OHIO RIVER VALLEY
SOILS SEMINAR XLII

October 21, 2011 | Cincinnati, OH

LESSONS LEARNED: FAILURES AND FORENSICS

Planned Agenda – Friday, October 21, 2011

6:30-7:30 am	Exhibitor Registration and Setup
7:30-8:15 am	**Registration**
8:15-8:30 am	Welcome Remarks – Jared M. McFaddin, P.E., City of Cincinnati
8:30-9:05 am	“Lessons Taught by Excess Pore Water Pressure and Pyritic Sulfur” – George Webb, P.E., HC Nutting, A Terracon Company
9:05-9:40 am	“The DNA of the Leaking Basement Epidemic” – Stuart Edwards, P.E., Verdant Energi and Environment
9:40-10:15 am	“Repair of Ground Movement in a Filled Slope in an Abandoned Quarry” – Farrokh Screwvala, Ph.D., P.E., President, Farrokh N. Screwvala Inc.
10:15-10:35 am	**Break**
10:35-11:10 am	“Risk Considerations for Geotechnical Construction” – George Burke, P.E., D.G.E.
11:10-11:45 am	“Lessons Learned When Mitigating Liquefaction Potential Using Vibrocompaction and Stone Columns” – Rick Deschamps, Ph.D., P.E., Nicholson Construction
11:45-12:35 pm	**Lunch**
12:35-12:45 pm	Keynote Introduction – Joseph D. Hauber, P.E., Thelen Associates, Inc.
12:45-1:40 pm	**Keynote:** “The Practice of Forensic Engineering” – Patrick C. Lucia, Ph.D., P.E., Principal, Geosyntec Consultants
1:40-2:15 pm	“Forensic Geotechnical Engineering for the Insurance Industry: Earthquake, Blasting, Weather, and Ground Related Claims” – John S. Nealon, P.E., Thelen Associates, Inc.
2:15-2:45 pm	**Break**
2:45-3:20 pm	“Temporary Earth Retention – Project Performance: East Bank at the Flats, Cleveland, Ohio” – Rick Slack, P.E., Richard Goettle, Inc.
3:20-3:55 pm	“Consolidation Analyses of Greater Cincinnati Soils” – Nishant Dayal, P.E., HVJ Associates, Inc.
3:55-4:30 pm	“Observational Method Using Real Time Surface Settlement Monitoring – The South Toulon Tunnel Project” – Boris Caro Vargas, Soldata, Inc.
4:30-4:45 pm	Closing Remarks

Proceedings of the 42nd Annual Ohio River Valley Soils Seminar

Lessons Learned:
Failures and Forensics

October 21, 2011
Millennium Hotel Cincinnati
150 West Fifth Street
Cincinnati, Ohio

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Lessons Taught by Excess Pore Water Pressure & Pyritic Sulfur

George C. Webb, P.E., LEED AP, M. ASCE [1] and
Andrew Bodocsi, PhD, P.E., M. ASCE [2]

Abstract: This paper will address two issues which can cause major problems for a project, if they are not recognized prior to construction. This includes the dramatic influence excess pore water pressures can have on global slope stability and the heaving potential due to secondary mineral growth of certain slag materials.

The impact of excess pore water pressures is presented through the example of the Anderson Township Park-N-Ride earthen embankment in Cincinnati, Ohio. Slope failure occurred in this 55 ft. high embankment due to the presence of high moisture cohesive soils in the lower lifts of this controlled fill. The fill embankment was constructed quickly and the excess pore water pressures could not dissipate, resulting in a massive landslide. The lesson learned is to emphasize the importance of holding firm on maintaining the specified moisture content range of the fill soil and to avoid using unconfined compression tests to determine the suitability of a controlled fill.

The second point of discussion includes a "Green" case history where granular on-site surface paving materials were unknowingly recycled by the project plumbing contractor for use as trench backfill. The granular material was a slag which contained pyritic sulfur. The paper describes conditions, the cause of heave and presents solutions which were considered for repair. The lesson learned is to never use slag inside of a building.

Introduction

The goal of this paper is to present two case histories which describe field conditions where failures occurred such that these conditions can be recognized and avoided on future projects. The first case history deals with a landslide which occurred due to excess pore water pressures developing in an unsaturated controlled fill. The second case history addresses heave problems as associated with secondary mineral growth in trench backfill consisting of recycled on-site material.

Case History 1: Anderson Township Park-N- Ride – Cincinnati, Ohio

A large landslide, encompassing approximately 1-½ acres, occurred on October 1, 2004 at the Anderson Township Park-N-Ride development in southeastern Cincinnati, Ohio. This 200 car Park-N-Ride facility was the first phase of a 21 acre Public Development which would eventually include construction of a large (38,000 SF), two-story Anderson

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Township office and banquet facility with dramatic public grounds, including multilevel ponds, landscape areas and open park spaces.

Prior to construction, the Park-N-Ride Phase I project site had approximately 65 feet of relief, with grades sloping down from the existing shopping mall grade at approximate elevation 800 feet in the south, to creek level elevation 735 at the toe of slope. H.C. Nutting (HCN) performed a geotechnical study for this work in 2003. This study showed the Park-N-Ride site had previously served as a waste-soil spoil area, when the original Beechmont Mall (due south of the site) was constructed and during the 1981 Mall expansion. Further, fill had been placed in the west portion of the site when the Hamilton County Five-Mile connector roadway was constructed. Prior to construction of the Park-N-Ride embankment, it was known that 30+ foot deep uncontrolled fill was present on this site. HCN soil test borings showed a wide variety of soils and shale fragments had been placed on this site, both in an uncontrolled (end-dumped) and compacted condition. Organic matter, cinders and shale were included in this old fill. Moisture contents were variable within the fill soil, with very moist to wet conditions being noted in some of the test borings. Field reconnaissance during the geotechnical study showed a preconstruction landslide was present which encompassed approximately 27,000 sf, (0.6 acre) immediately upslope of the existing creek. This landslide was attributed to the original 3H:1V grade, the existing poor quality fill and undercutting of the toe of slope by creek erosion.

Significant grading was planned for the Park-N-Ride facility in order to create a relatively flat, 2-acre parking lot for 200 vehicles and a waiting area /restroom facility. A maximum 55-foot high embankment having a proposed 3H:1V slope was to be constructed over a known layer of uncontrolled fill. It was recognized that the original slope was only marginally stable and consisted primarily of uncontrolled fill. Analyses were completed in the geotechnical study which indicated that a portion of the existing uncontrolled fill (amounting to approximately 25,000 cu. yds.) would need to be removed in order to prepare the site to receive the proposed embankment fill. Specifically a 50 to 60 foot-wide horizontal bench was initially excavated at the toe of slope to expose undisturbed, horizontally bedded shale and limestone bedrock. This shale bedrock served as the foundation of the embankment. The bedrock belongs to the Fairview Formation, which traditionally consists of about 65% shale and 35% thin limestone layers. Bench drains were constructed and controlled fill was placed directly on the undisturbed shale surface. It was recommended by HCN to dry and reuse the majority of the existing lean clay and weathered shale fill soil as the new structural fill for the embankment construction, with the fill soil to be uniformly compacted to 95% Standard Proctor density (ASTM D-698). This was a fast-tracked project, with the goal of completing the required embankment before the traditional fall rains commenced. Clearing and grubbing operations began on August 26, 2004 with earthwork operations immediately following.

The Phase I fill embankment was constructed between September 6, 2004 to September 24, 2004, reusing the onsite fill soil as a structural fill. HCN was on site on a daily basis to observe and spot-check the in-place density of the fill, as it was being placed. Field observations showed the fill was typically placed in 8 to 12-inch loose lifts, with each lift being uniformly compacted to at least 95% Standard Proctor density (ASTM D-698), though the soil was typically wet of optimum. Field observations noted pumping action being observed in the fill under load of the Cat 825 sheepsfoot roller. In addition, the suitability of the fill was judged during fill placement by using pocket penetrometer testing, to provide an estimate of the unconfined compressive strength and undrained shear strength of the fill. Typically, the pocket penetrometer showed an unconfined compressive strength greater than 2.5 TSF.

Fig. 1 plots the field moisture contents vs. time of the field density tests which were performed from September 7 to 24, 2004. Fig. 1 plots the individual density test moisture (percentage points above and below optimum moisture) for each day of testing. The job specification called for the soil moisture content to be ±3% of optimum moisture. The plot shows two tests had a moisture content less than –3% of optimum and 50 tests where the moisture content was greater than +3% of optimum. Many of the high moisture tests were recorded on September 7 and 8, 2004, near the bottom of the embankment fill.

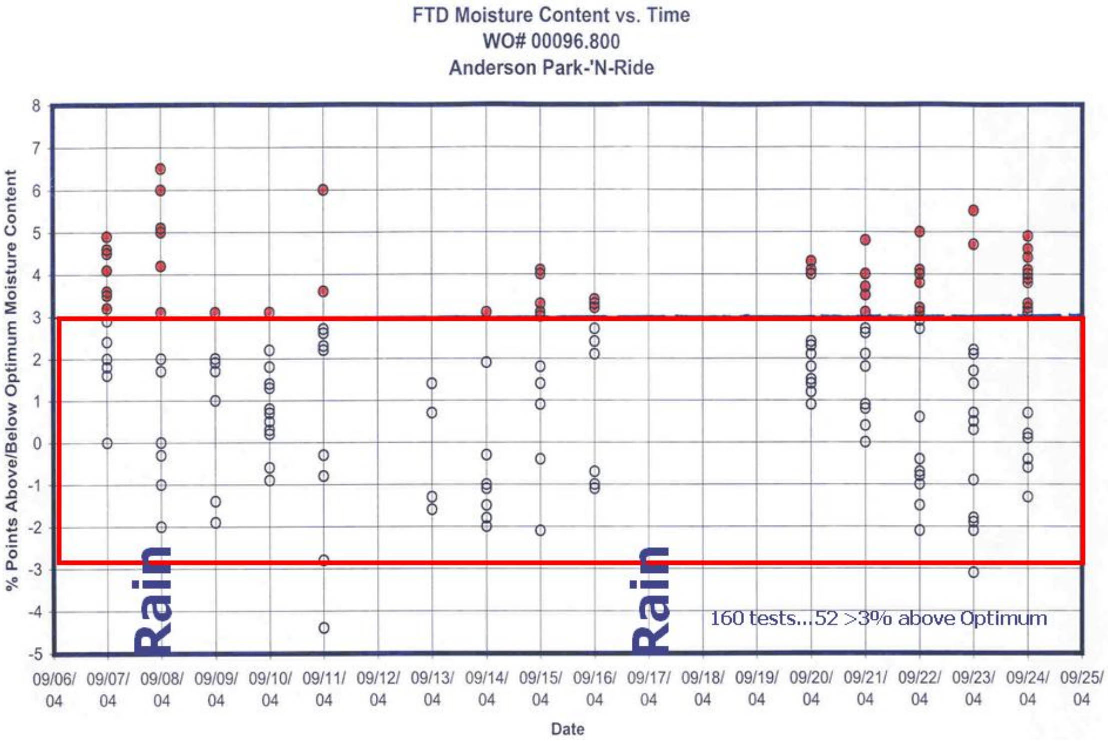

Fig. 1. Field Density Test Moisture Content vs. Time

On October 1, 2004, the Hamilton County Project Engineer called HCN, indicating hillside movement had occurred overnight within the recent, quality-controlled fill embankment. Following a site meeting to observe conditions, a drill rig was immediately dispatched to the site to obtain factual information and to install inclinometers. Three test borings were performed which extended to undisturbed shale bedrock. Lateral movement occurred within the landslide during drilling operations. The inclinometer casings sheared within 2 days of installation, providing a definite plane of movement within the failure mass, at the inclinometer casing. The slip plane occurred primarily within the lower portion of the new controlled fill (see Fig. 4).

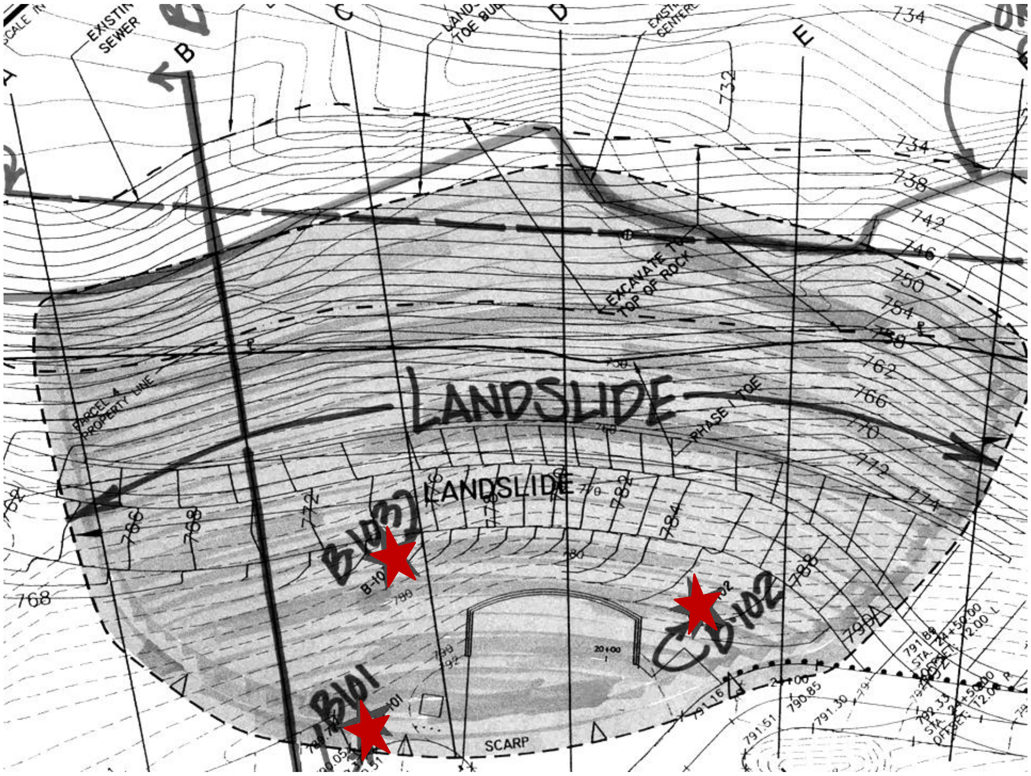

Fig. 2. Test Boring Location Plan

The photos in Fig. 3 illustrate site conditions on October 2, 2004, the day after movement was first noted.

Analyses

Following drilling operations and laboratory testing, numerous slope stability analyses were performed evaluating both rotational and translational modes of failure. The analyses were based on the Rankine Block Method (Modified Janbu Method) and the Modified Bishop Method. In the Rankine Block Method, the failure surface was searched based on restricting the slip through the observed toe bulge, head scarp, and piezometer/inclinometer shear/pinch depths. In the Modified Bishop Method, the slope failure surface initiation and termination boundaries were set, but the slip surface was not restricted.

Slope stability analyses were performed using PCSTABL5M as developed by Purdue University in conjunction with the Indiana Department of Highways. Analyses were performed at two cross sections; B-B and E-E (see Fig. 2 for location of these cross sections). We evaluated the impact pore water pressures would have on the as-built embankment, using the soil parameters from our original, 2003 analyses. The failure model showed known points of movement (survey located head scarp and toe bulge, plus inclinometer and piezometer data), using a water table slightly higher than ½ the distance between the top of rock and grade to yield a safety factor of 0.98 (failure condition).

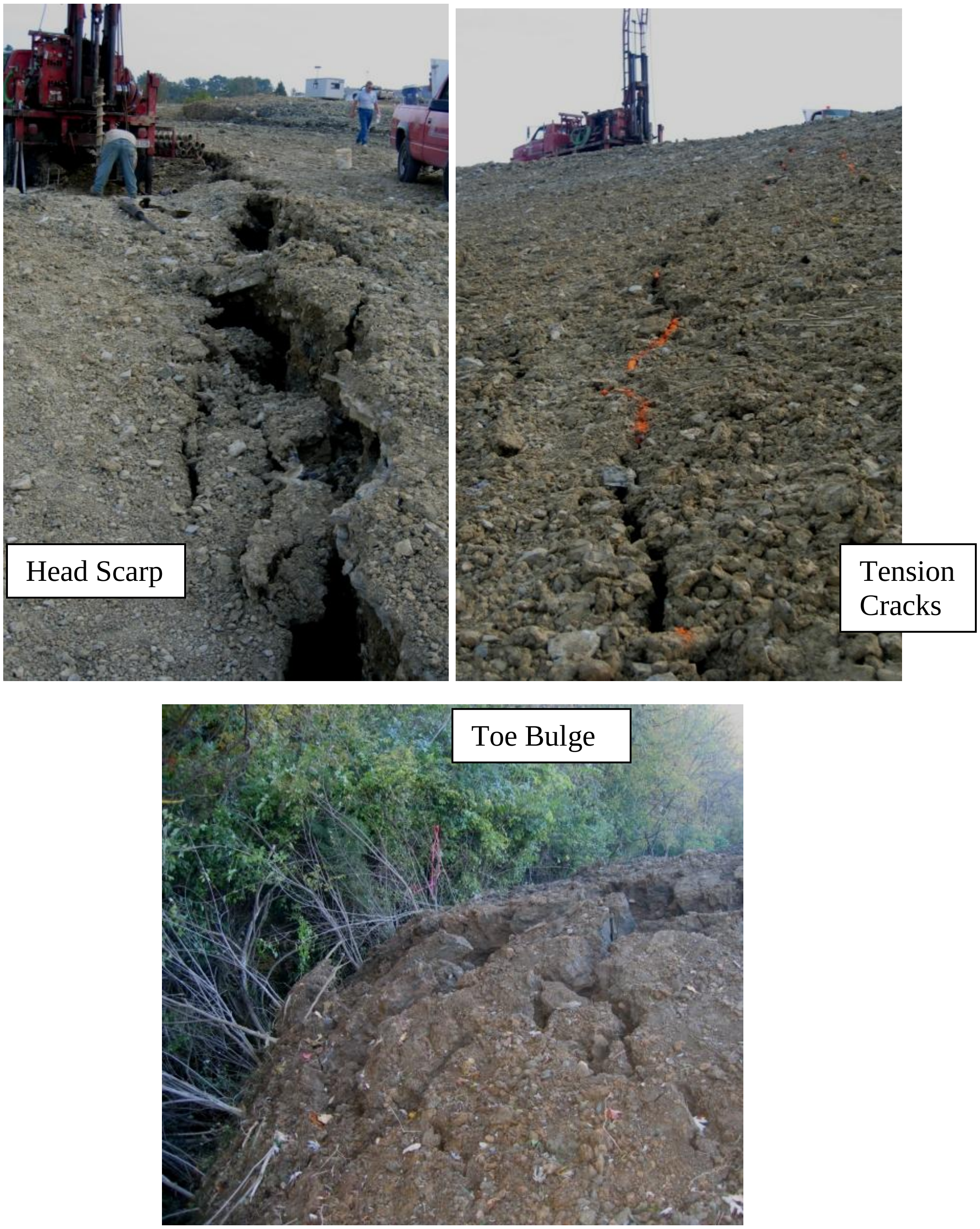

Fig. 3. Landslide Photos from October 2, 2004

In addition to the slope stability analyses, we studied and interpreted the orientation of the tension cracks which developed, the soil conditions encountered in the test borings and the results of the slope inclinometers to determine the cause of failure. Visual observations indicate that the slide started at the west end and progressed to the east.

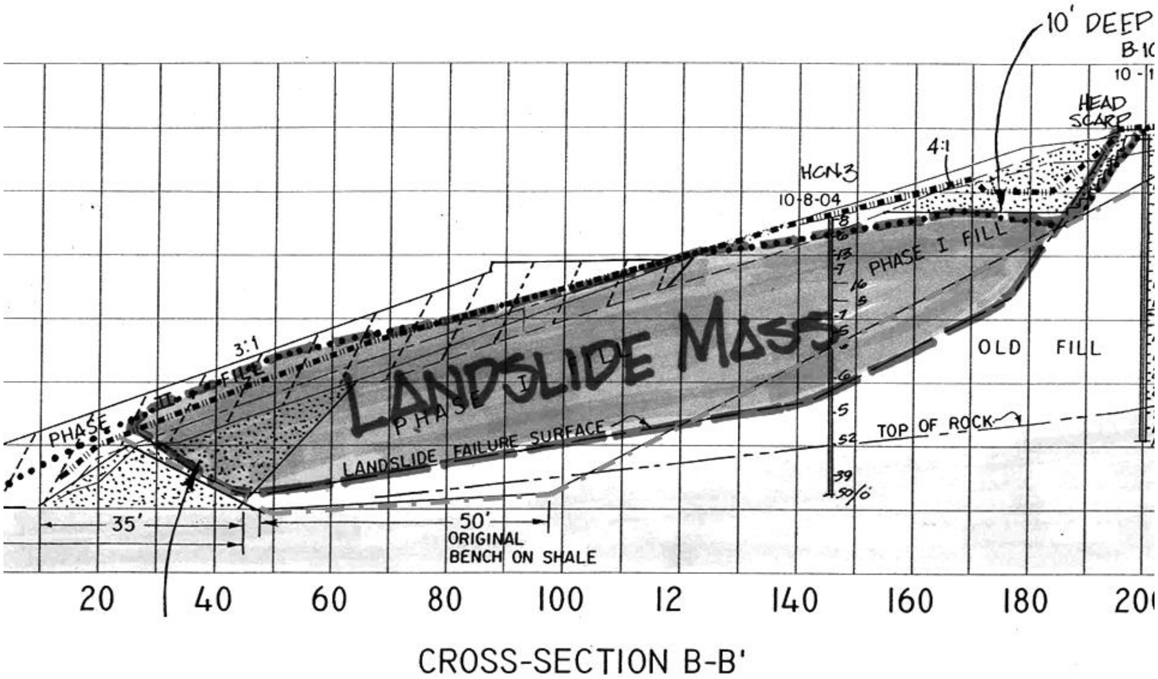

Fig. 4. Cross section showing the embankment failure mass

Remediation

Remediation consisted of reworking a portion of the existing fill, such that the soil was moisture-conditioned prior to re-use as a structural fill, plus the construction of a buttress fill. Analyses showed the proposed Phase II embankment construction, together with removal and lime treatment of a portion of the landslide mass, was sufficient in order to achieve both short term and long term stability. Specifically, HCN recommended that the landslide remediation plan include the use of onsite glacial till (which was naturally near optimum moisture content) or to treat the very moist to wet existing fill soil with 2% quicklime, when needed, to reduce the moisture content of the old and new fill soil.

Our analyses used the following effective stress (long-term) soil parameters to determine the size of the stabilizing buttress fill. It was fortunate in that a significant portion of the stabilizing buttress consisted of the planned Phase II embankment. In these analyses, the fill soil parameters were the same parameters as used in the original HCN 2003 slope stability analyses, which we felt were somewhat conservative, especially for the controlled fill. We also performed analyses using the results of the laboratory tests (consolidated-undrained triaxial tests, with pore water measurements), which generally confirmed these parameters.

Table 1. Effective Stress (Long-Term) Condition Soil Parameters

Soil Type	Angle of internal Friction (ϕ)	Cohesion (C) - psf	Unit wet Weight (γ) – pcf
September 2004 Fill	18	50	130
Original Fill	18	50	120
New Lime Mod. Fill	28	300	125
Weathered Shale	30	1000	140

Summarized in Table 2 are the results of our slope stability analyses, at cross sections B-B and E-E. The failure models included the surveyed head scarp and toe bulge, as well as the points of movement as defined by the two inclinometers and the piezometer.

Table 2. Slope Stability Analyses Results

Case	Cross Section	Condition	Min. SF	Max Depth of Failure
1	B-B	Short Term –Excess PWP – Failure Condition	0.98	28’
2	B-B	Long -Term-Only Phase II Buttress Fill to El 770	1.64	28’
3	B-B	Long-Term - Toe Cut to 24’ – Phase II Buttress Fill to El 770	1.69	28’
4	B-B	Long Term -Toe Cut to 34’ – Phase II Buttress Fill to El 770	1.74	28’
5	E-E	Short Term – Excess PWP – Failure Condition	0.98	17’
6	E-E	Long-Term -Toe Cut to 24’ – Phase II Buttress Fill to El 770	1.60	17’
7	E-E	Long-Term -Toe Cut to 34’ – Phase II Buttress Fill to El 770	1.75	17’

The analyses which were performed to evaluate the Anderson Park-N-Ride landslide used total stress (undrained) soil conditions for the October 2004 landslide, to reflect short-term conditions. Effective stress (drained) conditions to evaluate the long-term slope stability following remedial action. The analyses showed and the field instrumentation indicated that the October 2004 landslide failure slip plane occurred primarily within the recently placed, Phase I embankment fill.

Discussion

When a cohesive fill embankment is constructed quickly, the soils are loaded under what is termed an ‘undrained’ condition from the dead load of the fill soil, in conjunction with the low permeability characteristics of the cohesive fill. Our analyses indicate the failure mechanism at the Park-N-Ride project was the result of the development of high pore water pressure due to the weight of the fill above, coupled with relatively high moisture contents and high plasticity (low permeability) fill soils. When the embankment load was applied to the very moist fill in the lower elevations (from the weight of subsequent lifts of compacted fill), the load was carried by the 'trapped' excessive moisture within the soil, instead of 'soil particle to soil particle' transfer of load. This is termed excess pore water pressure and is the result of saturated, plastic cohesive soils not draining quickly enough (due to the low permeability) to allow the pore water pressures to dissipate. When this condition is recognized, the placement of the fill is stopped, until the excess pore water pressures dissipate to an acceptable level. This delay allows most of the loading to be carried by the soil particles, resulting in a more stable condition. For the Park-N-Ride embankment, the heightened pore water pressures resulted in a distinct reduction in the effective soil shear strength, which resulted in the observed landslide. This reduction in soil strength due to excess pore water pressures (u) is shown as the expression:

$$S = C + \left(\frac{1}{3}(\sigma_1 + 2\sigma_3) - u \right) \tan(\phi') \qquad \text{(Equation 1)}$$

where S = Soil Strength, C = Cohesion, u = Pore water pressure, σ_1 = major principal stress, σ_3 = minor principal stress, and ϕ' = effective strength angle of internal friction.

Thus, with an increase of pore water pressure (u), the normal effective stress will decrease. As a result, the soil strength will decrease, as long as the pore water pressure is present.

When the development of the excess pore water pressures is recognized in the field, staged construction methods are used to allow the excess water pressure to dissipate. Typically, this condition develops in saturated soils, below the water table. Staged construction methods require the embankment to be constructed in increments, such that there is a delay of several days or weeks, before placing the next interval of fill. HCN has been involved with hundreds of embankments which have been constructed successfully with the benching, finish grade slope and compacted fill methods as were performed for the Park-N-Ride development. The difference with the Park-N-Ride embankment construction was we did not recognize the potential of developing excess pore water pressures in the lower portion of the unsaturated fill embankment (which was above the static water table) and was significantly wet of optimum. Due to the weight of the embankment above, the bottom portion of the new fill compressed, became saturated and developed high pore pressures and lost shear strength.

The lessons learned from this landslide include the following:

1. Follow the project specifications when placing compacted fill, giving equal weight to soil moisture and to field dry density. Recognize the soil moistures greater than 3 percentage points above optimum can cause problems.

2. Recognize the potential for excess pore water pressures to develop in wet soils below an embankment, especially when the wet soil is overlain by a tall embankment (say greater than 40 feet tall).
3. Recognize that fast-tracked fill placement can often result in development of excess pore water pressures within the embankment fill. Staged construction, with pore pressure measurements, could be required to allow excess pore water pressures to dissipate, to avoid landslide conditions.
4. Recognize the importance of using lime or cement stabilized soils as a tool to reduce soil moisture, especially when drying conditions are limited (i.e., during wet weather).
5. Do not use unconfined compression test values (pocket penetrometer values) when evaluating the suitability of a lift of compacted fill soil. Instead, rely on field density testing, natural moisture content, as compared with appropriate Proctor values.

Case History 2: Floor Slab Heave from Recycled Materials

A second case history involves the use of on-site surface paving materials as backfill material. This material was used by the plumbing contractor as granular fill within utility trenches below the floor slab of a new grocery store. With the Green movement, recycling materials is an important consideration during new construction. This case history emphasizes the need to fully understand the chemical makeup of recycled materials which will be incorporated into the construction of a building.

Background information

A proposed grocery store, consisting of new construction, was to be built in a strip mall located in the Western Hills neighborhood of Cincinnati, Ohio. Grading work involved as much as 40 feet of quality-controlled fill in the south end of the store to a maximum 14 foot cut in the north end of the store, where undisturbed gray shale bedrock was exposed. This site was formerly a drive-in theater, which had a surface pavement consisting of crushed slag and cinders. This granular surface material was stripped and stockpiled on-site by the developer. This recycled material is the culprit of this forensic study.

The 65,000 sf grocery store was successfully constructed in 1996. The building is supported on shallow foundations with an independent (not tied to the perimeter walls) slab-on-grade floor slab. The floor slab is a nominal 5-inch thick non-reinforced concrete slab, which is supported on a 4-inch thick aggregate base.

Heave and related distress was first noticed in 2001, when cracking and limited heave was noted in the floor slab. No heave was observed below the building foundations. The floor slab cracking pattern was documented and it was clear that the distress followed the buried utility lines, with cracking being most prominent in the bakery/deli area and in the "Back-Room" (hallway) in the rear of the store. Floor slab heave occurred throughout the store, including areas where shale bedrock was near subgrade elevation, as well as in areas of deep controlled fill. By 2006, heaving of the floor slab in the "Back-Room" had grown to 2-⅛ inches at the rear exterior man-door, creating a serious trip-hazard. Also, heave occurred in the bakery area, where the top of the ovens were pushing up against the ceiling tiles. Frozen food cases were also showing differential movement, due to heaving in local areas of the interior floor slab.

Field Investigation

A test pit was performed within a utility trench in the "Back-Room" where the greatest amount of heave had occurred. The test pit extended 27-inches below finish floor level. Six bag samples, amounting to approximately 400 pounds of backfill were recovered for visual review and laboratory testing, including the dense grade aggregate base below the slab plus the trench backfill. The encountered trench backfill had secondary mineral growth within the pore spaces of the granular material and was difficult to excavate by pick and hand shovel. The majority of the trench fill consisted of crushed limestone, with a silt and clay binder. Some small, infrequently occurring (<0.1%) gravel-size inclusions were noted, which were interpreted as a slag material.

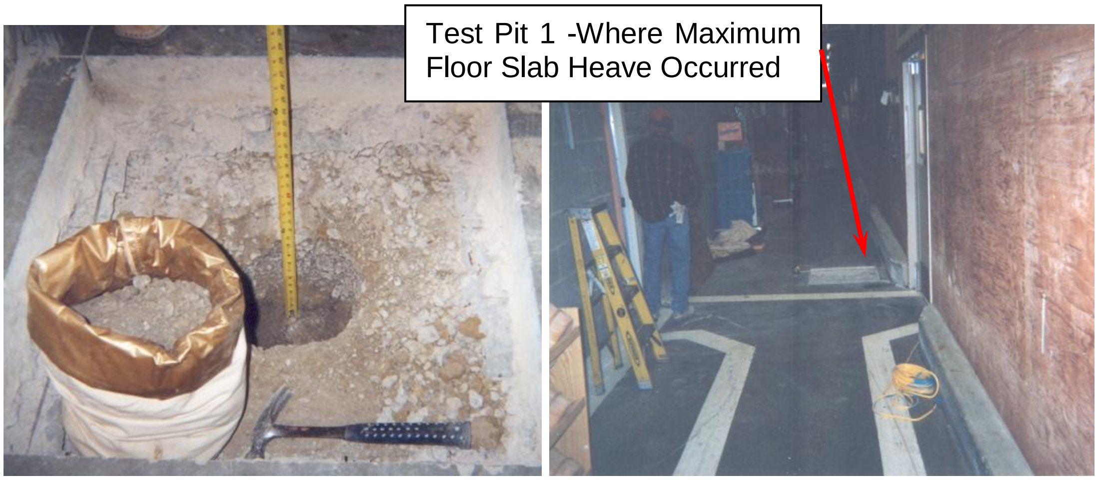

Fig. 5. Photos of Test Pit 1 in "Back-Room"

Six samples of the recovered crushed stone were tested for pyritic sulfur. All six tests showed the presence of pyritic sulfur, ranging from 0.17 to 0.21% pyritic sulfur. The soil had a strong alkaline pH (pH=9 to 12), which favors the formation of Ettringite, one of the most expansive sulfate-based minerals. Literature[1,6] indicates pyritic shale content greater than 0.1% by weight has caused heave in structures.

Heave of the floor slab in the "Back Room" was monitored using a dial gauge which was securely mounted to the exterior building wall. The purpose of the monitoring program was to determine the rate of growth (heave) and to determine if the secondary mineral growth slowed with time. Monitoring was performed from November 20, 2006 to November 9, 2010 and showed approximately 20.3 mm (~ ¾- inch) movement over a 4-year period. Further, the rate of heave showed a rather constant rate, as illustrated in Fig. 6.

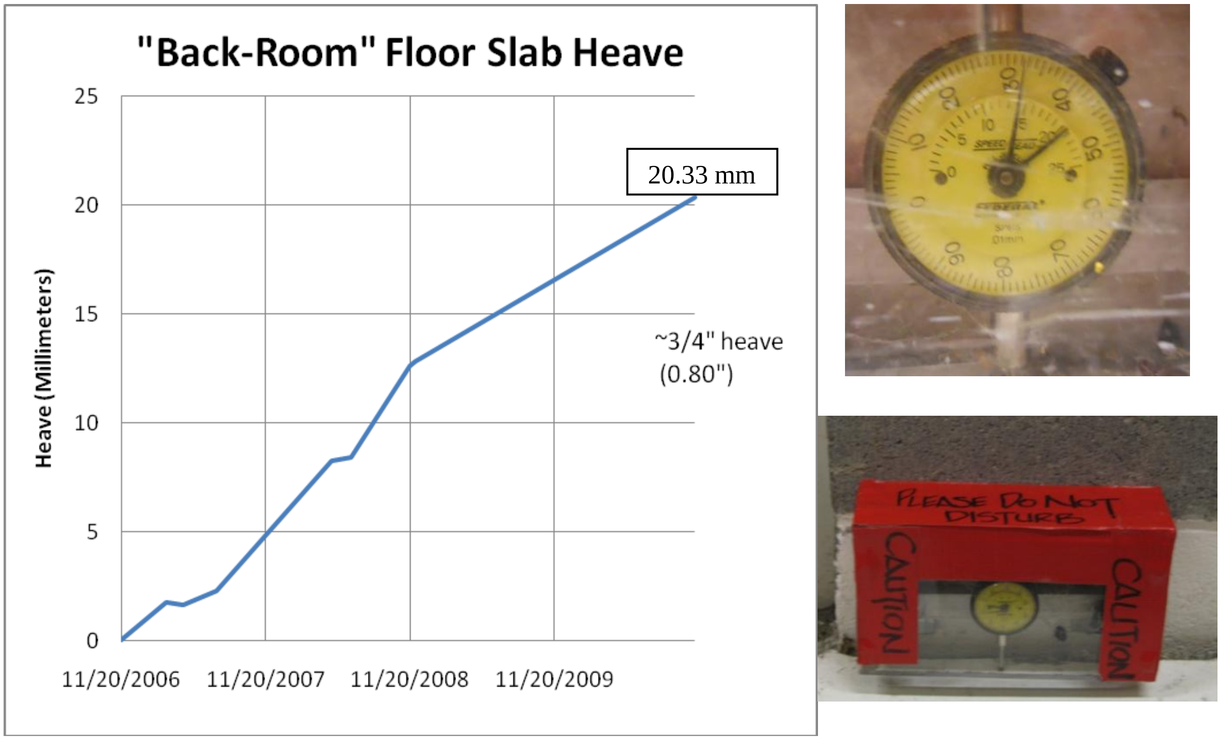

Fig. 6. Floor slab heave versus time

Based on the data developed, it was predicted that floor slab heave would continue. Several remediation options were considered, including:

1. Removal of the contaminated trench fill and replacement with inert compacted fill that is certified to be free of soluble sulfate.

2. Remove a portion of the fill and replace with a compressible Geofoam.

3. Remove a portion of the contaminated fill and replace with a structural floor slab that would span over the excavated trench void.

4. Dissolve the pyritic sulfur with silica fume and then flush the contaminate out of the trench backfill. The silica fume would serve to "capture" the pyritic sulfur and convert it into a non-expansive mineral before it had time to form the secondary mineral Ettringite.

In spring 2011, sections of the floor slab were removed within the heaved areas. This work was performed as a part of a store remodel project. The soils which were exposed included widespread, though random sections of trench backfill consisting of black cinders and slag. The cinders and slag were tested for pyritic sulfur content, which showed pyritic sulfur content to vary from 0.07 to 0.20 %, by weight. In addition, clean sand backfill which was present below the cinders was tested, which showed pyritic sulfur content to vary from 0.03 to 0.04%. This indicated that leaching of sulfur from the cinder backfill and contamination of the underlying sand had occurred.

Illustrated in Fig. 7 are the conditions which were exposed after the floor slab had been removed.

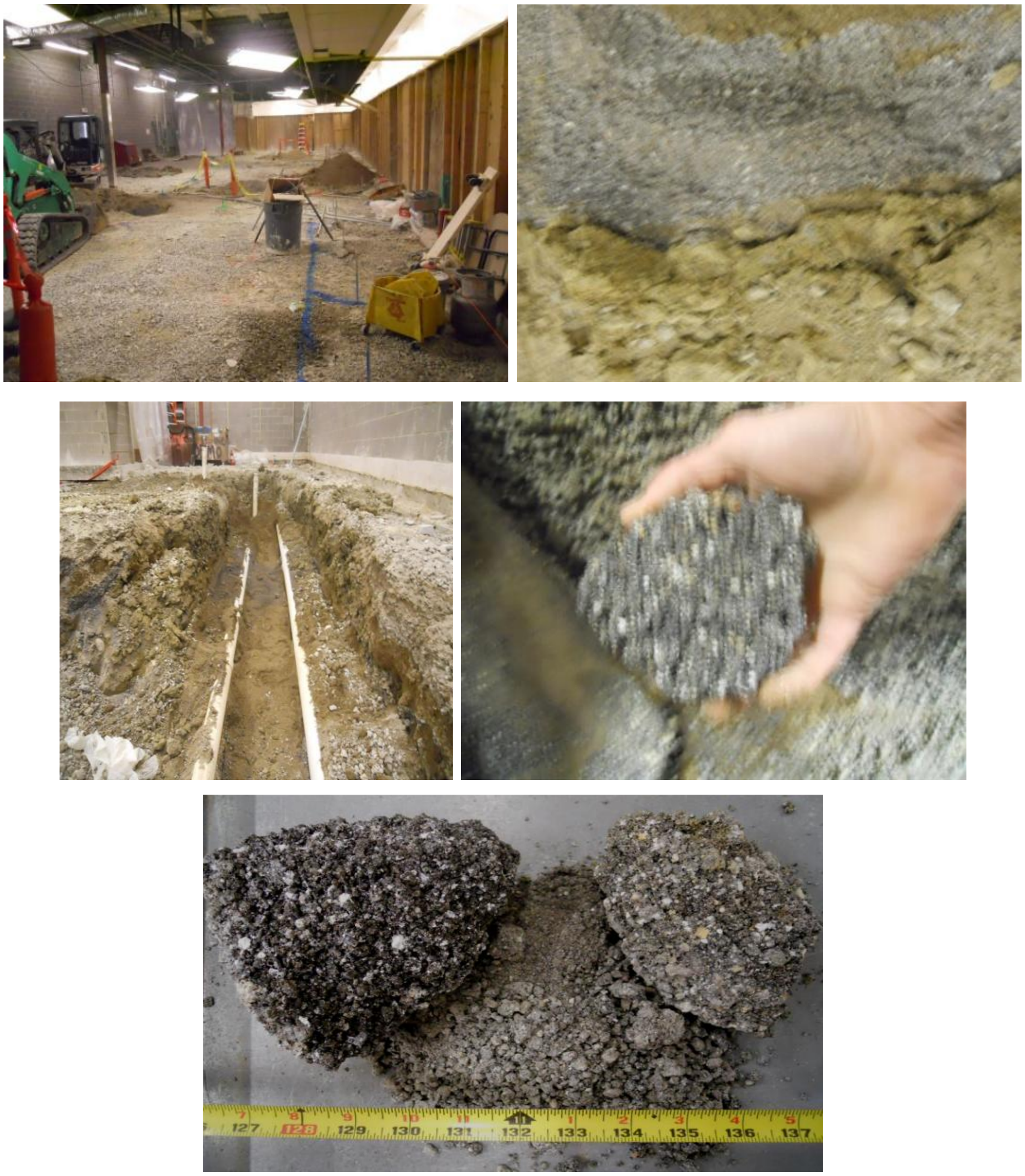

Fig. 7. Photos showing bands of cinder and slag backfill within the utility trenches and gypsum inclusions within the cinder and slag backfill.

The trench widths exceeded 11 feet in some areas. Due to this variation in the trench width, the contractor opted to simply remove the cinder and slag backfill and a portion of the underlying contaminated sand backfill. The undercut areas were backfilled with State approved inert granular fill, which was placed in lifts which were uniformly compacted to 98% Standard Proctor density (ASTM D-698).

The lessons learned include:

1. Never use slag or cinder material as a fill soil within the building footprint.
2. When reusing on-site materials for controlled fill, perform appropriate laboratory testing to determine if pyritic sulfur is a concern.
3. Recognize that heave pressures from secondary mineral growth will easily lift a floor slab (and can lift a two story building, if placed below the building foundation).

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The DNA of the Leaking Basement Epidemic

Stuart Edwards, P.E., L.M. ASCE[1]

Abstract: The 'Waterproofing Contractors' section of many local Yellow Pages is a strong indicator of the scale of the leaking basement epidemic in the United States. The paper draws from a detailed case history to look at its causes and financial ramifications, and suggests ideas for mitigation. The case history tracks a foundation rehabilitation project at an upscale residence in Cincinnati and the evolution of a broader forensic evaluation of contributory causes. As a structural solution, the outstanding performance of FRP strapping and chemical grout injection was critical to the success of stabilizing the structure, but the regrading of the lot and the replacement of all roof drain leaders was equally important to its long term success. The paper also examines the motivations for and effects of earlier, unsuccessful attempts to remedy the problems and, finally, presents the author's experience in assessing the diminished value of several similarly afflicted properties, and the impact that the problem has on the overall value of our national housing stock.

It concludes that most often the failures are systemic involving multiple factors, but that very significant amongst these is the role of roof drainage, the long-term deterioration of the drainage systems and consequent loss of functionality. This is compounded by poor design decisions that result in walls unable to support the lateral loads that are created when hydrostatic pressures develop, and poor construction techniques that fail to ensure stable backfill behind basement walls.

Solutions range from more proactive maintenance of the drainage systems to strengthening of walls. However there must first be more public awareness of the nature and causes of the problem, and its equity sapping effects. The opportunity for intervention presents itself when property transfers occur; sadly this is almost never considered by the home inspection or lender community.

Introduction

Claiming to have decoded the DNA of the leaking basement epidemic is probably overstating the case just a little. But in some respects the metaphor is entirely appropriate. It will be shown that the problem is complex and multifaceted, and that there are a handful of variables that are intertwined and can act together in various combinations to create the malady that is afflicting so many residences. If, at the most fundamental level, the role of DNA is the long term storage of information [1] then, hopefully, this paper can contribute to the collective knowledge of how the problem develops, and how it can be mitigated. The basic ideas were first published in 2006 [2] and this represents an update with some additional case histories, further explanation of the failure mechanism, and discussion of mitigation.

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The ‘Waterproofing Contractors’ section of many local Yellow Pages is a strong indicator of the scale of the leaking basement epidemic in the United States. In Cincinnati, Ohio, for example, there are 12 pages of them representing some 34 contractors who serve a population of about 2 million in the tri-state region [3]. What has happened to the housing stock that makes it necessary for the ‘Basement Doctor’, a Superman look-a-like and ‘Gotta Crack Call Jack’ to exist?

In attempting to answer this question, the paper draws primarily from a detailed case history where a home with seriously distressed basement walls was purchased for investment. Evidence that there is indeed an epidemic became obvious from screening a large number of additional investment prospects. From an engineering stand-point, the scale of the problem was puzzling and an effort was made to understand the causes and financial ramifications. Ideas have been developed on how to mitigate the problem and the huge financial burden that it represents.

Case History

Equity Erosion

The home is a colonial style, two-story, brick, single-family residence built in 1951. In May 2003 it was offered for sale at a price of $790,000, and shortly thereafter an offer was accepted contingent upon the usual inspections. Removal of some paneling in the basement revealed that the walls not only leaked, but that the west wall was seriously cracked. That deal fell through and the house was returned to the market at a price of $649,000. Foundation problems tend to have a chilling effect on property value, and the eventual purchase price was $555,000; close to the value of the 1.25 acre lot.

This may be an extreme case, but clearly a cracked basement wall is not just a nuisance, it is potentially a huge financial liability. The seller lost 30 percent (%) of the value of a premium property, and arguably 100% of the value of the structure. There was a large equity and the owner could absorb the loss, but if the purchase price had been below the remaining mortgage balance, default and foreclosure would have been likely. It should be clear then, that this is not just a problem for homeowners, but for mortgage lenders too.

Property Condition

The house is a two-story brick structure with a tile roof. Except for the garage and laundry room that have slab-on-grade foundations, there is a complete basement with 8-inch thick, non-reinforced concrete walls. The cracked west basement wall beneath the living room had bowed inwards 1-3/4 inches and the larger cracks were open about 3/8 inches. Leakage occurred along some of the cracks during and after rainfall.

Historically, previous owners had taken measures to mitigate leakage by installing an interior perimeter interception and drainage system along the westerly basement wall. These measures were reasonably effective at controlling water once it entered the basement. A remedial system in the non-paneled portion of the basement included the use of large fiber-glass sheets riveted to the wall that allowed water to run unobserved behind them into the sub-floor drainage system. A detrimental aspect of this system was that cracks could no longer be seen either, and so were ‘out of sight, out of mind’.

The performance history of the walls was pieced together roughly from available records of a) efforts made by previous owners to alleviate their water problem and b) periodic remodeling initiatives. A major remodeling appears to have taken place in about 1963 that included finishing a family recreation room in the portion of the basement beneath the living room. While not conclusive, it suggests that the owner was not overly concerned about the moisture conditions or the state of the west wall at that time. The work involved the attachment of 1 inch x 2 inch furring strips to the concrete walls and installation of rough finished 1/2 inch pine paneling. This remodeling is significant in that it undoubtedly prevented direct observation of the condition of the west wall in this part of the basement in the ensuing years.

In 1988, an effort was made to control water seepage through two cracks that had developed in the northern part of the west basement wall, beneath the dining room / first floor hall. This remediation consisted of a trench drain leading to a sump, and fiber glass panels attached to the wall that directed water from the cracks to the drain. The sump appears to have been designed to discharge into the drainage system provided for the roof drain down spouts. The floor drain portion of this system was still operational in 2003 and appears to have successfully controlled the flow of seepage water within this part of the basement.

In June 2000, a contractor proposed to conduct remedial work on the recreation room drainage system. A new catch basin and metal grate were constructed at the outside door to the room, and new discharge piping installed.

May 2003 saw further remedial drainage works related to the recreation room. A drain was installed on the outside of the west basement wall in an apparent effort to relieve hydrostatic pressure on the wall and redirect surface water flows. The wall was also reportedly waterproofed. Two cracks in the east wall of the furnace room were 'injected', although no performance warranty was offered. Significantly, both allowed water into the basement during and after periods of heavy rain on several occasions in 2003 and 2004. The remediation in 2003 was an apparently failed attempt to deal with the drainage issues in preparation for sale of the property.

In hind-sight, it might have been more cost effective to demolish this structure and rebuild a modern style house. However, it was a strikingly good looking and potentially functional home. This, together with the perceived environmental benefits of ‘recycling’ it argued in favor of renovation. Naturally, a key element of this was to stabilize the structure and deal with the water. Fortunately, the brick shell of the house was perfectly intact and beyond the basement walls themselves, cracked plaster was the only damage to the building envelope.

Stabilizing the Basement Walls and Eliminating the Leakage

A completely dry and structurally sound basement was essential for an upgraded family room that would be a valuable feature of the home. To accomplish this it was first necessary to answer some fundamental questions.

If the walls had performed satisfactorily for a decade or two after the house was completed, what happened to change things? Walls do not simply ‘wear out’ like a shingle roof. Concrete continues to gain strength (slightly) indefinitely and so should have improved over time rather than deteriorating. The only other variable is the loading condition. In addition to the weight of the house, the major load on the west basement wall is the pressure created by soil backfill acting laterally, together with hydrostatic pressure (again a lateral load) caused by water that happens to become dammed up against it. The pressure created by the backfill is unlikely to change significantly as long as the moisture conditions remain fairly constant. This leaves hydrostatic conditions. The key issue here is not whether hydrostatic pressure can create a critical load condition, but why did it increase over time?

The answers to this question appeared to be obvious with the first heavy rains when the gutters started to overflow, flooding the area adjacent to the house and above the basement wall. It was assumed that the gutters and/or down-spouts were blocked. Upon further investigation, it was found that the down-spouts were clear, but had a significant head of water in them. The down-spout drains were blocked. Further investigation of several of the offending pipes revealed that they were blocked solid, from end to end, with roots and decaying organic matter.

Another observation during times of rain was that surface water pooled for long periods of time in the front yard adjacent to the house. The lot slopes down from the street towards the house which tends to impede the natural drainage causing ponding at several locations along the west wall. It is hard to believe that the original builder paid so little attention to surface drainage. Even if it was marginal to start with, most likely it was functional. So an additional factor was needed to explain changes that could account for subtle grade modifications in the front yard near the house.

The answer was not obvious until some months later during installation of an in-ground utility sink in the laundry room. Having cut a rectangle out the concrete slab-on-grade it was obvious that the soils beneath it had subsided about 12 – 15 inches. This would have been backfill behind the adjacent basement wall (furnace room west wall) and the surface of the soil showed the clear imprint of the concrete slab that had been in direct contact at the time it was poured. With periodic, prolonged periods of saturation caused by overflowing gutters and ponding, it is reasonable to surmise that poorly compacted clay backfill had settled significantly over time. With only modest surface drainage grades to start with, it is easy to see how backfill-related subsidence adjacent to the house, could have resulted in enough grade changes to disrupt surface drainage and cause ponding.

The problem was now clearly defined and there were two actions that had to be taken to eliminated the source of the water, and help reduce the loads on the basement wall. First, all the down-spout drains had to be replaced, and second, the front yard near the house had to be regraded to provide proper drainage away from the structure. Regrading involved creating a swale the length of the house about 15 feet from it. All the front landscaping was sacrificed and had to be replaced, but the result was a remarkable and immediate improvement. Watching a torrent of water several inches deep in the swale during a storm drove home just how serious the stress on the building envelope had been when much of this had just dammed up against the house. While the regrading was in progress, new 6-inch diameter drains were installed and connected to the down spouts. Large diameter piping was selected after seeing the non-functional state of the original 4-inch lines. The pipes were laid to daylight in the side yard, and discharged onto a small rock apron protected by dry stone wing walls. This will allow future inspection and maintenance of the pipes, a feature that was not present previously when disposal of roof run-off relied on tile drains.

Three reasonably feasible options for strengthening the basement walls were considered:

Complete replacement
Soil anchors
Reinforcement
    a) steel
    b) externally bonded fiber reinforced polymer (FRP)

Bonded FRP was selected primarily because it is minimally disruptive (compared to total replacement), unobtrusive when complete (compared to most steel reinforcement and anchoring systems) and overall costs were reasonable. As a secondary factor, contractors that are familiar with this technology also tend to be knowledgeable about epoxy crack injection for both leak abatement and structural restoration. This was important for the other objective, the dry basement. All cracks were injected with epoxy to restore waterproofing. Several of the larger cracks were injected with structural epoxy to restore strength to the distressed concrete section.

The FRP product consisted of 6-inch wide kevlar/carbon-fiber woven ‘straps’ bonded vertically to the surface of the concrete with epoxy every four feet along the wall. This creates the equivalent of tensile reinforcement on the bowed face of the wall. Initially, straps were applied only to the recreation room west wall since it was in the worst condition. Subsequently, the decision was made to reinforce the whole of the west basement wall.

The installation contractor provided a lifetime warranty on the straps. However, the approach to design seemed simplistic and could most generously be described as ‘empirical’. Straps are always placed 4-foot on center. That is the design. To evaluate this further, structural analysis of the wall was conducted for both before and after remediation scenarios. The American Concrete Institute (ACI) design and construction guide ACI 440.2R-02 [4] provides information on the analysis of bonded FRP, and the British Cement Association's PC-based software package for the analysis of simple reinforced-concrete structures [5] was used for a parametric analysis of the bending stresses in the wall. The generalized loading conditions are shown in Fig. 1.

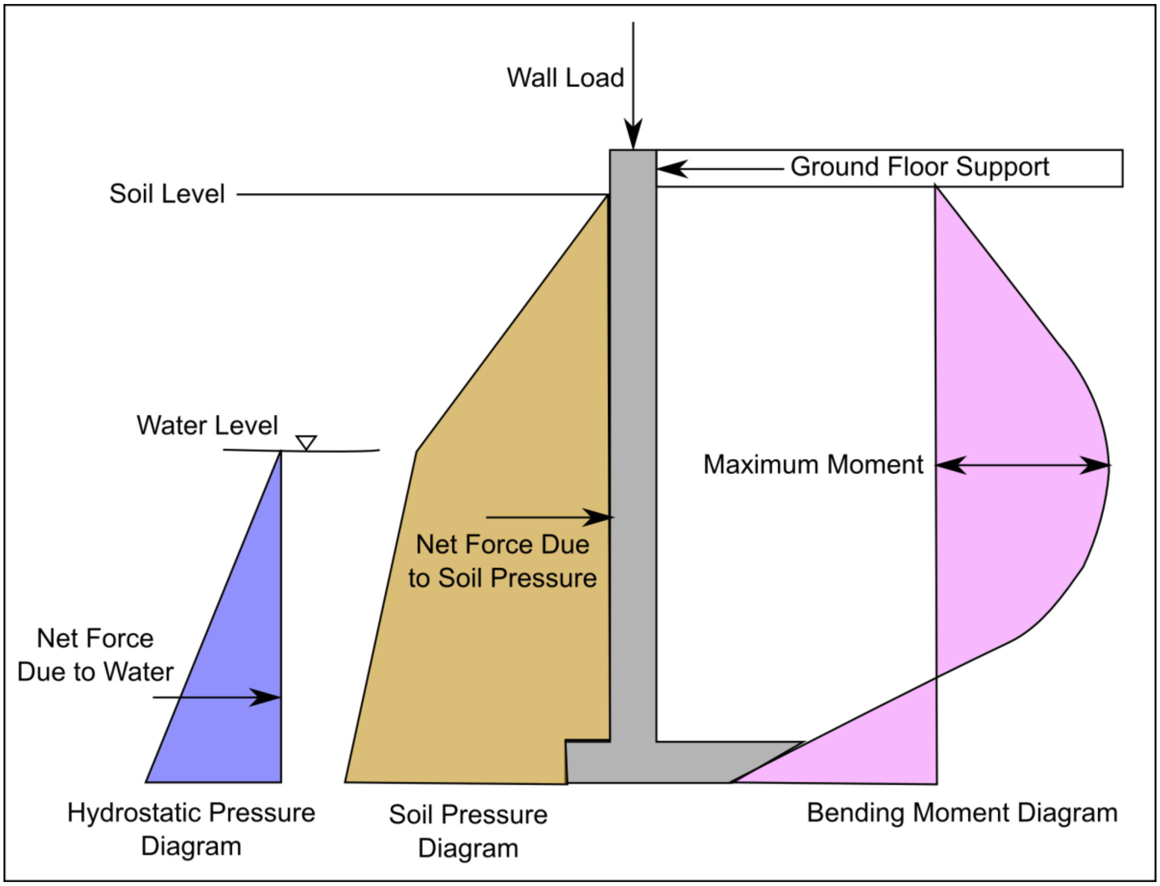

Fig. 1. Basement Wall Loading Conditions - schematic, not to scale

Field measurements of the geometry of the structure, laboratory strength testing of a concrete core recovered when forming a penetration for new HVAC duct-work, and estimates of soil properties based on field observation were input to the spread-sheet model. The results of interest are the maximum bending moment creating tension on the inside face of the wall, and the moment resistance of the concrete section.

Prediction of the lateral load imposed by fine grained backfill is challenging. On the one hand, the short term strength of clays can be quite high and, unlike sands, they often can be seen to stand vertically for extended periods in excavations. In this mode, the imposed lateral forces are nominal. However, if they subsequently become saturated - particularly for extended periods - consolidation will occur and high lateral forces are likely to develop. Poorly compacted clay backfill is particularly susceptible to this process as discussed above. For purpose of analysis, it was assumed that the 'stable' phase of the backfill history was past and that the material had become saturated. At this point the lateral soil pressure was assumed to be close to the 'at rest' condition, there having been little or no movement of the concrete wall. Subsequently, as the wall cracked and deflected, the pressures were more likely to be 'active', and the hydraulic loading moderated by the presence of cracks that allowed dissipation of water pressure into the structure.

The retained soil is silty clay and, in the absence of strength test results, a range of possible friction angles were examined. The graph below (Fig. 2) shows the maximum moment in the wall for soil friction angles between 20 and 40 degrees and a range of ground water levels (hw) from full depth (water at the ground surface) to no water. The soil at the site is assumed to have a friction angle towards the bottom of the range; say 20 – 30º. The 30º - 40º range is shown for comparison and would be representative of a granular backfill.

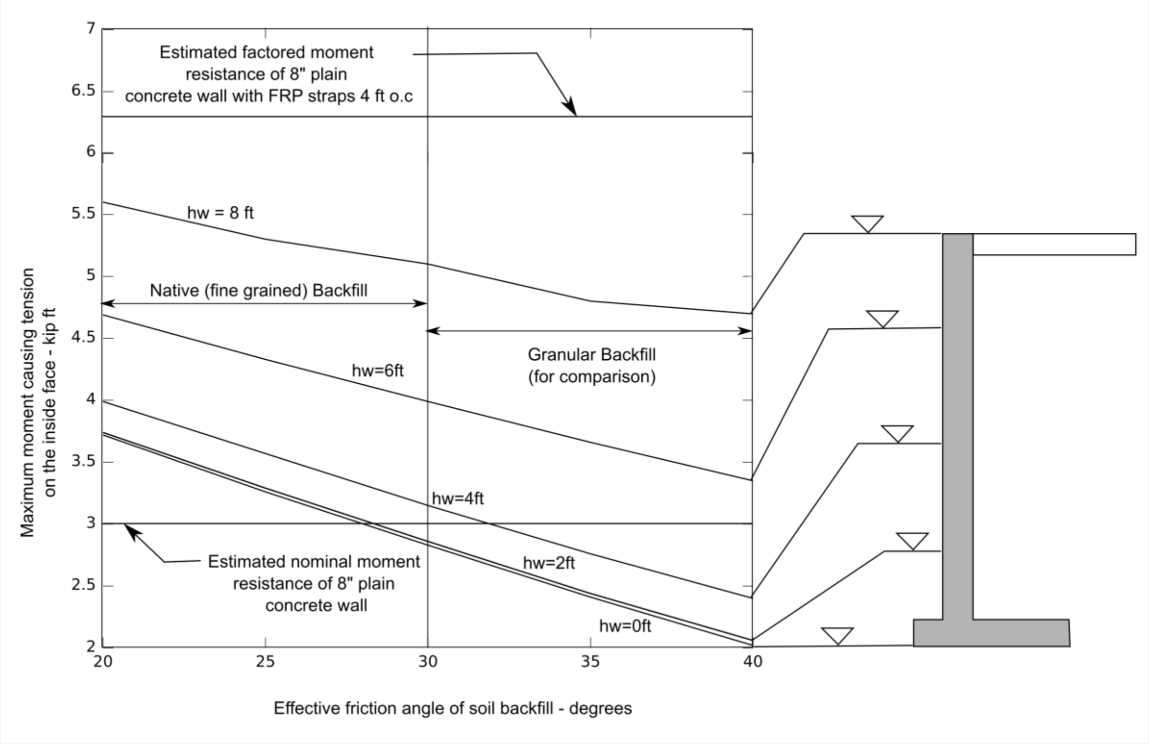

Fig. 2. Bending Moments and Structure Resistance

The impact of a rising water table is clear. At low water levels, (up to about 50% of the height of the wall) the presence of the water does not significantly increase the maximum moment. However, as the water level rises beyond this, it increases quite dramatically. For a 25-degree friction angle soil, the initial ‘dry conditions’ moment (3.2 kip ft) increases by about 10% for the first 4 feet of the water level increase and by ~70% to 5.4 kip ft for the next 4 feet of rise.

The calculated moment resistance of the wall once the straps have been applied is 6.3 kip ft. This is computed using material factors for the concrete and FRP of 1.5 and 1.17 respectively, and exceeds the maximum nominal moment that would be anticipated under extreme hydrostatic loading conditions. If the lateral load is factored using 1.35 for horizontal earth pressure (at rest)[8], the factored resistance exceeds the factored load up to the point where the water table is within 2 ft of the ground surface.

Based on the loads imposed by fine grained soils, the moment resistance of the wall prior to cracking (assuming nominal water pressure and 'at rest' soil pressure) must have been between 3 - 4 kip ft. The calculated nominal moment of resistance of a plain concrete wall 8-inches in thickness is on the order of 3.0 kip ft. This is based on the commonly assumed value for the tensile strength of concrete (10% of compressive strength) using the compressive strength results from a single test. The analysis of the wall as a two-dimensional 'strip' of the overall structure may be conservative because it ignores three-dimensional support effects.

The net result of the remedial work was that the basement remained completely dry and stable during and after significant rainfall events thereafter.

Costs

As a residential remedial project this was an expensive repair, although compared to the loss of equity that the former owner suffered, it is almost trivial. The costs are typical of those that must be incurred (scaled as appropriate for the size of the property) to properly stabilize a structure when the basement has reached an advanced state of disrepair.

Carbon fiber/Kevlar straps	$8,000	
Crack injection (structural and waterproofing combined)	$6,000	
Regrading and downspout drain replacement, reseeding	$5,000	
Landscaping and walkway replacement	$5,000	
Total (2004)		$24,000

The portion of the basement wall that was strengthened measured about 100 linear feet and cost about $80/foot. About 170 linear feet of cracks were injected at $35/ foot of crack (equivalent to about $60/linear foot of wall). The total concrete repair costs were, therefore, on the order of $140 per linear foot of wall. Regrading required the creation of 90 feet of swale, about the same amount of buried 6" diameter pipe and two head walls. The affected area that was hydro-seeded was on the order of 2,000-2,500 square feet.

Discussion

Each element of this remedial program could be viewed separately: the strengthening of the wall, the leak-proofing of the wall, the provision of new downspout leader drains and the modification of the surface grades to promote drainage. Indeed, all are quite independent activities and this is generally how they are sold by specialty contractors. But this misses the bigger point and, perhaps, provides a clue as to why the leaking basement problem is widespread. At its root was a systemic failure, the seeds of which were planted before the builder left the job site in 1951. It involved every phase in the life cycle of the building from conception to (very nearly) its destruction.

First was a failure of the design process and, more importantly, of that type of design that relied on simple building code 'engineering'. The wall should have been designed to withstand much higher lateral loads. Granular backfill and a drain at the heel of the wall would have been helpful too as the above graph shows. However, in 1951 all the code required was that a non-reinforced concrete wall be at least 8 inches thick [6]. In defense of the code it must be said that this specification was followed by a statement putting responsibility squarely back on the designer’s shoulders:

> “When the stresses due to earth pressure, surcharges and superimposed loads exceed the maximum working stress permitted by this Code for the material used, and the additional stresses are not otherwise provided for, the wall thickness shall be increased to bring the stresses within the required limits.”

A little reinforcing steel might have been more useful; but at the least some extra concrete would have helped. The International Residential Code [7] would, today, require #6 bars 24 inches on center in this wall or a 12-inch thick plain concrete wall.

Severe subsidence of the backfill around the basement was a construction failure attributable to the builder. This outcome was reasonably predictable and suggest that high levels of construction quality control (by today’s standards) are not likely to have been employed during this era of home construction.

Tile drains for disposal of roof run-off are bound to fail eventually. Even with the relatively crude early leaf guards on the gutters (frequently wire mesh), some solid matter is washed into them. This will eventually clog the drain and it will cease to function as anything more than a small reservoir. Note that there is a crop of modern leaf guard designs that are extremely effective in preventing the ingress of solids to the drainage system, and these are highly recommended. Drains that discharge to the surface stand a better chance of long term survival, but it is not unusual for their discharge points to become blocked with debris and for roots to invade them. This speaks to a failure of maintenance. Once the drain fails, the roof run-off can only overflow the gutter, discharging on top of the poorly compacted basement backfill below.

And finally, there are some institutional failures that contributed to the problem. When the renovated house was sold, the new owner commissioned a detailed inspection. The inspector was, overall, very thorough. However the section dealing with the drains was painfully weak:

> “Gutters: Aluminum; Downspouts: Aluminum; Leader/Extension: Storm water runoff from the downspouts is to the drain tile. The underground storm drains were not inspected as part of this evaluation.”

He had been told that the drains had been replaced and was shown the new surface discharge points, but that is not really the point. In his summing up where he suggested a multitude of additional inspections (termites, radon etc) nothing more was said about the leaders/extensions that were not (by his own admission) inspected.

Is this Really an Epidemic?

In the introduction, it was suggested that the number of basement waterproofing contractors was a strong indication that there is a leaky basement epidemic. A lot more study could be done to decide what constitutes an epidemic and whether this is one or not. It is interesting to note that on the street where the case history is located at least two other houses had basement leakage problems out of a total of eight; almost 40%. Qualitatively the writer has observed that about 50% of open houses visited have or have had leaking basements, or show signs that they soon will. On one street in a community east of Cincinnati, there were four adjacent houses that, in 2006, were either for sale or being renovated. All seem to be infected.

House prices in that community had been rising at about 3.5% per year for the previous few years, so while it is not exactly San Diego Ca., it was not in terminal decline either. It tends to be a place where first time buyers can find a small house for less than $100,000 (median home price in this part of the community was $95,000 in 2005) that is comfortable and close to the city. The four houses in question were built between 1946 and 1953 and are located adjacent to undeveloped woodland on the lower slopes of a hill. The following table shows the recent changes in value that were triggered, at least in part, by deterioration of a basement wall.

Table 1. Equity Change in Properties with Distressed Basements

Street Number	County Valuation ($)	Sale Price (1) ($)	Sale Price (2) ($)	Change in Equity (%)
6926	88,400 (2002)	65,000 (1993)	34,700 (2006)	-61
6924	83,000 (2005)	40,000 (2006)	83,000 (2006)	0
6920	82,800 (2002)	54,000 (2004)	18,400 (2010)	-78
6916	86,500 (2005)	54,000 (2005)	93,500 (2006)	8

Three houses were for sale in 2006 and all had distressed basements. Based on exterior symptoms it appears that the forth one does too. Distressed here is defined as meaning that there is horizontal cracking at the mid point of the concrete block basement walls on the ‘up-hill’ sides of the property, and evidence of leakage. Damage to the above ground portion of the structures varies from minor to serious with significant cracking of the brick around door and window penetrations at 6920. Ironically, that is the house where the owner expected the highest price, and where in 2010 the property changed hands for what is essentially the land value.

The houses were in various states of repair: considerable remodeling work had been done at 6920 while 6926 was in need of a total rehab. A common factor is that (in 2006) in at least three of the four cases there had been a foreclosure within the past two years [9]. It seems likely that in these cases, the cost of repairing the structures may have contributed to the owner’s decision to default on their mortgage and walk away. Using the prices developed above, and assuming that two walls of the typically 38 x 23 foot basements need to be stabilized, an owner would be looking at an expense of more than $8,000.

Replacement of downspout drains and improving surface drainage could easily add another $1,000 - $2,000. This is a large sum for a low or moderate-income family to be faced with for repairs and it is easy to see why abandonment could be seen as the better option. Even if an owner elects to sell the house, the condition of the basement is unlikely to escape an appraiser's eye, and any chance of financing is diminished until repairs are completed. The financial result is probably the same – foreclosure. In the present real estate market the climate is significantly worse than in 2006 and even a small defect can be enough to deter an otherwise willing buyer.

In a recent (2011) evaluation of a property on behalf of a prospective buyer, a retaining wall forming one wall of the garage had clearly become slightly tilted and was pushing the double overhead door framework out of alignment. There was a classical hair-line mid-height horizontal crack with evidence of (minor) seepage along the retaining wall. Deteriorated surface drainage patterns appear to have contributed to the problem and stabilization measures were estimated to cost on the order of $10,000. After attempting to negotiate a lower price, the buyer abandoned the transaction and purchased another property. In this case the owner lost a sale and most likely had to discount the property to the next prospective buyer since she now had knowledge of the problem.

This is, admittedly, a small sample. However, the following statistics are striking. The median house age in the Cincinnati MSA was 36 years in 1998 (the most recent data) [10]. This probably means that it is nearly 50 years now and one can be sure that of the more than 400,000 single family units in the MSA, the 200,000 down-spout drainage systems that are now older than the median are not improving with age. Many in the 20 – 40 year age bracket may also be starting to fail. So if this is not an epidemic, it will be soon.

Solutions

Once a basement wall has failed, nothing short of a major remedial project will cure the situation. The emphasis must therefore be on prevention. Since one of the most serious culprits seems to be the downspout drains and their progressive deterioration, this is where most attention needs to be directed. It will always be difficult to convince people to replace blocked drains promptly, but there is one point at which outside influence can be brought to bear: during a property transaction. The lender has the option of making the loan or not and it should be contingent on a more rigorous inspection regime when it comes to this issue.

This raises the question of what an appropriate inspection regime would look like. Here, the engineering profession could usefully apply some thought and develop ideas on how to assess the condition of a roof water disposal system. Perhaps a simple hydraulic load test or the equivalent of a 'percolation test', such as is used to develop design parameters for on-site sewage disposal systems, could be developed. Attempting to fill the downspout with water from a hose is a fairly simple test; if it can be filled, it has probably failed.

Or perhaps it should always be assumed that the drains have failed if the house is more than 20, 30, or 40 years old, and should be replaced. This puts the cost burden on the current owner who, after all, is the party that ‘enjoyed’ the drains while they were still functioning. Assuming that they could be replaced for an average cost of $2,500, this would seem to be a prudent expenditure in light of the cost of repairing the basement – or worse yet, loosing the value of the house entirely.

Another approach would be to retrofit walls that were designed using now obsolete standards to more modern specifications. Externally bonded FRP is an excellent choice for this and can add substantial bending stress resistance to non-reinforced walls. FRP and epoxy injection combine to form a powerful tool for remediation once damage has been done to a basement. Development of this technology should continue with a view to reducing the cost and refining the design process.

The design life of tile drain systems that are still functioning may be significantly lengthened through the use of modern high efficiency gutter guards that prevent most solids from entering the system. These are not inexpensive but can be highly effective.

Particular care should be taken to maintain surface drainage away from the house so that accumulations of water adjacent to the basement walls are not encouraged. Wherever gutters have overflowed, the soil beneath them may have become saturated and consolidated. In addition to a new increment of lateral load induced stress caused by the soil, this has created an accumulation point for water that also allows the development of high hydrostatic loads.

Conclusions

Evidence presented in this paper suggests that there is indeed an epidemic of leaking basements. Because of the vast number of older houses it is fair to say that it is far from over. Most often the failures are systemic involving multiple factors. Very significant among these is the role of the downspout drain, its frequent deterioration over time and consequent loss of functionality. Combined with this, poor design decisions have resulted in a large number of basement walls that are unable to support the kind of lateral loads that can be expected if high hydrostatic pressures develop following downspout leader drain failure.

These two factors combine with predictable results and often cause a significant loss of property value. This may be so serious that it can influence the decision of an owner to abandon the home so creating liability for lending institutions.

There are solutions to the problem that range from proactive maintenance to strengthening of the walls. However, there must first be more public awareness of the causes and liabilities involved. This can best be accomplished through more focused attention during the inspection and appraisal process when property transfers occur.

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Repair of Ground Movement in a Filled Slope in an Abandoned Quarry

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Abstract: Cleveland Metroparks Euclid Creek Reservation was conceived in 1941 and built on both sides of Euclid Creek in the City of South Euclid, Ohio. Euclid Creek Parkway was built on the west side of Euclid Creek. It is a heavily used road that connects Highland Road to Green Road. A hiking trail was constructed just east of Euclid Creek Parkway and is extensively used. The south end of Euclid Creek Parkway was constructed over a 50 ft. deep sandstone quarry. Some time before April 1997 ground movement had occurred, along a 225 feet long section, just east of Euclid Creek Parkway, in the area of the former quarry. The hiking trail was damaged and the east edge of Euclid Creek Parkway was exposed to future undermining. The subsurface investigation in the slide area took into account the uncertainty in the expected conditions and required special measures to confirm that a geogrid reinforced slope was feasible, and could in fact be reasonably completed given the actual ground conditions. The design of the geogrid reinforced slope allowed sufficient flexibility to accommodate variations in the actual ground conditions. The work was completed in 1998 and has resulted in an economical and successful repair.

Introduction

Some time prior to April 1997 ground movement affected about a 225 feet long section on the east side of Euclid Creek Parkway. The hiking trail located on the east side of the Parkway was damaged. Fig. 1 shows the north end of the ground movement as it appeared on May 22, 1997. Cleveland Metroparks wished to reconstruct the hiking trail for the benefit of the park users. Because of the difficult ground conditions, the slide was located in the area of a former stone quarry, Cleveland Metroparks asked for an initial evaluation of the feasibility of four ways to reconstruct the slope. They were, relocating the hiking trail from the east side of Euclid Creek Parkway to the west side, reconstruct the slope from the bottom up using existing materials supplemented with new materials as needed, constructing a bridge to support the hiking trail and reconstructing the slope using geogrid reinforcement. Relocating the hiking trail to the west side of Euclid Creek Parkway, was not an attractive option. Fig. 2 shows the ground conditions on the west side of Euclid Creek Parkway, in the area of the ground movement. It shows that the ground slopes up steeply and rock is present near the surface. There was limited room available on the west side of the Parkway. Rock excavation, allowing for appropriate support of adjacent properties and the safety issues raised by asking hikers to cross a busy street made relocating the hiking trail an undesirable choice. Reconstructing the slope using existing materials and supplementing them with new materials, as needed, could be done. The volume of earth work needed for this approach would be substantially larger

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Fig. 1. North end of the ground movement on May 22, 1997

Fig. 2. Ground conditions on the west side of the Parkway in the slide area

than the volume of earth work needed for a geogrid reinforced slope, resulting in substantially greater cost. The hiking trail could be supported on a bridge, but that would not provide any support for the Parkway, which could later be undermined. The feasibility of repairing the damage using a geogrid reinforced slope, therefore, became the primary focus of the work.

The Subsurface Investigation

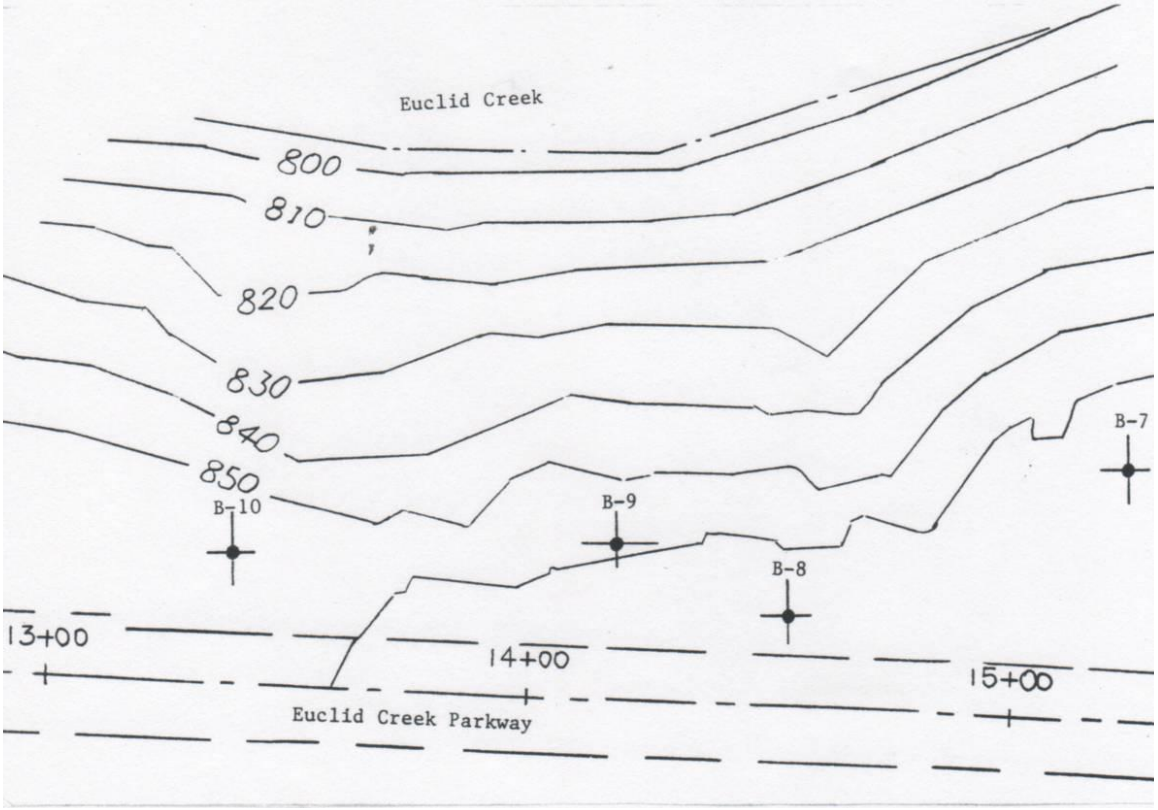
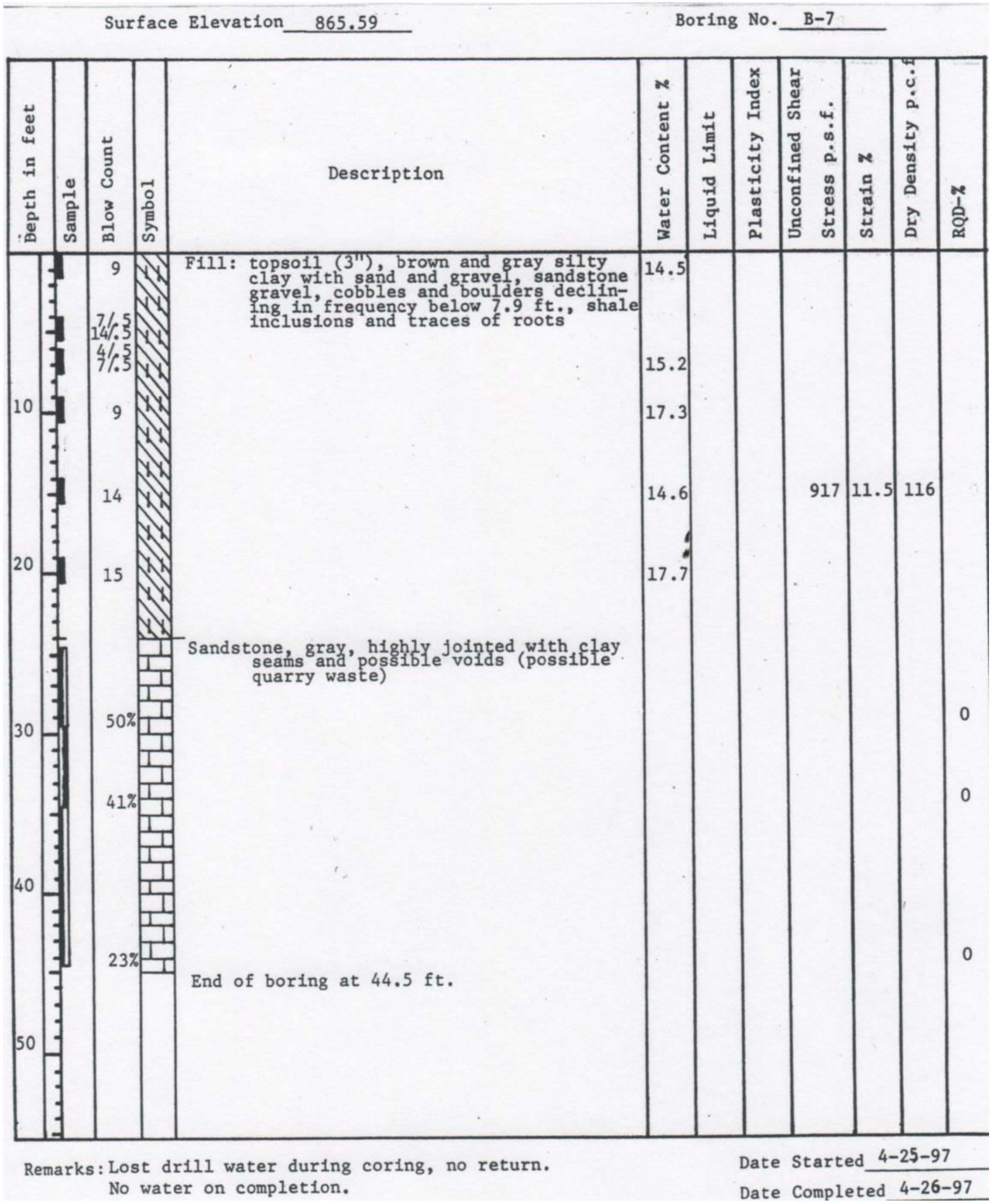


Fig. 3. Locations of borings

Fig. 3 shows the locations of four borings, B-7, B-8, B-9 and B-10 that were drilled to understand the ground conditions at and near the slide. Borings B-8 and B-10 were drilled just west of the edge of the ground movement. They encountered intact rock, sandstone and shale, at elevations 848.2 and 852.78 respectively. The upper 5 feet of fill at both locations appeared to be material placed to facilitate construction of the park. The fill below the upper 5 feet at boring B-8 appeared to consist of quarry waste. Boring B-9 was drilled by hand in the area of the ground movement. Quarry waste fill was encountered to the shallow depth that could be penetrated. Boring B-7 was drilled south of the area of ground movement to get some understanding of the ground conditions near the eastern edges of the ground movement. Quarry waste fill was encountered to elevation 821.59 in boring B-7 at which depth drilling was stopped. The borings did not



Surface Elevation 865.59

Boring No. B-7

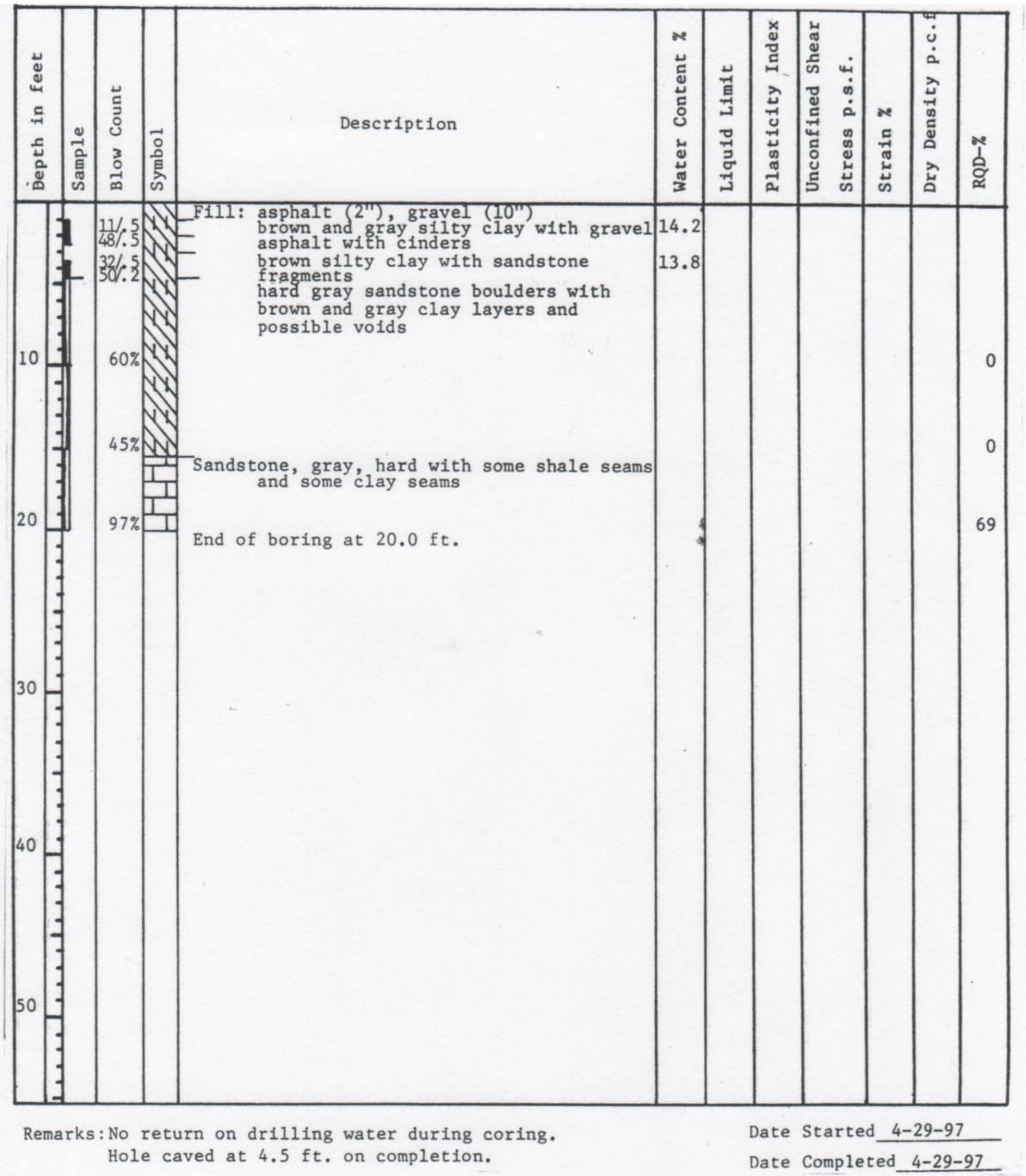
Depth in feet	Sample	Blow Count	Symbol	Description	Water Content %	Liquid Limit	Plasticity Index	Unconfined Shear Stress p.s.f.	Strain %	Dry Density p.c.f	RQD-%
		9		Fill: topsoil (3"), brown and gray silty clay with sand and gravel, sandstone gravel, cobbles and boulders declining in frequency below 7.9 ft., shale inclusions and traces of roots	14.5						
		7/.5 14/.5 4/.5 7/.5			15.2						
10		9			17.3						
		14			14.6			917	11.5	116	
20		15			17.7						
				Sandstone, gray, highly jointed with clay seams and possible voids (possible quarry waste)							
30		50%									0
		41%									0
40											
		23%									0
				End of boring at 44.5 ft.							
50											

Remarks: Lost drill water during coring, no return.
No water on completion.

Date Started 4-25-97
Date Completed 4-26-97

Fig. 4. Laboratory log of boring B-7

Surface Elevation 863.70 Boring No. B-8

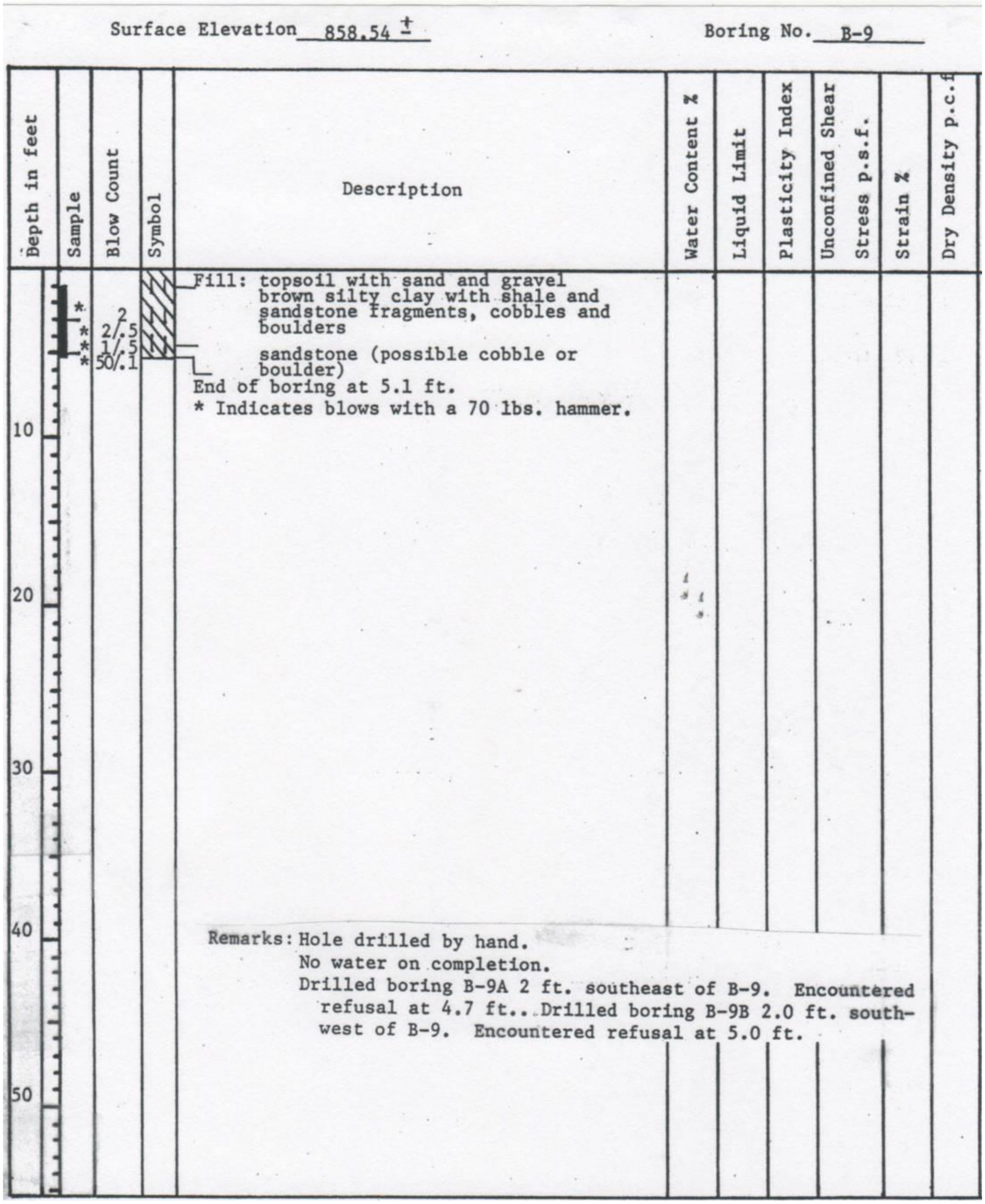


Depth in feet	Sample	Blow Count	Symbol	Description	Water Content %	Liquid Limit	Plasticity Index	Unconfined Shear Stress p.s.f.	Strain %	Dry Density p.c.f	RQD-%
		11/.5		Fill: asphalt (2"), gravel (10")							
		48/.5		brown and gray silty clay with gravel	14.2						
				asphalt with cinders							
		32/.5		brown silty clay with sandstone fragments	13.8						
		50/.2		hard gray sandstone boulders with brown and gray clay layers and possible voids							
10		60%									0
		45%									0
				Sandstone, gray, hard with some shale seams and some clay seams							
20		97%									69
				End of boring at 20.0 ft.							
30											
40											
50											

Remarks: No return on drilling water during coring.
Hole caved at 4.5 ft. on completion.

Date Started 4-29-97
Date Completed 4-29-97

Fig. 5. Laboratory log of boring B-8



Surface Elevation 858.54 ±

Boring No. B-9

Depth in feet	Sample	Blow Count	Symbol	Description	Water Content %	Liquid Limit	Plasticity Index	Unconfined Shear Stress p.s.f.	Strain %	Dry Density p.c.f
	*	2		Fill: topsoil with sand and gravel brown silty clay with shale and sandstone fragments, cobbles and boulders						
	*	2/.5								
	*	1/.5		sandstone (possible cobble or boulder)						
	*	50/.1								

End of boring at 5.1 ft.

* Indicates blows with a 70 lbs. hammer.

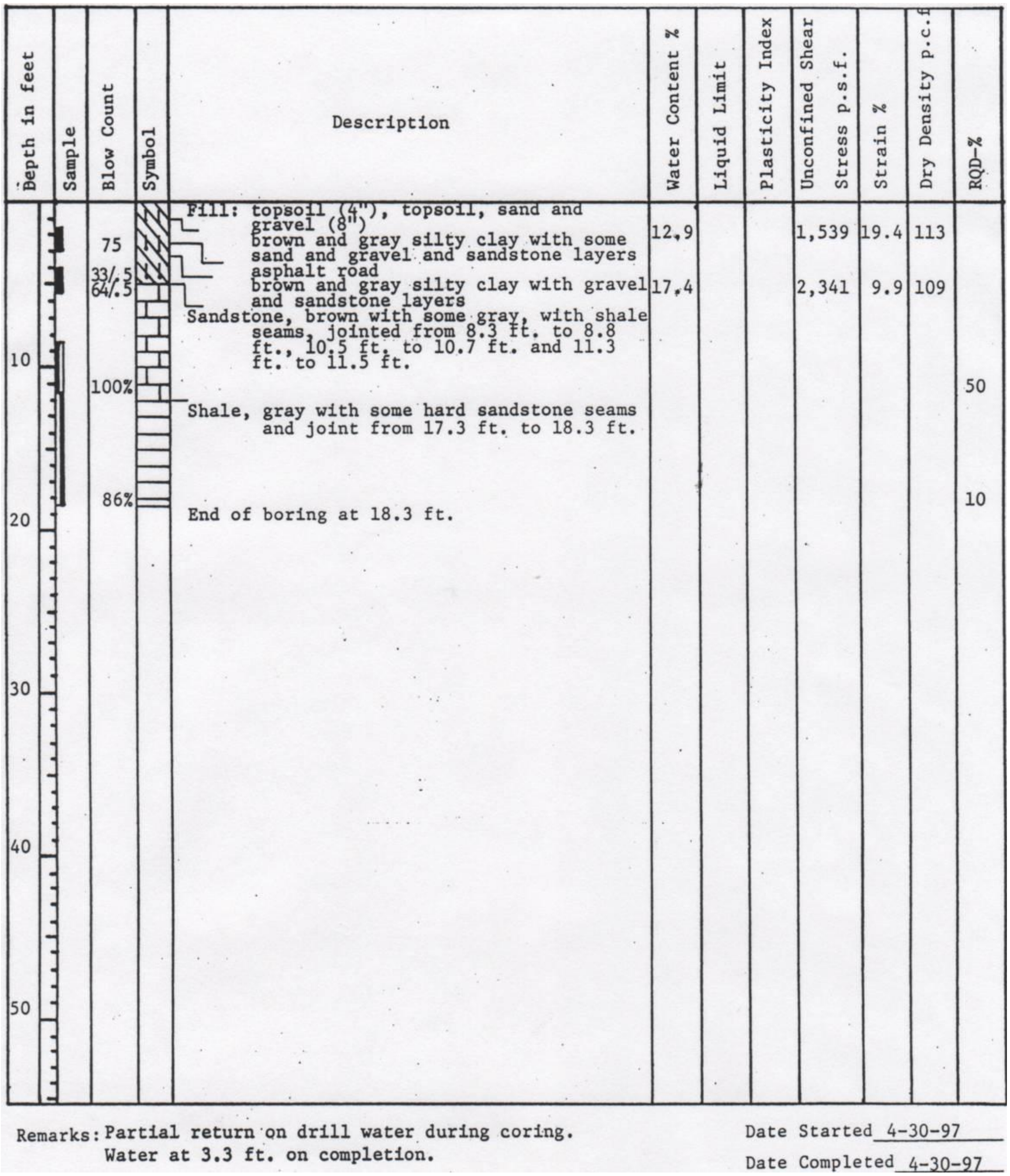
Remarks: Hole drilled by hand.
No water on completion.
Drilled boring B-9A 2 ft. southeast of B-9. Encountered refusal at 4.7 ft.. Drilled boring B-9B 2.0 ft. southwest of B-9. Encountered refusal at 5.0 ft.

Date Started 5-2-97

Date Completed 5-2-97

Fig. 6. Laboratory log boring B-9

Surface Elevation 857.78 Boring No. B-10



Depth in feet	Sample	Blow Count	Symbol	Description	Water Content %	Liquid Limit	Plasticity Index	Unconfined Shear Stress p.s.f.	Strain %	Dry Density p.c.f	RQD-%
				Fill: topsoil (4"), topsoil, sand and gravel (8")							
		75		brown and gray silty clay with some sand and gravel and sandstone layers	12.9			1,539	19.4	113	
				asphalt road							
		33/.5 64/.5		brown and gray silty clay with gravel and sandstone layers	17.4			2,341	9.9	109	
				Sandstone, brown with some gray, with shale seams, jointed from 8.3 ft. to 8.8 ft., 10.5 ft. to 10.7 ft. and 11.3 ft. to 11.5 ft.							
10		100%									50
				Shale, gray with some hard sandstone seams and joint from 17.3 ft. to 18.3 ft.							
		86%									10
20				End of boring at 18.3 ft.							
30											
40											
50											

Remarks: Partial return on drill water during coring.
Water at 3.3 ft. on completion.

Date Started 4-30-97
Date Completed 4-30-97

Fig. 7. Laboratory log of boring B-10

provide enough information on the ground conditions that would be encountered near the toe of a reconstructed slope. Four test pits were excavated with a telescoping back hoe to shed some light on this. A test pit excavated near boring B-8, very roughly 32 feet from the center-line of the road, appeared to encounter a ledge of rock very roughly about 15 feet below the road level. A test pit excavated near boring B-9 very roughly 35 feet from the center-line of the road, encountered rock more or less 15 feet below the road level. The conditions in these test pits were such that encountering of rock had to be determined partly by feel. It was, therefore, not possible to establish with certainty that rock had been encountered. A test pit excavated between borings B-9 and B-10 revealed a rock slope under shallow overburden. The rock appeared to slope down very roughly at a grade of 1 vertical to 1.5 horizontal. The area north of approximate station 13 +75 appeared to be outside the former quarry. The laboratory logs of borings B-7 through B-10 are included as Figs. 4 through 7.

The Design of the Reinforced Slope

The geogrid reinforced slope was designed using the criteria provided in Tensar Technologies, Inc.'s technical note TTN:SR1, dated February 1990. The Tensar criteria are based on the assumption that the top of the slope is horizontal, but the top of the slope being repaired is not. Because of the actual ground conditions, sound rock is present close to the face of the repaired slope, it was decided that it was acceptable to do so. In addition, a conservative approach was adopted by using the design length needed for the maximum height of slope, for the entire repair. A grade of 0.5 horizontal to 1.0 vertical was selected for the repaired slope. The Tensar Technologies Inc. recommendation of wrap around construction and a maximum spacing of 2 feet layers were adopted. ODOT #58 gravel was selected as the material for the repair. A factor of safety of 1.5 was used to select the length of the reinforced slope. The survey revealed a maximum height of reinforced slope of 15 feet. An equivalent surcharge load of 2 feet was used for the design. This resulted in a required length of 10 feet for the geogrid reinforced slope. Fig. 8 shows a schematic representation of the construction of the geogrid reinforced slope.

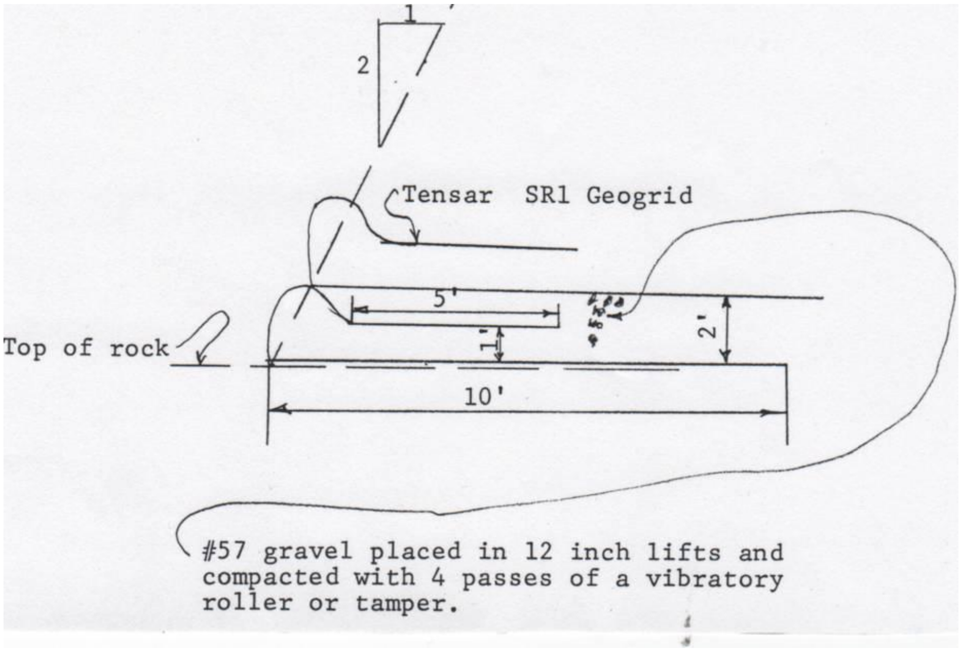
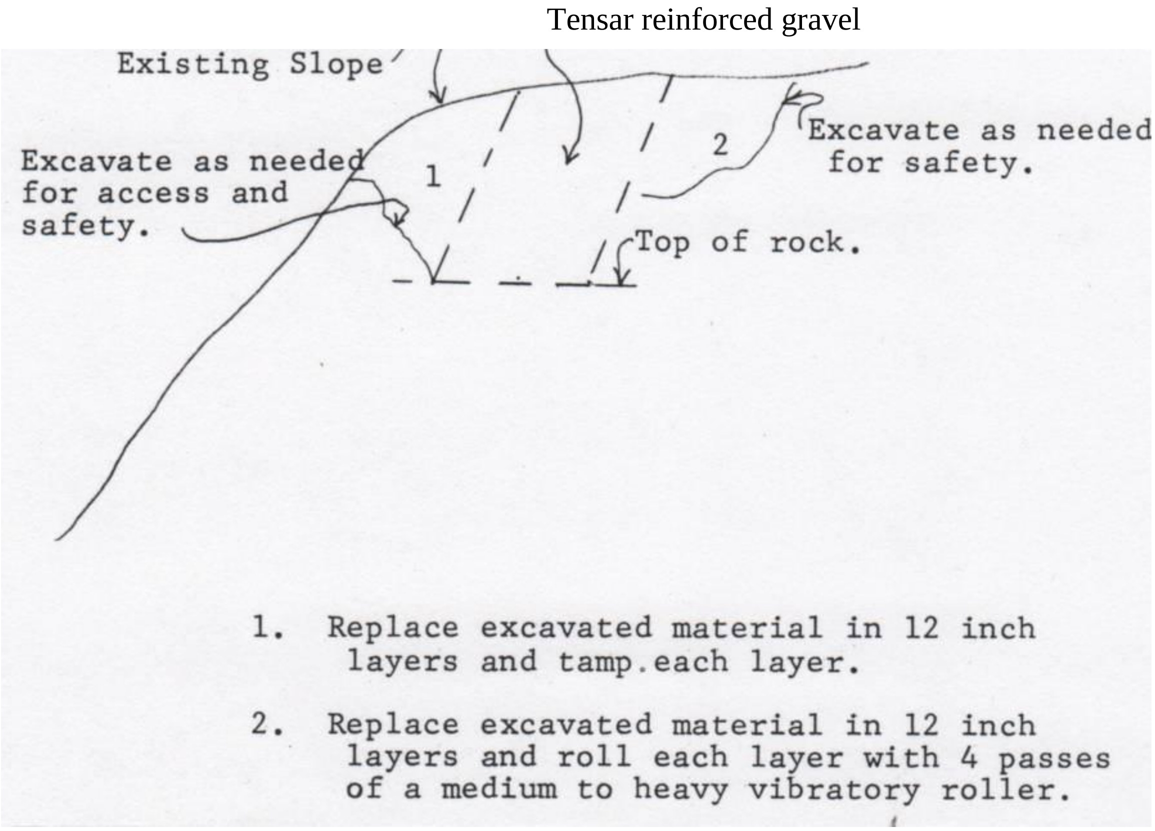


Fig. 8. Schematic construction procedure for the geogrid reinforced slope



1. Replace excavated material in 12 inch layers and tamp.each layer.
2. Replace excavated material in 12 inch layers and roll each layer with 4 passes of a medium to heavy vibratory roller.

Fig. 9. Schematic representation of required construction procedure

The design specified the construction procedure to be followed by the contractor. Fig. 9 above is a schematic representation of the construction procedure that was to be followed. The construction of the geogrid reinforced slope was to begin at the lowest level and proceed upwards. The geogrid reinforced slope was constructed in layers as shown in Fig. 8. As the reinforced geogrid slope progressed upwards the excavated material on the downhill side was to be compacted back in place. As long as this material remains in place, the geogrid reinforced slope will not be fully loaded.

Figs. 8 and 9 show idealized ground conditions that are unlikely to be encountered in reality. Allowance was made in the design to create an appropriate bearing surface for the start of the geogrid reinforced slope by placing concrete when needed. The following notes were added to the drawings:

1. Begin work at the north end and proceed to the south end till the work is completed.

2. Elevations of the rock surface are approximate. Record the actual elevation of the rock surface and notify the engineer of any changes from the expected rock levels.

3. The ground at the north end of the work area is expected to be a rock slope which may have been filled over for the construction of the road. The ground at the south end of the work area is expected to have been part of a quarry which was filled to

construct Euclid Creek Parkway. Notify the geotechnical engineer when the rock surface is exposed for observation. Do not begin constructing the geogrid reinforced slope until the rock surface has been observed by the geotechnical engineer.

4. It is likely that the actual ground conditions will vary from the expected ground conditions. An effort has been made to accommodate these changes without significantly delaying or disrupting the operations of the contractor. The construction procedures should be designed to accommodate the expected changes.

Construction of the Repair

Fig. 10. Excavation at the north end of the slope repair

Excavation for the toe of the repaired slope was started at the north end on September 10, 1998. Fig. 10 shows the conditions that existed at the start of the excavation. The excavation for the toe of the repaired slope was completed in shale at the south end on September 14, 1998. Elevations of the level of the rock were obtained at 9 to 13 feet intervals and information was prepared for the contractor to place concrete to allow the repair to begin at the appropriate level. Fig. 11 below shows a block of concrete at the north end of the repair to avoid extensive rock excavation.

Fig. 11. Block of concrete placed at the north end to avoid rock excavation

Fig. 12 shows the southward extension of the concrete foundation for the geogrid reinforced retaining wall. The formwork used was simple. For small drop heights concrete was placed directly from a truck. For deeper levels the bucket of a backhoe was used to place the concrete. Fig. 13 12 shows the form work for a concrete block foundation in an area of sloping rock. Fig. 14 shows the extension of the concrete block foundation in the area of sloping rock. Fig. 15 shows a longitudinal view of the concrete block foundation for the support of the geogrid reinforced retaining wall.

Fig. 12. Southward extension of the concrete foundation for the geogrid wall

Fig. 13. Form work in area of sloping rock

Fig. 14. Extension of concrete block foundation in area of sloping rock

Fig. 15. Concrete block foundation looking south to north

Fig. 16 below shows the start of the construction of the geogrid reinforced retaining wall. Fig. 17 below shows a view of the construction of the geogrid reinforced wall at mid-height. The backhoe can be seen replacing the previously excavated downhill slope.

Fig. 16. The start of the geogrid reinforced wall

Fig. 17. The geogrid reinforced wall at mid height

Fig. 18 below shows a view of the geogrid reinforced wall looking south to north. The wall is wider to the south because the slope of the road rises from north to south. Fig. 19 below shows the completed slope.

Fig. 18. The geogrid reinforced slope looking south to north

Fig. 19. The completed repair

The Performance of the Repair

Fig. 20. The hiking trail on July 17, 2011

Reference

Tensar Earth Technologies, Inc. (1990) Slope Reinforcement With Tensar Geogrids Design And Construction Guideline, TTN:SR1

Risk Considerations for Geotechnical Construction

George K. Burke, P.E., D.G.E.[1]

Abstract: All geotechnical engineers understand that current investigation techniques and budgets offer only a very small amount of information from which one must interpret to great distances in all three dimensions. This interpretation is usually based on an understanding of local ground experience and geology combined with the investigations which usually improves the result.

This paper will focus on how this interpretation affects geotechnical construction, and offers some points worthy of consideration for all parties concerned. Evaluating risk is a complex issue and differs depending on the construction considered and the potential affects and impacts. Construction risks are multifaceted and include schedule, production, materials, and other time-related costs, but this paper deals primarily with "technical" risks. However, it should be understood that it is generally not possible to separate risks as they are not independent elements.

Introduction

Risks in geotechnical construction include all the risks of general construction, plus the risks associated with the unknown. In 1995 at a Hayward Baker internal conference, Dr. Elio D'Appolonia presented the differences between the knowns, the known-unknowns, and the unknowns. His point was that shared experience is a risk management tool in our profession to minimize the unknowns and identify the known-unknowns. By "shared experience" Dr. D'Appolonia meant collected project information and outcome…today's Enterprise Resource Planning systems!

Risk today is managed implicitly from our experience (Lane, 2003). The challenge is the effective communication of this experience; a difficult thing to accomplish.

Today, construction methods are much faster than ever, and require fewer people to perform the work (Trenter, 2003). This consequently means there is less time to react to hazards that are revealed, and fewer people to identify and mitigate them.

I agree with all of these previous industry experts. So, why is it that managing ground conditions pose such a challenge for designers? These reasons were noted (Clayton, 2001) in the article "Managing geotechnical risk: time for change":

- Geologic setting and history…how did it become as it is?
- The properties and distribution of the ground and groundwater beneath a construction site are pre-existing, and largely out of our control.

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- Ground and groundwater conditions can be highly variable, from place to place, and with depth.
- Construction in the ground is normally carried out at the start of any project, so that delays at this stage will affect the latter stages of construction.

So to summarize this section... the geotechnical risk exists… it is there… deal with it. Since we investigate less than one part in a million, we know little about it. And lastly, we are expected to design to it, as best we can, with what we do know. So if you can't avoid it, manage it!

Geotechnical Construction Risks

When a construction opportunity is presented, what do I ask myself about it?

1. *Do I have a good-enough understanding of the subsurface conditions?*

The first place I look of course is the geotechnical investigation report. These reports vary widely depending on the preparer, and more recently depending on what the customer is willing to spend on investigation. Developers often commission an investigation designed to determine the "feasibility" of a site, and this investigation becomes the information we are obliged to "design" to.

It should be said that this is not always the case as there are many excellent investigation reports I review each year, but the percentage of "feasibility" investigations is ever increasing.

Points of particular interest in reviewing reports include:

- Site history… what it is like now comes from the investigation… but what was it before?
    - Structures, utilities or old foundations
    - Old fills and surcharge loads
    - Stream beds, lakes or beaches
    - Quarries and mine activities

- Actual sampling and in-situ testing
    - SPTs
      With what equipment and
        + How were the holes drilled?
        + Continuous sampling or at some depth intervals?
        + Is the log some driller's interpretation, was there a trained inspector, or is it from laboratory classifications?
        + How was the groundwater level determined?
        + What, if any, corrections are in order in the evaluation of the N-value identified?

    - Are there any CPTs?
        + Do the CPTs corroborate the SPTs?

  - Are there other types of in-situ tests, or site reconnaissance observations of value?

- Laboratory testing
  - Is there information to base a professional design for the geotechnical construction?

- Interpretations
  - In most cases, the local consultant is best experienced to make site interpretations
  - Is there enough investigation and historic awareness to make low-risk interpretations?

- Recommendations
  - Is the variability too great to make the right recommendations?
  - Do the recommendations consider the risks for uncertain conditions?
  - If there is insufficient or unsatisfactory information, is there enough schedule time to do more investigation?

The objective here is to minimize the assumptions made and use directly correlating parameters for design. Another aspect is to identify the hazards that may be nearby or impacted by the new construction.

2. *Are the objectives of the construction clearly understood***?**

Geotechnical construction techniques vary considerably. Everything from stabilizing the ground for tunneling operations, to resisting the forces from a potential landslide. The construction can be integral to the intended product, or perform an auxiliary function that protects structures from nearby construction. Regardless, the specialist contractor deserves to know the objectives, in priority, so that he can best evaluate the risks of the work.

If there are design requirements and/or tolerances, then they must be clearly communicated. If there is a specified verification method, then it too must be clearly communicated and understood. Doing this in a way that acceptance/rejection is clear to all parties is not as easy as it sounds, which is why preconstruction meetings are so valuable.

Offering a bid proposal to do the work, exclusive of any exceptions to the terms and conditions of a contract, is considered acceptance of the constructability of the work in accordance with the plans and specifications. This certainly justifies having a clear understanding of what is expected by the owner.

3. *Can the requirements of the specification be met?*

Often is the case that the specification preparer is not expert in the technology or expected performance of the technology. This can lead to misunderstandings, and often verification by a means unsuited to the product, or minimum acceptance values improbable to achieve. Be certain expectation is consistent with the technology.

This is generally not the case for traditional geotechnical construction like piles and spread footings where specifications have been re-written and evaluated many times. But it is not yet routine to use ground improvement or specialized grouting systems, and one certainly cannot be an expert after a single experience.

A specifier must understand the method(s), product(s), and verification to be sure it is constructible as specified. This requires more than a simple literature search. It demands the same learning processes used to become an engineer, seeking out experts; questioning important aspects; understanding variability and how to design for it; and understanding what is controllable and what is not.

4. *Does the schedule impact achieving the specification requirement?*

Contract duration is rarely considered when specifications are prepared. But the specification sometimes requires that verification be performed after work has been turned over to others, or the schedule is impacted on account of this. Scheduling the work and assessing constructability in the desired timeframe is critical to meeting project objectives; nobody would argue with this.

Schedules drive the timing of construction tasks, and one's ability to achieve them can be dependent on specifications and project objectives. Everything from the time for consolidation to testing anchors quickly so that the next cut of excavation can proceed.

5. *Are the materials available to support my project needs?*

Again, in many cases the specifications, plans and contract documents may be prepared months prior to contract execution. With just-in-time inventory becoming more prevalent with suppliers, access to what is needed becomes more difficult and more costly. Advocating for specification flexibility is the best solution whenever possible. When not possible, advocate that the owner either purchase the materials separately, or arrange for the inventory separately.

An example of this might be for a wet soil mixing project, where the owner paid for a lab study to identify binder dosage to achieve in situ requirements, but didn't realize that one of the binders must be transported a long distance. Since materials are often 40% of the construction cost, this impact is very significant to both cost and schedule.

6. *Is there another way to achieve the objectives?*

Every specialty contractor believes they thrive in this environment. Nobody expects the consultant or owner to know all the options, especially when the value of them comes into play. Several years ago the author presented a paper meant to provide values for ground improvement solutions (Burke, 2003), but these are "ballpark" values and every site and subsurface condition will have nuances that affect these values.

Risk can be reduced by identifying these technologies that qualify for meeting project objectives, and having the specialist contractor propose his solution(s) and risk-reward associated with the project. Any specific concerns should be stated, along with requirements and tolerances for the verification.

7. *Do I possess the right resources to meet the specification and achieve the objective safely and on-schedule?*

Every specialty geotechnical construction technology requires experienced workforces, the right equipment, the right procedures, and suitable verification. If one of these is missing, the project is in danger of having problems, and becomes a serious risk to all parties concerned.

Smaller contractors have this concern routinely, but even larger contractors get busy and stretch their resources.

Prequalifications help in this regard, but they should be followed with preconstruction submittals offering specific project resources to be employed. This serves to benefit all parties.

8. *Are there operational or contractual issues that could impact construction?*

This question is a big category of things, from liquidated damages to a restricted jobsite with too many contractors working in the same space.

The project site likely needs specialty geotechnical construction because the ground conditions contain some risk. The specialty contractor may well be expert in his understanding of the construction technology, but his subsurface understanding at least at the project start, is no greater than the owner's. The risk must be fairly allocated in the project contract.

One of the biggest risks seen is when working for government agencies. Often is the case that the agency does a good job with site investigations and design, and the specification to build the work. But the agency bureaucracy often separates the construction oversight and contracting elements, somewhat separating the designer from the construction. This often leads to literal interpretation or misunderstanding of the specifications by agents unaware of the design or design intent.

The consequences of this can be extraordinary, and harmful to all parties <u>and</u> the project. There needs to be some continuity of the design aspects carrying through construction, and clear and easy communications throughout.

Biggest Challenges (Greatest Risks)

In my 30+ years of geotechnical construction experience, I have been challenged by many things. These challenges are just that, but they only scare me when consideration is not given to all aspects of the work. So what are the biggest challenges in geotechnical construction?

1. Stopping groundwater: Creating a groundwater barrier requires perfection in situ. Hydrostatic forces are real and continuous and must be understood. Even at low

gradients, any imperfection will be exposed. And worse yet, it may be difficult or impossible to identify them.

2. Resisting large forces: This could be a support system for compressive or lateral forces, and are compounded by seismic and hydrostatic forces. The load instilled by a large landslide can be extraordinary, and the means to add resistance can be very complicated, in three dimensions.

3. Large area loads: Dams, embankments, and tanks cover very large areas that impact the foundation to great depths. These areas are often large enough, that even if the ground is reasonably consistent, the character of it may vary enough to cause problems.

Risk Example:

Many years ago we were asked for a design-build solution for an excavation support system for a new cut-and-cover tunnel in an airport. The geotechnical investigation was reasonably thorough, identifying a largely sandy fill profile with high groundwater table. The specification was open to all methods, but restricted the amount of groundwater that could be pumped from the excavation.

A reinforced concrete box tunnel was the objective, and it was supported on driven pipe piles in the excavation (Figs. 1 & 2).

The specialty contractor designed and installed a cement-bentonite slurry wall, with steel sheet piling set within it. This was installed coincident with the pipe piles, and followed by a jet grouted base seal for groundwater control.

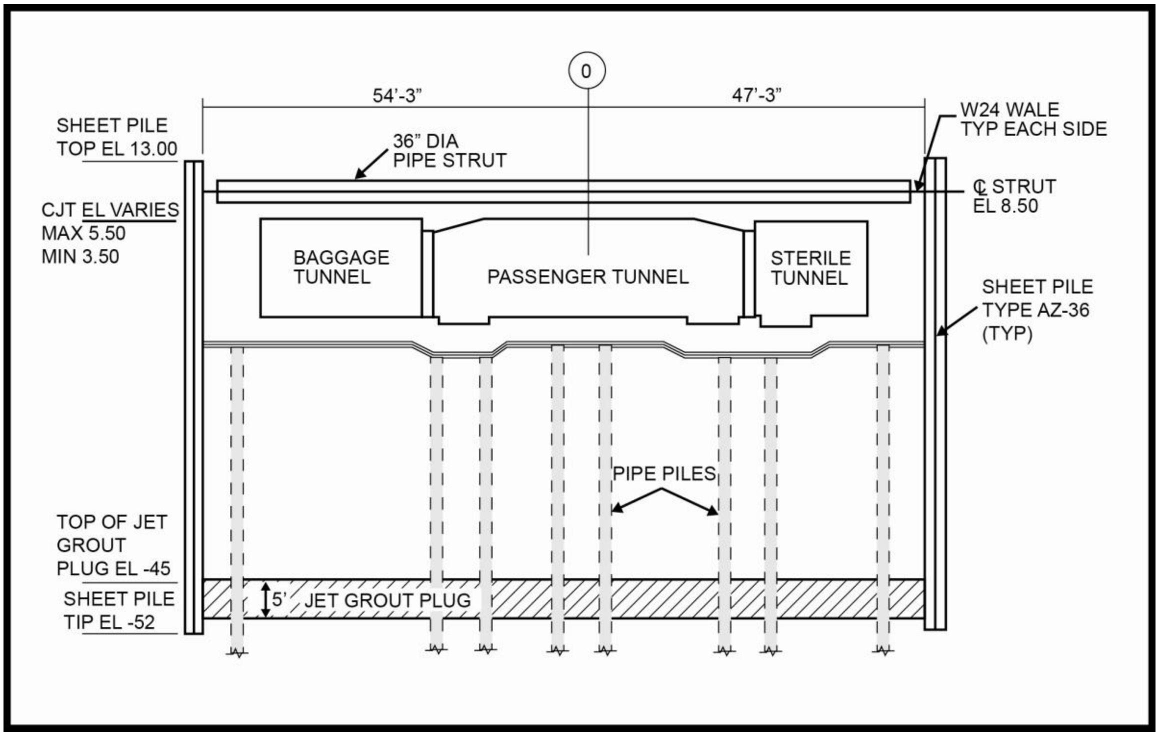

Fig. 1. Tunnel cross section

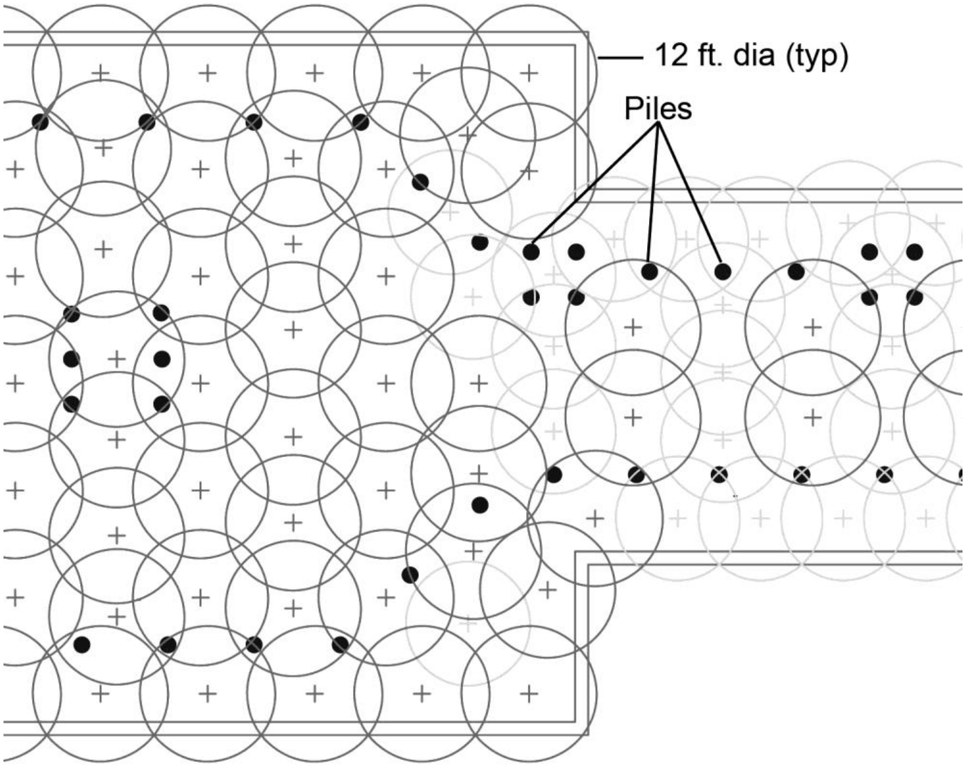

Fig. 2. Tunnel partial plan

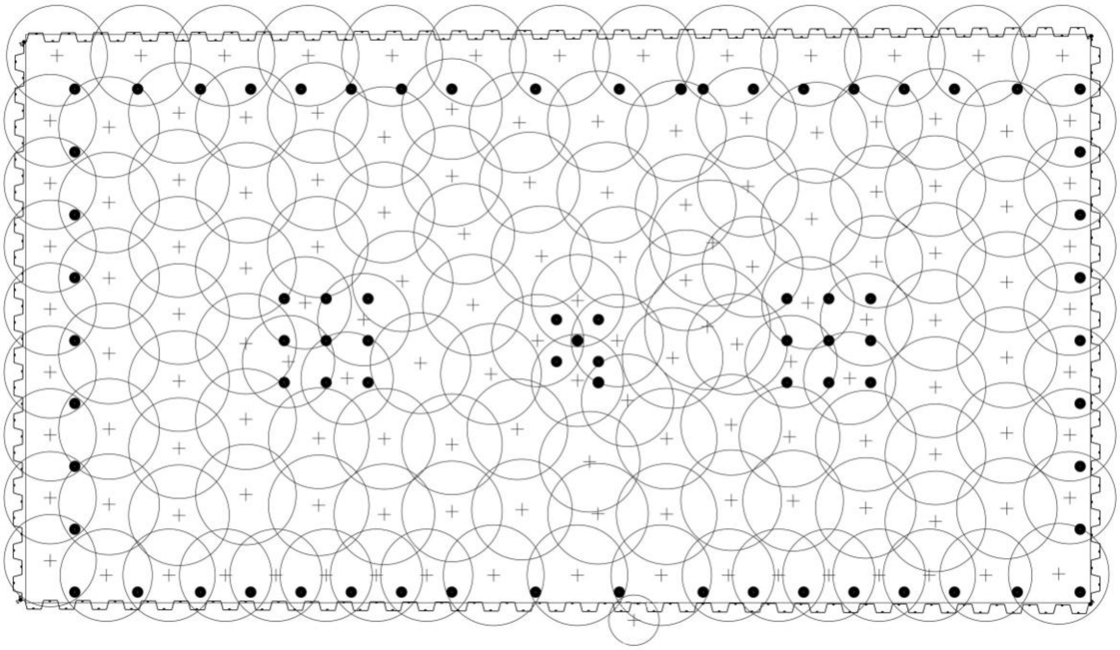

Fig. 3. Plan view of cement-bentonite slurry wall surrounded by sheetpiles, with jet grouted base seal

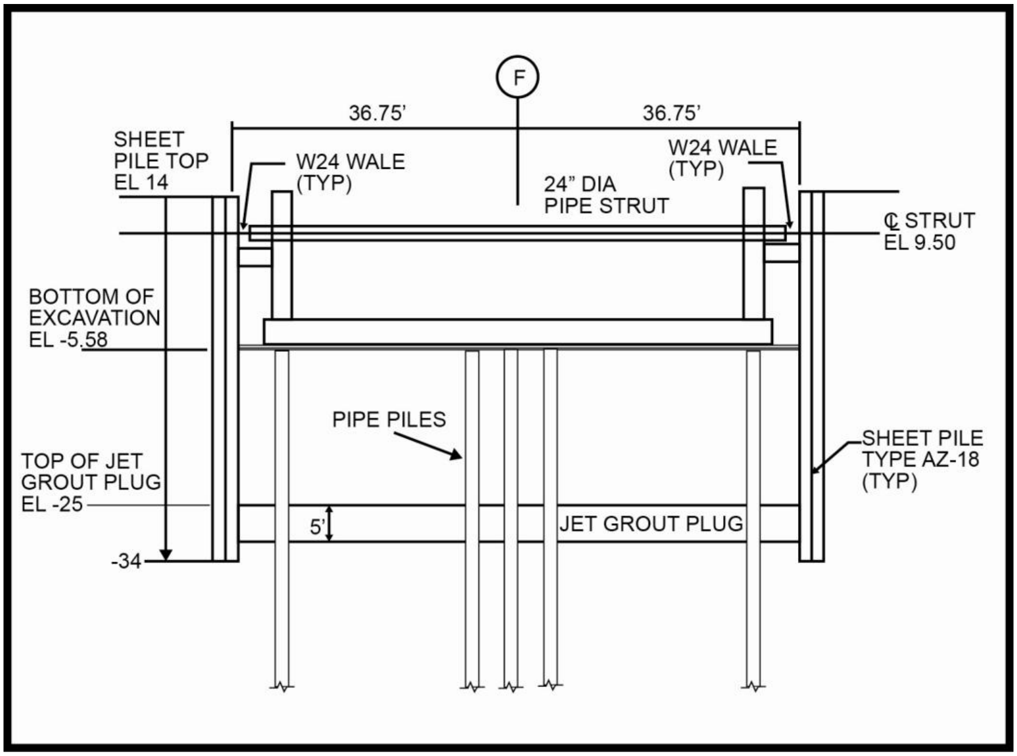

Fig. 4. Pump Station section view

First installed was a pump house cofferdam (8,150 sf jet grouted base seal installed after sheet piling was driven and the interlocks externally jet grout sealed, Figs. 3 & 4) and it was constructed quickly and easily, and the groundwater seal worked very well (Fig. 5). Construction progressed with few problems.

Fig. 5. Pump house and cofferdam with effective groundwater seal

The tunnel was a much larger structure, with 3 segments totaling 58,700 sf phased into the civil work. This also progressed on schedule, but when phase 1 was excavated, the allowable pumping criterion (130 gpm) was marginally exceeded. This volume of water could easily have been tolerated by simple sumping for construction, but a distant plume of jet fuel contamination was detected to have moved toward the excavation. This alerted the state agency overseeing environmental protection, which stopped all construction.

Risks identified before construction included:

- Schedule: liquidated damages included in the prime contractors contract
- Groundwater pumping tolerance: limited pumping for a large area was understood. Previous work like this was effective, but sister companies warned it requires the highest degree of QC. There was no aquiclude to key the wall into at reasonable depth.
- Selection of the best people and equipment for the project, and assignment of full time quality control staff.

Risks illuminated from post-project scrutiny included:

- QC elements were evidence of a job well done in all aspects

- Although the sequence of installation of the jet grouted base seal took into consideration the pipe piles, the effects from pile installation, and/or the effects of jetting near an obstruction could have caused a leak(s)
- The sandy fill material at this site was found to be laden with all sorts of clay balls, marine organics, and other elements that were the result of dredging and hydraulic filling. Un-engineered fill is just that.
- Although the contractor was prepared to clean contaminated groundwater pumped from the site, it was unforeseen that a project shutdown would result from moving a contaminated plume.

Significant schedule time was lost, and the tunnel was constructed in smaller sections to reduce the pumping requirements, all at significant extra expense to the contractor.

Summary

Geotechnical construction is risky business. Subsurface conditions cannot be perfectly represented. “Known-unknowns” should be anticipated whenever possible, but to say that the specialist contractor is the “expert” does not mean he possesses divine understanding any more than others.

Managing these risks is the best course of action, by all parties involved. This paper describes some of the considerations that should be addressed, and offers an example of what can go wrong when they are not.

The author would like to acknowledge the reviewers of this paper and the hundreds of projects that inspired this paper.

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Lessons Learned When Mitigating Liquefaction Potential Using Vibrocompaction and Stone Columns

Rick Deschamps, P.E., PhD[1]

Abstract: Recent changes in the International Building Code (IBC) have led to more stringent requirements for assessing and controlling the risk of liquefaction in many areas in the U.S. The assessment of liquefaction risk is an uncertain process that includes many variables, and is generally based on crude empirical models. When it is determined that there is an unacceptable risk of liquefaction, treatment is needed to make a developing site viable. Vibro-methods, including vibro-compaction and stone columns, are commonly selected as the most efficient approach to provide site improvement. However, there can be "conflicts" between what is achievable with these techniques and with the interpreted requirements of the empirical models. The presentation will describe conflicts between what can be accomplished with the vibro-methods relative to the requirements of the empirical models used for liquefaction assessment. It will be shown that there are situations were where the perceived degree of improvement needed cannot be achieved with some intermediate soil types.

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The Practice of Forensic Engineering

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Abstract: The practice of forensic engineering is one of the most interesting and challenging for geotechnical engineers. Geotechnical engineers are naturally drawn to failures as opportunities to increase our judgment by learning how applications of our knowledge may have failed to achieve the intended result. The challenge for many engineers is that these failure studies typically must be done within the context of a legal system where the primary objective is to assign or allocate responsibility for the failure. The adversarial process can be intimidating, especially for engineers not accustomed to the process. Engineers typically work together collaboratively in an atmosphere where they challenge each other to produce the best engineered result. In litigation, collaboration is displaced by criticism of even the most scientifically grounded professional opinions. Forensic engineering must therefore meld the best science of failure analysis to the art of conflict resolution, where the financial consequences of the resolution can be a matter of grave consequence to the responsible party.

Introduction

This paper is not intended to specifically address technical approaches to evaluating causes of failure, although several case histories will be discussed. Technical approaches for evaluating failures vary greatly depending upon the problem and are discussed in great detail in other publications. The intent of this paper is to discuss the framework under which a technical evaluation of a failure takes place in the legal system and how under some circumstances the requirements of the legal system can affect the work and the opinions of the experts.

The discussion in this paper is intended to be about the practice of forensic engineering as practiced by individuals who serve as expert witnesses. Admittedly, expert witnesses comprise a small percentage of the geotechnical community, but their practice has an impact on almost every company that engages in geotechnical engineering. Most engineers live in fear that someday they will be entangled in this process. In today's litigious society a failure typically results in the intersection of the legal and the engineering professions, two professions in which the members are highly trained and intelligent. An evaluation of a failure warrants the time and expense of a forensic evaluation is generally done for the purpose of allocating responsibility. In fact, the geotechnical expert's client is typically a lawyer who is retained by the defendant geotechnical engineering company. Engineers too often think of the evaluation of a failure only in terms of the technical issues involved. However, that is only part of the practice of forensic engineering; the evaluation of the failure takes place within the context and rules of the legal system and it is that involvement of the legal system that, in my experience, frustrates most engineers. A definition of forensic engineering edited to apply to geotechnical forensic engineering is as follows:

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> **Forensic geotechnical engineering** is the application of geotechnical engineering to answer questions of interest to the legal system. The word forensic comes from the Latin adjective forensis, meaning "of or before the forum." In Roman times, a criminal charge meant presenting the case before a group of public individuals in the forum. Both the person accused of the crime and the accuser would give speeches based on their side of the story. The individual with the best argument and delivery would determine the outcome of the case. This origin is the source of the two modern usages of the word forensic, as a form of legal evidence and as a category of public presentation.

As defined above the word forensic is both as a form of legal evidence and a category of public presentation. By the time both parties to the dispute get to court, both believe that their evidence will determine the outcome of the litigation. While the evidence must meet the standards of the court, the ability of the expert to present often complicated technical facts to either a judge or lay person in a jury can be the key to the outcome of the conflict. Unfortunately, the facts themselves are only a part – a very important part, of course– of the resolution of the conflict. A poor presentation of the facts by both sides in the conflict can result in a judge or jury having to decide the meaning and implication of complex technical issues based on their own life experiences and education.

The evaluation of a failure within the legal system has to incorporate the laws of science, the practice of engineering, and the rules of evidence within the court system. Typically an expert will have an opinion on the cause or causes of the failure, which will point liability towards one or more of the participants in the project. The basis of that expert's opinion can vary depending on the state or the court system in which the case is being tried. If the finding of the court is that there was negligence on the part of the engineer, that finding must be based on a determination that the engineer failed in their duty to meet the standard of care practiced by other engineers in their profession at the same time.

Allocation of responsibility leading ultimately to a financial sharing of the costs of the failure is that portion of the forensic study that most engineers find exceedingly difficult. The forensic evaluation leads to an opinion on the cause of the failure and then to responsibility for that cause. The technical evaluation then turns toward an accusation of responsibility to either an individual or company. Engineers are typically ill equipped to deal with this part of the forensic process. Lawyers are on the other hand quite skilled at conflict and the resolution of conflict through mediation, arbitration, or trial. Solving the conflict ultimately becomes the point of the forensic evaluation; this happens not only within the laws of science and engineering, but also within the rules of discovery and evidence within the court. A number of the issues that the forensic engineer must deal with in this practice are discussed below with a few case histories to illustrate the issues. The discussion is from the perspective of a geotechnical engineer who frequently intersects with the legal profession. The opinions on how the legal system works relative to geotechnical engineering are just that: opinions based on observations over 25 years of forensic investigations and expert testimony.

How Do We Define Fault – Standard of Care

In the allocation of responsibility regarding the engineer's performance the issue is always directed at the engineer's compliance with the standard of care. An engineer is not held to a strict liability standard where the performance of their work is guaranteed. The courts recognize that the practice and standards of engineering can vary over time. ASFE states that the standard of care is "… that level of skill and competence ordinarily and contemporaneously demonstrated by professionals of the same discipline practicing in the same locale and faced with the same or similar facts and circumstances." More recently courts have ruled that the geographic location may be a factor but not a determining one.

The implications of the standard of care defense for the defendant geotechnical engineer is that in the forensic evaluation the plaintiff's expert is looking for factors that contributed to the failure that were not representative of the standards met by other engineers. These can include many things such as the number of borings drilled, the frequency of sampling, the number and types of tests conducted, the interpretation of the data, the assumptions made in the analyses, the types of analyses performed, the recommendations made based on the results of the analyses, and the performance of the engineer during construction observation services.

In the practice of geotechnical engineering the engineer cannot specify the materials for the project. The engineer is given a site for which he or she may only observe a small percentage of the material and can test an even smaller percentage to evaluate its properties. The engineer must then interpolate between the limited data, apply reasonable assumptions to an analysis, provide the client with design and construction recommendations, and then provide construction observation services that will result in a project that performs to the client's expectations. At each step in the process substantial judgment is required.

Given so many opportunities to use an individual's judgment, how does anyone evaluate compliance with the standard of care? Two engineers with different levels of experience in a particular geologic formation may conclude that quite a different number of borings and tests are required. The extremes are easy; if no borings were drilled, no tests conducted, and recommendations were based entirely on prior experience, most would conclude that in most circumstances that would be outside the standard of care. Textbooks, handbooks, and a variety of publications give guidance on conducting studies that, in most cases, are conservative and defer to the judgment and experience of the engineer. While the extreme cases are easy, and quite rare, most of the time the evaluation of the compliance with the standard of care by engineers falls into a gray area that results in a conflict with opinions by plaintiffs and defense experts.

The definition of standard of care as proposed by ASFE and incorporated in other definitions seems simple enough. Merely polling other engineers is likely to produce results with a distribution of responses that correspond to the complexity of the problem. Ultimately the question is answered, as far as the court is concerned, by the experts in the case. The plaintiff's expert will present an argument that the defendant geotechnical engineer failed to meet the standard of care either based on their experience or by reciting technical documents that, in their opinion, establish the failure on the part of the engineer.

The defense expert will, of course, counter those arguments with their own experience and documents supporting the work of the defendant geotechnical engineer.

The difficulty with this system is that, in my experience, many plaintiffs' experts are highly educated, intelligent engineers with excellent academic credentials but with limited experience practicing the profession for which they now offer opinions on how others are conducting their work. Most practicing geotechnical engineers are reluctant to criticize their peers and rarely take on the role of the plaintiff's expert. The standard of care is never written down in a manner that has gained universal acceptance and therefore is always subject to debate.

In establishing whether negligence has occurred the testimony of the expert is subject to the rules of the court. As discussed below, under one set of rules of admissibility of evidence great weight is given to the education and training of an engineer to testify as an expert. Under another set of rules, while great weight is given to the education and training of the expert, a substantial weight is given to published peer-reviewed literature on the technical matter. Geotechnical engineers often believe that their technical strength is the local knowledge they have gained over many years of practice, which often is not recorded in publications. As in Roman times, the arguments are presented before the forum or in the present day equivalent, a judge or jury, and the side with the best facts and presentation will prevail.

The Rules of Expert Evidence versus the Practice of Engineering

The forensic evaluation is typically intended to come to an opinion as to the factors that led to the failure and were ultimately responsible for the failure. Geotechnical engineering, probably more than any other civil discipline relies on both the science of engineering and the empirical knowledge gained by an engineer in many years of practice. That combination of science and empiricism is considered the definition of the Standard of Care. The plaintiff's expert must reach the conclusion and offer the opinion that the defendant geotechnical engineer breached the standard of care at one or more points in the process leading to design. Because such arguments are so subjective, the defendant geotechnical engineer often says, "How can he say that?" There are in fact rules in the submission of evidence and the development of opinions. Experts are generally given great latitude in developing and presenting opinions, but there are rules nonetheless.

There are two main rules for the admissibility of expert testimony evidence in the United States. In all federal courts and in some state courts Daubert (Dejnožka, 2004), decided by the U.S. Supreme Court in 1993, is the basis for admissibility of evidence. In other states Frye (Dejnožka, 2004) is considered the basis by which admissibility of evidence is judged, and a few states have their own rules. These two rules have different criteria as to the admissibility of expert testimony in trial. The status of Frye or Daubert in state court varies. As shown in Table 1, 30 states have adopted Daubert or have rules of evidence consistent with Daubert, 14 states have rejected Daubert, and seven states have neither rejected Daubert nor accepted it. Daubert is the standard for expert testimony in all federal courts.

Consistent with Frye and as contained in the California Evidence Code 720, to qualify to give expert testimony:

> "(a) A person is qualified to testify as an expert if he [or she] has special knowledge, skill, experience, training, or education sufficient to qualify him as an expert on the subject to which his testimony relates."

The opposing side can object that the expert lacks the qualifications to testify as an expert but the court is given broad discretion in this judgment and commonly the testimony is admitted and the judge or jury must decide what weight to give the testimony.

Generally under Frye, judges are not considered "gatekeepers," meaning that the testimony is typically allowed and the jury or judge can weigh the testimony relative to testimony of other experts. The opposing side can challenge the testimony when that testimony is based on assumptions not supported by the record, is considered speculation, or based on factors or methods not accepted by other experts. Under Frye in California expert testimony must establish general acceptance and that correct scientific procedures were used. General acceptance implies that a consensus of the relevant qualified scientific community accepts the technique. This interpretation is consistent with the standard of care definition. However, the discretion of the court in allowing opinions in testimony is broad and can often lead to the situation when the defendant geotechnical engineer asks, "How can he say that?," the answer simply is that it is allowed by the law based on the expert's qualifications. Under Frye it is really up to the judge or jury to decide the credibility or believability of the testimony.

In 1993 the Supreme Court ruled that the trial judge must "ensure that any and all scientific testimony or evidence admitted is not only relevant, but reliable." This ruling established the trial judge as the gatekeeper of expert testimony in federal court and in those states that have adopted Daubert. The rules of admissibility of expert testimony are based on four criteria:

- Whenever a scientific theory or technique is used in the development of an opinion, has the theory or opinion been tested?
- Has the scientific theory or technique been subjected to peer review and publication?
- Are there standards to control the application of the scientific theory or technique and is there a known or potential error rate?
- Has the scientific theory or technique gained acceptance within the relevant scientific community?

Importantly Daubert requires that testimony be based on scientific knowledge. That knowledge must be more than a subjective belief or unsupported speculation and must apply to a body of known facts. The methodology used in arriving at the opinion must evaluate a testable hypothesis, been subject to peer review, and have a known or potential rate of error. Daubert does not require expert testimony to be accurate to 100% certainty since it is recognized that uncertainties exist in science and engineering.

Table 1. The Status of Daubert in State Courts 2006

States Adopting Daubert		
Alaska	Maine	Oklahoma
Arkansas	Massachusetts	Oregon
Connecticut	Michigan	Rhode Island
Delaware	Mississippi	South Dakota
Georgia	Montana	Tennessee
Idaho	Nebraska	Texas
Indiana	New Hampshire	Utah
Iowa	New Jersey	Vermont
Kentucky	New Mexico	West Virginia
Louisiana	Ohio	Wyoming

States Rejecting Daubert		
Arizona	Kansas	Pennsylvania
California	Maryland	South Carolina
Colorado	Nevada	Washington
District of Columbia	New York	Wisconsin
Florida	North Dakota	

States Neither Accepting Nor Rejecting Daubert		
Alabama	Minnesota	Virginia
Hawaii	Missouri	
Illinois	North Carolina	

The differences between Frye and Daubert can be seen in a dispute between plaintiffs and defense experts. Assume a slope failure occurs during construction adjacent to existing homes. A lawsuit is filed against the geotechnical engineer of record for the slope failure. The slope movements do not cause any visible or measurable damage to the homes. The plaintiff's expert claims that the slope movements have resulted in a reduction of lateral confinement for the soil under the existing homes. The loss of lateral confinement has, in their opinion, resulted in lateral expansion of the soil and the opening of cracks that cannot be seen at the surface. It is hypothesized that over the passage of time the infiltration of water from irrigation and rainfall will result in migration of surface soils into those cracks and eventually lead to differential settlement at the surface and subsequent damage to the homes. The plaintiff's expert acknowledges that the cracks cannot be detected as the process of investigating the cracks will obscure them.

The above example happens frequently in litigation. An expert will take an event such as a slope failure adjacent to homes and state that loss of lateral support leads to horizontal movement in the slope. That statement leads to a theory that the consequence of that event will have a future impact of substantial magnitude on the property. The current damage that will eventually lead to future damage is not testable, and therefore results in the proverbial dueling experts. Under Frye this testimony can be allowed based on the credentials of the expert. The credibility of the testimony will be determined by a judge or jury weighing the testimony against the testimony of other experts. Under Daubert this testimony can be challenged by the opposing side and a hearing can be held on the admissibility of the expert's testimony; the judge will have to make a determination as to its admissibility based on the criteria by Daubert. The defense will argue that the testimony amounts to nothing more than a subjective belief or unsupported speculation. There is a lack of peer reviewed literature supporting this hypothetical scenario. Additionally, the fact that this is not a testable hypothesis will increase the likelihood that the testimony will not be admitted.

Considerations for the Forensic Investigation

The forensic investigation starts with the fact that a failure has occurred likely as a result of a geologic feature, soil property, groundwater condition, loading condition, constructed feature, negligence on the part of the engineer, construction defect, or some other condition that differs from those assumed during design. All parties begin the process knowing that something is different than assumed by the engineer. The question is whether that thing could have been foreseen, was knowable, was the result of an error in calculations, an omission, a negligent act on the part of the engineer, or a defect in construction caused by the contractor. The burden of proof rests with the plaintiff to demonstrate that the geotechnical engineer breeched the standard of care. In some cases the plaintiff's expert may simply review the existing data and conclude that the engineer was negligent in the number of borings, tests conducted, analyses performed, or observations made during construction.

In other cases there may be extensive investigations that uncover a previously unknown condition. The plaintiff's expert, in reviewing the existing or new information, will arrive at a completely different interpretation of data. The plaintiff's expert will conclude that the result of their investigation and their interpretations conclusively show the geotechnical engineer of record breached their duty to meet the standard of care. In addition to the conclusion on standard of care he or she will present a cost associated with mitigating the damages associated with the failure.

The time for investigation is limited to the discovery period defined by the court. The plaintiff presents its case and then the defense has the opportunity to conduct whatever investigation is felt necessary to rebut the allegations. Typically there is but one opportunity to conduct the defense's investigation. A consideration the defense will evaluate is "did the plaintiff prove their case?" Often in a standard of care case additional investigation does not necessarily help the defense's case regarding the negligence aspect. If in the course of their investigation the plaintiff uncovers some unknown condition. The defense will argue that the defendant engineer conducted the investigation, testing, analyses, and observations in accordance with the standard of care. The newly uncovered condition was not knowable or foreseeable. The plaintiff's investigation was done with the benefit of hindsight, unavailable to the geotechnical engineer at the time of their investigation.

Additional investigation is useful for the defense to develop its own cost of repair, regardless of how strong they may believe their case to be in arguing that the geotechnical engineer is not liable for the damage. In the court's deliberations the judge or jury may determine that an allocation of responsibility is appropriate and the defense would prefer that the allocation consider costs they believe to be more realistic. The defense expert must consider that the plaintiff may say that they accept the expert's cost estimate and ask that they conduct whatever repairs are believed necessary and guarantee the work. This is typically a negotiating strategy and is done prior to court as part of settlement discussions. In my experience, I have never seen a defendant geotechnical engineer agree to resolve a problem that they do not believe they have any responsibility for, for a plaintiff who has sued them, and then guarantee the work.

Summary of the Litigation Process

After the investigations have been completed and both sides in the conflict have an idea of the costs involved and an understanding of the strengths and weaknesses of their arguments, the process of financially resolving the matter can truly begin. In mediation I have seen mediators, when meeting privately with the engineers and their attorneys, ask the attorneys to tell their engineer client the cost of going to trial, a cost that the attorney would accept as a lump sum price. It is common that these costs can range from hundreds of thousands of dollars to over a million dollars. Generally attorney's fees are not recoverable unless the original contract provides for these fees.

Engineers often have difficulty agreeing to settle a case when, in their view, the facts do not indicate that they were negligent. Ultimately at this point the decision becomes based on business facts. The costs yet to be incurred and the risks of losing must be considered. The cost, both financially and emotionally, of the involvement of very senior personnel in the firm must also be considered. It is at this point that the ability of attorneys to deal with conflict far outweighs that of the engineer. More than 90% of all litigation cases I have been involved in have settled prior to trial.

The costs associated with settling the case can be mitigated by the quality of the forensic investigation. Having the best facts along with the best presentation will convince the other side of the weakness of their case and move settlement toward a reasonable amount. These arguments are illustrated in the case histories that follow.

Case Histories

The case histories described below are intended to demonstrate outcomes in court when juries have to decide technical matters based on their own life experiences. In one case a simple technical fact is well presented before a jury, contrasted with the second case where several complex technical issues were presented to a jury. Based on polling of the jury after they reached their verdict it was clear that all the experts had failed to present convincing arguments. The last case history presents a failure and the outcome was more in line with what all engineers would like to see as a result of a forensic investigation.

Good Facts and a Good Presentation

The first case history involves a very familiar scenario in geotechnical engineering. The project was the installation of utilities over a length of several miles with trench depths reaching 25 feet. Placing and compacting backfill in trenches at depths to 25 feet can be difficult and present challenges for all involved. The geotechnical engineer did not test the compaction at depths below 10 feet due to concerns about safety. All the compaction tests met the project specifications as the specifications were understood by all at the time. Shortly after completion of the backfilling of the trenches, settlements of up to four inches of the backfilled trenches had occurred at numerous locations.

After the settlements had been observed the geotechnical engineer evaluated the characteristics of the backfill through a series of borings. Samples were taken through the full depth of the backfill, density and water content measurements were made, and the compressibility of the backfill was measured. The degree of compaction of the samples was measured by combining the individual samples collected in the borings and then developing a compaction curve using ASTM 1557. The measured dry densities of the soil samples were typically from about 65 pounds per cubic foot (pcf) to about 80 pcf, with water content from about 35% to about 50%. The engineer concluded that, based on the lowest measured maximum dry density of 87 pcf, samples at dry densities less than 78 pcf would fail to meet the 90% relative compaction criteria. Additionally, based on compression test data, the engineer estimated that at the insitu densities and water content, settlements on the order of 1.5 inches to 3.5 inches were to be expected. The engineer concluded that the entire depth of backfill did not meet specifications and should be removed and recompacted and the settlement was due to construction defect.

The owner of the project demanded the contractor remedy the defect. The contractor declared bankruptcy at which point the owner called the bond the contractor put up for the project. At this point I was requested by the contractor’s bonding company to review the data and the conclusions reached by the geotechnical engineer. The low dry density and high water content data certainly seemed quite inappropriate for a backfill of this use and likely to produce the results observed at the site. Despite what may appear to be obvious, it is very important to review the available documents and to the extent possible to understand the history of the project prior to reaching opinions regarding causation.

The original geotechnical report for the site recommended that the compaction requirements for the site be in accordance with CalTrans standard CTM 216, with the caveat that dry density be used. This is an important caveat since CTM 216 is not a dry density specification, as stated in the general scope for CTM 216:

> “Relative compaction in this method is defined as the ratio of the in-place wet density of a soil or aggregate to the test maximum wet density of the same soil or aggregate when compacted by a specific test method.”

When the final specifications for the project were produced, the compaction requirement for trenches included the use of CTM 216 without any caveats as recommended by the geotechnical engineer on the use of dry density as opposed to a wet density; additionally there was a requirement that 90 percent compaction be achieved at depths below 12 inches. The engineer’s recommendation in their report to use a dry density specification was not carried forth in the final specifications for the project, despite the geotechnical engineer’s review of the final plans and specifications.

In subsequent meetings with all the parties it was pointed out that the investigation by the geotechnical engineer on the characteristics of the backfill was requiring the contractor to meet a standard that was not part of the construction documents. Despite what was intended on the part of the engineers regarding the use of a dry density compaction requirement that was not what the contractor was required to do by the contract. The engineers believed that they had significant experience in prior projects with the CTM 216 specification and that the fault had to be on the part of the contractor regardless of whether a wet or dry density specification was used. A decision was made by the owner of the project to remove all the backfill and recompact the same fill in place with the CalTrans CTM 216 specification as a dry density specification.

All the parties were allowed to sample the original backfill during the removal process. The wet density and water content data collected as the soil was excavated agreed fairly well with the data previously collected in borings by the geotechnical engineer. Despite the depth of the trenches and length of the project, there were a limited number of tests taken during construction. All the samples taken during construction met the specification both for wet densities and for dry densities.

Samples taken after construction generally had slightly lower wet densities and higher water content than samples tested during construction. However, attempting to determine density and water content data existing at the time of compaction at a later date can be imprecise. The data does show that soils at the recorded densities and water content will not perform as expected for a project of this type. The soils have very low dry densities and high water content and are susceptible to significant settlement. In my opinion, the data did indicate that based on the CTM 216 wet density specification as included in the project specifications, it was highly probable that the contractor placed the soils in accordance with the contract specifications.

The obligation of the contractor is to build the project according to the plans and specifications. The specifications, as included in the contract documents, required that the soils be placed in accordance with CTM 216, which is a wet density specification. The placement conditions of the backfill at the time of construction cannot be determined to an absolute certainty; however, in my opinion, it is highly probable that the compacted soils would have met the specifications. It is also true that the soils as they existed at the time of sampling following construction did meet the specifications. Based on the results of investigations the contractor’s bonding company filed a lawsuit against the owner of the project and the geotechnical engineering to recover the bond money that had been paid. The complaint alleged that the settlement problems were the result of negligence and not a construction defect. The project had been built according to the specifications.

When a failure occurs it is universally true that the attorneys for the parties go back to the contract documents to understand what was agreed to by the parties at the initiation of the project. They are not looking for what people intended to do or what people thought should be done, but what was actually agreed to be done. At times this can be a matter of dispute; however, in this project it was clear that the parties contractually agreed to place the soils in accordance with CTM 216. That compaction specification was inappropriate for these soils and would allow for placement of backfill that would meet the specifications but would have a performance that would not meet the expectations of the project.

Several attempts were made to mediate the litigation that were ultimately unsuccessful. Lacking a resolution, the parties were committed to going to court for a jury trial. The contractor's bonding company's position was that the design was defective as a result of negligence on the part of the engineer. The negligence on the part of the engineer was the failure during their review of the specifications to note that the specifications did not meet their recommendations. The specification was inappropriate for this site, not consistent with the standard of care, and was always likely to lead to failure. The defense had several legal arguments as to the ability of the plaintiff to bring suit, but also argued that this specification was appropriate, used substantially in the practice, and the cause of the failure was defective construction.

The technical issues in the case were narrow and apparently easily understood by the jury. The inappropriateness of the specification was demonstrated to the jury by a container of soil that given its weight, would not meet the specification. However, for a total density specification, merely adding water to the soil to increase its total weight eventually would allow the soil to meet the specification. This simple demonstration helped the jury to understand the technical issues being debated by the experts. The jury returned a verdict in favor of the contractor and its bonding company for nearly the full amount of the damages against the owner of the project and the engineer.

Convincing a jury or judge in a case involving technical matters is a matter of having the best facts supporting the case and being able to present them in a way a lay person can understand, for either complex or simple technical issues.

Good Facts and a Bad Presentation

The second case history is far more complex technically. The site was a residential development with about 1400 home sites, and involved a massive grading project, with lots constructed on fill over gently sloping hillsides. The project had, in addition to the housing lots, a significant amount of open space surrounding the homes. Shallow failures occurred in the open space causing concern on the part of the Homeowners Association (HOA). A lawsuit was initiated by the HOA against the developer and subsequently all the parties involved in the design and construction of the project were parties to the lawsuit. A series of investigations were conducted by several different geotechnical consulting firms representing the different defendants. During the original investigation for the development, a number of existing landslides and colluvium were found in areas that were to receive fill for construction of lots for homes. Records by the geotechnical engineer of record of removal of the colluvium and landslide deposits during construction were sparse or non-existent. Additional investigation through boring and tests pits indicated that perhaps as many as 30 areas where homes were built could be fill constructed over soils that had been intended to be removed. The homes had been built several years prior to the litigation and some had existed up to about five years. No homes had been damaged by earth movement although, as indicated, landslides had occurred in adjacent open space.

The issue at dispute was the extent to which the stability of the existing fills supporting homes would need to be mitigated through construction of buttresses and installation of drainage to achieve appropriate factors of safety. The issues debated by the experts included:

- The areas underlain by the colluvium and/or landslides;
- The properties of the colluvium;
- The residuals strengths of the colluvium and landslides;
- The assumptions on groundwater levels;
- The effects of earthquake loading on soils at their residual strength; and
- Appropriate factors of safety to be used.

A cross section typical of the conditions at the site is shown below in Fig. 1. While there was significant scatter in the soil data, there was agreement on the strength properties for the fill and colluvium. There was disagreement on the residual strength properties to be used, the location of the groundwater table, what would be appropriate factors of safety, and the meaning of calculated factors of safety for landslides at their residual strength.

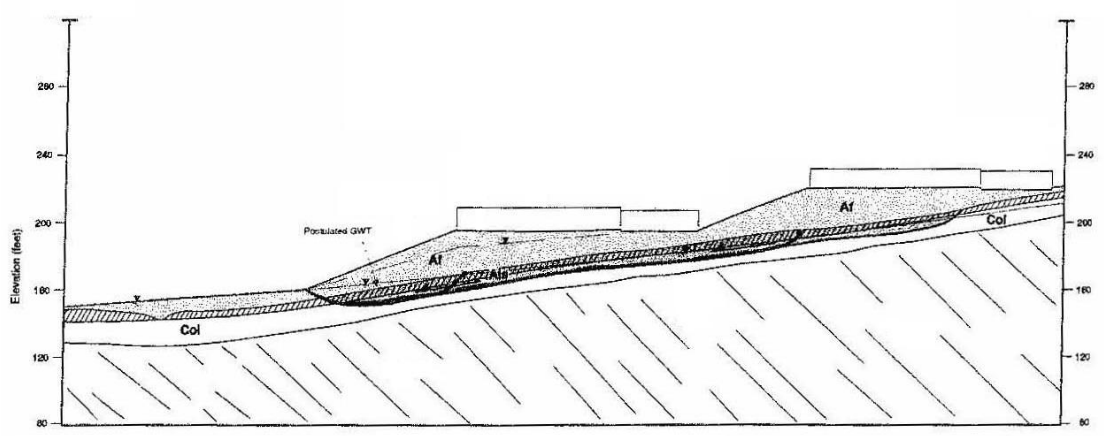

Fig. 1. Typical Cross Section

The disagreement on the residual strength was primarily due to the curved nature of the residual strength envelopes. The calculated vertical stresses on nearly all the residual surfaces ranged from 500 pounds per square foot (psf) to about 2500 psf. Depending on the depth of the slide plane, a different linear approximation of the strength envelope was utilized, incorporating a different value of cohesion (c) and friction angle (phi) to get the correct shear strength for the appropriate range of stresses. Others choose to use a singular value of c and phi to approximate the shear strength at all depths.

There was significant disagreement on the appropriate factor of safety to be used. Commonly a factor of safety of at least 1.5 is used in residential construction. I argued that the use of a factor of safety of 1.5 or even higher is appropriate at a stage of investigation where data is being collected on the site for design purposes. However, when a failure has occurred or through substantial additional investigation the uncertainty on the data is greatly reduced, allowing for a lower factor of safety. This is particularly relevant when residual strengths are being used. The uncertainty associated with the strength of the soil when residual strengths are used is quite small. The uncertainty due to a lack of long term data on the phreatic surface led to conservative assumptions on location of groundwater. The conservative assumption that groundwater was at or near the ground surface combined with the equally conservative assumption on strength through the use of residual strengths warranted the use of a factor of safety lower than 1.5. It was my opinion that the only true variables in the analyses were the location of the groundwater surface and the shear strength of the soil. Give the conservative assumptions on both variables a factor of safety of 1.25 was appropriate.

Additionally, there was significant debate on the use of seismic coefficients and the meaning of factors of safety against earthquake loads. There are well established methods to establish the factor of safety against seismically induced deformations. These methods involve total stress conditions and the use of undrained strength properties. However, it was my opinion that these methods were not necessarily applicable to calculation of deformations of slopes where the soils are at their residual strengths. The residual strength of soils is calculated or measured as an effective stress condition. The use of an effective stress parameter in a total stress analyses would be inconsistent with basic soil mechanics.

There was no literature available on this topic to provide insight on this matter. In a State like California where Daubert is not accepted the testimony of all experts on this matter is allowed and the jury must decide this issue, weighing each expert's opinion. In other states where Daubert has been accepted and an opinion lacked peer reviewed technical literature the judge could be asked to not allow that opinion in the trial, essentially having the judge serve as the "gatekeeper" as intended by Daubert.

In developing recommendations as to which sites needed mitigation substantial cost differences occur when analyses are done with factors of safety of 1.25 versus a factor of safety of 1.5. The differences in assumptions by the experts resulted in an order of magnitude difference in the cost estimates to mitigate the problems at the site. My estimate was that approximately $2.5M dollars were required to provide the homes with an adequate factor of safety while the opposing expert estimated the cost at about $25M based on the assumptions in their analyses.

The trial was heard by a jury and lasted about five months. Testimony by experts lasted for several weeks. As an example of the uncertainty of jury decisions, the geotechnical engineer, who was a minor party to the case, settled for the full amount of their insurance policy while the jury was deliberating. When the jury returned their verdict they did not find that the geotechnical engineer was guilty of negligence. The jury came back with a verdict that the developer and contractor owed the HOA about $6.5M.

The jury members who agreed to be interviewed by the lawyers after the trial as to how they arrived at their verdict indicated that they never understood what the experts were talking about. At the end they did believe there was a problem and they thought the HOA should have money available to fix any issues that developed so they figured out the dollar amount on their own. This case illustrates the perils of allowing a jury to decide complex technical issues. It is not clear from the jury’s perspective who had the best facts; it is clear that both sides had a poor method of presenting their facts and the story of their case. The issues were very complex and difficult to present in a simplified form. This case history also demonstrates the perils of letting technical issues go to jury trial for decisions.

Forensic Engineering as It Should Be

The third case history involves slope stability problems that occurred during the early phases of the Panama Canal widening project. The Panama Canal in the Gaillard Cut area is well known by geotechnical engineers as historically unstable. The widening of the Gaillard Cut area was a major part of the expansion of the Panama Canal designed to increase the flow of traffic and increase revenue. At the time of the beginning of the project the United States still had authority over the Canal.

During this early phase of the Canal widening a major landslide occurred at an area known as Hodges Hill. Historically this area was well known to be unstable and had been studied by Casagrande. Fig. 2 from Casagrande shows the general vicinity of Hodges Hill in 1912, 1915 and 1947. Over a period of about 35 years the slope progressively moved to become stable at its residual strength. The original designers were unaware of the concept of residual strength and believed that the soils could sustain a much steeper slope. The slopes are comprised of formations ranging from basalts to clay shale with low residual strengths. A cross section of the lithology is shown in Fig. 3, where the various soil and rock types can be seen. The harder basalts overlay the weaker shale making back calculation of the residual strength a complex undertaking.

The geotechnical engineers at the Panama Canal Commission and the designers of the project are a very experienced and knowledgeable group with a good understanding of the properties and behavior of soils at their residual strength. They were very aware of the history of the slopes and the fact that the existing factor of safety was very near to unity, and that over steepening of the slopes during construction would result in a high probability of failure. The intent of the design was to cut the slope back to an angle equal to the original slope angle before removing the toe in what was called the “wet excavation” that would create the additional width for the canal. With this sequence of excavation the overall factor of safety should remain equal to or greater than the original factor of safety.

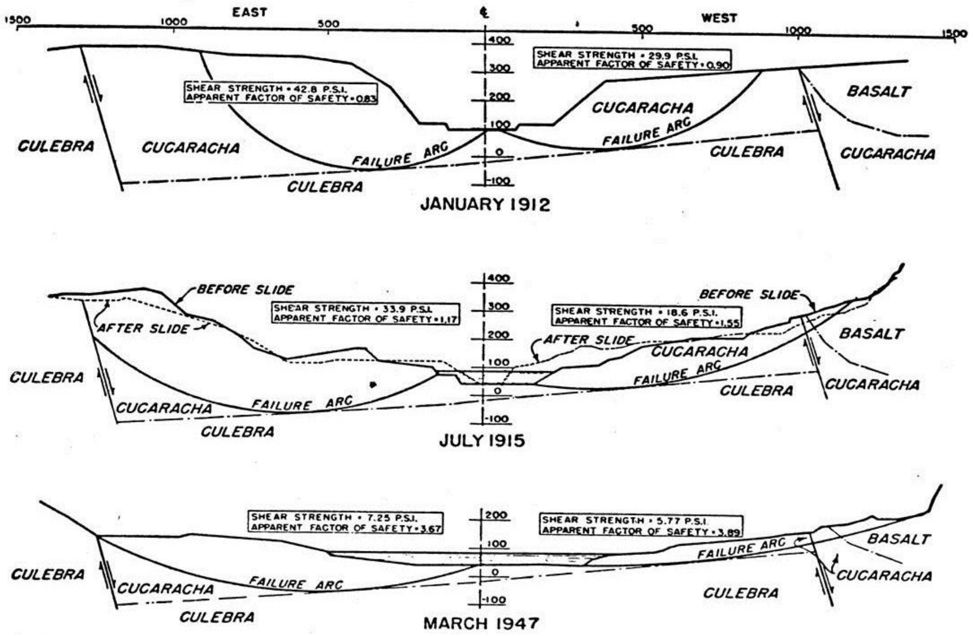

Fig. 2. Historical Landslide Stability

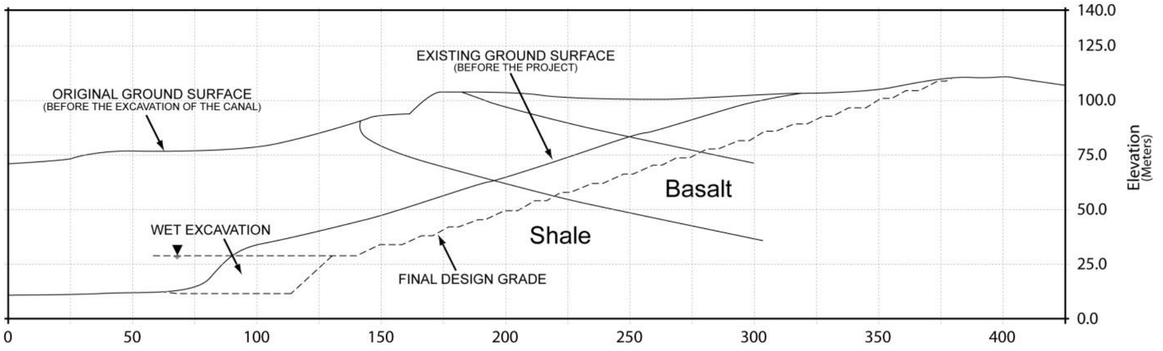

Fig. 3. Geologic Cross Section

The construction specifications were clear as to how the excavation was to be sequenced, as taken from the specifications and shown below:

> “General …The initial part of the excavation at any location shall be performed by complete progressive benching commencing from the top of the cut and working down to the lowest bench. Any other method of constructing the slopes shall be submitted for approval of the Contracting Officer. The contractor shall never leave slopes at angles exceeding design grades during excavation work.”

The landslides occurred in late July of 1997 and in January 1998.The slides occurred from about an elevation of 85m down to the Canal elevation of about 26m. This area of Hodges Hill has been subject to previous movements in 1910 and 1912. My review consisted of a site visit, extensive meetings with the staff of the Panama Canal Commission reviewing the design and subsequent construction of the cut slope, and the analyses of the failures. The specific issues to be addressed in my evaluation were as follows:

- Was the original design performed correctly?
- Was the slope constructed in accordance with the plans and specifications?
- What was the cause of the 1997 and 1998 landslides?
- What are the impacts of the slope failure on the wet excavation project?

In 1968 cracks were discovered on Hodges Hill at approximately the same location as the 1997 and 1998 landslides. When the movements causing the cracks observed in 1968 actually took place is unknown. Lutton's conclusion regarding the 1968 cracks is as follows (Lutton, 1975):

> "Activity in April 1968 is seen in retrospect to have fallen short of failure. There seems to be no strong evidence or reason to suppose a through going sliding surface had developed. It was suggested in a letter to members of a board of consultants on the problem (Memo, E and C Director, July 16, 1968) that a logical shear zone pattern was not in evidence from data collected on subsurface motion. The depths and areas in which motions were recorded suggested shearing activity on several planes with separate blocks of material shifting independently. No reactivation followed despite the fact that at least two heavy rains fell that wet season."

No subsequent significant movements were noted in the Hodges Hill slope even during the record rainfall year of 1986.

The project was required to facilitate the subsequent wet excavation for the widening of the Canal. The basic criteria for the design of this and other cut slopes is that the factor of safety at all stages of construction and at completion of excavation be greater than or equal to the factor of safety that existed prior to excavation. This criterion is imposed due to difficulty of evaluating the actual factor of safety in the Cucaracha Shale and the underlying Culebra Formation.

The Hodges Hill slope was considered stable based on criteria used at the Canal (factor of safety greater than 1.0) and based on its historical performance and the results of an analytical evaluation. The analytic evaluation used the existing slope geometry, assumptions on piezometric surfaces and strengths, and assumed modes of failure.

For the purpose of design, piezometric data was available from piezometers installed in 1968 and from piezometers installed in 1995. The highest piezometric levels from the period of 1968 to 1972 were compared to data collected from 1995 to 1997. The data from the 1968 to 1972 period was more conservative (higher piezometric evaluation) than the 1995 to 1997 data for the purposes of evaluating the stability of the slope and the design of the excavation of the slope. Using residual strength values for the Cucaracha and Culebra Formations, the calculated factor of safety of the slope was less than one, indicating the slope was unstable. Since this conclusion contradicted the observed performance of the slope, the higher, fully softened strength values were appropriately used for design.

The analyses for design of the cut slope assumed that the Culebra and Cucaracha Formations had similar strength properties. Evaluation of both fully softened and residual strength envelopes derived from previous analyses and laboratory testing for the Culebra and Cucaracha Formations indicate that the Culebra Formation has similar strength to the Cucaracha Formation. For the purpose of design, the assumption that both strengths are the same is appropriate. The analyses considered several modes of failure including failures at the bottom of the wet excavation cut and failure surfaces within the slope. The various modes of failure considered were sufficient to evaluate the most critical failure surfaces.

Based on the fully softened shear strength parameters used in the design, the slope was considered stable in its pre-construction configuration. If the slope angle was maintained or flattened during and after construction, the factor of safety of the slope would not be decreased from its pre-construction stable state. During construction this critical condition was to be maintained.

The specifications clearly require an unloading of the slope from the top to the bottom. With the final slope equal to or flatter than the existing slope the basic design criteria would not have been violated.

The design was performed in an appropriately conservative manner. The design evaluated the previous evidence of movement in the slope and through careful analyses concluded that the slope was not at its residual state allowing the use of the fully softened strength. This conclusion was consistent with that of Lutton and with the prior performance of the slope. The specifications provided a means for excavating the slope in a way that the basic design criteria would be met at all stages of construction and at completion. It was my opinion that the design was performed correctly and was consistent with the standard of care normally exercised by engineers.

Based on review of cross sections provided by the contractor as a basis of progress payments, the geometry of the excavation can be evaluated at any time during construction. The cross sections clearly indicate that the excavation did not proceed in accordance with the required specifications. The cross section, shown in Fig. 4, at approximately the center of the 1997 and 1998 landslides clearly shows that, by July 1997, prior to the landslide, substantially more excavation had occurred at the toe than at the top of the slope. This was in violation of the specifications and in violation of the design criteria.

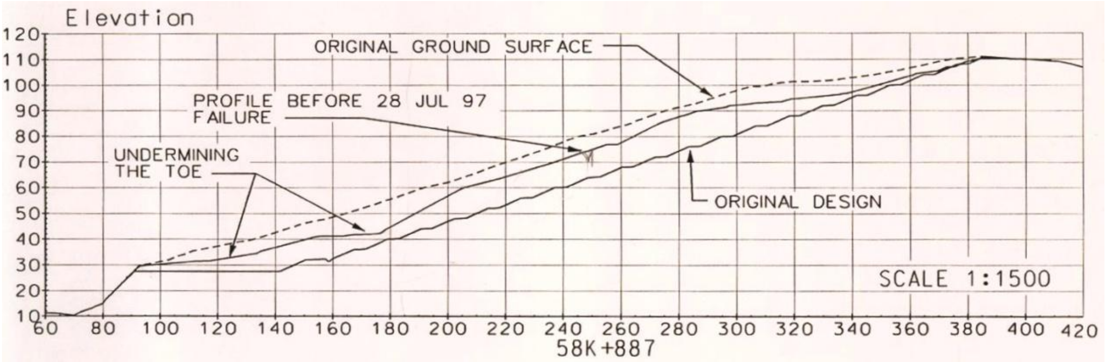

Fig. 4. Cross Section of Excavation Prior to Failure

The 1997 landslide at the Hodges Hill excavation is a result of the failure of the contractor to follow the plans and specifications. The overexcavation of the toe of the slope resulted in a steeper slope than existed prior to construction. This violated the basic design criteria of maintaining the factor of safety equal to or greater than the factor of safety that existed prior to construction. The excavation sequence during and prior to July 1997 resulted in a reduction in the factor of safety to less than one and the subsequent movement of the slope. Back analyses of the failed slope indicated a significant reduction in the factor of safety and the resulting reduction in the friction angle due to the movement of the slope.

The January 1998 movement was almost entirely within the 1997 movements and was merely a continuation of the earlier movements with the strength reduced from fully softened to a residual value. After the 1997 movements the slope was in an unstable condition and small, seemingly insignificant changes in the geometry of the slope could lead to additional movements.

The original design required that following the completion of the project and the subsequent wet excavation project the factor of safety of the slope would not be reduced below its value prior to the dry excavation. The slope movements of 1997 and 1998 resulted in a reduction of the strength of the soils in the slope. At the slope configuration existing following the 1997 and 1998 movements the factor of safety of the slope has been reduced due to the reduced soil strength. Excavation of the toe of the slope, as proposed for the wet excavation project, would likely result in a significant failure of the slope and potentially a major impact on the Canal. It was my recommendation that the Panama Canal Commission continue with its plan, which included the following:

- Installation of instruments to evaluate the depth of movements,
- Continued reevaluation of the reduced friction angle of the slope using the 1997 piezometric data,
- Design of reconstructed slope angles that result in a slope with the same degree of stability as originally considered in the design, and
- Allowing the contractor to continue the current excavation in accordance with the plans and specifications.

I advised the Commission that continuing excavation on the slope may result in additional movements of the impacted area. At that time all the evidence indicated that the movements were entirely within the slope and not impacting the Canal. It was my opinion that any additional movements would be insignificant, and similar to the movements observed in January 1998 of a few feet. Localized regrading may be required to flatten slopes to mitigate movements prior to development of a mitigation plan for the entire slope.

While the conclusion was that the landslide was a construction defect, it was pointed out to the Commission that the specifications for the sequencing of the excavation were clear the plans were less specific. The plans showed the initial and final configurations of the slope but made no mention of the required sequence of construction. For the purposes of a construction claim against the contractor there was enough data to pursue such a claim. However, one of the issues was to address future construction issues to facilitate a more efficient project. I recommended that future plans be more specific on the proposed sequence of construction.

Secondly, the contract had a single unit price for excavation. The excavation at the top of the slope was in basalt while the excavation at the toe of the slope was in shale. The contractor initially started the excavation at the top of the slope but moved to the toe, likely as a result of the slower rate of excavation in the basalt. The cash flow for the contractor on the project could be improved by the faster rate of excavation of the shale. I recommended that the Commission consider a provision in the bid documents for different unit rates for excavation at the top of the slope as opposed to the toe of the slope to provide an incentive to the contractor.

Thirdly, the Commission had a division of labor based on the responsibility of different departments within their organization. The construction management group was separate from the design group. A lack of communication between the designers and the construction managers on the importance of the excavation sequence contributed to the failure. I recommended that a member of the design team be involved in the construction of the project and future projects.

While a claim against the contractor was contemplated, to the best of my knowledge, none was ever filed. While this case history could be cited as forensic engineering in a perfect world, it remains a unique case history in my experience. This project was at the beginning of a project of major scope in the world and could be considered a “lessons learned” situation for the remainder of the project, where potential financial savings could be achieved.

Conclusions

The practice of forensic geotechnical engineering is the application of geotechnical engineering to answer questions of interest to the legal profession. In today’s litigious society the failure of a geotechnical engineering project brings together the engineering and legal professions to resolve the conflict of responsibility for the failure. The geotechnical engineer must apply science and engineering within the rules of the legal system in order that their work can be effective in helping to bring about a resolution to the conflict. The rules of the legal system governing the admissibility of opinions can vary from state to state and between state and federal court.

When an expert presents their opinion on the cause or responsibility for a failure, that opinion must be well founded and presented in a way that the judge or jury understands the technical matters at dispute in the litigation. Judges and jurors must reach a decision in the matter. When left with a poor presentation or poorly understood conflicts on complex technical matters they must use their own life experiences to decide responsibility for the failure. While jurors and judges do their best to sort out the issues the results can often be confusing. Settlement of the dispute prior to proceeding to trial is almost always the preferable outcome.

A thoughtful, high quality forensic investigation consistent with good science and engineering combined with an ability to clearly present the matters being disputed will always aid in the settlement of the dispute.

Acknowledgments

The author appreciates the support of a number of individuals over the course of a career engaged in forensic geotechnical engineering. Dr. Chris Hunt of Geosyntec Consultants and Julie Ryan, formerly of Geosyntec Consultants worked on many of the major forensic geotechnical investigations that comprise my experience and tirelessly worked to meet the difficult deadlines litigation often imposes. My knowledge of the legal process was gained through the many fine attorneys I have worked with in my career, but none more than Paul Sanner, General Counsel for Geosyntec Consultants; as my good friend and counselor he has constantly and gently reminded me of my limitations in my knowledge of how the law works. Lastly, I thank my good friend and mentor Dr. J.M. Duncan of Virginia Polytechnic Institute and State University, who taught me more about engineering and life than any other person in my life and who reviewed this paper providing many useful comments that greatly improved the content.

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Forensic Geotechnical Engineering for the Insurance Industry: Earthquake, Blasting, Weather, and Ground Related Claims

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Abstract: Many homeowners' insurance claims are made for damage to structures that is attributed to such causes as earthquakes, blasting, and severe weather events, when in fact, the actual causes are related to soil settlement, soil heave, earth pressures on basement walls, moisture extraction by mature trees, and normal shrinkage cracking of concrete. We have observed that homeowners generally do not look for cracks or other distress in their homes until they believe they have a reason to do so, such as following an earthquake, blasting, or severe weather event. Other claims are made for damage to homes in cases where neither the homeowner nor the adjuster has a good idea what the cause might be. We use case histories to promote understanding of some of the common causes and telltale signs of residential structure distress, which can assist in deciphering the available evidence and determining the validity of claims made for seismic, blasting, and severe weather events as well as other ground-related causes.

Introduction

Forensic engineering for the insurance industry is an interesting branch of the discipline of geotechnical engineering. In many cases, homeowners use simple cause-and-effect relationships to explain the causes of damage to their homes, i.e., noticing new cracking during or following recent blasting activity, or following an earthquake. In other cases, cracking or settlement damage appears in 20- to 60-year-old homes that have never experienced any distress, and neither the homeowner nor the insurance adjuster can determine the cause. In most of the cases we have investigated for the insurance industry, the actual causes of the distress turn out to be different from those assumed by the homeowner or the insurance adjuster. This paper uses case histories to promote understanding of some of the common causes and telltale signs of residential structure distress. This understanding can assist in deciphering the available evidence and determining the validity of claims made for seismic, blasting, and severe weather events as well as other ground-related causes.

Earthquake Claims

The United States Geological Survey (USGS) maintains a program on its website called "Did You Feel It?" that allows the public to report felt seismic events. Based on an individual's observations during, or response to, a given earthquake, a Modified Mercalli Intensity (MMI) is assigned. The MMI is an earthquake rating scale based on

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observations made during the event. The "Did You Feel It?" responses are organized by zip code, and are useful in gauging public perception of a given seismic event.

The most recent earthquake event to be widely felt in the Cincinnati, Ohio area was the Bellmont, Illinois earthquake that occurred on April 18, 2008. The USGS received a total of 42,174 "Did You Feel It?" responses to this event. Because felt earthquakes in the Cincinnati area are relatively rare, it is useful to compare the public response to the Bellmont event to one of the same magnitude that occurred in the Greater Los Angeles, California area on July 29, 2008.

Table 1. Comparison of Two Earthquakes of the Same Magnitude

LOCATION	Bellmont, IL	Greater Los Angeles Area, CA
DATE	4/18/08	7/29/08
MOMENT MAGNITUDE	5.4	5.4
TOTAL RESPONSES	42,174	41,148
TOTAL ZIP CODES	4,319	814
MMI V RESPONSES	592 (1.4%) from 20-610 km	6,192 (15.0%), all but one from <56 km
MMI VI RESPONSES	327 (0.8%) from 4-381 km	1,464 (3.6%), all but 3 from <20 km
MMI VII RESPONSES	6 (0.01%) from 11 to 35 km	0

The MMI V through VII criteria are as follows:

Table 2. The Modified Mercalli Intensity Scale (UPSeis [2011])

INTENSITY	EQUIVALENT RICHTER MAGNITUDE	WITNESS OBSERVATIONS
V	4.0 to 5.0	Felt by almost everyone, some people awakened. Small objects moved. Trees and poles may shake.
VI	5.0 to 6.0	Felt by everyone. Difficult to stand. Some heavy furniture moved, some plaster falls. Chimneys may be slightly damaged.
VII	6.0	Slight to moderate damage in well built, ordinary structures. Considerable damage to poorly built structures. Some walls may fall.

These two earthquakes occurred in the same year, had the same magnitude, and had almost the same number of “Did You Feel It?” responses. The far smaller number of zip codes reporting in the Greater Los Angeles Area reflects the higher population density there, while the far greater number of zip codes reporting in the Bellmont event likely also reflects the fact that central US earthquakes typically have much wider felt areas than western US earthquakes. A smaller percentage of the respondents to the Bellmont, IL earthquake rated the event at MMI V or greater; however, among those who did, the range of distances of the respondents from the epicenter was much greater.

We investigated two claims for damage resulting from the Bellmont earthquake. Both occurred in the same community in southeastern Indiana, roughly 225 km from the epicenter, where the USGS estimated the peak ground acceleration (PGA) to be about 0.03g. The first claim was at a 60-year-old, wood-frame, single-story-plus-basement house in which the concrete masonry unit (CMU) basement walls were not structurally connected to the first floor framing. The homeowner reported that most of the front and right side CMU walls had been replaced in 2002 because of distress. About a week after the Bellmont earthquake, the homeowner noticed that the CMU basement walls were distressed again. He then filed his claim for earthquake damage.

Fig. 1. Non-replaced corner of basement walls, showing previous tuckpointing.

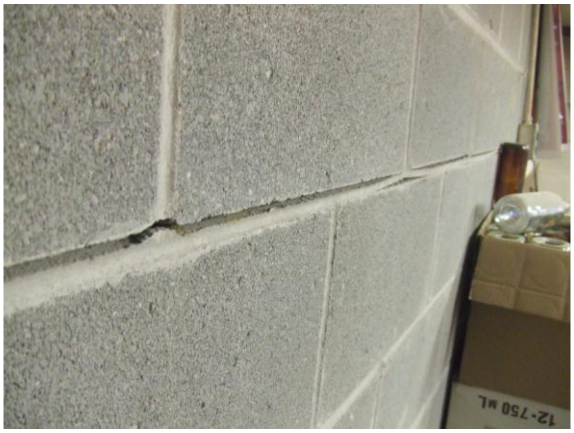

Fig. 2. View of inward bowing of replaced portion of basement wall.

The corner of the basement walls that was not replaced in 2002 (Fig.1) showed previous tuckpointing. The front CMU wall showed a horizontal mortar joint separation (Fig. 2) about 2/3 of the wall height above the floor. The section of wall above the separation was plumb, and the section below was tilted top-inwards about ½ inch per foot.

We noted that the downspouts were connected to flexible, perforated, PVC drainage pipes that extended below grade. The homeowner said that as part of the 2002 repair work, all of the downspouts were connected below grade by the same flexible, perforated pipe. He said the buried pipe ran into the back yard, and did not have an above-grade outlet. The only option for discharge of the roof drainage was dissipation in the subsurface out in the yard and along the below-grade basement walls.

Some of the backfill along the front (west) wall had settled and experienced grade reversal. Grades were sloped towards the house adjacent to the west wall.

Minor cracking was observed in the first-floor plaster walls. The homeowner said all of the first-floor plaster distress was old.

Within the spectrum of possible shaking damage, the damage threshold of interior plaster in the upper levels of a structure can be expected to be among the lowest. This is because the effects of shaking are amplified in the above-grade portions of a structure, where there is no earth restraint to limit deflections, and where the construction materials (e.g., plaster) are weaker than the CMU in foundation walls. The distress to the basement CMU walls was consistent with that expected from a combination of active earth and hydrostatic pressures acting on a long CMU wall. Below-grade CMU walls are not typically designed to withstand a combination of active earth and hydrostatic pressures.

The roof drainage from the house was directed into a below-grade, perforated, PVC pipe with no above-grade outlet. This pipe ran along the below-grade, distressed, exterior walls of the house, and its drainage was directed to the gravel backfill against the below-grade walls. Because the pipe has no above-grade outlet, the roof drainage was discharged along the below-grade walls once the pipe filled and backed up. This provided a significant amount of water to the subsurface that was generating hydrostatic pressures against the CMU walls, for which they were not designed. We concluded that the distress to the basement walls was not caused by the earthquake, but by hydrostatic pressures that the CMU walls were not designed to resist.

The second Bellmont earthquake claim was at a 45-year-old (±), wood-frame, single-story-plus-basement house with CMU basement walls. Site grades sloped towards the house along one side. The long walls contained window wells with no drains. The homeowner said that the basement had experienced water problems for years, and that he had repaired a horizontal crack in the north wall some years ago. The cracks were mostly horizontal and occurred along the three below-grade walls. Near the corners, the cracks were stair-stepped through the CMU mortar. The cracks showed evidence of previous patching and repainting. The homeowner said there was no distress to the first floor.

Review of the photographs shown below (Figs. 3 and 4) indicates that the distress to the below-grade CMU walls predated the Bellmont earthquake. (It is interesting that the previous repairs were far more distinct on the photographs than with the naked eye.) We concluded that the distress was caused not by the earthquake, but by hydrostatic pressures that the CMU walls were not designed to resist.

Blasting Claims

Shaking caused by blasting can elicit a response from property owners similar to shaking caused by an earthquake: they inspect their homes or businesses for damage following the event, and can discover cracks or other distress that may have existed prior to the event, but which they had not noticed before because they did not previously have a reason to look.

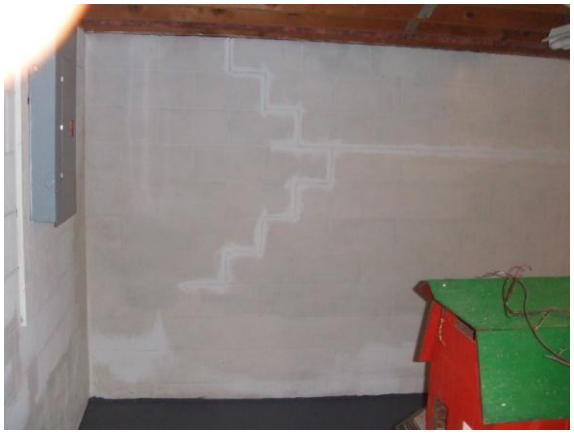

Fig. 3. Previous repairs visible on basement walls.

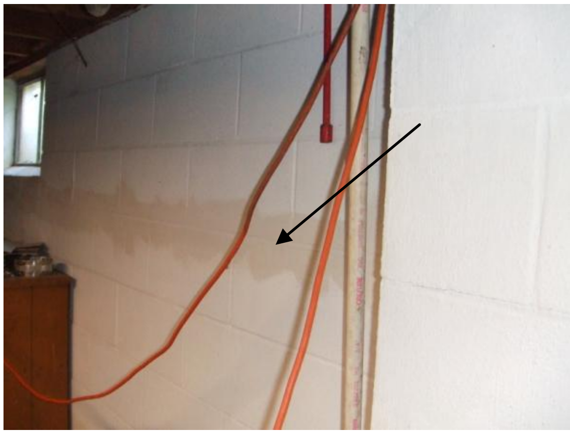

Fig. 4. Previous repairs visible (darker paint) on basement walls.

Blasting standards set forth in 805 KAR (Kentucky Administrative Regulations) 4:020 state that "…in all blasting operations…the maximum peak particle velocity of the ground motion in any direction shall not exceed two (2) inches per second at the immediate location of any dwelling house, public building, school, church, commercial or institutional building." Many states use the same 2 in/sec criterion. Shaking damage can occur even in cases where the ground motion caused by blasting does not exceed the 2 in/sec criterion. In cases where seismograph records have been provided in our investigation of claims, we note that most of these cases involved ground motions considerably less than 2 inches per second.

Strains caused by normal environmental factors must be considered as well. Stagg et al. (1984) noted that a 10°F change in temperature can cause the equivalent amount of strain within a wood frame structure as a ground vibration of 1.0 to 3.2 in/sec. They also noted that a 10 percent change in humidity can cause the equivalent amount of strain as a ground vibration of 1.0 to 2.4 in/sec, and that slamming the front door can cause the equivalent amount of strain as a ground vibration up to 1.9 in/sec.

The importance of seismograph records and pre-blast surveys cannot be overemphasized as valuable tools in the investigation of blasting claims. Seismograph records document the peak particle velocities (PPV) and accelerations (PPA) acting on a given structure by blasting operations. Where recorded PPV and PPA values are very high or very low, they can add confidence to the engineer's conclusions regarding the likelihood that blasting operations did or did not cause damages to structures. The instrument records velocity and acceleration in the longitudinal, transverse, and vertical directions (with respect to the orientation of the instrument), so that the magnitude of the PPV and PPA can be determined at the location of the seismograph.

Pre-blast surveys are indispensable tools in resolving blasting claims. A good pre-blast survey uses photographs and observations to thoroughly document the condition of a structure prior to blasting. The pre-blast survey can prove the prior existence of cracks and other distress that are being attributed to recent blasting activities, and also form a basis for structure owners to establish legitimate claims for damage that does occur from blasting activities. Dowding (1985) discusses the multiple natural causes of crack development within structures, and notes that any postblast inspection at low vibration levels is likely to find new cracks from natural aging unless a pre-blast survey is conducted immediately before blasting.

An important point to keep in mind is that the blast damage threshold of interior drywall in the above-grade level of a structure can be expected to be reached before the blast damage threshold of below-grade walls (even mortared CMU walls). This is because the effects of shaking are amplified in the above-grade portions of a structure, where there is no earth restraint to limit deflections, and where the construction materials (e.g., drywall) are weaker than the CMU walls. We have investigated claims of blast damage to concrete slab-on-grade driveways and patios. Siskind (2005) notes that vibrations cannot crack concrete or anything in or on the ground (such as driveways) unless peak particle velocities are very high, far above 2 inches per second. Cracks would not be expected to occur in driveways without extensive superstructure damage to the associated houses.

We investigated a blasting claim at a 5-year-old, single-story house supported over a crawl space. No pre-blast survey was performed, and no other claims were made by others in the vicinity. The homeowner first noted distress to the house during a period of reportedly heavy blasting for a roadcut at least 427 m (1400 ft) from the home. On the east side, where the exterior wall is 14.0 m (46.0 ft) long, little damage was observed, including three hairline drywall cracks, a jagged compression crack above a doorway (Fig. 5), and a discernable slope of 1.4 percent in the kitchen floor. On the west side, where the exterior wall is 18.3 m (60.2 ft) long, the homeowners had to lower the latch plates on the doors to get them to close. Gaps had formed in the hardwood floor near the front door and office. The wood floor had developed a discernable slope of 0.9 percent.

Fig. 5. View of jagged crack above doorway.

Fig. 6. View of wood beam in crawl space.

Fig. 7. View of wood beam support in crawl space.

Fig. 8. View of wood beam support in crawl space.

Review of the crawl space showed that the first floor joists were supported on two poorly constructed wood beams (Fig. 6). These beams were constructed by nailing together three 2x10 wood members. The joints between the center and outer members were staggered to provide some continuity to the beam. The beams were supported by wood columns or CMU pillars placed at or near the joints where the individual lengths of the outer 2x10s ended (Figs. 7 and 8). In addition, the average span of the first floor joists was about 6.1 m (20 feet). The southern wood beam was 11.9 m (39 feet) long, and was supported by two wood columns and one CMU pillar. The four sections of beam between the supports were sagging. Wood shims had been placed between the wood beams and the CMU pillars in order to level the beams during construction; the shims were crushed in one location.

We concluded that the distress to the house was related to the sagging of the floor support system caused by poor construction and support of the wood beams, and by excessive spans (up to 6.1 m [20 ft]) of the floor joists.

Another blasting claim included cracking in a CMU wall of a basement-level garage. The house was less than 40 years old. The cracking was stair-stepped through the mortar and showed about 1.6 mm (1/16 inch) of horizontal offset (Figs. 9 and 10). The homeowner said he had painted this wall about a year before our site visit, and that the crack had been caused by nearby blasting that had occurred about a month before our site visit.

A close inspection showed that paint spanned the two sides of the crack, and in some cases, filled the crack altogether (Figs. 11 and 12). Based on the homeowner’s report that he had painted the wall about a year prior to the site visit, we were able to conclude that the crack had not been caused by the recent blasting the month before, without our knowing anything else about the proximity or intensity of blasting.

Fig. 9. Stair-stepped cracking in CMU wall of basement-level garage.

Fig. 10. View of 1.6-mm (1/16-inch) horizontal offset in crack.

Fig. 11. View of year-old paint spanning sides of crack.

Fig. 12. View of year-old paint filling crack space.

We investigated a blasting claim at a 20-year-old, single-story-plus-basement residence that had been owned by the claimant for about 3 years. Blasting for a roadcut occurred over a span of 5 to 6 months at a distance of about 610 m (2,000 ft) from the house. The homeowner said that no pre-blast survey had been done.

The homeowner made a claim for damage to the CMU basement walls. No drywall or other damage was reported.

Review of the basement showed inward buckling of the front CMU basement wall that had been restrained by installation of steel tube columns (Figs. 13-16). These columns had been in place at the time of purchase three years before, and only occurred at locations where the wall had experienced buckling. Poor patchwork at the top of one of the columns showed it had been installed after the basement was finished (Fig. 14).

Fig. 13. Horizontal cracking behind steel tube column in garage.

Fig. 14. Evidence that column was installed after basement was finished.

We concluded that the intensity of shaking necessary to cause the basement CMU walls to buckle would have caused extensive damage to the upper floor of the house. Since the steel tube columns had been placed by a previous owner, and had only been placed where the wall was buckled, we concluded that the buckling of the CMU wall predated the blasting, and was likely the result of active earth and/or hydrostatic pressures acting laterally on the wall.

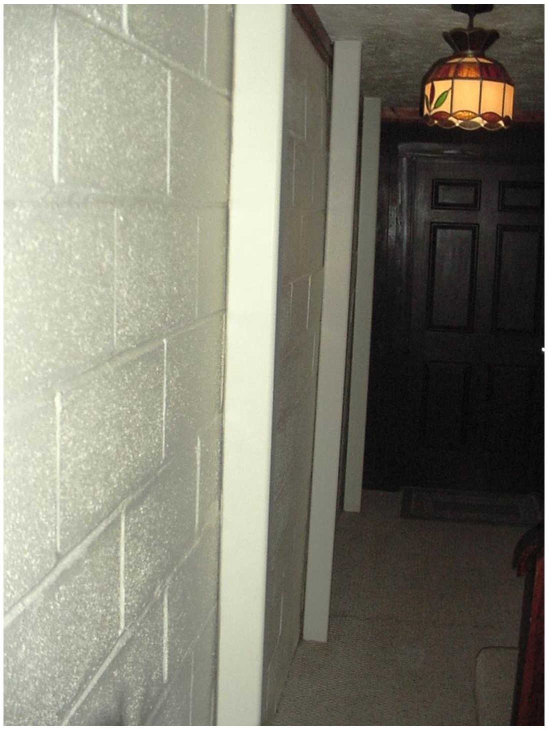

Fig. 15. View of inward buckling of wall restrained by steel columns.

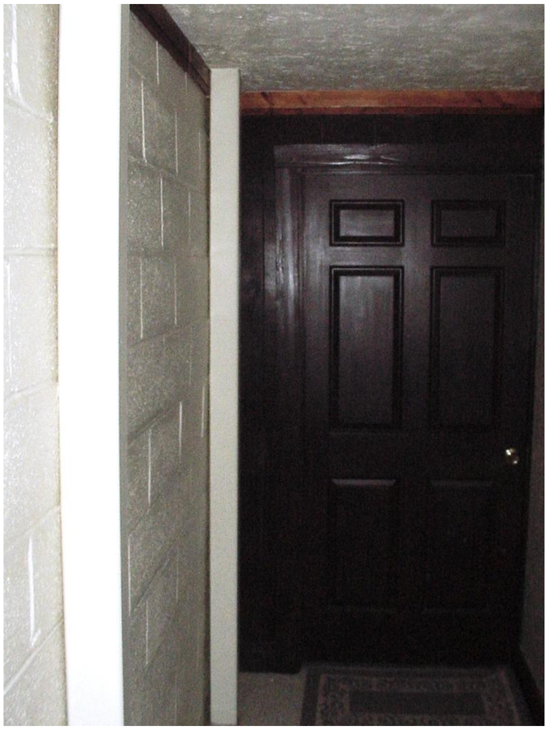

Fig. 16. Another view of inward wall buckling restrained by steel columns.

The value of seismograph records in reviewing blasting claims is illustrated in the case of roadcut blasting occurring over an 8-month period and within 91 m (300 ft) of a single-story, 12-year-old house. The blasting was done to remove bedrock composed roughly of 75 percent limestone and 25 percent shale. No pre-blast survey was performed, which is unusual for planned blasting activity occurring so close to a house.

Fig. 17. View of jagged crack at upper left window corner.

Fig. 18. Closeup of jagged crack shown in Fig. 19.

Fig. 19. View of jagged crack at lower right window corner.

Fig. 20. Extension of crack shown in Fig. 19.

The front wall of the house was oriented perpendicular to the direction of the road, so that the seismic waves generated by the blasting were essentially directed parallel to the front and rear walls and perpendicular to the side walls. The front door has a window on either side, one in a bedroom and the other in a living room. The living room window had a crack in the lower pane. The window frame was racked[1], with 2.5-mm (0.1-in) gaps at the lower right (south) and upper left (north) corners, looking out from the inside. A jagged, horizontal crack, 1.3 mm (0.05 in) wide, extended to the left and through the drywall from the upper left corner of the window (Fig. 17). Another jagged, horizontal crack, 2.5 mm (0.1 in) wide, extended to the right and through the drywall from the lower

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[1] Racking of a window or door frame occurs when differential movements cause the corners to lose their right angles, or "squareness". The frame assumes a parallelogram shape, as opposed to that of a rectangle.

right corner of the window (Figs. 18-21). These cracks exhibited lateral offsets of features on the upper and lower crack edges. The crack off of the bottom right corner turned downward at its south end, and exhibited compression damage across the crack.

Fig. 21. Closeup of crack shown in Fig. 20, showing horizontal offset and crushing of drywall caused by shear.

The bedroom door is set near the front wall of the house, adjacent to the front door of the house. The doorframe was racked, showing relative downward movement of the hinged side, which was nearest the front exterior wall. A slight vertical crimp was visible in the drywall above the door.

A single seismograph record provided for our review indicated a peak horizontal particle velocity of 1.20 in/sec and a peak horizontal particle acceleration of 1.26g, or 1.26 times the force of gravity. We concluded that the house had experienced damage caused by the blasting because 1) racking of the window, in combination with the jagged nature of the cracking and crushing damage across the cracks, suggested dynamic horizontal movement; 2) the house was supported on or near the limestone bedrock surface, which made static heave or settlement mechanisms unlikely; and 3) the likelihood that the short distance from the house to the blasting zone, in combination with the large peak horizontal particle acceleration, placed unique and unusual stresses on the house.

We investigated a blasting claim in which the blasting interval passed within 100 feet of the claimants' home. The blasting spanned two months, and the work included both a pre-blast survey and seismograph monitoring during blasting. The affected property was at the top of a roadcut that was being widened, and therefore the blasting occurred at a lower elevation than the house. This was critical because the blasting generated vertical as well as horizontal blast waves. A horizontal blast wave will shake the ground (and thus the house) back and forth, causing compression and shear. A vertical blast wave will lift the ground (and thus the house), and once the wave passes, the house will then react to the influence of gravity in free fall. If the structure is moving downward in momentary free fall while the next vertical wave is lifting the ground beneath it, the resulting strains can be intensified. Houses in this section of the country are typically not designed for dynamic vertical movements.

Over the 2-month blasting period, the seismograph records documented peak horizontal velocities as high as 2.1 in/sec; peak vertical velocities as high as 1.6 in/sec; peak horizontal ground accelerations as high as 1.8g; and peak vertical ground accelerations as high as 1.4g. These motions caused extensive damage to a brick planter on the back patio (Figs. 22-24), as well as some shearing damage to drywall inside the house (Figs. 25-28). The drywall cracks exhibited jagged surfaces, horizontal offsets, and compression damage.

Fig. 22. Pre-blast photo of brick planter.

Fig. 23. Post-blast photo of brick planter.

Fig. 24. Post-blast photo of brick planter. Roadcut being widened is between the wall and the house in the background.

We concluded that the damage to this residence was caused by blasting on the basis of 1) the substantial horizontal and vertical ground motions recorded by seismograph at and near the home, 2) the jagged shear cracks that showed evidence of shearing and compression damage across the sides of the cracks, 3) the destruction of the brick planter, which was not likely due to static mechanisms, and 4) comparison of pre-blast and post-blast photographs. Although the pre-blast photographs did not show the entire length of the planter, we concluded that the pre-blast documenter would have taken more extensive pictures of the planter if it had exhibited any form of distress or damage prior to blasting.

Fig. 25. Jagged horizontal crack in interior stairwell.

Fig. 26. Jagged crack near ceiling between stairwell and entry to second-floor bedroom.

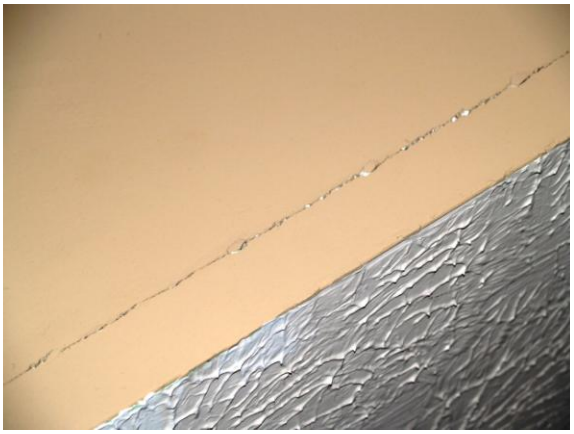

Fig. 27. Jagged crack at ceiling of second-floor dormer, showing lateral offsets.

Fig. 28. Jagged crack at ceiling of second-floor dormer, showing lateral offsets.

Weather Claims

Thus far, we have established the importance in claims review of not only understanding what other mechanisms may have acted to cause the reported damage, but of also understanding whether or not the damage may predate the reported date of loss. This is also important in the review of weather-related claims.

We investigated a claim for damage to a farmhouse by a tornado that destroyed a barn and outbuildings less than 43 m (140 ft) away (Fig. 29). The farmhouse was a single-story, wood-frame structure with a brick façade and a partial basement. The age of the house was unknown; it had been purchased by the homeowner in about 1973. The tornado that destroyed the barn and outbuildings also tore eaves, shutters, and vent covers off of the house.

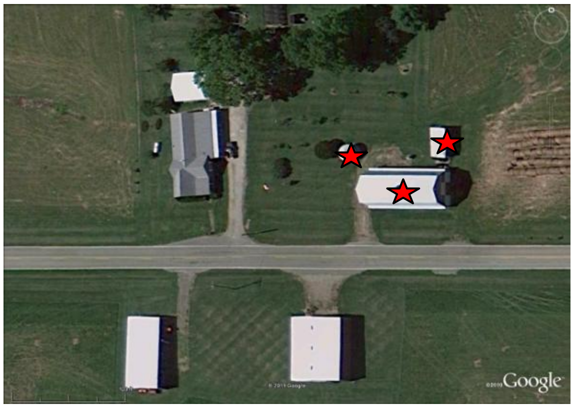

Fig. 29. Aerial view of farm. Buildings marked by stars destroyed by tornado. West (left) wall of large barn about 43 m (140 ft) from house.

Fig. 30. West wall of house. Heaviest damage to brick façade occurs over partial basement (see window well).

Fig. 31. Previously-caulked damage over upper left corner of picture window. Subsequent movement has squeezed caulk out of cracks.

Fig. 32. Previously-caulked damage beneath picture window.

The exterior brick façade contained numerous cracks, many of which exhibited previous caulking and tuckpointing. The wall with the greatest amount of cracking to the brick façade was the west wall, which also receives the brunt of storms that move from west to east (Figs. 30-32). The wall with the least amount of cracking was the east wall. A large amount of cracking occurred above the window corners, where the rusted ends of the steel window lintels were exposed (Fig. 33). The inside drywall on the first floor could not be observed for damage because the house had been completely paneled.

Review of the partial basement showed CMU construction of the basement walls. Water damage was visible on the inside of the west basement wall, and inward bowing of this wall was evident (Fig. 34).

Fig. 33. Horizontal cracking caused by expansion of rusting steel lintel at top of door frame.

Fig. 34. Water damage visible to west basement CMU wall. This wall has bowed inwards under active earth and hydrostatic pressures.

Based on the amount of patching and caulking visible on the brick façade, we concluded that the façade had experienced much damage over the time preceding the tornado strike. The greatest amount of damage to the west exterior wall was largely in response to the inward bowing of the CMU basement wall, likely under the influence of active earth and hydrostatic pressures. We concluded that much of the caulked and uncaulked cracking above the window and door corners was related to rusting and expansion of their steel lintels. When the steel lintels rust, they expand in volume, causing separation and vertical movements at horizontal mortar joints. These gaps commonly propagate upwards as stair-stepped cracks. Some of the cracks had extended themselves since they had last been caulked or patched. Given the tendency of brick and mortar cracks to propagate with time, especially during the ongoing process of rusting of steel window and door lintels, there is a strong possibility that the uncaulked brick cracks were already present at the time of the tornado strike. We were unable to determine or estimate the probability that they were extended further by stresses caused by the tornado, but were able to assign other causes to them besides the tornado.

Ground-Related Claims

In general, structural distress caused by static, ground-related mechanisms such as soil settlement under loading or vertical movements of moisture-sensitive soils are not covered under homeowners' insurance policies. It is useful to summarize the nature and causes of such movements.

Soil settlement can be caused by supporting a structure over compressible soils. If these soils are in a very moist and softened condition at the time of construction, the new construction loading can induce compression of the soils and subsequent settlement of the structure from the time the construction loads are applied. If the compressible soils are in a relatively dry condition at the time of construction, they may be capable of supporting construction loads as long as they remain relatively dry. However, if those soils gain access to water some time after construction, due to causes such as utility leaks, above-normal precipitation, or changes in grading and drainage caused by nearby development, they can then soften and compress under load, causing settlement some time after construction.

Upward or downward vertical movements can also be caused by seasonal hydration or dehydration of moisture-sensitive soils. These soils are in great abundance in the Greater Cincinnati Area and in other parts of Kentucky, and exhibit the ability to increase in volume when exposed to moisture and to decrease in volume when they lose moisture. Mature trees are capable of causing significant seasonal fluctuations in the moisture contents of these soils, and to depths of several feet below the ground surface. During drought seasons of the year when the trees are actively extracting moisture from the ground, the direction of vertical movement is downward. During the wetter winter and early spring months, when the trees are dormant and not extracting moisture, the soils rehydrate and expand; the resulting direction of vertical movement is upward. If the feeder roots manage to extend themselves close to or beneath the house foundations, the seasonal effects become more severe. Monthly weather summaries made available online by the National Weather Service are valuable for establishing precipitation trends over time in cases where distress may have been caused by seasonal moisture extraction by mature trees.

Soil settlement under load, or downward vertical movement caused by moisture extraction, can cause cracking to develop in brick façades or drywall that is routinely interpreted by homeowners as being caused by blasting or earthquake damage. In many cases, the movements and subsequent cracking occur years or decades after construction of the house, due to the time it takes for the soils to become wetted or for the tree to reach maturity and vastly extend its root system. The years- or decades-long delay in the start of distress to the house, after a long period of suitable performance, is confusing to homeowners and insurance adjusters alike.

In general, if a mature tree is close enough to the house to allow its canopy to reach the roofline, it can induce seasonal movements of moisture-sensitive soils. We have been told by a certified arborist that certain species of trees can extend their feeder roots out to distances of two to three canopy diameters.

We investigated a unique case of significant distress to a single-story-plus basement house that was at least 36 years old. The basement floor slab was buckled out of level in various directions. The front basement CMU wall had been buckled into the interior space, and the drop ceilings had buckled (Figs. 35 and 36). Along the back wall of the house, lateral offsets in the brick façade near the base of the wall showed that the first floor of the house had been thrust to the rear by almost 41 mm (1.6 inches) (Fig. 37). A large horizontal gap in a brick mortar joint on the rear wall had opened up where the rear wall had buckled outward (Fig. 38).

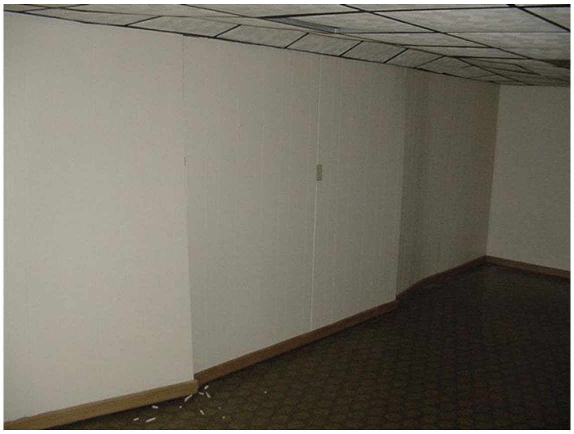

Fig. 35. View of buckled basement walls and drop ceiling.

Fig. 36. Buckled basement wall caused buckling of kitchen countertop.

Fig. 37. Horizontal offset in brick on rear wall caused by thrusting of first floor to the rear.

Fig. 38. Buckling of brick façade on rear wall caused by thrusting of first floor to the rear.

The homeowner had filed a claim for earthquake-related damage many years before our investigation. The claim had reportedly been denied.

A published geologic map indicated that the surficial bedrock at the house location was a fissile to chunky, thin bedded, expansive clay shale that "expands and becomes very plastic when wet", and that was an "extremely poor foundation material". The geologic map indicated that the shale had been shown to exhibit swell pressures as high as 4,100 pounds per square foot, and that it "requires special attention for engineering purposes".

We recognized the possibility that the basement had been excavated into this expansive shale, and the possibility that the basement walls may have been backfilled with the excavated shale materials. Our report concluded that it was likely that the buckling of the CMU walls and the concrete floor slab were related to wetting and expansion of the shale. The ground surface was depressed where the basement walls had buckled in (Fig. 39), which exacerbated the problem by directing more surface runoff towards the house. Because repair work had been previously done on the basement walls, we also concluded that the basement backfill zone might have been wider than usual. We concluded that lateral pressures caused by the deflecting front foundation wall were thrusting the first floor joists to the rear, which in turn deflected the rear wall outward and damaged the brick façade (Figs. 37 through 40). The lateral pressure also buckled the suspended basement ceiling, and likely the floor slab as well. The first floor appeared to be moving laterally on the floor diaphragm with minimal visible damage to the first floor interior.

Fig. 39. Depression in ground surface resulting from inward buckling of front basement wall.

Fig. 40. Short front porch wall has stayed in place while front porch slab is dragged to the rear along with the first floor.

The slab-on-grade garage had experienced settlement as well (Figs. 41 and 42). A mature tree was located just off the corner of the garage, which was also surrounded by many shrubs. The vegetation had induced settlement of the structure through seasonal moisture extraction. Figs. 34 and 35 are from photographs taken on July 23, 2003.

The resolution of the case is interesting. The homeowner's policy excluded coverage for damage caused by earth movement, but endorsed coverage for damage caused "directly or indirectly" by water. The principal author indicated that a possible conflict existed, since 1) swelling could be considered “earth movement”, but that 2) the floor and wall heave caused by the swelling were indirectly caused by water. The insurance company concluded that the basement damage was covered under the homeowner’s policy. The first floor of the house was jacked up and off of the basement walls, after which the basement walls and floor were removed and rebuilt.

The conclusion of this particular case points out the wisdom of having the insurance adjuster research the homeowner’s policy to determine whether ground-related causes of distress to the home are excluded or are endorsed under certain circumstances.

Fig. 41. View of front wall near junction of basemented (to left) and non-basemented (to right) portions of the house.

Fig. 42. Settlement of slab-on-grade garage corner caused by seasonal moisture extraction from mature tree off left side of photo.

Conclusions

We have discussed insurance claims for damage or distress to structures related to earthquake, blasting, weather, and ground-related causes. It is important to understand the following points when investigating insurance claims attributed to these causes:

1. Home and property owners generally do not look for cracks or other distress in their homes until they believe they have a reason to do so, such as following an earthquake, blasting, or severe weather event. Once a crack is found that had not been noticed previously, the home or property owner will tend to look carefully for more cracks or distress, and may find features that had also escaped previous notice.

2. Other claims are made for damage to homes in cases where neither the homeowner nor the adjuster has a good idea what the cause might be. Understanding the common causes and telltale signs of residential structure distress can assist in deciphering the available evidence and determining the

validity of claims made for seismic, blasting, and severe weather events as well as other ground-related causes.

3. Within the spectrum of possible shaking damage (either from earthquake or blasting), the damage threshold of interior plaster in the upper levels of a structure can be expected to be among the lowest. This is because the effects of shaking are amplified in the above-grade portions of a structure, where there is no earth restraint to limit deflections, and where the construction materials (e.g., plaster) are weaker than the CMU in foundation walls.

4. Shaking damage can occur even in cases where the ground motions caused by blasting do not exceed the 2 inch-per-second criterion held as a standard by many states.

5. Strains in construction members caused by normal environmental factors such as temperature and humidity changes can equal or exceed those caused by ground motions exceeding the 2-inch-per-second criterion. There are multiple natural causes of crack development within structures; any post-blasting or post-earthquake inspection at low vibration levels is likely to find new cracks from natural aging.

6. We cannot overemphasize the importance of seismograph records and pre-blast surveys as valuable tools in the investigation of blasting claims. Seismograph records document the peak particle velocities (PPV) and accelerations (PPA) acting on a given structure by blasting operations. A good pre-blast survey can prove the prior existence of cracks and other distress that are being attributed to recent blasting activities, and also form a basis for structure owners to establish legitimate claims for damage that does occur from blasting activities.

7. Monthly weather summaries made available online by the National Weather Service are valuable for establishing precipitation trends over time in cases where distress may have been caused by seasonal moisture extraction by mature trees.

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Temporary Earth Retention – Project Performance East Bank at the Flats, Cleveland, Ohio

Frederick W. Slack, PE, M. ASCE [1]

Abstract: The performance of an earth retention system is impacted by several factors. Certainly, the design parameters selected will have an impact. However, the methods of installation, the conditions in which the earth retention system was installed, the duration of use of the system and the subsequent activities planned after the system has been installed could have an impact on the overall performance of the earth retention system.

This presentation will focus on a project in Cleveland, Ohio that experienced larger than expected movements in an anchored soldier pile and wood lagging temporary earth retention system. A brief review of the project requirements and the design methods used will be presented. The magnitude and cause of the movements will be examined. The methods taken to halt the movement and prevent future will also be discussed.

Project Information and Location

The project is located on the west side of downtown Cleveland, OH, along the east bank of the Cuyahoga River in an area called “The Flats”. Once an industrial area, this neighborhood has evolved into a business and entertainment area. This specific project encompasses one building of a multi-building project. This building is a high-rise office tower and parking garage. The site is located behind buildings that front on West 9th street, between Front Avenue to the north and Main Avenue to the south. (Refer to Fig. 1).

The site is located over the west slope of a deep pre-glacial valley that was eroded into the bedrock. This ancient valley was then filled with alluvium and lacustrine deposits. Existing grade varied between 580 and 620. Rock elevations varied from 430 to 460. Generally speaking, the site stratification can be described as man-made fill (4’ to 15’ from grade); alluvial deposits of sand and silt (15’ to 30’ from grade); lacustrine deposits of soft to medium stiff clay and silt layers (30’ to 80’); glacial till consisting of silt, sand and clay (80’ to rock at +/- 135’). A typical soil boring is shown in Fig. 2.

A three sided temporary earth retention system was needed to allow the building excavation to proceed. Original bottom of excavation was set at elevation 580. The grade at Front Avenue to the north varied from approximately elevation 595 to 605, west to east. Grade behind the buildings on West 9th varied from approximately 605 to 620, north to south. Along Main Avenue, grade sloped from elevation 620 to 590 east to west. Fig. 3a shows a plan view of the earth retention system.

The project Geotechnical report was dated January 30, 2008. Richard Goettle, Inc. (RGI) bid on the project in June 2008 and was awarded the work for the temporary earth

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retention system in July 2008. You may recall that this period of time was quite volatile economically. Projects started and stopped, funds were available and then they were withdrawn. Of course, this project was going to be different and the work started in late July 2008. Work progressed through August and into September. A monitoring system had been established at the beginning of the project and on September 9th, wall movements increased and a crack appeared in the parking lot behind the buildings along West 9th Street. The monitoring showed continued movement to the point at which a berm was constructed in front of the wall on September 17th to arrest the movement. Obviously, the owner and neighbors were concerned. At this point, an investigation into the cause of the movement was started and a remediation plan was developed. Remediation of these problems will be addressed in the following sections.

After the movement was addressed and stopped, so did the project. At the end of October 2008, the project was placed on hold. Economic conditions and tenant difficulties led the developer to stop work on the project.

In January 2011, we were awarded a new contract to modify and add more earth retention to that already in place, as shown in Fig. 3b. During the shutdown period, the building and garage foundations were modified and additional excavation was needed and portions of the existing earth retention needed to be modified. Work on the new contract started in mid January 2011 and was completed at the end of March 2011. Heavy spring rains, poor site grading and maintenance led to a small portion of the new work experiencing movements similar to the original project movements. A plan of action was developed and the remediation of the new movement was installed in early May 2011.

As of this writing, the project's foundation is installed and the project is proceeding without further difficulties.

Earth Retention System Design

The earth retention system was designed using tried and proven methods used on previous projects in downtown Cleveland. Surcharge pressures were added where necessary. The system selected as most appropriate was an anchored soldier pile and wood lagging wall. Ground water and/or dewatering were not issues. Soldier pile and lagging systems have performed well on previous projects and are a cost effective system for the correct conditions. A recap of the design parameters are shown in Figs. 4a and 4b. The design was submitted for approval in early July and accepted.

Soldier piles were installed with a diesel pile hammer. The piles were spaced at 8'-0" to 10'-0" intervals along the perimeter. Soldier pile toe embedment varied from 6 to 10 feet. Three inch thick untreated hardwood lagging was used to protect the retained earthen face. To anchor the wall in the soils present, an augured typed anchor was selected. The anchors were inclined at 20o to 25o to accommodate the anchor installation and to clear the utilities found around the perimeter of the site. The anchors were 14" in diameter, installed with a continuously flighted auger. A high strength bar was inserted in the fresh structural grout to complete the anchor. This type of anchor looks very much like an augercast pile being installed, only nearly horizontal.

In late July, the bottom of excavation was lowered by nearly 3 feet. To accommodate this change, an additional row of anchors was added to compensate for the loss of toe support in the soldier piles.

Wall Movement Investigation and Remediation

The monitoring system that was established for the project consisted of optical measurements at specific points checked at regular intervals. Once the movement concerns were heightened, the frequency of measurement increased, up to daily readings. As a team, the owner, general contractor, geotechnical engineer and RGI decided that the installation of inclinometers would be useful, specifically to rule out global stability problems. The inclinometers were installed in early September 2008. Readings indicated that the movements were not related to global instability, but rather a movement of the soil wedge behind the wall. Wall movement at the top of the soldier pile in the worst area was typically 2”, up to a localized worst reading of 4”. The lateral and vertical movements at the top of the pile were approximately the same. The question now was why did the wedge move and how can the movement be stopped?

As the data was examined, a pattern of first vertical downward movement and then a responsive lateral movement was detected. The movements always occurred in this repetitive cycle. It was determined that the soldier piles were being drawn downward by the vertical component of the anchors. As the piles and walers moved downward, the stressing length of the tensioned anchors was shortened, allowing the anchor to lose load. The loss of anchor load allowed to soldier pile to move laterally. Once the pile moved laterally, it re-engaged the anchor, the anchor picked up load (and vertical component) and the process repeated itself.

This type of movement has happened in the past; however, it usually occurs for only one cycle of movement. The movement cycles on this project kept repeating until movements become intolerable. It was clear to RGI, that if the vertical movement of the soldier piles could be stopped, the lateral movements would also stop.

This being the case, a system of support piles was installed to act as vertical support elements for the soldier piles. Walers and brackets connected the soldier piles to the added support piles. Since a stabilizing berm was added to the front of the wall, the support piles were installed from the top of the berm; at a point about one-half way up the full wall height. The supplemental support system was installed from mid September to the end of October 2008.

It should be noted that while the movement that led to the installation of the stabilizing berm and support system occurred behind the buildings along West 9th Street, a similar pattern of movement was starting to develop along Main Avenue. To be proactive and since we were at the site and had the necessary equipment, it was decided to install a support system along Main Avenue. Figs. 5a and 5b show an elevation view of the West 9th Street wall remediation and a typical section showing the various elements of the remediation system.

The movements that occurred on the work performed in 2011 were also created by loss of vertical support and were remediated by stopping the vertical movement of the soldier pile. However, because of space limitations and site specifics, the details used to halt the movement were different. Figs. 6a and 6B show the methods used for the 2011 project. In both instances, the object was to stop the vertical movement of the soldier piles. The resulting details are quite different. This difference was dictated by the conditions present at the site when the remedial measures were constructed.

Conclusions

The fact that an earth retention system moved is not at all unusual. The amount of movement in this instance and the rate at which it took place demanded that some type of remedial action was necessary.

For the 2008 project, the loss of toe support was aggravated by the lowering of the excavation by nearly 3 feet. This action led to a reduced toe capacity (less pile embedment) and additional vertical load (the additional row of anchors to compensate for the loss of toe lateral support) – a literal double whammy. Also, the decision to drive the piles in lieu of drilling the piles in place had an impact on the available vertical support provided by the soldier pile embedment.

The second project at the site in 2011 was a different matter. Several contributing factors were involved in this instance:

- The new piles for this project were drilled into place, which greatly increased the vertical capacity provided by the toe. The greatest movement occurred in the original piles re-used in the new project.

- The time that elapsed between the two projects amounted to nearly 27 months (November 2008 to mid January 2011). In that time period, the owner did not engage a contractor to be responsible for maintaining the site in a manner that would keep the bottom of excavation drained or direct the surface water from behind the wall. The hard northern Ohio winter in 2010/11 and the heavy spring rains in 2011 compounded the problems that occurred.

- It is important for an owner/developer to realize that if a project is going to be mothballed for over 2 years, some level of maintenance is required to keep the site conditions under control so that existing construction previously installed is not impacted.

Construction contacting has its risks. A contractor that works below grade in soil and rock has the typical contracting risk plus the additional risk the unseen subsurface materials can provide - it is true that Mother Nature can be a cruel lady at times. There are times that it is necessary to step up and fix problems. It should be remembered, however, that there are simple steps that can be taken by other parties that will help mitigate problems.

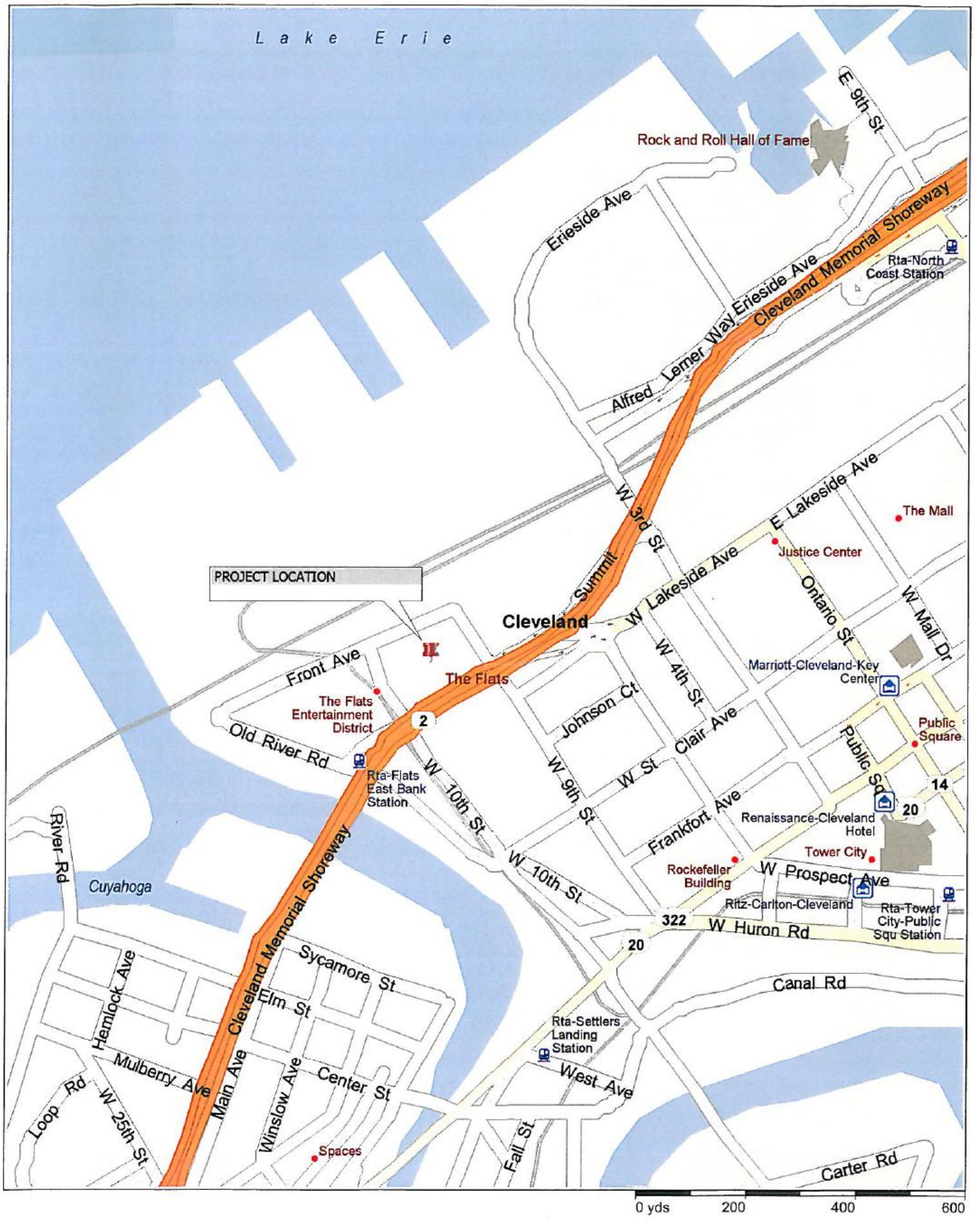
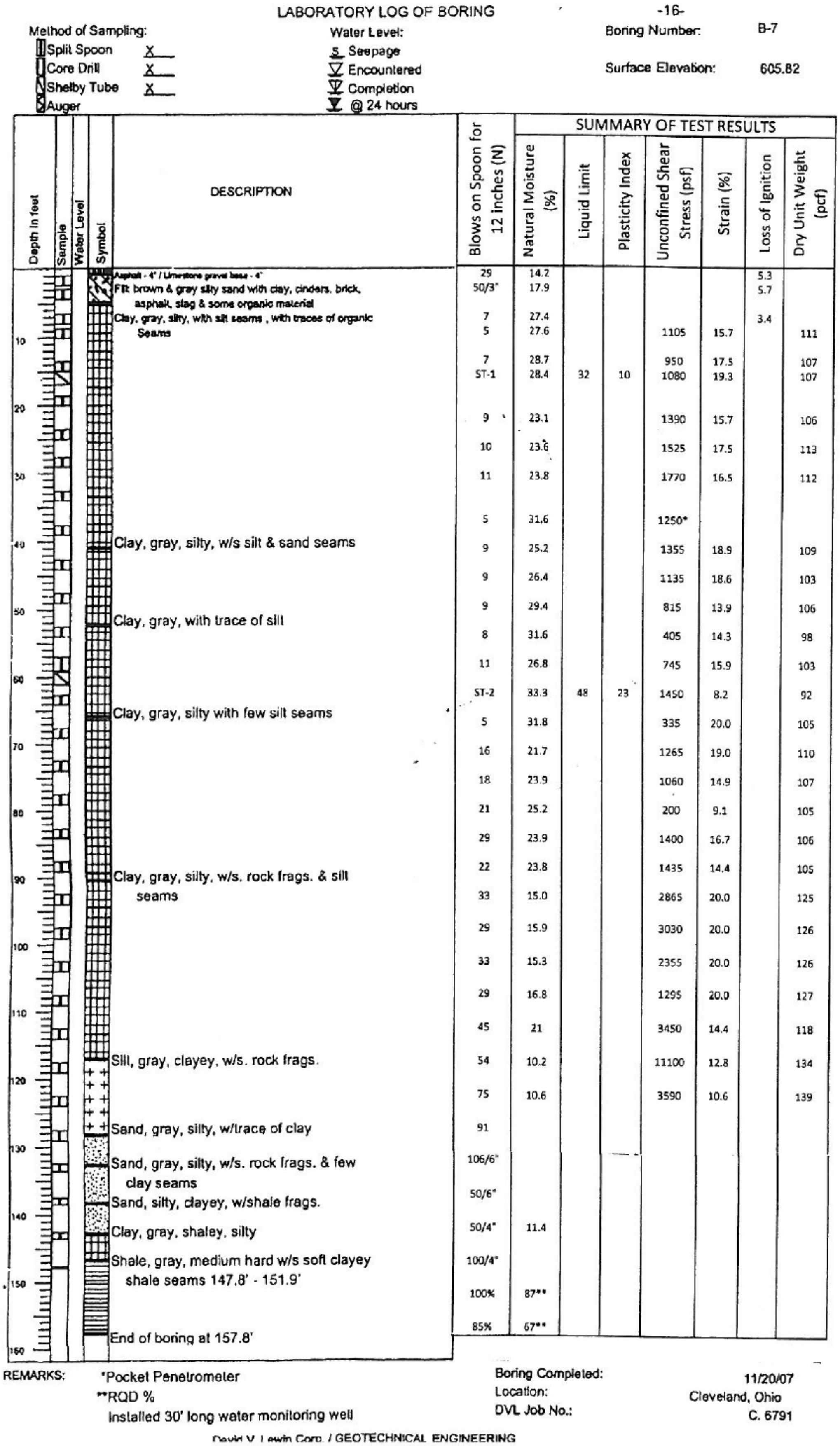


Fig. 1. Project site map



LABORATORY LOG OF BORING

-16-

Method of Sampling:
Split Spoon X
Core Drill X
Shelby Tube X
Auger

Water Level:
S Seepage
Encountered
Completion
@ 24 hours

Boring Number: B-7

Surface Elevation: 605.82

Depth in feet	Sample	Water Level	Symbol	DESCRIPTION	Blows on Spoon for 12 inches (N)	SUMMARY OF TEST RESULTS Natural Moisture (%)	Liquid Limit	Plasticity Index	Unconfined Shear Stress (psf)	Strain (%)	Loss of Ignition	Dry Unit Weight (pcf)
				Asphalt - 4" / Limestone gravel base - 4"	29	14.2					5.3	
				Fill: brown & gray silty sand with clay, cinders, brick, asphalt, slag & some organic material	50/3"	17.9					5.7	
				Clay, gray, silty, with silt seams , with traces of organic Seams	7	27.4					3.4	
10					5	27.6			1105	15.7		111
					7	28.7			950	17.5		107
					ST-1	28.4	32	10	1080	19.3		107
20					9	23.1			1390	15.7		106
					10	23.6			1525	17.5		113
30					11	23.8			1770	16.5		112
					5	31.6			1250*			
40				Clay, grey, silty, w/s silt & sand seams	9	25.2			1355	18.9		109
					9	26.4			1135	18.6		103
50					9	29.4			815	13.9		106
				Clay, gray, with trace of silt								
					8	31.6			405	14.3		98
					11	26.8			745	15.9		103
60					ST-2	33.3	48	23	1450	8.2		92
				Clay, gray, silty with few silt seams								
					5	31.8			335	20.0		105
70					16	21.7			1265	19.0		110
					18	23.9			1060	14.9		107
80					21	25.2			200	9.1		105
					29	23.9			1400	16.7		106
					22	23.8			1435	14.4		105
90				Clay, gray, silty, w/s. rock frags. & silt seams								
					33	15.0			2865	20.0		125
					29	15.9			3030	20.0		126
100					33	15.3			2355	20.0		126
					29	16.8			1295	20.0		127
110					45	21			3450	14.4		118
				Silt, gray, clayey, w/s. rock frags.	54	10.2			11100	12.8		134
120					75	10.6			3590	10.6		139
				Sand, gray, silty, w/trace of clay	91							
130				Sand, gray, silty, w/s. rock frags. & few clay seams	106/6"							
				Sand, silty, clayey, w/shale frags.	50/6"							
140				Clay, gray, shaley, silty	50/4"	11.4						
				Shale, gray, medium hard w/s soft clayey shale seams 147.8' - 151.9'	100/4"							
150					100%	87**						
					85%	67**						
160				End of boring at 157.8'								

REMARKS: *Pocket Penetrometer
**RQD %
Installed 30' long water monitoring well

Boring Completed: 11/20/07
Location: Cleveland, Ohio
DVL Job No.: C. 6791

David V. Lewin Corp. / GEOTECHNICAL ENGINEERING

Fig. 2. Typical boring log

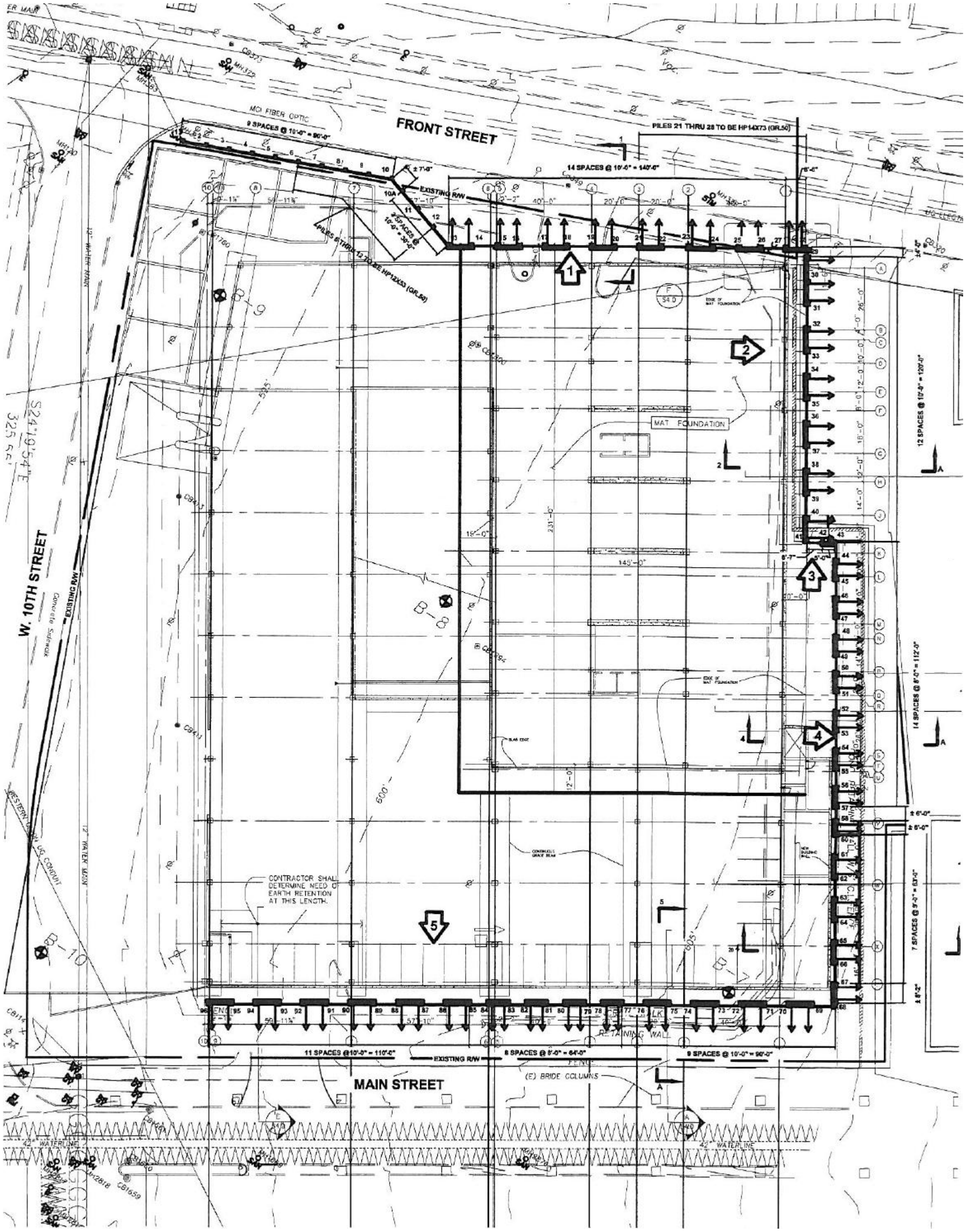


Fig. 3a. Plan View - 2008 Earth Retention System

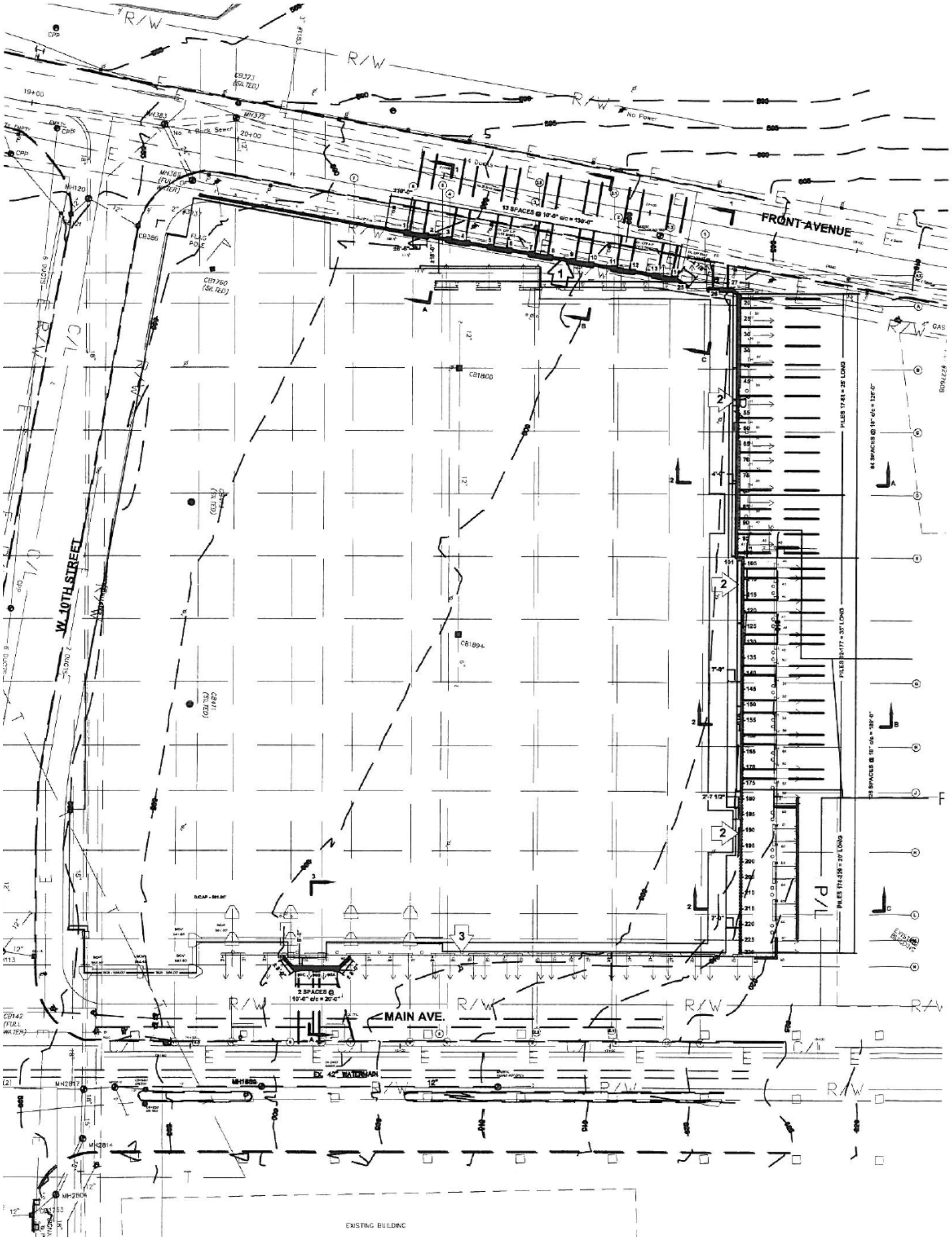


Fig. 3b. Plan View - 2011 Earth Retention System

Retained Material - Fill, Silt and Clay $5 < n < 29; n_{avg} = 10$

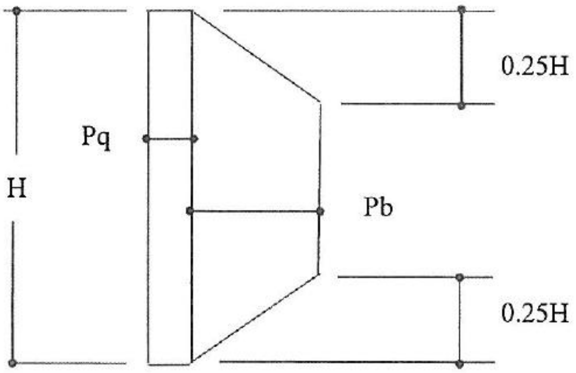
Toe Material - Silt and Clay $5 < n < 18; n_{avg} = 10$

Silty CLAY, SILT
5<n<29; avg "n" value = 10 (Retained)
5<n<18; avg "n" value = 10 (Toe)
Use c = 1,250 psf, g = 130 pcf

TABLE 1-2 SHEAR STRENGTH of COHESIVE SOILS

Consistency	*Very Soft*	*Soft*	*Medium*	*Stiff*	*Very Stiff*	*Hard*
q_u *= unconfined compression strength tons per sq. ft.*	0	0.25	0.50	1.00	2.00	4.00
Standard penetration resistance, N = no. of blows per foot	0	2	4	8	16	32
Unit weight, pcf (saturated)	100 - 120		110 - 130	120 - 140		130+
Identification characteristics	*Exudes from between fingers when squeezed in hand*	*Molded by light finger pressure*	*Molded by strong finger pressure*	*Indented by thumb*	*Indented by thumbnail*	*Difficult to indent with thumbnail*

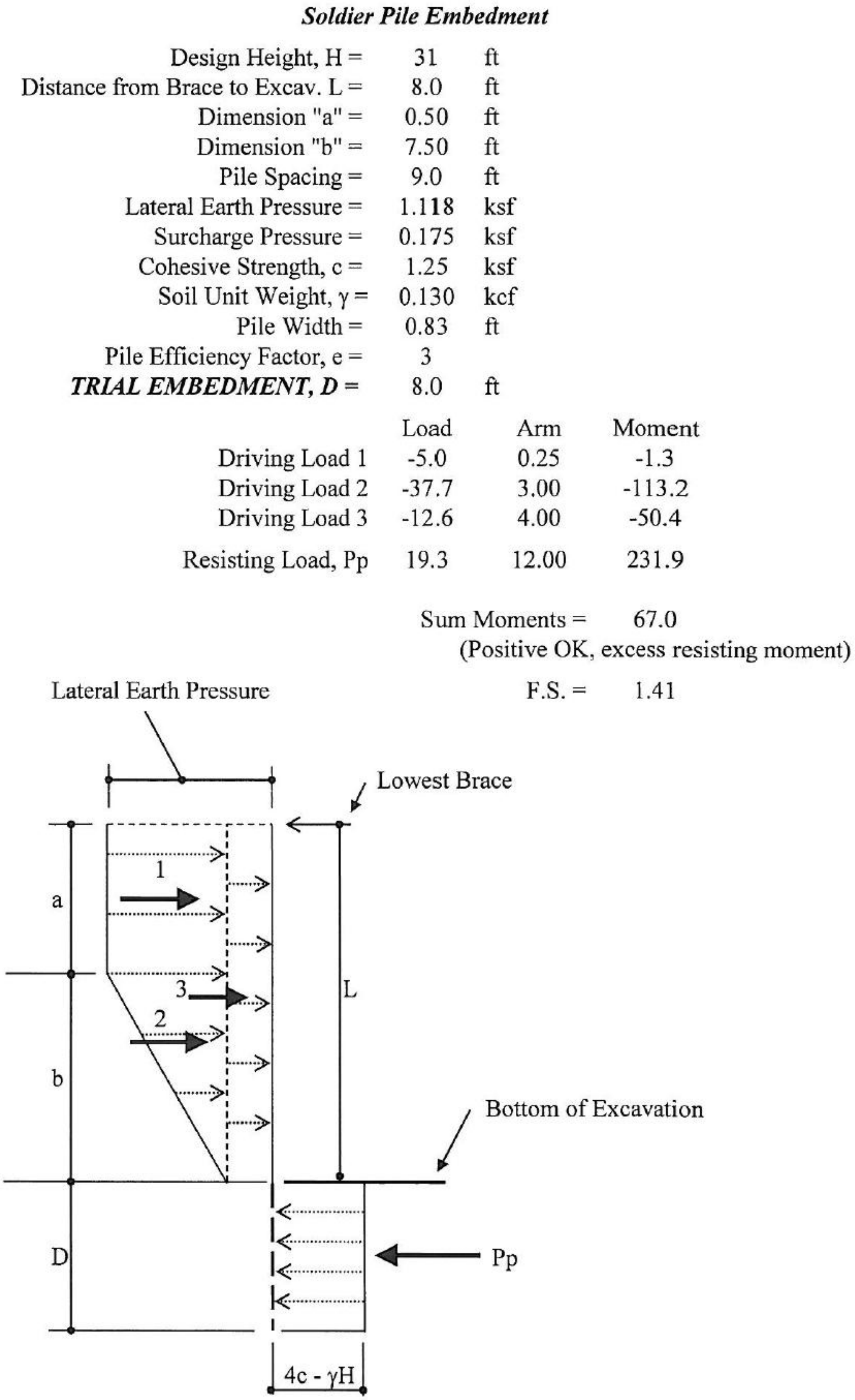
Reference: Foundation Design; Wayne C. Teng, pages 12 and 15



Pq = Surcharge Pressure, as applicable

Pb = Lateral Earth Pressure = 0.2 γ H = 26H

Fig. 4a. Soil properties and earth pressures



Soldier Pile Embedment

Design Height, H = 31 ft
Distance from Brace to Excav. L = 8.0 ft
Dimension "a" = 0.50 ft
Dimension "b" = 7.50 ft
Pile Spacing = 9.0 ft
Lateral Earth Pressure = 1.118 ksf
Surcharge Pressure = 0.175 ksf
Cohesive Strength, c = 1.25 ksf
Soil Unit Weight, γ = 0.130 kcf
Pile Width = 0.83 ft
Pile Efficiency Factor, e = 3
TRIAL EMBEDMENT, D = 8.0 ft

	Load	Arm	Moment
Driving Load 1	-5.0	0.25	-1.3
Driving Load 2	-37.7	3.00	-113.2
Driving Load 3	-12.6	4.00	-50.4
Resisting Load, Pp	19.3	12.00	231.9

Sum Moments = 67.0
(Positive OK, excess resisting moment)

F.S. = 1.41

Fig. 4b. Soldier pile embedment design

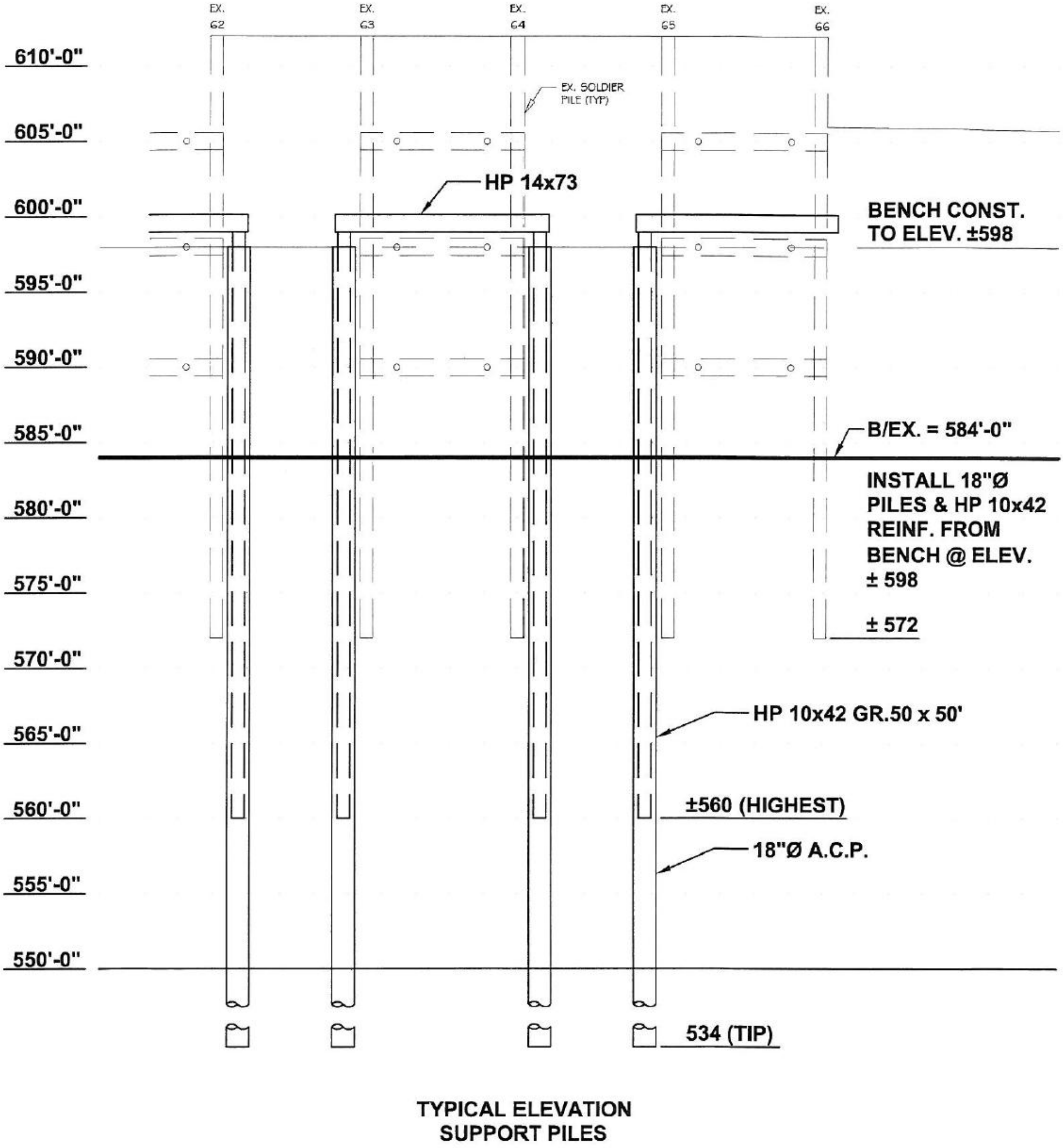


Fig. 5a. 2008 Remediation Elevation

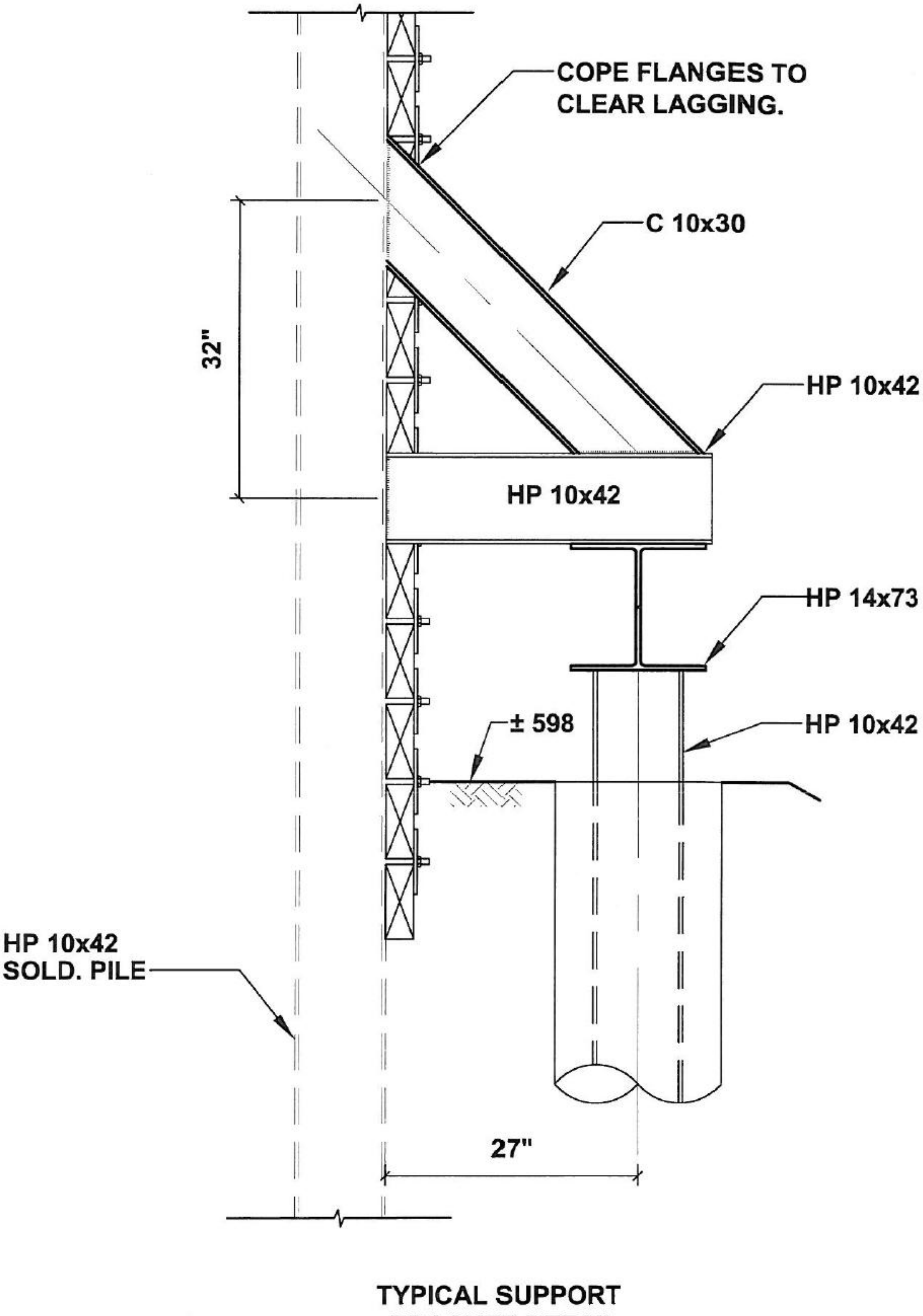


Fig. 5b. 2008 Remediation Section

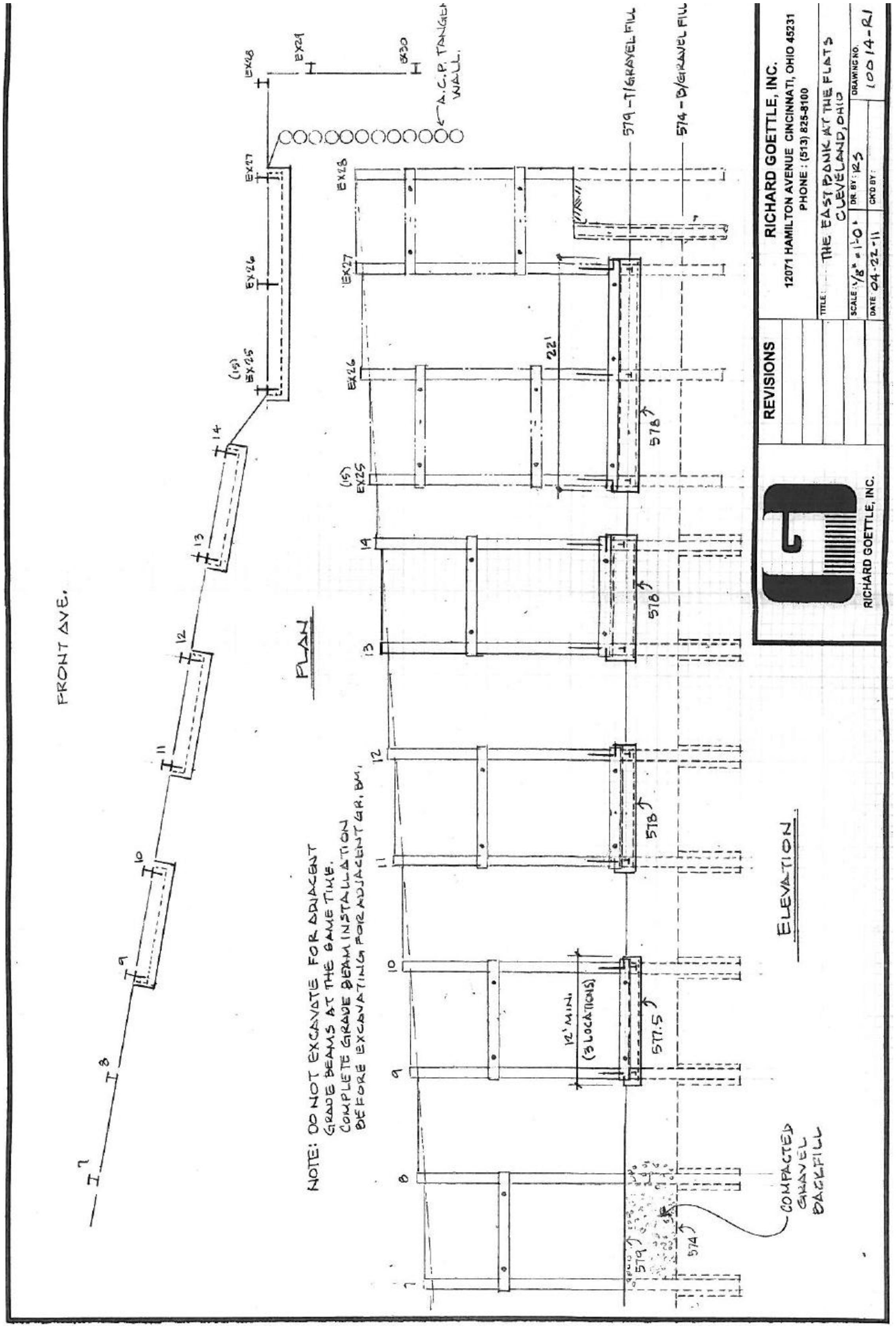


Fig. 6a. 2011 Remediation Elevation

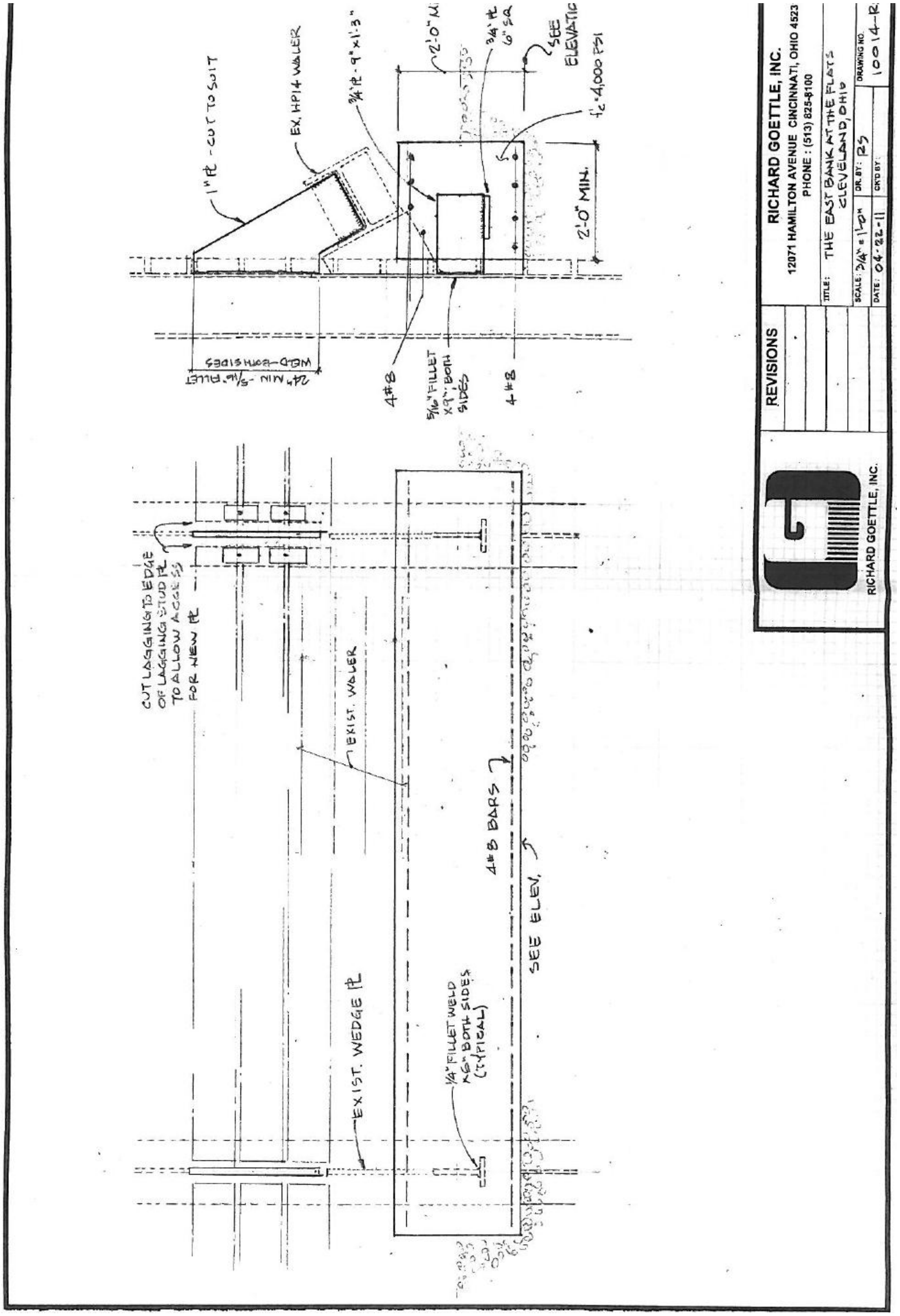


Fig. 6b. 2011 Remediation Section

Consolidation Analyses of Greater Cincinnati Soils

Nishant Dayal, P.E., M. ASCE[1] and Dr. Mark T. Bowers, Ph.D., P.E., M. ASCE[2]

Abstract: Single and multiple regression analyses were carried out on data obtained from 81 consolidation tests that were conducted on undisturbed samples. The samples are specific to the Greater Cincinnati and Northern Kentucky area. Several empirical correlations between compression index and single and multiple soil properties are proposed. The proposed correlations are also compared with existing empirical equations.

Six case studies are presented where consolidation settlements are evaluated using the new correlations. These settlement values are then compared with previously predicted as well as measured values of settlement.

The soils in the Greater Cincinnati area are typically inorganic clays with low to medium plasticity. Normally consolidated soils were encountered with an increase in depth. The proposed empirical correlations involving compression index, void ratio, dry unit weight, and water content can be reasonably used to determine the compression index for the soils of Greater Cincinnati and Northern Kentucky area and/or similar soils. The predicted values of the settlement utilizing the new equations prove to be remarkably close and conservative to the actual recorded magnitudes and previously predicted values of settlement for most of the cases studied in this paper.

Introduction

This paper is a result of Master's thesis (Dayal, 2006) research carried out at the University of Cincinnati. The topic was chosen to facilitate a proper understanding of the compressibility of Greater Cincinnati/Northern Kentucky soils and provide a tool to the local engineers to predict settlements accurately in the event of insufficient information.

Terzaghi's experiments on clay paved the way for the birth of modern soil mechanics (Terzaghi, 1922). Some of these experiments marked the beginning of Terzaghi's consolidation theory. This theory explained the load transfer from pore water in the clay strata to the mineral skeleton of the soil. Terzaghi (1925) concluded that the relationship between void ratio and pressure for the virgin section of the compression curve could be illustrated by a logarithmic curve. One of the greatest difficulties in obtaining an accurate magnitude of consolidation is the ability to simulate field conditions in the laboratory. Much research (Casagrande, Schmertmann, Silva, Butterfield, Becker et al., Jacobsen, Kim et al.) has been carried out and scientists have developed several empirical methods to estimate the preconsolidation pressure. The most popular of these are Casagrande's (1932) graphical method and Schmertmann's method (1955).

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The biggest hurdle in the determination of maximum past pressure is sample disturbance. While soil disturbance is still not understood properly, it lowers the magnitude of preconsolidation pressure and the volume of voids for any given value of effective overburden pressure. It has been observed that the slope of the recompression curve is less than that of the field virgin compression curve even with high quality sampling and testing (Holtz and Kovacs, 1981). The evaluation of the slope of the field virgin compression curve (known as compression index, C_c) was made possible by Schmertmann (1955) with his graphical method. The procedure involves a typical $e-\log p$ plot. The magnitude of compression index for any soil is very important in the determination of settlement under the application of load. Since this is purely empirical, scientists have developed correlations (Table 1) between compression index and other soil parameters based on the available data and other assumptions.

Table 1. Previous Correlations

Correlation	Reference	Applicability	Remarks
$C_c = 0.007\ (LL - 10)$	Skempton (1944)	Remolded clays	LL = Liquid Limit e_o = Initial void ratio PI = Plasticity Index ω_n = Natural moisture content G_s = Specific gravity of soil solids n_o = initial porosity
$C_c = 1.15\ (e_o - 0.35)$	Nishida (1956)	All clays	
$C_c = 0.54\ (e_o - 0.35)$	Nishida (1956)	All clays	
$C_c = 0.30\ (e_o - 0.27)$	Hough (1957)	Inorganic cohesive soil; silt; some clay; silty clay; clay	
$C_c = 0.009\ (LL - 10)$	Terzaghi and Peck (1967)	Undisturbed, Normally Consolidated clays	
$C_c = 0.014PI + 0.02$	Nacci et al. (1975)	North Atlantic clays	
$C_c = 0.01(\omega_n - 5)$	Azzouz et al. (1976)	All clays	
$C_c = 0.141G_s^{1.2}[(1 + e_o)/G_s]^{2.38}$	Rendon-Herrero (1983)	All clays	
$C_c = 0.2343(LL/100)G_s$	Nagaraj and Murty (1985)	All clays	
$C_c = (LL - 13)/109$	Mayne (1980)	All clays	
$C_c = 0.01\omega_n$	Koppula (1981)	All clays	
$C_c = 0.334\ (e_o - 0.15)$ $C_c = 0.839\omega_n$	Endley et al. (1996)	Gulf Coast clays	
A set of correlations	Yoon et al. (2004)	Marine (Korea) clays	
$C_c/n_o = 0.0109C_c + 0.0018$	Park and Kuomoto (2004)	Ariake (Japan) clays	
$C_c/n_o = 0.0115C_c + 0.00269$			

Methods and Data

Cincinnati and its vicinity are underlain by an arrangement of horizontal beds of shale and limestone, the former exceeding the latter by a significant degree in thickness (Fenneman, 1916). These bedrock formations are primarily Ordovician in age (445-510 million years ago). Colluvium, usually overlaying the bedrock, is characterized by high plasticity and low shear strength, and is formed by the weathering of shale.

Glacial till, outwash sand and gravel, alluvium, lacustrine clays, and fill are among the other soils found in Cincinnati (Johnson, 2002). Glacial till consists of clay, silt, sand, gravel, and boulders and was deposited by melting glaciers. Outwash sand and gravel are the result of glacier retreat. Lacustrine sediments usually consist of fine sand, silt, and clay deposited on lake floors. These soils have low shear strengths. Alluvial sediments are carried by streams and rivers and deposited as these rivers slow down.

The total settlement S for normally consolidated clay with a thickness H, initial void ratio e_o, compression index C_c, effective overburden pressure, σ'_{vo}, and induced stress $\Delta\sigma$, may be expressed as

$$S = \frac{C_c}{1+e_0} H \log \frac{\sigma'_{vo} + \Delta\sigma}{\sigma'_{vo}} \qquad \text{(Equation 1)}$$

C_c can be interpreted as the slope of the "virgin compression" part of the $e - \log p$ curve obtained from consolidation tests carried out on undisturbed soil samples. The current study is based on selected 81 consolidation test results on undisturbed soil samples from Cincinnati and Northern Kentucky provided by Thelen Associates Inc., H.C. Nutting Company, and Ohio Department of Transportation. The Atterberg limits were available for 65 test data and only these were used for the analysis. The data used to evaluate the correlations exclude the consolidation test results present in the case studies later in this paper. While the specifics of sampling and testing procedures are not covered exclusively, their effects may be significant. Hence, the results should be viewed with these limitations in mind.

Characteristics of Data

Casagrande (1932) introduced correlations between Atterberg limits and many properties of silts and clays, such as their dry strength, compressibility, consistency, and their reaction to the shaking test by means of a plasticity chart (Fig. 1). The chart is divided into six regions, three above the A-line and three below it. Fig. 1 illustrates the number of samples that fall in each category based on the Unified Soil Classification System. This plasticity chart indicates that most of the soils are inorganic clays of either low plasticity or medium plasticity.

Table 2 provides a summary of statistical parameters for different soil properties of the available data. The coefficient of variation for different soil properties is a measure of their dispersion. The statistical parameters of liquid limit (LL) and plasticity index (PI) show high dispersion tendencies. The liquid limit has a range of 102 and the plasticity index has a range of 86.8. These values are very high. The dry unit weight has a range of 66.3 pcf. These values indicate the vast range of the type of soils in the Cincinnati – Northern Kentucky area. Fig. 2 illustrates that the OCR decreases with depth for Greater Cincinnati soils. This observation is significant considering the glacial past of the area. Figs. 3 to 7 present plots illustrating the data scatter for various soil parameters. Table 3 presents the data evaluated by some of the correlations introduced previously by various scientists. The ratio of C_c predicted and C_c measured, however, is evaluated based on the data collected for this research. Table 4 introduces the new correlations.

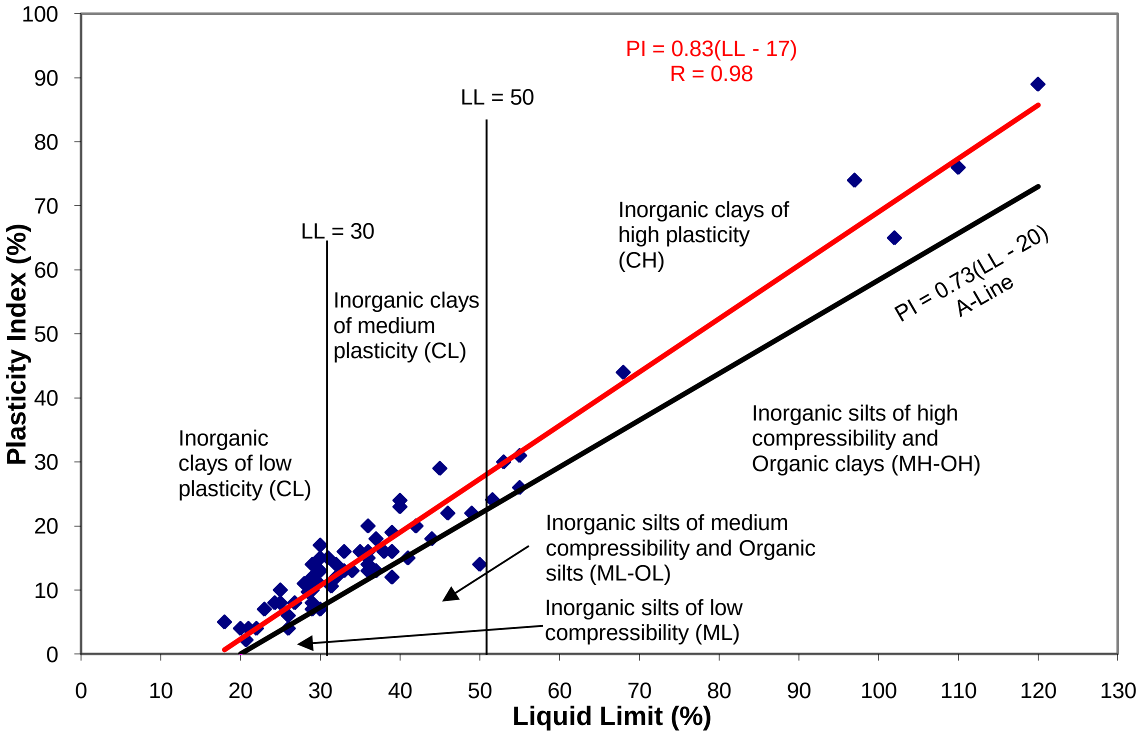

Fig. 1. Classification of soil samples (Casagrande, 1932)

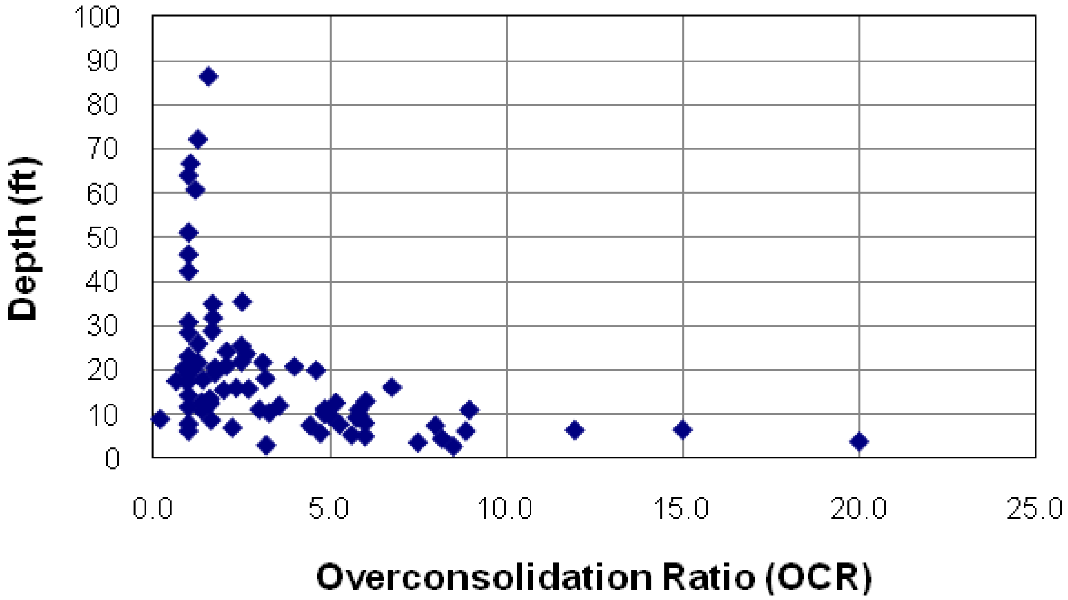

Fig. 2. Depth Vs Overconsolidation Ratio

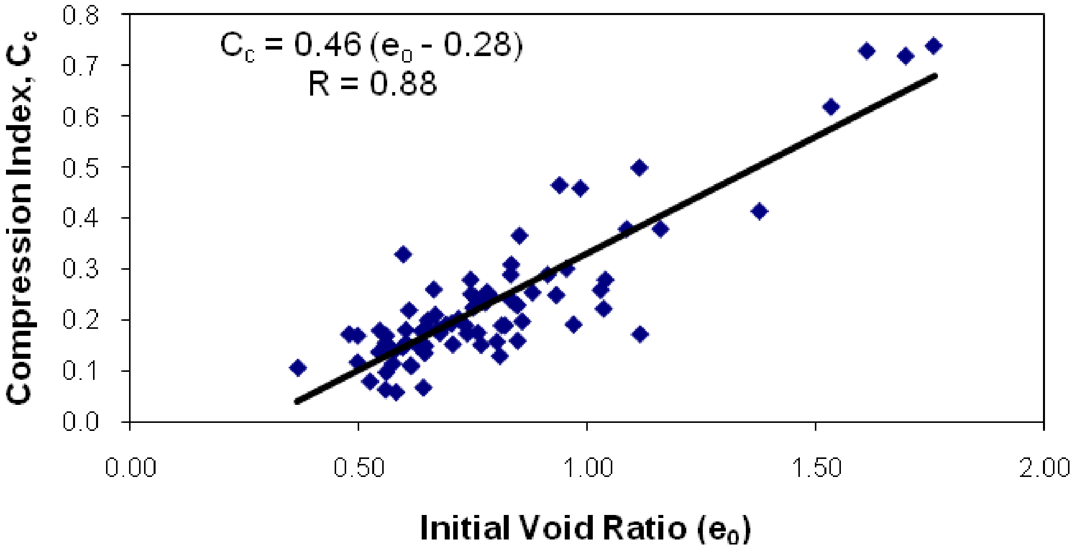

Fig. 3. Compression Index Vs Initial Void Ratio

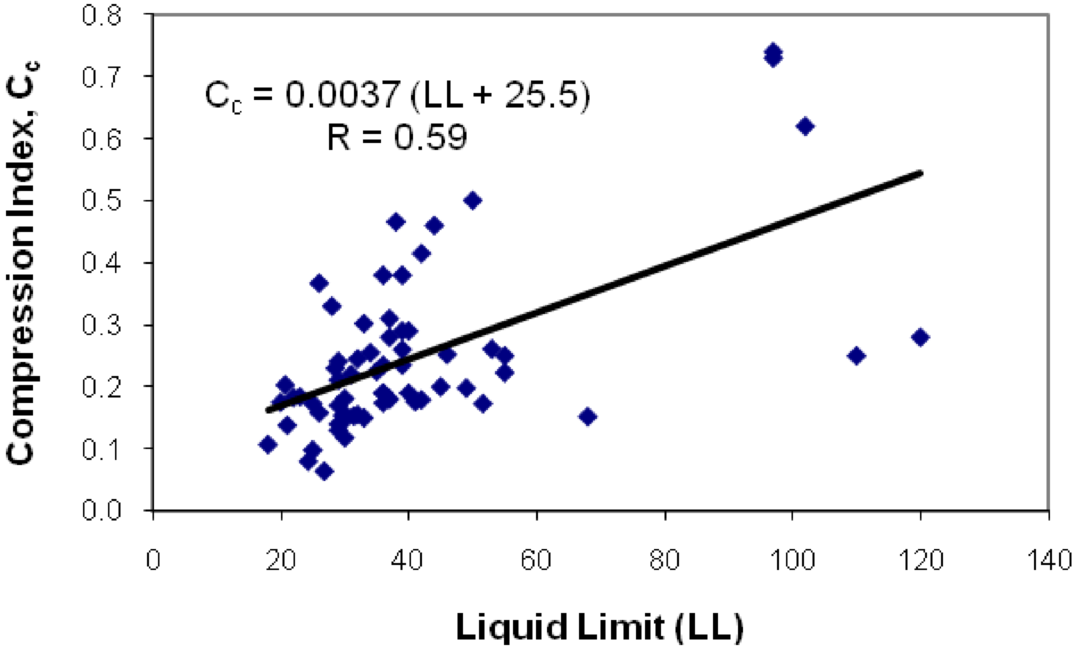

Fig. 4. Compression Index Vs Liquid Limit

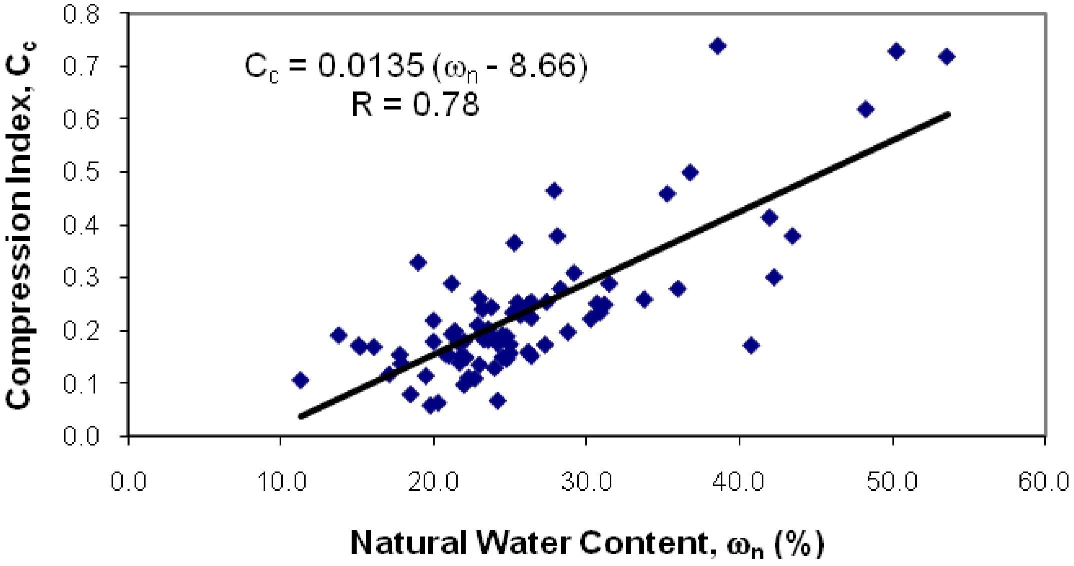

Fig. 5. Compression Index Vs Natural Moisture Content

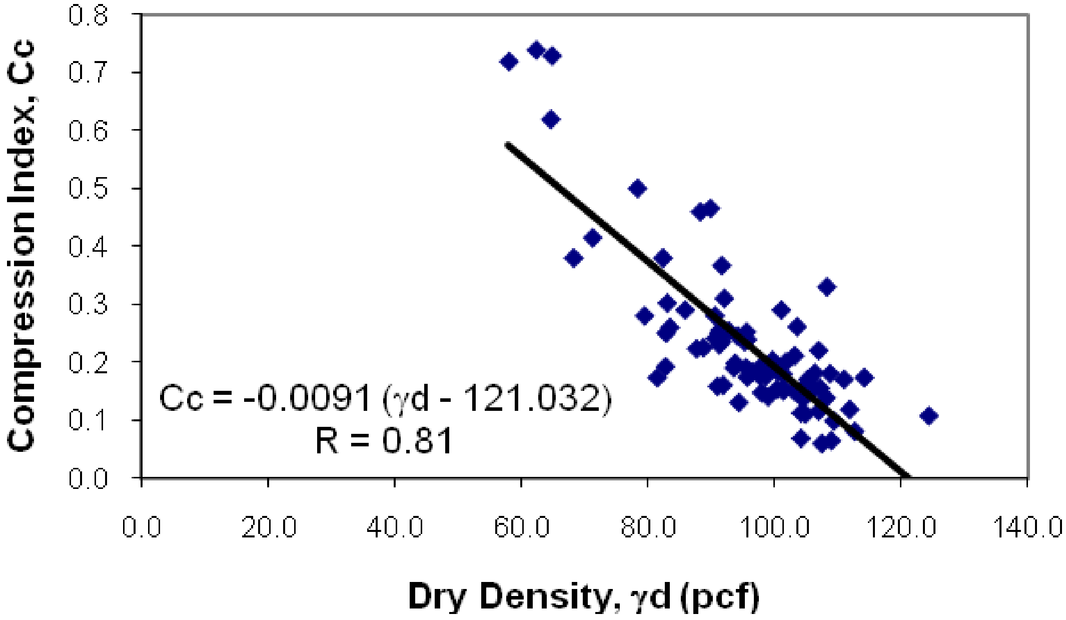

Fig. 6. Compression Index Vs Dry Density

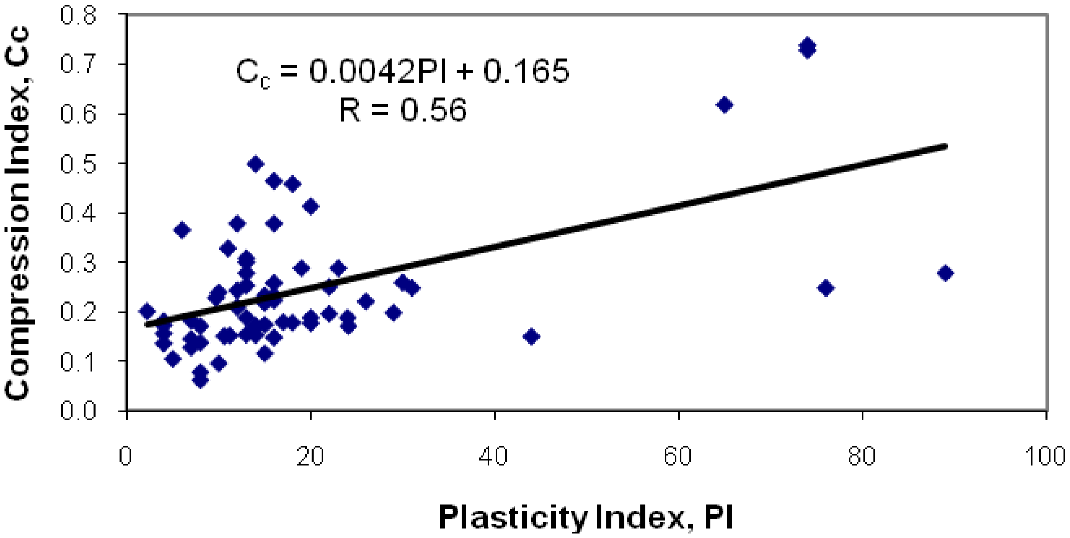

Fig. 7. Compression Index Vs Plasticity Index

Table 2. Summary of Statistical Parameters for Different Soil Properties at Greater Cincinnati Region

Soil Parameter	Min	Max	Median	Coefficient of Variation	Average, μ_x	Standard Deviation, σ_x	No. of Samples
Water Content, ω_n (%)	11.3	53.6	24.2	0.307	26.0	8.0	81.0
Liquid Limit, LL (%)	18.0	120.0	36.0	0.526	40.4	21.2	65.0
Dry Density, γ_d (pcf)	58.0	124.3	96.4	0.130	95.3	12.4	81.0
Initial Void ratio, e_o	0.3667	1.7597	0.7460	0.335	0.7938	0.2661	81.0
Specific Gravity, G_s	2.52	2.88	2.70	0.021	2.70	0.06	81.0
Plasticity Index, PI (%)	2.2	89.0	14.0	0.936	19.3	18.1	65.0
Compression Index, C_c	0.059	0.740	0.192	0.591	0.236	0.139	81.0

Table 3. Ratio of Predicted and Measured Compression Indices using Single and Multiple Soil Parameters for Greater Cincinnati Soils

Correlation	Reference	Applicability	C_c predicted/C_c measured*
$C_c = 0.007 (LL - 10)$	Skempton (1944)	Remolded clays	0.928 (0.512)
$C_c = 1.15 (e_o - 0.35)$	Nishida (1956)	All clays	2.271 (0.844)
$C_c = 0.54 (e_o - 0.35)$	Nishida (1956)	All clays	1.066 (0.396)
$C_c = 0.30 (e_o - 0.27)$	Hough (1957)	Inorganic cohesive soil; silt; some clay; silty clay; clay	0.724 (0.257)
$C_c = 0.009 (LL - 10)$	Terzaghi and Peck (1967)	Undisturbed, Normally Consolidated clays	1.193 (0.659)
$C_c = 0.014PI + 0.02$	Nacci et al. (1975)	North Atlantic clays	0.995 (0.881)
$C_c = 0.01(\omega_n - 5)$	Azzouz et al. (1976)	All clays	1.031 (0.436)
$C_c = 0.141G_s^{1.2}[(1 + e_o)/G_s]^{2.38}$	Rendon-Herrero (1983)	All clays	0.879 (0.319)
$C_c = 0.2343(LL/100)G_s$	Nagaraj and Murty (1985)	All clays	1.158 (0.529)
$C_c = (LL - 13)/109$	Mayne (1980)	All clays	1.076 (0.676)
$C_c = 0.01\omega_n$	Koppula (1981)	All clays	1.305 (0.560)

*Note: The values in the table are averages with standard deviation in parenthesis.

Table 4. New Correlations between Compression Index and Single and Multiple Soil Parameters for Greater Cincinnati Area

No.	New Correlations (Greater Cincinnati and Northern Kentucky)	Correlation Coefficient, R	C_c predicted/C_c measured*
1.	$C_c = 0.46 (e_o - 0.28)$	0.88	1.085 (0.385)
2.	$C_c = 0.0037 (LL + 25.5)$	0.59	1.159 (0.475)
3.	$C_c = 0.0135\omega_n - 0.1169$	0.78	1.120 (0.480)
4.	$C_c = - 0.0091 \gamma_d + 1.1014$	0.81	1.076 (0.400)
5.	$C_c = 0.0042PI + 0.165$	0.56	1.183 (0.501)
6.	$C_c = 0.46e_o - 0.049G_s + 0.0023$	0.88	1.078 (0.379)
7.	$C_c = 0.4965e_o - 0.0014\omega_n - 0.123$	0.88	1.076 (0.379)
8.	$C_c = - 0.247e_o + 0.004LL + 0.01\omega_n + 0.021$	0.64	1.171 (0.491)

*Note: The values in the table are averages with standard deviation in parenthesis.

Compression Index using Single Soil Parameter

Most of the available regression equations are linear in nature and generally relate to a single soil parameter as summarized in Table 2. In the present research, the relationships between compression index and soil parameters are obtained for initial void ratio, natural water content, and dry unit weight with high correlation coefficients. The relationship between the compression index and the plasticity index gives the least value of correlation coefficient. This is expected due to a significant scatter present in the data (Fig. 7). A good correlation coefficient value of 0.88 is obtained between the compression index and initial void ratio.

The average values and the standard deviations (in parenthesis) of the ratios of compression index derived from the regression equation to the measured compression index are 1.085 (0.385), 1.120 (0.480), and 1.076 (0.400), respectively, where the initial void ratio, natural water content, and dry unit weight are a function of the equation. The new correlations (Table 4) were obtained with the help of regression analysis. The best fit curves between different soil properties were determined by the least squares method which provides an optimum fit for the data because it minimizes the sum of the squares of the ordinate differences.

A summary of available correlations is provided in Table 3 with the average values of the ratio of predicted to measured compression index with standard deviation in parenthesis. It shows that most of the existing correlations either under-predict or over-predict the settlement. While it is more conservative to over-predict the settlement, it should not be significantly large. In this respect, Nishida (1956), Mayne (1980), and Azzouz et al. (1976) provide good estimations for the consolidation settlement for Greater Cincinnati soils. All the new equations involving correlations between compression index and soil parameters over-predict the settlement by a small amount.

Compression Index using Multiple Soil Parameters

The regression equation as proposed by Renden-Herrero (1983), which is a function of specific gravity of solids and initial void ratio, underestimates the compression index for Greater Cincinnati soils while the correlation obtained by Nagaraj and Murty (1985) overestimates it. Under such circumstances, it is more plausible to use the latter equation in order to predict the settlements, however, the new correlations obtained in this analysis will provide conservative settlement values and low standard deviations for various combinations of initial void ratio, natural water content, and liquid limit. In the equations involving multiple soil parameters (Table 4 - Equations 6, 7, and 8), standard errors of 0.0677, 0.0675, and 0.1065 have been obtained respectively. Therefore, prediction of the magnitude of primary consolidation using the currently derived equations in lieu of the correlations from previous studies is statistically improved and recommended for the soils of the Greater Cincinnati area.

Case Studies

Six project reports were obtained from the H. C. Nutting Company. These reports had most of the required information (including the monitoring data) to enable a complete settlement analysis to be undertaken. The reports have been studied and presented hereafter in the form of case studies in this paper. The settlements have also been evaluated using the newly obtained correlations and compared with the previously estimated and the actual recorded values. In these case studies, only the new correlations involving single soil parameters, specifically, initial void ratio, dry density, and/or natural water content have been used to evaluate the compression index.

Some reports lacked all the required data for analysis. This was either because of limited laboratory testing or loss of information in the filing process. Several assumptions and some judgment calls were made based on the available information to re-evaluate the settlements. The results of consolidation tests carried out in these case studies were not used in the formulation of new correlations.

1. Expansion of Warehouse and Distribution Facility, Claryville, KY.

The project site is located to the east of US 27 in Claryville, KY. The project involved the expansion of an existing warehouse and distribution facility to the west, covering an area of approximately 155 ft x 295 ft in size. The floor slab of the proposed facility was to be at the same elevation as that of the existing structure. The maximum column loads were estimated to be approximately 80 kips, with floor loadings of approximately 2000 psf.

The natural soils consist of a stiff to hard sandy lean clay containing gravel, which is glacial till, stiff to very stiff fat plastic clays, and thick lakebed deposits consisting of stiff to very stiff varved clays. All borings terminated in very highly weathered soft shale bedrock of Ordovician age belonging to the Kope Formation. The Kope Formation is the thickest of all outcropping formations in the Cincinnati region at 200 to 250 feet of thickness with 65 to 80 percent shale (Potter, 1996).

A soil surcharge 8 feet high was placed over the entire building site in two phases. Five settlement plates, SP-1 through SP-5, were installed to monitor settlement. The recorded settlement readings (Table 5) registered by the plates indicated a maximum settlement of 5.40* inches and a minimum settlement of 1.56 inches. The earlier predicted value of the settlement under the weight of 8 feet surcharge was approximately 1.3 – 1.6 inches. Settlement plates are not a very reliable source of measurement because they are often prone to erroneous results in the event of improper installation, neglect, and inadequate maintenance.

The authors of this paper estimated the settlement to be in the range of 5.8 – 5.9 inches with the use of new correlations involving initial void ratio, dry density, and several assumptions. It should be noted that significant settlements were still taking place when the settlement monitoring program was abandoned. In the last 6 days, SP-5 and SP-1 experienced 0.48 inch and 0.60 inch of settlement respectively. SP-2 recorded 1.2 inches of settlement in the last five days. SP-3 recorded 0.36 inches of settlement in the last twelve days, however, it was noted in the original report that settlement reading was highly questionable at SP-2 for the last week and no reasons were provided. Hence, it is likely that the monitoring program may have been abandoned too soon.

Table 5. Maximum Settlement Recorded at the Warehouse and Distribution Facility

SP-1 (in)	**SP-2 (in)**	**SP-3 (in)**	**SP-4 (in)**	**SP-5 (in)**	**Estimated with New Correlation (in)**
4.08	5.40*	1.56	2.04	2.76	5.8 – 5.9

*Highly questionable

2. Large One-Story Structure, Newport, KY.

The project site is located to the east of I 471 and south of the Ohio River in Newport, KY. The project involved a one story building covering an area of approximately 300 ft x 400 ft in size. Two methods of providing the roof were considered by the author of the project report: (a) The maximum column loads were estimated to be approximately 35 kips and uplift forces of 8 kips utilizing a flexible roof support on cables; (b) A conventional long-span steel joist spanning between a steel frame structure with loads of approximately 125 kips.

The site is located within the present Ohio River Valley. Bedrock exists 80 to 90 feet below grade through the eight test borings drilled. The bedrock is the typical local Cincinnati Series layered gray shale and limestone of Ordovician age. Five to ten feet of glacial till was encountered above the bedrock in several borings and medium dense outwash sand and gravel overlaid the glacial till and bedrock for the most part. At several of the test borings above the sand, stiff to very stiff silty clay was found. At least five borings illustrated the presence of rubble fill at 5 to 15 feet. This fill consist of clay, sand, concrete and limestone slabs, cinders, brick, glass, metal, and wood. Ground water was present typically at a depth of 30 to 35 feet.

The existing rubble fill was to be removed from the building area to a distance of 15 to 20 feet outside of the area for the placement of structural fill. Approximately 15 feet of fill was recommended. Based on the results of laboratory consolidation tests, 6 to 7 inches of settlement was estimated at the location of approximately 26 feet of soft to medium stiff silty clay.

The consolidation tests indicated that the soil was overconsolidated at depth 2-4 ft. but normally consolidated at 10 – 12 ft. The overconsolidation in the upper 2 to 4 ft was most likely due to desiccation. The authors of this paper estimated the settlements to be 1.17 – 1.24 ft using new correlations.

The maximum value of recorded settlement was 1.2 ft as opposed to the previously estimated settlement of 6 – 7 inches. In fact, it took only three months to settle more than 9 inches and reached about 1.2 ft in about 16 months.

3. Five Story Steel Frame Structure, Milford, OH

The proposed building was a 4-story, steel frame construction with brick veneer without any basement. Measuring approximately 192 ft x 88 ft, exterior and interior column loads were estimated to be 650 kips and 500 kips respectively. Ground floor level was proposed approximately 4 to 23 feet above the existing grades (elevation 607) at the beginning of construction.

The construction site was located at the base of the Illinoian till terrace (~300,000 years). This is characterized by minor sand and gravel at higher elevations overlying interbedded till, sand, and gravel and commonly thickest in the preglaciated tributaries of the Ohio River, Mill Creek, and East Fork of the Little Miami River (Potter, 1996). Ten Standard Penetration Test borings were performed and subsoils consisting of topsoil, glacial till, alluvial, glacial outwash, and lakebed deposits were encountered. Lakebed deposits were encountered only in two borings. It was observed that the basement outwash and lakebed soils were eroded by glacial meltwater and replaced by alluvium. The SPT blows for alluvium ranged from 2 to 20 and the unconfined compressive strength ranged from 0.5 to 1.9 tsf. The consolidation test indicated that this alluvium was overconsolidated.

The authors of this paper estimated the settlement to be 5.6 to 5.8 inches based on the subsurface profile at the location of the thickest compressible layer as determined from the test borings. The results of the settlement monitoring program showed a settlement of 6.8 inches due to a 33 feet high surcharge. It should be noted that the settlement plate was located at about 30 feet east of the location of the current evaluation.

4. Two Story Steel Frame Structure, Springdale, OH

The proposed structure was two storied with ground floor on grade, occupied an area of 230 ft x 420 ft, and had a steel frame with maximum column loads of 154 kips and 30 ft x 30 ft bay spacing. The site was located in Springdale, north of Cincinnati. Previous grades within this portion of the tract ranged from approximately 851 to 863 and the proposed finished floor was to be at 870.5.

Geologically, the site is situated within the area abraded by both the Illinoian and Wisconsin glaciers. The soils at the site ranged from outwash sand and gravel to lakebed clays. The test borings exposed fill, top soil, and alluvium for the first several feet. About 6 feet of soft wet high silt content materials were found within the lower eastern swale. Below this layer existed glacial till with high SPT N-values and shear strength in the west with gradual reduction in its thickness towards the swale in the east. The above layer is underlain by varved silt, silty clay, and medium stiff unweathered till. At one of the borings, the lake bed is mixed with outwash sand, gravel, and silt. Layers of dark silt, moderate to highly plastic clays, sand and gravel, and buried glacial logs were also found in this zone. Compact sand and gravel outwash or stiff to hard weathered glacial till (presumably of older Illinoian age) was found around the lowest bottom elevation (804.5) explored. The bedrock was assumed to be at a depth of 70 feet below the average existing grade. The groundwater level was encountered at a depth of 3 to 4 feet below the existing grade in eastern swale areas and at increased depths elsewhere.

A maximum settlement of 4 inches was previously predicted for the foundation soils on the eastern swale areas due to the superimposed load of the embankment plus the building. These settlements were expected to occur within 60 days of the application of loads. Six settlement plates were installed at locations specified by the project engineer. Placement of additional surcharge was recommended to expedite the settlement process. Consolidation tests carried out on the samples obtained from test borings indicated the soil to be overconsolidated.

The settlement was estimated for the location of the south-east corner of the proposed building because maximum amount of settlement was expected at this location. New correlations were used to evaluate the compressibility of soil. The settlement estimated by the authors of this paper ranged from 4.7 to 6.7 inches. The maximum actual settlement recorded illustrates the settlement to be around 6 inches which is close to the current estimated value. Thus the estimated settlement using the new correlations proves not only accurate but also slightly conservative.

5. Aeration Basins, North Bend, OH

The report involves two aeration basins along the Ohio River shore in North Bend. Settlements and bearing capacity were predicted assuming an 11 feet high soil surcharge. Settlement monitoring was conducted. The diameter of the basin was assumed to be 100 feet with 1 ksf design load for evaluation purposes.

The test borings indicated the presence of approximately 30 feet of alluvium underlain by dense sand and gravel. These are mostly believed to be glacial drift of Wisconsinan and Illinoian age. These test borings were probably conducted 10 to 30 years prior to this project. It should be noted that there were no consolidation tests carried out for this specific project and location.

The construction of both the aeration basins was similar with some minor differences. Three settlement plates were placed across the footprint of each of the aeration basins. These plates were roughly aligned perpendicular to the river, one at the center while the other two were to be placed 15 to 20 feet inside the perimeter of each of the basins. Settlements were previously predicted for the following cases to be:

1. Under design loading (1 ksf): 8.2 inches
2. Undercut 6 feet and replaced with ODOT 304 with an additional soil surcharge 11 feet high: 7.0 inches

The settlements were evaluated by assuming the profile to be normally consolidated. This was probably a conservative estimate since there were no consolidation tests conducted during this project. However, only the bottom layer (below 13 feet) was assumed to be normally consolidated for computations. The initial void ratio values were used from the previous consolidation test results referenced in the original report. Settlements were currently predicted for the following cases with the use of new correlations to be:

1. Under design loading (1 ksf): 6.3 – 7.0 inches
2. Undercut 6 feet and replaced with ODOT 304 with an additional soil surcharge 11 feet high: 6.2 – 6.8 inches

Settlements recorded for the following case with the use of six settlement plates (Table 6): Undercut 6 feet and replaced with ODOT 304 with an additional soil surcharge 11 feet high:

Table 6. Settlements Recorded at the Aeration Basins

A (in)	B (in)	C (in)	D (in)	E (in)	F (in)
4.2	4.2	2.5	5.0	8.0	5.0

The settlement values estimated with the use of new correlations compare well with the observed values.

6. Large One-Story Structure, Cincinnati, OH

The location includes areas both east and west of Paddock Road, south of Seymour Avenue but this report addresses only a portion of the area east of Paddock Road. The proposed building was a single level, 600,000 sq. ft. structure with expected floor loading to be around 150 psf and interior column loads around 150 kips. The finished floor elevation was assumed to be at 551 with a subgrade at 550.

The site was divided into three principal areas on the basis of topography and soil conditions: deep valley, buried slopes, exposed slopes and upland area. The soils forming the upland area and underlying the slopes were deposited during the Illinoian glacial period and include till, outwash, and lake bed clays. The valley was probably formed during the recession of Illinoian ice. The uppermost soils encountered in the valley were basically fine textured alluvium.

The typical profile encountered in the deep valley consisted of about 10 feet of alluvium underlain by about 30 feet of lake bed clays. The unconfined compressive strengths of the alluvium ranged from 2 to 4 ksf and that of the lake bed clays around 1.5 ksf. Two consolidation tests conducted on samples from this clay indicated that the clay was overconsolidated. However, at greater depths, the soil was found to be normally consolidated at the middle to southern part of the site.

The material underlying steep exposed hillsides and the upland area was compact glacial till, well graded material consisting of clay through gravel sizes, and at least one layer on a heavily overconsolidated Illinoian silty lake bed deposit. The groundwater level varied considerably in all the borings and was particularly shallow closer to the Bloody Run Channel. The sand layers beneath the buried slopes were free of water due to the high permeability of the sands.

Table 7 presents the recorded settlements at the site. The earlier predicted settlements for the valley area based on a 1000 psf surcharge and consolidation test results lie in the range of 9 – 15 inches. The settlements estimated with the use of newly obtained correlations were in the range of 10 – 12 inches.

Table 7. Recorded Settlements

Settlement Plate	Recorded Settlement (inch)
SP1	1.7
SP2	1.7
SP3	1.7
SP4	5.8
SP5	4.7
SP6	8.4

The recorded settlement of plates 1 through 3 was very uniform varying from 1.69 to 1.72 inches. Settlements observed at plates 4 through 6 were much greater varying from 4.7 to 8.4 inches. The observations also indicate that the settlements were still continuing, albeit at a much smaller rate, when the monitoring program was called off. The total amount of fill placed at the 6 locations varied from approximately 9 feet to 10.5 feet, the final 6 feet of this being temporary surcharge

Conclusions

Soil parameters such as initial void ratio, liquid limit, plasticity index, dry unit weight, natural moisture content, specific gravity of soil solids, and compression index were obtained from data collected from various local geotechnical firms. These data were then used to develop several empirical equations for predicting the compressibility of soils found in the Greater Cincinnati area. Further, six cases were considered to validate the proposed empirical equations.

Soils in the Greater Cincinnati area were typically found to be inorganic clays with low to medium plasticity. The overconsolidation ratio shows a decrease with an increase in depth for the observed samples.

Most of the newly obtained empirical equations are statistically improved over their predecessors, however, those suggested by Nishida (1956), Azzouz et al. (1976), and Mayne (1980) can be considered reliable in predicting the compression index for Greater Cincinnati area soils as well. The correlation involving initial void ratio and compression index has a good correlation coefficient of 0.88 and is highly recommended. The other correlations between compression index and soil properties such as dry unit weight and natural water content may also be used. The proposed empirical equations involving liquid limit and plasticity index have low correlation coefficients due to a higher degree of scatter in the data. The use of these equations should be avoided.

Three new equations involving multiple soil parameters have been proposed in this paper two of which have high correlation coefficients (0.88). These two equations show relatively low percentage of standard errors (~ 7%) having initial void ratio as the common parameter and should be used in the event of the availability of required data.

The case studies present a widely existing problem of insufficient data for a complete analysis. The greatest challenge was to obtain cases with every aspect of settlement analysis especially settlement monitoring and laboratory test results. The consolidation tests, if conducted, were not commensurate with the number of test borings and/or the depth of the hole. Moreover, the authors believe that the time of monitoring the settlement may have been inadequate for some of the cases analyzed in this paper as significant settlements were still taking place when the monitoring program was called off. It would appear that industry relies more often on the experience and judgment of the engineer than on necessary in situ and laboratory testing.

The predicted values of settlement utilizing the new equations proved to be remarkably close and conservative to the actual recorded magnitudes of settlement when compared to previously predicted values of settlement for most of the cases studied in this paper

(Table 8). However, it should be noted that the total number of cases studied in this research is not enough to be conclusive and therefore, the proposed equations are to be used intelligently depending on the subsurface conditions, laboratory test results, and engineering judgment.

Table 8. Case Study Summary

Case Study	Max. Settlement Recorded (inch)	Max. Settlement Predicted in the Original Report (inch)	*Max. Settlement Predicted with the use of New Correlations (inch)	Remarks
1	5.4[†]	1.6	5.8 – 5.9	Monitoring time ~ 40 days
2	14.4	7.0	14.1 – 14.9	Monitoring time ~ 22 months
3	6.8	---	5.6 – 5.8	Monitoring time ~ 60 days
4	~ 6.0	4.0	4.7 – 6.7	Monitoring time ~ 5 months
5	~ 8.0	7.0	6.2 – 7.0	Monitoring time ~ 70 days
6	8.4	15.0	10.3 – 12.0	Monitoring time ~ 5 months

*Several new correlations were used and therefore, a range of maximum settlement values are provided.

[†] Highly questionable

Time rate of settlement is an important component of consolidation analysis. It is recommended to consider time rate of settlement wherever possible. The authors suggest a similar study which extends the present research by incorporating the pore pressure measurements and coefficient of consolidation.

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Observational Method Using Real Time Surface Settlement Monitoring – The South Toulon Tunnel Project

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Abstract: The Toulon South Tunnel in France is the second part of a project started 20 years ago. After the problems and delays suffered during the construction of the North Tunnel, due to a complex geologic context of highly fractured and soft rocks under a highly urbanized environment, a very dense automatic monitoring system was implemented to control the settlements of surface (more than 1000 points), the structures adjacent to the tunnel alignment and the underground galleries being constructed. The latest automatic instrumentation systems were installed, using for example AMTS capabilities in terms of "automatic prismless settlement monitoring". All the data (geotechnical, structural and geological) was gathered in a global automatic data acquisition system and continuously available on a web based interface. Three different threshold levels (notification, anomaly, alert) were set up, each one with a differential and an absolute value for settlements. Depending on the actual geological site conditions and the actual monitored settlements, different supporting systems and excavation techniques were used. Furthermore, the monitoring data was also used to predict and anticipate the future settlement, by comparing it continuously with the theoretical Peck settlement curves. This method allowed a better risk management by limiting the settlement on existing structures and helped to secure the Works progress schedule, for a project that lasts from July 2007 to March 2011. The implementation of this observational method application is presented in this paper.

Introduction

Project History

The Toulon tunnel project is a key component of the Highway corridor on the French Riviera between Marseille and Nice and is a fundamental transportation infrastructure for the region. The first design studies started in the 1960's. The tunnel project is made of 2 separate tunnels: South and North tunnel. The North tunnel was built between 1992 and 2002 after 10 years of difficult construction (for example, on March 15th, 1996, a collapse of the tunnel in the "Marchand Area" generated a sink hole in the city Center, causing 20 months delay). The South tunnel alignment runs parallel to the North tunnel at the same depth below Toulon historical downtown (20 to 40m depth with some areas with less than 10m of cover). The Risk management program was studied to be a State of the Art program in urban tunneling construction.

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The Hydro Geological Context

The whole project area is covered with backfills and colluviums of 1 to 4m thick. The Schist, marls and sandstones, randomly distributed along the alignment below the first layer have different characteristics. While the Schist are pretty compact, the marls are fractured. The other layers have very different characteristics, and many faults are located along the alignment, especially in the limestones. The Breccia are heavily decompressed and deformable, as well as the other soils located in the Marchand Area. This 80m long area is where the settlements need to be controlled with an extensive monitoring program. The complete Geological profile is presented in Fig. 1.

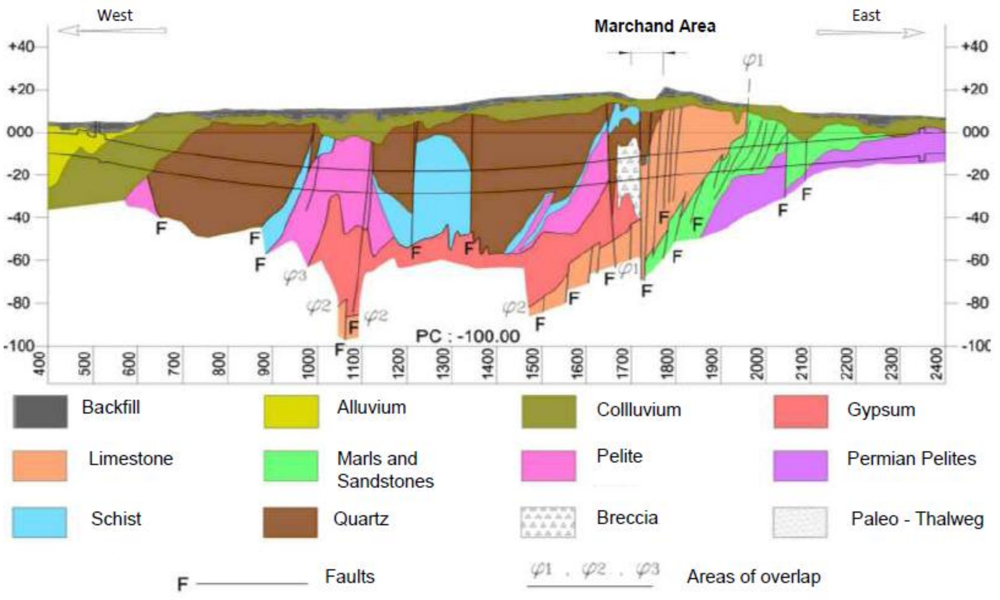

Fig. 1. Geological profile along the alignment

The superficial water table is relatively stable and is located in the superficial layer (Colluvium). The deep water tables are located in the Limestones and fractured sandstones. These water tables are independent, as they are separated by impermeable layers. This heterogeneity is hard to monitor precisely as the permeability conditions vary drastically along the alignment.

The Existing Buildings

All the alignment is heavily built. Most of the existing buildings above the tunnel alignments were build in the 50's in armed concrete, and are superficially founded.

Fig. 2. Aerial view of the alignment

In the “Marchand Area”, 2 buildings (named K6 and K7) were located in the settlement trough in the most deformable area and were the most heavily instrumented.

The Design

The Design Factors

The tunneling design was focused to control the settlements and to limit the deformations of the existing buildings. This approach was chosen to avoid the problems encountered during the first phase and because it appears to be the only way to deal with the highly complex hydrogeological profile. 3 steps were followed to achieve this purpose:

- Evaluation of the admissible deformations for each building and classification in 13 different sectors
- Definition of 3 levels of absolute and differential settlements (notification – anomaly – alert) for each of the 13 sectors
- Design of specific supports and choice of the excavation techniques to respect the deformation criteria defined in step 2

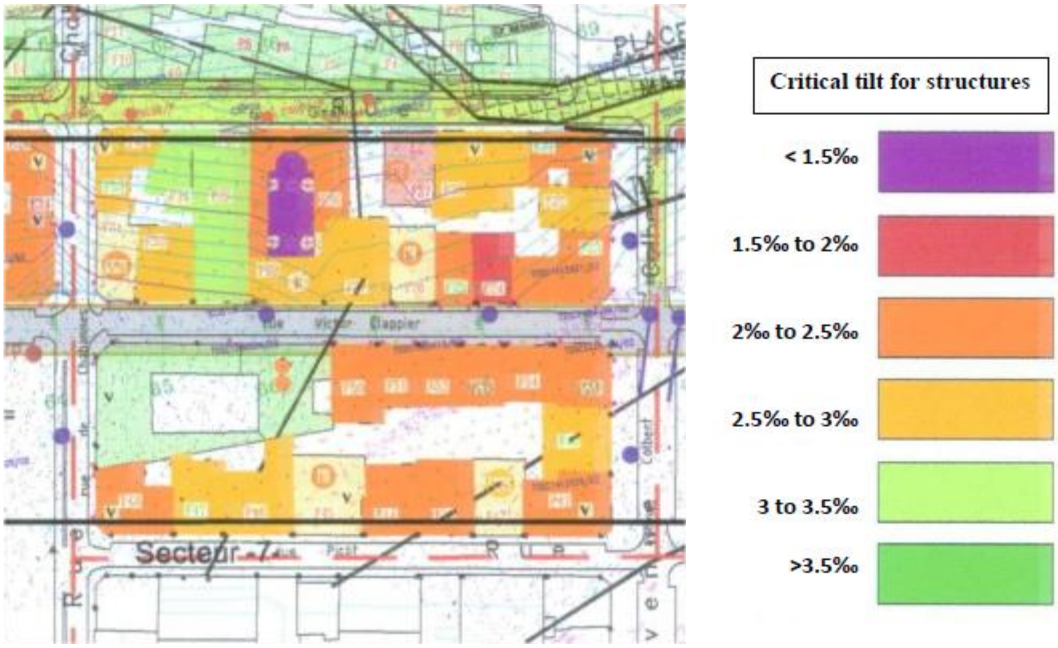

Fig. 3. Definition of critical deformation

Types of Supports and Tunneling Techniques

The tunneling section is located between 2 “Cut and cover” sections of each portal. The 1818m of tunnel are excavated using the traditional method full section. The tunneling is started at 3 locations, in addition to the 2 East and West ends, a specific shaft in the “Marchand Area” is used to start the 3rd front of tunneling. 9 different support designs were defined, from “light” support systems with only one I beam arch to more complex support systems with umbrella arch, shotcrete and 2 levels of I beams closely spaced.

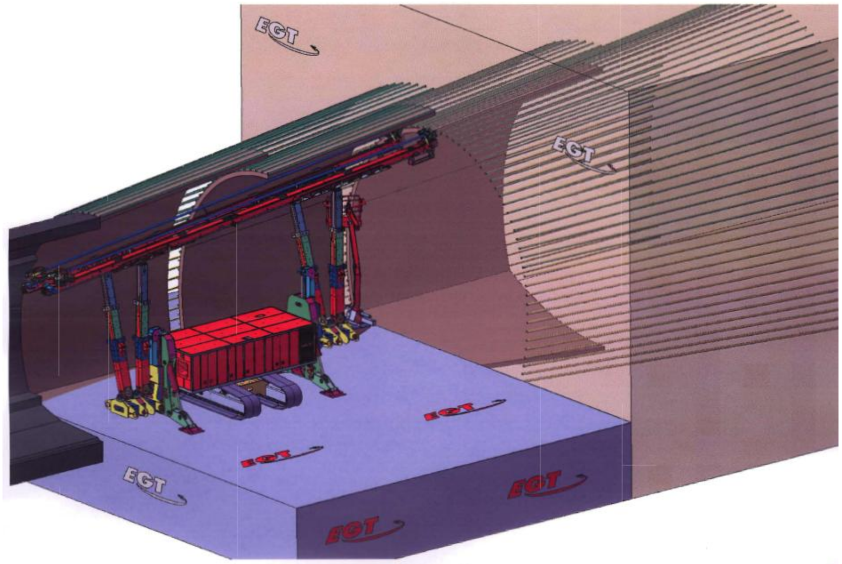

Fig. 4. Umbrella arch technique

The Monitoring Program

An extensive monitoring program was implemented along the alignment, inside and outside the tunnel. As in other sensitive project where Risk management is critical, most of the instruments installed were automated and all the data management system was linked to a web based interface software (Geoscope) for a real time access to all the project data. **The monitoring program was mainly based on the use and capabilities of Automatic Motorized Total Stations (AMTS)**, and locally completed with "traditional" geotechnical and structural instrumentation as detailed in this section.

Ground Instrumentation outside the tunnel

From the surface were installed:

- Multiple borehole extensometers (3,4 and 5 anchors) were installed to measure the vertical ground displacement at several depths between the surface and the tunnel crown
- Manual inclinometers on both sides of the tunnel and in the vicinity of the Shafts
- Automatic Piezometers to measure water level and pore pressure

The manual data collected from the inclinometers was downloaded into the database, as well as the whole quality control reports of the project, so all the information related to the project was accessible at anytime from one single source. The software was able to generate in real time water level isolines as shown on Fig. 5.

Fig. 5. Water level tables isolines along the alignment

Instrumentation inside the Tunnel

In addition to the standard manual survey, the convergence was continuously measured every 4.5m by using Automatic motorized total stations and 3D prisms attached to the support system as shown on Fig. 6.

Fig. 6. AMTS and targets inside the gallery

In the critical sections, additional automatic instrumentation was installed (radial extensometers, strain gages on the I profiles, pressure cells in the shotcrete).

Structural Instrumentation on Existing Buildings

Once again, most of the instrumentation was automated. The following instruments types were installed on the critical buildings as determined by the pre construction expertise:

- 3D targets monitored with AMTS
- Tiltmeters and tilt beams on vertical structural elements
- Automatic and manual crackmeters
- Strain gages on key structural elements
- Vibration monitoring units

Due to the quantity of automatic data to be transmitted, a specific local Wi-Fi network was set up in the city.

More than 1,500 3D prisms were installed on existing buildings and monitored every 2 hours for the duration of the project with a **required accuracy of 0.3mm.** This criteria was achieved thanks to specific filters on the raw data and least square calculation tools included in the global data management software.

Automatic "Prismless" Surface Monitoring Using AMTS

The most innovative solution used on this project was the automatic surface settlement monitoring using AMTS called *"CENTAURE"*. **With no need to install any kind of reflector on the monitored area** (The system still requires 3D reference targets installed outside the zone of influence to correct the position of the AMTS and achieve the accuracy required), the system was configured to automatically measure the settlement of a virtual mesh of 1,800 surface settlement points on the existing roads with 36 AMTS positions along the alignment **with an accuracy of 0.5mm at a 40m distance** (the maximum monitoring distance is limited by the reflective capabilities of the surface). This technique has been used successfully on several other tunneling projects since then, and more generally on projects where the access to the area to be monitored is limited. Examples of applications on Existing highways are shown on Fig. 7. This technique can substitute any type of "standard" manual surface leveling, and provides safer "real time" risk management, as well of a better understanding of the settlement trough geometry (Fig. 9).

On the specific Toulon South Tunnel project, this system was combined with the automatic monitoring of structures using traditional 3D prisms "*CYCLOPS©"*. An example of this combined AMTS application is shown on Fig. 8.

In this case the monitoring frequency depended on the location of the excavation front; the monitored "zone of influence" was 45m before and after the front with 5-points arrays every 9m or 4.5m depending on the sensitivity of the area. The monitoring frequency depended on how critical the structure was and on the alert level (from 1 to 8 measures/day).

Once the data was processed, automatic isolines of settlement were generated by the software to have a global "3D visualization" of the construction impact on the surface and buildings (Fig. 9).

Fig. 7. 3D monitoring points (green circles) and virtual mesh of automatic

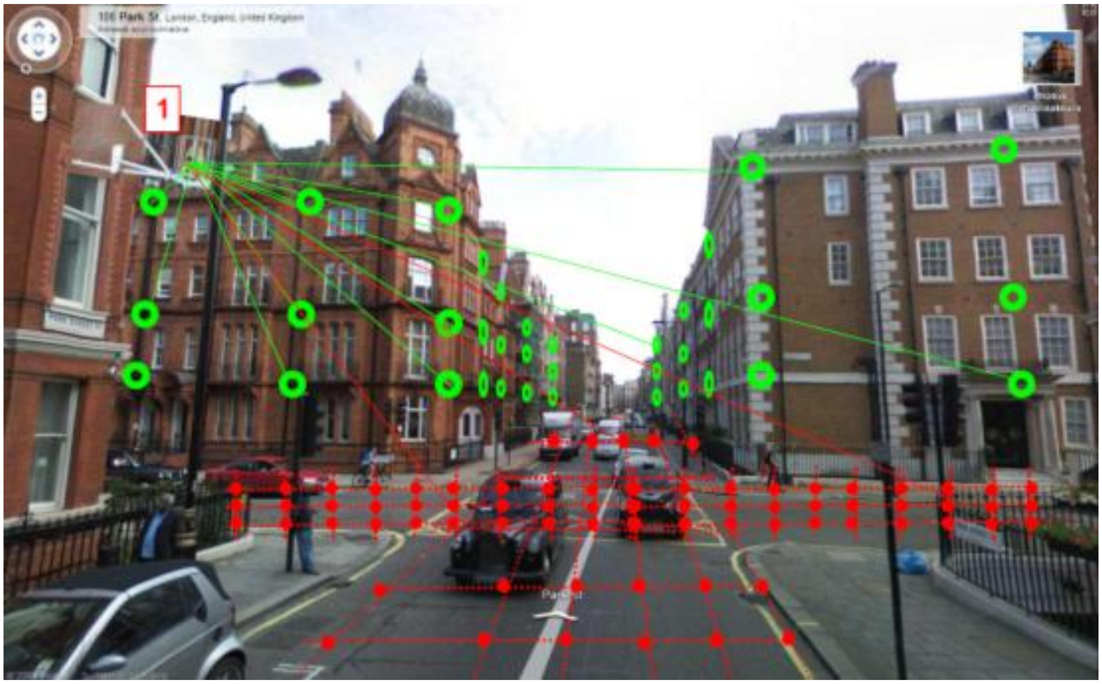

Fig. 8. Example of combined AMTS system: 3D monitoring points (green circles) and virtual mesh of automatic surface settlement points (red points)

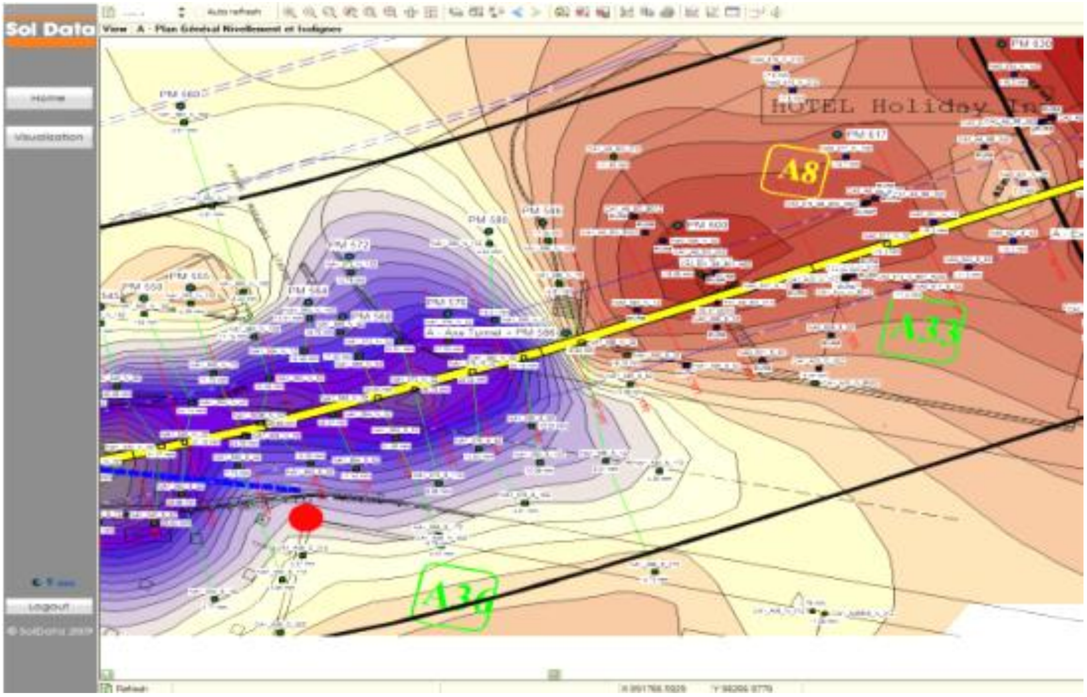

Fig. 9. Example of Isolines of settlement

The Observational Method

Construction Method and Support System

On the majority of the excavation projects, the choice of the best suitable supporting system is done based only on the parameters available ***before*** construction (geologic profile, geotechnical investigations logs). On the south Toulon project, in addition to the pre-construction data, the Construction method and the supporting system was chosen also with the data retrieved ***during*** construction. In addition to the analysis of the on site geological front excavation section, the measured deformations inside and above the tunnel were used to optimize the design.

Based on the analysis and the **prevision** of anticipated settlements in comparison to the initial calculated settlement ant the 3 levels defined by the Engineer, the Tunneling contractor had to whether:

- reduce the quantity of support,
- use the specified support system
- increase the supporting system by proposing a reinforced design

Distance to Facegraph

The objective was to precisely monitor the initial start of the settlements a few meters ***in advance*** from the tunnel position to help the tunneling contractor with the choice of support system. For this specific application, a new monitoring software feature has been developed to compare the reference settlement curves (based on the Peck probabilistic approach) and the curves resulting from the measured deformations, at different positions before or behind the excavation front position. This tool, called the distance to face graph, was part of the real-time monitoring software Geoscope.

This special feature requires accurate measurements. Requirements were a 0.5 mm accuracy for surface vertical movements and a 0.3 mm accuracy for 3D buildings movements. This precision was achieved thanks to specific AMTS features (CYCLOPS and CENTAURE – See paragraph on "Automatic "prismless" surface settlement monitoring using AMTS"). Through filtering programs, weather corrections least square approximation and median calculation, raw data is transformed into accurate data and transmitted into the monitoring web based data management system.

The distance to face graph can be refreshed every day with the median of the previous 24 hours to be have an updated anticipation of the final settlement and optimize the construction process (supporting system, excavation speed) as well as reduce the risk of damage to the existing structures.

An example of the output of this feature is shown on Fig. 10. On the same graph are visualized the 3 different theoretical curves (each of these referring to an alert level defined during the design process) and the 5 surface points monitoring data, in relation to their distance with the excavation front. "Before" the excavation reaches the monitored section, settlement starts to occur. At that stage, the calculation enables to anticipate the final settlement (once the excavation front is away from the monitoring section); if this "anticipated" settlement exceeds the alert level the design is reviewed for the settlement to stay within the project limits of building deformation specified during the evaluation of admissible deformations.

In addition to these "longitudinal" sections the settlement was also monitored and compared to the calculated data on the transversal sections. Sections of settlement trough were retrieved using the monitoring data and the Gauss analysis.

In both cases (longitudinal and transversal), the tilt was retrieved by using the monitored data of 2 adjacent points and compared to the maximum allowable for the building; this information was essential to evaluate the possible damage. The transversal differential deformations (up to 2.8mm/m) were more damageable to the buildings than the longitudinal ones.

Unpredicted Behavior in the Marchand Area

This area was anticipated to be the most critical on the project, as most of the problems encountered during the North tunnels were located in the vicinity of this area. The design of the support system was the heaviest of all the alignment, with Umbreall arch, double 220 I beam and 180 I beam. The fiberglass blots on the section were repeated every 4.5m.

Since the beginning of the excavation in that area, important settlements were measured on 2 buildings (K6 and K7). These early settlements would have resulted in unacceptable final deformation for the buildings after the tunneling. An additional compensation grouting program was implemented to prevent further damages to the buildings. The initial settlement hollow below the K6 and K7 buildings is shown in Fig. 11.

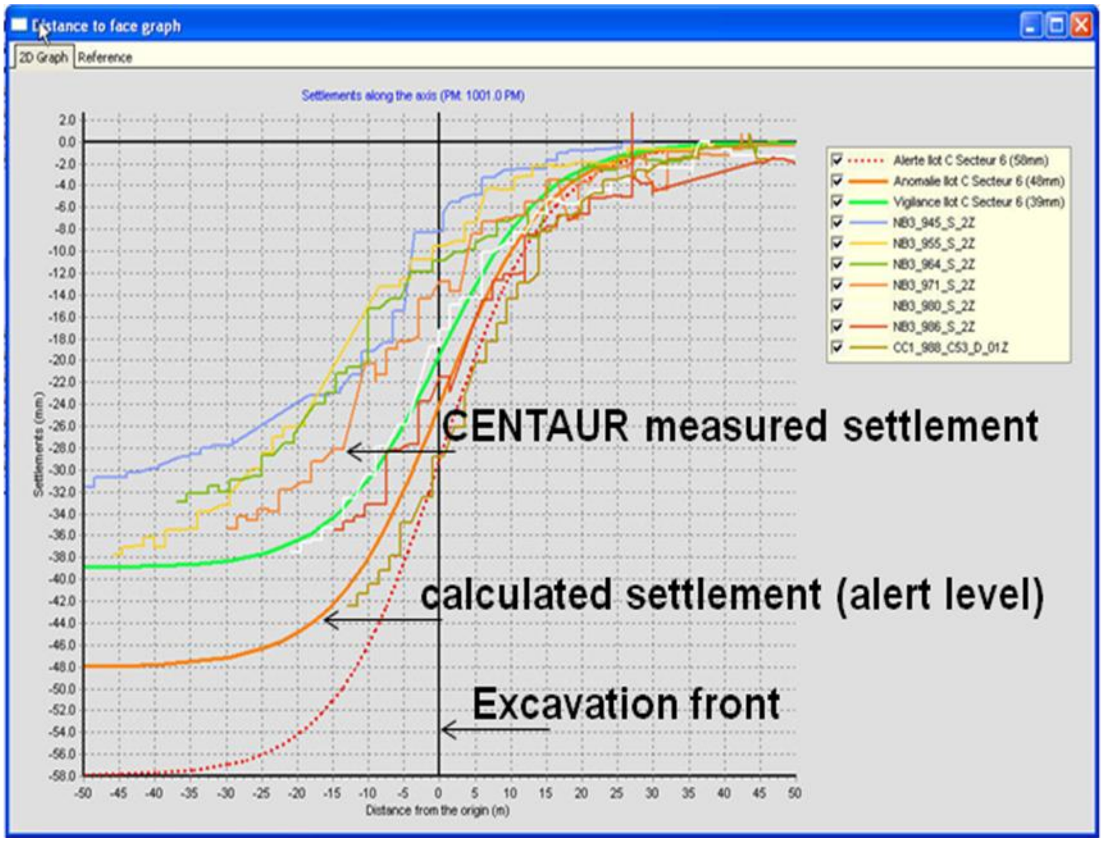

Fig. 10. Comparison between calculated settlement curves and actual measured data before (right) and after (left) the excavation front

Thanks to the real time settlement monitoring program and feedback analysis, major damages were avoided and construction could proceed.

The real time surface monitoring program was combined with the compensation grouting monitoring program to have a global control on the operations and ensure the safety of this sensitive operation. Fig. 12 shows the evolution of settlement of the K6 building in relation to the tunnel excavation progress, as well as the control of settlements made possible thanks to the compensation grouting program.

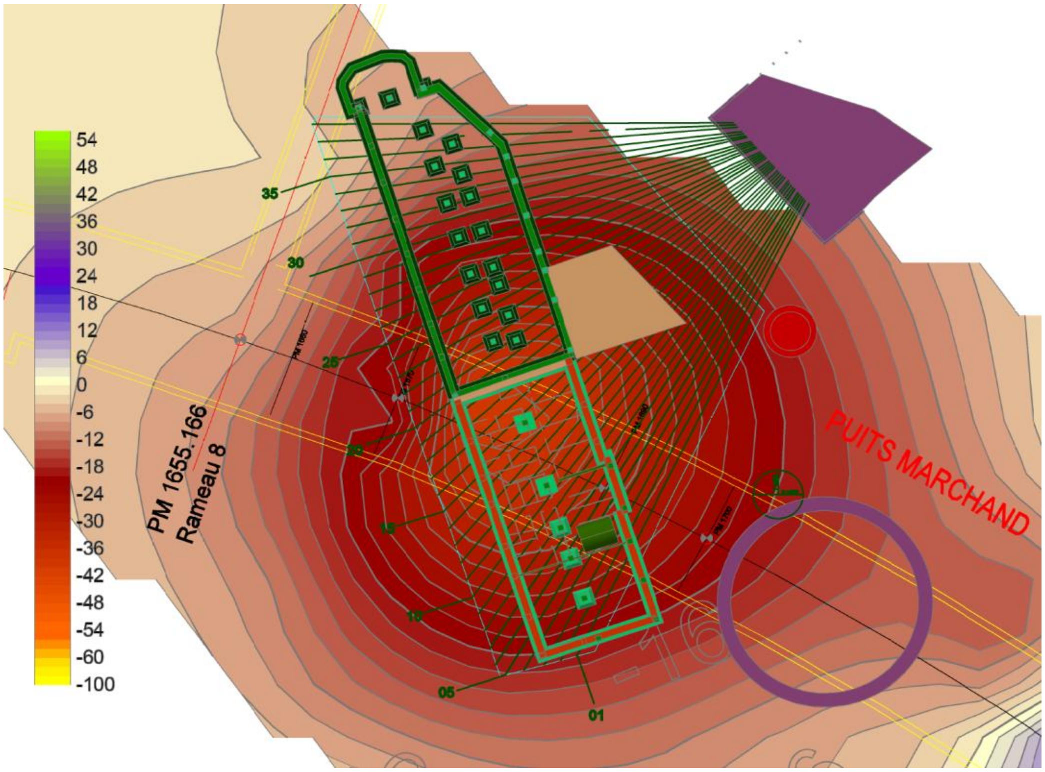

Fig. 11. Settlement Hollow in the Marchand area and compensation grouting program

Fig. 12. Tunnel progress (green line) and settlement on road surface (red line) before and after compensation grouting works

Conclusion

The Toulon South tunnel project is a typical example of challenging urban tunneling project in a difficult environment. Thanks to the lessons learned during the construction of the North Tunnel, most of the damages have been avoided. For this purpose, innovative monitoring solutions combined with a powerful data management systems have been implemented. However, in some specific areas the initial design was not enough to anticipate completely some unexpected and abnormal soils behaviors. But the real time monitoring program helped to limit the impact of such phenomena and certainly avoided the same type of collapse that happened during the first phase.

The applications of such approach are unlimited. Not only this method could allow cost savings optimizations during the design process, but also limit the impacts of all the risk factors associated with a construction project (delay, collateral damages, cost overruns).

All this dynamic process is only possible if the 3 actors (design Engineer, monitoring company, General Contractor) work together and use the right tools.

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Starting Work Right Isn't Always Enough

Farrokh N. Screwvala, Ph.D., P.E., F.ASCE, L.M.ASCE[1]

Abstract: A house was designed and built in 1992 at a sloping site in Brecksville, Ohio. A geotechnical investigation was obtained and drawings were prepared with the assistance of a registered structural engineer. The owner retained a contractor to build the house. The contractor was able to persuade the owner that money could be saved by ignoring some of the recommendations of the original geotechnical engineer and the original structural engineer. Sadly, but not surprisingly, the house developed structural distress. Another structural engineer and two other geotechnical engineers were retained to remedy the house as litigation ensued. The needs of the house were more pressing than the speed of the litigation. Structural repairs necessitated by the changes of the contractor and underpinning of some of the foundations to the depths required by the original geotechnical engineer were hastily implemented to avoid a catastrophe. Recommendations of expensive retaining walls and deep seated failure have yet to be realized.

Introduction

According to county records, the property at which the house was built was owned by a builder in 1992. The records also show that a home was built at the property, that title to the property and the home were transferred to a family in September 1993 and that they still retain that title. In April 1993 the County published topographic maps of the area. An excerpt from one of those maps is included as Fig. 1. The home that is the subject of this paper is shown in red. By December 1997 the home had experienced substantial structural distress which was urgent enough to warrant immediate repair. Litigation was underway to allocate responsibility for the damages that resulted. The author first viewed the condition of the house on December 2, 1997. The information obtained from the investigation into the causes of the damage to the house and the lessons that can be learned from them are the subject of this paper. Fig. 2 shows an excerpt from the 1948 and 1953 County topographic map on which the location of the house has been superposed. Even though this information was not used in the litigation that ensued, it offers independent corroboration of the conclusions reached and may be of some interest to readers.

The House on December 2, 1997

Fig. 3 shows a photograph of the downhill face of the house. Fig. 4 shows a close up of the east end of the downhill face. Tie rods have been inserted to prevent failure of a section of the house that was required to function as a retaining structure. Fig. 5 shows a view of the main section of the downhill wall. The original foundation has been replaced with a grade beam which is supported on Atlas piers. Tie rods have been inserted through the grade beam to provide lateral support. Fig. 6 shows the west wall of the

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house being underpinned by an Atlas pier. Fig. 7 shows some of the damage that was visible at the west end of the house.

Fig. 1. Excerpt from the April 1993 County topographic map.

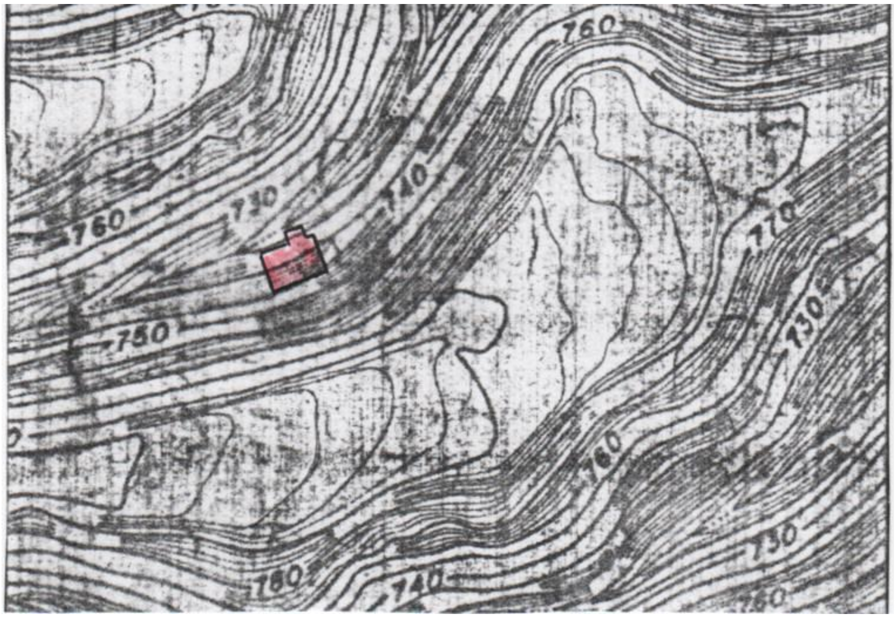

Fig. 2. Excerpt from the 1948 and 1953 County topographic map.

Fig. 3. The downhill face of the house.

Fig. 4. East end of the downhill face.

Fig. 5. Underpinned and tied back downhill foundation wall.

Fig. 6. West wall being underpinned with an Atlas pier.

Fig. 7. Cracks in west wall.

The Design of the House

During deposition the owner testified that he represented to the City of Brecksville that a variance was needed to make the lot buildable, because of a severe drop off, and that he relied on the builder for his belief that the lot was buildable. He also testified that he retained Geotechnical Engineer A to perform an investigation of subsurface conditions at the lot to fulfill a requirement of the City. The report was completed on July 24, 1992.

Geotechnical Engineer A's report identified slope failures in fill, which was reportedly placed in 1991, provided reasonable means for correcting failures in existing fills and avoiding failures in new fills. Criteria for the depth and proportioning of foundations and for the inspection of the foundation excavations were provided. It was required that basement walls be reinforced.

Structural Engineer A was retained to prepare a foundation plan consistent with Geotechnical Engineer A's recommendations. A foundation plan was prepared and completed on July 29, 1992. It showed foundations that were designed consistent with Geotechnical Engineer A's recommendations and required 16" and 12" Ivany block construction for some of the foundation walls. The foundation plan was reviewed by Geotechnical Engineer A and on July 29, 1992 a letter was sent to the owner that the foundation plan was consistent with his recommendations.

The Construction Contract

The owners entered into a construction contract with the builder on September 3, 1992. Item I of the specifications in the contract read in part:

"EXCAVATING (PER INSTRUCTIONS FROM GEOTECHNICAL ENGINEER A, INCLUDING FOLLOWING RECOMMENDATIONS PAGES 1-7) AND GRADING."

Item II of the specifications in the contract read in part:

"II CONCRETE FOOTINGS (PER ATTACHED DRAWING - STRUCTURAL ENGINEER DESIGN)
    (A) Concrete footings shall be a nominal 8" in thickness and 4" wider than the foundation walls on each side of the wall itself."

Item II (A) was inconsistent with the foundation design criteria of Geotechnical Engineer A. Item I and Item II (A) of the construction contract could therefore not both be complied with. Most of the foundations shown on Structural Engineer A's drawing were more than 8" thick and more than 4" wider than the foundation walls on each side. Therefore, the reference to "(PER ATTACHED DRAWING - STRUCTURAL ENGINEER DESIGN)" could not have been to the drawing prepared by Structural Engineer A.

Item III of the specifications to the contract read in part:

"III FOUNDATION WALLS (PER ATTACHED DRAWING - STRUCTURAL ENGINEER DESIGN)
    ……All basement foundation walls to be 12 courses of 12" concrete block."

Structural Engineer A's drawing called for some of the foundation walls to consist of 12" and 16" Ivany block walls. The "PER ATTACHED DRAWING" referred to in the contract was, therefore, not the drawing prepared by Structural Engineer A. Since the contract required the use of unreinforced foundation walls, the contract intended to violate Item I of the specifications to the contract.

In his deposition, one of the owners testified that he was a registered professional engineer in Ohio. He also testified that he was not one of the people who wanted Geotechnical Engineer A's report and that he did not read it when he received it. The deposition revealed that the owner assumed that the builder did not have any professional qualifications other than his experience building houses. The deposition also established that the builder explained to the owner what an Ivany block wall is, that he felt that Geotechnical Engineer A's design and Structural Engineer A's design were not needed, and that a savings of $15,000 could be realized by ignoring their recommendations, just prior to signing the contract. According to the deposition, the owner never discussed the alleged over design with the City, Structural Engineer A or Geotechnical Engineer A and the discussion with the builder was the only attention given by the owner to Geotechnical Engineer A's report and Structural Engineer A's foundation design.

Evidence Obtained During the Repair

Damage to the house became noticeable in 1994. The owners retained Structural Engineer B and Geotechnical Engineer B to evaluate the damage and help stabilize the house. Geotechnical Engineer B discovered that the foundation of the wall shown on Fig. 3 was at approximate elevation 726.5. Structural Engineer A's drawing showed that the lowest level of the foundation was to be at elevation 722 with upward steps permitted based on an on-site approval of the bearing surface by a "qualified representative of Geotechnical Engineer A". Structural Engineer B required that the tie rods that are visible in Figs. 3 through 5 be installed to maintain the structural integrity of the house. Structural Engineer B also prepared a sketch dated September 26, 1994 showing an underpinning detail for the foundations of the house. The underpinning was installed and performed satisfactorily until it was removed to install the permanent underpinning, shown in Fig. 5, some time after July 29, 1996. The installer of the permanent underpinning confirmed that the walls were built using unreinforced block. One of the borings drilled for Geotechnical Engineer B determined that six feet of soft clay was present at a location where Geotechnical Engineer A had required its removal and replacement with granular, free draining, inert soils placed in a specified manner.

Issues Raised by the Expert Witnesses

There were no significant structural issues to be resolved. Geotechnical Engineer B became the expert witness for the owner, Geotechnical Engineer C was also retained as an expert witness for the owner and the Author became the expert witness for Geotechnical Engineer A.

Geotechnical Engineer B started with visual observation, did a later study based on drilling borings and a final study based on additional borings and instrumentation. He concluded that ground movements were limited to near surface materials. He concluded that "Readings from inclinometer well M-2 indicate that the failure plane has occurred between 9 and 11 feet or between elevations 713 and 711." He recommended remedial procedures to correct deficiencies in the weakened structure that was built.

Geotechnical Engineer C provided a preliminary slope study to the owner as part of a proposal for additional work at the property on March 27, 1995. The stability study was based on previously developed data and Geotechnical Engineer C's experience and understanding. Geotechnical Engineer C concluded that the results of the analysis were not believable and arbitrarily adjusted the soil strength parameters to decrease the resistance along the failure plane by a factor of 3 to coincide with the preconceived notion of a deep-seated failure plane. The proposed additional work was the construction of a retaining structure down slope of the house at a cost in the low six figures. The contractor submitting the proposal for the wall stated that the proposed repair was intended to work only in the absence of a deep seated failure. Instrumentation installed by Geotechnical Engineer B after the stability study and the performance of the house disproved the existence of a deep seated failure plane.

The Author disagreed with Geotechnical Engineer B's interpretation of the inclinometer data. The Author believed that the recorded inclinometer data was small enough to be within or close to the precision limits of the equipment used. The level to which the down hill wall had been temporarily underpinned had not been recorded. The Author was able to use other data to calculate the level of the underpinning. It coincided with the lowest foundation level required by Geotechnical Engineer A. Since the temporary underpinning had performed satisfactorily, it proved by an actual load test that Geotechnical Engineer A's foundation design criteria was appropriate. The reconstruction of the elevation of the temporary underpinning was a focus of attention during the Author's deposition. When the lawyers finally grasped the accuracy of the calculation, the owner's lawyer complained that with all the engineers involved no one had bothered to record the level.

The Present Condition of the House

A copyrighted aerial photograph of the house, taken in 2010, that is available on the Internet, shows the house in good condition. The decks seen in Fig. 3 are no longer present. No retaining structure is visible downhill from the house.

Conclusions

Designing work correctly does not ensure that it will be completed correctly. In our society that is an undesirable but necessary cost of doing business. Design professionals are governed by state registrations laws and the Code of Ethics of organizations such as ASCE. Geotechnical Engineer C's assertion of a deep seated failure was frivolous and added unnecessary cost to a project that was already troubled. A look at Figs. 1 and 2 shows that the weight of approximately 45 feet of soil was removed from the slope before the construction of the house. This shows that the assertion of a deep seated failure was even more egregious. Most professionals comply with registration laws and Codes of Ethics on their own. In the rare instances where this does not occur, enforcement of the registration laws and Codes of Ethics would benefit society and the profession.

Monitoring of Geohazards Impacting Highway Projects using TDR

Kevin M. O'Connor, P.E., Ph.D., M.ASCE[1]

Abstract: Due to risk to the traveling public, and liability concerns, it is not sufficient to be aware of a potential geohazard without timely updates and assessment of the risk involved. Real time monitoring of geohazards with Time Domain Reflectometry (TDR) is being implemented for a variety of applications specifically for risk assessment. TDR is cable radar which is used to interrogate coaxial cables thousands of feet in length. The cables are monitored with battery powered automated data acquisition systems. When deformation at any location along a cable exceeds an alarm level, this activity is recorded and downloaded to a base station server via the internet. Data is displayed on a web page for viewing by project personnel, and email notification can be sent whenever the magnitude of deformation reaches action levels. This paper summarizes highway projects where TDR is being used to monitor mine subsidence, sinkhole subsidence, slope movement, and bridge foundation scour. The projects illustrate the cost-effective benefits of remote, automated real time monitoring which include early warning of movement, safety of the traveling public, and liability limitation for owners.

Introduction

TDR is analogous to radar in a coaxial cable. Consequently, it is possible to display all reflections along a cable and identify the type and location of cable deformities producing these reflections. As shown in Fig. 1, a metallic coaxial cable can be placed in a drill hole and anchored to the walls by tremie placement of an expansive cement grout. When localized shear movements in the surrounding rock or soil are sufficient to fracture the grout, cable deformation occurs and can be detected using a TDR cable tester which launches a voltage pulse along the cable. At each location where deformation is occurring, a portion of the voltage is reflected back to the TDR unit which displays the reflections. Travel time of each reflection distinguishes the location(s) where cable deformation is occurring, and the reflected signal magnitude can be employed to quantify the magnitude of cable deformation (Dowding et al, 1988; O'Connor and Dowding, 1999). When a cable is crimped prior to placement in the hole as shown in Fig. 1, a reflection from each crimp serves as a distance reference marker in the TDR record.

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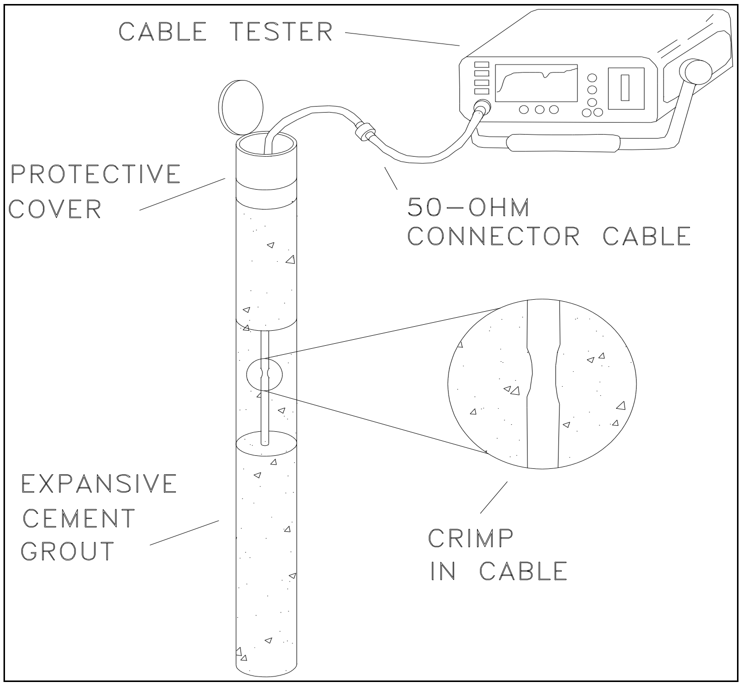

Fig. 1. Schematic of cable installation for deformation monitoring

Highway Applications

TDR technology has been used in a variety of applications for highways in diverse configurations. These have ranged from soil moisture measurements to conductivity measurements to deformation measurements (O'Connor and Dowding, 1999). This paper focuses on applications where TDR has been used to monitor ground movement associated with mine subsidence, karst sinkhole subsidence, abutment slope movement, and scour of bridge foundations.

Inactive Mine Subsidence – S.R. 91, Saltville, Virginia

Closure activities at the USG facility in Saltville, Virginia included realignment of an existing highway outside the predicted limits of long term subsidence (O'Connor et al., 2004). Concerns about the possibility of subsidence along State Route 91 being induced by construction activities, or in the former plant area where excavated rock was being placed, motivated the installation of a real time monitoring system (Fig. 2). Rapid development of sinkhole features was of particular concern because they posed nearly non-predictable and sudden safety hazards. The goal at Saltville was to provide an early warning system for catastrophic subsidence.

The sensitivity of TDR to rock mass movement is related to the proximity of cable installations to subsiding ground. The lateral extent of monitoring is enhanced by installation of cables in trenches over a wide area, and the depth of monitoring is enhanced by installation of cables in deep vertical or angled holes beneath critical structures.

Fig. 2. Installing cable in a trench along SR 91; cables were also installed in angled holes drilled below the road from the berm (see Fig. 3)

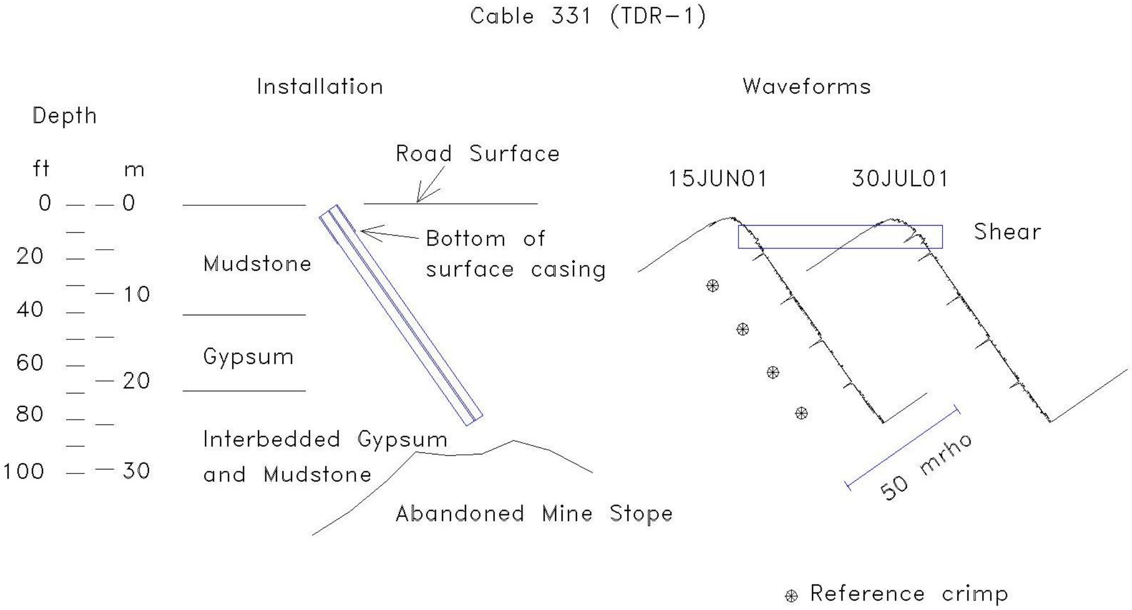

Fig. 3. TDR waveforms associated with shearing of cable installed in angled hole

Quantification of cable deformation due to ground movement can be illustrated by cable TDR1 which was installed in an angled hole (Fig. 3) and was sheared as subsidence occurred. The deformation occurred at a depth of 10 feet (3 m) at the bottom of the surface casing that was left in place and produced a TDR reflection that increased in magnitude over time. Also note the reflections associated with reference crimps made every 20 feet (6 m) along the cable during installation. A time history of the cable deformation is plotted in Fig. 4 (TDR reflection magnitude was converted to deformation magnitude using calibrations that have been developed). It can be seen that accelerated movement correlated with precipitation events. The rate of movement decreased from

0.04 mm/day to 0.01 mm/day which was similar to the rate of movement measured with precise levels surveys (0.06 mm/day to 0.03 mm/day)

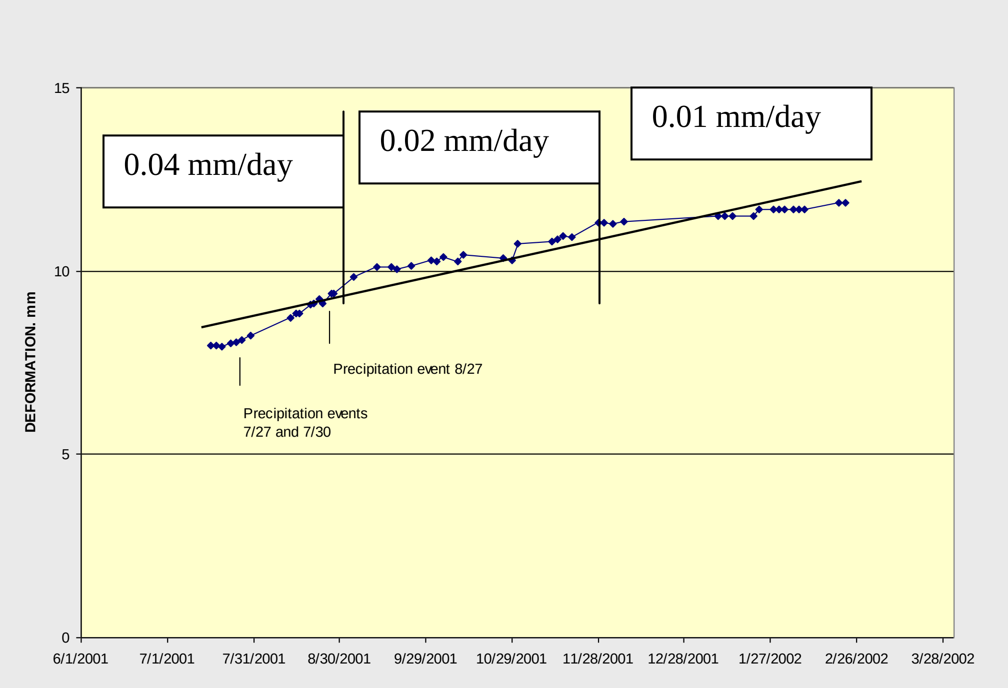

Fig. 4. Time history of shear deformation magnitude at a depth of 10 feet

Manual monitoring of three cables began in June 2001. The system was ultimately expanded to automated real time monitoring of 21 cables with lengths that vary from 10 m (30 ft) to 270 m (886 ft) for a total of over 2500 m (8200 ft) of cable. They were installed in angled holes beneath the existing highway, in trenches along the highway, and in trenches over the former plant area where rock excavated for the new alignment was being placed. The automated monitoring incorporated a call back capability. Whenever the difference between the baseline profile and the current profile at any location along a cable exceeded a preset alarm threshold, the datalogger would initiate a call to assigned personnel.

Monitoring Subsidence in Karst – I-70, Frederick, Maryland

The area around Frederick, Maryland has historically been impacted by the development of sinkholes. The area has an abundance of active, cover-collapse sinkholes – a situation complicated by population growth and concomitant development pressures. Altering surface drainage and lowering the groundwater table by quarry activities exacerbate the hazard (Brezinski et al, 2004).

This project dates back to 1985 when the Maryland State Highway Administration (SHA) initiated planning studies for interchange improvements and widening of I-70 through the Frederick area. Particular attention was given to the issues of storm water management and risk of sinkholes. The studies determined that, in this area, no runoff left the area as surface water, and numerous sinkholes were found. Observations indicated that, during 100 year storms, the roadway additions would produce a volume of runoff that would exceed the acceptance capacity of the sink system and flooding would occur. Also, the use of sinks to receive highway drainage was no longer acceptable from both environmental and roadway stability perspectives.

SHA investigated several alternatives for handling the storm water runoff. The plan for disposal into Carroll Creek, which parallels the roadway to the north, was selected even though it required pumping facilities and large storage ponds. This required that the Monocacy Boulevard embankment be constructed and certified as a dam. Additional the storage ponds (Fig. 5) would need to be lined to prevent seepage into the underlying soils.

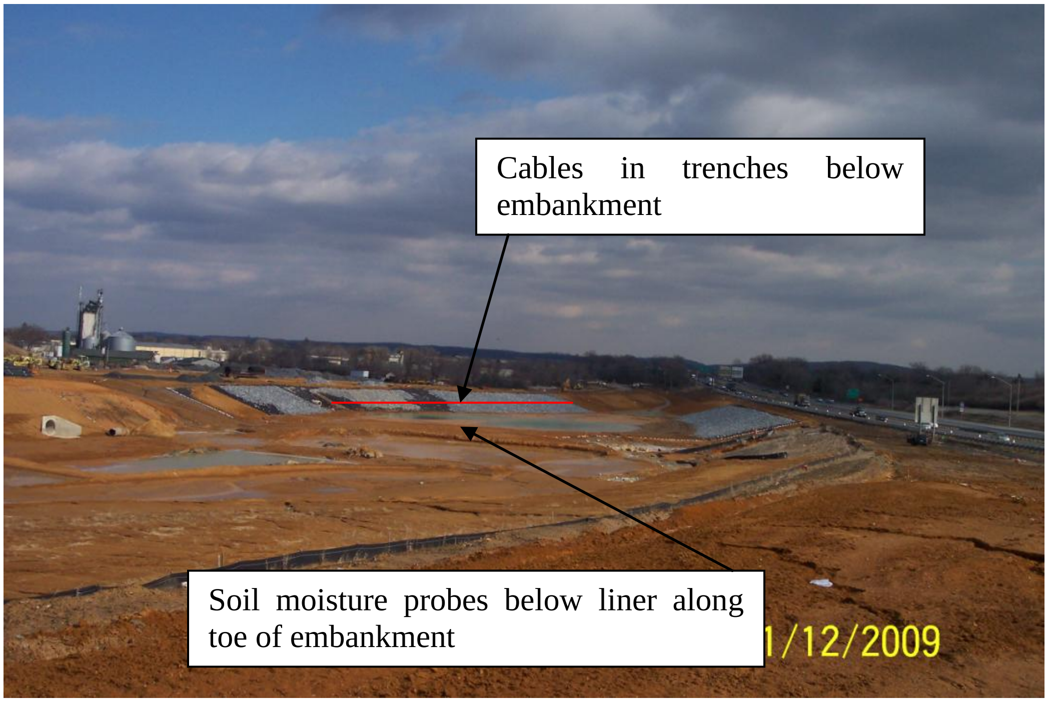

Fig. 5. Cables were installed in trench prior to construction of Monocacy Boulevard embankment; I-70 is on the right side of the photograph

Instrumentation was installed under areas of the Monocacy Boulevard embankment (background in Fig. 5) and the Route 85 embankment. The purpose of the instrumentation was to monitor the embankment for ground movement. The monitoring systems consist of coaxial cables placed in trenches backfilled with flowable backfill beneath the two embankments (the Monocacy Boulevard embankment is shown in Fig. 5). The cables are connected to two remote sinkhole alert system stations (one is shown in Fig. 6) and monitored using TDR. The four cables installed under Monocacy Blvd were each 800 feet (244 m) long. The four cables under Rt 85 were each 250 feet (76 m) long.

Soil moisture sensors were installed below the clay liner along the toe of the Monocacy Boulevard embankment (Fig. 5) to monitor for leaks in the pond liner. The soil moisture sensors were installed in a trench before placement of the liner and connected to the DAS shown in Fig. 6. Consequently, moisture spikes measured at these sensor locations could be associated with subsurface seepage below the Monocacy Blvd embankment and below the liner. It is anticipated that in the case of a leak developing in the liner that several adjacent soil moisture sensors would respond simultaneously.

Fig. 6. Data acquisition system on north berm of Monocacy Boulevard embankment

Data is downloaded via wireless modem to an SHA server and plots are viewed on any computer connected to the SHA network.

Abandoned Mine Subsidence – I-70, Zanesville, Ohio

In the area east of Zanesville, soil profile sheets for the original 1962 construction of I-70 indicated that coal was outcropping within the natural slopes. Construction diaries from the 1962 and 1963 original mainline construction, both eastbound and westbound, indicated that subgrade subsidence occurred during construction in the area noted by subsidence in Fig. 7. As a result of the subsidence, the mainline was block cut immediately beneath the areas designated for pavement to the depth of the mined interval and replaced with compacted engineered fill. However, no mine remediation measures were applied to areas immediately outside of the pavement footprint or east of the block cut area.

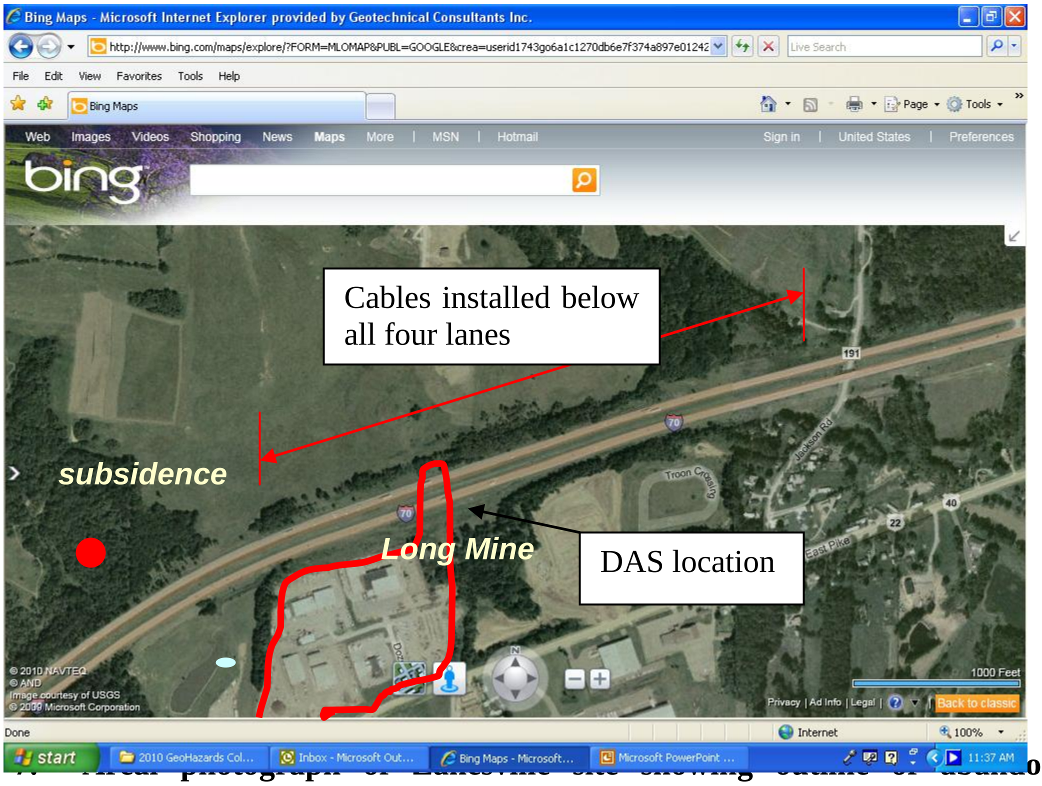

Fig. 7. Aerial photograph of Zanesville site showing outline of abandoned underground mine and a subsidence sinkhole on the north side of I-70

Historic mine maps for the area on file with the Ohio Department of Natural Resources, Division of Geological Survey, indicated that three mapped areas of previous underground coal workings are located within a 500 ft (150 m) buffer from the project limits of these workings. One map indicated that a haulage way is located under IR 70. A second underground mine map for the Long Mine (Fig. 7) revealed a set of haulage ways extending across all four lanes of IR 70 with mine rooms constructed to the west under westbound IR 70. The remaining maps do not show any direct workings under the roadway, but contain notations about areas being worked out or mined out east of the haulage ways in the direction of the roadway.

A TDR system was installed to monitor any vertical earth movement/subsidence that may occur (similar to the system discussed in O'Connor et al, 2002). Installation involved placing a prepared coaxial cable in a horizontal directional drill (HDD) hole (with grout) as shown in Fig. 8. One cable was installed under each lane, 4 cables east of the DAS location (Fig. 7 and Fig. 10), and 4 cables west of the DAS location, in HDD runs of 800 to 1000 feet (245 to 305 m), cable was pulled back into each hole and grouted (Fig. 9). After splicing cable runs together, the total cable lengths ranged from 1800 to 3000 feet (550 to 915 m).

Fig. 8. Horizontal direction drilling below eastbound lanes of I-70

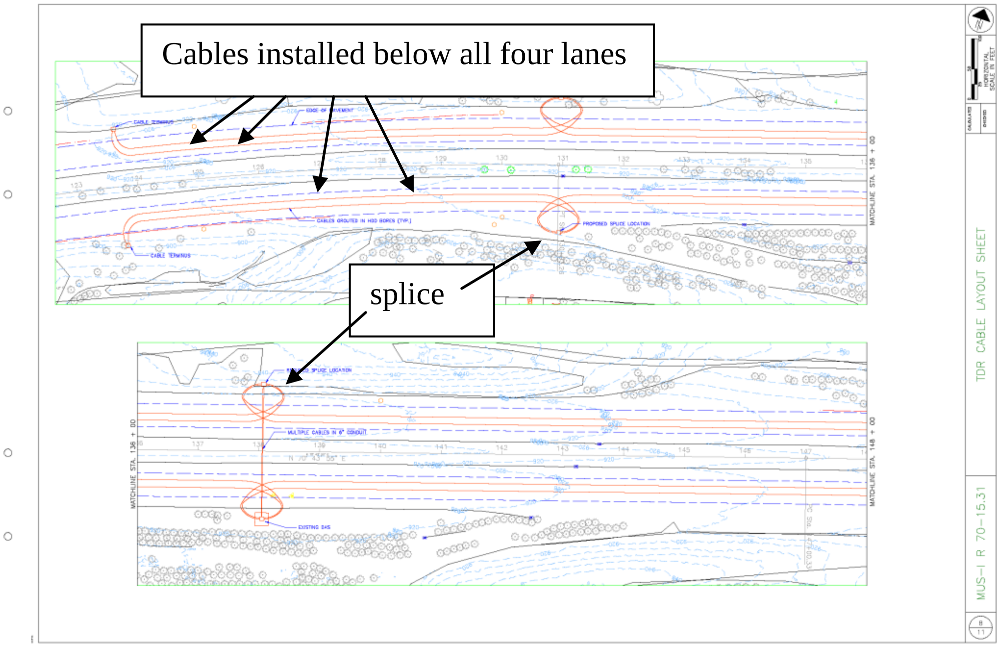

Fig. 9. Cables were pulled into HDD holes and runs were spliced together

The system is composed of the eight spliced coaxial cables connected to a multiplexer and a TDR cable tester, which are controlled by a programmable datalogger (Fig. 10). Remote communication with the datalogger is via a wireless modem.

Fig. 10. The head ends of the eight spliced coaxial cables were connected to the data acquisition system (DAS)

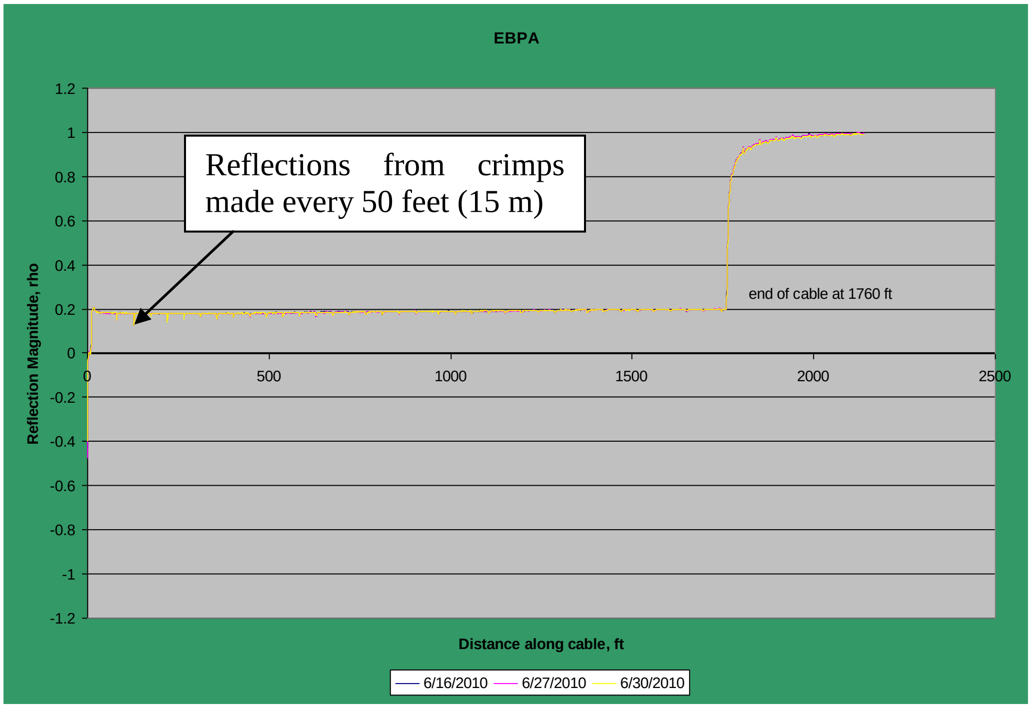

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Fig. 11. TDR waveform for one cable; note reflections from crimps made every 50 feet (15 m) along cable during installation

Each coaxial cable was crimped every 50 feet (15 m) to create a known pattern of reference points in the TDR waveform (Fig. 11). The system interrogates each coaxial cable and logs each location along a cable where deformation is occurring that exceeds a preset alarm level. The log file includes the interrogation date and time, cable ID, location(s) along the cable, and the magnitude of deformation at each location. The log file is downloaded over the internet through the system's IP address, and a notification is sent to designated personnel.

Abutment Slope Movement – I-64, Sulphur, Indiana

The abutments of two highway bridges on I-64 in Sulphur, Indiana were constructed by building embankments over existing soils on the west abutment and over rock on the east abutment. The embankment fill consists of stiff silty clay and rock fragments and, on the west abutment, it overlies native alluvial soft clay and looses sand. The underlying rock on both abutments is shale and limestone. The shale is soft and erodible while the limestone is conspicuous as ledges in exposures along the highway.

Inclinometer casing and coaxial cables were installed in separate holes on both the east and west (foreground in Fig. 12) abutment slopes. Four cables were installed in holes that ranged from 68 to 95 feet (21 to 29 m) deep.

Fig. 12. Drilling vertical hole in west abutment slope to install coaxial cable.

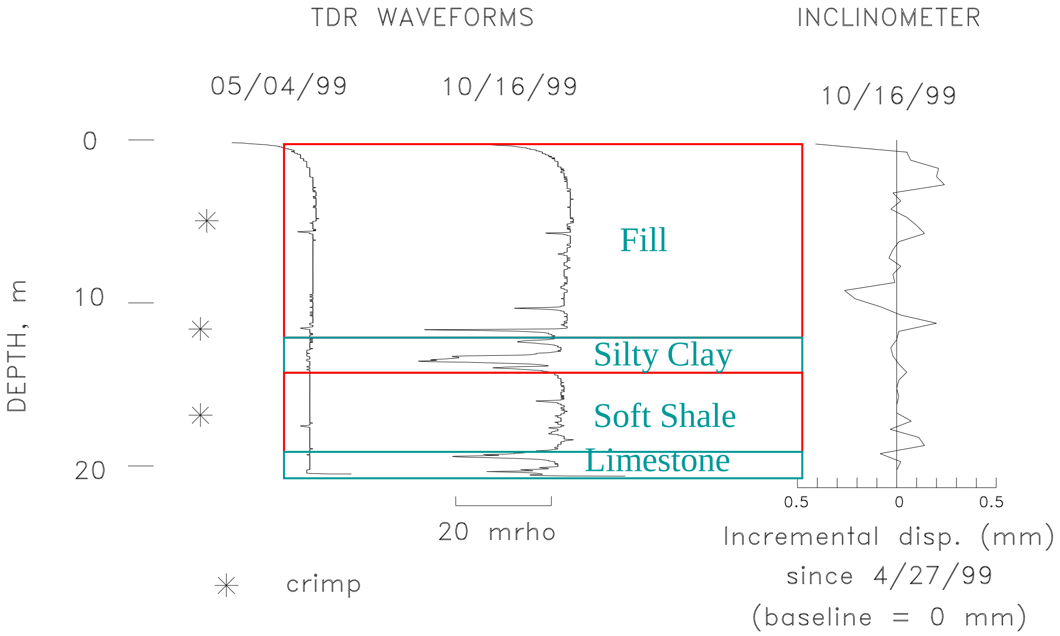

Fig. 13. TDR waveforms for cable installed on east abutment slope; inclinometer plot is shown for comparison.

TDR data was acquired with cable tester and laptop computer at the site. The TDR waveforms and inclinometer incremental displacement profile shown in Fig. 13 were acquired on the east abutment where the embankment was constructed over rock. Both the coaxial cable and inclinometer casing were deformed at depths of 12 and 19 m (40 and 62 ft). However, deformation of the cable was also detected at depths of 11, 13, 16, and 20 m (36, 43, 53, and 65 ft). The fill extends to a depth of 13 m (43 ft) where it rests on the soft shale which is underlain by limestone at a depth of 20 m (65 ft). The water table is located at the top of the shale layer. The largest TDR reflections correspond to depths at which contacts exist, 29 and 21 mrho at a depth of 19 m (65 ft) at the contact between shale and limestone. By comparison, the inclinometer incremental displacement was only 0.2 mm at depths of 12 and 19 m (40 and 62 ft). This implies a sensitivity which varies between 21 mrho / 0.2 mm to 29 mrho / 0.2 mm or 105-145 mrho / mm. These values are considered to be unusual since a typical value (based on laboratoryand field studies) is 16.5 mrho / mm. Reasons for such a large difference in response between TDR and inclinometers are discussed in the conclusion.

Scour of Bridge Foundation – Horse Creek, California

Long term movement of a bridge on Highway 96 over the Klamath River had been documented in inspection reports. Pier 2 (background in Fig. 14) had been moving toward the river and the rocker had been rotated clockwise to within 50 mm (2 in.) of its limit of rotation. During the summer of 1997, an over-height truck caused damage to each of the transverse truss members of the bridge. During inspection of this damage, the Caltrans Maintenance Supervisor noted that the bridge was tilted over Pier 2. Upon taking measurements, it was found that this pier had translated 305 mm (12 in.) horizontally downstream and settled 406 mm (16 in.). It was discovered that there was a scour pocket beneath Pier 2. Furthermore, there were horizontal cracks indicating that a hinge had formed and the pier rotated.

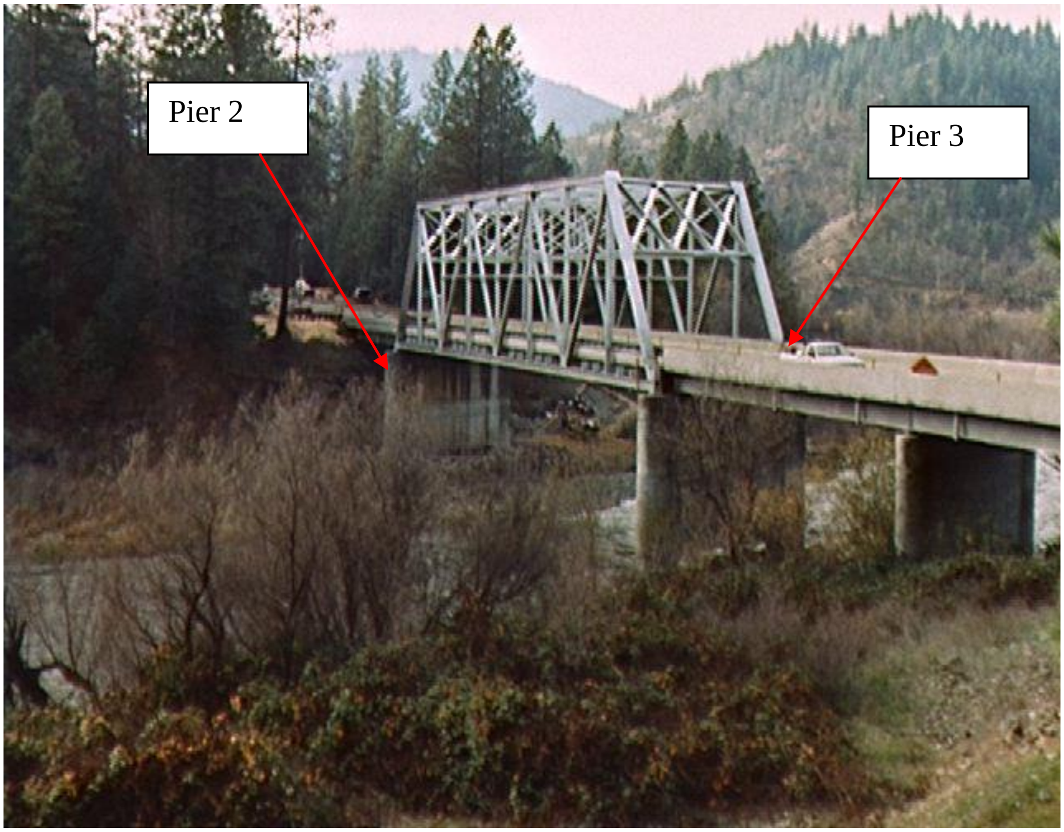

Fig. 14. Bridge over Klamath River; scour pocket developed under Pier 2 in the background; cables was installed through the foundation of Pier 3 in foreground.

Under an emergency contract, a contractor was retained to repair Pier 2 and relevel the bridge. Water was pumped from the scour pocket which was found to be 1.8 m deep by 9 m wide by 18 m long (6 ft deep by 30 ft wide by 60 ft long). 67 m^3 (88 $yd^{3)}$ of concrete was pumped into the pocket. Under the direction of the Caltrans Resident Engineer, the original scheme for releveling was modified. Eight 1-m (36-in). diameter by 18 m (60 ft) long cast-in-place drilled piers were constructed; one at each corner of the two pier footings. Using the drilled piers as a reaction, Pier 2 and the bridge were jacked back into position. After releveling was completed, riprap was placed around Piers 2 and 3 for scour protection.

There were few complications while constructing the cast-in-place piers and the operation went so smoothly that appropriated funds were available for instrumentation and research prior to any further work on Pier 3. The decision was made by Caltrans to install monitoring cables beneath Pier 3 to monitor for scour of the graphitic schist rock.

A major design consideration was physical location of the hole location. The exploration borings were done by Caltrans by drilling an access hole through the bridge deck then installing steel casing from the deck down into the river bed. The hole was advanced through this casing. This approach was adopted to install the TDR monitoring cable since the casing could be left in place to act as a protective steel conduit for the cable. Furthermore, the hole would be placed inside the curtain wall to provide additional protection. There was limited access to accomplish this since it was not desired to cut any holes in the steel girders supporting the bridge.

It was proposed that a monitoring cable should be installed to monitor the following modes of deformation:

1. shear along the interface between the concrete footing and graphitic schist,
2. shear along a bearing capacity slip surface within the schist which was approximated as a circular surface with a radius of 1.5 m (equal to one-half of the footing width of 3 m), and
3. assume that the rock adjacent to the footing is scoured out and bearing capacity failure occurs as shear along a circular surface in the schist with a radius equal to the footing width of 3 m (10 ft).

Furthermore, based on information gathered prior to, and during, the site visit it was also apparent that the bridge had the capacity to remain standing even though one footing of Pier 2 had been completely undermined by scour. The possibility exists that scour beneath Pier 3 could occur without any of the hypothesized modes of deformation occurring. So, the cable would also need to monitor:

4. scour of the graphitic schist beneath the upstream footing (of Pier 3) without deformation within the rock or movement of the pier.

A 124 mm (4-7/8 in.) diameter hole was drilled through the deck (Fig. 15) and top of the concrete pier and to a depth of 305 mm (1 ft) in the concrete footing. Wireline casing was set and a 102 mm (4-in.) diameter hole was drilled through the concrete footing to a depth of 8 m (26 ft) into the graphitic schist. The casing was removed and a 102 mm (4-in). ID steel pipe was placed inside the curtain wall between the hole in the footing and hole at the top of the pier. Solid aluminum foam dielectric coaxial cable was installed inside the steel pipe to the bottom of the hole and grout was tremied into the hole and steel pipe (Fig. 15 and Fig. 16).

Fig. 15. After placing the cable, grout was tremied into the hole below the footing and into the steel pipe between the top of footing and top of pier

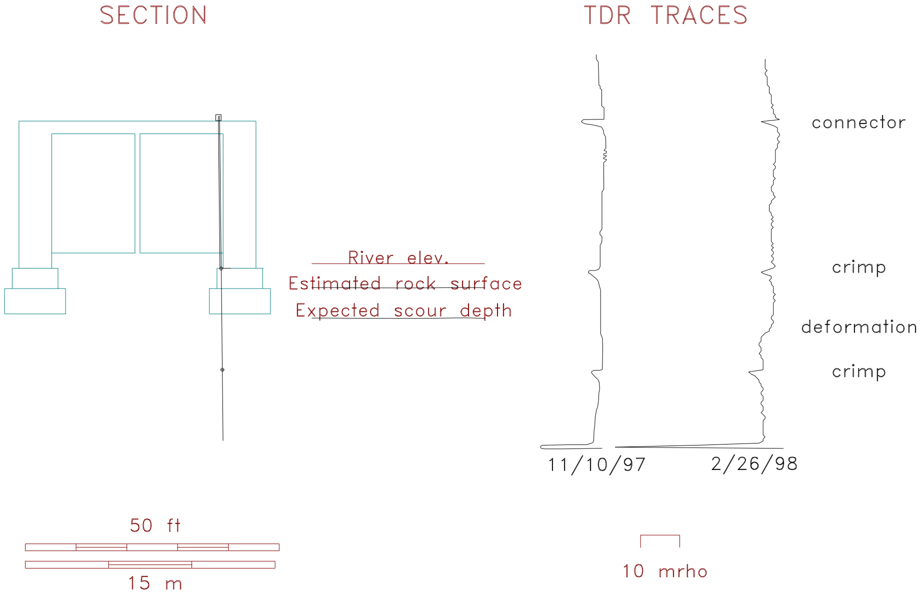

Fig. 16. Schematic of cable installation and TDR waveforms

During on-site discussion of cable installation, data acquisition and interpretation, the author made Caltrans aware of other pertinent applications of TDR technology. In particular, the use of air-dielectric cable to monitor water level in wells (Dowding et al, 1996a; Nicholson et al, 1997). This stimulated discussion and this application was implemented for the following reasons: (1) river water levels would vary daily and these changes would verify that the monitoring system was operating properly, (2) changes in water level could be correlated with deformation, and (3) many agencies would be interested in the water level information.

A 25 mm (1 in.) ID steel pipe was attached to the outside of the 102 mm (4 in.) diameter steel pipe and an air dielectric coaxial cable was placed in this pipe. The bottom of the pipe is resting on the top of the footing. Lead cables extend across the bridge to the DAS installed on the west abutment (background in Fig. 14).

Real Time Monitoring and Alarm

When geotechnical measurements are used for activation of alarm systems, the rationale for the measurements is significantly different from the rationale for performance monitoring. Measurements must be made in real time and be available to several responsible parties simultaneously (O'Connor, 2006; O'Connor et al., 2001).

Algorithms can be programmed into dataloggers or it may be more efficient to implement the algorithms at base stations. A basic algorithm based on TDR measurement of cable deformation is illustrated in Fig. 17 where the current waveform and baseline waveform are plotted. The baseline waveform is stored in the datalogger which computes the difference between the current and baseline waveforms. Whenever the difference at any point is greater than the specified value, this is considered to be an alarm condition at that location.

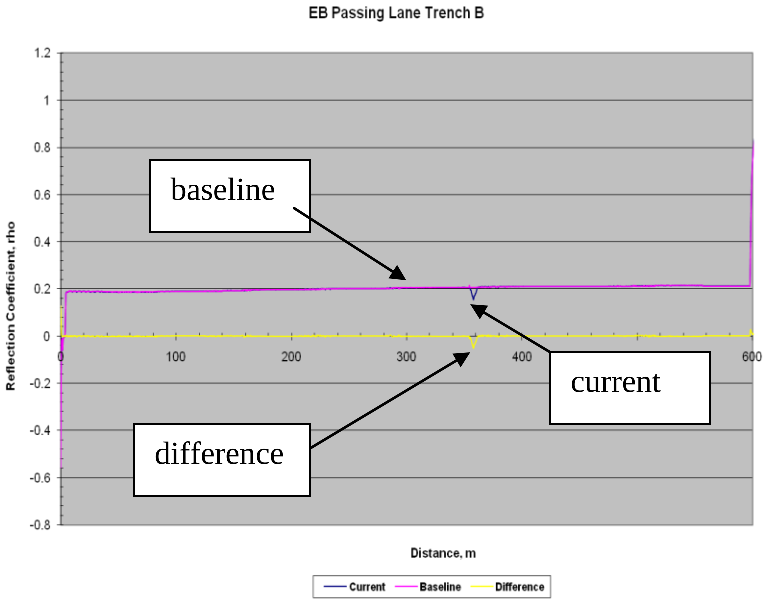

Fig. 17. Baseline waveform, current waveform with a reflection at 360 m, and the difference waveform

In response to alarm calls from the TDR monitoring systems installed at Saltville, Virginia, it was vital to isolate the cause. It was necessary to download and analyze TDR data files to determine if the alarm condition was associated with ground movement and the rate at which movement was occurring. Based on this information, it was possible to make an informed decision when talking with site personnel about the appropriate plan of action. Once it was determined that the alarm call was triggered by a condition that is persistent, it is typical to validate the condition using on-site manual interrogation of the coaxial cable, identify the physical location, conduct a visual assessment, and develop a plan of action.

Action plans are developed and refined as data is collected and action levels are refined. The data collected varies from real time downloading of large data files to real time reception of a small data packet. Systems have been installed with features that reduce spurious alarms. In Manhattan, Kansas, redundant cables were installed in a single trench, and alarm conditions must exist simultaneously on both cables to activate an alarm call to the base station. The interrogation cycle needed to be repeated once every ten minutes, so data is not stored during each cycle. It is only stored once every three hours, and the alarm notification is sent to another remote acquisition system which then requires notification by several other sensors to activate a system.

Recently, another approach has been utilized in which activity log files for each cable are maintained by the datalogger and downloaded whenever they updated. These files are a log of activity for the cable in which the location along the cable and magnitude of deformation are logged with a time stamp. The activity files are sent via email to assigned personnel.

Conclusion

Geotechnical applications of TDR are continuing to evolve and usage is increasing, particularly for monitoring slope movement. It has evolved into real time monitoring with alarms and a variety of notification schemes. This has created the need to develop action plans based on various alarm levels.

Quantification of deformation is based on extensive laboratory calibration testing and correlation with inclinometer measurements (O’Connor and Dowding, 2000). Quantification of water level monitoring is based on laboratory and field studies (Dowding et al, 1996a; Nicholson et al., 1997).

Comparison with Inclinometers

Plastic inclinometer casing and solid metallic coaxial cables deform differently when subjected to very localized shearing. Metallic coaxial cable deforms easily when subjected to highly localized shear and has been found useful in rock where deformation occurs along joints, bedding planes, and fractures. On the other hand, inclinometer probes are sensitive to gradual changes in inclination of the inclinometer casing. Localized shearing of inclinometer casing causes kinking such that a probe cannot be moved through the deformed casing. The thinner the localized shear zone, the greater the TDR response and the smaller the slope inclinometer response (O’Connor and Dowding, 1999). Thus, in situations involving both general shear and localized shear, it should be expected that the two technologies will respond differently (Dowding and O’Connor, 2000).

Comparison of slope inclinometer and TDR responses for cases presented by Dowding and O'Connor (2000) indicate that both technologies provide useful information. TDR technology is especially sensitive to localized shear so it is most responsive to concentrated shear strain. On the other hand, slope inclinometers are especially sensitive to gradual changes in inclination so they are most responsive in soils undergoing general shear. TDR technology will also respond to abrupt changes in shear strain at the boundaries of a thick shear band. These differences do not imply that either technology is more correct; rather, the two technologies respond optimally under different conditions.

Finally, solid aluminum coaxial cables can be installed in deformed inclinometer casing to allow continued monitoring. The results of installing and monitoring coaxial cables installed in deformed inclinometer casing indicate that this will be effective whether the casing has been installed in rock or in soil. Such retrofitting allows continued monitoring of subsurface deformation without the need to drill additional holes.

Typical Costs

Coaxial cables have been developed for a variety of uses in large volumes so costs are low due to the economy of scale created by industrial users. There are several manufacturers of TDR units which are a basic tool in the communications industry. A major development was the remote TDR unit that is available which can utilized with a programmable datalogger for automated remote monitoring (Campbell Scientific, 2006; Dowding et al, 1996b; Nicholson et al, 1997). The following table itemizes (a) typical hardware costs, and (b) installed cost for cable used for deformation monitoring.

Table 1. Typical Costs

MTDR unit	$3,600
Cable on spool	$1 / ft
Remote DAS	$10,000
INSTALLED CABLE COST	
Trench	$7 / ft
Vertical hole	$15-$25 / ft
Angled hole	$30-$45 / ft
Inside inclinometer casing	$3 - $5 / ft

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CHRONOLOGY OF OHIO RIVER VALLEY SOIL SEMINARS

ORVSS I	BUILDING FOUNDATION DESIGN AND CONSTRUCTION, October 16, 1970, Lexington, KY
ORVSS II	EARTHWORK ENGINEERING, START TO FINISH, October 15, 1971, Louisville, KY
ORVSS III	LATERAL EARTH PRESSURES, October 27, 1972, Fort Mitchell, KY
ORVSS IV	GEOTECHNICS IN TRANSPORTATION ENGINEERING, October 5, 1973, Lexington, KY
ORVSS V	ROCK ENGINEERING, October 18, 1974, Clarksville, IN
ORVSS VI	SLOPE STABILITY AND LANDSLIDES, October 17, 1975, Fort Mitchell, KY
ORVSS VII	SHALE AND MINE WASTES: GEOTECHNICAL PROPERTIES, DESIGN, AND CONSTRUCTION, October 8, 1976, Lexington, KY
ORVSS VIII	EARTH DAMS AND EMBANKMENTS: DESIGN, CONSTRUCTION AND PERFORMANCE, October 14, 1977, Louisville, KY
ORVSS IX	DEEP FOUNDATIONS, October 27, 1978, Fort Mitchell, KY
ORVSS X	GEOTECHNICS OF MINING, October 5, 1979, Lexington, KY
ORVSS XI	EARTH PRESSURE AND RETAINING STRUCTURES, October 10, 1980, Clarksville, IN
ORVSS XII	GROUNDWATER: MONITORING, EVALUATION AND CONTROL, October 9, 1981, Fort Mitchell, KY
ORVSS XIII	RECENT ADVANCES IN GEOTECHNICAL ENGINEERING PRACTICE, October 8, 1982, Lexington, KY
ORVSS XIV	FOUNDATION INSTRUMENTATION AND GEOPHYSICAL EXPLORATION, October 14, 1983, Clarksville, IN
ORVSS XV	PRACTICAL APPLICATION OF DRAINAGE IN GEOTECHNICAL ENGINEERING, November 2, 1984, Fort Mitchell, KY
ORVSS XVI	APPLIED SOIL DYNAMICS, October 11, 1985, Lexington, KY
ORVSS XVII	NATURAL SLOPE STABILITY AND INTRUMENTATION, October 17, 1986, Clarksville, IN
ORVSS XVIII	LIABILITY ISSUES IN GEOTECHNICAL ENGINEERING AND CONSTRUCTION, November 6, 1987, Fort Mitchell, KY
ORVSS XIX	CHEMICAL AND MECHANICAL STABILIZATION OF SOIL SUBGRADES, October 21, 1988, Lexington, KY
ORVSS XX	CONSTRUCTION IN AND ON ROCK, October 27, 1989, Louisville, KY
ORVSS XXI	ENVIRONMENTAL ASPECTS OF GEOTECHNICAL ENGINEERING, October 26, 1990, Fort Mitchell, KY
ORVSS XXII	DESIGN AND CONSTRUCTION WITH SYNTHETICS, October 18, 1991, Lexington, KY
ORVSS XXIII	IN-SITU SOIL MODIFICATION, October 16, 1992, Louisville, KY
ORVSS XXIV	GEOTECHNICAL ASPECTS OF INFRASTUCTURE RECONSTRUCTION, October 15, 1993, Fort Mitchell, KY

CHRONOLOGY OF OHIO RIVER VALLEY SOIL SEMINARS (CONTINUED)

ORVSS XXV	RECENT ADVANCES IN DEEP FOUNDATIONS, October 21, 1994, Lexington, KY
ORVSS XXVI	SITE INVESTIGATIONS: GEOTECHNICAL AND ENVIRONMENTAL, October 20, 1995, Clarksville, IN
ORVSS XXVII	FORENSIC STUDIES IN GEOTECHNICAL ENGINEERING, October 11, 1996, Cincinnati, OH
ORVSS XXVIII	UNCONVENTIONAL FILLS: DESIGN, CONSTRUCTION AND PERFORMANCE, October 10, 1997, Lexington, KY
ORVSS XXIX	PROBLEMATIC GEOTECHNCIAL MATERIALS, October 16, 1998, Louisville, KY
ORVSS XXX	VALUE ENGINEERING IN GEOTECHNICAL CONSULTING AND CONSTRUCTION, October 1, 1999, Cincinnati, OH
ORVSS XXXI	INSTRUMENTATION, September 15, 2000, Lexington, KY
ORVSS XXXII	REGIONAL SEISMICITY AND GROUND VIBRATIONS, October 24, 2001, Louisville, KY
ORVSS XXXIII	GROUND STABILIZATION AND MODIFICATION, October 18, 2002, Covington, KY
ORVSS XXXIV	APPLICATIONS OF EARTH RETAINING SYSTEMS AND GEOSYNTHETIC MATERIALS, September 19, 2003, Lexington, KY
ORVSS XXXV	ROCK ENGINEERING AND TUNNELING, October 20, 2004, Louisville, KY
ORVSS XXXVI	GEOTECHNICAL INNOVATIONS IN TRANSPORTATION ENGINEERING, October 14, 2005, Covington, KY
ORVSS XXXVII	INNOVATIONS IN EXPLORATION OF SUBSURFACE VOIDS, October 27, 2006, Lexington, KY
ORVSS XXXVIII	CIVIL INFRASTRUCTUREAND THE ROLE OF GEOTECHNICAL ENGINEERING, November 14, 2007, Louisville, KY
ORVSS XXXIX	URBAN CONSTRUCTION, October 17, 2008, Covington, KY
ORVSS XL	GEOTECHNICAL ENGINEERING AND ENERGY INFRASTRUCTURE, November 13, 2009, Lexington, KY
ORVSS XLI	NATIONAL INFRASTRUCTURE: DAM AND LEVEE SAFETY, October 20, 2011, Louisville, KY
ORVSS XLII	LESSONS LEARNED: FAILURES AND FORENSICS, October 21, 2011, Cincinnati, OH