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Effect of Sloping Backfills on Geosynthetically Reinforced Walls

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ABSTRACT: The effect of backfill slope on the performance of geosynthetically reinforced soil walls is investigated using finite element analyses, with particular reference to the location of the potential failure surface. The methodology involves initial validation of the numerical model against instrumentation records from a full-scale wall and subsequent parametric study of different wall and surcharge configurations. For practical purposes, the location of the potential planar failure surface is found to be independent of the presence of a sloping backfill on the top of the wall. Moreover, the normalized summation of reinforcement tensions along the critical planar surface is found to depend only on the sloping backfill geometry. The use of the Rankine surface is shown to be a conservative, but suitable, design basis for geosynthetically reinforced walls with sloping backfills.

1 INTRODUCTION

Although sloping backfills behind geosynthetically reinforced soil walls are common, some aspects of their design and performance have not been fully investigated. The state-of-practice for design of soil walls reinforced with extensible inclusions and having horizontal backfills has been to consider a Rankine failure surface as the locus of maximum tensile forces (Mitchell and Christopher, 1990). In the case of reinforced soil walls with surcharges induced by sloping backfills (Fig. 1), the same potential failure surface defined by an angle of $45°+\phi/2$ from the horizontal has also been generally considered suitable for design. Since the anchorage length for pullout resistance verification is the reinforcement length behind this surface, correct location of the potential failure surface has major implications on the verification of the wall internal stability.

Instrumentation records from full-scale geosynthetically reinforced walls with sloping backfills (Christopher et al, 1990; Simac et al, 1990) indicated that the theoretical Rankine surface appropriately represented the reinforcement maximum tension line. However, further verification of the location of the potential failure surface is necessary to extend current design methods to different wall and backfill characteristics. Accordingly, a finite element (FE) study was undertaken to investigate the validity of current design assumptions for geosynthetically reinforced soil walls with sloping backfills. The study involved two steps: (1) the finite element prediction of the behavior of an actual instrumented geosynthetically reinforced soil wall with sloping backfill surcharge, and (2) a parametric study, using calibrated input parameters obtained from the previous step, to investigate the effect of surcharge geometry and wall design characteristics on the location of the potential failure surface.

Although a number of successful FE analyses of metallic- and geogrid-reinforced soil retaining walls have been validated against field records, this is not the case for the more flexible geotextile-reinforced structures. A review by Yako and Christopher (1987) identified approximately 200 reinforced walls and slopes that had been constructed in North America using polymeric reinforcements. The number has certainly grown significantly since then. However, of the reviewed projects, only 13 had well-documented instrumentation. Of these, only five

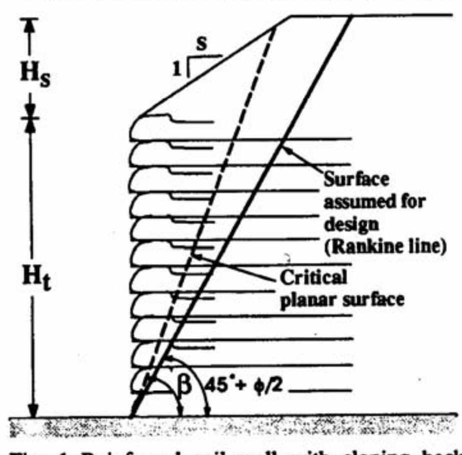

Fig. 1 Reinforced soil wall with sloping backfill showing potential failure surface assumed in conventional design.

provided stress-strain information, and these were all geogrid-reinforced structures. Consequently, much of the field experience to date has provided only qualitative assessments of the design variables in flexible reinforced structures, whereas, quantitative data are needed to substantiate design modifications.

The FE analysis of a well-instrumented geotextile-reinforced wall was performed to add quantitative information to the existing instrumentation records and to provide a calibrated modeling procedure for the parametric study that followed (Zornberg and Mitchell, 1994). The results of a finite element parametric study are presented herein to investigate the influences of the sloping surcharge geometry (fill slope and surcharge height) and of wall design characteristics (wall height and reinforcement stiffness) on the location of the potential planar surface.

2 VALIDATION OF THE FINITE ELEMENT MODEL AGAINST FIELD INSTRUMENTATION RECORDS

The FE analyses in this study were done using the code SSCOMP developed originally by Seed and Duncan (1984), and subsequently modified by Collin (1986) for analysis of reinforced soil structures. Additional modifications were implemented for the purposes of this study. SSCOMP is a general, plane strain, soil-structure interaction program for static analyses of geotechnical structures including consideration of compaction-induced stresses and deformations. Nonlinear stress-strain and volumetric strain behavior of soil is modeled using the hyperbolic formulation proposed by Duncan et al. (1980). Reinforcements are modeled using elastic bar elements, and soil-structure interaction is modeled using interface elements. The program has been used successfully by previous investigators to predict the behavior of large model walls in centrifuge tests and full-scale instrumented reinforced soil walls.

In order to validate the FE model for the analysis of flexible reinforced structures, instrumentation records from a 12.6 m high geotextile-reinforced retaining wall were compared to numerically predicted results. The wall under study, referred to as the Rainier Ave. wall, was higher than any geotextile-reinforced wall built previously and supported a 5.3 m high surcharge fill (Christopher et al., 1990; Allen et al., 1991). An extensive instrumentation program was developed to evaluate the structure performance during construction and after placement of the inclined surcharge fill. The wall was designed using 0.38 m reinforcement spacing, and the specified geotextile strength was varied with the height of the wall to more closely match design strength requirements. Accordingly, three different polypropylene slit film wovens and a polyester multifilament woven were selected as geotextile reinforcements. The instrumentation consisted of bonded resistance strain gauges and mechanical extensometers to monitor geotextile strains, inclinometer tubes to measure lateral displacements within the reinforced soil mass, optical and photogrammetric surveys to evaluate face displacements, and stress cells to monitor vertical stresses beneath the wall.

Considering the difficulty in modeling flexible reinforced structures, extra care was required for the determination of appropriate mesh layout, material parameters, and analysis sequence. The FE mesh selected for the final analysis consisted of 1698 nodes, 1661 plane strain elements for soil representation, and 561 bar elements for simulation of the reinforcements. Mesh discretization between reinforcement layers was found essential for the proper representation of the soil layer behavior. Parameters for the nonlinear soil representation were obtained after calibration of the hyperbolic model using stress-strain results from triaxial tests performed on backfill soil samples. Very good representation of the deviatoric stress-strain behavior was obtained by the hyperbolic model. Further details of the wall characteristics and modeling procedures are given by Zornberg and Mitchell (1994).

One of the most important parameters to be selected in the FE analysis of a reinforced soil wall is the in-situ tensile stiffness of the geotextiles. Although results from unconfined wide width tensile tests were available for the geotextiles used in the structure under study, an increase in stiffness and strength is expected in geotextiles under the confinement of soil. Since it is essentially impossible to determine the confined stiffness directly from the instrumentation data, the in-situ moduli were deduced by trial and error matching of the numerical results to the instrumentation records of the Rainier Ave. wall. As expected, the back calculated stiffness values were higher than the values obtained from wide width testing, and the increase depended both on the woven geotextile material and on the in-situ confining pressures. For polypropylene materials, in-situ stiffness was found to increase with confining pressure from roughly twice to approximately four times the unconfined stiffness. The increase in the only polyester material used as reinforcement (in the zone from 9 to 12 m from top of the wall) was found to be less than four times the unconfined modulus.

Good matching was obtained between the numerical results and the various instrumented responses of the wall: tension distribution in the geotextile reinforcements, lateral displacements within the reinforced soil mass, lateral face displacements, and vertical stresses beneath the wall. Fig. 2 shows the comparison between FE and field results for the lateral displacements at the location of an inclinometer tube installed 2.7 m behind the wall face. Both numerical results and field values show a lateral displacement increase of roughly 2.5 cm near the top of the wall caused by the surcharge. The reader is referred to Zornberg and Mitchell (1994) for additional numerical results of the different aspects of the wall response. After successfully validating the finite element model, the effect of sloping backfills was investigated by analyzing the effect of a surcharge fill on the performance of the Rainier Ave. wall and of generic geosynthetically reinforced walls.

3 LOCATION OF THE CRITICAL PLANAR SURFACE IN THE RAINIER AVE. WALL

The potential slip surface in a reinforced soil wall is assumed to coincide with the locus of maximum tension forces in the reinforcements. This locus has been considered to be linear in structures with extensible reinforcements. Since a parametric study of the location of the potential failure surface would be simplified if this surface is assumed to be planar, a systematic methodology was used to determine this critical plane.

The location of potential failure surfaces in nonreinforced soil structures has already been investigated using FE analyses (e.g., Duncan, 1992). In these studies, the numerically predicted shear stresses along a trial surface are compared to the ultimate shear resistance available along that surface. This approach can be extended to reinforced soil structures to investigate trial planar failure surfaces.

For each trial failure surface forming an angle β from the horizontal (Fig. 1), a Reinforcement Tension Summation (RTS) is determined by adding the reinforcement tensions along that surface. The value of the Factor of Safety in each trial can also be determined using numerically obtained soil stresses and reinforcement tensions. The surface with the maximum RTS is the critical planar surface since, considering simplifying assumptions, it can be demonstrated that the plane with a minimum Factor of Safety corresponds to the surface with a maximum RTS (Zornberg and Mitchell, 1993).

Fig. 3 shows the critical surfaces for the Rainier Ave. wall, before and after surcharge placement, determined by the search process. RTS values at each trial plane forming an angle β from the horizontal are also indicated in the figure. The critical plane before surcharge placement forms an angle β=70.75° from the horizontal, whereas the critical plane after surcharge placement forms an angle β=68.6°. The Rankine plane, defined using the friction angle obtained from triaxial tests on backfill specimens (ϕ=43°), forms an angle β=66.5° from the horizontal. Thus, for the wall under study, the anchorage length for pullout safety can be conservatively estimated both before and after surcharge using a potential failure surface defined by the theoretical Rankine line. A Factor of Safety against sliding along the critical planar surface of approximately 3 was calculated for the Rainier Ave. wall (Zornberg and Mitchell, 1993). Such a high safety factor suggests that current design procedures for geosynthetically reinforced structures are conservative in determining reinforcement strength requirements.

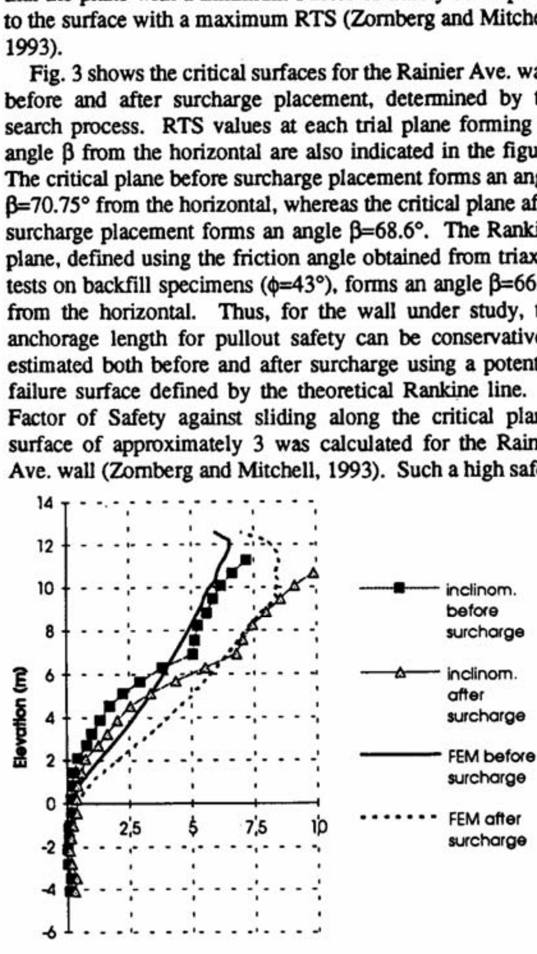

Fig. 2 Lateral displacements at an inclinometer located 2.7 m behind the face of the Rainier Ave. wall.

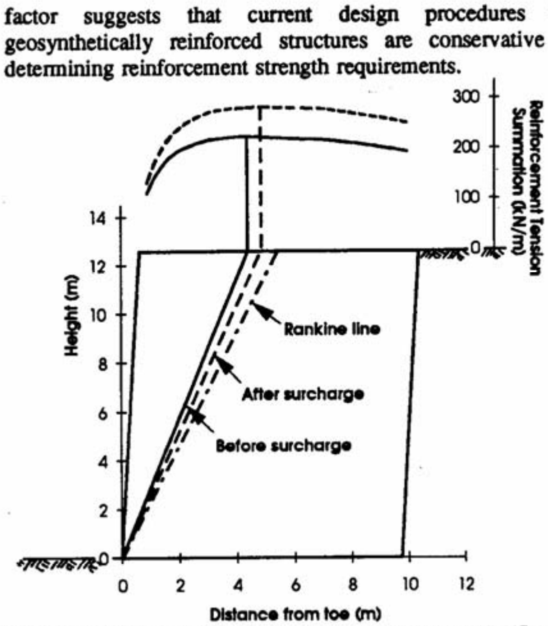

Fig. 3 Location of the planar surfaces with maximum RTS at the Rainier Ave. wall.

4 PARAMETRIC STUDY: EFFECT OF SLOPING BACKFILL GEOMETRY

Using calibrated input parameters and modeling procedures obtained from the analysis of the Rainier Ave. wall, a parametric study was done to investigate the effect of sloping surcharge geometry on the performance of geosynthetically reinforced soil walls. Two variables were used to define the surcharge geometry, namely, the slope *s:1* of the backfill behind the top of the wall (Fig. 1), and the surcharge height *Hs* (or the ratio with the wall height, *Hs/Ht*). The parametric study was done on a structure with the dimensions, reinforcement characteristics, and backfill soil properties of the Rainier Ave. wall. Reinforcement Tension Summations were determined along the critical planar surfaces.

For a given surcharge geometry (defined by *Hs/Ht* and *s*), the critical planar surface is defined as the plane along which the Reinforcement Tension Summation, obtained from the finite element analyses, reaches a maximum. Although the locus of the numerically obtained maximum reinforcement tensions does not necessarily correspond to a planar surface, the plane with the maximum RTS was generally a good linear fit of the actual locus. For this parametric study a linear fit is particularly suitable since the effect of surcharge loadings on the location of the critical surface can be easily quantified. In this case, a straight forward index to measure the effect of the sloping surcharge is the ratio β/β_o, where β is the angle from the horizontal of the critical planar surface after placement of a given surcharge, and β_0 is the inclination of the critical plane before surcharge.

In the case of the Rainier Ave. wall (*Hs/Ht*=0.42, *s*=1.75), the inclination of the critical planar surface decreased about 2°, that corresponds to a ratio β/β_0=0.97. It is considered that this small change in the location of potential failure surface can be neglected for practical design purposes. These numerical results are consistent with reinforcement strain records obtained at the Rainier Ave. wall, that showed no change in the location of the maximum reinforcement tensions after surcharge placement.

Fig. 4 shows the effect of sloping backfill height on the location of the critical planar surface considering a surcharge slope equal to that of the Rainier Ave. wall (s=1.75). The figure shows that the slight changes in the location of the critical planar surface (≈2°) occur after placement of relatively low surcharges (Hs/Ht≈0.2). Then, the location of the critical planar surface remains constant with additional surcharge placement. Numerical results obtained considering placement of sloping backfill surcharges up to the wall height (Hs/Ht=1.0) are indicated in the figure. For any sloping surcharge height Hs, the inclination of the critical planar surfaces was always greater than the angle formed by the theoretical Rankine line. It may be concluded that, both before and after surcharge placement, the state-of-practice use of the Rankine line as internal failure surface is a conservative design assumption for determination of pullout resistance.

Fig. 5 shows the effect of sloping backfill height on the Reinforcement Tension Summation. As would be expected, RTS values increase with increasing surcharge heights. However, the tension summation achieves a maximum value at Hs/Ht≈0.30 and no further increase is observed with additional surcharge beyond this value. The Reinforcement Tension Summation after placement of 12.6 m of surcharge (Hs/Ht=1.0) is only 26% higher than the RTS value before surcharge.

The influence of the surcharge slope s on the location of the critical planar surface was investigated by performing a series of finite element analyses on a reinforced wall designed as the Rainier Ave wall, but with surcharges placed at different slope angles. Slopes varying from s=1.0 to s=3.0 and surcharge heights up to 12.6 m (Hs/Ht=1.0) were considered in these analyses. The normalized inclinations β/β_0 of the critical planar surface are indicated in Fig. 6 showing that the surcharge effect on the location of the potential failure surface is basically independent of the slope s. In all cases, the surcharge causes only a slight decrease in the angle β of the critical surface. This decrease always occurs after placement of relatively low surcharges, from Hs/Ht≈0.10 (for the case s=3.0) to Hs/Ht≈0.20 (for the case s=1.0).

The influence of the surcharge slope s on the Reinforcement Tension Summation, calculated along the critical planar surface, is indicated in Fig. 7. RTS values are normalized in relation to RTS_0, the Reinforcement Tension Summation before surcharge placement. As expected, there is an increase in the calculated RTS values with increasing surcharge slopes. In all cases, and particularly for the cases with lower surcharge slopes, the maximum RTS is achieved at relatively low surcharges. The

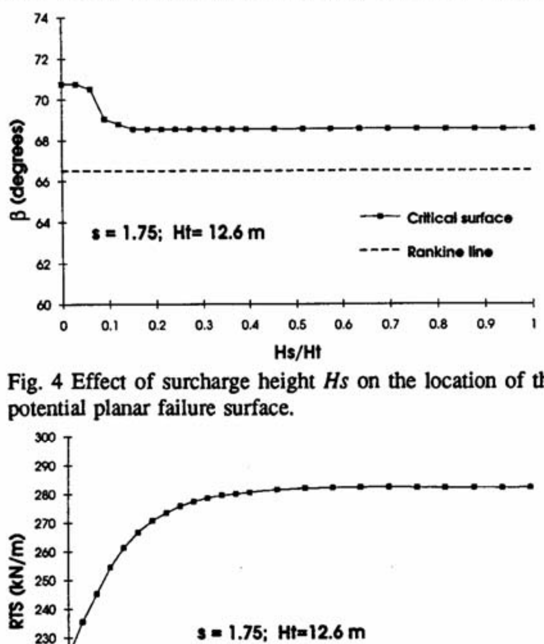

Fig. 4 Effect of surcharge height Hs on the location of the potential planar failure surface.

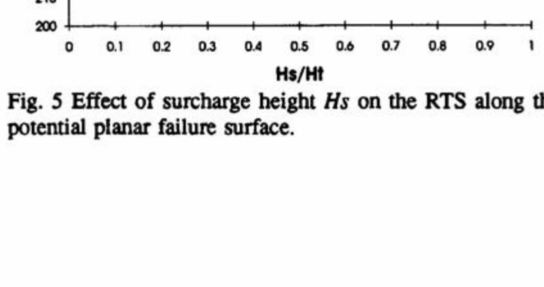

Fig. 5 Effect of surcharge height Hs on the RTS along the potential planar failure surface.

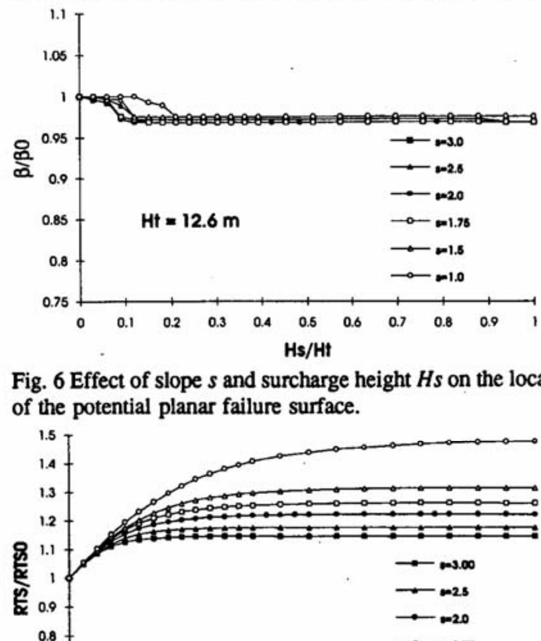

Fig. 6 Effect of slope s and surcharge height Hs on the location of the potential planar failure surface.

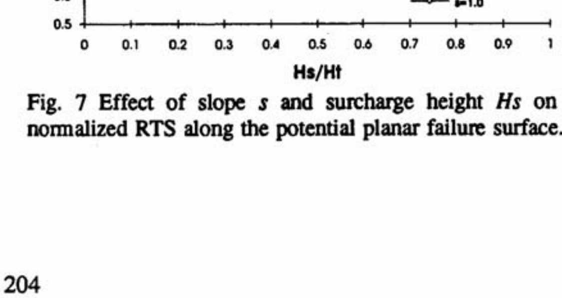

Fig. 7 Effect of slope s and surcharge height Hs on the normalized RTS along the potential planar failure surface.

increase in RTS values due to the surcharge goes from 14% for a slope s=3.0 to 47% for a surcharge slope s=1.0 .

In conclusion, the effect of the sloping backfill surcharge on the location of the potential failure surface was observed to be very small, and can be neglected for practical design purposes. This observation is found valid independently of the geometry of the sloping surcharge. Although the magnitude of the Reinforcement Tension Summation after surcharge placement depends on the inclination and height of the surcharge, the maximum RTS value is achieved at relatively low surcharge heights. The implication of this observation is that, beyond a certain surcharge height *Hs*, there are no additional reinforcement requirements with further surcharge loads; e.g., the height of the surcharge fill becomes of no importance once *Hs*/*Ht* exceeds about 0.2 for a backfill slope defined by s=2.

5 PARAMETRIC STUDY: EFFECT OF WALL DESIGN CHARACTERISTICS

The parametric analyses in the previous section showed that the location of the critical planar surface is almost independent of the sloping surcharge geometry. The validity of this observation for different wall design characteristics, namely, wall height and reinforcement stiffness has also been investigated herein. The effect of wall height was evaluated by a parametric finite element study of generic 6.5, 9.5, and 12.6 m high walls. A constant reinforcement spacing of 0.38 m, a constant reinforcement stiffness of 1820 kN/m, and a reinforcement length of 80% of the wall height were adopted. Selected soil properties were those obtained for the Rainier Ave wall.

Fig. 8 shows the effect of wall height on the inclination β of the critical planar surfaces after surcharge placement (slope s=2.0). The pattern of the results is similar to that obtained for the analyses done to investigate different surcharge geometries. The general observation that the surcharge has only a small effect on the location of the potential failure surface is valid, independently of the height of the reinforced soil wall. Almost no change at all is observed in the slope of the potential failure surface for the 6.5 m high wall.

The influence of wall height *Ht* on the normalized Reinforcement Tension Summation calculated along each critical planar surface is indicated in Fig. 9. The maximum RTS is always achieved at relatively small surcharge heights. The calculated RTS values collapse into a single normalized RTS curve, showing that the normalized values are essentially independent of the height of the wall.

The effect of reinforcement stiffness on the location of the critical planar surface was also investigated. These analyses were performed on 6.5 and 9.5 m high walls. A constant reinforcement stiffness was adopted for the full vertical section of the walls, with selected values ranging from 364 to 2912 kN/m. This range encompasses confined stiffness values of commonly used geosynthetic reinforcements. The reinforcement layout in these generic walls was based on the one used in the Rainier Ave. wall analysis, with constant reinforcement spacing of 0.38 m and reinforcement length 80% of the wall height. Soil parameters were those obtained for the Rainier Ave. wall, and the surcharge slope in these analyses was s=2.0 .

The effect of surcharge placement on the location of the critical planar surface in 9.5 m high walls reinforced using different reinforcement stiffnesses *J* is shown in Fig. 10. The figure shows the normalized inclination of the critical planar surface (β/β_0) as a function of the sloping surcharge height. It may be observed that the influence of the surcharge on the location of the potential failure surface is small for the range of reinforcement stiffnesses considered in the study. The observed trend is that the more flexible reinforced soil walls show less change on the location of the critical planar surface as a result of surcharge placement. For the 6.5 m high wall, the location of the critical planar surface was essentially unchanged after surcharge placement, independent of the reinforcement stiffness.

The influence of reinforcement stiffness *J* on the normalized Reinforcement Tension Summation, calculated along each critical planar surface is indicated in Fig. 11. The maximum RTS is always achieved under relatively low surcharges (*Hs*/*Ht*≈0.2). Since all RTS curves essentially collapse into a single normalized curve, it may be inferred that the normalized Reinforcement Tension Summation is independent of the reinforcement stiffness. Results from the analysis of a 12.6 m high wall designed having four vertical sections with different reinforcement stiffnesses (as the Rainier Ave. wall, but with a surcharge slope s=2.0) also fit very well into the normalized RTS

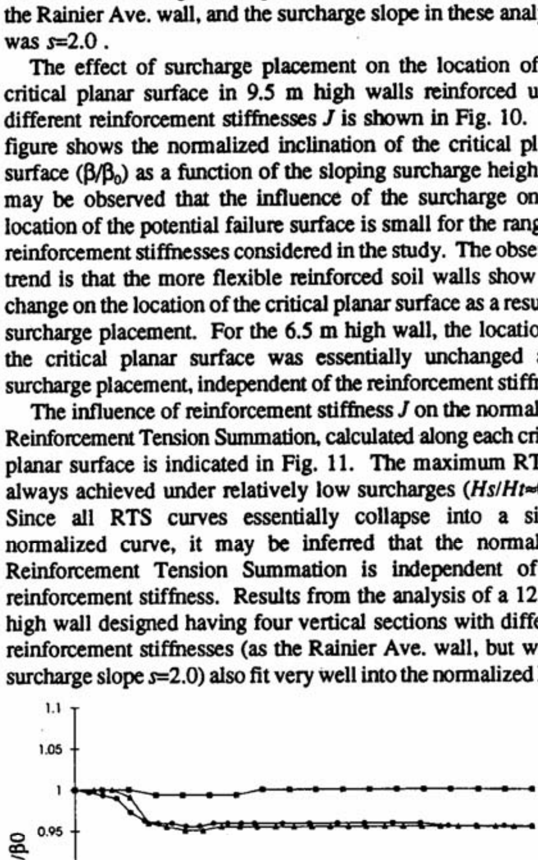

Fig. 8 Effect of wall height *Ht* on the location of the potential planar failure surface - Normalized curves.

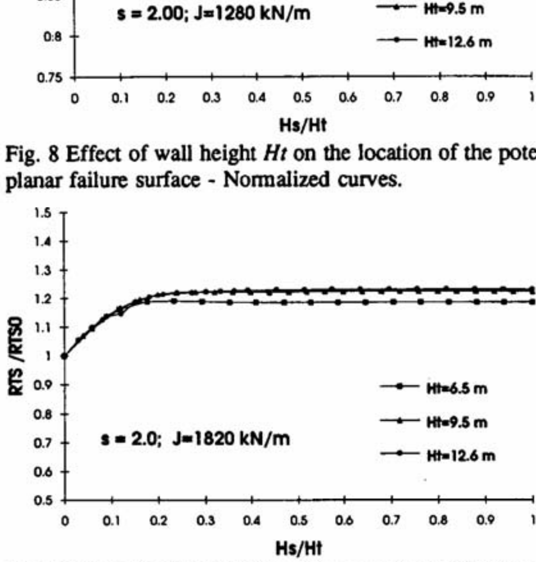

Fig. 9 Effect of wall height *Ht* on the normalized RTS along the potential planar failure surface.

curve. Moreover, it may be observed that the normalized RTS curves in Figs 9 and 11 essentially collapse into a unique plot. This suggests that the normalized Reinforcement Tension Summations are a function only of the surcharge geometry (Fig. 7), being independent of the wall height (Fig. 9), and of the reinforcement stiffness (Fig. 11).

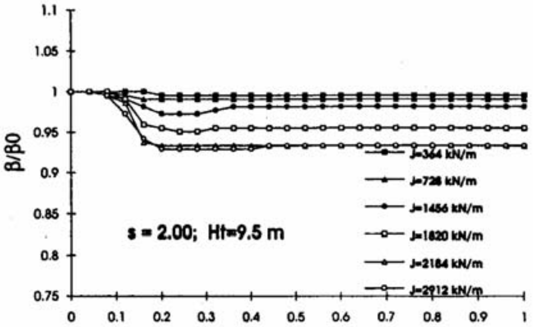

Fig. 10 Effect of reinforcement stiffness *J* on the location of the potential planar failure surface - Normalized curves.

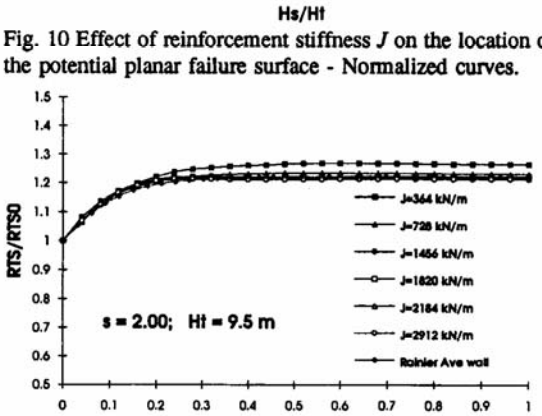

Fig. 11 Effect of reinforcement stiffness *J* on the normalized RTS along the potential planar failure surface.

6 CONCLUSIONS

The analysis of a well-instrumented soil wall with sloping backfill that was reinforced using geosynthetics was done to validate a finite element model. A parametric study was subsequently done to investigate the effect of sloping backfills on the location of the potential failure surface. The following conclusions can be drawn from this study:

- For practical purposes, the location of the critical planar potential failure surface is independent of the presence of a sloping backfill surcharge on the top of the wall. This was found to be true independently of the sloping surcharge geometry (surcharge slope and surcharge height), and of wall design characteristics (wall height and reinforcement stiffness).
- Reinforcement Tension Summation (RTS) values under surcharge loading can be normalized to the RTS value before surcharge placement. Normalized RTS values are only a function of the surcharge geometry, being independent of the wall height and of the reinforcement stiffness. The maximum RTS value is achieved at relatively low surcharge fill heights.
- The Rankine failure surface provides a conservative, however suitable, design basis for separation of the active and resistant zones within geosynthetically reinforced walls with sloping backfills. The required reinforcement length for pullout resistance purposes can be taken as the reinforcement length behind this surface at each reinforcement level.

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VOLUME 1

REINFORCEMENT APPLICATIONS AND RELATED RESEARCH

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