Implementation of a Bearing Capacity Design Procedure for Railway Subgrades: A Case Study

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Application of a bearing capacity design approach to evaluating railway subgrades is illustrated. The bearing capacity design approach was developed as part of an overall research program to assess the stability of track subgrades. The development of the design procedure was published earlier. The additional strength of the subgrade soil resulting from matric suction was incorporated into the bearing-capacity design procedure. The bearing-capacity design procedure was used as a complementary tool in conjunction with stress analyses and slope stability analyses for the Canadian Pacific Railways Floodway Trackage, Emerson Subdivision, Winnipeg, Manitoba, Canada. Stresses in the subgrade were predicted by using the GEOTRACK computer program. The ultimate bearing capacity was determined by using bearing-capacity theories modified to accommodate layered track systems and the additional strength of the soil resulting from soil suction. Slope stability modeling of the embankment slopes completed the analyses. Results of the complementary analyses were used to select appropriate design alternatives.

A bearing-capacity approach to railway design was developed at the University of Saskatchewan, Saskatoon, Canada, as part of an ongoing investigation into the stability of track subgrades (*1,2*). The design approach combines a computer stress analysis, using the computer program GEOTRACK, with conventional bearing capacity modified for layered systems and the influence of soil suction. Comparison of subgrade stress with subgrade strength (i.e., bearing capacity) provides a measure of the factor of safety against failure. Development of the design procedure has been documented earlier (*1,2*).

The objective in the development of the bearing capacity design procedure was to incorporate the soil suction term into a bearing-capacity approach to subgrade design. Soil suction, or matric suction, is a fundamental stress state variable defined as negative pore-water pressure u_w referenced to pore-air pressure u_a (*3*). Incorporation of soil suction into the bearing capacity design procedure permits the design procedure to use the additional strength of the soil that results from subgrade soil suction. Changes in subgrade strength owing to ponding or long-term evaporation may be quantified through the use of the soil suction term.

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The bearing-capacity design procedure, incorporating subgrade soil suction, was used in the analysis of a railway embankment near Winnipeg, Manitoba, for Canadian Pacific Railways Ltd.

Winnipeg, which is situated on a highly plastic lacustrine clay of glacial Lake Agassiz at the confluence of the Red and Assiniboine rivers, has been plagued by intermittent flooding since 1826 (*4*). The Red River Floodway was constructed in the late 1960s to divert the floodwaters of the Red River around the Greater Winnipeg Area (Figure 1). The Canadian Pacific (CP) rail trackage was relocated in 1966 and 1967 (*5*) as part of the floodway project. The level of the track was raised approximately 3 m and a bridge was constructed across the floodway channel to cross the floodway. The floodway trackage embankment was designed with a 6.7-m top and 4:1 side slopes and was constructed of locally available highly plastic clay (*5*).

Maintenance was required to alleviate subgrade problems within a year of construction. Slope indicators installed in 1969 recorded movements of 22 mm/yr for 4 years before they were destroyed. Both shallow and deep-seated slip surfaces have been identified, and water has been observed to pond in depressions below the ballast. French drains and berms have been used over the years to attempt to stabilize the subgrade. Increased train loads and traffic volumes have aggravated embankment instability problems. Daily track lifting and realignment during wet periods were required on the floodway trackage, resulting in costly maintenance and the danger of derailment (*5*).

Analyses indicate that within the immediate vicinity of the floodway bridge the subgrade soil has been sheared past its peak strength and only a low residual strength is being mobilized. Figure 2 illustrates the severity of both bearing capacity and slope stability failures through the embankments near the floodway bridge.

FIELD AND LABORATORY INVESTIGATIONS

Eight testholes were drilled adjacent to the floodway bridge in 1980 (*5*). Samples were collected for Atterberg limits, density measurements, consolidation tests, direct shear tests, and triaxial repeated loading tests. Analyses were performed and remedial measures were suggested but never adopted owing to financial constraints.

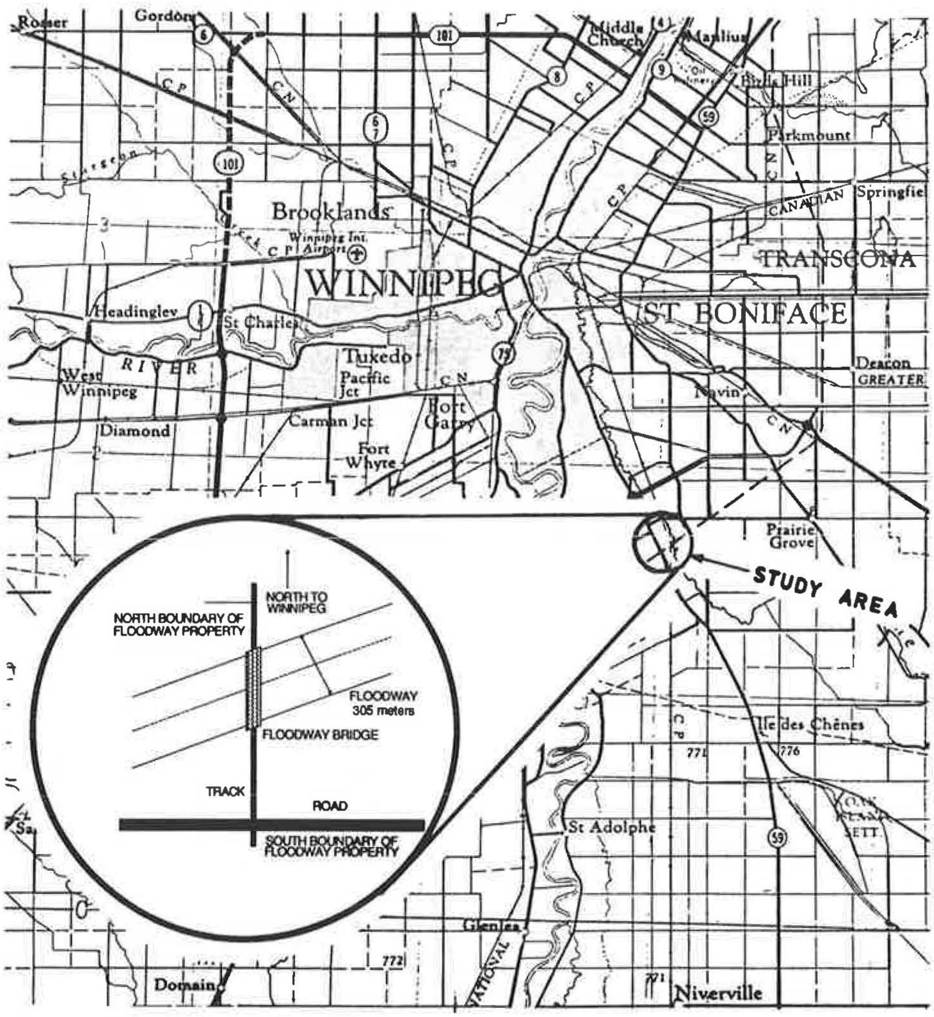

FIGURE 1 Location map of the study area.

FIGURE 2 Photograph of failures at the site.

Liquid limits ranged from 63 to 90 percent, and plastic limits varied from 18 to 29 percent. The average density was 1.95 g/cm^3. The swelling pressure, as measured from two consolidation tests, was about 275 kPa. The corrected swelling pressure was about 325 kPa. Results of five direct shear tests performed on the subgrade clay indicated a residual effective friction angle between 5.5 and 10.5 degrees with a cohesion intercept ranging from 5.5 to 20.0 kPa. Results of the triaxial repeated loading tests indicated an extreme variation in the resilient modulus value of 44.8 MPa to 177.8 MPa (*5*).

Subsurface conditions were again studied in March 1988 (*6*). The drilling program included 12 specified holes, plus 2 additional test holes and 6 holes for follow-up samples. The stratigraphic cross section interpreted from the test holes is presented in Figure 3. The ballast consisted of gravel ranging in size from 12 to 50 mm diameter, with an increasing sand fraction with depth. Thickness of the ballast varied from 0.20 to 0.45 m, averaging 0.30 m. Sub-ballast consisted of fine-to-medium sand with some gravel and a trace of silt. The sub-ballast thickness ranged from 0.45 to 1.55 m and averaged 1.15 m. Clay fill consisted of stiff to very stiff, highly plastic, mottled dark gray-green-to-black clay. Slicken-sided planes were abundant. Construction layering was also evident in some samples. A mixed zone between the clay fill and the sub-ballast was observed and consisted of a mixture of sub-ballast materials and softened clay fill. The natural strata consisted of an upper clay layer, a layered silt, a silty clay layer, and a highly plastic clay. The water table was identified approximately 5.5 m below the top of the ballast (*6*).

Fifty-two undisturbed Shelby tube samples and two bag samples were tested at the University of Saskatchewan. Index

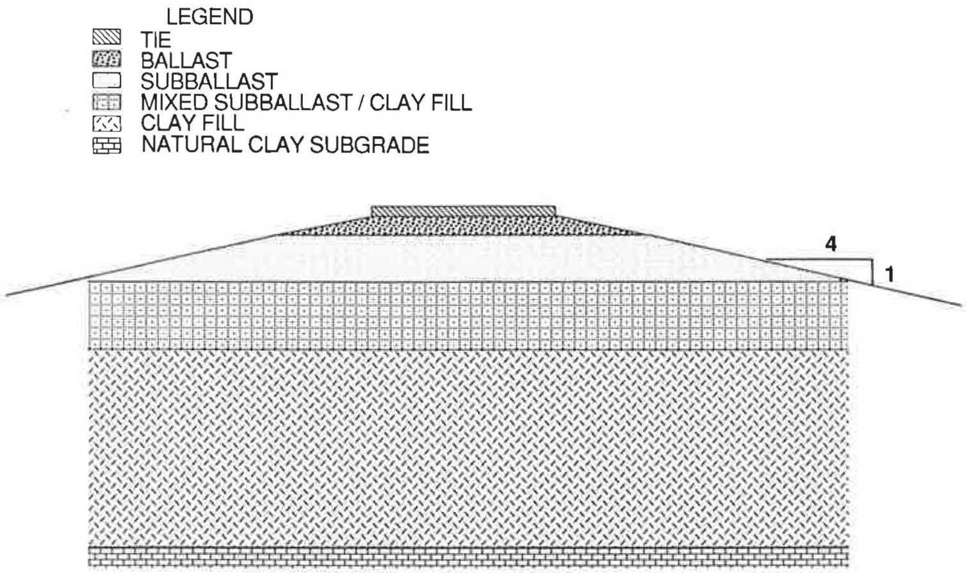

FIGURE 3 Stratigraphical cross section of embankment.

tests, matric suction measurements, and shear strength tests were performed.

The natural water contents varied between 23.3 and 52.6 percent with an average of 33.3 percent. Liquid limits varied between 65 and 98 percent with an average of 76 percent. Plastic limits varied between 21 and 29 percent with an average of 23 percent. The average plasticity index was determined to be 52 percent. The average specific gravity was measured and recorded as 2.71 (*7*).

Soil suction was measured in the laboratory on Shelby tube samples by using thermal conductivity sensors. A 125-mm (5-in.) long portion from the center of each Shelby tube sample was double wrapped in plastic film, confined with a layer of masking tape, and double wrapped in aluminum foil to prevent moisture loss during suction measurements (*8*). A thermal conductivity sensor, calibrated to measure soil suction (*9*), was installed into each sample (see Figure 4). Additional details on the use of thermal conductivity sensors to measure matric suction in the laboratory have been described by Sattler and Fredlund (*8*).

Measured negative pore-water pressures are illustrated in Figure 5. Measured values varied from 0 to 280 kPa. A tendency exists for the sample to expand in all directions when Shelby tube samples are released from the confinement of the Shelby tube. Expansion of the sample results in an increase in matric suction. The overburden pressure was subtracted from each measurement to compensate for this expansion, assuming a coefficient of lateral earth pressure at rest K_0 equal to 1 and a B pore pressure parameter equal to 1 (*8*). The resulting negative pore-water pressures are plotted in Figure 6. The measurements from all the test holes have been plotted on the same graph so the variability does not reflect the changes in any one test hole. The hydrostatic pore-water pressure line is illustrated in Figure 6 to aid interpretation.

Direct shear measurements were conducted to determine the rate of increase in shear strength for increasing soil suction ϕ^b. The angle measured in the laboratory was 25 degrees, which is slightly higher than the peak effective friction angle for the material. Interpretation of the results suggests that

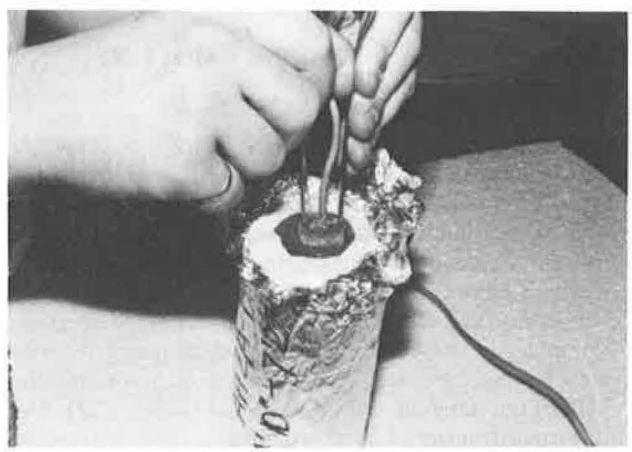

FIGURE 4 The installation of a thermal conductivity sensor.

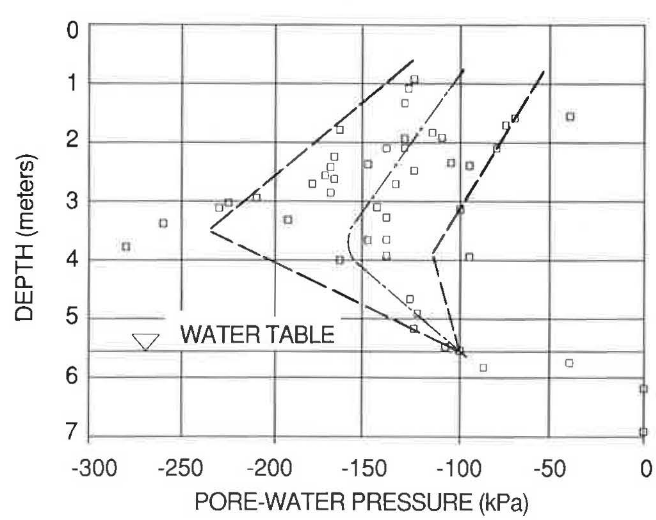

FIGURE 5 Measured negative pore-water pressures, using thermal conductivity sensors.

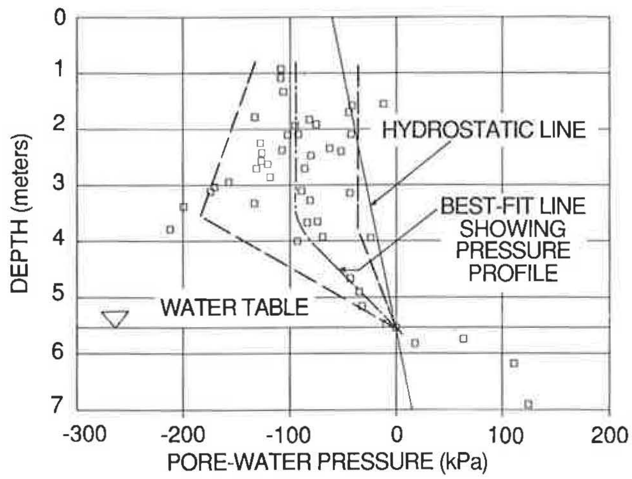

FIGURE 6 Negative pore-water pressures adjusted for overburden.

essentially the peak ϕ^b angle had been measured because research into appropriate values for ϕ^b suggest that ϕ^b should not exceed ϕ' (*7,10*).

STRESS ANALYSES AND BEARING CAPACITY ANALYSES

Railway design has been based on the concept of allowable stresses in the rails, ties, and subgrade. Most of the design aspects are quite empirical. The bearing capacity approach provides a measure against which subgrade stresses can be compared. The computer program GEOTRACK, documented by Chang et al. (*11*), was used for the prediction of stresses in the subgrade. The bearing capacity of the subgrade was computed by using conventional bearing capacity theories adapted for layered systems and then by incorporating the strength of the subgrade soil owing to soil suction (*1,2*). The bearing capacity factor of safety is defined as subgrade bearing capacity divided by subgrade stress, providing a comparative tool for analysis.

Input parameters required for the GEOTRACK model include the repeated loading moduli for the ballast, sub-ballast, and subgrade materials and for the two design materials, roller-compacted concrete and hot-mix asphalt. Reasonable values chosen to represent the ballast and subballast moduli were 241 MPa and 138 MPa, respectively (*11,12*). A design modulus value of 52 MPa was chosen to represent the softened upper layer of clay subgrade, and the lower stiffer layer was modeled with a modulus of 96 MPa on the basis of measurements conducted in 1980 (*5*). A value of 24,800 MPa was chosen for a repeated loading modulus for roller-compacted concrete (*13*), and a modulus of 8300 MPa was used for hot-mix asphalt (*14*). The GEOTRACK computer program, using those parameters, computed stresses at the subgrade surface for each of the design alternatives.

The conventional bearing capacity equation was used to compute subgrade bearing capacity q_u with the incorporation of soil suction as shown:

$$q_u = cN_c + \frac{1}{2}\gamma BN_\gamma + q_0N_q \tag{1}$$

where

$$c = c' + (u_a - u_w)\tan\phi^b \tag{2}$$

where

c = total cohesion,
c' = effective cohesion,
$(u_a - u_w)$ = soil suction or matric suction,
ϕ^b = rate of increase in shear strength with respect to soil suction,
γ = total unit weight,
B = bearing width,
q_0 = surcharge loading, and
N_c, N_γ, N_q = bearing capacity factors.

The effect of the ballast and sub-ballast layers were accommodated by using the concept proposed by Broms (*15,1,2*).

The parameters required for computing the subgrade bearing capacity include c', ϕ', ϕ^b and the design soil suction. Changes in pore-water pressure owing to train loading are considered to be insignificant relative to changes in pore-water pressure owing to the environment for an unsaturated soil where the pore-water pressures are negative. A design soil suction value was chosen to reflect the near-minimum suction expected over a period of several years. The design soil suction value for the subgrade was selected as the mean in situ suction minus 1 standard deviation. The selected value was equal to 55 kPa (see Figure 6).

A cohesion intercept of 2.5 kPa was selected along with an effective friction angle of 10 degrees on the basis of results of the direct shear measurements and modeling experience. An appropriate design value for ϕ^b was presumed to be on the order of 15 degrees (*10*). Experimental research studies indicate that ϕ^b should not be greater than the ϕ'. Therefore, a value of 10 degrees was selected for ϕ^b (*7*).

Design charts were developed for four alternatives:

- Alternative 1. Increased sub-ballast thickness (Figure 7);
- Alternative 2. Increased sub-ballast thickness with impermeable membrane (Figure 8);
- Alternative 3. Hot-mix asphalt layer (Figure 9); and
- Alternative 4. Roller-compacted concrete layer (Figure 10).

Four stress analyses were conducted to produce the four alternatives presented. Four suction values were selected for computing four bearing capacities for each alternative. Each chart indicates the relative increase in bearing capacity factor of safety for increased depth of granular material. The four values for bearing capacity are represented as four contours of soil suction on each chart to indicate the relative increase in factor of safety for increasing soil suction.

The increased sub-ballast alternative will serve to reduce stresses transmitted to the subgrade and therefore will increase the bearing capacity factor of safety. Alternative 2 with increased sub-ballast and impermeable membrane reduces stresses transmitted to the subgrade and also increases watershed capabilities of the subgrade so that a larger design suction value may be used. The hot-mix asphalt and roller-compacted concrete alternatives significantly reduce stresses transmitted to the subgrade and also provide for increased watershed

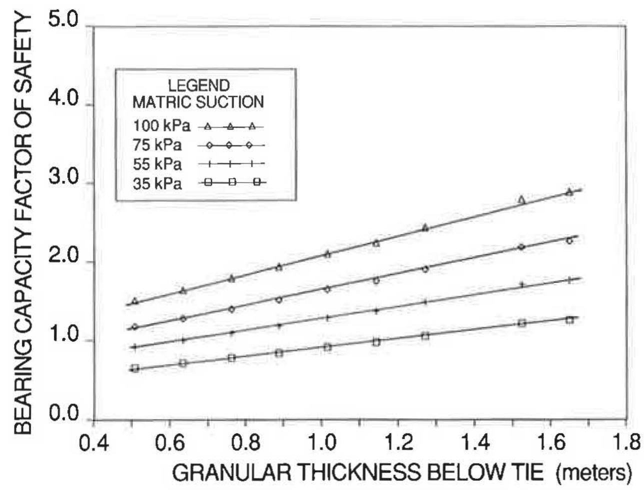

FIGURE 7 Bearing capacity factor of safety versus granular thickness below the tie for Alternative 1, increased sub-ballast.

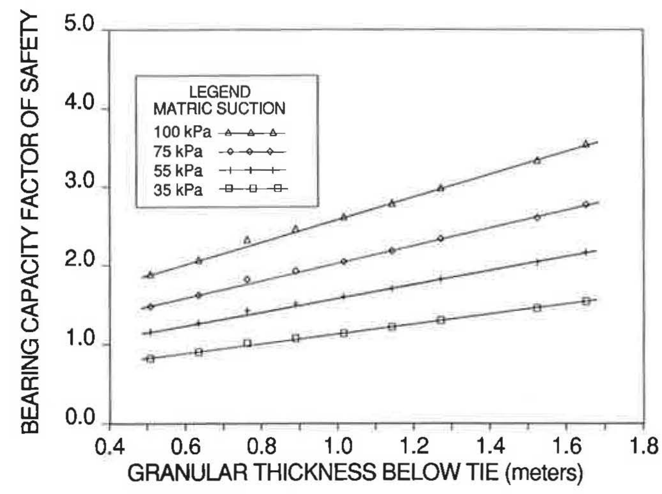

FIGURE 9 Bearing capacity factor of safety versus granular thickness below the tie for Alternative 3, hot-mix asphalt.

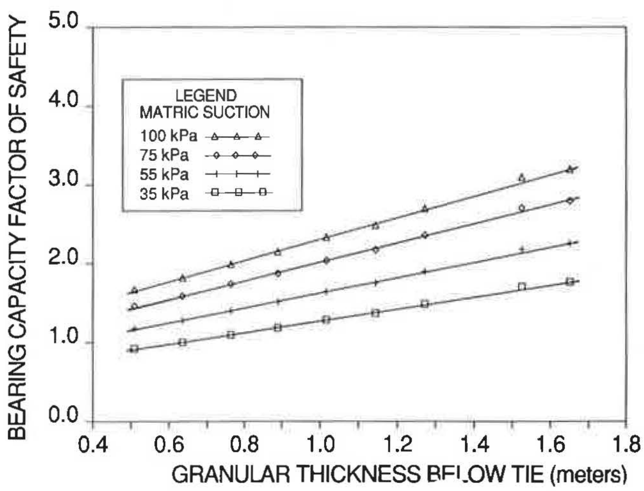

FIGURE 8 Bearing capacity factor of safety versus granular thickness below the tie for Alternative 2, increased sub-ballast with impermeable membrane.

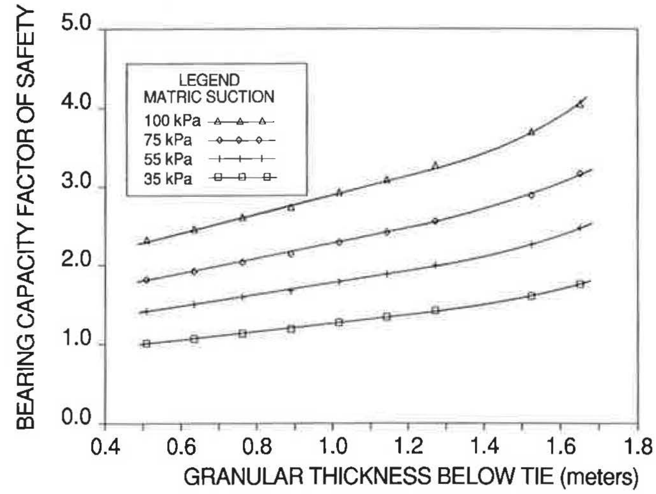

FIGURE 10 Bearing capacity factor of safety versus granular thickness below the tie for Alternative 4, roller-compacted concrete.

capabilities of the subgrade. For comparison purposes it was assumed that 75 kPa of suction could be maintained within the subgrade embankment for Alternatives 2, 3, and 4 (*7*).

For a bearing-capacity factor of safety equal to 2.0 and the assumed soil suction, the relative depths of granular material required beneath the tie for each of the alternatives are increased sub-ballast (55 kPa suction), 1.91 m; increased sub-ballast with impermeable membrane (75 kPa suction maintained), 1.40 m; hot-mix asphalt (75 kPa suction maintained), 1.02 m; and roller-compacted concrete (75 kPa suction maintained), 0.71 m. The corresponding thicknesses for a bearing-capacity factor of safety equal to 2.5 are 2.57, 1.83, 1.40, and 1.22 m, respectively.

The figures indicate relative depths of granular required to prevent bearing capacity failures. However, the site investigation revealed that the ultimate design must provide protection against both bearing capacity and slope-stability failures.

SLOPE-STABILITY ANALYSES

Slope-stability analyses were conducted by using the PC-SLOPE computer program (*16*). Two stabilizing berms were studied to increase the factor of safety of the embankment slope: a gravel berm with 4 : 1 side slopes and a clay berm with 5 : 1 side slopes. The effects of berm width and soil suction in the subgrade were investigated by using Bishop's Simplified method (*7*).

The water table was assumed to remain at 5.5 m below the top of the ballast. Zero suction was assumed for the ballast and sub-ballast layers. Constant suctions, somewhat greater than hydrostatic negative pore-water pressures, were assumed in the clay fill and upper natural subgrade strata, whereas hydrostatic pore-water pressures were assumed below the water table. Train loading was increased by 50 percent to simulate dynamic loading and was applied as a surcharge load to the top of the ballast (*7*).

Existing conditions, before remedial work, were used to calibrate the model for an appropriate matric suction value. With zero matric suction the factor of safety was computed at 0.684 with the train load applied and 0.884 without the train load. At the preceding two factor of safety values the embankment would have failed. Therefore, the incorporation of soil suction was justified. The factor of safety was computed at 0.926 with the train load and 1.205 without the train load by using the design matric suction value of 55 kPa (7). The fact that the existing embankment had been failing under train load suggested that the factor of safety with respect to slope stability was close to 1.0. The analysis of the existing condition developed an appreciation for the significance of the matric suction component of shear strength.

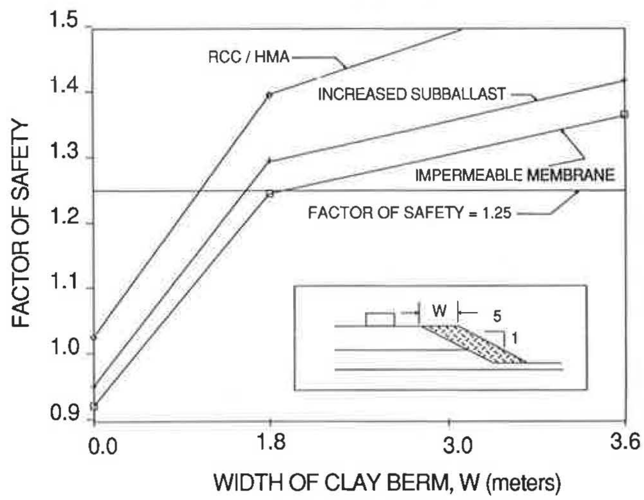

FIGURE 11 Slope stability factor of safety versus width of clay berm.

The four alternatives discussed earlier were analyzed for slope stability for each of the two berm designs. The clay berm produced a higher factor of safety than the gravel berm for the same width of berm and was due to the flatter slope selected for the clay berm. Therefore, only the clay berm is illustrated in Figure 11, which presents the factors of safety with respect to slope stability for each of the four alternatives. Without berming (i.e., berm width equal to zero) the factor of safety with respect to slope stability of the embankment slope ranges from 0.92 to 1.02, indicating a stabilizing berm was required for each design alternative considered. The roller-compacted concrete and the hot-mix asphalt produced similar results from a slope stability perspective. For the same berm width, roller-compacted concrete and hot-mix asphalt produced higher factors of safety than either increased sub-ballast alone or increased sub-ballast with an impermeable membrane.

COMPARATIVE DESIGN ALTERNATIVES

Analyses identified four equivalent design alternatives for remedial work on the floodway trackage railway embankment. A bearing capacity factor of safety equal to 2.0 and a factor of safety with respect to slope stability of 1.25 were suggested for design. The four recommended alternatives are illustrated in Figure 12.

Alternative 1 (increased sub-ballast thickness) consists of 229 mm of ballast and 1.68 m of sub-ballast (i.e., 1.91 m total), requiring the removal of existing ballast, sub-ballast, and approximately 1 m of original clay fill. A clay berm with 5:1 side slopes and 1.6 m top width is required for slope stability.

Alternative 2 (increased sub-ballast with impermeable membrane) consists of 229 mm of ballast, 1.17 m of sub-ballast, the removal of material to the bottom of the mixed

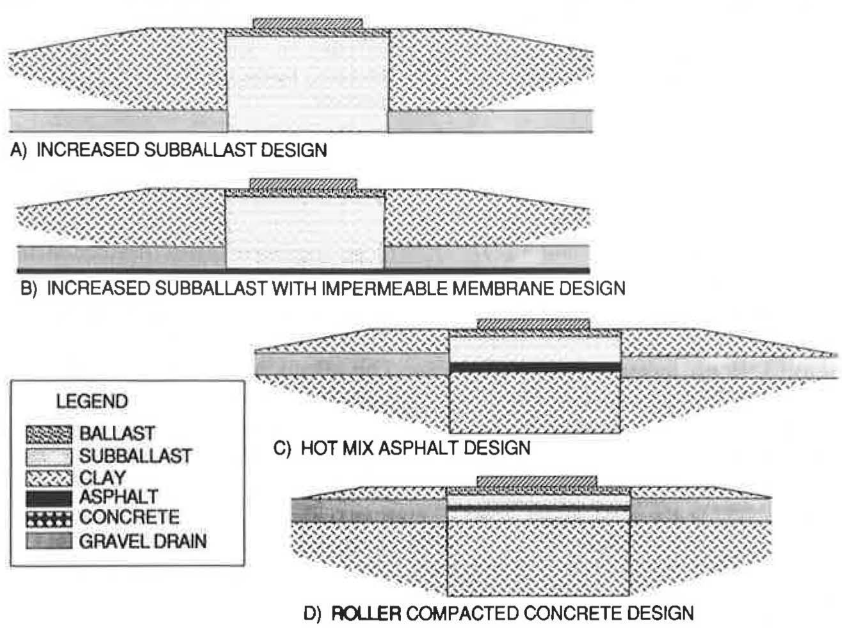

FIGURE 12 Comparison of the four design alternatives.

zone, and the recompaction of 127 mm of clay fill below the impermeable liner. A 1.9-m-wide clay berm is required.

Alternative 3 (hot-mix asphalt) is constructed of 229 mm of ballast, 0.79 m of sub-ballast, 229 mm of hot-mix asphalt, and 330 mm of compacted clay fill required to replace the mixed zone. The width of clay berm required is 1.14 m.

Alternative 4 (roller-compacted concrete) requires the same berm width as Alternative 3 but should be constructed from 229 mm of ballast, 330 mm of sub-ballast, and 203 mm of roller-compacted concrete. An additional 152 mm of sub-ballast is required below the concrete to resist sulphate attack from the clay subgrade. Removal of the mixed zone requires recompaction of an additional 635 mm above the natural clay subgrade.

The remedial design implemented consisted of removal of material to the bottom of the mixed zone. The embankment was constructed by using an increased thickness of glacial till as sub-ballast and a clay berm with a gravel drainage layer connected to the sub-ballast beneath the track. The glacial till material was locally available at considerable saving. The concepts presented by the design report facilitated the choice of material and design thicknesses.

CONCLUSIONS

The case study illustrates the complementary nature of the bearing-capacity design procedure in analyzing railway subgrade problems. Computation of subgrade bearing capacity provides a measure against which subgrade stresses can be compared. Traditionally, railway subgrade design has been based on limiting the subgrade stress to some value that is based on experience. The bearing capacity procedure accounts for the fact that subgrades may accommodate differing levels of stress, depending on material properties and environmental influences.

The investigation revealed the significance of using matric suction in the computation of shear strength for both slope stability and bearing-capacity problems.

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