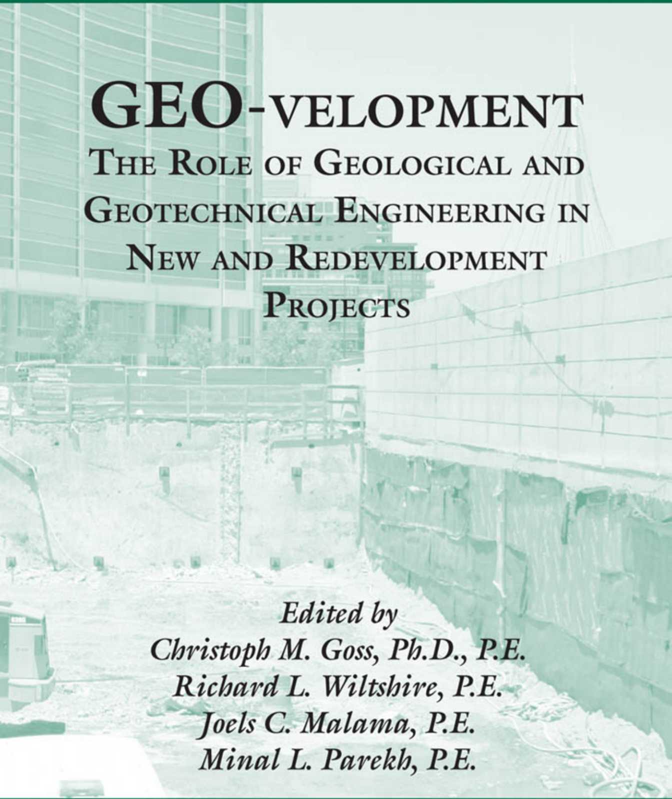
GEOTECHNICAL PRACTICE PUBLICATION NO. 5



GEO-VELOPMENT

THE ROLE OF GEOLOGICAL AND GEOTECHNICAL ENGINEERING IN NEW AND REDEVELOPMENT PROJECTS

Edited by

Christoph M. Goss, Ph.D., P.E.

Richard L. Wiltshire, P.E.

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GEO INSTITUTE

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PROCEEDINGS OF THE 2008 BIENNIAL GEOTECHNICAL SEMINAR

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Colorado Association of Geotechnical Engineers

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Preface

Across all lands and time, engineers have developed their environment to obtain housing, water, transportation, mineral resources, and protection. As structures wore out or societal needs changed, the environment would be redeveloped. Inevitably, the development would start from the ground up, in the realm of the geotechnical engineer, who would make the foundation of the dam, building, or canal strong and clean. Geotechnical engineering in Colorado began around the 8th Century A.D. at Mesa Verde, when the first reservoir was constructed by the Ancestral Puebloans. In the 19th Century, geotechnical engineering was focused on mining and irrigation. As cities grew in the 20th Century, the development focus shifted to large buildings, roads, airports, bridges, and dams. Today, Colorado's geo-community continues its work to develop new areas and re-develop the old. We hope that this collection of seminar papers, presenting Colorado's geotechnical practice and experience related development and re-development, will be of value to others worldwide.

Since 1984, the ASCE Colorado Section's Geotechnical Group, in collaboration with the Rocky Mountain Section of the Association of Environmental and Engineering Geologists and the Colorado Association of Geotechnical Engineers, has organized a biennial series of geotechnical seminars on a wide variety of themes that have been attended by as many as 270 civil/geotechnical engineers, geologists, and other geo-professionals. The geotechnical seminars have been held at area universities or hotels and have offered the opportunity for sharing ideas and experiences among Colorado's diverse geo-disciplines. Since 2004, ASCE's Geo-Institute has published the papers of these seminars in Geotechnical Practice Publications, allowing the experiences to be shared with a worldwide audience.

The GEO-velopment Steering Committee convened in August 2007 and held monthly meetings to plan for the 2008 Biennial Geotechnical Seminar. The Steering Committee members included Joels Malama (Conference Chair), Dr. Christoph Goss, Mark Brooks, Dr. Bill McCarron, Minal Parekh, Becky Roland, Mark Vessely, Leslie Jansen, Steve Bryant, Jere Strickland, Keith Seaton, Melanie Longi, Joe Kerrigan, Chris Wienecke, and Richard Wiltshire.

Christoph Goss, Richard Wiltshire, Joels Malama, and Minal Parekh

Acknowledgements

The GEO-velopment Steering Committee wishes to take this opportunity to thank all of the authors and reviewers of our papers, which are herein presented as Geotechnical Practice Publication No. 5. The authors have spent many hours in preparing and finalizing their papers, which will be presented at the 2008 Biennial Geotechnical Seminar on November 7, 2008. These papers have been reviewed by a volunteer group of Denver area geo-professionals who put in their valuable time and helped make these papers even better. The Geo-Institute's Committee on Technical Publications completed its review of our GEO-velopment papers in a very timely manner and their adherence to our aggressive publication schedule is greatly appreciated. We would also like to acknowledge the assistance of Donna Dickert of ASCE's Book Production Department for putting this publication together.

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Downtown Redevelopment with Complex Site Constraints

James W. Niehoff[1], M. ASCE, P.E.

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ABSTRACT: When a developer selected a site in Denver for a new office building, there were numerous challenges to be faced. Due to zoning and height restrictions, the 8 story building required 4 levels of below grade parking, extending over 6 meters (20 feet) below the water table in coarse sand. On the northwest and southwest sides of the site, an existing basement wall was present at the property line, supporting critical utilities in the backfill zone. On the northeast side of the site, a portion of a newly completed building was supported on shallow footing foundations located about 6 meters (20 feet) above final proposed grade at the property line. As the new building was to extend to the property line, there was no room for a temporary excavation bracing system. All of these factors dictated a creative approach to site preparation and building construction. The ultimate approach to construction involved the installation of a secant wall around the entire periphery of the site, extending into bedrock to cut off the majority of the groundwater, provide lateral and vertical support for the adjoining building, protect adjoining utilities and serve as the final basement wall for the office building.

INTRODUCTION

During the past few years, the Lower Downtown section of Denver, Colorado has experienced a period of significant development and redevelopment. Once a predominantly commercial/warehouse district, the area now includes Coors Field (a major league baseball park), as well as numerous shops, restaurants, condominiums, and offices.

While older buildings dating to the 1800's largely have been refurbished and converted to new use, a few more recently constructed buildings with less historical significance have been razed to make way for new development. One such building was the former United States Post Office Processing Center located within the city block bordered by 15th , Wynkoop, 16th and Wewatta Streets at the western extreme of the Lower Downtown District. This large, utilitarian structure was constructed in the middle part of the 20th Century and included one to two basement levels and 4 levels above grade (Figure 1). Following acquisition of the property by a national developer, the building was demolished to street level and the basements were filled with the resulting demolition debris.

Figure 1: Post Office Processing Center

Due to the large size of the site, it was subdivided into two parcels. The northeastern half was developed first, with an 8 story office building, currently housing the Denver Office of the Environmental Protection Agency (EPA). This entire parcel incorporated 2 levels of below-grade parking. The overlying 8-story office has a slightly smaller footprint, leaving an at grade access drive along the southwestern property line. This office building is primarily supported by drilled pier foundations extending into the Denver Formation bedrock. However, the exterior wall of the parking deck running along the shared southwestern property line was supported by a strip footing foundation bearing at a depth of about 7 meters (22 feet) below adjacent street grades.

Following substantial completion of the EPA building, plans were developed for construction of a second office building on the remaining parcel of land. This was also to be an 8-story office building to house a prominent local insurance company. Due to local building height restrictions and tenant requirements, plans called for the building to incorporate 4 levels of below-grade parking. This requirement would ultimately create significant complications for the project and necessitate innovative approaches to both design and construction.

PROJECT CHALLENGES

The 1515 Wynkoop Project, as it was ultimately designated, faced a number of significant challenges including high groundwater, the presence of existing structural elements, and the need to protect adjacent sensitive utilities and structures during and after construction.

The subsurface profile in the project site area consists of Quaternary-aged alluvial sands and gravels extending to depths of about 13 to 14 meters (40 to 45 feet) below grade atop the Denver Formation (Niehoff, 2006). The bedrock consists of weakly to moderately cemented sandstone/claystone which extends to a depth of over 100 meters (300 feet). The Denver Formation materials are relatively impervious, and as a result, support aerially extensive perched groundwater. The system is recharged locally by the infiltration of surface water through the pervious surficial sands and by Cherry Creek, which is located two blocks to the south of the site. While fluctuations in the water table occur seasonally, groundwater is typically found about 8 to 9 meters (24 to 27 feet) below the level of the surrounding streets. Given that the parking levels would need to extend to a depth of 13 to 15 meters (40 to 45 feet) below street level, groundwater would need to be considered both during and after building construction.

The post office building that had previously occupied the site had been supported by a system of footing foundations bearing in dense sand strata at the approximate level of the groundwater system. The foundations around the periphery of the building extended to the property line and supported a thick reinforced concrete basement wall. The foundations and basement walls had been left in-place during building demolition and the open basement areas had been filled with construction debris. New construction would require excavations to depths of over 6 meters (20 feet) below the bearing elevations of the existing foundations and the base of this wall. Due to the presence of critical utilities, including fiber optic cables, water and gas lines within 1 to 2 meters (3 to 5 feet) of the existing basement walls, it would not be possible to install a new excavation bracing system at the property line to allow for the removal of these basement walls.

The new structure was also designed to extend to the site's northeastern property line. This would require a cut extending to a depth of over 6 meters (20 feet) below the footing foundations supporting the exterior wall of the EPA building parking structure. A section through the site presenting this geometry is presented on Figure 2, below.

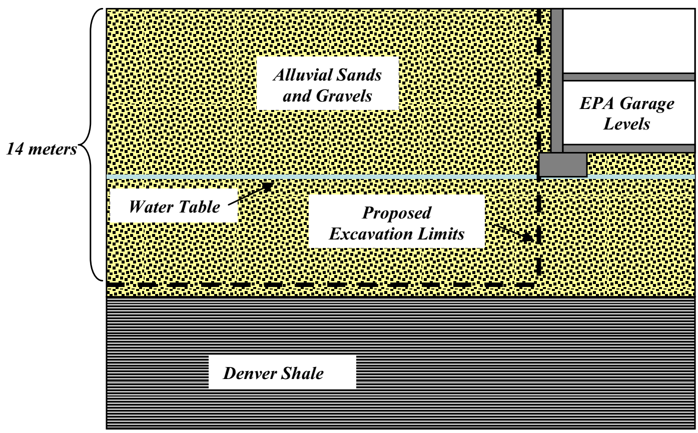

Figure 2: Section though the site presenting project geometry

Due to the unique challenges posed by this structure, design was undertaken as a team effort including the owner/developer, architect, geotechnical, structural and civil engineers as well as the general and specialty contractors.

DESIGN PROCESS

The challenges described previously were considered both individually and together during the design process. In general, the design needed to consider temporary and permanent groundwater control, earth retention, and support of critical adjoining structures and utilities, while providing for maximum interior space.

Due to the presence of critical utilities within a short distance of the property line, the design team made the decision to incorporate the existing basement walls associated with the demolished postal facility into the new structure. To provide for adequate lateral support for these walls until they could be framed into the new structure, temporary earth anchors needed to be drilled through the wall and into the surrounding soils. These walls also required vertical support as the excavation proceeded downward below the wall base. This issue was considered as part of the shoring system for the lower portion of the excavation.

The presence of shallow groundwater was considered a critical issue in the design and construction of the deeper portion of the basement planned for the building. Initially, consideration was given to the installation of a temporary dewatering system to allow for construction to proceed in the dry, followed by the installation of a permanent system of drains, sumps and pumps to maintain water below basement levels. This option was rejected by the design and construction team for several reasons. First, the quantity of water expected to be pumped from the coarse sands and gravels beneath the site was expected to exceed 4 cubic meters (1000 gallons) per minute. This quantity of water could not be discharged into storm drains, but would need to be filtered and piped to Cherry Creek several blocks away. The high pumping rate would result in a general lowering of the groundwater elevation within a significant radius of the site, possibly causing ground subsidence beneath streets and nearby buildings. Finally, a permanent dewatering system would be expensive to operate and maintain and would require extensive backup pumps and electrical supply systems to keep the basement levels from flooding in the event of electrical or mechanical system failure.

In lieu of removing groundwater, the design team concluded that a cut off-wall extending to bedrock would be preferable both for construction and subsequent operation of the building. A number of options were considered for this purpose including driven sheet piling, concrete diaphragm walls installed through a slurry trench, a program of jet grouting, and even a ground freezing approach. The driven sheet pile wall option was rejected due to the potential for damaging vibrations. Diaphragm walls were not selected due to high cost, and lack of availability of local contractors and equipment. The jet grouting option was considered to be viable for treatment of the sandy overburden, but the specialty contractor was concerned that a seal could not be established between the treated sand and the underlying bedrock. Ground freezing was rejected due to the risk of the heave of treated soils beneath the EPA Building. Ultimately, a secant wall system was selected for use in this application. A secant wall is constructed by installing a series of overlapping drilled pier foundations. To install the wall, a template is placed on the ground surface (Figure 3), and even numbered piers are drilled through the overburden soil and into

Figure 3: Secant Wall Template

bedrock, then filled with a weak concrete mix. Once these "non-structural" piers are installed, odd numbered pier locations are drilled, each of which penetrates about 12 to 15 centimeters (6 inches) into the non-structural piers on either side. These piers are filled with high strength concrete as well as a reinforcing cage to provide structural support for the wall. This type of wall system presented many advantages. First, it could be installed from either the existing ground surface or from the basement level of the post office with only minor disturbance to adjoining soils and structures. Second, the drilling equipment would be powerful enough to penetrate well into the low permeability Denver Formation and form an effective groundwater cutoff. Third, the inner surface of the wall could be finished with shotcrete and fitted with haunches to allow for its use as the permanent basement wall. Finally, it could be used to structurally support the existing footing foundations for the exterior EPA wall as well as the existing post office basement walls that were to remain.

The secant wall incorporated 69-centimeter (27-inch) diameter piers, spaced 51 centimeters (20 inches) center to center to create an 18 centimeter (7 inch) overlap. One to three rows of tie-back anchors were employed to provide supplemental lateral restraint for the secant wall during the construction process. These were in addition to tie-backs used for temporary support of the pre-existing post office basement walls. All tie-backs were to be cut after basement levels were framed in.

CONSTRUCTION AND MONITORING

Construction of the secant wall was initiated along the Wynkoop Street side of the site, where no existing basement walls were to remain and where the wall was to extend from the surface to the lowest portion of the basement (Figure 4). This construction of this section of the wall was used to refine the installation procedure and to confirm that the wall could be drilled without excessive vibration or disturbance to adjoining soils, streets and buildings. For each pier, a 69-centimeter (27-inch) diameter casing with cutting teeth was first advanced into the ground to a depth of about 14 meters (45 feet), penetrating a short distance into the Denver Formation Shale. Then the casing was cleaned out using an auger, and the pier was advanced a minimum depth of 1-1/2 meters (5 feet) into relatively unweathered shale to create a positive groundwater barrier and to provide lateral resistance to earth pressures. Following the drilling process, reinforcing steel was placed in the structural piers and concrete was introduced by the free fall method to form the pier.

Figure 4: Secant wall installation

Concurrent with initial pier installation, a construction monitoring program was designed for the EPA Building. This initially included a condition survey of the parking deck portion of the structure, that included documentation and instrumentation of thin cracks in the wall on the shared property line. This was followed by the installation of accurate survey points on the shared wall, a laser beam target system, and a series of geophones to detect the level of vibrations resulting from excavation, demolition of remaining footing foundations and secant wall installation. Surveys were conducted prior to excavation to establish baseline elevations and horizontal positions and to detect variations in measurements due to temperature fluctuations. During construction, surveys of the horizontal and vertical position of targets were conducted once or twice a week, depending upon the location and type of on-going construction activity. Additional interior conditions surveys of the EPA Building parking structure were conducted on a monthly basis. Vibration monitoring was continuous for the duration of the excavation and secant wall construction.

Installation of the secant wall generally proceeded according to schedule, except where unanticipated subsurface obstructions including steel H-Piles were encountered, causing minor delays. Wall installation was found to only generate vibrations with velocities on the order of 0.01 meters per second (0.4 inches per second) within 1.5 meters (5 feet) of the drilling equipment. This was considered well within a tolerable range for adjoining buildings and utilities. Additionally, surveys indicated that lateral movements of the existing EPA building foundations and exterior walls were limited to 0.6 centimeters (¼ inch) or less.

Upon excavation to the lowest basement level, the wall itself was found to be structurally continuous, and relatively impervious, effectively cutting off the majority of groundwater to the interior of the excavation (Figure 5).

Figure 5: View of postal facility basement and secant walls 3 meters (10 feet) below groundwater elevation (the EPA Building is to the right).

As part of wall construction, interceptor drains consisting of slotted PVC pipe, 2.54 cm (1 inch) in diameter, were installed between individual piers prior to applying the shotcrete facing to collect groundwater that might pass through the walls at overlap points. These were connected to a subslab drainage system and a series of sumps that had been designed to collect and remove 0.75 cubic meters (200 gallons) of water per minute.

The estimated total quantity of water entering the site following excavation to the lowest basement level was measured to be less than 0.4 cubic meters (100 gallons) per minute. Much of this was judged to be upward seepage through isolated pervious seams in the Denver Shale, rather than through the wall.

Overall, the wall system met the project objectives and allowed for the construction of below grade building levels to proceed in a relatively dry environment without adverse impacts to surrounding infrastructure elements and buildings. The use of a permanent cut-off wall will significantly reduce building energy costs related to groundwater removal and minimize impacts to adjoining development.

CONCLUSIONS

Construction in a downtown setting can pose significant challenges, particularly when height restrictions necessitate multiple basement levels, and adjoining buildings and utilities preclude the use of conventional excavation retention systems. For such projects, input from all stakeholders is essential in the consideration of workable and cost-effective design and construction alternatives. For this project, a secant wall proved to be an ideal solution to the challenges of high groundwater, earth retention, and support of adjoining structures.

ACKNOWLEDGMENTS

The author would like to acknowledge the project team for 1515 Wynkoop, which included Hines (owner and developer), Gensler (production architects), Jirsa Hedrick & Associates (structural engineers), Professional Service Industries (geotechnical engineer), J. R. Harris Company (secant wall design), Martin & Martin (civil engineer), Holder Construction (general contractor) and Malcolm Drilling (secant wall contractor).

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Development Constraints in Areas with Sugar Beet Spoils

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ABSTRACT: Development in areas with sugar beet spoils may result in unexpected complications and project constraints. Mitigation of beet spoils through design, removal and replacement, or avoidance can be costly. Identification of beet spoil materials prior to construction can allow mitigation methods to be incorporated into the construction sequence and lessen unforeseen construction costs.

The agricultural boom of the sugar beet industry of Colorado in the early part of the 20th century carried with it a significant byproduct: sugar beet spoils from processing. Flushed from the sugar beet processing plants, the byproducts were distributed over vast open areas. Beet spoil materials were commonly found covering large expanses of ground, or having leached into the soils, in areas up to 4 miles away from the processing plant.

New developments have pushed into areas with beet spoils. Structures and roadways constructed over beet spoils have experienced construction difficulties and failures. New developments in these areas are almost certain to have problems if the beet spoils are not identified and mitigated prior to development. Beet spoils have peculiar engineering properties, are subject to piping, and can cause poor pavement performance including sinkholes in roadways, subgrade consolidation, and loss of adequate support. Beet spoils, often very high in soluble sulfates, react negatively with concrete and chemically treated subgrades, causing deterioration and other failures.

Geotechnical investigations and research of historical site use are critical in identifying sites with potential sugar beet spoils. Identification of beet spoils prior to construction can result in significant savings during development and in the long term.

INTRODUCTION

History

The semi-arid climate of Colorado seemed like a lost cause for agriculture in the early to mid 1800's. As settlers from the gold rush began to pour into the area in the late 1850's, it became imperative to find fruits and vegetables that could grow in such a dry climate. One crop that did seem to thrive was the sugar beet.

The existence of sugar beets in Colorado agriculture dates back to the late 1860's with the first processing plant being constructed in Grand Junction in 1899. Sugar beet production spread to the foothills and eastern plains by the early 1900's and by the 1930's, Colorado became one of the largest producers of sugar beets in the United States. The Western Sugar Beet Company was the largest of the sugar beet producers in Colorado. The production of beet sugar spanned the Front Range corridor of Colorado, reaching through Fort Collins, Denver, Sterling, and Fort Morgan. (Figure 1).

FIG. 1. Sugar beet processing plant 1904, Fort Collins, CO.

The agricultural boom of the sugar beet industry carried with it a significant byproduct, the spoils from processing. Typically, the spoils were flushed from the sugar beet plants after processing; the byproduct, often in the form of slurry, was discharged over local farm fields (Figure 2). In some cases, the spoils were dried and placed in large spoil piles (Figure 3). The remaining pulp was often used as feed for cattle at nearby ranches.

Sugar beet production declined as the production of cane sugar and corn syrup became more profitable for farmers, leaving behind abandoned processing plants and their waste products.

Production of Beet Sugars and Resulting Beet Waste

Through photosynthesis, sugar beets can produce up to 20 percent of their weight as simple sugars. These sugars include glucose and fructose, which are combined to produce sucrose ($C_{12}H_{22}O_{11}$). The remaining 80 percent of the sugar beet is composed of plant pulp and impurities, which are extracted through a complex process.

FIG. 2. A flume for conveying beat spoils across the Cache la Poudre River, Fort Collins, CO.

In general, sugar beets were taken to a factory and placed into a diffuser, which used water to dissolve the sugars into a raw juice. The raw juice was then mixed with a lime concentration (calcium hydroxide or $Ca(OH)_2$) during a carbonization process to help precipitate some of the impurities retained in the mixture. The mixture was then passed through a filter system to remove the precipitate. Carbon dioxide was added to flocculate the remaining lime from the mixture in the form of calcium carbonate or chalk. Another set of filters removed the calcium carbonate precipitate and impurities from the processing including sulfates, phosphates, citrates, and oxalates. The mix was then treated with sulfurous acid to rebalance the pH and bleach the sugars and filtered again for precipitates. With the impurities of the sugar beet removed, the resulting beet liquor was used to produce several sugar products such as table sugar and syrup. The products of each filtering made up the beet waste that was either piled near the plant or washed as a slurry out of the plant to nearby lagoons or fields to dry.

At different times during the history of beet processing, the beet spoil was mixed or treated with different materials in an effort to use it to as soil amendments for agriculture. Although the modifiers were not well documented and success of these efforts is unknown, the result was variations in the chemistry of the beet spoils over time. In areas with a longer history of deposition, the specific chemistry of spoils may change with depth.

DISTRIBUTION OF BEET SPOILS

Sugar beet spoils can often be found adjacent to processing plants, but are also found distributed several miles from the plants. The beet spoils located near the processing plants typically occur in large piles and are locally referred to as, "beet lime”. A prime example of this are several very large piles south of the sugar beet processing plant located on State Highway 119 / Ken Pratt Boulevard in Longmont, CO. The difference in pile or slurry deposits appears to be a function of the processing filter system and the method the plants selected to transport the materials out of the plant.

Spoil piles we observed were fairly clean, white to gray in color, and look like a chalky in appearance. Beet spoils that had been transported away from the processing plants often covered large expanses of ground ranging from less than approximately 4,000 square meters (1 acre) to several square kilometers (100 acres). In the Fort Collins, CO area, beet spoil deposits have been observed within a 6.5-kilometer (4 mile) radius of a processing plant. The spoils are typically distributed in the form of slurry that was flooded onto fields or low-lying areas and allowed to dry. The deposits exposed along the Cache la Poudre River and adjacent areas range from less than 0.3 meter (1 foot) thick up to 4.5 meters (15 feet) thick (Figure 3). The beet spoils that were deposited as slurry range from fairly clean deposits that are white to gray in color to deposits mixed with, or leached into, native soils. The mixed deposits were often gray to light brown in color. Typically, only the lower 0.3 to 1 meter (1 to 3 feet) of beet spoils are mixed with native soils. Vegetation in areas of beet spoils is typically sparse compared to adjacent areas with no spoils.

FIG. 3. Beet spoil slurry deposit exposed on the north bank of Cache la Poudre River, Fort Collins, CO.

ENGINEERING PROPERTIES OF BEET SPOILS AND ASSOCIATED SOILS

CTL|Thompson, Inc has conducted several geotechnical investigations in the Front Range corridor for new developments, pavement design, and utility improvement projects in areas containing beet spoils. In addition, we have conducted an investigation in existing beet piles to determine their engineering properties. In general, the beet spoils placed as piles have SPT values that range from 0 to 3 for a 30-centimeter (12-inch) drive. Beet spoils placed as slurry and mixed with soils have slightly higher SPT values, depending on moisture content, that range from 0 to 10 for a 30-centimeter (12-inch) drive.

Visual identification can be simple when the beet spoils are relatively pure. The material is white to gray and chalky or appears silty. When mixed with soils, the proper identification becomes more difficult. Often, only laboratory testing can verify the presence of beet spoil. However, light colored soils with higher amounts of low plastic fines (silt-like) are a good indication of possible beet spoil materials.

Site Conditions and Observations

During our investigations, we conducted several site visits to areas with beet spoil deposits. Typically, these areas were relatively flat and often occupy flood plains adjacent to active river systems and lowland areas, prime for distribution of spoil slurries. In several cases, we observed sinkholes and piping features, some of which have significantly affected site improvements. At a location in Fort Collins, CO, we observed conical sinkhole features ranging from less than 0.3 meter (1 foot) to 15 meters (50 feet) in diameter (Figure 4). The depths of the sinkhole features observed ranged from less than 0.3 meter (1 foot) to 1.3 meters (4 feet). The depth of these features appeared to be controlled by the depth of the beet spoil deposit and the depth to ground water. It appears the mechanism for sinkhole formation is removal of beet spoil materials through piping, by solution or transportation (Figure 5). At one location, we observed surface water entering a pipe feature (Figure 6).

FIG. 4. Large sinkhole features in beet spoil slurry deposit, Fort Collins, CO.

FIG.5. Pipe features in beet spoil slurry deposit, Fort Collins, CO.

FIG. 6. Surface water entering piping feature in beet spoil slurry deposit, Fort Collins, CO.

Laboratory Testing

Laboratory testing of beet spoils has produced a wide range of results for moisture content, dry density, liquid limits, plasticity index, pH, soluble sulfates, and gradation. A summary of laboratory testing conducted on beet spoil piles is presented in Table 1, and a summary of laboratory testing conducted on beet spoils deposited as slurry is presented in Table 2.

Moisture contents of beet spoils deposited in piles range from 7.8 to 96.7 percent, while beet spoils deposited as slurry has moisture contents ranging from 5.2 to 50.1 percent. All of the samples tested were collected from above the groundwater table. The dry density of the beet spoils tested ranges from 675 to 1,400 kilograms per cubic meter (42 to 87 pounds per cubic foot).

Table 1. Laboratory Test Results of Beet Spoil Pile

	SPT Blows per 0.3 m (12")	**Moisture Content** (%)	**Dry Density** kg/m^3 (pcf)	**Liquid Limit **** (%)	**Plasticity Index** (%)	**pH**	**Soluble Sulfates** (%)	**Minus 200** (%)
	3	47.9	1393 (87)			8.8		
	1	47.4	1977 (86)	NL		8.6		93
	4					8.6		87
	35	7.8				8.7		22
	4	78.9	720 (45)				0.5	
	4	47.1	1105 (69)	52	20	8.7		58
	3	86.5	736 (46)				0.12	
	2	85.8	768 (48)	NL	NP	8.5		94
	2	85.3	704 (44)	NL	NP	8.7		95
	3	96.7	673 (42)				0.07	
	2					8.7		88
	2	73.6	849 (53)	NL	NP	8.5	0.009	94
	9	85.2	801 (50)	NL	NP		0.13	96
Ave	5.7	67	913 (57)	-	-	8.6	0.06	81
Max	35	96.7	1393 (87)	-	-	8.8	0.5	96
Min	1	7.8	673 (42)	-	-	8.5	.009	22

** NL indicates a liquid limit could not be obtained due to the plasticity of the sample.

Table 2. Laboratory Test Results of Beet Spoils Deposited as Slurry

	SPT * Blows per 0.3 m (12")	Moisture Content (%)	Dry Density** kg/m^3 (pcf)	Liquid* Limit (%)	Plasticity Index (%)	pH	Soluble Sulfates (%)	Minus 200 (%)
	NA	8.2	NA	31	16			56
	NA	5.2	NA	NL	NP	8.5	0.1	30
	NA	12.9	NA					60
	NA	32	NA	48	31		1.8	54
	NA	35.8	NA			8.1		63
	NA	31.9	NA	57	39			57
	NA	46.8	NA			8.3	1.9	65
	NA	43.6	NA			8.1	2.0	90
	NA	50.1	NA			8.0	1.8	60
Ave	-	29.6	-	45	29	8.2	1.5	59
Max	-	50.1	-	57	39	8.5	2.0	90
Min	-	5.2	-	31	16	8.0	2.0	30

* SPT and dry density values are not provided because slurry deposits were sampled in bulk.
** NL indicates a liquid limit could not be obtained due to the plasticity of the sample.

The results of Atterberg tests for the beet spoils were variable. In beet spoils from both piles and slurry deposits, liquid limits ranged from 31 to 57 percent and plasticity indexes between 16 and 39 percent. Many of the samples tested appeared to be non-plastic (NP) and non-liquid (NL) materials. During our laboratory testing, we encountered numerous samples of beet spoils from piles and slurry deposits that became very hard (apparent cementation) when submerged in water. In some cases, this occurred in less than one hour. These same samples, when dried, became a powdery, chalky like substance.

The pH of samples tested both from piles and slurry deposits were fairly consistent ranging from 8.0 to 8.8. When tested with 1 molar hydrochloric acid, the samples reacted moderately to vigorously.

The results of soluble sulfate testing from the beet spoil piles ranged from 0.009 to 0.5 percent. Samples of spoils from slurry deposits tested ranged from 0.1 to 2.0 percent. Native soils in the area of the beet spoils tested were typically alluvial in origin, consisting of cobbles, gravels, sands, silts, and clays. Our experience indicates soluble sulfates in the native alluvial deposits are generally very low (less than 0.01 percent).

The results of our gradation testing from both the beet spoil piles and slurry deposits indicated 22 to 96 percent passing the #200 screen. We believe the results of these tests were likely skewed due to flocculation of the material and the reaction to water described above.

DEVELOPMENT CONSTRAINTS

The explosion of growth along the Front Range corridor and on the Western Slope of Colorado has been significant over the last decade. Much of the development has occurred in areas formerly used for agricultural purposes, many of them adjacent to old sugar beet processing plants. The distribution of beet spoils in areas around processing plants is significant, ranging from less than an acre to several hundred acres. In the Fort Collins area, we have observed beet spoil deposits up to 6.4 kilometers (4 miles) from processing plants. Much of the property around the processing plants in this area is now slated for redevelopment or has recently been developed. Beet spoils and soils mixed with beet spoils provide poor support characteristics for roadways, structures, and other improvements.

Although the differences between the piled spoils and slurry spoils are minimal, the slurry spoils tend to affect larger areas and have leached into the native soils. Whereas the piled spoils can be more readily identified and removed, the leaching effect of the slurry spoils can affect much larger volumes of soil and can be harder to identify.

Roadways

Roadway construction in areas impacted by beet spoils is problematic at best. If not mitigated prior to construction of roadways, the support characteristics result in erratic pavement performance. The presence of piping and sinkhole features can cause local subsidence features in the pavement. Overall, pavements constructed on these materials have significantly elevated maintenance costs.

When soft ground or expansive soils are encountered in areas to be paved, the pavement subgrade is often stabilized using a fly ash, cement, or lime treatment. The high concentration of soluble sulfates found in beet spoils and soils mixed with beet spoils reacts negatively with common chemical treatments. The addition of calcium contained in the fly ash, cement, or lime often results in the formation of ettringite, a highly expansive mineral known to form heave features in pavements.

Structures

The very soft ground condition in areas of beet spoils often cannot support even lightly loaded residential and commercial structures without significant mitigation. Structures constructed with shallow foundations bearing on beet spoils are subject to large amounts of settlement. Settlement may be related to the soft ground conditions or piping and sinkhole formation. In addition to settlement issues, concrete foundation elements in contact with beet spoils are subjected to sulfate attack. Based on our experience with soluble sulfate concentrations in beet spoils ranging from 0.009 to 2 percent, according to the American Concrete Institute (ACI), the majority of sites would classify as Class 2 or Class 3, also known as severe and very severe exposure to soluble sulfates.

Other Improvements

Other improvements, such as flatwork, sidewalks, curb and gutter, and wet utilities constructed in areas with beet spoils can be significantly affected. Flatwork and sidewalks can undergo differential settlement, cracking, and sulfate attack. Due to the erratic behavior of beet spoils, grade critical wet utilities and curb and gutters can lose required slope and perform poorly. This may result in ponding issues and even flooding. Drainage channels such as stormwater channels cut into the beet spoils typically have unstable slopes that require extra support and are highly susceptible to erosion. In addition, drainage channels or waterways cut into beet spoils with piping and sinkhole features can either collect or lose water through these features.

BEET SPOIL MITIGATION

The development constraints placed on a site due to the presence of beet spoils can be significant, resulting in costly site mitigation, underutilization of a site, or even site avoidance. It is critical to identify if beet spoils are present at site before development begins. Only once the extents of beet spoils at a site are fully characterized can different means of mitigation be evaluated.

Site Investigation

Typically in the Front Range corridor of Colorado, geologic/preliminary geotechnical investigations and Phase-I environmental site assessments are conducted as part of the due diligence process when a site is being considered for purchase or development. Understanding potential geologic and geotechnical constraints of a site is the key to successful development. When subsurface conditions are understood prior to site development, the owner has the information needed for development planning and budgeting purposes.

Investigations conducted in areas containing beet spoils have not always identified the materials correctly. In the Fort Collins area, several investigations have misidentified beet spoils as wind blown loess deposits or alluvial silts. A proper site investigation in areas near sugar beet processing plants should include looking for evidence of beet spoils. Typically, geotechnical engineers and geologists are not aware of the extent of these materials.

Roadways

Typical mitigation for roadway construction in areas of soft soils includes removal and replacement, chemical treatment (including fly ash, cement, or lime), or mechanical stabilization such as geosynthetic fabrics. As discussed above, chemical treatment of beet spoils and soils containing beet spoils is problematic due to high soluble sulfate contents.

Mechanical stabilization can be a cost-effective way to stabilize pavement subgrade. A recent example of the need for mechanical stabilization over beet spoils occurred in Longmont, Colorado. The Colorado Department of Transportation (CDOT) needed to construct a 4.5 meter (15 feet) high embankment, which would support a rigid pavement. They needed a cost-effective solution that would mitigate differential settlement to avoid excessive pavement cracking. Geogrid was selected for incorporation in the embankment to solve this challenge (Figure 7).

Two additional options considered by CDOT included building a bridge to span the beet spoils or over-excavation and replacement of the beet spoils. Over-excavation and replacement of the beet spoil soils was estimated to have cost \$4,000,000, while the bridge option was estimated at \$2,000,000. The geogrid reinforced embankment option had a cost of approximately \$1,000,000 significantly less than the other options.

Another method used along the Front Range area to deal with beet spoil material is to relocate the material to non-structural areas and mix it with clean soils to “dilute” the effects of the beet spoils. We understand this method was used at the CDOT project in Longmont to relocate beet spoil affected materials into the embankments along the roadway.

FIG. 7. Construction of roadway embankment over beet spoils and stabilization of subgrade using Geogrid.

Structures

To eliminate potential settlement of structures constructed over areas of beet spoils, foundation systems that transfer the loads to a deeper, more stable, soil or bedrock layer can be used. Depending on the thickness of beet spoils at the site and the subsurface conditions, removal and replacement with select materials is an option. Beet spoils are often placed in low-lying areas with shallow ground water, resulting in a need for site dewatering during over-excavation and replacement. Depending on the size of the development, dewatering and purchase of clean fill can be cost prohibitive.

Other Improvements

Site improvements such flat work, sidewalks, curb and gutter, and grade critical wet utilities are somewhat problematic in areas of beet spoils. For these improvements, over-excavation and replacement of beet spoils is likely the most cost-effective solution for these improvements. For grade-critical wet utilities such as sanitary sewer and storm water, over-excavation or crowding aggregate below the improvements are viable options.

The variable nature of beet spoils combined with the negative reactions with commonly used stabilizers leaves a void of available chemical mitigation options. Research potential exists to find alternative chemical mitigation methods that can alter the chemical composition of the beet spoils to encapsulate the beet spoil or cement the beet spoil and improve the engineering characteristics.

CONCLUSIONS

Site development of areas underlain by beet spoils or soils mixed with beet spoils is problematic. Roadways constructed in these areas typically require stabilization. Roadways constructed over beet spoils without mitigation will result in poor pavement performance, differential settlement, and regular maintenance. Beet spoils do not have adequate support characteristics for even lightly loaded structures. Deep foundations or over-excavation and replacement can be used as support. Other site improvements such as flat work, sidewalks, curb and gutter and grade critical wet utilities can also present challenges in areas underlain by beet spoils.

The key to a successful development in an area with beet spoils is knowing the extent of the materials at the site. A proper geologic/geotechnical investigation is necessary to understand the subsurface conditions at a site. Conducting an appropriate investigation will result in understanding the site constraints and allowing for proper planning and development budgeting.

ACKNOWLEDGMENTS

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Ground Modification by Sub-Excavation
For Subdivisions on Expansive Soils

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ABSTRACT

Deep over-excavation and recompaction at high moisture content is a process currently used to address swelling soil conditions for residential subdivisions and light commercial construction. Sub-excavation to relatively deep depths was initially conceived based on observations of performance of structures on steeply dipping bedrock. CTL | Thompson Inc. first used this approach in 1994/95 to address swelling issues in a large subdivision north of Golden which was underlain by steeply dipping expansive bedrock. The process was then used in other areas underlain by steeply dipping bedrock adjacent to the Front Range. Subsequently, as developers gained confidence and experience, the method spread to areas where the bedrock is relatively flat lying.

The paper presents results, in summary form, of experience gained during the last 10 years throughout the Denver metro area, specifically at three sites: one underlain by dipping bedrock and two in flat lying bedrock locations. Graphs of swell potential before and after treatment and comparison of moisture content, dry density and swell potential as affected by sampling procedure are included. Typical quality control issues are identified. The process has been successfully used on several thousand single-family residential lots in the Denver metro area. Data developed over the last 10 years should give confidence to others as they discuss this approach with their clients.

INTRODUCTION

Deep excavation and replacement of expansive clays and claystones compacted to high moisture content was adopted in the Denver area as a ground modification technique in the mid 1990's. From the 1960's to the 1990's relatively thin fills constructed with non-expansive soils were used to improve performance of commercial construction. Performance of the replacement fills was mixed. Extensive monitoring of a subdivision located in an area of shallow, steeply dipping bedrock indicated better performance of the structures where there were 3 meters (10 or more feet) of cover over the bedrock in basement areas. This monitoring led to the concept of ground modification using over-excavation and recompaction in an area of steeply dipping bedrock.

The initial application of this ground modification used a minimum depth of over-excavation of 3 meters (10 feet) below the required basement excavation. At the time, this resulted in excavations about 4.9 meters (16 feet) below the finish lot grade for the pre-construction pad. The basements extended about 1.8 meters (6 feet) below pad grade and the soils from basement excavation were used to shape individual sites and develop surface drainage.

Typical practice is to use about 3 meters (10 feet) of moisture stabilized soil below the deepest required house excavation with 1 to 1.5 meters (3 to 5 feet) of stabilized soil below pavements. The deep excavation extends at least 1.5 meters (5 feet) beyond the footprint of the residence. Typical specifications require placement of the moisture stabilized soil at moisture contents between optimum and 4 percent above optimum as determined by ASTM D 698. Compacted density is usually specified as at least 95 percent of the maximum determined using ASTM D 698. Our firm frequently specifies 1 to 4% above optimum. Range of allowable variation from specified density and moisture is specified both by the test and day. Potential heave needs to be considered during the preliminary geotechnical study to establish the depth of modification for a specific subdivision or filing. The intent is to achieve very uniform conditions. The desired end result should be a low swelling, very impervious fill.

The required depth of over-excavation generally ranges between 4.9 and 6.1 meters (16 to 20 feet) below finished over-lot grade. Some subdivisions have been constructed which required over-excavation to 6.1 or more meters (20 feet) depth. In one of the early projects with steeply dipping bedrock, finite element modeling (FEM) was used to evaluate propagation of linear heave through the stabilized zone. The FEM model was used to judge the required thickness of stabilization assuming that a sharp heave occurred in the bedrock below the stabilized zone, McOmber (2003). Greater depths of over-excavation were also used in some flat lying situations where very high swell occurred in preliminary testing. In addition to evaluation of swell potential data, selection of the depth of sub-excavation should consider the depth of significant wetting. Wetting depth is discussed by Thompson (1997) and McOmber (2003).

Currently the ground modification procedure generally occurs in 3 forms. The most reliable form is over-excavation of an entire subdivision or filing. Projects ranging from 20 to more than 300 lots have been completed. This modification occurs as part of overlot grading prior to utility or curb and gutter installation. The second form occurs when a builder/developer purchases developed lots with curb and gutter and utilities in place in the streets. Groups of lots can be treated using the over-excavation method, but it is difficult to get ramp areas and the edges and corners of excavations uniformly moisture treated and compacted.

The least desirable method is a single or 2 lot over-excavation site. Uniform wetting, mixing and compaction are difficult in limited work areas. The example data included in this paper are from sites where modification occurred during overlot grading of multi-lot subdivisions.

ENGINEERING SERVICES

Multiple engineering services by the geotechnical consultants are necessary to achieve a good product in any ground modification scheme. Typically, the relative expansive nature of the soils and bedrock is evaluated in a preliminary site study for zoning. The studies may recommend deep foundations and structural basement floor systems with ground modification alternatives to allow shallow foundations and either slabs-on-grade or structural basement floor systems. In the steeply dipping bedrock areas, some county regulations require the over-excavation procedure. The preliminary site study usually recommends a depth of over-excavation followed by compaction at specific moisture and density. A general civil engineering firm prepares grading plans and defines the location and extent of over-excavation to meet the geotechnical recommendation. Specifications must address the range of allowable moisture as criteria.

Full time observation and testing by the geotechnical consultant is normally required on over-excavation modification projects. Tracking daily variation in moisture content with maximum variation specified is normal. A graph of variation of average moisture content against compaction report number is shown on Fig. 1. In addition, thin wall tube samples (hand drive samples) are obtained frequently to confirm percent swell and provide a check of nuclear moisture/density tests. The field quality tests are reviewed both daily and weekly to evaluate the likely performance characteristics of the stabilized fill.

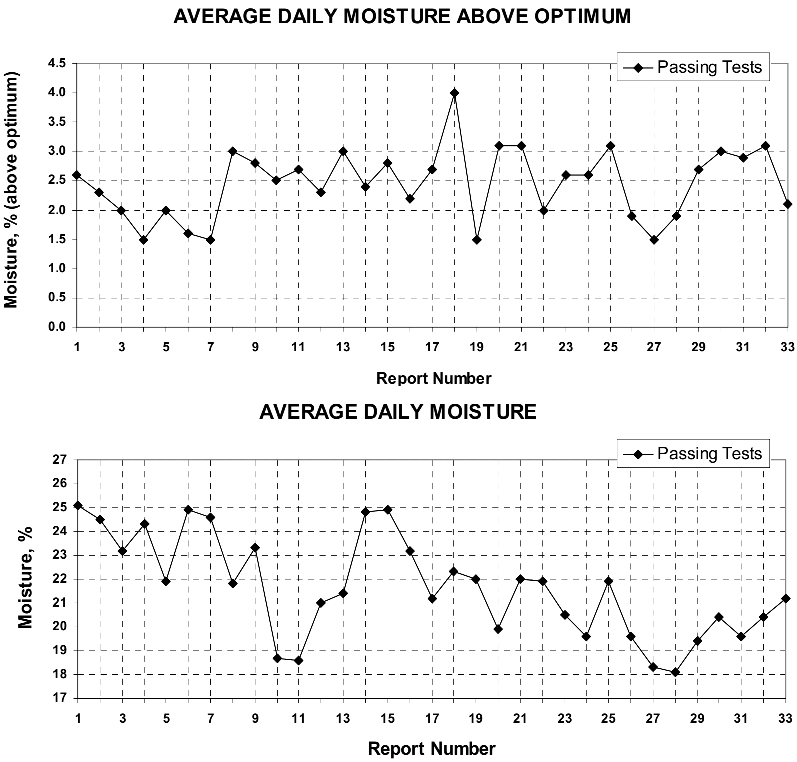

FIG. 1. Daily Moisture Content and Deviation from Optimum

After the over-excavation process is complete, utility installation, curb and gutter and other preconstruction activities proceed similar to land development where over-excavation is not done. The authors' practice is to drill and sample each lot after the ground stabilization. Thickness and performance characteristics of the moisture stabilized fill are confirmed using swell/consolidation tests. Sampling is performed using both California barrel samples and 76 mm (3.0 inch) diameter Shelby tube samples. The measured swell in laboratory tests appears to be influenced by the sampling method. This issue is discussed under **IMPLEMENTATION ISSUES AND PERFORMANCE**.

EXPERIENCE 1994 TO 2008

Records were searched at the authors' firm to identify subdivisions, filings, and lots where over-excavation was used for ground modification. Projects involving 10 or more lots were selected and included subdivisions with and without ground modification. Multi-family or apartments were excluded.

Approximately 1000 project numbers were identified as meeting the 10 or more lot criteria. This includes all subdivision subsurface investigations in a 15-year period. The database was too large to complete evaluation for this paper. After determining the size of the database, we decided to limit the data analyzed for this paper to only projects involving ground modification. Most of the projects where data was used in this study were completed between 2003 and 2008. The data from one very large project completed during 2000 and 2001 were used because of extensive swell test data that was available for over 300 lots. The data from the 2003 to 2008 period included 11 different subdivisions spread over the Denver metro area. Several of the subdivisions had multiple filings. The data analyzed were developed from 24 project numbers.

Three subdivisions with different geologic conditions located in the northeast to southwest parts of the Denver metro area were selected to demonstrate typical geotechnical data. The data included soil characteristics before ground modification, data collected during compaction control of the modification process and test results from confirmation borings for design of the structures.

The subdivision with dipping bedrock is located in northwest Douglas county adjacent to the area described by Noe (2005). The adjacent subdivision experienced severe foundation failure in the late 1980's, Thompson (1995). The geotechnical characteristics reported are primarily from the sections of the site underlain by steeply dipping portions of the Pierre Shale formation (Kpu). Bedding dips on the order of 60 degrees to the east. The subdivision comprises 333 lots and ground modification to depths of up to 6.1 meters (20 feet) was required.

A subdivision located in the south metro area several miles east of the dipping bedrock area and underlain by Dawson and Arapahoe formations (TKda) was selected to demonstrate geotechnical data from a flat lying bedrock site. The soils and bedrock at this site were less expansive than the dipping bedrock site and the representative site selected for the northeast part of the Denver metro area.

The third site selected from the authors' database is located in the northeast part of the Denver metro area where considerable new development has occurred in the last 5 years. The site is underlain by the Denver formation (TKd). Preliminary data indicated the near surface bedrock to be highly expansive. The author's experience with other sites in the north Denver area with similar bedrock indicated frequent movement of drilled pier foundations.

DIPPING BEDROCK SUBDIVISION

Swell testing from the preliminary geotechnical study showed that more than 60% of the near surface samples possessed high to very high swell potential. In Thompson (1997), the range of swell potential and local classification are presented. More than 68% of the high swelling samples were in the very high classification. Swell test results shown in Fig. 2 indicated that compaction at moisture contents between 1 and 4 percent above optimum moisture content resulted in swell potential in the range of 1.5 to 2%. ASTM D 698 was the procedure used for remolded moisture and density. The specification recommended for the ground modification required compaction to at least 95% of maximum density as determined by ASTM D 698. The moisture was required to be between 1 and 4% above optimum. Tests less than 0.5% above optimum were rejected. The daily average placement moisture was specified to be at least 2% above optimum. Fig. 1 shows variation of field moisture content by compaction report number (day). The field moisture content is plotted by departure from optimum.

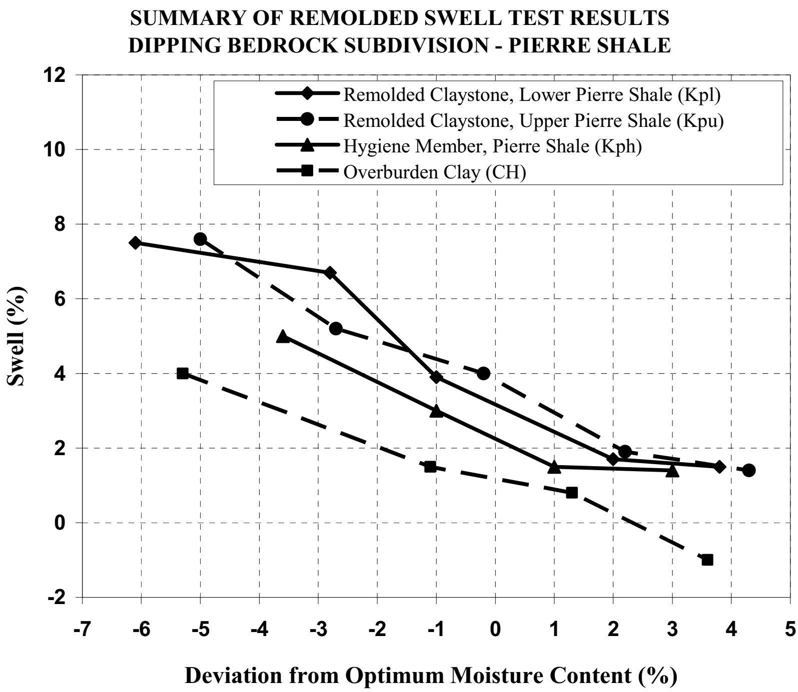

FIG. 2. Effect of Remolded Sample Moisture Content on Swell

During the ground modification process, a total of 369 field moisture/density tests were performed using the nuclear method with occasional sand cones for a check. Hand driven, thin walled samplers, 51mm (2.0 inches) O.D. and 49 mm (1.935 inches) I.D., were obtained (35 samples). These samples were tested for swelling characteristics in the laboratory. The nature of the sampling device and details of the laboratory test procedure are presented in Thompson (1995). Using the above described quality control process, a relatively uniform fill was placed in the over-excavated areas.

After completion of the site grading, borings were drilled on each lot. These borings were sampled using both a modified California barrel and 76mm (3.0 inches) Shelby tubes. A comparison of swell test results using the three different sampling methods is shown on Fig. 3.

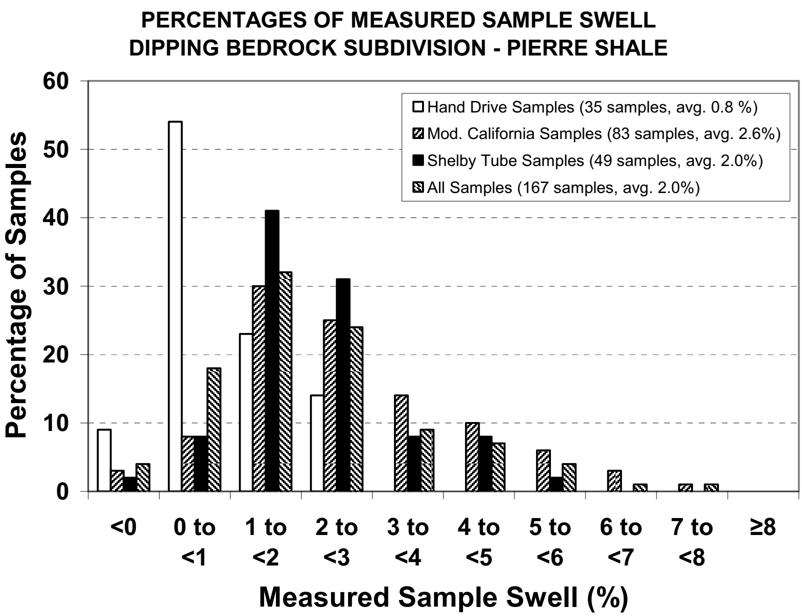

FIG. 3. Comparison of Sample Swell

The hand drive samples on average exhibited 0.8% swell. The samples obtained using the modified California barrel averaged 2.6% swell and the Shelby tube samples averaged 2.0% swell. The laboratory remolded samples indicated 1.5 to 2.0% swell, the difference between field and laboratory remolding is discussed under **IMPLEMENTATION ISSUES AND PERFORMANCE**.

SOUTH METRO SUBDIVISION

A similar field quality control approach was used in the subdivision in flat lying beds in the south metro area. The filing where data were used for this study comprised 134 lots. The average for preliminary swell tests was 4.5%. Total soil suction liquid limit and plasticity index averaged near 3100kPa (4.5 pF), 50 percent and 31 percent respectively. After site grading was complete, samples from individual lot borings exhibited an average swell potential of 1.1% and suction of 545kPa (3.74 pF). Table 1 presents a summary of soil and swelling characteristics prior to ground modification. Table 2 presents subsurface data after ground modification. The swell potential prior to modification was not unusually high. The builder/developer client preferred shallow foundations and selected the ground modification approach. The test data in Table 2 show that 21% of the samples compressed when wetted under the 47.9 kPa (1,000 psf) test load. For this moderately swelling site, ground modification by over-excavation and recompaction at high relative moisture resulted in samples from more than 78% of the lots exhibiting swell potential less than 2%.

Table 1. Summary of Swell Test Results Before Sub-Excavation South Denver Subdivision

Soil Type	Compression	Range of Measured Swell (%)*			
		Low 0 to <2	Moderate 2 to <4	High 4 to<6	Very high ≥ 6
	Number of Samples and Percent				
Sandy Clay	3 18%	4 23%	3 18%	6 35%	1 6%
Weathered Claystone Bedrock	0 0%	0 0%	1 17%	3 50%	2 33%
Claystone Bedrock	2 3%	29 51%	20 35%	5 9%	1 2%
Claystone/ Sandstone Bedrock	1 14%	2 29%	1 14%	2 29%	1 14%
Overall Number	6	35	25	16	5
Overall Percent	7%	40%	29%	18%	6%

***Swell measured after wetting under an applied pressure of about 47.9KPa. (1,000 psf)**

Table 2. Summary of Swell Test Results After Sub-Excavation
South Denver Subdivision

Soil Type	Compression	Range of Measured Swell (%)*			
		Low 0 to <2	Moderate 2 to <4	High 4 to<6	Very high ≥ 6
	Number of Samples and Percent				
Fill	50 21%	185 78%	1 0.5%	1 0.5%	0 0%
Claystone Bedrock	0 0%	0 0%	1 100%	0 0%	0 0%
Overall Number	50	185	2	1	0
Overall Percent	21%	78%	0.5%	0.5%	0%

***Swell measured after wetting under an applied pressure of about 47.9 kPa (1,000 psf)**

NORTH METRO SUBDIVISION

The Denver formation occurs at shallow depth below the north Denver metro subdivision. The formation has highly plastic claystones interbedded with sandstone layers. Test data from the preliminary subsurface exploration indicated high to very high swell potential for most of the site. Filing 1 comprises 218 lots. All of the lots were sub-excavated during overlot grading. The depth of ground modification was 16 feet. Field compaction records indicate 13 Proctor tests (ASTM D 698) were used during grading. The average maximum density was 1,709 kg/m³ (106.7 pcf) with a range of 1,586 to 1914 kg/m³ (99 to 119.5 pcf) Two of the Proctors had maximum dry densities of 1,810 and 1914 kg/m³ (113 and 119.5 pcf). These high unit weights are for the sandstone portions of the formation. Most of the maximum Proctor density values ranged between 1586 and 1698 kg/m³ (99 and 106 pcf). Excluding the sandstones, the average maximum dry density was 1674 kg/m³ (104.5 pcf). The average density obtained by the hand drive method during placement was 1634 kg/m³ (102 pcf). Distribution of density test results using hand drives, California samples and Shelby tubes is shown on Fig.4.

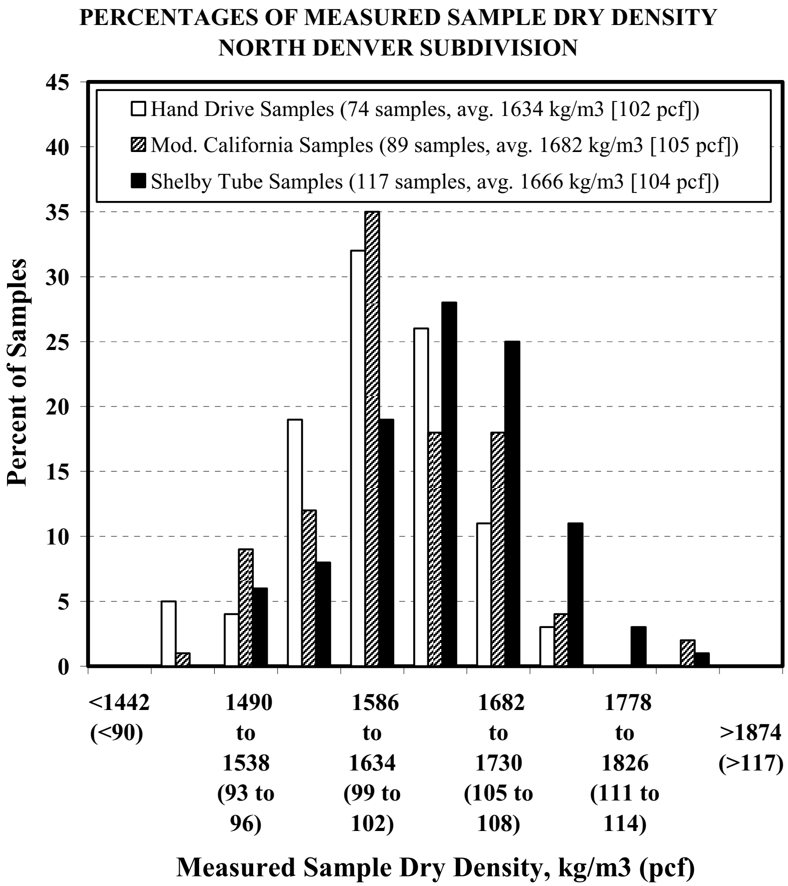

FIG. 4. Comparison of Sample Dry Density

Excluding the very sandy materials, the average optimum moisture content for the Proctor tests was 19.7 %. Distribution of sample moisture content is shown on Fig. 5. The average moisture content for all samples is 20.2%. The hand drive samples averaged 21.4% which corresponds well with the nuclear field moisture content data.

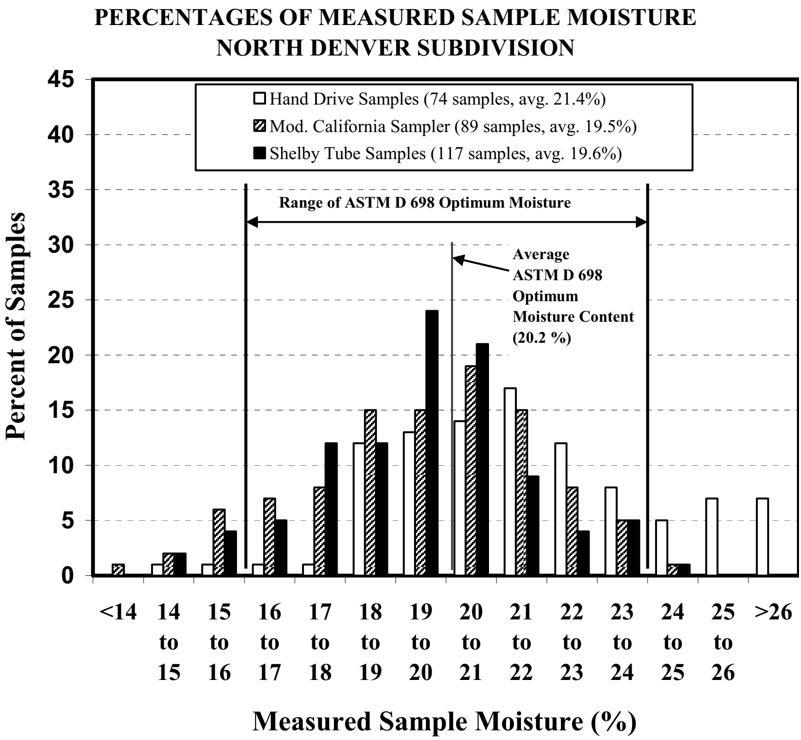

FIG. 5. Comparison of Sample Moisture Content

Liquid limits for some of the claystone samples were near or above 50 percent with plasticity indexes of 28 to 36 percent. The average optimum moisture content and range for optimum as determined by the Proctor tests is shown on Fig. 5. Distribution of swell test results is presented on Fig.6. Pre-construction swell potential by depth is compared to swell potential by depth after modification on Fig. 7.

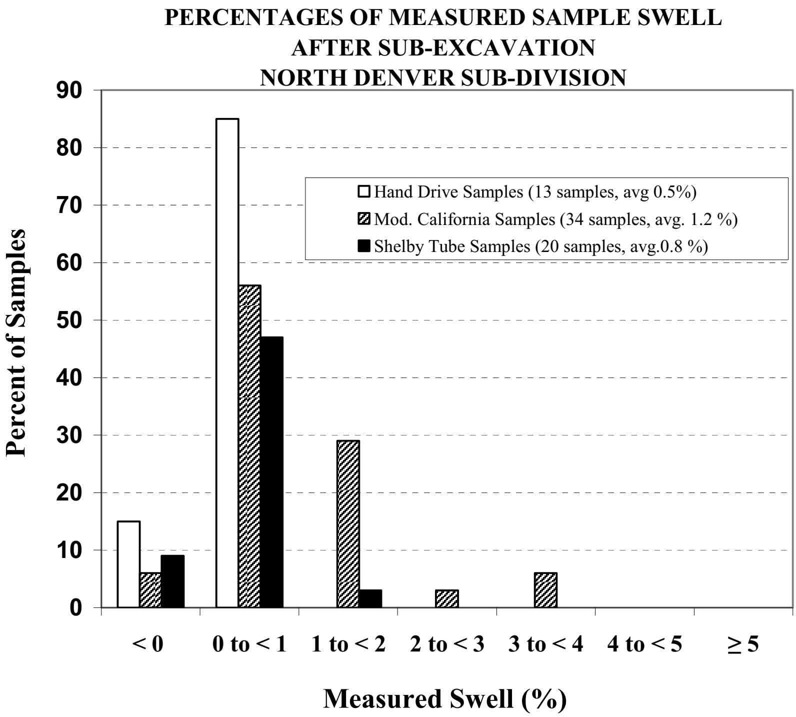

FIG. 6. Measured Swell After Sub-Excavation

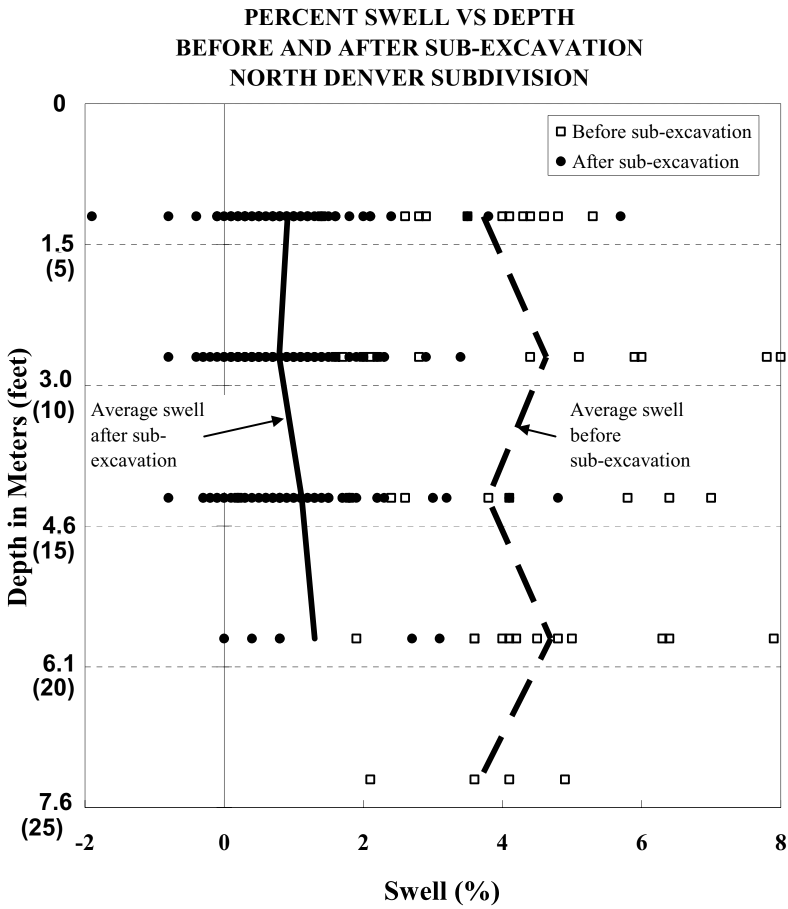

FIG. 7. Comparison of Swell Before and After Sub-Excavation

TRENDS

The data show reduction in swell potential after ground modification. The distribution graphs reflect the variation of plasticity and sand content of the soils and bedrock. The end result is a relatively uniform fill of high moisture and low permeability. The permeability is inferred from the typical Atterberg limit and percent passing the No. 200 sieve tests. The data from the 3 subdivisions indicate the samples typically exceed 40% passing the No. 200 sieve, and most exceed 60% passing. Typical liquid limit values range from 40 to 60 percent.

While the fill is relatively uniform as compared to the natural condition, the distributions of both moisture and density demonstrate the need for careful monitoring of the ground modification process to achieve a low swelling end product. Swell test results of hand drive samples obtained during fill placement tend to be lower than tests on samples obtained after completion of grading. The moisture and density determined using the hand drive procedure correlate with the nuclear field density test results.

IMPLEMENTATION ISSUES AND PERFORMANCE

Overall the reported problems by builder/developer clients have been less severe than with historical practice using drilled piers. Our clients report relatively minor cosmetic cracking, believed to be settlement issues. In a few instances, settlement has been significant enough to merit remediation by compaction grouting or underpinning. In many of these cases, poor drainage caused excessive wetting of the fill.

Two subdivisions in the steeply dipping bedrock areas were constructed using this technique where there was an extensive record of foundation failure and litigation in the adjacent subdivisions, Noe (2005). We are aware of no problems in the adjacent subdivisions where ground modification occurred, except for one house where lateral foundation wall movement occurred. The structures are at least 6 years old, certainly adequate time for swelling damage to arise. Some of the significant heaving in one of the adjacent non-modified subdivisions is discussed in McOmber (2003).

Problems have occurred with survey location of the zone to be sub-excavated. The basic footprint of the house must be determined at the time of grading. Changes in house design and location on a lot have created issues with respect to the location of the treated zone. Cul-de-sac lots and lots at the end of a sub-excavated area have the greatest frequency of layout issues. Ramps down into the excavated area should be planned carefully, and considered when foundation type is selected. This is a particular problem when attempting use of this stabilization method in a subdivision after curb and gutter and street utilities are in place, or when only part of a site is sub-excavated. Changing the type and footprint of a house between grading and house construction has caused problems with part of the structure on treated soils and part on an untreated area. The geometry of the excavation should allow good compaction of fill. Special attention is merited for compaction in excavation corners and adjacent to cut slopes. Clients should be warned of these issues and can address the potential problems with larger sub-excavation zones during grading to provide flexible sites.

The builder/developer client must be willing to support the cost of additional mixing water, processing equipment and geotechnical monitoring to achieve success with this ground monitoring procedure. The stiff clays and claystones are broken to small pieces with large disks. The soils break down with adequate water and processing with a disk, however, the break down is not as complete as pulverized laboratory samples. Small gravel size pieces and occasional chunks of stiff clay or claystone occur. We believe these small pieces may explain some of the extremes in the distribution diagrams of swell, density and moisture data. The California sampler and the Shelby tubes are more likely to subject the sample to densification by the sampling process than the hand drive tubes. We believe this difference in sampling methods may account for the differences shown on the distribution diagrams.

CONCLUSIONS

1. Deep sub-excavation was developed as a method to address the high frequency of drilled pier foundation movement in steeply dipping expansive claystone.

2. Good performance of structures in the steeply dipping hazard zones after sub-excavation led to use of sub-excavation in areas where the claystones are nearly horizontally bedded.

3. The first use of deep sub-excavation occurred in the early 1990's. Reported performance has been good. Claims and litigation regarding foundation performance on sub-excavated sites have been less frequent than with other methods.

4. Most problems reported to the authors' firm have been structures where sub-excavation limits did not match the footprint of the structure, or when residences are sited over ramps, or excavation corners or edges. These issues have been attributed to layout problems during sub-excavation, changes in the structure location after grading or the contractor's excavation approach. Good surface drainage remains a key design and construction issue.

5. Comprehensive construction monitoring is essential to achieve the desired mass of uniform low swelling, low permeability fill.

ACKNOWLEDGEMENTS

We thank the clients who had the confidence to proceed with this modification method during the early applications. Several engineers at our firm assisted in compilation of the extensive data collected as part of the monitoring during the sub-excavation process. The data included should help other engineers evaluate this technique for sites with similar subsurface conditions.

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Remedial Measures for Spillway Stability Problem on the Colorado Front Range

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ABSTRACT: Stability problems with dam spillways on existing landslides and appropriate mitigation measures to remedy them are ongoing concerns for geotechnical engineers. The first successful application of drilled piers as stabilizing structures under the spillway was at the Trinity Dam in Sicily, Italy. Based on the available literature the first application of drilled piers to stabilize a spillway chute in the U.S. was at the Ralston Dam in Colorado.

Ralston Dam, which is a 53.3-meter-high (175-foot) zoned fill dam located 24 km (15 miles) northwest of downtown Denver, was built in the late 1930s. The concrete spillway for the dam was built on the Pierre Shale Formation in the left abutment. Since reservoir operations began, mass movements have occurred downstream of the chute area. With modifications that include: 1) a stabilizing berm with a toe trench that improves the global stability; 2) seepage prevention and control measures that prevent the infiltration of water; and 3) the structural support of the replaced chute section with drilled piers and reinforced concrete the probability of future displacements or other problems have been reduced for the Ralston Dam Spillway. Over the last two years Ralston Dam spillway performed without any problems.

INTRODUCTION

Stability problems related to clay shale deposits have occurred worldwide. Some of these problems are related to water retaining structures. Some geologic formations, such as Pierre Shale (Oahe Dam, U.S.), Bearpaw Shale (Fort Peck Dam, U.S.; Gardiner Dam, Canada), Cucaracha Clay Shale (Panama; Panama Canal zone), and Yazli Formation (eastern Turkey; proposed Alpaslan 2 Dam site), are especially problematic because of the 50% to 75% strength reduction on the slide planes after initial sliding. With the exception of the Cucaracha Clay Shale, all of the formations mentioned above contain volcanic ash that was altered into bentonite and montmorillonite layers over geologic time.

Physically, clay shales are transitional materials between rock and soil; therefore, they exhibit properties of both. This has been a source of problems for geotechnical engineers, who traditionally analyze geological materials either with rock mechanics or soil mechanics, but rarely in terms of both. Furthermore, clay shales tend to transition from rocklike behavior to soil-like behavior within a relatively short time period. This transition can be due to unloading, weathering, swelling, and subsequent increase in water content of shales. Such relatively rapid changes in material properties create challenges in classification and in engineering design.

Slaking and fracturing within clay shales is a significant example of the transitional behavior. Botts (1986) indicated that clay shales progress through four stages of alteration, including (1) a rock-like mass in which the strength is controlled primarily by the orientation of and the strength along intact fractures, (2) a partially softened rock-like mass with its strength controlled primarily by the strength and orientation of soft, filled fractures, (3) a highly-softened mass consisting of a matrix of soft clay surrounding stiff, intact cores, and finally (4) a fully-softened, remolded clay. Similarly clays undergoing swell-shrink cycles transform to solid, semisolid, plastic, viscous liquid, and liquid states (Fang, 1997). The analysis of a clay shale undergoing progressive softening along fissures is complex, and generally requires some understanding of principles from both soil and rock mechanics.

RALSTON DAM

Ralston Dam, which is a 53.3-meter-high (175-foot) zoned fill dam located 24 km (15 miles) northwest of downtown Denver, was built in the late 1930s The facility includes a concrete spillway constructed on Pierre Shale in the left abutment of the dam. A landslide in 1942 damaged a section of the spillway chute that was subsequently repaired. Additional landslides requiring periodic spillway maintenance have occurred since then. In 1982, a tension crack beneath the right wall of the spillway was filled with 42 m^3 (55 yd^3) of sand and grout. In 1984, Chen Associates installed piezometers and inclinometers to monitor the slope below the spillway. In 1994, spillway joints were sealed and clean-outs were constructed. In April 2005, it was determined that the flow of water in the spillway removed a section of the spillway chute floor and that water was leaking into the spillway foundation. Concurrently, piezometers within the slope recorded a rise in the phreatic surface level, creating concerns for further landslide movements. In 2006 and 2007, URS was contracted by Denver Water to develop a more permanent solution by the installation and monitoring of inclinometers and piezometers; design and construction of slope stabilizing measures; and replacement of the distressed section of the chute.

The geologic unit under the left abutment of Ralston dam is Cretaceous-age Pierre Shale. The Pierre Shale in this area is steeply dipping, about 52^o to the southeast (Van Horn, 1972). Costa and Bilodeau (1982) described the Pierre Shale in the region as follows: "…contains thin beds of montmorillonite and mixed-order clay minerals and exhibits moderate to high swell potential. It is over-consolidated, generally easy to excavate at shallow depth, and only moderately erodible. Slope stability is good where the shale is undisturbed, and in cuts less than 45 degrees where groundwater is not present…"

Field investigations in 1984 and 2006 indicated 1.5 to 6 meters (5 to 20 feet) of colluvium and fill on the slope. A plan view of the Ralston Dam spillway chute area and the analyzed section of the slide geology are provided in Figures 1 and 2, respectively. Samples collected in the vicinity of the chute indicate a highly fractured Pierre Shale formation with clay fillings.

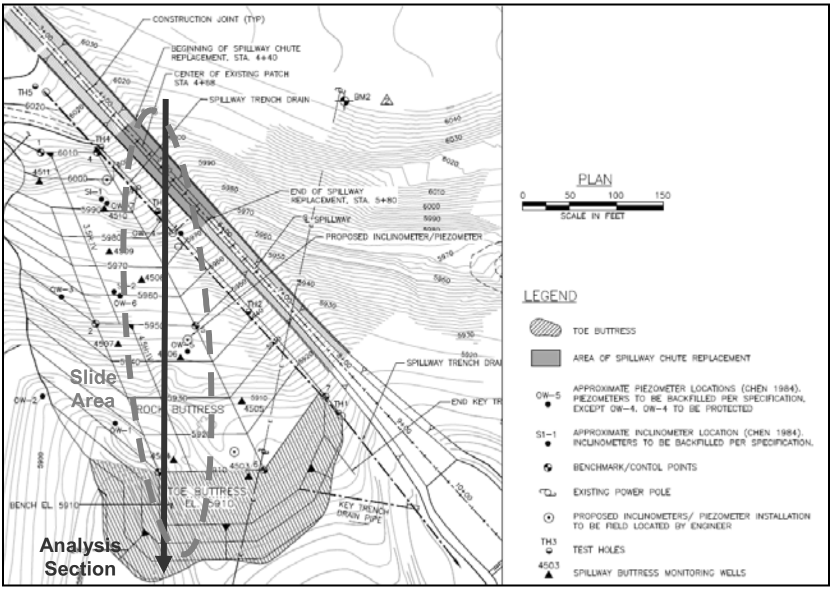

FIG. 1. Plan View of Ralston Dam Spillway Chute Area

We modeled the existing conditions at the site based on the information from Chen Associates (1984). Site topography data, historical photographs of the site, and the piezometric data were provided by Denver Water (2005).

During the 1984 exploration program, piezometers were set within the slide mass (Figure 1). Monthly readings of these piezometers were recorded by Denver Water. The records indicate periodic increases in water levels in some of the piezometers along the slide plane. The elevated piezometer readings correspond to operation and leakage from the spillway. According to Denver Water (2005), previous slope movements have been correlated with the elevated piezometric levels. Cycles of wetting and drying of spillway foundation during operations probably initiated swelling and movement of the slide. This is consistent with typical slope failure mechanisms. Therefore, we used the highest piezometric reading for each piezometer for the stability analysis.

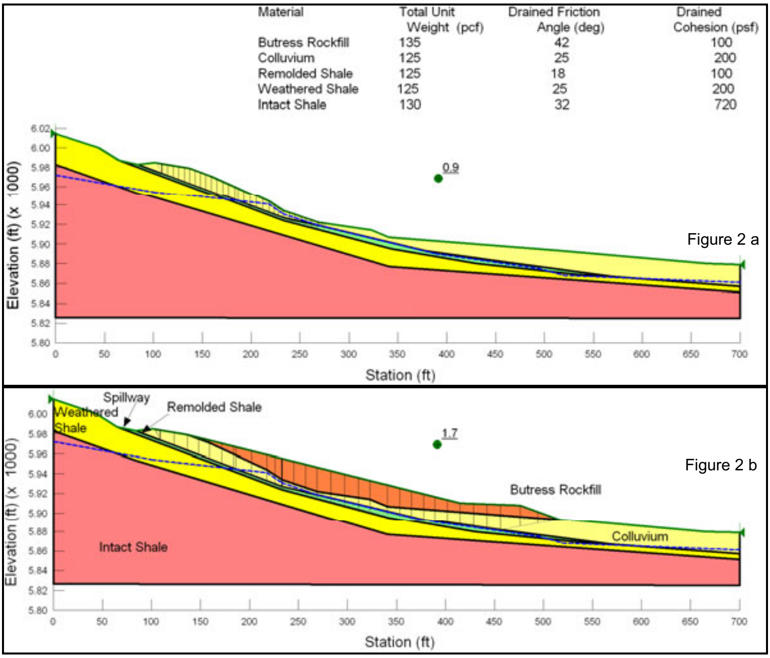

FIG. 2. Analyzed Section of the Slide

Chen & Associates (1984) delineated the approximate slide area based on aerial photographs and topography. We reviewed this delineation and updated the topography in the area. Based on these evaluations, we selected a critical cross-section through the historical slide (Figure 2a). According to the stability analysis, this section had the least stable slope geometry through the slide. This cross-section was used both to evaluate the existing conditions and to design the buttress.

ENGINEERING PROPERTIES OF THE FOUNDATION AND STABILITY ANALYSIS

Once a slide occurs, the residual strength conditions should be considered for the slip plane (Stark and Eid, 1997). Residual friction angles (ϕ_r') of clays and shales were reviewed in detail by Kenny (1967) and Mesri and Cepeda-Diaz (1986). The latter used pulverized shale for their tests to maximize disaggregation. Observations were recorded and empirical relationships were developed among residual friction angle, liquid limit, clay size fraction, and intact friction angle.

Liquid limit (LL) is a function of particle size, platiness and mineralogy. There is an inverse correlation between residual friction angle and liquid limit as provided in Mesri and Cepeda-Diaz (1986) and as adapted in Figure 3. Ralston Dam foundation laboratory test results generated 42, 54, 56, 57, and 62 as potential values for the LL. The average LL value of 54 is shown in Figure 3 as the representative value for the Ralston Dam spillway foundation.

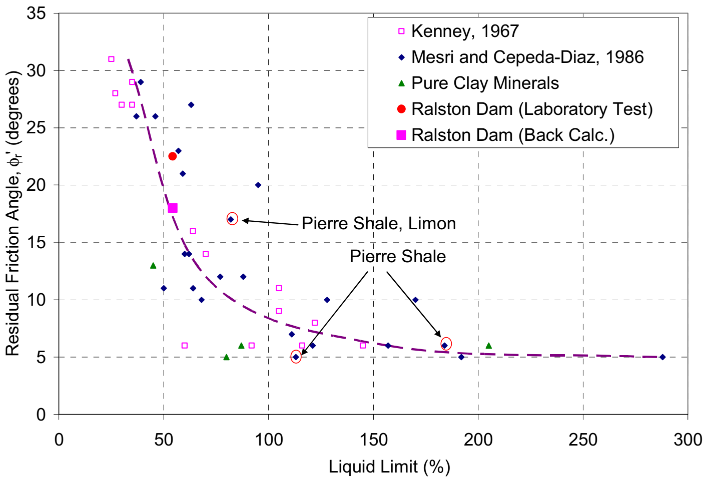

FIG. 3. Relationship Between Residual Friction Angle and Liquid Limit [adapted from Mesri and Cepeda-Diaz, (1986)]

Clay size fraction (CF) can be determined from the hydrometer test results and from Stoke's law, represented by particles with an equivalent spherical diameter of less than 2μm. There is a negative correlation between residual friction angle and liquid limit as provided in Mesri and Cepeda-Diaz (1986) and as adapted in Figure 4. Ralston Dam foundation laboratory test results provided 47% and 48% as the values for the CF. The average, CF = 47.5%, is shown in Figure 4 as the representative value for the Ralston Dam spillway foundation.

There is a positive correlation between residual friction angle and peak friction angle as provided in Mesri and Cepeda-Diaz (1986) and as adapted in Figure 5. In order to estimate the residual friction angle directly, a laboratory test program was implemented in 1984. Two California liner samples were recovered from the Ralston Dam site and reverse shear box tests were performed to estimate the residual shear strengths of the samples. Samples were sheared for three cycles. Laboratory test results were 22 and 23 degrees for ϕ_r'. The average value of 22.5 degree is shown as "Ralston Dam (Lab. Test)" in Figures 3, 4, and 5.

We conducted limit equilibrium stability analysis with Slope/W (GeoSlope International, 2004) to back-calculate the residual friction angle on the potential failure plane. This analysis yielded $\phi_r' = 18$ degrees as the probable residual friction angle and is shown as "Ralston Dam (Back Calc.)" in Figures 3, 4, and 5.

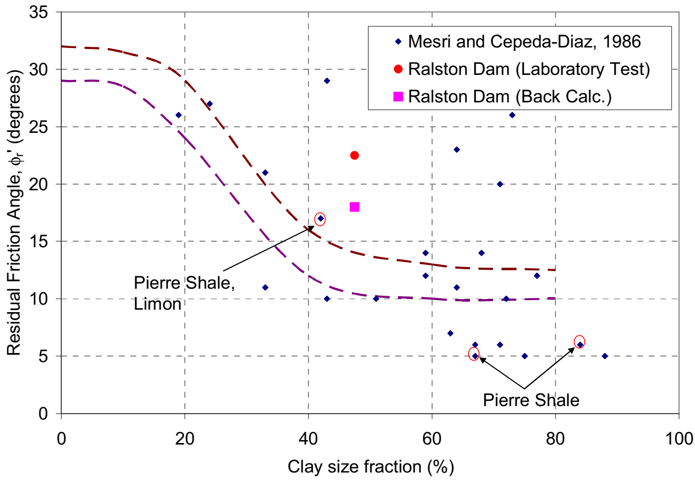

FIG. 4. Relationship Between Residual Friction Angle and Clay Fraction [adapted from Mesri and Cepeda-Diaz, (1986)]

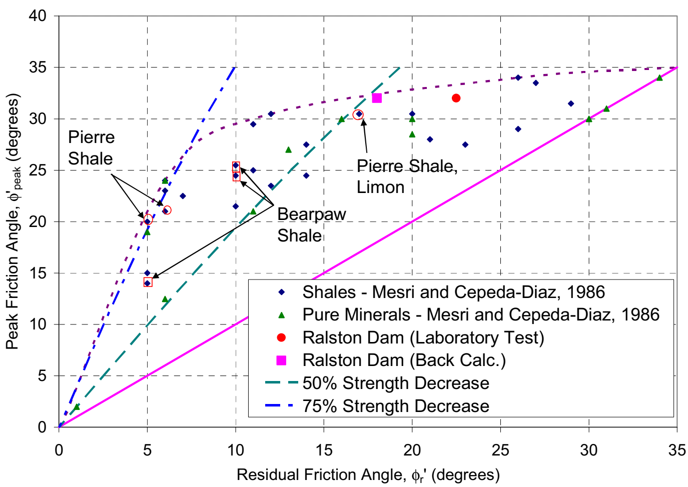

FIG. 5. Relationship Between Peak and Residual Friction Angles [adapted from Mesri and Cepeda-Diaz, (1986)]

Two methods were used to estimate the peak friction angle of the Pierre Shale at Ralston Dam:

1. Laboratory test results provided 22, 29, 31, 32, and 42 as the values for plasticity index. Based on the correlation developed by Ladd et al. (1977) on normally consolidated clays and measured plasticity index values, 25 to 33 degrees provide the possible friction values. For over-consolidated clay and shale friction values are expected to be higher.
2. Rock mass strength was estimated using the RocLab (Rocscience, 2005) program resulting in $\phi' = 32.6$ degrees and c = 34.5kPa (5 psi) as shown in Figure 6.

Finally, based on engineering experience and available literature (Hoek and Bray, 1981) 32 degrees was selected as the peak friction angle of the shale mass for the stability analysis (Figure 5). Additional calculations using the data presented by Mesri and Cepeda-Diaz (1986) indicate that the intact strength of Pierre Shale and Bearpaw Shale reduces 48% to 76% from peak to residual conditions. For Pierre Shale from Limon, Colorado the strength reduction was 48%. Strength reduction observed in the Ralston study was also [1-tan(18)/tan(32)=0.48] 48%, which is within the expected range of reduction values.

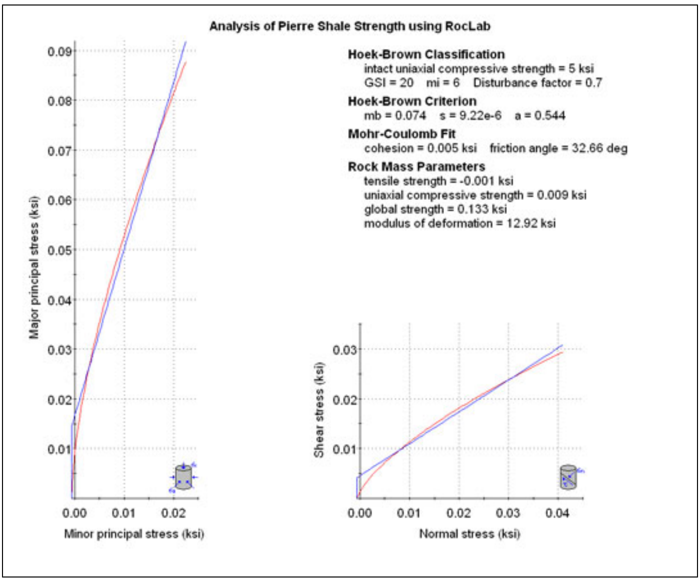

FIG. 6. Pierre Shale Strength Estimate from RocLab.

Fell and Jeffery (1987) recommended using the fully softened friction angle for fissured clays and indicated that the softened friction angle is generally equal to the peak friction angle. Stark and Eid (1997) confirmed that high plasticity (LL>50), stiff, fissured clays have the same peak and softened value, however, the mobilized friction angle for first time slide could be less (as low as the average of residual strength and fully softened strength). For the fissured Pierre Shale, the average value

(ϕ' = 25 degrees) was used in this study. This is consistent with our stability calculations with the exception of the low cohesion value c = 9.6 kPa (200 psf). Engineering properties for the stability analysis and the factor of safety (FOS) values with the stabilizing berm are provided in Figure 2b.

The stability analyses were performed using drained strength parameters. The low likelihood of elevated piezometric levels coinciding with pore pressure buildup within the non-saturated, over-consolidated clays present at the site did not warrant an undrained analysis.

MITIGATION MEASURES

Schuster (2006) classified mitigation procedures and applications for dams constructed over landslides into two categories, each with several possible mitigation options:

1) Abutment stabilization: removal of landslide deposit; flattening of abutment slope; earthfill or rockfill buttresses; dam serving as a buttress; concrete cutoffs or keys; retaining walls; anchors; gunniting; dental work.

2) Seepage reduction measures: impervious cutoffs; drainage systems; impervious curtains, membranes; and blankets.

He also mentioned relocating or reinforcing the spillway.

The spillway of Trinita Dam in Italy, Sicily, was built on piles on an existing flow-earth slide (Catalano et. al., 2000). Even though, the landslide reactivated in 1965 and 1981, it did not impact the functionality of the spillway.

For the Ralston Dam Spillway we utilized a rockfill buttress, a concrete cutoff, and a drainage system. In addition, the chute was reconstructed with reinforced concrete walls and floor supported by a grade beam and was founded on drilled piers. Mitigation measures for the Ralston Dam spillway are shown in Figure 1.

To design the rock buttress, we analyzed the stability of the existing slope (FOS = 1.0) with varying rock buttress configurations until a theoretical minimum factor of safety exceeding 1.5 was reached for the potential failure surfaces. Both shallow and deep non-circular slides were analyzed for the buttressed slope with elevated piezometric conditions. The results of the analyses, shown on Figure 2b, indicated that the theoretical minimum factor of safety for the proposed buttressed slope exceeded 1.5.

The recently constructed 32 meters (112 feet) long section of the spillway chute is supported by drilled piers and was constructed as a reinforced concrete grade beam (frame). This design allows the reconstructed section of spillway to support the deadload of the chute, support the soil loads against the walls, and resist uplift and swell pressures. The drilled piers are 46 cm (18-inch) diameter, spaced at 2.4-meters (8-feet), and are socketed a minimum 4.6 meters (15 feet) into intact claystone. Installation of the drilled piers is shown in Photo 1. The new spillway chute was designed as a structural frame supported by drilled piers. The internal dimensions at the new chute match the existing chute geometry and consist of a 3-meter-wide (10-foot) slab and sloped training walls approximately 3 meters (10 feet) high. The walls were designed to be 25 cm (10-inches) thick and the slab is 30 cm (12-inches) thick.

Drilled piers and the frame system are capable of supporting the chute in case there is additional movement at the slide plane. We checked and determined acceptable deflections in the piers with GROUP v.7 program (Ensoft, 2006) as shown in Figure 7.

4-foot deep reinforced concrete cut-off walls were also included in the chute reconstruction to reduce down-slope underseep and to increase sliding stability. 10-inch diameter, perforated HDPE underdrain pipes was placed under the chute as shown in Photo 2. The new underdrain pipes were connected to the existing 15 cm (6-inch) diameter clay-pipe drains at the upstream and downstream ends of the new spillway chute sections. The new underdrain pipe was connected to a new 30 cm (12-inch) diameter trench drain with a 25 cm (10-inch) diameter solid pipe.

The foundation for the new training walls was protected with a minimum of 5 cm (2 inches) of shotcrete within 24 hours of removal of the existing concrete walls. The spillway floor slab foundation was protected with 7.5 cm (3 inches) of shotcrete. The shotcrete was provided to protect the foundation surface from degradation resulting from weathering or construction activities.

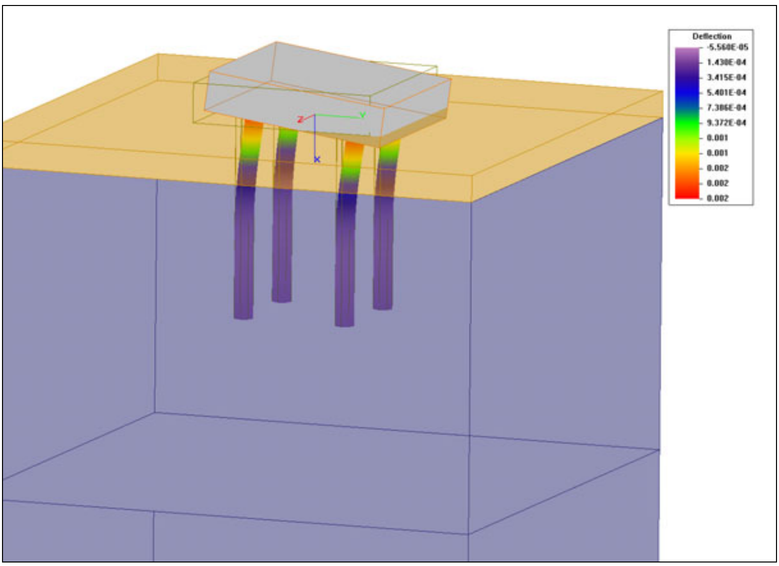

FIG. 7. Spillway Chute Slab Deflections

Photo 1. Drilled Pier Installation

Photo 2. Underdrain Replacement

OBSERVATIONS AND PERFORMANCE

First Trial

The first trial of the new spillway took place on September 6, 2006. Water flow was approximately 4000 lt/s (142 cfs) through the spillway. The spillway performed as expected with the underdrain pipes conveying approximately 76 to 114 liters (20 to 30 gallons) of clear water per minute. Water elevations at piezometers P-1 and P-2 rose 0.3 meter and 3 meter (1 foot and 10 feet), respectively. Geomembrane patches that were fixed at new spillway slab joints were ripped off by the hydrodynamic force of the flowing water. However, no damage or displacement was observed on the spillway slab.

Inspection Program

The Ralston Dam spillway stability is currently monitored by piezometers, inclinometers, and settlement monuments. Instruments are monitored and observations are made every two-weeks by the care taker. Visual observations of Ralston dam during the January and May 2008 inspection are provided in Photos 3 through 6. Inclinometer readings indicate no deflections.

Table 1. Time Event Log for Ralston Dam Spillway

Date	Time	Observances
06/09/06	0900 hrs	Water started over spillway for trade ditch.
06/09/06	1200 hrs	Approximately 142 cfs going over spillway.
06/09/06	1400 hrs	**New drain at bottom of spillway running approximately 10gpm (Water clear).**
06/09/06	1800 hrs	Drain at bottom of spillway running approximately 20 gpm. **(Water clear).**
06/09/06	2000 hrs	Drain at bottom of spillway running approximately 20 gpm. **(Water clear).**
06/10/06	0630 hrs	**Drain at bottom of spillway running approximately 30 gpm. (Water clear).**
06/10/06	1200 hrs	Drain at bottom of spillway running approximately 20 gpm **(Water clear).**
06/10/06	1830 hrs	**Patch put in by contractors ripped out.**
06/10/06	1900 hrs	Piezometer readings are at normal ranges.
06/11/06	1700 hrs	Piezometer readings are at normal ranges.
06/11/06	1800 hrs	Drain at bottom of spillway running approximately 20 gpm. **(Water clear).**
06/12/06	1800 hrs	Piezometer readings are at normal ranges.
06/12/06	1900 hrs	Drain at bottom of spillway running approximately 20 gpm. **(Water clear).**
06/13/06	1700 hrs	Piezometer readings are at normal ranges.
06/13/06	1800 hrs	Drain at bottom of spillway running approximately 20 gpm. **(Water clear).**
06/14/06	1930 hrs	**Piezometer #2 shows a 4 ft. rise in reading from normal, all other normal.**
06/14/06	2030 hrs	Drain at bottom of spillway running approximately 20 gpm. **(Water clear)**
06/15/06	1900 hrs	Water off of spillway.
06/16/06	1100 hrs	Drain at bottom of spillway running approximately 2 gpm. **(Water clear).**
06/16/06	1400 hrs	**Piezometer #1 shows a 1 ft. rise in reading from normal.**
06/16/06	1415 hrs	**Piezometer #2 shows a 10 ft. rise in reading from normal. All others normal.**
Note: Normal readings for piezometers were determined from most recent readings prior to water on spillway.		

Source: Ralston Dam Caretaker's Log 6/9/2006 – 6/16/2006 Denver Water.

Photo 3. Overview of Ralston Dam left abutment and spillway from the right abutment

Photo 4. Overview of the chute and repair area from the stilling basin


Photo 5. Repair area with no translation or deformation


Photo 6. Overview of the chute and repair area from the crest

CONCLUSION

The Ralston Dam Spillway had stability problems during its lifetime because of an existing slide within the Pierre Shale foundation. With modifications, including 1) a stabilizing berm with a toe trench that improves global stability, 2) seepage prevention measures that prevent the initiation of sliding, and 3) structural support of the chute with drilled piers and reinforced frames, the probability of future displacements was reduced for the Ralston Dam Spillway. This study also leads to the following observations:

1. Reverse shearbox test results do not necessarily provide the "true" (disaggregated) residual shear strength of clays and shales and tend to overestimate the residual strength (27% in this case). Other laboratory tests such as rotational shear test or back calculating the residual friction angle can be more reliable for existing slides.
2. Cycles of wetting can reactivate slides. Spillway chutes are prone to cracking and leakage at joints due to hydraulic forces, thermal cycles, and foundation movements. Older drainage control measures (clay tile underdrain systems) may be damaged and may not be able to sufficiently evacuate seepage and thus unable to prevent the saturation of the foundation materials.
3. Deep foundations can provide structural reinforcement in case all slide prevention measures such as drainage and buttress fail.
4. With improvements of the foundation materials, the Ralston Dam spillway has successfully performed without any problems over the last two years.

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Sinkhole Remediation and Geomembrane Lining of RCC Reservoir No.1

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Abstract

Rifle Correctional Center (RCC) Reservoir No. 1 is classified by the Colorado State Engineer's Office (SEO) as a Minor Size Low Hazard Dam. The dam is located on the western slope approximately ten miles north of the town of Rifle, CO. The reservoir was originally constructed in 1997 by excavating into the natural hillside on the eastern and northern side of the reservoir. The excavated soils were used to construct a non-zoned homogeneous earthen dam embankment on the western and southern sides of the reservoir. The reservoir impounds approximately 4.2 ha-m (34 acre-feet) of water used to augment Middle Rifle Creek and provide fire suppression water for the Rifle Correctional Center.

Upon initial filling unacceptable seepage out of the reservoir was observed. In 1998 soils in the cut areas of the reservoir were amended by mixing dry bentonite with the native soils in an attempt to reduce seepage. During normal reservoir operation desiccation cracking of the soils developed and the seepage was not reduced to an acceptable level. In 2006 a sinkhole was observed in the northern portion of the reservoir during a routine SEO dam inspection and a zero storage restriction was placed on the reservoir.

Geotechnical investigations determined that the dam and reservoir are founded on an alluvial fan deposit that is locally blanketed with coarse colluvial soils in the area of the sinkhole. Remediation of the sinkhole required the design and construction of a two stage soil filter. Due to the history of unacceptable seepage out of the reservoir

a geomembrane liner system was designed and installed to reduce the reservoir seepage losses to an acceptable level for normal operation of the reservoir.

The chosen lining material was a 1.5 mm (60 mil) Hypalon geomembrane liner underlain with a 0.45 kg (16 oz) geotextile fabric. The construction was completed in September of 2007. The reservoir is expected to be filled and return to normal operation for the 2008 water year.

Introduction

Rifle Correctional Center (RCC) Reservoir No. 1 is an off-stream water storage reservoir located on the Colorado Department of Correction's (CDOC) Rifle Correctional Center approximately ten miles north of Rifle, Colorado. The reservoir serves as an augmentation water reservoir and also will provide fire suppression water in the event of a fire at RCC. The augmentation water is returned to Middle Rifle Creek to restore depletions of water used by the facility. The site is bounded on the east by Country Road 219 and on the west by Middle Rifle Creek in Garfield County, Colorado. The goal of this project was to repair a sinkhole that developed in the reservoir and reduce excessive seepage so that the reservoir could be successfully operated as an augmentation and exchange reservoir as required for lawful operation by the CDOC.

Site Description

RCC Reservoir No. 1 is located in the mountains of western Colorado and is classified by the State Engineer's Office (SEO) as a Minor Size Low Hazard Dam, and has a reported capacity of approximately 4.2 ha-m (34 acre-feet) at the normal high water line of elevation 1857.5 meters (6094.0 feet). RCC Reservoir No. 1 is an off-stream reservoir impounded on the west and south sides by an earthen embankment dam approximately 385 meters (1,270 feet) long and approximately 6 meters (16 feet) high with a crest width of 3.5 to 4 meters (11 to 13 feet), and was originally constructed in 1997 as a non-jurisdictional dam. In 1999 improvements were made to the increase the capacity of the reservoir that required improvements to emergency spillway. These changes modified the reservoir to a jurisdictional reservoir as classified by the SEO. The upstream and downstream slopes of the dam are approximately 3:1 (horizontal to vertical). The reservoir is normally filled by diversions out of Middle Rifle Creek.

The reservoir was constructed by excavating into the natural hillside on the eastern and northern sides of the reservoir. The excavated soils were used to construct a homogeneous earthen dam embankment on the western and southern sides of the reservoir. The excavated (cut) slopes on the east are up to 9.75 meters (32 feet) high.

Project Background

Upon initial filling of RCC Reservoir No. 1 unacceptable seepage out of the reservoir was observed. In 1998 the soils in the cut areas of the reservoir were amended by mixing dry bentonite, a high swelling clay mineral rich soil, with the existing embankment soils in an attempt to reduce the seepage losses. However, during normal operation desiccation cracks developed in the reservoir during periods of little to no storage. As a result the seepage losses were not reduced to an acceptable level for normal reservoir operation.

During an SEO dam safety inspection in March of 2006, a sinkhole was observed in the bottom of the northern portion of the reservoir. Upon discovery of the sinkhole, the SEO placed a storage restriction on the reservoir until the sinkhole was investigated, analyzed and SEO approved repairs to the reservoir could be completed.

The engineering firm of Deere & Ault Consultants, Inc. (D&A) was retained by GMS, Inc. in September 2006 to develop a design for the rehabilitation of RCC Reservoir No. 1. The work included the peer review of the geotechnical investigation performed by Kumar and Associates (K&A); design recommendations for the sinkhole repair; design of a geomembrane lining system; design of reservoir improvements including a new staff gauge and erosion control improvements to the inlet rundown channel; preparation of construction documents, construction engineering and coordination with the SEO for design approval.

Geotechnical Investigations

K&A performed the first of two geotechnical investigations in November 2006. A total of eight test pits were excavated with a backhoe. Six of the test pits were excavated approximately 1.5 meters (5 feet) deep in the reservoir basin and two were excavated approximately 3.7 meters (12 feet) deep between the embankment and Middle Rifle Creek. Drive tube samples were collected in each of the test pits for soil testing and classifications. Near the sinkhole, the soils consisted of approximately 0.3 meter (1 foot) of sandy and clayey fill overlying clayey gravel. Along the embankment in the southern portion of the reservoir, the soils consisted of approximately 0.3 meters (1 foot) of sandy and clayey fill overlying sandy clay.

K&A performed a second geotechnical investigation in December 2006. Six exploratory borings were advanced to depths between 12.2 and 15.2 meters (40 and 50 feet) below the surface. Four of the borings were completed as permanent piezometers, two in the foundation soils for the embankment and two in the embankment soils. California liner samples and standard split-spoon samples were collected at regular intervals during the drilling for soil testing and classifications. The soils encountered in the bore holes consisted of approximately 0 to 25 feet of sandy clay fill overlying approximately 0 to 12.2 meters (0 to 40 feet) of sandy clay with zones of clayey gravel with sand. Bedrock was not encountered in the borings.

Various physical and engineering laboratory tests were performed on soil samples from the geotechnical investigations. Laboratory tests included: moisture content, dry density, grain size analysis, Atterberg Limits, and pinhole dispersion testing. The laboratory testing indicated that the site soils generally classify as low plasticity sandy clays and clayey gravels with sand.

Sinkhole Formation and Leakage

From the geotechnical investigations it was determined the reservoir and embankments are founded on an alluvial fan deposit, locally blanketed with coarse colluvial soils. The geotechnical investigations indicate the presence of high permeability colluvial soils under the north end of the reservoir. These colluvial soils coupled with the development of desiccation cracking of the bentonite-amended native soil liner, allowed at least one seepage path to develop, resulting in an unacceptable rate of seepage from the reservoir. A small sinkhole was observed in the northern portion of the reservoir where cobble and boulder sized colluvial deposits occured at shallow depths beneath the liner. A full reservoir on the clayey liner material on top of the coarse colluvial material apparently created a hydraulic gradient high enough to initiate internal erosion and material migration (piping), and the formation of the sinkhole.

Filter Analysis

To reduce the potential for material migration between incompatible foundation soils, K&A completed a filter analysis based on the soil gradations collected during the geotechnical investigations. Their analysis can be summarized as follows:

1. In subgrade areas of gravel, cobbles, and boulders with less than 30 percent by weight finer than 9.5 mm (3/8-inch) and less than eight percent finer than the No. 4 sieve, a two-stage filter will be required. An appropriate graded filter system can be constructed by placing 30.5 cm (12 inches) of CDOT Class A aggregate (slightly modified to allow <10 percent larger than 7.6 centimeters (3 inches) overlain by 15.2 centimeters (6 inches) of aggregate meeting an AASHTO M-6 gradation (modified to allow up to two percent -200). Both filter layers should be placed at 70 percent relative density and extended at least 10 feet in all directions beyond the gravel and cobble subgrade areas.

2. In subgrade areas where more than 30 percent by weight is finer than 9.5 mm (3/8-inch), less than eight percent finer than the No. 4 sieve, but with less than 15 percent finer than the No. 30 sieve, a single-stage filter should be sufficient. A 15.24 centimeters (6 inches) layer of aggregate meeting the AASHTO M-6 gradation noted above should be placed over the subgrade soils and compacted to 70 percent of relatively density.

Excavation for Construction of the Soil Filter

Slope Stability Analysis

A two-dimensional slope stability analysis utilizing the computer program UTEXAS3 was performed by K&A, modeling the maximum embankment section with an exposed liner system to verify the embankment meets minimum requirements of the SEO. A summary of the slope stability results is provided in the table below:

Loading Condition	Calculated Factor of Safety	SEO Minimum Factor of Safety
Long Term Steady State Full Reservoir Upstream	3.0	1.5
Pseudo-Static Earthquake Upstream	2.7	1.0
Rapid Drawdown Upstream	2.2	1.2
Long Term Steady State Full Reservoir Downstream	2.2	1.5
Pseudo-Static Earthquake Downstream	1.6	1.0

Based on these analyses, the embankment would meet or exceed the SEO slope stability criteria for the required loading conditions with or without an exposed geomembrane liner.

General Project Design

The scope of the design for the rehabilitation of RCC Reservoir No.1 included mainly these three design items:

1. A graded filter system to reduce the potential for internal soil erosion caused by the coarse and variable subgrade soils.

2. A liner system designed to reduce reservoir seepage losses to an acceptable level for the proposed reservoir use and operations.

3. Other reservoir improvements designed to enhance the reservoir operations, including a new sloping staff gauge and inflow rundown channel.

While there were several ways to reduce seepage and enhance the safety of RCC Reservoir No. 1, the installation of an exposed geomembrane liner was selected for the following reasons:

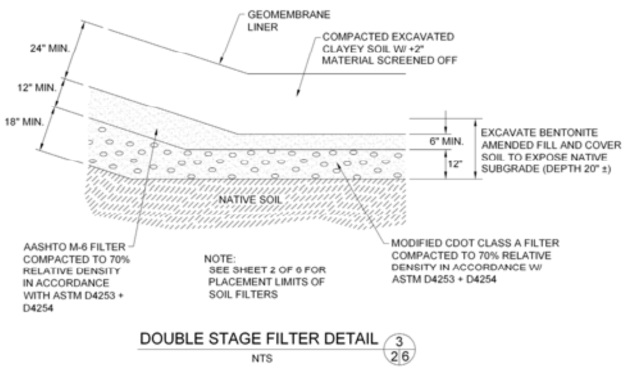
- The area of the reservoir was not large and could be lined economically.

- Embankment and cut slopes were generally gentle (3:1 horizontal to vertical or flatter).

- Natural low permeability soils for the construction of a typical compacted soil liner were not available on site in sufficient quantities.

- Access to the reservoir can be readily restricted to prevent liner damage from vehicles, foot traffic, animal damage, and vandalism.

Based on the geotechnical studies, internal erosion or piping of finer grained soils into and through the existing clayey gravel colluvium foundation soils beneath the reservoir appeared to be the primary source of seepage out of the reservoir. Prior to lining the reservoir, a graded soil filter was installed beneath the bentonite-amended soil liner and the geomembrane liner. In the event of damage to the primary geomembrane liner, the graded filter should prevent additional piping or sinkhole development in the reservoir basin.



Construction Detail of the Design Soil Filter

In addition to the two stage filter, no less than 15.2 cm (6 inches) of clayey soils was placed over the filter zones as a natural soil secondary liner. The material used for construction was the original bentonite-amended liner soils excavated from the reservoir basin to allow placement of the filter on the natural subgrade.

Filter Design

Design of a graded soil filter for RCC Reservoir No. 1 required dispersion and gradation testing. The dispersion tests evaluate if the soils are prone to piping. Gradation testing provides a basis for design of a graded filter soil that sufficiently decreases the gradient and reduces the migration of fine grained soils preventing the creation of a “pipe” or sinkhole.

The dispersion tests indicated the existing amended soil liner ranges from non-dispersive to slightly dispersive. This indicated the main factor in the sinkhole development was the incompatibility of the liner soils with the underlying coarse grained colluvial soils.

The alluvial fan and colluvial soils resulted in the deposition of an extremely variable subgrade soil matrix. During the geotechnical investigation the coarse soils in the northern portion of the reservoir were identified as the most likely cause of leakage out of RCC Reservoir No. 1. The variability of the soils made it difficult to design a single soil filter suitable for all the reservoir subgrade encountered in the geotechnical investigations. However, a filter system that is theoretically suitable for the most gap graded soil should provide suitable protection for the more well graded soils.

The initial filter design included areas of two stage soil filter construction as well as areas of single stage filter construction. In areas where the coarse subgrade soil areas had less than 30 percent by weight finer than 9.5 mm (3/8-inch) and less than eight percent finer than a No. 4 sieve, a two-stage filter was required for filter capability. The first stage consisted of 30.5 cm (12 inches) of CDOT Class A aggregate meeting the following gradation:

Sieve Size	% Passing by Weight
75 mm (3”)	90 - 100
19.0 mm (3/4”)	20 - 90
No. 4	0 - 20
No. 200	0 - 3

The second stage of the filter consisted of 15.2 cm (6 inches) of AASHTO M-6 filter material meeting the following gradation:

Sieve Size	% Passing by Weight
9.5 mm (3/8")	100
No. 4	45 - 100
No. 16	45 - 80
No. 50	10 - 30
No. 100	2 - 10
No. 200	0 - 2

In areas of subgrade characterized by finer soils with more than 30 percent finer than 9.5 mm (3/8-inch) by weight, greater than eight percent finer than the No. 4 sieve, but less than 15 percent finer than the No. 30 sieve, 15.2 cm (6 inches) of the single filter of AASHTO M-6 aggregates was considered adequate to meet filter criteria.

In order to simply the construction, the two stage filter was used to protect the entire northern portion of the reservoir as required to be necessary during the design of the sinkhole remediation.

Construction of Two Stage Soil Filter

Geomembrane Liner Design

There were numerous factors that were considered for selection of the type of geomembrane liner suitable for this project. Factors considered included:

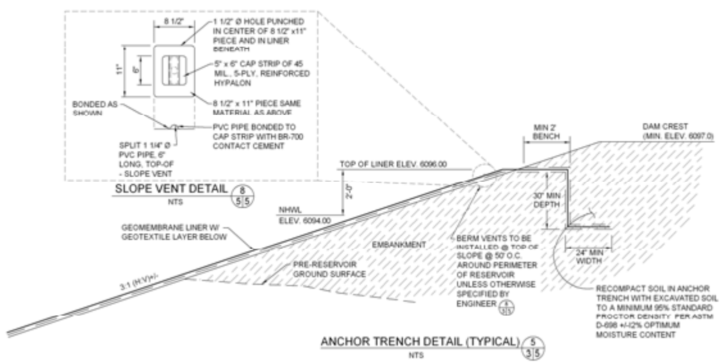
- Ease of installation and field seaming required during construction.
- Durability under environmental conditions including exposure and ice loading.

- Compatibility with proposed reservoir operations and ancillary facilities.
- Ease of repair.
- Highest level of assurance that the seepage was reduced.

These and other factors were discussed in depth with the client and consultants as well as representatives from several geosynthetic manufactures and contractors. Based on these discussions, 1.5 mm (60 mil) M 529 Hypalon, (Dupont's registered trademarked name of chlorosulfonated polyethylene), was selected for use as the geomembrane liner for RCC Reservoir No. 1. Criteria that made Hypalon the choice on this project included:

- The liner would be exposed: Hypalon is considered to be very durable when exposed to ultra-violet radiation.
- Hypalon has less expansion and contraction due to thermal variations than other geomembrane reservoir liner options evaluated.
- Hypalon is marginally more difficult to repair than other geomembrane liners as the surface may need to be pretreated in preparation of patch repairs, but can be patched using adhesives by trained staff of the CDOC. Abrasion resistance of the Hypalon material made it superior to a reinforced polypropylene geomembrane liner system for this application.

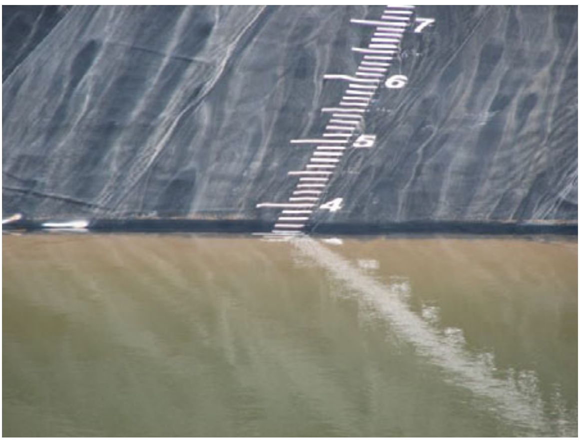
The liner system was constructed to extend two feet above the high waterline (HWL). The liner material was secured from pullout by laying the top of the geosythetic and cushion fabric along a horizontal bench with a width of 0.6 meters (2 feet) then into an anchor trench with dimensions of 0.75 meters (2.5 feet) deep by 0.60 (2.0 feet) wide. The liner on the platform and in the anchor trench was then buried with compacted clayey soils.



Prior to installation of the geomembrane liner, the subgrade of the reservoir basin and upstream embankment slope was prepared by removing sharp rocks, branches and trash prior to rolling with a smooth drum vibratory compactor. A 0.45 kg (16 oz) geotextile fabric between the subgrade and geomembrane liner was installed to help protect the liner from puncturing during construction and reservoir operations.

Reservoir Improvements

The original staff gauge installed prior to this project was designed to accommodate 18-inches of riprap that was never placed. The gap beneath the staff gauge and the embankment slope exposed it to damage resulting in reservoir level reading inaccuracies. As part of this project, a new staff gauge meeting SEO requirements, was installed on the geomembrane liner near the outlet works. Numbers and elevation "ticks" were cut out of white Hypalon material and permanently affixed to the black geomembrane liner. Whole numbers and elevation ticks at tenths foot increments were placed based upon surveyed elevations along the exposed liner slope.



Staff Gauge During Initial Fill of RCC Reservoir No. 1

The existing riprap rundown inlet channel located in the northeast corner of the reservoir was removed during the installation of the geomembrane liner. A new rundown channel was constructed from the Hypalon liner material in the same location. A 30.48 cm (12-inch) PVC pipe discharges from the Middle Rifle Creek diversion structure in the northeast corner of the reservoir. Diversion records indicate that a normal maximum diversion is approximately 0.01 cms (0.5 cfs). The new rundown channel consisted of a chute constructed of two 15.2 cm (6-inch) diameter sand filled ballast tubes and an extra layer of 1.5 mm (60 mil) Hypalon. The water will be discharged onto the extra layer of geomembrane and confined by the two ballast tubes. A stilling basin to reduce energy upon initial filling was included at the toe of the slope by adding sand filled Hypalon tubes perpendicular to the flow.

Reservoir and Embankment Monitoring

The geotechnical investigations and analysis indicate the sinkhole probably developed due to a vertical hydraulic gradient between the reservoir lining of bentonite amended soils and the coarse colluvium subgrade soils. Based on a review of the geotechnical data and the site conditions, the potential for total undermining of the dam embankment structure causing catastrophic failure with a sinkhole appears to be remote. However, recommendations were developed for regular embankment monitoring and include:

- Monthly measurement of the reservoir water level, piezometer water values, and toe drain flows prior to and through construction of the liner. Continued monitoring of the reservoir level, piezometers, and toe drain discharges during reservoir filling and on through normal operations.

- Monthly visual inspection of the toe drain for cloudy or murky water, as well as visually note the general condition of the embankment slopes documenting any cracks or sloughs.

- Measurement of groundwater levels during operation of the reservoir to verify groundwater levels are lower than the level of water stored in the reservoir. If groundwater levels rise above the water level in the reservoir, additional liner ballasting may be required.

- Annually complete a thorough inspection of the liner while the reservoir is at the lowest pool level to observe any significant damage and perform any maintenance and repairs necessary to maintain the integrity of the geomembrane liner.

RCC Reservoir No. 1 with Hypalon Liner Installed

Conclusions

The sinkhole repair and relining of RCC Reservoir No. 1 was successfully completed in time to fill the reservoir and meet mandated fire protection water requirements for the Rifle Corrections Center facilities and in preparation for use as augmentation water for the 2008 water year. To date, seepage out of the reservoir has been within acceptable levels as determined by discussions with the Hypalon manufacturer and installer and operational minimums set by the CDOC to meet augmentation requirements. The CDOC is proceeding with improvements to the RCC water delivery system that will benefit from the rehabilitation of RCC Reservoir No. 1.

The improvements to RCC Reservoir No. 1 should satisfy the design criteria of reduction of seepage losses, providing addition protection against internal erosion of the liner soils, while increasing the overall stability and operational control of the reservoir. The soil filter protects the structure from internal erosion due to piping and the geomembrane liner helps reduce seepage and allows the reservoir to operate as intended.

RCC Reservoir No. 1 Partially Full in Fall of 2007

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Transient Seepage Analyses of Soil-Cement Uplift Pressures During Reservoir Drawdown

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ABSTRACT: The south Denver metro area is experiencing significant growth and development, which has resulted in an increased demand for residential and commercial water. A new 61-meter-high (200-foot-high) zoned earthen dam is currently being constructed to provide an additional 9.2 x 10^7 m^3 (75,000 acre-feet) of water storage. Upstream slope protection will consist of soil-cement, which is not free-draining like riprap. During periods of sustained reservoir drawdown, uplift pressures could occur on the soil-cement if the reservoir is lowered faster than water within the upstream shell can drain. Damage to the slope protection could occur if the uplift pressures exceed the dead weight of the soil-cement. A steady state seepage analysis cannot model the changing boundary conditions associated with reservoir drawdown. This paper describes transient seepage analyses that we performed to evaluate possible uplift pressures on the soil-cement. During the transient seepage analyses, we developed unsaturated hydraulic conductivity functions and non-linear transient boundary conditions, and investigated the model sensitivity to its inputs. Parametric analyses indicated that the expected uplift pressures are highly sensitive to the hydraulic conductivity of the upstream shell and drainage provided to it. The model results were used to design a drainage system within the upstream shell that is expected to prevent the development of excess uplift pressures beneath the soil-cement slope protection.

INTRODUCTION

Significant growth and development in the south Denver metro area has resulted in an increased demand for municipal water. Rueter-Hess Dam and Reservoir in Parker, Colorado is currently under construction, and when completed will consist of a 61-meter-high (200-foot-high) zoned earthen dam that will provide 9.2 x 10^7 m^3 (75,000 acre-feet) of water storage. Upstream slope protection will consist of soil-cement, which is not free-draining like riprap. Uplift pressures could occur on the soil-cement during periods of sustained reservoir drawdown if the reservoir is lowered faster than water within the upstream shell can drain. Damage to the slope protection could occur if the uplift pressures exceed the dead weight of the soil-cement. To accurately model the changing boundary conditions associated with reservoir drawdown, we performed transient seepage analyses using SEEP/W© (Krahn, 2004) to evaluate uplift pressures on the soil-cement. This paper describes the development of the transient seepage models, our interpretation of model results, and the resulting design of a drainage system that is expected to prevent the development of excess uplift pressures beneath the soil-cement.

BACKGROUND

Rueter-Hess Dam is a zoned embankment with a central/sloping core, a granular upstream shell, and upstream and downstream stability berms. Soil-cement slope protection will extend from the dam crest to the upstream stability berm. A representative cross section used in our analyses is shown on Figure 1. This analyzed section consists of the upper upstream portion of the maximum embankment section. Material specifications for Zones 2 and 2A fill are presented in Table 1 and gradations for Zones 2 and 2A are shown on Figure 2. The Zone 2 gradation shown on Figure 2 is a representative average gradation for this material zone, while the Zone 2A gradation used for our analyses was conservatively considered as being near the fine end of the specification.

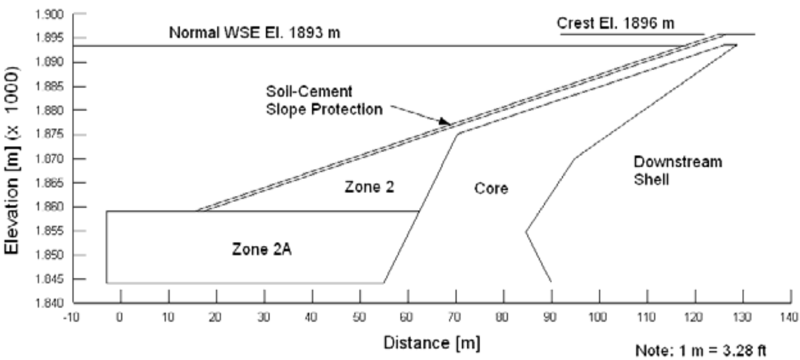

FIG. 1. Analyzed Section.

Table 1. Specifications for Zones 2 and 2A Fill

Attribute	Zone 2 (Upstream Shell)	Zone 2A (Upstream Berm)
Maximum Fines Content (P_{200}) (%)	12	30
Maximum Plasticity Index (PI) (%)	10	15
Maximum Particle Size (cm)	15 (6 in.)	15 (6 in.)

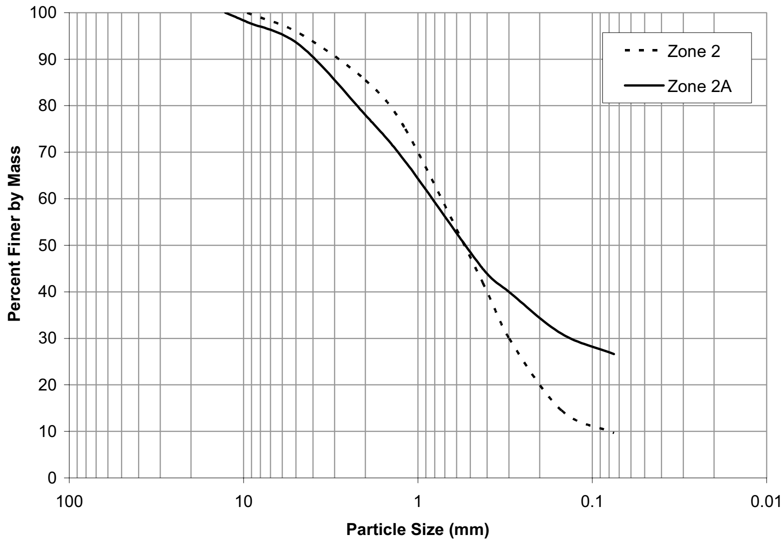

FIG. 2. Gradations for Zones 2 and 2A Fill.

The design flow rate of the Rueter-Hess outlet works conduit was originally 1.4 m^3/s (50 ft^3/s). Initial transient seepage analyses were performed to support design of a toe drain system that would alleviate the excessive soil-cement uplift pressures associated with this drawdown rate. After these initial seepage analyses had been performed and soil-cement construction had begun, the project design criteria were modified to increase the design flow rate to 8.8 m^3/s (310 ft^3/s). We then performed additional transient seepage analyses using this higher drawdown rate to design a mid-level drainage system.

The following section describes the development of material properties and boundary conditions required for transient seepage analyses, and modeling procedures.

MODEL INPUTS AND PROCEDURE

Material Properties

Saturated Hydraulic Conductivity and Anisotropy Ratio

The horizontal saturated hydraulic conductivity (k_h) for the Zone 2A was based on falling head tests (ASTM D 5084, Method C) performed on five specimens of Zone 2A fill that were compacted using modified Proctor energy (ASTM D 1557). No hydraulic conductivity tests were performed on Zone 2 fill, but three tests were performed on the filter drain material, which has about 3 percent fines and conforms to the requirements for ASTM C 33 fine aggregate. Since the gradation requirements for Zone 2 (Table 1) are intermediate between those of Zone 2A and the drain material, we assigned the Zone 2 a saturated hydraulic conductivity between the design values used for the Zone 2A and the drain. Horizontal hydraulic conductivities for Zones 2 and 2A and the drain material are presented in Table 2.

Table 2. Hydraulic Conductivity Values

Material	Range of Laboratory Hydraulic Conductivities (ASTM D 5084 Method C) (cm/s)	Design Horizontal Hydraulic Conductivity, k_h (cm/s)
Zone 2 (upstream shell)	--[a]	2.0×10^{-2} [b]
Zone 2A (upstream berm)	2.4×10^{-4} to 1.4×10^{-3}	8.0×10^{-4}
Drain material	4.7×10^{-2} to 8.3×10^{-2}	7.0×10^{-2}

Notes:

a. No laboratory tests were performed on Zone 2 material.
b. Design conductivity for Zone 2 is intermediate between those for Zone 2A and the drain material.

Zones 2 and 2A were both assigned anisotropy ratios of 0.25, which is the ratio of the vertical to horizontal saturated hydraulic conductivities (k_v/k_h). This value is in agreement with previously published values for compacted embankment shells (USBR, 1987).

Unsaturated Hydraulic Conductivity Functions

Since drainage of the upstream shell during drawdown conditions will include unsaturated flow above the piezometric surface, we developed hydraulic conductivity functions for Zones 2 and 2A to more accurately model the unsaturated flow. These functions represent decreases in hydraulic conductivity as the degree of saturation decreases.

SEEP/W© automatically generates unsaturated hydraulic conductivity functions for user-specified saturated hydraulic conductivities and soil water characteristic curves. A soil water characteristic curve depicts the decrease in degree of saturation as matric suction increases (Lu and Likos, 2004).

We used empirical relationships presented by Yang et al. (2004) to estimate soil water characteristic curves using the model by Fredlund and Xing (1994). The Fredlund and Xing (1994) model of a soil water characteristic curve requires the empirical curve fitting parameters *a*, *n*, and *m* to relate volumetric water content to suction within a soil. Empirical relationships presented by Yang et al. (2004) allow for the estimation of these parameters from the slope and D_{10} of a soil's grain size distribution curve. Soil water characteristic curves that we estimated for Zones 2 and 2A are shown on Figure 3. The hydraulic conductivity functions for Zones 2 and 2A that were developed from these soil water characteristic curves and the respective saturated hydraulic conductivities are presented on Figure 4.

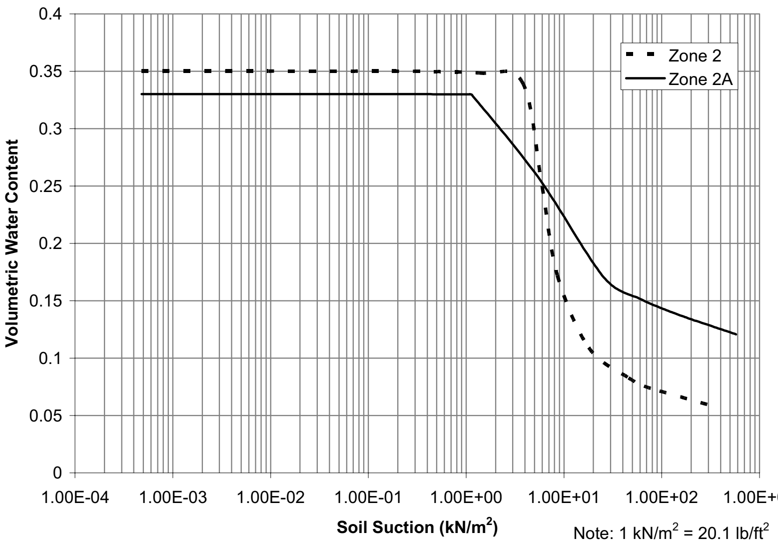

FIG. 3. Soil Water Characteristic Curves for Zones 2 and 2A.

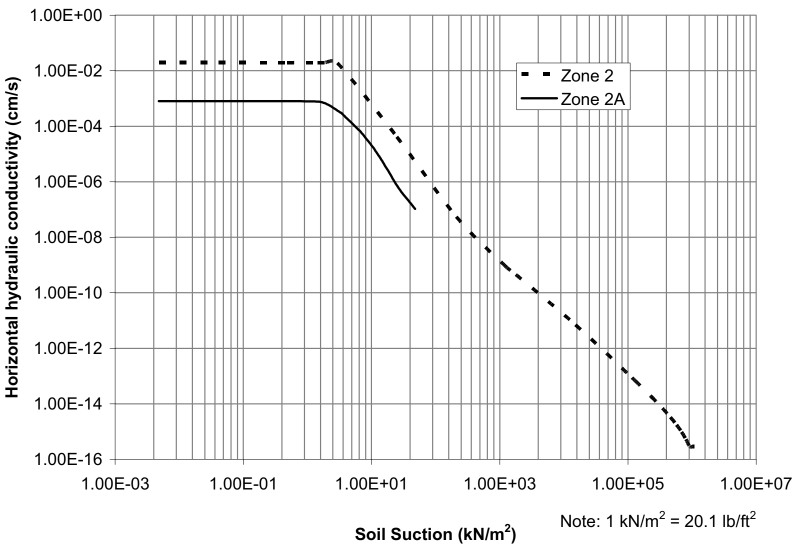

FIG. 4. Hydraulic Conductivity Functions for Zones 2 and 2A.

Boundary Conditions

Two sets of boundary conditions are required to perform a transient seepage analysis: an initial boundary condition and a transient boundary condition.

Initial Boundary Condition

An initial boundary condition is required to define pore water pressures throughout the model at the beginning of the transient analysis. The initial boundary condition in our models applied a total head on the upstream embankment slope equal to the reservoir level at the initiation of drawdown, and all other model edges were modeled as no-flow boundaries. This created an initial hydrostatic pressure distribution throughout the model that we felt reasonably represented the pore water pressures that would exist upstream of the core under steady state seepage. We analyzed uplift pressures generated by drawdown from both the normal pool (El. 1893 m (6212 ft)) and an intermediate pool (El. 1884 m (6180 ft)).

We also investigated the initial boundary condition of allowing steady state seepage through the core. Results were not significantly different from those obtained from the hydrostatic initial boundary condition, and thus the hydrostatic case was used for simplicity.

Transient Boundary Condition

Transient boundary conditions were created to model confined flow through Zones 2 and 2A (Figure 1). Locations where water was allowed to leave the model (the surface of the Zone 2A berm and drain locations) were assigned head functions (total head versus time) to simulate drawdown of the reservoir. These head functions were developed from the reservoir's elevation-capacity curve and the design flow rates from the outlet works. The head functions used for 1.4-m^3/s (50-ft^3/s) and 8.8-m^3/s (310-ft^3/s) discharge from the normal pool (El. 1893 m (6212 ft)) are shown on Figure 5. The core and soil-cement were assumed to be relatively impermeable, and thus were modeled as no-flow boundaries.

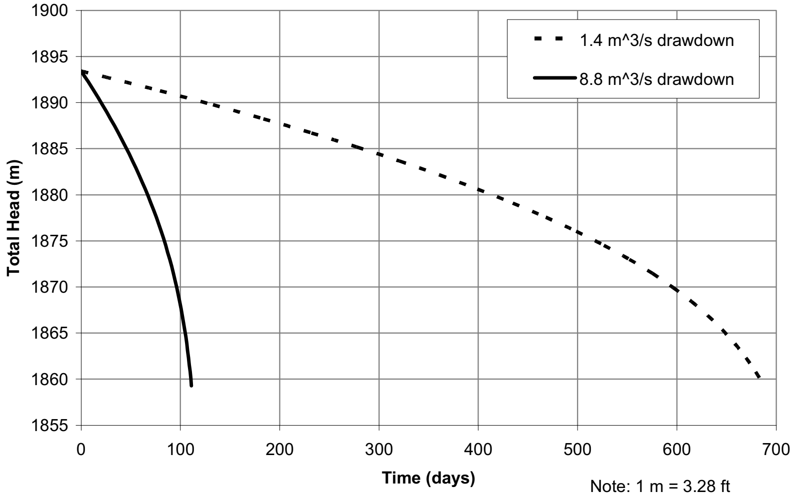

FIG. 5. Head Functions for 1.4-m^3/s (50-ft^3/s) and 8.8-m^3/s (310-ft^3/s) Drawdown from the Normal Pool (El. 1893 m (6212 ft)).

Finite Element Mesh

We used an unstructured triangular finite-element mesh as shown on Figure 6 because of the geometry of the Zone 2 region. This mesh was developed so that nearly all elements would be equilateral or isosceles right triangles, which are highly desirable to improve computational accuracy (Krahn, 2004).

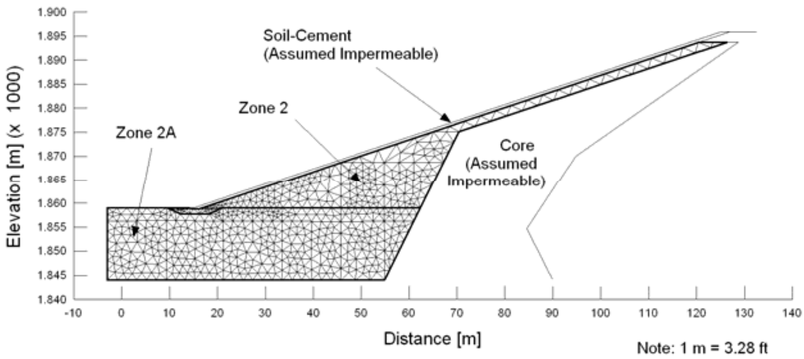

FIG. 6. Modeled Finite-Element Mesh.

Modeling Procedure

Model set-up consisted of meshing the regions as shown on Figure 6, assigning the material properties discussed above to the regions, and assigning the boundary conditions discussed above to the appropriate nodes. Time steps within the transient analysis were then adjusted until acceptable water balance errors were obtained. In our opinion, a water balance error less than approximately 0.85 m^3 (30 ft^3) would be acceptable, which is approximately 0.2 percent of the meshed cross sectional area.

Once a model with an acceptable water-balance error was developed, Node Information was viewed within SEEP/W© to identify the pore water pressures at each time step for each node located along the upstream slope of the Zone 2 (base of the soil-cement). These pressures are only those existing within the soil, and thus we subtracted the pressure obtained at each node from the hydrostatic pressure within the reservoir at each particular node elevation and time increment in order to obtain the net uplift pressure on the soil-cement slope protection.

MODEL RESULTS AND INTERPRETATIONS

The soil-cement slope protection on the 3H:1V upstream slope was designed as a series of horizontal steps, but it will be shown in this paper's figures as a 0.9-meter-thick (3-foot-thick) plate for simplicity. During interpretation of the model results, we assumed that the soil-cement could withstand a net uplift pressure equal to its slope-normal component of dead weight, which is 18.2 kPa (380 lb/ft^2).

Initial Transient Seepage Analyses

The initial analyses were performed early in the design process using the design discharge of 1.4 m^3/s (50 ft^3/s) to support design of a toe drain at the base of the Zone 2.

Model Results

The results of net uplift pressure versus position within the embankment for different durations of sustained 1.4-m^3/s (50-ft^3/s) drawdown are shown on Figure 7. Figure 7 shows that the maximum net uplift pressure on the base of the soil-cement would be about 11 kPa (230 lb/ft^2) after 240 days of sustained 1.4-m^3/s (50-ft^3/s) drawdown if a 0.9-m-thick (3-ft-thick) toe drain was included at the bottom of the soil-cement.

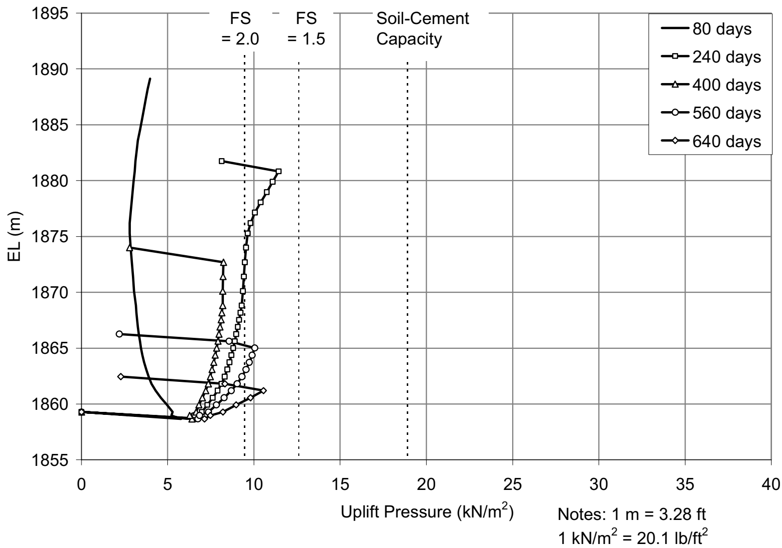

FIG. 7. Uplift Pressure versus Elevation for Various Times of Sustained 1.4-m^3/s (50-ft^3/s) Drawdown from the Normal Pool with a Toe Drain and Using Design Hydraulic Conductivities for Zones 2 and 2A.

Parametric analyses were also performed to investigate model sensitivity. We investigated changes in predicted uplift pressures to changes in saturated hydraulic conductivities of Zone 2, the inclusion or omission of the toe drain, and the thickness of the toe drain. These analyses indicated that the predicted uplift pressures were most sensitive to the inclusion or omission of a toe drain, but increasing the thickness of the drain had minimal impact on the predicted uplift pressures. Predicted uplift pressures under 1.4-m^3/s (50-ft^3/s) drawdown, if no toe drain is provided, are shown on Figure 8. If no drain is included, uplift pressures were predicted to approach the soil-cement capacity for even short drawdown durations, and uplift pressures exceeded the soil-cement capacity for sustained drawdown durations longer than 400 days.

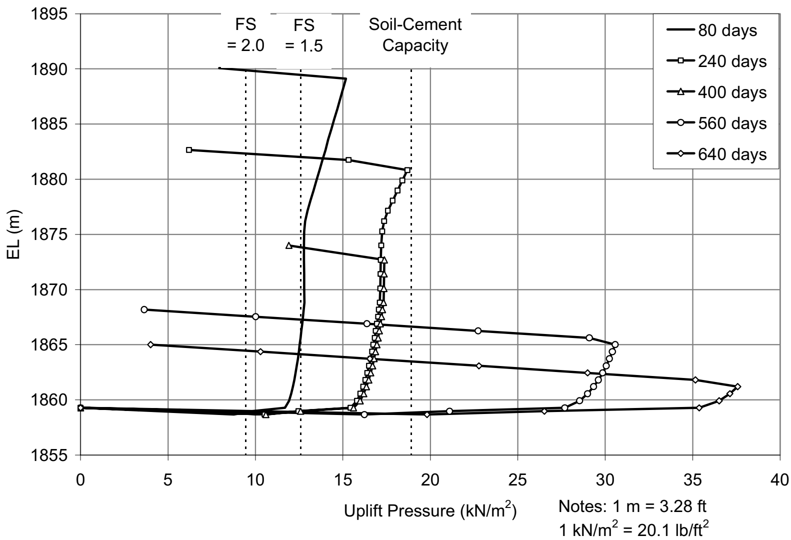

FIG. 8. Uplift Pressure versus Elevation for Various Times of Sustained 1.4-m^3/s (50-ft^3/s) Drawdown from the Normal Pool without a Toe Drain and Using Design Hydraulic Conductivities for Zones 2 and 2A.

Interpretation of Model Results

From the results of these initial analyses (Figures 7 and 8), we concluded that a 0.9-m-thick (3-ft-thick) toe drain consisting of Zone 2 material should extend below the soil-cement and daylight on the upstream berm, as shown on Figure 9. This drain would provide a safety factor of approximately 1.65 against soil-cement uplift failure for sustained drawdown rates of 1.4 m^3/s (50 ft^3/s) (Figure 7). The seepage analyses indicated that exit gradients within the toe drain would be less than 0.4 and exit velocities would be less than 1.5 m/day (5 ft/day), and thus we felt that a weighted filter would not be required above the Zone 2 toe drain.

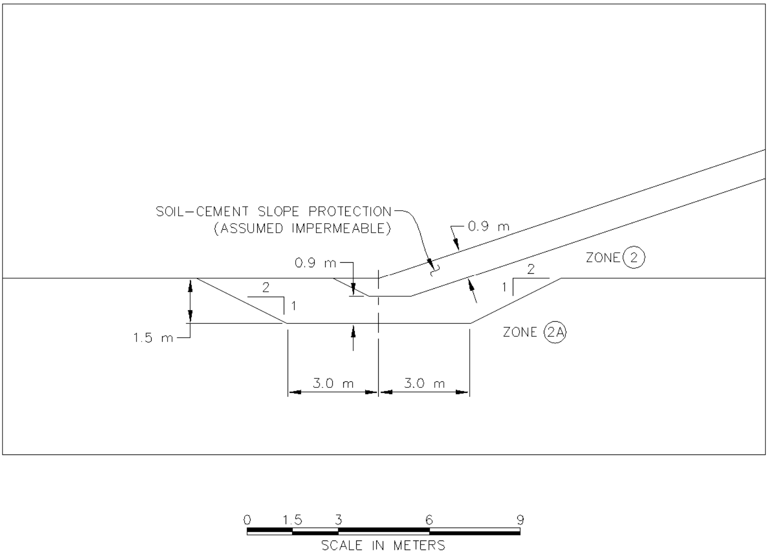

FIG. 9. Toe Drain Design.

Subsequent Transient Seepage Analyses

Subsequent transient seepage analyses were required after the outlet works design discharge had been increased to 8.8 m^3/s (310 ft^3/s). By this time, the toe drain shown on Figure 9 had already been constructed and soil-cement had been placed to approximately El. 1877 m (6159 ft). These analyses investigated whether or not an additional mid-level drain near the top of the completed soil-cement (approximately El. 1877 m (6159 ft)) would be required to relieve uplift pressures under the increased drawdown rate. This mid-level drain was modeled as a node with the head-function boundary condition for 8.8-m^3/s (310-ft^3/s) drawdown that is shown on Figure 5.

Model Results

Using the design hydraulic conductivities presented in Table 1, model results indicated that the maximum uplift pressures would be not be significantly reduced by including a mid-level drain. Analyses were also performed using saturated hydraulic conductivities that were approximately half an order of magnitude lower than those reported in Table 1 for Zones 2 and 2A. The lower conductivity for Zone 2A is comparable to the lowest test result (ASTM D 5084 Method C). Predicted uplift pressures using these lower conductivities and only allowing drainage through the toe drain are shown on Figure 10. Figure 11 shows the predicted uplift pressures if a mid-level drain is included near El. 1877 m (6159 ft).

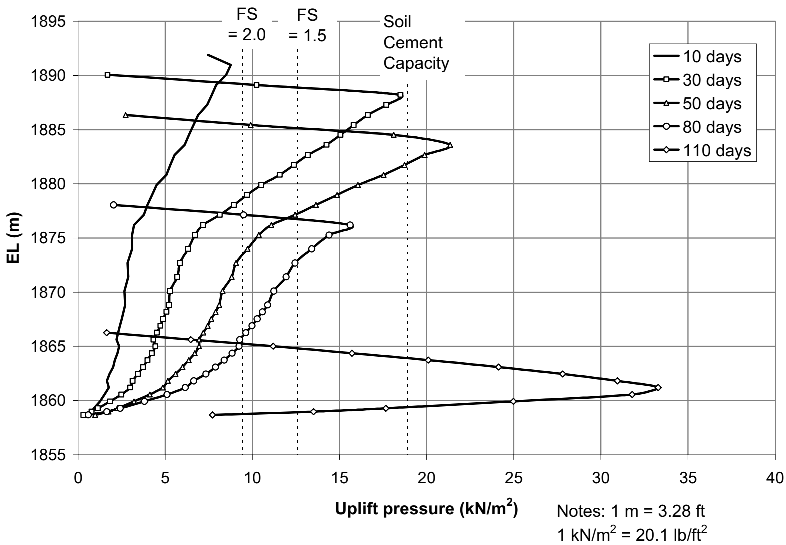

FIG. 10. Uplift Pressure versus Elevation for Various Times of Sustained 8.8-m^3/s (310-ft^3/s) Drawdown from the Normal Pool with no Mid-Level Drain and Using Lower Hydraulic Conductivity Values for Zones 2 and 2A.

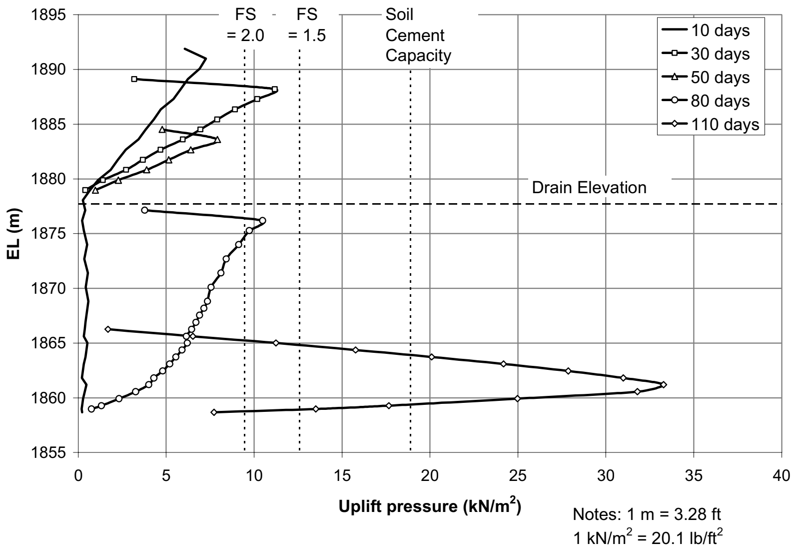

FIG. 11. Uplift Pressure versus Elevation for Various Times of Sustained 8.8-m^3/s (310-ft^3/s) Drawdown from the Normal Pool with a Mid-Level Drain and Using Lower Hydraulic Conductivity Values for Zones 2 and 2A.

Interpretation of Model Results

From the model results discussed in the previous section and shown on Figures 10 and 11, we concluded the following:

- If the design hydraulic conductivity values for Zones 2 and 2A (Table 1) are accurate representations of the in-situ materials, the toe drain provides sufficient drainage and there is no justification for installing a mid-level drain.
- If the in-situ Zones 2 and 2A have conductivities approximately equal to the lower values that were analyzed, installing a mid-level drain provides a significant increase in the factor of safety against soil-cement uplift failure for sustained 8.8-m^3/s (310-ft^3/s) drawdown that lasts up to 80 days.
- Installing a mid-level drain will not improve the factor of safety against soil-cement uplift failure for sustained 8.8-m^3/s (310-ft^3/s) drawdown that lasts longer than 80 days (reservoir drawn down from El. 1893 m (6212 ft) to El. 1876 m (6155 ft)), and soil-cement failure is expected to occur after 110 days of sustained drawdown.

Based on the model results, it was our opinion that it was prudent to include a mid-level drain to provide redundancy, facilitate drainage, and reduce soil-cement uplift pressures for the following reasons:

- Zone 2A laboratory test results indicate that its conductivity may be as low as the lower-conductivity value that was modeled.
- Vertical drainage through the Zone 2 will be controlled by the lowest conductivity of a horizontal lift (Cedergren, 1989)
- Siltation on the berm surface, which was not included in any model, could reduce the capacity of the toe drain.
- Model results indicated that soil-cement uplift pressures induced by a short-duration 8.8-m^3/s (310-ft^3/s) drawdown from an intermediate pool elevation (El. 1884 m (6180 ft)) were higher than those caused by the same duration drawdown from normal pool.

MID-LEVEL DRAIN DESIGN

Since the mid-level drain had been modeled as a node with a head-function, which has an unlimited flow capacity, additional analyses were required to support the design of the actual drainage system. We designed the drain to convey the maximum predicted flow at this location, which is 6.3×10^{-6} m^3/s per lineal meter (6.8×10^{-5} ft^3/s per lineal foot) of embankment and occurs after 70 days of sustained 8.8-m^3/s (310-ft^3/s) drawdown from the normal pool. The resulting drain is shown on Figure 12 and consists of a continuous slotted 15-cm (6-inch) PVC pipe encapsulated in a gravel pack beneath the soil-cement at approximately El. 1877 m (6159 ft). A solid 15-cm (6-inch) PVC pipe is provided every 200 meters (650 feet) and will extend through the soil-cement and discharge on the upstream embankment slope. This system is filter compatible with the Zone 2 material and provides sufficient drainage capacity without exceeding 3/4-full flow in the pipes.

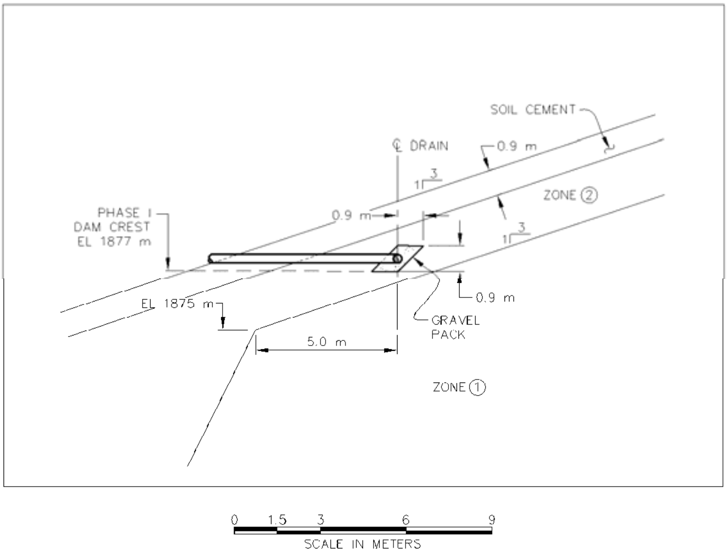

FIG. 12. Mid-Level Drain Design.

CONCLUSIONS

This paper described transient seepage modeling that was used to predict uplift pressures beneath soil-cement slope protection during sustained periods of reservoir drawdown. Model results were used to design a drainage system that is predicted to provide an adequate factor of safety against soil-cement uplift failure. Construction costs for the drain are anticipated to be approximately \$270,000, which is a small investment in comparison to the \$6.5-million soil-cement that it is designed to protect.

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CONSIDERATIONS FOR DETECTION OF INTERNAL EROSION IN EMBANKMENT DAMS

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ABSTRACT

The paper presents considerations to develop monitoring systems for internal erosion (IE) detection in embankment dams. The need for, and location of, instrumentation systems within a dam must be evaluated by a risk analysis, which develops potential failure modes and detailed information on the concerns leading to those failure modes. Important considerations include:

- Characteristics of the embankment materials that may be sensitive to IE.
- Detection of seepage concentrations in sections parallel to the embankment profile.
- In areas of concentrated seepage flows, detailed information of the ground water characteristics should be obtained using state of the art instrumentation.
- Proper monitoring procedures for the seepage collected by drain system.
- Design of data collection systems to obtain continuous readings for long term monitoring.

References of where these specialized instruments have been placed and successfully monitored are included.

1. INTRODUCTION

Internal erosion is a process that develops concentration of seepage flows and in time may develop large cavities that may produce embankment failure with a catastrophic and uncontrolled release of the reservoir.

Embankments at the verge of failure or under failure due to IE were detected by visual inspection most of the times and seldom by instrumentation, because the instruments were not placed in the right place or missed the critical readings. However, after stress signs are detected or suspected installation of additional instrumentation is highly effective and common practice in Reclamation to monitor the performance of the embankments during and after remedial work. Before enhancement of a monitoring system, a risk analysis should be performed to provide vital information for a proper design, which should comply with the risk analysis concerns that were identified by the failure modes. The concerns are the potential deficiencies that may have developed during construction and performance.

It should be noted that most of the monitoring systems on existing embankments were not designed to detect IE, and when used, they were installed to determine water levels for stability reasons or to gain other insights about embankment performance. Those instruments were unable to detect particle migration that may lead to deficiencies because of their own limitations and perhaps the frequency of readings.

It appears that to detect internal erosion, an instrumentation system has to be efficient and cover large areas as a typical visual inspection does. A toe drain system is an example of covering covers large areas, and most of the times the toe drains have detected internal erosion development, but they do not give information of the extent and location of the IE. The seepage flow collected by the toe drain system with continuous monitoring can give quick information on pH, temperature, and quantities of seepage flow, which can give useful clues to the performance of the soil and can, expand future studies on the IE consequences.

An instrumentation system has to have an effective combination of instruments, and placed in areas where concentrated flows are suspected. Those areas can be located by geophysics surveys such as with an active thermal survey using fiber optics or resistivity survey according to the concerns identified by the risk analysis; both survey methods should be capable to operate with continuous readings for long periods.

To obtain detailed information of the concerns, the monitoring programs should utilize instruments with multiple purposes, and be placed in sequence of surveillance coverage, from large areas to critical areas. The instrumentation should include multipurpose piezometers, ultrasonic acoustic sensors, geophysics surveys, flow analyzers and data collection systems.

2. EMBANKMENT DAMS

Embankment dams are earthfill structures built with various types of materials designed for storing large volumes of water for reservoirs. The reservoir is used mainly for flood control, irrigation, and power generation. Seepage occurs through the embankment and foundation. Thus, it is important to understand the mechanics of the internal erosion that may occur in the embankment and foundation. Also, their post construction performance should be reanalyzed to improve the monitoring system if required.

3. INTERNAL EROSION

Internal erosion is the result of forces acting on the soil particles and the soil response to the seepage flow.

FORCES ACTING IN SOIL PARTICLES. Figure 1 presents the forces acting at a particle level. The sketches show the external loading force and the contact forces, the buoyant weights and the viscous drag developed by seepage flow, the capillary forces and electrical forces that define the soil performance.

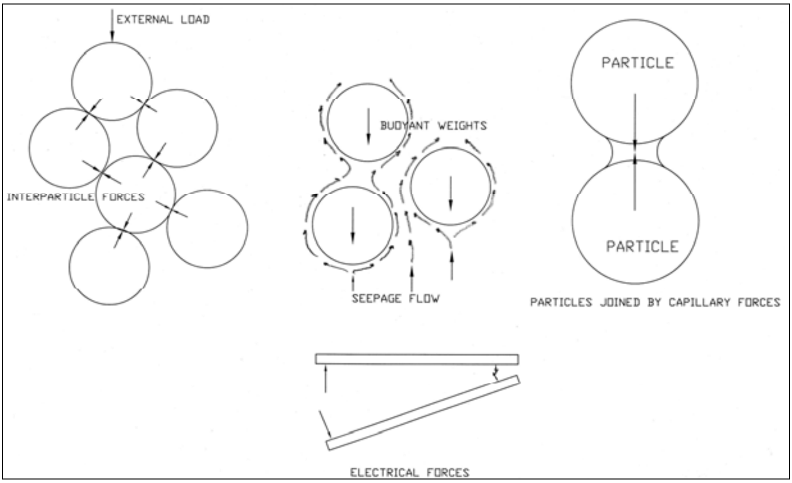

Figure 1. Forces acting on soil particles

3.2 SOIL RESPONSE TO SEEPAGE. The soil response to the effects of seepage flow is manifested by particle detachment and particle transport. Solutioning of soil particles is not included in this paper.

Particle detachment is the separation of a particle from the soil mass. The ease of separation depends on the physical and chemical properties of the soil, as well as seepage forces acting on the particle.

Particle transport depends on the hydraulic shear stress developed by the eroding seepage water and the seepage path conditions. IE starts with the colloidal-size particles, for which detachment and transport are more intense in the dispersive clay group. The particle transport becomes a concern when the erosion rate of the soil is significant and visible. The erosion rate depends mainly on the hydraulic shear stress, and the physical characteristics of the voids and pores along the seepage path that may lead to plugging and/or unplugging conditions. Also, of course, the magnitude of the hydraulic shear forces has the largest influence on the outcome of the transport.

3.3 TYPES OF INTERNAL EROSION. The basic types of IE are manifested in two forms: suffusion and piping. Practitioners on this subject have indicated several types of internal erosion. However, these types are just based on location and degree of IE intensity. Mc Cook (2004) presents a well documented explanation of piping and IE failure modes.

Suffusion, also known as internal instability, involves movement of fine particles by seepage forces acting through the constrictions, voids and pores between larger particles. This erosion may leave behind an intact soil skeleton formed by the coarse soil particles, or may collapse. Thus, suffusion may be described as the result of internal instability and IE. Seepage flow through embankment cracks produced by mechanical forces and large pore spaces can produce large voids or cavities**. Figures 2 to 3** show the basic concepts that will define the unavoidable suffusion effects, which may vary from negligible to catastrophic. Figure 2 shows: (a) Individual clay platelet group arrangements, (b) Individual silt or sand particle arrangements, (c) Clothed silt or sand particle arrangements and (d) Clay platelet group arrangements. Mitchell (2005), show more information of the particle arrangements, assemblages of the particles and the type of voids, in pages 111 and 112

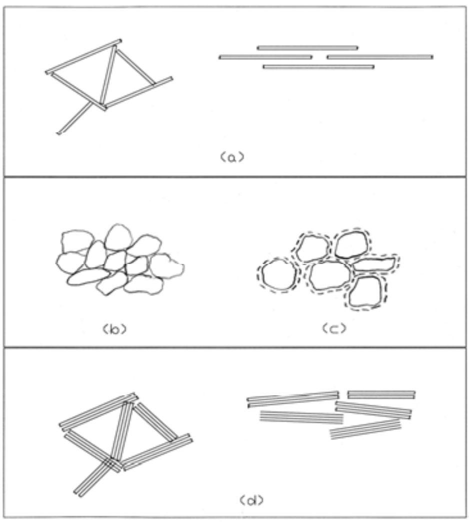

Figure 2 Schematic representations of particle arrangements

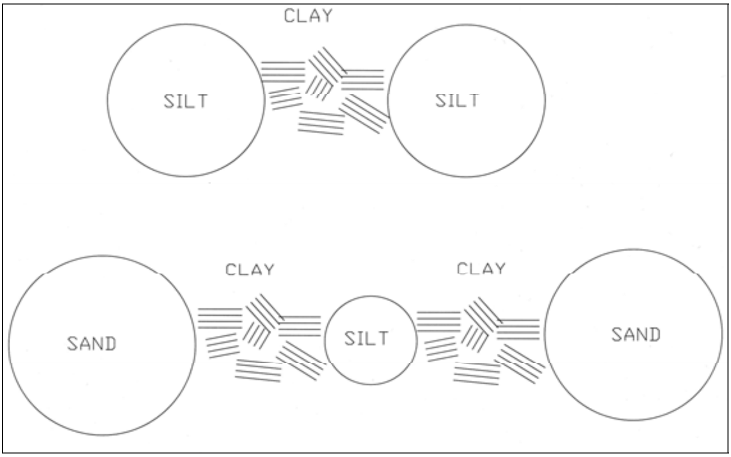

Figure 3. Schematic representations of particle assemblages

Figure 3 shows the basic composition of connectors between silts and silt/sand particles. Combinations of regular and irregular aggregations grouping clay particles, interweaving bunch that include silts, sands, clay and granular matrices may occur.

Piping. The sequence of detachment and transportation of particles may develop a back erosion condition known as piping, and starts at the seepage exit face where there are no constrains to prevent transport of the particles. High exit gradients are basic ingredient necessary for piping occurrence. Some practitioners consider piping a special case of suffusion.

4. MECHANICS OF INTERNAL EROSION

The mechanics of IE or the process to present the development of IE requires a full knowledge of what type of soil is involved, which may present clues of how IE could be controlled.

4.1 SOIL TYPES. The soil particles are divided in six groups according to their size: clay, silt, sand, gravel, cobbles and boulders. The clay particles are the bonding elements in a soil aggregate, and have special properties when they are in contact with water. The soils are a variable mixture of one or several sizes of soil particles. For engineering purposes, on IE, the soils are divided in cohesive and non-cohesive (or granular soils). The cohesive soils are more resistant to erosion due to a large content of the clay particles. However, some clays are structurally unstable in saturated conditions, because those clay particles detach spontaneously from each other even in quiet water as indicated by Mitchell on page 239. This type of clay is known as dispersive clay, and can be identified by laboratory testing. Halliburton presents a well documented report describing identification and treatment of dispersive clays.

Clay erosion can also start in desiccation cracks, settlement cracks or in channels of high permeability that may exist or develop inside the embankment core or Zone 1. Some soils have the ability to retain water within their pores, the content of which is critical to the seepage flow velocity. Thus, it is important to know if the soil is saturated or unsaturated, and if the properties of the reservoir and pore water may help the dispersion of clay particles. The IE mechanism is a complex phenomenon that involves the soil structure and the nature of the interaction between the clay surface and the eroding water. But, to understand this process, we can simplify it as presented below.

4.2 INTERNAL EROSION STAGES. IE consists of the detachment and transport of particles. The location of the internal erosion in progress defines failure modes that highlight the characteristics that are helpful to control the erosion process.

To explain the mechanics of IE, Foster and Fell (1999) uses a sketch shown in figure 4, which includes four stages: initiation, continuation, progression and breaching. Descriptions of these stages are presented in this figure. It is pointed out that the intensity of these stages is a function of the soil properties and the IE concepts discussed before. Usually, IE is detected in the stage of progression because of larger seepage flows, wet areas, erosion tunnels and higher pore water pressure (PWP).

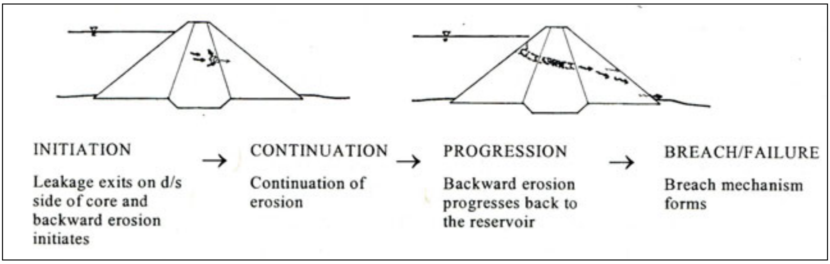

Figure 4. Internal erosion process

4.3 CONTROLLING. The most popular methods for IE control in the embankment are filters to prevent particle migration, and foundation cut-offs to minimize the hydraulic conductivity, thus reducing the seepage forces. According to the ionic composition of the water and the clay type, the resistance to erosion can be predicted and improved (if required) with special additives to reduce particle detachment. Thus, it is required to know the factors that may affect IE. Foundation seepage flow can also be reduced with grout curtains and sheet piles.

5. FACTORS AFFECTING EROSION

5.1 DURING INITIATION STAGE. Laboratory results indicate that the stress required to initiate clay erosion is affected by the concentration of ions (anions & cations) in seepage water, clay type, pH, temperature measurements, volume of water content in the voids, and thixotropy.

Pore water ions. Mitchell (2005) indicates on page 31 that the ions are atoms that carry a positive or negative electric charge as a result of having lost or gained one or more electrons. The ions are either cations (+) or anions (-). The clay particle has a net negative charge that can attract cations, and as a result, it separates from the "soil group". The cations are replaceable by other cations. Ordinarily, small cations tend to displace large cations; i.e. A typical replaceability is $Na^+ < Li^+ < K^+ < Cs^+ < Mg^{2+} < Ca^{2+} < Ba^{2+} < Cu^{2+} < Al^{3+} < Fe^{3+} < Th^{4+}$. Thus, the behavior of clays in contact with water is dependent on the ionic composition and concentration. In brief, clay in water with cations Na^+ is susceptible to erosion.

Dispersive clays. The dispersive potential of clays is measured mainly by two indices that are obtained by two tests known as the Sodium Absorption Ratio (SAR) and Exchangeable Sodium Percentage (ESP). Knowing these values, there are charts that help to identify the behavior of the clay. These charts are: Total ionic concentration vs Exchangeable sodium of soil, and Sodium absorption ratio vs. Total cation concentration in water. Halliburton presents (pages 109 to 114) several charts showing relationships for various clay types.

SAR . Is the concentration of sodium ions in the seepage water divided by the square root of the average of concentrations of calcium and magnesium ions, expressed in milliequivalents per liter**.**

ESP. It is determined by dividing the concentration of sodium ions on the exchange complex by the cation exchange capacity (CEC) of the soil. But, since CEC is difficult to obtain, it was replaced by an empirical formula which is:

$$ESP = [100(-0.0126+0.01475SAR)] / [1+(-0.0126+0.01475SAR)] \qquad (1)$$

Also, Aitchison and Woods reported that earthdam failures due to dispersive clay piping can be attributed to two types of dispersion: the post-construction dispersion of micro aggregates (clay) or the movement of dispersed clay particles through and out of a pervious soil.

5.2 DURING CONTINUATION AND PROGRESSION STAGES

Permeability. Permeability varies during the life span of the embankment, and may vary because of losing fine grains, flocculation or deflocculation of clay particles along the seepage flow, and/or reduction of void size due to swelling. Monitoring of this parameter may provide clues to assess the overall IE condition. The mechanism of permeability reduction is expected to be more influenced by the swelling property of the clay particles. Mitchell in page 133 indicates that in the macrospores of a bentonite–sand mixture, the bentonite has swelled to completely fill them. Aitcheson and Woods presented the results showing a boundary between flocculation and deflocculation within the relationship Total Cation Concentration vs. SAR. Investigation on cation concentration was made concluding that different electrolyte concentrations may affect the permeability of the soils due to swelling and breakdown of the soil mass.

The structure of the soil and the osmotic influences set up the swelling effects on the surface of the clay. The osmotic influences are due to the differences of concentration gradients between the pore and the eroding fluid (clay-water interface). The properties of swell and slaking phenomena can accelerate or stop further erosion. The amount of osmotic swell induced in a clay-water-electrolyte system is dependent on the structure of the system and the concentration difference between the pore and eroding fluid.

Flocculation. According to the electrolyte concentration of a silty clay sample, research found that a 10% to 15% of permeability reduction may be considered as a threshold concentration for flocculation. Clay in suspension may experience flocculation that may affect its performance. The flocculation of clays is influenced by the attraction and repulsion forces acting between particles. Flocculation exists within a zone limited by a boundary that indicates the beginning of the deflocculation zone, according to tests performed on an illite sample. The flocculation depends also on the surface potential of soil colloids and on the pH level. However, those effects are minor. Aitcheson and Woods present rules with respect to the chemical treatment to reduce the deflocculation effect. Consequently, the chemistry of the eroding water and pore water should be monitored.

Deflocculation. Several embankment dams have failed, and it was postulated that post-construction deflocculation was the cause of these failures, based on the observed piping traces. Studies on the deflocculation process were performed, and some conclusions were drawn. Compaction and low permeability are factors that can minimize the deflocculation effects significantly. Aitcheson and Wood present interesting concepts about this topic.

Moisture content during compaction. A discussion of moisture content during compaction and its effects on the permeability were discussed by others concluding that compaction on the wet side of optimum is recommended for lower permeabilities.

Other factors. Studies on many dam failures have identified several reasons why piping may occur. Examples are: Soil pipes may develop from cracks where permeabilities were in the range of 10^{-6} to 10^{-8} cm/sec, and rapid reservoir filling, leaks leading to hydraulic fracturing, seepage with total salt concentrations of 15 meq/l or less in the seepage water, may disperse clay particles.

5.3 Other methods to predict internal erosion. Tests to predict erosion were developed in terms of indexes that vary from extremely rapid to extremely slow as presented by Wan et all (2002) with their apparatus Hole Erosion Test. Briaud presents another apparatus known as Erosion Function Apparatus that measures the erosion rate.

6. PIT FALLS IN CURRENT METHODOLOGY TO DETECT IE

The current methodology to detect IE consists of piezometers, seepage drainage systems, and geophysical methods with only one survey. Embankments are monitored by piezometers, toe drains, weirs and visual inspections. But, many dams had failed without warning, because the stress signals were missed due to: instrumentation being placed in the wrong place, periodic measurements missed the critical measurement, missing measurements of soil parameters dealing with the failure type, and incomplete data regarding internal erosion cavity.

7. RISK ANALYSIS AND RISK ASSESSMENT

To determine the need for an instrumentation program, a risk analysis is performed to identify failure modes and the details of what could be wrong with the embankment and foundation performance. This analysis leads to a question for future monitoring "what kind of parameter should be monitored and to what extent". If the analysis indicates a potential for failure exists, then the consequences of failure are measured by economic losses and loss of lives.

Risk analysis leads to estimated numerical values that allow us to predict embankment performance in terms of annualized loss of life. The analysis is based on tables that contain historical probabilities of cracks and high permeability zones in the embankment, and historical probabilities for piping through/into the foundation. The study is complemented by a risk assessment, which combines the risk analysis results with other factors, and helps to make a decision about improving the safety of the embankment, as presented by Fell and Foster (2007). Also, the risk analysis shows: identification of potential deficiencies leading to failure and details of these deficiencies, which will help to determine the proper instrumentation to be used.

8. INSTRUMENTATION FOR MONITORING SYSTEMS

Safer performance of the embankment can be achieved with a monitoring program that includes continuous data on all the areas of concern. It implies the use of the right types and quantities of instruments within a reasonable cost. The instruments proposed here should be able to perform a long term monitoring surveillance, and their objectives include: locations of concentrated seepage flows, and measurements of pore water pressures, seepage velocities, permeabilities, variations of clay in suspension, and cavity surveillance, when found. The clay dispersibility and the quality of the pore water content will be analyzed by water chemistry tests on the samples obtained in-situ. These considerations do not include filter and drain systems.

8.1 BAT PIEZOMETERS. Emerging technology has developed piezometers such as the BAT GMS (Ground Monitoring System) piezometer presented by BAT Geosystems

(2007). This Bat piezometer has additional devices for measurements of temperature, and permeability, and sampling seepage pore water. Knowledge of the chemistry of the water is important to predict qualitative potential for clay erodibility rates. Several BAT piezometers can be part of a BAT wireless system to monitor the ground water conditions. A description of each part of this device is presented below:

Tip. The tip of the BAT piezometer is a device known as the BAT Filter Tip MkIII, which has a one-inch-diameter extension pipe that extends to the ground surface. The tip has a septum that can connect to auxiliary devices that measure PWP, permeability (K), and can sample pore water. The tip installation can be made by push-in method or placing in a borehole. The type of tip installation can be simple or cluster.

Accessories. The accessories for this piezometer are: sensors, test containers for K, and evacuation sample tubes. The sensors, test containers, and evacuation sample tubes are interchangeable parts of the BAT piezometer. The sensor is the intelligent sensor (IS) that measures PWP, and includes a temperature gauge. To engage with the tip or other elements to perform additional tasks, the IS sensor has an injection needle that can be connected to the septum of the tip, or to the test container. The septum is a device that makes the connection between an injection needle and the septum itself leak proof. Double injection needles are available to connect the interchangeable units. **Figure 5** shows this detail, and also shows the connection to auxiliary devices such as the Test Container for the permeability test, or the evacuation sample tube for the ground water sampling.

Readout devices. Piezometric readings can be made manually or automatically. The automatic readings can be made for a stand-alone piezometer or for IS-network piezometers. A logger unit is attached to the extension tube. The readings from the logger unit are read by the IS Field Unit system, in engineering units, making barometric compensations, programming future readings and downloading data. The IS-network can be composed of several filter tips and can be connected to one module.

Transmitters. A logger unit can be connected to a GMS system for a wireless transmittal. Powered by ordinary alkaline batteries, the GMS module can log and transmit data during a period of several months.

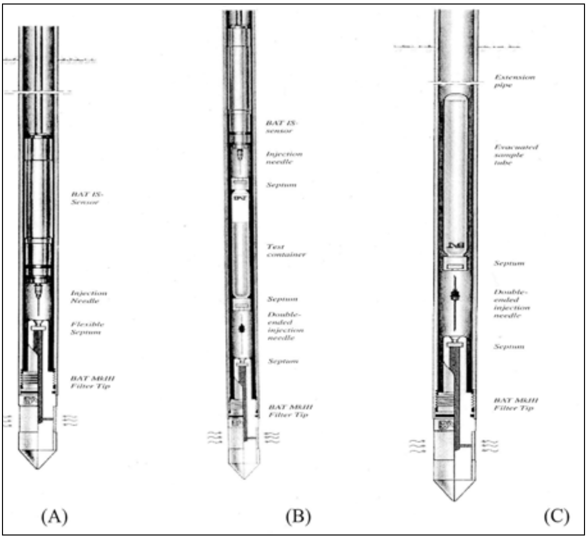

Figure 5. BAT GMS piezometers to measure PWP (A), Permeability (B) and sampling (C)

8.2 VW PIEZOMETERS. Slope Indicator (2007) describes the VW (vibrating wire) piezometer which converts water pressure to a frequency signal by a diaphragm, a tensioned steel wire and an electromagnetic coil. The piezometer is designed such that a change in pressure on the diaphragm causes a change in tension on the wire. When excited by the electromagnetic coil the wire vibrates at its natural frequency. The vibration of the wire in the proximity of the magnetic coil generates a frequency signal that is transmitted to the readout device. The readout device processes the signal, applies calibration factors and displays a reading in the required engineering units. This new designed piezometer can also be installed as a multi-level VW piezometer system. A PVC pipe provides a way to control the precise installation depth of each piezometer. The pipe protects the cables and allows a bentonite-cement grout to fill the borehole and the pipe itself forms a system where each piezometer is isolated from the others.

8.3 ULTRASONIC ACOUSTIC SENSORS

It is not unusual to discover or suspect cavities along the seepage path in the embankment or foundation. The geometry and the size of the cavity are essential pieces of information for a proper treatment. Thus, a device to measure a cavity should be available when it is needed. A search of these devices was performed and the following information was found:

Ultrasonic sensors are often used in robots for obstacle avoidance, navigation and map building. Two basic types of sensors are popular: Based on Polaroid for camera range findings and Hitechnic. The Polaroid device is described in Ultrasonic Acoustic Sensing (2007), which shows its sketch with one sensor A, the Hitechnic device with two sensors B and C. Sound waves emitted by these devices will measure the distance between the sensors and the cavity wall. The sensor A is a transmitter and receiver for the Polaroid type. In the Hitechnic type, the sensor B is a receiver and the sensor C is the transmitter. From the Hitechnic ultrasonic sensor web page, we learn that their "ultrasonic range sensor works by emitting a short burst of 40 kHz ultrasonic sound from the piezoelectric transducer "C". A small amount of sound energy is reflected by the wall in front of the device "C" and returned to the detector, another piezoelectric transducer "B". The receiver amplifies and sends these reflected signals to a micro-controller which determines how far away the objects are, by using the speed of sound in air. The calculated range is then converted to a constant current signal and sent to a central station. The Hitechnic sensor has separate transmitter "C" and receiver "B" components as indicated before. The assessment of the cavity survey may have a better quality using the Hitechnic sensor.

Excessive seepage has developed huge cavities in embankment, foundations or below the reservoir. The dimensions and geometry of the cavity is helpful to determine and assess the extent of the damage and potential branching for design of a proper fix of the cavity.

A simple mechanism can perform the survey of the cavity, and store the survey data in a readout device. The simple mechanism may consist of an ultrasonic sensor that is placed in the cavity thru a cased drilled hole, at an elevation from the cavity floor. The sensor is operated from the top of the hole by a mechanism that allows 360 degrees rotation. Several surveys at different elevations of the cavity hole will give good information.

8.4 GEOPHYSICAL METHODS

There are many geophysical methods, but the active thermal survey with fiber optics and the resistivity survey are selected as the most appropriate methods to perform the initial step of the monitoring program along longitudinal sections of the embankment or across the river valley.

Active Thermal Survey. The main element of the active thermal survey is known as a distributed temperature sensing (DTS) device that is also known as fiber optic wire. The measurements made on this DTS are calibrated to translate the measured thermal response to flow velocity and degree of saturation. These two parameters are quite useful for a computerized seepage analysis.

The basic concept for this application is that the embankment temperature depends on the reservoir and the climate, which creates temperature waves that propagate through the embankment and foundation. **Figure 6** show these heat waves by using arrows that represents the propagation of the temperature waves. The seepage flow controlled by the impervious element (Zone 1) is slow. The effect of temperature in the flow depends on the height of the embankment, and should not be ignored in the analysis. Thus, the thermal properties of the materials such as the thermal conductivity and thermal heat capacity should be well determined.

It will be quite useful to know the actual seepage flow location across longitudinal sections for the assessment of the seepage conditions of the embankment. Thus, the use of distributed temperature sensors such as the fiber optics gives a high-quality product. The use of fiber optics can be done in new and implemented in existing dams (as was done in several countries) either horizontally buried or vertically in standpipes.

A qualitative evaluation of the seepage can be performed by studying the seasonal variation, where the large temperature variation indicated higher seepage flow as shown on **Figure 7.** An example of temperature measurements by a sensor that was subjected to different seepage flows can be found in Johansson.

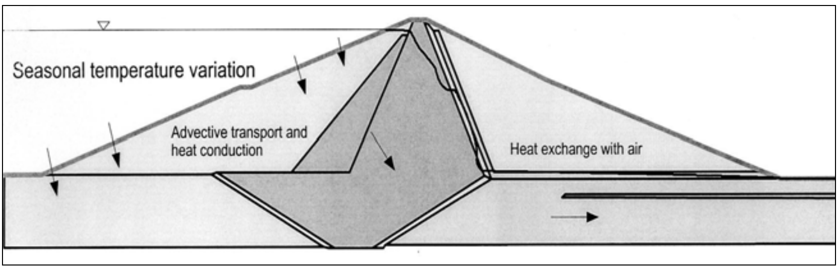

Figure 6. Geothermal flow in embankments

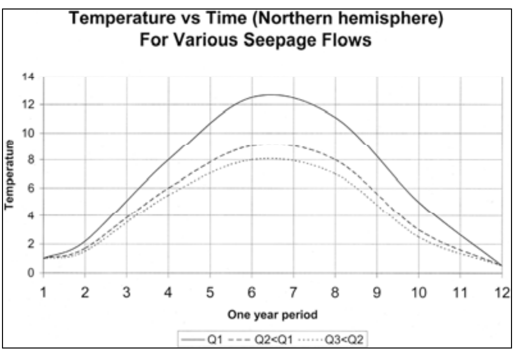

Figure 7. Temperature measurements

Resistivity surveys. The electrical resistivity method of subsurface exploration is based on measuring the resistance of the materials through which electrical current is passing. This paper presents the use of the Werner method as presented in Deutsches Talsperrn Komitee (2007). The resistivity of any material depends on its porosity and the seepage salinity. The separation of the electrodes depends on the type of investigation and the required depth of interest.

The electrodes are made of stainless steel for long term monitoring, and should be installed where the temperature and humidity vary as little as possible (better would be below the freezing depth).

The resistivity of the embankment materials should be expected to vary significantly. The bedrock is usually highly resistive when of sound quality, while in weathered, fractured or mineralized zone conditions the resistivity can be significantly low. Zone 1 usually has low resistivity; filters have higher resistivity if unsaturated, and the saturated resistivity depends on the resistivity of the water. The resistivity of Zone 3 is larger than the resistivity of the filter. Resistivity of the rock fill has thousands of ohm-meters, but the resistivity of the rock fill on the upstream side of the embankment is often reduced by the water.

Resistivity is temperature dependent, and the seepage induced temperature variations can be detected by repeated resistivity measurements. The performance of this method is shown on the examples presented below.

Example of temperature measurements made at Hylte Dam (Sweden) using fiber optics can be found on Figure 5, page 9 of Johansson, which shows the location of the fiber optic cable, and Figure 6 page 10 shows the temperature measurements along the cable.

Example of resistivity measurement method at Sadva Dam
Sadva Dam is located in Sweden, south of the Arctic Circle. The dam consists of a main dam, dike, and several appurtenant structures. The maximum section is about 31 m high. Resistivity surveys were taken with high and low reservoirs, using a mobile version of the data acquisition system installed at the site. The results on the upstream profile of the survey indicate the existence of areas with high and low resistivity. Details of this example can be found in the report presented by Johansson 2003.

Reclamation performs resistivity surveys and figure 11 presents an example of a survey performed by Markiewicz. The information presented on this figure can help to assess the material type, and when compared with previous measurements (made with the exact same conditions), it will help to assess the water content seasonal variations. The three figures correspond to Measured (Upper) and Calculated (middle) Apparent Resistivity Pseudo section and the Inverted Resistivity Section (lower).

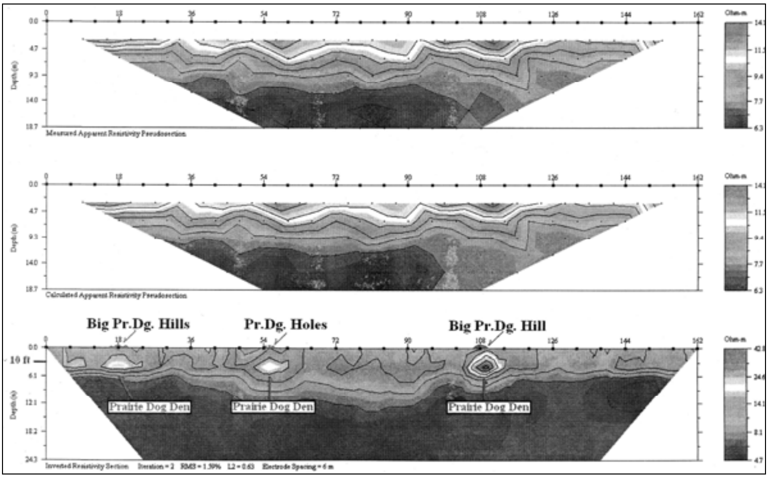

Figure 11. Survey Results

9. CONCLUSIONS

9.1 PIEZOMETERS. The use of BAT piezometers may significantly improve studies on seepage; making prediction of internal erosion more meaningful. Such improvements could lead to better remedial and preventive measures to protect the embankment performance. The improvements may consist of monitoring potential permeability variations, changes in pore water chemistry, and developing relationships such as permeability and pore water pressures that could give clues for better assessments.

Modern VW piezometers may be advantageous to measure water pressures in a multilayer foundation along a borehole; the borehole is grouted along its entire extension, and they do not need sand filters or bentonite seals for proper operation. Connections to readout devices can be made with fiber optics, eliminating the effects of lightning strikes.

9.2 FLOW SENSORS FOR WEIRS. The flow collected by the weirs can provide more information on IE by using more sophisticated measuring devices such as the ultrasonic and electromagnetic sensors, and the multiparameter sensor, which will provide data on flow and water chemistry parameters.

9.3 ULTRASONIC ACOUSTIC SENSORS. Knowledge of the cavity size will help to identify the direction of the seepage flow, and the assessment of the foundation conditions will be improved. Since erosion occurs on softer and erodible portions, the shape of the cavity may indicate branching of the cavity towards other soft zones.

9.4 THERMAL SURVEYS USING FIBER OPTICS. The benefits are permanent monitoring of the seepage flow (and indirectly, monitoring of internal erosion) through the downstream longitudinal section of the river valley. These longitudinal sections may include portions of the embankment as required. The intrinsic immunity to lightning strikes and other interferences that common conductors have are no longer a problem. Comparisons of seasonal variation and changes of seepage flow through the years will help to make a proper assessment of the internal erosion, and maintain a safe performance of the embankment.

9.5 RESISTIVITY SURVEY. The benefits of the resistivity survey are permanent monitoring of the seepage flow through the downstream side of the embankment and foundation that can be performed whenever it is needed. Comparisons of seasonal variation and changes of seepage flow through the years will help to make a proper assessment and maintain a safe performance of the embankment. It may need more time to perform the survey, but it may include improvements to more direct measurements of seepage, i.e. from existing toe drains.

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Reservoir Embankments of Mesa Verde

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ABSTRACT: Reservoir construction by early Americans at Mesa Verde from A.D. 750 to 1180 utilized embankments to create water storage impoundments. An interdisciplinary team of engineers, geologists, sedimentation experts, and archaeologists has evaluated reservoirs at Mesa Verde National Park. Excavations and test hole drilling have provided detailed reservoir cross-sections that were analyzed as to timelines, side slopes, soil gradation, and volume of storage.

Spoil zones for dredged sediment were identified. Thin ash deposition layers provided field evidence of prehistoric forest fires, sandy deposits represented periods of flooding. Testing of ancient pollen told of concurrent agricultural activity in the upstream drainage basin. Six berm-building phases were distinguished using stratigraphic analyses. Failure of the inside face of a too-steeply sloped berm occurred at Morefield Reservoir in about A.D. 900, but was corrected in subsequent years.

It was determined that the early Americans of Mesa Verde were successful water harvesters and reservoir builders under difficult conditions. These ancient people were good engineers with innate geotechnical knowledge.

INTRODUCTION

The study of the reservoirs built by the ancient Americans of Mesa Verde pushes back the record of successful water system and geotechnical engineering in North America by about 1,000 years. Much has been learned about the knowledge, skills, and history of the early Mesa Verdeans. The benefit of field research into the reservoir practices of the Ancestral Puebloans of Mesa Verde is that modern engineers can learn how these people dealt with reservoir seepage, slope stability, and dredging to maintain storage capacity. Field research at Mesa Verde was, in many ways, forensic engineering. From 1995 through 2005, engineers, geologists, sedimentation experts, and archaeologists affiliated with Wright Paleohydrological Institute (WPI) studied four Ancestral Puebloan reservoirs at Mesa Verde National Park in Southwestern Colorado.

In WPI's interdisciplinary team investigations at the four sites, an objective was to conduct a thorough technical evaluation of the many theories previously presented on the functions that each of the four sites had served. Theories on these functions ranged from ceremonial locations for social gatherings, to meeting places for leaders of the community, to ceremonial burial sites, to water storage facilities. The sites, now known as Morefield Reservoir, Far View Reservoir, Sagebrush Reservoir, and Box Elder Reservoir, were not all recognized as reservoirs when studies began, and, indeed, the Box Elder site was not even discovered until 2001 when its mound was exposed due to a forest fire.

Starting with the "Morefield Mound", the conclusion was reached that it provided storage for water based on the presence of layered redoximorphic soils (color patterns in the soil caused by saturated conditions) and evidence of earthwork. Larry V. Nordby, Research Archeologist and Field Director of the Archeological Site Conservation Program at Mesa Verde National Park, after visiting the Morefield site and examining the sediment layers in the excavated cut and standing on top of the mound, stated there was no longer any theory about the site; it was a water storage facility. Further studies found similar evidence that indicated that all four archaeological sites were reservoirs constructed and used by the Ancestral Puebloans in the A.D. 750 through 1180 period.

These ancient engineers had some geotechnical savvy. The recent evidence shows that without formal training or written language, the ancient people of Mesa Verde discerned a great deal about embankment construction, seepage, slope stability, dredging, and other geotechnical considerations. They operated and maintained these public works projects for as long as 350 years. In the case of Morefield Reservoir, WPI researchers found evidence that the Ancestral Puebloans learned the hard way about the appropriate sloping of berms. Evidence of a failure that occurred around A.D. 900 provides a record of what happened.

Table 1. Site Locations and Approximate Dates and Periods of Use

Reservoir	Location	Time Span (A.D.)	Period
Morefield	Morefield Canyon	750-1100	Pueblo I Pueblo II
Far View	Chapin Mesa	950-1180	Pueblo II Pueblo III
Sagebrush	Unnamed Mesa	950-1100	Pueblo II
Box Elder	Prater Canyon	800-950	Pueblo I Pueblo II

RESERVIOR CONSTRUCTION

Physical features at the four reservoirs varied considerably. The locations and approximate periods of use, based on potsherds and carbon dating of artifacts in the sediments, are shown in Table 1. Morefield (Figure 1) and Box Elder (Figure 2) Reservoirs were located in canyon bottoms, both with upstream drainage basins of about 10 square kilometers (about 4 square miles). At both sites, the original impoundment was dug by hand either in, or adjacent to, the main canyon thalweg. The initial pits varied from an estimated 1 to 2.5 meters (about 4 to 8 feet) deep. The original source of water into these two canyon-bottom ponds was likely from groundwater.


FIGS. 1 and 2. The raised earthen mounds of Morefield (L) and Box Elder (R) Reservoirs are located in canyon bottoms.

Diversion of water from the main canyon thalweg became a later source of water for Morefield and Box Elder Reservoirs. At Morefield, the location of the final intake canal or ditch is easily identified by the hand-placed stones leading from the main channel along the bank of the terrace and raised berm to the reservoir. At Box Elder Reservoir, only a few stones were found as evidence of an intake ditch along the upstream terrace.

Far View (Figure 3) and Sagebrush (Figure 4) Reservoirs were located on mesa tops, with limited catchment areas. For Far View Reservoir, the original excavation to the sandstone bedrock was about 1.5 to 1.8 meters (5 to 6 feet); while at Sagebrush Reservoir, the depth to the sterile clay bottom was from 1.2 to 1.5 meters (4 to 5 feet) below the natural ground elevation. Each of these mesa tops had identifiable access entrances to the water storage area. Inflow of water to the reservoirs from the drainage basins was from agricultural areas and compacted topsoil.


FIGS. 3 and 4. The Far View Reservoir (L) remains easy to recognize because of its rock wall construction. The Sagebrush Reservoir (R), marked by two intersecting archaeological excavation trenches, is shown from above.

Morefield and Box Elder Reservoirs

Sediments from the upstream drainage basins were diverted into the reservoirs along with the inflow of water. Over the years, the sediment accumulation reduced the reservoirs' storage volume, so the Ancestral Puebloans occasionally dredged the sediments with flat stones, digging sticks, and baskets to maintain an adequate domestic water pool. The dredged material became berm material which provided definition and raised the reservoir perimeters. These berms consisted of clays and silt-size material which, under the compaction of foot traffic, resulted in excellent embankments. The dense clay controlled seepage.

Stratigraphic interpretation of the sediments exposed in the archaeological trench dug at Morefield in 1997 (Figure 5) provides a history of maintenance activities for the water storage facility. Based on the relative position and shape of the sand-rich units observed on the trench wall and using the principles of overlap and truncation, at least six reservoir periods were evident (Figure 6). Each period represents multiple runoff events during which sediments were transported into the reservoir, reducing storage capacity. Truncation of some of the sand-rich units shows that these sediments subsequently were removed and discarded over the adjacent berm area. For instance, the abrupt termination of sandy zones shown in Figure 6 tells us that dredging occurred and that the sandy zones to the side were spoil areas. The six reservoir periods represent deposition of an estimated 14.3 meters (47 feet) of sediments during a 350-year period, an average deposition rate of 40 millimeters (1.6 inches) per year.

Because dredging did not remove all the sediment during each cleaning, it was not long until the water storage ponds began to rise in elevation and take the form of mounds into which water would no longer flow by gravity. The early people determined that water could be diverted from the canyon bottoms into a delivery canal leading to the rising ponds, but sediment deposits still had to be occasionally cleaned out and cast to the side, where berms were formed. After 350 years, by A.D. 1100, this process brought the Morefield Reservoir up in elevation about 6.4 meters (21 feet) above the original reservoir bottom.

FIG. 5. This archaeological trench was excavated in May 1997 to allow analyses of the sediment layers.

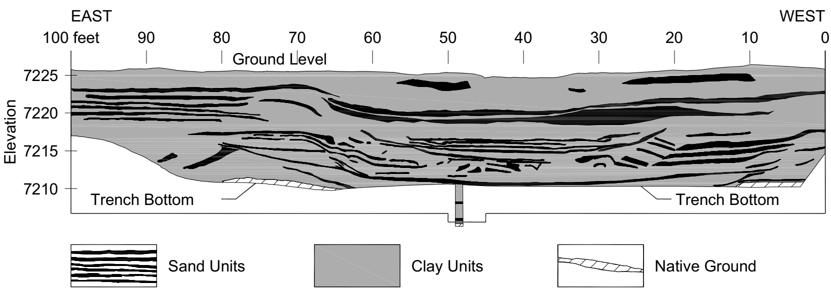

FIG. 6. The truncation of sediment layers at Morefield Reservoir indicates its maintenance history.

The present-day reservoir mound at Morefield is 4.9 meters (16 feet) above the valley bottom, with a circular 61-meter (200-foot) base and a flat top 40 meters (130 feet) in diameter. At Box Elder Reservoir, the mound rises 6 meters (20 feet) above the existing canyon thalweg and is elliptical with a minimum diameter of about 36 meters (120 feet) and a maximum diameter of about 49 meters (160 feet.) Analyses of the dense berm material of the mound were aided by Mr. Richard Wiltshire of the U.S. Bureau of Reclamation. Increased sedimentation rates were a result of both natural occurrences, such as forest fires, and man-induced activities, such as upgradient agricultural land use practices.

The volume of total sediment deposited in the canyon-bottom Morefield and Box Elder Reservoirs was estimated at 12,180 cubic meters (430,000 cubic feet [ft^3]) and 4,300 cubic meters (152,000 ft^3), respectively. The computed annual sediment yield for the drainage basins was 3.19 cubic meters per square kilometer (0.0067 acre-feet per square mile [AF/mi^2]) for Morefield Canyon and 2.76 cubic meters per square kilometer (0.0058 AF/mi^2) for Prater Canyon. Considering the sediment that may have bypassed the diversion structure or overflowed the canal or reservoir, the sediment yield for both drainage basins was probably about 4.76 cubic meters per square kilometer (0.01 AF/mi^2) per year.

Far View and Sagebrush Reservoirs

After the early success of water storage facilities at both Morefield and Box Elder Reservoirs, similar water storage ponds were subsequently constructed at the Far View and Sagebrush Reservoir sites. Earlier archaeological excavations were done at Far View Reservoir (then known as Mummy Lake) in 1967 under the direction of Dr. David Breternitz of the University of Colorado. The exploration trenches at Sagebrush Reservoir shown in Figures 4 and 6 were dug in 1972-1974, under the direction of Dr. Jack Smith.

The pond size varied from the approximate 27-meter (90-foot) circular dimension at Far View Reservoir to an elongated pit with a short width of about 16 meters (55 feet) to a maximum width of about 24 meters (80 feet) at Sagebrush Reservoir. The depth of the ponds was limited, which required construction of a stone wall completely around the storage area. To provide storage for the dredging material, a parallel stone wall was built around a portion of the downslope side of the storage pit. At Far View Reservoir a third wall was discovered buried between the outer walls that exist today. The set of walls on the south and east sides of the Far View Reservoir are from 3 to 6 meters (10 to 20 feet) apart. The area between the two walls was filled with sediments from both the initial pit excavation and dredged sediments from the reservoir.

The berms and fill at Far View Reservoir represent approximately 840 cubic meters (1,100 yd^3), roughly 200 percent of the excavated natural soils. Without considering wind-deposited sediments, the sediment volume at Far View Reservoir would indicate an average inflow of sediment of about 1.8 cubic meters (2.4 yd^3) per year over 230 years. Wind and water erosion of the banks, if considered, would tend to raise this sedimentation rate. The unit sediment yield in cubic meters per square kilometer per year was similar to Morefield Reservoir.

The plotted profile of Sagebrush Reservoir's Trench I included in the Smith report (1999) identified an excavated depression in the southern part of the reservoir. This depression was an excavation pit dug to a depth of about 1.3 meters (4.3 feet) in a likely circular pattern 7.9 meters (26 feet) in diameter probably for containment of about 45,412 liters (12,000 gallons) of water for domestic use. Spoil from the excavation was likely deposited around the southern area as a berm surrounding the excavation. The bottom of the pit was the sterile red clay stratigraphic formation, which created an ideal impervious bottom layer. This excavation was considered to be Phase I of the water development project.

During Phase III, stones were placed to confine the berm and to surround the excavated pit in a pattern similar to that on the adjacent mesa's successful Far View Reservoir development. Stones were also used to define a narrow entranceway about 0.8 meters (2.7 feet) in width at the northwestern part of the reservoir.

Data from the Smith report (1999), along with the soil testing by augers in 2000, and 2001 topographic mapping of the reservoir, were used to estimate the volume of sediment deposited in Sagebrush Reservoir. The first step in these computations was to identify the volume of the initial excavation under Phase I for the approximate 7.9-meter (26-foot)-diameter pit. As dug to a depth of about 1.3 meters (4.3 feet), the material excavated from the pit and deposited in the berm surrounding the storage area was about 46 cubic meters (60 yd^3).

Total sediment deposited in Sagebrush Reservoir over the 150-year period from A.D. 950 to A.D. 1100 was about 170 cubic meters (220 yd^3). This included sedimentation of the excavated depression of Phase I. It did not include the approximate 84 cubic meters (110 yd^3) of sediment subsequently deposited by wind after abandonment of the reservoir, to a depth of about 0.3 meter (1 foot) overlay of the reservoir sediment deposits. Some dredging of deposited sediments took place during the period of occupation at the site. Some of these sediments would have been placed in the area between the two walls of stone and primarily to the south of the reservoir. Evidence shows that the dredged material was also cast to the reservoir sides. It is estimated that about 10 cubic meters (13 yd^3) of sediment dredging would have been added to the berm, primarily from Phase I activities. This gives a total computed volume of sediment deposited in the 150-year period of occupation at the site of about 180 cubic meters (233 yd^3).

The Sagebrush Reservoir berm, as it exists today, was built during its Phase III period. The southerly portion of the berm was raised above the natural earth surface to tend to account for the higher ground surface reservoir edge to the north. The south berm represents about 0.7 meters (2.4 feet) of elevation gain. It was this construction that significantly increased the storage volume. This construction provides evidence of good engineering, knowledge of water containment principles and ability to work within the natural constraints of elevation differences. The berm contains about 100 cubic meters (130 yd^3) of fill material.

ANCIENT BERM FAILURE

About A.D. 900 the ancient operation of Morefield Reservoir experienced a berm failure that is recorded within the sediments of the mound. The berm had an interior slope of 4 to 1 and was about 1.2 meters (4 feet) high with five horizontal sandy layers (Figure 7). The berm consisted mostly of dense silt and clay, however. Evidence indicates that during dredging operations the stability of the berm was impacted.

A slip failure occurred that caused the 1.2-meter (4-foot)-wide displaced berm section to slide down about 0.6 meter (2 feet) while rotating clockwise, causing an upward bulge 1.8 meters (6 feet) out from the toe. The stratification evidence (Figure 7) led the field researchers to conclude that weakening of the toe of the interior berm likely caused the apparent slip failure. Examination of sandy strata markers tells a story of continued reservoir operations with the failed berm continuing to serve to impound stored water until the next berm building phase.

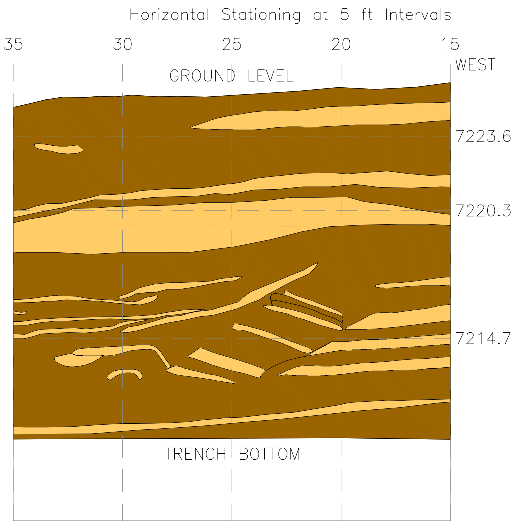

FIG. 7. Displaced sediment layers are evidence of a prehistoric berm failure at Morefield Reservoir.

SEDIMENT AND SOIL SAMPLING

The May 1997 excavation of a 38.1-meter (125-foot)-long trench to a depth of 4.9 meters (16 feet) across the Morefield Reservoir mound was a unique undertaking during the WPI investigations (Figure 5). The excavation was completed under a permit from Mesa Verde National Park. The backhoe excavation was much deeper, and was near the same location as the shallower trench dug in 1967 under the direction of Dr. Smith and Ezra Zubrow of the University of Colorado Mesa Verde Research Center. The WPI research team observed and documented the sediment layering and the exposed characteristics of material along the south trench wall. The backhoe-excavated trench depth was limited to 4.9 meters (16 feet) due to density of the deposited sediments. Hand augering of an additional 1.5 meters (5 feet) identified the original natural soil horizon below the bottom of the trench for a total reservoir height of 6.4 meters (21 feet).

At the Morefield Reservoir mound, the sediment layering exposed on the trench wall provided clear evidence that waterborne sediments had been deposited within a reservoir. Sediment layers of fine sand-size material, with traces of charcoal, proved the inflow of sandy material occurred occasionally, likely due to extreme thunderstorm events and drainage basin erosion (Figure 8). The larger portion of the exposed sediments on the trench wall, about 65 percent, was a densely compacted clay matrix. Over the 350-year life of the reservoir, there were about 21 instances of measurable sand to sandy clay depositional occurrences that would have represented larger inflows to the reservoir during canyon flooding periods. There were approximately 14 different thin, continuous layers of charcoal deposits, which likely represented fluvially transported charcoal from forest fires. Sediment samples of the exposed material were collected for particle size gradation analysis.

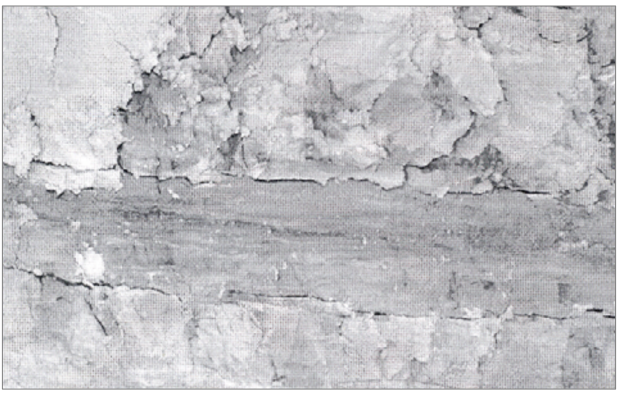

FIG. 8. Layer with fine sand size sediments and charcoal at Morefield Reservoir.

Sediment sampling at Box Elder, Far View, and Sagebrush Reservoirs consisted of hand augering and classification of soils extracted from auger holes at several locations within the reservoir area. Samples collected at the mesa-top Far View and Sagebrush Reservoirs showed iron staining, proving that the depressions had held water. There was also evidence of the textural interlayering in some samples and variations in soil classification. Auger hole samples showed that the native soil samples collected outside the reservoir were relatively uniform in color and texture compared to the reservoir sediments.

Mechanized drilling of four holes in May 2003 at Box Elder Reservoir, using U.S. Bureau of Reclamation equipment, enabled analyses of the deep native soil horizon near the reservoir center at a depth of over 6.2 meters (20.2 feet) or 617 centimeters (243 inches). Continuous samples collected by the jackhammer drilling rig were analyzed by Doug Ramsey of the Natural Resources Conservation Service. Selected samples were later tested by the U.S. Bureau of Reclamation Soils Laboratory at the request of Richard Wiltshire. The U.S. Bureau of Reclamation testing included grain-size gradation and Atterberg limits testing.

ASCE RECOGNITION

On September 26, 2004 Pat Natale, Executive Director of the American Society of Civil Engineers (ASCE), dedicated these four prehistoric reservoirs in Mesa Verde National Park as a National Historic Civil Engineering Landmark (Figure 9). The dedication of the reservoirs as an ASCE landmark was special, as ASCE has recognized only four other National Historic Civil Engineering Landmark sites in the state of Colorado. The recognition of these water storage projects was a tribute to the early Mesa Verdeans who successfully undertook these water-storage projects.

FIG. 9. Dedication of the ASCE Landmark plaque occurred on September 26, 2004. ASCE Executive Director Pat Natale, Ken Wright, David Breternitz, and former Mesa Verde National Park Research Chief Linda Towle are shown.

The success of the four reservoirs used as domestic water supplies and storage facilities was remarkable; however, they required extensive maintenance throughout their useful life. We believe that ultimately the maintenance and hand dredging of sediments became too great due to excessive sediment deposition, not unlike the accumulation of sediment in many reservoirs throughout the world in operation today.

ACKNOWLEDGEMENTS

Reservoir analyses were aided by federal agencies such as the U.S. Bureau of Reclamation, the National Park Service, the U.S. Geological Survey, and the Natural Resource Conservation Service. Individual participants included specialists in geology, hydrology, soil science, water law, surveying, archaeology and palynology. The ten years of research studies were undertaken by volunteers and experts serving the Wright Paleohydrological Institute, under permits from the National Park Service and with financial support from the Colorado Historical Society. The assistance of the staff of Mesa Verde National Park is gratefully acknowledged.

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Brownfield Development Challenges at a Former Manufacturing Facility, Engineering "Opportunities" Make the Difference

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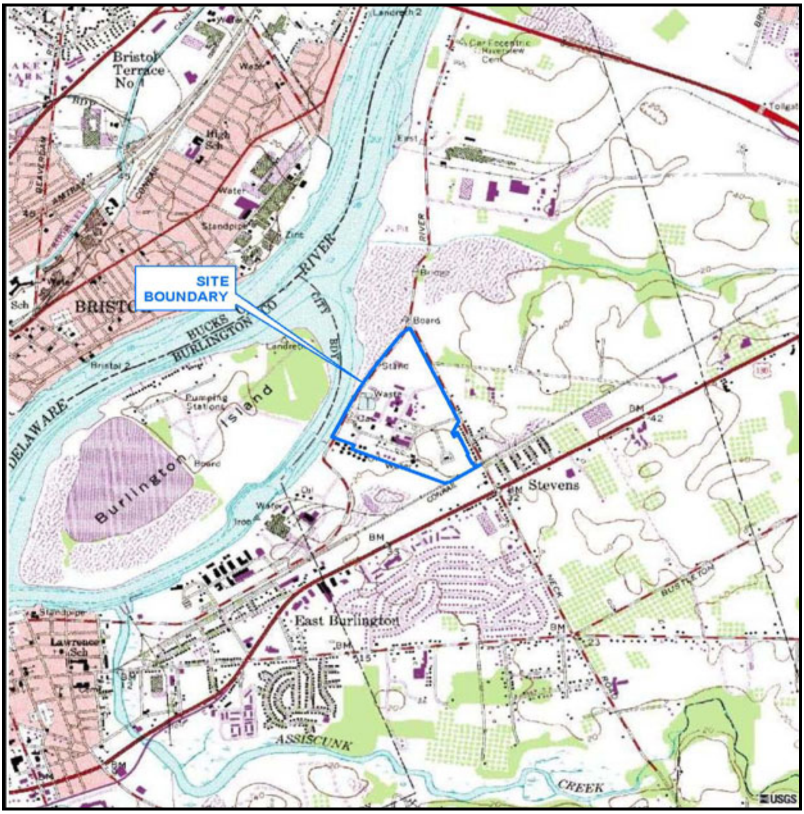
ABSTRACT: The Former Hercules Neck Road facility occupies approximately 55 hectares located 2.4 kilometers northeast of Burlington City in Burlington County, New Jersey. Hercules conducted manufacturing operations from 1945 to 1992. Products manufactured included hard and liquid wood rosin based resin products used by the food and cosmetic industries; dimethylterephthalate, a raw material used for the manufacture of XR-5 fabrics, an agricultural herbicide; and Ethylene Oxide Adduct, a detergent ingredient. The site is located in a low lying area along the Delaware River with ground elevations from 3 to 7.6 meters above mean sea level. The 100-year and 500-year flood elevations are 3.8 and 4.6 meters above MSL respectively. For re-development purposes, the on-site water treatment storage facility consisting of 5 lagoons was converted, in-place, to an on-site repository, containing approximately 45,875 cubic meters of chemically stabilized sludge in addition to soil and concrete impacted with polychlorinated bi-phenyls. The stabilized lagoon system covers approximately 1.3 hectares, complete with an engineered cap. Approximately 191,100 cubic meters of material was imported to construct the site above the 100 year flood plain, in compliance with development requisites. Import material consisted of a modified / processed dredge material. The processed dredge material was trucked approximately 130 kilometers from Newark Bay and was augmented with approximately 8% Portland cement.

1.0 INTRODUCTION

Viridian Partners LLC (Viridian) is a Colorado based Brownfields development company that acquires environmentally impaired properties and repositions them for resale once remediated. Viridian assesses properties throughout the country including several former mine sites and municipal landfills in Colorado. These properties are generally challenging from a technical standpoint and provide an excellent opportunity to use innovative remedial technologies to mitigate complex environmental issues while positioning the property for reuse.

1.1 Background

In 2005, the former Hercules Incorporated manufacturing facility (Site) located in Burlington, New Jersey was acquired as a Brownfield redevelopment project. The Site is located in west central New Jersey northeast of Philadelphia, Pennsylvania and south of Trenton, New Jersey. The Site occupies approximately 135 acres with approximately 915 meters (3,000 feet) of frontage on the east bank of the Delaware River immediately east of Burlington Island. The Site is approximately 2.4 kilometers (1.5 miles) northeast of Burlington City in Burlington County, New Jersey. The site location is illustrated in Figure 1.



Map Source: USGS, Bristol, Pennsylvania-New Jersey Quadrangle, 1981.

FIG. 1. Site Location Map.

A Remedial Action Work Plan (RAWP) was negotiated with the New Jersey Department of Environmental Protection (NJDEP) that defined the remedial strategy for cleaning up the site. As part of the remedial action, the site was divided into four separate Operable Units (OUs). OU-1 consists of the portions of the Site associated with contaminated soil and sediment in and near the storm water drainage system. OU-2 consists of contaminated soils at the Site that are not in or near the storm water drainage system and not in or near the former wastewater lagoon system. OU-3 consists of the soils and sediments within and near the former wastewater lagoon system; and OU-4 consists of impacted shallow groundwater at the Site. The delineation of the operable unit boundaries are illustrated in Figure 2.

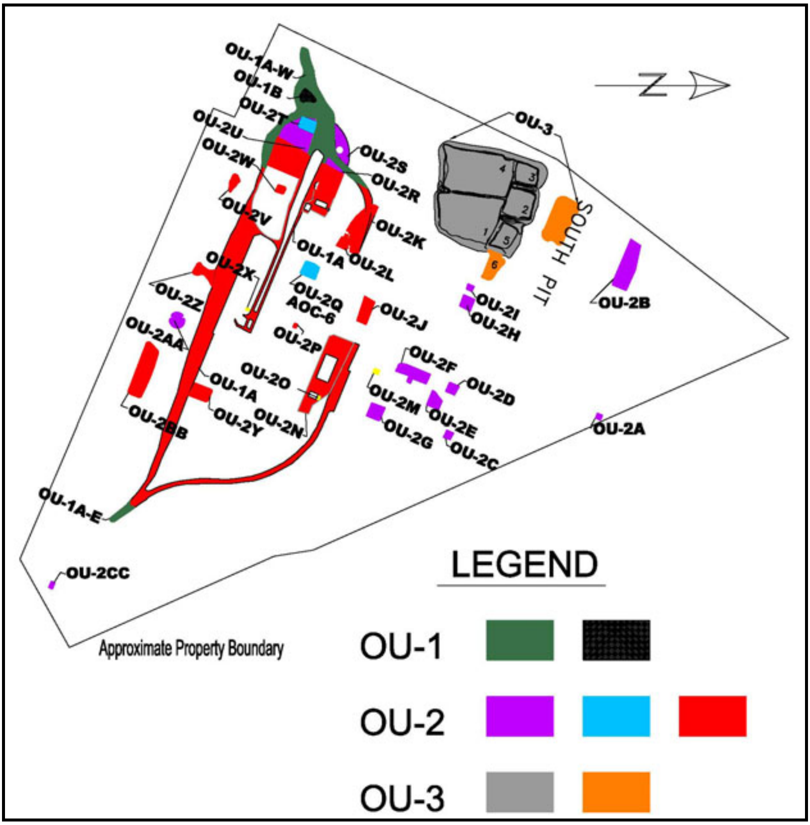


Fig. 2. Delineation of Operable Units at the Former Hercules Inc. Neck Road Facility

OU-1 and OU-2 consisted of soil and sediments that were impacted by numerous chemical constituents including heavy metals, total petroleum hydrocarbons (TPH) and Polychlorinated Biphenyls (PCBs). The remedial action objectives for these

Operable Units included the characterization and removal of contaminants above the appropriate remedial standards and utilizing the excavated materials as part of the OU-3 closure activities. As part of the final remedial measures, future site development would provide for an impervious engineered cap with approximately 24 hectares (60 acres) of warehouses and parking areas.

The remedial objective for OU-3 was the consolidation of contaminated materials within the wastewater lagoon system together with other contaminated soil and concrete from OU-1/OU-2 and construction of an on-site repository.

OU-4 consisted of the shallow on-site groundwater which was to be monitored over a 2 year period to assess whether the quality of the groundwater was improving following the removal of source contaminants within the overlying soil as part of the OU-1/OU-2 remedial efforts.

Since this site was purchased with the intent to redevelop, the RAWP incorporated some aspects of development as part of the cleanup strategy in order to get the site to a 'pad ready' condition as quickly as possible. This approach not only prepared the site for future development it also reduced the development costs by utilizing existing engineering support and site infrastructure available for the environmental processes to complete development activities. This integrated approach to Brownfield development not only expedited the development activities it also allowed for 'qualified' personnel to manage 'unexpected' environmental issues discovered as part of the development work.

1.2 Site History

Hercules Incorporated (Hercules) owned the site from 1945 to 2005. Manufacturing operations at the plant were conducted between the mid-1940s until 1992. Principal products manufactured at the facility included hard and liquid resin products used by the food and cosmetic industries. Hercules entered into an Administrative Consent Order (ACO) with the NJDEP on May 6, 1992 and all site operations ceased. In 1993, Hercules began decommissioning the site infrastructure and all original buildings on the site were subsequently demolished. As part of Viridian's remedial activities, concrete foundations and building pads were removed in 2007. Concrete associated with these structures was crushed and later utilized as part of the development activities.

As part of the purchase and sales agreement, Viridian assumed the liabilities and obligations for the Site remediation pursuant to a new ACO, dated August 30, 2005. Remedial action activities have been performed under the 2005 ACO for the various operable units, including OU-1, OU-2 and OU-3 since 2006.

1.3 Physical Setting

The Site is located in a low-lying area adjacent to the Delaware River. The land surface elevation prior to remediation and re-development preparation activities varied from approximately 2.9 meters (9.5 feet) above mean sea level (AMSL) on the southwest to 6.7 meters (22 ft) AMSL along the eastern property boundary. In general, the site slopes from the northeast to the southwest.

The western edge of the property borders the east bank of the Delaware River. The 100-year flood water surface from the Delaware River is approximate elevation 3.8 meters (12.5 feet) AMSL, based on the Federal Emergency Management Administration (FEMA) Flood Insurance Rate Maps.

Historically, surface runoff at the Site was managed via a series of man-made storm water drainage ditches. There are no natural streams on the Site. Drainage ditches associated with the prior land use received non-contact cooling water, steam condensate and sand filter backwash from the water treatment plant.

1.4 Summary of Constituents of Concern (COCs)

The major COCs at the Site were asbestos-containing materials (ACMs), polyaromatic hydrocarbons (PAHs), polychlorinated biphenyls (PCBs), total petroleum hydrocarbons (TPHs) and volatile organic compounds (VOCs). OU-1/OU-2 COCs consisted primarily of PCBs spread over a significant part of the site while OU-3 COCs were generally low level VOCs. The approved work plan provided that on-site soils with PCB concentrations less than 100 mg/kg were acceptable for use as a stabilizing material for the wastewater lagoon system closure.

2.0 OU-1 AND OU-2 REMEDIAL CONSTRUCTION ACTIVITIES

As a result of historic manufacturing activities onsite, surface soils and sediments were found to be contaminated. The primary contaminant of concern was PCB which was discovered to be above the applicable standards. PCB contaminated soils were excavated and were managed on-site or were transported off site for disposal. Approximately 16,800 cubic meters (22,000 cubic yards) of PCB contaminated soil remained on-site and were later used in the wastewater lagoon system solidification process.

As part of the remedial activities to address the PCB contaminated soil in OU-1 and OU-2, NJDEP required that the site be capped with an engineered cover to ensure that impacted soil that may not have been identified during site characterization and that could possibly be in exceedance of the applicable non-residential standards, were covered to prevent possible human exposure in the future.

Proposed development plans were approved by the Burlington Township planning board in October of 2007. The approved plans consisted of the redevelopment of 40.5 hectares (100 acres) that included three large warehouses, associated parking areas, access roads and storm water detention ponds. Subsequent to the excavation of the contaminated soil and the township approval of the development plans, Viridian evaluated potential fill materials to be utilized for construction purposes to bring the site elevation up to meet the development plan in addition to elevating the site above the 100 year flood plain. Based on cut and fill requirements, approximately 191,140 cubic meters (250,000 cubic yards) of imported fill were required to meet the developments plans and FEMA flood level stipulations. Due to the enormity and economic impact of the fill requirements, potential off-site fill sources were carefully evaluated for geotechnical as well as analytical characteristics. Numerous sources were evaluated to assess quality and cost and it was determined that dredge material being removed from the Newark Bay area as part of a Corps of Engineers channel maintenance project would provide for the most cost effective solution to the site grading imbalance.

The import fill material source that was eventually selected and approved by NJDEP was processed dredge material (PDM) from Newark Bay. The use of the PDM as fill, created an effective “engineered cap” overlying formerly impacted areas and provided a good source of fill for future site development.

2.1 Processed Dredge Material (PDM) Handling

Viridian entered into a contract with a vendor who handled the dredged material once the dredge scowls were full and brought to dock. The dredge was removed from the scowls with clam shell type buckets and placed into a hopper fitted with a 10 centimeter (4-inch) grizzly. The consistency of the dredge at this stage is very loose and would classify as moderately viscous slurry. As the dredge material passes through the grizzly, it is conveyed to a pug mill that amends the dredge material with Portland cement. The vendor, through prior pilot tests and other previous project uses, determined that the amount of Portland necessary to create a batch with a consistency that can be further handled, transported and placed within a 48 to 72 hour period was determined to be between 5 and 8%. In order to meet the site fill requirements, nearly 15,000 – 15 cubic meter (20 cubic yard), tri-axel truck loads of PDM were received at the site over a 7 month period. In delivering the PDM to the site, each truck traveled 210 kilometers (130 miles) roundtrip.

2.2 Engineering Characteristics and Placement of On-Site and PDM Fill

In general, the site fill consisted of two material types; on-site material consisting predominately of silty, fine sand and PDM consisting of silt amended with Portland cement. Engineering characteristics and placement methods for each of these materials are summarized in the following sections.

The on-site fill material consisted of sandy over-bank deposits of the Delaware River. This material proved to be forgiving and very constructible. Due to the presence of this material to depths ranging from 9 to 15 meters (30 to 50 feet) below ground surface, the site would drain rapidly after heavy precipitation allowing for construction activities to continue within a day or two of receiving 5 to 8 centimeters (2-3 inches) of precipitation.

The PDM consisted of silt amended with 5-8% Portland cement. The PDM was very sensitive to moisture and temperature variations and had to be stockpiled at times to allow for suitable placement conditions. Optimum temperatures for placement of PDM proved to be above 20 degrees Celsius. Engineering characteristics associated with these materials are summarized in Table 1, Table 2 and Table 3..

Table 1. Range of Engineering Index Properties of Fill Materials Placed

Material Source	Grain Size Distribution			Atterberg Limits		
	Gravel (%)	Sand (%)	Silt/Clay (%)	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index (%)
On-Site	0 - 32	66 - 97	1 – 16	0 - 8	12 - 18	4 - 12
PDM	0 - 0.1	16 - 39	61 - 91	1 - 5	11 - 15	9 - 14

Table 2. Range of Density and Permeability Properties of Fill Materials Placed

Material Source	Maximum Dry Density (kg/m^3)	Optimum Moisture Content (%)	Permeability (cm/sec)
On-Site	1,930 – 1,990	13 – 14	1.5×10^{-3}
PDM	60 - 85	25 - 42	$2.1 \times 10^{-5} – 1.6 \times 10^{-6}$

Table 3. Range of Strength Properties of Fill Materials Placed

Material Source	Direct Shear		Unconfined Compression (kg/m^2)
	Internal Friction,[1] (degrees)	Cohesion (kg/m^2)	
On-Site	NT	NT	NT
PDM	30	1,011	10,545 - 19,825

Note: 1. Shear values represent ultimate stress.
2. NT denotes not tested.

Placement of on-site materials was accomplished primarily with earth moving scrappers. In instances where scrapers could not be used, excavation with excavators and hauling with end dump trucks was also utilized. Engineered lifts for this material were maintained at 20 to 25 centimeters (8-10 inches) compacted to a minimum of 95% maximum dry density as determined by ASTM D 1557 with the use of 10 ton vibratory smooth drum compactors.

Placement of the PDM was accomplished by end dumping from tri-axle trucks as they arrived at the site and then depending on site conditions this material was either stockpiled or immediately spread in engineered lifts that had to be maintained at no greater than 15 centimeters (6 inches) placed. Compaction of the PDM to a minimum of 95% maximum dry density as determined by ASTM D 1557 was achieved with both vibratory smooth drum and sheeps-foot compactors.

Once the site was graded to meet the approved site plan, a Remedial Action Report (RAR) was submitted to NJDEP detailing the scope of remedial activities related to the OU-1/OU-2 removal efforts. As part of the RAR, a No Further Action (NFA) was requested based on an engineered cap utilizing the PDM.

3.0 OU-3 REMEDIAL CONSTRUCTION ACTIVITIES

In order to understand and appreciate the sequence of events that lead to the eventual remedial process selected for OU-3, historic information and background are discussed in the following section.

3.1 Background

During historic production activities at the Hercules facility, there were a total of six lagoons and one pit (South Pit) comprising the OU-3 waste water lagoon containment system. The lagoons and South Pit contained varying amounts of sludge and water derived from the former operation of the waste water treatment system. During site closure activities conducted in 1995, Hercules installed a 30 mil reinforced polyethylene geomembrane liner over Lagoons 1 through 4 to prevent the accumulation of precipitation within the lagoons and to provide a temporary cap over the sludge. Historic information regarding 'As-Built' detail for the construction of the original OU-3 waste water storage lagoons was not available, however a review of available records suggests that the lagoons were constructed in sequential order as needed to provide on-site storage and containment of the waste water sludge and have been in-place for approximately 4 decades.

The sludge contained within the lagoons ranged in depth from approximately 0.5 to 2.4 meters (1.5 to 8 feet). The sludge consistency varied from very loose, loose and sticky/cohesive to viscous, stiff, tarry and resin-like. It was estimated that the total volume of sludge in all six lagoons and the South Pit was approximately 16,160 cubic meters (21,140 cubic yards) including a tarry vegetative layer on Lagoon 6. In addition to the sludge, the lagoons contained a significant volume of water estimated to be 4,467 cubic meters (1.18 million gallons).

The initial remedial design report detailed the lagoon closure activities and proposed using rubble fill as a means of "stabilizing" the lagoon sludge once the water was removed. This closure plan proposed using rubble fill to create a structural matrix within the sludge which would transfer the overlying cap load through the sludge to the soil underneath the lagoons.

The rubble fill concept was approved by the NJDEP in early 2006. In July 2006, a significant error in the initial calculations used to develop the rubble fill stabilization process was discovered. The initial calculations used to determine the actual volume of rubble fill needed to stabilize the lagoon sludge was significantly underestimated. While initial rubble fill volume estimates to complete the stabilization work was calculated to be approximately 83 cubic meters (22,000 cubic yards), further review of the original calculations suggested that the actual rubble fill volumes, needed to stabilize the sludge and lagoon water, would require approximately 190 cubic meters (50,000 cubic yards) of material. Further, this revised volume estimate anticipated that the rubble fill material would be well graded and provide for 50% pore volumes within the rubble matrix to contain the sludge and water. Based on this new information and concern about the constructability of capping the lagoon system, Viridian suspended work related to the rubble fill closure of OU-3.

Based on the engineering concerns related to the increased volume of rubble fill required to close OU-3, coupled with the anticipated size of the remediated lagoon system as a result of the additional rubble fill volume a cost analysis evaluation was completed. This evaluation was necessary in order to determine if the rubble fill approach was still viable or whether an alternative approach would be more appropriate.

In October 2006, Golden Engineering with assistance from TG Labs Company prepared a modification to the Remedial Action Work Plan (RAWP) which proposed a stabilization process in which the sludge would be mixed with chemical reagents allowing the treated sludge to form bonds which would develop a stronger structural matrix (Golden, 2006). The sludge stabilization process for closure of the OU-3 lagoon system provided the following improvements over the previous rubble-fill closure concept:

- A more structurally stable closure is achieved with the chemically-stabilized sludge;
- The chemically-stabilized matrix has lower leachability than the rubble-stabilized sludge;
- The chemically-stabilized closure requires less overall earthwork resulting in a smaller footprint since the overall volume and porosity of materials is reduced;

- The previously proposed retaining walls around portions of the closure were eliminated; and
- The chemically-stabilized process significantly reduced the remediation budget by nearly $2 million.

The modification to the RAWP was approved by the NJDEP in November 2006 and this process was implemented in June 2007 and has proven to be successful as proposed.

3.2 Stabilization Process

Sludge stabilization activities at OU-3 began in June 2007. The sludge was initially hauled to a central processing area by placing the liquid sludge into end dump trucks with excavators. At the process area the sludge was dumped and then placed into a pug mill using a front end loader. Sludge stabilization products consisted of Portland cement (Type I), and proprietary chemical reagents. These reagents along with site soils were mechanically mixed to create a solidified matrix and placed within the limits of the lagoon system.

The chemical reagents were introduced to the sludge and blended with sandy soils impacted with low levels (< 100 ppm) of PCBs. Once mixed, the processed sludge was transported back to the lagoons. Approximately 76 cubic meters (20,000 cubic yards) of impacted soil and approximately 83 cubic meters (22,000 cubic yards) of lagoon sludge were processed in this manner by November, 2007.

The sludge consistency varied from very loose, loose, soft and sticky/cohesive to viscous, stiff, tarry and resin-like. In addition to the sludge, the lagoons contained a significant volume of water that was estimated to be approximately 4,467 cubic meters (1.18 million gallons). Actual field observations indicated that the water was more of an emulsified liquid intermixed or entrained within the sludge and did not readily segregate from the sludge.

During the removal of the sludge from the lagoons, the base of each lagoon was inspected to assess whether the sludge continued into the subsurface or if there was a base to the lagoons which prohibited migration of the sludge into the groundwater. There was no synthetic liner in the bottom of the lagoons; however, there did appear to be a native clay and or clayey silt layer that provided a natural barrier for the lagoons. The bottom of the lagoon system was slightly above the groundwater level. Due to the proximity of the groundwater to the base of the lagoons, only visual inspections were performed to verify that all of the sludge had been removed for treatment.

Regulatory (NJDEP) changes to the management of concrete as part of the demolition of historic manufacturing facilities required that on-site concrete be sampled and tested for PCBs and PAHs prior to removal or demolition activities. As a consequence, results of the sampling indicated that there was concrete on-site which had elevated PCB concentrations. NJDEP agreed that the concrete with PCB concentrations less than 100 mg/kg could be placed within the lagoons and therefore approximately 3,820 cubic meters (5,000 cubic yards) of impacted concrete was crushed to minimize particle size to less than 46 centimeters (18 inches) and dispersed within the lagoons along with the stabilized sludge.

Final stabilization activities were completed in December 2007. In total, approximately 16,667 cubic meters (21,800 cubic yards) of sludge, 4540 cubic meters (1.2 million gallons) of lagoon waste water, 27,340 cubic meters (35,760 cubic yards) of excavated soils (PCB or other) along with 1,640 cubic meters (2,150 cubic yards) of Portland cement and 380 cubic meters (500 cubic yards) of chemical reagents were utilized in the stabilization process. In addition, approximately 3,820 cubic meters (5,000 cubic yards) of concrete, brick and other debris were placed within the lagoon. In summary, approximately 31,350 cubic meters (41,000 cubic yards) of materials were added to the lagoon system during the closure stabilization process.

3.3 Final Grading

Subsequent to the stabilization process, the accumulated stabilized sludge mound was rough graded to the approximate final configuration. The final configuration reflects significant consolidation compared to the original 'rubble fill' plan previously approved by NJDEP. The final shape of the lagoon area was optimized in order to reduce the footprint of the lagoon and minimize the amount of unproductive area on-site without compromising the integrity of the cap and liner system.

The stabilized sludge was rough graded for winter shut down from December 2007 to February 2008. As spring weather allowed, the final stabilized lagoon configuration was constructed. The final configuration reflects a modification to the original lagoon footprint. This modification was made to provide optional land use at the north end of OU-3. The revised lagoon footprint places the northern closure limits approximately 37 meters (120 feet) to the south of the original northern limits. This adjustment provides approximately 910 square meters (9,800 square feet) of otherwise restricted land use. The final stabilized sludge lagoon system covers an area of approximately 1.3 hectares (3.1 acres). The top of the closure is approximately El. 12 meters (40 feet) AMSL and the side slopes are 5(horizontal):1(vertical).

3.4 OU-3 Cover System and Drainage Control

After the processed sludge material was graded to its final configuration, a 15 centimeter (6 inch) layer of PDM was placed over the stabilized sludge to provide a cushion layer to help protect the overlying geomembrane from possible puncture. As previously discussed, the PDM is a silty sediment that has been amended with Portland cement.

The PDM used as a cushion layer has a maximum aggregate size of 0.6 centimeter (¼-inch) to provide a suitable base for the geomembrane. The PDM was placed in a single 15 centimeter (6-inch) lift, graded and compacted. The compacted PDM cushion layer provides an excellent bearing surface for the overlying geomembrane liner system. Approximately 3,364 cubic meters (4,440 cubic yards) of compacted PDM was utilized as a cushion layer beneath the geomembrane as part of the cover system.

The geomembrane used is a textured, 40-mil linear low-density polyethylene (LLDPE-T) material that covers the limits of the stabilized lagoons and terminates within an anchor trench at the perimeter of the OU-3 closure limits. A geocomposite drainage network (GDN) was placed over the geomembrane layer and beneath the cover soil to intercept precipitation infiltration through the cover soil and direct any moisture to peripheral drainage swales. The material installed was a double sided 170 grams per 0.8 square meters (6 ounce/square yard) geosynthetic fabric with a HDPE drainage net.

Gas relief vents were installed on top of the lagoon closure and extend approximately 4 feet through the geomembrane into a gravel pack placed within the stabilized sludge mound to allow for the escape of any potential gas. Following the placement of the GDN and gas vents, 45 centimeters (18 inches) of cover soil and 15 centimeters (6 inches) of topsoil were placed on top of the stabilized mound to provide sufficient growth media to support a vegetative cover. At the conclusion of capping activities, perimeter drainage swales were constructed around the base of the slope to control and direct surface and infiltrating storm water run-off away from the lagoon closure.

3.5 Material Properties

The engineering properties of the materials that were processed, placed and used for construction of the respective OU-3 components were verified and maintained as part of the Quality Control and Quality Assurance protocols that were established prior to construction. The engineering properties of the materials that were used are summarized in Table 4, Table 5 and Table 6.

Table 4. Range of Engineering Index Properties of Materials Placed for OU-3 Closure

Material Type	Grain Size Distribution			Atterberg Limits		
	Gravel (%)	Sand (%)	Silt/Clay (%)	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index (%)
PDM	0	16 - 39	61 - 83	1 - 3	11 - 14	8 - 13
Stabilized Sludge Material	2 - 12	54 - 71	26 - 38	NT	NT	NT
Cover Soil	0 - 32	66 - 97	1 – 16	0 - 8	12 - 18	4 - 12
Top Soil	6	72	22	NT	NT	NT

Note: NT denotes not tested.

Table 5. Range of Density and Permeability Properties of Materials Placed for OU-3 Closure

Material Type	Placed Dry Density (kg/m^2)	Placed Moisture Content (%)	Permeability (cm/sec)
PDM	960 – 1,315	27 - 58	$1.6x10^{-6}$
Stabilized Sludge Material	1,490 – 1,685	15 - 17	NT
Cover Soil	1,870	13.6	$1.5x10^{-3}$

Note: NT denotes not tested.

Table 6. Range of Strength, Properties of Materials Placed for OU-3 Closure

Material Type	Direct Shear		Unconfined Compression,[2] (kg/m^2)
	Internal Friction,[1] (degrees)	Cohesion,[1] (kg/m^2)	
PDM	30	1,011	19,830
Stabilized Sludge Material	28 - 36	480 – 1,200	32,340 – 73,820
Cover Soil	NT	NT	NT

Note: 1. Shear values represent ultimate stress.
2. Reported values based on minimum 28 day compressive strength.
NT denotes not tested.

4.0 REMEDIATION STANDARDS ACHIEVED

Remedial activities related to OU-1, OU-2 and OU-3 have been completed consistent with the approved Work Plans. The site meets state non-residential cleanup criteria with a final development plan which supports warehouse and light industry.

4.1 OU-1 and OU-2 Engineering Controls

OU-1/OU-2 remedial efforts provided for the removal of contaminated soil, the utilization of these soils as part of the OU-3 closure activities and the engineered cap to isolate the remaining contaminants. The engineered cap consisted of the import of nearly 191,140 cubic meters (250, 000 cubic yards) of PDM to establish a barrier and provide the necessary fill to meet the desired development.

4.2 OU-3 Engineering Controls

OU-3 remedial activities included the stabilization of the lagoon sludge in a matrix with adequate geotechnical properties to support the engineered cap. The stabilization process included mixing 31,350 cubic meters (41,000 cubic yards) of on-site materials with proprietary chemical reagents to solidify the lagoon sludge and provide a suitable subsurface for an engineered cover.

Contaminant concentrations in soils above the NJDEP Residential Direct Contact Soil Cleanup Criteria (RDCSCC) will remain in place at the OU-3 closure. Therefore, engineering controls are required to prevent any potential exposure to contaminated media. The as-constructed engineering controls for the OU-3 closure consist of a geosynthetic liner and soil cover system with the following components:

- A textured geomembrane consisting of a 40-mil linear low-density polyethylene (LLDPE-T) placed over compacted and graded PDM material over the stabilized sludge and waste materials in the OU-3 closure at 5h:1v slopes;
- A geocomposite drainage network (GDN) placed over the LLDPE-T; and
- A final soil cover consisting of 18 inches of clean soil plus 15 centimeters (6 inches) of topsoil with seeding and erosion control.

5.0 SITE DEVELOPMENT CONSIDERATIONS

As previously discussed, the site was purchased by Viridian with the intent of developing it for future warehouse and light industry. The remedial strategy for meeting the environmental clean-up standards was integrated with the final development plan in mind and numerous aspects of the re-development plans were completed as part of the remedial activities. This integrated approach to remedial activities provided for an expedited development schedule, allowed for development infrastructure (i.e., storm sewers, detention basins, building pad construction, and site plan approval) and reduced both engineering and construction costs.

Figure 3 illustrates the final site development plan as currently envisioned with three warehouse buildings south of River Road and one warehouse building north of River Road.

Fig. 3. Aerial View of the Site with Proposed Warehouse Building Locations.

The following summarizes construction items completed as part of re-development activities:

- Placement of approximately 191,140 cubic meters (250,000 cubic yards) of structural fill needed for future site development imported and placed as part of the remedial activities;
- Construction of five on-site storm water detention basins needed for future site development installed as part of the remedial activities;
- Construction of future site building pads as part of the 'soil cap' necessary to meet the remedial action objectives;
- Installation of storm water sewers necessary for site development as part of the storm water management controls needed to complete the remedial activities;
- Preparation and approval of the Final Site Plan and development entitlements during the implementation of the remedial action;

- Utilization of engineering and site infrastructure necessary to complete the remedial activities to complete development activities;
- A one year jump on the site development as a result of integrating future site work as part of the remedial action;
- Management of unforeseen ‘environmental’ issues discovered during the completion of development activities;
- ‘Significant’ cost savings resulting from integration of environmental and development activities.

6.0 ACTUAL REMEDIAL AND CONSTRUCTION COSTS

The total estimated cost to complete the remedial action, including the soil removals and capping for OU-1 and 2 and the stabilization of lagoon sludge, consolidation with soil and crushed concrete, regrading, capping and perimeter drainage system for OU-3 was approximately $6 million. In addition, Viridian was able to integrate an additional $4 million of development related activities as part of the work completed alongside the environmental remedy.

7.0 CONCLUSIONS

As presented, the recognition of opportunity to implement creative geotechnical and environmental technology has provided further advancement in brownfields re-development.

Operable Units 1, 2 and 3 were remediated in accordance with the Administrative Consent Order (ACO) with the NJDEP; the NJDEP-approved the Revised RAWP prepared by Roux Associates, dated March 29, 2001 (Roux, 2001); the RAWP/RD Addendum prepared by Tetra Tech EM. Inc., dated November 15, 2005; (TtEMI, 2005) and the NJDEP approval of the Modification to the RAWP for closure of OU-3 (NJDEP, 2006). In accordance with the NJDEP TRSR requirements (N.J.A.C. 7:26E-8.2), a Deed Notice, will be filed with Burlington County Clerk’s Office and the NJDEP. The site closure activities at OU-3 are also in conformance with the requirements of the Burlington Township Planning Board.

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Chilled Mirror Measurements of Soil Water Retention Curves

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1. ABSTRACT

The capability of a chilled mirror device for measuring soil water retention curves was investigated on fine grained soils having plasticity indices ranging from about 15 to 80. The measurements were obtained over a wide range of moisture contents on each test specimen to avoid data scatter from material variability. The response times for equilibrating a soil specimen in the sealed chamber of the chilled mirror device varied from about 70 to 300 minutes, increasing with higher plasticities and lower water contents. Chilled mirror measurements of soil water retention curves on fine grained soils were obtained in days rather than the six to eight weeks required with conventional pressure plate methods. The semi-logarithmic display of soil water retention curves on low plasticity clays includes one linear segment between suction levels of 0.01 to 100 MPa and water contents ranging from air dry near the liquid limit of the material. The semi-logarithmic display of soil water retention curves on high plasticity clays can be approximated with a bi-linear relationship consisting of one relatively steep linear segment in the high suction range, and a second relatively flat linear segment in the low suction range. Some characteristics of these linear and bi-linear relationships are inconsistent with the assumptions underlying McKeen's expansive soil classification system. All of the samples tested show zero-water-content intercepts that are higher than McKeen's benchmark intercept of 5.25 log kPa. Two of the three samples exhibiting bi-linear soil water retention curves have slopes that fall in negligible categories for swelling potential, even though they have high plasticity indices, high activities that suggest the clay fraction is smectite, and substantial values of percent swell in the ASTM Swell Consolidation Test.

2. INTRODUCTION

The soil water characteristic curve (SWCC) describes the constitutive relationship between soil suction and soil water content. This curve varies widely with soil type. It also varies depending on whether the soil water content is increasing or decreasing. SWCCs are commonly referred to as soil water retention (SWRC) or soil water adsorption (SWAC) curves.

The SWCC has long been recognized as a useful concept for characterizing and predicting unsaturated soil behavior in terms of changes in suction that are caused by changes in the water content (w) of unsaturated soils. However, practical applications of this curve have been restrained by limitations in methods for measuring soil suction. The need for more rapid and reliable methods recently has stimulated geotechnical interest in a commercially available device for measuring total suction using the chilled mirror dew point technique (Gee et al. 1992, Petry and Bryant 2002 and 2007, Bulut et al. 2002, Leong et al, 2003, Lu and Likos 2004).

These studies examine the credibility of the chilled mirror dew point technique by comparing total suctions measured using chilled mirror devices with total suctions measured with filter paper and psychrometer techniques. Some of these references show very close agreement (Gee et al. 1992; Lu and Likos 2004). Others show significant discrepancies (Petry and Bryant 2002, Bulut et al. 2002, Leong et al, 2003).

A recent study (Patrick et al. 2007) shows these discrepancies can be attributed to (a) errors in chilled mirror total suction measurements due to incomplete equilibration in the sealed test chamber of the chilled mirror device, and (b) errors in estimated filter paper total suction values due to natural variations of the zero-water-content intercept in the log total suction vs. water content relationship.

The objectives of this study were (1) to clarify the time needed to equilibrate specimens in the sealed test chamber of the chilled-mirror device, and (2) to compare soil water retention curves (SWRCs) obtained using the chilled-mirror device with the assumptions underlying McKeen's expansive soil classification system (McKeen 1992).

3. BACKGROUND

Commercial chilled mirror devices of current interest to geotechnical researchers are the Decagon WP4 and WP4-T Dewpoint PotentiaMeters, hereafter referred to as the WP4 and WP4-T. These models are being marketed for measuring water potential (or suction) in soils and rocks, and represent a modification of previous devices termed *water activity meters* (models CX1 and CX2), which were used in the food service and soil science industries (Scanlon et al. 2002). These two machines are very similar with one exception: the WP4-T allows the user to control the temperature at which total suction measurements are made. Decagon claims that these machines can measure water potential from 0 to -60 MPa with an accuracy of ± 0.1 MPa from 0 to -10MPa and ±1% from -10 to -60 MPa (Decagon Devices, Inc. Website, 2004). Both of these devices were used in the studies cited above.

In Decagon's chilled mirror devices, a test specimen is inserted into a sealed chamber that contains a mirror together with means to chill and detect condensation on the mirror. The temperature at which condensation begins determines the water potential in the head space of the sealed chamber. When a test specimen is inserted into the chamber, moisture is transferred between the specimen and the head space until equilibrium is reached. At equilibrium, the water potential in the head chamber equals the water potential in the specimen. In soils, the water potential is usually called suction.

Petry and Bryant (2002) published research aimed at evaluating whether the Decagon WP4 PotentiaMeter could provide a more convenient and consistent way to measure soil suction, compared with the filter paper method (FPM) which was adopted as an ASTM standard test method (D5298) in 1994. They anticipated the need, and developed a protocol, for equilibrating a soil specimen with the headspace in a sealed specimen cup, prior to inserting the cup with specimen in the sealed test chamber of the chilled mirror device. Following an initial equilibration period of about 16 to 24 hours, they removed the sealed lid from the specimen cup and inserted the cup with soil specimen into the sealed test chamber of the device. Following the guidelines in Decagon's Operator's Manual, readings were taken and repeated until two consistent consecutive readings were obtained. This usually took between 5 and 10 minutes. They concluded that the WP4 is more practical, less costly in time and expense, and less likely to contain errors than the FPM. This conclusion has recently been further substantiated with a round-robin testing program involving three geotechnical companies from Texas, one each from Colorado and New Mexico, and geotechnical engineering laboratories in BYU and UMR (Petry and Jiang 2007)

In 1992, McKeen proposed a practical approach for classifying expansive soils and estimating their potential heave based in part on a semi-logarithmic version of the SWCC). McKeen derived this approach from extensive field studies and laboratory measurements of filter paper (FP) total suction versus water content and volume change relationships for expansive soils in Texas and adjacent states (McKeen 1981, 1985, 1992). These studies show the semi-log SWCC) tends to be a straight line between the levels of 98 MPa (6 pF) and 98 kPa (3 pF). These studies also show the extrapolated intercept for this relationship at zero-water-content (ZWC) is consistently near 5.25 log kPa (174,385 kPa), and that the slope of this semi-log SWCC varies substantially with the swelling potential of the material. McKeen used this "benchmark" ZWC intercept, together with his field and laboratory data, to develop the simplified classification criteria illustrated in Fig. 1. According to this figure, swelling potential varies inversely with the slope of the SWCC, $\Delta h/\Delta w$, where h is the suction in units of either log kPa or pF and w is the water content in g/g of dry soil weight. In Fig. 1, the slopes of the four boundaries between the five categories of swelling potential are -20, -13, -10, and -6 (log-kPa or pF).

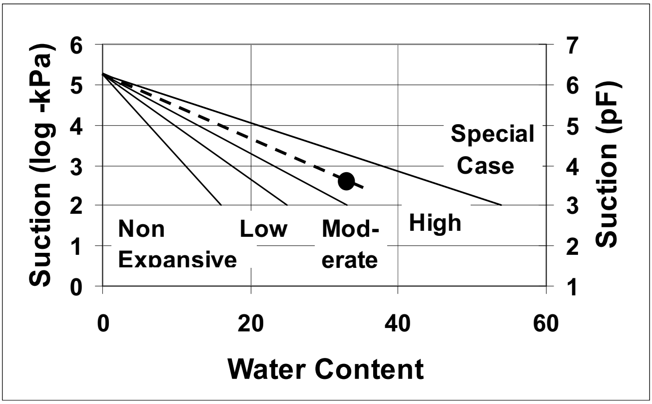

FIG. 1. McKeen's expansive soil classification system.

Fig.1 also illustrates how a log SWCC for a soil sample can be estimated from a single data point consisting of soil suction and water content measurements. The black circle represents such a data point on an expansive soil sample. The dashed line represents the water retention curve for this sample.

Likos et al (2003) investigated the validity of this assumed (ZWC) intercept for a suite of 80 shale specimens from the Colorado Front Range Corridor (Krosley 2000). At least four data points were obtained for each specimen, initially by measuring the total suction at the specimen's natural water content, and then by incrementally allowing the specimen to air dry and measuring the suction with a new piece of filter paper. The actual ZWC intercept for each specimen was found by regression. The results show the actual intercepts vary randomly from 4.65 to 6.4 log -kPa (5.65 to 7.4 pF). In contrast, the ZWC intercept assumed in McKeen's model is 5.25 -kPa (6.25 pF).

A more recent study by Patrick et al (2007) compared chilled-mirror soil suction measurements with those estimated from FPM soil suction measurements using the assumptions and concepts underlying McKeen's classification system. The results show the discrepancies are consistent with (a) possible errors in chilled mirror soil suction measurements due to incomplete equilibration in the sealed test chamber of the chilled-mirror device, and (b) possible errors in estimated filter paper soil suction values due to natural variations of the ZWC intercept in the log SWCC relationship.

4. MATERIALS AND METHODS

Table 1 summarizes the geotechnical properties of the six samples used in this study. These samples had previously been analyzed by Advanced Terra Testing Inc. (ATT) using ASTM procedures for grain size, plasticity, and swell-consolidation tests. Samples 1, 2, and 3 were from various locations in the United States. Samples 3, 4, and 5 were taken from the lower member of the steeply dipping Pierre Shale Formation about three miles north of Golden Colorado. Each sample was thoroughly remolded and moistened to a water content near its liquid limit and then allowed to equilibrate in a sealed glass jar for at least two or three days before testing.

Suction measurements were obtained with a Decagon WP4-T chilled mirror device in an isolated small room in the ATT Laboratory where temperature variations were usually less than a couple of degrees centigrade. For each sample, a test specimen was placed in a stainless steel specimen cup, weighed, and inserted into the sealed specimen chamber in the WP4-T device. Pairs of suction and water content measurements were then obtained at a series of water contents, or stages, on each sample ranging from near its liquid limit to air dry. After each suction measurement the specimen and its cup were weighed, air dried to allow an increment of water to evaporate, weighed again, and reinserted in the sealed specimen chamber. Following the final suction measurement, each specimen was oven dried and weighed again. These weight measurements allowed the water content of the test specimen to be determined at the beginning and the end of each suction measurement, in accordance with the conventional method for measuring water contents (ASTM D2216-98).

Table 1. Geotechnical Properties of Samples Used in This Study

Sample	1	2	3	4	5	6
% finer 200 sieve	-98	-- --	98	93	99	69
% Clay	55	-- --	95	34	55	43
Liquid Limit (LL)	58	101	32	62	68	37
Plastic Limit (PL)	22	23	20	17	19	19
Plasticity Index (PI)	36	79	13	45	50	18
Activity (PI/[%Clay])	0.65	-- --	0.14	1.32	0.90	0.42
Hygroscopic w (%)	5.4	5.3	2.4	5.3	3.1	0.9
% Swell	-- --	-- --	-- --	4.8	7.5	0.0
% Swell Surcharge (psf)	-- --	-- --	-- --	619	619	472
% Swell Surcharge (kPa)	-- --	-- --	-- --	30	30	23

Before and following the pairs of suction and water content measurements on each sample, the calibration of the device was checked by measuring the suction of a 0.5 N KCl solution sample, for which the suction is independently known. A fresh sample of Decagon's KCl Performance Verification Standard was used for each calibration check. The sample of KCl was placed in a plastic specimen cup and inserted into the sealed specimen chamber of the WP4-T device.

The procedures used to setup and calibrate the WP4-T device and to conduct soil suction measurements were obtained from Decagon's Operator's Manual, Version 2.1. According to this manual each suction measurement, which usually takes about 4 minutes, can be conducted in either normal or continuous modes. In the normal mode, the operator initiates each reading and records the result manually. In the continuous mode, the device automatically conducts suction measurements about every four minutes and documents each measurement on a computer. In this study, most of the suction data were obtained using the continuous mode and a computer. Thereby, extensive data were obtained for defining the variation of suction with time on the KCl Performance Verification Standard sample and also on the soil test specimen as it equilibrated with the head space in the test specimen chamber. The duration of continuous mode suction measurements was increased early in the testing program from ½ to about 24 hours.

5. EXPERIMENTAL RESULTS

Equilibration of Calibration and Soil Test Specimens

Fig. 2 shows the variation of suction with time for the 0.5 N KCl sample used to verify the calibration of the WP4-T prior to conducting soil suction measurements on Sample 4. The curve consists of a well defined band within which the values of over 300 continuous successive readings fluctuated. This figure shows about 70 minutes were needed for the suction to reach a constant value. Similar calibration tests were run before and after the series of soil suction measurements on most of the samples listed in Table 1. All of the calibration measurements show approximately the same behavior and the same equilibration response time.

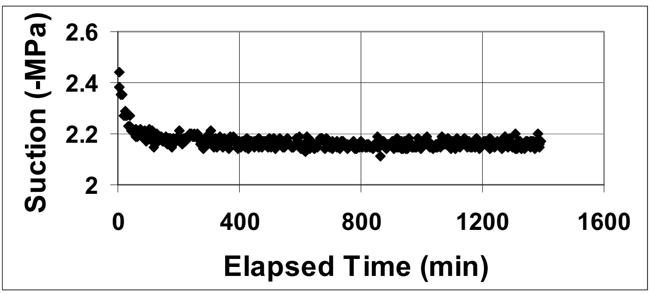

Fig 2. Equilibration of the 0.5N KCl sample used to verify the calibration of the WP4-T prior to soil suction measurements on Sample 4.

Table 2 summarizes the index properties and the equilibration data obtained on the Sample 4 test specimen at each stage of the test. The table also shows the equilibration data on the 0.5N KCl samples tested before and following the measurements on the test specimen. The index properties include the water content (w) and the liquidity index (LI = [w-PL]/[LL-PL]), where LL and PL are the liquid and plastic limits of the specimen, respectively, The characteristics of the equilibration data include (a) the equilibration time, (b) the initial, equilibrium, and final suction values, and (c) the loss of moisture from the test specimen during each of the tests.

These data show the three types of suction versus time curves illustrated in Fig 3, which consist of bands of continuous consecutive readings similar to those in Fig. 2.

The lowest curve illustrates the behavior of relatively wet samples having water contents generally between their plastic and liquid limits. Their response times are on the order of 100 minutes. Thereafter, the suction values remained approximately constant for periods on the order of 1440 minutes (24 hours). The middle curve illustrates samples having water contents between their plastic limit and hygroscopic moisture condition. This curve differs from the lower curve in two respects: longer response times (200 to 300 minutes) followed by a slow increase in suction with time. The upper curve was obtained on the sample after it had been air dried. This curve shows the suction gradually increased throughout the measurement period at a much slower rate than the preceding stages. Similar variations in suction versus time curves at decreasing water contents were obtained in all of the samples listed in Table 1.

The data in Table 2 show the weight of both the calibration specimen and the soil specimen decreased about 0.05g to 0.08g for all but the two driest sample stages. Since the dry soil weight of the sample 4 test specimen was 9.874g, the equivalent loss of moisture content during any but the driest sample stages was about 0.7%.

Table 2. Equilibration Data Obtained On Sample 4 at Each Test Stage

	Water Content (% dry soil wt)	Liquidity Index	Equilibra-tion Time (min)	Initial Suction (-MPa)	Equilib-rium Suction (-MPa)	Final Suction (-MPa)	Moisture Loss (g)
Calibration	-- --	-- --	70	2.45	2.19	2.16	-- --
1	59	0.94	100	0.47	0.21	0.19	0.075
2	53	0.80	100	0.63	0.25	0.20	0.071
3	48	0.69	100	0.57	0.30	0.30	0.069
4	37	0.44	100	1.35	0.50	0.45	0.071
5	30	0.28	100	1.75	0.90	0.85	0.064
6	24	0.14	100	3.00	1.80	2.00	0.068
7	17	-0.10	100	6.80	5.80	6.90	0.068
8	11	-0.15	300	33	23	27	0.055
9	6	-0.25	300	165	130	138	0.019
10	5	-0.28	300	246	247	248	0.001
Calibration	-- --	-- --	70	2.72	2.21	2.20	0.051

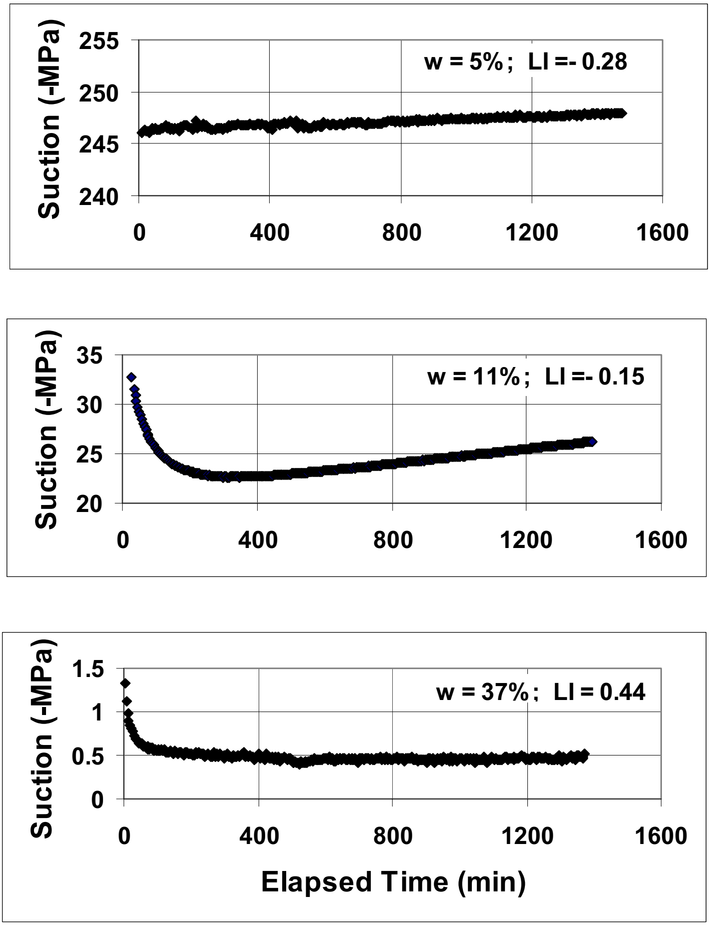

Fig. 3. Equilibration of Sample 4 at three test stages. The water content (w) and liquidity index (LI) are displayed for each stage.

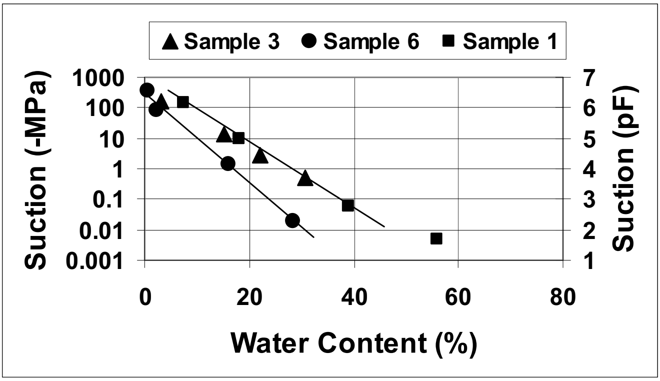

Fig. 4. Semi-log display of SWRC data on Samples 1, 3, and 6 together with linear trend lines through most of the data points for Samples 1 and 6.

Soil Water Retention Curves

The two types of soil-water retention curves (SWRCs) found in this study are displayed in Figs. 4 and 5. Each figure also includes two trend lines that are linear on the semi-log plot. In Fig. 4, the trend lines illustrate that the data for samples 1, 3, and 6 are linear between suctions of 98 kPa to 980 MPa, consistent with McKeen's assumption (1992). In Fig.5, the trend lines illustrate that the data for samples 2, 4, and 5 fall on more complex relationships. Fig. 6 illustrates that these

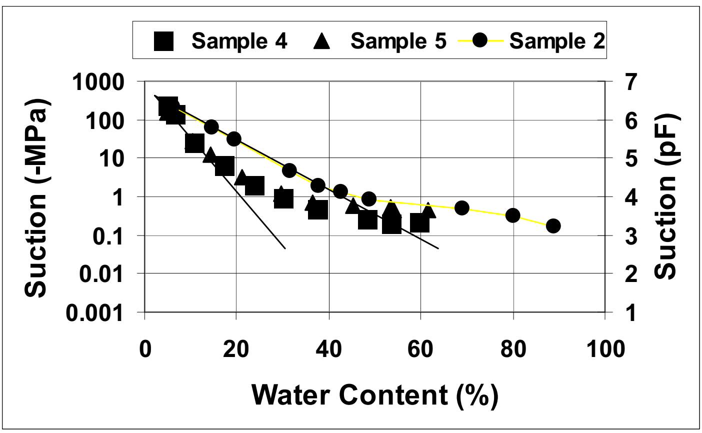

Fig. 5. Semi-log display of SWRC data on Samples 2, 4, and 5 together with linear trend lines through data points for Samples 2 and 4 in the range of low to very low water contents.

relationships can be approximated with two linear semi-log trend lines that are hereafter referred to as bi-linear.

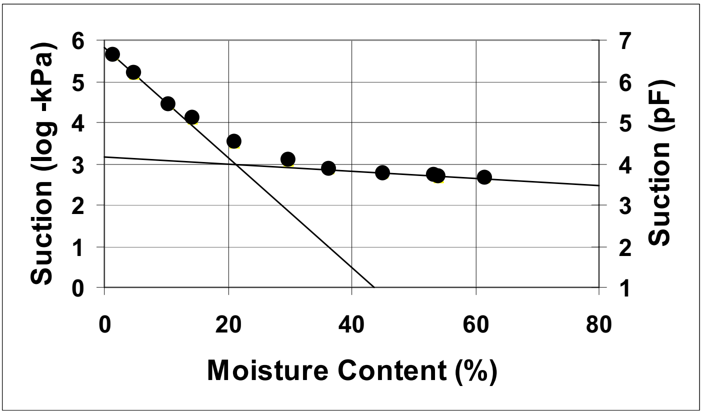

Fig. 6 Bi-Linear approximation of the SWRC data for Sample 5.

The characteristics of the SWRCs for all of the samples tested in this study are summarized in Table 3. This table presents the sample information in columns arranged with their PIs increasing from left to right. The SWRC characteristics include the slope and zero-water-content (ZWC) intercept for each of the regression lines through the linear array of data points at both the high suction range (HSR) and the low suction range (LSR) ends of the SWRC. They also include the water content (w) and liquidity index (LI) where the linearity ends on the HSR part the SWRC.

This table shows the extent to which the SWRC characteristics can be related to conventional geotechnical index and swell-consolidation properties. The three samples with the lower PI values appear to be consistent with HSR SWRC trend lines, as shown in Fig. 4, and the three samples with the higher plasticity index values exhibit bi-linear behavior, as shown in Figs. 5 and 6. In addition, the end of linearity on the HSR part of the SWRC occurs at relatively high LI values on low plasticity soils, and very low LI values on high plasticity soils. In contrast, the PI does not correlate with either the HSR SWRC slopes or the ZWC intercept values. The samples with the lower PI values have SWRC slopes ranging from -9.20 to -15.43. The samples with high PI values have slopes ranging from -6.56 to -16.29. In both the low and high suction ranges, the ZWC intercept values are higher than McKeen's assumed value of 5.25 log kPa.

Table 3. Characteristics of SWRCs for All the Samples Tested

Sample	3	6	1	4	5	2
% Clay	95	43	55	34	55	-- --
Liquid Limit (LL)	32	36.8	58	62	68	101
Plastic Limit (PL)	20	18.8	22	17	19	23
Plasticity Index (PI)	13	18	36	45	50	79
Activity (PI/[%Clay])	0.14	0.42	0.65	1.32	0.90	-- --
Hygroscopic w (%)	2.4	0.9	5.4	5.3	3.1	5.3
% Swell	-- --	0.0	-- --	4.8	7.5	-- --
Surcharge for % Swell (psf)	-- --	472	-- --	619	619	-- --
Surcharge for % Swell (kPa)	-- --	23	-- --	30	30	-- --
Slope of LSR SWRC (log -kPa)	-- --	-- --	-- --	-2.72	-0.87	-0.65
Slope of HSR SWRC (log -kPa)	-9.20	-15.42	-10.67	-16.29	-13.36	-6.56
ZWC Intercept (log -kPa)	5.51	5.51	5.93	6.18	5.83	5.74
End of HSR Linear SWRC (%w)	>32	>30	>48	>10	>12	>40
End of HSR Linear SWRC (LI)	>1.0	>0.8	>0.8	-0.15	-0.15	0.22

DISCUSSION

Equilibration Process

In Fig. 3, the equilibration process during the initial 300 to 400 minutes is consistent with the transfer of moisture between the test specimen and the headspace in the sealed test chamber of the chilled mirror device. The two lower curves show decreasing suctions as the humidity in the headspace increases after sealing the test chamber. The upper curve does not show a similar decrease because the sample was initially air dry, and hence in equilibrium with the laboratory environment.

However, the slowly increasing suctions following equilibration, as seen in the middle curve in Fig. 3, were not anticipated. These occurred primarily during test stages on specimens having water contents between their plastic limits and their hygroscopic moisture conditions where LI values are negative. To determine whether water vapor could be escaping from the sealed test chamber to the laboratory environment, weight measurements were begun before and following each test run on both the soil and 0.5N KCl solution specimens. The measurements in Table 2 show roughly the same moisture loss for all but the two driest test stages where the water vapor gradient between the headspace and the laboratory environment would be very small. Because the rates of suction increase following equilibration are small compared with the rates of change before equilibration, it appears reasonable to assume that, after equilibration, a test specimen remains in equilibrium with the headspace even though moisture is escaping from the head space at a slow rate.

Equilibration Time

Our approach for equilibrating soil test specimens with the head space of the sealed test chamber of the chilled-mirror device differs from the protocol proposed by Petry and Bryant (2002) in the following respect. In this study the entire equilibration process occurred after a cup with specimen was inserted in the sealed test chamber of the chilled-mirror device. In Petry and Bryant's protocol, a soil specimen is equilibrated for a 16 to 24 hour period in a sealed specimen cup prior to inserting the cup with specimen into the sealed test chamber of the chilled mirror device. The question arises whether these approaches are equivalent.

One concern regarding Petry and Bryant's protocol is the need to remove the seal and lid from the specimen cup prior to inserting the cup with specimen into the test chamber of the chilled mirror device. It seems unlikely that disturbance of the vapor pressure in the headspace of the specimen cup could be avoided when the sealed lid is removed from the cup before inserting the cup with specimen into the test chamber. In this regard, we have observed that allowing a specimen to equilibrate in a sealed specimen cup, regardless of location, can be accompanied by the accumulation of condensed water on the surface of the sealed specimen lid. This water is no longer a part of either the soil specimen or the headspace of the sealed test chamber of the chilled-mirror device.

Another concern regarding Petry and Bryant's protocol and Decagon's Operator's Manual is the guideline that suction readings be taken repeatedly until two consistent consecutive readings were obtained. In this study we found that successive readings during a 24 hour period fluctuated appreciably within a well defined band. It follows that the equilibration time needs to be defined by the behavior of the band, and not by the consistency of consecutive readings.

Both of these concerns were avoided in this study by monitoring the equilibration process continuously in the sealed specimen chamber of the chilled-mirror device using the continuous mode for the device and a computer to log the individual readings.

Comparison of Soil Water Retention Curves with McKeen's Expansive Soil Classification System

Fig. 4 and Table 3 show the SWRCs for samples 1, 3, and 6 which have relatively low plasticity indices (13, 18, and 36), and activities (0.42 & 0.65). These SWRCs are generally consistent with the linearity of the suction versus moisture content relationships in McKeen's expansive soil classification system illustrated in Fig. 1 (McKeen 1992). The log suction versus water content data for each sample are linear over suctions ranging from 98 MPa to 98 kPa (3-6 pF), and water contents ranging from their shrinkage limits to near their liquid limits. The slopes of these relationships, which range from -9.2 to -10.7 log kPa, are close to the boundary slope (-10) between the medium and high categories of swelling potential in Fig. 1.

In contrast, Fig. 5 and Table 3 show the SWRCs for samples 2, 4, and 5 have more complex relationships that can be approximated with bilinear relationships, as illustrated for sample 5 in Figs. 5 and 6. Moreover, two of these samples (4 and 5) have SWRC slopes in the high suction range that deviate from linearity at relatively high suctions (>10 MPa) and relatively low water contents (<15%). In addition, the high suction range SWRC slopes of samples 4 and 5 are inconsistent with McKeen's classification system because they fall in low to negligible categories for swelling potential, even though they have PIs of 45 and 50, high activities (0.9 and 1.3) suggesting the clay fraction is smectite, and they exhibit substantial values of percent swell in the ASTM Swell Consolidation Test (D4546 method B modified).

All of the samples tested show ZWC intercepts that are higher (5.51 to 6.18 log kPa) than McKeen's benchmark intercept of 5.25 log kPa. This result is consistent with the previous study by Likos et al (2003) where ZWC intercepts ranging from 4.65 to 6.4 log kPa were obtained on 80 expansive soil samples from diverse locations in the Colorado Front Range Corridor.

Advantages of Chilled-Mirror SWRC Measurements

Petry and Bryant (2007) have recently demonstrated that chilled-mirror suction measurements take far less time and are less difficult to perform successfully, compared with filter paper measurements concerning expansive soil behavior. Our results are consistent with this conclusion, even though our equilibration response times are much longer than those mentioned in Decagon's Operator's Manual and in previous geotechnical studies concerning the WP4 and WP4-T potentiameters.

This study illustrates how chilled-mirror SWRC measurements can be used for evaluating whether the expansive behavior of a specific soil is consistent with the assumptions underlying McKeen's expansive soil classification system, and may be useful for finding improvements to his system model. The need for such evaluations is demonstrated by the discrepancies between McKeen's benchmark ZWC intercept and the measured values that have been reported in this study and in the previous study by Likos et al (2003) for clay shales in the Colorado Front Range Corridor The need is further demonstrated by the finding in this study that, at least some of the relatively high plasticity clays in the Colorado Front Range Corridor exhibit bi-linear behavior, and that the slopes of these relationships do not fall in reasonable categories of swelling potential in Fig. 1.

This study shows that chilled mirror SWRC measurements of fine grained soils can be obtained in far less time than with conventional pressure plate methods. With the latter, in our experience, six to eight weeks are generally required. In this study, about two weeks were needed to clarify the equilibration process for about 24 hours at each of 10 data points on moderate to highly plastic soil specimens. For practitioners, we anticipate a chilled mirror SWRC could be generated in a few days because the actual equilibration times varied from about 1 to 5 hours, and that fewer data points would suffice for many applications.

Another advantage of chilled-mirror SWRC measurements, compared with conventional pressure plate methods, is that the volume of the test specimen does not need to be controlled. In conventional pressure plate technology, suction measurements are affected by changes in the volume of a test specimen that tend to occur as soil water is either inserted or expelled from the specimen. Errors and uncertainties from such errors are of particular concern for expansive soils.

CONCLUSIONS

The capability of a chilled mirror device for measuring soil water retention curves was investigated on fine grained soils having plasticity indices ranging from about 15 to 80. The measurements were obtained over a wide range of moisture contents on each test specimen to avoid data scatter from material variability.

The results demonstrate that (1) the response time for equilibrating soil suction at each water content varies from about 70 to 300 minutes, increasing with higher plasticities and lower water contents, (2) chilled mirror measurements of soil water retention curves on fine grained soils can be obtained in a few days rather than the six to eight weeks required with conventional methods, (3) the semi-log display of soil water retention curves on low plasticity clays includes one linear segment between suction levels of 0.01 to 100 MPa and water contents ranging from air dry near the liquid limit of the material, and (4) the semi-log display of soil water retention curves on high plasticity clays is more complex. It can be approximated with a bi-linear relationship consisting of one relatively steep linear segment in the high suction range, and a second relatively flat linear segment in the low suction range.

One inconsistency between these linear and bi-linear SWRCs and the assumptions underlying McKeen's expansive soil classification system is that all of the samples tested show ZWC intercepts that are higher than McKeen's benchmark intercept of 5.25 log kPa. This discrepancy can be avoided by finding the actual intercept with chilled mirror SWRC measurements. Another inconsistency found in this study is that two of the three samples exhibiting bi-linear SWRCs have slopes that fall in negligible categories for swelling potential, even though they have high plasticity indices, high activities that suggest the clay fraction is smectite, and substantial values of percent swell in the ASTM Swell Consolidation Test. Because these samples came from the lower Pierre shale near Golden CO, this inconsistency suggests that McKeen's expansive soil classification system may not be directly applicable to all of the soils in the Colorado Front Range Corridor.

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Modeling the Effects of Intrinsic Damping in Soil-Structure Interaction

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ABSTRACT: The dynamic behavior of a soil-structure infrastructure system is governed by its elastic, inertial, and dissipative characteristics. Finite element procedures for representing the stiffness and mass of an infrastructure system's individual components and their assembly into a global system representation have been well established. However, means for representing intrinsic damping, a material's capacity for mechanical energy dissipation, have lagged in comparison. This is disadvantageous in geotechnical studies of soil-structure interaction, where it is necessary to preserve the distinct dissipative characteristics of both the natural earth and manmade construction materials. In this paper, equivalent linear models for mathematically representing a soil's intrinsic damping based upon common geotechnical laboratory procedures are presented. Finite element procedures for using these models in representing nonuniform intrinsic damping in multi-degree-of-freedom soil-structure systems are reviewed. A representative soil-structure system is analyzed to illustrate the applications of the procedures the differences in their predicted responses. It is shown that three independent equivalent linear models based upon three different theoretical premises result in virtually identical dynamic predictions when uniform intrinsic damping is examined. However, when nonuniform intrinsic damping is considered, the three methodologies result in widely divergent responses. Reasons for the disagreements in responses are discussed based upon the analytical simplifications and assumptions made in each model. Recommendations are made to help guide the analyst in using these procedures for modeling nonuniform intrinsic damping in soil-structure systems.

INTRODUCTION

Many civil engineering structures must be considered in conjunction with the surrounding soil to predict their response to dynamic design loads (Safak 1995, Stewart et al. 1999, Feriani et al. 2000, Snieder and Safak 2006). Examples include tunnel linings installed through natural earth materials subjected to the repeated passage of subway trains; short, stiff structures such as nuclear power reactors constructed in seismically active areas; and residential homes situated near highly trafficked roads that may experience damaging effects from ground-borne vibration. Additionally, current trends in geotechnical engineering research and practice are moving towards the improved characterization and retrofitting of existing critical soil-structure infrastructure systems by working in tandem with structural engineers and geologists in considering infrastructure behavior from a holistic, system-wide view. Dynamic soil-structure interaction may therefore represent an important design consideration for a multitude of new or rehabilitated infrastructure systems for practicing geotechnical engineers and researchers.

In predicting the dynamic response of a soil-structure system, it is necessary to preserve the individual materials' contributions towards the overall response. Each material contributes its own elastic, inertial, and dissipative characteristics, represented by its stiffness, mass, and intrinsic damping, respectively. Finite element procedures for representing the stiffness and mass of the soil and structural components have been well established (Bathe 1996). However, modeling techniques for intrinsic damping are considered ambiguous at best (Raggett 1975, Bergman and Hannibal 1976, Ungar 1992, Feriani and Perotti 1996). In 1973, Roesset et al. noted that, "The most troublesome aspect of analyzing soil-structure interaction is defining damping in the system in a useful, meaningful way;" 35 years later, and this issue still has yet to be satisfactorily resolved.

In the following paper, background on the nature of intrinsic damping and experimental procedures for determining a soil's intrinsic damping will be discussed. Methods for using these observations in finite element modeling of dynamic soil-structure interaction will be presented. These methods include nonclassical damping matrix assembly, weighted modal analysis, and linear hysteretic damping. A representative example of a soil-structure system will be used to illustrate the effects of the assumptions of the intrinsic damping models upon overall system response. Recommendations on using these methods for soil-structure interaction are then presented, based upon the limiting assumptions invoked in the models.

BACKGROUND

An infrastructure system's dynamic behavior will be heavily influenced by its materials' intrinsic damping characteristics. Simiu and Scanlan (1972) present an example in the analysis of a high-rise, modeled using one material with one intrinsic damping characteristic represented by a critical viscous damping ratio. Their comparison of the structure's predicted peak displacements and accelerations under a dynamic load for two critical viscous damping ratios is summarized in Table 1.

Table 1. Effects of Intrinsic Damping on System Response (after Simiu and Scanlan, 1972)

Response Quantity	Critical Viscous Damping Ratio = 1.0%	Critical Viscous Damping Ratio = 5.0%
Peak Displacement	1.49 m	1.18 m
Peak Acceleration	0.16 m/s^2	0.069 m/s^2

While the influence of intrinsic damping on system behavior is readily apparent, methods for modeling its causes and effects are not. A material's intrinsic damping is not the result of a unique phenomenon. Intrinsic damping more accurately refers to a collection of multiple atomic-level actions, such as interface friction and internal hysteresis, which results in an overall, observable effect (Bishop 1955, Lazan 1968, Bergman and Hannibal 1976, Ungar 1992). It is currently impractical for typical civil engineering purposes to directly link these atomic actions to system response.

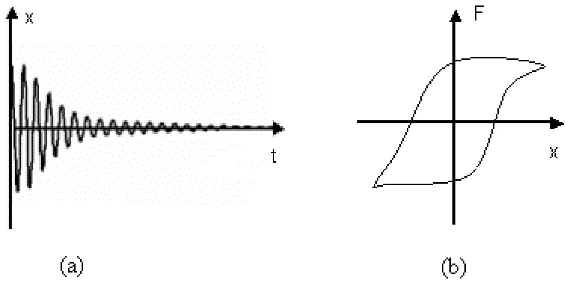

FIG. 1. Schematic of observations of intrinsic damping from (a) resonant column procedures and (b) dynamic cyclic triaxial tests.

A material's intrinsic damping is therefore usually approximated using data from one of several experimental procedures. For soil materials, two common laboratory procedures for determining a measure of intrinsic damping are resonant column and dynamic cyclic triaxial tests (Hardin 1965, Hardin and Black 1968, Whitman 1970b, Lai and Rix 1998, Kramer 1999, ASTM 2000). Both are single-degree-of-freedom (SDOF) tests that generate a measure of SDOF dissipation. A resonant column test involves the torsional, axial, or flexural excitation of a soil sample contained within a triaxial cell. The general procedure for intrinsic damping measurement involves the sample being excited to resonance, at which point the excitation is abruptly stopped and the decaying free vibration response recorded (Fig. 1a). From this decaying response, a logarithmic decrement, L, may be observed as:

$$L = \ln\left(\frac{x_n}{x_{n+1}}\right) \tag{1}$$

where x_n represents the amplitude of one peak and x_{n+1} the amplitude of the subsequent decaying peak. L is an indicator of the dissipative characteristics of the soil material, as it measures a relative reduction in dynamic motion due to the internal friction of the soil material. A dynamic cyclic triaxial test involves the cyclic straining of a soil sample to generate dynamic hysteresis loops displaying forcing load (F) vs. SDOF displacement (x) (Fig. 1b). Hysteresis loops reflect deviations from perfect elasticity; the amount of energy dissipated per cycle, D, may be observed as the area contained within the hysteresis loop. It has been noted that, for soil materials, the hysteresis loops are approximately constant with respect to forcing frequency over a wide range of frequencies, indicating that the amount of energy dissipated in a soil is dependent only upon the magnitude of the excitation and not its speed (Dobry et al. 1971, Lai and Rix 1998).

It is immediately apparent from the hysteresis loops that the stress-strain behavior of a soil is nonlinear, indicating that the amount of energy dissipated is also nonlinear. Although nonlinear intrinsic damping models exist, equivalent linear models are often used instead. In an equivalent linear model, the nonlinear hysteresis loop is replaced by an elliptical loop with the same area and slope (Lazan 1968, Whitman 1970b, Finn 1988, Jones 2001). Although the true stress-strain path is lost, this is often considered a reasonable approximation, as the amount of energy dissipated per cycle as well as the limiting case of linear elasticity are preserved. Additionally, the use of an equivalent linear model results in increased mathematical tractability and ease in interpretation of experimental data: the intrinsic damping may be represented as an equivalent force written within the context of a linear ordinary differential equation instead of a series of nonlinear, incremental equations, and material properties may be directly extrapolated from laboratory tests without considering the geometric configuration of the experimental system. Equivalent linear models have therefore been adopted for their mathematical convenience in as opposed to their accuracy in describing stress-strain paths or the fundamental nature of the atomic actions involved in intrinsic damping.

Two of the most common SDOF equivalent linear models are the Kelvin-Voight (KV) model and the linear hysteretic (LH) model. The restoring force for a KV model is given as:

$$F = kx \pm c\varpi\sqrt{R^2 - x^2} \tag{2}$$

where k represents system stiffness, c a viscous dashpot coefficient, R the maximum amplitude of cyclic vibration, and x the displacement of the system. Eq. 2 is composed of elastic and dissipative components that trace an elliptical loop of area D. D may be found by integrating Eq. 2 over one cycle (x = 0 to x = $\frac{2\pi}{\varpi}$) as:

$$D = \pi c\varpi R^2 \tag{3}$$

Eq. 3 indicates that the dissipative characteristics of the KV model are frequency-dependent. As the cyclic driving frequency decreases, the area of the loop decreases, indicating that dissipation is dependent not only on the magnitude of the cycle loading but also the rate at which the material is loaded. Although this model is attractive in that it results in a mathematical model with a closed-form analytical solution in the time domain, its frequency dependence does not accurately describe the behavior of many civil engineering materials, particularly soils, which have been observed to dissipate energy independent of forcing frequency.

The SDOF LH model was developed in response to this divergence from observed behavior. The LH model redefines the dashpot coefficient as:

$$c = \frac{h}{\varpi} \tag{4}$$

such that the restoring force may now be described as:

$$F = kx \pm h\sqrt{R^2 - x^2} \tag{5}$$

The dashpot coefficient given by Eq. 4 is inversely proportional to the forcing frequency, which, when substituted into Eq. 3, leads to a constant amount of energy dissipated per cycle. However, although the LH model more closely adheres to observed behavior for soil materials, it is defined only for harmonic motions of known driving frequencies. Transient vibration is not clearly defined in the time domain; frequency domain analysis, whereupon signals are represented as weighted summations of harmonic signals, must be performed. Eq. 5 is therefore more accurately described in complex modulus notation as:

$$F = kx + ihx \tag{6}$$

where x is expressed is a harmonic motion of generic forcing frequency, ϖ. It can be seen that the model is dependent upon the existence of an external harmonic loading, rendering the interpretation of a free vibration condition incomplete. Additionally, many researchers have noted that the LH model displays indications of noncausal behavior; the predicted response slightly anticipates the forcing function within the time domain (Scanlan 1970, Crandall 1970, Inaudi and Kelly 1995, Feriani and Perotti 1996). This mathematical anomaly has been well documented, but satisfactory physical interpretations have yet to be developed.

Therefore, it is important to note that the equivalent linear KV and LH models have limitations in their applicability and mathematical rigor, which result from the inaccuracy in describing atomic dissipation as an overall force within the scope of linear ordinary differential equations. In discussing equivalent linear models, Scanlan (1970) notes, “we do not in general pretend that a given mathematical model for damping is more than a poor crutch, yielding perhaps acceptable results in limited ranges, but certainly not implying any detailed explanation of the underlying physics.” The complementary pros and cons of the KV and LH model are listed in Table 2.

Table 2. Comparison of the Kelvin-Voight and Linear Hysteretic Models

Model	Pros	Cons
Kelvin-Voight	_Mathematical clarity:_ Closed form, analytical solutions are readily available in both the time and frequency domains for free vibration and any forced vibrations.	_Divergence from observed behavior:_ Model predicts that the amount of energy dissipated per cycle is dependent upon external frequency.
Linear Hysteretic	_Agreement with observed behavior:_ Model predicts that the amount of energy dissipated per cycle is independent of forcing frequency.	_Mathematical ambiguity:_ Analysis of forced, nonharmonic vibration must be performed within the frequency domain. There is no definition for a free vibration solution. The model also displays mathematical noncausality.

MODELING TECHNIQUES AND CHALLENGES

Relationship Between Geotechnical Experimental Data and Mathematical Models

Equivalent linear damping models represent restoring forces that may be used in conjunction with d'Alembert forces to represent the three components necessary in describing dynamic behavior: elasticity, dissipation, and inertia. These three components combined represent SDOF oscillators. The challenge for the geotechnical engineer is to relate the SDOF experimental data to an appropriate SDOF mathematical model to represent a soil material's intrinsic damping. In this manner, the geotechnical engineer may examine the effects of the soil foundation's dissipative contributions upon the behavior of the overall soil-structure system.

Common representative intrinsic damping values that may be generated from the combination of experimental data and equivalent linear models are critical viscous damping ratios (ξ) and constant loss factors (η). Critical viscous damping ratios imply the usage of resonant column procedures and the KV model, while loss factors imply the usage of dynamic cyclic triaxial tests and LH models. These relationships occur due to the nature of the two modeling techniques as previously discussed: the KV model is defined for free vibration, which is what is measured during a resonant column procedure, while the LH model describes frequency-independent damping, which is what is observed in a dynamic cyclic triaxial test. However, these quantities may be "translated" between the two mathematical models. This is particularly important, as ξ values, not η values, are traditionally used in reference materials to describe intrinsic damping characteristics of common construction materials.

The critical damping ratio represents a "per cycle" measure of the logarithmic decrement as measured from the free vibration response of a resonant column procedure:

$$\xi = \frac{1}{2\pi} L \tag{7}$$

The critical viscous damping ratio is defined in conjunction with a SDOF KV oscillator; the equation of motion is expressed in the time domain as:

$$m\ddot{x}(t) + c\dot{x}(t) + kx(t) = P(t) \tag{8}$$

where c and k are defined as previously, and m, x(t), and P(t) are the system's mass, the time-varying displacement response, and the time-varying external forcing function, respectively. The equivalent frequency domain expression is represented as:

$$(-\varpi^2 m + i\varpi c + k)x(\varpi) = P(\varpi) \tag{9}$$

The critical viscous damping ratio may be related to the viscous dashpot in a SDOF KV oscillator by:

$$c = 2m\omega\xi \tag{10}$$

Where ω is the natural frequency of the single-degree-of-freedom system under consideration:

$$\omega = \sqrt{\frac{k}{m}} \tag{11}$$

This value for c is based upon the experimental observation of the logarithmic decrement and may be directly used in the mathematical representation of dissipation.

An experimentally observed loss factor from a dynamic cyclic triaxial test may be defined as:

$$\eta = \frac{D}{2\pi U} \tag{12}$$

This is a normalized, per cycle measurement of dissipation, where U, the maximum stored strain energy, is defined as:

$$U = \frac{1}{2} kR^2 \tag{13}$$

D, the observed area of the hysteresis loop is independent of frequency; this may therefore be used in conjunction with a SDOF LH oscillator. A SDOF LH oscillator is properly defined in the frequency domain only; its equation of motion is given as:

$$(-\varpi^2 m + ih + k)x(\varpi) = P(\varpi) \tag{14}$$

Therefore, using Eq. 3,4, 12, and 13 the loss factor for a SDOF LH oscillator is defined as:

$$\eta = \frac{h}{k} \tag{15}$$

And Eq. 14 for a SDOF LH oscillator may be rewritten as:

$$(-\varpi^2 m + k(1 + i\eta))x(\varpi) = P(\varpi) \tag{16}$$

The SDOF LH oscillator utilizes a frequency independent complex stiffness quantity that incorporates both elasticity and dissipation; η may be observed from a dynamic cyclic triaxial test and be directly used in this model.

To translate between the two quantities, it can be shown that ξ may be related to η by assuming equal, normalized measures of energy dissipation at system resonance; as the transfer function for damping force is governed near and around resonance, this is a reasonable assumption. If this assumption is made, it can be shown, using Eq. 3, 7, 10, 12, 13, and 15 that:

$$\eta = 2\xi \tag{17}$$

Therefore, experimental or reference data defined with respect to either model may be translated for usage in the other model. Fig. 2 summarizes the procedures for determining how data may be used to mathematically model intrinsic damping using either the KV or LH oscillators.

It is important to note that, in this study, a loss factor is defined as derived from experimentally observed quantities that are constant with respect to frequency; this is often the case for natural earth materials. Other formulations involve the conversion of experimentally observed critical viscous damping ratios into mathematically modeled, frequency-dependent loss factors, leading to a more general viscoelastic definition. As this paper is focused towards the practical application of experimental geotechnical data in mathematical models, this discussion is omitted.

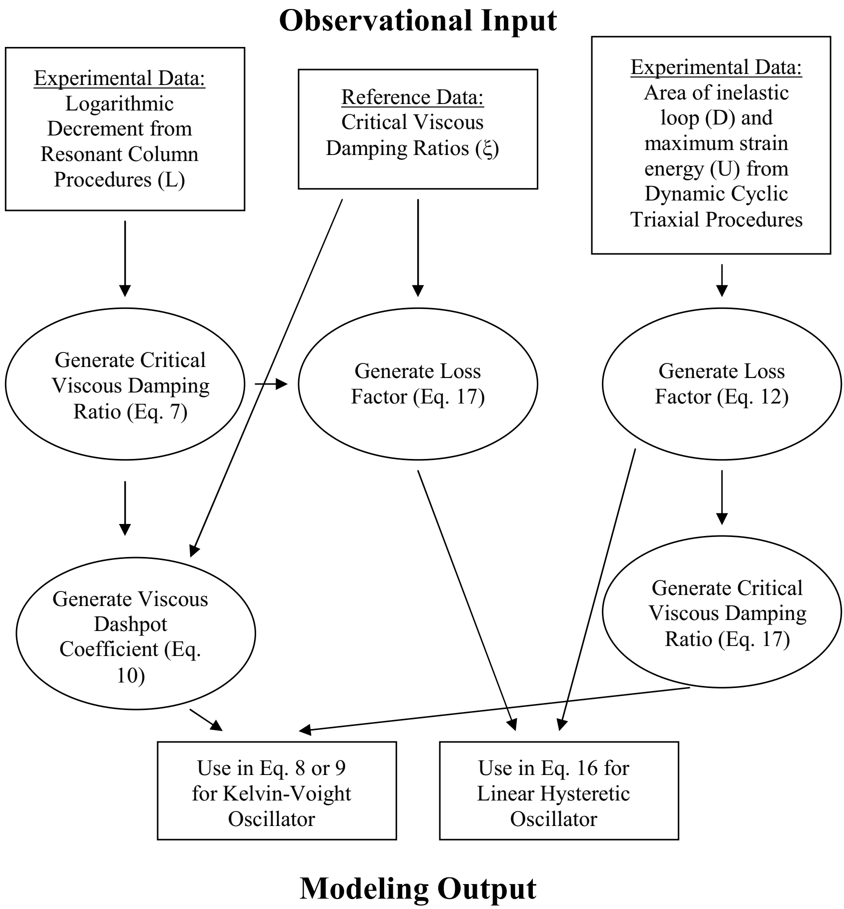

FIG. 2. Flowchart for Using Observed Geotechnical Data in Equivalent Linear Models of Intrinsic Damping.

Extension to Multi-Degree-of-Freedom, Soil-Structure Systems

To model a realistic soil-structure infrastructure system, a multi-degree-of-freedom (MDOF) model typically must be used. Techniques for extending SDOF representations of stiffness and mass towards MDOF stiffness and mass matrices have been well-defined using the finite element method; however, equivalent techniques for MDOF intrinsic damping matrices are poorly justified in comparison. The scalar dissipative quantities ξ and η for the SDOF KV and LH models are determined by relating the SDOF mathematical models to SDOF experimental data; there are no analogous, accepted experimental techniques for MDOF models. A summary of three existing proposed MDOF techniques (nonclassical damping; weighted modal analysis; and linear viscoelastic, linear hysteretic representation) for capturing the dissipative characteristics of both soil and structural materials in a composite system is as follows.

Nonclassical Damping: In nonclassical damping matrix assembly, individual proportional intrinsic damping matrices are assembled for the soil and the structure with coefficients of stiffness and/or mass proportionality based upon an assumed range of natural frequencies of the soil-structure system's undamped mode shapes (Chopra 2001). The soil and the structure's intrinsic damping matrices are then assembled via typical finite element procedures into a global intrinsic damping matrix for the entire system. However, it has been noted that the rationale for describing intrinsic damping as a weighted sum of the stiffness and mass matrices is performed for mathematical convenience, not due to an actual relationship between dissipation and stiffness and/or mass. Bergman and Hannibal (1976) note that proportional damping "rarely, if ever, realistically models actual damping characteristics" and relies upon user judgment in determining an appropriate range of frequencies. Nonclassical damping matrix assembly may also result in complex mode shapes that are not the same as those for the undamped case. Although nonclassical damping has acknowledged drawbacks in appropriately modeling multiple intrinsic damping characteristics, variations on it have been widely adopted and worked into commercial finite element codes as the procedure can easily be incorporated into finite element frameworks.

Weighted Modal Analysis: Whitman (1970a) describes an intuitive procedure initially proposed by Biggs for incorporating both soil and structural materials' intrinsic damping contributions by using an energy approach and modal time history analysis. The basic principle behind this methodology involves the extension of the definition of a material loss factor to a system loss factor, with a composite system comprised of j distinct components defined by a modal system loss factor given by:

$$\eta_{system} = \frac{\sum \eta_j U_j}{\sum U_j} \tag{18}$$

A system loss factor is determined for each mode shape, where the strain energies of each component are determined based upon the deformations implied in the mode shape. Using this system loss factor, modal analysis may be performed, with each mode shape idealized by a SDOF KV oscillator. The intrinsic damping is then inserted as a modal damping ratio into each mode's equation of motion through the relationship between the system loss factor and the critical viscous damping ratio (Eq. 17), and the system may be analyzed via traditional system-level modal analysis. Curry (1979) analytically verifies this methodology for simple, one-dimensional parallel systems but notes that it overestimates the composite intrinsic damping for simple, one-dimensional series system, thus underestimating the predicted magnitude of the dynamic response. Gasparini et al. (1981) specifically note that, "if one of the loss factors is large and different from the others, the procedure may grossly overestimate the effective damping of series systems." This may be a serious concern in for the goetechnical engineer in analyzing soil-structure interaction problems, where the intrinsic damping values of soil materials may be several times larger than manmade construction materials.

Linear Hysteretic Representation: Methods for incorporating intrinsic damping on the material level may be expressed by interpreting the LH model in terms of linear viscoelastic constitutive models. Linear viscoelastic materials are materials whose stress-strain behaviors follow the following relationship:

$$\begin{aligned} P_1\sigma_{ij} &= Q_1\varepsilon_{ij} \\ P_2\sigma &= Q_2\varepsilon \end{aligned} \tag{19}$$

Where σ_{ij} and ε_{ij} are the deviatoric components of the stress and strain matrices; σ and ε are the spherical components; and P_1, Q_1, P_2, and Q_2, are linear differential operators with coefficients defined by the chosen rheological model (Gasparini et al. 1980). Once these operators are established, Eq. 19 may be rewritten for a material undergoing a cyclic loading of forcing frequency ϖ in the form of a complex modulus:

$$\underset{\sim}{\sigma} = \underset{\sim}{E}(\underset{\sim}{I} + i\underset{\sim}{N}(\varpi))\underset{\sim}{\varepsilon} \tag{20}$$

Where $\underset{\sim}{\sigma}$ represents stress, $\underset{\sim}{E}$ the material's elasticity matrix, $\underset{\sim}{I}$ an identity matrix of appropriate dimensions, i the imaginary constant, and $\underset{\sim}{N}(\varpi)$ a diagonal matrix representing the frequency-dependent phase lag between stress and strain, $\underset{\sim}{\varepsilon}$. Once the complex moduli are determined for the soil and structural materials, they may be incorporated into the derivation of the component elements' stiffness matrices. These stiffness matrices may then be assembled into a global infrastructure representation through typical finite element procedures. It is assumed that the complex global stiffness will take into account both the elastic and dissipative effects of each material and that the individual intrinsic damping contributions will be preserved.

To use the linear viscoelastic framework for soils analysis, a complex modulus representation based upon a frequency independent, linear hysteretic model may be used. This results in the frequency independence noted in soil materials, and the translation of experimental data from typical soil testing devices into the mathematical framework is relatively straightforward. A complex modulus representation of linear hysteretic damping is one in which the operators defined in Eq. 19 are:

$$\begin{aligned} P_1 &= 1 \\ Q_1 &= 2G + \frac{2G\eta}{\varpi}\frac{d}{dt} \\ P_2 &= 1 \\ Q_2 &= 3K + \frac{3K\eta}{\varpi}\frac{d}{dt} \end{aligned} \tag{21}$$

Where G is the shear modulus of a material, K the bulk modulus, and η a constant loss factor. If the operators are mathematically manipulated into a complex modulus representation, Eq. 20 yields:

$$\underset{\sim}{\sigma} = \underset{\sim}{E}(\underset{\sim}{I} + i\underset{\sim}{N})\underset{\sim}{\varepsilon} \tag{22}$$

Where the diagonal matrix representing the phase lag is now frequency-independent and defined as:

$$\underset{\sim}{N} = \begin{bmatrix} \eta & 0 & 0 & 0 \\ 0 & \eta & 0 & 0 \\ 0 & 0 & \ddots & 0 \\ 0 & 0 & 0 & \eta \end{bmatrix} \tag{23}$$

These constant loss factors may be readily determined from the SDOF experimental and modeling procedures as previously described. Once this complex modulus is assembled for the individual soil and structural elements, it may be used to generate stiffness matrices for each element that may then be assembled together into a global system representation, incorporating both elastic and dissipative characteristics.

It is important to note that every technique proposed involves simplifying assumptions that obfuscate the true nature of the component dissipative characteristics. These techniques additionally have not been experimentally validated for real situations of soil-structure interaction, casting doubt upon their usage as predictive tools for design.

ANALYTICAL STUDIES

To illustrate the assumptions of the proposed MDOF modeling techniques and their resulting implications, an idealized example soil-structure situation is examined (Fig. 3). The structure is a relatively stiff, squat structure, one in which soil-structure interaction may play a significant role. Plane frame elements are used to assemble the steel frame structure. The columns are W12X136 steel beams and the girder is a W24X76. The frame structure rests upon concrete mat and a dense, dry sand foundation, both of which are discretized using linear plane strain elements. The sand foundation is assumed to overlay a much stiffer soil layer. The relevant material properties and input data required to analyze this system using all four MDOF techniques previously described are summarized in Table 3.

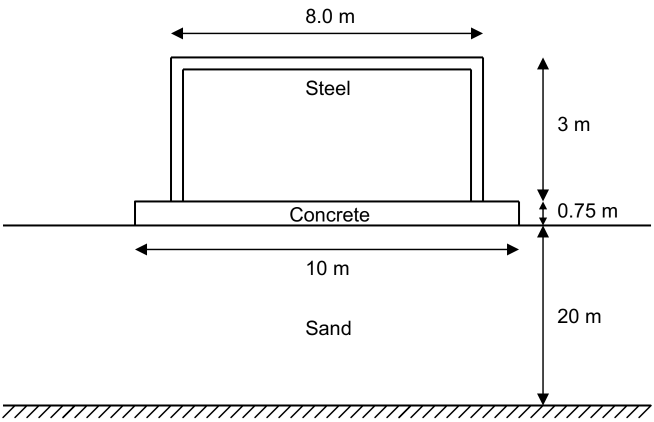

FIG. 3. Schematic of Example Soil-Structure Interaction Situation (not to scale)

Table 3. Material Properties for Example Soil-Structure Interaction Situation

Material	Density (kg/m^3), ρ	Elastic Modulus (N/m^2), E	Poisson's Ratio, ν	Shear Modulus (N/m^2), G	Critical Damping Ratio, ξ
Steel	7850	2.00E+11	0.29	7.75E+10	0.02
Concrete	2300	2.40E+10	0.15	1.04E+10	0.02
Sand	2000	7.50E+07	0.4	2.68E+07	0.08

To illustrate the influence of considering the effect of the variable intrinsic damping properties in a soil-structure interaction situation, the three methodologies for incorporating intrinsic damping (nonclassical damping, weighted modal analysis, and linear hysteretic damping) were used on both the structural system alone and the composite soil-structure system combined. The structural system alone is variable in elasticity and inertia but uniform in intrinsic damping; the composite soil-structure system has variation in its materials' stiffness, mass, and dissipative characteristics. The natural frequencies of the structural system alone and the composite soil-structure system were found through eigenvalue analyses of the undamped systems; they are 265 radians/sec and 20.7 radians/sec, respectively. The dynamic test loading is a horizontal point force applied at the roof of the structure. The load linearly ramps from magnitude 0 at time = 0 to a magnitude of 445 kN (100 kips) at time = 0.01; the loading function is then immediately zero for the rest of time. The monitored displacement is the horizontal motion of the roof. For nonclassical damping matrix assembly and weighted modal analysis, the first ten natural frequencies and ten mode shapes shall be used; the method for determining the weighting factors for the stiffness and mass coefficients for the nonclassical damping matrix is that described by Chopra (2001).

Structural System Alone: Uniform Intrinsic Damping Results

The purpose of this set of simulations is to examine the results from the three MDOF techniques for modeling intrinsic damping when the intrinsic damping properties of a system are uniform. Although the stiffness and inertial properties of the steel and concrete are different, their intrinsic damping properties are assumed to be the same. This represents a control test for a MDOF system where elasticity and inertia may vary across materials, but dissipative characteristics do not. Zero displacement boundary conditions were applied to the bottom of the concrete mat. The displacement responses from all three analysis methods are shown in Fig. 4. All three methods produce virtually identical results when applied to a system of uniform intrinsic damping. The absolute magnitude of the dynamic displacement is 0.015 m, which (as a cross-check) is approximately equivalent to the magnitude of the static displacement in response to a static loading of magnitude 445 kN. Additionally, the frequency content of the three dynamic responses (Fig. 5) are all dominated at 256 radians/sec, the first natural frequency of the frame and mat.

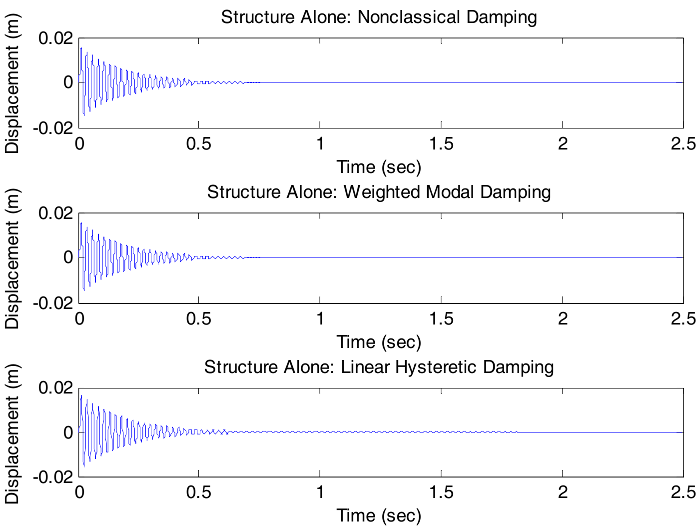

FIG. 4. Comparison of Dynamic Responses for the Structural System Alone.

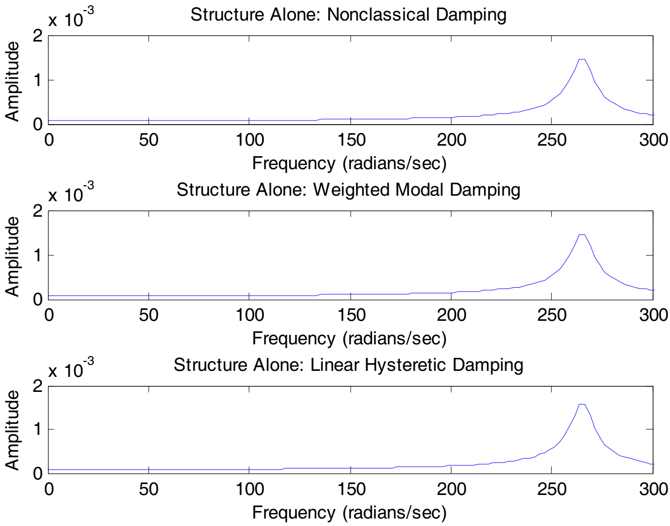

FIG. 5. Comparison of Frequency Content for the Structural System Alone.

Composite Soil-Structure System: Nonuniform Intrinsic Damping

The purpose of this set of simulations is to examine the differences in the results from the three MDOF techniques for modeling intrinsic damping when the intrinsic damping properties of a system are nonuniform. The results of the three methods applied to the composite soil-structure system are widely divergent. Although the methodologies used are the same as those applied to the structural system alone, the data vary widely in their magnitude, duration, and general behavior when applied to a system of nonuniform intrinsic damping. The time domain responses are pictured in Fig. 6, and their corresponding non-zero frequency contents are shown in Fig. 7.

The application of a nonclassical damping procedure towards the system results in a high frequency vibratory signal of very short duration followed by a creep response of the system back to equilibrium. The maximum magnitude of the signal is much lower than that of the static response or that of the dynamic response of the structure alone. Additionally, the frequency context is shifted much lower than the first natural frequency of the frame and the mat alone or of the undamped total soil-system; the expected natural frequencies are undetected in the resultant frequency spectrum. In the absence of an analytical solution for the composite system's response considering nonuniform intrinsic damping, these wide disagreements in magnitude and frequency content compared to the static and uniform intrinsic damping situations undermine the believability of this predicted response. This strong divergence most likely reflects the arbitrary modeling decision to represent an intrinsic damping matrix as a weighted sum of the stiffness and mass matrices.

The application of weighted modal analysis results in an even smaller displacement response than that predicted by the nonclassical damping matrix method. The magnitude diverges even more significantly from the order of magnitude observed in the static and uniform intrinsic damping cases, again undermining the credibility of this methodology's results. This divergence may be attributed to the fact that the soil-structure system under consideration is not a simple parallel system. It is instead heavily indeterminate with several elements in series. Additionally, the intrinsic damping values of the soil materials are four times as large as those of the steel and concrete. Therefore, as previously discussed, for simple series systems with significantly different intrinsic damping properties, the overall effect of intrinsic damping in more complex series systems with widely different intrinsic damping properties is overestimated, thus resulting in an underestimation of the magnitude of the dynamic response. Even though the frequency content of the resultant signal appropriately reflects the first natural frequency of the composite soil-structure system, the predicted magnitude is incorrect due to the redundancy of the system and the large difference between intrinsic damping values.

The results of the linear hysteretic damping analysis of the composite soil-structure system are identical to those from the structural system alone. The static response of the soil-structure system when subjected to a static loading of 445 kN is 0.0144m; this is on the same order of magnitude of the dynamic displacement as predicted by the linear hysteretic model, lending credibility to the magnitude of the results. However, the frequency content does not match that expected for the soil-structure system combined but merely of the structure alone. Any dissipative contribution of the soil foundation is completely overshadowed by the structural system's response.

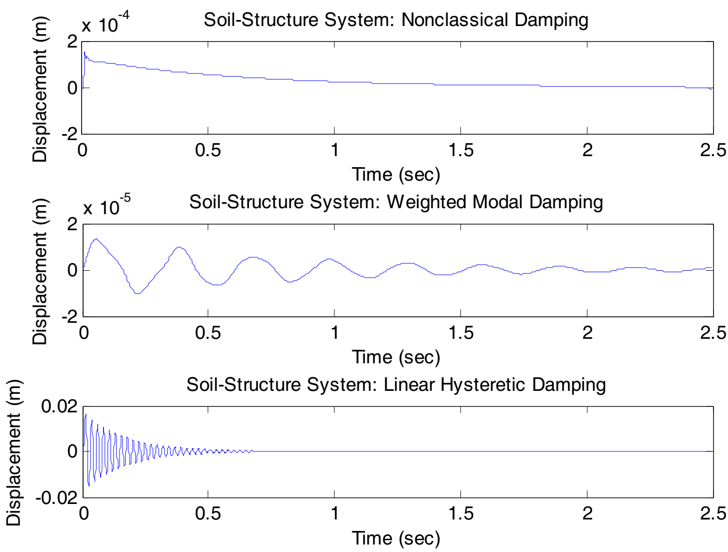

FIG. 6. Comparison of Dynamic Responses for the Soil-Structure System.

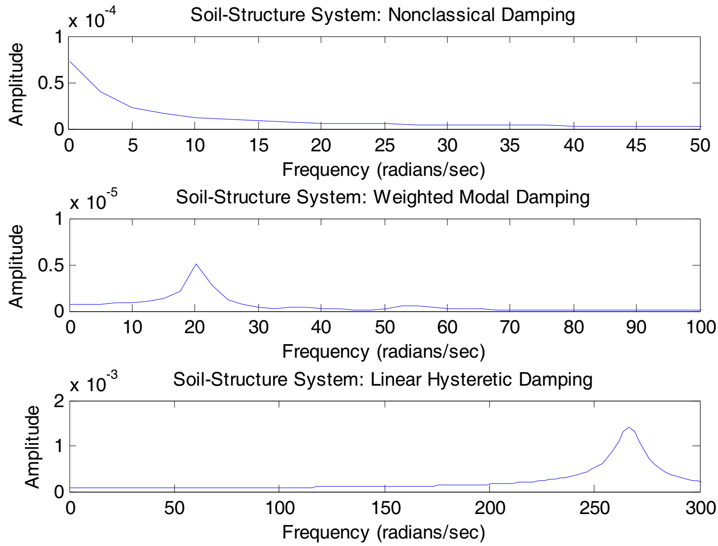

FIG. 7. Comparison of Frequency Content for the Soil-Structure System.

CONCLUSIONS

The geotechnical engineer or analyst must be prepared to identify and incorporate the dissipative effects of the soil foundation upon an infrastructure system's dynamic behavior in order to guide the structural engineer in making appropriate design choices; he or she must also be prepared to acknowledge the assumptions and their resulting limitations involved in modeling the overall system behavior. A review of intrinsic damping mechanisms and their importance in civil engineering dynamics and soil-structure interaction is presented. Geotechnical procedures for determining measures of intrinsic damping are reviewed, and means for using these measures in equivalent linear modeling procedures are detailed. Three common methods for using these measures in modeling MDOF systems with uniform and nonuniform intrinsic damping (nonclassical damping assembly, weighted modal analysis, and linear hysteretic damping) are reviewed, and comparisons of the three methods applied to a representative soil-structure system are shown in a series of analytical studies.

From the analytical studies, the following observations may be made:

(1) The three methodologies produce essentially identical displacement responses when applied to a system of uniform intrinsic damping. This indicates that the three methodologies, although they start from different analytical premises and assumptions, yield similar, reasonable results of expected orders of magnitude and frequency contents in situations of uniform intrinsic damping.

(2) The three modeling methodologies produce widely different results when a

system with nonuniform intrinsic damping characteristics such as a soil-structure system is examined. The addition of variable intrinsic damping highlights the differences in modeling assumptions as follows:

a. In nonclassical damping matrix assembly, the displacement diverges from expected orders of magnitudes and frequencies. This illustrates that the analytical assumption of representing intrinsic damping as a weighted combination of stiffness and mass is not physically justified.
b. Weighted modal analysis techniques applied to redundant systems with multiple different intrinsic damping values result in unrealistically low displacement responses. This illustrates the fact that weighted modal analysis techniques overestimate the composite dissipative effect. This was analytically shown by Gasparini et al. (1981) in simple series systems of rheological units and has been shown here to extend to more realistic infrastructure systems. However, weighted modal techniques do appropriately predict the time variation of response. This is due to the fact that weighted modal analysis techniques are based upon mode shapes and natural frequencies.
c. Linear hysteretic models result in displacements of appropriate magnitudes. However, the frequency content of the overall system is not preserved. This illustrates the fact that linear hysteretic models are based upon complex stiffness properties, which may be related directly to displacements. The frequency content is not sufficiently modified to reflect the composite system.

All of the suggested procedures should be used with caution, recognizing their assumptions and limitations. This is particularly true for infrastructure systems with nonuniform intrinsic damping such as soil-structure systems. Considering that (1) there are no analytical solutions for the dynamic responses of systems with nonuniform intrinsic damping and (2) the suggested procedures have not been experimentally validated for a real soil-structure system, the author's recommendations for modeling and checking the results from the dynamic analysis of a soil-structure system considering nonuniform intrinsic damping are as follows:

(1) Unless the analyst has a strong a priori idea of the predicted response and is willing to iteratively calibrate his or her weighting factors to reflect the anticipated results, the author recommends that nonclassical damping matrix assembly be generally avoided. Although the procedure is mathematically simple and is incorporated into multiple commercial finite element codes, usage of different weighting factors may result in significantly different predicted responses. The amount of user judgment required is extremely high, and the technique is not physically justifiable.
(2) If the system is a relatively simple system that may be modeled or simplified as a predominantly parallel system, the analyst should use weighted modal analysis. This technique has been shown to be analytically coherent with viscoelastic constitutive models for simple rheological units in parallel (Gasparini et al. 1981). However, if the system must be modeled as a redundant series system, weighted modal analysis may be able to capture the predominant frequencies of the system but may not be appropriate in terms of capturing the

magnitude of the displacement.

(3) The validity of the frequency content of the response from recommendation (2) may be cross-checked by performing the eigenvalue problem on the undamped composite system. As the natural frequencies of the damped system should be approximately the same as the damped system, this may provide a reasonably justifiable check upon the frequencies of the responses. The validity of the magnitude of the response from recommendation (2) may be cross-checked by using a linear hysteretic model to model the system and comparing magnitudes of the results; however, it is important to note that the linear hysteretic model has not been analytically or experimentally verified, so this check is merely performed as an approximate credibility check. If wide divergences are noted, this may help inform the analyst of issues from his or her analysis techniques, and these concerns should be noted in the presentation of the results.

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Asset Management Systems for Retaining Walls

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ABSTRACT: Retaining walls are structures that are an important part of many development and re-development projects. Individually, they may not be large or expensive, but often owners can be responsible for large numbers of walls and in total the walls constitute an important asset, and one that can be difficult to manage. Difficulties stem from the dispersed nature of the asset, the different types of retaining wall construction, and the different purposes the walls serve. Asset management systems are costly to implement because of upfront data collection and management needs. In fact, some owners have judged the cost to be greater than the expected benefits, and have taken no action. The National Park Service and the Federal Highway Administration have developed an asset management system for retaining walls that could be a cost effective model for others to adopt. The system is both versatile and simple, and in the past two years more than 3,200 walls have been inventoried and assessed. This paper discusses the considerable effort made to keep the program simple enough to be attainable and yet valuable for its primary purposes.

INTRODUCTION

Retaining walls are often an overlooked critical asset of America's infrastructure. Like some other geotechnical engineering features, retaining walls are often overlooked because they are constantly around us. Each year the United States constructs millions of square meters of retaining walls for private and public projects. Retaining walls save space, reduce impacts, and allow owners to get the most out of a given property or right-of-way. Thus, retaining walls are a very important part of development projects today.

If an owner has many walls of a certain type, all built within a certain time frame, and the design life is limited by a component of the wall, many walls could need replacement at approximately the same time. In this situation and in many others the associated costs can be planned for and minimized through asset management. Not only are there direct cost considerations, there are indirect costs that can be managed. With respect to highways, these indirect costs are usually related to increased congestion, environmental impact, safety and economic impact. For others, the indirect costs may be primarily opportunity costs.

Asset management is achieved through a process of (a) conducting inventory, (b) assessing condition of the inventory, and (c) analyzing results to make predictions and plan for future needs. The inventory is conducted to define the size of the asset, how many walls, how many square meters of wall face, types of walls, locations, etc. In a sense, this information is static unless new walls are built or existing ones removed. The condition assessment defines the current state of the asset in terms of needs. This information is temporal; it changes through time as age and extreme events, for example, affect walls. Analysis principally involves study of the rate of change of condition and, as such requires condition assessment at multiple times. In the analysis process, data are used to minimize life-cycle cost by determining the optimum wall types with respect to performance and optimum times for intervention (e.g. maintenance).

Financially speaking, the potential value of an asset management system is the difference in the life-cycle cost of an asset maintained without one, and of the same asset managed optimally with one. Whether this value is 5% or 50% of the asset value, the authors aren't yet sure. However, in addition to managing costs, inventories and condition assessments are made for safety reasons. The National Bridge Inventory for highway bridges, and dam safety programs, for example, have this as a primary objective.

FIG. 1. Retaining wall inspection at Mesa Verde National Park.

This paper presents the experience of the National Park Service (NPS) and the Federal Highway Administration (FHWA), Office of Federal Lands Highway (FLH), in developing a retaining wall inventory for the NPS to use in managing the retaining walls that form part of their roadway asset. Given the purpose of this particular inventory, retaining walls in road and highway development, such as shown in Figure 1, are highlighted. It is thought that the asset management system and the observations presented here will be valuable to many owners and developers, both public and private.

EXAMPLES OF WALL MANAGEMENT PROGRAMS

Twenty three state departments of transportation, other federal agencies and several major municipalities were canvassed in 2004 for experiences with retaining wall inventories. Most of the managing agencies have bridge and/or roadway inventories tied to an asset management system, but very few have gone beyond acknowledging the desire to incorporate retaining walls in their asset programs. In fact, only seven departments and one municipality had any substantive experience with attempting to identify and inventory wall assets. In most of these cases, inventories have been limited to simple cataloguing systems tied to existing bridge or roadway infrastructure surveys, only include new walls – ignoring the backlog of existing structures, or have only focused on one particular wall type (e.g., mechanically stabilized earth (MSE) structures).

The city of Cincinnati was found to be particularly advanced with respect to the interests of the NPS. The city has been using a retaining wall inventory and inspection system since 1990. The system allows the city to maintain a prioritized list of repairs and replacements based on regular inspections. In addition, the system helps them respond rapidly to public concerns of their walls. They currently track 6,796 retaining walls that affect their right-of-way, including privately owned walls. These walls are equivalent to approximately 250 kilometers in length. Of the retaining walls within the right of way, the City is responsible for maintaining 1,827 of the public owned walls equivalent to approximately 100 kilometers in length. The city allocates a fund to manage capital expenditures to repair or replace the 1,827 city owned retaining walls. Their maintenance program is also funded to perform routine inspections and maintenance. The city engineer's office finds this program to be a very useful tool to manage the earth retaining structures asset.

Another key finding was the experience of the Colorado Department of Transportation (CDOT). CDOT contracted a study the University of Colorado titled Feasibility of a Management System for Retaining Walls and Sound Barriers in the early 2000's (Hearn, 2003). The objective of the study was to review retaining wall management system practices, identify alternatives, and make recommendations for a proposed retaining wall management system for CDOT. The system could then be used to predict the future condition and performance of retaining walls and sound barriers under various budgeting scenarios, and could be integrated with other ongoing CDOT asset management. Since 1998, the CDOT Bridge Branch has issued structure numbers and retains inventory data for newly constructed retaining walls in the bridge database. However, there are neither structure numbers nor inventory data for the older walls in the state. Currently the CDOT database contains approximately 1,000 retaining walls with structure numbers in the inventory system, which is estimated to include approximately 650 thousand square meters of wall. There are also an estimated 3,000 additional walls without a structure number. CDOT elected not to fully implement the recommendations in the Hearn report due to the significant financial and time resources required for implementation. The recognition of this action guided many decisions of the NPS wall asset management program. It was very important to keep implementation costs as low as practical.

NATIONAL PARK SERVICE NEEDS

The NPS is responsible for the management and maintenance of nearly 9,000 kilometers of paved roads and parkways across more than 250 park properties nationwide. NPS retaining walls range in age from new to over 80 years and range in condition from very good to poor. The NPS wanted to know how many walls they owned, and if the walls were concentrated in certain parks or regions. They wanted to know if an inventory of a select few parks could be used, at least initially, to forecast the number and types of walls, and recommended actions in other parks. Because many national parks were developed at approximately the same time, the NPS has a primary concern that there may be a large future need if many walls began requiring extensive rehabilitation at approximately the same time.

The NPS recognized minimizing indirect costs associated with wall maintenance and replacement and assuring public safety as additional goals. Many parks have limited seasons during which they are open or when the majority of visits occur and many parks constitute a primary tourism draw for communities outside the parks. Thus, the indirect costs are very high if it is necessary to close a park or park route because of wall rehabilitation or replacement.

The NPS has an inventory of both roads and bridges, and would like to be able to tie their wall data to the existing databases. Thus, there needs to be compatibility in the data collected and not redundancy. Some individual parks had begun local inventories and condition assessments of walls but these were not useful in tying to other inventory systems for comparing needs between parks.

PROGRAM CONCEPTION

After reviewing examples of wall management programs, federal programs on similar assets, and NPS needs, we recommended an approach based heavily on Hearn (2003). Hearn describes concepts and terms such as ‘Inventory Data’, the static data that describe the wall; for example, its function, type and location, ‘Appraisal Data’, ‘Element-Level Data’, and the idea of components. We also took concepts and definitions on risk from Allen (1997) and Bowles (2002).

Appraisal data are the data that describe how well the design, materials and layout meet current standards. Appraisal data are valuable because design standards, material specifications, and site-specific design criteria, can be expected to change through time. For example, a wall in apparently good condition could be given a low rating because it wasn't designed according to American Association of State Highway Transportation Officials (AASHTO) standards, or geometrically it is now in a poor location, etc.

Elements are the materials that comprise the structure. They have condition data tied to them – data that can change through time. Wall elements are parts of the wall that can be observed, and are defined by their form, material, and use. Using the approach presented by Hearn, standard wall elements such as piles, lagging, and anchor heads would be assessed individually. Elements not integral to the wall but adjacent or attached to it would also be assessed, for example, guard wall, road surface, and soil or rock slopes. To the extent practical the assessment of the individual elements would be limited to the factual description of their condition at the time of inspection, and would not be an assessment of perceived past performance. Many parts of retaining walls are buried and cannot be seen, but are of great importance with respect to a wall's ability to perform. These parts are called 'components' by Hearn, who states that the condition of components can be assessed from the performance of the wall. Of course, it would be preferable to directly measure the condition of buried components such as anchors, concrete, geosynthetics, drainage, and steel, but this is not practical for thousands of walls. So, in the absence of direct measurement, a category of performance is used to assess condition of wall components (buried wall elements) in aggregate.

We also tried to work in concepts of ductility and a recording of age. Age is an indicator of fatigue of elements and components and is particularly valuable for assessing wall components, which cannot readily be observed or inspected. Years of anticipated design life remaining is the recommended measure of age, or remaining life. It is possible that some walls would possess a negative value, which means they have outlived their design life.

Ductility is a measure of resilience, flexibility and redundancy in a design that allows early signs of distress to be observed and monitored before a failure condition is achieved. A ductile design poses lower risk than a brittle design wherein deficiencies are not notable until it is too late to mitigate failure, such as a 'fracture-critical' bridge. For example, a rigid, cast-in-place concrete cantilever wall might hide settlement of backfill, foundation scour, and deterioration of reinforcements until the conditions are so poor that the facing would fail. A wire-faced MSE wall might show more deformation earlier, as a result of these conditions, and allow a greater opportunity for the asset management process to work and deploy resources to repair the wall before the risks and costs got too high.

Other key findings from review of current practice were related to risk. A common definition of risk is the product of the likelihood of failure and the consequence of failure. The rating system based on performance, age, ductility, and wall elements provides a measure of the likelihood of failure but not its consequence. Failure can be broadly defined as unmet expectations, and expectations of a retaining wall associated with a road in a National Park include meeting certain cultural attributes, being aesthetic, minimal impact to surrounding features, requiring limited maintenance, having a long life, providing for safe transportation both on the road and otherwise adjacent to the structure. The consequence of failure to meet some of these expectations (public safety, loss of life) is higher than others (increased maintenance needs). Consequence is also tied to the proximity of adjacent elements (pavement, fences, buildings, etc.) and strongly to presence of people. A wall 15 meters from the shoulder of a seldom used road would have a smaller consequence of failure than the same wall 1.5 meters from the shoulder of a busy highway. A factor representing the consequence of failure is recommended here to convert a wall condition rating to risk, and to allow ranking of walls based on their risk. Allen (1997) presents definitions that were used categorize risk and these formed the basis of the categories included here.

The concept of data reliability is a valuable contribution from Bowles (2002). The term 'apparent' is used to qualify 'pass' and 'not pass' ratings for dam assessments when insufficient data are available 'to make assessments with the normal level of confidence associated with good practice in a particular country or jurisdiction'. The term 'apparent' means the ratings are judgmentally assigned and additional investigation is needed. Since retaining walls have an associated broader range of risk (sometimes very low) than dams, a more gradational assessment of data quality would be desirable.

Conceptual Objectives

The inventory and assessment should capture three broad categories of information. Static data should be collected to describe the wall in specific attributes and terminology that is completely consistent with other systems describing other assets. Condition data should be collected and used to determine wall ratings and prioritize needs. These data would be collected at recurring times to allow the rate of change in condition to be assessed. Finally, if actions are recommended, a conceptual plan and cost should be indicated. Despite the advent of field-worthy electronic data collection devices (hand-helds and laptops) keeping data collection to a 1-page paper form would be best for simplicity and minimizing costs as the system is implemented coast-to-coast.

A conceptual rating and prioritization system is shown in Figures 2 and 3. The required input is a Condition Score and a Data Reliability factor for the categories of Performance, Age, Ductility, and for appropriate Wall Elements and Adjacent Elements, and a Failure Consequence Factor. Each of these would vary from 1 to 10 and would need definitions for at least 3 to 5 of the scores - allowing interpolation between. A rating matrix template could be developed for each wall type with a unique list of Wall Elements and agreed upon weighting factors for that general type of wall. The Rating is the calculated score as a percentage of the maximum possible score. So that walls with different numbers of Wall Elements, Adjacent Elements, or weighting factors can be compared with one another, each wall is evaluated with respect to its own maximum possible score. Figure 2 shows how this would look on a data input form for a wall with 3 wall elements and 3 adjacent elements. The weighting factors shown in all figures are for illustration only, appropriate values to be determined at a later stage based on validation of calculated scores and actions with independently derived recommendations from experienced engineers.

	Condition Rating			Reliability Factored Rating	
		Input Required		Input Required	
Category	Technical Significance	Condition Score	Cat. Rating	Data Reliability	Cat. Rating
Maximum Value	*10*	*10*		*10*	
Performance	6	10	60	3	180
Remaing Age	4	10	40	9	360
Ductility	2	5	10	9	90
Wall Element 1 (Code #)	7	4	28	9	252
Wall Element 2 (Code #)	4	4	16	9	144
Wall Element 3 (Code #)	2	4	8	9	72
Adj. Element 1 (Code #)	4	2	8	9	72
Adj. Element 2 (Code #)	2	2	4	9	36
Adj. Element 3 (Code #)	1	2	2	9	18
APPRAISAL	(APPRAISAL not used in rating)	TOTAL:	176	*RATING CERTAINTY:*	1224
8		MAXIMUM POSSIBLE SCORE:	320	*70*	3200
(out of 10)		*RATING:*	55		38

FIG. 2. Conceptual data input form.

The Condition Rating assumes all Condition Scores are accurate, though some may not be. For example, what is behind the stone face of a wall may be unknown so appraising it versus current design standards must be done with uncertainty. Similarly, there may be no way to know if a wall was built with a bulge or bulging is a result of poor performance. To account for this type of uncertainty Condition Scores would be factored by a factor indicating Data Reliability; a 10 could indicate direct observation, a 5 could be a soundly based assumption, and a 1 could be an assumption with vary little basis. A Reliability Factored Rating could then be calculated such that it is also measured against the maximum possible score for the wall. The comparison of the two ratings for the wall gives a general assessment of the reliability of the wall rating and whether there is large uncertainty in the actual wall condition. The right hand side of Figure 2 shows how this could work; Rating Certainty is the percentage the reliability factored rating is of the original rating.

The Data Reliability represents one form of risk, the risk being that the wall condition is different than assessed, and this could be called Data Risk. What could be called Performance Risk is the primary risk associated with the wall and it is related to the likelihood of failure (given by the Condition Rating) and the consequence of failure, given by the Failure Consequence factor. These two forms of risk can be considered jointly and plotted as shown in Figure 3. The calculated values of Condition Rating and Rating Certainty in Figure 2 can be used for an objective assessment of wall conditions and risks, and to make decisions with respect to actions to take; for example, is it valuable to investigate a wall more carefully, or not. Referring to Figure 3, walls that have a low Performance Risk are of low priority regardless of their Data Risk, indicating that resources are best allocated elsewhere. However, if the Performance Risk was higher it would be important to reduce the Data Risk by further investigation, etc.

Additional objectives are to record conceptual cost estimates and to assure consistency between different review teams and inspection cycles. Therefore, recommended actions should be described with reconnaissance level cost estimates developed onsite. The cost estimates should be of sufficient detail and accuracy to evaluate cost-benefit and to evaluate approximate overall budget needs. Specific verbal descriptions of scores for individual wall elements, adjacent elements, failure consequence, and appraisal should be developed and used to help make ratings independent of individual assessors and repeatable at each inspection.

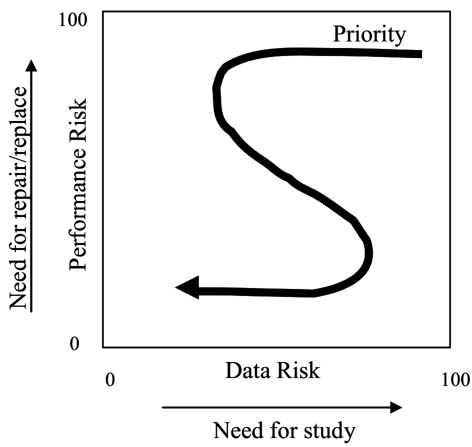


FIG. 3. Chart illustrating prioritization based on risk.

NPS RETAINING WALL INVENTORY PROGRAM

Following conceptual development in 2006, FLH and NPS team members undertook (a) defining approximately 65 wall data attributes that are logged, measured, calculated or assessed during field inventories; (b) development of field data collection procedures, field forms, and associated field guides and cost information; and (c) the development of a Microsoft Access-based, fully searchable database. Not all parts of the concept were carried forward because a primary emphasis was to keep the inventory simple and efficient so that implementation costs were as low as possible. The actual inventory program is described in this section.

Wall Function	Wall Type	Architectural Facing	Surface Treatment	Wall Element
Fill Wall	Anchor, Tieback H-Pile	Brick Veneer	Bush Gun	Piles and Shafts
Cut Wall	Anchor, Micropile	Cementitious Overlay	Color Additive	Lagging
Head Wall	Anchor, Tieback Sheet Pile	Fractured Fin Conc.	Galvanized	Anchor Heads
Bridge Wall	Bin, Concrete	Form-lined Concrete	Painted	Wire/Geosyn. Facing
Slope Protection	Bin, Metal	Plain Concrete	Preservative	Bin or Crib
	Cantilever, Concrete	Planted Face	Silane Sealer	Concrete
	Cantilever, Soldier Pile	Sculpted Shotcrete	Stain	Shotcrete
	Cantilever, Sheet Pile	Shotcrete	Tar Coated	Mortar
	Crib, Concrete	Steel/Metal	Weathering Steel	Block/Brick
	Crib, Metal	Stone	Other	Placed Stone
	Crib, Timber	Simulated Stone		Stone Masonry
	Gravity, Block/Brick	Stone Veneer		Foundation Material
	Gravity, Mass Concrete	Timber		Wall Drains
	Gravity, Dry Stone	Other		Architectural Facing
	Gravity, Gabion			Traffic Barrier/Fence
	Gravity, Mortared Stone			Road/Shoulder
	MSE, Geosynthetic Face			Upslope
	MSE, Precast Panel			Downslope
	MSE, Segmental Block			Lateral Slope
	MSE, Welded Wire Face			Vegetation
	Soil Nail			Culvert
	Tangent/Secant Pile			Curb/Berm/Ditch
	Other			Overall Performance

FIG. 4. Key Retaining Wall Attributes

The key attributes recorded are shown in Figure 4. The concept of ductility was dropped altogether because it was difficult to define and to relate to financial objectives. The concept of data certainty was simplified to scale of 1 to 3 for individual elements and the query for the overall wall - “Is investigation required?”. Similarly, the appraisal of the wall was simplified to include just one of three responses: (1) Does not meet any known standards, (2) Consistent with other structures of its type/period with good performance, and (3) Meets AASHTO design standards. Furthermore, the appraisal was kept separate from numerical condition ratings.

Although seemingly straightforward, the apparent simplicity of describing, measuring and evaluating retaining walls can be deceiving. For example, in some circumstances it can be very difficult for inventory teams to determine whether a structure qualifies for inclusion in the inventory, or how to classify a particular wall's function. For example, is a wall present on the inside of a switchback a fill wall or a cut wall? Should a wall such as shown in Figure 5 be considered a wall with a culvert, or a culvert headwall? Is a wall such as shown in Figure 6 an integral part of the bridge wingwall and, therefore covered under the bridge inventory program, or does it primarily support the bridge approach. During the development of this program, inventory teams were often challenged to best describe such unique wall conditions. Definitions on the requirements for walls to be included in the inventory were established as shown in Figure 7. Small changes in these requirements greatly change the size of the inventoried asset.

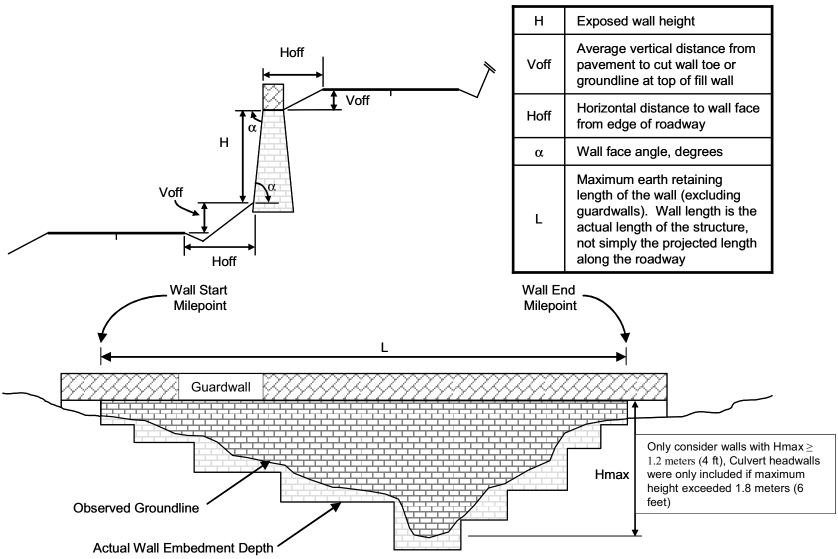
FIG. 5. A retaining wall with dual functions at Glacier National Park.

Similar challenges were also met with definitions of elements and their conditions. The definition of 'attachments' or 'adjacent elements' evolved to include other wall elements that would have a somewhat minor impact on the assessment of the wall (e.g. architectural facing). These elements were distinguished from other wall elements as secondary versus primary, and were assigned lower weighting factors in calculating scores. Element conditions are described within four general distress categories: Corrosion/Weathering, Cracking/Breaking, Distortion/Deflection, and Lost

Bearing/Missing Elements. The condition of all wall elements can be described based on these four potential means of deterioration. Directing observations in this way was done so that examples of different numeric ratings could be provided to inspectors and that inspectors with diverse backgrounds would arrive at similar scores.

FIG. 6. A structure that grades from slope protection to a bridge approach wall.

The weighting factors for the various elements and adjacent elements were heavily discussed. It is recognized that some elements are more critical than others to the overall wall performance. It was decided that the observed wall elements including the overall wall performance would all be weighted equally. Adjacent wall elements such as slopes, guardrails, fences, vegetation, drainage, and roadway would have a graduated weighting factor. The objective of looking at adjacent elements is to observe the performance of those elements as it relates to the wall performance. These elements, which are often linear, provide a good indication of wall movement and performance issues. If the element was performing well the weighting factors would be such as to not affect the score from the wall elements. If the adjacent elements were performing poorly then they would affect the overall wall rating. There was much discussion on what these numbers should be. We came to the conclusion that weighting factors should be reevaluated after data collection when it would be possible to compare weighted ratings to independent assessments.



H	Exposed wall height
Voff	Average vertical distance from pavement to cut wall toe or groundline at top of fill wall
Hoff	Horizontal distance to wall face from edge of roadway
α	Wall face angle, degrees
L	Maximum earth retaining length of the wall (excluding guardwalls). Wall length is the actual length of the structure, not simply the projected length along the roadway

FIG. 7. Retaining wall geometric definitions

Experience of wall inspectors was also a critical discussion. We wanted to simplify the data and address all possible questions in order that non-engineers could go out and inspect the walls. We came to the conclusion that there was no practical way to anticipate all possible wall settings and combination of geometry, loading, and geotechnical parameters. An engineer is needed in the field to make decisions on the wall condition and action requirements. Inspection teams can be lead by an engineer with other less experienced personnel.

In summary, wall attributes within five general data categories are described, measured, evaluated and/or rated to define and quantify assets:

- Wall Location Data: Walls are located by park name, route number/name, side of roadway, wall start and end milepoint. This was done for consistency and ease of integration with the Road Inventory.
- Wall Description Data: Walls are described by function, type, year built, architectural facings and surface treatments. Measurements are recorded for wall length, maximum height, face area, face angle, and vertical and horizontal offsets from the roadway. Average wall height is then calculated from the area and length of wall. These represent exposed areas of the wall. Photos are also logged for each wall, noting location relative to the roadway, major wall features, and overall element conditions.
- Wall Condition Assessment: Primary and secondary wall element conditions are described relative to extent, severity and urgency of observable distresses,

and then numerically rated, giving due consideration to data reliability. The overall performance of the wall system (global performance of the entire wall system) is also evaluated and rated, with all ratings weighted and combined to arrive at a final, overall wall condition rating.

- Wall Action Assessment: Objective consideration is given to (1) the final wall element condition numerical rating, (2) any identified requirements for further site investigations (measure of data reliability), (3) the apparent design criteria employed (e.g., AASHTO), (4) any cultural concerns, and (5) the consequence(s) of wall failure to determine a recommended action. The recommended courses of action are no action; monitor the wall; conduct maintenance-level work; repair wall elements; replace wall elements; replace the entire wall.
- Work Order Development: Brief, yet descriptive work orders are provided when maintenance, repair or replace actions are required. Unit costs for major work items are generated from the cost guide, available Park cost data, etc., to arrive at preliminary estimates of cumulative deferred maintenance.

Of the 23 primary and secondary wall elements defined, only those applicable (generally 5-15) are described in the field via a written "Condition Narrative" – a concise, descriptive narrative of element condition sufficient to characterize severity, extent and urgency of element distresses. Elements are described within the four general distress categories: Corrosion/Weathering, Cracking/Breaking, Distortion/Deflection, and Lost Bearing/Missing Elements. Condition ratings are then determined through the application of a 1-10 Element Condition Rating scale.

The requirement for sound engineering judgment is most apparent in the manner in which recommended wall actions are determined. Whereas similar condition-based inventory systems may directly correlate a numerical rating to a specific action, the wall inventory program assessment methodology develops a numerical condition rating for applicable wall elements which is then objectively considered relative to other influencing factors to arrive at a recommended action. Other factors include such things as the consequences of wall failure, the cultural/historic significance of the structure – a very important aspect of the park program, and the reliability of the condition assessment data. The result is the selection of an appropriate action founded on a well-documented element condition and wall performance assessment, and encompassing documented judgment. Except for when 'No Action' is recommended, the actions are supported by repair or replace work orders and associated cost estimates. The current wall assessment methodology meets the comprehensive goals of identifying walls in need of maintenance, repair or replacement, estimating the current cost of recommended actions, allowing statistical assessments of wall elements throughout the entire inventory, and it provides the baseline for all future wall assessments.

RESULTS FROM INITIAL PARK INVENTORIES

Twenty-six (26) parks have been inventoried with close to 3,200 walls recorded as of April 2008. These walls are estimated to be worth over $400 million if they needed to be replaced in kind. The effort has identified close to $10 million in work orders covering maintenance, repair, and replacement costs. Details are provided in Anderson et al (2008).

Of the six wall functions inventoried – fill walls, cut walls, headwalls, switchback walls, bridge walls and slope protection – approximately half represent outboard fill walls. If culvert headwalls are also considered as a type of fill wall, then nearly 90 percent of all walls are designed and built to retain fill. Clearly, culvert headwalls supporting roadway assets comprise an overwhelmingly large percentage of the wall database. This leads to the question, as owners are moving to inventory and manage their culvert assets as well, as to where culvert headwalls should be included. The results show that culvert headwalls are typically small gravity structures in generally good condition. Inclusion of these structures within the inventory tend to bias and mask database performance trends for what could be considered the more traditional retaining walls – suggesting they are more appropriately assessed under culvert inventories. In comparison, cut walls comprise approximately 10 percent of the inventory, and a very small percentage of the walls are classified as slope protection, switchback walls, or bridge walls.

Although 17 unique wall types were inventoried, very few dominate the database. Nearly all culvert headwalls, and 50 percent of all walls, are mortared stone masonry gravity structures. Dry-laid stone masonry walls comprise another 25 percent of the inventory. It should also be noted that most of these stone masonry structures were built in the first half of the twentieth century. Of the 15 different wall types making up the remaining 25 percent, concrete gravity and concrete cantilever walls are relatively common. The inventory has only a few segmental block Mechanically Stabilized Earth (MSE) walls and metal crib walls, and only one MSE wall with a geosynthetic wrapped face. The distribution of wall types is probably indicative of the setting where the walls are constructed and the relatively narrow time frame during which most were built. Different owners may find a completely different distribution.

The number of wall elements rated is different for different types of walls; generally ranging from 5-15 elements depending on the number of wall components and setting features. Nevertheless, when overall wall ratings are calculated, the maximum, mean and minimum ratings remain generally consistent across the various wall types, indicating the wall inventory program successfully quantifies wall condition within a reasonable band and with enough variation in scores that prioritization is possible.

Thus far in the program, and for most wall types with significant populations, about 25 percent of the walls require some type of corrective action. Most walls with recommendations for action require either maintenance or repairs to localized elements. These are relatively low cost endeavors that could be incorporated in a routine maintenance program. Only 3 percent of all walls have recommendations to replace all or part of the wall, suggesting that the asset as a whole is still in acceptable condition, and that a recurring maintenance program would likely keep the asset in a functional condition.

Work orders, defining general work items and associated costs, are prepared any time a maintenance, repair or replace action is recommended. As expected, maintenance recommendations are most common and least expensive, averaging about $4,000 per wall. Recommendations to repair or replace localized wall elements are less common, and have average costs ranging from $25,000 to $35,000. Total wall replacement costs average about $150,000 each.

These findings characterize the asset and have produced immediate value for the NPS. Additional value will be realized as identified actions are addressed and as subsequent inspections of the same walls track changes in conditions through time.

CONCLUSIONS

A retaining wall asset management system has been developed and deployed for NPS retaining walls. The system is simple yet versatile for many types of walls and wall functions. The data collected are static, inventory data on wall type and location, and temporal data on the condition of specific elements of the wall and adjacent features. Condition ratings are considered individually for wall elements and summed to provide an overall rating for the wall. Appraisal with respect to design standards, consideration of the consequence of failure, the urgency of needs and the data reliability are considered in making a recommendation for future action, but are not part of the numerical condition rating. The recommendations fall into one of the following five categories: No-action/monitor, Maintenance, Repair Elements, Replace Elements, and Replace Wall. Thus, each wall receives a numerical rating and a recommended action.

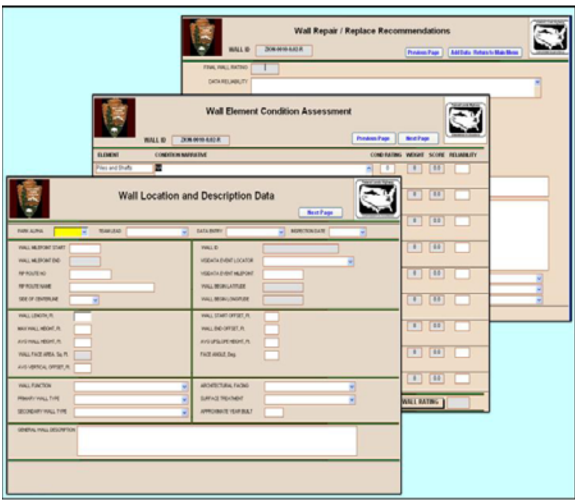


FIG. 8. Database input screens.

The system, which is described in detail by DeMarco (2008) is designed to produce reliable, repeatable information by inclusion of careful definitions and a systematic approach to data collection. Data were collected on a one, two-sided, page of paper in a format similar to Figure 2 and entered into a Microsoft Access database through similarly structured input forms (Figure 8). It was not possible to consider, in advance, all possible wall types, observations and needs so that inexperienced personnel could simply follow the process and arrive at meaningful scores and recommended actions. Thus, some engineering experience is needed by the teams doing the inventory and assessment.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the support of Dave Keough of the National Park Service, John Thiel of the Eastern Federal Lands Division of FHWA, Rich Barrows of the Western Federal Lands Division of FHWA, and Stephen Lewis, contractor to the Central Federal Lands Highway Division. Their contributions in developing the system, and the contributions of many other Federal Lands employees in conducting the inventory, helped make this program a success.

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Case History - Southlands Orchard Road Retaining Walls

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ABSTRACT: This paper will discuss the design considerations for the Southlands Orchard Road Retaining Walls located in Aurora, Colorado. The project had many challenging aspects that had to be accounted for by the design team. These challenges included large wall obstructions, high applied bearing pressures, and project constrains with utility easements. This paper will focus on the segmental mechanically stabilized earth (MSE) retaining walls that were designed for the proposed Orchard Road. The segmental MSE retaining walls were designed to include an overall grade separation of 17.4 meters (57 feet) at its tallest point, which is one of the tallest segmental MSE block retaining walls that have been constructed in the Denver Metro Area to date. Due to the significant grade separation many factors influenced the wall design including wall terracing, foundation improvements, high groundwater conditions and a wall system selection. The paper will discuss the challenges and lessons learned with the design and construction of the Southlands Orchard Road segmental retaining walls.

INTRODUCTION

The proposed residential development of the Villages at Southlands created the need for a four-lane route, East Orchard Road, between South Aurora Parkway and Powhaton Road. The proposed Orchard Road would require retaining walls to account for a 17.4 meter (57-foot) grade separation. Approximately 3,345 square meters (36,000 square feet) of segmental MSE block retaining walls were installed, with as many as five tiers in some areas. Within this paper, the MSE block retaining walls will be referred to as segmental retaining (SR) walls. In addition to the overall height of the structure requiring foundation improvements and settlement concerns, the design also had to consider wall obstructions, utility easements, and high groundwater constraints.

SITE CONDITIONS

The project site is located approximately one mile northeast of the interchange of E-470 and Smoky Hill Road on the east side of the Southlands Mall in the city of Aurora, Colorado. The retaining wall construction is associated with a 202,344-square meter (50-acre) multi-family residential development named Villages at Southlands. South Aurora Parkway borders the proposed development to the west and Murphy Creek running from the south to north borders the development to the east. The proposed four-lane Orchard Road is located along the northern limit. The general site map is illustrated in Figure 1.

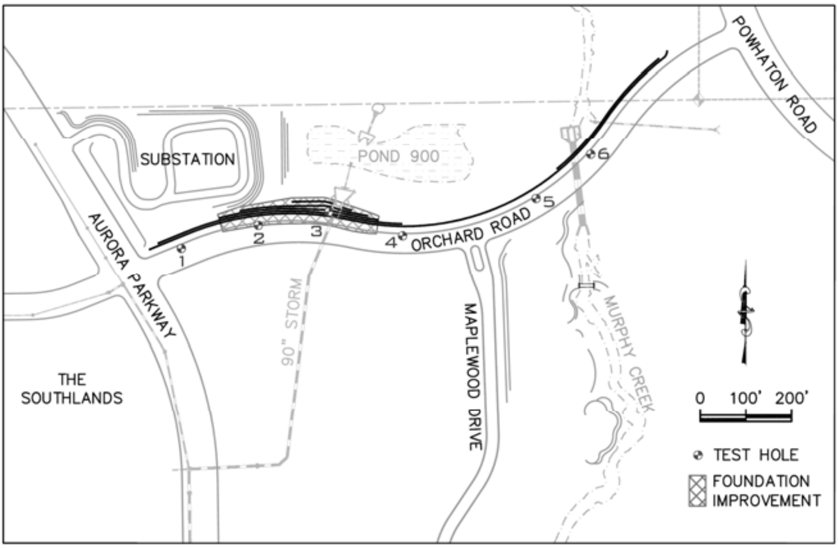

FIG. 1. General site map (from CLC Associates).

Site Grading

The original topography at the development site was generally declining to the north with an elevation change of over 36 meters (120 feet) in approximately 610 meters (2,000 feet) across the site. The site topography was also sloping downward generally from South Aurora Parkway on the west side to Murphy Creek on the east side. An existing regional storm drainage, which serves a major portion of the Southlands Mall area, traversed the northwestern section and discharged storm water to a regional pond (Pond 900) located on the north side of the proposed Orchard Road.

The proposed site grading generally called for cut slopes up to approximately 15 meters (50 feet) in depth along the southern limit and fill retaining walls nearly 18 meters (60 feet) in height along the northern limit. As a result, a multiple-tier retaining wall system was required at the locations as shown in Figure 1 to support the proposed Orchard Road and achieve the grade separation between Orchard Road and Pond 900 for approximately 518 liner meters (1,700 linear feet).

Subsurface Conditions

A total of six (6) test holes were drilled near the retaining wall site at the approximate locations shown in Figure 1. The subsurface conditions encountered in the test holes typically consisted of isolated areas of controlled fill or natural sandy clay underlain by sandy claystone to clayey sandstone bedrock at depths ranging from approximately 1.8 to 8.8 meters (6 to 29 feet) below the existing ground surface. Groundwater was encountered in the test holes located near Murphy Creek and the regional drainage at elevations near the drainage channel level. Additionally, the penetration resistance values indicated that relatively soft overburden soils existed in the vicinity of the regional drainage, which coincides with the area of maximum wall height. The boring log obtained from the test hole located near the regional drainage is presented in Figure 2 (GROUND, 2007).

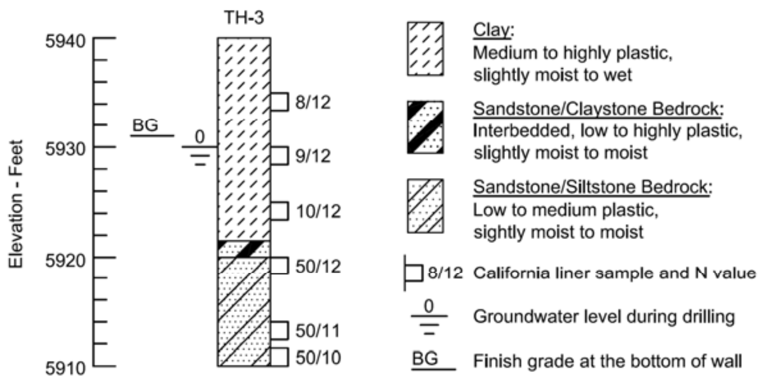

FIG. 2. Boring log near regional drainage with maximum wall height.

DESIGN OF SEGMENTAL RETAINING WALLS

To achieve grade separations up to 17.4 meters (57 feet) in height within a 9.1-meter (30-foot) horizontal space, construction of SR wall is typically an economical approach in a fill situation. Therefore, a multiple-tier segmental retaining wall system similar to those constructed throughout the Southlands Mall Development was selected in the early design phase.

Wall Geometry

Based on the right-of-way line, which is located at 4.0 meters (13 feet) behind the curb line on the north side of Orchard Road and the grade differences provided by the Civil Engineer, the horizontal wall alignments and vertical wall terracing were laid out as the first task of the design process. The proposed wall layout consisted of up to 5 tiers of walls at a typical toe-to-toe spacing of 2.1 meters (7 feet). A maximum exposed wall height of 3.0 meters (10 feet) was selected for the middle walls. The height of the top tier wall was reduced to accommodate the underground utility easement. The height of the bottom tier wall was increased due to the penetration of an impact stilling basin structure. Slopes between the tier walls and above the top tier wall were typically set at 4 (H) to 1 (V) for a proper drainage. A typical cross-section of the wall with the maximum design wall height is shown in Figure 3.

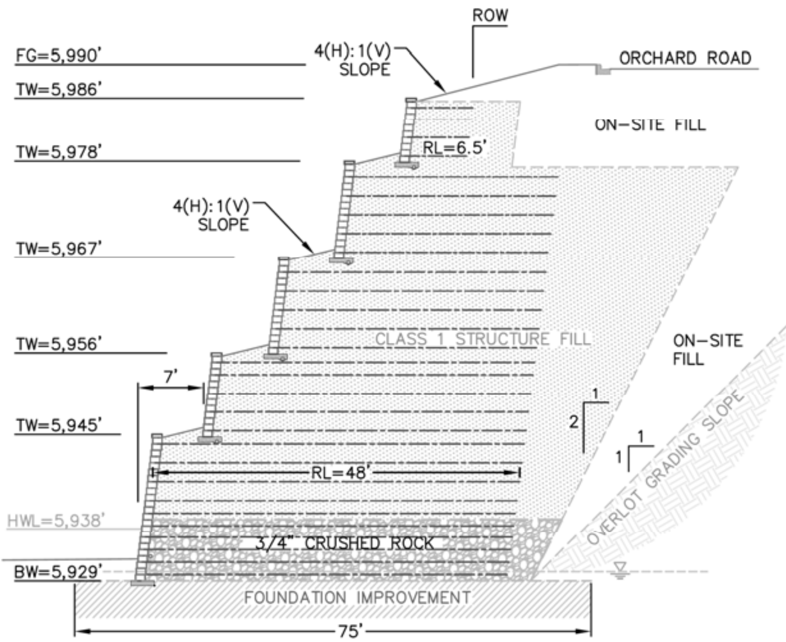

FIG. 3. Typical wall section with maximum design wall height.

Design Methodology

The SR wall design calculations were performed in accordance with the NCMA design procedures (NCMA, 2002) by using the SRWall® computer program. These design calculations evaluate the external stability, internal stability, and facing stability to determine required geogrid strengths, elevations, and reinforcement lengths.

Global stability analysis was performed on various numbers of tier walls by using the Slope-W® computer program. The Morgenstern-Price Method was used to calculate the factors of safety. A surcharge load of 12.0 kPa (250 psf) was applied at the location of Orchard Road for traffic loadings. Adverse groundwater conditions based on the anticipated 100-year high water elevation were incorporated in the analysis. The reinforcement lengths were adjusted to achieve a minimum factor of safety of 1.5.

SPECIAL DESIGN CONSIDERATIONS

A variety of unique, complex, and compounding design challenges were considered in the SR wall system design. For aesthetic considerations, the 17.4-meter (57-foot) grade separation was achieved using multiple wall tiers. Such a significant grade separation also created concerns regarding foundation bearing capacity and anticipated settlements. In addition, high water conditions had to be analyzed and both utility easements and wall obstructions were considered to account for the drainage structures.

Wall System Selection

Due to the complexity of the design challenges, the developer and the engineer wanted to be confident with the SR wall system utilized on the project. A complete SR wall system package including block, geogrid, and connectors specifically designed to work and interact with each other provides reliability. Careful consideration was also dedicated to selecting an experienced SR wall installer.

Typically, connection strength and facing stability are significant factors in the design of an SR wall, especially for taller wall heights. A positive mechanical connection of the geogrid to the segmental block facing units was considered beneficial to the project for reliability, dependability, and cost effectiveness. Components of the system required high block compressive strengths, dependable geogrid long term design strengths, and proven connection test results. The developer chose to use the same SR wall system as was constructed throughout the Southlands Mall Development for the multiple-tier retaining walls.

Multiple Tier Walls

The proposed retaining walls consist of up to 5 tier walls at approximately 2.1-meter (7-foot) horizontal spacing. The retaining wall design considered the surcharge loads, including the live load above the top tier wall and the dead load of the upper tier walls applying to the lower tier walls. The surcharge loads in the multiple tier wall system were approximated with a top-down approach by using the two-tier wall example as provided in the NCMA Design Manual (NCMA, 2002).

First, as a single-tier wall, the live load above the top tier wall was used in the design calculations for the top tier wall. Next, the live load above the top tier wall and the dead load of the top tier wall were used in the design calculations for internal and external stabilities of the second tier wall, respectively. Subsequently, the bearing pressure at the bottom of the top tier wall and the dead load of the second tier wall were used in the design calculations for internal and external stabilities of the third tier wall. The approximation of surcharge loads was repeated for the rest of lower tier walls so that the tier wall surcharges were carried through the wall system.

Typically, the tier wall heights and tier spacing vary at different locations and different tiers. As a result, the surcharge conditions for internal and external stabilities at each tier can vary from place to place. Therefore, the top-down approach for surcharge loads as described above was used in the design calculations for each of the representative wall sections.

Storm Drainage Structures

In order to maximize the usable area of the development, the project converted the existing regional storm drainage, which runs through the northwestern section of the site, to a 229-centimeter (90-inch) storm pipe installed at the bottom of the overlot fills. The storm water would be discharged to an impact stilling basin prior to being released to Pond 900 to the north. The impact stilling basin is a reinforced concrete enclosed box structure 8.2 meters (27 feet) in width, 6.1 meters (20 feet) in height, and 10.7 meters (35 feet) in length entirely embedded at the bottom of the tallest section of the walls. The north end of the box structure left a 7.6-meter by 4.6-meter (25-foot by 15-foot) opening on the wall face.

To incorporate the penetration of the box structure in the wall design, the bottom tier wall, which is 4.9 meters (16 feet) in height adjacent to the box structure, abuts the front edge of the box wall on each side of the box structure. The second tier wall from the bottom runs to the sides and directly over the top of the box structure. The 8.2-meter (27-foot) wide box structure was designed for a 240 kPa (5,000 psf) surcharge pressure. Nineteen (19) dowels were integrated into the top of the box structure along the alignment of the second tier wall at 45.7 centimeters (18 inches) on centers to provide sliding resistance.

A two-tier wall system and Orchard Road would run across the existing Murphy Creek channel. The project installed a reinforced concrete twin box culvert 1.8 meters (6 feet) in width and 1.8 meters (6 feet) in height each to allow Murphy Creek to flow beneath the roadway embankment. The site grading allowed the finish grades at the bottom of the wall to be set at approximately 1.5 meters (5 feet) above the top of the box culvert. Therefore, box culvert penetration on the walls was not required for Murphy Creek.

High Water Conditions

The anticipated normal water level in front of the impact stilling basin was located at approximately 1,807.9 meters (5,931.5 feet), which is more than 0.6 meters (2 feet) above the bottom of the lower tier wall. The 100-year flood level on the north side of the impact stilling basin was located at an elevation of 1,809.8 (5,937.8 feet), which is approximately 2.6 meters (8.5 feet) above the bottom of the lower tier wall.

In order to alleviate the hydrostatic pressure build-up in the wall system during and after storm events, the structure backfill in the lower tier wall below 1,810.2 meters (5,939 feet), where a submerged condition was anticipated, 1.9-centimeter (3/4-inch) clean crushed rock backfill was used. The zone of clean crushed rock is shown in Figure 3. A layer of filter fabric was placed between the crushed rock and other backfill materials for separation and filtering. Approximately 1,223 cubic meters (1,600 cubic yards) of clean crushed rock was used in the project.

Utility Easement

The City of Aurora allowed the geogrid reinforcement to extend beyond the right-of-way line located at approximately 2.1 meters (7 feet) behind the upper tier wall, but not to pass the back of curbs on the north side of Orchard Road. Nevertheless, the City also required that the dry utility easement to be located immediately inside the right-of-way line be clear from any obstacles for a depth of at least 3.7 meters (12 feet) below the roadway grades.

To meet the City's requirement, the length of reinforcement at the upper tier wall had to be limited to 2.0 meters (6.5 feet), which is 75 percent of the design height for the upper wall. Results of the SR wall design calculations indicated that the limited reinforcement length with the use of structure backfill in the reinforced zone satisfied the internal and external stabilities for the upper tier wall. Global stability analysis was then performed on the upper one, two, and three tier walls. Results of the global stability analysis demonstrated that the structure backfill at the upper tier wall should extend to approximately 2.1 meters (7 feet) behind the reinforced zone to satisfy global stability of the upper tier wall. A section view of the extent of structure backfill is presented in Figure 3. No modification to the original wall design was needed for the lower wall tiers.

FOUNDATION IMPROVEMENTS

The method of ground improvement that was selected for this project consisted of the Geopier® soil reinforcement, Rammed Aggregate Pier® (RAP) system. This technology was selected due to the anticipated foundation pressures that would be exerted on the native soils, which would exceed the bearing capacity of the native soils. The RAP system was selected over the traditional overexcavation and replacement option based on cost and scheduling advantages of the RAP system.

Rammed Aggregate Pier System

The RAP elements were installed by removing the weak native soils and replacing the soil with highly-compacted, stiff RAP elements (i.e. well-graded aggregate). The aggregate is placed in the drilled cavity and compacted in thin lifts, 30.5 centimeters (12 inches) or less, using a patented beveled tamper sized to fit within the diameter of the drilled cavity. The compaction energy that is used requires a hydraulic hammer with limited amplitude, high magnitude of force with high frequencies of 300 to 600 cycles per minute. This compaction effort results in creating high lateral stress build up of the matrix soils and creates a stiff aggregate pier. Figure 4 illustrates a typical RAP element section and construction sequence.

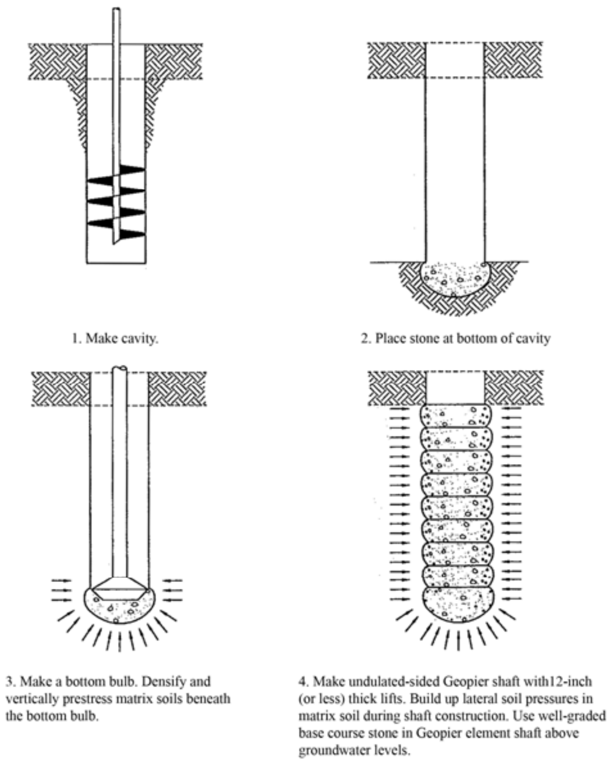

FIG. 4. Typical RAP element section and construction sequence.

Based on the available geotechnical data, the theoretical allowable bearing capacity was approximately 192 kPa (4,000 psf). However, the retaining wall design indicated a calculated bearing pressure of 287 kPa (6,000 psf) at the bottom of the tall wall section. To improve the foundation soils, the RAP elements were used to increase the allowable bearing capacity at the bottom of the wall. The allowable bearing pressure incorporating the RAP elements were evaluated using a procedure developed from Rankine lower bound planar approaches and modified by a factor to account for limit-equilibrium behavior. The suggested procedures were presented in Barkdale and Bachus (1983), with a slight modification suggested by Hall et.al. (2002) and FitzPatrick (2004), and include the following steps:

a. Determine the composite strength parameter values based on a weighted average of the matrix soil, RAP elements shear strengths, and the stress concentration factor for the vertically-stratified zone beneath the wall.
b. Calculate the Rankine lower bound solution for bearing capacity by equilibrating the average stresses acting within two blocks of slipping soil as shown in Figure 5.
c. Apply a conversion factor to the Rankine lower bound solution to arrive at an upper bound solution. The conversion is based on comparisons between Rankine lower bound solution and Terzaghi upper bound solution for a number of similar cases.
d. Calculate factor of safety by dividing the Terzaghi upper bound solutions by the bearing pressure exerted by the wall.

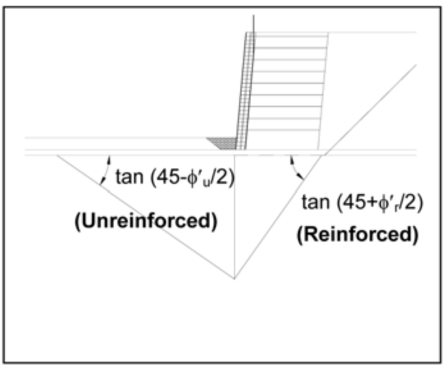

FIG. 5. Lower solution for bearing capacity.

Using the design approach described above, the design calculations indicated RAP elements should be installed at an area ratio of five to seven percent; resulting in spacing's of 2.1 to 3.0 meters (7 to 10 feet) on center. The RAP elements increased the allowable bearing pressure to 287 kPa (6,000 psf), and increased the composite foundation soil's shear strength to 28 degrees. A total of 327 RAP elements, 76.2-centimeter (30-inch) diameter by 4.6-meter (15-foot) deep, were installed over an area of approximately 2,834 square meters (30,500 square feet).

Global Stability Consideration

In addition to offering the required foundation bearing capacity support, the RAP system also aided in addressing the global stability of the tiered SR walls. Utilizing the same geogrid layout, the RAP elements increased the global stability of the tiered wall from 1.02 to 1.32. Figure 6 shows the global stability analysis prior to the foundation improvement and Figure 7 shows the global stability analysis after the foundation improvements from the RAP elements were applied.

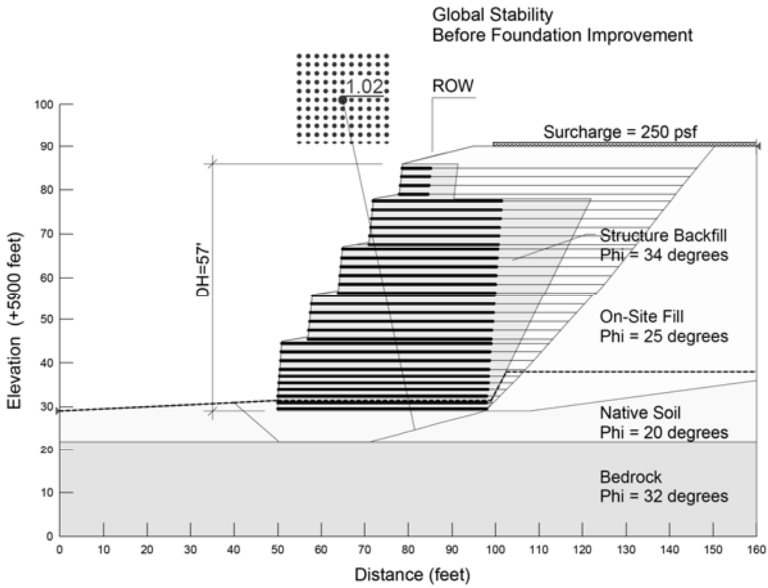

FIG. 6. Global stability before foundation improvement with RAP elements.

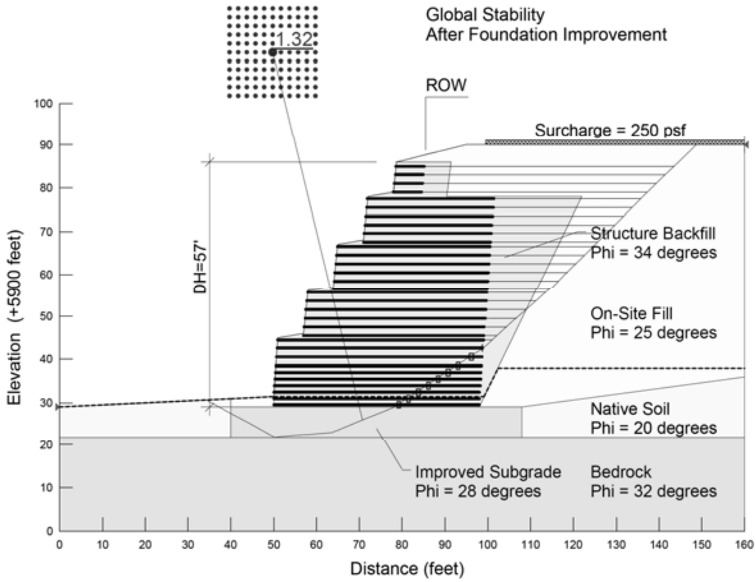

FIG. 7. Global stability after foundation improvement with RAP elements

Embankment Settlement

After all the internal and external stability concerns were identified, the settlement of the structure was analyzed. The proposed SR walls were constructed to retain fills up to approximately 18.3 meters (60 feet) in height as well as to support a four-lane asphalt paved roadway with traffic loading, curbs, gutters, and sidewalks. Deflection of the SR walls, settlements of the fill materials (reinforced backfill, retained backfill, and overlot fills), and differential settlement between different backfill materials in the tall wall sections can potentially result in distresses of the roadway pavement and adjacent concrete installations, even if the fill and backfill materials are properly placed and compacted.

Granular materials are commonly used to reduce potential settlement of tall embankments. However, suitable granular soil was not readily available on or near the project site. Due to the large scale of the project, the quantity of imported materials would significantly impact the project cost. Value engineering was exercised to evaluate the costs of importing granular soils and the associated geogrid reinforcement. Based on the preliminary pricing and anticipated performance of the walls, the design incorporated imported Class 1 structure backfill for the reinforced zone and on-site material for the retained zone. The two different types of materials were both placed and compacted by the wall contractor as construction of the walls progressed to maintain quality and consistency of the fill placement.

In order to reduce the long-term differential settlement along the interface between structure backfill and on-site material, two modifications were made to the configuration of the structure backfill. The interface plane between the structure backfill and on-site fill was laid back to a 1 (H) to 2 (V) slope for three or more tier walls. Additionally, on-site fill was used in the top 3.7 meters (12 feet) across the interface plane to provide a cap below the roadway pavement. The extents of structure backfill in a five-tier wall section are illustrated in Figure 3. It was estimated that approximately 30,582 cubic meters (40,000 cubic yards) of structure backfill and over 15,291 cubic meters (20,000 cubic yards) of on-site fill were placed during the wall construction.

Foundation Settlement

Results of the wall design calculations indicated a maximum bearing pressure of approximately 287 kPa (6,000 psf) at the bottom of the five-tier wall, which is beyond the bearing capacity that the native soils can support. Foundation improvement was necessary for the tall walls to reduce foundation settlement. The design team defined the SR wall settlement criteria to be approximately 7.6 centimeters (3 inches) of post construction settlement. The area that required foundation improvements is shown as a hatched area in Figure 1 and is also shown in Figure 3.

A two-layered approach was used to analyze the settlement beneath the wall, as described by *Geopier Foundation Company Technical Bulletin No. 06* (2005). This approach evaluates the settlement within the RAP reinforced zone and below the RAP elements. The upper zone consists of a stiffened crust that has reduced compressibility, thus reducing the settlement of the embankment within the RAP element improved zone. The settlement below the RAP elements improved zone is considered the lower zone and is evaluated using traditional geotechnical analysis approaches. The total settlement (s_{total}) for embankment settlement is the sum of the upper zone settlement (s_{UZ}) and lower zone settlements (s_{LZ}).

Upper Zone Settlement (s_{UZ})

In the RAP element improved zone, upper zone settlement is determined by calculating the top of pier stress (q_{RAP}) using the following equation:

$$q_{RAP} = q\left[\frac{n_s}{n_s R_a - R_a + 1}\right] \qquad \text{Eq. 1}$$

Where q is the average applied pressure, R_a is the area replacement ratio of the RAP elements to matrix soil, and n_s is the stress concentration ratio between the RAP elements and the matrix soil.

With the piers being installed for the areas of concern, the design team was able to use the on-site modulus test to estimate the settlement of RAP element reinforced zone. This is performed by dividing the top-of- pier stress (q_{RAP}) by the RAP element stiffness modulus (k_{RAP}):

$$s_{UZ} = \frac{q_{RAP}}{k_{RAP}} \qquad \text{Eq. 2}$$

The upper zone settlement provided the design team a determination of the deflection of the RAP elements; however, it did not account for the native soil deflection with respect to the RAP elements. Instrumented field tests have shown that only minor differential settlement is observed between the top of the RAP elements and the matrix soil under reinforced mass loadings (Minks 2001and White 2002). With the height of the SR wall on this project the amount of differential settlement between the RAP elements and the matrix soil is minor compared to the height of the reinforced mass. This is related to the development of a plane of equal settlement caused by soil arching of the embankment material and the stiff RAP element. (Terzaghi, 1936).

Lower-Zone Settlement (s_{LZ})

The lower zone settlement is calculated by analyzing the zone located below the RAP elements improved zone, the settlement is analyzed by using classical geotechnical approaches, either elastic settlement analyses or consolidation analyses. The following expressions are used:

$$s_{LZ} = \frac{\Delta q H}{E}, \text{ and} \qquad \text{Eq. 3}$$

$$s_{LZ} = c_c \left[\frac{1}{1+e_o}\right] H \log\left[\frac{p_o + \Delta q}{p_o}\right] \qquad \text{Eq. 4}$$

Where H is the thickness of the lower zone, E is the elastic modulus (matrix soil), C_c is the coefficient of compressibility (matrix soil), e_o is the soil ratio (matrix soil), p_o is the vertical effective stress at the mid-point of the compressible layer, and Δq is the average bearing pressure applied by the embankment. The applied bearing pressure is determined by the stress influence factor, I_σ, however this factor is assumed to be 1 with large embankments and MSE walls.

As was shown on Figure 2, the presence of sandstone/siltstone bedrock at the depth of 5.5 meters (18 feet), no lower zone of settlement was calculated due to this stiff layer. Therefore, no lower zone settlement was expected in these structures.

CONCLUSIONS

This paper highlighted the design challenges of the Southlands Orchard Road retaining walls. Due to the 17.4-meter (57-foot) grade separation and other critical design constraints, careful consideration was put into the selection of the segmental MSE retaining wall system. A complete package retaining wall system with a positive mechanical connection was chosen for reliability and efficiency. To address foundation bearing capacity and global stability concerns, Rammed Aggregate Pier elements were utilized.

- Over 3,345 square meters (36,000 square feet) of segmental block retaining wall was installed with an overall grade separation height up to 17.4 meters (57 feet), and including as many as 5 tiers. The retaining wall selection was important given the wall heights and additional project variables. A complete package wall system was selected with a positive mechanical connection of the geogrid reinforcement to the block facing units.
- 327 RAP elements, 76.2-centimeter (30-inch) diameter by 4.6-meter (15-foot) long, were installed to address the foundation bearing capacity and settlement concerns resulting from the applied pressures of the retaining wall. The RAP system also provided global stability support of the tiered wall layout.
- In addition to the tall wall heights and loading conditions, the wall design had to also take into account utility easement constraints, wall obstruction details created by drainage penetrations, and high groundwater conditions.

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Garfield County Regional Airport Runway Upgrade Project

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ABSTRACT: The Garfield County Regional Airport in Rifle, Colorado sits atop a mesa high above the surrounding terrain. In order to accommodate a larger class of jet aircraft and improve safety, the main runway will be upgraded and realigned. The new runway will require expansion of the mesa's flat top by means of 30 meter (90 foot) high fill slopes. Space for runway expansion is constrained on both ends of the existing mesa by county roads and Interstate 70. Fill slopes steeper than can be attained with conventional earth fill are required. An alternatives analysis identified earth fill amended with cement (soil-cement) as the preferred method to allow increased slope angles and meet geometric constraints. A soil-cement mix design was formulated using on-site materials. Slopes as steep as 1H:1V are proposed in the constrained areas. Flatter earth fill slopes were designed for less constricted areas. In addition to the unique application of soil-cement as a slope buttressing material, very loose wet materials were encountered in several borings during site investigations. Consideration of these potentially loose saturated materials underneath 90-foot high fill slopes presented a difficult design challenge. Olsson Associates were the prime consultant on the project.

INTRODUCTION

The authors' involvement with the Garfield County Regional Airport Project began in 2003 with an evaluation of the airport layout and geometric constraints that could affect the proposed upgrade of Runway 8/26 to meet the requirements of airport reference code (ARC) D-III. The Federal Aviation Administration (FAA) subsequently made the decision to upgrade to ARC D-II criteria while meeting some requirements of ARC D-III. The ARC standards mostly relate to runway and taxiway alignments, lengths and widths. The decision to upgrade the runway was based on the increasing use of the airport by business jet traffic, which exceeded 7,000 operations per year in 2006.

In January 2004, Vector Colorado LLC (acquired by Tetra Tech in January 2007) participated in an Airport Layout Plan (ALP) update that detailed the investigation of a number of alternatives for upgrading the runway. The geotechnical constraints and cost implications of various alternatives were investigated, including concrete retaining walls, mechanically stabilized earth (MSE) walls, and reinforced steepened slopes (RSS). At the time, the airport Master Plan Update included construction of a 30 meter (90 foot) high reinforced concrete retaining wall structure on the west end of the runway. The ALP update considered a number of alternatives, including runway re-alignment, runway extension to the east, runway extension to the west and a combination of those alternatives. In 2005, the FAA announced that they would like to proceed with one of the alternatives identified by the study with a runway length matching the existing length of 2,135 meters (7,000 feet). Though the runway length remained the same, the required runway safety areas on either end of the runway increased. The runway sits upon an elongated mesa oriented generally east-west. Expansion of the available ground to both the east and west is constrained by steep topography (Figure 1).

It was determined during the investigation for the 2004 ALP that all of the runway upgrade alternatives would require some fill slopes steeper than those which can be supported by the onsite materials without the use of retaining walls, soil reinforcement (MSE, RSS) or soil amendments (cement or fly-ash). Soil stabilization and reinforcing techniques were considered to allow runway upgrade fill slopes to be as steep as practical within the constraints of the geotechnical limitations of the site geology and available fill materials. Reinforced concrete retaining walls, reinforced steepened slopes and MSE walls were all evaluated and found to be too expensive. Cement or fly-ash amended soil has been used to allow slopes as steep as 1H:1V in other applications (mainly water resources). Soil-cement was chosen as an innovative and economical method to increase slope angles in the constricted areas.

The detailed design of the project was divided into a number of stages to facilitate advancing the project within available funding. In 2006, the first stage of the detailed design involved preparing preliminary plans to support 404 and 401 permit applications for re-routing a portion of Dry Creek to the southwest of the airport. As part of the Dry Creek relocation, drop structures will be required to provide a stable slope in the new channel. The availability of suitable rip-rap and boulders was poor and at high cost, so soil-cement was investigated as a drop structure material.

The impact of the 401/404 permit applications on the geotechnical aspects of the project was limited to providing some back-up testing to verify that the soil-cement would be suitable for the proposed fill slopes. In 2007 the geotechnical aspects of the project were advanced to final design.

The following sections of this paper discuss the site geology, geotechnical investigations, subsurface conditions, borrow sources for the project and the soil-cement design. The first stage of construction went out to bid at the end of April, 2008. A 30 meter (90 foot) traditional earthfill embankment is being constructed on the east end of the mesa during the 2008 construction season. The overall project will require approximately 2.75 million cubic meters (3.6 million cubic yards) of fill.

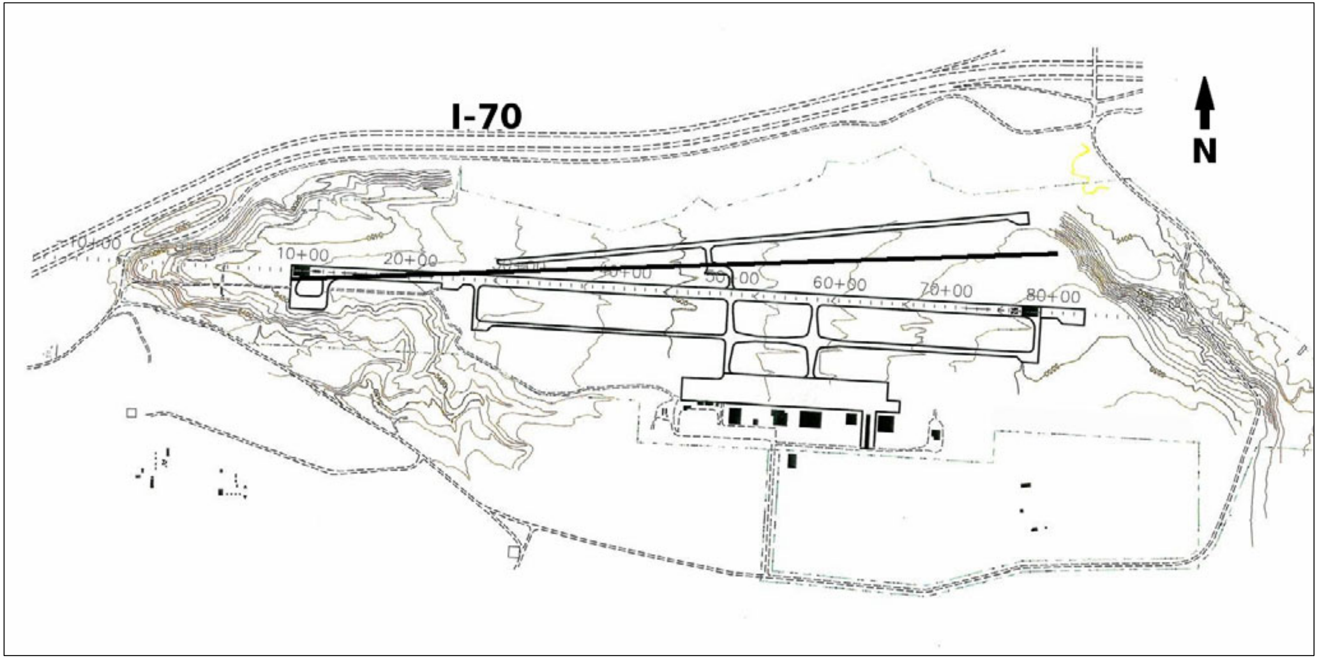

Figure 1. Site overview with topography. The existing runway and facilities are shown, with the approximate proposed runway alignment illustrated as a thick black line.

LOCAL GEOLOGY

The Garfield County Regional Airport is located on top of a broad mesa topographically situated approximately 30 to 45 meters (100 to 150 feet) higher than the majority of the surrounding terrain. The topography on top of the mesa is very flat while the side slopes fall away steeply.

The mesa is comprised of claystone, mudstone and siltstone bedrock of the Shire Member of the Wasatch Formation which is overlain by wind-blown deposits of fine sand and silt (loess). The bedrock locally contains interbedded thin sandstone units while the loess locally contains small amounts of clay and calcium carbonate cementation.

The mesa sideslopes consist of exposed bedrock, alluvial terrace deposits, colluvium and sheetwash deposits. The alluvial terrace deposits consist of a poorly sorted mixture of sand, gravel and cobbles which have been locally mined and processed for use as aggregate.

GEOTECHNICAL INVESTIGATIONS

During the initial geotechnical investigation the existing runway and apron area subgrades were examined along with the area beyond the end of the mesa. Potential sources of borrow material were also investigated.

The initial geotechnical investigation included a total of 25 boreholes to depths of 4 to 13 meters (12 to 42 feet). The borings were advanced using a rubber-tired truck-mounted CME-75 drill rig using hollow stem auger methods and included Standard Penetration Tests (SPT) on nominal 1.5 meter (5 foot) centers. The SPT samples were supplemented with Modified California Tube samples and Shelby Tube samples.

Additional geotechnical investigations were performed in 2007 in support of the 404/401 permit process and final design. The objective of the 2007 geotechnical investigations was to determine the geotechnical parameters necessary for final design of the county road and Dry Creek relocations as well as the earth fill and soil-cement embankments.

A total of eleven borings were drilled in 2007 at select drop structure locations and on the east and west ends of the mesa. The borings were advanced with an AP1000 percussion hammer truck-mounted drill rig using nominal 10-inch diameter dual wall drill rods with cuttings removed by air using reverse-circulation methods to drill through the coarse cobble-rich alluvial deposits present. Standard Penetration Tests (SPT) and Modified California Tube samples were collected from the borings and bulk samples of select on-site materials were obtained.

Groundwater was encountered at depths ranging from 5 to 21 feet in the borings on the east end of the mesa. No groundwater was encountered in borings on the west end or those drilled for the drop structures. At the west end of the mesa power lines, the intersection of two county roads, numerous utilities and two irrigation ditches limited drill rig access during the 2007 investigations.

SUBSURFACE CONDITIONS

Borings conducted on top of the mesa during the 2004 investigation encountered loess to the full depth of each boring (3 to 5 meters, 12 to 17 feet). SPT N-values in this material ranged from 3 to 33 blows per 30 centimeters (blows per foot, bpf). Higher N-values were attributed to the presence of slight calcium carbonate cementation. Lower N-values may represent material that had previously been disturbed as a result of construction activity at the site. Typical N-values obtained in the loess on top of the mesa were on the order of 8 to 12 bpf.

Laboratory testing conducted on samples of the loess indicate the deposits consist of silt with fine sand and clay. The material classifies as low plasticity clay (CL) to non-plastic silt (ML) in the Unified Soil Classification System (USCS) with liquid limits of non-plastic to 36 and a plasticity index (PI) of non-plastic to 23. The loess was also tested for possible use as a fill borrow source area. This testing included Modified Proctor compaction (ASTM D1557) and California Bearing Ratio (CBR, ASTM D1883).

The geotechnical borings conducted at the foot of the mesa beyond the west end of the existing runway and south of the airport encountered variable fine grained sheetwash and alluvial deposits overlying dense alluvial mixtures of sand, gravel and cobbles. All of the 2004 borings terminated with auger refusal in the dense alluvial deposits and resulted in the use of the percussion hammer drill for the 2007 investigations. One of the 2004 borings encountered very soft, wet, silty sand / sandy silt at a depth of 3 to 3.5 meters (10 to 11.5 feet) along Dry Creek on the south side of the mesa. The soft material was located beneath denser material of similar composition and above very dense coarse grained alluvial / colluvial material.

Three additional borings were drilled in the area to investigate the continuity of the soft material. Although material of similar composition was encountered in one of the additional borings to a depth of 5 meters (16.5 feet), SPT N-values indicated the material to be medium stiff to very stiff at that location. The other two borings encountered very dense, coarse grained alluvium immediately below the ground surface. Borings were also conducted along Dry Creek during the 2007 investigation and no soft zones were encountered. The soft zone encountered in the 2004 investigation on Dry Creek is thought to be an isolated pocket, perhaps an old stream channel that was filled in with silt and clay as back channel deposits.

During the 2004 investigation, three borings were conducted at the foot of the mesa beyond the east end of the existing runway. These borings encountered approximately 2.5 to 7 meters (8 to 23 feet) of stiff to very stiff loess overlying weathered claystone / siltstone bedrock.

The 2007 geotechnical investigation told a different story. Some unanticipated very soft fine alluvium material was encountered in several boreholes on the east end of the mesa: the drill rods sank under the weight of the rods with no hammer blows in some zones and SPT N-values were zero to three. This material was not encountered during the previous investigation as the previous borings were all drilled on the upper two terraces of three topographic terraces in the alluvial valley on the east end of the mesa. The lowest blow counts and worst conditions were encountered near Mamm Creek in the vicinity of the northeast edge of the proposed fill. The soft material was encountered on the lowest terrace in boreholes TT07-2 and TT07-2A (See Figure 2). The fine alluvial deposits encountered in these boreholes extend to a depth of approximately 8 meters (25 feet). Water was encountered in this area at depths ranging from 1.5 to 6.5 meters (5 to 21 feet). The samples that were tested in the laboratory classified as silty sands (SM) and low plasticity clays (CL) under the USCS system.

It was extremely difficult to obtain high quality samples suitable for laboratory testing during the site investigation due to the nature of the saturated materials. Typical of alluvial deposits, there was significant vertical variability, often with three or more distinct material types occurring within a single SPT sample.

Discussion

A supplemental geophysical investigation was conducted in early spring 2008 to verify the amount of soft material. A seismic refraction survey off the east end of the mesa in the suspected soft zone area (see Figure 2) indicated that the soft zone likely includes the entire extent of the lower topographic terraces underneath the embankment. The lower terraces include an area of 5600 square meters, 60,000 square feet) and the soft zone extends to an average depth of approximately 5 meters (15 feet), for a total volume of approximately 27,500 cubic meters (36,000 cubic yards). The project schedule and funding did not allow for additional investigations to obtain higher-quality samples for laboratory testing or cone penetration testing that would be required for a more detailed analysis of the material and staged construction analyses. Therefore the soft zone under the embankment was excavated and replaced prior to embankment construction in 2008. During construction, the soft zone was found to extend to depths ranging from 10 to 15 feet and it was underlain by a dense gravel layer. The embankment itself will consist of approximately 475,000 cubic meters (620,000 cubic yards) of fill.

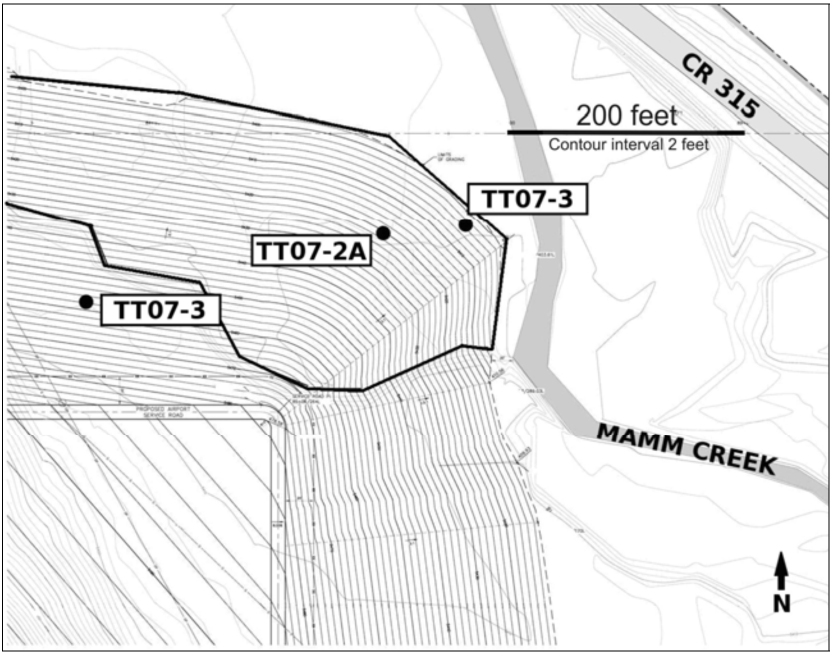

Figure 2. East end of the mesa showing both existing and proposed topography. The darker line outlines the approximate extent of the very soft, wet materials that underlie the proposed 30 meter (90 foot) embankment.

BORROW SOURCES

Geotechnical investigations at the site included assessing potential borrow sources for earthwork construction. The results of the investigation indicated two primary material types as potential borrow sources for construction: the loess deposits on top of the mesa and the coarse sand, gravel and cobble alluvial terrace deposits located along the flanks of the mesa and elsewhere in the general area.

The loess is present in large quantities across the entire surface of the mesa and is therefore readily available for use. Although this material is fine grained and generally exhibits considerably lower strength characteristics than the coarse alluvium, it is proposed for use on most fill slopes of 2.5H:1V or flatter on the project.

The coarse alluvial terrace deposits observed at the site form excellent high-strength fill material and are proposed for use for conventional and zoned earthfill slopes up to 2H:1V and as aggregate for the soil-cement. At the airport site the deposits are primarily located on the northern and western fringes of the mesa top and side slopes and contain limited quantities of available fill that will require selective borrowing. Concurrent with the airport upgrades, County Road 319 southwest of the airport is to be realigned and upgraded. This road upgrade requires excavation of a hillside composed of a significant volume of coarse alluvium, thus generating more alluvium available for use as borrow. Some of the alluvial material may also be used as aggregate for base coarse, asphalt, or concrete.

During the 404/401 permit process a geotechnical investigation was conducted to estimate the extent of the coarse alluvium as well as its suitability for use as a construction material. There was found to be an adequate supply of coarse alluvium meeting FAA requirements for use as subgrade, subbase, base course and asphalt or concrete aggregate. The material will have to be uncovered, excavated, screened, crushed, sized and recombined in order to meet the specific requirements of each product. It is thought that this can be done economically and contractors will be allowed opportunity to make use of this on-site material. The material will also have to be screened for use as aggregate in soil-cement.

SOIL-CEMENT

Because the project requires fill slopes steeper than those that can be supported by the onsite materials without the use of retaining walls or soil amendments, soil-cement was selected as the preferred method to increase slope angles in the required areas. The high cost of retaining walls and other reinforced soil slope options was also a factor in the decision to use soil-cement. Because soil-cement was proposed for use in some site fills, it was decided to also use soil-cement for the drop structures used to realign Dry Creek. Soil-cement will also be used for outlet and channel protection near the intersection of County Roads 319 and 346 on the west end of the mesa.

During preliminary design stages for this project, the use of the on-site loess material amended with fly ash or cement to obtain a material with adequate strength for the steep fill slopes required was evaluated. In 2006, bulk samples of the loess material were collected and two amended samples were tested. The amendments considered included 10% Type C fly ash and 5% cement by dry weight. It was determined that the fly ash did not react in a cementitious manner with the loess, but the addition of cement did increase the strength of the material.

Despite the strength gain imparted to the loess by the addition of cement, further research into soil-cement applications revealed that the loess material has too high a fines (material finer than a No. 200 sieve or 0.074 mm) content to produce strength properties that would be adequately resistant to freeze-thaw weathering. Generally, a material with more than 8% of fines is not suitable for use as soil-cement aggregate for this type of application. The soil-cement mix design evaluation therefore proceeded in 2007 using the on-site coarse alluvium. Initial testing was performed on samples from three test pits on the south side of County Road 319. Initial testing included gradations and Atterberg Limits. The material from one test pit was found to have too high of a fines content to warrant further testing. The material from the two other test pits was very similar. All of the cobbles and gravel greater than 38 millimeters (1.5 inches) in diameter were screened off, as that is the maximum particle size that is practical to use in soil-cement for this application. The oversize particles from the combined samples amounted to 17% of the total sample weight.

The resulting gradation of the soil-cement aggregate is similar to that used for roller-compacted concrete (RCC). For the purposes of this project, the nomenclature of the fill material was of little consequence, so the decision was made to retain the name soil-cement.

Based on the initial laboratory testing and the soil's AASHTO classification (A-2-4), three trial mixes using 5%, 7% and 9% cement by dry weight of soil aggregate were subjected to further testing. Testing proceeded with a Modified Proctor compaction test on the mixture at the median cement content (7 %). Eight 150 millimeter diameter by 300 millimeter high (6-inch by 12-inch) cylinders were prepared for each of the specimens to be tested. Unconfined Compressive Strength (UCS) testing was performed at 7, 28 and 90 days. A portion of the cylinders were also subjected to freeze-thaw testing using the methods outlined by Choi and Groom (2001). A summary of the soil-cement testing is presented in Table 1.

Table 1. Soil-Cement Testing Results

Trial Mix	**Average 7-day Compressive Strength (kPa, (psi))**	**Average 28-day Compressive Strength (kPa, (psi))**	**Average 91-day Compressive Strength (kPa, (psi))**	**12-cycle Freeze-Thaw Weight Loss (%)**
# 1 - 5 percent Cement	4,830 (700)	5,650 (820)	8,140 (1180)	1.5
# 2 - 7 percent Cement	10,070 (1460)	10,620 (1540)	13,510 (1960)	1.3
# 3 - 9 percent Cement	14,200 (2060)	17,370 (2520)	20,550 (2980)	0.5

The soil-cement mix design for this application was developed to produce a minimum 7-day compressive strength of 6,900 kPa (1,000 psi) and a low weight loss during the freeze-thaw test. The cement content obtained in the laboratory was increased by 2% for construction to account for variations in natural materials and construction processes in the field. Based on the laboratory testing results, a cement content of 8% was recommended for the project.

Stability analyses conducted using the material properties determined in the laboratory for on-site materials and conservatively assumed parameters for the soil-cement indicated that a soil-cement buttress 3 to 9 meters (10 to 30 feet) wide can support the proposed 20 to 30 meter (60 to 90 feet) high fill slopes on the west end of the mesa. The parameters used for the stability analyses of the soil-cement embankment are presented in Table 2.

Table 2. Selected Material Properties for Stability Analyses

Material	Unit Weight (kN/m^3, (psf))	Angle of Internal Friction (degrees)	Cohesion (kPa, (psf))
Soil-Cement	23.6 (150)	45	47.9 (1000)[1]
In-Place Loess	18.8 (120)	30	0
Loess Fill	21.2 (135)	32	0
Coarse Alluvium (In-Place and Fill)	19.6 (125)	38	0
Sandstone Bedrock	24.6 (157)	45	23.9 (500)
Claystone Bedrock	20.9 (133)	26	23.9 (500)

Note 1: Stability Analyses were also conducted assuming that the soil-cement had no cohesion in order to simulate the possibility of potential interlift "cold joints". The embankment was found to have adequate stability with this scenario as well.

In many cases, the fill slope will be a composite of soil-cement with free-draining alluvium behind the soil-cement walls and the use of loess as the remaining fill. Many of the proposed slopes include a 3H:1V loess fill slope at the toe and 2.5 meter (8 foot) wide benches to improve safety. Figure 3 shows the existing and proposed topography of the west end of the mesa and Figure 4 shows a proposed cross-section (Section 1 from Figure 3).

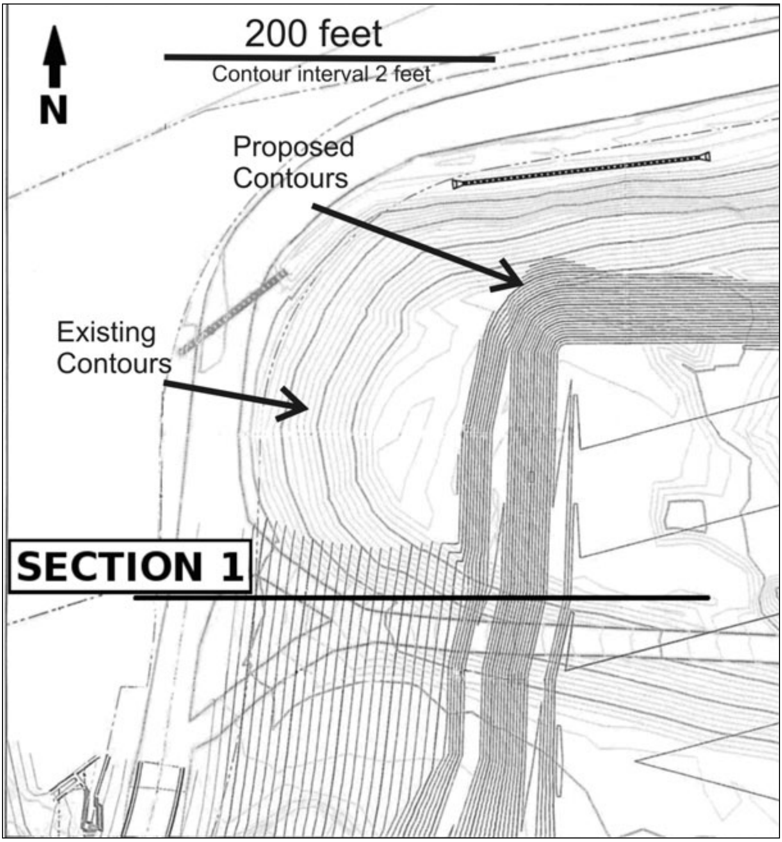

Figure 3. Existing and proposed topography on the west end of the mesa.

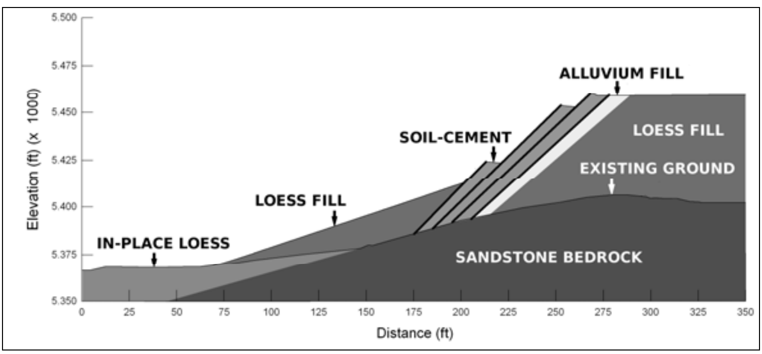

Figure 4. Section 3 through a proposed 1H:1V soil-cement composite fill slope.

SUMMARY

The initial geotechnical investigations conducted at the Garfield County Regional Airport indicated that the construction of large embankments at either end of the mesa to accommodate runway upgrades is technically feasible. The initial investigation determined that fill slopes of 2.5H:1V or flatter could be constructed using conventional earthfill methods and readily available fill material (loess). Fill slopes of 2H:1V could be constructed using the select borrow material derived on site (alluvium or a composite slope with alluvium and loess). Steeper fill slopes with 1.5H:1V to 0.5H:1V could be constructed with soil-cement or other methods

Because more conventional reinforced earth slopes and concrete or MSE retaining walls were deemed too expensive, soil-cement was selected as an innovative and cost-effective method to increase slope angles to meet the demanding geometric constraints of the site. Coarse alluvium from on-site sources was selected as the aggregate for the soil-cement, resulting in an acceptable compressive strength and exceptional freeze-thaw resistance with a cement content of 8% by dry weight of aggregate. Stability analyses indicated that a soil-cement buttress 3 to 9 meters (10 to 30 feet) wide can support the proposed 1H:1V embankments on the west end of the mesa.

During an initial geotechnical investigation, a soft zone was encountered in a boring on the south side of the mesa. This was thought to be isolated as subsequent surrounding borings gave no indication of similar soft material. The extent of this soft zone will be monitored during construction in that area. However, on the east side of the mesa a large area of soft material was encountered and the construction schedule and budget necessitated that the material be removed prior to embankment construction.

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