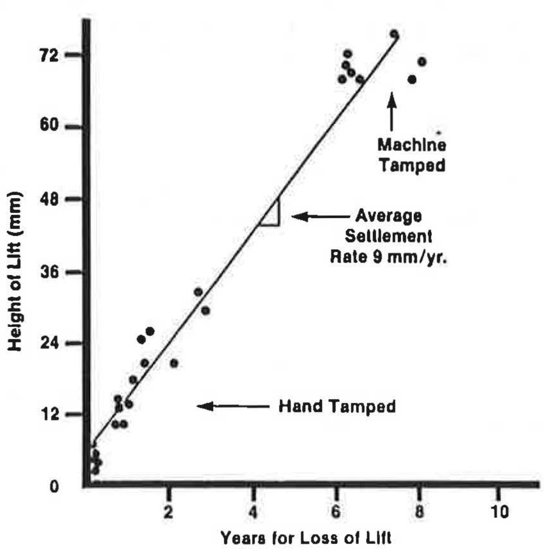


FIGURE 11 Loss of track surface lift.

The subgrade settlement rates in mm per month have been plotted against depth of granular as shown in Figure 12. The Edmonton area settlement points fell in a reasonably smooth curve, but settlement rates for the Montreal sites were lower. There are two explanations: the Edmonton subgrade consisted of 53 to 67 percent clay, 25 to 44 percent silt, and 3 to 8 percent sand, whereas the Montreal subgrade consisted of 15 percent clay, 50 percent silt, and 35 percent sand. In addition, traffic densities were heavier around Edmonton, and there were far greater concentrations of 100-ton cars. As track surface maintenance is conducted on a time schedule, the monthly or yearly rates of settlement could be useful for maintenance planning.

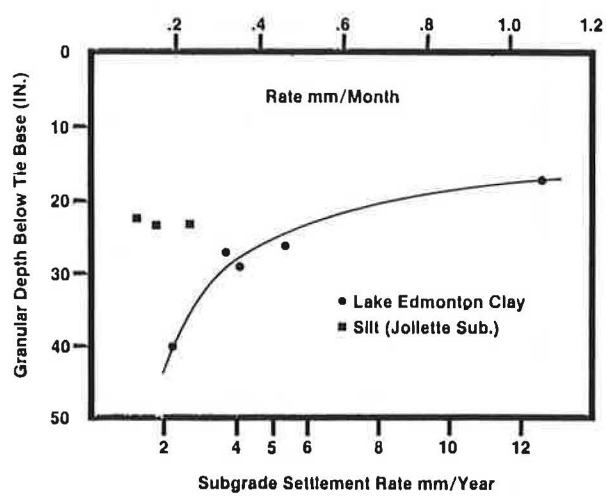


FIGURE 12 Subgrade settlement with time.

Subgrade settlement rates per million gross tons (MGT) of traffic have been plotted against depth of granular as shown in Figure 13. The curve fit is quite smooth for both the Edmonton and Montreal subgrade soil types. At 17-in. granular depth, the subgrade settlement rate was roughly 10 times greater than it was at 30-in. granular depth. If the traffic density at the 17-in. granular site was suddenly increased to the same level as at the 30-in. granular site, the track would literally sink out of sight and the Sangudo Subdivision experience would be re-

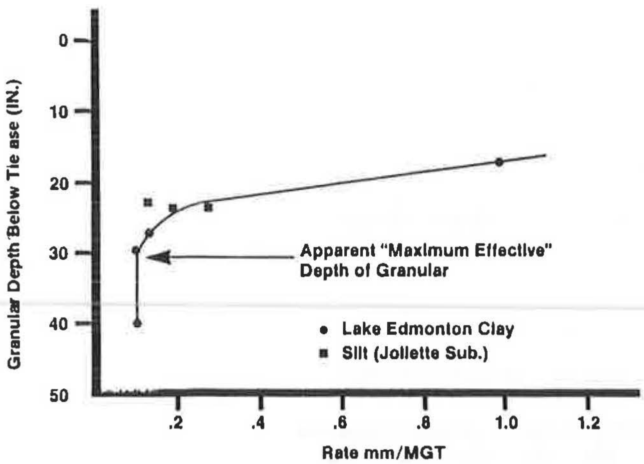


FIGURE 13 Subgrade settlement with traffic.

peated. Note also from this graph that no apparent benefit was derived in increasing the granular depth from 30 in. to 40 in. Although this graph has accounted for traffic density, it still has not taken account of track loading spectra, and there were significant differences in concentrations of 100-ton cars at the various sites.

Based on the amount of information available, but without any detailed assessment of the amount of track surface maintenance required at the various sites, it was somewhat arbitrarily decided that a subgrade settlement rate of 5 mm per year would provide for an acceptable track maintenance regime. A preliminary guideline for selection of granular depth requirement is shown in Figure 14. This applies basically to Lake Edmonton clays.

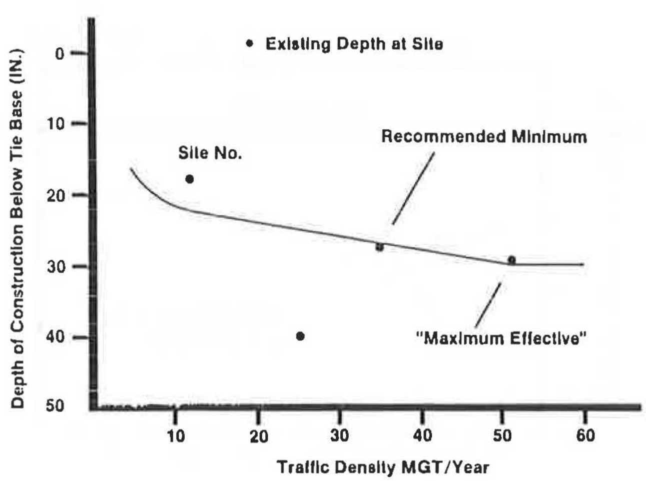


FIGURE 14 Preliminary granular depth design chart.

At the low traffic densities that are common on branch lines, granular depths of 17 to 20 in. would provide acceptable maintenance levels. At Site No. 17 with 12 MGT of traffic, 5 in. additional granular would be required to bring track settlement rates into line. At Sites No. 11 and No. 15, the granular depths appeared to be satisfactory for the current traffic densities of 35 and 50 MGT. At Site No. 14 with 40-in. granular depth and traffic at 25 MGT, there is far more granular depth than is required. The graph shows that there is little to be gained by increasing granular depths beyond 30 in.

Field measurement work was terminated in 1982, and McGill University laboratory soils work was terminated in late 1983. These terminations were the result of staff reductions and a heavy influx of more urgent shorter term projects. Railroad researchers are on the firing line, and they are far too few in numbers to enjoy the luxury of sitting in ivory towers.

It is not possible to state that this project has been successfully concluded, because there is still much cross-correlation of data that should be carried out. The final reports on field measurement and laboratory work at McGill University are still to be issued, and this paper is the closest that can be considered a project final report.

In retrospect, we undertook more than we could afford with our available resources, and we ran up blind alleys in some of the analysis work. However, the measurement and data analysis techniques required to complete the job have been developed, and those parameters that should and should not be recorded have been identified. Perhaps other railroad research agencies will continue this research.

What would have been preferable was a granular depth selection chart related to traffic density, track loading spectra, and a range of subgrade soil types. This chart would be directly applicable for new track construction and for existing tracks faced with significant increases in traffic. One of the problems with tracks constructed many years ago and having had many surface lifts in their lifetime is that nobody really knows how much granular depth is present. Although it could be determined through countless diggings, it would be far more practical if elastic deflections could be measured with a moving track modulus measurement car. Not only would soft spots be immediately identified, but the magnitudes of deflections would indicate how much ballast surface lift would cure the problem. The track geometry car surface roughness measurements in this study correlated quite well with subgrade settlement measurements, indicating a close link between depth of granular and track maintenance requirement.

It was too optimistic to believe that a 4 1/2 year study with limited resources would provide a usable granular depth selection chart. After all, after more than 100 years of railroading there is still no universally accepted roadbed design theory.

ACKNOWLEDGMENT

The author acknowledges the financial assistance given by the Transport Canada Transportation Development Centre during this study.

Publication of this paper sponsored by Committee on Track Structure System Design.

Demonstration of the Rail Energy Cost Analysis Package: The Route Perspective (RECAP II)

MICHAEL E. SMITH

ABSTRACT

A demonstration of the Rail Energy Cost Analysis Package: The Route Perspective (RECAP II) is presented. RECAP II is a computer model that can be used to perform engineering-economic cost analysis of specific rail movements. The program was designed in response to the need for costing tools that are adequate for analyzing changes in railroad technology. Existing costing tools, such as the Uniform Rail Costing System, assume fixed technology, and therefore cannot be used to assess the value of technological advances. The RECAP II model calculates costs by determining the resources consumed in making a specific movement. To perform this function, the model contains a Train Performance Simulator that calculates train running time and fuel consumption. With this information, and with track maintenance costs developed from a matrix of marginal costs of track maintenance, RECAP II determines the cost of running a specific consist over a specific route. Through a demonstration run detailed in the paper, RECAP II is shown to be a powerful decision-making tool. With its help, informed choices can be made among many service and technology options.

For many years, railroads have relied exclusively on accounting procedures and techniques to determine the cost of moving freight. By far the greatest source of railroad cost information in the past has been Rail Form A and its successor, the Uniform Rail Costing System (URCS). These accounting-based costing methods were developed primarily to meet the needs of regulatory cost analysis. That is, they were used to determine the variable cost of given moves in order to establish maximum legal freight rates.

INTRODUCTION

The basic approach to accounting-based costing is to determine unit costs (e.g., cost per switch engine minute) by regressing all expenses associated with an expense category (e.g., the time-related expense of running switch engines) against the number of service units (e.g., number of switch engine minutes) associated with that expense. After the resulting fixed and variable costs were developed, costs for a specific move were calculated by determining the number of service units of each category consumed and multiplying by the variable cost rate and adding fixed costs. A thorough description of the URCS system is available in Uniform Railroad Costing: 1980 Rail Cost Survey (1).

Although such systems may be quite useful for overall pricing strategy or for regulatory analysis, they are not particularly well suited to analyzing alternative service options and alternative technologies for two major reasons. First, the unit costs developed are highly aggregate, as they must be, having been developed as averages for an entire railroad. Because of this aggregation, cost differences resulting from special or individual cases cannot be accounted for. For example, a URCS-based analysis of the cost of moving a consist over severe grades would yield the same cost as running the same consist over flat territory. A second difficulty occurs because accounting-based cost analysis assumes fixed technology. For example, road-haul fuel costs are usually determined by multiplying appropriate unit costs by gross ton-miles and by locomotive unit miles. If a more efficient locomotive is used, or if cars are streamlined, the cost analysis procedure would overestimate fuel costs. Therefore, such cost analysis procedures are usually not adequate for assessing the value of technology improvements.

To better evaluate technological improvements, an engineering-based economic model is more useful. An engineering economic model uses engineering knowledge to determine the actual resources used in a specific move and then to calculate costs by determining the price and quantity of each resource. The Rail Energy Cost Analysis Package: The Route Perspective (RECAP II) is a tool that can be used to determine point-to-point rail costs based on an engineering analysis of the movement.

Before specifically discussing RECAP II, however, it is useful to make a few points about engineering economic models in general. Although such models are useful for analyzing service and technology alternatives, they are of limited use in overall pricing strategy or regulatory analysis. This is because an engineering economics analysis will generally provide a reasonable estimate of the difference in cost between two alternatives, but will usually provide only a lower bound estimate on the absolute cost of any given alternative. By modeling a specific circumstance and determining costs by evaluating resources consumed in that circumstance, engineering economics models can miss certain components of cost, such as administrative overhead and costs associated with unforeseen events (e.g., train accidents).

Given these constraints, potential applications for RECAP II can be listed. The following is a brief, but not exhaustive, list of potential uses for RECAP II:

1. Comparison of relative costs of different routes;

2. Determination of the cost and operating effects of alternative train makeup strategies;
3. Determination of the effect on all categories of the cost of reducing tare weight, increasing lading, or both;
4. Calculation of cost savings generated by reducing aerodynamic drag or reducing bearing resistance;
5. Calculation of absolute and relative cost impacts of changing crew compensation methods and costs;
6. Determination of cost and operating impacts of various refueling strategies; and
7. Calculation of cost and operating impacts of improving locomotive performance through increased traction, increased horsepower, and decreased weight.

RECAP II, then, is a powerful tool to assist railroad management in making operating and investment decisions that affect the cost of operation over specific routes.

PROGRAM LOGIC

RECAP II is composed of several submodels. It is primarily built around the Train Energy Model (TEM), which is an enhanced Train Performance Simulator (TPS) (2) developed by the Association of American Railroads (AAR) Technical Center in Chicago, Illinois. The full RECAP II model contains a TEM module, as well as a Driver program, a Cost model, and a matrix of data generated by the Track Maintenance Cost Model (discussed later). A flow chart for RECAP II is shown in Figure 1.

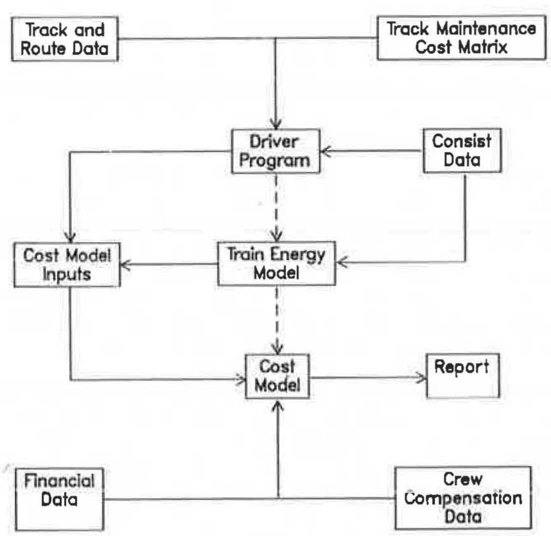


FIGURE 1 Flowchart for RECAP II.

RECAP II is written in FORTRAN and is currently designed to run on a Digital Equipment Corporation model 2060 (DEC 20) computer. Railroads that are members of the AAR may establish an account for access to the DEC-20 computer at the AAR Technical Center in Chicago, Illinois. RECAP II can then be executed remotely via modem. In the future (sometime in 1985), a version of RECAP II will be available for the IBM Personal Computer (PC). The PC version will be written in FORTRAN and BASIC.

The program begins by executing the Driver program. The Driver program reads two input files, the track and route profile file and the consist and route data file. These files contain the information necessary to specify the train consist and the route over which it runs. The files are based on similar files used to run TPS models. After the Driver program has read the input files, the program then creates a cost analysis file to be read by the costing subroutines. The cost analysis file contains the following information:

- Consist general data: makeup, gross weight, net weight, cubic capacity.
- Locomotive data: purchase prices, availability ratios, running cost per mile.
- Car data: daily rate, per-mile rate.
- Crew data: crew change points, crew district boundaries.
- Track maintenance cost data: incremental track maintenance cost for each track segment.

The last data category (track maintenance cost) is determined based on input of a track maintenance cost matrix. The matrix provides the marginal track maintenance cost per gross-ton-mile based on the following characteristics: (a) type of rail; (b) gross tons per year traversing that segment; (c) type of consist (standard unit train, heavy unit train, intermodal, mixed freight, or empty); (d) grade; and (e) curvature. The costs in the matrix were determined by executing the Track Maintenance Cost Model (TMCOST). TMCOST is an AAR-enhanced version of the Canadian Institute for Guided Ground Transport Rail Wear Cost Model (3). The track maintenance cost matrix provided with RECAP II is based on industry averages.

After the Driver program has produced the cost analysis file, it passes control to the TEM. The TEM is a train simulator similar to the TPS, with the exception that the TEM produces slightly more information about energy use. In modeling the train over the route, the TEM will handle the train by attempting to match the train's speed to the speed limit, as found in the track file. For trains that run substantially below the speed limit, the speed limits in the track file should be adjusted accordingly. The TEM stops the train at user-specified points for a user-specified dwell time. This feature is useful in including stops and signal delays that the user believes are representative of the average case. Like the Driver program, the TEM produces a file for input to the Cost model. This file contains the following information for a large number of points on the route:

1. Distance in feet,
2. Elapsed time since beginning of trip,
3. Cumulative fuel consumption, and
4. Current throttle notch position.

The frequency of entries in this file varies; however, it averages about one entry for every 200 to 500 ft. The user specifies the mile per hour increment that must occur to create an entry in this file. The TEM also produces a summary report on train makeup and the train's trip history. Later, the Cost model appends cost information to this report.

After the Driver program and the TEM have developed the appropriate files, control is passed to the Cost model. The Cost model determines the following costs of running the specified consist over the specified route:

1. Crew costs,
2. Locomotive ownership and running costs,
3. Car ownership and running costs,
4. Fuel cost, and
5. Track maintenance costs.

Also, the model will determine each of these costs per gross ton-mile, per net ton-mile, and per cubic foot-mile. In the case of a train running empty, the costs per net ton-mile will be set to arbitrarily large numbers.

To determine these costs, the Cost model requires additional inputs; therefore, data are read from the financial data file and the crew compensation file. With that additional information, the Cost model then has all the data it needs to calculate costs.

Crew costs are calculated by the time-distance formula used throughout the industry. The Cost model reads through the train running history as output by TEM and stops when the distance marker exceeds the distance marker for the first crew change point. At this point, the elapsed time and elapsed distance are used to calculate crew basic pay, in accordance with the following formula:

$$p = b + m*(100 - d) + b*[t - (d/12.5)]*.1875$$

where

- p = basic pay,
- b = basic daily rate,
- m = mileage rate (or zero, if d is less than 100),
- d = distance traveled, and
- t = elapsed time.

To the basic pay is added a fixed arbitrary allowance to yield gross pay. Gross pay is then multiplied by 1 plus the fringe benefits rate to yield total compensation.

Locomotive ownership and running costs are calculated on the basis of locomotive data in the consist file, the elapsed time and distance calculated by the TEM, and, because locomotives are a capital asset, the information in the financial data file. Locomotive ownership costs are developed by first calculating an equivalent hourly rent on the equipment. This equivalent rent is calculated on the basis of each locomotive's purchase price and the present value of depreciation tax shelters and investment tax credits. The equivalent hourly rents are multiplied by elapsed time in hours to yield locomotive ownership costs. Locomotive running costs are determined by multiplying the running cost per mile (user input) for each locomotive by the number of miles traveled.

Car ownership costs are determined for each car by dividing the per diem rate by 24 and multiplying by the elapsed time in hours. The per diem rate input by the user should be the actual rate divided by the proportion of time that the car is in service. Car running costs are determined by multiplying each car's mileage rate by the number of miles traveled.

Fuel costs are determined in a straightforward manner. The fuel consumption calculated by the TEM is multiplied by the price per gallon specified by the user to determine fuel cost. Track maintenance cost is calculated in the Driver program, as described earlier.

SAMPLE ANALYSIS

As an example application for RECAP II, consider the following. A railroad operates a unit coal train in a mountainous area. Railroad officials are trying to decide whether to replace 100-ton coal cars (263,000 lb gross weight) with 112-ton coal cars (286,000 lb gross weight). They would like an estimate of the line-haul operating costs of moving the coal using each car type.

The first step in analyzing the alternatives is to prepare the data needed to execute RECAP II. The following information is needed: (a) track and route profile data, (b) consist and route data, (c) financial data, and (d) crew compensation data.

For this example, it is assumed that most of the track and route profile data are available from existing files used in executing a TPS. In addition to the data usually found in a TPS route profile deck, the following information is also required: (a) gross tons of traffic at each point, (b) crew district boundaries, (c) crew change points, (d) track type at each point, and (e) track lubrication data. For this example, it has been assumed that all the track on the route is 132 lb/yd hardened continuous welded rail, track is unlubricated, and gross tons of traffic is 30 MGT/year on each track at all points. Crew district boundaries and crew change points are given on the system schematic shown in Figure 2.

A summary of the route data available is provided in Table 1. Together with the system map in Figure 2 and the requirement to model a train from Hub A to

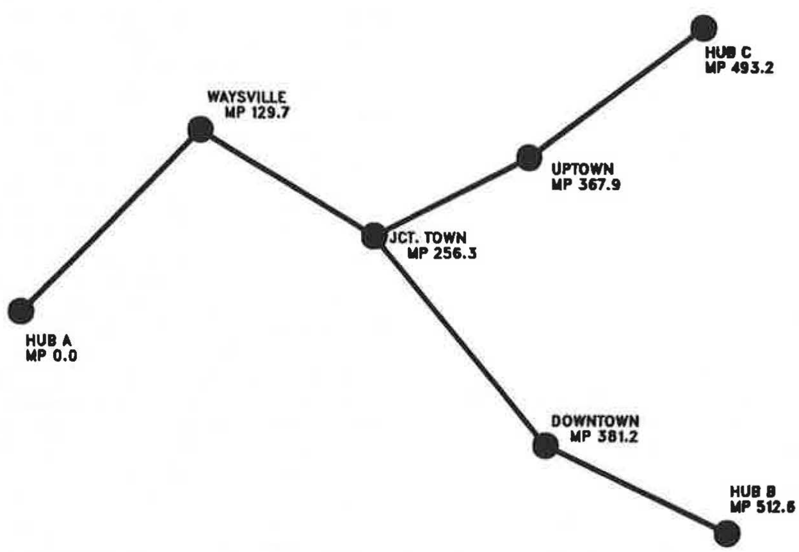


FIGURE 2 System schematic. (All stations shown represent crew district boundaries and actual crew change points for this route.)

TABLE 1 Summary of Track and Route Profile Data (File name = TRAK)

Segment No.	Begin Mile	End Mile	Begin Station	End Station	On Route
1	0.0	129.7	Hub A	Waysville	Yes
2	129.7	256.3	Waysville	Junction Town	Yes
3	256.3	367.9	Junction Town	Uptown	No
4	367.9	493.2	Uptown	Hub C	No
5	256.3	381.2	Junction Town	Downtown	Yes
6	381.2	512.6	Downtown	Hub B	Yes
7	512.6	381.2	Hub B	Downtown	No
8	381.2	256.3	Downtown	Junction Town	No
9	493.2	367.9	Hub C	Uptown	No
10	367.9	256.3	Uptown	Junction Town	No
11	256.3	129.7	Junction Town	Waysville	No
12	129.7	0.0	Waysville	Hub A	No

Hub B, a route can be selected consisting of segments 1, 2, 5, and 6.

The consist and route data file is the main driving file that the user is likely to change most often. It contains the following information: (a) general job data, (b) route segment list, (c) general train data, (d) data on locomotive units, and (e) data on cars.

A summary of the general job data is provided in Table 2. The first item is for selection of summary or detailed printed reports. Because RECAP II is only capable of summary printouts at this time, they have been selected. As indicated in Table 1, there are four route segments involved in this analysis. The track file information item indicates that the route profile deck is on disk with file name TRAK.KED. The coasting overspeed values indicate that the train may be permitted to exceed the posted limit by 10 mph or 25 percent (whichever is less) before applying brakes. The next three items are switches for certain summary printouts. All have been set to 1 to indicate a desire that they all be printed. The cost of fuel is found to be 85¢ per gallon for this railroad. The iterative velocity increment is the change in speed that will cause the simulation model to update the train movement. The setting of 1.0 mph is standard. The duplicate segment switch has been set to zero, thereby disallowing the same route segment from appearing twice in the route description.

TABLE 2 Job Data

Item	Value
Printout type	Summary
Number of route segments	4
Track file information	DSK: TRAK
Costing overspeed (mph)	10
Coasting overspeed (%)	25
Throttle position summary	1
Velocity range summary	1
Energy use summary	1
Cost of fuel	85.0
Iterative velocity increment	1.0
Duplicate segment switch	0

The next set of data in the consist and route data file identifies the route segments used in this run. These data are indicated as follows:

Item	Value
Milepost at special begin point	None
Segments on route	1, 2, 5, 6
Milepost at special end point	None

The first item is a milepost at any special begin point. That is, if a trip begins in the middle of the first segment listed, this is the milepost where the trip begins. In this case, the trip begins at the beginning of the first segment, so there is no special begin point. Similarly, there is no special end point, as indicated in the last data item. The remaining data item is the list of route segments, given in order traversed, defining this trip.

The next set of data required for the consist and route data file contains general information about the train. These data are given in Table 3. The name given to this train is COAL. The departure time is set at 00:00 (midnight) so that all times given are elapsed times. Wind is assumed calm. The train is given seven locomotives. The adhesion limit is 0.23; that is, the power axles can transmit a force, longitudinal to the rail, no greater than 23 percent of the weight on that axle before slipping. The 23 percent figure is representative of dry rail under normal conditions. The maximum speed of the train is set at 80 mph. This will override any greater speed limits found in the route profile data (if any). The brake pipe pressure (air brakes) is set at 80 psi. The coasting overspeed switch is off; so rather than apply brakes, the train will be allowed to exceed speed limit slightly when coasting.

TABLE 3 General Train Data

Item	Value
Train name	COAL
Departure time	00:00
Wind speed/direction	0
Number of locomotives	7
Adhesion limit	0.23
Maximum speed	80
Brake pipe pressure	80
Coasting overspeed	0
Consist code	2
Number of cars	101
Curving resistance code	0
Resistance equation code	2
Tangent lubrication resistance factor	1

The consist code of 2 tells RECAP II that this is a unit train with standard car loaded-weights of 263,000 lb. The number of cars in this train is 101 (100 hoppers and one caboose). The curving resistance code of 0 indicates that the cars contain standard trucks and that the standard Davis formulation for curving resistance is to be used. The resistance equation code of 2 selects the Davis equation for train resistance as modified by the Canadian National. The program will calculate reduction of resistance due to tangent lubrication through the input of the tangent lubrication resistance factor. The value of 1 here indicates that 100 percent of tangent rolling resistance will be used and no resistance reduction due to lubrication on tangent track is being modeled.

Data for the locomotive units in the consist are given in Table 4. In this train there are seven SD-40-2 units in the consist. The data for each, as given in Table 4, are relatively straightforward and standard for these units. The last few items on the data list could use additional illumination, however. The low-speed and high-speed breaks refer to points on a graph of tractive effort versus velocity where the slope of the curve changes. The transmission conversion factor is an indicator of transmission efficiency. No resistance coefficients are being supplied, so the model will assume standard coefficients. The economic factors of life, availability ratio, and running/cost per mile are all

TABLE 4 Locomotive Data

Item	Value
Name	SD-40-2
Total weight	195 tons
Weight on drivers	195 tons
Unit length	66 ft
Rated horsepower	3,000 HP
De-rated horsepower	3,000 HP
Total number of axles	6
Number of driven axles	6
Energy rate running	0.0559
Energy rate idling	0.092
Low-speed break	12
High-speed break	13
Transmission conversion factor	308
Resistance coefficients	Standard
Economic factors	
Price new ($)	1,250,000
Life	20 years
Availability ratio	0.90
Running cost/mile ($)	1.16

based on the railroad's experience with this model locomotive. The price new would be the current purchase price for an SD-40-2 locomotive or its equivalent.

Data for the cars in the consist are provided in Table 5. The car name is an arbitrary label that must be unique for each type of car in the train. Because RECAP II will be run separately for the heavy and light cars, both cars are labeled Hopper without risking ambiguity. Data on the 100-ton hopper are given in the first column and data on the 112-ton hopper are given in the second column. The loaded weight for all cars is equal to the tare weight of the car plus the weight of the lading. The length of the car is the coupler-to-coupler length. The remaining general data are self-explanatory. The resistance coefficients used in this case are standard Davis equation coefficients. The cross-sectional area of the heavy car has increased to account for its larger size, however. The streamlining and exponent values for aerodynamic characteristics do not apply here. The per diem and mileage charges shown in the economic data section are the time and distance rates for the cars when off-line. It should be noted that the 112-ton capacity hoppers may be more expensive or have a shorter life, thus requiring a higher per diem; or they may be more costly to maintain, requiring a higher mileage charge. In this study, costs are compared assuming no increase in these figures; however, performing alternative analyses with variations in these amounts would be a useful sensitivity study.

The crew compensation data file contains the following four numbers on crew pay: basic daily rate, mileage charge, arbitrary allowance, and fringe benefit rate. The file requires these four numbers for each crew staffing the train as it traverses the route. In this example, all crews have identical pay rates, as follows:

Basic daily rate	394.16
Mileage rate	3.6629
Arbitrary allowance	184.88
Fringe benefit rate	0.35

For a typical crew traveling 129 miles in 7 hours, cost would be calculated as follows:

Crew cost = (Basic daily rate + 29 x (mileage rate) + arbitrary allowance) * (1 + fringe benefit rate)

or

Crew cost = (394.16 + 29 x 3.6629 + 184.88) * (1.35)
Crew cost = $925.11

The financial data file contains information about projected inflation rates, discount rates, and tax rates. In this example, the following values were used:

Item	Value (%)
Inflation rate	5
Discount rate	8
Tax rate	46
Investment tax credit rate	10

The output file from RECAP II contains an echo of inputs, an analysis of the simulated train run and a cost analysis. Because this paper focuses mainly on the cost analysis features of RECAP II, only those output pages relating to cost are provided here. The cost analysis begins on page 16 of the output, shown in Figure 3.

Page 16 of the output shows the crew cost summary. Starting, ending, and elapsed time and miles are shown along with a variety of cost categories. In this case, each crew receives a total compensation of $900 to $950 per trip, which is approximately $7.00 to $7.25 per mile. Because no trip exceeded 8 hours, no overtime pay is necessary.

Page 17 of the output (Figure 4) provides a summary of equipment costs. The table indicates running and ownership costs for each type of equipment in the train. In this case, most of the cost is car running cost. Overall, equipment costs are about 3.7 times the crew cost.

Page 18 of the output (Figure 5) shows a fuel cost summary divided by stopping points along the route. The information provided in this table is essentially the same as in the run summary and timetables noted earlier; however, the emphasis in this table is to clearly show the cost of fuel. In this case, fuel costs are about 2.7 times crew cost and about 2/3 of the equipment cost.

A track maintenance cost summary is provided on page 19 of the output (Figure 6). The costs given in this table are marginal costs. That is, the amounts represent the track maintenance money that would have been saved if the operation of this particular consist had not occurred. As expected, the cost of maintaining curved track is much higher per mile than the cost of maintaining tangent track.

The last page of the output (Figure 7) summarizes the cost for this run. For each category of cost, the following unit costs are provided: cost per gross ton-mile, cost per net ton-mile, and cost per cubic foot-mile. The total cost for this run is $30,982.47 or 0.60¢ per net ton-mile (NTM). If the cost appears low, remember that the cost of the

TABLE 5 Car Data

General data			
Car name	Hopper	Hopper	Caboose
Loaded weight	131.5 tons	143.0 tons	30 ton
Length	52 ft	52 ft	35 ft
No. of axles	4	4	4
Lading volume	4000 ft^3	4450 ft^3	0
Tare weight	31 tons	31 tons	30 tons
Resistance coefficients			
Rolling friction	0.6	0.6	0.6
Bearing friction	20	20	20
Flange friction	0.010	0.010	0.010
Air drag	0.0005	0.0005	0.0005
Cross sectional area	140	148	140
Streamlining	n/a	n/a	n/a
Exponent	n/a	n/a	n/a
Economic data			
Per diem ($)	18.00	18.00	9.00
Mileage charge ($)	0.13	0.13	0.07

CREW COST SUMMARY:

CREW NUMBER	CREW DISTR.	START TIME	END TIME	TOTAL TIME	START MILE	END MILE	TOTAL MILES	BASIC PAY	O-MILE PAY	O-TIME PAY	ARBIT. ALLOW.	BENEFIT ALLOW.	TOTAL COMP.
01	1	0: 0	5:38	5:38	0.00	129.67	129.67	394.29	108.60	0.00	184.80	240.69	928.38
02	1	5:38	11:13	5:34	129.67	256.27	126.59	394.29	97.34	0.00	184.80	236.75	913.17
03	1	11:13	16:35	5:23	256.27	381.20	124.93	394.29	91.25	0.00	184.80	234.62	904.97
04	1	16:35	20:10	3:35	381.20	512.55	131.35	394.29	114.74	0.00	184.80	242.84	936.67
TOTAL		0: 0	20:10	20:10	0.00	512.55	512.55	1577.16	411.93	0.00	739.20	954.90	3683.19

FIGURE 3 Crew cost summary—base case.

EQUIPMENT COST SUMMARY:

EQUIP-MENT TYPE	NUMBER OF UNITS	PURCHASE PRICE (EACH)	HOURLY OWNSHP COST	TOTAL HOURS OPER.	TOTAL OWNERSHIP COST	RUNNING COST CTS/MILE	TOTAL MILES OPER.	TOTAL RUNNING COST	TOTAL COST
SD-40	7	1250000.	9.75	20.17	1376.66	116.000	513.	4161.90	5538.56
HOPPER	100		0.75	20.17	1512.85	13.000	513.	6663.13	8175.99
CABOOSE	1		0.38	20.17	7.56	7.000	513.	35.88	43.44
TOTAL	108				2897.08			10860.91	13757.99

FIGURE 4 Equipment cost summary—base case.

FUEL COST SUMMARY:

STOP NUMBER	MILE-POST	ARRIVE TIME	DEPART TIME	IDLE TIME	FUEL CONS. AT STOP	CUM. FUEL CONSUMED	CUM. FUEL EXPENSE
1	129.33	3:36	5:36	2: 0	77.28	2670.82	2270.19
2	255.97	9:11	11:11	2: 0	77.28	6255.90	5317.51
3	380.91	14:33	16:33	2: 0	77.29	8829.41	7505.00
******	512.60	20:10	****FINAL DESTINATION****			11564.33	9829.68

FIGURE 5 Fuel cost summary—base case.

TRACK MAINTENANCE COST SUMMARY:

SEGMENT	LENGTH	TOTAL MGTM	CONSIST MGTM	MILES OF TANGENT	TANGENT TRACK MAINT. COST	MILES OF CURVE	CURVED TRACK MAINT. COST	TOTAL TRACK MAINT. COST
HUB A	129.70	3891.00	1.88649	114.38	766.26	15.32	123.80	890.06
WAYSVILLE	126.60	3798.00	1.84140	95.99	645.68	30.61	275.17	920.85
JCT. TOWN	124.90	3747.00	1.81667	96.38	650.18	28.52	277.76	927.94
UPTOWN	131.40	3942.00	1.91121	99.29	669.04	32.11	303.93	972.97
TOTAL	512.60	15378.00	7.45577	406.04	2731.16	106.56	980.66	3711.82

FIGURE 6 Track maintenance cost summary—base case.

COST OF RUNNING SPECIFIED CONSIST:

CATEGORY	TOTAL COST	COST /GTM	COST /NTM	COST /CFM
CREW COST	3683.19	0.00049405	0.00071503	0.00001797
LOCOMOTIVE COSTS:				
OWNERSHIP COST	1376.66	0.00018466	0.00026726	0.00000671
RUNNING COST	4161.90	0.00055827	0.00080796	0.00002030
CAR COSTS:				
OWNERSHIP COST	1520.42	0.00020395	0.00029516	0.00000742
RUNNING COST	6699.01	0.00089859	0.00130050	0.00003268
FUEL COST	9829.48	0.00131850	0.00190822	0.00004794
TRACK COST	3711.81	0.00049789	0.00072058	0.00001810
TOTAL COST	30982.47	0.00415592	0.00601471	0.00015112

FIGURE 7 Total cost summary—base case.

empty return trip has not been tabulated, and no overhead costs are included.

As mentioned earlier, the primary purpose of this study is to determine whether coal can be transported more inexpensively using 286,000-lb cars instead of 263,000-lb cars. Because a run of RECAP II has provided costs for a run of 263,000-lb cars, it is now necessary to run the model using 286-lb cars.

The input file for making the second run will differ only slightly from the first input file. The job data (Table 2) and route data will remain the same. The general train data (Table 3) will have two changes. First, the number of cars in the consist will be reduced from 101 to 91 to keep total lading weight as constant as possible. The base train contained 100 hopper cars, each with 100.5 tons of lading, for a total of 10,050 tons. This train contains 90 hopper cars, each with 112 tons lading, for a total of 10,080 tons. Second, the consist type code will be changed from 2 (standard unit train) to 1 (heavy unit train).

None of the inputs that describe the locomotives (Table 4) has been changed. The inputs that describe the cars (Table 5) inputs are shown in the second Hopper column of Table 5.

A portion of the output generated by running RECAP II with the revised input file is shown in Figures 8-12. The crew cost summary (Figure 8) indicates that there is no significant difference in crew costs for the two runs. Because crew rates are mostly determined by miles traveled, this is not unexpected.

The equipment cost summary (Figure 9) indicates a significant decrease of $825 from the previous run. This is partially because the second run took 3 minutes less than the first run. This 0.25 percent reduction in running time is due to the lower train resistance caused by reduced train gross weight (discussed in detail later). This running time reduction is responsible for saving 0.0025 x 13,758, or $34 in equipment ownership costs. The remaining $791 savings is a result of 10 fewer cars required to carry the same lading.

The fuel cost summary (Figure 10) indicates that this run used 11,610 gallons of fuel, which is 46 gallons more than the previous run, for an increase in cost of $39. The slight increase is due to the greater aerodynamic cross section of the 112-ton cars. This is offset somewhat by the reduction in train gross weight. Although each loaded car is 8.75 percent heavier than before, there are 10 percent fewer cars. As a result, the train is lighter by 280 tons. Because lighter trains offer less resistance to acceleration, the travel time for the trip is reduced; however, the faster speed and greater aerodynamic cross section of the high-capacity car result in a slight net increase in fuel use.

The track maintenance cost summary (Figure 11) indicates that the track maintenance costs are 9.3 percent higher when running a consist with the heavier cars. On the one hand, higher track maintenance costs may be expected because of the larger axle loads. On the other hand, a reduction in track maintenance costs may be expected because fewer total gross tons are moving over the track. The effects of these two components can be evaluated by examining the Consist MGTM column and the Total Track Maintenance Cost column. For the first run, dividing the $3,711.82 total cost by the 7.45577 million gross ton-miles (mgtm) yields a track maintenance cost of $497.85 per mgtm. In the second case, the track maintenance cost per mgtm is

CREW COST SUMMARY:

CREW NUMBER	CREW DISTR.	START TIME	END TIME	TOTAL TIME	START MILE	END MILE	TOTAL MILES	BASIC PAY	O-MILE PAY	O-TIME PAY	ARBIT. ALLOW.	BENEFIT ALLOW.	TOTAL COMP.
01	1	0: 0	5:37	5:37	0.00	129.67	129.67	394.29	108.58	0.00	184.80	240.68	928.38
02	1	5:37	11:11	5:34	129.67	256.26	126.60	394.29	97.35	0.00	184.80	236.75	913.17
03	1	11:11	16:34	5:22	256.26	381.20	124.94	394.29	91.27	0.00	184.80	234.62	904.97
04	1	16:34	20: 7	3:33	381.20	512.55	131.35	394.29	114.74	0.00	184.80	242.84	936.67
TOTAL		0: 0	20: 7	20: 7	0.00	512.55	512.55	1577.16	411.93	0.00	739.20	954.90	3683.19

FIGURE 8 Crew cost summary—heavy train.

EQUIPMENT COST SUMMARY:

EQUIP-MENT TYPE	NUMBER OF UNITS	PURCHASE PRICE (EACH)	HOURLY OWNSHP COST	TOTAL HOURS OPER.	TOTAL OWNERSHIP COST	RUNNING COST CTS/MILE	TOTAL MILES OPER.	TOTAL RUNNING COST	TOTAL COST
SD-40	7	1250000.	9.75	20.12	1372.93	116.000	513.	4161.90	5534.82
HOPPER	90		0.75	20.12	1357.87	13.000	513.	5996.82	7354.69
CABOOSE	1		0.38	20.12	7.54	7.000	513.	35.88	43.42
TOTAL	98				2738.34			10194.59	12932.94

FIGURE 9 Equipment cost summary—heavy train.

FUEL COST SUMMARY:

STOP NUMBER	MILE-POST	ARRIVE TIME	DEPART TIME	IDLE TIME	FUEL CONS. AT STOP	CUM. FUEL CONSUMED	CUM. FUEL EXPENSE
1	129.33	3:35	5:35	2: 0	77.28	2689.89	2286.41
2	255.97	9:10	11:10	2: 0	77.28	6276.36	5334.91
3	380.91	14:31	16:31	2: 0	77.28	8869.17	7538.80
******	512.60	20: 7	****FINAL DESTINATION****			11609.93	9868.44

FIGURE 10 Fuel cost summary—heavy train.

TRACK MAINTENANCE COST SUMMARY:

SEGMENT	LENGTH	TOTAL MGTM	CONSIST MGTM	MILES OF TANGENT	TANGENT TRACK MAINT. COST	MILES OF CURVE	CURVED TRACK MAIT. COST	TOTAL TRACK MAINT. COST
HUB A	129.70	3891.00	1.85017	114.38	853.27	15.32	128.41	981.68
WAYSVILLE	126.60	3798.00	1.80595	95.99	718.36	30.61	283.18	1001.54
JCT. TOWN	124.90	3747.00	1.78170	96.38	725.47	28.52	289.04	1014.51
UPTOWN	131.40	3942.00	1.87442	99.29	744.66	32.11	313.82	1058.48
TOTAL	512.60	15378.00	7.31224	406.04	3041.76	106.56	1014.45	4056.21

FIGURE 11 Track maintenance cost summary—heavy train.

$4,056.21 divided by 7.31224 or $554.72. Thus, cost per mgtm rose 11.4 percent. Train gross weight declined from 14,545 tons to 14,265 tons, a decrease of 1.9 percent. These two effects explain the observed increase of 9.3 percent in total track maintenance cost. Although in this case track maintenance costs rose only 9.3 percent, it must be recognized that the entire route was assumed to contain 132 lb/yd hardened, continuous welded rail. With lighter weight rail and possibly poorer ballast, a shift from 100- to 112-ton hoppers could result in a much larger increase in track maintenance cost.

The total cost for the second run is shown in Figure 12. For this run, the total cost is $30,540.57. The cost is 1.4 percent less than the cost for the first run, yet the train was carrying slightly more lading. The extent of actual savings is better measured by comparing costs per NTM. The second run's cost per NTM of 0.591¢ represents a 1.5 percent reduction from the first run's costs. Therefore, for this 513-mile trip the line-haul cost of moving one ton of coal can be reduced from $3.0855 to $3.0537 by using the heavier cars.

Several observations about this analysis are in order. First, the costs presented do not include any burdens of administrative overhead. Further, no switching, terminal, or dispatching costs have been included. Such additional costs must be measured in order to make pricing decisions. However, RECAP II can be used to determine the extent to which any change in technology or operating strategy will affect total avoidable line-haul costs.

To conduct a more complete analysis, several other operating strategies should also be tested. The operating strategy chosen in light of the new technology was, in the case presented, to keep the lading weight of the train constant. This resulted in a smaller train gross weight with fewer cars. This analysis would be reasonable if the size of the train were mostly dependent on the amount of commodity to be shipped. However, because the alternative train has a lower gross weight, it is reasonable to try running the model with one fewer locomotive in the consist to determine (a) whether the train will be able to complete the run and (b) whether the benefits of dropping a locomotive will exceed the cost of adding time to the trip.

If the size of the train is mostly dependent on the size of available locomotive consists, it may be worthwhile to make a run with sufficient cars to make the two trains nearly equal in gross weight. If train size is mostly dependent on the logistics of assembling a certain number of cars in one place, then it may be worthwhile to make a run where the number of cars is held constant. The latter case may, however, result in the need for a larger locomotive consist due to greater train gross weight. In either of these two cases it would be necessary to compare costs on a per NTM basis because total NTM will vary considerably from the base case.

Table 6 gives a summary of the results of many of these additional runs. The base train loaded run and the 90-car heavy train loaded run are the two cases that have been reported. The 92-car case represents the results when the loaded train of heavier cars is sized to match the gross weight of the base train. The 100-car heavy train case represents the situation where the number of cars available for loading represents the train size constraint (i.e., it has the same number of cars on the base train). For each case, the corresponding empty run costs are given in the table.

The second section of Table 6 gives the cost by category for the various cases. As can be observed, all costs decline uniformly for the heavier cars and longer trains, except track maintenance costs, which rise due to the heavier wheel loads.

The third column in the table is the sum of the loaded and empty run costs and represents the total line-haul round trip costs. Of course, these costs cannot be compared directly because a different amount of lading is carried in each case, as indicated in the NTM column. Correcting for NTM, it is easy to see that costs decline as heavier cars are used, and decline still further as more cars are added to the train. Also, the table indicates that by considering the empty return, the cost advantage

COST OF RUNNING SPECIFIED CONSIST:

CATEGORY	TOTAL COST	COST /GTM	COST /NTM	COST /CFM
CREW COST	3683.19	0.00050375	0.00071290	0.00001794
LOCOMOTIVE COSTS:				
OWNERSHIP COST	1372.93	0.00018778	0.00026574	0.00000669
RUNNING COST	4161.90	0.00056923	0.00080556	0.00002027
CAR COSTS:				
OWNERSHIP COST	1365.42	0.00018675	0.00026428	0.00000665
RUNNING COST	6032.70	0.00082510	0.00116766	0.00002939
FUEL COST	9868.22	0.00134968	0.00191004	0.00004807
TRACK COST	4056.22	0.00055477	0.00078510	0.00001976
TOTAL COST	30540.57	0.00417706	0.00591128	0.00014878

FIGURE 12 Total cost summary—heavy train.

TABLE 6 Costs for Several Alternative Trains

	Base Train ($)	Heavy Trains 90 Cars	92 Cars	100 Cars
Total Costs				
Cost of loaded run	30,982	30,541	30,944	32,502
Cost of empty run	22,407	21,653	21,891	22,826
Round trip cost	53,389	52,194	52,835	55,328
Net ton-miles (000s)	5,152	5,167	5,282	5,741
Cost per 1,000 NTM	10.364	10.100	10.003	9.637
Percent savings from base	-	2.5	3.5	7.0
Cost by Category (per 1,000 NTM)				
Fuel cost	2.9150	2.8510	2.8289	2.6671
Crew cost	1.4299	1.4257	1.3947	1.2831
Track cost	.8201	.8775	.8751	.8667
Equipment cost	5.2797	4.9472	4.9044	4.7473

of the 90-car heavy train over the 100-car standard train is increased because there are fewer cars on the return trip.

From the preceding analysis, the following conclusions can be drawn:

1. By using 286,000-lb cars instead of 263,000-lb cars for this coal movement, in trains carrying nearly the same amount of lading, line-haul costs can be reduced by 2.5 percent, from $10.36 per 1,000 NTM to $10.10 per 1,000 NTM.
2. The various categories of round trip cost are affected as follows: (a) crew costs--down 0.3 percent; (b) equipment costs--down 6.3 percent; (c) fuel costs--down 2.2 percent; and (d) track maintenance costs--up 7.0 percent.
3. Significant additional economies are available using longer trains.

CONCLUDING REMARKS

The uses for the RECAP II computer model have been described and a case example showing how the model can be used in route-specific rail cost analysis has been provided. RECAP II has been shown to be a powerful model that can evaluate the changes in avoidable line-haul operating costs that can be expected from the introduction of new technology or from changes in operating strategies. Because it is an engineering-based costing tool, RECAP II, unlike accounting-based cost models, is able to evaluate cost changes that result from new technologies and new operating strategies. The RECAP II model cannot be used, however, for regulatory cost analysis because engineering-based costing models only provide a lower bound for absolute cost. Care should be taken, therefore, that the RECAP II be used only to analyze changes in cost rather than absolute costs.

REFERENCES

1. Uniform Railroad Costing System: 1980 Rail Cost Study. Bureau of Accounts, Interstate Commerce Commission, Dec. 1982.
2. S.M. Howard, L.C. Gill, and P.J. Wong. Review and Assessment of Train Performance Simulation Models. In Transportation Research Record 917, TRB, National Research Council, Washington, D.C., 1983, pp. 1-6.
3. E.R. McIlveen, et al. Rail Wear Cost Model User's Manual. Canadian Institute of Guided Ground Transport Report 81-19, Transport Canada Report TP-3727E, Ottawa, Ontario, April 1982.

Publication of this paper sponsored by Committee on Track Structure System Design.

Techniques for Controlling Rail Corrugation

SO'N T. LAMSON

ABSTRACT

Corrugation on low rails in curved track is a common and significant problem on North American railways. Research findings on probable causes are described and practical methods of keeping this problem under control are suggested. The role of wheel and rail contact stresses in the plastic deformation of rail surface, a necessary condition for corrugation development, is examined, and the use of high-strength steel and rail profile grinding to control rail corrugation through reducing or preventing rail surface plastic deformation is discussed. Results of recent field trials of rail corrugation control using a rail profile grinding technique are presented.

Current interest in heavy freight and high-speed passenger train operations has brought increasing attention to the century-old problem of rail corrugation--a form of periodic rail surface deformation that causes severe vibration in vehicles and track. Vibration in turn generates noise, an environmental problem that has become a sensitive issue in densely populated areas. In terms of vehicle and track maintenance, vibration can cause severe damage to wheels and rails as well as to ties and ballast.

Railway rail corrugation is generally classified into two groups according to wavelength. Long-wave corrugation has wavelengths on the order of one-half to one tie spacing, and a peak-to-peak amplitude from 0.020 to 0.070 in. or more. In North America it is common to heavy freight train operations such as coal lines. At the trough of the corrugation, where the heavily loaded wheels (30,000 lb or more) repeatedly pound the rail, severe plastic flow and fatigue damage are often observed.

Short-wave corrugation has wavelengths in the range of 2 to 4 in., and peak-to-peak amplitude is usually less than 0.005 in. It is more common in high-speed passenger train operations and in light rail transit systems. This problem (called "roaring" rail) has long been of considerable concern in Europe. With the recent stepped up development of light rail transit systems, this problem is receiving increased attention in North America. Short-wave corrugation does not usually cause serious damage to vehicles and track structures, at least in the short term. However, the noise and vibration, especially in underground transit systems, are particularly troublesome because of the increasing sensitivity of the public.

In North America, long-wave corrugation is a common problem on low rails in curved track and requires periodic grinding. The cost of grinding could run into millions of dollars for a major railroad. Without grinding, however, the consequences would be far worse: speed restrictions or increased rail fatigue failure, or both, due to severe wheel and rail vibration. In some heavy-haul railway operations with high axle loads (up to 33 tons) and high traffic density [up to 50 million gross tons (MGT) a year on single track], corrugation could grow from a few thousandths of an inch to between 0.020 and 0.040 in. in 6 months. At this level there is usually severe wheel- and rail-contact-generated track vibration, and speed restrictions are sometimes necessary (actual imposition depends on, among other things, the sensitivity of a particular track formation to vibration). To maintain the required track capacity, these slow orders must be removed as soon as possible. If rail grinding is not available, the rails will have to be replaced. Thus, if slow orders are not acceptable and rail grinding is not available, corrugation could drastically reduce the service life of rails. A cost survey by Roney (1) suggests that the cost of rail corrugation to the Canadian railways is about $30 million (1984 dollars) a year.

The focus of this paper is mainly on the North American long-wave corrugation problem, with identification of practical measures that could help reduce its magnitude.

CORRUGATION THEORY

To find a permanent solution to the problem of rail corrugation, its cause or causes must be identified. To date there have been a number of theories--some based on general observations, others based on in-depth study. Of the latter, there are two categories: contact resonance and stick-slip. The contact resonance theories consider resonance vibration of the wheel and rail system as the primary cause of rail corrugation. Sources of resonance frequency may be found in various possible vibration modes of wheels, axles, rails, and rail and tie systems. The stick-slip theories concentrate mainly on the frictional contact instability between wheels and rails during vehicle movement through curves. These theories, although not yet proven, could be used to explain the corrugation phenomenon in curved track under heavy axle load. These theories are reviewed in the light of the North American rail corrugation problem.

Theories on the stick-slip phenomenon explain that the contact between wheel and rail is not stable but alternates from stick to slip. During the stick cycle rail wear is small, but during the slip cycle significant wear occurs because of relative wheel and rail sliding under very high contact stresses.

King and Kalousek (2) are among a number of researchers who explain the stick-slip theory in the context of wheelset curving behavior. This theory hypothesizes that during curve negotiation, the effective rolling radius is the same on both wheels, even though the wheel on the high rail (outer) side has a longer traveling distance. To accommodate this difference, the wheel on the high rail side must rotate faster than the one on the low rail side. Because the wheelset is an integral component, this instantaneous rotational speed differential places the axle in torque until the torsional reaction becomes greater than the adhesion forces at the wheel and rail interface. The axle then twists back, resulting in wheel and rail slip. The stick-slip cycles set the axle into torsional vibrations which in turn perpetuate the stick-slip cycles, causing corrugation to increase.

The assumption of equal effective rolling radius on high and low rail sides does not hold, because, for a conical wheel tread in a flanging configuration, the wheel on the high side travels on a larger wheel radius. The stick-slip theory is, however, still valid if a torque of significant magnitude exists on the axle.

Various studies into the curving behavior of three-piece freight trucks have shown that large creep forces exist between wheel and rail interfaces, and that for a given wheelset, especially the leading one, the creep force orientation is such that a large torque is applied to the axle (3-5). Figure 1 shows an example of a force diagram for a leading wheelset as predicted by a steady-state, truck-curving computer model (6). The magnitude of torque on the axle is about 22,000 lb/ft in a 3-degree curve. Apart from the existence of a sufficiently large torque, the occurrence of stick-slip further requires that wheel and rail friction have a falling characteristic at high creepage and that the wheel and rail creepage condition be severe enough to give an operating point in the falling portion.

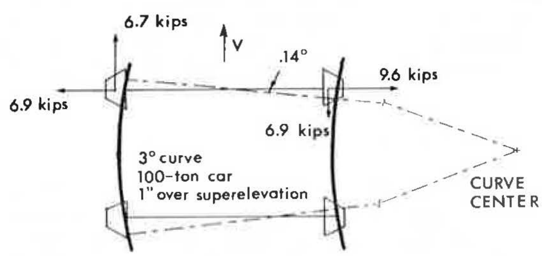


FIGURE 1 An example of force diagram for the leading wheelset in a curve.

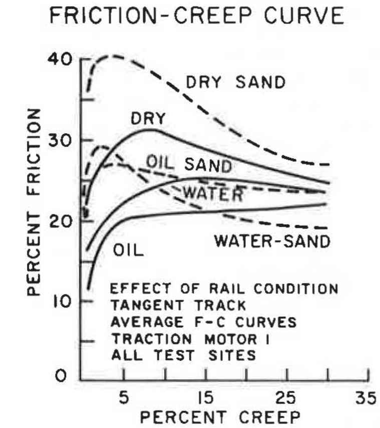


FIGURE 2 Average friction-creep curves.

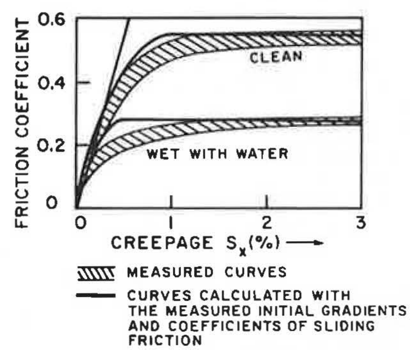


FIGURE 3 Comparison of the longitudinal traction/creep relationship of clean rollers and rollers wet with water.

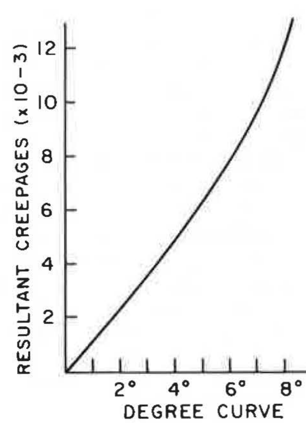


FIGURE 4 Resultant creepage versus degree curve at balance speed.

Figure 2 shows some sample friction-creepage curves for various wheel and rail contact conditions. It is evident that a falling characteristic due to creep saturation occurs at a few percent creepage for all cases except those with water and oil contamination. However, most experimental results for the lower range of creepage, such as those in Figure 3, show a somewhat different characteristic. These indicate that creep saturation begins at about 1 percent creepage but show no falling characteristic up to 3 percent creepage.

Figure 4 shows the resultant creepage on low rail in a range of curves. These data are derived from computer simulation of the curving behavior of a three-piece freight truck with 35-ton axle load. Generally creepage increases with the degree of curvature and is about 1 percent in 8-degree curves.

These data (from Figures 3 and 4) indicate that stick-slip as a result of friction and creepage falling characteristics is not likely. It may be suspected, however, that in the field other conditions, such as wheel and rail contamination and vibration, may exist but are insufficiently accounted for in the laboratory. These may create the necessary falling characteristics for stick-slip to occur. In a recent paper, Clark (7) reported experimental work to verify the stick-slip theory. The results show that for a creepage condition greater than 1 percent, considerable vibration occurred, and in some cases a corrugation-like pattern appeared on the rail surface.

PLASTIC DEFORMATION

Stick-slip vibration may create periodic variation in the tangential force component at wheel and rail contact. However, for this force to generate periodic plastic deformation as observed in typical North American corrugated rails, the force magnitude must exceed the elastic limit of the rail steel. Thus, it is necessary to know the magnitude and nature of the wheel-loading environment at the wheel and rail interface, as well as the resistance of the rail steel to plastic deformation under actual track operations. A sample calculation that illustrates the magnitude of the wheel-loading environment relative to the strength of the rail steel follows. For a 100-ton car with 36-in. diameter wheels, running at slightly under balance speed (1 in. over-superelevation) in a 3-degree curve, the wheel loading on the low rail is:

1. Vertical:

Static wheel load	32,000 lb
Vertical load transfer	3,200 lb
Steady-state wheel load	35,200 lb
Dynamic load factor (assumed)	10%
Dynamic wheel load	38,720 lb
Compressive contact stress:	
• average wheel and new rail (tread is slightly hollow with a concave radius of 30 in.)	195,000 psi
• new wheel and new rail (tread is 1 in 20)	227,100 psi
• worn wheel and new rail (tread is hollow with false flange radius of 4 in.)	430,400 psi

2. Tangential (estimation based on steady-state curving simulation of three-piece truck):

Lateral creep force	8,000 lb
Longitudinal creep force	7,000 lb
Resultant creep force	10,600 lb

3. Ratio of tangential to vertical force 0.275

Both standard carbon and low-chrome alloy rails are used in North America. Carbon rails are used mainly in tangent track and shallow curves (e.g., less than 3 degrees). Low chrome rails are used in sharper curves as a measure against the more severe wheel-load environment. Yield strengths of standard carbon and chrome rails are about 75,000 psi and 100,000 psi, respectively. Even after taking into account a constraint fractor of 1.7, these yield strengths are much lower than the estimated range of contact stresses (195,000 to 430,000 psi). Therefore, plastic deformation of rail would occur under the first wheel passage. For subsequent wheel passages, the residual stress created under earlier wheels improves the resistance of the rail steel to further plastic deformation. This phenomenon, called shakedown, has been the subject of intensive research in the field of contact mechanics. Johnson (8), in a recent comprehensive review of this subject, proposed an estimate for the elastic and shakedown limits applicable to rail steel in rolling contact as shown in Figure 5. According to this estimation, the shakedown limit for standard carbon and chrome rails, in the absence of tangential force, is 202,000 psi and 270,000 psi, respectively. However, if a tangential force of 0.275 times the vertical is also present, these shakedown limits are reduced to about 150,000 psi and 200,000 psi for standard carbon and chrome rails, respectively.

These estimates indicate that:

1. Standard carbon rails would be plastically deformed under the passage of most wheels, and

2. Chrome rails would be plastically deformed under the passage of new and worn (i.e., with false flange) wheels. For average wheels, the likelihood of plastic deformation is less than certain, because the shakedown limit under 0.275 creepage is only slightly below the contact stress.

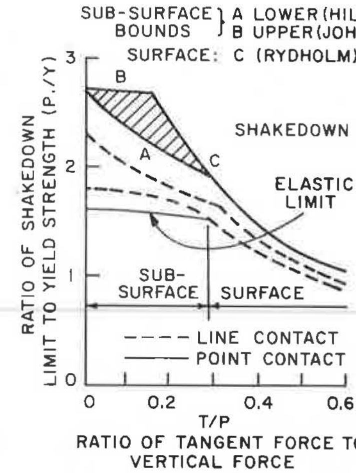


FIGURE 5 Elastic and shakedown limits in rolling contact.

For a track carrying 50 mgt of 32-ton axle loads per year, and assuming a wheel condition distribution of 15 percent new, 75 percent average, and 15 percent worn, the plastic deformation cycle for standard carbon rail and chrome rail would be about 1,500,000 and 500,000 times a year, respectively.

Thus, curved track is generally too hostile an environment for the standard carbon and low allow rail steels currently in common use. These rails would likely corrugate in the presence of vibration.

PRACTICAL SOLUTION

The preceding discussion indicates that low rails in curves are subject to such a severe loading environment that unless hardened rails are used, there is potential for widespread corrugation. High wheel loads (due to heavy cars) and creepage (due to poor curving of three-piece trucks) place the present rail steel on the threshold of plastic deformation. With wheel and rail contact being steel on steel (low damping), and with limited adhesion (stick-slip), there is potential for vibration. Uniformly spaced tie support could add to the vibration problem as well. Knowing that these two factors, that is, plastic deformation and contact vibration, are essential in the development of corrugation, the solution obviously is to eliminate or reduce their severity. To achieve this goal when there is no option of replacing existing trucks with ones of steerable design, or replacing existing track with a continuously supported structure, or reducing axle load, the author believes the following remedial actions could be taken:

1. Profile grinding. At present, rail is ground periodically to remove corrugation. This grinding operation can be adapted to produce an optimal rail profile as well, without additional cost. The objective is to create a rail profile that conforms to existing wheel profiles, given the existing range of track gauge error. On the low rail, the primary aim is to ensure that wheel and rail contact takes place away from the false flange should the wheel be worn and the track gauge widen. If this is achieved, a potential contact stress of 430,000 psi under worn wheels can be reduced to 195,000 psi, making the low

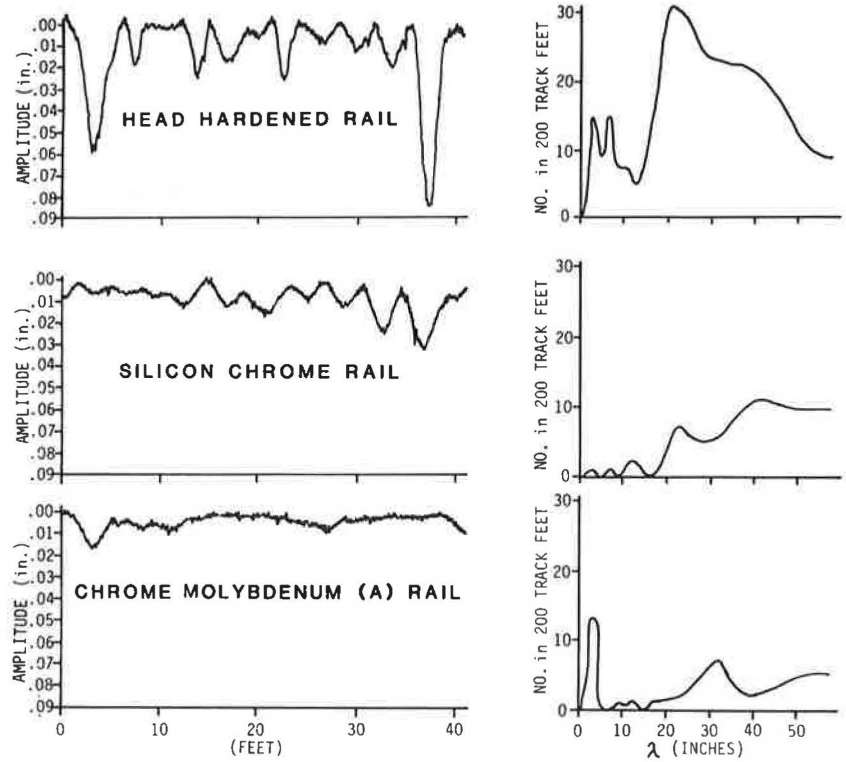


FIGURE 6 Typical initial rail profiles.

rail less susceptible to plastic deformation. This technique will be described in the case study.

2. Grinding of new rail. The surface of new rail normally is not smooth. Apart from mill scales, there are irregularities with wavelengths from a few inches to several feet. An example is shown in Figure 6. This roughness can excite wheel and rail vibration at a time when the rail steel has not yet hardened to its potential. Early removal of surface roughness would reduce the initial plastic deformation and produce a smoother hardened rail surface at a later stage.

3. Superelevation. Track should be elevated at balance or at a slightly deficient level to avoid wheel load transfer to low rails. In a mixed-traffic situation, superelevation should be designed to accommodate the heavier traffic categories.

4. Heat-treated rail. Heat-treated rails such as the head-hardened type could be used to improve rail resistance to plastic deformation. With a yield strength (2 percent proof stress) of about 125,000 psi, they would be able to resist plastic deformation under most wheel loads except the false flange type. When used with a profile grinding program, which is effective in avoiding false flange contact, heat-treated rails would provide satisfactory resistance against plastic deformation, and hence rail corrugation.

5. Rail lubrication. The standard practice of lubricating the gauge face of the high rail effectively transfers some of the creepage from low rail to high rail. Figure 7 is a three-piece truck curving simulation result showing the reduction of longitudinal creepage and creep force on the low rail as the friction factor on the high rail gauge face decreases. This has the dual effects of reducing the likelihood of wheel slip and increasing the shakedown limit of the low rail. Daniels and Blume (9) reported a correlation study at FAST in which rail corrugation growth was much lower during the periods when rail lubrication was used than during the

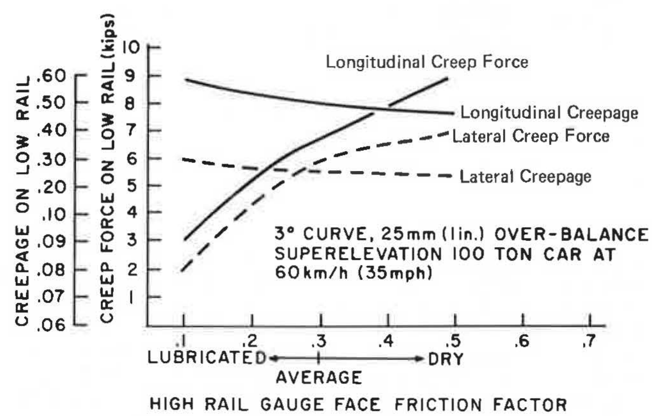


FIGURE 7 Effect of rail lubrication on creepages and creep forces.

periods without lubrication (Figure 8). Because trackside lubricators are already being used on most railways, a field trial to validate this effect could be carried out quite readily.

There are also other measures that may be less readily achievable but are nonetheless feasible:

1. Wheel remachining. More frequent wheel inspection and remachining would reduce the percentage of defective wheels such as those with false flange or flat spots. If this action is coordinated with a program of profile grinding and the use of heat-treated rail, the problem of rail corrugation would be greatly reduced. For several years the Hamersley Iron and Mount Newman Mining Railways in Australia (33-ton axle loads) have been experiencing minimum rail corrugation problems using the preceding strategy.

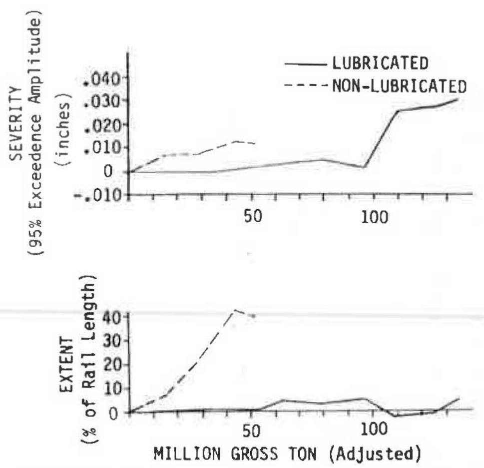


FIGURE 8 Rail corrugation growth by lubrication condition; standard carbon rail, 5 degree curve, hardwood ties.

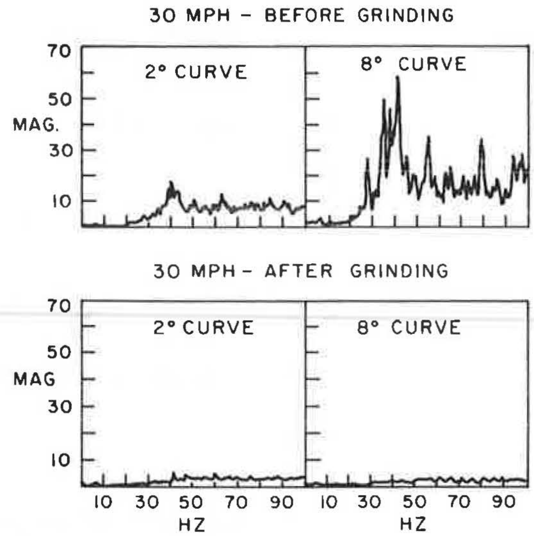


FIGURE 9 Measurements of rail vibration.

2. Track gauge control. The biggest problem with timber track, as far as corrugation is concerned, is gauge widening, which causes false flange contact on low rails. The use of concrete ties solves this problem but introduces high dynamic load because of larger tie mass and stiff rail pad. For timber track, a practical solution could be to insert one steel tie, for example, every five tie spacings. The spacing, however, should not be uniform (i.e., not always one every five ties) as this may introduce a periodic stiffness variation. Gauge rod is another, perhaps less practical, option.

CASE STUDY: PROFILE GRINDING TEST ON CP RAIL - 1982 (10)

Outline

In May and June 1982, a rail profile grinding test was conducted on a section of curved track on the Canadian Pacific (CP) main line through the Rocky Mountains. The first objective was to reduce the problem of corrugation on low rails in curves. For this, two curves were selected: an 8-degree curve with severe corrugation (average depth 0.0595 in.) (1.44 mm) on the low rail (chrome), and a 2-degree curve with typical corrugation (average depth 0.0335 in.) (0.84 mm) on the low rail (standard carbon). Measurements of rail vibration under a 30-mph test train (three locomotives, five 100-ton cars) are shown in Figure 9. These indicate the severity of the problem, especially in the 8-degree curve.

Investigation

Track gauges and wheel profiles were surveyed in order to design a suitable rail profile to redress the problem of corrugation on low rails in curves. This survey revealed that in curves where rail corrugation occurs, the static track gauge is typically 0.25 in. wider than usual. Under traffic, there will be up to 0.25 in. of additional gauge widening due to lateral wheel forces (typical magnitude of 4 to 12 kips outward on both high and low rails). The survey of wheel profiles showed that there was a large percentage of wheels with about 0.25 in. flange wear, and a small but significant percentage of wheels having false flange (i.e., reversed tread radius on the field side). The combined effect of gauge widening and wheel wear creates an effective total gauge widening under wheel passage of about 0.50 to 0.75 in. (0.25 in. flange wear + 0.25 in. gauge face wear + up to 0.25 in. dynamic gauge widening). This led to a kind of false flange contact problem, as shown in Figure 10.

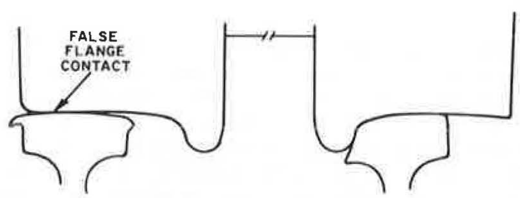


FIGURE 10 Wheel and rail contact geometry in curves with wide gauge.

Design of the Ground Rail Profile

The preceding investigation led to the conclusion that false flange contact is a primary contributing factor to the problem of corrugation on low rails. To avoid false flange contact (without resorting to the remachining of worn wheels and the prevention of track gauge widening), the field side of the low rail must be ground down so that wheel and rail contact takes place only on the gauge side. Figure 11 shows diagrammatically what must be done. In reality, grinding of the field side had to be reduced because of the high cost of grinding and short track time. A cost-effective grinding program is one that

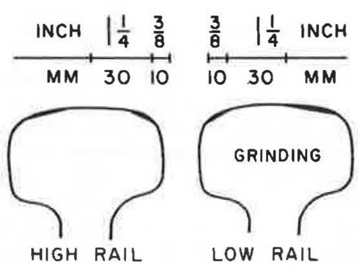


FIGURE 11 Proposed ground rail profile for curves on CP Rail.

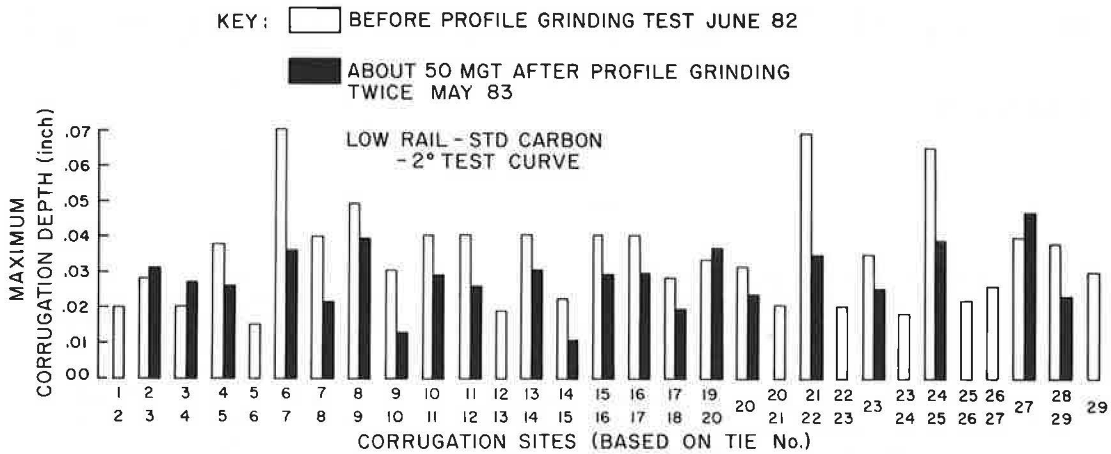


FIGURE 12 Effect of profile grinding on the formation of rail corrugation.

removes only sufficient rail metal to avoid wheel and rail contact in the false flange area between grinding sessions. At the time of the next session, this grinding can be repeated, hence excessive grinding is not necessary each time.

Test Results

Corrugation regrowth was measured periodically after the test grinding. Figure 12 shows the corrugation levels in the 2-degree test curve 1 year after the test grinding. The rail had been subjected to a total of two profile grinding sessions. The open bars represent corrugation levels existing about 6 months (about 25 mgt) after conventional (symmetrical) grinding. The full bars represent corrugation levels 6 months after profile grinding. Except at 4 locations where minor increases occurred, the remaining 26 locations showed a significant reduction in rail corrugation levels.

Further Work

Following the previously described test, CP Rail adopted the designed profile rail as a provisionary standard for the main line. A more comprehensive field test involving 12 curves is being carried out. The objective of this test is to further examine the performance of the designed rail profile and to explore its refinement.

ACKNOWLEDGMENTS

This paper is based on a number of research projects that were funded either by the BHP Melbourne Research Laboratories; Hamersley Iron Company; Mount Newman Mining Company joint research program; or by the Canadian Institute of Guided Ground Transport, Transportation Development Centre, Canadian Pacific Railway, Railway Advisory Committee; or Speno Rail Services joint research program. The author is grateful to these organizations for their support. The author also is very appreciative of the contribution of Sherrill Arnold in the preparation and editing of this paper.

REFERENCES

1. M.D. Roney. Economic Aspect of the Wear of Rail on Canadian Railways. Presented at International Symposium on Contact Mechanics and Wear of Rail/Wheel Systems, Vancouver, B.C., July 1982.
2. F.E. King and J. Kalousek. Rail Wear and Corrugation Studies. AREA Bulletin 656, Vol. 77, 1976, p. 429.
3. D. Boocock. Steady State Motion of Railway Vehicles on Curved Track. Journal of Mechanical Engineering Science, Vol. 2, No. 6, 1969.
4. D.E. Newland. Steering a Flexible Railway Truck on Curved Track. Trans. ASME, Journal of Engineering for Industry, 1969.
5. P.P. Marcotte. K.J.R. Mathewson, and R. Young. A Practical Model of Rail Vehicle Curve Negotiation. Presented at 7th Symposium of International Association of Vehicle System Dynamics, Cambridge, England, Sept. 1981.
6. S. Marich and S.T. Lamson. A Strategy and Procedure for the Development and Application of a Bogie Curving Model. Presented at Railway Engineering Conference, Sydney, Australia, Sept. 1981.
7. R.A.P. Clark. Rail Corrugation--Recent Theories. Track Technology for the Next Decade. University of Nottingham, Nottingham, U.K., July 1984.
8. K.L. Johnson. Inelastic Contact: Plastic Flow and Shakedown. Presented at International Symposium on Contact Mechanics and Wear of Rail/Wheel Systems. Vancouver, B.C., July 1982.
9. L.E. Daniels and N. Blume. Rail Corrugation Growth Performance. Presented at Second Heavy Haul Railway Conference, Colorado Springs, Colo., Sept. 1982.
10. S.T. Lamson. Rail Profile Grinding. CIGGT Report 82-7. Canadian Institute of Guided Ground Transport, Queen's University, Kingston, Ontario, Canada, 1982.

Publication of this paper sponsored by Committee on Track Structure System Design.

The Evolution of Washington Metro's Track Standards

ARTHUR J. KEFFLER

ABSTRACT

In 1969 the opportunity to design the construction of a completely new transit track system using the latest technology occurred with the decision to construct the Washington Metropolitan Area Transit Authority Metro system. The system offered the track designer a wide spectrum of challenges. The 101 route miles of double track system included underground structures of several types, at-grade track, and several extensive aerial structures. In the 15 years since the first phase of Metro track was designed, many of the design features have evolved from their original concept to significantly different concepts as a result of construction problems, maintenance problems, and new developments. Presented here are brief descriptions of these evolutions with respect to direct fixation track including grout pads, anchorage systems and the fasteners themselves, rail, rail welding, ballasted track tight radius curves, and special trackwork designs including inserts and methods of support.

Washington, D.C., is a city of wide boulevards and numerous monuments. In spite of the broad streets and many open spaces, by the late 1950s the traffic to government offices and commercial centers in the area had become intolerable. The rush hours produced long lines of traffic, and midday traffic created unacceptable levels of congestion. Clearly the time had come for a complete analysis of the city's future transportation needs. It would take more than highways and buses to accommodate the traffic anticipated by the year 2000 by which time the metropolitan area population was expected to grow from 2 million to 5 million.

SYSTEM BACKGROUND

The transportation analysis concluded that a heavy rail transit system with an extensive feeder bus system was needed. The selected alternative comprised nine radial alignments emanating from the central business district into suburbs near the Capital Beltway (Interstate 95 and 495), which circles the city at a radius of about 10 miles. (See Figure 1.). Among its many tasks, De Leuw, Cather and Company was given responsibility for the design of the trackwork system including final design and provision of engineering service during construction.

The heavy rail system will eventually comprise 101 route miles of dual track with 86 passenger stations. To serve the 5 routes and 600 revenue cars operating on them requires 1 major repair yard, 4 service and inspection yards, and 3 storage yards. The breakdown of track mileage between types and stages of construction is given in Table 1.

INITIAL TRACK STANDARDS

A chronicle of the evolution of trackwork standards from their inception to the standards currently in effect must begin with a brief review of the original standards and how they were developed. In 1967 an in-depth investigation, which included detailed questionnaires, was conducted of the eight heavy rail transit systems operating in North America. The findings were published in 1968 in a report titled "Trackwork Practices of North American Rapid Transit Systems (1)." The study provided the Metro designers an extensive enumeration of the materials, concepts, and methods of transit track installation then in current use.

Working from background information obtained from the survey of transit systems, De Leuw, Cather engineers developed a second report, "Recommended Trackwork Standards" (2). Every component of the track system was analyzed and alternatives compared. In some cases, where analytical methods were unavailable or inadequate, the recommendations were based on the experience of the operating transit systems. The following paragraphs contain the principal recommendations made in the report, which was published in 1969.

Direct Fixation Rail Fasteners

The development of direct fixation rail fasteners, particularly for transit application, was (and still

TABLE 1 Metro System Trackwork Statistics

Category	In Service (as of 12/84)	Under Construction	Future	Total
Main line track (miles)				
Underground DF[a]	68.68	0.66	23.66	93.00
Aerial DF	11.91	0.46	1.00	13.37
Total DF	80.59	1.12	24.66	106.37
Ballasted track	41.73	17.12	35.36	94.21
Total all main line	122.32	18.24	60.02	200.58
Yard track (ballasted)	19.01	5.73	10.56	35.30
Passenger stations				
Underground	38	0	12	50
Aerial	3	0	0	3
At grade	19	4	10	33
Total	60	4	22	86
Special trackwork units				
Main line DF	96	4	22	122
Main line ballasted	67	20	36	123
Subtotal	163	24	58	245
Yard, ballasted	148	37	84	269
Total	311	61	142	514

[a]DF = Direct fixation.

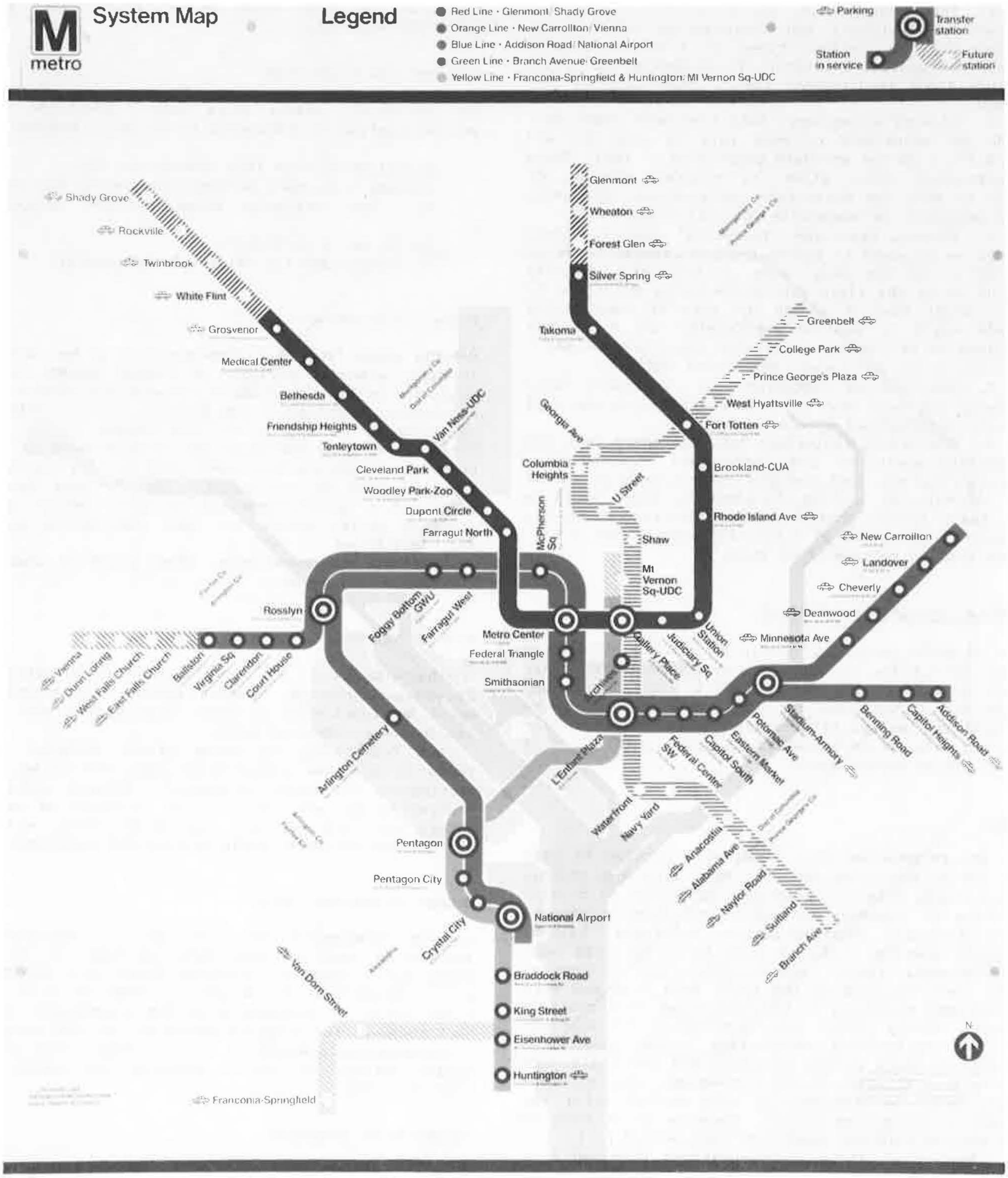


FIGURE 1 Metro system map.

is) in a state of evolution. Several fasteners were available which, with modifications, could meet the criteria developed for Metro and provide a satisfactory solution to Metro installation and maintenance plans.

It was recommended that the criteria for rail fasteners be fully developed during the final design stage and incorporated in a detailed performance specification. The criteria developed included the following:

1. Service life. Rail fasteners were to be designed for a minimum service life of 50 years, which was equivalent to the calculated life of tangent rails with Metro loadings, assuming favorable operating conditions. It was recommended that laboratory testing of rail fasteners to predict service life cover a minimum of 3 million load cycles. Up to 10 million cycles could be required to establish a definite trend or pattern of performance characteristics.

2. Interchangeability and simplicity. Standardization of fasteners and minimization of fastener components for various types of track structures would be considered during final design to limit future stock requirements and to facilitate maintenance.

3. Lateral adjustment. Rail fasteners would provide for adjustment of each rail of +3/8 in. and -5/8 in. from the standard gauge line of rail. These requirements would allow for accurate gauging of rail to meet the prescribed construction tolerances and regauging to compensate for rail wear.

4. Lateral stability. Individual rail fasteners would be designed to resist service lateral loads of 3,900 lb on the gauge side of the rail head and 2,700 lb on the field side while being subjected to a vertical load of 13,500 lb. Each of the lateral loads would be applied separately, and a maximum deflection of the rail head at a point 5/8 in. below the top of the rail would not exceed 1/8 in.

5. Longitudinal restraint. Rail fasteners would provide not less than 3,600-lb restraint to the rail in the longitudinal direction.

6. Electrical isolation. Rail fasteners in a dry condition would not pass more than 1 microampere between the rail and fastener bolts with a potential of 100 volts DC applied. In addition, there would be at least 10,000 ohms impedance to frequencies between 20 cycles and 10 kilocycles tested with a maximum applied voltage of 50 volts AC.

Roadbed Sections and Ballast

For at-grade sections, 12 in. of crushed stone ballast over 8 in. of subbalast was recommended. Ballast shoulders were to be 10-in. wide for timber ties and 12-in. wide for concrete ties. In underground and aerial structures, direct fixation fasteners were to be installed on grout pads and held in place by anchor bolts.

Rail

It was recommended that 115RE rail section be used on the Metro system for both main line and storage yard track. This recommendation was based on consideration of numerous variables including structural and electrical characteristics, stiffness, height, support spacing, life of rail based on head wear predictions, future availability, and subsequent cost over the life of the rail. Main line and yard track were generally to be constructed with control-cooled carbon steel rail conforming to American Railway Engineering Association (AREA) specifications. On sharp curves both high and low rails were to be heat treated. It was recommended that all running rails be continuously shop welded using the electric flash-butt method. Thermite welds were to be used to join the lengths of shop-welded rail.

Adoption of these recommendations provided continuously welded rail throughout the system except in the following situations where bolted joints were recommended to join

1. Switch rails with closure rails,
2. Rails at insulated joints,
3. Rail ends with crossing and turnout frogs, and
4. Rails with stock rails of turnouts.

Special Trackwork

It was recommended that all special trackwork be heat treated and conform to the latest specifications of the AREA folio of Trackwork Plans and Specifications (3).

Turnouts and Crossovers

The following turnout sizes were recommended for standardization of components in the Metro system:

- Junctions of main line routes: No. 15.
- Permanent turnback service crossovers: No. 10.
- Main line emergency crossovers and turnouts: No. 8.
- Yards: No. 6 (minimum).
- Center storage tracks: No. 4 equilateral.

Frogs and Crossings

Movable point frogs and crossings were to be avoided in Metro whenever possible. A special design using short curves between frogs for system crossovers was prepared that minimized the distance between points-of-switches. By making all the curves within the crossing the same radius as the closure curve in the turnouts, speed was not restricted any further than that required by the turnout itself. This design permitted a more economical crossing design with crossing angles within the AREA permissible range for rigid frogs.

Railbound manganese steel frogs would be used in all special trackwork.

Special Trackwork Fastening

In the underground and nonballasted aerial portions of Metro, switches, turnout frogs, and crossings would be fabricated on steel base plates and installed on elastomeric pads.

The feasibility of using plated switches and frogs in at-grade construction also was to be investigated. The plates in special trackwork would be secured to the wood switch ties by means of screw spikes or lock spikes. All other rods, clips, braces, and so forth, would be standard components.

Method of Joining Rails

It was recommended that joint bars of appropriate section be used to join the rail ends of switch rails and turnout and crossing frogs with abutting rails. Joint bars would also be used to join the stock rails of turnouts with the continuous rails just ahead of the point of switch and at heel blocks.

Closure rails would be cut to length from shop-welded strings of CWR to minimize the number of bolted joints.

EVOLUTION OF STANDARDS

From the standards described in the previous sections, many changes have taken place over the years. Each final design phase of trackwork has resulted in a reevaluation of the previous designs. This reevaluation occurs automatically and is directed primarily at specific maintenance and construction issues that surfaced during previous phases. Each new design is also influenced by advances in technology. There have been 10 trackwork final design phases. These designs are referred to as TW-1 through TW-10. The track designed under each contract is currently in the following status: TW-1 through TW-6, TW-8 and TW-9 are in service. TW-7 is under construction. TW-10 is under design.

Some of the revisions to the standards have been relatively simple matters of changing dimensions, or materials; some have been more of a procedural nature wherein the method of procurement, quality control testing, or construction method specified has been changed, and finally, a few have been major reworkings of original concepts. In this latter category are two items that will be discussed in detail: direct fixation fasteners and fastener support system.

A number of less extensive changes, but no less important in terms of construction or future maintenance costs to the system, include:

- Rail procurement specifications;
- Joining continuous welded rail (CWR) strings in the field;
- Ties;
- Special trackwork sizes; and
- Bonded inserts, joints and gauge plates in special trackwork.

Each of these will be briefly discussed in this review of Metro track standards.

Direct Fixation Fasteners

A direct fixation fastener features an elastomer pad, steel plate, or plates, and various anchoring and insulating components used to attach the rail directly to the tunnel invert or aerial structure deck. The projected Metro system requires approximately 490,000 direct fixation fasteners. To date, 340,000 direct fixation fasteners have been placed under contract; 333,000 are in service, and the remaining 7,000 are in various stages of installation. About 150,000 fasteners remain to be purchased.

Procurement History

To ensure the highest possible ride quality, noise and vibration control, electrical insulation, rail-creep prevention, gauge-holding capability, ease of maintenance, and longevity, a comprehensive series of stringent acceptance and quality control tests was developed. They governed the procurement of direct fixation fasteners from TW-1 through TW-8. Laboratory repetitive loading tests were required to provide an indication of future service life. The most severe test was the 3 million cycle combined lateral and vertical repeated load test. The decision to limit the test to 3 million cycles was based on the reasonable assumption and extensive American Association of Railroads (AAR) research laboratory experience that a fatigue failure had a high probability of occurring during that exposure.

The nonproprietary performance specifications prepared for Metro did not include detailed designs but consisted of laboratory performance tests with acceptance criteria and a minimum of dimensional constraints to ensure interchangeability as well as economy and ease of maintenance. They also stipulated the sampling and minimum quality control tests required. This approach has the merit of allowing suppliers maximum freedom to develop solutions within the specified parameters and thus fulfill Urban Mass Transit Administration's objective of procurement from multiple, competitive sources.

Results have been mixed. Three bids were received for two of the seven procurements; two contracts generated two bids each; two were supplied directly by the track installation contractor and one contract was supplied through a sole bidder. Four different designs of fastener went through the series of acceptance tests at significant cost and with delays incurred in the process. Several variances from the specifications were granted to the manufacturers to keep the construction on schedule.

Experience

Metro has approximately 8 years of operating experience with its 27,000 TW-1 direct fixation fasteners. Figure 2 shows their configuration. No significant problems requiring replacements have been encountered with the TW-1 fastener although some have been replaced due to excessive corrosion in portions of the subway having water intrusion problems.

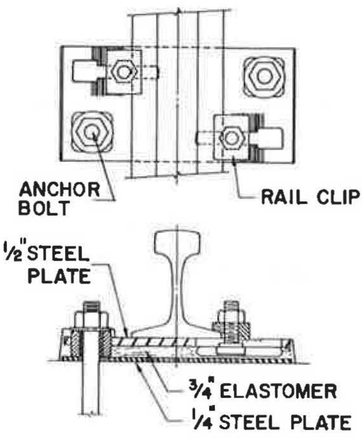


FIGURE 2 TW-1 fastener.

A total of 178,000 fasteners, shown in Figure 3, have been installed in TW-2, 3, and 4. After approximately 6 years of operation, many of the welds on the studs that join the top and bottom plate and provide the resistance to lateral deflection have failed as a result of fatigue.

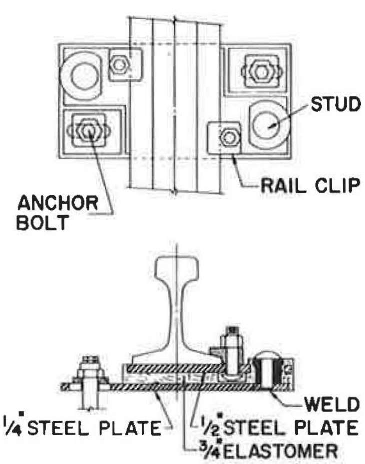


FIGURE 3 TW-2,3,4 fastener.

In TW-5 and 6, 104,000 fasteners similar to those shown in Figure 2 were installed with about 58,000 of these in service for about 3 years. An additional 7,000 fasteners were bought in the TW-5 and 6 procurement for installation in TW-7. Another 50,000 of these fasteners have been installed by maintenance forces as replacements for the fatigue-failed TW-2, 3, and 4 fasteners. With up to 4 years of service on some of these fasteners no major service problems have been identified.

The last fastener procurement was for TW-8 and included 21,000 units. The configuration of this fastener is shown in Figure 4. The fasteners have been in service for nearly 2 years and have shown no defects to date. Because of the cutaway top plate around the anchor bolts, lateral loads are not evenly distributed to both anchor bolts. Because of this, TW-8 fasteners were installed only on tangents and curves of greater than a 2,000-ft radius.

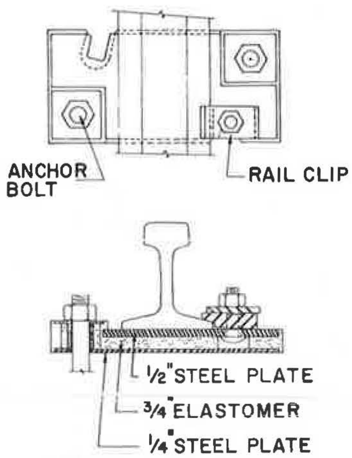


FIGURE 4 TW-8 fastener.

New Specification Development

The experience with fasteners through TW-8, especially the fatigue-failed TW-2, 3, and 4 fasteners, led to a research program to develop a better understanding of the loading environment and a major revision to the procurement specification to improve the performance and maintainability of future fasteners.

One of the research programs was conducted by the Transportation Systems Center (TSC), Cambridge, Massachusetts. The objective of this program was, in part, to obtain realistic estimates of the fastener service loads. A load cell was designed that could be inserted between the direct fixation track and the concrete invert. Once installed, the fastener forces (vertical, lateral, and roll moment) could be measured in track under service loads. Loads were measured on a 755-ft radius curve with standard WMATA fasteners and with fasteners modified to make a major reduction in lateral stiffness and only a minor reduction in roll moment stiffness. The standard fastener had a lateral rail head displacement of 0.09 in., whereas the soft fastener displaced slightly less than 0.25 in.

Vertical loads on both fasteners were measured to be about one-half the load specified for repeated load testing. As expected, the laterally softer fastener allowed better distribution of the wheel load to adjacent fasteners; the softer fastener carried less than one-half the load that the stiff fastener carried. Unexpectedly, the laterally soft fastener also experienced a 15 percent reduction in the wheel-to-rail load to be distributed to the fasteners.

Field tests of a specially designed vibration attenuating fastener concluded that lowering the vertical spring rate of the fastener to about 70,000 lb/in. produced significant attenuation of ground-borne vibration that would permit elimination of expensive floating slabs in many locations.

The new TW-10 specification reflects the findings of the preceding studies. The new fastener will be more in tune with its environment and more compatible with the WMATA vehicle. The changes to the specification are in three areas: physical characteristics, mechanical properties, and test load environment. There are several changes to the physical characteristics of the new fastener. The configuration limitations have been relaxed from previous specifications, but are still sufficient to permit interchangeability. Allowing the overall size and thickness to vary within the limits shown in Figure 5 gives suppliers more freedom to develop their designs.

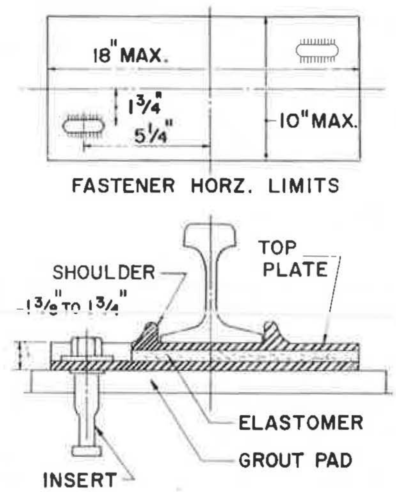


FIGURE 5 TW-10 fastener envelope.

The rail will be held to the fastener by a resilient one-piece threadless spring clip. Although previous specifications did not prohibit spring clips, the lateral load test forced the supplier to use a rail clip that could resist a large toe load, making the spring clip infeasible as a result. The field measurement data demonstrated that the qualification test configuration generated too large a roll moment compared to the in-service measured roll moment. With the test loads and configuration properly coordinated to the in-service fastener load environment, the roll moment is reduced to a level that requires only minimal toe load. The rail spring clip, which will provide the minimum toe load, is more tolerant of the variables at the rail-fastener interface. The use of spring clips in place of threaded parts facilitates installation and maintenance. It also eliminates the corrosion problems associated with threaded parts.

The lower longitudinal restraint provided by the spring clip compared to the rigid bolted clip makes it possible to use one type of spring clip fastener throughout an aerial structure. With the rigid clip, which has high longitudinal restraint, it was necessary to use a second type of fastener with very low longitudinal restraint to keep the CWR-aerial structure interaction forces within practical limits. The spring clip fastener will reduce procurement and inventory requirements from two types of fasteners interspersed throughout the length of the structure to one design used throughout.

The fastener's rail seat will have nonadjustable shoulders that will provide positive positioning of the rail. This will facilitate rail installation and renewal. Lateral adjustment of the fastener will be accomplished with the fastener anchor bolts. Future anchor bolts will screw into female inserts installed in the concrete track bed. The inserts will have adequate bearing area on the bottom of the fastener so that the fastener will be securely clamped by the anchor bolts.

Although no major change to the electrical insulating characteristics has been made over the years, the new fastener will have a surface leakage distance of 3/4-in. instead of the previous 1/2-in. in response the the problem of stray current leakage in wet areas of tunnels.

The mechanical properties of the new fastener have been substantially changed. The new fastener is more resilient vertically and laterally and, as previously discussed, has less longitudinal restraint than previous Metro fasteners.

The new fastener will have an average dynamic vertical spring rate of 70,000 lb/in. compared to 150,000 lb/in. or more for previous designs. This change results in better distribution of wheel loads to adjacent fasteners and greater attenuation of vibrations.

The new specification requires the fastener to be much softer laterally than the old specification permitted. Figure 6 compares the lateral deflection acceptance ranges. The required deflection assures distribution of the lateral wheel-to-rail forces to adjacent fasteners and a reduction in the wheel-to-rail loads.

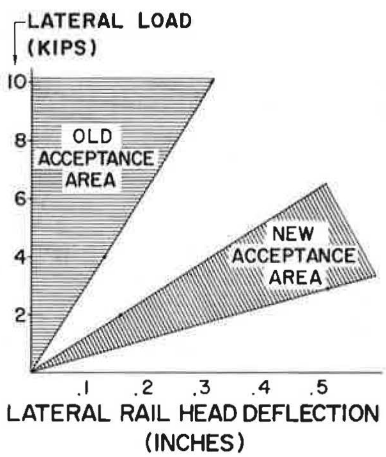


FIGURE 6 Static load acceptance criteria.

The qualification tests have been changed to make them compatible with the field conditions. The magnitude of loads, the manner in which the loads are applied, and the number of cycles of loading have been revised from previous specifications. The most significant change is in the repeated load tests. Formerly, four separate sequences of tests were performed on separate fasteners. The new specification requires one set of four fasteners to undergo a single sequence of tests. A successful fastener will have to survive more than 10 million cycles of load application.

Because the load environment is probabilistic, three load cases in a ratio of 98:1:1 will be applied for the 9 million cycles of repeated lateral and vertical loads. Figure 7 shows the load cases. The 98 percent load case applies relatively low forces whereas the 1 percent cases each apply a combination of high vertical and lateral loads. To duplicate the field conditions requires specifying the loading rate as well as the load amplitudes. Figure 8 shows the load rates in seconds for the repeated load test. To account for torsional resistance of rail and reduced roll moment, the lateral loads are applied at a point 3 in. above the base of the rail instead of at the gauge point on the head of the rail.

	CASES A	CASES B	CASES C		LOAD POINT ON CURVE EXH. VII
VERTICAL LOADS	-261	-324	-213	LBS	A
VERTICAL LOADS	+6	+7.5	+5	KIPS	B
VERTICAL LOADS	+3.9	+4.9	+3.4	KIPS	C
LATERAL LOADS	-104	-212	-212	LBS	D
LATERAL LOADS	+2.4	+5	+5	KIPS	E
LATERAL LOADS	+.9	+1.9	+1.9	KIPS	F
	98*	1*	1*		

*NO. OF CYCLES OF EACH CASE PER 100 CYCLES OF REPEATED LOAD TEST.

FIGURE 7 Load cases.

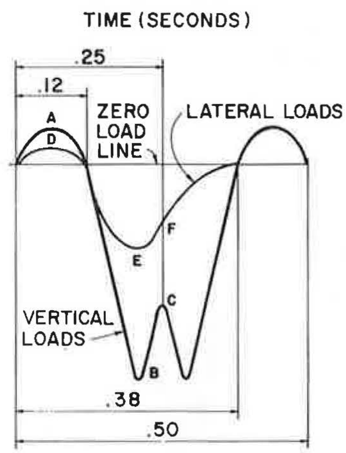


FIGURE 8 Load curves.

Fastener Support System

The second major change or evolution from the originally recommended standards involved the method of supporting direct fixation fasteners.

History

Over the several phases of the trackwork installation, a variety of methods of supporting the direct fixation fasteners has been specified. All of them fall under the single pour methodology, using a thin grout pad on the concrete invert to achieve the final elevation required for support of the fasteners and using anchor bolts that pass through the grout pad into the concrete. invert slab.

In TW-1, the grout pads were placed on cleaned roughened invert and were made of 3,500 psi concrete grout. Fully threaded steel rods 10 in. long by 7/8-in. diameter were drilled and set in the invert. Subsequent phases added progressively more sophisticated materials and procedures to meet construction and design requirements. On aerial structures, in TW-2 and TW-4, epoxy resin with quartz sand aggregate was used for the grout material. Delamination during curing due to high shrinkage led to a cooler epoxy-modified concrete grout being specified for TW-5, 6, and 8. In underground sections the strength of the portland cement grout was increased to more than 7,000 psi to obtain higher early strengths. This, in turn, led to the use of bonding agents to overcome shrinkage stresses during curing. High strength steel was specified for the anchor bolts beginning with TW-3. In TW-8, galvanized, corrosion-resistant steel bolts were specified.

As part of the final design of TW-7, an in-depth value engineering study of the fastener support system was undertaken to develop a system that, in addition to meeting design requirements, offered improved constructibility and maintainability. All previous systems were analyzed along with those of other transit systems. Several alternatives were considered and the more promising ones were completely developed and costed for comparison.

The selected configuration specified for TW-7 (see Figure 9) was 4,000 psi portland cement grout placed on cleaned invert without a bonding agent. The size of the pads was increased in both width and minimum length. Pads greater than 2 in. thick were reinforced with wire mesh. The fully threaded anchor bolt was replaced with a female threaded insert. The inserts were made of corrosion resistant steel and were epoxy coated. Quality control tests were specified to ensure minimum bonding and to insert pullout strengths throughout the installation.

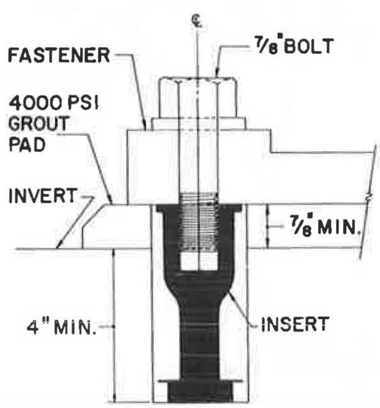


FIGURE 9 Fastener installation.

Several advantages to this new system were foreseen. The lower strength grout reduced the shrinkage problems and lessened the delamination due to differential movement during both curing and thermal expansion on aerial structure pads. Because the lower strength grout would have lower bonding requirements, the bonding agent was eliminated. The use of the threaded insert allowed a lower pullout requirement because the high strength connection required for the fastener would now be entirely between the metal components and would not involve the pullout strength of the threaded rod in the invert. The insert allows removal of a single fastener with minimal disturbance to adjacent fasteners. Shimming of fasteners can be done without raising the fastener more than is needed to insert the shim, which simplifies final profiling of the track. The epoxy coating of the insert provides added electrical isolation of the rail from ground.

The remaining paragraphs are a brief summary of several significant evolutions of WMATA standards.

Spectral Trackwork

Although most special trackwork details are in accordance with AREA recommendations and have remained unchanged, the following improvements have evolved.

Size of Turnouts

The only change from the originally recommended turnout sizes and crossing geometry to be used on Metro has been the elimination of the No. 4 equilateral turnouts in center storage tracks. The high rate of wear experienced by these turnouts during the interim period, when the center storage track also serves as the temporary turnback for the route until it is extended, has resulted in the installation of No. 6 equilaterals that have flatter curves and are less subject to wear. At the National Airport station, the No. 4 equilateral that was used for turnback service wore so rapidly that it was replaced with a No. 6 equilateral guarded turnout even though this required a special design and an expensive change-out performed under service.

Bonded Inserts

The use of bonded joints through main line special trackwork required installation of epoxy-bonded inserts at the heel of the switch rail and at the heel of the frog to transfer thermal stresses from the running rails to the closure rails and then back to the running rails thereby maintaining the continuity of the stress. Use of the bonded joints serves to stiffen the turnout and produces an improvement in ride quality. The inserts at the heel of the switch were designed to allow removal of the switch rail without disturbing the bonded insert. Three yards were built with bonded joints and inserts. Because of the concentration of turnouts in ladder tracks, the continuity of the rail caused the lateral stresses from thermal expansion to make lining the ladders difficult. Therefore, in yards, bolted joints are now used in turnouts. This eliminates the need for bonded inserts and results in less expensive procurement, construction, and maintenance.

Support Plates

In accordance with the recommended standard, all direct fixation special trackwork units are supported on steel plates that rest on 3/4-in.-thick elastomer pads. The plates are fastened to the invert with 7/8-in-diameter bolts. The rail components are held to the plates by a crane rail type clamp device that is removable, thereby allowing replacement of the rail without disturbing the plates. This permits rapid change-out of frogs and stock rails, important on transit systems because of the limited maintenance time available. The concept was extended to main line ballasted trackwork in TW-1 and in TW-3 to yard turnouts.

Over the years minor problems with alignment of main line turnouts on direct fixation have led to design modifications to the plates. More clamps were added in the area around the bonded inserts, and gauge plates were added at the switch points and frogs were added to prevent lateral movement.

CONCLUSION

The Metro system carries more than one-third of a million passengers each day. It does this with a high degree of reliability, speed, and comfort. The completion of the 101-mile system remains the principal goal of WMATA in spite of funding cutbacks that have delayed expansion of the system.

Only the more significant changes to the trackwork standards that have evolved through 10 design phases have been discussed. Lesser changes and the many standards that have not been changed have been passed over due to lack of time and space.

At De Leuw, Cather and Company, the changes that have been made are viewed as the result of advancing technology and as responses to construction and maintenance as proof of the dynamic nature of the state-of-the-art in transit track. We look forward to the future as an opportunity to further advance the state-of-the-art and to improve on what is already one of the finest transit systems in the country.

REFERENCES

1. Trackwork Practices of North American Rapid Transit Systems. WMATA Rapid Transit Development Program. De Leuw, Cather and Company, Washington, D.C., 1968.
2. Recommended Trackwork Standards. WMATA Rapid Transit Development Program. De Leuw, Cather and Company, Washington, D.C., 1969.
3. Trackwork Plans and Specifications. American Railway Engineering Association, Washington, D.C., 1984.

Publication of this paper sponsored by Committee on Track Structure System Design.

Track Rehabilitation and New Construction in An Operating Environment at BART

VINCENT P. MAHON

ABSTRACT

The Bay Area Rapid Transit District is completing the last phase of a three part construction and track rehabilitation project designed to relieve a bottleneck in the city of Oakland downtown "wye". This paper contains a before, during, and after description of the Oakland downtown junction area in sufficient detail to illustrate the impact of the construction project on system operations. Also outlined are the planning, coordination, management, and cooperation required between all divisions of the power and way maintenance, train operations, and engineering departments to complete the project on time and minimize the impact of construction on revenue schedules. Several innovative railroad construction methods are detailed without which this project could not have been completed within the minimal impact mandate.

Decreasing train headways on the Bay Area Rapid Transit District (BART) system involves several problems, one of which is a bottleneck in the city of Oakland downtown "wye" junction. The BART KE Line Expansion is a three part project designed to eliminate this bottleneck and provide additional operational facilities in support of the Close Headways program.

Project construction began in April 1980, and included the following tasks: building approximately 2.4 miles of track; changing the alignment by approximately 12 ft of more than 700 ft of revenue track; building and installing 15 main line turnouts; precisely locating existing facilities in the work site area; relocating or reconstructing existing facilities in conflict within new work; constructing two aerial structures; installing subsurface raceways and conduits; upgrading traction power facilities; and installing additional train control and communication equipment.

The probability was high that this construction project would interfere with revenue operations because of its magnitude and location. BART management mandated that the project be completed without impact on peak revenue service schedules and with only minimum effect on service during the lightest patronage periods.

BART TRACK SYSTEM DESIGN DETAILS

The BART revenue track system is divided into four double track lines covering a three-county area. The lines are designated as follows: the Alameda (A) line, the Contra Costa (C) line, the Richmond (R) line, and the San Francisco (M) line. Geographically, the system track layout can be visualized as an X configuration, with the Oakland downtown wye area serving as the junction point of these lines (see Figure 1).

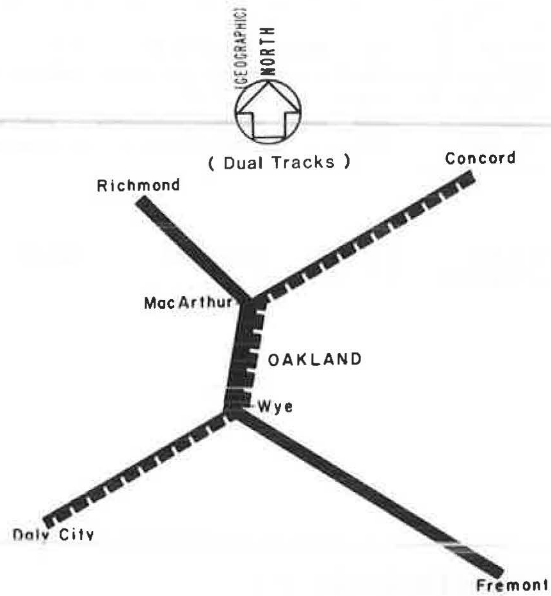


FIGURE 1 BART system simplified track configuration.

The design of the BART track system makes it possible to provide direct (without turn back) train service as follows:

- A-line to M-line and M-line to A-line,
- A-line to R-line and R-line to A-line,
- A-line to C-line and C-line to A-line,
- C-line to M-line and M-line to C-line, and
- R-line to M-line and M-line to R-line.

The junction configuration that makes this possible is shown in Figure 2. This junction covers a distance of more than 2.5 miles. As indicated in Figure 2, the C and R lines merge to a common line, previously designated the K-line, in the area of the MacArthur Station (K30). The K-line tracks then continue through downtown Oakland to a turnout area designated A05 where they diverge to either the A-line or the M-line.

The junction at the north end of the MacArthur Station (K30) where the C and R lines merge and diverge is designated the K35 turnout area. Tracks in this area run at grade, on elevated structure, and underground to provide a no-crossing traffic junction. From this junction four tracks (designated C1, C2, C3, and C4 tracks) serve the four passenger loading platforms at the MacArthur Station.

To the south of the MacArthur Station is a turnout area designated K25. The C1 and C3, and the C2 and C4 tracks merge and diverge here to the common C1 and C2 tracks of the K-line. The K25 turnout area is the north end of the downtown Oakland bottleneck.

The track and automatic train control systems were designed to allow C, K, and R line trains to

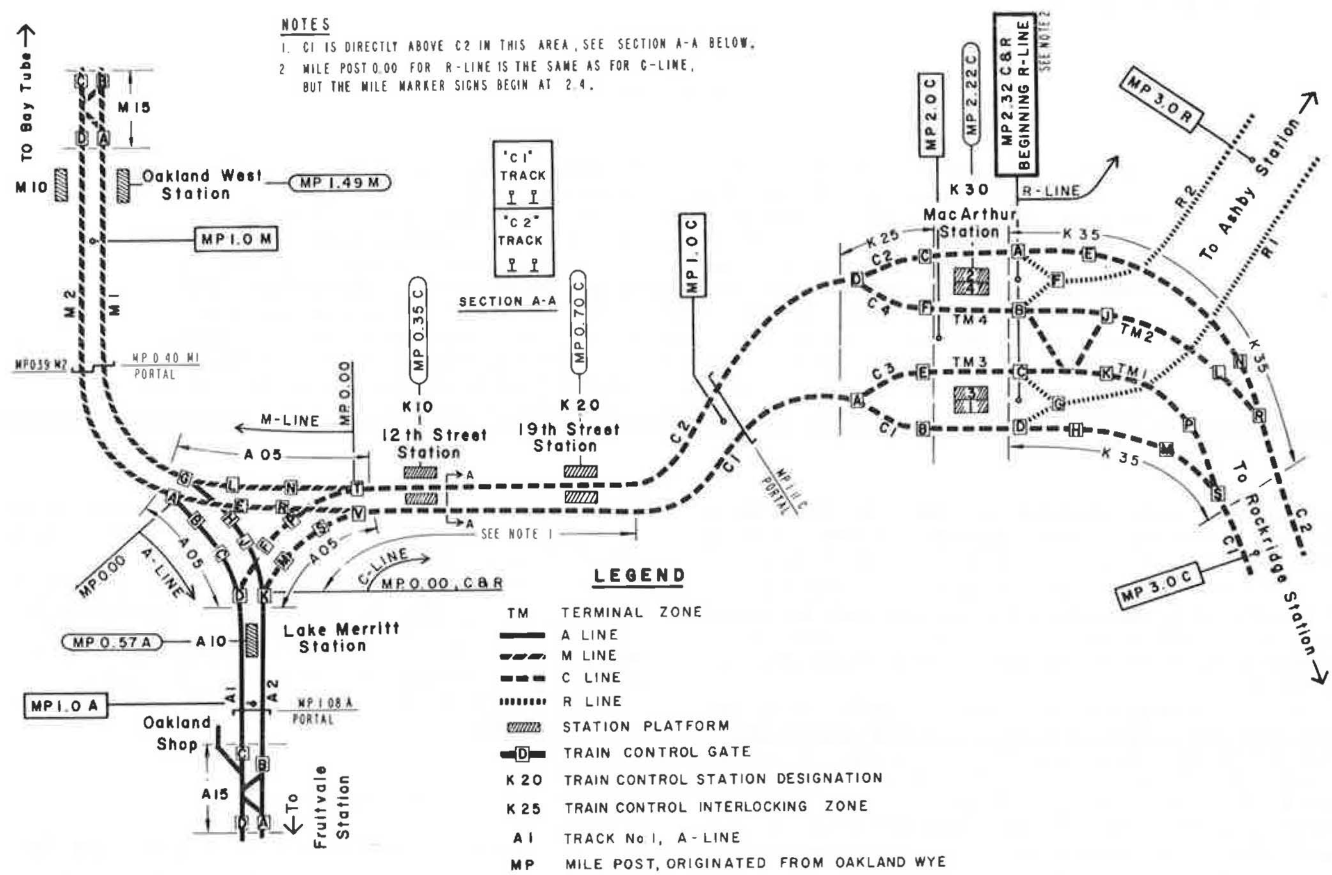


FIGURE 2 BART Oakland Junction detailed track configuration—before KE expansion.

switch tracks and turn back in the K35/K30/K25 area, either automatically or by remote control from the BART Central Train Operations Center (BART Central). The configuration also allows C and R line trains to turn back and reroute between the two lines.

At the south end of the K-line is the A05 turnout area. It is also called the Oakland Wye because of its physical configuration. The Oakland Wye is a multilevel subway no-crossing traffic junction between the A, K, and M lines that allows all combinations of any two lines to merge to the third line. A three-dimensional perspective of this junction is shown in Figure 3. The south end of the downtown Oakland bottleneck originates at A05 gates T and V. The process of merging the A and M lines into the common K-line completes the design problem. The probability is high that malfunctioning wayside or train equipment in the K-line area during peak revenue service would bring the compact BART system to a grinding halt. On the other hand, the two-track K-line limits the maximum train density and minimum system headway of the BART system.

BENEFITS OF THE KE EXPANSION PROJECT

The designers of the BART system recognized the bottleneck problem early in the BART project. A solution was designed, but because of insufficient funding, only the most costly portion was completed during initial system construction. When the subway through downtown Oakland was constructed, a third subway trackway designated the X trackway was built parallel to the C1 and M1 trackways between the 23rd Street and Washington Street portals. It was intended that this trackway would be finished and put into service as the BART system matured and the need for additional train capacity became a reality.

The KE Expansion, as the project is known, provides two major system improvements. First it adds a third revenue track, the CX/MX track. This track originates at the K25 turnout area, proceeds southward through a new turnout area designated K23, through the downtown Oakland subway, and terminates at a new turnout area south of A05 between the M1 and M2 tracks designated the M03 area. The CX/MX track can carry automatically routed train traffic destined for San Francisco from MacArthur Station (K30) or San Francisco-originated traffic destined for MacArthur Station, while the C1 and C2 tracks carry train traffic between the A-line and MacArthur Station. The CX/MX track has full passenger platform services at the two downtown Oakland stations, 12th Street Station (K10), and 19th Street Station (K20). When not used for revenue operations, the CX/MX turnouts provide switching capabilities for cutting bad order train equipment out of the revenue traffic pattern. Alternatively, the CX/MX track may be used to provide storage for standby trains scheduled for use during peak traffic periods.

The KE Expansion project also adds a spur track, with complete automatic routing and turn-back capabilities, in the at-grade and aerial area of the new turnout area designated K23. This gives the train operating department room to store either bad order equipment or standby trains.

The appearance of the completed KE Expansion project is shown in Figure 4. Contrast the junction configuration shown in Figure 4 with that shown in Figure 2 to observe the operational upgrade provided by the KE Expansion project. This project is a major step toward the goal of 72-train peak-period service, Furthermore, this project provides a new and important benefit for wayside maintenance--efficient single-track train operations between K30 and A05. Without the CX/MX track, single tracking operations are highly disruptive to train operations because of the M and A line merge and diverge.

KE EXPANSION PROJECT DETAILS

The KE Expansion project covers a distance of approximately 3 miles. It begins immediately south of MacArthur Station (K30), continues south to the 23rd Street subway portal and through the X subway to the Washington Street portal, then south approximately 0.3 miles to the south end of the M03 turnout area.

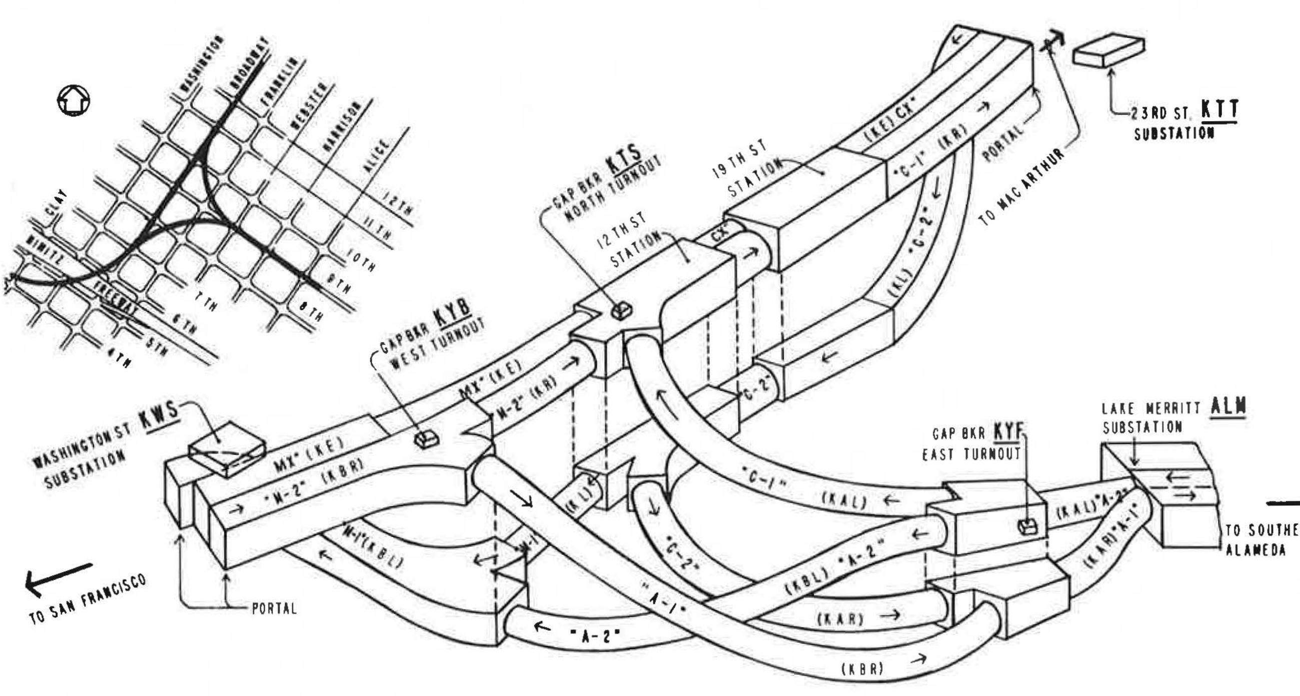


FIGURE 3 BART Oakland Wye perspective.

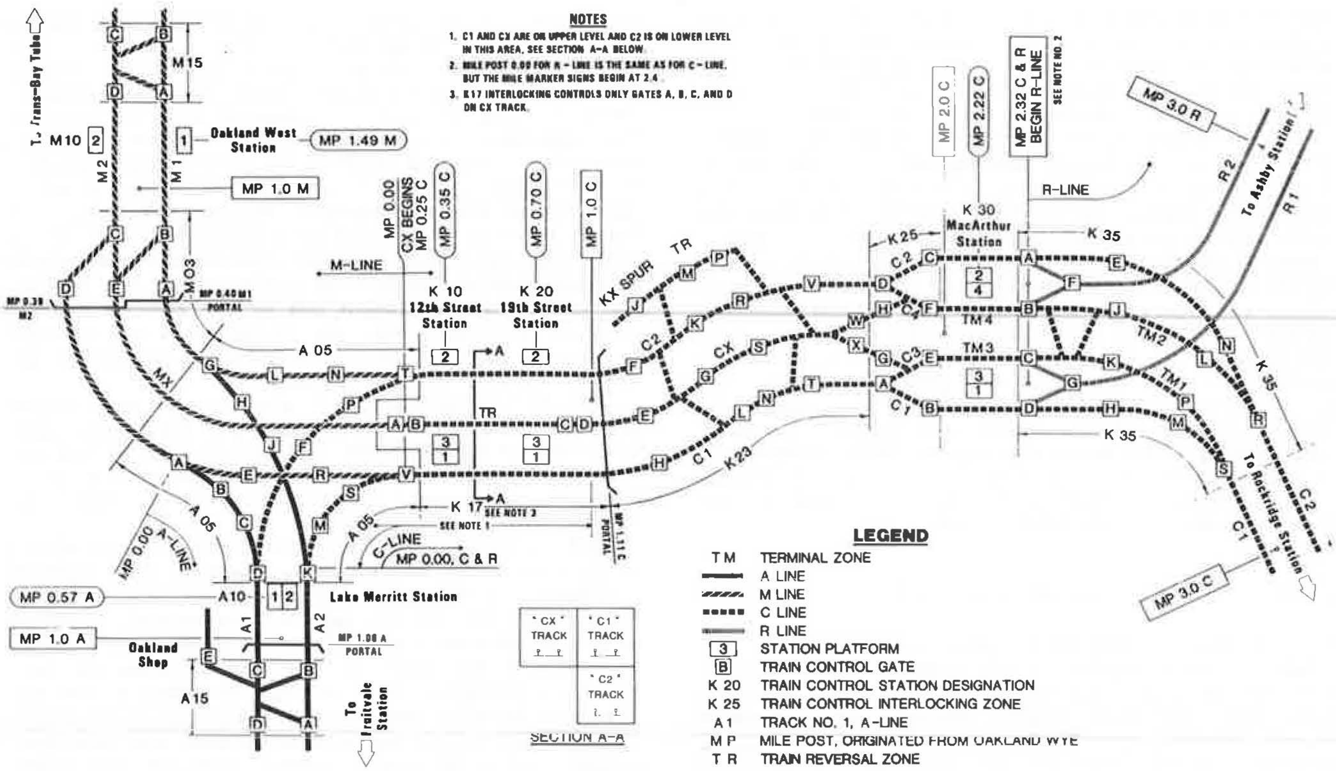


FIGURE 4 BART Oakland Junction detailed track configuration—after KE expansion.

The entire KE project is divided into three phases, designated A, B, and C. A detailed scope of each phase is discussed below.

Phase A Scope

Phase A, now complete, consisted of building the M03 turnout area and the CX/MX track from M03 to the Washington Street portal and through the X subway to the 23rd Street portal. The project included all electrical and subway facilities work, as well as station finish work at the two Oakland downtown passenger stations (12th and 19th Street Stations). Included was the BART Central electrification control board revision reflecting the KE Expansion details.

Phase A construction began April 1, 1980. Power and Way Maintenance Department track and traction power division employees installed the three M03 switches into the M1 and M2 revenue tracks and performed the associated electrical work, which was completed on June 21, 1980.

Work Area Considerations

All work not affecting revenue train operations continued around the clock on a closely supervised schedule. Work that affected train or employee safety was scheduled for nonrevenue periods, which normally extends from 0145 hr to 0445 hr (a 3-hr period). If the work required a longer time period, late night single track revenue operations were established to free a single track from 2200 hr, providing an additional 3.75 hr of track time. If an even longer period was required, late night single track revenue operations were scheduled from 2200 hr on a Saturday night until the end of revenue service, then reestablished at the start of revenue service Sunday morning and lasting until 1000 hr. This provided a 12-hr work period during a time when single track revenue operations have a minimal impact on revenue operations and inconvenience the smallest number of patrons.

Safety protection requirements for automatic train single track revenue operations are complex and time consuming to establish. Additionally, most of the wayside work required electrical third rail safe clearance protection for personnel, another time consuming effort. To maximize the work period, close coordination between the issuing authority (BART Central), the establishing agent (train control or traction power personnel, or both), and the requester was necessary. The power and way maintenance controllers stationed in BART Central provided this coordination effort. They worked closely and efficiently with all parties concerned to ensure that the track area was protected and prepared for work, then cleared and released on time for revenue operations.

Contract Work

Following the initial work by BART employees, contractor personnel began work in the X trackway beyond the clear point of the M1 and M2 tracks on April 25, 1980. The initial contractor work involved the construction of safety barricade fencing to isolate the contractor work area from the revenue trackway. From within the fenced area, the contractors built the CX/MX track, installed and completed the traction power third rail system, completed all the subway equipment and services, and performed the passenger station finish work. This work was monitored by a BART resident engineer.

The track, traction power third rail system, and subway facilities were completed by January 1982. The contract as a whole was completed in March 1982, but the barricades were retained until May 1984 for personnel safety and project security. Control of the area was turned over to the BART Train Operations Department, and they currently use the CX/MX track to store bad order trains during the revenue day.

Phase B Scope

Phase B, also now complete, consisted of several major tasks including the following: building and installing eight main line switches into the C1 and C2 revenue service tracks in the K23 and K25 turnout areas; changing the C2 track alignment by approximately 12 ft for a distance greater than 700 ft; constructing two aerial structures to accommodate the KX spur track; installing all traction power third rail system modifications in support of all switch and track construction and modifications; and constructing subgrade to accommodate relocation of drainage and electrical facilities. Work under Phase B was performed in three separate areas (designated A, B, and C) between MacArthur Station and the 23rd Street portal. Specific location detail is shown in Figure 5.

The tasks in Phase B were covered under two different contracts administered in two different ways. The contract covering the construction of two aerial structures to accommodate the KX spur track was a typical construction contract; that is, the contractor routinely performed and supervised the work while a BART resident engineer monitored the contract (same as the contractor work in Phase A).

The remaining tasks in Phase B, that is, those related to track realignment, turnout construction and installation, and subgrade construction, all involved the operating revenue system. Before the contract was drafted, the major concern of BART management was how to make a contractor responsible for performing the work while not interfering with revenue service. All of the usual contractor incentives and monitoring methods were considered, but they all had drawbacks.

It was finally decided that all contractor work would be performed according to work directives issued by the Power and Way Maintenance Department project director during the progress of the contract. The contractor would furnish labor, equipment, and certain materials, and then perform work as directed on a time and material basis. The contractor, in effect, became an extension of the Power and Way Maintenance Department work force. The responsibility to administer the contract was assigned to the department manager, power and way maintenance. A team from among power and way maintenance department personnel was assembled to handle the day to day details--a first at BART.

The contract not only spelled out the technical details and standards of the job, but also all the relevant operation rules and procedures that employees must follow while working in the active revenue environment. Furthermore, it defined the hours of work and the limits of revenue interference. All work was required to be performed after revenue or under late night revenue conditions with special train protection procedures.

Phase B construction began July 20, 1981 in area C. BART and contractor personnel built a temporary road vehicle crossing over the C3 track near Maintenance of Way Access 7 (MW07). This crossing was connected to MW07 and would remain until the completion of Phase C.

With completion of the crossing and road access construction on July 31, 1981, the appearance of areas A, B, and C changed rapidly. During the months of August and September, contractor personnel performed drainage structure relocation and underground trenching construction for electrical conduit installation under and between the revenue operational C1 and C2 tracks. The subsurface construction between the C1 and C2 tracks at the switch locations designated 3, 4, 5, 7, 9, 10, 15, and 16 on Figure 5 had to be completed on schedule. Furthermore, because of the proximity of the excavations to the revenue tracks, extensive shoring and cribbing was required to maintain the stability of the area for normal revenue train activity. The winter rainy season tested this stability, as construction continued through the wettest winter in Bay Area history--a full 200 percent above normal.

In general, all work on the C1 and C2 tracks was performed by BART personnel, whereas work under and between the tracks was performed by contractor personnel under the constant and direct supervision of BART supervisory personnel. Under the contract, all contractor work was scheduled from 2200 hr to 0445 hr. Because the period from 2200 hr to 0145 hr was still in the revenue operations period, unique train safety protection schemes were planned and implemented to minimize effect on train schedules and maximize personnel and train safety.

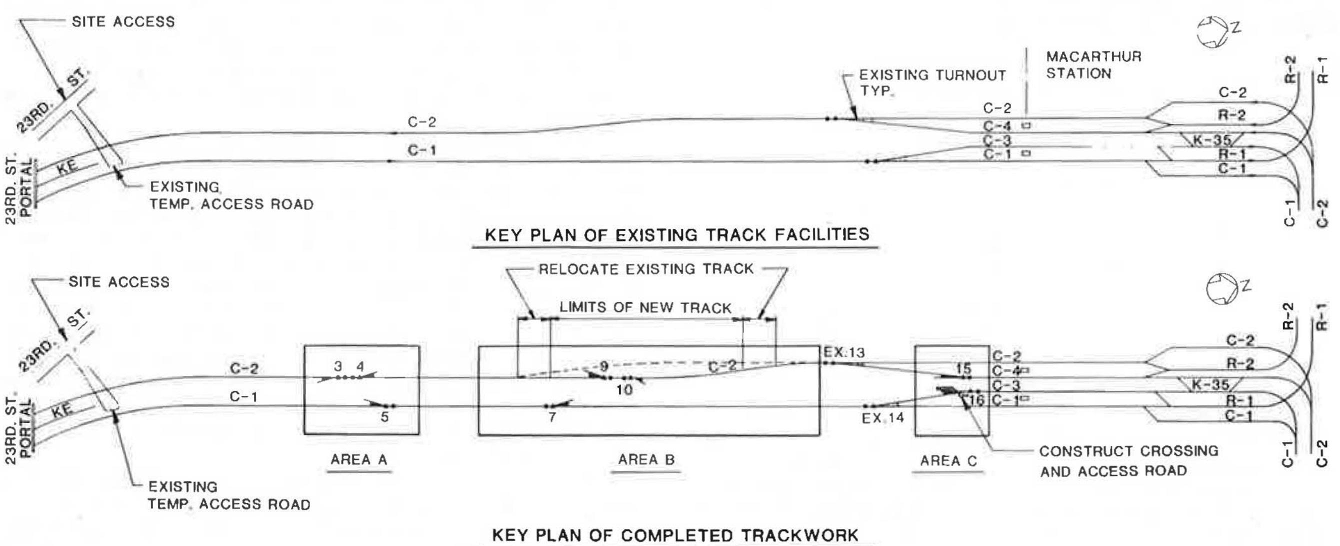


FIGURE 5 Phase B key plan.

Work Area Safety

All train traffic in the construction area was restricted to road manual operation while work was being performed. This safety measure was imposed with three levels of protection. First, BART Central ordered each train to stop at a stated track milepost location. Second, the wayside automatic train control speed encoding equipment (mux boxes) for all track signal blocks in the work area and for 1,000 ft preceding it were adjusted to transmit automatic zero mile per hour speed codes. Third, wayside flagmen were stationed in the work area, and BART Central released each train under the control of the flagmen and with road manual movement orders.

Under normal operating conditions in this area, the automatic train operation (ATO) maximum speed is 50 mph. Under road manual orders, the maximum allowable train speed is 25 mph, with the qualification to proceed with caution. As the trains moved beyond a stated milepost location, they were returned to ATO to proceed at speeds normal for the area. Work area limits were selected to correspond with train control track circuit limits. In this way necessary work area protection was afforded and the length of road manual train movements were minimized.

This level of protection was established every night promptly at 2200 hr and retained until 0145 hr. If either of the revenue tracks were required to be taken out of service, single tracking operations would be established. Frequently, third rail electrical safe clearance protection would be required during single tracking periods and between the nonrevenue hours of 0145 hr and 0445 hr. Regardless of the type or degree of protection needed, establishing and then removing it daily in a timely manner required a monumental coordination effort. The Power and Way Maintenance Control Center performed this coordination with almost flawless precision.

Turnout Construction Considerations

Turnout construction is a time consuming and labor intensive task. Phase B required that eight turnouts be installed in the C1 and C2 revenue tracks. The track in this area is constructed of continuously welded rail (CWR). The locations of all rail welds in the area had to be determined in order to lay out the exact switch placement and determine where the CWR would be cut. The turnouts to be installed included three number 10, three number 15, and two number 20 switches (see Table 1).

TABLE 1 Phase B Turnout Legend

Location Number	Turnout Type	Turn Direction	Remarks
3	15	Left	Curved
4	10	Left	Curved
5	15	Right	Curved
7	15	Left	Straight
9	10	Right	Straight
10	10	Right	Straight
15	20	Right	Straight
16	20	Left	Straight

During the design stage it was decided to build and install jointless switches. This decision required that the heel and toe of the frogs and the heel of the switch points be redesigned. Furthermore, field welds were required in lieu of joint bars throughout the switch area. This feature keeps the rail profile through the switch compatible with the welded rail environment in which it is installed. The quality of the ride through the switch is greatly improved as compared with the ride through a jointed switch.

Constructing the turnouts in place would have continuously disrupted revenue service over a period of several months. The mandate of not disrupting revenue service prevented this approach. Instead, the switches were constructed on special "roller ties" in between the C1 and C2 tracks, immediately adjacent to the location where they would be installed. The switches would then be pulled into place when the existing track was removed.

This decision revealed the need for several other decisions to be made and identified several requirements to be satisfied. Included among these considerations are the following: What is the exact location of the new switch? What is the exact location of the switch construction area? When would excavation construction in the area be complete? How could the revenue rail be cut and the ties and track removed in a short time period? What kinds of skids could be used to help move the switch from its construction site to its installation site? What methods could be used to make the move?

The Rail Removal Solution

Two major innovative ideas dictated solutions for most of the problems. First, the rail removal solution. On some night before the switch installation date, BART track personnel would cut the CWR at two locations, unfasten it from its tie clips and lift it off to the side of the right-of-way (leaving the ties in place). Twenty foot lengths of rail were then installed on the ties, making up 20-ft long rail panels that were jointed together, aligned, and surfaced. The panels were temporarily bonded together using number 4/0 cable welded to rail anchors that were attached to the rail. Upon completion of train control signal testing and adjustment, the area was returned to revenue service.

The second part of the rail removal task took place the night the turnout was to be pulled in. (The rail removal task will be discussed first and the turnout pulling process later.) With a crane on rail and in position to lift a 20-ft rail panel, a flat car was moved up behind the crane (see Figure 6). The panel joints, temporary rail bonds, and third rail sections were disconnected. The rail panel was then lifted, swung to the rear of the crane, and lowered to the flat car. The crane and flat car would then back up to position the crane to lift at the center of the next panel, and the disconnecting, lifting, and loading process would be repeated.

This procedure was repeated until all the panels were removed. It was done as rapidly as possible, as this was just the beginning of the night's work and the area had to carry revenue traffic in the morning over a new turnout. After the rail panels were loaded onto the flat car, the crane and flat car would reposition adjacent to the area where the 700 plus feet of realigned C2 track was to be located. The panels were lifted off the flat car and precisely positioned where the C2 track was to be relocated. The short rails were then released, leaving the ties properly spaced for the new C2 track construction and the short rails available for the next CWR rail section removal.

Several factors entered into the decision to make the rail panels 20 ft long. Thirty-nine-foot panels were considered, but when calculated the weight was found to be almost 2,000 lb heavier than the maximum safe load of the crane. A 30-ft panel was within the

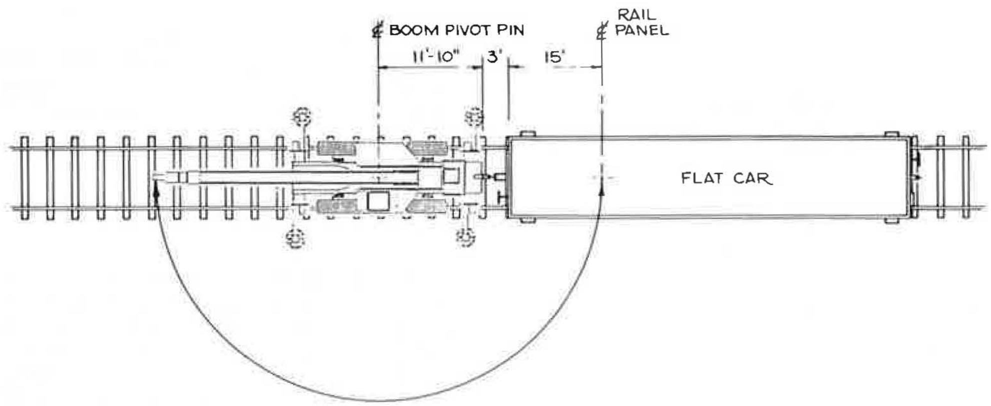


FIGURE 6 Crane and flat car detail.

safe load range, but it was determined that 20-ft (actually 19 ft 6 in., or one-half of the 39-ft rail) lengths were the best compromise because of the locations of the CWR welds and the ties to be moved.

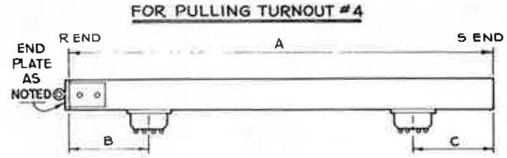
The long sections of rail that were released from service were available to the contractors as construction material. Other turnout construction materials were received at the BART Hayward rail yard, loaded onto the BART rail or work train during the day, and delivered at night to the construction site about 22 miles away.

The Switch Installation Solution

The process of pulling a switch into a space made available by rail removal began before the first turnout tie was ever laid. After the decision was made to build the switches adjacent to where they would be installed, it became clear that the elevation of the area between the C1 and C2 tracks would have to be increased; a survey indicated that the average difference in elevation in the toe of ballast to the top of rail was 2.4 ft for the C1 track and 2.0 ft for the C2 track. Of course, the exact amount of raise depended on the pulling-skidding method.

The pulling-skidding method to be used was a major concern until flat-topped chain-type roller assemblies (see Figure 7) were discovered. The method developed rapidly following this discovery. The roller assemblies were attached to the bottom of 12 and 15-ft long wooden ties to form a mobile platform base on which to build the switch. For turnout number four, the first turnout built and moved this way, seven ties were used in this manner. Refer to Figure 8 and Table 2 for construction details of the roller ties and their specific placement locations. A bridle was attached to the end plates of ties number 2, 3, 5, and 6 to provide a place to pull. The placement of the roller ties, the roller assemblies on the ties, and the bridles was calculated to give the best mechanical advantage and switch stability during the pull.

FIGURE 7 Flat-topped chain-type roller assemblies.



NUMBER	A	B	C	REMARKS
1	12'	3'-2"	3'-2"	
2	12'	3'-2"	3'-1"	END PLATE
3	12'	2'-8"	2'-7"	END PLATE
4	12'	1'-7"	2'-6"	
5	15'	1'-7"	3'-11"	END PLATE
6	15'	1'-7"	2'-6"	END PLATE
7	15'	1'-7"	1'-2"	

FIGURE 8 Roller tie construction detail.

The next consideration was the ballast surface the switch would move over during the pull. To provide a smooth and low-friction surface, 6-in. channel iron was used as a track for the chain roller assemblies. Before any of the ties for the new switch were laid, the required elevation of the channel iron was determined and the elevation of the area between the C1 and C2 tracks was raised. The placement of the roller ties was then determined and the roller tracks (channel iron) were installed. For turnout number four, they were placed between the C1 and C2 tracks and also under the C2 track, all at the proper elevation. To accomplish this, ballast had to be removed from between the ties and under the track (see Figure 9). This was typically done before the rail was cut. Once installed, the ballast

TABLE 2 Roller Tie Placement Detail

Tie Number	Station Location	Tie Length (ft)	Channel Length (ft, in.)	Adjacent Spacing (ft, in.)	PS[a] Distance (ft, in.)
1	990+36.07	12	25-6		10-5.500
				10-5.500	
PS[a]	990+46.53	-	-		-
				9-8.125	
2	990+56.21	12	25-6		9-8.125
				20-8.875	
3	990+76.95	12	26-6		30-5.000
				20-1.875	
4	990+97.11	12	27-6		50-7.000
				20-1.875	
5	991+17.27	15	29-0		70-8.875
				19-2.125	
6	991+36.45	15	31-0		89-11.00
				0-5.750	
PF[a]	991+36.93	-	-		90-4.750
				[illegible]-10.625	
7	991+47.81	15	31-6		101-3.375

[a]For reference only; roller ties are not used at point-of-switch (PS) or point-of-frog (PF) locations.

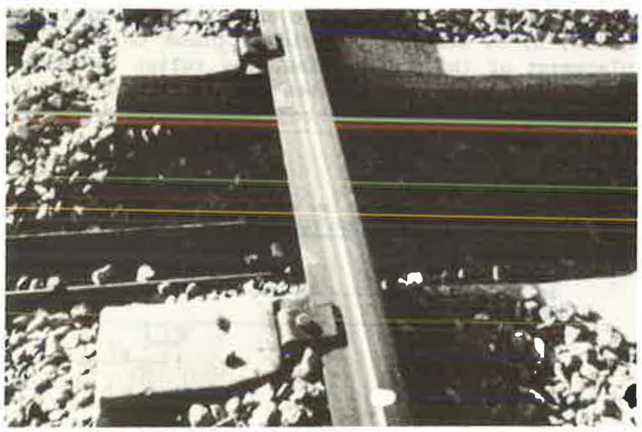


FIGURE 9 Roller channel installation.

was replaced to help hold the channel iron in place during the switch construction period.

The switch construction was handled by contractor personnel under the direction of BART supervisors. The project director issued a roller tie construction detail (Figure 8), a roller tie placement detail (Table 2), and a turnout tie data table for each s itch to be built. When the switch was built and ready to be installed, the switch and channel were jacked up so that the bottom of the ties would have approximately 2 in. clearance above the ballast. The ballast was then tamped tightly under the channel. All that remained was to perform the installation. The installation was typically scheduled for a Saturday night. Single tracking operations would be established from 2000 hr with an estimated completion time of 1000 hr for returning the track to revenue service Sunday morning. The installation was performed by both BART and contractor personnel.

The final consideration was the pulling method. At this point several viable alternatives were available, but one method stood out from all the rest. The entire Phase B construction site sits in the median of a California freeway, at grade level. The California Department of Transportation was most cooperative in closing the left-most traffic lanes, which allowed a heavy equipment wrecker to be positioned to pull the switches into position. After months of research, planning, and preparation, the outcome was anticlimatic. The average pull took less than 4 min to position the switch. We were prepared for less desirable circumstances, however, and had simultaneously developed an alternate plan to pull from the opposite track. In addition, the roller-channel method was designed and installed so that the turnout could be manually moved if necessary.

Important Details

To this point the major highlights of BART's instant track switch installation program have been discussed. However, there are more details that complete the program and without which there would not have been the same measure of success.

While building each switch, the third rail insulators were installed on the appropriate ties and the switch machine was installed and adjusted. Before installation day, the contractor cut and bolt-jointed back the third rail in the future turnout area.

On turnout installation day, the first task was to remove the previously bolt-jointed third rail section. As the rail panels were lifted, front loader tractors began removing ballast from the track bed using the roller channels as an elevation reference. When all the rail panels were removed and the ballast was at the correct elevation, the channels were verified to be free of ballast and at the correct elevation and alignment.

The heavy equipment wrecker was then connected to the bridle cables and the pull was made. Any necessary incremental position adjustment was manually made, the switch was then jacked up and the roller ties were removed. The switch was lowered to the track bed and the ends of the switch were joined to the ends of the existing rail, using hydraulic rail pullers as necessary to close the rail gap.

The end loader tractors were then used to replace the ballast and correct the grade. BART track personnel then manned the BART tamping and lining equipment to precisely correct surface and alignment. Final ballast dumps were made with the BART air-controlled ballast cars; then it was time to complete the rail bond and electrical third rail installation. The area was cleared of all equipment and personnel and made revenue ready. When complete, the train control technicians performed their final tests and adjustments. When they were clear, a test train was brought into the area for road manual and ATO test runs.

This procedure was repeated throughout the installation of all eight turnouts. The only other major task was the realigned C2 track. The contractor built approximately 717 ft of new track adjacent to the existing C2 track using the methods previously mentioned. The ends of the existing C2 track were then cut and pulled into alignment with the new track. This task was as successful as the others.

Phase B was a big success story for the Power and Way Maintenance Department. Everyone associated with the project grew personally and professionally. The final work was completed in January 1983.

Phase C Scope

Phase C work consists of track and turnout construction, electrification and traction power construction and equipment installation, and train control and communications equipment installation. With the completion of this contract, the KE Expansion project will be complete and revenue ready. Work under this contract began in March 1983 and completion is anticipated in November 1985.

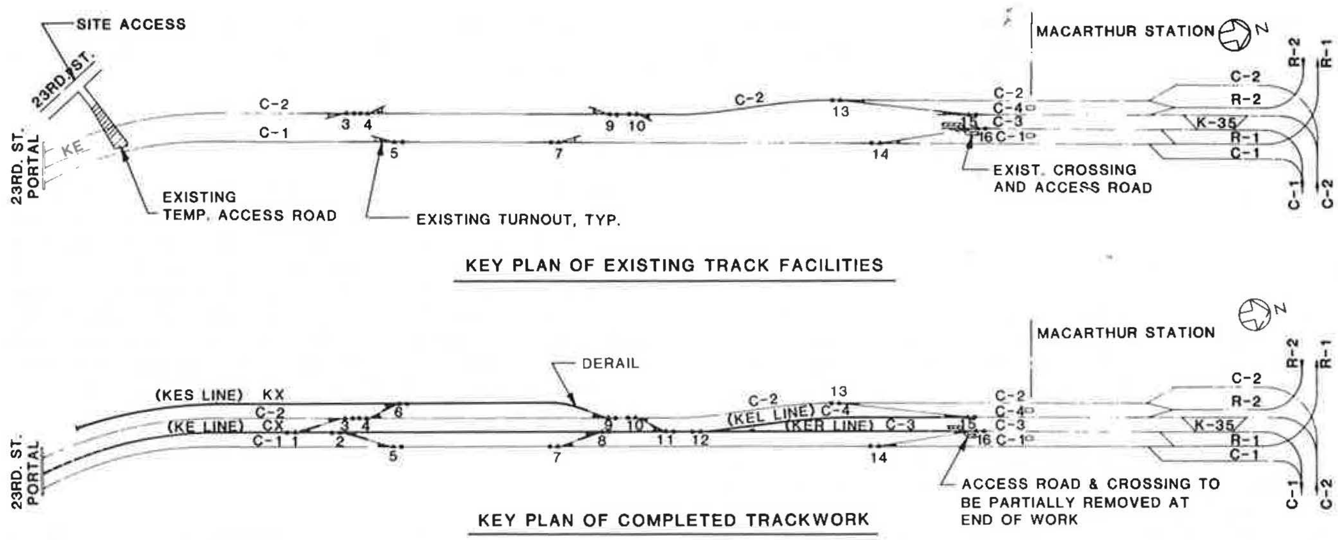


FIGURE 10 Phase C key plan.

The work area of this contract extends from MacArthur Station at the north, south along the C1 and C2 trackway to the X trackway at the 23rd Street Station portal, then south through the X trackway to the Washington Street portal, and south to the M03 turnout area. Also included in this work area are the train control equipment rooms at the 12th Street, 19th Street, MacArthur, and Lake Merritt Stations, and the traction power facilities in the work area.

Work Area Safety

The area between MacArthur Station and the 23rd Street portal and the area between the Washington Street portal and the M03 turnout are revenue-active. To protect contractor personnel, wooden safety barricade fencing is required to isolate the work area from the revenue area. Furthermore, close supervision by the BART resident and field engineers is required when any traction power or train control equipment installations are performed on the active revenue system.

The majority of the work is performed during the day by contractor personnel. To provide them with wayside vehicle access, the C3 track has been removed from service for the duration of the project. To accomplish this, K25 switch number 15 was aligned and spiked in the normal direction (K25 A to B), electronic route prohibits were placed for routes K25 A to E and K25 E to A and through K35 C. In addition, a knife switch was installed to make it possible to isolate the entire C3 track third rail section, and a physical gate was installed across the C3 track behind the clear point of the C1 track. Finally, a safety barricade fence was erected to isolate the revenue operational C1 track from the C3 track crossing and road extension to MW07.

The contractor then had road vehicle and equipment access to the work area. Although the C3 track is scheduled to remain out of service for the remainder of the KE project, all construction and all equipment storage within the C3 track is required to be beyond the C3 track clear point or be mobile such that under BART emergency conditions the C3 track can be cleared and returned to service by BART personnel within 60 min of notification.

Track and Turnout Construction

Track and turnout construction under Phase C was totally different from Phase B. Participation of the Power and Way Maintenance Department in construction under this contract was limited to work train operation (such as delivering material from the BART Hayward rail yard or ballast dumping), operating the BART mechanized track surfacing and lining equipments, installing one derailer, and changing several wood ties that extended into the revenue area. The contractor, on the other hand, built six turnouts and approximately 1.5 miles of track.

In general, the contractor's work consisted of building the second half of the turnouts that BART personnel installed in the C1, C2, C3, and C4 tracks under Phase B as well as the track to connect to them. See Figure 10 and Table 3 for details. The contractor also built all temporary safety barricade fencing and is responsible for all permanent fencing construction. At the end of the construction phase, the contractor will remove all temporary fencing, in addition to all temporary work area access points, including the two temporary access roads at 23rd Street and MW07 and the stairway at the Washington Street portal. Track and turnout construction was completed in April 1984.

Electrical Construction

Electrical tasks under Phase C consists of the construction, installation, and cut over to service of traction power third rail equipments, traction power control system equipments, and electrical utility equipments.

TABLE 3 Phase C Turnout Legend

Location Number	Turnout Type	Turn Direction	Remarks
1	15	Left	Curved
2	15	Right	Curved
6	10	Left	Curved
8	15	Left	Straight
11	10	Right	Straight
12	20	"Y"	Equilateral

The subgrade conduit runs for this work were built during Phase B. The external conduit runs will be built under Phase C. It should be noted that although the contractor pulls in the control cables, the termination of these cables into the working system is to be performed by BART maintenance personnel. The electrical part of this contract was completed in February 1984.

Train Control Construction

The train control tasks under Phase C include the following: the installation and cut over to service of all train control wayside equipment electrical power services; all the additional wayside and train control room equipments needed to provide local and remote supervised and controlled ATO and manual train operations on all new track and through all new turnouts; all new wayside maintenance communications equipments; and the additionally required train destination sign equipment at the 12th and 19th Street Stations. The operational characteristics of the KE Expansion project were previously detailed in Figure 4, completion is expected in November 1985.

SUMMARY

On completion of the KE Expansion project, San Francisco Bay Area commuters will experience an increased level of service even though it is only a part of the Close Headways program. We at BART are confident that the investment of $22,000,000 for this expansion will result in a level of service improvement that will aid BART in continuing to gain ridership through the process of decreasing the perceived advantages of alternate Bay Area transportation methods.

Publication of this paper sponsored by Committee on Track Structure System Design.

Estimates of Rail Transit Construction Costs

DON H. PICKRELL

ABSTRACT

Reliable estimates of the costs of constructing new rail transit facilities are necessary to evaluate the growing number of proposals to build new rail lines and extend existing ones. Yet the construction cost estimates used in past studies have often been erroneous, even when they have been based on detailed engineering analyses of proposed projects. Further, rail construction costs appear to have increased rapidly in recent years, even after being adjusted to reflect general price inflation throughout the economy. New estimates of the costs of constructing rapid transit and light rail facilities are reported. These estimates are developed by statistically allocating (via regression analysis) total expenditures for 18 rapid transit and 14 light rail construction projects among their individual components. The results include estimates of unit costs for building rapid transit and light rail lines and stations underground, at grade level, and on elevated structures, including construction outlays and expenses for acquiring the necessary land at typical prices. Some uncertainty exists about the cost estimates for individual rail system components (lines and stations) developed here, but the procedure for estimating them allows this uncertainty to be explicitly quantified. Yet the best estimates of line and station costs suggest that local transportation planners and consultants have seriously underestimated the likely expense for building almost every new rail line or system extension now under serious consideration in the United States.

The recent resurgence of interest in major new rail transit investments among both professional transportation planners and political decision makers, after several decades of widespread disinvestment in rail transit facilities, has focused considerable attention on the costs of constructing new rapid transit and light rail lines. Reliable estimates of these costs play a critical role in evaluating the growing number of proposals to build new rail systems or extend existing ones, as their suitability depends at least in part on how those costs compare with the potential resource savings and other benefits such investments offer. Despite the obvious importance of using reliable cost estimates in such evaluations, past studies of rail transit's suitability have often relied on simple per-mile cost figures derived from limited construction experience. Even when sophisticated engineering cost studies have been undertaken, actual construction costs have typically been much higher than the original estimates produced using their detailed methods.

Another impetus for studying rail construction costs is what appears to be their extremely rapid escalation in recent years, even after taking account of the persistent general price inflation that prevailed throughout the 1970s and early 1980s. For example, Boston's 5.4-mile, 7-station northern extension of its Massachusetts Bay Transportation Authority (MBTA) Orange Line was completed in 1975 for slightly less than $300 million (for comparative purposes this and all subsequent construction costs are reported in equivalent 1983 dollars). [Note that where they were available, annual construction outlays were converted to 1983 dollars using changes from the year in which they were incurred to 1983 in construction cost indices for individual U.S urban areas reported in Engineering News Record (1). Where annual outlays were not available, total project expenditures were adjusted by the change in the appropriate construction cost index between the middle year of the project and its 1983 average value.] Yet an almost identical extension of the Chicago Transit Authority (CTA) Milwaukee line had been constructed only 5 years earlier for about $180 million, whereas the only modestly more extensive 7.7-mile, 9-station Baltimore Mass Transit Administration (MTA) Phase I line, which opened in late 1983, cost more than $900 million to build.

A third reason to investigate further is the puzzling variation in the costs of building what appear to be similar rail transit systems: the 13-mile (90 percent in tunnel), 17-station second segment of the Washington, D.C. Metrorail system was constructed for approximately $980 million, whereas the 13.7-mile (only 40 percent of which is underground), 15-station Metropolitan Atlanta Rapid Transit Authority (MARTA) Phase A project in Atlanta required nearly $1.7 billion to complete.

One question that naturally arises is to what extent these differences can be explained by the extensiveness and capacities of the individual facilities constructed, rather than by harder to identify factors such as variation in local construction prices, geologic and topographic considerations, or effectiveness in project management. Reported here are the results of a preliminary statistical analysis of the costs of constructing 32 recent rail transit systems and line extensions in U.S. and Canadian urban areas. The basic approach used parallels those of previous engineering and accounting-based studies of rail project costs, insofar as an attempt is made to develop estimates of the unit costs for constructing various functional components of transit systems (such as guideway, tunnels, or stations).

This study differs because an attempt is made to estimate the specific costs typically associated with such individual functional units by relating the actual total expenditures for transit construction projects to their respective combinations of those components (using regression analysis), rather than by allocating accounting expenditures or contract prices to specific system components. Despite this difference, the results obtained appear to be consistent with those of previous studies, although somewhat higher unit cost estimates are obtained here than in previous studies. One advantage of this approach is that the results it produces may be more broadly applicable to the problem of forecasting the costs of completing the various rail transit projects now planned or underway, because such results incorporate information on virtually every recent rail transit construction project in North America.

FACTORS LIKELY TO AFFECT RAIL CONSTRUCTION COSTS

Rail transit systems consist of several basic functional components or units: the track or guideway; the right-of-way on which it is located (which can be in underground tunnels, at or slightly below the land surface, or on elevated structures); passenger stations, transit vehicles; and fixed facilities such as yards, depots, and maintenance garages. Much of the wide variation in the costs of constructing rail transit lines is undoubtedly introduced by expenses for acquiring or constructing the right-of-way on which the guideway is located. (Even if building a rail line entails no direct expenditure for land acquisition--such as where land already under public ownership or an inactive railroad right-of-way is available--it will impose real and substantial opportunity costs for right-of-way, because any land it uses certainly has some value in alternative uses that is obviated by locating a transit facility on it.) These land acquisition costs are certainly an important source of potential variation in the costs of providing surface or elevated rail rights-of-way, although it is difficult to specify in advance exactly how extensive land requirements are in specific corridors, and how they are likely to differ between at-grade and elevated alignments. Expenses for right-of-way land can be largely (but certainly not completely) avoided by locating rail lines underground, but only by substituting the high attendant expense of constructing tunnels.

Station locations, passenger handling capacities, and architectural characteristics also appear likely to be among the critical determinants of construction costs. Surface stations are able to use the simplest passenger access facilities and platform designs, and are likely to offer the fewest complications in construction procedures. Thus they would be expected to exhibit considerably lower installation costs than stations of equivalent capacity situated on elevated structures or in underground excavations. On the other hand, as with the guideway itself, land requirements for surface stations may be considerably larger and thus more costly than those for elevated stations. Although underground placement of stations can again substantially reduce land acquisition requirements, excavation and construction costs can be substantial, especially where they must be designed to accommodate large passenger volumes.

Certain physical features of stations, some of which are determined by the anticipated volume of passenger traffic, also appear likely to have a pronounced effect on construction costs. These include total station size or volume, the specific platform layout used, and the number and capacities of passenger access and egress facilities. Some design considerations such as depth underground or architectural elaborateness may also affect station construction costs, even though they may not affect actual passenger-handling capacity or other dimensions of in-use performance. Unfortunately, most of these design parameters are site-specific as well as difficult to measure explicitly, so their individual effects on typical station construction expenses are difficult to isolate.

Although this range of potentially important determinants of rail project costs is quite wide, a logical first step is to investigate the association between actual expenditures for individual rail transit construction projects and readily available measures of their makeup. The two most directly observable characteristics of individual projects are their line lengths and numbers of stations, each of which can be classified according to their location underground, at grade level, or on elevated structures. The makeup of 18 recent rail rapid transit and 13 modern light rail transit construction projects is given in Tables 1 and 2. Because the spacing of stations appears to be relatively consistent among the various projects, it also appears logical to test the simple association of project costs with only the length of right-of-way of each of these three types, leaving implicit the exact number of stations provided.

TABLE 1 Rail Rapid Transit Construction Project Characteristics

		Two-Track Miles/Number of Stations[a]		
City	Project	Tunnel	Surface	Elevated
Cleveland	Initial Line	-	14.9/15	-
	Airport Extension	0.3/0	3.8/3	-
Philadelphia	Lindenwold Line	-	14.5/13	-
	Snyder-Pattison Extension	1.2/1		-
San Francisco	BART System	20.0/14[b]	27.0/7	24.0/13
Washington, D.C.	Metrorail Phases I-IVA	22.4/28	13.3/11	1.5/2
	Phases V-VI	12.5/9	10.7/8	1.0/2
Atlanta	Rail Phase A	5.5/8	5.8/7	2.4/2
Baltimore	Metro Phase I	4.5/6	-	3.2/3
Boston	Red Line South	-	9.5/5	-
	Red Line Northwest	3.2/4	-	-
	Orange Line North	1.0/2	4.4/5	-
Miami	Metrorail N-S Line	-	1.7/0	19.3/20
New York	63d Street tunnel	12.0/0	-	-
	2d Avenue tunnel	7.2/0	-	-
Chicago	Dan Ryan Line	-	9.4/9	1.1/0
	Milwaukee Extension	1.2/2	3.9/4	-
	O'Hare Airport Extension	0.6/1	6.6/3	-

[a]Equivalent miles of two-track line.
[b]Includes approximately 4-mile transbay "tube" (no stations).

A SIMPLE EMPIRICAL MODEL OF PROJECT COSTS

The foregoing discussion suggests two basic models that can be used to relate each project's total costs to its readily measurable characteristics:

$$TC = a_0 + a_1*UGMI + a_2*AGMI + a_3*ELMI \tag{1}$$

and

$$TC = b_0 + b_1*UGMI + b_2*AGMI + b_3*ELMI + b_4*UGSTNS + b_5*AGSTNS + b_6*ELSTNS \tag{2}$$

where

TC = total project expenditures (in 1983 dollars),
UGMI = miles of two-track line in tunnel,
AGMI = miles of two-track line at-grade,
ELMI = miles of two-track line on elevated structures,

TABLE 2 Light Rail Transit Construction Project Characteristics

		Two-Track Miles/Number of Stations		
City	Project	Tunnel	Surface	Elevated
Buffalo	Initial Line	5.2/8	1.2/6	-
Calgary	Southeast Line	0.7/0	6.9/11	-
	Northeast Line	-	6.1/7	-
Edmonton	Initial Line	-	3.5/4	-
	Downtown Subway	0.9/3	-	-
	North Extension	-	1.4/2	-
San Diego	San Ysidro Line	-	15.8/18[a]	-
San Francisco	MUNI/BART Tunnel and Line Extension	5.7/4[b]	13.3/7	-
Boston	Green Line Riverside Branch reconstruction	-	12.0/0[c]	-
Newark	Subway rehabilitation	4.3/4[d]	-	-
Pittsburgh	Tunnel reconstruction and South Hills Line	1.0/3	12.3/7	-
Portland	Banfield Line	-	15.1/16	-
Toronto	Scarborough Line	-	2.7/2	4.3/6
Vancouver	New Westminister Line	0.9/2[e]	3.7/5	8.7/8

[a]Total length is 15.8 miles, of which 1.7 two-track miles and 12 stations were newly constructed.
[b]Part of tunnel and four stations jointly used by BART system.
[c]Minor rehabilitation of 13 stations accompanied line reconstruction.
[d]Tunnel and stations not rebuilt, but line substantially rehabilitated.
[e]Including 0.7 miles in existing single-track tunnel expanded to double-track capacity.

UGSTNS = number of stations underground,
AGSTNS = number of stations at grade, and
ELSTNS = number of stations on elevated structures.

In model 1, a_1, a_2, and a_3 correspond respectively to the unit--in this case, per mile--construction costs of underground, surface, and elevated rapid transit line segments, inclusive of station construction costs. Analogously, the coefficients b_1, b_2, and b_3 in model 2 represent the unit construction costs of these three types of line segments exclusive of the costs of constructing stations, which are represented by b_4, b_5, and b_6 for underground, surface, and elevated stations.

The interpretation of the terms a_0 and b_0 is more ambiguous, but ideally they represent expenditures for planning and constructing the minimal complement of ancillary facilities necessary to supplement the system described by the line-mile and station variables. Including these terms acknowledges that some project construction costs may not be uniquely associated with a specific structural component of the project. One complication in their interpretation arises, as previously discussed, because new systems will generally require installation of such facilities, whereas projects that represent line additions or extensions of existing systems may not require significant expansion of their capacities. [Note that average station spacings for underground, surface, and elevated alignments are 0.86, 1.34, and 1.42 miles for the 18 heavy rail projects included in this analysis, and 0.84, 1.08, and 0.78 miles for the 14 light rail projects studied.]

Further, if the scale of vehicle storage and maintenance facilities is closely correlated with line mileages or number of stations, their costs will be subsumed within the line and station unit cost estimates instead of being incorporated into the intercept terms. In this case the intercept terms will capture the effects of any remaining expenses not associated with the included measures of the scale of the project (such as planning expenditures) and may be mistakenly interpreted if they are regarded simply as the costs of constructing fixed facilities. Recognizing these potential complications in their interpretation, variants of both models that exclude their respective intercept terms were also estimated and are compared to the specifications that include them in the discussion that follows.

EMPIRICAL ESTIMATES OF UNIT COSTS

A variety of methods can be used to estimate the parameters of these models ($a_0,..,a_3$ and $b_0,..,b_6$). Among them are allocation of expenditure accounts or individual contract awards to individual functional units, engineering-based estimation of resource requirements (labor, materials, etc.) for constructing individual components, and assignment of individual contract awards to particular system components. Another approach is to statistically estimate the parameter values using a sample of observations on project costs and their individual component make-ups. When this method is employed, a residual term is implicitly specified for each model; it corresponds to the variation in individual project costs that is not accounted for by the variables included in the model.

Tables 3 and 4 contain ordinary least squares estimates of unit costs for heavy and light rail transit project components, derived by computing the coefficient values that minimize the sum of the squares of these unexplained residual terms. (Because each project's total expenditures are expressed in equivalent 1983 dollars, the resulting estimates of unit costs for project components can also be interpreted in 1983 dollars.) Of course, the small sample sizes lead to considerable uncertainty surrounding the specific unit cost estimates, but on the whole they exhibit surprising consistency and precision, particularly considering the variety of projects represented and the diversity of their designs and locations. Further, even these simple models account for 60 to 80 percent of the variation in expenses among individual projects.

A few specific implications of the estimates reported in Tables 3 and 4 are particularly noteworthy. First, the intercept terms are consistently only about as large as their standard errors, suggesting that there is a low probability that the true values of a_0 and b_0 differ significantly from zero. Second, it is interesting to note that the model including only the line length variables (model 1, reported with and without the intercept term as models 1 and 1a in Tables 3 and 4) accounts for more than one-half of the wide variation in the costs of both heavy and light rail projects, despite its simple specification.

This may occur partly because the range of station spacings within each type of project is not extremely wide (it averages about 1.35 miles for rapid rail systems and 0.96 miles for light rail lines, although for both modes the average figure varies considerably among underground, surface, and elevated alignments). Nevertheless, some improvement in the explanatory power of the models is achieved by separately specifying line lengths and numbers of stations, as evidenced by the smaller standard errors in estimating total project costs with this slightly more complex version of the model (reported as models 2 and 2a in Tables 3 and 4). [Note that adjusted R^2, the conventional goodness-of-fit measure, declines despite this improvement in precision, in response to the reduction in the already limited number of degrees of freedom imposed by the more complex specification.]

The data in Table 5 summarize the best point estimates of line and station construction costs obtained from the two samples of rail transit projects. As indicated in Table 5, rapid transit and light rail lines in underground tunnels including conventionally spaced stations (1.16 and 1.30 per mile, respectively) typically cost about $137 and $114 million per mile to construct (again, these and all subsequent estimates are reported in 1983 dollars). Thus some limited cost savings on a line-mile basis appear to be possible using light rail tech-

TABLE 3 Least-Squares Regressions of Rail Rapid Transit Project Construction Costs on Project Characteristics (n = 18)

	Coefficient (Standard Error) in Specification			
Variable	1	1a	2	2a
Constant	34.1 (67.9)		74.9 (62.4)	
Two-track miles in tunnel	137.1 (8.0)	136.5 (8.3)	100.4 (11.2)	102.8 (10.0)
Two-track miles at grade	27.8 (8.1)	30.8 (6.8)	17.8 (7.5)	22.3 (12.2)
Two-track miles on elevated	49.3 (8.4)	55.3 (9.3)	36.5 (17.1)	39.3 (14.6)
Stations underground			36.0 (11.4)	39.5 (11.2)
Stations at grade			6.7 (4.6)	9.7 (5.0)
Stations on elevated			23.0 (16.1)	22.9 (16.3)
Adjusted R^2 of regression	0.75	0.80	0.59	0.64
Standard error of estimate[a]	199.6	211.6	135.7	154.4

[a]Millions of 1983 dollars, around a mean of $987.7 million.

TABLE 4 Least-Squares Regressions of Light Rail Transit Project Construction Costs on Project Characteristics (n = 14)

	Coefficient (Standard Error) in Specification			
Variable	1	1a	2	2a
Constant	56.0 (43.7)		7.4 (52.5)	
Two-track miles in tunnel	98.9 (15.7)	104.1 (16.5)	60.2 (45.1)	67.5 (39.0)
Two-track miles at grade	16.2 (5.6)	16.5 (3.8)	10.4 (6.8)	11.0 (5.2)
Two-track miles on elevated	67.7 (11.5)	72.0 (11.3)	65.9 (11.1)	66.2 (10.4)
Stations underground			31.8 (22.7)	34.2 (26.5)
Stations at grade			8.7 (5.7)	8.9 (5.1)
Adjusted R^2 of regression	0.64	0.68	0.54	0.60
Standard error of estimate[a]	95.3	97.9	91.4	86.8

[a]Millions of 1983 dollars, around a mean of $249.0 million.

TABLE 5 Estimates of Unit Construction Costs for Rail Transit Projects

Component	Typical Rapid Transit Unit Construction Cost[a] (millions of 1983 dollars)	Typical Light Rail Unit Construction Cost[b] (millions of 1983 dollars)
Two-track mile, in tunnel:		
Including stations	137	114
Excluding stations	103	68
Two-track mile, at grade:		
Including stations	31	17
Excluding stations	22	11
Two-track mile, on elevated structure:		
Including stations	55	72
Excluding stations	39	-
Underground stations	40	34
At-grade stations	10	9
Elevated stations	23	-

[a]Source: Parameter estimates for specifications 1a and 2a in Table 3, rounded to nearest million.
[b]Source: Parameter estimates for specifications 1a and 2a in Table 4, rounded to nearest million.

nology, although rapid rail transit probably still offers lower costs per unit of passenger-carrying capacity because its maximum capacity is nearly twice that of most light rail systems.

Comparable figures for surface lines are somewhat closer--typically about $31 and $17 million for rapid and light rail lines--including costs for stations at representative spacings of 1.34 and 1.42 miles (corresponding to station frequencies of 0.77 and 0.93 per mile). For lines on elevated structures, estimated rapid transit and light rail construction costs, including stations at typical spacings of 1.42 and 0.84 miles (0.70 and 1.19 stations per mile), are respectively about $55 and $72 million per mile. Although superficially surprising, the higher cost estimate for light rail than for rapid rail transit is no doubt partly explained by the fact that each line-mile of light rail typically includes nearly twice as many stations as each mile of rapid transit line. In addition, only recent examples of elevated light rail lines actually represent an experimental, intermediate-capacity technology, so its slightly higher expense is less surprising.

Disaggregating into line segments and stations, constructing underground rapid transit lines normally entails an expenditure of about $103 million per mile, somewhat higher than the typical $68 million value that appears to be typical for light rail lines constructed in underground tunnels. Underground stations for these two types of lines appear to have similar costs, typically reaching nearly $40 million for those serving heavy rail systems and about $34 million for those serving light rail lines in tunnels. For surface facilities, light rail lines have apparently been only about one-half as costly to construct as their heavy rail counterparts--about $11 million versus $22 million per mile, excluding stations--whereas surface stations for the two types of lines appear to be closely comparable in expense ($9 to $10 million).

Thus it appears that significant construction cost savings can be achieved by cities that choose to employ light rail rather than full-scale rapid transit. Any potential savings from installing light rail facilities underground or at grade apparently stem primarily from the slightly lower costs for right-of-way and line construction, rather than from significant savings in constructing stations or other facilities. Yet the estimation results summarized in Table 5 show surprisingly similar per-mile costs for the two types of lines when stations are included, regardless of whether they are placed in underground, surface, or elevated alignments. This suggests that much of the potential cost savings from light rail may have been sacrificed by incorporating more frequent stations, perhaps in an effort to improve its passenger collection and distribution capabilities to compensate partly for its slower line-haul speed.

EXAMINING THE OUTLIERS

The various specifications given in Tables 3 and 4 generally perform surprisingly well in reproducing the costs of constructing the samples of projects from which they are estimated. Most of the project costs estimated using the different variants of the models fall within 10 percent of their inflation-adjusted total costs. Yet as the data in Table 6 indicate, there are consistently two groups of outliers, or projects with actual costs that are not predicted accurately by the models estimated here. Among heavy rail construction projects, the predicted costs of Boston's MBTA Red Line Northwest extension are consistently only 60 to 65 percent of actual expenditures, whereas those for Atlanta's MARTA Rail Phase A are only 85 to 90 percent of actual outlays. Calgary's Southeast light rail line also appears to have been considerably more costly than predicted by the various models.

At the same time, according to the data in Table 6, predicted costs for two rapid transit projects in Chicago--the Dan Ryan line and O'Hare Airport extension--are considerably above (150 to 200 percent) their actual values. Similarly, the San Ysidro light rail line in San Diego was considerably less costly to construct than anticipated by any of the models estimated here: its estimated costs are about one-third higher than actual construction outlays.

TABLE 6 Predicted versus Actual Costs for Selected U.S. Rail Transit Construction Projects

Project	Predicted Project Cost (millions of 1983 dollars)	Predicted Cost as a Percent of Actual Cost (%)
Boston Red Line Northwest Extension	490[a]	62.6
Calgary Northeast Line	121[b]	75.9
Atlanta Rail System Phase A	1,449[a]	87.5
San Diego San Ysidro Line	127[b]	114.7
Chicago O'Hare Airport Extension	277[a]	150.8
Chicago Day Ryan Line	321[a]	200.7

[a]Predicted using unit cost estimates reported in Table 3 and project descriptions in Table 1.
[b]Predicted using unit cost estimates reported in Table 3 and project descriptions in Table 2.

There are several possible explanations for such large forecasting errors in a few specific locations. First, local construction costs vary among geographic areas in response to differences in prevailing wage rates and delivered prices of construction materials, and they may vary in ways that contribute to the observed pattern of errors. yet the urban area construction cost indices used to adjust project expenditures suggest exactly the opposite pattern: with some minor differences depending on the date for which they are examined, typical construction costs are less than 80 percent of the national average in Atlanta and only about 95 percent of the nationwide figure in Boston, whereas they are no lower in Chicago or San Diego than for the average of large cities nationwide.

Another possible explanation is differences among projects in the cost of land acquisition for rights-of-way, because none of the specifications tested explicitly controls for differences in unit land prices, but land purchase costs are included in some of the project expense totals. This may help explain the surprisingly low costs of projects in San Diego, which makes extensive use of an unused railroad line, as well as in Chicago, where newly constructed transit lines extensively occupy freeway medians. Differences in the effectiveness of local project management could also partially account for the large errors in predicting the costs of these specific projects, although the market in the type of large-scale public works construction management services utilized for rail transit projects appears to be national in scope, and thus unlikely to give rise by itself to such wide variation.

Other possible explanations for the project outliers remain; for example, two of the unusually high-cost projects were new lines that required construction of depot and maintenance facilities, whereas the unusually low-cost line extensions in Chicago apparently utilized existing yards and maintenance facilities. Station designs also appear to vary in ways that could explain some of the wide deviation from the more typical cost experience: two of the stations serving the MBTA Red Line northwest extension, for example, incorporate extensive facilities to serve passengers arriving by automobile or bus, whereas several stations in the MARTA system are architecturally elaborate and designed to accommodate very high passenger volumes. Finally, particularly difficult geologic conditions or tunnel construction characteristics may partially explain the atypically high cost experience, because three of the four projects for which costs are substantially underpredicted incorporate extensive underground facilities.

FORECASTS OF FUTURE SYSTEM COSTS

To illustrate the applicability of the cost estimates developed here, the data in Table 7 compare their predictions of the costs for constructing several planned rail transit systems and line extensions with those prepared by consultants or local planning organizations. These projects range in scale from an 11-mile light rail line planned to utilize almost entirely existing rights-of-way in Rochester, New York, to the nearly 40-mile, 26-station final segment of the Washington, D.C. Metrorail system. (Again, considerable care has been taken to express all figures in 1983 dollars to ensure their comparability, but in a few cases it has not been possible to produce a completely reliable current-dollar estimate from published figures.) Because details about planned station spacings and locations are not available in every case, some of the cost estimates were constructed using the simple forms of the unit cost models presented in Tables 3 and 4, which represent per-mile cost estimates including stations at typical spacings. In most cases, enough detail about their planned configurations was available to allow use of the separate unit cost estimates for line and stations given in Table 5 to produce total cost estimates.

As the data in Table 7 indicate, there is a pronounced tendency for consultants and local planning organizations to substantially underestimate the costs of constructing currently planned light rail systems, compared to those implied by recent U.S. and Canadian experience in building similar systems. That experience--as embodied in the unit cost estimates developed here--implies construction costs ranging from 23 percent (for St. Louis' planned Clayton Airport light rail line) to 188 percent (for an 18-mile light rail line now under construction in Sacramento, California) higher than their consultants' or local planning organizations' most recent published projections. If there is any notable tendency among these discrepancies, it is the somewhat surprising one that those systems that have been studied more recently and in greater detail by consultants and local planners--particularly the light rail lines now under construction in Sacramento and San Jose, California--have projected costs that are more unrepresentative of recent experience than are lines in the early planning phases in cities such as St. Louis, Rochester, and Columbus.

This result may be attributed partly to the difficulty in accurately converting distant future cost estimates for this latter group of cities to current dollars, but it is difficult to tell how much this might contribute to their apparently more realistic estimates. In any event, it appears clear that the currently projected costs of the several U.S. light rail systems now in varying stages of planning and construction are generally much too low to be consistent with typical recent experience. In fact, most of the light rail line construction cost estimates given in Table 7 are apparently also much too low to be consistent with even the comparatively favorable cost record established in the construction of San Diego's celebrated "budget" light rail line.

TABLE 7 Comparison of Published and Author's Cost Estimates for Planned Rail Transit Projects in U.S. Cities

Urban Area	Project Description	Line Miles (Stations)	Construction Cost Estimates[a] (millions of 1983 dollars): Published ($)	Using Table 5 ($)
Sacramento	Light rail at grade	18.3(20)	132 FD	380
San Jose	Light rail at grade	19.7(25)	278 FD	442
Columbus	Light rail at grade	10.6(11)	102 AA	216
Detroit	Light rail in tunnel	15	1,500 PE	1,710
Rochester	Light rail (20% in existing tunnel, 80% at or near grade)	11.4(14)	95 AA	251[b]
St. Louis	Light rail at grade	18.0(26)	350 AA	432
Los Angeles	Heavy rail in subway	17.4(18)	3,325 FD	2,512
Honolulu	Heavy rail in subway	7.8(11)	825 AA	1,243
Houston	Heavy rail (10% in subway, 70% elevated, 20% at grade)	18.5(22)	1,880 FD	1,292
Washington, D.C. (Phases VIA-VIII)	Heavy rail (40% in tunnel, 5% elevated, 55% at grade)	39.6(26)	2,185 FD	2,884
Atlanta (Phases B2 and C)	Heavy rail (20% in tunnel, 30% elevated, 50% at grade)	19.0(11)	857 FD	1,045

[a] "Published" figures are from Alternatives Analyses (AA), Preliminary Engineering Studies (PE), and Final Design (FD) estimates.
[b] Assumes no tunnel construction or rehabilitation expenditures.

The comparisons given in Table 7 between officially projected costs for constructing new heavy rail systems or line extensions and those estimated using the models developed here are more equivocal. On one hand, some of the same tendency to underestimate costs in comparison to recent historical experience is evident. Again surprisingly, it arises mainly in the two cities--Washington, D.C. and Atlanta--that provide much of the recent U.S. experience with constructing heavy rail systems. Certainly the outlying portions of these systems are likely to be less costly to build than their earlier downtown segments, because the at-grade and elevated rights-of-way that can be more readily utilized in suburban locations tend to be less costly than subway lines. Nevertheless, planners in these two cities appear likely to have underestimated the costs of completing their systems even by comparison to the recent experience with building these typically less costly surface and elevated lines in their own and other large U.S. cities such as Baltimore, Miami, and San Francisco.

In contrast, planners currently project that the costs of constructing the entirely new heavy rail systems currently under study in Los Angeles and Houston will substantially exceed those that would be predicted from typical recent experience. The implied unit cost for constructing Los Angeles' underground line would approach that (nearly $250 million per mile) for the most expensive recently constructed subway line, Boston's MBTA Red Line Northwest extension, whereas that for Houston's mixed-alignment system (about 10 percent in tunnel, 20 percent at grade, and 70 percent elevated) would approach the figures for building the most expensive comparable system in recent history, Atlanta's Phase A project. Some of this result may be caused by an inadequate adjustment for the inflation rates anticipated over their construction horizons, although it is difficult to specify how much. It may also partly reflect local planners' recognition of unusually high construction costs in these particular urban areas; local building cost indices show that typical construction costs in Houston and Los Angeles are currently 134 percent and 120 percent of their national average figure (2). In any event, it appears that planners and consultants in these cities have been considerably more cautious in preparing cost estimates for their planned rail systems than have their counterparts in cities planning to build light rail lines or complete planned rapid transit systems.

ACKNOWLEDGMENT

This research was conducted at the Transportation Systems Center under the sponsorship of the Urban Mass Transportation Administration, U.S. Department of Transportation (DOT) but the results reported here do not represent an official view of the DOT. Helpful guidance and comments were provided by Douglass Lee and Larry Doxsey, also of the Transportation Systems Center, Boris Pushkarev, and several anonymous TRB referees.

REFERENCES

1. Engineering News Record. July 14, 1977; July 17, 1980; July 16, 1981; July 14, 1983.
2. Engineering News Record. July 5, 1984.

Publication of this paper sponsored by Committee on Track Structure System Design.