FIELD PERFORMANCE OF STEEP WALLS OF NONWOVEN AND WOVEN GEOTEXTILES REINFORCING POORLY DRAINING SOILS

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ABSTRACT
Instrumentation, construction and performance of two 5.4 m-high steep geotextile-reinforced soil walls with fine local soils are described in this paper. A relative weak and extensible nonwoven geotextile and a stronger woven geotextile were compared. Both structures were instrumented in order to obtain internal horizontal displacements, facing displacements and matric suction. Interestingly, periods of medium and heavy rainfall were registered during the construction, providing a condition in which the advancement of wetting front could be affecting the performance of the structures. Matric suction of backfill soil was monitored by tensiometers installed along to the wall height. Data gathered from this program showed good performances of both walls, even after the backfill matric suction reduction. However, weak and extensible nonwoven geotextile-reinforced soil wall performed relatively well when compared to the stronger woven geotextile. Therefore, effect of soil confinement on strength properties of nonwoven geotextiles reinforcements, as well as the internal drainage provided by their hydraulic properties, may support the performance of the nonwoven geotextile-reinforced soil wall. Factors of safety and failure surface by limit equilibrium and Rankine earth pressure theory were compared to the measured strains and agreement of Rankine failure surface were observed either in nonwoven and woven sections. However, working stress method using Rankine stress state was more conservative then limit equilibrium analysis.

1. INTRODUCTION

The project consists of a geotextile-reinforced soil wall design as part of a retaining wall structure of the *Bairro Novo* residential condominium in Campinas, Sao Paulo, Brazil. The structure is a steep reinforced soil wall (1H:10V) with heights up to 9 meters constructed with a poorly draining local soil as a retaining structure along 300 m of the border of the natural embankment. A compacted unreinforced embankment of 4.5 meters high was constructed on the top of the retaining reinforced soil wall. The geosynthetic stabilized earth (GSE) walls were designed using woven geotextile reinforcements with adequate tensile strength. In order to compare the performance of two different geotextile reinforcements, an experimental section with 5.6 m high was constructed with a nonwoven geotextile in order to evaluate the efficiency of this type of reinforcement since mechanical properties and soil-reinforcement interaction may be improved in working conditions. Both sections, woven (W) and nonwoven geotextile (NW) reinforced walls were instrumented to compare performances at the same conditions and geometries. An interesting aspect is the use of a nonwoven reinforcement with 40% of the woven geotextile tensile strength. Therefore, a weak nonwoven geotextile was compared to a stronger woven geotextile. Figure 1 is a view of both instrumented sections after construction of the reinforced soil structure and during the construction of the compacted embankment on the top of the wall.

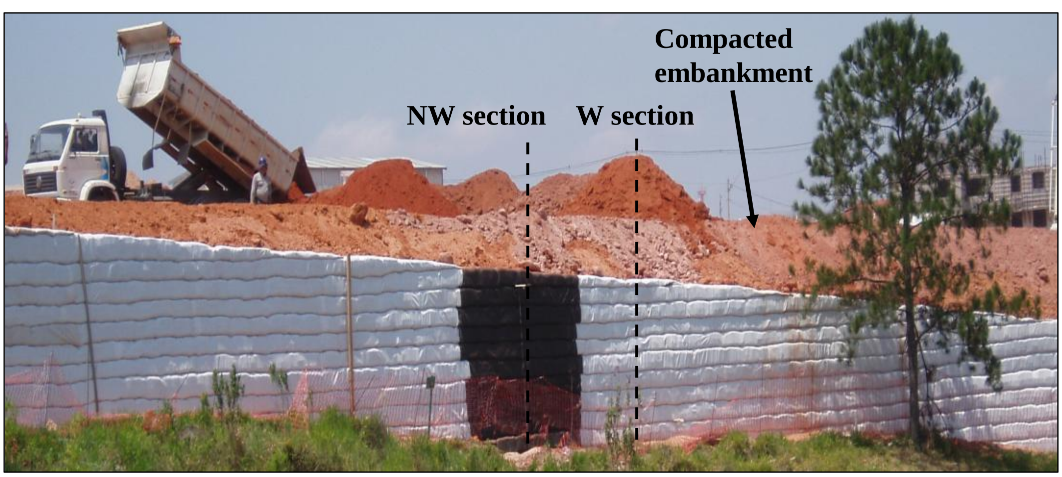

Figure 1. End of construction of experimental reinforced soil sections.

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This work intents to compare the results of instrumentation and design aspects of two geotextile reinforced soil walls. The woven geotextile section was properly designed to assure the safety in long term, following the FHWA guidelines and factors of safety, while the design guidelines used for the nonwoven geotextile section predicted failure of the structure. The conservatism of design approaches can be discussed for nonwoven geotextiles reinforced soil walls, since the mechanical properties of reinforcements used in designs are not realistic with the working conditions, and the hydraulic properties have particular effects with water presence.

2. DESIGN CONSIDERATIONS

2.1. Soil characteristics and compaction

The decision of using a local fine soil was the main aspect to become this technique competitive against others retaining wall solutions. The on-site soil was excavated from a natural slope in the same place where the geotextile reinforced soil wall was constructed. The soil is a nonplastic silty sand with 33% passing through No. 200 sieve (#200), maximum density of 19.4 kN/m^3 and optimum water content of 11.1% from modified Proctor tests (ASTM D1557-09). Shear strength parameters of soil obtained from CD triaxial tests are cohesion of 19 kPa and friction angle of 27^o. According to AASHTO (2002) specifications, reinforced soil wall backfills with no more than 15% passing U.S. 200 sieve are required to allow free drainage of water along the reinforced mass; otherwise, appropriated drainage system is required. In this design, drainage channels were constructed in the toe of the structure, between the reinforced soil wall and the unreinforced natural fill. No drainage layers were constructed on the top of structure; therefore, internal draining capacity of nonwoven geotextiles may be an important function in this design.

The backfill was compacted at ± 2% at the optimum water content of the modified Proctor curves using a vibratory roller (sheep foot pad) DYNAPAC$^@$ for compaction in areas distanced 100 cm from the face (Figure 2a). A softer compaction using impacting rammer was conducted at 100 cm from the face, in order to avoid excessive face displacements or local failures during construction.

(a) (b)

Figure 2. (a) Compaction and (b) soil bag facing construction.

2.2. Geotextile selection and layout

Woven geotextile reinforcement with tension strength of 50kN/m was selected by designers, while an experimental section with nonwoven geotextile reinforcement with tension strength of 20 kN/m was constructed for comparison purposes. The main aspect of selecting a weaker nonwoven geotextile for comparison purposes is based on experimental evidences in which improvements on strength and stiffness of nonwoven geotextile may occur under confinement conditions (McGown et al. 1982, Ling et al. 1992). Additionally, permeable reinforcements can be more efficient to reinforce poorly draining soils (Zornberg and Mitchell 1994, Mitchel and Zornberg 1995) since pore water pressures may be dissipated by reinforcements and an internal drainage is imposed. Both sections were constructed with reinforcement layers vertically separated each 40 cm and reinforcement length of 700 cm. Geometry and configuration is presented in Figure 3. A wrapped-around technique was used to form the wall face made from bags filled with manually compacted local soil (Figure 2b). Soil bags were used to replace formworks to facilitate the construction process. After completing the construction, a grass cover will be placed on it by seeding to provide a natural aspect to both the reinforced wall and the unreinforced embankment.

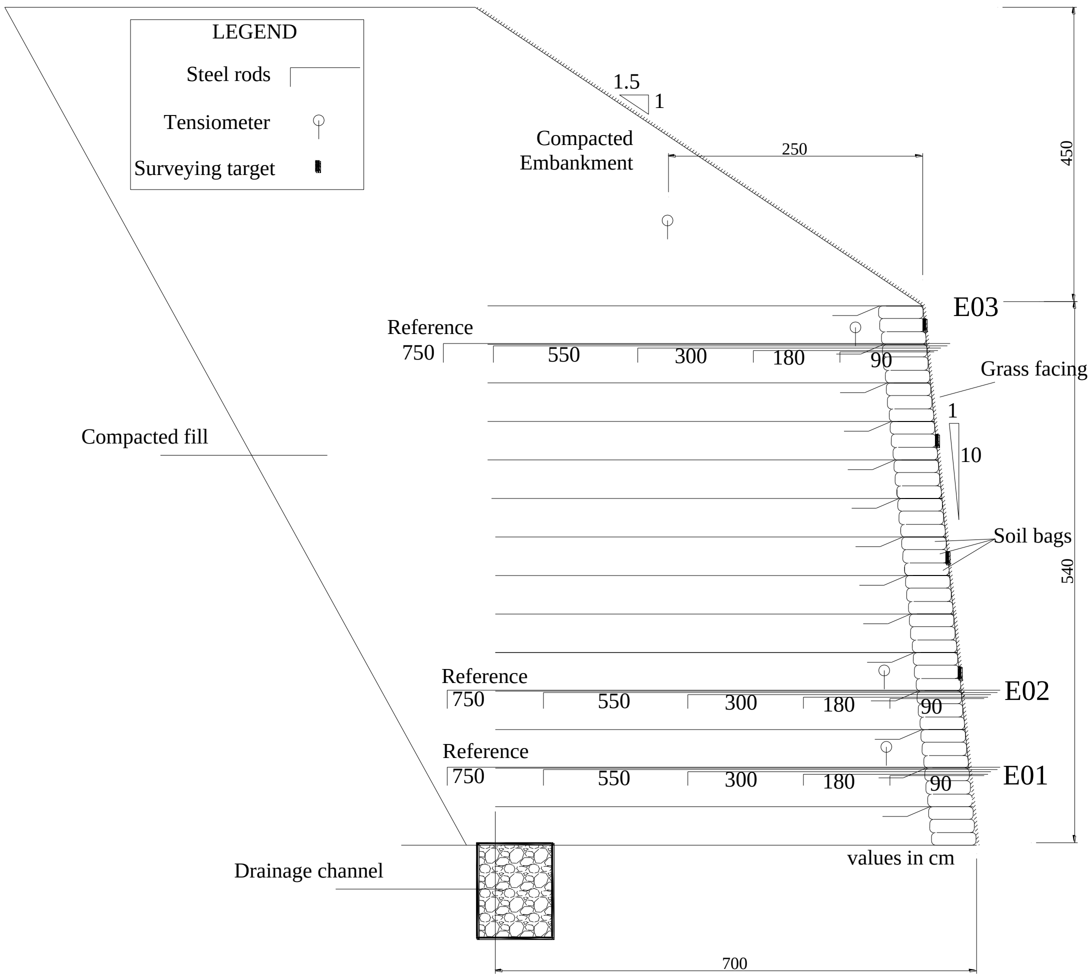

Figure 3. Cross section of the NW and W experimental walls.

2.3. Design methodology

The design of the woven geotextile wall was based on FHWA guidelines for reinforced soil walls, excluding recommendations about soil gradation. The working stress method according to Mitchell and Villet (1987) was used in this design. Rankine active earth pressure was considered for internal and external analysis. External analysis took into account were sliding, overturning and bearing capacity of foundation. Overall failure using a method of slices was conducted in order to verify potential failures outside of the reinforced zone. Internal stability analysis consisted of breakage and pullout of geotextile analysis. Reduction factor against construction damages, creep and degradation were considered to obtain the allowable tension strength of the woven reinforcement.

Factor of safety showed stability of this structure externally and internally. Otherwise, when nonwoven geotextile reinforcement with 40% of allowable tension strength of the woven reinforcement was used, the failure of the wall was expected due to the reinforcement breakage at the lowest layers. No reduction factor was used in this case.

3. INSTRUMENTATION

The instrumentation program was designed to monitor the performance of wall during and after construction. The instrumentation technique involved the measurement of internal displacements by means of steel rods, topographic survey for face displacements and matric suction monitoring by tensiometers. Figure 3 shows the location of the monitoring instruments to evaluate the performance of the walls. The internal displacement measurement system consisted in smooth metal rods spiked in soil at different distances from the face along the reinforcement length. A reference rod is placed outside the reinforced zone and relative measurements between reference rods and rods spiked inside the reinforced area were conducted. Metal rods were placed in points at 90, 180, 300 and 550 cm from the face. Tensiometers were installed at 150 cm from the face. All instruments were placed in 3 rows in different heights, designated as E01 at 80 cm from the toe, E02 at 160 cm from the toe, and E03 at 500 cm from the toe, as illustrated in Figure 3.

Surveying targets were attached at the exposed wrapped-around face at 160, 280, 400 and 520 cm from the toe. Figure 4 presents a picture of the metal rods and a tensiometer closed to the face, as well as the two instrumented wall sections (Figure 4b). Readings for the instruments have been acquired from the beginning of the wall construction and they are still being acquired. Results presented in this paper are partial results, and future results will be reported later in the literature.

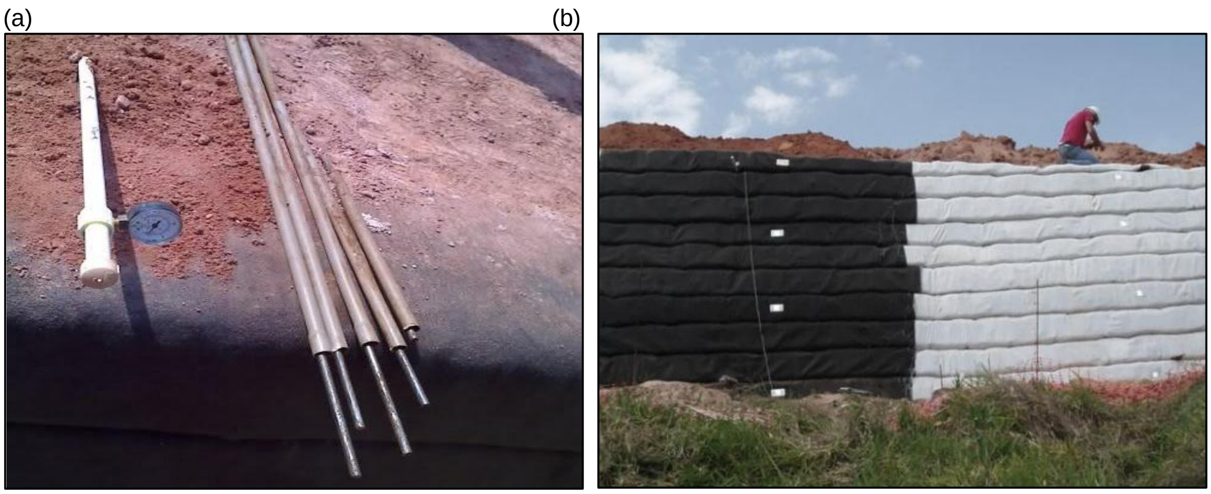

Figure 4. Instrumentation: (a) metal rods and tensiometers; (b) two instrumented sections.

4. CONSTRUCTION

The construction of the entire structure took 86 days from the preparation of the foundation to the final of construction of the unreinforced embankment on the top of the reinforced walls. The preparation of the foundation and drainage systems started in August 2010. The construction of the reinforced wall started in September 2010 and it finished in December, 2010. Thereafter, the unreinforced embankment was constructed and it was concluded in late January 2011. Figure 5a shows the steps of construction versus time of performance monitoring. Rainfalls during and after construction are presented in Figure 5b. The reinforced wall was constructed in periods of light rainfalls. During the first month of construction (September 2010), 11 days of rainfall were registered reaching a total volume of 110 mm. On the second and last month of construction of the reinforced structure, 13 days of rainfalls had occurred and the accumulation of 95 mm was registered. During the unreinforced embankment construction, heavy rainfall periods started with accumulated rainfall intensity of 330 mm for 18 rainy days. After the end of construction, the most heavy rainfall period was registered with intensity of 490 mm during 18 days. Figure 6 depicts the structure during construction of the reinforced embankment between December 2010 and January 2011, presence of water is clearly observed for this period.

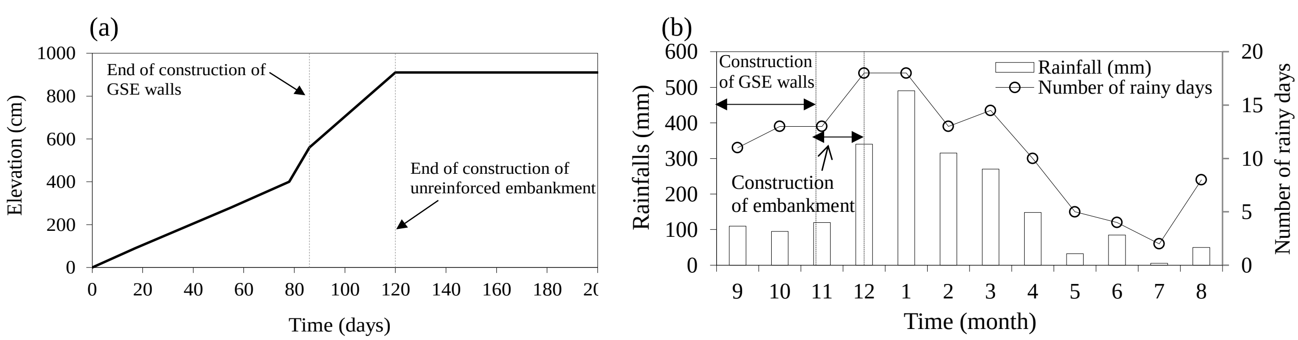

Figure 5. Construction and rainfalls: (a) Period of construction of reinforced structure (GSE) and unreinforced embankment, and (b) rainfall events during and after construction.

Figure 6 – View of geotextile reinforced soil walls during construction of the unreinforced embankment.

5. RESULTS AND DISCUSSIONS

5.1. Matric suction measurements

Matric suction of the fill was monitored only for the nonwoven geotextile section at points close to the face (150 cm). Matric suction along the elevation of the structure, after the end of wall construction, is presented in Figure 7. It is clearly observed no complete saturation of backfill during the period of monitoring, with minimum values of matric suction of 5 kPa. However, a significant decrease of this parameter was measured from the end of construction (86 days) to 329 days of working. The presence of water is evidenced and the wetting reaches the unreinforced embankment at 720 cm from the toe in 39 days after construction (125 days). After 114 days, the water advances the instrumented row E03 at 520 cm from the toe. Additionally, the lowest values of matric suction were observed 243 days after construction distributed along all instrumented rows (329 days). Interestingly, matric suction increments had occurred in this period for upper layers of the nonwoven geotextile section that is an indicative of drying or drainage of water from the reinforced zone. Also, dry periods can be observed in this time.

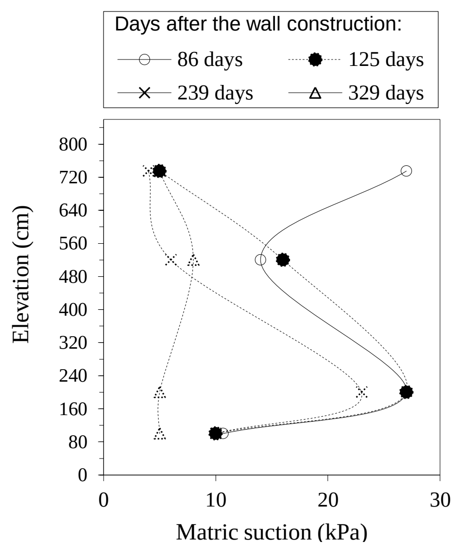

Figure 7. Distribution of matric suction along the elevation of the structure after construction.

5.2. Strains in the geotextiles

Reinforcement strains were obtained from the relative horizontal displacements between the reference rod and the metal rods attached along the reinforcement length at different distances. The distribution of relative displacement along the reinforcement between metal rods and wall face after 329 days from the end of the wall construction is presented in Figure 8, in this figure, sigmoidal curves fitting the raw data is drawn in order to have a smooth representation of the distribution of displacements along the reinforcement length (Zornberg and Arriaga, 2003). The displacements of nonwoven geotextiles were higher than woven geotextiles, with higher values at the lowest layer. In the woven geotextile section, higher displacements were observed in the upper layers, which can be an evidence of the saturation effect, since the permeable reinforcement could be dissipating the pore water pressures what could not be occurring for woven geotextiles.

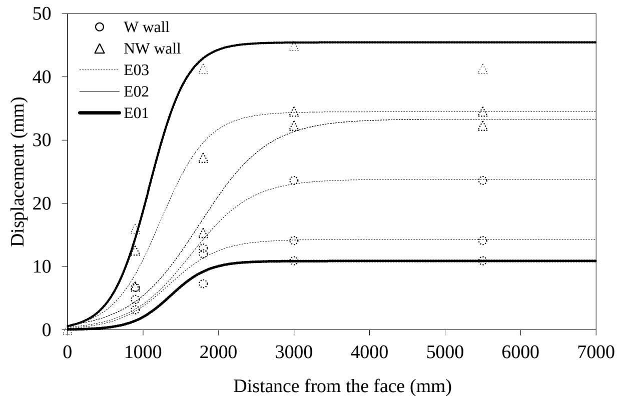

Figure 8. Distribution of relative displacements between rods and face along the geotextile length for nonwoven and woven sections.

The sigmoidal fitting shown in Figure 8 was also used to evaluate the distribution of strains along the reinforcement as presented by Zornberg and Arriaga (2003). Geotextile strains values can be obtained by calculating relative movements between the mechanical extensometers (metal rods) and dividing them by the initial distance between rods. However, the use of this technique is not useful in this case, since the distance between measured points are not small enough to get a real strain between points. Consequently, the raw data from extensometer displacement was initially smoothed by fitting the data to a sigmoidal curve and the distribution of strains along the geotextile length could be obtained by deriving of the displacement function as:

$$\varepsilon = d\left(\frac{1}{a+be^{-cx}}\right)/dx \tag{1}$$

where d is the extensometer displacement, x is the distance from the face to the measured point, and a, b and c are parameters defined by the fitting of sigmoidal curves to the raw data using the minimum squares technique.
The distribution of strains for the nonwoven and woven sections in the different rows of instrumentation (E01, E02 and E03) is presented in Figure 9. An interesting aspect is that both section presented the same levels of strains in working conditions for the instrumented rows E02 and E03, even the tension strength of nonwoven geotextiles be 40% of tension strength of woven geotextiles. However, the strains on the base of the structure were smaller for the stiffer reinforcement, i.e., woven geotextile. Explanations of this behavior can be around the major effect of water pressures in woven geotextiles than nonwoven geotextile that can provide dissipation of water, resulting in same levels of strains. If it is true, values of strains would be smaller without the water effect. Therefore, the soil-geotextile interaction could be better for nonwoven geotextile section. However, though the effect of wetting could be affecting the woven geotextile-soil interaction, the nonwoven geotextile chosen for this comparison is less stiff resulting in high or similar values of strains.

Additionally, mechanical properties of nonwoven geotextile under confined conditions could be improving the reinforcement stiffness. Among the reasons for improvement are: effect of impregnation of soil particles into the nonwoven geotextile structure (Mendes e Palmeira, 2008) and confinement stress effect. McGown et al. (1982) compared the confined and unconfined creep tests and observed an effective decrease of creep strains of nonwoven geotextiles in soil confinement conditions.

Figure 10 shows the maximum peak strains distribution with time in the geotextiles along the elevation of both wall sections. So that, strains development could be observed during the period of construction of the unreinforced embankment on the top of the reinforced wall. After this period no additional displacements were registered for the NW section. On the other hand, additional displacement of 0.5% was noticed in the W section after 239 days of lifetime. Since matric suction decreases between 239 days and 329 days of working, and additional strains were observed in this period only for woven geotextile (permeable reinforcement) and no additional strains were developed for permeable reinforcement (nonwoven geotextile), this is an evidence that the hydraulic capacity of nonwoven geotextile can be influencing the interaction between soil and reinforcement.

Considering that the nonwoven geotextile has 40% of the woven geotextile tension strength, and the strains levels were very similar, it can be concluded that the nonwoven geotextile is positively affected under confined conditions, and the woven geotextile can be affected negatively under the wetting conditions.

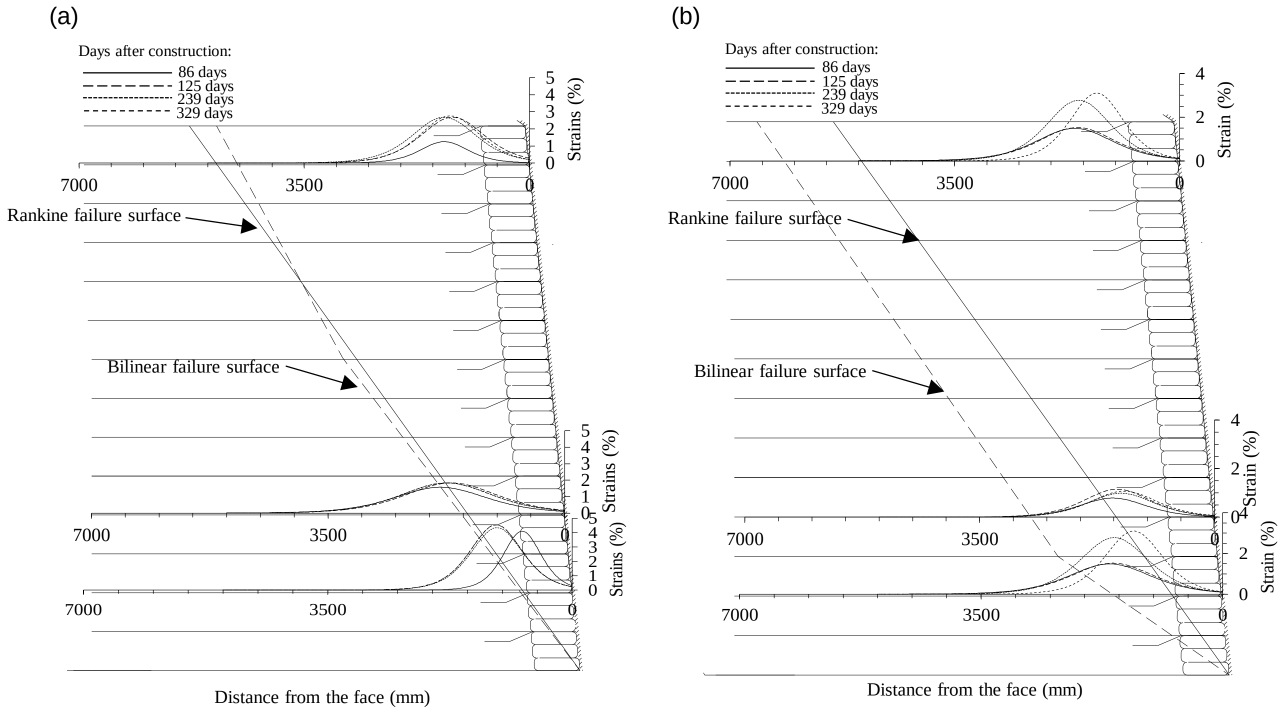

Figure 9. Geotextile strains distribution for (a) NW section and (b) W section in the instrumented rows E01, E02 and E03.

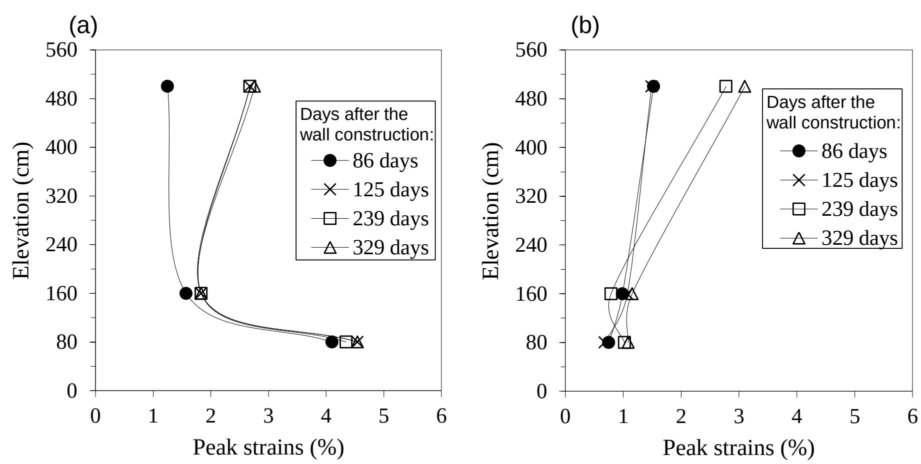

Figure 10. Distribution of geotextiles peak strains along the elevation of (a) NW section and (b) W section.

5.3. Implication on design

Conventional designs of reinforced soil walls use of working stress methods with Rankine or Coulomb failure surface and stress states in order to determine the reinforcement vertical spacing. In case of reinforced slopes, conventional limit equilibrium analyses are used taking into account the stabilizing moment provided by the reinforcements. Figure 9 shows the location of maximum geotextiles strains during times of measurement of the instrumentation for both geotextiles sections. Additionally, theoretical possible location of peak strains of Rankine failure surface and limit equilibrium analyses are indicated. The peak strains measured for both geotextile sections have an agreement with Rankine failure surfaces only in the lowest instrumented layers. There is no clear understanding about this behavior, but the effect of infiltration of water locally near the face or/and compaction stress during construction may be changing the location of the maximum tension forces and, consequently, the peak strains.

Limit equilibrium analyses were carried out to compare to the location of the measured peak strains and the Rankine failure surface (Figure 9). For the nonwoven section, the bilinear surface approached to the Rankine failure surface and, consequently, discussions above were valid. On the other hand, the slip surface for the woven geotextile is quite different and no agreement with measured points was verified. In this analysis, the factor of safety of the internal stability of the nonwoven and woven geotextile section was 1.64 and 3.02, respectively. Therefore, no failure was predicted contrasting the failure predicted by the working stress analysis for the nonwoven section, in which the minimum factor of safety was 0.88. In case of woven section, minimum factor of safety was 2.19. Therefore, by comparing the factor of safety predicted by both methods and the level of measured strains, the use of earth pressure theories to predict reinforcement loads shows to be conservative. In this case, limit equilibrium analysis seems to be more realistic.

Figure 11 presents the measured maximum peak strains and their location along the elevation of both sections and the maximum peak strains and their respective location from other geotextiles retaining walls reported in the literature. Although the location of the maximum peak strain has considerably differences when compared to the other walls here reported, structures reported by Tatsuoka and Yamauchi (1986) and Wu (1992) show similar peak location and approximated levels of peak strains. In cases which the peak strain location are not the same, levels of strains are similar. For the woven geotextile section, the experimental structure reported by Wo and Kim (2006) show similar values of maximum peak strains. In many cases, nonwoven geotextiles and woven geotextiles-reinforced walls shows similar level of strains as those observed in the experimental sections presented in this work. Therefore, design approaches should consider mechanical properties of geotextiles in soil confinement conditions. Apparently, the use of strength parameters of nonwoven geotextiles from unconfined tension and creep tests are not consistent with the real behavior in reinforced soil walls applications. This comparison is also useful to validate the instrumentation for internal displacement measurements used in this work since the level of strains and location were consistent with other walls reported in literature. The steel rods used for internal displacement monitoring is an inexpensive and straightforward technique that could be used to monitor different reinforced soil walls.

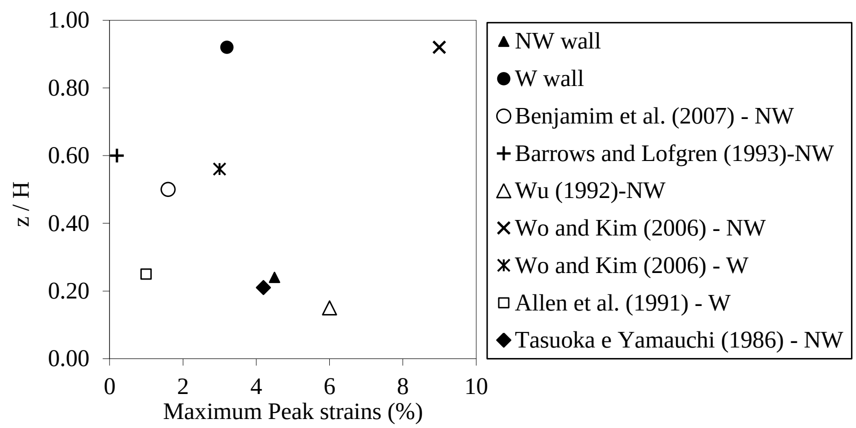

Figure 11. Comparison of maximum peak strains and peak strains locations between the NW and W section and cases reported in the literature.

6. CONCLUSIONS

Design, construction and instrumentation results from two reinforced soils walls were reported in this work. A behavioral comparison between a weak nonwoven and a stronger woven geotextile reinforcing poorly drained soils was described. The woven geotextile section was properly designed taking into account safety factor according to the FHWA guidelines, assuring the safety of the structure; on the other hand, approached design predicted the failure of the nonwoven geotextile section. Based on the instrumentation data and design aspects, the following conclusions can be drawn:

- Use of Rankine stress state and unconfined mechanical parameters of nonwoven geotextiles are a source of conservatism for nonwoven geotextile-reinforced soil walls designs;
- Limit equilibrium analysis was less conservative either for woven and nonwoven geotextile sections;
- In general, Rankine failure surface showed better agreement with measured peak strains than bilinear surfaces from limit equilibrium analysis.

- Levels of strains for both evaluated sections were similar even though the tension strength of nonwoven geotextile is 40% of the woven tension strength. This is a strong evidence of the confinement effect and draining capacity of nonwoven geotextiles. Additionally, nonwoven geotextile strains were totally mobilized during construction of the compacted unreinforced embankment on the top of the structure, no additional strains were observed even in periods when the reduction of matric suction was noticed. On the other hand, for the woven section, additional strains were observed not only during construction but still during lifetime.

- Level and location of peak strains showed consistency with the performance of geotextile-reinforced soil walls reported in the literature.

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