Deformation Evaluation Procedure for Reinforced Soil Walls

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ABSTRACT A deformation evaluation procedure for reinforced soil walls is proposed based on the findings of parametric studies using SSCOMP. This deformation evaluation procedure may be used as a serviceability design check for reinforced soil walls, constructed with either inextensible or extensible reinforcements, standard or non-standard configurations. The maximum facing deformation for a wall with "standard" configuration is first determined from a design chart produced using a well-validated finite element program SSCOMP. The deformation of walls with non-standard configurations can then be computed by multiplying the standard wall deformations by appropriate modification factors, termed "deformation indices". Use of the deformation evaluation procedure will give estimates of specific deformation values at working stress conditions. The proposed deformation procedure was validated by field walls and test walls with different reinforcements and configurations.

1. INTRODUCTION

Current reinforced soil walls designs consist of determination of the reinforcement types, lengths, cross-sections and spacings to insure external stability and internal stability. Comprehensive and up-to-date surveys and discussions of the current design methods can be found in Mitchell and Villet (1987) and Christopher et al., (1990). While these empirically based design methods consider overall wall stability and rupture and pullout capacities of the reinforcements, they do not explicitly consider wall deformation under working stress conditions. One reason for this may be that deformations are relatively small for the typical conditions of uniform wall geometry and loading from which the design procedures were developed. However, in situations where project requirements dictate wall structures with non-standard geometry or loadings that fall outside the range considered by the empirical design methods, the deformation behavior may become relatively more important. For example, the use of shortened reinforcement layers because of topographic or economic constraints will result in increased deformations which could possibly become the critical item for acceptable wall performance. Walls with large external loadings or sloping backfills, and walls where performance criteria require very small deformations are also cases in which wall deformation might become critical. For example, the vast number of reinforced soil walls built for highway application are constantly subjected to the action of vertical and horizontal loads during their service life. It is important to limit the cumulative deformations of these walls in order to satisfy the serviceability criteria. Hence an evaluation of wall deformation can be an essential step in satisfactory design.

2. PARAMETRIC STUDY BY SSCOMP

Parametric study of the major factors influencing the deformations of reinforced soil walls was done using SSCOMP. The program SSCOMP is a two-dimensional plane-strain finite element code, which models soil-structure interaction during incremental placement of soil layers and compaction (Seed and Duncan, 1984; Boulanger et al., 1991).

SSCOMP is capable of predicting deformations of reinforced soil walls if incremental construction is properly modeled. The present study showed that major factors affecting the deformation prediction by SSCOMP are the uncertainty in estimating soil modulus and stiffness, compaction stress, number of fill placement lifts, softening depth to model multiple compaction passes, and the number of rows of soil elements between the reinforcements.

It has been shown that although the final stresses in a homogeneous embankment do not differ appreciably with number of fill placement lifts, the final displacements are greatly affected (Kulhawy, 1977). The present study showed similar effects in reinforced soil walls. Reducing the number of placement lifts to shorten computation time may overestimate or underestimate the deformation by 15 to 50%, whereas the reinforcement tension prediction was insensitive to the number of placement lifts.

The effect of the repeated application of lateral stresses by multiple passes of compaction equipment is simulated in SSCOMP by specifying a depth, termed softening depth, below the surface being compacted in which compaction induced lateral stresses are not relieved by immediate lateral displacements. The softening depth concept allows multiple compaction passes to be modeled by a single solution increment, thus greatly reducing analysis time. The present study showed that lateral wall deformations increase very rapidly with increase of softening depth, whereas maximum reinforcement tension remains the same. Selecting the softening depth to be less than half the reinforcement vertical spacing gave satisfactory results with respect to both tensions and deformations.

The study also showed that a sufficient number of rows of soil elements should be provided between layers of reinforcement for realistic stress and strain distributions to be developed, and hence the correct prediction of deformations. For a reinforced soil wall with typical reinforcement type, spacing and length, backfill material, and loading conditions, at least four rows of soil elements are needed to give deformations prediction within 10% of the "actual" values. Similar results had been observed by Rowe and Ho (1988).

The soil and reinforcement stiffness has to be accurately estimated or measured for the proper prediction of deformation behavior. This is particularly important for geotextile reinforcements where research has shown that conventional unconfined stress-strain tests underestimate the actual reinforcement stiffness in the field, under confined conditions (Zornberg and Mitchell, 1993).

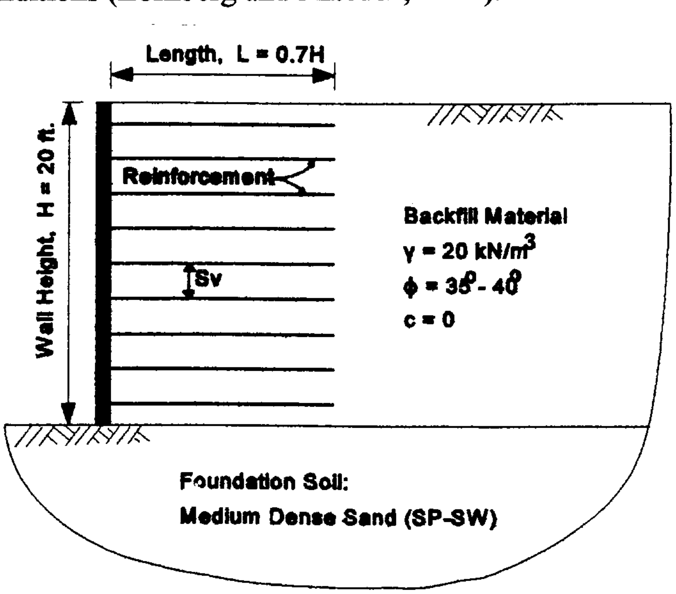

Fig. 1: Cross-section of baseline configuration

Some preliminary results of the parametric study of the deformation behavior were summarized in Chew et al. (1991). Geometric factors (such as length, spacing etc.), material factors (such as reinforcement type, facing type etc.), and loading factors were included. The extent to which these parameters influence the deformations was analyzed and presented in the form of a modification factor. This modification factor, termed Deformation Index DI, indicates the relative deformations with respect to a Baseline configuration. The baseline case represents standard wall configurations: L/H =0.7, H=20 ft., uniform reinforcement spacing, moderate compaction, level backfill and no external loading (Figure 1).

Some of the important findings from the parametric studies are as follows:

a. The relative stiffness of the reinforcement system, S_r, has a strong influence on deformation. S_r is defined as

$$S_r = \frac{E_r A_r}{S_v}$$

where E_r, A_r are the Young's Modulus and cross-sectional area of reinforcement respectively, and S_v is the uniform vertical spacing between reinforcements. Using gravelly sand with a friction angle of 35°-40° and other typical soil parameters, a design chart for $(\delta/H)_{base}$ with respect to various value of Reinforced Soil Relative Stiffness S_r was produced as shown in Figure 2. H is the wall height and δ is the maximum horizontal wall face deformation.

b. Reinforcement length has a strong influence on lateral wall facing deformations, particularly when the length is shorter than about 0.7H (see Figure 3). Deformation increases rapidly as length is reduced below this value.

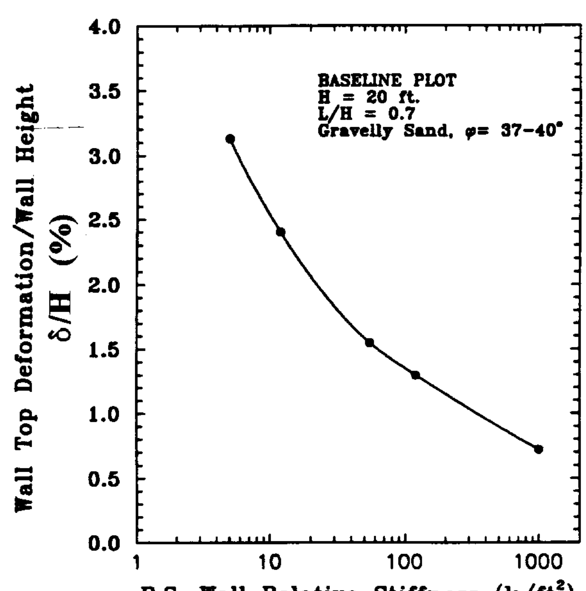

Fig. 2: Design chart for δ/H (baseline) as a function of Sr

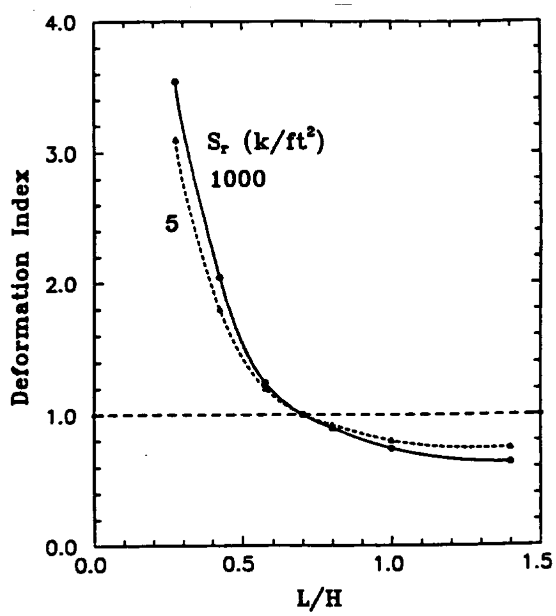

Fig. 3: Effect of reinforcement length on wall face deformations

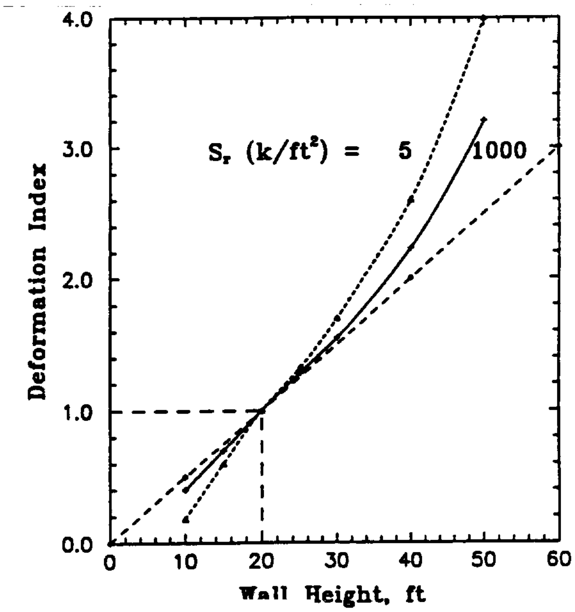

Fig. 4: Effect of wall height on wall face deformations

c. Higher wall gives larger deformation. The effect of wall height seem to be more significant for system with lower relative stiffness as shown in Figure 4.

d. Uniform surcharge increases deformations in proportion to loading magnitude, as long as the wall system is not near overall failure.

e. Sloping backfill causes moderate deformation increases which are shown to be dependent on slope angle and slope height as shown in Figure 5.

f. Strip loading can cause significant increases in deformation, with the amount of deformation strongly dependent on the location and magnitude of the loading. An optimum positioning of the loading may result in minimum deformation (Chew et al., 1991).

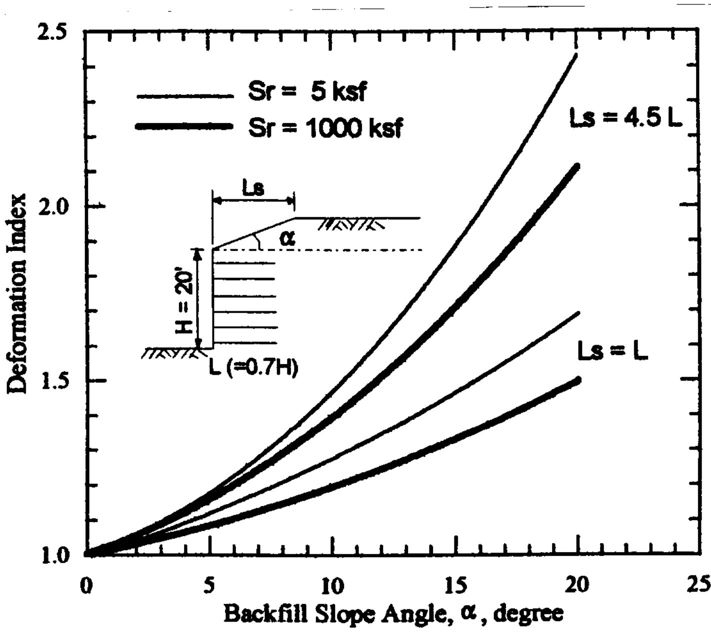

Fig. 5: Effect of sloping backfill on wall face deformations

3. PROPOSED DEFORMATION EVALUATION PROCEDURE

A proposed design methodology consists of the following:

1. Design the basic configuration using the current design methods that are based on stress considerations. This will enable the selection of required reinforcements types, cross-sectional area, length and vertical spacings, This is a stress-based design with usual minimum factors of safety. After this preliminary design, the Reinforced Soil Relative Stiffness, S_r, can be computed.
2. Determine $(\delta/H)_{base}$ for the equivalent Baseline Standard Wall using design chart as shown in Figure 2. Note that this baseline case should have L/H =0.7, H=20 ft., uniform reinforcement spacing, moderate compaction, level backfill and no external loading. In cases where backfill material and soil-reinforcement interface properties are very different from the typical values used to produce Figure 2, a FEM analysis using computer program such as SSCOMP is then needed to determine $(\delta/H)_{base}$.
3. For the particular wall design, determine the variations in the wall's configuration, loading condition etc. from the baseline condition and obtain the corresponding deformation index from Figures 2, 3, 4 and 5.
   The actual wall face deformation δ is then obtained from:

$$\frac{\delta}{H} = \left(\frac{\delta}{H}\right)_{base} \cdot DI_{L/H} \cdot DI_H \cdot DI_q \cdot DI_{slope} \cdots$$

   where DI_x is the Deformation Index for the particular variation of the wall configuration, x.
4. Modify the design if the δ/H value obtained from step #3 is not acceptable for that particular application. Repeat step #1-#3 with the new design configurations.
5. Evaluate reduced system stiffness due to material

creeping and corrosion, if any. Hence, the long term δ/H may be evaluated by

$$\left(\frac{\delta}{H}\right)_{long\ term} = \frac{\delta}{H} \cdot DI_{reduced\ Sr}$$

The reduction of long term stiffness can be estimated from long term material testings (McGown et al. 1984) or from field experiences on long term performance.

4. VALIDATION BY CASE STUDIES

Two case studies are presented here. Walls with different geometry, reinforcement type and applications were chosen to study the applicability of the proposed procedure. One wall was reinforced with geogrids, and the other one with geotextiles. The main objectives of the study were to evaluate the wall face deformation using the proposed procedure and to see how well these predictions compared with the measured values. Other field measurements such as soil stresses and strains in the walls are not discussed in this paper.

4.1 CASE STUDY 1: Seawall in Gaspe Peninsula, Quebec, Canada

Project Description A case study for a 325 ft (99 m) long, 17.5 ft (5.3 m) high polymer-geogrid reinforced soil seawall located along the coast of the Gaspe Peninsula, near Saint Lawrence River inlet in Quebec, Canada was presented by Berg et al., (1987). This seawall, which was constructed in August 1985, is exposed to severe climatic conditions involving wave action and freeze-thaw cycles. An extensive monitoring program was installed to observed the wall face movements, soil mass movements and geogrid strains for a period of time.

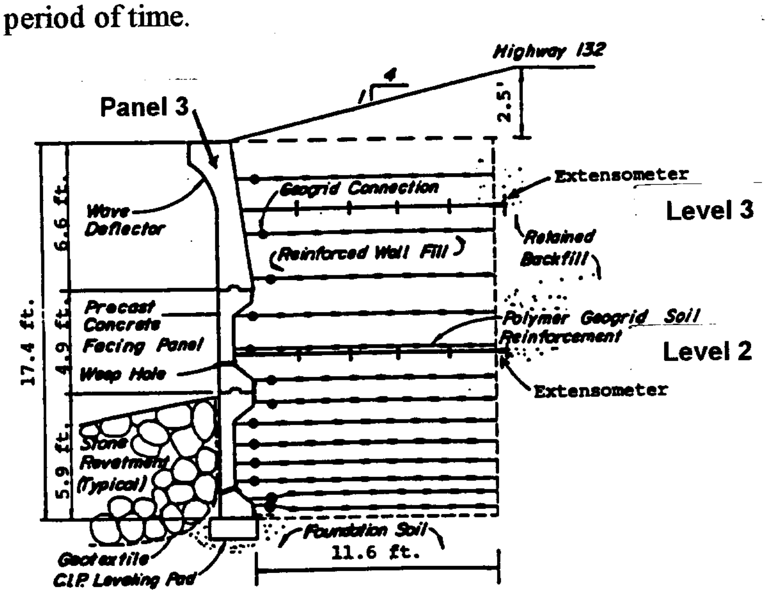

Fig. 6: Cross-section of the reinforced soil seawall at Gaspe Peninsula, Quebec (After Berg et al., 1987)

The cross-section of the seawall is shown in Figure 6. The precast concrete facing panels were 4.9 to 6.6 ft (1.5 to 2.0 m) high and 10 ft (3.0 m) wide. Panels placed at the top of the wall are shaped to act as wave deflectors. The geogrid reinforcement utilized was a 920 g/m^2 HDPE product. A total of thirteen 12.4 ft (3.78 m) long geogrid layers was used. This produced a relative stiffness value of 120 ksf. On-site beach gravel was used as the fill material for the bottom 3.3 ft (or 1 m). Fill was placed by bucket from a crane operating in front of the wall and then spread by a light dozer. Subsequently, selected fill was used for both the reinforced wall fill and the retained backfill. The fill is a well-graded, sandy gravel with a maximum size of 150 mm. The fill was spread with a dozer and compacted by a 5-ton roller compactor. A lightweight, hand-operated vibratory tamper was used close to the wall face.

Observed Wall Movements The horizontal wall movements with time are summarized in Figure 7 for the top layer of geogrid. The series of readings shown in Figure 7 started on July 30th, 1985, when the fill had already reached the lower quarter of panel 3 and the observed maximum wall face movement was about 15 mm (0.59 inch). Subsequently, with the erection of panel 3 and backfilling, additional movement of 25 mm (0.98 inch) was reported at the top of the wall by August 6th 1985. With the completion of the highway embankment by November, 1985, an additional outward movement of 23 mm (0.90 inch) was observed. No appreciable movement occurred after that, even during winter months.

Wall Deformations by the Proposed Method Using the proposed deformation evaluation procedure, the maximum outward wall face deflection can be computed as follows:

<u>(A) At the end of wall construction, August 6th, 1985:</u>

From Figure 1, $(\delta/H)_{base}$ is found to be 1.2% for a baseline wall with Reinforcement Soil Relative Stiffness S_r of 120 ksf. Correction for wall height, using DI_H of 0.72 obtained from Figure 4, resulted in a predicted wall face

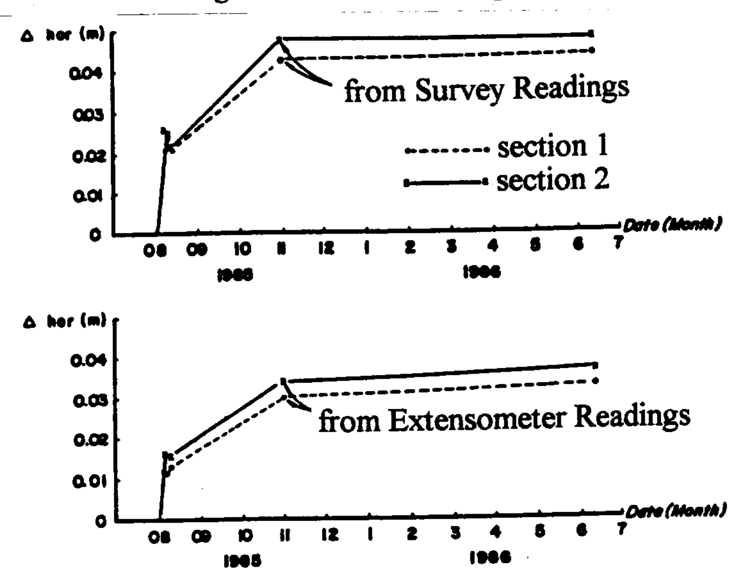

Fig. 7: Horizontal wall movements (at level 3) with time (After Berg et al., 1987)

deformation of 46 mm (1.80 inch), which compared well with the measured value of 40 mm (1.57 inch) at the end of wall construction.

(B) With the road embankment constructed:

The road embankment is sloping at an angle of about 11.3°. According to Figure 5, this will give a DI_{slope} of 1.30 for the case where Ls=L. Hence, the predicted deformation is about 60 mm (2.36 inch), which is within 5 percent of the measured deformation.

Field Measurements on "Coherent Block" Movement The extensive measurements of the trial sections also enable the evaluation of the "coherent block" movement at the back of the reinforced soil block. The extensometers were installed at levels 2 and 3 where wall face surveys were taken. The anchor plates of these extensometers were embedded right at the end of the reinforcements. If one assumes that the anchor plates at the end of the extensometers have not moved, the relative movements between these plates and the face of the wall should correspond to the movements of the wall. The wall face movements deduced from the extensometer readings are also plotted in Figure 7. From this figure, it can be seen that the movements observed by surveying are larger than those measured by the extensometers. This difference indicated that the anchor plates located at the ends of extensometers have been moving outward. This is the first direct measurement of a "coherent block" movement.

The information from survey results and extensometer results were analysed and plotted as Figure 8. This plot shows that the reinforced soil block moved as a coherent block. This "coherent block" movement is more like a rotation about the base during the construction of the wall. The observed top movement at the back of the reinforced soil block is of the order of H/600 (9 mm or 0.35 inch) in this case. During the construction of road embankment, the "coherent block" movement seemed to be more "translational" than "rotational". The observed block translation is equivalent to H/750.

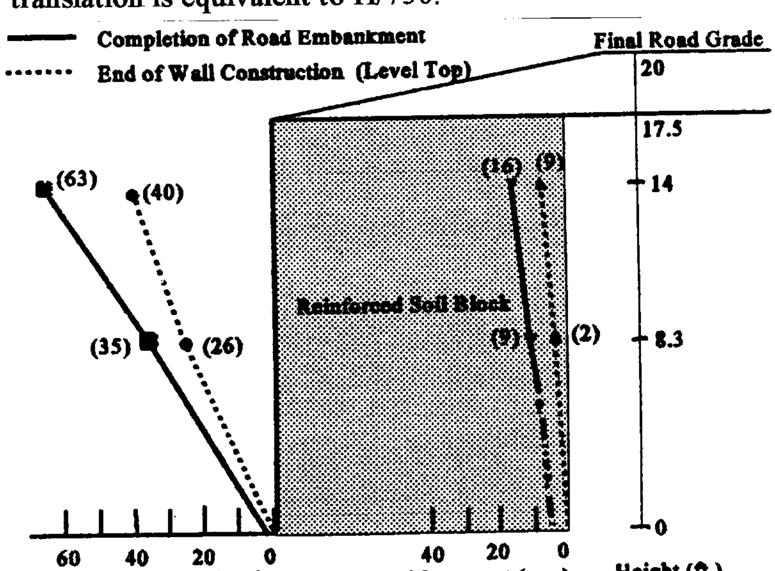

Fig. 8: Wall face and "coherent block" movements

4.2 CASE STUDY 2: "Mur Ebal" Full Scale Geotextile Reinforced Abutment, BASt, Germany

Project Description Balzer et al. (1991) reported the results of one set of full scale tests to study the performance of geotextile reinforced abutment built according to the French procedure named "Mur Ebal".

A needle punched non-woven polyester geotextile was used as reinforcement layers. The full scale abutment structure with a total height of 2.88 m had five geotextile reinforcement layers, each approximately 0.50 m in height. The geotextile was wrapped around to form an almost vertical wrapped-around wall face. The effective length of reinforcement was 2 m. The geotextile was tested in wide-width (20 inch) strip tests, and the tensile modulus was estimated to be about 10-20 k/ft from unconfined stress-strain tests, which gives a Sr value of about 6-12 ksf. The fill material used was gravelly sand with a friction angle of 39°.

Observed Wall Deformation The measured maximum deformations of the wrapped wall face was 11 mm at the end of wall construction. The full scale test abutment was subjected to a loading and unloading cycle until about 110 kN was reached. This corresponds to a strip load of 62 kN/m over a 0.9 m width. Then the loading were increased stepwise until failure occurred at about 640 kN. The development of the wall face deformations were moderate up to 70% of the failure load.

Prediction from Proposed Method

(A) At the end of wall construction :

Considering a baseline wall with Reinforcement Soil Relative Stiffness S_r of about 6 - 12 ksf, $(\delta/H)_{base}$ is found to be 2.9% to 2.6 % from Figure 1. Corrections for wall height with DI_H = 0.20 (Figure 4) resulted in the predicted deformation of 15 - 17 mm, which is over conservative by 30-50% as compared to the measured value of 11 mm.

Back calculation of the relative stiffness to match the wall deformation of 11 mm would suggest that the Reinforced Soil Relative Stiffness Sr was of the order of 21 ksf, and hence the reinforcement tensile modulus was of the order of 35 k/ft. The ratio of in-situ reinforcement modulus to the modulus from laboratory unconfined wide-width strength tests is about 2 to 3.5, which is consistent with other published results (Zornberg and Mitchell, 1993)

(B) With Concentrated Strip Load

The relative wall face deformation with respect to the strip load, normalized by the failure load, is shown in Figure 9. Also on this plot is the relative wall face deformation with respect to the strip loads on a reinforced soil wall containing inextensible reinforcements (Sr=1000 ksf). It can be seen from Figure 9 that the softer system gives a larger relative wall movement than the stiffer system for the same normalized strip loads. Figure 9 also shows that a strip load of about 70% of the failure load will double the wall face deformation for the geotextile reinforced wall.

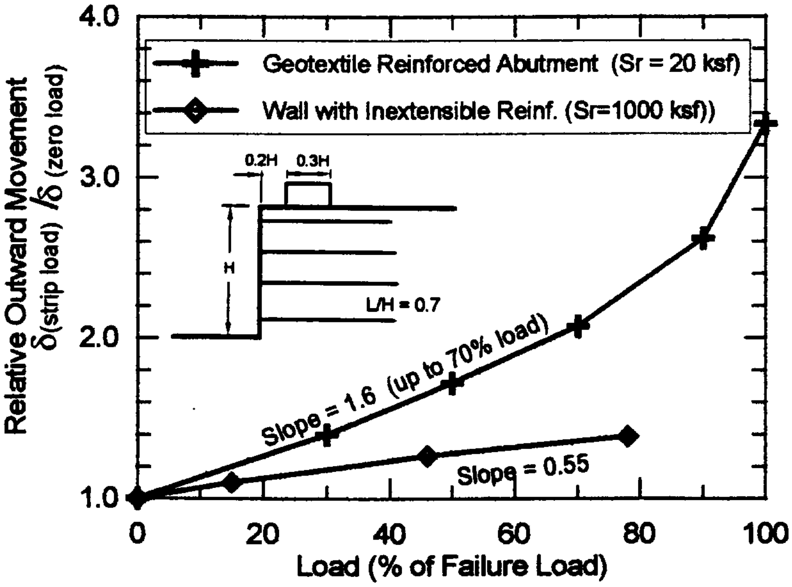

Fig. 9: Relative wall face movement with strip loads

5. CONCLUSIONS

A procedure was proposed to evaluate the maximum horizontal wall face deformations of reinforced soil walls. This procedure was based on the results of a parametric study using finite element program SSCOMP. The proposed procedure was then validated by case studies of a number of field walls and full-scale test walls, of which two were presented in this paper. Walls with different reinforcement materials, different facing elements, different height, subjected to various loading conditions for different applications were deliberately chosen to examine the applicability of the proposed evaluation procedure. Overall, the proposed deformation evaluation procedure gave very comparable results, generally within 15%, of the measured deformations.

Some key lessons learned from these two applications are:

1. Observations of the geogrid reinforced seawall at Gaspe Peninsula, Canada, confirmed that a reinforced soil block moves as a coherent block. The "coherent block" rotated about the base on the order of H/600 for a 17.5 ft high wall. The additional sloping backfill (for road embankment) caused the "coherent block" to translate by about H/750 in this case.
2. For the geotextile reinforced wall, the prediction using stiffness value obtained from laboratory tests gave much larger movements than the field measured values. Back calculation showed that an in-situ stiffness of 2 to 3.5 times the laboratory unconfined stiffness would match the field deformation very well.

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