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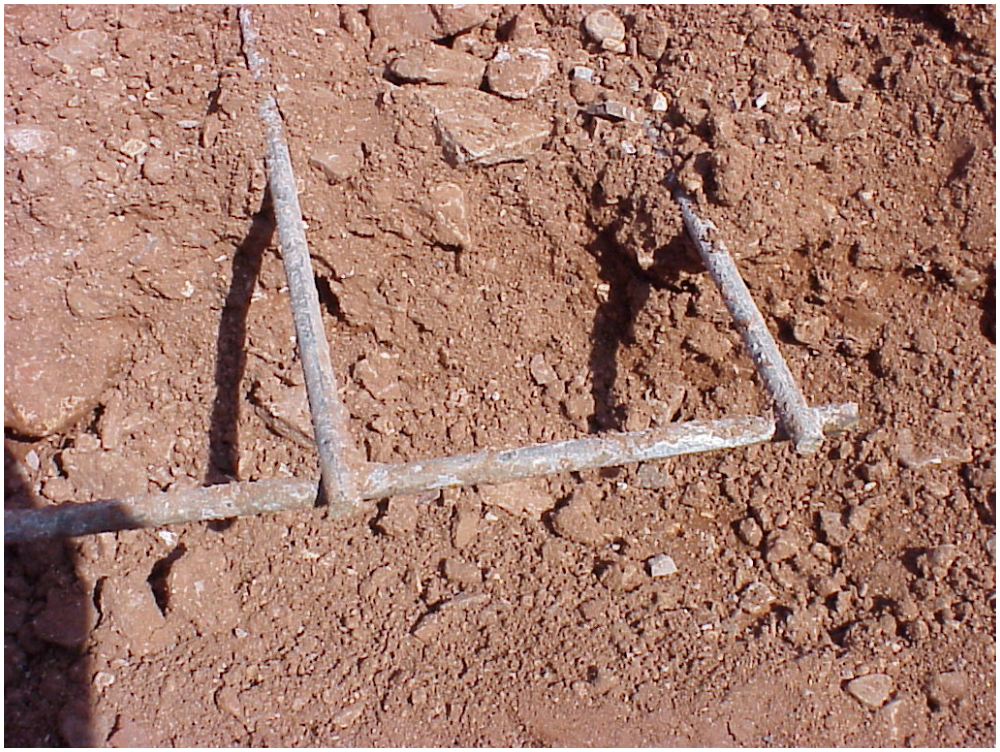

Corrosion Monitoring of Hot Springs VSL Mechanically Stabilized Earth Wall

Study SD2004-02
Final Report

Prepared by
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South Dakota Department of Transportation
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16. Abstract

In 2004 the replacement of the PCC pavement atop a VSL Retained Earth™ wall in Hot Springs, SD provided an opportunity to evaluate the performance of the galvanized reinforcement mesh. Concerns about deicing salt infiltration and high levels of sulfate in the granular backfill prompted an examination of reinforcement mesh in the wall. This was further complicated by a settlement problem in the wall which occurred during construction and created concerns with respect to corrosion of the concrete panel connectors as well as the bent mesh adjacent to the wall panels. The initial plan was to install a corrosion monitoring system in the wall to provide data as to expected remaining life if the corrosion present was severe. Initial examination of mesh near the abutments of a bridge connecting wall elements showed no evidence of corrosion occurring. The installation of the system was put on hold until excavation and coring was accomplished. All samples of mesh examined and tested demonstrated a remarkable lack of corrosion with little loss of galvanizing.

The corrosion monitoring program was abandoned as it was deemed superfluous. The unique properties of the red shale (Spearfish) used as backfill combined with the relatively free draining nature of the backfill near the wall panels (due to an interconnected system of voids) resulted in no significant penetration of deicing chemicals into the backfill and a relatively dry environment within the backfill. In fact, no area of excavated backfill exhibited *in situ* moisture at the first level of mesh reinforcement.

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Introduction

In 1983, a VSL Mechanically Stabilized Earth (MSE) retaining wall was completed in the city of Hot Springs, South Dakota as a replacement for an aged viaduct built in the 1930's. A reinforced earth retaining wall was selected as the optimal solution to three problems:

1) The cost for an equivalent replacement structure was prohibitive
2) The decorative panel façade of the wall would provide an aesthetically appropriate blend with the local sandstone building stone construction required as part of an Historic Preservation District Ordinance.
3) Lack of available right-of-way adjacent to the old viaduct which limited construction options.

Unfortunately, the wall suffered significant settlement prior to opening to traffic causing severe longitudinal cracking in the concrete pavement constructed atop the wall and compelling the replacement of several concrete facing panels and raising questions with respect to reinforcing mesh corrosion, voids along the interior of the wall panels, bent reinforcing grid along the wall faces and, especially, corrosion of the mesh attachment sites along the wall facing. The mesh was attached using threaded ferrules connected to coil bolts embedded in the panel concrete and, though these were galvanized, water infiltration through the voids system made their condition after years problematic. In 2004 Project NH0018(117)39 was initiated to replace the portland cement concrete pavement atop the wall and several approach slabs to two structures connecting wall segments. At the time of plans development a decision was made to include tie-back inspection of the retaining wall as an initial phase of construction with provision to proceed based on the results of that inspection. The evaluation process was accomplished as part of this research project.

Background

At the time of original construction, the wall design was based on a proprietary system from Reinforced Earth Company™ using reinforcing straps as the soil reinforcement medium. The VSL system, Retained Earth™, was accepted as a value engineering alternative and retrofit onto the existing plans. One primary difference between the systems at that time was the use of ASTM A-185 welded wire mats in the VSL system made up of ASTM A-82 wire with a diameter of ⅜" meeting the requirements of ASTM A-123 for hot dip galvanizing. Construction began in the late summer of 1982 with a cascade of problems on the project including vertical and horizontal alignment of panels, compaction, lift thickness, mesh alignment, application of corrosion protective grease on the ferrules, repair of damaged mesh galvanizing and installation of instrumentation. A significant error occurred when the abutment footing of an overpass structure connecting wall segments was cast one foot out of alignment, necessitating a redesign and lengthening of the bridge. On May 5, 1983 the falsework was removed from this structure and immediate settlement became apparent. A settlement of 0.3 feet on the abutment occurred at the misaligned footing with a settlement of 0.2 feet on the other abutment. The wall section adjacent settled as well but errors in instrumentation installation and set-up made the degree of settlement difficult to ascertain. The confined backfill settled with respect to the foundation soils resulting in severe longitudinal cracking of the portland cement concrete roadway above. This project was not a propitious start for MSE walls in South Dakota.[(1)]

Corrosion Concerns

The VSL proposal had specified an average galvanized coating thickness of 2.3 oz/ft of mesh with no galvanizing less than 2.0 oz./ft. The granular backfill used for the wall came from the Nelson Pit and had the following chemical properties:

Granular Backfill		Georgia DOT Specification
pH (24 hours)	7.82	6.0-9.5
pH (48 hours)	8.82	
Resistivity (24 hours)	8333 ohm-cm	>10,000
Resistivity (48 hours)	6250 ohm-cm	
Chloride	3.6 ppm	<20

Sulfate	14 ppm	<15
Alkalinity ($CaCO_3$)	57 ppm	$CaCO_3$ acidity > 15 ppm

The original granular backfill specifications in the plans called for resistivity > 3,000 ohm-cm with a pH between 5.5 and 9.0. The reason for the confusion with respect to the alkalinity/acidity $CaCO_3$ requirement is the titrant used to determine it. If the titrant is alkaline, the result is reported as acid $CaCO_3$. If acidic titrant is used, the result is alkaline $CaCO_3$. The Georgia DOT specifications referred to above were used as a basis for evaluating prospective granular backfill as GDOT had extensive experience with MSE wall construction and had refined their backfill requirements over time.

Although the granular backfill met specification requirements, the source of the material raised concerns as the pit lies in the Spearfish formation, a red shale noted for the presence of frequent gypsum lenses and notorious for sinkholes. Since the formula for gypsum is $CaSO_4 \cdot 2H_2O$ it was feared that the backfill might contain a significant amount of sulfate which could become soluble and contribute to corrosion under the wrong conditions. Figure 1 below provides an illustration of this potential corrosion issue with a prominent gypsum lens visible in the bottom center.

Figure 1 Excavated granular backfill at station 25+44, WBL. Note large gypsum lens.

The occurrence of such large pieces of gypsum throughout the granular backfill is somewhat ironic considering that the gradation requirements for the backfill called for 100% of the material passing a 6" sieve and no more than 25% retained on a 3" sieve. Acceptance samples tested over the course of construction all had 100% of the material passing the 4" sieve so the presence of oversize material was never indicated during construction.

The extensive longitudinal cracks in the PCC surfacing atop the wall allowed the influx of deicing liquids into the gravel cushion beneath the pavement during the winter after construction. Although these cracks were routed and sealed with silicone joint sealant the following year, the inability of the silicone to completely seal some of the wider cracks made the ongoing infiltration of chlorides extremely likely. This situation was exacerbated by the accumulation of salt/sand on the structure each winter. Corrosion concerns were further prompted as a result of samples of gravel cushion and backfill obtained from coring in the summer of 1987. The samples proved to have relatively high chloride contents and the pH of two samples near the downhill end of the wall was slightly acidic. Unfortunately, these results have been lost in the intervening years but were the basis for a recommendation in 1987 that any future work on the Hot Springs MSE wall incorporate an evaluation of the corrosion situation of the mesh into the plans.

A further cause for concern with respect to corrosion was the fact that the apparent mechanism behind the settlement created a strong likelihood for major voids directly behind panels where the compactive efforts during construction were inadequate. In addition, the contractor failed to adequately coat the connectors with anti-corrosive grease. The potential for deicing chemicals or water to intrude through these voids into the critical attachment zone, coupled with the possibility of water creating a corrosive environment due to the presence of gypsum, made determining the current condition of the mesh imperative.

Plan Provisions for Tie-Back Inspection

The plans for project NH0018(117)39 called for exposing and evaluating the condition of the mesh at 9 sites throughout the wall after concrete pavement slabs were removed. Areas investigated included the west abutment of the Garden Street bridge, directly beneath some of the most severe longitudinal cracking and the lower elevations at the east end of the structure. Excavation and core locations are listed in Table 1 and shown in Figure 2. Because of the unknown character of the mesh condition, the plans called for up to five

Table 1: Excavation and Core Locations

Location (Station)	Side	Reinforcement Length (feet)	Excavation	Core
23+05	Right (EB)	16	X	
23+14	Left (WB)	23	X	
23+82	Right (EB)	16	X	X
23+87	Left (WB)	16	X	X
25+44	Both	14	X	X
25+89	Both	14	X	X
West Abutment	Centerline	23	X	

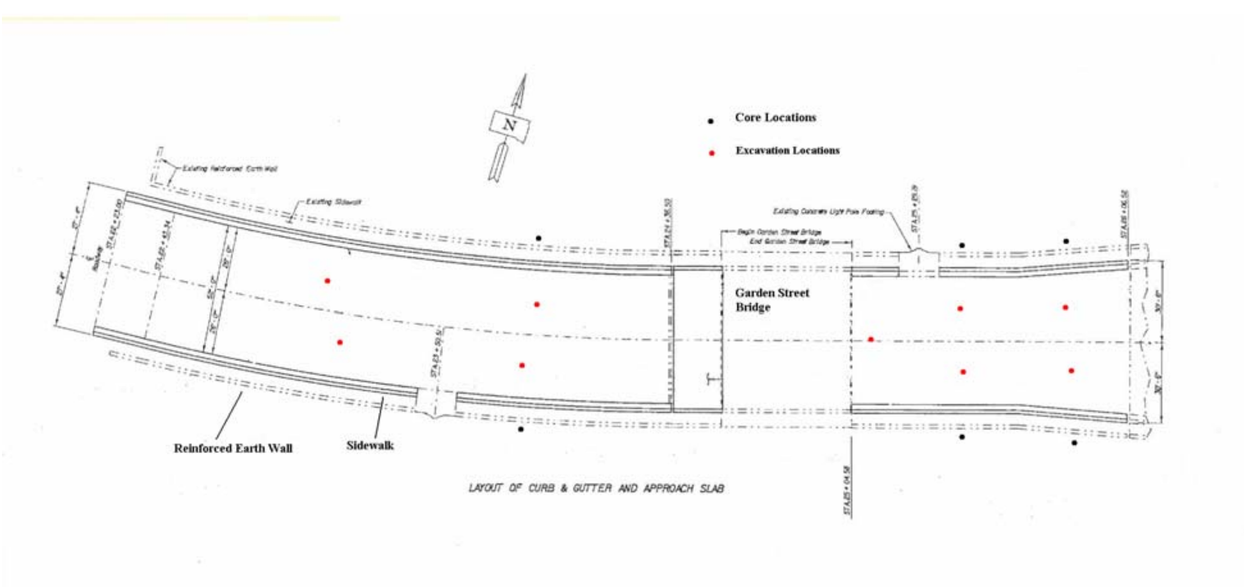

Figure 2 Excavation and core locations

working days after the reinforcement was exposed for corrosion evaluation. The plans also called for the installation of separator fabric on top of the granular backfill the length of the reinforcement to be covered with an aggregate base course to bring the areas up to grade, if necessary. The decision of whether or not to place the separator fabric was to be made after the corrosion evaluation was complete.

Corrosion Monitoring in Cooperation with FHWA

Because of the strong likelihood of corrosion issues within the wall, FHWA was contacted to see if they could provide technical expertise and loan equipment for monitoring corrosion rates within the wall, if necessary. FHWA established Demonstration Project 82 Reinforced Soil Structures MSEW and RSS during the 1990’s to generate design and construction guidelines for mechanically stabilized earth walls. They also produced *Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes* [(2)] as a product of this effort which gives procedures for setting up and methods for interpreting corrosion monitoring on both new and existing structures.

Mr. Barry Berkovitz, FHWA Regional Geotechnical Engineer, was contacted and asked to provide technical assistance in establishing a corrosion monitoring program for the VSL wall at Hot Springs. Unfortunately, Mr. Berkovitz was unable to participate in a site visit for health reasons so Mr. Rich Barrows, another Regional Geotechnical Engineer for FHWA, attended a site inspection/corrosion monitoring meeting in Hot Springs on May 3, 2004. Mr. Barrows discussed the installation procedures and recommended following the guidelines developed in *A Rational Field Process for Corrosion Evaluation of Mechanically Stabilized Earth Walls*[(3)].

Hot Springs Wall Site Visit May 3, 2004

On May 3, 2004 Mr. Barrows and personnel from the SDDOT Office of Research, Geotechnical Section and Custer Area Office conducted a site inspection of the VSL wall at Hot Springs. The goal of the trip was to develop a testing plan for monitoring the reinforcement mesh corrosion using a linear polarization resistance approach. At that time a decision was made to coordinate coring locations for corrosion probe and instrumentation installation with the existing stationing for excavation and evaluation of mesh as per the project plans. An attempt was made to get some idea of the current mesh condition by climbing up on the east and west abutments of the Garden Street bridge to expose reinforcing mesh and determine the current condition. Surprisingly, every sample of mesh exposed on top of the wall at the abutments on both sides of the bridge were in excellent condition with no evident steel section loss though the galvanizing appeared to have interacted with the red shale material in an unusual manner to form a crust on the steel. Figure 3 shows a mesh connector on the east abutment which appears to be in excellent condition.

Mesh steel was also exposed at the south end of the west bridge abutment as this represented an area where deicer runoff was draining into the backfill material at the top of the abutment. Figure 4 illustrates its condition and, again, there was little evidence of corrosion problems beyond the expected loss of galvanizing even though this site represented worst conditions. Based on these tentatively positive results, a decision was made to defer further planning for corrosion monitoring equipment installation until after excavation of the backfill had given a clearer picture of the need for such an extended effort. This dovetailed nicely with the fact that the equipment to conduct the monitoring would not be available for some time.

Deadwood Reinforced Earth Retaining Wall Site Visit

In 1986 a reinforced earth retaining wall was constructed in Deadwood, South Dakota employing epoxy-coated straps for the reinforcement.[(4)] At the time of construction, 36” samples of black steel, galvanized and epoxy-coated straps had been inserted into 6” PVC pipes into a bed of fine granular backfill material held in place by a filter fabric and wooden box. The reason epoxy coating had been specified for the reinforcement was the backfill material used. It consisted of mine tailings from the nearby Homestake Gold Mine containing high levels of sulfide ores. Chemical characterization of the backfill compared to the material used in the Hot Springs wall is shown in Table 2.

Figure 3 Mesh connector at east abutment of bridge

Figure 4 Mesh at southwest corner of west abutment

Figure 5 Severe corrosion of steel reinforcement straps in test location on Deadwood wall

Figure 6 Galvanized reinforcement straps showing zinc reaction product

Figure 7 Epoxy-coated reinforcement straps at Deadwood wall

Table 2: Comparison of Corrosivity of Hot Springs and Deadwood Backfill Materials

Granular Backfill	Hot Springs	Deadwood	Georgia DOT
pH	7.82	6.8	6.0-9.5
Resistivity (ohm-cm)	8333	2,800	>10,000
Chloride	3.6 ppm	n/a	<20
Sulfate	14 ppm	1580	<15

Sulfide ores are decomposed by chemical and biological processes into acidic sulfates which was confirmed by the rapid appearance of a pure gypsum efflorescence buildup along the wall within a few years after construction. Type V cement was specified for the concrete wall panels for the same reason.

As part of the initial effort to set up a corrosion monitoring program on the Hot Springs wall, the same group visited the wall at Deadwood to inspect the remaining test straps and compare their condition with the mesh reinforcement exposed at Hot Springs. The severe corrosion evident in the black steel straps (Figure 5), due to an acidic sulfate environment, confirmed the wisdom of using the epoxy coating. Steel loss was so advanced in these test specimens that the straps could not be pulled out from the wall interior as every attempt resulted in the steel breaking off under the stress. The galvanized steel samples (Figure 6), on the other hand, looked relatively good with no apparent steel section loss. Unfortunately, the galvanizing protection appeared to be mostly exhausted with surface rusting occurring in addition to the formation of zinc reaction products. There is a strong likelihood that these specimens will resemble the black steel samples in a decade or less. The epoxy steel straps were in excellent condition (Figure 7) with no evidence of active steel corrosion occurring. The rust colored material coating the straps in the picture is merely backfill that has undergone extensive degradation.

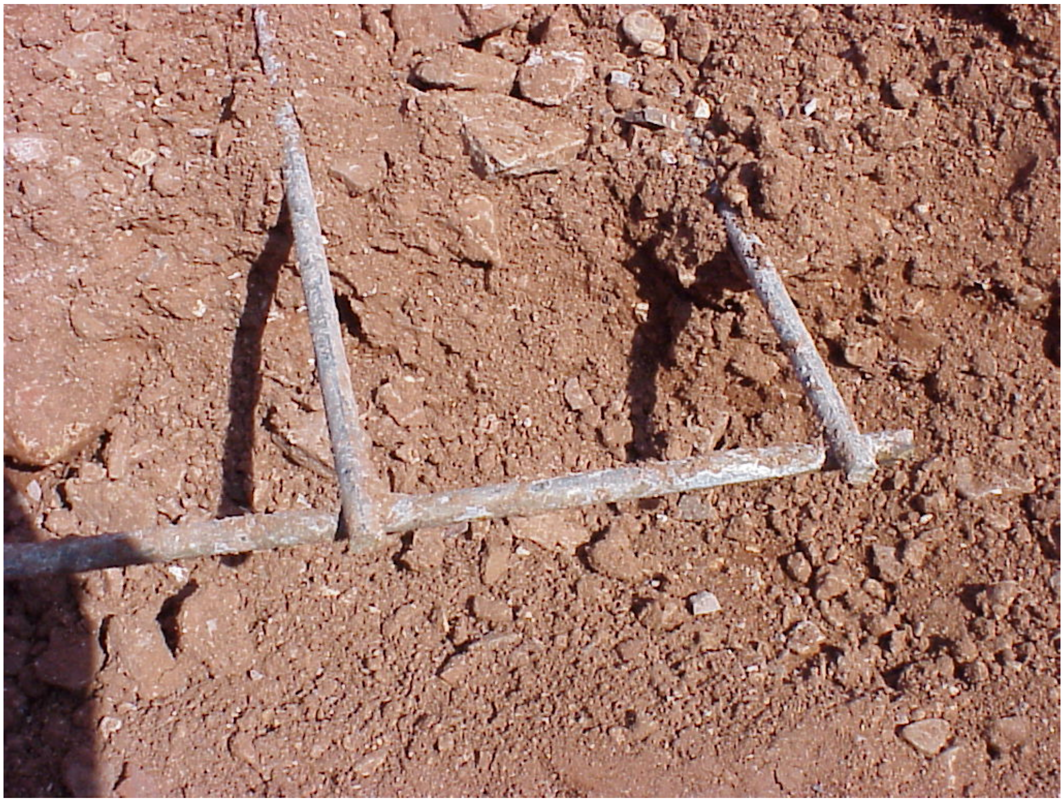

Figure 8 Reinforcement mesh at station 25+89 WB lane

Figure 9 Reinforcement mesh at station 25+44 WB lane

Figure 10 Closeup of reinforcement mesh showing coating of shale/galvanizing

Figure 11 Mesh at station 25+89 EB lane showing severe deformation

Excavations to Top Layer of Mesh Reinforcement

In August of 2004, construction had proceeded to the point where excavation had exposed reinforcing mesh in the westbound lane of the structure. The first mesh examined, at station 25+89, was in excellent condition with no indication of steel loss or rust formation, as revealed in Figure 8. Figure 9 shows the mesh at station 25+44 in the same lane. Virtually every location where mesh was exposed exhibited the same reassuring lack of steel corrosion, including those areas where chloride levels in the gravel cushion above the backfill had been elevated. There was no evidence of any electrochemical activity beyond the strange interaction between the galvanizing and the red shale. Figure 10 is a close-up of the mesh at station 25+44 in the eastbound lane. Note the marked visual difference between this mesh and the galvanized strap in Figure 6 which shows typical galvanic protection with an abundance of white reaction products from the decomposition of the zinc. The only problem encountered in the process of evaluating mesh condition was the evidence of severe deformation of the mesh in several locations as shown in Figure 11. This was undoubtedly due to the settlement of the wall which occurred after the falsework was removed during construction.

As part of the evaluation procedure at each location, the galvanizing thickness was measured using an Elcometer™ and the half-cell potential was determined using a $Cu/CuSO_4$ half-cell. Obtaining stable potential readings at each location was difficult as the backfill material was uniformly dry and digging a hole into the backfill to get to moist material did not work. A gallon of water was used at each site and allowed to sit for at least five minutes before taking potentials. Each value in the table below represents an average of eight or more readings for both galvanizing thickness and potentials. Although many of these readings appear somewhat low, indicating the possibility that the galvanizing may be nearing exhaustion. There was no visual evidence of this being the case and the much more likely probability was that the backfill moisture was insufficient to support normal electrochemical activity.

Table3: Galvanizing Thickness and $Cu/CuSO_4$ Half-Cell Measurements on Excavated Mesh

Location (Station)	Side	Reinforcement Length (feet)	Average Galvanizing Thickness (mils)	Average Half-Cell Potential (mV)
23+05	Right (EB)	16	7.5	-0.946
23+14	Left (WB)	23	8.6	-0.890
23+82	Right (EB)	16	6.9	-0.803
23+87	Left (WB)	16	10.0	-0.922
25+44	Right (EB)	14	5.8	-0.791
25+44	Left (WB)	14	6.5	-0.673
25+89	Right (EB)	14	7.6	-0.697
25+89	Left (WB)	14	6.4	-0.708
West Abutment	Centerline	23	9.7	-0.885
		Overall Average	7.7	-0.813

Coring near Base of Wall

Six inch cores were cut from the wall (Figure 12) at accessible locations corresponding to the areas where mesh had been exposed above. The primary purpose of the cores was to inspect the condition of the mesh in the critical zone where it connected to the concrete panels and to determine how badly the poor consolidation and settlement had affected its integrity. The inspection of the first layer of reinforcement mesh indicated that there was little need to instrument the wall and monitor corrosion. In every case where a core was drilled, the area behind the concrete was generally part of a network of voids with relatively large chunks of granular backfill (many in excess of 6”) providing the framework for the voids. The mesh was also uniformly bent downward, in many cases at angles approaching 45^o. No evidence of any significant corrosion was forthcoming at any of the six core locations as illustrated in Figure 13. The beginning of a typical bend in the mesh just off the wall face is shown in Figure 14.

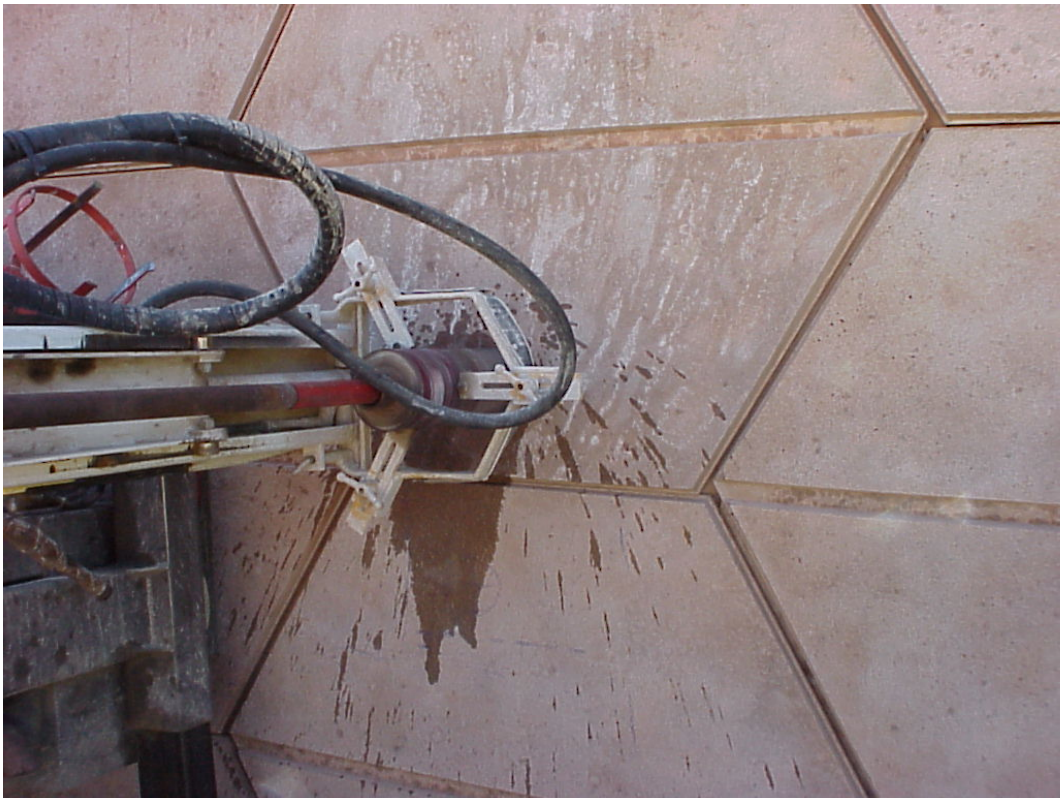

Figure 12 Coring operation at wall base

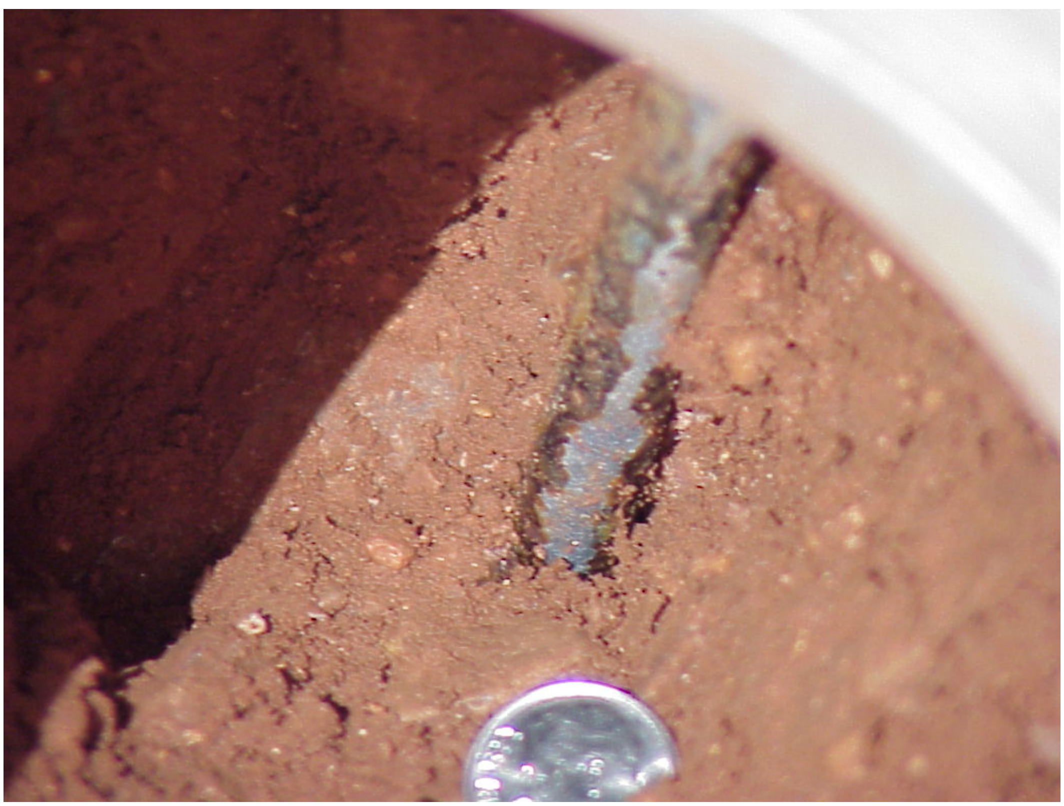

Figure 13 Mesh reinforcement in core hole at station 23+87 left

Figure 14 Beginning of bend in mesh at interior wall face station 23+87 left

Figure 15 Panel connector near base of wall station 25+89 right

Another concern was the condition of the connectors between the concrete panels and the mesh. The position of the core holes was adjusted to cut through the panels either just above or just below the nominal position of a connector at each panel drilled with the terrain determining which approach was most feasible. After coring, every effort was made to excavate sufficient backfill material to expose and photograph the connectors if lighting permitted. Mirrors were used to illuminate the hole interiors with sunlight and an additional mirror used where possible. Every connector was found to be in pristine condition which was not a real surprise considering there was insufficient electrolyte (soil) adjacent to them to support corrosion. Retention of water in the same area was problematical considering the backfill along the wall face was obviously free draining. Fortunately, the point where the steel had yielded and bent when the wall settled was in the same environment so that stress corrosion at these bends is not an issue. A typical connector inside the lower wall is shown in Figure 15.

Discussion

The assessment of the VSL mechanically stabilized earth wall at Hot Springs as to potential problems with future reliable performance proved relatively straightforward and uncomplicated. The considerations which had prompted the need for the evaluation at the time of the replacement of the badly cracked PCC pavement were all well founded and the subsequent results could have had a much more dismal outcome. The issue with deicing chemicals penetrating into the granular backfill and compromising the mesh integrity is reinforced by the severe corrosion of steel railing elements along the sidewalks where several rail base plates were so badly corroded they had to be replaced. Tie bars in the pavement also exhibited significant corrosion. If these chloride salts had been able to permeate into the backfill there is little doubt that corrosion could have compromised a significant portion of the top mat of reinforcement. Additionally, the backfill material used had a considerable amount of gypsum as one of the aggregate minerals and, had this sulfate ever solubilized, mesh corrosion throughout the wall section would have been accelerated tremendously.

Figure 16 Steel railing post showing severe corrosion around base

One of the major factors behind the surprising lack of corrosion occurring within the wall is illustrated in Figures 3 and 4 above where water was used to clean off the mesh prior to examination. The water tended to pool on the surface of the backfill forming a standing puddle that disappeared very slowly. In every case where excavation into the backfill exposed mesh, the backfill material was bone dry and water was needed to get a stable reading with a half-cell. The average half-cell readings in Table 2 indicate, on the basis of the expected potentials, that either the galvanizing is mostly gone or the lack of moisture is affecting the potential values. The thickness readings do not support the galvanizing approaching exhaustion which leads to the undeniable conclusion that the use of a totally inappropriate backfill material has resulted in a considerable extension of potential wall service life. The unanticipated low permeability of the backfill also minimized the penetration of chlorides from the surface.

The fact that the Hot Springs VSL wall suffered an alarming settlement before construction was completed without its structural integrity being compromised argues strongly for the resilient nature of MSE walls in general. The almost total lack of consolidation along the wall faces with the extended network of voids apparent from coring played a part in the settlement and damage to the pavement above but also provide a further explanation for the condition of the mesh. The western side of the wall rises at an 8% grade just off the Garden Street Bridge which means runoff is rapid. This is the area where most of the pavement cracking occurred immediately after settlement. The evidence of high levels of chlorides in the gravel cushion just below the pavement from 1987 indicates that the deicing salts penetrated below the pavement but did not travel any significant distance into the backfill. Any water moving through the gravel cushion would hit the almost impermeable barrier of the backfill interface and continue downgrade until reaching the area of the west bridge abutment. Any water approaching the concrete face panels would probable have rapidly drained to the base of the wall. The one location where moisture was evident in the backfill was in fact along this edge but the mesh examined at this location was in excellent condition as well. A careful examination of concrete face panel joints throughout the wall structure did not reveal any evidence of efflorescence or water leakage or transport of materials. This can be contrasted with the obvious efflorescence accumulating on the wall face in the Deadwood structure as shown in Figures 5-7.

Conclusions

1) The VSL mechanically stabilized earth wall on US 18 in Hot Springs, SD is in unexpectedly good condition despite lack of compliance with specifications and the resulting settlement, pavement cracking and influx of deicing chemicals.
2) There is no apparent corrosion to the mesh reinforcement examined in the Hot Springs wall. The locations selected for excavation or coring give a good cross section of the various corrosion environments expected and the results represent an optimal situation.
3) The interaction of the red shale backfill and the zinc galvanizing on the mesh is atypical and appears to form a protective coating of shale combined with zinc reaction products.
4) The VSL wall should achieve its design life of at least 75 years based on the fact that no steel corrosion is apparent as of yet and the rate of the corrosion, once initiated will be substantially lower than expected.
5) The use of epoxy-coated reinforcing straps in the Deadwood Reinforced Earth™ retaining wall proved to be a wise decision based on the performance of the black steel and galvanized test straps embedded in the wall.

Recommendations

1) The results of this research indicate that the corrosion concerns, although justified, have no basis in fact and that future monitoring of wall performance at this installation can be done on a routine basis.
2) The use of epoxy-coated reinforcement straps at Deadwood proved to be a wise decision and should be considered as an alternative for galvanizing on installations where unavoidable corrosivity issues warrant its use.

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