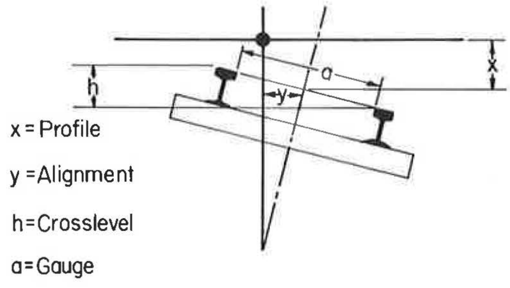

FIGURE 8 Track geometry measurements *(4)*.

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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

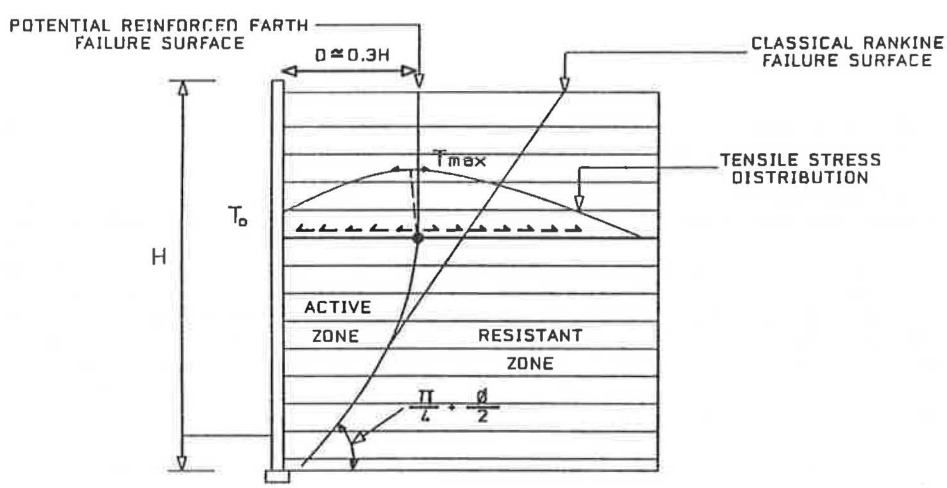


FIGURE 1 Tensile forces distributed along reinforcements.

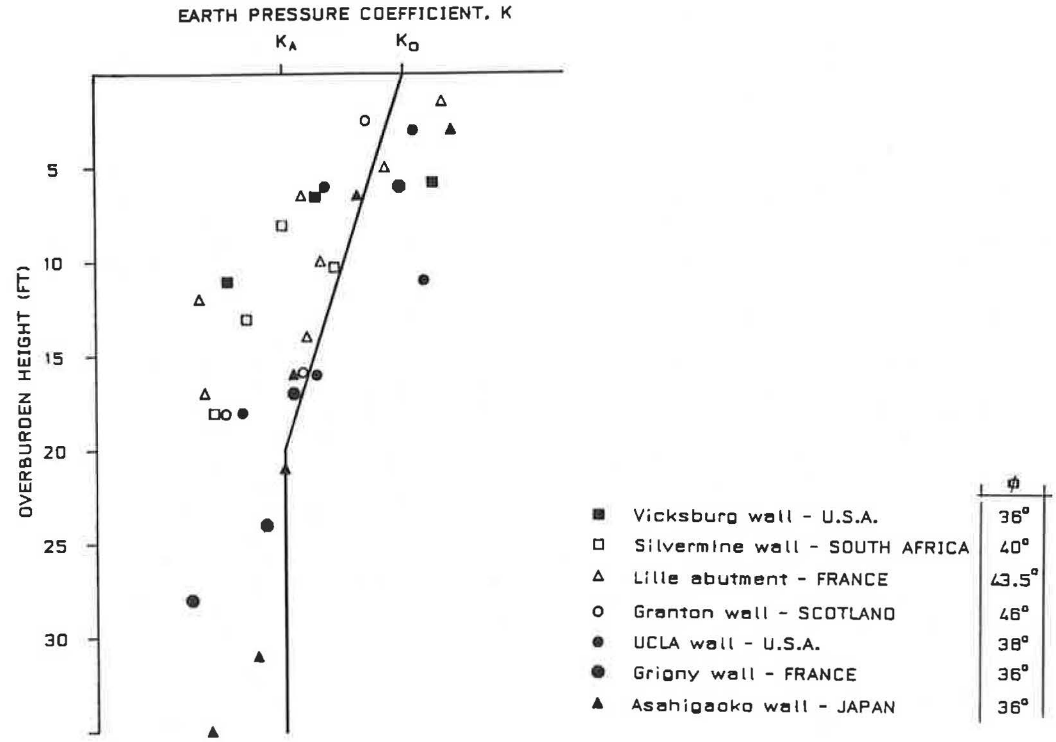


FIGURE 2 Variation of K from instrumentation of actual structures.

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).

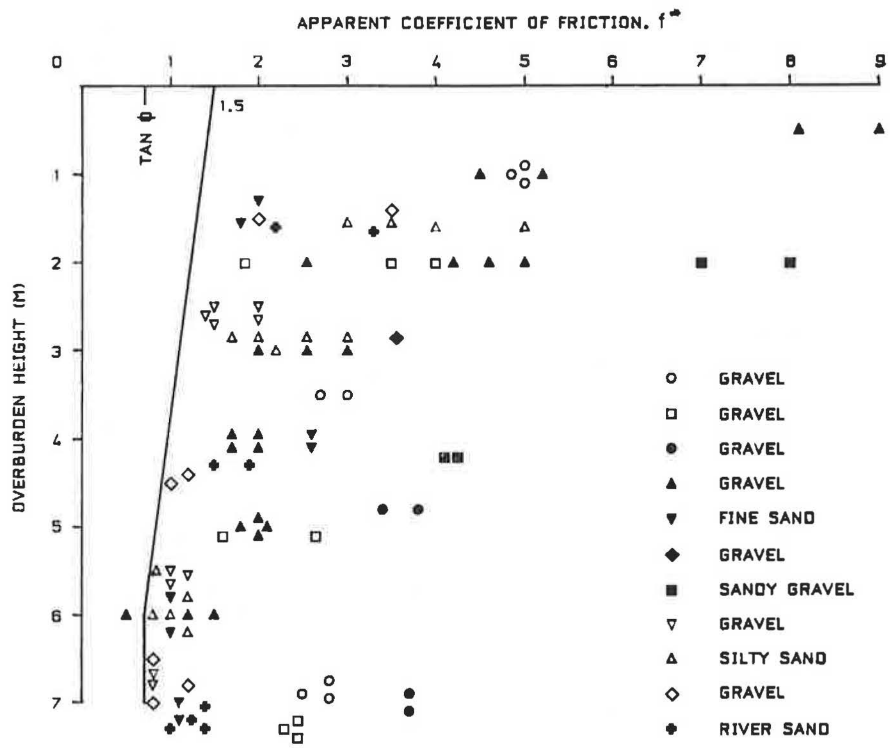
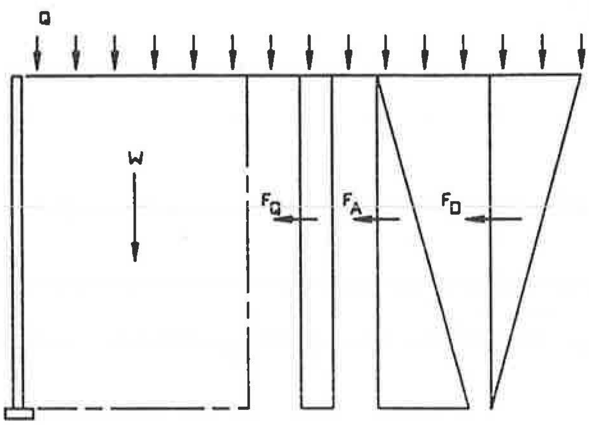


FIGURE 3 Values of apparent coefficient of friction (f*) from pullout tests.

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

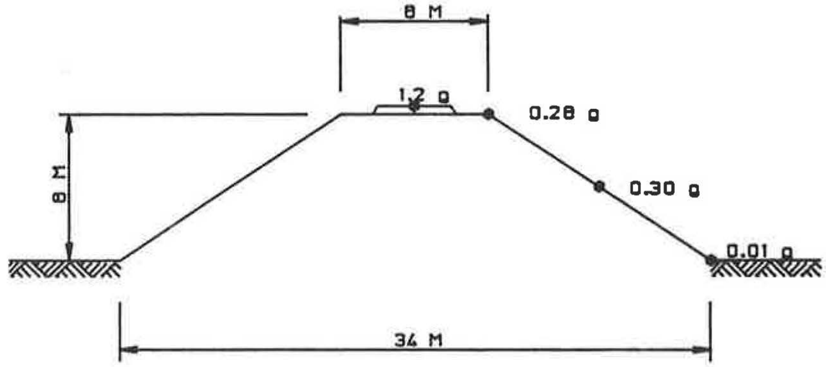


FIGURE 5 Vertical accelerations at the surface of an instrumented SNCF railroad embankment.

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-

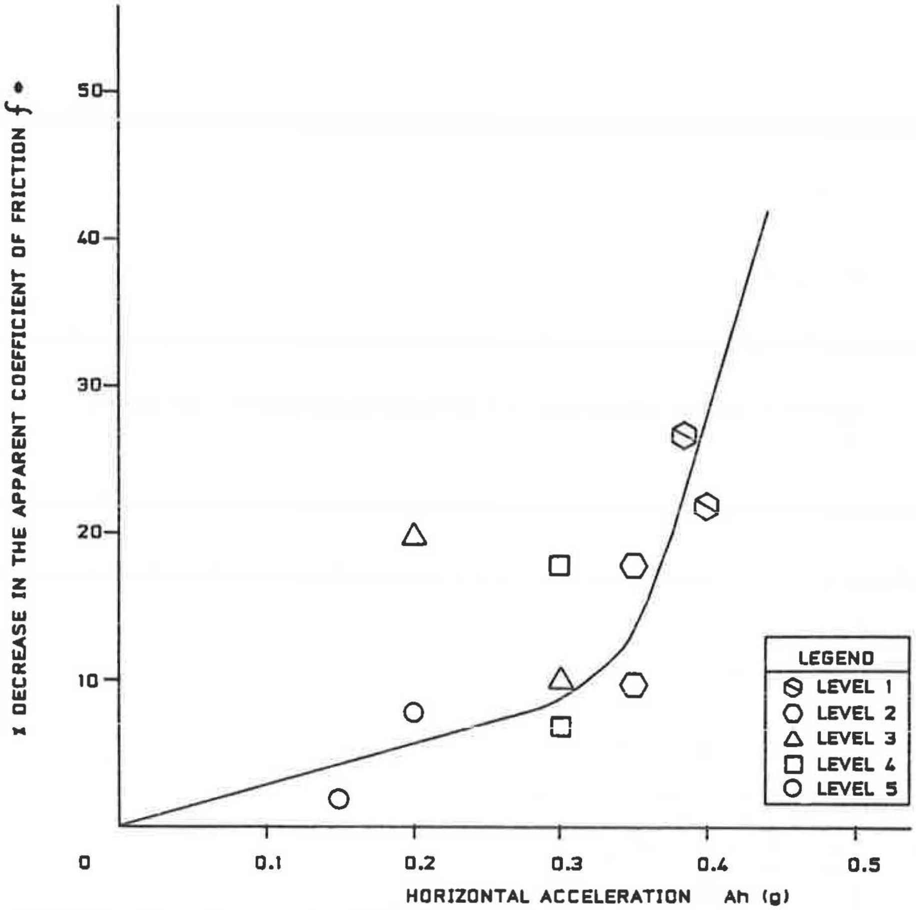


FIGURE 6 Effect of horizontal acceleration on apparent coefficient of friction.

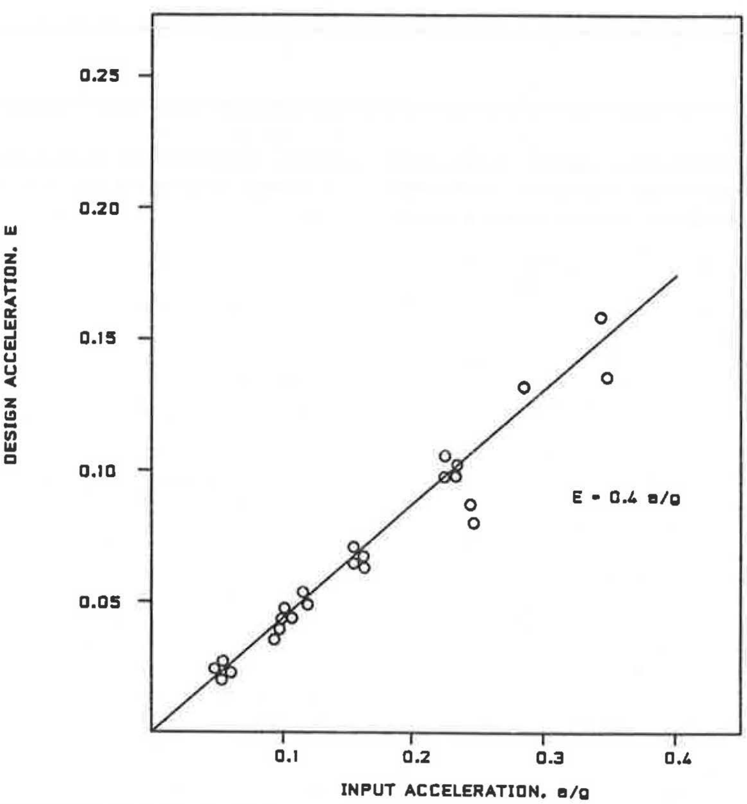


FIGURE 7 Relationship between design acceleration and input acceleration for Reinforced Earth walls.

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

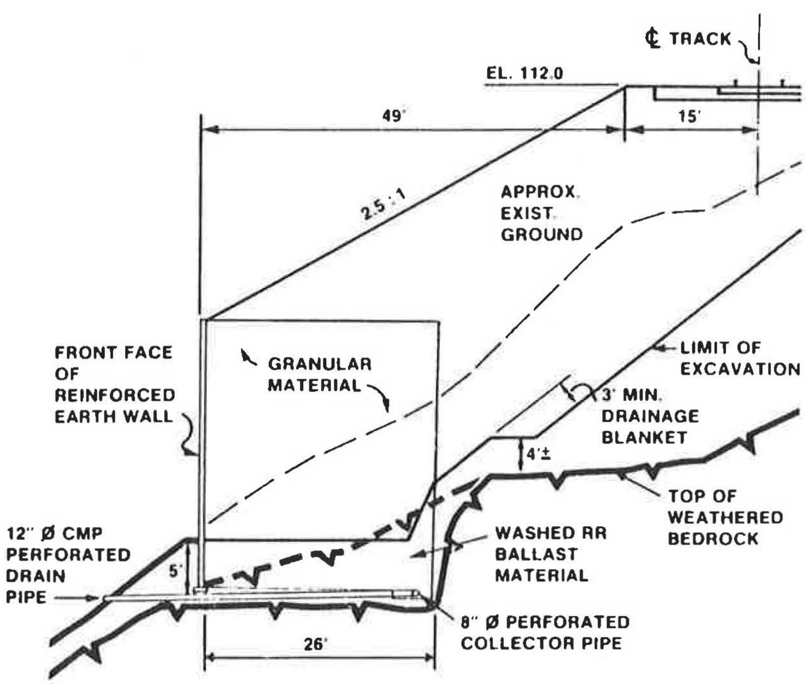


FIGURE 8 Typical cross section of Clinchfield stabilization project.

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.

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