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INVESTIGATING FAILURE OF A GEOSYNTHETIC-REINFORCED SOIL WALL IN BLACK HAWK, COLORADO

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ABSTRACT

Approximately 18 months after construction, an 11-m high rock faced Geosynthetic-Reinforced Soil (GRS) wall was found to suffer from subsidence of about 100 mm on the crest, along with rocks being dislodged from the wall face from time to time. In the ensuing three months after these signs of problems were first detected, the subsidence on the crest of the wall increased to about 250 mm with significant lateral bulging on the wall face. The wall comprised two sides. The problems occurred primarily on one side of the wall; while the other side showed little distress. The entire wall eventually had to be demolished and reconstructed. Prior to reconstruction, a forensic program was undertaken and analyses were performed to examine the causes of failure. This paper describes the geometry and properties of the wall, the events leading to failure of the wall, and the post-failure analysis. It was concluded that the failure likely stemmed from two causes: (1) poor compaction of the fill in harsh winter weather during which the wall was constructed, and (2) wetting of the fill on the side of wall where failure occurred, caused by discharge of water through an abandoned pipe behind the reinforced fill. The wall was reconstructed with well-compacted granular fills with drainage units installed behind the reinforced fill. The reconstructed wall has since performed satisfactorily.

INTRODUCTION

A rock-faced Geosynthetic-Reinforced Soil (GRS) wall was constructed in Black Hawk, Colorado in December 1996. The wall was 11.3 m high, and was constructed in bitter cold winter weather when the temperature often went well below freezing. In May 1998, approximately 100 mm of vertical settlement was observed in the northern part of the wall crest, accompanied by visible lateral bulging on the wall face. The settlement progressively increased to about 250 mm in August 1998, as shown in Figure 1. The lateral budging also became rather significant, especially near the mid-height of the wall, as shown in Figure 2. Due to the excessive deformation, the wall was demolished in October 1998, and was immediately reconstructed after demolition. The reconstruction was completed in February 1999. The reconstructed wall has since performed satisfactorily.

This paper describes the wall configuration, construction materials, events leading to failure of the wall, a forensic investigation program, laboratory tests for examining the effects of wetting, and analysis of the causes of the settlements.

Fig. 1. Settlement at the Wall Crest before Demolition

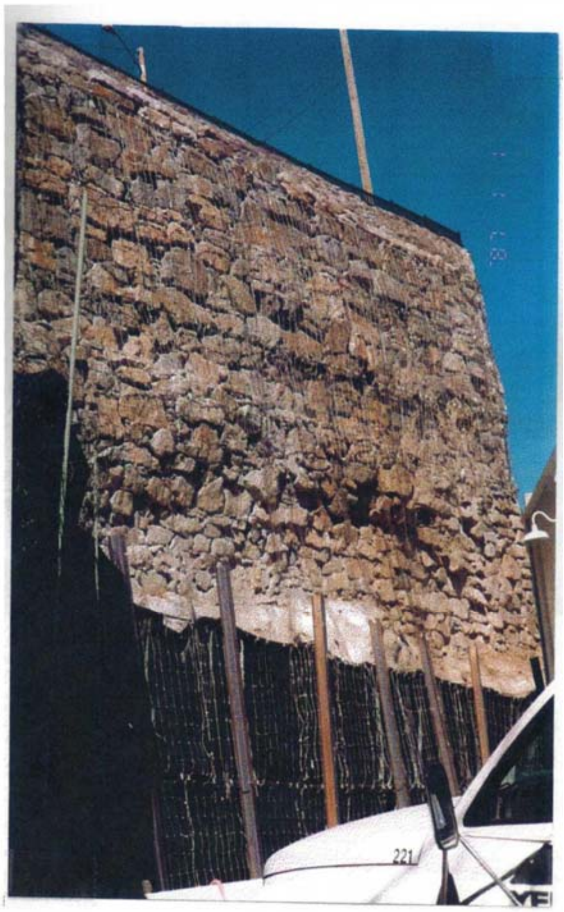

Fig. 2. Lateral Bulging at the Rock Facing before Demolition

PROJECT DESCRIPTION

Figures 3 and 4 show, respectively, the side and plan views of the rock-faced GRS wall. The wall was located in front of the City Hall Annex in Black Hawk, Colorado. The wall was 11.3 m high, and the crest of the wall was a paved parking lot for the City Hall Annex. The wall comprised two sides: the South wall and North wall (see Figure 4).

The wall was constructed over bedrock made of schist with granite inclusions. The bedrock had been stabilized by rock bolts with its face covered with shotcrete over wire meshes.

The facing of the GRS wall was formed by stacking rocks with sizes ranging approximately from 0.3 m to 0.9 m. Voids between the large rocks were filled with small rocks or "chinking" rocks.

The backfill of the GRS wall was a mixture of gravelly silts and gravelly sands obtained on-site. The backfill was reinforced with layers of geosynthetic reinforcement sheets placed horizontally at 0.3 m vertical spacing. The reinforcement was a polypropylene woven geotextile. The reinforcement length was 6.4 m at the top and gradually reduced with depth to accommodate the sloping rock cut behind the reinforced soil zone (see Figure 3). The front edge of each reinforcement sheet was sandwiched between vertically aligned rocks at the wall face to form a frictional connection between the reinforcement layer and facing rocks.

Drainage of the backfill was facilitated by three measures (1) a 150-mm thick layer of open-graded rocks was placed at the base of the wall, (2) a 300 mm by 300 mm triangular wedge of aggregates was installed behind each course of the facing rocks, and (3) strips of 300-mm wide geocomposite drainage units were placed behind the reinforced soil zone at 3-m intervals and extended underneath the shotcreted facing of the wall foundation.

The construction involved three major steps: (1) excavation of on-site soil and rock, (2) stabilization of the foundation, and (3) construction of the GRS wall. Approximately 3,000 m^3 of soil and 1,900 m^3 of rock were excavated. The excavated soil was employed as backfill for wall construction. The wall was constructed by a recurrent procedure that can be described by the following steps:

Step 1: lay a course of rocks of approximately 0.3 m in height;

Step 2: place backfill and compact the fill using a "jumping jack" compactor with a set pattern;

Step 3: lay a layer of reinforcement to the front edge of the rock face and covering the compacted fill;

Step 4: repeat Steps 1 through 3 until the design height is reached.

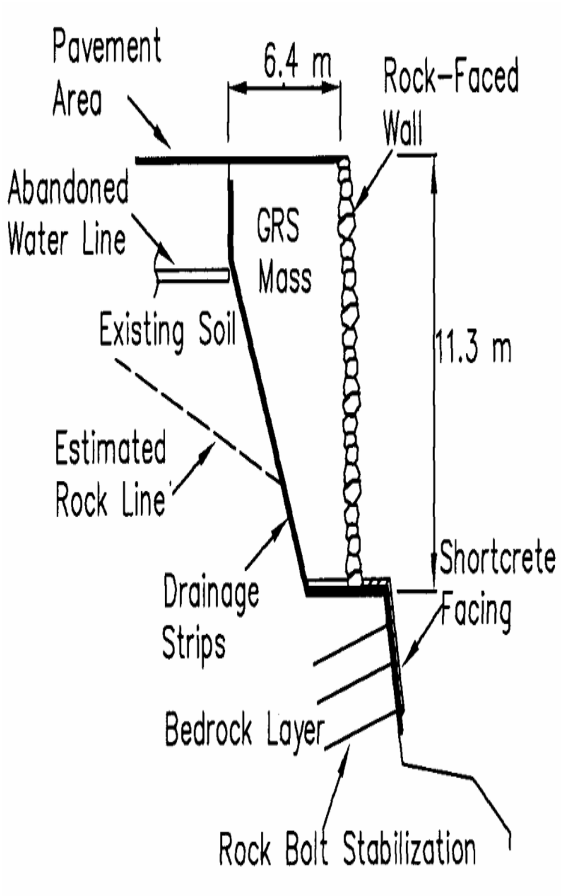

Fig. 3. Side view of the wall configuration.

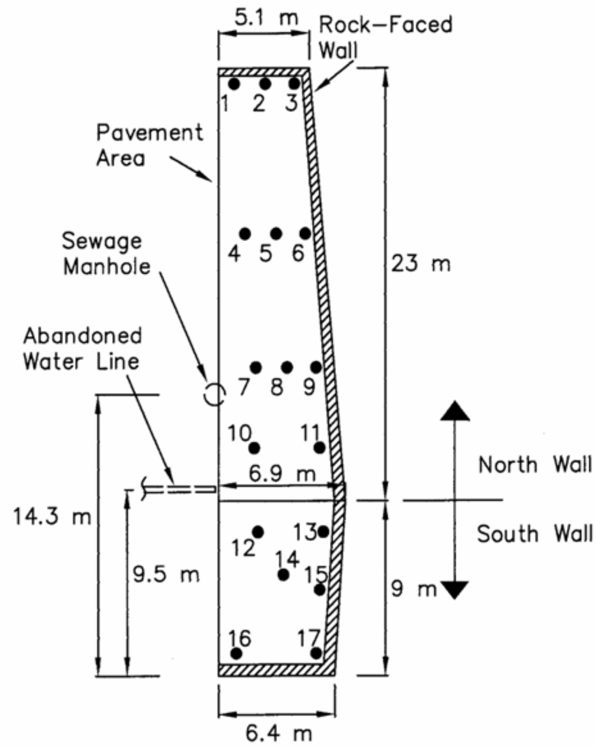

Fig. 4. Plan view of the wall configuration.

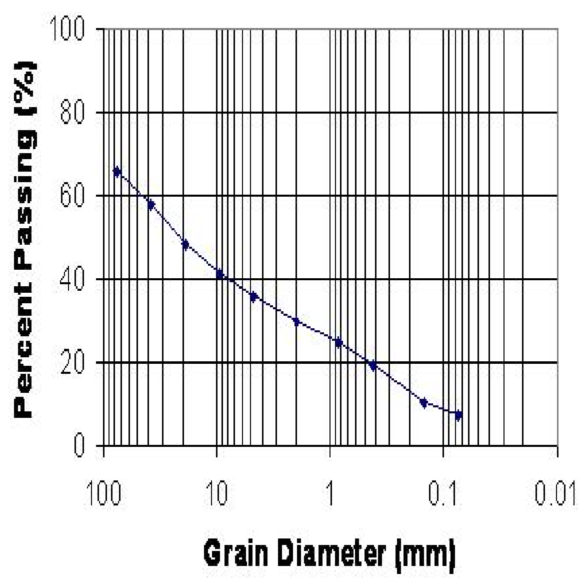

Fig. 5. A Gradation Curve of the Soil

MATERIALS

The backfill was a gravelly silt and gravelly sand, classified as GP-GM. The soil contained 20% to 50% of oversized particles (retained on the 19-mm sieve) and has about 7% of fines (passing the 0.075 mm sieve). A gradation curve of the soil is shown in Figure 5. The maximum dry unit weight and optimum moisture content for the minus 19-mm portion of the soil are 20.63 kN/m^3 and 8.8 %, respectively, per ASTM D1557 method C. The coefficient of permeability for the compacted gravelly soil is judged to be on the order of 10^{-3} cm/sec.

For the polypropylene multifilament-on-tape woven geotextile reinforcement, the wide width tensile strengths in the fill and warp directions, per ASTM D4595, are both 70.0 kN/m. The apparent opening size, per ASTM D4751, is 0.6 mm. The permittivity, per ASTM D4491, is 0.15 sec^{-1}. These values were provided by the manufacturer of the geotextile.

FORENSIC INVESTIGATION PROGRAM

A forensic investigation program was undertaken to investigate the causes of failure. The program consisted of visual inspection, settlement measurement of the paved area on the wall crest, and field density tests of the backfill. The field density tests, conducted during demolition of the wall, included nuclear density tests (per ASTM D2922) and a water replacement test in a test pit (per ASTM D5030).

Events and Visual inspection

The construction of the wall was completed in December 1996 when the weather was bitter cold. In May 1997, the wall face experienced tumbling of rocks, including small rocks and a few larger rocks of approximately 300 to 600 mm in diameter. Outward bulging near the mid-height and one-third from the top of the wall was apparent at some locations. A program of regular “scaling” of the wall was initiated after observing these problems. The “scaling” involved tapping with a hammer, prying each rock in the wall face, and dislodging any loose pieces.

In May 1998, about 100 mm of vertical settlement was observed in the paved area of the North wall. Some distinct cracks were also noted on the pavement. The cracks were nearly parallel to the wall face, approximately 6.5 to 7.5 m away from the wall face. However, the wall face condition appeared unchanged from that observed in May 1997.

In August 1998, the settlement increased to about 250 mm. The lateral budging had become rather significant, especially at the mid-height of the wall. The wall was subsequently demolished for reconstruction due to the excessive deformation and failing wall face.

During wall demolition, it was found that the reinforcement sheets connected to the facing were still sandwiched between the vertically adjacent rocks. No rupture of the reinforcement layers was detected. It was discovered that there was an abandoned water pipe about 2.7 m deep from the surface, behind the reinforced soil zone. The abandoned pipe was located at about 9.5 m from the south-end, and there was also a sewage manhole at about 14.3 m from the south-end (see Figure 4 for location). The abandoned water pipe apparently collected water from a large unpaved area behind the City Hall Annex.

Settlement Measurement

The settlement at different locations of the paved area (Pts. 1 to 17 in Figure 4) on the crest was measured. The survey measurement was taken on May 12, June 5, and August 21, 1998. The elevations recorded on May 12, 1998, approximately one year after initial wall distress was detected, were used as the reference for the subsequent settlement measurements on June 5 and August 21, 1998. The settlements of the pavement area measured between May 12 and June 5, 1998 (over 25-day period) and between June 5 and August 21, 1998 (over 45-day period) are listed in Table 1. From May 12 to June 5, 1998, the largest settlements in the North and South walls were 240 mm (at Pt.6) and 3 mm (at Pt.12), respectively. Between June 5 and August 21, 1998, the largest settlements in the North and South walls were 125 mm (at Pt.11) and 27 mm (at Pt.12), respectively. Note that the largest settlements all occurred adjacent to the wall face.

The settlement profiles relative to the wall geometry on May 12, 1998 are shown in Figure 6. The settlement profiles resemble a bowl shape with the smallest settlements being at the north- and south-ends. The settlement profile of the June 5 measurement was more uniform than that of the August 21 measurement which showed a dramatic increase of settlement near the middle of the North wall. A maximum settlement of 140 mm occurred between May 12 and August 21, 1998.

Table 1: Measured Settlements of the Pavement Area

settlements (mm)		
Point*	May 12 to June 5, 1998	June 5 to August 21, 1998
1	3	0
2	6	18
3	6	15
4	9	34
5	15	61
6	24	67
7	12	58
8	18	70
9	15	125

10	15	79
11	9	37
12	3	27
13	3	18
14	3	18
15	0	18
16	0	15
17	0	15

*North Wall (Pt. 1-11) and South Wall (Pt. 12-17)

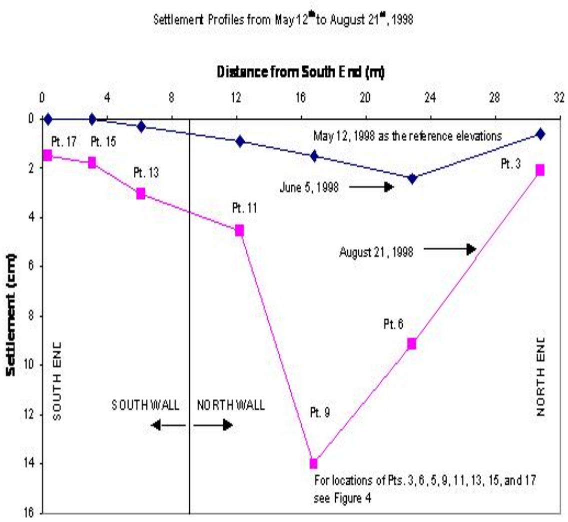

Fig. 6. Settlement Profiles between May 12 and August 21,1998

Field Density Tests

Nuclear density tests were performed at different depths during wall demolition. In addition, a water replacement test in a test pit was conducted to verify the density of the backfill measured by the nuclear density tests. Figures 7 and 8 show the dry unit weights and moisture contents at 6 m and 9 m below the wall crest. The average percentage of oversized material (larger than 19 mm) was 30%. The maximum dry unit weight, per ASTM D1557 method C with rock correction of 30%-oversized material, was 21.9 kN/m^3, with the optimum moisture content (OMC) being 6.3%. The average dry unit weight at the depth of 6 m was 17.8 kN/m^3 (i.e., 81 % standard Proctor relative compaction). The average dry unit weight at the depth of 9 m was 18.7 kN/m^3 (i.e., 86 % standard Proctor relative compaction). Figure 8 shows that the water contents increased from North and South ends toward the middle part of the North wall where the largest settlement occurred. The maximum water contents were 8.4% (i.e., 2.1% wet of OMC) at the 6 m depth and 7.9% (i.e., 1.6% wet of OMC) at the 9 m depth.

The water replacement test, per ASTM D5030, was conducted to verify the results of the nuclear density tests. The test procedure involved using a 1.2-m diameter circular template as a guide to excavate a test pit. The exposed surface of the test pit was lined with a flexible plastic sheet and the volume of the pit was measured by the replacement volume of water. The test pit was located approximately 24.0 m from the south end and 9.0 m below the wall crest. The material from the test pit was described as “a gravel with sand and silt”, moist, dark brown with 35 % of oversized particles (greater than 19 mm). The test indicated that the dry unit weight was 18.1 kN/m^3 and the moisture content was 5.8 %. These values are consistent with the average values determined from the nuclear density method.

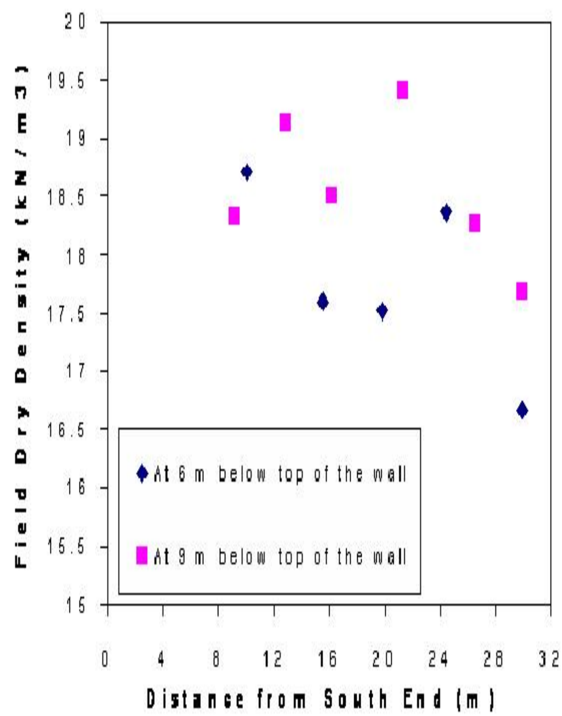

Fig. 7. Measured Field Dry Unit Weight

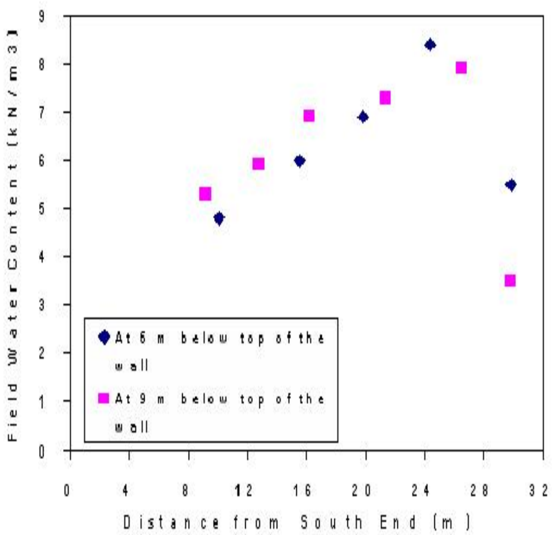

Fig. 8. Measured Field Moisture Content

SETTLEMENT ANALYSES

Backfill Settlement and Preliminary Settlement Analysis

In May 1997, approximately six months after the wall was constructed, the wall face bulged visibly and a few facing rocks became dislodged. Although not measured, it is likely that some settlements have occurred with the lateral deformation. After that event, little settlement was noted by visual inspection. In May 1998, the North wall exhibited a maximum settlement of approximately 100 mm. A settlement measurement program was then initiated. The North wall settled about 150 mm in the next 70 days (from May 12 to August 21, 1998). The maximum total settlement after the end of construction in the North wall was about 250 mm.

It should be noted that all soil retaining structures experience settlement at the crest after construction to some extent due to self-weight of the backfill. Previous field measurements have indicated that settlements up to 1% of the wall height after construction of a reinforced soil structure are not unusual (Findley, 1978; Jones and Hassan, 1992).

Figure 9 shows a probable scenario of the settlement history at the top of the North wall and the South wall. A total settlement of 50 mm (i.e., 0.45% of the wall height) due to self- weight of the backfill was assumed to have occurred from December 1996 (end-of-construction) to January 1998. It is assumed that the settlement rates between January and May, 1998 were constant, and equal to the settlement rates of the first set of the survey data recorded between May 12 and June 5, 1998. The rates of settlement were 1 mm per day in the North wall and 0.1 mm per day in the South wall. From Figure 9, the maximum total settlements after the end-of- construction were determined to be about 240 to 290 mm in the North wall (at Pts. 8 and 9, Figure 9) and 85 to 95 mm in the South wall (at Pts. 12 and 13, Figure 9).

When a GRS wall is well-designed and well-constructed, the rate of post-construction settlement will typically decrease with time. However, the rate of the post-construction settlement of a GRS wall may increase with time if there is significant time-dependent settlement in the foundation or if there is very large external loads applied on the crest. For this GRS wall, there was no sign of movement in the rock foundation beneath the wall. Also, the only external load on the wall crest was from the weight of vehicles in the parking area. This external load is considered negligible compared to the self-weight of the fill to have any significant effect on the wall deformation.

Upon examining Figure 9, the magnitude and the rate of post-construction settlement in the North wall were considered very unusual. There was a significant increase in the rate of settlement in the North wall after January 1998. The rate of settlement was approximately 5 to 10 times of that in the South wall. This is highly unexpected as the South wall had practically the same conditions as the North wall: they have presumably the same wall configuration, same backfill material, same soil placement procedure, and same construction procedure.

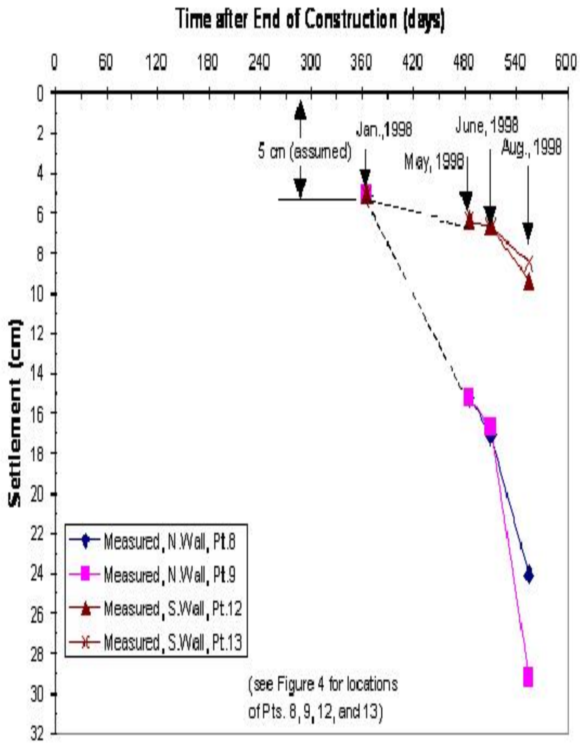

Fig. 9. Settlement versus Time after End-of-Construction

Based on the visual inspections and field measurements, the following scenario that led to wall failure is proposed. From the field density tests during wall demolition and judging from the soil type and the equipment used for compaction, the backfill was believed to have been lightly compacted with relative compaction of around 85%. After construction, the lightly compacted backfill settled considerably due to self-weight of the fill. The settlement subsequently led to some cracks in the pavement area. The surface water percolated through cracks and caused wetting of the backfill. Moreover, additional water was found to come from the abandoned pipe and sewage manhole, discovered during wall demolition. Backfill wetting accelerated the settlement and led to excessive settlement and distortion of the wall facing. The wetting-induced settlement hypothesis is examined in the following sections.

Previous Study on Settlement Due to Wetting of Granular Soils

Previous studies have shown that the settlement due to an increase of the water content (i.e., wetting) may occur in partially-saturated soils ranging in sizes from fine clays to boulders (Fumagalli, 1961; Dudley, 1970; Leonards and Altschaeffl, 1971; Jennings, 1967; Lawton *et al*, 1989). In view of the rather different deformation behavior of the North and South walls despite nearly identical material and geometric conditions, the deformation due to wetting was subsequently investigated in this study. The discovery of an abandoned water pipe behind the reinforced fill of the North wall also added to the likelihood that the excessive settlement in the North wall was a result of wetting.

Jennings and Knight (1957) proposed a “double-oedometer” test for estimating settlement of soils due to wetting. In the double-oedometer test, one-dimensional compression tests are performed on two specimens of the same soil: one prepared at its natural water content (termed the “dry” specimen) and the other is pre-wetted prior to testing (termed the “wet” specimen). The amount of compression due to wetting, or the wetting-induced strain (ε_{wet}), at any vertical pressure can be estimated from the difference in the axial strains of the “dry” and “wet” compression curves at that vertical pressure. It was observed that the amount of settlement was nearly the same whether the soil was loaded first and then wetted, or wetted first and then loaded. With field measurement data, Scherrer (1965), Dudley (1970) and Nobari and Duncan (1972) confirmed that the settlement due to wetting can be predicted fairly accurately using the results of the double-oedometer tests.

Large-Size One-Dimensional Compression Tests

A series of large-size (290 mm diameter) one-dimensional compression tests were conducted to examine the amount of compression due to wetting of the backfill that was used in the construction of the GRS wall. Only the minus 19-mm portion of the soil was tested. The specimen was prepared at a prescribed value of water content and compacted inside a rigid cylindrical mold, 290 mm in diameter and 380 mm deep, by the standard Proctor hammer. The specimen was loaded at a constant rate of 13.8 kPa per minute until a prescribed vertical pressure was reached. The vertical pressure was then maintained for one hour. The prescribed pressures employed in this study were 69, 138, and 207 kPa.

A total of six one-dimensional compression tests were conducted at dry unit weights of 17.53 kN/m^3 and 19.59 kN/m^3 (corresponding to 85% and 95% R.C., standard Proctor relative compaction) and at water contents of 5.8%, 9.8%, and 11.8% (-3%, +1%, and +3% of OMC). Table 2 shows the test designation, dry unit weights, and water contents of the tests.

Table 2: Large-Size One-Dimensional Compression Tests

Test	Dry Unit	Relative	As-Compacted
Designation	Weight	Compaction	Water Content
			(OMC. = 8.8 %)
	(kN/m^3)	(%)	(%)
1	17.53	85	5.8 (-3% of OMC)
2	17.53	85	5.8 (-3% of OMC)
3	17.53	85	9.8 (+1% of OMC)
4	17.53	85	11.8 (+3% of OMC)
5	19.59	95	5.8 (-3% of OMC)
6	19.59	95	11.8 (+3% of OMC)

Repeatability of the test procedure was first examined by comparing the results of Tests #1 and #2, conducted under identical initial density and moisture content. Figure 10 shows the applied pressure versus vertical strain relationships (referred to as "the compression curves") of Tests #1 and #2 (85% R.C., *w*=5.8%). The compression curves of Tests #1 and #2 are very similar with the maximum difference in vertical strain being 0.47% occurred at 207 kPa. The repeatability of the test procedure is considered satisfactory.

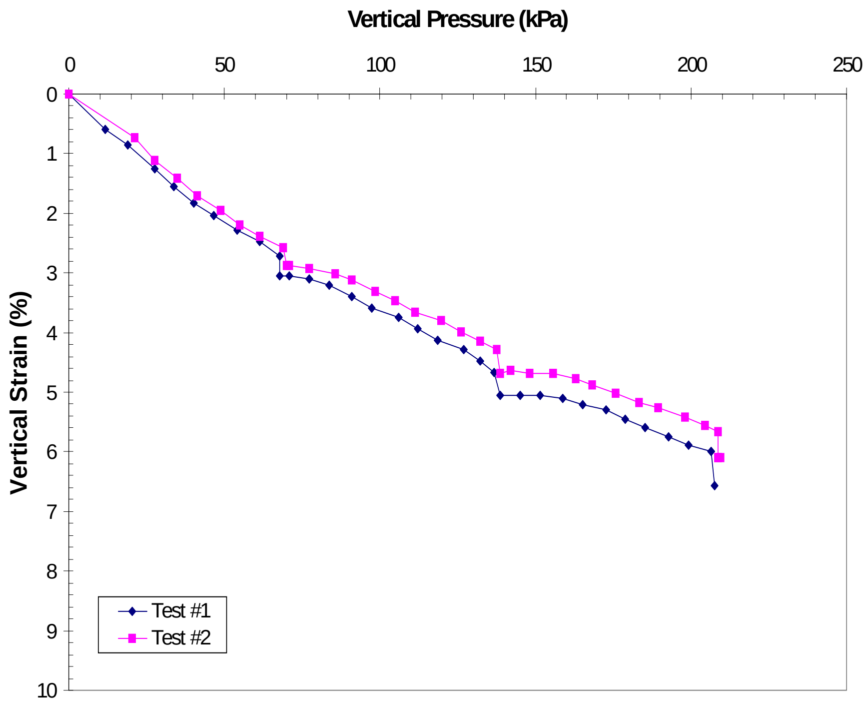

Fig. 10. Repeatability of the Large-Size One-Dimensional Compression Tests

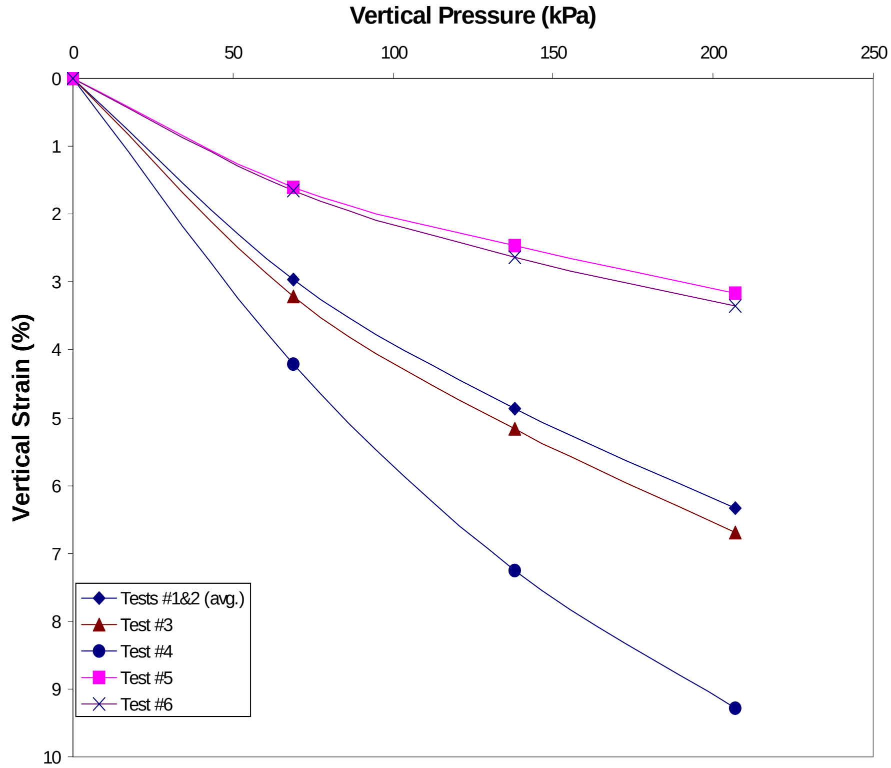

Fig. 11. Compression Curves at Different Water Contents

Figure 11 shows the compression curves of the 85% R.C. and 95% R.C. specimens at the different water contents. The vertical strains that occurred after the vertical pressure (69, 138, and 207 kPa) was applied for one hour were used to plot the compression curve. The vertical strains of the 95% R.C. specimens at water contents of –3% and +3% of OMC were, respectively, 1.60% and 1.66% at 69 kPa, 2.47% and 2.63% at 138 kPa, and 3.16% and 3.36% at 207 kPa. These results indicate that for the soil compacted to 95% relative compaction, the compression induced by wetting (water content changed from –3% to +3% of OMC) was fairly small.

The compression curves of the 85% R.C. specimens at water contents of –3%, +1%, and +3% of OMC show that the compressibility of the specimen increases much more pronouncedly with the increase in water content. The axial strains of the specimen at water contents of –3%, +1%, and +3% of OMC were, respectively, 2.96%, 3.22%, and 4.21% at 69 kPa; 4.87%, 5.17%, and 7.25% at 138 kPa; and 6.10%, 6.69%, and 9.29% at 207 kPa.

From Figure 11, the wetting-induced strain (εwet) of the 85% R.C. specimens at 69, 138, and 207 kPa was calculated. Figure 12 shows the relationships of wetting-induced strain versus vertical pressure of the 85% R.C. specimens as water content changed from -3% to +1% of OMC and -3% to +3% of OMC. The wetting-induced strain is seen to increase with the applied pressure. As the water content changed from -3% to +3% of OMC, the wetting induced strains were five to eight times of those when the water content changed from -3% to +1% of OMC.

Settlement Computations

Consider a soil is made of many sub-layers, the amount of settlement, *S*, that will occur in the soil due to an increase of water content can be computed as:

$$S = \sum (H \cdot \varepsilon_{wet}) \qquad (1)$$

in which H is the thickness of a soil layer, εwet is the average vertical strain induced by wetting of the sub-layer. The backfill in the North and South walls was each divided into several sub-layers for the settlement computations.

The calculations of settlements due to wetting were carried out with the following assumptions:
1) The deformation behavior of the wall approximates a one-dimensional compression condition. This assumption

agreed well with the observed deformation behavior that the wall deformation was predominantly vertical.

2) The backfill was compacted to about 85% standard Proctor relative compaction during construction. This was based on (a) the fairly low field density measured during wall demolition (R.C. = 81% to 86%), (b) the wall was constructed in a bitterly cold winter when the temperatures were often well below freezing, a condition usually led to low placement moisture content and density, and (c) the fill contained a large amount of large particles and made it difficult to compact to a high density.

3) The as-compacted moisture content was -3% of OMC. The wall was constructed in bitter cold weather, and the construction crew described that the fill was placed “rather dry”.

4) Between January and August, 1998, the ”average” water content of the backfill was assumed to change from -3% to +3% of OMC in the North wall and -3% to +1% of OMC in the South wall, with the latter being back-calculated from the measured behavior.

Figure 13 shows the calculated and measured settlement history of the walls from the end-of-construction (December 1996) to August 1998. The calculated settlements induced by wetting were 198 mm in the North wall and 28 mm in the South wall. The measured maximum settlements from January to August 1998 were approximately 190 to 240 mm in the North wall and 35 to 45 mm in the South wall. It is seen that the calculated settlements agree well with the measured values. This implies that the assumed wetting mechanism is probably not far from being true.

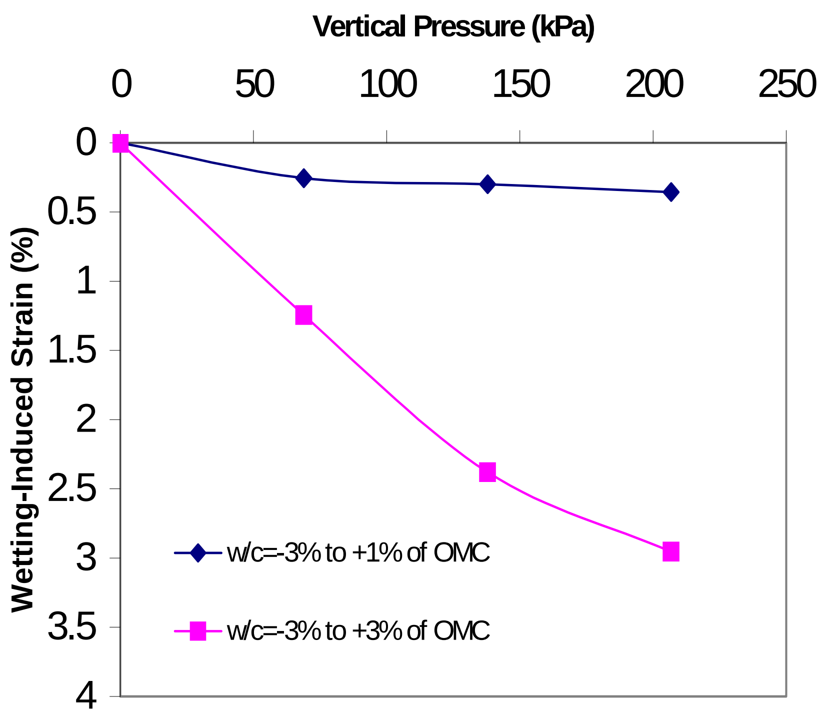

Fig. 12. Wetting-Induced Strain versus Vertical Pressure of the 85% R.C. Specimen

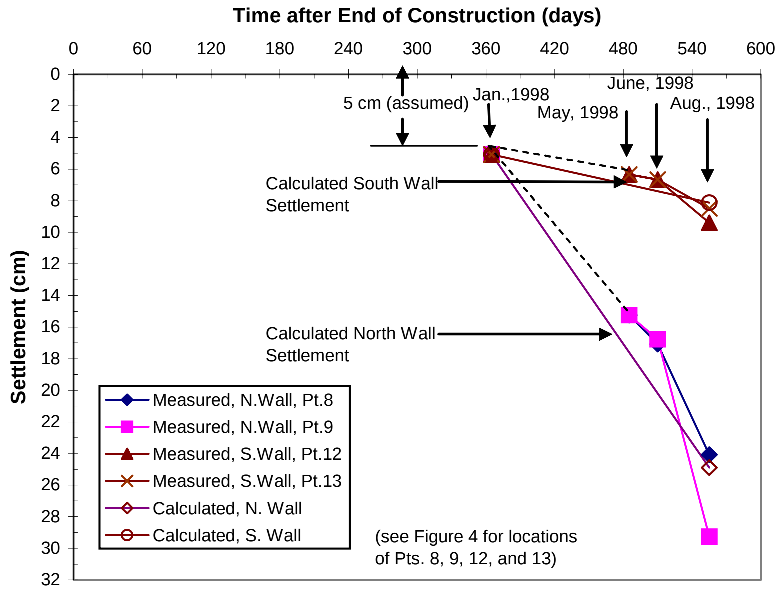

Fig. 13. Calculated and Measured Settlements from the End-of-Construction to August 1998

It is to be noted that when the wall was reconstructed in 1998, a slightly different type of backfill was employed and the backfill was compacted to 95% relative compaction. The wall has since performed satisfactorily.

SUMMARY AND CONCLUSIONS

A study was undertaken to investigate the causes of failure of an 11.3-m high geosynthetic-reinforced soil wall in Black Hawk, Colorado. The wall was constructed over stabilized bedrock and with a coarse granular backfill. The backfill was reinforced with layers of woven geotextile at a vertical spacing of 300 mm. The facing of the wall was formed by stacking large rocks aligned along the front face of the wall. The wall comprised two sections: the North wall and the South wall. The construction was completed in December 1996 in a bitter cold weather condition. In May 1998, some rocks were found dislodging from the wall face and approximately 100 mm of settlement was observed on the crest of the North wall. By August 1998, the settlement increased to about 250 mm and there was significant lateral bulging in the face of the North wall. The entire wall was demolished in October 1998, and reconstructed after demolition.

A forensic investigation program, including visual inspection, periodic settlement measurement at wall crest, and field density tests of the backfill, was carried out. The survey data showed that the maximum settlement (150 mm in 70 days) of the North wall was about five-times as large as that of the South wall (31 mm in 70 days). Field density tests, consisting of nuclear density tests and a water replacement test in a test pit, were conducted at various depths of the wall during demolition. The fill was found to be only 81% to 86% relative compaction of the standard Proctor test.

A series of large-size (290 mm diameter) one-dimensional compression tests were conducted to examine compression due to wetting of the backfill material. The tests were performed on "dry" (3% dry of OMC) and "wet" (1% and 3% wet of OMC) specimens at 85% and 95% relative compaction. The settlement due to wetting was calculated based on the one-dimensional compression test results and a "reasoned" initial dry density and moisture contents of the backfill before and after wetting. The calculated settlements agreed well with the measured data in both the South and North walls.

The findings of this study are summarized as follows:
1) Based on the field density tests at the time of wall demolition and judging from the soil type and the equipment used for compaction, the backfill is believed to be lightly compacted with relative compaction of around 85%. The relatively low placement density was a result of the use of a light-weight compactor (a "jumping jack") in 300-mm lifts and the presence of a large amount (about 30% by weight) of large particles (+19 mm) in the backfill. The placement water content was likely to be fairly dry, may be around 3% dry of the optimum water content. The fact that the wall was

constructed in bitter cold weather condition contributed to the low placement density and low placement water content.
2) The North and South walls had practically identical conditions in every respect: wall height, backfill type, backfill placement moisture and density, reinforcement and external loads; however, the two walls exhibited very different deformation behavior. The very large difference in deformation was judged to be due to the difference in moisture variation after construction. The fact that there was an abandoned pipe behind the backfill of the North wall collecting water from behind the City Hall Annex added to the dubiousness of this factor.
3) The results of the large-size double-oedometer compression tests conducted on the backfill showed that:
- The strain induced by wetting would be very small (on the order of 0.1% to 0.2%) if the soil had been compacted to 95% relative compaction.
- At 85% relative compaction, large vertical strains will occurred due to wetting. The strains induced by increasing the water contents from -3% to +3% of OMC were 1.2% at 69 kPa, 2.4% at 138 kPa, and 3.0% at 207 kPa. These strains are about five to eight times as large as those resulting from increasing the water content from –3% to +1% of OMC.

4) The settlements induced by wetting were calculated to be 198 mm in the North wall and 28 mm in the South wall. These settlements were in good agreement with the measured maximum settlements of 190 to 240 mm in the North wall and 35 to 45 mm in the South wall. The agreement implies that the excessive deformation in the North wall is very likely a result of wetting of the backfill after construction of the wall.

Like most earth structures, “failure” is typically a result of multiple causes. The findings of this study strongly suggest that the excessive deformation of the North wall was a result of two causes: (1) the placement density of the backfill was too low, and (2) there was substantial post-construction wetting of backfill in the North wall. With better compaction (say, 95% relative compaction), the excessive deformation would have been prevented. On the other hand, without the significant wetting after construction, the excessive deformation would not have occurred even with the low placement density, as evidenced by the satisfactory performance of the South wall.

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