TRANSPORTATION RESEARCH RECORD 1030

Track Maintenance and Classification Yards

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

Transportation Research Record 1030
Price $7.40
Editor: Elizabeth W. Kaplan
Compositor: Harlow Bickford
Layout: Marion L. Ross

modes
2 public transit
3 rail transportation

subject areas
21 facilities design
40 maintenance

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 maintenance and classification yards.

(Transportation research record ; 1030)
1. Railroads–Track–Maintenance and repair–Addresses, essays, lectures. 2. Railroads–Yards–Addresses, essays, lectures. I. National Research Council (U.S.). Transportation Research Board. II. Series.
TE7.H5 no. 1030 [TF530] 380.5 s [625.1'7] 86-901
ISSN 0361-1981 ISBN 0-309-03921-5

Sponsorship of Transportation Research Record 1030

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 Rail Freight Classification Terminal Design
James A. Wetzel, Consolidated Rail Corporation, chairman
J. Reilly McCarren, Consolidated Rail Corporation, secretary
Merrill J. Anderson, J. F. Brower, William F. Cracker, H. D. Crawley, Mark S. Daskin, Earl E. Frank, Thomas S. Guins, Robert L. Gwin, Hubert Hall, Dale A. Harrison, Robert E. Heggestad, Alain L. Kornhauser, Carl D Martland, John F. McGinley, Sr., Charles W. Morse, J. C. Pinkston, B. H. Price, William A. Schoelwer, Paul E. Van Cleve, Alexander Wilson, Peter J. Wong, Irwin P. Yatzkan

GROUP 3–OPERATION, SAFETY, AND MAINTENANCE OF TRANSPORTATION FACILITIES
D. E. Orne, Michigan Department of Transportation, chairman

Facilities and Operations Section

Committee on Railway Maintenance
Albert J. Reinschmidt, Association of American Railroads, chairman
William B. O'Sullivan, Federal Railroad Administration, secretary
Stephen C. Baran, Richard F. Beck, James R. Blacklock, H. D. Campbell, C. L. Coy, Ronald H. Dunn, Alfred E. Fazio, Thomas B. Hutcheson, Robert A. Kendall, Richard G. McGinnis, Willis H. Melgren, Guenther W. Oberlechner, Ernest T. Selig, William J. Semioli, Walter W. Simpson, Charles V. Slavis, Harry E. Stewart, Vincent R. Terrill

Edward J. Ward, Transportation Research Board staff

Sponsorship is indicated by a footnote at the end of each paper. The organizational units, officers, and members are as of December 31, 1984.

NOTICE: The Transportation Research Board does not endorse products or manufacturers. Trade and manufacturers' names appear in this Record because they are considered essential to its object.

Contents

Addresses of Authors

Deaton, Hubert, III, Schnabel Foundation Co., 1701 North Greenville Avenue, Richardson, Tex. 75081
Egan, Philip D., The Reinforced Earth Company, 1700 North Moore Street, Arlington, Va. 22209-1960
Elias, Victor, Victor Elias & Associates, 6809 Carlynn Court, Bethesda, Md. 20817
Markow, Michael J., Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Mass. 02139
Melhuish, Arthur W., Dowty Hydraulic Units, Ltd., Cheltenham GL51 0TP, England
Murakami, Kiyoaki, Japanese National Railways, Tokyo, Japan
Shahin, M. Y., U.S. Army Construction Engineering Research Laboratory, Champaign, Ill. 61820-1305
Turnquist, Mark A., School of Civil and Environmental Engineering, Cornell University, Ithaca, N.Y. 14853
Wong, Peter J., SRI International, 333 Ravenswood Avenue, Menlo Park, Calif. 94025

Application of Life-Cycle Costing and Demand-Responsive Maintenance to Rail Maintenance of Way

MICHAEL J. MARKOW

ABSTRACT

Although rehabilitation and renovation are becoming an increasingly important aspect of construction activity, comparatively little work has been devoted to the development of planning and management tools to evaluate different maintenance policies. In contrast with the design of new construction, the planning and management of maintenance (including routine maintenance, rehabilitation, and renovation) are more concerned with the long-term performance of an existing facility and associated life-cycle costs. Performance and costs are influenced not only by the quality of initial design and construction but also by the magnitudes and frequencies of the actual loads imposed, the actual environmental conditions encountered during service, aging, and the maintenance actually performed on the facility through its life. Because the interactions among these factors are complex, most managers extrapolate maintenance trends and costs from past experience and fail to investigate useful alternatives in maintenance policy through life-cycle costing. In work with the Red Nacional de los Ferrocariles Espanoles (RENFE) (Spanish National Railroad), some new concepts in maintenance management (referred to as a demand-responsive approach) were applied to study several different track maintenance policies. For each alternative not only total program costs, but also the impacts on future system condition, were computed. As a result of this research, a number of key findings emerged for RENFE top management. The analytical concepts of the demand-responsive approach and their application to the Spanish rail network are described.

The rehabilitation and renovation of existing, mature facilities are becoming increasingly important components of construction activity. However, comparatively little work has been devoted to the development of planning and management tools intended specifically for maintenance programs, particularly life-cycle costing of facilities. [For brevity, the term "maintenance" will be used in this paper in a broad sense, encompassing routine maintenance, rehabilitation (i.e., major repair), and renovation (i.e., substantial replacement of ballast, ties, and rail).] Yet, decisions regarding the planning, financing, budgeting, implementation, monitoring, control, and evaluation of maintenance are different from corresponding actions for new construction, in several ways:

1. Mature facilities require an understanding of the concepts underlying their performance, as opposed to their design. Good performance models, which can predict the behavior of a facility and its deterioration as a function of loading history, operating environment (temperature, moisture, soil conditions, etc.), aging, and past maintenance performed are scarce and have not been the focus of much research (in relation to design). Properly developed performance models could be used for design; design procedures cannot always be used for performance.
2. Because rail links rarely fail catastrophically, it is difficult to define the point of failure. This, in turn, complicates the specification of standards governing track performance, safety, and cost. In shifting industry emphasis from new construction to maintenance, engineers may have to rethink the process by which routine maintenance, rehabilitation, and renovation standards are now developed and enforced. This also holds for those facilities that do fail catastrophically (i.e., suddenly and with potentially large loss of life), but the ways in which specific technical and safety issues are addressed analytically will differ.
3. There is a need to understand the role of maintenance itself in influencing track performance. In virtually all performance studies, emphasis is placed on the effects of loads, environment, and aging; in contrast, there is little information about the influence of preventive or corrective maintenance on the subsequent rate of track deterioration.
4. Decisions to repair existing track are complicated by the wide range of activities possible (ranging from minor routine maintenance to major rehabilitation or reconstruction), problems in spatial and temporal allocation of resources throughout a network, and choices between investment and noninvestment policies (e.g., strengthening of track versus imposition of load limits).
5. As a result of Items 1-4, the optimization of maintenance policy is difficult. New concepts and analytic approaches need to be introduced among those responsible for transportation infrastructure, together with complementary support activities (e.g., inspection and monitoring, collection of relevant data, revisions to existing management systems and procedures, introduction of new technology).
6. The management of mature infrastructure implies an ability to evaluate life-cycle performance

and costs. Trade-offs must be measured in economic as well as technical terms to account for impacts to facility owners, users, and nonusers and to provide a consistent basis of measurement for decisions made at different points in a (possibly extremely long) life span.

In addressing these issues, the concepts needed to address facility maintenance itself will be summarized. Then analytic approaches that have been used to implement these concepts within workable procedures will be described. Finally, the application of these procedures to the rehabilitation of the Spanish National Railway will be illustrated.

CONCEPTS OF LIFE-CYCLE COSTING

Analysis of Track Maintenance Alternatives

This work is based on the premise that planning and managing facility maintenance requires a life-cycle approach; that is, consideration of the total costs of construction, routine maintenance, rehabilitation, and use of the facility throughout its service life. There are several reasons for this.

First, as the key component of a rail facility, the track embodies critical trade-offs among the economic costs of construction, maintenance, and rehabilitation (all engineering issues) and of train operation, travel time, accidents, and smoothness of ride (operational and marketing considerations). Furthermore, because these costs accrue in a time span that typically covers several decades, life-cycle costing is a natural and appropriate methodology for analyzing track investment strategies.

Second, for those facilities (such as track) that do not fail catastrophically, it is difficult to define the points at which the facilities require repair or renewal. This, in turn, complicates the specification of design, maintenance, and rehabilitation standards governing track performance, safety, and cost. Life-cycle cost analyses can be used, however, to estimate both the total and the marginal benefits and costs of alternative standards, thereby providing economic as well as engineering guidance on the selection of the appropriate track investment strategy.

Third, life-cycle cost analyses, if properly formulated, help in the understanding of the role of routine maintenance and rehabilitation in influencing track performance. This capability contrasts with, for example, conventional analyses of track design and construction, which emphasize the effects of train loads, environment, soils, and aging but which offer no information relating routine maintenance or rehabilitation policy to the subsequent rate of track deterioration. Where this gap in information exists, policy makers cannot analyze well the impacts of deferred maintenance, nor can they assess effectively potential trade-offs among initial design standards, construction quality, and future maintenance requirements. Such studies are feasible, however, using life-cycle cost analyses.

Fourth, decisions to repair or rebuild track are complicated by the wide range of activities possible (ranging from minor routine maintenance to major rehabilitation or reconstruction), problems in spatial and temporal allocation of resources throughout a network, and choices between investment and noninvestment policies (e.g., strengthening of track versus adjustments in train load or speed limits). Life-cycle analyses can illuminate the long-term costs and benefits of these different courses of action.

Demand-Responsive Approach

The implementation of life-cycle analyses of track required a new approach to looking at track performance and the factors that influence costs throughout its service life. This approach is referred to as "demand-responsive" because routine maintenance, rehabilitation, or reconstruction are viewed as responses to the demand for repair or renewal of the facility. This demand for work arises through both a physical dimension (the condition of the facility, which reflects the quality of initial design and construction; the accumulation of wear and damage from the combined effects of traffic loads, environment, and age; and corrections due to past repairs) and a policy dimension (standards of initial design and construction and the level of maintenance, rehabilitation, or reconstruction to be performed, expressed through quality standards). Furthermore, because the prediction of facility condition is central to the demand-responsive approach, the impacts, as well as the costs, of alternative investment policies can be computed.

Treating routine maintenance, rehabilitation, and reconstruction as demand-responsive activities requires that three additional elements be introduced within existing planning and management models. The first is that estimates of future resource requirements and costs cannot be extrapolated from past trends; they must instead be based on predictions of structural and operational deficiencies caused by use, environment, and age. The second is that, in designing models to be sensitive to the implications of different policies, there must be unambiguous statements of maintenance, rehabilitation, or reconstruction policies that define the types of preventive or corrective actions to be taken and when and where they are to commence. The third is that new relationships must be identified between the as-maintained state of the transportation facility and the impacts to both the transport agency and the traveling public to provide a measure of the benefits or liabilities of each policy at the costs computed. Organization of these ideas within a unified structure is shown in Figure 1; additional details on the demand-responsive approach and its applications may be found elsewhere (1-6).

Analytical Procedures

North American Experience

Many studies of track performance, track life, or track maintenance have been conducted by or for North American railroads. Although the models differ in their scope and approach, in general they attempt to predict the deterioration of one or more track characteristics as functions of several variables, such as annual tonnage (or traffic density), degree of curvature, weight of rail, and velocity of trains. Other factors, recognized on a more limited basis in selected models, include axle or wheel loads, rail age, ballast and tie condition, hardness of rail steel, distinction between jointed and welded rail, superelevation of track, and weight of ties.

Many North American models predict the expected lives of specific track components. Included here are predictions of rail head wear (7-9), rail life due to fatigue (10), overall rail life (11), and tie life (8,12). However, these models do not predict the actual performance of these track components (they predict only the time of useful or safe service), do not include costs, and are not sensitive to changes in maintenance policy. More general

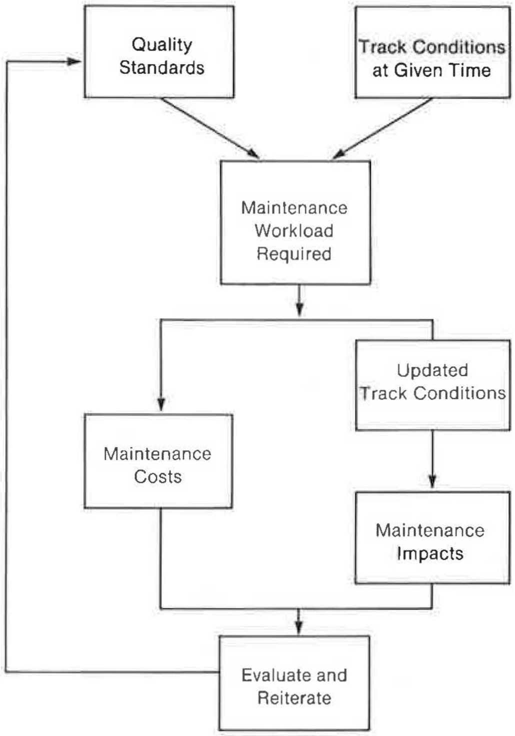


FIGURE 1 Demand-responsive approach to track investment.

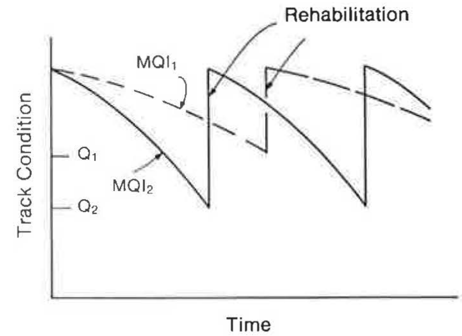


FIGURE 2 Effects of two different routine maintenance and rehabilitation policies on track condition.

models of track performance represent track deterioration or condition using a composite index (13,14). Other models (15) use a profitability criterion to analyze track costs. Although certain models (14,15) do include track maintenance costs within their formulations, these costs are treated as expenditures rather than as responses to demand for repair.

Incorporating the concepts in Figure 1 within analytical models requires an economic as well as an engineering approach. A brief description of how this is done is provided next.

Model Employed in Study

The analytical procedures needed to implement the management structure in Figure 1 are organized within a simulation model, the MIT Intercity Transportation Model (6). Each maintenance policy to be considered is tested by the model, which simulates the performance of the rail network; computes costs of routine maintenance, rehabilitation, and renovation; and predicts policy impacts on preservation of investment and rail operations through a given analysis period. This process is then repeated for several policy options to compare relative costs and impacts, to identify any additional policies that should be investigated, and to decide on a single policy that will form the basis for programming and budgeting future activities. Some examples of typical (but simplified) simulation results follow.

The prediction of track performance for two different policies of maintenance and rehabilitation or renovation is shown in Figure 2. (Track condition is identified in the accompanying figures as Q, a composite measure of several categories of track geometric deviation from the norm. A Q-value of about 100 denotes track in very good condition, whereas Q in excess of 300 reflects poor condition.) Note that, for Policy 1, both the quality standard (Q) and the quality of routine maintenance (denoted by the Maintenance Quality Index or MQI) are higher than for Policy 2. As a result, the average system condition is also higher for Policy 1. Predictions of system performance (i.e., histories of track condition) are accomplished using deterioration functions and specifications of routine maintenance, rehabilitation, and renovation policy. Whereas Figure 2 shows only two policies as examples, several policies may be simulated for comparison.

Costs for each policy are computed. The resulting cost histories for the two policies in Figure 2 are shown schematically in Figure 3. Routine maintenance

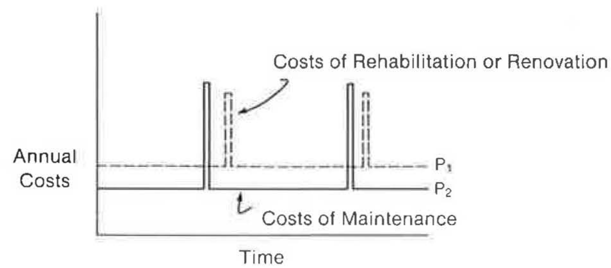


FIGURE 3 Agency costs for two track maintenance policies.

is costed on an annual basis with the better policy costing slightly more. Major repairs are represented by sudden peaks or spikes in the cost history. (Note that, under an inferior rehabilitation or renovation policy, both the magnitude of costs and the time intervals between successive performances of an activity may differ from those of better policies.)

Associated with the condition histories in Figure 2 are changes in impacts on the system, as shown in Figure 4. For simplicity a general benefits measure is shown. In reality several such functions could be developed for trip time and reliability, safety, comfort, and so on. The important thing to note is that the impact bears a direct relationship to the as-maintained condition of the track and is therefore sensitive to maintenance policy.

Results of the simulations in Figures 2-4 can be compared to identify the best policy, with or without budget constraints. To illustrate how this is done, assume that the benefits in Figure 4 can be reduced to monetary terms and thus compared directly to costs. Furthermore, it is assumed that instead of only two policies, as shown in Figures 2-4, several policies have been tested using the simulation model.

The results of each policy may be organized in terms of ascending costs to the transport agency. Because impacts are also in monetary terms (in this

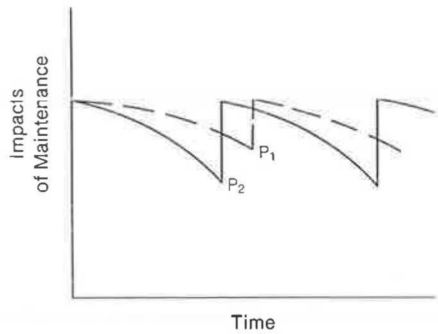


FIGURE 4 Impacts of two track maintenance policies.

example), they can be plotted on the same graph with costs for each policy. If maintenance policies are sensibly defined, more expensive policies (to the agency) should yield more advantageous impacts (i.e., greater reductions in costs associated, for example, with safety, travel time, or trip reliability), leading to the diagram that is Figure 5.

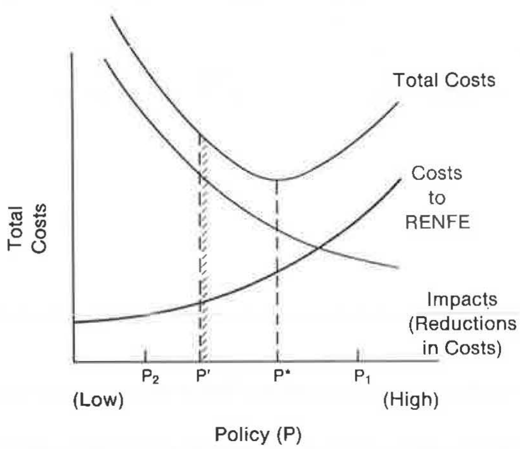


FIGURE 5 Optimization of track maintenance policies or strategies.

Identification of the most advantageous policy now becomes a question of minimizing total transport-related costs for the network configurations and traffic specified. In the absence of budget constraints, the appropriate policy is shown in Figure 5 as P*, because total costs (routine maintenance, rehabilitation, and renovation costs to the agency, plus costs associated with impacts of maintenance) are minimized at this point. If a budget constraint is imposed, the best policy that can be funded lies to the left of P*, perhaps at P'.

APPLICATIONS TO RENFE

The concepts illustrated in Figures 1-5 were applied to analyzing current condition and future maintenance policy on RENFE's track network, using the MIT Intercity Transportation Model. This work was done within the context of a multibillion dollar program proposed by RENFE, extending through the next decade, to upgrade its infrastructure and fleet and to expand and improve the level of transportation service provided.

Several categories of data were collected and analyzed to construct the case:

- Descriptions of the track network, including structural characteristics of each link, daily traffic levels, track classification, and so on;
- Deterioration relationships, developed from statistics on existing track condition provided by RENFE; and
- Routine maintenance, rehabilitation, and renovation policies to be tested, with unit costs to perform work under each policy.

The development of these data, and their relevance to the case, is explained in detail elsewhere (5).

Track Deterioration

In discussion with RENFE engineers, it was concluded that neither the North American models discussed earlier nor European practice reviewed by RENFE applied well to Spanish conditions governing track deterioration; therefore, deterioration models were estimated from data on track condition over time supplied by RENFE. These data were in the form of Q-values, representing composite indices of geometric deviations measured by an instrumental car in several dimensions (e.g., gauge, vertical profiles of both rails, superelevation, warp). At that time, RENFE had insufficient historical data from which to estimate deterioration curves; as an interim measure, cross-sectional data for each track class were used.

Examples of the analysis are shown in Figures 6 and 7 for high-standard and low-standard track, respectively. The change in track geometry (where greater deviations, implying worsening track condition, are denoted by higher Q-values) was found to be correlated with a statistic comprising the age of the track in years (A), traffic in gross tons per day (T), and weight of the steel rail in kilograms per meter (W). A function corresponding to the curves in Figures 6 and 7 was estimated for each of the track classes identified in Table 1 and separately for welded and for jointed rail. It was clearly stipulated, however, that these functions were preliminary and that further research by RENFE would be needed to develop more accurate models based on historical data and including the effects of routine maintenance policy.

Maintenance Policy

Four maintenance policies were tested (Table 2). These policies were sensitive to both Q-value (representing amounts of damage resulting in geometric deviations achieving a certain limit) and track age (to account for damage not correlated with geometric deviations). Note that different quality standards were defined not only for each policy but also for different classes of track. Policy 1 was regarded as the lowest standard and Policy 4 the highest.

The duration of this research project did not allow sufficient time to identify the relationship between maintenance policy and specific operational impacts (e.g., number of derailments, other safety considerations, line-haul travel times, etc.). In lieu of the types of impacts envisioned in Figure 4, the average measure of track condition throughout the network was predicted as an indication of the quality of service, safety, comfort, and speed that would be afforded passengers and freight. (The development of valid impact models relating maintenance policy to safety, operational efficiency, level of service provided, potential market share to be attracted and retained, and preservation of the track investment has been identified as a possible area of future research with RENFE.)

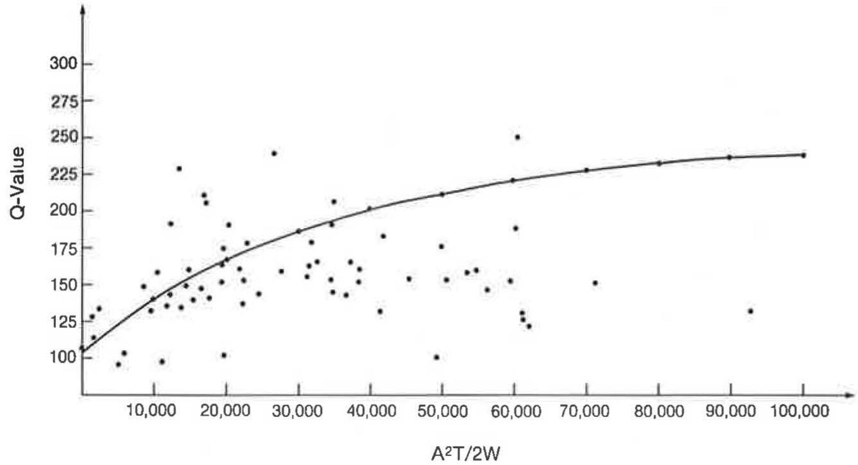


FIGURE 6 Deterioration relationship for arterial track (welded rail).

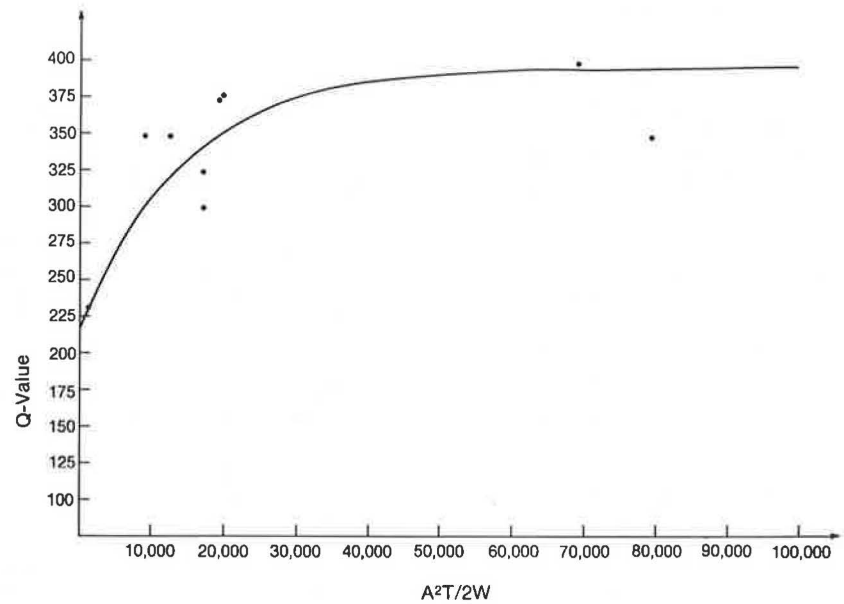


FIGURE 7 Deterioration relationship for secondary track (jointed rail).

TABLE 1 Operational Classes in RENFE Track System

Class	Kilometers	Explanation
Arterial	5534	Main or trunk lines
Principal	6426	Remaining lines serving primary traffic
Potential	2000	Backup system for primary network
Secondary	1137	Secondary lines

TABLE 2 Rehabilitation Thresholds for Four Track Policies

	Policy			
Track Class	1	2	3	4
Arterial	–	0 ≥ 225	0 ≥ 190	0 ≥ 160
Principal	–	0 ≥ 250	0 ≥ 210	0 ≥ 170
Potential	–	0 ≥ 250 or 300 Age ≥ 30	0 ≥ 210 or 275 Age ≥ 25	0 ≥ 275
Secondary	–	0 ≥ 338 Age ≥ 40	0 ≥ 300 Age ≥ 30	0 ≥ 300

RESULTS

Each of the maintenance policies defined earlier was simulated on the RENFE track network for the period 1982-1995. Results were computed in terms of both the costs of each policy to RENFE and the impacts, measured as track condition. This information was obtained (a) for each policy, (b) for each link, (c) for each class of track, (d) for each of seven maintenance zones, and (e) for the entire network. A summary of these results follows.

Figure 8 shows the distribution of costs for each policy for each track classification. The major portion of the budget for each policy is spent on the arterial and principal track classes. The allocation of a greater share of rehabilitation and renovation funds to the principal network does not imply that it is of greater importance. It is a reflection of the higher existing condition of the arterial network and the magnitude of expenditure required to restore the principal network to standard (i.e., to make up for the effects of deferred maintenance).

An analysis was also done of the cost of capital

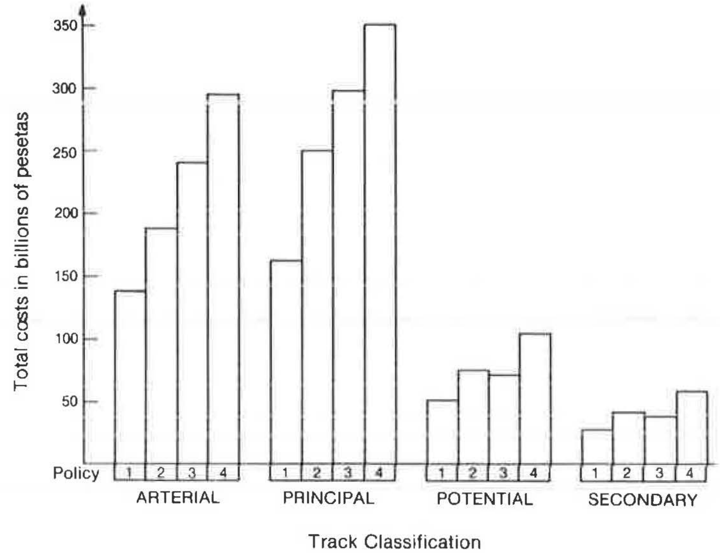


FIGURE 8 Costs versus track class.

repair of track (rehabilitation and renovation) as a percentage of total costs (capital repair plus routine maintenance). The highest such percentages were for Policy 4, which specified the highest standards for the RENFE system. Fifty-three percent of total costs were for rehabilitation and renovation under Policy 4, whereas only 31 percent of total costs were estimated for rehabilitation and renovation under Policy 2. Policy 1, which called for only routine maintenance and no track improvement, had a zero percentage.

Capital expenditures during the first year of the analysis were interpreted as the elimination of backlogged work. Taking the cost of work backlog by policy as a percentage of the overall cost of capital repair (renovation and rehabilitation), the percentage was found to be relatively low for arterial track, from about 20 to 40 percent depending on the policy chosen, indicating that arterial track is currently in good condition. (Track improvement here must be performed later in the 14-year analysis period as the track deteriorates.) Secondary track improvement, in contrast, was totally involved with the elimination of the backlog. When this work is completed, the track will deteriorate slowly due to light traffic volume. Only routine maintenance will be required later in the 14-year analysis period.

Figure 9 shows systemwide values of track condition (Q) averaged over the analysis period for each track class and policy tested. These results support the contention that a better maintenance policy substantially improves track condition. Policy 4 produces relative parity between the arterial and principal networks and provides approximately 12 000 km of premium track. Another significant factor is that Policy 4 is the only one of the four policies investigated that can transform the potential network into another 2 000 km of high-quality track to back up the other two primary networks.

Compared to Policy 4, Policy 3 results in a lower track condition on the arterial and principal networks. However, the harshest penalty in shifting from Policy 4 to Policy 3 is suffered by the potential network, which can no longer (from the standpoint of track condition) fulfill its intended role as a viable substitute for the arterial and principal networks.

A change from Policy 4 to Policy 3 will greatly

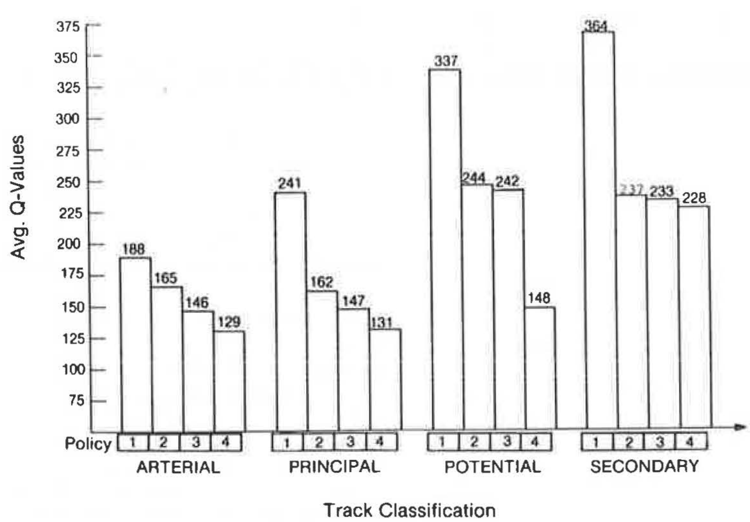


FIGURE 9 Average Q values versus track class.

affect the performance of the two major track classes, arterial and principal. The potential and secondary track classes remain relatively the same.

Policy 1 leaves the entire network in poor condition. The nonarterial classes each exhibit significant rates of deterioration. The only reason the arterial network does not itself exhibit worse conditions is, apparently, its current adherence to standards and that it has not suffered as much deferred maintenance as have the other networks in the past.

IMPLICATIONS FOR RENFE

As a result of this research a number of key findings emerged for the top management of RENFE:

1. For the best policy tested, the average Q-value is still being reduced with reasonable increments of cost. This means that it may be worthwhile to investigate even better maintenance policies.
2. As a measure of track condition, the Q-value is only an approximation of the benefits derived from maintenance. To determine the optimal policy requires at least an economic analysis of costs and benefits. Thus it will be necessary in the future to relate the Q-value of each link to factors such as operational efficiency and reliability, train safety, ride comfort, and so forth. These relationships form a potential topic for future research.
3. The elimination of backlogged work to bring the system up to standard will constitute a substantial percentage of all capital repairs to track during the next 15 years. This significant volume of work to correct deferred maintenance must therefore be accounted for in the definition and scheduling of track renewal projects.
4. The long-term benefits of the renewal program can be protected only if the rail system is maintained adequately and correctly in the future. Past policies of deferred maintenance cannot be continued.
5. The selection of maintenance policy also has strong implications for the operational roles that can be fulfilled by the different classes of track. In light of these considerations, RENFE managers are encouraged to define and test different maintenance policies in their planning of the long-term maintenance program and to include maintenance consideration in their capital budgeting.

CONCLUSION

This project has demonstrated the feasibility of applying demand-responsive concepts of maintenance planning and management to national transport networks. In addition to the case study presented in this paper, the approach has been used extensively in the analysis and evaluation of various surface transportation problems (1-6) and is, in general, applicable to other systems of infrastructure characterized by noncatastrophic failure. Furthermore, the approach addresses analytically many of the fundamental technical, economic, and financial differences between maintenance programs and new construction. This will take on added significance as several segments of the construction industry shift from the building of new facilities to the maintenance and preservation of existing mature infrastructure.

ACKNOWLEDGMENTS

The work reported in this paper was partly funded under a research contract between MIT and RENFE.

REFERENCES

1. J.J. Love III. A Demand-Responsive Approach to Railroad Track Maintenance Management. Master's thesis. Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Mass, June 1981.
2. M.J. Markow and B.D. Brademever. EAROMAR-2. Final Technical Report. FHWA, U.S. Department of Transportation, June 1981.
3. M.J. Markow. Economically Efficient Pavement Design and Rehabilitation Policies under High Traffic Volumes. Proc., Planning, Transportation, Research, Computation Summer Annual Meeting, Seminar J, University of Warwick, England, 1982.
4. M.J. Markow and T.K. Wong. Life-Cycle Highway Pavement Cost Allocation. In Transportation Research Record 900, TRB, National Research Council, Washington, D.C., 1982, pp. 1-9.
5. F. Moavenzadeh, M.J. Markow, et al. Analysis of Track Maintenance and Rehabilitation Policy. Center for Construction Research and Education, Massachusetts Institute of Technology, Cambridge, Mass., October 1982.
6. F. Moavenzadeh, M.J. Markow, et al. Intercity Transportation in Egypt: Summary Technical Report. Technology Adaptation Program, Massachusetts Institute of Technology, Cambridge, Mass., June 1983.
7. Rail Life. Report of Committee. AREA Bulletin, Nov.-Dec. 1957, Vol. 58.
8. J.C. Danzig, J.A. Russ, J.H. Williams, and W.W. Hay. Procedures for Analyzing the Economic Costs of Railroad Roadway for Pricing Purposes. On-Line Service, Inc., San Francisco, Calif., Vols. 1 and 2, Jan. 1976.
9. B. Hay, F. Martin, et al. First Progress Report: Evaluation of Rail Sections. Transportation Series 9. Civil Engineering Studies, University of Illinois, Urbana, June 1970.
10. A.M. Zarembski and R.A. Abott. Fatigue Analysis of Rail Subject to Traffic and Temperature Loading. Report R-315. Association of American Railroads Technical Center, Chicago, Ill., July 1978.
11. The Road Maintenance Cost Model. CIGGT Report 80-16. Canadian Institute of Guided Ground Transport, Queen's University, Kingston, Ontario, March 1981.
12. Preliminary Estimate of Deferred Maintenance in Track Materials for United States Class I Railroads. T.K. Dyer, Inc., Lexington, Mass., 1976.
13. J.E. Tyworth and A.J. Reinschmidt. Role of Safety and Train Speed in Track Maintenance Spending Decisions: A Case Analysis. Traffic Quarterly, Jan. 1981.
14. A.E. Fazio and R. Prybella. Development of Analytical Approach Track Maintenance Planning. In Transportation Research Record 744, TRB, National Research Council, Washington, D.C., 1980, pp. 46-52.
15. J.A. Bausch and R.R. Hooven. Deferred Maintenance: A Profit Maximizing Approach. Transportation Journal, Dec. 1977.

Publication of this paper sponsored by Committee on Railway Maintenance.

Dynamic Model for Scheduling Maintenance of Transportation Facilities

KIYOAKI MURAKAMI and MARK A. TURNQUIST

ABSTRACT

The optimum scheduling of maintenance for transportation facilities is addressed. The problem is described as a multiple-period resource allocation problem with constraints on both resource availability and state and decision variables. The problem is formulated as a nonlinear optimization problem and solved by using the generalized reduced gradient method. The model uses recursive formulas similar to those used in dynamic programming in order to calculate the partial derivatives of the objective function. The model is applied to an example based, in part, on actual data provided by the Japanese National Railways. Several tests are made to show the performance of the model, and the results are compared with those of two alternative solutions. The results show the usefulness of the model in a wide variety of applications and its superiority to the alternative solutions examined.

Maintenance of facilities is important in all transportation modes. Maintenance consumes a sizable, and increasing, fraction of total operating expenditures. For example, the 1982 Highway Cost Allocation Study (1) estimated 39 percent of all highway expenditures in 1985 would be for resurfacing, restoring, rehabilitating, and reconstructing (4R) work compared to 25 percent in 1978. In 1982 U.S. railroads spent $5.2 billion on maintenance of way and structures (19.7 percent of total operating costs), up from $3.5 billion (17.7 percent of total operating costs) 5 years earlier (2).

An important characteristic of many transportation facilities is that their condition declines nonlinearly over time. Figure 1 shows this point, using as an example the pavement present serviceability index (PSI) described by AASHTO (3). Because of this characteristic, the timing of maintenance expenditures is important. The marginal effectiveness of an additional dollar spent for maintaining a given facility depends greatly on the condition of that facility when the maintenance is performed. This characteristic is important not only for highway systems but for railroad and waterway facilities as well.

In this paper a general method for planning maintenance expenditures over multiple time periods is developed. The model includes the deterioration characteristics of the facilities under study, the dependence of maintenance effectiveness on current

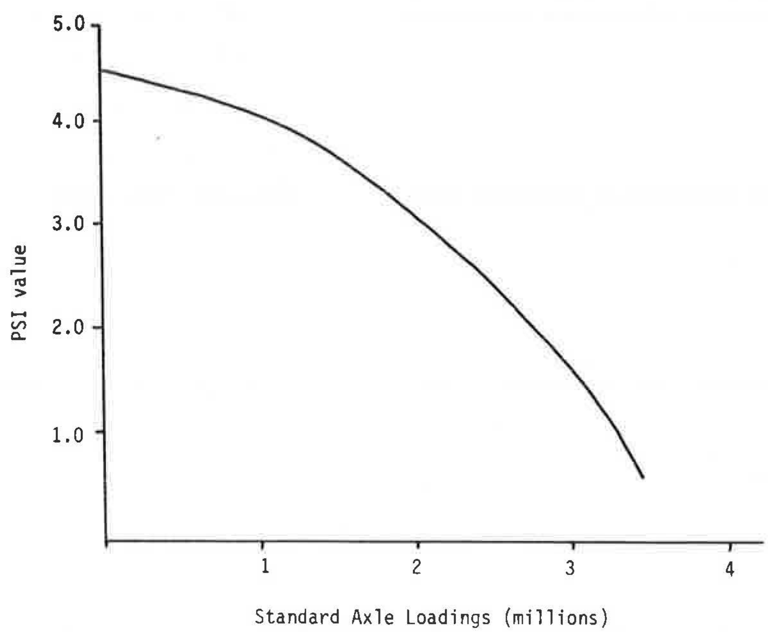


FIGURE 1 Decline of condition over time.

system state, and the potential variation in available maintenance resources across time periods.

The model is formulated as a nonlinear optimization problem that is readily solvable by standard nonlinear programming (NLP) computer codes. The model determines the allocation of available maintenance resources to each of several facilities (or sections of a single facility) in each time period, which maximizes an overall measure of system condition.

In the following section of the paper the structure of the model is described in greater detail. The third section presents an example application in railroad track maintenance and develops the mathematical formulation. The fourth section describes a solution method. Illustrative calculations for an example problem emphasize the relative advantages of the method. The last section presents conclusions and emphasizes the potential applicability of the model to highway and waterway problems, as well as to railroads.

MODEL DESCRIPTION

A basic premise of this model is that maintenance is a regular periodic activity and that the decisions that must be made are the amount of resources to be allocated to maintenance of a particular facility (or section of a facility) in each of a number of periods. Suppose that at the beginning of period j the state (or condition) of a specific section is given. In the absence of maintenance, that state will deteriorate during period j as a result of use and weather. The rate of deterioration is a basic element in this model.

This deterioration can be overcome by assigning maintenance resources to the section. The effectiveness of a particular set of resources in upgrading facility condition will depend on the current state of the facility, its structural characteristics (which determine "maintenance effectiveness"), and the available working time during the period (determined by traffic levels and climate).

All of these basic elements of the problem are combined in what is denoted the transition function, which specifies the facility state or condition at the beginning of period j+1 as a function of the state at the beginning of period j, the deterioration rate during period j, the maintenance resources allocated during period j, the available working hours during period j, and parameters determining the effectiveness of those resources. These elements are shown in Figure 2.

The assignment of maintenance resources is subject to the following constraints:

1. The available resources are limited;
2. The facility state for each section must stay above certain minimum standards; and
3. The resources to be assigned to a certain section may be bounded both from below and from above because of availability of facilities or operators or because of management policy.

One of the most important aspects of resource assignment is that it must be treated as a dynamic problem. In general, the three major parameters (deterioration rate, maintenance effectiveness, and available working hours) are not constant with location or time. For example, in track maintenance planning for railroads, available tamping hours vary from location to location and time to time because the actual work is performed during train intervals that are longer than a certain number of minutes. Therefore, in sections where the train frequency is high, the available tamping hours are extremely limited so the effectiveness of each tamping machine is reduced. Moreover, it is physically impossible to perform the work in heavy rain or when there is snow on the track. Because of the danger of rail buckle, the work is also restricted when the rail temperature is high.

In addition to the parameters, the constraints may be dependent on location or time. The bound on the facility state and the upper and lower bounds on the decision variables may differ from location to location, although they are usually considered constant with time. The total available resources also will frequently vary from one time period to another.

In this study the facilities are divided into several sections and the time into several periods such that the facility state within a section can be considered homogeneous. Thus the problem is how many resources should be assigned to each section in each period in order to achieve some overall objectives, subject to a set of constraints.

EXAMPLE APPLICATION

As an example of the general ideas expressed in the previous section, a specific application to railroad track maintenance planning is considered.

The track irregularity index (denoted the P-index) is used by Japanese National Railways (JNR) for the purpose of track state control. It shows the percentage of track irregularities within a certain

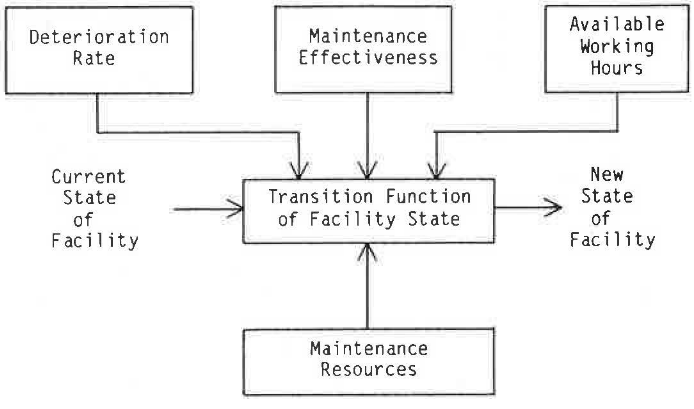


FIGURE 2 Elements of a maintenance model.

length of track (e.g., 10 m in the case of JNR) that exceed a predetermined critical value (e.g., 3 mm).

Each section of track has a length (ℓ_i) and a measure of importance, or weight (w_i), that generally reflects the volume and character of traffic over that section. Using these values, the objective of the problem may be formulated as follows:

$$\text{Minimize } T = \sum_{j=1}^{N} \sum_{i=1}^{n} w_i \ell_i P_{i,j+1} \tag{1}$$

where

T = total weighted P-index over n sections and N periods,
N = number of periods in the study period,
n = total number of track sections,
w_i = relative importance weight of section i,
ℓ_i = track length of section i,
$P_{i,j+1}$ = P-index of section i at the beginning of period j+1.

Changes in the P-index of section i are determined by usage of that section, its structural characteristics, climate, and maintenance resources allocated to it. In this case, the maintenance resource of interest is a tie-tamping machine that is used to level and align the track. To represent the dependence of the P-index on the decisions with respect to machine assignment, the transition function must be introduced. The transition function may be written as follows:

$$P_{i,j+1} = f_{ij}(P_{ij}, X_{ij} | d_{ij}, h_{ij}, e_{ij})$$

where

f_{ij} = transition function of track state for section i, period j;
X_{ij} = number of machines to be assigned to section i for period j;
d_{ij} = deterioration rate of section i during period j;
h_{ij} = available tamping hours in section i during period j; and
e_{ij} = an index that specifies the determinants of the tamping effectiveness of section i during period j.

The first two arguments are independent variables and the last three are parameters. For ease of notation, this function will be written as

$$P_{i,j+1} = f_{ij}(P_{ij}, X_{ij}) \quad \text{for } i = 1,\ldots,n;\ j = 1,\ldots,N \tag{2}$$

Note, however, that in general the deterioration rate (d_{ij}) may be dependent on the current state (P_{ij}) in which case this notation is a slight oversimplification. Also, there may be several different types of resources (such as machines with different production rates). In this case, X_{ij} to X_{ijk} could be generalized and the resources of class k assigned to section i for period j could be denoted. This makes the problem larger (more variables) but does not alter the basic structure of the problem or the solution method. Thus Expression 1 may be rewritten as

$$\min T = \sum_{j=1}^{N} \sum_{i=1}^{n} w_i \ell_i f_{ij}(P_{ij}, X_{ij}) \tag{3}$$

This minimization is subject to certain constraints:

1. The total number of machines is limited in each period:

$$\sum_{i=1}^{n} X_{ij} \le M_j \quad \text{for } j = 1,\ldots,N \tag{4}$$

where M_j is the total number of machines available in period j.

2. The track state of each section must satisfy a safety standard specified for the section:

$$P_{i,j} \le UBP_i \quad \text{for } i = 1,\ldots,n;\ j = 1,\ldots,N \tag{5}$$

where UBP_i is the upper bound P-index prescribed for section i.

3. There may be upper or lower bounds, or both, on the number of machines to be assigned:

$$LBX_i \le X_{ij} \le UBX_i \quad \text{for } i = 1,\ldots,n;\ j = 1,\ldots,N \tag{6}$$

4. Track states at the beginning of period 1 (P_{i1}) are given.

Physically, the decision variables X_{ij} must be integers. However, they will be defined as real in this study. This means that some of the machines can be transferred in the middle of a period, or some of the machines are shared among two or more sections. Whether or not this assumption is totally appropriate depends on the system under study. Nevertheless, they are defined as real for three reasons: First, this assumption provides a lower bound on the integer solution. Second, the real solution may be converted to an integer solution relatively easily. Third, the solution provided by the mathematical technique is not always the final decision because there are still some elements that cannot be formulated correctly or cannot be formulated at all. Human judgment is still important in the final decision.

Expressions 3-6 define an optimization problem that is, in general, nonlinear because of nonlinearities in the transition functions (f_{ij}).

The formulation presented here implies the need for a model of a multiple-period decision process. However, it may be asked why all periods need to be considered simultaneously instead of optimizing the problem period by period. Because the decision involves only maintenance, it might appear that the smaller the value of the objective function up to a certain period, the easier it would be for the value of the objective function to be minimized for the remaining periods. To see whether this is true, the following hypothesis will be examined. Let the system state in period j be the vector with n elements: $P_j = (P_{1j}, P_{2j}, \ldots, P_{nj})$.

Hypothesis: If there are two system states in period j, P_j^1 and P_j^2, such that the total weighted P-index up to period j is smaller when the system state is in P_j^1, then the minimum total weighted P-index through period j+1 derived from the system state P_j^1 is smaller than that from the P_j^2 when all the possible decisions in period j+1 are considered.

If this hypothesis holds, the problem can be decomposed into N (the number of periods) problems of one period each. The optimum solution (the best policy) would be obtained by the sequence of the N short-run optimum decisions. Unfortunately, this does not hold in general. Consider the following small example, in which two sections are considered. The track length, weight, upper bound P-index, initial P-index, deterioration rate for each section, and available tamping hours for each period and each section are as given in Table 1. Suppose that the

TABLE 1 List of Data and Parameters

	Section 1	Section 2
Track length (km)	100.0	100.0
Weight	1.0	1.0
Upper bound P-index	30.0	40.0
Initial P-index	20.0	30.0
Deterioration rate (d_{ij}) (P-index points/period)		
Period 1	10.0	10.0
Period 2	10.0	10.0
Tamping hours (h_{ij})		
Period 1	100.0	100.0
Period 2	50.0	100.0

total number of available machines is two for each period. Assume that both track structure and topographical conditions are the same for both sections and that there are no lower or upper bounds on the number of machines to be assigned.

The transition functions f_{ij} ($i = 1,2,$; $j = 1,2$) are given according to the two parameters, deterioration rate and available tamping hours. They are shown in Figure 3. The result of computation is given in Table 2. This example shows that while the total weighted P-index by decision 3 up to period 1 has the minimum value (5,300), the minimum total weighted P-index (11,900) derived from the best decision in the second period is greater than the total weighted P-index from decision 1 in period 1 (11,500) and from decision 2 (11,300). An intuitive interpretation of this example is that because the number of available tamping hours of section 2 in period 2 is smaller, it is advantageous to assign machines to section 2 in period 1 rather than in period 2.

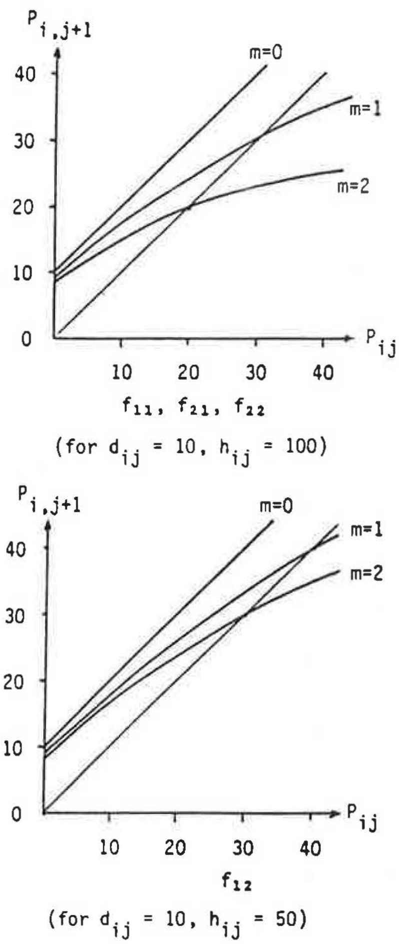


FIGURE 3 Example transition functions.

TABLE 2 Computed Results for Example Problem

Period 1 Decision[a]	Resulting Total Weighted P-Index	Period 2 Decision[a]	Resulting Total Weighted P-Index
1	6,000	1	Infeasible
		2	12,100
		3	11,500
2	5,400	1	12,000
		2	11,300
		3	Infeasible
3	5,300	1	11,900
		2	Infeasible
		3	Infeasible

[a] Decision 1 = assign both machines to Section 1, decision 2 = assign one machine to each section, and decision 3 = assign both machines to Section 2.

Hence the hypothesis is rejected. This simple example illustrates the need for a model of multiple periods, in which the dynamic aspects of the problem are represented properly.

SOLUTION OF THE PROBLEM

The model represented by Expressions 3-6 in the previous section is a nonlinear programming problem. In general, both the objective function and some of the constraints are nonlinear functions of the decision variables. In this section a procedure for obtaining solutions to the problem, using the generalized reduced gradient (GRG) method, is described.

The main idea of the GRG method is similar to that of the simplex method for linear programming. Using constraints, the vector of variables is partitioned into two subvectors: the vector of basic variables and the vector of nonbasic variables. The GRG method uses a gradient only in terms of the nonbasic vector, referred to as a reduced gradient, in order to improve the value of the objective function. For additional discussion of the GRG method, the interested reader is referred to Avriel (4).

To use the GRG method, it must be possible to compute partial derivatives of the objective function and all constraint expressions with respect to each of the decision variables. This can be done numerically, but, by using analytic expressions for the partial derivatives, both the speed and the accuracy of the algorithm can be improved. A discussion of the derivation of expressions for the required partial derivatives, using efficient recursive formulas, follows.

If $\{X_j\}$ is defined as the set of X_{ij} for $i = 1,\ldots,n$ (i.e., the allocations of machines to sections in period j), then $v_j(\{P_j\}, \{X_j\})$ can be defined as the value of the weighted P-index achieved in period j:

$$v_j(\{P_j\}, \{X_j\}) = \sum_{i=1}^{n} w_i \ell_i f_{ij}(P_{ij}, X_{ij}) \qquad (7)$$

Note that $v_j(\{P_j\}, \{X_j\})$ is simply a subset of the terms in the objective function 1, corresponding to a specific period j.

Also

$$T_j = \sum_{k=j}^{n} v_k(\{P_k\}, \{X_k\}) \qquad (8)$$

can be defined as the value of the objective function starting with period j and summed over the remaining periods, j, j+1,..., N. Note that a simple recursive equation can then be developed:

$$T_j = v(\{P_j\}, \{X_j\}) + T_{j+1} \quad (9)$$

The partial derivatives of the objective function (T) in Expression 3 with respect to X_{ij}'s can be computed as $\partial T/\partial X_{ij}$ (for i = 1,...,n, j = 1,...,N). Because a change in X_{ij} does not affect the value of T in stage 1 through state j-1, it is immediately apparent that the following is true:

$$\partial T/\partial X_{ij} = \partial T_j/\partial X_{ij}$$

Taking the partial derivative with respect to X_{ij} in Expression 9 yields

$$\partial T_j/X_{ij} = (\partial v_j/X_{ij}) + \sum_{\ell=1}^{n} (\partial T_{j+1}/P_{\ell,j+1}) (\partial P_{\ell,j+1}/X_{ij}) \quad (10)$$

Substitution for $\partial P_{\ell,j+1}/\partial X_{ij}$ in Expression 10 yields

$$\partial T_i/\partial X_{ij} = (\partial v_j/\partial X_{ij}) + \sum_{\ell=1}^{n} (\partial T_{j+1}/\partial P_{\ell,j+1}) (\partial f_{\ell j}/\partial X_{ij}) \quad (11)$$

Both $\partial v_j/\partial X_{ij}$ and $\partial f_{\ell j}/\partial X_{ij}$ can be computed; thus $\partial T_j/\partial X_{ij}$ can be computed if $\partial T_{j+1}/\partial P_{\ell,j+1}$ is known.

Expression 9 will again be used to compute this. Taking the partial derivatives with respect to $P_{\ell k}$ in Expression 9:

$$\partial T_j/\partial P_{\ell j} = (\partial v_j/\partial P_{\ell j}) + \sum_{k=1}^{n} (\partial T_{j+1}/\partial P_{k,j+1}) (\partial f_{kj}/\partial P_{\ell j}) \quad (12)$$

Equation 12 shows the rule to compute $\partial T/\partial P$ at a certain stage by using $\partial T/\partial P$ at the next stage. The boundary condition at state N+1 is

$$\partial T_{N+1}/\partial P_{i,N+1} = 0 \qquad \text{for } i = 1,\ldots,n \quad (13)$$

Then, starting from stage N+1 and computing backwards by using Equations 11 and 12, $\partial T_j/\partial X_{ij}$ can be computed for all i's and j's.

In the case of the present problem, Equations 11 and 12 can be simplified by using the independence of state variables with respect to space; that is,

$$\partial P_{\ell,j+1}/\partial X_{ij} = \begin{cases} \partial f_{\ell j}/\partial X_{ij} & (\text{if } i = \ell) \\ 0 & (\text{if } i \neq \ell) \end{cases} \quad (14)$$

$$\partial P_{\ell,j+1}/\partial P_{ij} = \begin{cases} \partial f_{\ell j}/\partial P_{ij} & (\text{if } i = \ell) \\ 0 & (\text{if } i \neq \ell) \end{cases} \quad (15)$$

Using Expressions 14 and 15 in Equations 11 and 12, respectively,

$$\partial T_j/\partial X_{ij} = (\partial v_j/\partial X_{ij}) + [(\partial T_{j+1}/\partial P_{i,j+1}) (\partial f_{ij}/\partial X_{ij})] \quad (16)$$

$$\partial T_j/\partial P_{ij} = (\partial v_j/\partial P_{ij}) + [(\partial T_{j+1}/\partial P_{i,j+1}) (\partial f_{ij}/\partial P_{ij})] \quad (17)$$

Next, the partial derivatives of the constraints are computed with respect to X_{ij} (for i = 1,...,n; j = 1,...,N). The first set of constraints, given by Expression 4, can be differentiated easily. For the second set of constraints, given by Expression 5,

$$P_{i,j} = f_{ij}(P_{i1}, X_{i1}, X_{i2},\ldots,X_{i,j-1}) \quad (18)$$

Equation 18 shows that $P_{i,j}$ is a function of the initial state (P_{i1}) and the sequence of the decisions that have been made in the section up to period j-1.

Taking partial derivatives of P_{ij} in Equation 18 with respect to $X_{k\ell}$ (k = 1,...,n and ℓ = 1,...,N) yields

$$\partial P_{ij}/\partial X_{k\ell} = \begin{cases} \sum_{n_1=1}^{n} \sum_{n_2=1}^{n} \ldots \sum_{n_m=1}^{n} [(\partial P_{ij}/\partial P_{n_1,j-1}) (\partial P_{n_1,j-1}/\partial P_{n_2,j-2})] \\ \ldots [(\partial P_{n_{m-1},\ell+2}/\partial P_{n_2,\ell+1}) (\partial P_{n_m,\ell+1}/\partial X_{k\ell})] \quad (\text{if } j \geq \ell) \\ 0 \quad (\text{otherwise}) \end{cases} \quad (19)$$

Again using Equations 14 and 15,

$$\partial P_{ij}/\partial X_{k\ell} = \begin{cases} [(\partial P_{ij}/\partial P_{i,j-1}) (\partial P_{i,j-1}/\partial P_{i,j-2})] \ldots [(\partial P_{i,\ell+2}/\partial P_{i,\ell+1}) \\ (\partial f_{i\ell}/\partial X_{i\ell})] \quad (\text{if } i = k \text{ and } j > \ell) \\ 0 \quad (\text{otherwise}) \end{cases} \quad (20)$$

Now all the partial derivatives required for the computation of the reduced gradient can be computed. Then, starting with an initial trial point and computing iteratively, the optimum solution can be obtained. The actual computations for the examples in this paper were done using the GRG2 package developed by Lasdon, Warren, and Ratnor (5).

ILLUSTRATIVE CALCULATIONS

Suppose that there are three track sections of interest with relative importance weights of 3.0, 2.0, and 1.0, respectively; these three sections may be assumed to be a main line, a sub-main line, and a local line, respectively. Considered will be the machine assignment planning for 1 year, divided into four periods; these four periods may be assumed to be four seasons (spring, summer, autumn, and winter).

Because of heavy snowfall during winter, available tamping hours are extremely limited in sections 2 and 3; on the other hand, tamping work is restricted during summer in section 1 because of the danger of rail buckling due to high temperature.

There are a total of 10 machines available per period. The management policy is to do tamping work no more than once within a period; that is, tamping distance executed in a certain section within a period is not to be more than the track length of the section.

Table 3 gives data for these three sections over the four periods. These data are based on values provided by JNR but do not necessarily correspond exactly with any specific sections of their system.

When X_{ij} machines are assigned to section i in period j, the distance tamped is computed as follows:

$$\ell_t = ch_{ij}X_{ij} \quad (21)$$

where

c = machine performance (distance/hour) and
h_{ij} = available tamping hours (hours/period).

The transition function $P_{i,j+1} = f_{ij}(P_{ij}, X_{ij})$ is given as

$$P_{i,j+1} = (ch_{ij}X_{ij}/\ell_i)\{-1 + [1 + 4a_i(P_{ij} + d_{ij}/2)]^{1/2}/2a_i\} + [1 - (ch_{ij}X_{ij}/\ell_i)]\,[P_{ij} + (d_{ij}/2)] + d_{ij}/2 \quad (22)$$

The derivation of this function is described in detail by Murakami (6).

TABLE 3 Data and Parameters for the Test Case

	Section		
	1	2	3
w_i	3.0	2.0	1.0
ℓ_i	225.3	241.4	217.3
a_i	0.05	0.025	0.015
UBP_i	35.0	37.0	39.0
$IP_i(=P_{i1})$	33.0	34.5	36.5
UBX_i		$\min(TM, \ell_i/c\ h_{ij})$	
LBX_i	0	0	0
Period			
1 d_{ij}	4.0	3.5	3.5
2	3.5	2.4	2.0
3	3.3	2.5	2.0
4	4.0	4.0	2.5
Avg	3.7	3.1	2.5
1 h_{ij}	50.0	60.0	60.0
2	40.0	80.0	100.0
3	50.0	60.0	80.0
4	40.0	30.0	20.0
Avg	45.0	57.5	65.0

Note: TM = total number of machines available per period, c = machine performance (= 0.32 km/hr), w_i = relative importance of section i, ℓ_i = track length of section i (km), a_i = tamping effect coefficient for section i, UBP_i = upper bound P-index for section i, $IP_i(=P_{i1})$ = initial P-index in section i, UBX_i/LBX_i = upper/lower bound number of machines to be assigned to section i, d_{ij} = deterioration rate of section i in period j, and h_{ij} = available tamping hours in section i during period j.

The optimal solution found for this problem is given in Table 4, and the best integer solution is given in Table 5. Note that the final P-index values for the integer and noninteger solutions are identical (to three significant figures). This indicates that the approximation involved in treating the problem with continuous variables is a reasonable one.

TABLE 4 Optimal Solution to Example Problem (machines assigned)

	Period			
Section	1	2	3	4
1	8.04	0.00	6.80	10.00
2	0.94	8.86	0.00	0.00
3	1.02	1.14	3.20	0.00

Note: Weighted average P-index after four periods = 29.1.

TABLE 5 Optimal Integer Solution to Example Problem (machines assigned)

	Period			
Section	1	2	3	4
1	7	0	7	10
2	1	9	0	0
3	2	1	3	0

Note: Weighted average P-index after four periods = 29.1.

As a further illustration of the effectiveness of the formulation described in this paper, the result of the optimization (as given in Table 4) has been compared with two alternative solutions:

- Assignment of a constant number of machines to each section for all periods (static solution) and
- Sequence of period-by-period optimal assignments (myopic solution).

Tables 6 and 7 show the results of these alternative solutions.

TABLE 6 Best Static Solution (machines assigned)

	Period			
Section	1	2	3	4
1	5	5	5	5
2	3	3	3	3
3	2	2	2	2

Note: Weighted average P-index after four periods = 33.0.

TABLE 7 Best Myopic Solution (machines assigned)

	Period			
Section	1	2	3	4
1	8.04	0.0	8.58	2.96
2	0.94	8.86	0.0	0.0
3	1.02	1.14	1.42	7.04

Noted: Weighted average P-index after four periods = 31.5.

The static solution yields a value of the weighted P-index at the end of the fourth period that is 10.8 percent worse than the optimal dynamic solution. The myopic solution is the same as the optimal solution in the first two periods and then differs over the last two periods. The myopic solution overallocates resources to section 1 in period 3, and then in period 4 it is forced to allocate most of the machines to section 3 in order to avoid violating the minimum standards in that section.

In contrast, the optimal solution looks at periods 3 and 4 together, recognizing that the machines can be used more effectively in section 1 in period 4 than in section 3. Thus it is seen that these two solutions are diverging, and, at the end of four periods, the myopic solution is about 6 percent worse than the optimal solution.

To illustrate the differences between the myopic period-by-period optimization and the dynamic optimization more completely, this test case was extended to 12 periods. The four-period seasonal cycle described in Table 3 was assumed to hold over two additional cycles, representing, in total, a 3-year planning horizon for the dynamic model.

Table 8 gives the machine assignments and resulting P-index values for the three sections over all 12 periods, allowing comparison of the myopic solution and the optimal solution. Note that the myopic solution follows a pattern of allocating just enough machine time to section 3 in each period to maintain the minimum required quality (maximum allowable P-index) with all remaining resources allocated to section 1 in the first, third, and fourth quarters of the year and to section 2 in the second quarter.

The optimal solution concentrates most maintenance in section 3 in two periods (2 and 7) and gen-

TABLE 8 Comparison of Myopic and Optimal Solutions

		Myopic Solution		Optimal Solution	
Period	Section	Machines Assigned	Resulting P-Index	Machines Assigned	Resulting P-Index
1	1	8.04	27.4	8.04	27.4
	2	0.94	37.0	0.94	37.0
	3	1.02	39.0	1.02	39.0
2	1	0	30.9	0	30.9
	2	8.86	26.0	3.25	34.5
	3	1.14	39.0	6.75	29.1
3	1	8.58	24.9	0	34.2
	2	0	28.5	10.0	26.6
	3	1.42	39.0	0	31.1
4	1	2.96	26.9	10.0	28.2
	2	0	32.5	0	30.6
	3	7.04	39.0	0	33.6
5	1	6.74	24.7	10.0	22.4
	2	0	36.0	0	34.1
	3	3.26	39.0	0	37.1
6	1	0	28.2	0	25.9
	2	8.86	25.5	9.38	23.8
	3	1.14	39.0	0.62	38.1
7	1	8.58	23.3	1.56	27.9
	2	0	28.0	0	26.3
	3	1.42	39.0	8.44	28.7
8	1	2.96	25.4	10.0	24.2
	2	0	32.0	0	30.3
	3	7.04	39.0	0	31.2
9	1	6.74	23.7	10.0	20.2
	2	0	34.5	0	33.8
	3	3.26	39.0	0	34.7
10	1	0	27.2	0	23.7
	2	8.86	24.7	9.38	23.6
	3	1.14	39.0	0.62	35.7
11	1	8.58	22.7	9.0	20.1
	2	0	27.2	0	26.1
	3	1.42	39.0	1.0	36.5
12	1	2.96	25.0	10.0	19.1
	2	0	31.2	0	30.1
	3	7.04	39.0	0	39.0

Note: Final average weighted P-index is 29.4 for the myopic solution and 26.1 for the optimal solution.

erally shows the pattern of using most resources in a single section in each period, rotating among the sections and keeping them all below the maximum allowable P-index. The weighted P-index achieved after 12 periods is 26.7, some 11 percent better than the myopic solution.

CONCLUSIONS

The formulation of a dynamic model for allocating maintenance resources across several facilities (or facility sections) over multiple time periods has been described. The basic structure of the model includes deterioration rates of the facilities over time, capability to represent variable effectiveness of maintenance resources in various sections over time, and variable amounts of total resources available over time.

The model has been formulated as a nonlinear optimization problem, and a solution method using the generalized reduced gradient approach has been described. This solution method has modest computation requirements and can be implemented using either commercially available software packages or custom software.

The formulation described here is based on using a single composite measure of facility condition (or state). The PSI rating for pavements and the P-index of track surface condition are examples of such measures. However, it must be recognized that single composite measures do not always represent all elements of a maintenance problem. They are simply an overall guide to relative facility condition.

In some applications, the state variables (facility condition measures) may not be independent from one section to another as assumed in the example in the previous section. The model can handle this complexity quite easily--the only change is that Equations 11, 12, and 19 are used in place of 16, 17, and 20 to compute the required derivatives.

An illustrative example based on data from Japanese National Railways has shown how the solution obtained from the optimization model compares to alternative, simpler, solutions: a static solution of doing the same thing each period and a myopic solution of optimizing one period at a time. The differences shown, even in this small example, emphasize the value of considering the dynamic elements of the problem.

The model described here is applicable to the analysis of maintenance planning problems in a variety of situations, including highway and waterway maintenance as well as railroad applications. For example, this model would be an effective complement to the Highway Condition Projection Model (HCPM) described by Hartgen (7) and used in New York State. That model is descriptive in nature, a "what-if" tool that predicts the effects of analyst-specified maintenance strategies. The model described here is an optimization model useful in designing those strategies.

In summary, this research provides an important new tool for the use of maintenance planners with the potential of helping make more efficient use of maintenance resources in many different applications.

REFERENCES

1. Final Report on the Federal Highway Cost Association Study. FHWA, U.S. Department of Transportation, 1982.
2. Railroad Facts, 1983 Edition. Association of American Railroads, Washington, D.C., 1983.
3. Interim Guide for Design of Pavement Structures. AASHTO, Washington, D.C., 1981.
4. M. Avriel. Nonlinear Programming: Analysis and Methods. Prentice-Hall, Englewood Cliffs, N.J., 1976.
5. L. Lasdon, A. Warren, and M. Ratnor. GRG2 User's Guide. Department of General Business, University of Texas, Austin, 1980.
6. K. Murakami. A Multiple Period Optimization Model for Scheduling Maintenance of Transportation Facilities. M.S. Thesis. School of Civil and Environmental Engineering, Cornell University, Ithaca, N.Y., 1984.
7. D. Hartgen. Long-Term Projection of Highway System Condition. In Transportation Research Record 940, TRB, National Research Council, Washington, D.C., 1984, pp. 8-15.

Publication of this paper sponsored by Committee on Railway Maintenance.

Development of U.S. Army Railroad Track Maintenance Management System (RAILER)

M. Y. SHAHIN

ABSTRACT

U.S. Army Facilities Engineers are responsible for the maintenance of more than 3,000 mi of railroad track. The track is dispersed in small lots and is analogous to industrial rather than commercial trackage. At present, there is no standard method for gathering track inventory and condition data and no standard method of determining the track's condition. In this paper an overview of the proposed U.S. Army Railroad Maintenance Management System (RAILER) and track evaluation concepts are presented. RAILER is to consist of subsystems for network definition, data collection including condition survey, data storage and retrieval, network data analysis, and project data analysis. The development of these subsystems is highly dependent on the track condition evaluation procedures that are used. Two major evaluation categories have been identified: track structural condition and track operational condition. Recommended procedures for performing the evaluation are presented.

U.S. Army Facilities Engineers are responsible for the maintenance of more than 3,000 mi of railroad track. The track is dispersed in small lots and is analogous to industrial rather than commercial trackage. Because the track does not compete well for maintenance funding, much of the maintenance and repair it needs has been deferred. If this trend continues, some of the track may deteriorate to a point where it could no longer support its mobilization mission. At present, there is no standard method for gathering track inventory and condition data and no standard method of determining the track's condition.

An intensive search was performed (1) to define maintenance problems and available maintenance management systems. The U.S. Army Major Command (MACOM) engineers, Strategic Mobility personnel, and track maintenance personnel were interviewed to obtain input about Army track maintenance problems. Twenty-seven large operating railroad firms, 14 firms operating short-line railroad tracks, the Federal Railroad Administration, and private railroad consultants were surveyed to determine what system, if any, they used for managing their track maintenance operations.

The search showed that there is no complete track maintenance management system that could be readily adapted to Army use; it also showed that the most efficient way of providing a track maintenance management system is to design one specifically tailored to the Army system of operation.

The U.S. Army Construction Engineering Research Laboratory (USA-CERL) was tasked with the development of such a system. USA-CERL has successfully developed an Army maintenance management system for pavements (PAVER) (2,3). It was decided that the generic concepts of maintenance management developed for PAVER be adopted, but that special attention be given to the technological differences between pavements and railroads.

In this paper an overview of the proposed U.S. Army RAILER and track evaluation concepts and recommended procedures is presented.

OVERVIEW OF THE RAILER SYSTEM

The basic subsystems of any facility maintenance management system consist of network definition, data collection including condition survey, data storage and retrieval, network data analysis, and project data analysis. The relationship among these subsystems is shown in Figure 1. The development of each of these subsystems for a given facility should be technologically based and cannot be blindly adapted from another facility's management system. The following is a brief description of each subsystem as envisioned for the RAILER system.

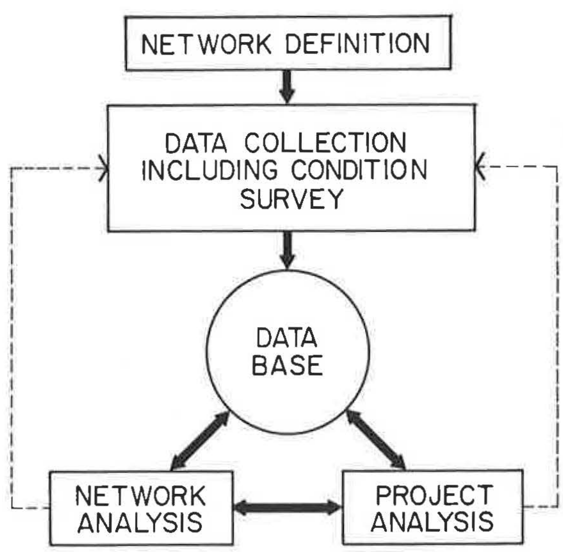


FIGURE 1 Generic facility maintenance management system.

Network Definition

Before anything can be managed, what is to be managed must be defined; railroad tracks are no exception. A track network could be defined in terms of mileposts, switch locations, grade crossings, and structures such as bridges. The network should be divided into sections that are uniform in construction and condition and that are subjected to similar traffic loadings. These sections represent the smallest management units in terms of major rehabilitation needs.

Data Collection

Data collection includes physical inventory of the track structure as well as the condition of each of its components. Data should also be gathered on the traffic that uses the track, including load intensity and number of repetitions. The details of the data collection and condition survey are determined on the basis of the needs of the condition evaluation and analysis techniques developed for both the project- and the network-level analysis. Exceeding these needs will not be cost-effective and could lead to failure of the entire management system.

Data Base

A data base can be manual (file cabinet) or automated (computer). In light of cost, expediency, and convenience, a computer system is much preferred. An inefficient or ill-designed data base will undoubtedly result in an inefficient overall system. The objective of the data base is to provide expedient and friendly data storage and retrieval. In the last 2 years, many "Data Base Manager" computer software packages have become available for microcomputers with features that were only available on large-frame computers before. Some of these packages offer excellent support for screen-formatted data entry, report generation using conversational language, and the ability to interface engineering analysis programs with the data base.

Network Analysis

The objectives of network analysis include budget planning, budget optimization, project identification and priority listing, and network inspection scheduling. To avoid duplication of efforts, it is best to coordinate the network inspection with the agency railroad track maintenance standards inspection. The development of the network analysis programs is a difficult task that requires the cooperation and involvement of the system's ultimate users.

Project Analysis

The primary objective of project-level analysis is to determine the best track rehabilitation alternative. This requires more detailed condition data than are needed for network analysis. One of the major factors in selecting the best rehabilitation alternative is life-cycle costing. Emphasis should be placed not only on initial rehabilitation cost but also on future maintenance costs associated with the alternative. An economic analysis procedure that can be used has been developed as part of the PAVER system (2,3). Guidelines for providing track information inputs to the analysis procedure still need to be developed.

The development of these subsystems is highly dependent on the track condition evaluation procedures used. This subject is addressed in the following section.

RAILROAD TRACK CONDITION EVALUATION CONCEPTS AND RECOMMENDED PROCEDURES

The railroad track and its support system have four main components: the rail and ties, which make up the basic track structure; and the ballast and subgrade, which make up the foundation. In addition to providing direct support for train traffic loads, each component, from the rail on down, distributes these wheel loads over an increasingly large area, thus minimizing pressure on the subgrade. For the track and foundation (track system) to withstand the loads imposed by train traffic, each component must have sufficient structural integrity to carry out its dual role of load support and load distribution.

In addition to providing structural support, the track system must also maintain track geometry: proper position and alignment of the two rails. A deterioration of either track strength or track geometry can make track unsuitable for service.

At present, there is no standardized method for evaluating track condition as a whole--a method that considers both track strength and operational condition. Definitions of major track condition categories and how to evaluate them follow.

- Structural Condition. This is a measure of the load-carrying capacity of the track structure. It takes into account both the magnitude of the wheel loads and the number of load repetitions the track system can handle before failure occurs. Structural condition is evaluated using a track modeling technique and knowledge of the strength of the individual components of the track system, including rail, ties, ballast, and subgrade.
- Operational Condition. This is a measure of maintenance and rehabilitation needs as well as of the safety of the track system. Operational condition is evaluated on the basis of the condition of the individual components of the track structure as well as on the geometric condition of the track.
- Evaluation
  1. Rail condition
     a. Internal defects (such as cracks)
     b. External defects (such as wear)
  2. Tie condition
     a. Number of defective ties
     b. Severity of defects
     c. Arrangement of defective ties
  3. Ballast and subgrade
     a. Degree of fouling and degradation
     b. Drainage condition
  4. Track geometry condition
     a. Gauge
     b. Crosslevel
     c. Profile
     d. Alignment

To achieve the objectives of this work, three subcontracts were awarded to three recognized consultants in the area of railroad engineering to perform preliminary studies and provide necessary background in the following three areas:

1. Tie condition evaluation (4),
2. Track geometry condition evaluation (4), and
3. Ballast and subgrade evaluation as well as overall track strength condition evaluation (4).

Meetings were also held with the U.S. Army Pavement and Railroad Maintenance Committee, which includes

Army railroad Major Command engineers. During these meetings, various track condition evaluation concepts were presented and critiqued. The recommendations presented in this paper are based on the consultants' reports, the author's views, and input from the Army Committee. Two major evaluation categories have been identified: track structural condition and track operational condition. Recommendations for evaluating each of these categories are presented in the following sections.

Structural Condition Evaluation

Two evaluation procedures are recommended. One procedure is to be approximate, but simple to use by Facilities Engineers without need for sophisticated testing or analysis. The other procedure is to provide in-depth analysis as a basis for determining cost-effective maintenance and repair alternatives. Both procedures are based, in principle, on mechanistic analysis of track behavior and on relating that behavior to track performance. The inputs for the approximate procedure, however, do not have to be based on direct measurements of material properties.

The overall structural evaluation of the track is a function of its components, including subgrade, ballast, tie, and rail, as well as the load to which the track is subjected. The effect of each track component on track structural condition indicators was studied. The study was performed in cooperation with Marshall Thompson of the University of Illinois, using the ILLI-TRACK computer system (5).

Typical results obtained using ILLI-TRACK are given in Table 1. By using subgrade strength, ballast thickness, tie spacing, rail size, and load as inputs to ILLI-TRACK, the following track structural condition indicators can be determined:

1. Tie reaction in kips [ballast bearing pressure can be computed as tie reaction divided by tie width multiplied by effective length (24 in.)],
2. Tie deflection,
3. Subgrade stress ratio (stress/strength), and
4. Rail bending stress.

TABLE 1 ILLI-TRACK Response Summary (4)

Ballast Thickness (in.)	Subgrade[a]	Maximum Tie Reaction (kips)	Tie Δ (mils)	Subgrade Stress (psi) $\sigma 1$	$\sigma 3$	σD	Relative Subgrade σ^b (%)
12	Medium	18.5	153	26.7	16.9	9.8	43
18	Medium	20.7	138	24.3	15.4	8.9	39
24	Medium	22.4	133	22.1	14.2	7.9	34
12	Soft	16.6	299	23.6	16.6	7.0	54
18	Soft	19.3	266	21.7	15.3	6.4	49
24	Soft	21.5	248	20.0	14.2	5.8	45
12	Very soft	15.4	493	–[c]	–	–	100
18	Very soft	17.4	479	17.1	13.0	4.1	66
24	Very soft	20.2	438	15.6	11.9	3.7	60
6	Medium	16.7	159	30.0	18.8	11.2	49
6	Soft	15.1	319	26.0	18.4	7.6	59
6	Very soft	18.2	553	34.2	28.0	6.2	100

[a]Subgrade strengths: medium – q_u = 23 psi; soft – q_u = 13 psi; very soft – q_u = 6.2 psi.
[b]Relative subgrade stress = 100 (subgrade stress/subgrade strength) = 100 (σ_D/q_u).
[c]Extensive subgrade and ballast failure. Stress data are not valid.

Each of these indicators can be used to determine the adequacy or inadequacy of the track to carry a specific load for a given number of repetitions.

For the approximate procedure, it is recommended that a parameter study be performed using ILLI-TRACK (or a similar mechanistic model) from which a nomograph, such as that shown in Figure 2, can be constructed. For the approximate procedure, a methodology needs to be developed for which all the inputs can be obtained by the Facilities Engineer's staff without the need for sophisticated testing or analysis. If track strength is determined to be inadequate or questionable, the detailed structural evaluation can be requested.

For the in-depth evaluation procedure, similar nomographs could be developed, but the input would require more direct measurements and the output should be in terms of allowable specific load value and associated number of repetitions. An alternate method for the in-depth evaluation is the direct use of the selected mechanistic model on a project-by-project basis.

A limited parameter study was performed using

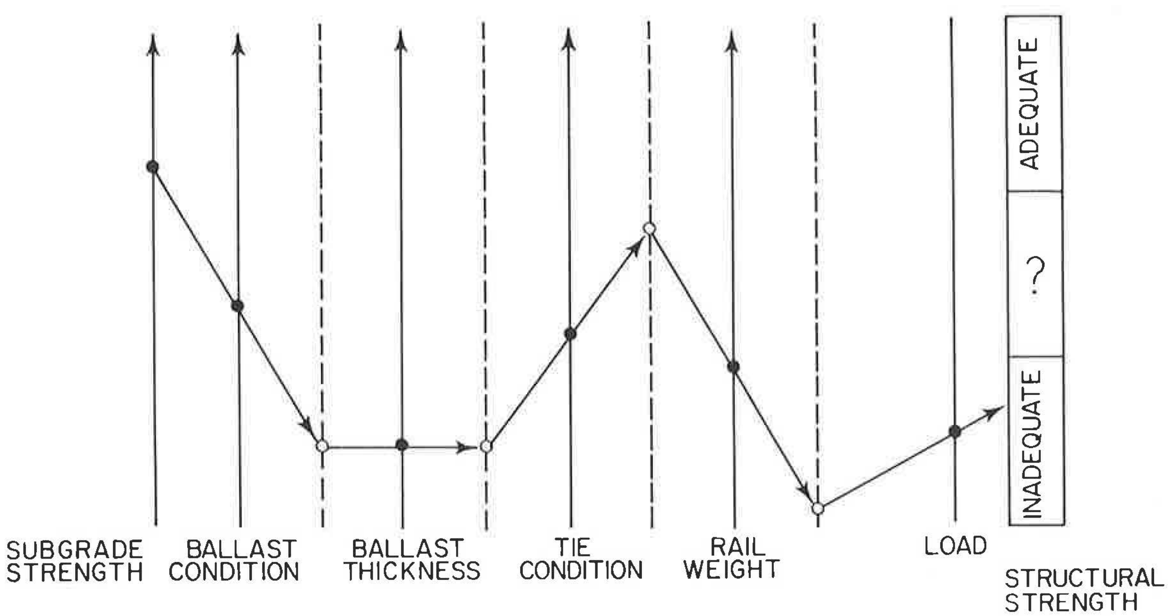


FIGURE 2 Conceptual nomograph recommended for U.S. Army railroad track structural evaluation.

TABLE 2 ILLI-TRACK Comparisons (5)

Rail Size	Tie Spacing (in.)	Maximum Tie Reaction (kips)	Tie Δ (mils)	Subgrade Stress (psi) σ1	σ3	σD	Relative Subgrade σ^a (%)	Rail Bending Stress (ksi)
132	20	16.6	299	23.6	16.6	7.0	54	19.3
132	40	30.3	456	25.8	16.4	9.4	72	22.1
90	20	19.5	310	24.4	16.5	7.9	61	28.9
90	40	31.7	459	26.4	16.3	10.1	78	32.8
60	20	26.0	331	26.3	16.8	9.5	73	43.0

Note: Ballast thickness = 12 in.; soft subgrade; and tie size = 9 in. wide by 7 in. thick.

[a] Subgrade strength: soft – q_u = 13 psi; relative subgrade stress = 100 (subgrade stress/subgrade strength) = 100 (σ_D/q_u).

ILLI-TRACK to illustrate the relative effect of rail size and tie spacing on the structural strength indicators. The results of the study are given in Table 2 and Figures 3-6. It should be noted that a tie spacing of 40 in. was used to simulate a case in which every other tie is bad, although further parameter studies should consider various arrangements of bad ties. However, from this limited study, the importance of both tie condition and rail size cannot be overemphasized. The study was performed assuming a soft subgrade. The significance of the subgrade class is clearly demonstrated in Table 1.

Operational Condition Evaluation

In many cases the operational failure of a track system may be caused by the gradual deterioration of the system components, localized defects, or improper track geometry. These conditions are often correctable, before failure, with an effective maintenance management system. They represent the operational condition of the track.

The operational condition of a track segment can be determined by measuring and inspecting

1. Rail condition,
2. Tie condition,
3. Ballast drainage condition,
4. Subgrade drainage condition, and
5. Track geometry.

Rail Condition

Rail condition is determined by inspecting both internal and external defects.

Internal defects must be detected with special equipment. These defects are potentially hazardous because they cannot be seen, and there are often no external indications of their presence. If not detected, an internal defect can grow until a rail break occurs.

External defects include rail head wear (both top and side), corrosion, cracks, and various surface defects. Sometimes these occur in combination with internal defects.

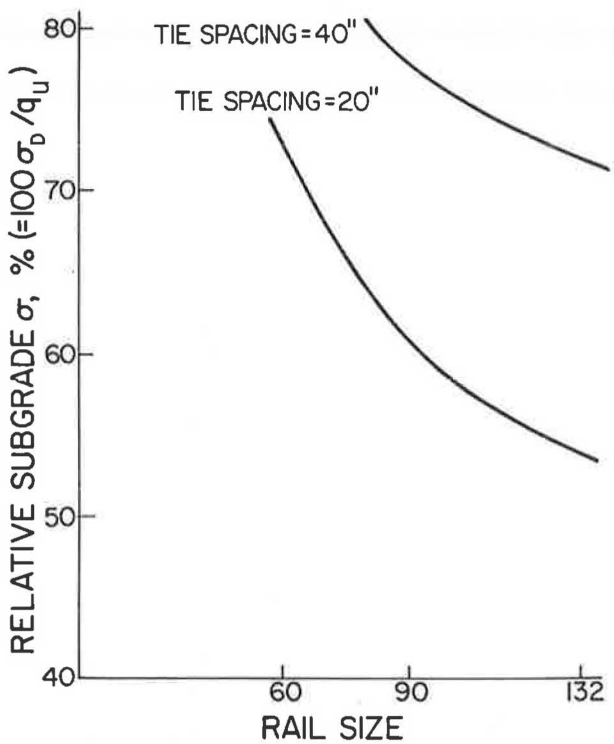


FIGURE 3 Effect of tie spacing and rail size on relative subgrade stress.

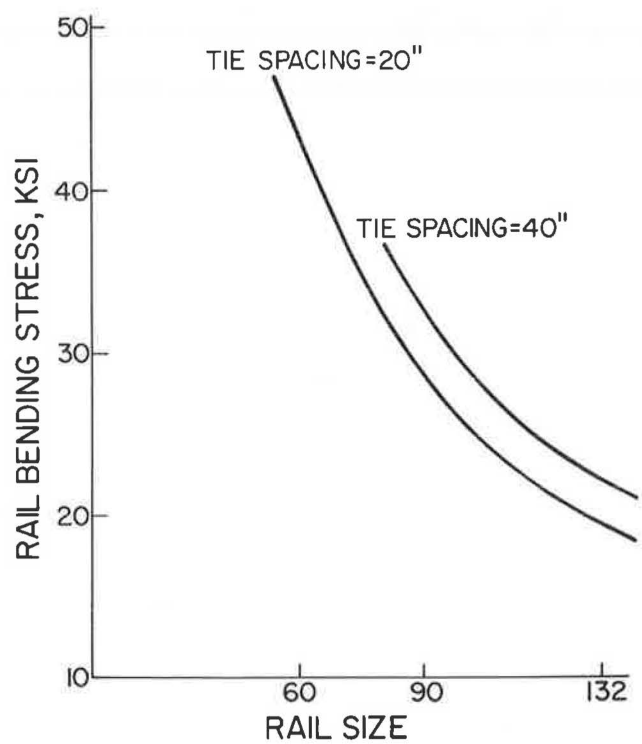


FIGURE 4 Effect of tie spacing and rail size on rail bonding stress.

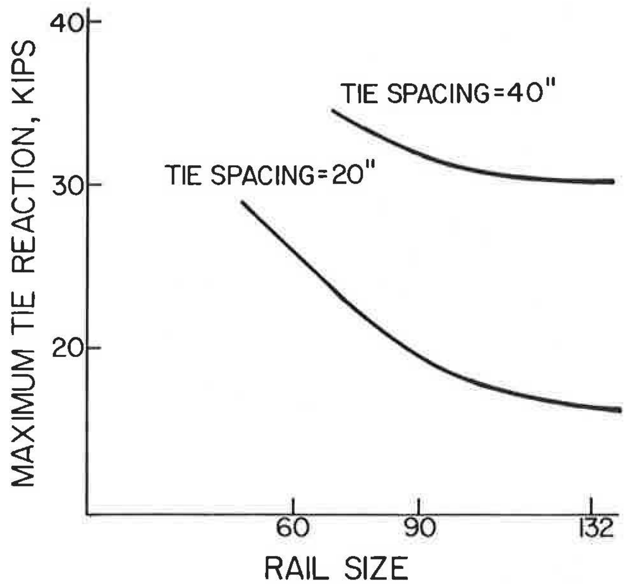


FIGURE 5 Effect of tie spacing and rail size on maximum tie reaction.

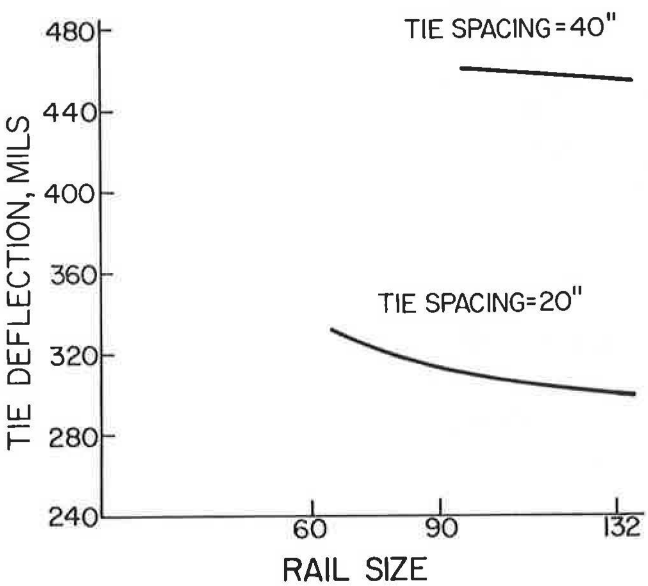


FIGURE 6 Effect of tie spacing and rail size on tie deflection.

In most cases, rail defects are corrected by replacing the defective section.

Tie Condition

Tie condition may be determined by combining a visual inspection procedure with calculations to produce a tie condition index. This index would indicate the overall condition of ties in a given track segment.

Tie defects may cause the loss of both vertical and lateral rail support, which leads to poor track geometry and loss of track load-carrying capacity. The need for tie replacement in a given track segment is determined by the number of defective ties, the arrangement of defective ties (i.e., the presence of consecutive defective ties), and the severity of the defects. Figure 7 shows typical tie defects.

Ballast Drainage Condition

The ballast section holds the track in vertical and horizontal alignment. To properly perform this function, ballast must drain well and not suffer significant particle degradation.

Visual inspection can be used to detect drainage problems and ballast deterioration. When such conditions exist, remedial action is required.

Subgrade Drainage Condition

Like the ballast section, the subgrade provides vertical track support. To do this, the subgrade must have sufficient strength and be properly drained. Visual inspection can be used to detect drainage problems and signs of subgrade failure.

Track Geometry

Track geometry is usually described by four parameters: gauge, crosslevel, alignment, and profile (Figure 8). For military railroads (or any other low-speed trackage), the most important geometric parameters are gauge and crosslevel. Ultimately, all track system components hold the rails in proper position; therefore, a track geometry defect usually indicates the failure of one or more of these components.

Track geometry measurements can be made with simple devices on unloaded track. However, without full-scale loading, the results may not accurately reflect what the position of the rails would be when subjected to actual train traffic. This is especially the case for track that is of light construction, is rarely used, has had minimal maintenance over the years, or has structural defects. A significant portion of Army track falls into at least one of these categories. Therefore, it was recommended that track geometry measurements be taken with engine- or car-mounted devices.

Geometry-measuring devices that mount on the engine or car are currently available. They allow measurements to be made and recorded continuously along the track under full-scale loads. In addition, this equipment is easily installed and removed.

SUMMARY

The concepts developed for the U.S. Army Railroad Track Maintenance Management System (RAILER) have been presented. RAILER will consist of subsystems for network definition, data collection including condition survey, data storage and retrieval, network data analysis, and project data analysis. Two major track evaluation categories have been identified: track structural condition and track operational condition. Current work efforts include the development of these elements.

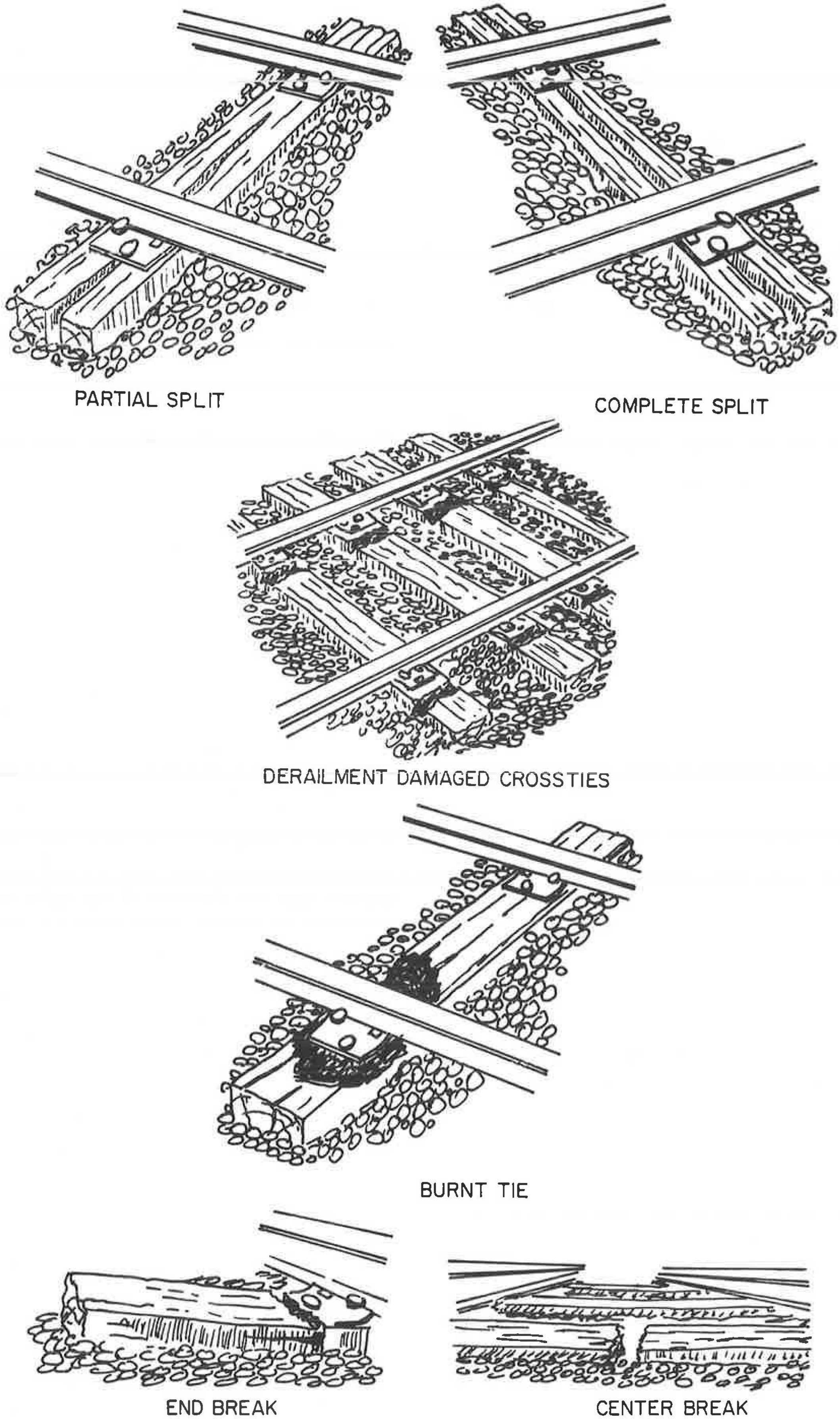


FIGURE 7 Typical bad ties.

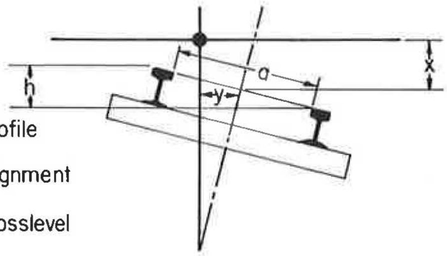


FIGURE 8 Track geometry measurements (4).

ACKNOWLEDGMENT

This research was performed under the sponsorship of the Office of the Assistant Chief of Engineers, U.S. Army Corps of Engineers. The views of the author do not purport to reflect the position of the Department of the Army or the Department of Defense.

REFERENCES

1. S.C. Solverson, M.Y. Shahin, and D.R. Burns. Development of a Railroad Track Maintenance Management System for Army Installations: Initial Decision Report. Technical Report M-85/04/A149491. U.S. Army Construction Engineering Research Laboratory, Champaign, Ill., 1984.
2. M.Y. Shahin and S.D. Kohn. Pavement Maintenance Management for Roads and Parking Lots. Technical Report M-294/ADA110296. U.S. Army Construction Engineering Research Laboratory, Champaign, Ill., and U.S. Air Force Engineering and Services Center, 1981.
3. M.Y. Shahin and S.D. Kohn. Overview of the PAVER Pavement Management System and Economic Analysis of Field Implementing the PAVER Management System. Technical Manuscript M-310/ADA116311. U.S. Army Construction Engineering Research Laboratory, Champaign, Ill., 1982.
4. M.Y. Shahin. Development of the U.S. Army Railroad Track Maintenance Management System (RAILER). Technical Report. U.S. Army Construction Engineering Research Laboratory, Champaign, Ill., forthcoming.
5. S.D. Tayabji and M.R. Thompson. Program ILLI-TRACK--A Finite Element Analysis of Conventional Railway Support System--User's Manual and Program Listing. FRA, U.S. Department of Transportation, undated.

Publication of this paper sponsored by Committee on Railway Maintenance

Use of Reinforced Earth® for Retained Embankments in Railroad Applications

VICTOR ELIAS and PHILIP D. EGAN

ABSTRACT

Since its introduction in the United States in 1969, Reinforced Earth® technology has been used in a variety of civil engineering projects, especially in the field of highway construction. In the last 5 years several Reinforced Earth structures have been built to provide direct support for railroad tracks, including retained fills, bridge abutments, and a foundation slab. Although they offer the economies normally associated with Reinforced Earth construction, these structures have also been designed for the vibratory loads and higher live loads associated with railroads. Design methods have been developed, on the basis of both research and experience, to produce structural designs that are responsive to these loading requirements. The behavior and failure mechanism of Reinforced Earth structures are discussed in this paper. The normal design procedure is described, followed by a detailed discussion of the dynamic effects of rail loading. Substantial research and field measurement of these effects have led to modification of the normal design procedure for the case of railroad-supporting structures. Three completed projects the design of which incorporates the results of this research are described. The economic impact of these modified design procedures has been found to be minimal.

Reinforced Earth is a composite material formed by the association of linear metallic reinforcements and granular soil. The principle behind Reinforced Earth is analogous to that of reinforced concrete: the mechanical properties of a basic material, in this case soil, are improved by reinforcing it in the direction parallel to the orientation of its greatest tensile strains. In a Reinforced Earth mass, frictional interaction between the soil and the reinforcements allows the soil, which can withstand only compressive and shear stresses, to transfer tensile stresses to the reinforcements.

BEHAVIOR OF REINFORCED EARTH

As a result of the continuous research and development that have been conducted on this construction system, the behavior of Reinforced Earth, under a variety of loading conditions, can be predicted with great accuracy. Research conducted to date includes finite element analyses, bidimensional and tridimensional model tests, and instrumentation of actual structures, all under a variety of loading conditions. Begun in 1969, this research effort has led to the emergence of a well-understood and widely accepted behavioral mode and failure mechanism. The key elements are summarized in the following subsections.

Failure Mechanism

The potential failure surface for Reinforced Earth, shown in Figure 1, is a potential failure surface for the reinforcements and a potential sliding surface for the soil. The shape of this potential failure surface is distinctive. Bidimensional and tridimensional model tests loaded to failure have shown that, unlike the classical Coulomb or Rankine failure surface, the failure surface of a Reinforced Earth wall is curvilinear. This difference in the shape of the potential failure surface is due primarily to the mechanics of movement. The reinforcements in the soil-reinforcement matrix restrain horizontal expansion within the active zone, especially in the upper portion of the wall.

As is shown in Figure 1, the potential failure surface with the soil-reinforcement matrix delineates two zones within the mass:

- An active zone, between the facing and the potential failure surface, where the shear stresses are directed toward the facing and
- A resistant zone, beyond the potential failure surface, where the shear stresses are directed away from the facing.

The two possible modes of internal failure that are associated with the curvilinear failure surface are as follows:

- Excessive movement of the reinforcing strips when the soil-reinforcement friction interaction is insufficient to resist the mobilized pullout force. To preclude such an occurrence, it must be ensured that the reinforcements have sufficient adherence length within the resistant zone. This is called the "effective length."
- Tensile rupture in the reinforcements. To prevent such failure, the reinforcements must be of high-strength materials that have a well-known stress-strain behavior and that do not yield excessively during loading.

State of Stress

The essential calculation in designing Reinforced Earth structures is the calculation determining the lateral or tensile stresses that must be resisted by the reinforcements. Overstress could promote tensile failure of the reinforcements, which in turn would produce a catastrophic structural collapse.

Schlosser (1) reports a summary of the variation of earth pressure with depth calculated from strain gauge measurements made at seven actual structures. The data, shown in Figure 2, consistently demonstrate that the horizontal earth pressure at the top of the structure approaches the at-rest condition (Ko); the reinforcements minimize horizontal deformations particularly well near the top of a structure. The horizontal earth pressure decreases with depth, approaching a constant value that is slightly less than the active earth pressure condition (Ka) at a depth of approximately 6 m (20 ft).

Adherence Between the Soil and Reinforcements

As observed during the bidimensional model testing, one mode of failure of a Reinforced Earth structure is slippage between the soil and the reinforcements, caused by lack of adherence. To adequately design the reinforcements for sufficient adherence, it is

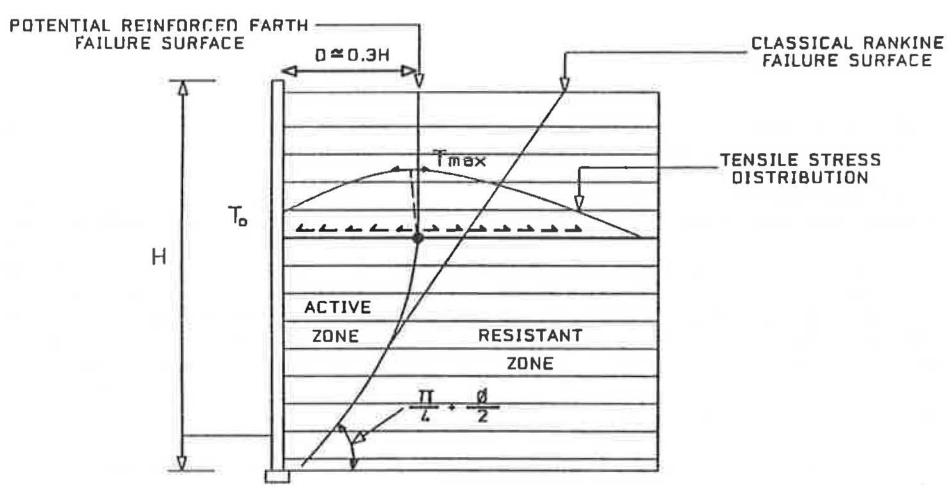


FIGURE 1 Tensile forces distributed along reinforcements.

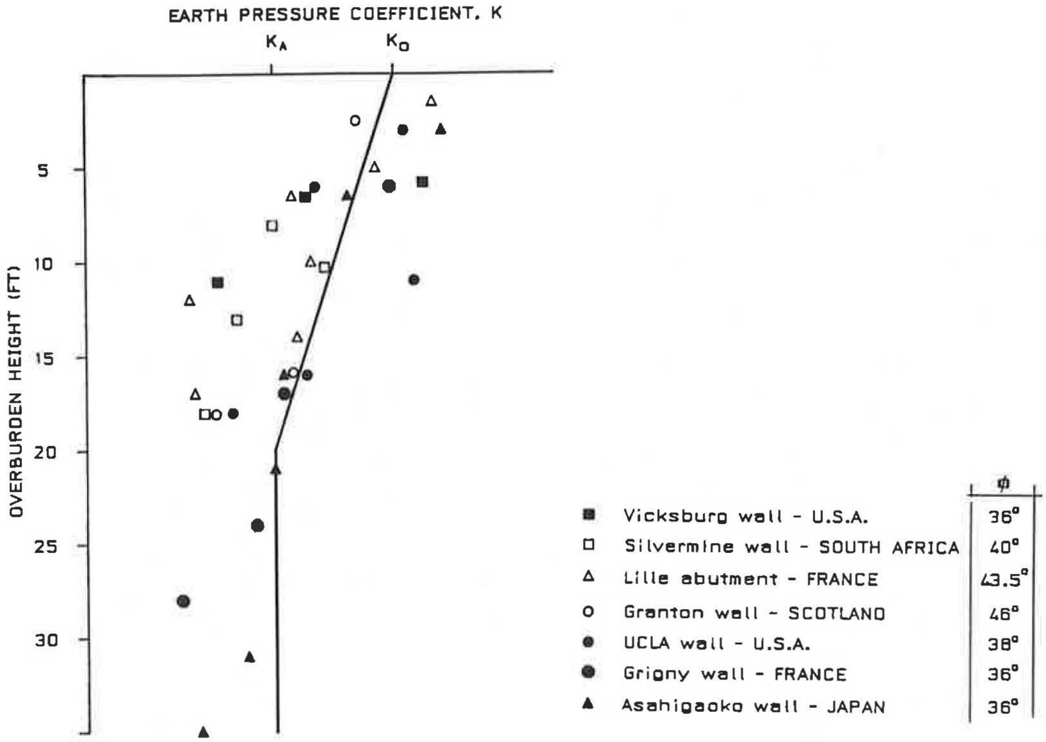


FIGURE 2 Variation of K from instrumentation of actual structures.

necessary to predict the friction mobilized along the soil-reinforcement interfaces. This sliding shear resistance between the soil and the reinforcements has been the subject of numerous research studies (2). Figure 3 shows typical values of the soil-reinforcement friction coefficient, also known as the apparent coefficient of friction (f*), based on pullout tests using ribbed reinforcements. Examination of the field results shows a clear trend toward high values of the apparent coefficient of friction (f*) near the top of the structure. Like the coefficient of earth pressure, the frictional coefficient decreases with depth until it reaches a constant value at approximately 6 m (20 ft).

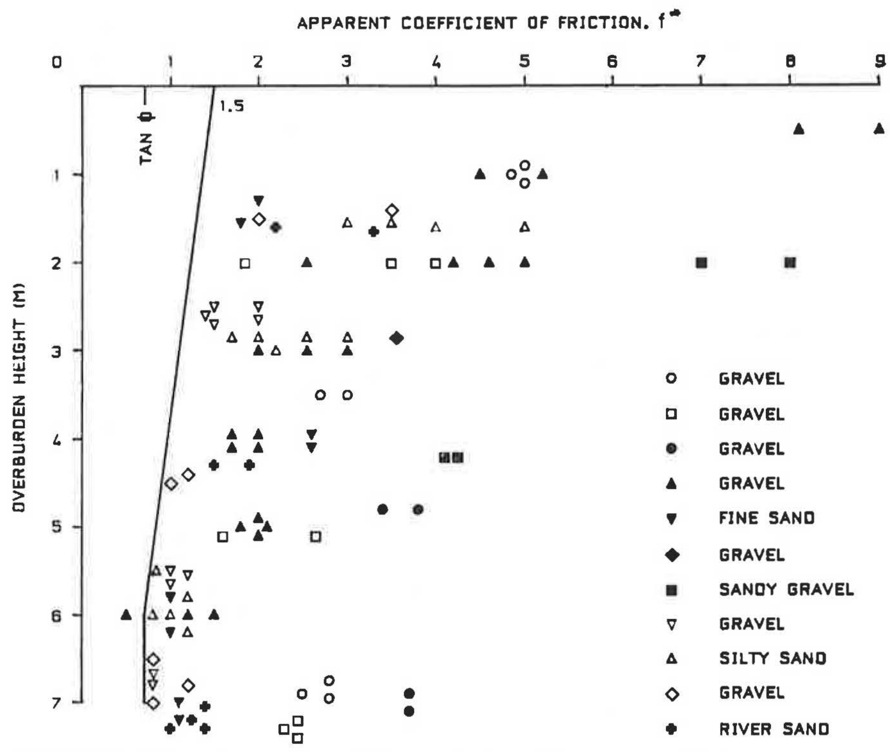
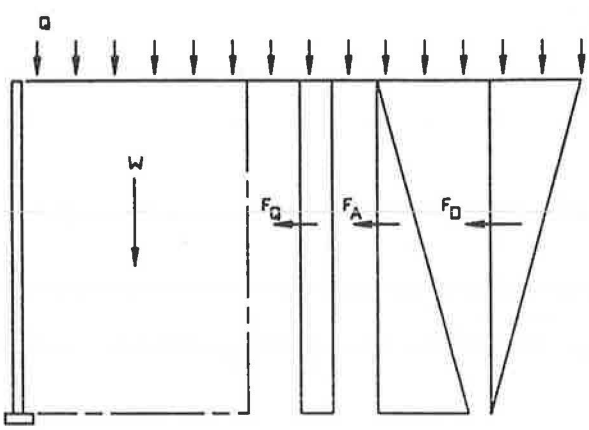


FIGURE 3 Values of apparent coefficient of friction (f*) from pullout tests.

DESIGN OF REINFORCED EARTH STRUCTURES

The state-of-the-art design procedure for Reinforced Earth structures consists of a local equilibrium analysis between the facing elements and the reinforcements. The analysis is predicated on the assumption that the soil-reinforcement matrix is in a state of limit equilibrium and that the principal directions of the stresses within the mass are vertical and horizontal. The reinforced volume is treated as a composite material that displays both frictional strength due to the granular backfill and pseudocohesional strength due to the restraint imparted by the reinforcements. The Reinforced Earth mass can thus be analyzed as a single gravity unit.

The Reinforced Earth gravity mass is designed to withstand the horizontal earth pressures normally associated with earth retaining structures including forces developed by seismic or dynamic events. These latter forces have been quantified on the basis of extensive research, and a predictive model has been developed to determine the additional tensile forces associated with these events. This pseudostatic method of dynamic analysis is based on data from model tests and the results from an instrumented test wall constructed at UCLA (3) to determine the dynamic response of Reinforced Earth walls to harmonic and random excitations. The application of these dynamic loads for external and internal stability considerations is shown in Figure 4.



WHERE

W - WEIGHT OF REINFORCED EARTH VOLUME

Q - SURCHARGE LOAD

F_Q - HORIZONTAL LOAD DUE TO SURCHARGE

F_A - HORIZONTAL LOAD DUE TO ACTIVE EARTH PRESSURE

F_D - HORIZONTAL LOAD DUE TO DYNAMIC EARTH PRESSURE

FIGURE 4 Application of loads associated with railroad structures.

For all Reinforced Earth structures, the external stability is checked using conventional methods before internal stability design. The checks on external stability include sliding and overturning calculations. For conventional Reinforced Earth structures that are not subject to large surcharge loads, the reinforcing strip length is generally 70 percent of the wall height. In addition, the contact bearing pressure of the Reinforced Earth mass on its foundation soil is determined using Meyerhof's suggested distribution. It should be noted that, when the adequacy of the foundation is checked for bearing capacity, the allowable bearing pressure determined is based on a reasonable factor of safety for a flexible structure applied to the ultimate bearing capacity of the foundation; an allowable bearing pressure limited by differential settlement considerations is not applicable to Reinforced Earth because the structure can settle differentially without structural distress. This method of analysis is contrary to that for a conventional rigid retaining structure, which is sensitive to differential settlement. For this latter type of structure, allowable bearing pressure is governed by the permissible differential settlement.

The internal stability design of the Reinforced Earth wall consists of checking each level of reinforcements for the two possible modes of failure; namely, lack of adherence or tensile rupture. This is done by developing the appropriate horizontal pressure distribution and designing the reinforcing strips with sufficient cross-sectional area and effective length to resist the horizontal loads with an adequate factor of safety. The horizontal pressure distribution is developed on the basis of the known variation of earth pressure with depth within the Reinforced Earth mass. The variation used in design is shown in Figure 2. The variation of the apparent coefficient of friction (f*), which is used in adherence calculations, has been defined through numerous laboratory and field pullout tests. Although the overall phenomenon is complex, in general the density and dilatancy of the granular backfill and the nature of the strip surface are the predominant factors. The variation used in the design of Reinforced Earth structures is shown in Figure 3.

DYNAMIC EFFECTS INDUCED BY RAIL LOADING

The design of a rail-traffic support structure requires an analysis of both the dynamic pressures associated with rail vibration and the effect of these pressures on the soil-structure interaction. For a Reinforced Earth structure, this analysis necessitates a knowledge of the variation in dynamic accelerations with depth and the effect of such accelerations on the apparent coefficient of friction (f*).

The variation of ground acceleration with depth is a complex analytical phenomenon. However, through field instrumentation, the level of vertical acceleration within a railroad embankment can be measured. One such study was performed by the French National Railroad (SNCF) (4). The purpose of the study was to define both the level and the limit of significant vibrations.

The SNCF study consisted of instrumenting a railroad embankment on the heavily traveled line between Paris and Marseilles. In cross section, the instrumented railroad embankment measured 8 m (26.25 ft) across the top and 34 m (111.55 ft) across the toe. The height of the embankment is approximately 8 m (26.25 ft). The line carries 22 passenger trains and 60 freight trains per day, typically traveling at 40 to 50 mph for freight traffic and 60 to 70 mph for passenger traffic. As shown in Figure 5, vertical acceleration measurements were made at four locations along the embankment. Three such cross sections were instrumented along a 192-m (630-ft) length of embankment. The average values of the vertical accelerations at each accelerometer location along the embankment cross section are shown in Figure 5. The measurements indicate that vertical accelerations decrease from 1.2 *g* at the top of the ballast to 0.28 *g* at a lateral location 4 m (13.1 ft) from the centerline of track, at the top of the

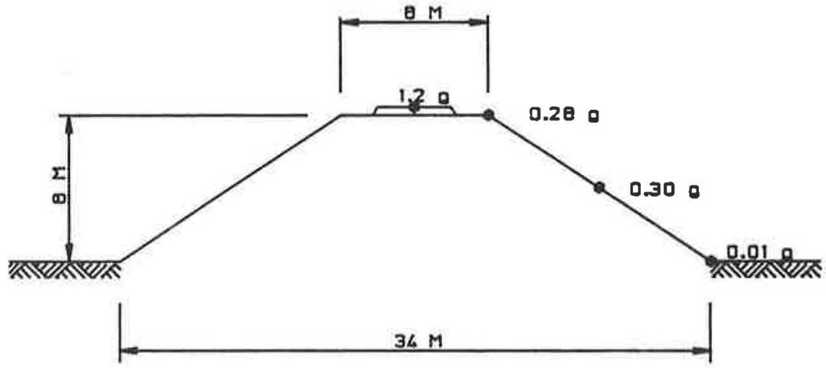


FIGURE 5 Vertical accelerations at the surface of an instrumented SNCF railroad embankment.

embankment. It appears that the magnitude of vertical accelerations decreases with depth, but the rate of decrease with depth through the embankment is not well documented.

The research data obtained in the SNCF study are not directly applicable to Reinforced Earth because only vertical accelerations were measured. Research conducted at UCLA (5) on dynamic behavior of Reinforced Earth conclusively demonstrated that vertical accelerations alone have no practical effect on the design and performance of the system; only horizontal accelerations produce significant increases in tensile forces and displacements within the structure. Therefore it is necessary to estimate the level of horizontal accelerations consistent with the vertical accelerations measured by SNCF.

In a recently completed research program, the Reinforced Earth Company duplicated levels of vertical acceleration and frequency of vibration consistent with the SNCF tests. At the same time the corresponding horizontal accelerations were measured to determine their influence on the apparent coefficient of friction (f*).

The frequency of vibrations transferred through the soil of the instrumented railroad embankment and measured by SNCF are generally in the 40- to 80-Hz band. Reinforced Earth Company's tests duplicated these frequencies and measured vertical accelerations using a 10-ton vibratory roller atop a Reinforced Earth test wall. The vibratory roller was placed in a concrete cradle close to the rear face of the precast concrete panels. During the test the measured frequency of vibration ranged from 50 to 70 Hz. When accelerometers were used at five vertical locations along the wall face, the peak vertical accelerations were generally in the range of 0.4 to 0.6 g. The horizontal accelerations measured during the dynamic testing were in the range of 0.1 to 0.4 g. From these data it appears reasonable to estimate that the anticipated horizontal accelerations are approximately 2/3 of the vertical.

The effect of the horizontal accelerations on the apparent coefficient of friction (f*) is shown in Figure 6. The percentage decrease in the apparent coefficient of friction was determined by measuring the pullout resistance of reinforcing strips in the test wall both before and during dynamic loading. Both the SNCF and the Reinforced Earth Company data strongly suggest that horizontal acceleration levels are relatively constant, in the range of 0.25 to 0.4 g, in the upper 3 to 4 m. Below this critical depth the level of horizontal acceleration decreases rapidly. Therefore, in the upper 3 to 4 m, the apparent coefficient of friction (f*) should be expected to decrease approximately 10 to 20 percent from those values normally associated with static loading. This influence, although only transient, must be considered during the design of a Reinforced Earth structure that supports rail traffic.

The dynamic pressures associated with rail traffic must be determined and their effects included in the analysis. Two dynamic forces, inertial and dynamic earth pressures, are considered in the design of earth retaining structures subject to earthquake-induced vibrations. The inertial force develops because of the acceleration of the active zone of the soil-reinforcement mass and occurs even if backfill beyond the reinforcing strips is not present. This force, internally generated, causes additional stresses that must be resisted by the tensile reinforcements. The second force, a dynamic active earth pressure, is caused by a potential sliding wedge of soil behind the wall. This latter force, affecting overall stability only, is not likely to develop because of the low total dynamic energy produced by rail traffic vibrations and their limited area of application on top of the retained embankment.

The inertial pressure can be estimated from data developed during the original model tests performed at UCLA (6) and subsequent Japanese prototype tests (7). These tests have shown that dynamic horizontal pressures increase with increasing input acceleration. Furthermore, for vibrations at or near the resonant frequency of the structure, a significant magnification of the input acceleration, and thus of the associated dynamic horizontal pressure, occurs.

On the basis of the aforementioned model and prototype testing, it is known that the resonant frequency of Reinforced Earth structures is in the range of 5 to 15 Hz. This range is well below the normal frequencies associated with vibrations induced by rail traffic. Therefore the design acceleration (E) should not be subject to magnification. The relationship between the design acceleration (E) and the input acceleration (a/g), based on the calculation method proposed by Richardson and Lee (6), has been developed during prototype testing by the Japanese. The relationship between design acceleration and input acceleration is shown in Figure 7. These data are valid for all frequencies except those near resonance.

Dynamic inertial pressures are greatest at the subballast level and decrease with depth, becoming insignificant at depths greater than 6 to 8 m because they are proportional to the level of horizontal accelerations. From analysis of the data developed at the UCLA test site it appears that an inverted triangular pressure distribution is an appropriate model for calculation. The overall dynamic inertial force (Fd) can be approximated as the product of the weight of the active zone of the Reinforced Earth wall times the design horizontal accel-

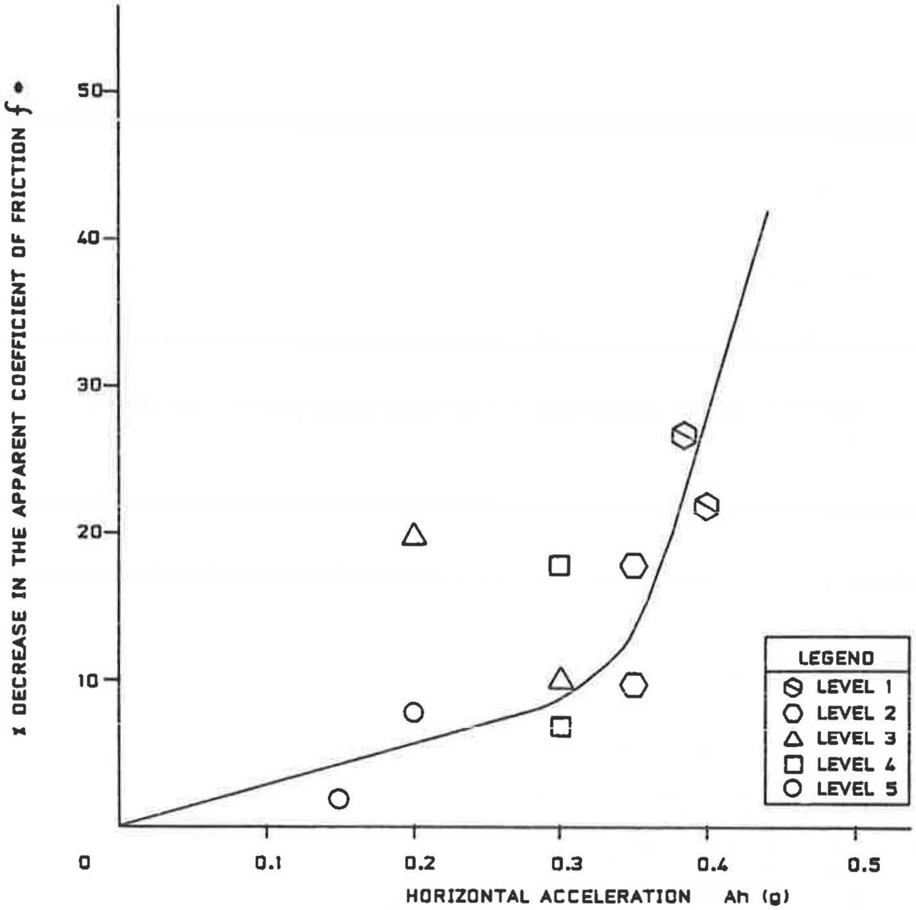


FIGURE 6 Effect of horizontal acceleration on apparent coefficient of friction.

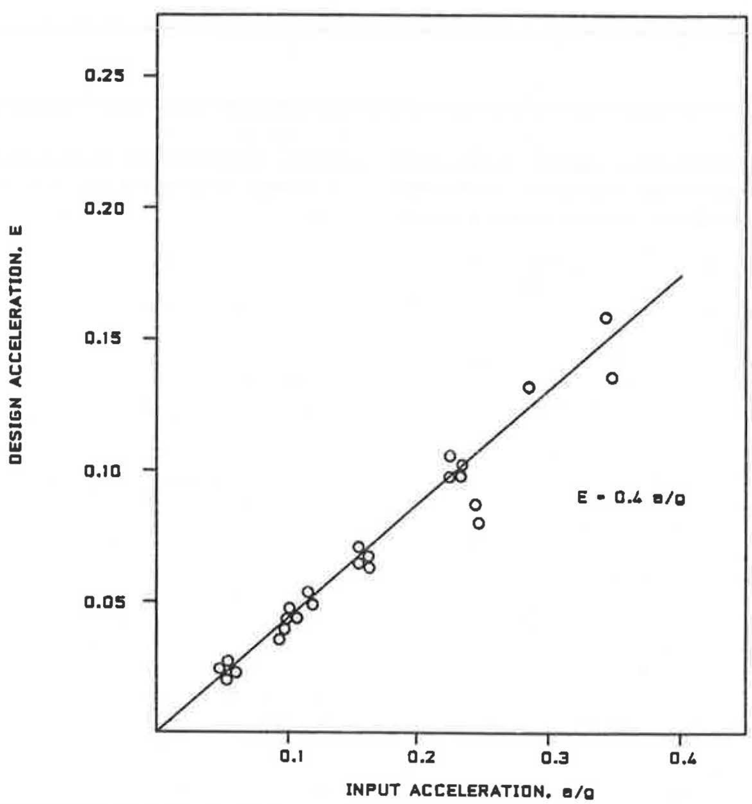


FIGURE 7 Relationship between design acceleration and input acceleration for Reinforced Earth walls.

eration (E) within the mass. The application of the dynamic inertial force is shown in Figure 4.

REINFORCED EARTH WALLS SUPPORTING RAIL TRAFFIC

To date, several Reinforced Earth walls have been constructed in the United States to support railroad traffic. These structures have been retaining walls, bridge abutments, or foundation slabs that distribute heavy rail loadings to soft foundations. The following discussions illustrate the use of Reinforced Earth for support of railroad traffic.

Clinchfield Railroad Project

During the spring of 1979 heavy rain caused the failure of an earth embankment supporting a line of the Clinchfield Railroad Company on Blue Ridge Mountain in North Carolina (8). After alternative repair techniques had been evaluated, a Reinforced Earth slide buttress was chosen. It was the first Reinforced Earth structure to be built to support live railroad loading in the United States.

The Clinchfield line, which descends the south side of the Blue Ridge Mountain, was constructed in 1906-1908 and is an engineering masterpiece, even by today's standards. From an elevation of 2,628 ft at Altapass, North Carolina, the line is on a 1.2 percent compensated grade, unbroken for 20 mi. The maximum curve is 8 degrees. The development loops require 18 mi of track to cover a straight-line distance of 2.3 mi. There are 17 tunnels in one particular 11-mi stretch.

The many high embankment fills were constructed by methods typical of that era, such as dumping without compaction. The native soil with which the fills were built is a micaceous, sandy clay, which becomes very unstable when wet. Each year from late January through April, several of the high fills on the Blue Ridge begin to settle, and it is an annual ritual during this period to patrol the mountain and correct the line and grade on the fills.

The month of March 1979 was very wet. There were continuous rains for several days, climaxed by a heavy deluge. The fill located at milepost 189.3 first slid away from the heads of the ties. Restoration work was under way, with on-track ditching equipment, when the heavy deluge hit the area. The entire fill then gave way, sliding down the mountain and leaving the track structure hanging in the air like a suspension bridge. Studies were then made of several alternative solutions. Reinforced Earth was selected because of (a) permanence, (b) short installation time, and (c) favorable costs compared to other methods. The design for repairing the line was a reconstructed earth fill buttressed by a Reinforced Earth wall. Figure 8 shows a typical section.

Work began in late October 1979 with excavation of the site and placement of filter media, subsurface drains, and the leveling pad. The first wall panels were placed on December 5, and the wall was completed on December 23. Construction required no special skills and was accomplished by a Clinchfield bridge and building force that had no previous experience with this type of construction. They were assisted by a local contractor.

The roadway fill above the Reinforced Earth portion was built with selected material and compacted. Filter fabric was laid on the finished grade with no subballast. The track was surfaced on an average of 8 in. of ballast and opened for traffic on January 7.

Joseph C. McNeil Generating Station, Burlington, Vermont

This project used Reinforced Earth to support a railroad unloading trestle at a 50-megawatt wood-burning power plant. The trestle supports a rail spur of the Central Vermont Railroad that is used to deliver wood chips to the largest wood-burning power plant in the world.

The ability of Reinforced Earth to withstand significant postconstruction settlement was one of the factors leading to its selection at the McNeil Station. Before construction, borings indicated that

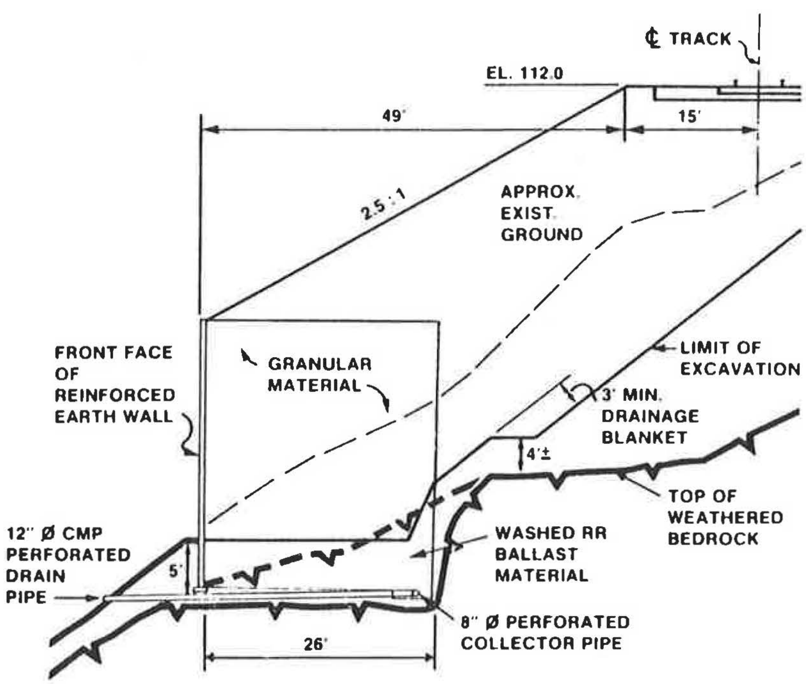


FIGURE 8 Typical cross section of Clinchfield stabilization project.

the first 35 ft of soil consisted of loose silts and fine sands to medium dense silts with some coarse sand and traces of gravel. These foundation conditions necessitated a deep foundation for any rigid system of abutments. In fact, the preliminary design of the structure included a reinforced concrete abutment founded on 136 piles of 100-ton capacity. Due to the prohibitive cost of such a system, the flexible system of Reinforced Earth abutments was chosen.

The Reinforced Earth abutments support 50-ft single span beams that impose a total load, including impact, of 26 kips per linear foot. The beams were placed on a 9-ft-wide bearing seat atop the Reinforced Earth volume. This design results in a bridge seat bearing pressure of 2.9 ksf applied to the Reinforced Earth abutments, which is well below the 4 ksf allowable bearing pressure for the abutment bearing seat.

The compressible soils at the McNeil site necessitated the founding of the Reinforced Earth walls on a 5-ft-thick mat of compacted gravel. Although the maximum anticipated settlement was approximately 6 in., measured total settlement at one of the abutments exceeded 16 in. The maximum differential settlement along the wall face is 8 in. in approximately 100 ft, or 0.67 percent.

The construction of the Reinforced Earth abutments was scheduled so that the abutments would be allowed to settle before placement of the bearing seat and superstructure. When 95 percent consolidation of the underlying foundation had occurred, the transverse differential settlement was accounted for by pouring the abutment bearing seat to the final design elevations. One of the completed Reinforced Earth abutments at this location is shown in Figure 9.

FIGURE 9 Reinforced Earth abutment at McNeil generating station.

Power Authority of the State of New York, Staten Island, New York

The project consists of a Reinforced Earth foundation slab, used to spread the heavy railroad loading to a stone column foundation below. The Reinforced Earth slab is a mat structure constructed of alternating layers of closely spaced horizontally bedded reinforcing strips and granular backfill. The mat was designed to span the area between stone columns, thereby bridging the existing soft foundation material and transferring the railroad loads directly to the stone columns. The project was built at an access line to a 700-MW fossil power plant for the Power Authority of the state of New York.

SUMMARY AND CONCLUSIONS

The completed projects demonstrate that Reinforced Earth technology is wholly applicable to and economical in a railroad environment. A rational design procedure has been developed to predict the effects of vibratory loading on the soil-structure interaction in a Reinforced Earth mass. From the experimental data presented, the effects of railroad traffic vibrations are manifested in (a) slightly lower apparent coefficient of friction (f^*), which is a function of the horizontal acceleration imposed by the rail traffic, and (b) higher stresses to be resisted by the reinforcements caused by the dynamic inertial forces developed by rolling rail traffic. The additional material costs consistent with full consideration of these design parameters are modest. Additional research is necessary to fully define the level of rail traffic-induced horizontal acceleration and its variation with depth, speed of rolling stock, and rolling weight.

REFERENCES

1. F. Schlosser. History, Current and Future Developments of Reinforced Earth. Proc., Symposium on Soil Reinforcing and Stabilising Techniques in Engineering Practice, Sydney, Australia, 1978.
2. D.P. McKittrick. Reinforced Earth: Application of Theory and Research to Practice. Proc., Symposium on Soil Reinforcing and Stabilising Techniques in Engineering Practice, Sydney, Australia, 1978.
3. G. Richardson, D. Feger, A. Fong, and K. Lee. Seismic Testing of Reinforced Earth Walls. Journal of the Geotechnical Engineering Division, ASCE, Vol. 103, No. GT1, Jan. 1977, pp. 1-17.
4. Measures d'accélération verticale sur la partie remblayée. R.6400-69-06. Société Nationale des Chemins de Fer Français, Paris, France, 1969.
5. W.E. Wolfe, K.L. Lee, D. Rea, and A.M. Yourman. The Effect of Vertical Motion on the Seismic Stability of Reinforced Earth Walls. ASCE Symposium on Earth Reinforcement, Pittsburgh, Pa., 1978.
6. G. Richardson and K. Lee. Seismic Design of Reinforced Earth Walls. Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, No. GT2, Feb. 1975, pp. 167-187.
7. K. Minami and K. Adachi. Examination About Characteristics and Designing Method of Reinforced Earth. Internal Report 1630. Construction Research Office of Civil Engineering Laboratory of Construction Ministry, Tokyo, Japan, May 1981.
8. J.A. Goforth and V. Elias. Interior Support Stiffens Embankment Renewal. Railway Track and Structures, Sept. 1980.

Publication of this paper sponsored by Committee on Railway Maintenance.

Tieback Wall Stabilization of Railroad Embankments

HUBERT DEATON III

ABSTRACT

Tieback walls are used to stabilize railroad embankments. Frequently, they can be built without interrupting traffic or relocating the tracks. The walls eliminate the temporary excavation support that may be required to construct other methods of stabilization. They may be built close to tracks or structures because they limit long-term soil movements. Tieback walls can stabilize embankments with deep failure surface. The design of a tieback wall for a railroad embankment is performed in a manner similar to that of other tieback walls. A geotechnical investigation should provide the design properties including location of the failure surface, unit weights of material, internal strength properties of material, strength properties along the failure surface, soil or rock classification, and groundwater information. In addition, tests must be made to determine the levels of corrosion protection that will be adequate to ensure that the tieback and the wall components will not corrode. A railroad embankment in the Blue Ridge Mountains was stabilized with a tieback, driven H-pile wall 40 ft high. Track traffic was not interrupted and future maintenance was reduced. Monitoring of the wall has indicated that it is performing as expected.

Grouted rock and soil anchors typically called tiebacks were originally developed as an alternate to bracing or massive cantilever sheeting for temporary excavation support. Their good performance and the ease with which their capacities may be verified has led to their use as a permanent means of supporting retaining walls. These walls are typically called tieback walls or permanently anchored walls. Their use is increasing throughout the United States both on public and on private works (1).

Tieback walls have several structural members (Figure 1). The tieback itself is a grouted anchor that transmits force from the wall through a posttensioned tendon. The tendon force is developed in the soil or rock by adhesion of the grouted anchor over a zone called the anchor length. The portion of the tendon that is between the anchor and the wall is called the unbonded length, and it elongates elastically during posttensioning (2). Tieback walls may have horizontal support members called elements (Figure 2) or vertical support members called soldier beams. The space between support members may be covered to further reduce soil movement or erosion. This cover is called the facing and it is typically cast-in-place or precast concrete, shotcrete, or treated timber.

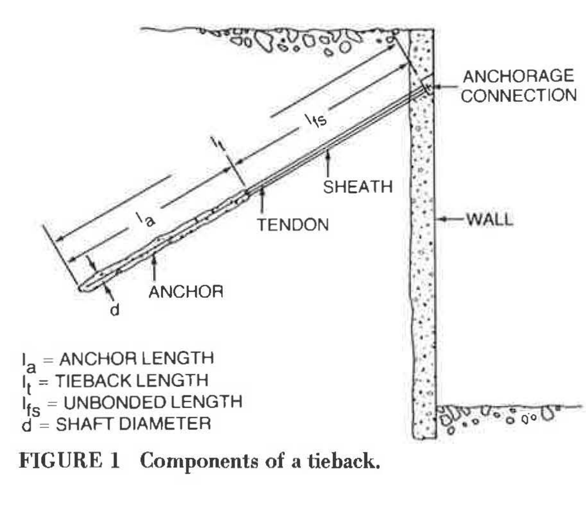


FIGURE 1 Components of a tieback.

APPLICATION

Tiebacks can offer significant advantages in the stabilization of existing embankments that support critical loads such as railroad traffic. Many railroad embankments have been constructed by dumping with no special compaction or selection of materials. Then fills are subject to slope failures that are normally repaired by dumping additional material. This may overload a marginally stable slope and further movement can result. Construction of a conventional retaining wall may require removal of a large portion of the embankment. Temporary shoring of the excavation may be required. Figure 3 shows a comparison of a conventional retaining wall with temporary shoring and a tieback wall. Alternately, relocation of the tracks away from the wall construction may be required. With the use of a tieback wall, the temporary shoring can become a permanent part of the wall, and relocation of the tracks is not necessary. In many cases, train traffic can proceed uninterrupted while the wall is being constructed because construction proceeds from the top down (Figure 4).

In cases in which embankment movement is generated by landslide conditions, tieback walls can be even more advantageous. Whereas nontieback walls must penetrate the landslide's failure surface, with a tieback wall only the tieback anchors are required to extend beyond the plane of movement. The wall itself may stop at whatever level produces stable slopes above and below the wall (3).

Because tieback walls are pretested to overloads in excess of anticipated service loads, their capacities are verified and soil movements may be kept small. By the use of instrumentation actual loads taken by the wall may be compared with design assumptions. The ease of verifying performance and the small magnitude of movement allow tieback walls to be located close to movement-sensitive facilities.

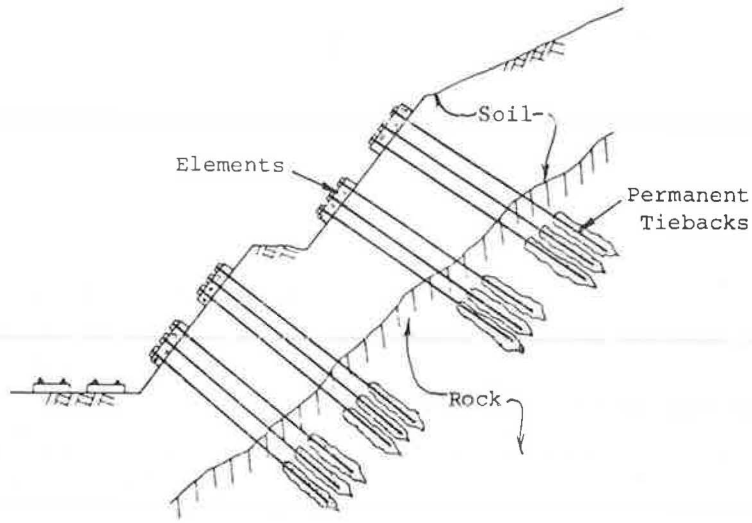


FIGURE 2 Tieback element wall.

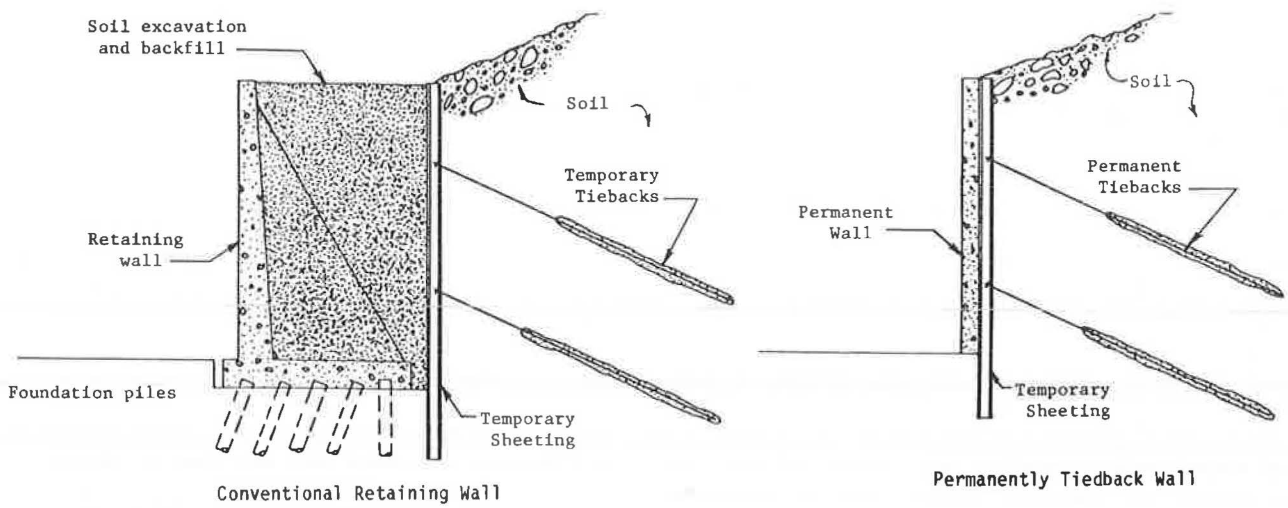


FIGURE 3 Comparison of conventional retaining wall and tieback wall.

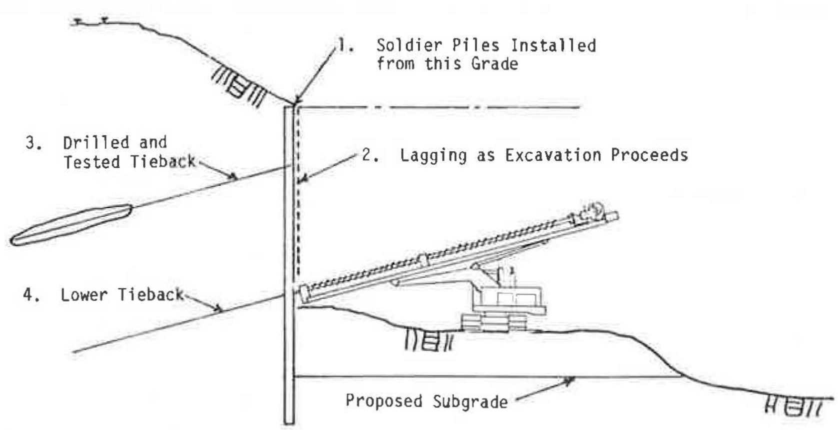


FIGURE 4 Tieback wall construction from top down.

Tieback walls may be desirable in any cut situation, but they are particularly useful when traffic cannot be interrupted, relocated, or where large deflections cannot be tolerated.

In evaluating the use of tiebacks for embankment stabilization, there are important preliminary considerations. It must be physically possible to install tiebacks and underground easements must exist or be secured. It must be possible to develop tieback capacity in the anchor length, and the soil must not be subject to excessive creep. Most fills and some in situ soils are not suitable for anchors. In rare cases the ground is so aggressive that tiebacks cannot be adequately protected from corrosion. If it appears to be feasible to use permanent anchors, their cost can then be compared with that of other possible stabilization methods.

DESIGN

Obviously, the design of tieback walls requires a thorough geotechnical investigation. It must be determined if the wall will be loaded by conventional lateral earth pressure or if the embankment is moving as a landslide. Among the information required for estimating design forces is the angle of internal friction, cohesion, depth to groundwater surface, and traffic surcharges. This geotechnical investigation should be made early in the design phase. To determine in situ properties of soil and rock, borings should be taken in the areas where the anchors will be made. For soils, standard penetration resistance, atterberg limits, and unconfined compression tests should be made. Rock cores should be taken with core recovery and rock quality designation (RQD) noted. The potential for corrosion in the embankment must be investigated. Levels of resistivity, soil and groundwater pH, soluble sulfate content, and presence of sulfide must be known to determine the degree of corrosion protection required for the work (4). A summary of some basic permanent anchor criteria follows.

Criteria to be examined in the preliminary evaluation of a tieback wall (1):

1. Geometry
   - Overburden depth greater than 15 ft (above anchor)
   - Tieback no steeper than 45 degrees below horizontal
2. Soil strength
   - For coarse-grained soils: standard penetration resistance greater than 10 blows/ft.
   - For fine-grained soils: unconfined compressive strength greater than 1.0 ton/ft^2 and consistency index (I_C) greater than 0.8:

$$I_C = (W_L - W)/(W_L - W_P)$$

where

W = natural water content,
W_L = liquid limit, and
W_P = plastic limit.

If it is decided to anchor in fills or soils that do not satisfy these criteria, a precontract testing program is recommended.

3. Aggressiveness of soil
   - Encapsulation of the tendon will be necessary if soil surrounding anchor has pH < 4.5, resistivity < 2000 ohm-cm, and any sulfides are present.
   - If soluble sulfate content is greater than 2000 mg/kg use ASTM Type V cement. If nearby buried concrete structures show signs of acid attack, portland cement grouts should not be used.

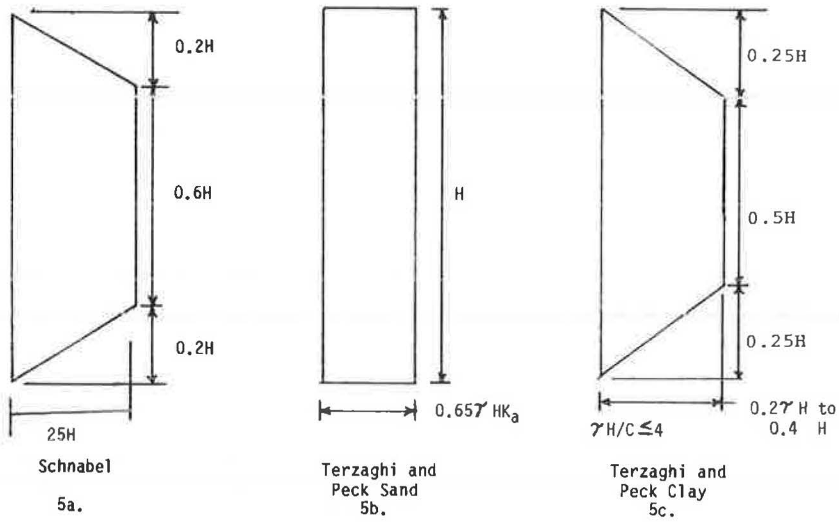
If the wall is subject to conventional lateral earth pressures, apparent earth pressure envelopes similar to those proposed by Terzaghi and Peck and discussed by Peck et al. (5) have been found to be appropriate in most soils (6) (Figure 5). If the wall is acting as a landslide restraint, the restraining force that will be supplied by the tieback must be estimated. This is typically done by limit equilibrium analysis. The location of the failure surface and the strength properties along the failure must be known (7). If an embankment is failing as a landslide, inclinometers may be necessary to determine the sequence and rate of movement. It is necessary to compare both sets of forces and design the wall for the controlling loading. The engineer must also determine what factors of safety are present in the method used to estimate forces. The calculated loads can then be adjusted to the desired factor of safety dictated by proper comparison of economics and risk. A final check must be made for overall external mass stability. The unbonded length of the tieback must extend behind the critical failure surface previously determined in the geotechnical investigation. The length of anchor behind the critical failure surface must result in the total tieback length being such that the soil or rock mass between the wall facing and the ends of the tiebacks will be stable. Thus the factor of safety along any potential failure surface behind the ends of the tiebacks will be greater than or equal to the factor of safety along the critical surface. The soil mass will be both internally and externally stable (8) (Figure 6).

TESTING

Although the capacity of tieback anchors may be estimated using empirical values, it is imperative that each tieback be tested after installation to verify the ability of the anchor to perform as expected. It is not economical to design tiebacks to such low capacities that variations in soil properties, material qualities, or workmanship may be ignored. There are three types of common tieback tests: the creep test (Figures 7 and 8) that estimates long-term load-carrying capacity, the performance test (Figure 9) that determines residual movement of the anchor at increments of load, and the proof test that verifies proper tendon elongation and ability of the anchor to carry the proof load (Figure 10). Typically 90 to 95 percent of the tiebacks may be proof tested only. A center-hole hydraulic jack and pump are used to apply the load in such a way that the entire tieback tendon is loaded simultaneously during testing. The tieback movement is measured by a dial gauge or a vernier scale independent of the tieback structure. Measuring the jack ram travel is not an accurate way to monitor movement.

The first few tiebacks and a selected percentage of the remaining tiebacks should be performance tested. The performance test is used to establish the load-deformation behavior of the tiebacks at a particular site. It also is used to separate and identify the causes of tieback movement and to check that the unbonded length has been established. The movement patterns developed during the performance test are used to interpret the results of the simpler proof test.

Performance testing is conducted by measuring the load applied to the tieback and its movement during



Comparison of Several "Apparent" Earth-Pressure Envelopes

1. Pressures based on construction sequence from top to bottom.
2. These diagrams do not apply to softer soils or to unusual soils.
3. These diagrams do not include major surcharges such as buildings, railroads, etc.
4. These diagrams do not include water pressure or special conditions such as ice lenses, tidal heads, etc.
5. These diagrams are more empirical than theoretical and cannot be used blindly.

FIGURE 5 Typical lateral earth pressures *(6)*.

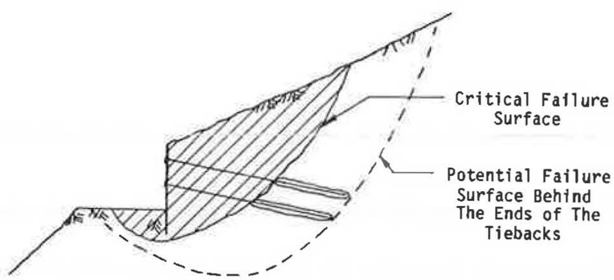


FIGURE 6 Stability analysis for determining unbonded and total tieback length.

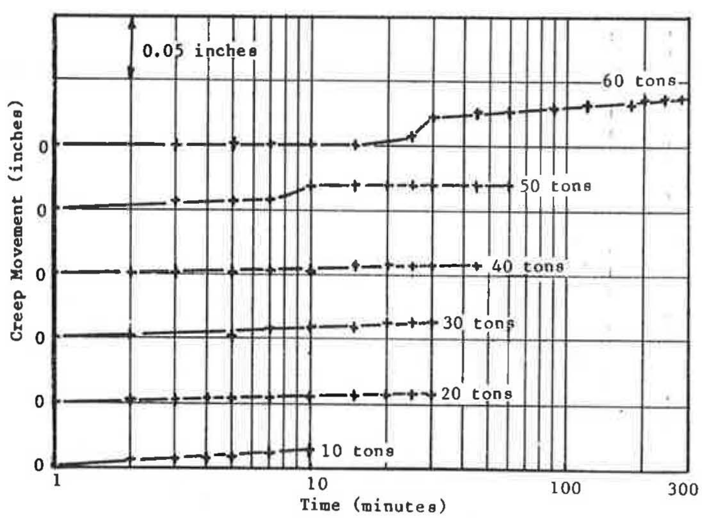


FIGURE 7 Creep test (1.0 ton = 8.9 kN, 1.0 in. = 25.4 mm).

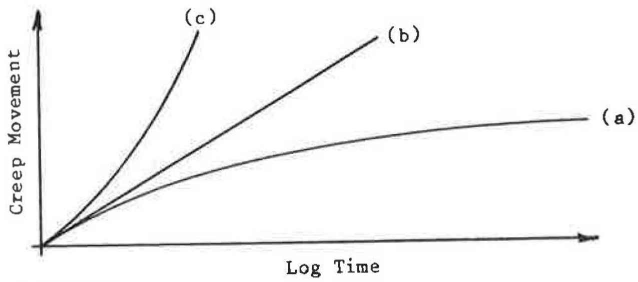


FIGURE 8 Characteristic creep curves.

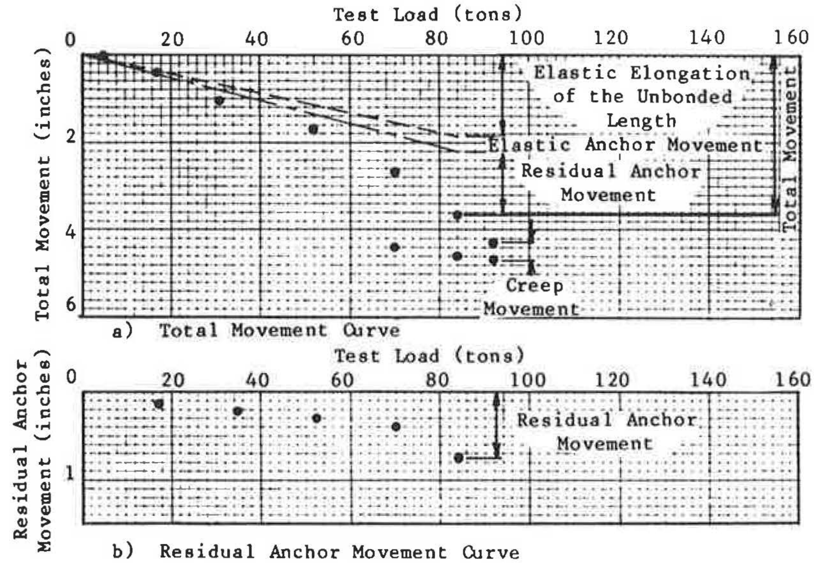


FIGURE 9 Performance test (1.0 ton = 8.9 kN, 1.0 in. = 25.4 mm).

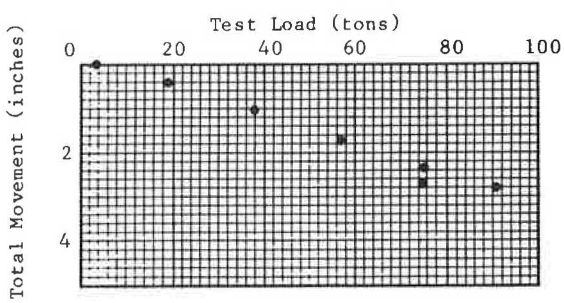


FIGURE 10 Proof test (1.0 ton = 8.9 kN, 1.0 in. = 25.4 mm).

incremental loading and unloading. Figure 9 shows a plot of a typical performance test. The upper graph in the figure shows the total tieback movement as a function of load, and the lower graph shows the residual movement of the anchor as a function of load. The residual movement (permanent set) of the anchor is the nonelastic or unrecoverable movement of the anchor that is measured when the load is released after each loading increment. The maximum load applied during the performance test is held constant for 10 min, and the movements are measured and recorded at 1, 2, 3, 4, 5, 7, and 10 min. If the tieback is not creep susceptible, the elongation between 1 min and 10 min normally will be less than 0.04 in. If so, the test can be discontinued. If the movements exceed 0.04 in., the maximum load should be held for 60 min so a creep curve can be plotted.

Each production tieback that is not performance tested should be proof tested. A proof test is a simple test that is used to measure the total movement of the tieback. Proof testing is conducted by measuring the load applied to the tieback and its movement during incremental loading. Figure 10 shows a typical proof test plot.

The load increments are the same as those used in the performance test, except that the maximum increment is normally equal to 1.20 times the design load. The maximum load applied during a proof test is held constant for 5 min and the tieback movement is recorded. If the movement during the 5-min observation period is less than 0.03 in., the test is discontinued. If the movement exceeds 0.03 in., the load should be maintained until the creep rate can be determined and compared to the creep behavior observed during the performance or creep tests.

Creep tests are performed to evaluate the long-term load-carrying capacity of tiebacks installed in cohesive soils and soft shales. They normally are made on the initial two performance-tested tiebacks. During a creep test, each increment of load is held constant for a certain time period and the elongations are recorded and plotted. Figure 7 shows a typical plot of creep movement versus time on a semilogarithmic graph, with each curve representing the creep movement at each load increment. Figure 8 shows the three characteristic types of creep curves observed during tieback testing. Curves (a) and (b) indicate acceptable behavior, provided that the creep movement estimated by projecting the creep rate over the life of the structure is not excessive. A creep rate of 0.08 in. per log cycle would produce a creep movement of approximately 0.5 in. over a period of 50 years. Curve (c) indicates that the tieback would continue to creep until it failed.

The primary purpose of any tieback test is to determine whether the tieback will carry the required load without unacceptable movement. Proper tieback testing will include

- Testing of tieback to an overload, typically 120 to 150 percent,
- Comparison of actual elastic elongation versus theoretical,
- Determination that the rate of creep is acceptable,
- Examination of the residual anchor movement, and
- Verification that all components of the tieback connections perform satisfactorily at an overload.

Any tieback that does not perform as designed must be replaced or incorporated in the wall at a reduced capacity.

CASE HISTORY: 4TH ROCKY FILL SLIDE STABILIZATION

Existing Conditions

A busy line of track of the Clinchfield Railroad (9) crossed on an embankment of uncontrolled fill. This fill blocked natural drainage paths and developed seepage pressures. The resulting slides were repaired by dumping additional material that increased embankment height and the potential for further movement. This method of maintenance increased the likelihood of a failure that would halt use of the line (Figure 11).

Geotechnical Investigation

The railroad first had a comprehensive geotechnical investigation performed. This investigation established the bottom of the shear surface, shear strength, and groundwater levels. It was verified that the fill was failing in a landslide mode and that movement was continuing.

The geotechnical consultant recommended use of a tieback wall to stabilize the embankment on the basis of the following project requirements:

1. The method of construction must not interrupt rail traffic or increase slope instability,
2. Train surcharge must be supported with small deflections,
3. Embankments above and below the wall must be stable,
4. Drainage must not be obstructed,
5. Maintenance must be reduced,
6. Construction schedule must allow completion during dry season, and
7. Costs must be competitive with other options.

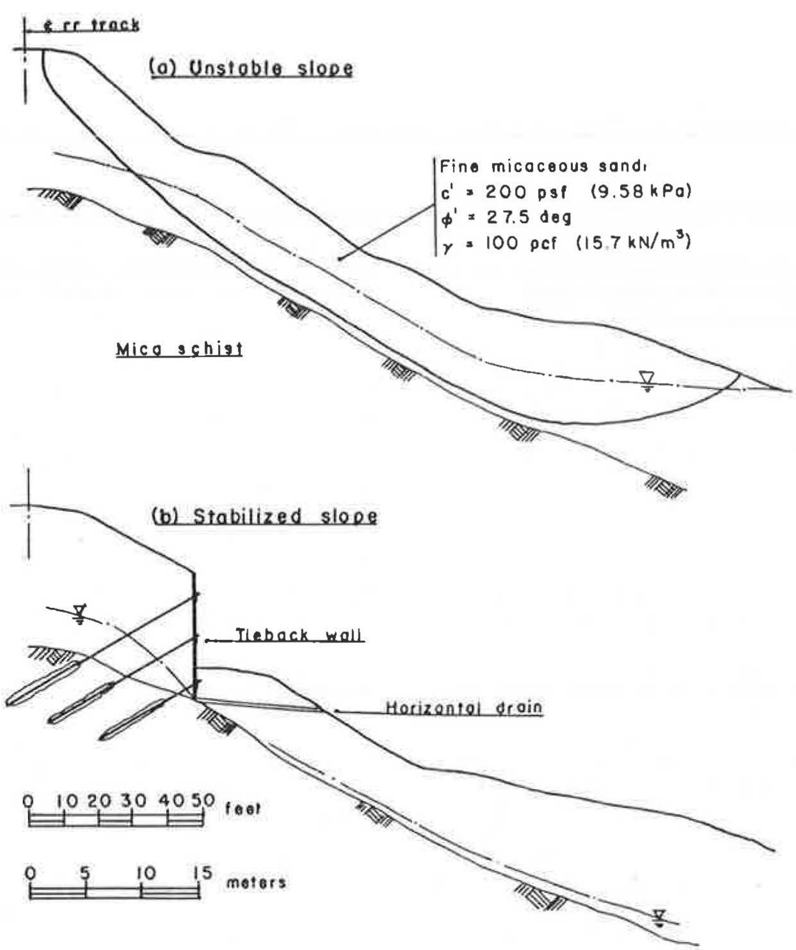


FIGURE 11 Fourth rocky fill slide profiles.

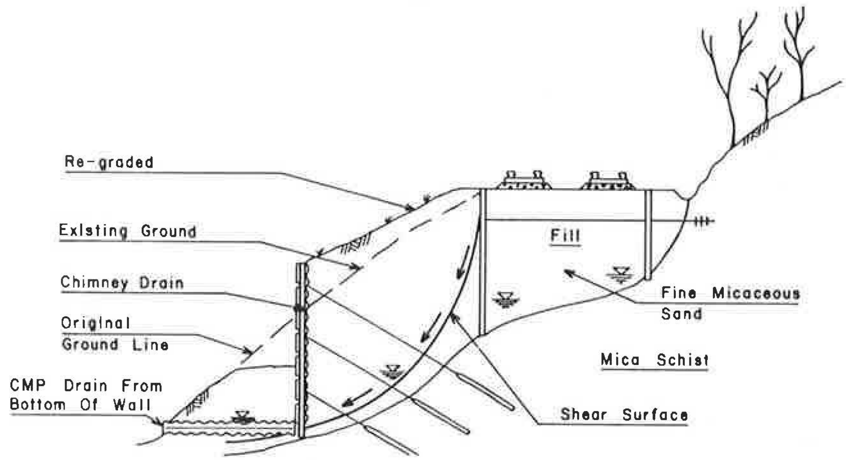


FIGURE 12 Active side hill embankment slide threatens main-line Clinchfield Railroad tracks.

Design

Several contractors were invited to submit detailed proposals for a design-build tieback wall under a performance specification (Figure 12). The design selected called for the following

1. Design height of 40 ft,
2. Driven H-piles to rock,
3. Tiebacks penetrating the failure surface,
4. Stability of the embankment below the wall ignored,
5. Comparison of lateral earth pressure and landslide forces,
6. Treated timber lagging,
7. Chimney drains with drainage fabric to prevent loss of ground,
8. Regrading of embankments above and below the wall to more stable slopes,
9. All tiebacks tested to overloads, and
10. Wall instrumented and its performance monitored.

Lateral earth pressures were computed by a conventional trapezoidal earth pressure diagram (Figure 13). The landslide stability analysis, which controlled in this case, used Janbu's (10) method to estimate the restraint force (Figure 14). This force was then distributed into the tiebacks. Tieback angles were selected and the capacity of soldier piles to support the vertical loads was examined. Tieback anchorage was required to be behind the shear surface. Because a possibility existed that the embankment below the wall could fail in some later slide, no passive resistance or stability factors were allotted to the lower embankment.

Construction

Soldier piles were driven from a bench cut into the embankment. Installation of wood lagging and drainage fabric proceeded concurrently with excavation down to the first tieback level. Wood lagging was not designed but was sized on the basis of judgment

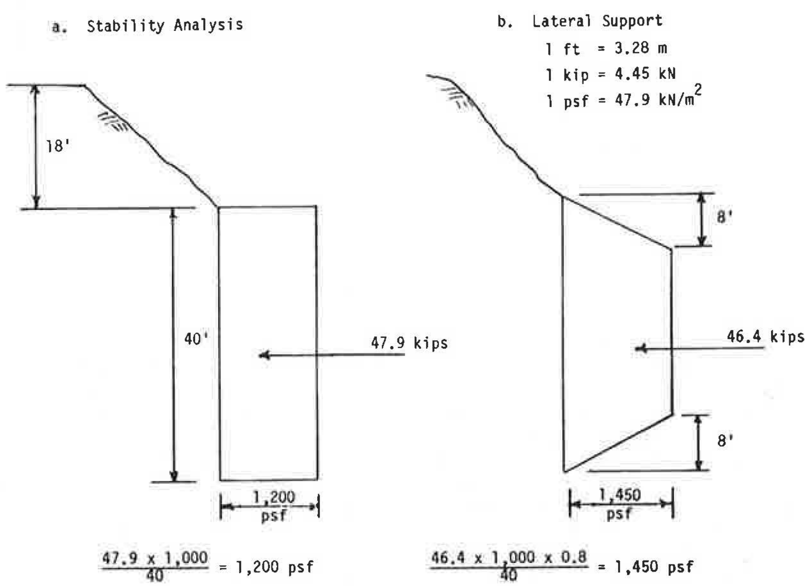
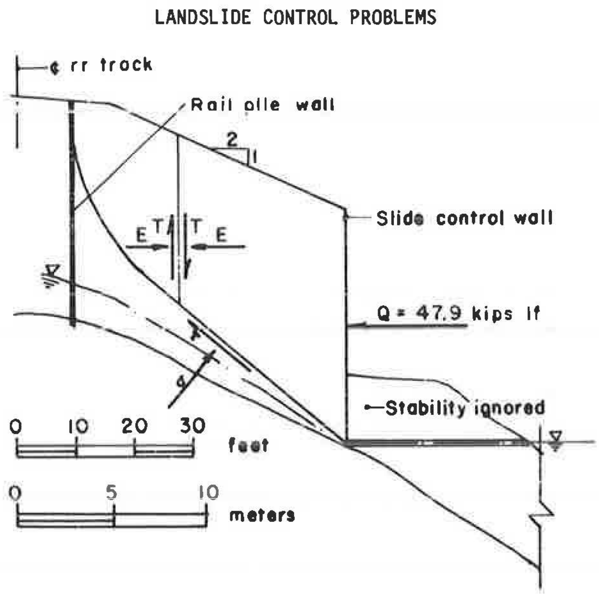


FIGURE 13 Design earth pressure envelopes for fourth rocky fill tieback slide control wall.



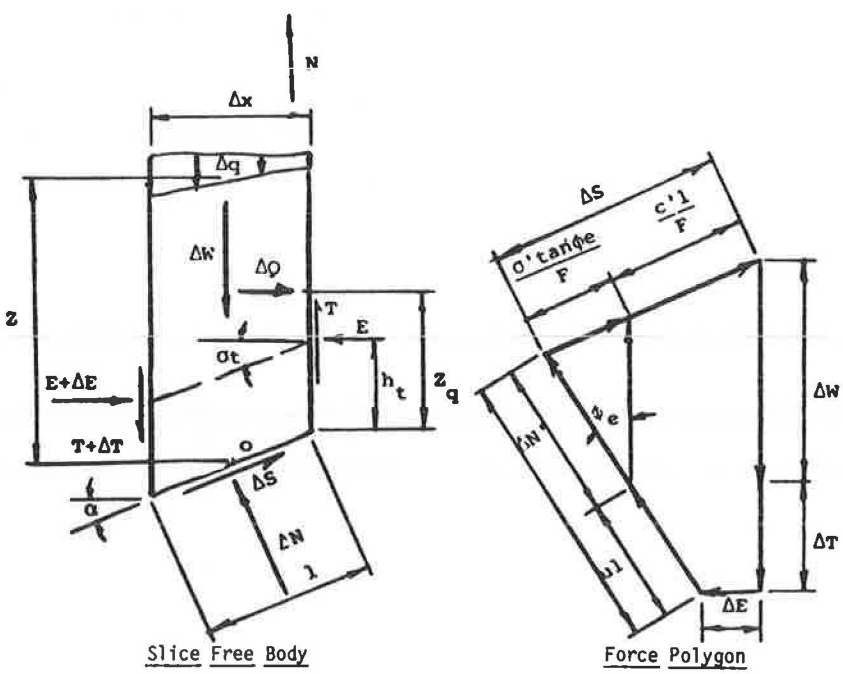


FIGURE 14 Model for stability analysis of fourth rocky fill.

for the soldier pile spacing used. The choice of a timber facing was dictated by economics. Tiebacks were installed, tested to an overload, and locked off. Any tiebacks that failed were replaced.

This procedure was repeated for the lower rows of tiebacks. Lagging and drainage were installed to the top of rock. Horizontal drains were connected to the chimney drains to conduct water down slope. By this method the water surface is kept at the top of rock. The lower slope was then regraded to approximately half-way between the second and third levels of tiebacks, burying the lagging, wales, and tiebacks. At the same time the upper slope was graded from top of pile to track elevation. All slopes were then seeded to protect them from erosion.

Performance

Initially the piles moved into the fill when the upper level of ties was tensioned. At the top of the pile, movements of as much as 8 in. were recorded. This is not unusual for tiebacks tensioned against fill.

Reference tapes were installed at the top of the piles and a reference line was established after all tiebacks had been tensioned and excavation was completed. Total outward movement of the piles in the first 7 months averaged 0.021 ft or approximately 1/4 in.

Two vertical rows of load cells were provided. The six load cells were monitored for a year after lock-off. These load cells showed an average of 4 percent increase in load over a 12-month period, most of which occurred in the first 3 months. Some maintenance of the timber lagging has been required. In general, performance of the wood facing has been as expected.

CONCLUSIONS

Tieback walls can be an economical way to stabilize railroad embankment.

Their construction usually does not interrupt railroad traffic.

A thorough geotechnical study is required.

The machanism by which the wall will be loaded must be known.

Tiebacks must be tested to verify design assumption.

Evaluation of wood lagging facings must include costs of periodic maintenance.

REFERENCES

1. D.E. Weatherby. Tiebacks. Report FHWA/RD-82/047. FHWA, U.S. Department of Transportation, July 1982.
2. T.C. Anderson. Earth Retention Systems Temporary and Permanent. Proc., 32nd Annual Soil Mechanics and Foundation Engineering Conference, 1984.
3. R.B. Reeves. Control of Landslide with Permanent Tiebacks. In Slope Stability and Landslides, University of Wisconsin Extension, Madison, Jan. 1982.
4. D.E. Weatherby and P.J. Nicholson. Tiebacks Used for Landslide Stabilization. ASCE Proc., Application of Walls to Landslide Control Problems, Las Vegas, Nev., April 1982.
5. R.B. Peck, W.E. Hanson, and T.H. Thornburn. Foundation Engineering, 2nd ed. John Wiley and Sons, Inc., New York, 1974.
6. H. Schnabel, Jr. Tiebacks in Foundation Engineering and Construction. McGraw-Hill Book Company, New York, 1982.
7. N.R. Morganstern and D.A. Sangrey. Methods of Stability Analysis. In Landslides Analysis and Control, R.L. Schuster and R.J. Krizek, eds., National Academy of Sciences, Washington, D.C., 1978, pp. 155-171.
8. H. Schnabel, Jr. Stability of Tiedback Excavations. Presented at ASCE Construction Excavation Meeting, Harrisburg, Pa., April 1982.
9. G.L. Tysinger. Slide Stabilization 4th Rocky Fill, Clinchfield R.R. ASCE Proc., Application of Walls to Landslide Control Problems, Las Vegas, Nev., April 1982.
10. N. Janbu. Slope Stability Computations. In Embankment Dam Engineering, R.C. Hirschfeld and S.J. Poulos, eds., John Wiley and Sons, New York, 1973, pp. 47-86.

Publication of this paper sponsored by Committee on Railway Maintenance.

Methodology for Allocating Loss and Damage to the Railroad Transport Cycle

PETER J. WONG

ABSTRACT

A methodology for allocating loss and damage costs to various parts of the railroad transport cycle is presented. Specific estimates of loss and damage attributed to line-haul shock and vibration and flat and hump yard coupling impacts are developed. In addition, loss and damage estimates are provided for various levels of overspeed impacts in hump yards.

In calendar year 1983, Association of American Railroads (AAR) statistics indicate that North American railroads paid out a total of $162 million in freight loss and damage (L&D) (from Information and Public Affairs, AAR). Industry sources indicate that the indirect costs to railroads and shippers of processing and handling L&D claims may be as much as eight times greater than the direct L&D payments (1). If this is true, the total costs of L&D to railroads and shippers may be on the order of $1.3 billion per year.

Even though the railroad industry has been vitally concerned with L&D for many years--the 1983 loss is the lowest since 1965--these figures indicate that the magnitude of the L&D problem is still large enough to warrant new approaches. Any decrease in the L&D payments translates directly to increases in net income (i.e., "bottom-line") on a one-to-one basis. Thus improvements to the L&D payment situation can significantly affect the health and viability of the entire railroad industry.

There is a lack of information about and understanding of the L&D costs that can be attributed to shock and vibration (rough handling) in the transport cycle. In particular, it would be nice to know how much L&D can be attributed to railroad yards versus the line haul. In yards, the amount of L&D that can be attributed to hump yards versus flat yards and how L&D increases with impact speeds in yards are unknown. Over the line haul, the contribution to L&D of longitudinal train slack action versus vertical vibration should be assessed.

This L&D transport cycle information is necessary to assess the benefit-cost impact of any specific proposed countermeasures to mitigate the effects of shock and vibration. Also, this L&D transport cycle information is important to assist in planning research priorities for future L&D countermeasures. Examples of potential areas for countermeasure development include

- Countermeasures to reduce L&D due to train slack action over the line haul,
- Countermeasures to reduce L&D due to vertical vibration over the line haul, and
- Countermeasures to reduce L&D due to overspeed impacts in hump or flat yards.

This research presents a methodology for allocating L&D to shock and vibration in the transport cycle. Although the data needed to perform this allocation precisely are lacking, estimates are developed on the basis of available data that allocate L&D to the following:

- Yards versus line haul,
- Hump versus flat yards, and
- Levels of overspeed impact in yards.

The approaches and procedures presented here represent a first approximate step in structuring a methodology. The method can be useful in developing L&D countermeasure technology.

METHODOLOGY OVERVIEW

The AAR each year provides aggregate statistics on L&D (2) in the following 12 categories:

1. Shortage, package shipment;
2. Shortage, bulk shipment;
3. All damage not otherwise provided for;
4. Defective or unfit equipment;
5. Temperature failures;
6. Delay;
7. Robbery, theft, pilferage;
8. Concealed damage;
9. Train accident;
10. Fire, marine, and catastrophies;
11. Error of employee; and
12. Vandalism.

Unfortunately, these 12 categories do not indicate the L&D due to shock and vibration (i.e., rough handling), which is germane to this work. Freight damage resulting from excessive shock and vibration would probably be listed in Category 3, "all damage not otherwise provided for," and Category 8, "concealed damage." Almost $71.4 million or 44.1 percent of the total 1983 freight loss and damage payments was classified in Category 3, and $0.5 million or 0.3 percent was classified in Category 8 (2). Other major factors that affect L&D in Categories 3 and 8 are inadequate packaging, improper loading, and claims incorrectly assigned to these categories.

The methodology consists essentially of allocating Categories 3 and 8 L&D to shock and vibration in the transport cycle using the "tree-structured top-down" approach shown in Figure 1. The steps can be summarized as:

- Step 1: Categories 3 and 8 L&D are allocated between shock and vibration versus "others."
- Step 2: Shock and vibration L&D is allocated between line haul and yard.
- Step 3: Line-haul L&D is allocated between shock (caused by the slack action of trains) and vibration.
- Step 4: Yard L&D is allocated to hump yards versus flat yards. (It is implicitly assumed that yard-related L&D is due to shock, not vibration, because the distances that cars travel in a yard are small and thus the exposure to vibration damage is minimized.)
- Step 5: Hump yard shock L&D is allocated to overspeed impact levels.

DETAILS OF METHODOLOGY

In this section the data and details for implementing each step in the methodology are presented.

Allocation to Shock and Vibration Versus Others (Step 1)

Categories 3 and 8 L&D amounted to $71.9 million in 1983. How much of this amount can be attributed to shock and vibration (i.e., rough handling) versus other causes such as inadequate packaging, improper loading, and claims incorrectly assigned?

The data to perform this allocation are scarce and imprecise. In their corrugated container study, Ostrem and Godshall (3) indicate that 80 percent of the damage could be attributed to rough handling. However, the Whirlpool appliance study (4) indicates that, at a minimum, 43 percent of appliance L&D could be attributed to shock and vibration. This leads to the conjecture that the percentage of damage that can be attributed to shock and vibration varies with the type of commodity under consideration.

The obvious answer to the problem is to get more data. However, in lieu of this possibility, an attempt is made to estimate the percentage of Categories 3 and 8 L&D that can be allocated to shock and vibration using a systematic procedure based on the data that exist.

Table 1, obtained from Braddock et al. (5), gives the gross claims paid by cause and commodity. Although the "damage" category in Table 1 contains damage causes in addition to Categories 3 and 8 (e.g., temperature failures, delay, fire, and train accidents), let it be assumed that the percentage of L&D for each commodity item under the "damage" category in Table 1 applies to Categories 3 and 8 L&D for each commodity group. It is known that 80 percent of corrugated container L&D can be attributed to rough handling and that a minimum of 43 percent of appliance L&D can be attributed to shock and vibration. The procedure will then be to associate each commodity grouping in Table 1 with either an 80 percent or a 43 percent L&D due to shock and vibration.

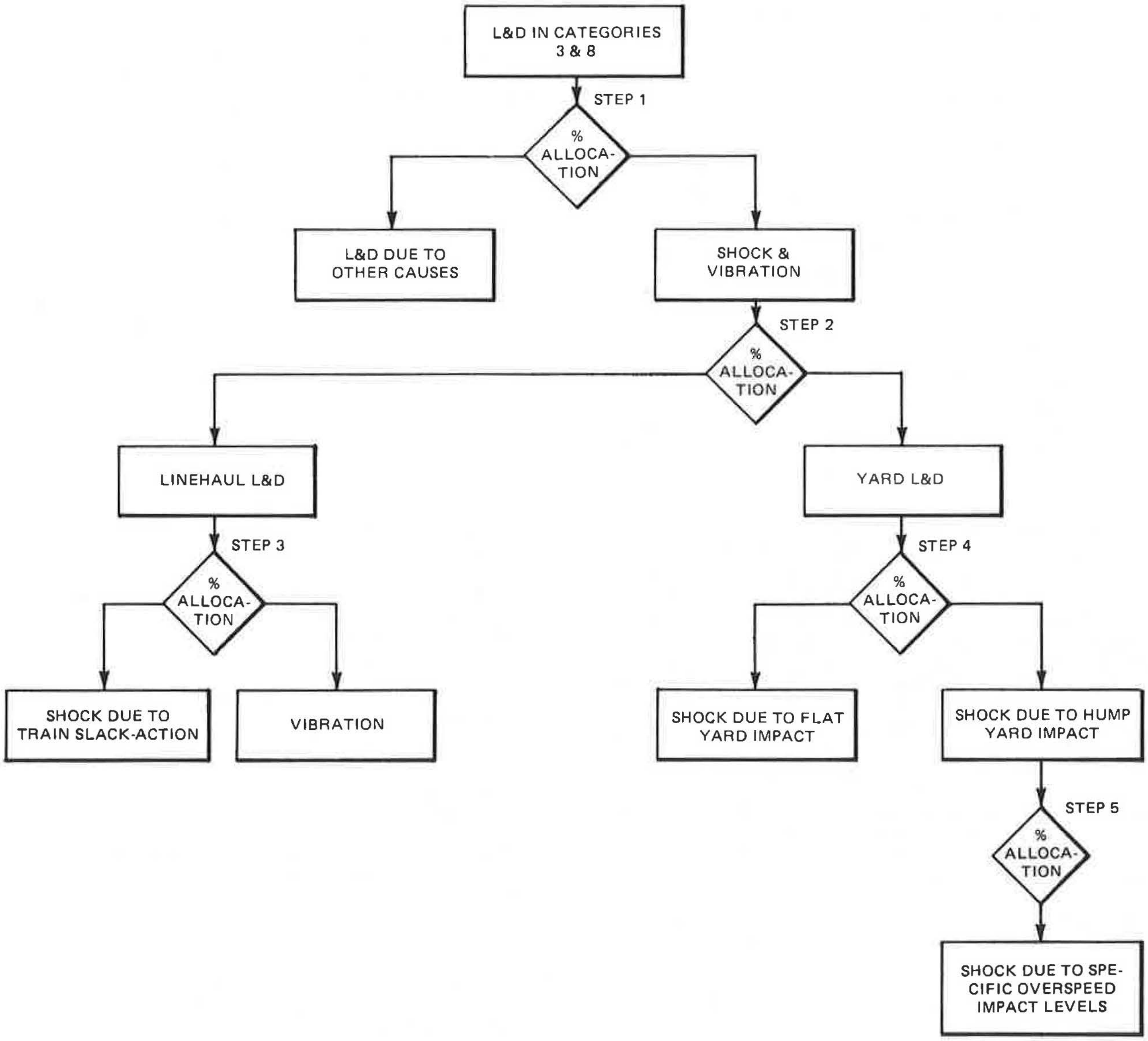


FIGURE 1 Methodology for allocating L&D in Categories 3 and 8 to the transport cycle.

TABLE 1 Gross Railroad Claims Paid by Cause and Commodity (5)

	Shortage		Theft		Damage		Total	
Commodity Grouping	Value ($)	Percentage	Value ($)	Percentage	Value ($)	Percentage	Value ($)	Percentage
Food and food products	12,522,827	28.5	3,295,481	12.6	203,849,028	45.0	219,667,336	42.0
Alcoholic beverages	1,889,409	4.3	915,411	3.5	3,623,983	0.8	6,428,803	1.2
Tobacco products	659,096	1.5	1,909,287	7.3	1,358,994	0.3	3,927,377	0.8
Wood products and furniture	2,592,445	5.9	680,020	2.6	44,846,786	9.9	48,119,251	9.2
Chemicals, petroleum, rubber, and plastic	5,800,046	13.2	2,196,987	8.4	32,162,847	7.1	40,159,880	7.7
Metal products and hardware	3,734,878	8.5	1,987,750	7.6	22,196,894	4.9	27,919,522	5.3
Machinery (except electrical)	2,372,746	5.4	732,329	2.8	13,136,937	2.9	16,242,012	3.1
Electric machinery, including appliances	2,153,047	4.9	3,263,326	12.5	22,196,894	4.9	27,619,267	5.3
Transportation equipment, including motor vehicles	8,304,612	18.9	10,331,071	39.5	45,299,784	10.0	63,935,467	12.2
Clothing and textiles	0	0	0	0	0	0	0	0
Jewelry and coins	0	0	0	0	0	0	0	0
Instruments	0	0	0	0	0	0	0	0
Medicines, drugs, and cosmetics	0	0	0	0	0	0	0	0
Others	3,910,637	8.9	836,948	3.2	63,419,698	14.0	68,167,283	13.0
Total	43,939,744	100	26,154,610	100	452,997,840	100	523,092,195	100
Percentage of total loss	8.4		5.0		86.6		100	

TABLE 2 L&D in Categories 3 and 8 Attributed to Shock and Vibration

Commodity Grouping	Value ($)	Percentage of Loss Due to Damage	Percentage Attributed to Shock and Vibration	Percentage of Total L&D Due to Shock and Vibration
Food and food products	203,849,028	45.0	80	36.00
Alcoholic beverages	3,623,983	0.8	80	0.64
Tobacco products	1,358,994	0.3	80	0.24
Wood products and furniture	44,846,786	9.9	43	4.26
Chemicals, petroleum, rubber, and plastic	32,162,847	7.1	61.5	4.37
Metal products and hardware	22,196,894	4.9	43	2.11
Machinery (except electrical)	13,136,937	2.9	43	1.25
Electric machinery, including appliances	22,196,894	4.9	43	2.11
Transportation equipment, including motor vehicles	5,299,784	10.0	43	4.30
Clothing and textiles	0	0	–	–
Jewelry and coins	0	0	–	–
Instruments	0	0	–	–
Medicines, drugs, and cosmetics	0	0	–	–
Others	16,419,693	14.0	61.5	8.61
Total	452,997,840	100		63.89
Percentage of total loss	8.4			

Table 2 gives percentages assigned to commodities on the basis of whether the commodity group is "closer to" corrugated containers or to appliances. In difficult cases, it was assumed that the percentage of L&D attributed to shock and vibration is the average of 43 percent and 80 percent (i.e., 61.5 percent). If this procedure is followed, and the percentage of L&D is "weighted" by commodity group by the percentage that the commodity contributes to total L&D, then the following is obtained: approximately 64 percent (actually 63.9 percent) of Categories 3 and 8 L&D can be attributed to shock and vibration. (It should be noted that if it is assumed that the percentage allocation lies between 43 and 80 percent, and the arithmetic mean of these two numbers is taken, the estimate is 61.5 percent.)

Using the 64 percent estimate,

Total 1983 Categories 3 and 8 L&D allocated to shock and vibration = 0.64 ($71.9 million) = $46.0 million (1)

If $46.0 million is divided by 18,800,172 revenue car loadings for 1983 (6),

Average 1983 L&D payments per loaded trip due to shock and vibration = $46.0 million/18,800,172 car loadings = $2.45 (2)

A number of experienced railroad personnel believe that the 64 percent allocation is too low and should be closer to 70 percent. On the other hand, there are people who believe the allocation should be closer to 50 percent.

Allocation to Line Haul Versus Yard (Step 2)

There exist few data that allow the allocation of the $46.0 million L&D costs associated with rough handling to line-haul train slack action and vertical vibration versus coupling impacts in yards.

B. Gallacher, formerly assistant to the Chief Engineer of the Southern Pacific Transportation Company, has recorded data on shifted loads of lumber and pipes occurring in yards versus the line haul. His data indicate that 55 percent of the shifted loads occurred in yards versus 45 percent in the line-haul movement. If it is assumed that there is a correlation between the percentage of shifted loads and the percentage of L&D, it can be assumed that 55 percent of the L&D occurs in yards. Using these percentages,

Total 1983 Categories 3 and 8 L&D allocated to line-haul train slack action and vertical vibration = 0.45 ($46.0 million) = $20.7 million (3)

Total 1983 Categories 3 and 8 L&D allocated to coupling impacts in yards = 0.55 ($46.0 million) = $25.3 million (4)

If these numbers are divided by 18,800,172 revenue car loadings for 1983 (6),

Average 1983 L&D payments per loaded trip due to line-haul train slack action and vertical vibration = $20.7 million/18,800,172 car loadings = $1.10 (5)

Average 1983 L&D payments per loaded trip due to coupling impacts in yards = $20.7 million/18,800,172 car loadings = $1.35 (6)

Some industry personnel believe that the 55 percent allocation of L&D to yards is too low and should be closer to 60 percent.

Allocation to Line-Haul Shock Versus Vibration (Step 3)

In the previous section, $20.7 million L&D has been allocated to line-haul shock and vibration. The shock is mainly due to train slack action; the vibration is mainly the vertical component from the wheel-rail interface. Currently, there do not exist any data by which to allocate L&D between line-haul shock and vibration. A number of industry personnel believe that train slack action is the major cause, whereas others feel that vertical vibration is the main cause. Their viewpoints may depend on the commodity with which they are most closely associated. The author suspects that the type of commodity being transported has great bearing on whether line-haul shock or vibration is the major L&D cause. Because of the lack of data, this allocation cannot be made. This is clearly an area where more data are required.

Allocation to Flat Versus Hump Yards (Step 4)

The total Categories 3 and 8 L&D due to coupling impacts in yards is $25.3 million; the average L&D per loaded trip due to coupling impacts in yards is $1.35 (see the previous section).

Petracek et al. (7) indicate that 80 percent of total U.S. switching occurs in flat yards and 20 percent in hump yards. Therefore it will be assumed that a car spends 80 percent of its yard time in flat yards and 20 percent in hump yards.

However, simply allocating 80 percent of the L&D costs to flat yards and 20 percent to hump yards will not work because the relative L&D in hump and flat yards is unequal. More specifically, the relative time spent in flat and hump yards should be "weighted" by the relative damage occurring in flat versus hump yards; this "weighted relative time" should be used to apportion L&D to the time a loaded car spends in hump yards. In particular,

Total 1983 Category 3 and 8 L&D associated with time spent in hump yards
= {0.2 (hump damage)/[0.8 (flat damage) + 0.2 (hump damage)]} $25.30 (7)

Average L&D per loaded trip associated with time spent in hump yards
= {0.2 (hump damage)/[0.8 (flat damage) + 0.2 (hump damage)]} $1.35 (8)

These equations could be solved if some idea could be gotten of the relative damage occurring in flat versus hump yards. An attempt to estimate the relative damage in flat versus hump yards is made in the remainder of this discussion.

Simmons and Shackson (8) indicate that "the damage impulse increases as the square of the speed." Furthermore, in Figure 2, reproduced Simmons and Shackson (8), acceleration in g's at the car floor is plotted versus impact speed in miles per hour. It appears that there is little if any damage at 4 mph coupling and that the damage impulse increases with the squared difference between 4 mph and the coupling speed. Therefore the following relationship between damage and speed will be assumed. Damage is proportional to the squared difference between 4 mph and coupling speed, i.e.,

$$\text{Damage} = (\text{coupling speed} - 4\ \text{mph})^2 \qquad (9)$$

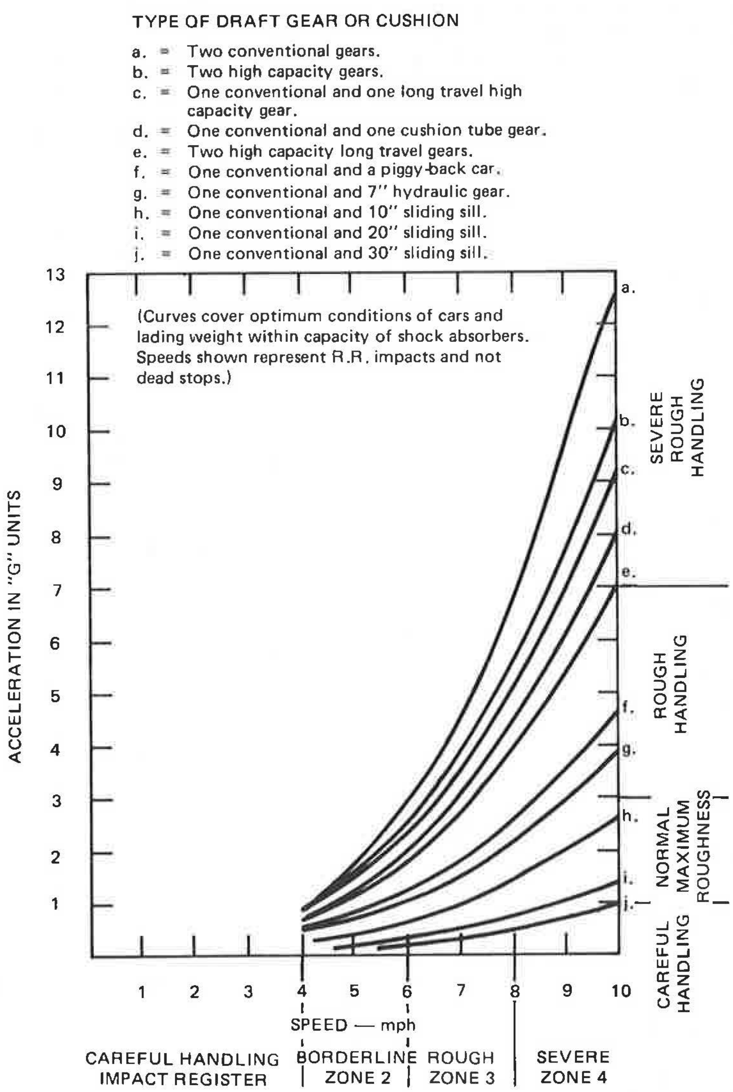


FIGURE 2 Car floor acceleration versus impact speed.

TABLE 3 National Careful Car-Handling Observation Day Results

Coupling Speed (mph)	Retarder/Hump Yards Percentage of Total		Flat Switching Yard Percentage of Total	
	1969	1970	1969	1970
4 or less	51.6	65.6	71.2	80.0
4.1 to 4.9	23.1	12.7	20.7	11.0
5.0 to 5.9	13.1	12.6	5.7	5.6
6.0 to 6.9	5.0	3.6	1.3	2.0
7.0 to 7.9	4.7	3.7	0.7	0.9
8.0 to 8.9	1.1	1.1	0.2	0.3
9.0 to 9.9	0.9	0.5	0.2	0.1
More than 10	0.5	0.2	0.1	0.1
Sample size	3,949	10,493	14,642	26,933

Table 3 gives the frequency of occurrence of overspeed impacts in flat versus hump yards for data taken in 1969 and 1970 by the Association of American Railroads. The 1969 and 1970 AAR data have been combined, and in Figures 3 and 4 these frequencies of occurrence versus coupling speed and (coupling speed - 4 mph)^2 for both flat and hump yards have been plotted. [For each occurrence of coupling in an interval (e.g., 5 mph to 6 mph), it is assumed that the coupling speed is at the mean or midpoint of the interval (e.g., 5.5 mph).] Using the assumption of Equation 9, the relative damage in flat versus hump yards is proportional to the areas under the curves in Figures 3b and 4b, respectively. In particular, the area under Figure 3b representing damage in flat yards is 0.508, and the area under Figure 4b representing relative damage in hump yards is 1.718. Therefore, substituting these values into Equations 7 and 8 gives

Total 1978 Categories 3 and 8 L&D associated with the time spent in hump yards
$$= \{0.2\ (1.718)/[0.8\ (0.508) + 0.2\ (1.718)]\}\ \$25.3\text{ million} = \$11.6\text{ million} \quad (10)$$

Average 1983 L&D per loaded trip associated with time spent in hump yards
$$= \{0.2\ (1.718)/[0.8\ (0.508) + 0.2\ (1.718)]\}\ \$1.35 = \$0.62 \quad (11)$$

The corresponding cost associated with flat yards is simply found as follows:

Total 1983 Categories 3 and 8 L&D associated with time spent in flat yards
$$= \$25.3\text{ million} - \$11.6\text{ million} = \$13.7\text{ million} \quad (12)$$

Average 1983 L&D per loaded trip associated with time spent in flat yards
$$= \$1.35 - 0.62 = \$0.73 \quad (13)$$

Equations 11 and 13 indicate the average L&D per loaded trip associated with time spent in hump and flat yards, respectively. A more interesting statistic would be the average L&D per hump yard coupling or per flat yard coupling. The calculation is performed as follows: Petracek et al. (7) indicate that a loaded car, on the average, goes through six yards on its loaded trip journey. Because a loaded car is assumed to spend 20 percent of its yard time in hump yards and 80 percent of its yard time in flat yards (7), it is assumed that a loaded car on the average goes through 1.2 hump yards (i.e., 1.2 = 0.2 x 6) and 4.8 flat yards (i.e., 5.8 = 0.8 x 6). If it is further assumed that a car has only one coupling per hump yard or flat yard (i.e., it is assumed that the number of rehumped or reswitched cars is small), then using the results from Equations 11 and 13

Average 1983 L&D per hump yard coupling
$$= \$0.62/.2(6) = \$0.52 \quad (14)$$

Average 1983 L&D per flat yard coupling
$$= \$0.73/.8(6) = \$0.15 \quad (15)$$

Allocation to Hump Yard Overspeed Impact Levels (Step 5)

In the previous section it was calculated that the average L&D per hump yard coupling is $0.52. Using this average value, the expected L&D associated with various levels of coupling speed can be calculated in the following manner.

Figure 4a shows the frequency of occurrence of coupling impact speed for the following intervals: less than 4 mph, 4 to 5 mph, 5 to 6 mph, ..., greater than 10 mph. The percentages in Figure 4a are interpreted as probabilities of occurrence. It is also assumed that all couplings occurring between 4 and 5 mph take place at the mean interval value of 4.5 mph; similarly it is assumed that couplings in the other intervals occur at the mean interval values of 5.5 mph, 6.5 mph, ..., 10.5 mph. (It is assumed that cars coupling at speeds greater than 10 mph all couple at 10.5 mph.) Let $D_{4.5}$, $D_{5.5}$, ..., $D_{10.5}$ represent the unknown value of L&D due to couplings at the mean interval values of 4.5 mph, 5.5 mph, ..., 10.5 mph. The definition of average (or expected value) allows the following equation to be written:

Average 1983 L&D per hump yard coupling
$$= \$0.52 = .613(0) + .156_{4.5} + .127D_{5.5} + .04D_{7.5} + .011D_{8.5} + .01D_{9.5} + .003D_{10.5} \quad (16)$$

where the probabilities are taken from Figure 4a, and $D_{4.5}$, $D_{5.5}$, ..., $D_{10.5}$ are the unknowns. Note that the 61.3 percent of the cars that couple at less than 4 mph are assumed to have "zero" L&D.

To aid in the solution of Equation 16, the assumption stated in Equation 9, namely that damage is proportional to the squared difference between coupling speed and 4 mph, is used again. Using this assumption, the following relationships can be written:

$$D_{5.5} = [(5.5 - 4)^2/(4.5 - 4)^2]D_{4.5} = 9D_{4.5} \quad (17)$$

$$D_{6.5} = [(6.5 - 4)^2/(4.5 - 4)^2]D_{4.5} = 25D_{4.5} \quad (18)$$

$$D_{7.5} = [(7.5 - 4)^2/(4.5 - 4)^2]D_{4.5} = 49D_{4.5} \quad (19)$$

$$D_{8.5} = [(8.5 - 4)^2/(4.5 - 4)^2]D_{4.5} = 81D_{4.5} \quad (20)$$

$$D_{9.5} = [(9.5 - 4)^2/(4.5 - 4)^2]D_{4.5} = 121D_{4.5} \quad (21)$$

$$D_{10.5} = [(10.5 - 4)^2/(4.5 - 4)^2]D_{4.5} = 169D_{4.5} \quad (22)$$

Equations 17-22 can be substituted into Equation 16 yielding one equation and one unknown, $D_{4.5}$, as follows:

$$\$0.52 = .152D_{4.5} + .127(9)D_{4.5} + .04(25)D_{4.5} + .04(49)D_{4.5} + .011(81)D_{4.5} + .01(121)D_{4.5} + .003(169)D_{4.5} = 6.867D_{4.5} \quad (23)$$

By solving Equation 23 for $D_{4.5}$ and using Equations 17-22, expected L&D can be calculated for var-

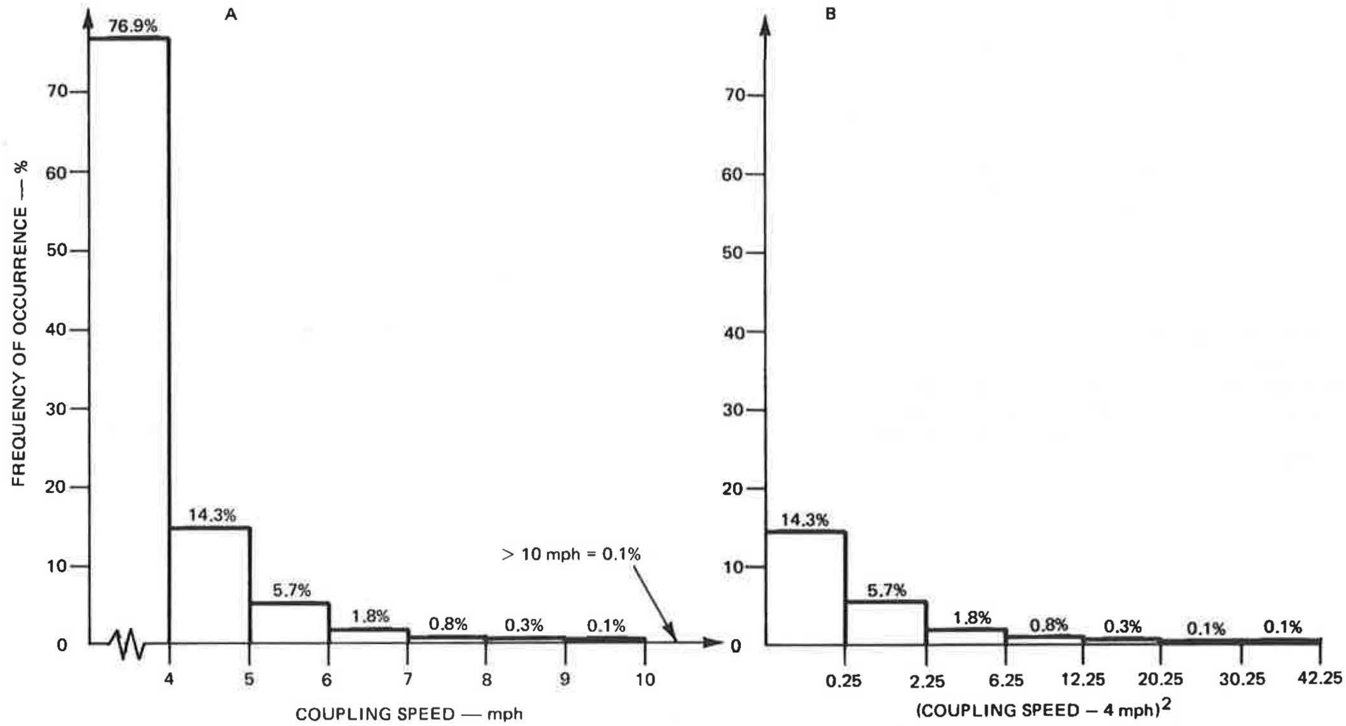


FIGURE 3 Frequency of flat yard impacts for coupling speed and (coupling speed – 4 mph)^2.

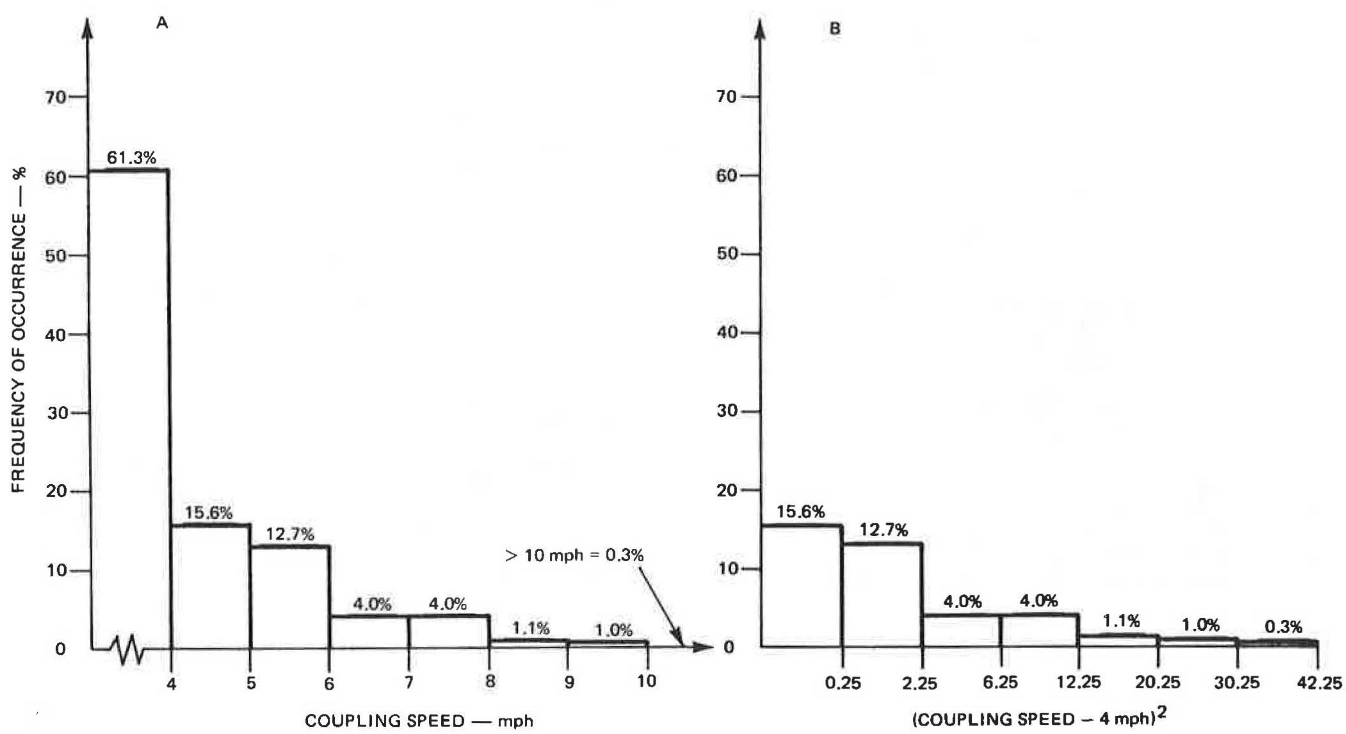


FIGURE 4 Frequency of hump yard impacts for coupling speed and (coupling speed – 4 mph)^2.

TABLE 4 L&D Versus Overspeed Impact

Overspeed Impact	Expected 1978 L&D per Occurrence ($)
4 mph	0.00
4-5 mph	0.08
5-6 mph	0.68
6-7 mph	1.90
7-8 mph	3.71
8-9 mph	6.14
9-10 mph	9.17
10 mph	12.81

ious levels of overspeed impacts; the results are given in Table 4.

Figure 5 shows a summary of the findings about allocation of payouts to the various causes shown in Figure 1. The dollar amounts are obviously only as good as the sketchy data used and are shown to illustrate the method.

CONCLUSIONS AND RECOMMENDATIONS

A methodology for allocating loss and damage costs to shock and vibration (i.e., rough handling) in various elements of the transport cycle has been presented. Although the quantitative estimates are important, the methodology itself, viewed as a "prototype," is more important. It is likely that the methodology can be refined to give more precise estimates with more definitive empirical data. In particular, the methodology presented here could form the basis of an experimental plan to obtain more refined estimates of L&D costs.

Because the magnitude of loss is large when both direct and indirect costs are considered, it is clear that the potential for improvement is great and that continued effort should be made to develop countermeasures to reduce the loss and damage due to shock and vibration in line haul and in yards (especially hump yards).

Research is needed to obtain better data. Data from other than National Car Handling Day should be used. Frequency of impact today should be deter-

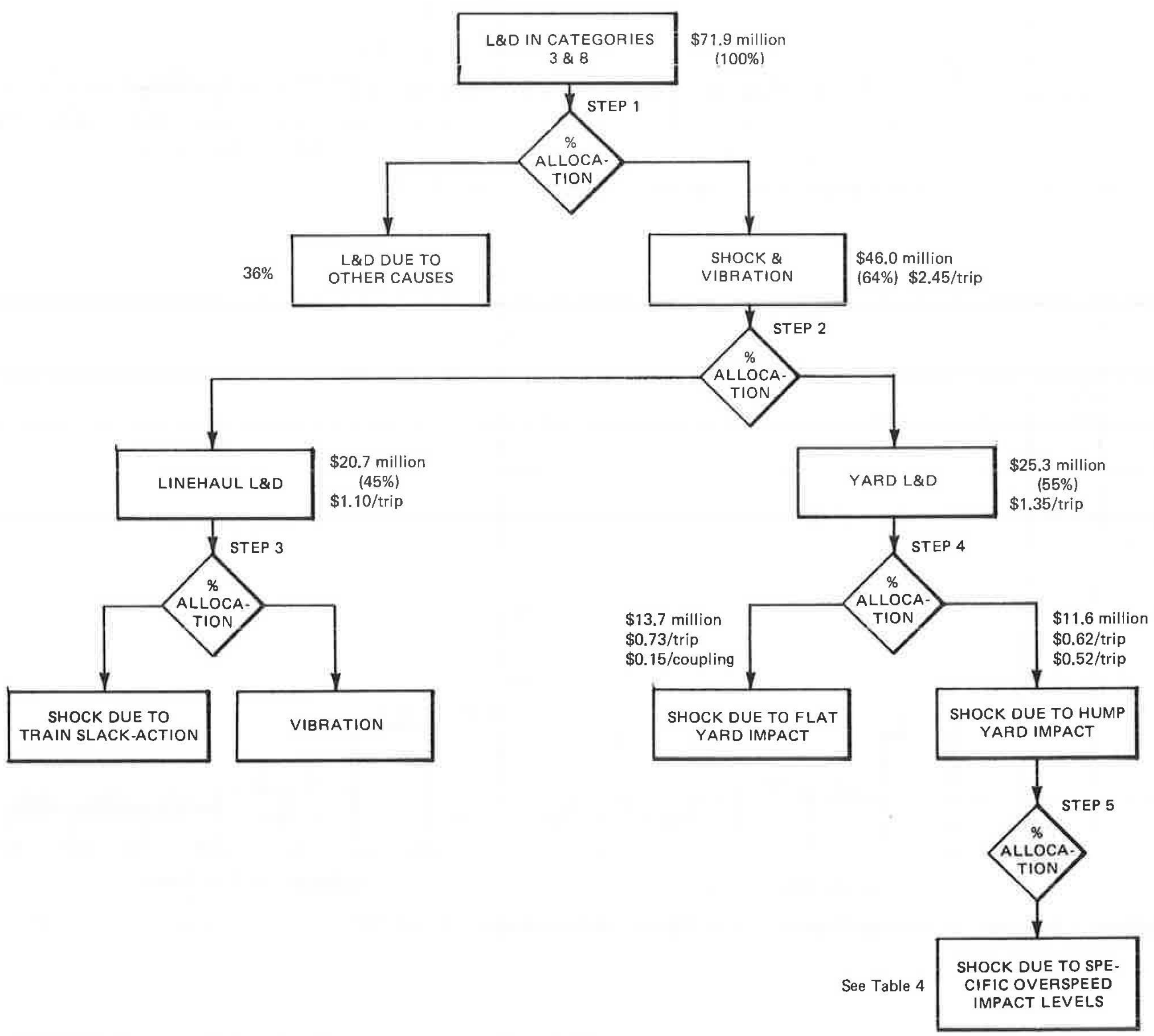


FIGURE 5 Summary of allocation of payments to causes shown in Figure 1.

mined. Obviously the only data available may not represent today's practices so up-to-date data are needed to validate the procedure. Efforts should be made to determine if shifted loads are more susceptible to damage than loads that have not shifted.

An extensive bibliography on loss and damage is presented elsewhere (9).

ACKNOWLEDGMENTS

This research was supported by the Federal Railroad Administration (FRA). The project technical monitor was Claire Orth. The author wishes to acknowledge the support and encouragement of Phil Olekszyk (FRA) and William Cracker (formerly of FRA). Valuable information to the project was supplied by Barney Gallacher (retired, Southern Pacific Transportation Company) and Robert Robins (Weyerhauser Company).

REFERENCES

1. W.S. Mielziner. Shippers and Carriers Find New Ways to Control Damage Through Utilization of Impact Recorders. Traffic World, March 26, 1966.
2. Freight Loss and Damage, 1983. Freight Claim and Damage, Association of American Railroads, Chicago, Ill., 1983.
3. F.E. Ostrem and W.D. Godshall. An Assessment of the Common Carrier Shipping Environment. Report FPL 22. Forest Products Laboratory, Madison, Wis., 1979.
4. Report on Survey of Whirpool Corporation by Task Force for Prevention of Damage to Household Appliances. Freight Loss and Damage Prevention Section, Association of American Railroads, Washington, D.C., 1970.
5. An Economic Model of Cargo Loss: A Method for Evaluating Cargo Loss Reduction Programs. Report PB-210-223. Braddock, Dunn and McDonald, Inc., McLean Va.; U.S. Department of Transportation, May 1972.
6. Yearbook of Railroad Facts: 1983 Edition. Economics and Finance Department, Association of American Railroads, Washington, D.C., 1983.
7. S.J. Petracek et al. Railroad Classification Yard Technology: A Survey and Assessment. Report FRA/ORD-76/304. SRI International, Menlo Park, Calif.; Transportation Systems Center, U.S. Department of Transportation, Cambridge, Mass., July 1976.
8. L.C. Simmons and R.H. Shackson. Shock and Vibration of Railroad Movement of Freight. Presented at the American Society of Mechanical Engineers Winter Annual Meeting, New York, New York, Nov. 29-Dec. 4, 1964.
9. P.J. Wong. Allocating Loss and Damage to the Railroad Transport Cycle. Report FRA/ORD-81/64 (NTIS No. PB82-195587). SRI International, Menlo Park, Calif.; FRA, U.S. Department of Transportation, Aug. 1981.

Publication of this paper sponsored by Committee on Rail Freight Classification Terminal Design.

Economic Design Methods for Automated Miniyards

ARTHUR W. MELHUISH

ABSTRACT

Changing traffic patterns and operating methods will continue to reduce the number of cars to be classified in yards. This trend promotes a need for economically designed, built, and operated miniyards. Such small-scale yards can be designed in ladder track or balloon formation, both with minihumps and suitable for 1,000 to 2,000 cars per day throughput. To attain low-cost, efficient operation of these yards they will need to be automated in an economical manner with automatic route setting and simple car speed control. The system described in this paper could control the humping procedure to give continuous, discontinuous, and manual modes of car throughput as appropriate to the measured rollability category and track address for each car.

The concept discussed here is that of an unpretentious economic electronic system to both detect and signal on the hump, and to interface with the automatic route setting.

Significant cost savings in retarder control can be made in the initial design stages by

- Using a minimum practical design rollability value (1) and
- Adopting a double standard for car running performance (i.e., accepting those cars that will only reach clearance along with those that will sustain separation).

To compensate in operation for the dilution of the initial design criteria, supplementary operating aids are proposed to

- Control car separation in ladder track yards (in coordination with the route setting controls) by determining car release interval periods according to track destination and thus signal the release of each car and
- Detect cars that have rollability values outside the design bandwidth and then signal appropriate actions.

LADDER TRACK YARD

General

The throughput in ladder track yards depends on operator experience and judgment in the cutting to ascertain adequate separation.

Although some degree of performance is achieved by operator knowledge of destination and by observation, it is thought that the operation, and thereby throughput efficiency, could be enhanced by providing a suitable timing and signal aspect system to control the cutting sequence.

The criteria for the use of the system would be

- Continuous retarder control imposed in the yard to administer the speed of the cars and
- The movements of point switches supervised by automatic route setting.

Such a system could not be applied in a yard where the car speed would not be controlled, but, by imposing retarders to continuously control the speed of all cars to a known value, it is possible to predict the initial separation period needed for the various switch destinations.

With the advent of small self-contained retarders of the Dowty type, which can be installed through the turnouts, it is now possible to impose such retarder control in ladder track yards. The Dowty-type retarder is a small, self-contained hydraulic unit that is quite different in concept and application from the large clasp retarders that have traditionally been used in North American yards.

The purpose of the system would be to create a controlled initial separation between cars at the beginning of the run so that the last ladder track switch, common to the routes for two consecutive cars, could be operated.

The initial separation period would also be kept to a minimum to promote a good throughput rate. In addition, with the speed measurement facility within the system, cars with rollability factors outside the design parameters could be detected and appropriate actions initiated.

Economic Design Parameters

The cost of the retarder equipment in a yard is proportional to

- The car throughput rate. For this type of yard, which is only intended to handle low throughput rates, this value will be low, on the order of 1,000 to 2,000 cars per day.
- The maximum distance from hump crest to clearance marker. This distance is exceptionally high in ladder track formations but can be made acceptable by using continuous speed control and by employing supplementary aids to signal the discontinuous humping moves dependent on car destination.
- Allowable car separation. This distance can be kept to a minimum by employing car detection devices in place of track circuits that are dependent for length on the maximum distance between trucks.
- Maximum axle weight. This value is considered standard for all yards.
- Maximum rollability value. This factor has a prominent effect on yard cost and performance and therefore needs to be kept as low as practicable in the design stages. For the purposes of discussion, the yard shown in Figure 1, with 2 lb per ton minimum rollability and dual maximum values of 5 lb per ton to sustain separation and 8 lb per ton to reach maximum clearance, is assumed. To enable a practical operation to be based on such a narrow design rollability bandwidth, a supplementary operating system could be used to categorize rollability, determine car address, and signal appropriate operating modes and actions.

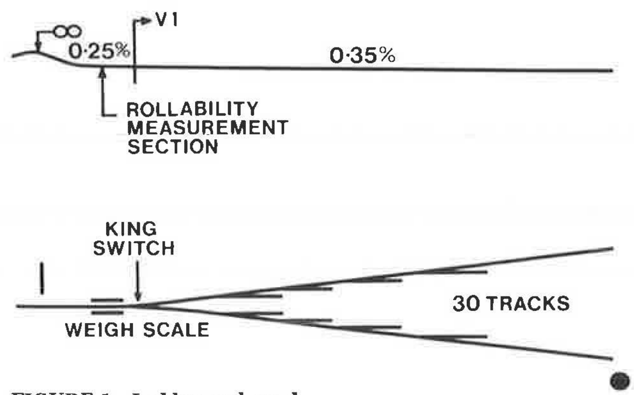


FIGURE 1 Ladder track yard.

Description of Yard Design and Performance

Figure 1 shows a ladder track yard made up of 30 class tracks with an accelerating hump and a weigh-scale track.

Cars would be cut loose at the apex to accelerate down the hump and over the weigh scale to arrive at the King switch with velocity V_1. The weigh-scale track would have a gradient of 0.25 percent so that a 5 lb per ton rollability car would traverse it at constant velocity.

Retarders would be installed on the hump and in the King switch to control the maximum speed to V_1. This retarder control would continue throughout the switches on the ladder lead tracks so that the nominal velocity (V_1) is maintained throughout.

A profile would be selected to ensure that

- A 5 lb per ton rollability car would accelerate to V_1 and continue with constant velocity along the ladder lead tracks and

• An 8 lb per ton rollability car would roll past the farthest clearance marker.

After establishing a constant nominal velocity, by applying continuous control, it is possible to construct a time-distance curve, as shown in Figure 2, to establish the variable release interval periods (RIPs). RIPs can be established for every switch, but for illustrative purposes a simplified method has been adopted here that uses only three different periods (i.e., RIP minimum, RIP mean, and RIP maximum) compatible with the minimum, mean, and maximum distances to run.

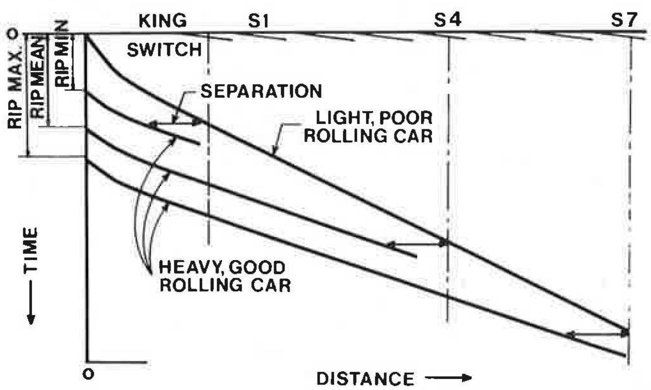


FIGURE 2 Time-distance curves.

Conceptual Study of Supplementary Operating Aids

Suitable process control programs for the supplementary operating aids and the route progression system would need accommodation within a minicomputer with a suitable timer appended. An interface would be needed to receive signals from three car detectors and a manual switch and to transmit commands to a two-aspect color light signal (Figure 3).

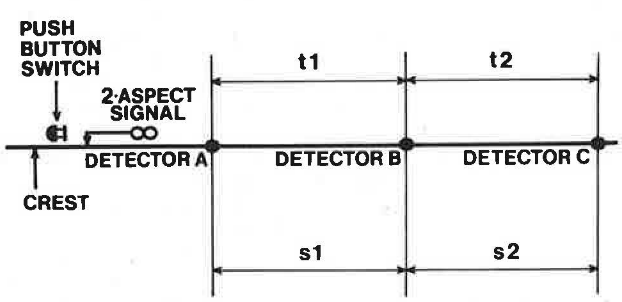


FIGURE 3 Signal and detection equipment—hump track.

Two consecutive timed sections of equal length (S1 and S2), would be located between the hump and the King switch. The measurement of the time taken for a car to traverse the first section would be stored and used as the base time (t_1). The measurement of the time taken (t_2) for a car to traverse the second section would be compared with the recording from the first section to determine the car rollability category.

If $t_2 > t_1$, the car would be exhibiting a higher rollability tendency than that acceptable within the design for sustaining separation. (Note that the gradient of 0.25 percent on the weigh-scale track is equal to the design rollability ratio of 5 lb per ton.) In this event the system would adopt the manual mode and the operator would visually monitor the car's progression through the yard.

The basic operation shown in Figure 3 is envisaged as follows:

1. Signal aspect at green. Leading car can be released.
2. Detector A activated. Start first timing period (t_1) and the RIP signal aspect to red. Interrogate route setting program to obtain addresses of leading and following cars.
3. Determine last common ladder track switch for both car destinations. If common switch is King, select minimum RIP; if switches S1-S4, select mean RIP; and if switches S5-S7, select maximum RIP (Figure 2).
4. Detector B activated. End first time period (t_1) and start second time period (t_2).
5. Detector C activated. End second time period (t_2).
6. Compare t_2 with t_1 to ascertain rollability category.
7. If valid, go to Step 8. If invalid go to Step 9.
8. At end of selected RIP, signal aspect to green. Next car can be released.
9. Adopt manual mode. Activate flashing red alarm signal aspect. Operator to monitor car clear through system.
10. Operator activates manual push button switch. System reverts to automatic mode with signal aspect at green. Next car can be released.

Additional Facilities for Consideration

In this study the aim has been simplicity in the system design. With an expanded design study, preferably for a nominated project, it is believed that a practical system could be attained. The need to achieve a simple design with economy of costs is fully recognized. But, on the other hand, it is also recognized that in development the system could be extended to provide additional refinements, such as

1. Radar speed measurement could be used to measure car velocities from which acceleration could be determined and thus rollability factors for an enhanced number of categories.
2. Cars indicating a rollability value above 8 lb per ton could be detected and rejected from running through the yard by use of a reject track. The reject track could be via S1 (Figure 2) or constructed by adopting a lap switch for the King switch position.
3. The process control program would include an RIP for each individual switch; this could be further embellished by introducing a maximum rollability value appropriate for the distance to run to each switch and judging each car's compatibility during operation.
4. The theoretical RIPs should be adjustable in the commissioning stage in order to make allowance for operator reaction time.

BALLOON FORMATION YARD

General

This type of yard layout was originally adopted to overcome the large difference between maximum and minimum distances to the switches experienced in ladder track yards. A balloon yard is designed for higher car thoughput rates than is a ladder track yard, and this throughput would be achieved by employing a constant humping velocity. Therefore the RIP associated with discontinuous humping is not needed. However, supplementary operating aids could be usefully employed to determine rollability categories and thus accept or reject cars as applicable.

A suitable car retarder system would need to be employed to ensure car separation in the switching area and to control overall car performance, together with automatic route setting.

For normal operations the humping speed would be constant and the separation sustained for cars having average (R avg) rollability values. A maximum design rollability factor (R max) could be determined to ensure that all cars within this limit clear the switching area. By measuring the rollability and categorizing at the hump, the following appropriate actions could be signaled.

1. The majority of cars will have average rollability values and the humping process will be continuous.
2. If the rollability value is above average but below maximum the humping would stop for the operator to visually monitor the car's progress through the switching area.
3. If the rollability value should be above maximum the car could automatically be switched to a reject track to prevent congestion in the switching area.

Predicted Economies

Example 1

Figure 4 shows the basic features of a small balloon yard that should be capable, in approximate terms,

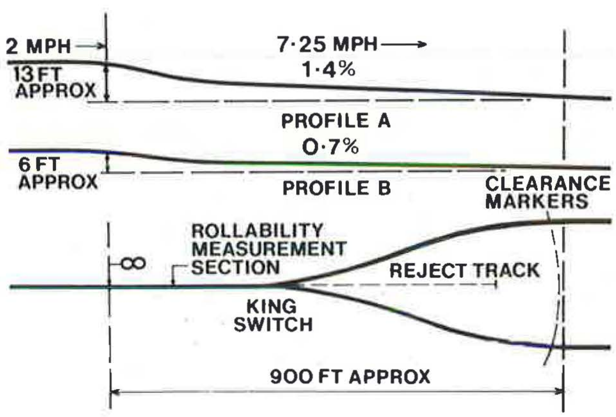


FIGURE 4 Track diagram and switching area profiles.

of handling 180 cars per hour over the hump, which, with an operating efficiency of 40 percent, should result in a throughput of 1,700 cars per day.

By adopting a design rollability ratio of 28 lb per ton maximum, it would be possible to cater to the car performance of 100 percent of the fleet. To cater to this maximum rollability ratio, and to sustain separation for the car rollability bandwidth of 2 lb per ton to 28 lb per ton, the hump crest would need to be on the order of 13 ft above the clearance markers (profile A in Figure 4). Approximately 12.2 ft of retardation energy head would be needed to control the heavy, low rollability cars in the switching area.

Example 2

By adopting rollability values of 2 lb per ton minimum and 10 lb per ton to sustain separation, 90 percent of the car population could be serviced. If a 14 lb per ton maximum, to reach clearance, were adopted, an additional 6 percent of the car population could be serviced. If supplementary operating aids were applied to assist the humping process, 90 percent of the cars could be continuously humped at 2.0 mph and a cumulative 96 percent of all cars would pass clearance. Four percent of the cars (i.e., those with above 14 lb per ton rollability) could be switched to a reject track to avoid stalling in the switching area. Continuous humping would be interrupted for 10 percent of the cars, when a manual mode of operation would be adopted.

If it is assumed that a car would take 1 min to clear the switching area, with the humping stopped, the average throughput could be on the order of 168 cars per hour or 1,600 cars per day when operating at 40 percent efficiency.

A hump height of only 6 ft above the clearance markers would be needed to cater to the rollability bandwidth of 2 lb per ton to 14 lb per ton (profile B in Figure 4). Approximately 5.2 ft of retardation energy head would be required in the switching area to control the heavy, low rollability cars.

A comparison of the examples reveals that the diluted design criteria employed in Example 2, compensated for by the application of the supplementary operating aids, could achieve an estimated 43 percent saving in the required switching area retardation energy (i.e., retarder costs) for only a 5.9 percent reduction in car throughput. Because of the variations in different types of retarder system performance and price, and also the international variations in exchange rates and in labor and material costs for supply, shipment, and installation, no attempt has been made to convert the 43 percent saving in retardation energy into a monetary value.

CONCLUSION

In this paper the aim has been to describe the proposed system in basic and simple terms. There is no doubt that the system could be enhanced and expanded to include more rollability categories, distances to switches information, and car performance data.

Traditionally, ladder track yards and small hump yards, where high throughputs are not required, have been designed to rely heavily on manual operation. In recent years, some automatic route setting systems have been employed in these types of yards, and perhaps now, with the addition of a car retarder speed control system, the way is open to employ new methods to improve operating efficiency.

A system of supplementary operating aids as described herein could be a way to reduce the initial capital cost of miniyards and might thereby encourage both designers and operators to adopt full automation for small, low-throughput yards and thus reap the benefits of improved operating efficiency.

REFERENCE

1. Railroad Classification Yard Technology Manual, Vol. 3: Freight Car Rollability. Office of Research and Development, FRA, U.S. Department of Transportation, 1982.

Publication of this paper sponsored by Committee on Rail Freight Classification Terminal Design.