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LESSONS LEARNED FROM THE PERFORMANCE OF A DEGRADABLE SHALE EMBANKMENT: CASE STUDY

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ABSTRACT

Degradable shales are sedimentary rocks with properties that can vary from those of solid rock to soil like materials if subjected to wetting and drying cycles. If the potential for degradation of the shale is not identified on time, rock like particles could be used as strong embankment material. After wetting and drying cycles, degradation occurs and the initial large voids formed between the generally uniform rock particles are filled with smaller fragments, resulting in significant settlements and slope instability. This paper present a case study of a 40 foot (12.2m) approach embankment having 1.5H:1V slopes that was unintentionally constructed with degradable shale. Preliminary testing showed that even though some samples were clearly degradable shale other samples exhibited durability indexes greater than the values generally accepted as durable rock. After embankment completion a settlement monitoring program was instituted for the prescribed quarantine period prior to construction of the bridge abutments. The embankment settled more than initially predicted and finally stabilized after more than 1 feet (0.305m) of internal deformation. A laboratory testing program was developed to investigate the causes of the observed degradation.

INTRODUCTION

Shales are sedimentary rocks with properties that can vary from those of solid rock to soil like materials. Soil-like shales can be generally classified as clayey, silty, or sandy shales, while rock-like shales can be classified as calcareous, siliceous, ferruginous, carbonaceous, or clay bonded shales depending on their constituents (Winterkorn and Fang 1975). When exposed to wetting and drying cycles, rock-like shales retain their integrity while soil-like shales exhibit slaking. The compressive strength of shale can vary from less than 25 psi to more than 15000 psi (172- 103421 kPa) depending on cementation, while values for Young's modulus have been reported from less than 20000 psi up to $2x10^6$ psi (137895 – $1.38x10^7$ kPa) (Winterkorn and Fang 1975).

The three main causes of slaking are: a) tensile failure due to an increase of pore air pressure generated by compressed entrapped air - capillarity suction, b) tensile or shear stresses generated by differential swelling due to osmotic swelling or surface hydratation, and c) dissolved cementing agents (Huber 1997). Previous research has focused on studying these mechanisms and how slaking is progressively generated at the microscopic detail, being influenced by several variables such as pore diameter, pore shape, and pore roughness (Botts 1986, Vallejo et al. 1993, Vallejo and Stewart-Murphy 2001).

The main problems associated with degradation of shales are excessive settlement and slope instability (Wu et al. 1993). When degradation occurs, the initial large voids formed between the generally uniform rock particles are filled with smaller fragments, resulting in settlements at the ground surface and generating dense zones that affect the hydraulic conductivity and drainage pattern of the rockfill (Huber 1997). As reported by FHWA (1980), excessive settlements varying from 1 to 3 feet (0.30-0.91m) have been measured in shale embankments used for highway applications. Continuing settlement has lead in many cases to slide failures and structure repairs.

The more severe settlement and slides are mainly related to (FHWA 1980): use of nondurable shale as rockfill (progressively slake and soften in the presence of water), mixing shale and overburden soils with harder rock (preventing adequate compaction), lack of adequate benching and drainage of underlying slopes.

SHALE DURABILITY: IDENTIFICATION, CLASSIFICATION, AND EXPECTED PERFORMANCE

The principal tests used to determine the durability of shales are the jar slake test, the slake durability test**,** the modified soundness test (also known as the modified sulfate soundness test) and the point load test. Huber (1997) reported a comprehensive summary of other tests also used to determine the durability of shales; some interesting features of these tests are the performance of consecutive cycles of wetting-drying and the analysis of changes in grain size distribution.

Underwood is considered to be the first to present a simple way of estimating in situ behavior of shales based on engineering properties (Huber 1997). He developed a table of average range of values (properties) that correlate with the expected favorable vs. unfavorable performance. He also indicated in his table what are the potential problems regarding each property. The expected performance of shale aggregates can also be estimated with the Franklin classification system (Walkinshaw and Santi 1996). The material is classified based on the durability rating, R, obtained after the slake durability test (2 cycles), the Atterberg limits, and the point load strength test had been performed. Once the shale rating is found, special charts can be used to determine lift thicknesses and compacted densities, shear strength parameters, allowable slope angles, and recommended embankment height. These values should be used carefully since they only provide rough estimates.

The most accepted criterion to classify shales is the FHWA-Strohm, Bragg, and Zeigler system. The jar slake test is used first to discard the less durable material (I_j<2). The slake durability test is then used (two cycles) to classify the shale. Visual inspection is also used to identify nondurable materials. According to FHWA (1980), shale needs to be classified as one of the following: 1) Soft nondurable- soillike, 2) Hard nondurable – soillike, 3) Hard durable – rocklike. In general, only material with a slake durability greater than 90% can be classified as Durable Rock. This system is based on actual performance data, and is related to recommended construction criteria.

COMPACTION

As pointed out by Huber (1997), compaction is probably the most important issue regarding the stability of the embankment since compaction will minimize settlements, improve the shear strength, and limit the infiltration. In general, specifications call for stringent compaction control and thinner lifts when the shale is nondurable. Usually construction specifications and recommendations from transportation state agencies in the United States suggest a conservative lift thickness of 8-12 inches (0.203-0.305m) for nondurable shales, with compaction requirements (density-moisture content) similar to the values required for regular soil. On the other hand, there is not a consensus on the lift thickness for durable shale. While some agencies do not allow the use of shales for rock embankments, other agencies allow up to 36 inch (0.91m) lifts. Many others do not have a clear specification regarding the use of shales in embankments. These discrepancies evolve from the past experiences of each agency, highlighting the importance of local experience on the design of shale embankments.

Besides compaction, preventing saturation is a key element on the design of shale embankments. The use of underdrains, rock drainage pads, horizontal drains, and vertical drains are highly recommended (FHWA 1980). Surface water should also be controlled to prevent infiltration. Some options of controlling surface water infiltration are: pavements subdrains, paved median and shoulder ditches, or an impervious cap of compacted shale or soil (Huber 1997).

CASE STUDY: GENERAL DESCRIPTION

Project Description and Location

The U.S. 220 Transportation Improvement Project consisted of the construction of a new four lane limited access highway extending from the northern end of the existing Tyrone Expressway to the western end of Mount Nittany Expressway (U.S. 322). As part of this project, structure bridges No. 111 and 112 were designed to carry the northbound and southbound lanes over the Nittany & Bald Eagle Railroad. The north and south embankments are common for both bridges. The bridges are located in Worth Township approximately 1.2 miles (1.9 km) southwest of Port Matilda, Centre County, Pennsylvania (USA).

This case study focuses on the settlement recorded on the north embankment. The approximate height of the embankment was 40 feet (12.2 m), having a 1.5H: 1V slope. It was recommended to construct a rock core of Best Available Rock (BAR) to provide a stable slope. Provisions for a construction of a rock toe and a rock blanket were also specified. Settlement platforms were installed at several locations divided into two categories: Type 1 located at the existing ground before the construction of the embankments, and Type 2 located at the top of the embankment (Fig.1).

Subsurface Conditions and Initial Concerns

As found during the geotechnical subsurface investigation for structure 111 (abutment 2), the thickness of the soils encountered by the borings varied from 35.2 to 35.4 feet (10.7-10.8 m). The soils were colluvium, alluvium, and residuum. The colluvium generally consists of a cohesive mixture of clay and silt with sand and rock fragments with cobbles and potentially boulders. The alluvium is a granular mixture of sand and gravel with variables amounts of silt and clay. The residuum is derived from the underlying bedrock and is primarily clay with shale fragments. The bedrock is a soft to medium hard shale. The groundwater level varied from 6.4 to 11.3 feet (1.9-3.4 m) below the ground surface during the investigation. Similar conditions were encountered during the geotechnical subsurface investigation of structure 112 (abutment 2).

The proposed bridge abutments and wingwalls were designed to be supported on driven piles (driven to absolute refusal). One of the major geotechnical concerns identified during the foundation design of the structures was the embankment induced settlement. A quarantine period of a minimum of 3 months was originally recommended based on the expected deformation of the original ground. For the abutment 2 (north abutment) of structure 111 a total settlement of 3.93 inches (9.98 cm), with a component of consolidation settlement of approximately 2.92 inches (7.42 cm) was estimated due to the embankment loading. Similarly a total settlement of 3.75 inches (9.52 cm), with a component of consolidation settlement of approximately 2.87 inches (7.29 cm) was estimated for abutment 2 (north abutment) of structure 112.

Construction and Initial Performance of the Embankment

Construction of the embankment was completed near the end of February 2004. Density tests could not be performed during construction due to the oversized particles. The only criterion used to verify compaction was no deformation under the weight of the construction equipment. Shale was used to construct the embankment. This shale was hard to break and initially behaved as strong rock.

A few months after completion, significant settlement was observed on the embankment. Plots of settlement vs. the square root of time were showing that more settlement could be expected. The extended quarantine period could not be released due to the observed deformation.

ANALYSIS OF THE AVAILABLE INFORMATION AND SETTLEMENT RECORDS

Slake Durability Tests from Anticipated Project Cut Areas

Slake durability tests from anticipated cut areas are shown in Table 1. The minimum, average, and maximum values were 25.6%, 75%, and 94.7% respectively. The material was primarily nondurable shale (slake durability index less than 90%). However, during construction the material was not treated as nondurable and most likely it was not placed in thin compacted layers. Table 1 shows that some of the shale had a slake durability index greater than 90%.

Table 1. Slake Durability Tests from Anticipated Cut Areas

Rock Description	**Slake Durability Index (%)- Second cycle**
Shale	94.7
Claystone	70.5
Shale	80.3
Calyey Shale	94.0
Clayey Shale	70.7
Shale	25.6
Claystone	90.5

Recorded Settlement

Figures 2 to 5 show the measured deformation at the settlement platforms during a period of approximately 676 days (26^2 days) since construction started. Two different time scales are used for the settlement platforms since Type 1 platforms were installed before the construction of the embankment. For the Type 1 platforms, the time presented on the graphs is absolute time while for the Type 2 platforms time starts at 0 on a relative scale, this starting point corresponds with approximately 256 days (16^2 days).

The graphs for Type 2 platforms show the total settlement at the top of the embankment which includes the settlement of the original ground surface. To obtain the compression of the embankment, the settlement recorded for the Type 1 platforms should be subtracted from the measurements of the Type 2 platforms. The scattering of the results presented in Figures 2 to 5 is not fully explained by trends such as smaller settlement at the toe or near the edge of the embankment. The conditions of the original soil and the embankment randomly changed in a close proximity. The data is better analyzed as average and maximum values.

Table 2 presents a summary of the deformations. As predicted during the design phase, the average settlement of the original ground surface was close to 4 inches (0.10 m). On the other hand, the maximum net settlement of the shale embankment was greater than 1 foot (0.305m), corresponding with a net compression greater than 3% of the original embankment height.

Table 2. Summary of recorded deformations (1 in = 2.54 cm)

		Struct. 111	**Struct. 112**
Settl. at original ground surface: Type 1 (in)	Min.	4.0	1.3
	Est. Avg.	4.8	3.8
	Max.	5.6	6.7
Settl. at top of embankment: Type 2 (in)	Min.	7.5	9.4
	Est. Avg.	12.0	13.0
	Max.	21.7	18.2
Embankment max. deformation (in)		16.9	14.4
Embankment max. strain		3.5%	3%

where *Embank. max. deformation* = (Max. Type 2 deformation) - (average Type 1 deformation), and *Embank. max. strain* = (Embankment max. deformation)/(embankment height).

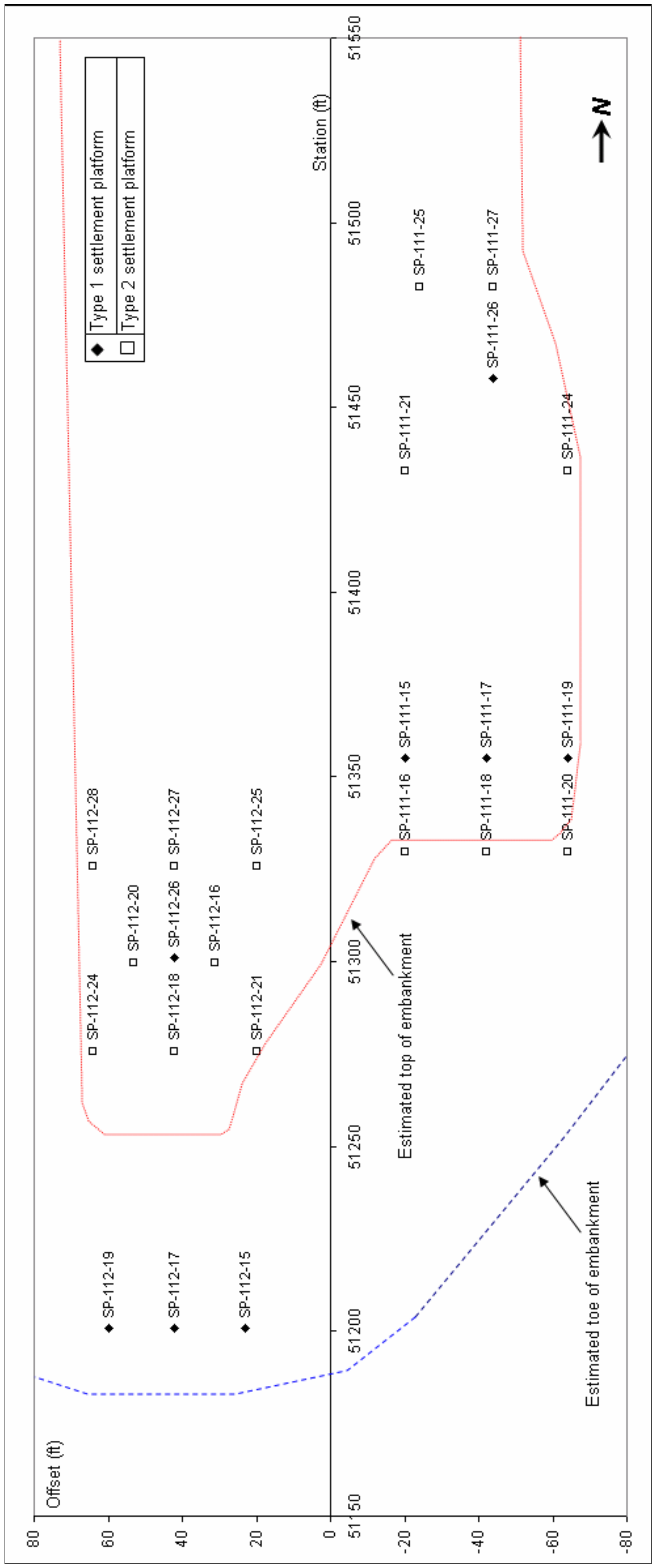

Fig. 1. Location of the settlement platforms (1ft=0.305m)

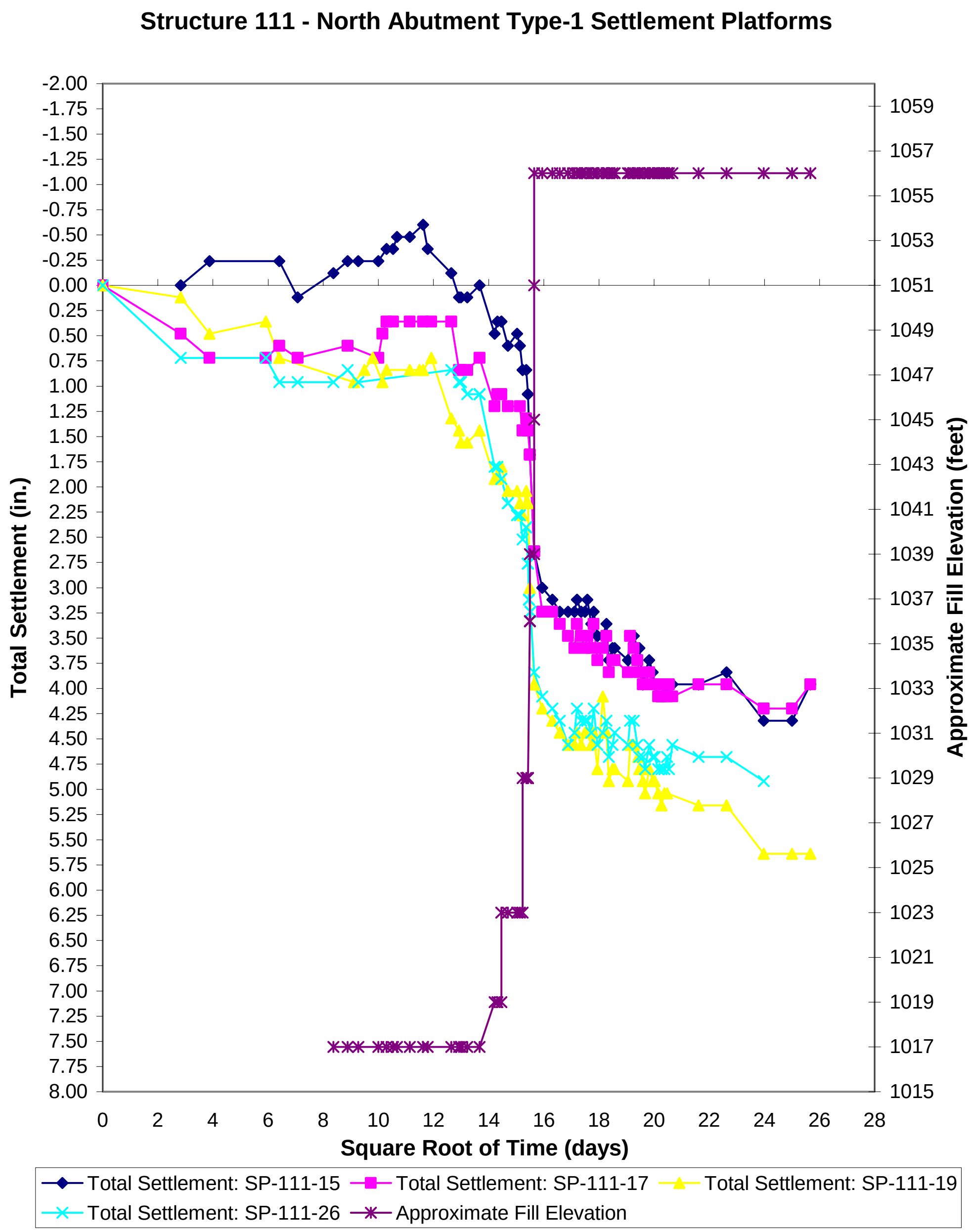

Fig. 2. Structure 111 – North Abutment Type 1
(1 inch= 2.54cm, 1 feet= 0.305m)

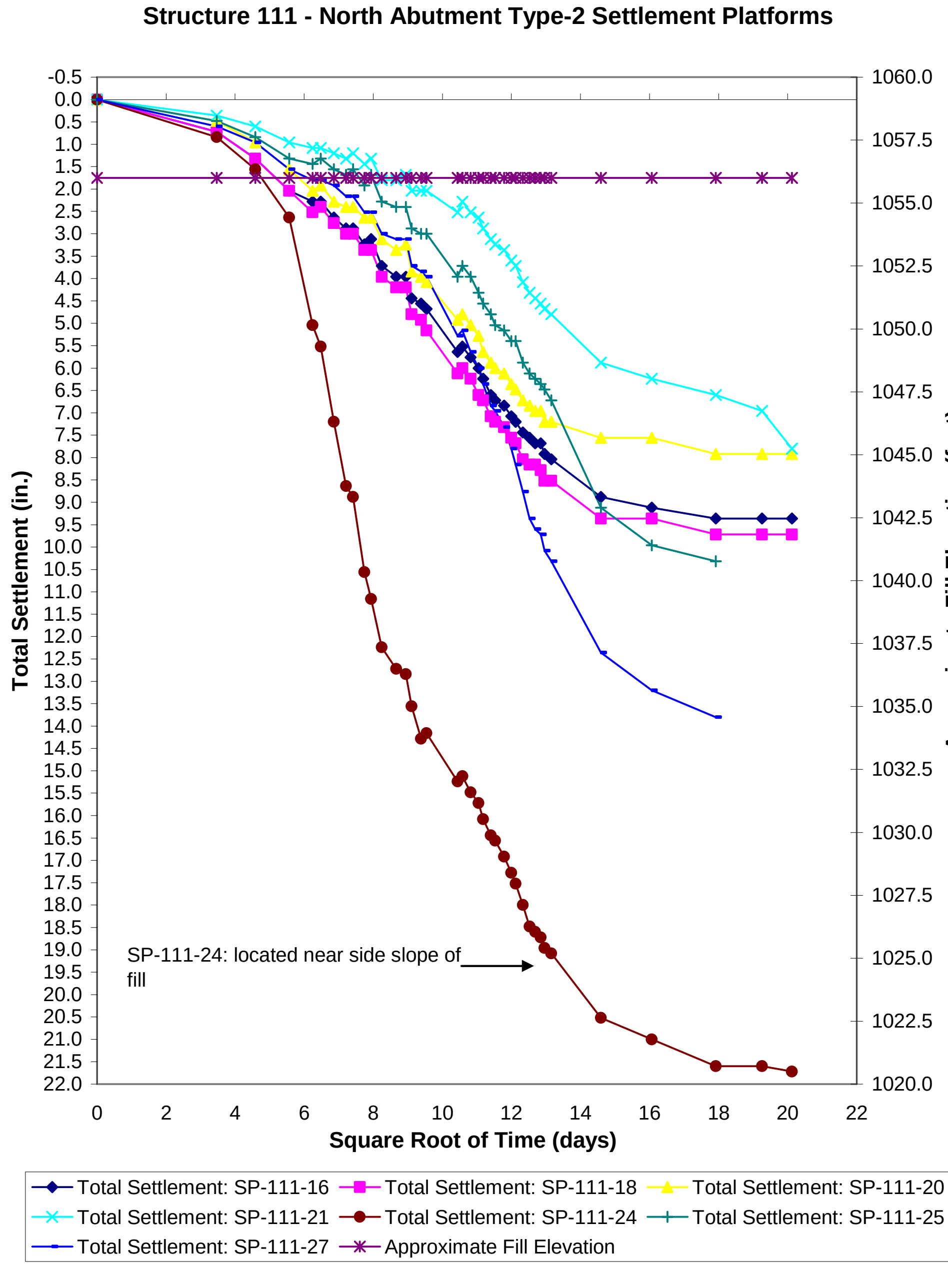

Fig.3. Structure 111 – North Abutment Type 2
(1 inch= 2.54cm, 1 feet= 0.305m)

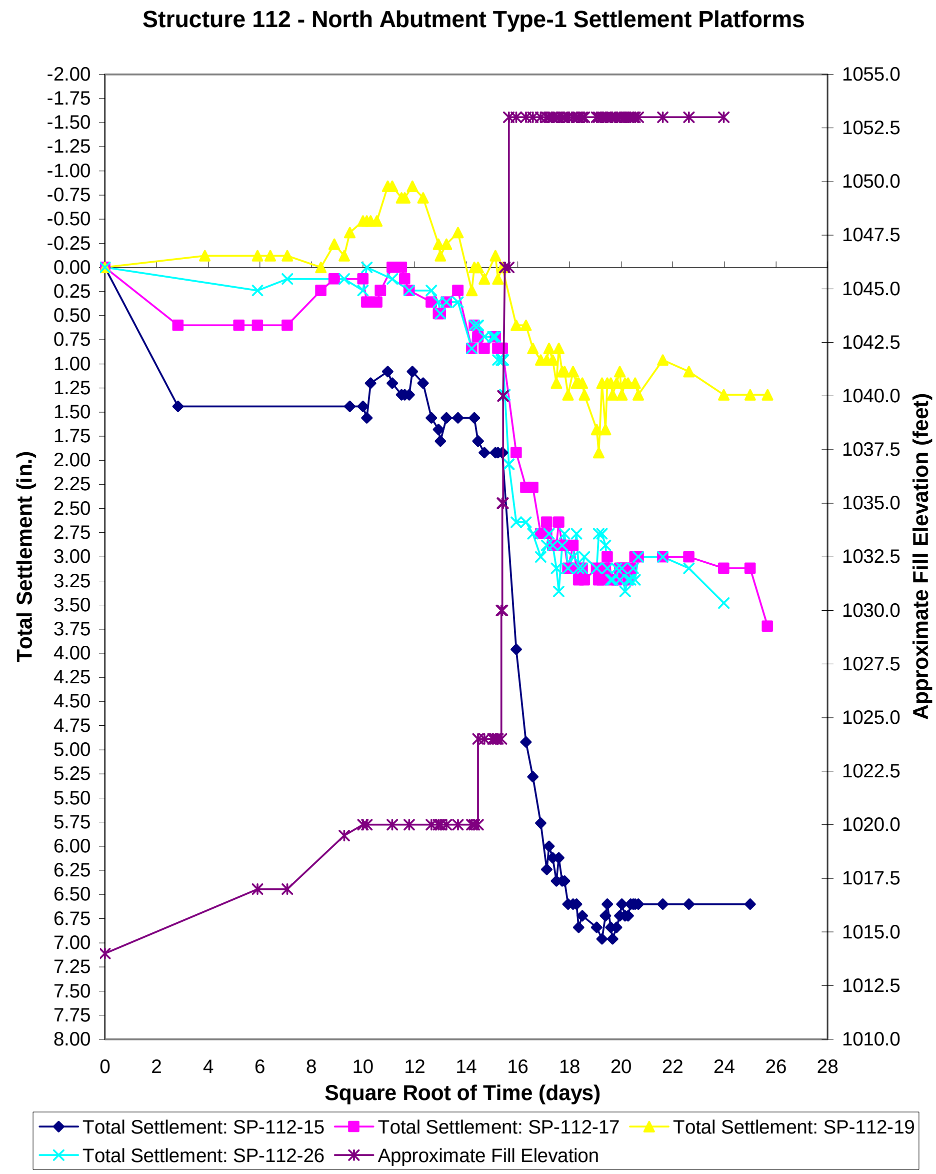

Fig. 4. Structure 112 – North Abutment Type 1
(1 inch= 2.54cm, 1 feet= 0.305m)

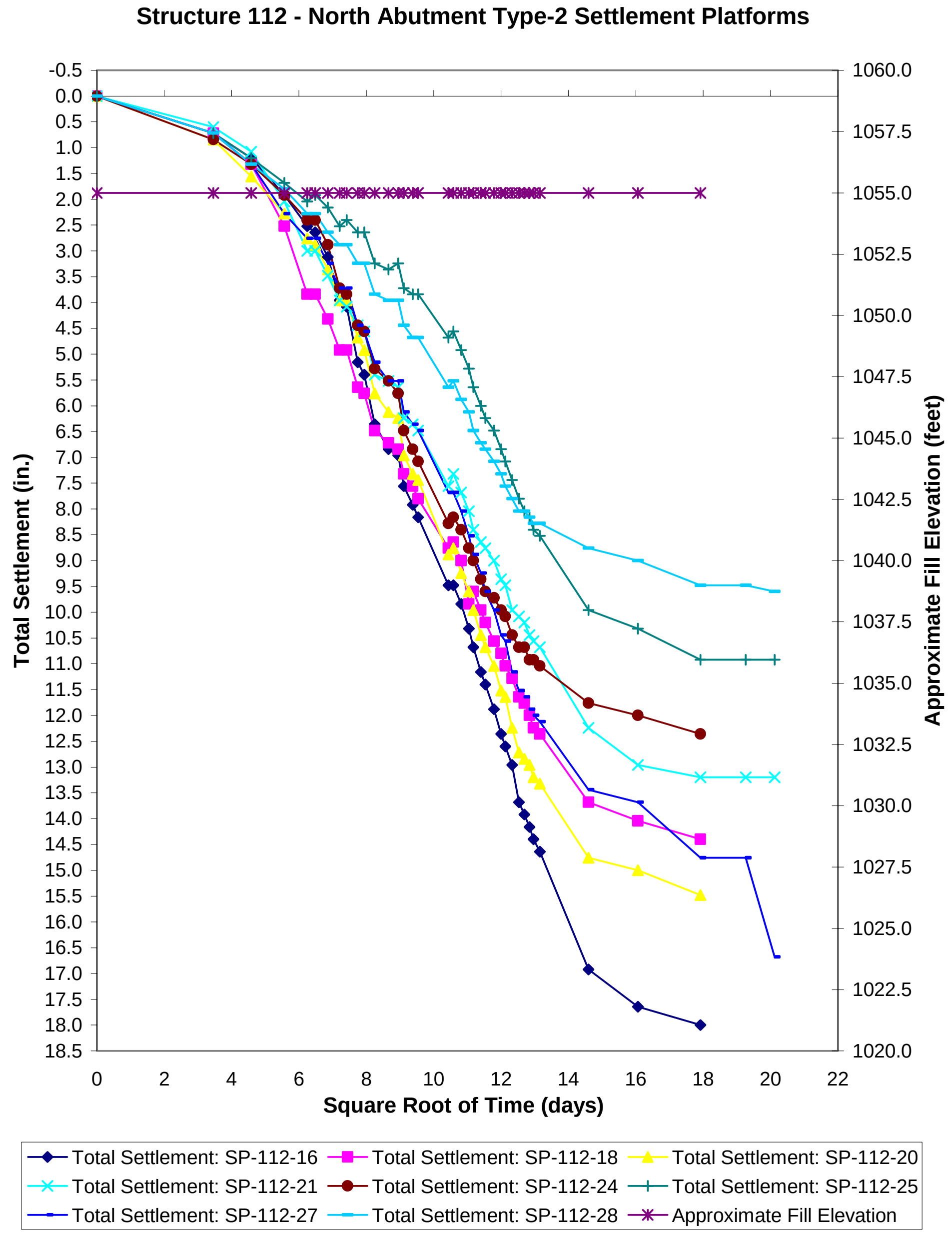

Fig.5. Structure 112 – North Abutment Type 2
(1 inch= 2.54cm, 1 feet= 0.305m)

Comparison of recorded data and published information for rock embankments

Johnson et al. (2002) documented the construction, instrumentation, and performance of a 95 feet (29 m) highway embankment constructed with good quality sandstone. The average embankment compression was approximately 2% of the embankment height. This 2% was divided into 1.5% occurring during construction and 0.5% occurring in a 4 months period after construction. The total embankment compression and the stabilization time reported by Johnson et al. (2002) are considerably smaller than those recorded for the degradable shale embankment (almost 1 year = 19^2 days). Oldecop and Alonso (2007) presented a summary of settlements recorded on rockfill dams built worldwide during the twentieth century. The maximum settlement reported on their compilation was close to 1.6% the height of the rockfill dam. This upper limit is almost half the settlement recorded on this case study.

LABORATORY PROGRAM: SHALE DEGRADATION

A laboratory program was developed to understand the observed shale degradation on the embankment. The laboratory testing was divided into two main areas: material characterization and predicted performance. The material characterization consisted on testing for durability properties. The tested samples were divided into two main groups: weathered and unweathered samples. Weathered samples were collected from the embankment and from a representative quarry closely located to where the shale of the embankment was originally extracted. Unweathered samples were also collected from the same quarry. Performance tests consisted on edometric compression (one dimensional confined compression) under constant load. The performance tests were also conducted on weathered and unweathered samples.

Wet-dry Durability and Slake Durability Tests

On the wet-dry durability tests both weathered and unweathered samples had a total loss of 100%. For the unweathered sample, 94.8% of the loss came from splitting and cracking resulting in fragments bigger than the No. 4 sieve (4.75 mm). Only 5.2% of the loss in the unweathered sample corresponds with particles smaller than the No. 4 sieve. For the weathered sample, 44.5% of the loss corresponded with particles bigger than the No.4 sieve, and 55.5% with particles smaller. The weathered sample was more affected by wetting and drying than the unweathered sample.

The slake durability tests showed a small difference for the weathered and the unweathered samples. The slake durability index for the weathered and the unweathered samples was 60.9% and 64.9% respectively. Both samples exhibited values below the standard for durable rock (90%). The results for the unweathered sample were slightly lower than the average value reported during the design phase of the highway project (75%).

Point Load Tests

The point load test (point load index) provides an estimate of the strength of the material and it can be correlated with unconfined compression values. The influence of the number of wetting/drying cycles on the strength of the shale was studied using this test. The obtained results are presented on Table 3.

Table 3. Results of the point load tests (1psi= 6.89kPa)

Description	**No. of tests**	**Estimated Unconfined Compression (psi)**		
		Min.	**Ave.**	**Max.**
Unweath.:				
No wetting	10	865	3570	6958
1 cycle w/d	10	0	180	546
2 cycles w/d	10	0	0	0
Sat. no drying	10	0	82	371
Weathered:				
No wetting	10	560	2639	5638
1 cycle w/d	10	0	0	0
2 cycles w/d	10	0	0	0
Sat. no drying	10	0	0	0

Unweathered samples were tested dry and after 1 and 2 cycles of wetting/drying (different samples for each test). Some of the samples subjected to one cycle had no strength and the average was significantly lower than the average of the dry samples. The samples subjected to two cycles disintegrated, crumbled, or did not have any remaining strength and no values could be recorded. Unweathered samples were also tested after the first wetting but prior to drying, they were termed as saturated. The results showed that this was a slightly more critical condition than a complete first wetting/drying cycle.

Weathered samples were subjected to the same conditions mentioned before. The measured strength of the dry samples was significantly smaller than the strength of the dry unweathered samples. No strength was exhibited by samples subjected to one or two wetting/drying cycles, or subjected to wetting and tested saturated.

Edometric Compression

Edometric compression (confined one-dimensional compression) tests on weather and unweathered samples were performed to study the deformation induced by particle degradation. The effects of wetting/drying cycles were investigated under simulated field conditions. A standard CBR mold was used on the test. The mold was a rigid metal cylinder with an inside diameter of 6 inches (15.2 cm) and a height of 7 inches (17.8 cm). During each test, a rigid metal disk with a diameter slightly smaller than the diameter of the cylinder was placed at the top of the tested sample. The load was then applied on the metal plate and kept constant for the remaining part of the test. The mold was inside a metal container, which allowed saturating the sample. The water from the container could be removed allowing the tested sample to drain (the mold had some small holes that drain water out from the sample). Each sample was subjected to several wetting/drying cycles while keeping constant the applied vertical load. The applied pressure was 2400 psf (114.9 kPa) which roughly corresponds with the vertical stress at the middle height of a 40 feet (12.2 m) embankment. The samples had an initial uniform grain size distribution with an average size of 0.87 inches (2.21 cm). The results are shown on Fig.6.

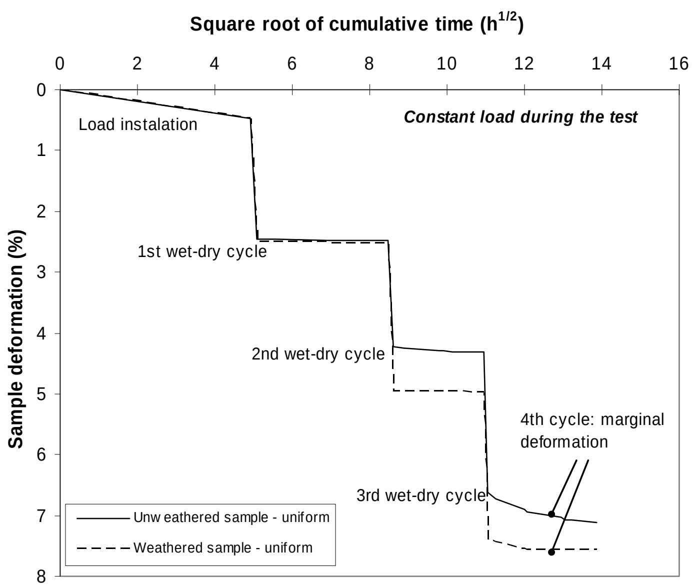

Fig. 6. Deformation vs. time

Deformation is presented as a percentage of the initial height of the sample. Square root of time is used as it is commonly employed for consolidation plots. As a result of the load installation, a small settlement was recorded on both samples (less than 0.5%). After the samples stabilized, the first wetting/drying cycle took place. The samples were kept saturated until they stabilized, after this the water was allowed to drain out and the samples were again allowed to stabilize. Almost all the deformation took place rapidly during the wet part of the cycle. The same procedure was repeated for the second, third, and fourth cycles. The fourth cycle produced marginal settlement. The tests were stopped since the samples reached a stable state independent of wetting/drying cycles. The behavior of the samples was identical up to the second cycle, when the weathered sample exhibited more deformation than the unweathered sample. At the end of the fourth cycle the unweathered sample had a total settlement of 7.1% while the weathered sample had a total settlement of 7.6%. In general for the two samples, the deformation occurred during the first hours of the wet part of each cycle and was insignificant during the drying stages.

Fig.7 shows the results of the sieve analyses performed at the end of the tests. The samples became a well graded mixture of particle sizes. The weathered sample was subjected to more degradation than the unweathered sample as reflected on the coefficients of uniformity.

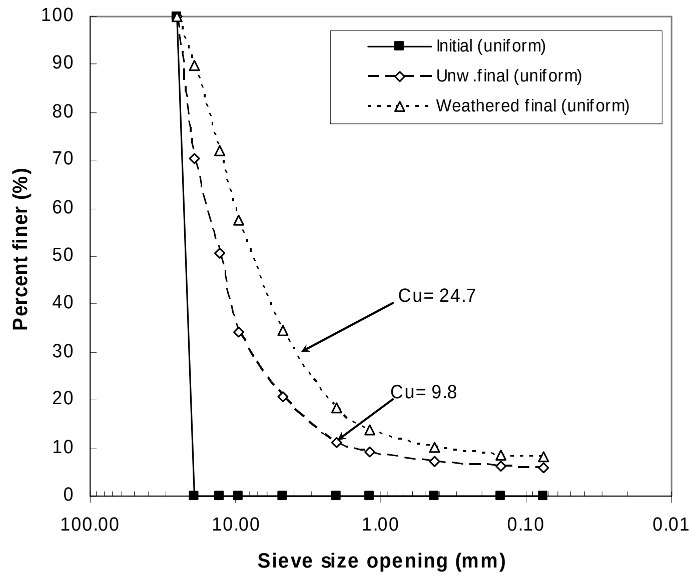

Fig.7. Grain size distribution of the initially uniform samples

Comparison of Field Records vs. Laboratory Test Results

The final settlements on the laboratory samples are approximately two times the normalized settlements recorded on the real embankment. Some of the reasons for this difference are: the shale embankment was not constructed with a controlled uniform material, the embankment is not completely saturated at the same time and drainage allows smaller periods of saturation, due to scale effects more deformation is associated with the degradation of a single particle in the laboratory test than in the real embankment, the embankment could have had better compaction compared to the laboratory sample.

The trends identified on the laboratory tests agree with the field records. The durability tests and the point load tests showed that the shale could not stand more than a couple of wetting/drying cycles before loosing all the strength. This was also observed on the compression tests where only 4 cycles were necessary to achieve a stable state. Deformation of the samples and the embankment took place during the first hours after water was added to the shale (rain events in the embankment). Complete saturation of the sample accelerated the degradation process compared to the embankment where not all the material was saturated, resulting in a longer stabilization period. After the shale degraded both the embankment and the laboratory samples stabilized with no extra deformation due to wetting/drying cycles.

CONCLUSIONS

Shale degradation was the main cause of the embankment deformation. As a result of infiltrating water, the shale fragments broke down to smaller sizes and filled the voids between the remaining big particles causing considerable settlements.

Although the strength of the shale particles rapidly reduced after a few wetting/drying cycles, the stabilization period extended for over a year since not all the embankment was saturated at the same time and drainage structures helped to minimize the exposed time. After degradation, the material developed a well graded mixture of sizes preventing further fragmentation and settlement.

The lessons learned from this experience are not new in relation to the FHWA recommendations developed many years ago. Good compaction, carefully controlling lift thickness, avoid mixing durable and nondurable material, and inducing degradation at the time of compaction are excellent practices. Even though hard rock may be seen as durable it can behave as a soillike material. It is important that this happen before continuing with the next lift.

The construction of the bridges started after the embankment stabilized as concluded by the monitoring program. Currently no problems have been reported regarding the stability and deformation of the embankment and the structures.

This case study can be used to highlight the importance of establishing a monitoring program and a quarantine period. The cost of a monitoring program is insignificant compared to the cost of the necessary repairs if the bridge structure is already in place when the deformation occurs.

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