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MITIGATION OF ROADWAY SETTLEMENT ABOVE BURIED CULVERTS AND PIPES

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Abstract

This report identifies the likely causes of settlement and roadway damage at culverts sites on Wyoming highways. These causes are:

- Poor material used as backfill around and above culverts
- Inadequate compaction
- Low soil cover, as defined as 5m or less

Based on the results of field and laboratory investigations certain design and construction measures show the greatest potential for reducing the above causes. These are:

- The use of high quality granular materials compacted to a high density for backfilling culverts
- Avoiding the use of highly plastic, compressible fine-grained soils
- The use of controlled low strength materials (CLSM), commonly referred to as flowable fill, for backfilling culverts

Of the options tested, the use of a well compacted granular backfill or CLSM backfill showed the greatest potential for minimizing roadway settlement. However, CLSM consisting of sand, fly ash, and water appears to have numerous advantages from a construction viewpoint. Mix design should be established on the basis of ultimate strength and ease of excavation.

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SI* (Modern Metric) Conversion Factors

Approximate Conversions from SI Units

Symbol	When You Know	Multiply By	To Find	Symbol
Length				
mm	Millimeters	0.039	inches	in
m	Meters	3.28	feet	ft
m	Meters	1.09	yards	yd
km	Kilometers	0.621	miles	mi
Area				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	Hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
Volume				
ml	Milliliters	0.034	fluid ounces	fl oz
l	Liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
Mass				
g	Grams	0.035	ounces	oz
kg	Kilograms	2.202	pounds	lb
Mg	Megagrams	1.103	short tons (2000 lbs)	T
Temperature (exact)				
°C	Centigrade Temperature	1.8 C + 32	Fahrenheit Temperature	°F
Illumination				
lx	Lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
Force and Pressure or Stress				
N	Newtons	0.225	Poundforce	lbf
kPa	Kilopascals	0.145	pound-force per square inch	psi

Approximate Conversions to SI Units

Symbol	When You Know	Multiply By	To Find	Symbol
Length				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
Area				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
Volume				
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	l
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
Mass				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lbs)	0.907	megagrams	Mg
Temperature (exact)				
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celsius temperature	°C
Illumination				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
Force and Pressure or Stress				
lbf	pound-force	4.45	newtons	N
psi	pound-force per square inch	6.89	kilopascals	kPa

CONTENTS

FIGURES

TABLES

EXECUTIVE SUMMARY

Settlement problems over culvert installations on Wyoming roadways are examined and potential corrective measures are discussed. Over 160 culvert sites in Wyoming were inspected as part of a field investigation. Of these sites, fifteen locations were drilled and sampled. Based on field investigations and information obtained from WYDOT personnel, three probable causes of settlement are identified: 1) inadequate compaction, 2) shallow cover of fill above culverts, and 3) the use of plastic, compressible soils derived from bentonitic Cretaceous shales as fill material. Design and construction measures for minimizing roadway settlement over culverts are reviewed. The two primary alternatives are reinforced backfill and the use of controlled low-strength material (CLSM) for backfill. Of the options tested, the use of a well compacted granular backfill or CLSM backfill showed the greatest potential for minimizing roadway settlement.

1: INTRODUCTION

Installation of culverts, water lines, stock passes, and other similar structures across roadways typically involves excavation of existing ground followed by placement of the structure, backfilling of the excavation with compacted soil, and construction of a pavement over the backfill. If vertical deformation occurs in the soil or rock beneath the structure, in the backfill material, or in the structure itself, it is likely to be manifested as vertical deformation, or settlement, at the surface of the roadway. The result of roadway settlement may be relatively minor and practically unnoticeable by motorists, or it may cause a significant "dip" in the roadway, posing a safety hazard and possibly vehicular damage. In extreme cases, settlement may result in cracking and premature deterioration of the pavement, requiring an overlay or pavement replacement. Figure 1 depicts the general nature of the problem. Delta (Δ) as seen in Figure 1 is the maximum deflection due to settlement.

Settlement of highway pavements overlying buried culverts has been identified as a problem by the Wyoming Department of Transportation (WYDOT). In some locations, settlement is severe enough to pose a hazard to driver safety and has created the potential for vehicular damage. Even moderate settlement decreases highway quality and contributes to pavement damage that requires costly maintenance and possibly early replacement of some pavements before the design life has been achieved.

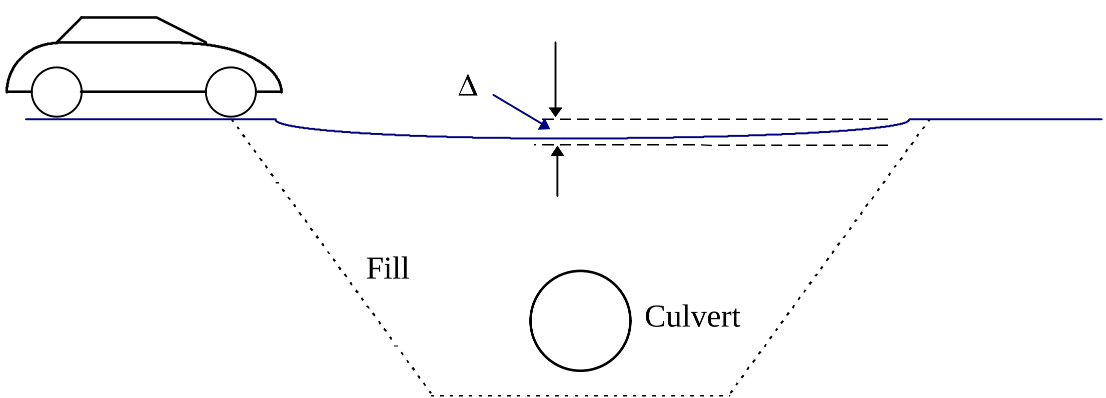

Figure 1. Roadway Settlement Over Culvert

The goal of this report is to develop recommendations for design and construction procedures to eliminate or minimize damaging roadway settlement above culvert crossings. Preliminary investigations revealed the following WYDOT perceptions: (1) the number of sites in Wyoming at which excessive settlement has occurred is not known, however the general perception of maintenance personnel is that the problem is widespread and not limited to any particular region of the state; (2) the mechanism(s) by which excess settlements occur are not known; and (3) existing design and construction procedures are not resulting in satisfactory performance. The research described in this report is aimed at further defining the scope of the problem, identifying the controlling mechanisms, developing methods to prevent or minimize future settlement, and presenting the findings in the form of recommendations for design and construction of backfilled culverts and pipes for implementation by WYDOT personnel.

OBJECTIVES

The research objectives are to:

1. Determine if roadway settlement appears to be related to culvert or pipe characteristics, geological environment, construction practices, or other factors.
2. Establish the mechanism(s) of settlement, for example, compression settlement of fill around and above culverts and pipes under traffic loads, settlement of soil beneath culverts and pipes, deformation of the culverts and pipes, or some combination of causes.
3. Evaluate design and/or construction procedures to prevent or minimize roadway settlement.

METHODS

To achieve the stated objectives, research was carried out in two phases. The first phase consisted of field and laboratory investigations to establish the most likely cause(s) of settlement. The second phase consisted of experimental evaluation of design and construction procedures to eliminate or minimize roadway settlement for future culvert installations. Based on the results of the experimental study, recommendations are made that will provide procedures to minimize roadway settlement.

SCOPE OF STUDY

A review of the literature for establishing possible mechanisms causing roadway settlement and possible methods of minimizing or reducing settlement is provided in Chapter 2. Chapter 3 describes a field investigation that was performed to characterize the problem and to identify mechanisms of settlement. Chapter 4 describes the laboratory testing performed to classify soils, evaluate settlement, and measures to reduce settlement. Laboratory load tests were conducted on small-scale models of culverts buried in a soil backfill. These load tests were used to evaluate the effects of backfill properties and soil reinforcement on surface deflections. Results obtained from the laboratory and experimental testing are discussed in Chapter 5. Chapter 6 presents the final conclusions, recommendations, and summary of the research.

2: LITERATURE REVIEW

This chapter presents a review of the literature on soil compaction and other methods to control roadway settlement.

METHODS TO CONTROL ROADWAY SETTLEMENT

Measures that can be taken to reduce or eliminate settlement of roadways above culverts and pipes can be placed into two categories: (1) improvements in compaction specifications and procedures and (2) other ground modification techniques in addition to compaction. Improvements in compaction specifications and procedures involves more detailed screening of soils to evaluate their suitability as backfill, specifications tailored for specific soil types and compaction equipment, and improved quality assurance and control during field compaction. Ground modification techniques include soil reinforcing and the use of controlled low strength materials.

Compaction Specifications and Procedures

The backfill of underground structures, pipes, and culverts must be performed correctly and according to specification requirements (Waidelich 1990). Improper placement of backfill material or poor compaction can result in undesirable settlements and subsidence of the pavement (Waidelich 1990) and possibly failure of the buried structure (Ariema and Butler 1990).
A possible, and likely, source of roadway settlement in backfilled areas is compression of the backfill soil. The Wyoming Department of Transportation Standard Specifications for Road and Bridge Construction (1996), subsection 206.037, requires the following:

> “a. ***Backfill Material***. Backfill material placed under, adjacent to, and over pipe conduit, precast and cast-in-place structures, shall be a fine, compactible, excavated soil or granular fill material. Backfill material shall not contain frozen lumps, chunks or highly plastic clay, or stones which would damage the structure.
>
> b. ***Placing and Compacting Backfill***. Backfill will be placed in loose layers approximately 200 mm thick, and each layer compacted to the density specified for the type of structure. Backfilling and compacting shall be done without damage directly on or against the structure.”

The specification also states:

> “Backfill material for structures, drainage conduits, pipelines and all other facilities, shall be compacted to a density of not less than 95 percent of maximum density at optimum moisture content (standard proctor), in accordance with subsection 203.035.”

Materials used as backfill could show considerable variability in terms of geotechnical properties, in particular with regard to compressibility. For example, a "fine, compactible, excavated soil" could contain a significant percentage of fines (silt and clay size particles), and may exhibit a high degree of compressibility even when compacted to 95 percent of maximum dry density. A "granular fill material" would be expected to exhibit lower compressibility, especially under static loads such as overburden stress. Granular soils may also exhibit settlement under repeated loading caused by traffic and could be prone to collapse settlement under certain conditions. Table 1 summarizes the range of properties that might be expected for different compacted soils. Note that compressibility generally is greater for cohesive (fine-grained) soils than for cohesionless (coarse-grained) soils.

There is a general perception within the engineering community that densification of soils induced by compaction precludes the possibility of further densification and resulting settlement. However, as pointed out by Noorany and Stanley (1994), Lawton et al. (1992), and others, compaction of soil according to a set of standard specifications often does not insure satisfactory performance. Noorany and Stanley (1994) identified several factors that have led to damaging settlement of compacted fills in California. The two most important factors were:

1. difficulty in making proper fill observations and testing and
2. post-construction wetting, leading to collapse settlement.

Noorany (1990a, 1990b) reports that several sites studied in California exhibited "ample evidence of inadequate compaction, thick lifts, an abundance of nested oversize cobbles, highly expansive soils and not enough nonexpansive select material, an inadequate number of field density tests, mismatch and other fill testing problems."

Settlement of compacted fills caused by wetting is another problem that has gained recognition in recent years. Numerous studies have demonstrated that compressibility of compacted cohesive soils depends not only on soil density but also on initial moisture content (Barden et al. 1973, Booth 1975). Hausmann (1990) summarizes the general findings as follows:

> "A soil compacted dry of optimum shows less settlement than a soil compacted wet of optimum, if it is left unsaturated. If soils are saturated after being subjected to a significant compressive load, additional settlement may occur. This "collapse" settlement is most significant for soils compacted dry of optimum water content, where maximum dry density was achieved during compaction."

Table 1. Typical Properties of Compacted Soils (Design Manual 7.2 U.S. Navy, 1982)

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Moisture Percent	Typical Value of Compression: At 1.4 tsf (20 psi) Percent of Original Height	Typical Value of Compression: At 3.6 tsf (50 psi) Percent of Original Height	Typical Strength Characteristics: Cohesion (as compacted) psf	Cohesion (staurated) psf	∅ (Effective Stress Envelope Degrees)	Tan ∅	Typical Coefficient of Permeability ft/min.	Range of CBR Values	Range of Subgrade Modulus k lbs/cu in.
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5 x 10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt	120 - 135	12 - 8	0.5	1.1	----	----	>34	>0.67	>10^{-6}	20 - 60	100 - 300
GC	Clayey gravels, poorly graded gravel-sand-clay	115 - 130	14 - 9	0.7	1.6	----	----	>31	>0.60	>10^{-7}	20 - 40	100 - 300
SW	Well graded clean sands, gravely sands	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	>10^{-3}	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	>10^{-3}	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	5 x >10^{-5}	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	2 x 10^{-6}	5 - 20	100 - 300
SC	Clayey sands, poorly graded sand-clay-mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	5 x 10^{-7}	5 - 20	100 - 300
ML	Inorganic Silts and clayey silts.	95 - 120	24 -12	0.9	1.7	1400	190	32	0.62	5 x 10^{-7}	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay.	100 - 120	22 -12	1.0	2.2	1350	460	32	0.62	>10^{-5}	----	
CL	Inorganic clays of low to medium plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	5 x 10^{-7}	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	----	----	----	----	----	----	----	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	5 x >10^{-7}	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	>10^{-7}	15 or less	50 - 150
DH	Organic clays and silty clays	65 - 100	45 - 21	----	----	----	----	----	----	----	5 or less	25 - 100

Notes:
1. All properties are for condition of “Standard Proctor” maximum density, except values of k and CBR which are for “modified Proctor” maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR Data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value show.
4. (----) indicates insufficient data available for an estimate.

Lawton et al. (1989) also adds:

> "The drier the soil at compaction, the greater both the maximum collapse or swelling at a particular vertical overburden stress and the critical overburden stress. Compacting at water contents near or wet of the line of optimums reduces collapse as well as swelling. Wetting induced collapse is reduced because the wetter, more compressible as-compacted soil has already experienced a substantial amount of load-induced compression before being permitted to take up additional water."

Considering cohesionless soils, submergence of partially saturated fine uniform sands, loess, and soils containing soluble salts has been observed to cause sudden settlement, or collapse. The cause of submergence may be a rising water table, surface infiltration, or moisture buildup beneath covered areas (such as pavements). The addition of water under a constant pressure leads to a sudden decrease in void ratio and settlement. The mechanism of collapse is believed to be a loss of apparent cohesion, which may be provided by capillary tension or the cementing action of salts (Lawton et al. 1992). Many natural soil deposits in Wyoming contain soluble salts, especially gypsum, and there are numerous wind-deposited silt and sand deposits that may form collapsible soils. It is noted that most documented cases of collapse settlement have involved deep-seated collapses, under overburden stresses somewhat higher than would exist in most backfills for culverts and pipes buried to depths of 6 m or less. Lawton et al. (1992) states that under the proper conditions, most compacted soils will densify when given free access to water. Lawton et al. (1992) also adds that the most critical conditions are the degree of saturation, dry density of the soil, and the total overburden pressure. Soils most prone to collapse settlement are those compacted dry of optimum to relatively low dry unit weights, under high overburden stresses (> 96 kN/m^2).

In addition to the two factor cited above, other reported sources of settlement in compacted fills include repeated loading (Shackel 1976), shrinkage of expansive cohesive soils (Ladner and Hamory 1974), and frost susceptibility of silty soils (Hausmann 1990).

Reinforced Backfill

Besides compaction, which would be classified as mechanical ground modification, other ground modification techniques may be required to minimize settlement. One technique that appears to have a high potential for addressing this problem is soil reinforcing. In recent years, great strides have been made in the development of low-cost, durable materials for soil reinforcing, and the applications have become widespread. The purpose of reinforcing a soil mass is to improve its strength and bearing capacity and reduce settlement and lateral deformation. This is accomplished by placing inclusions in the soil which are able to resist tensile stresses and which interact with soil through friction and/or adhesion. One type of reinforcing which may be applicable to backfilled culvert and pipe installations is geogrids. Geogrids are open-meshed polymeric sheets, relatively flexible and lightweight. They are easily handled in the field and connections of adjacent sheets can be made with interweaving cords. Geogrids are available in various grades of strength and stiffness and are fabricated by stretching of prepunched sheets of high-density polyethylene (HDPE) or polypropylene. Figure 2 shows a geogrid placed on a compacted soil lift.

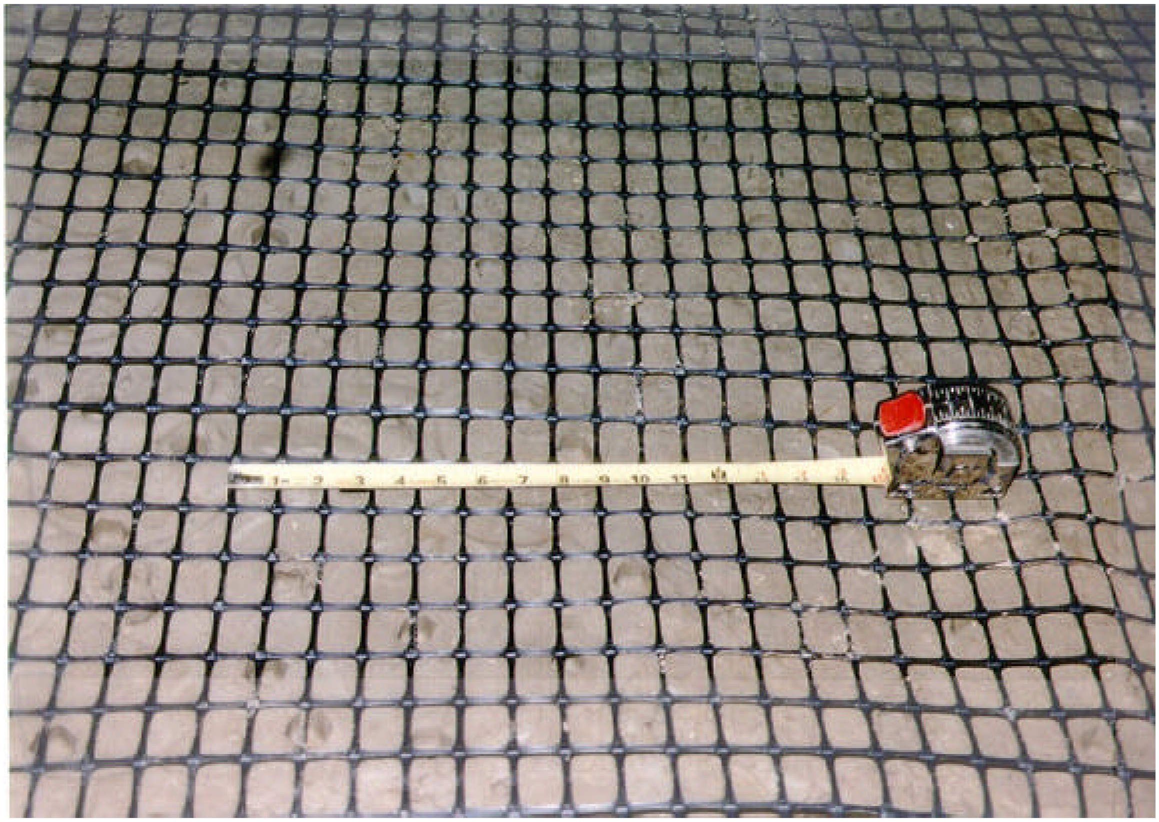

Figure 2. Geogrid Reinforcing On Compacted Soil Lift.

A recent study conducted by Bauer (1994) found that "geogrids used as overlays for flexible conduits at shallow depth were also found effective in reducing surface settlements and deformations of the model conduits." The study involved the test setup shown in Figure 3, in which the backfill above the buried conduit was reinforced with one or more layers of geogrid. Figure 3 also shows load-settlement responses for cover soils reinforced with two layers of geogrid, compared to cover soil with no reinforcing. These results suggest the potential for geogrid reinforcing to control surface settlements above buried conduits.

Additional testing has been conducted using geotextiles and geogrids to reinforce soil for bearing capacity of rectangular and spread footings. Yetimoglu et al. (1994) and Adams and Collins (1997) both reported that bearing capacities were increased using geosynthetic reinforcing. Adams and Collins (1997) indicate that the use of geosynthetics can increase the ultimate bearing capacity of shallow spread footings by a factor of 2.5. The geogrid improved the bearing capacity of the foundation system by a factor of 2.3. The improvement in ultimate bearing capacity occurred at settlements between 10 and 20 mm. Figure 4 illustrates typical results obtained by Adams and Collins. Note that for any given bearing stress, the settlement of the reinforced soils is significantly less than that of the unreinforced (control) soil.

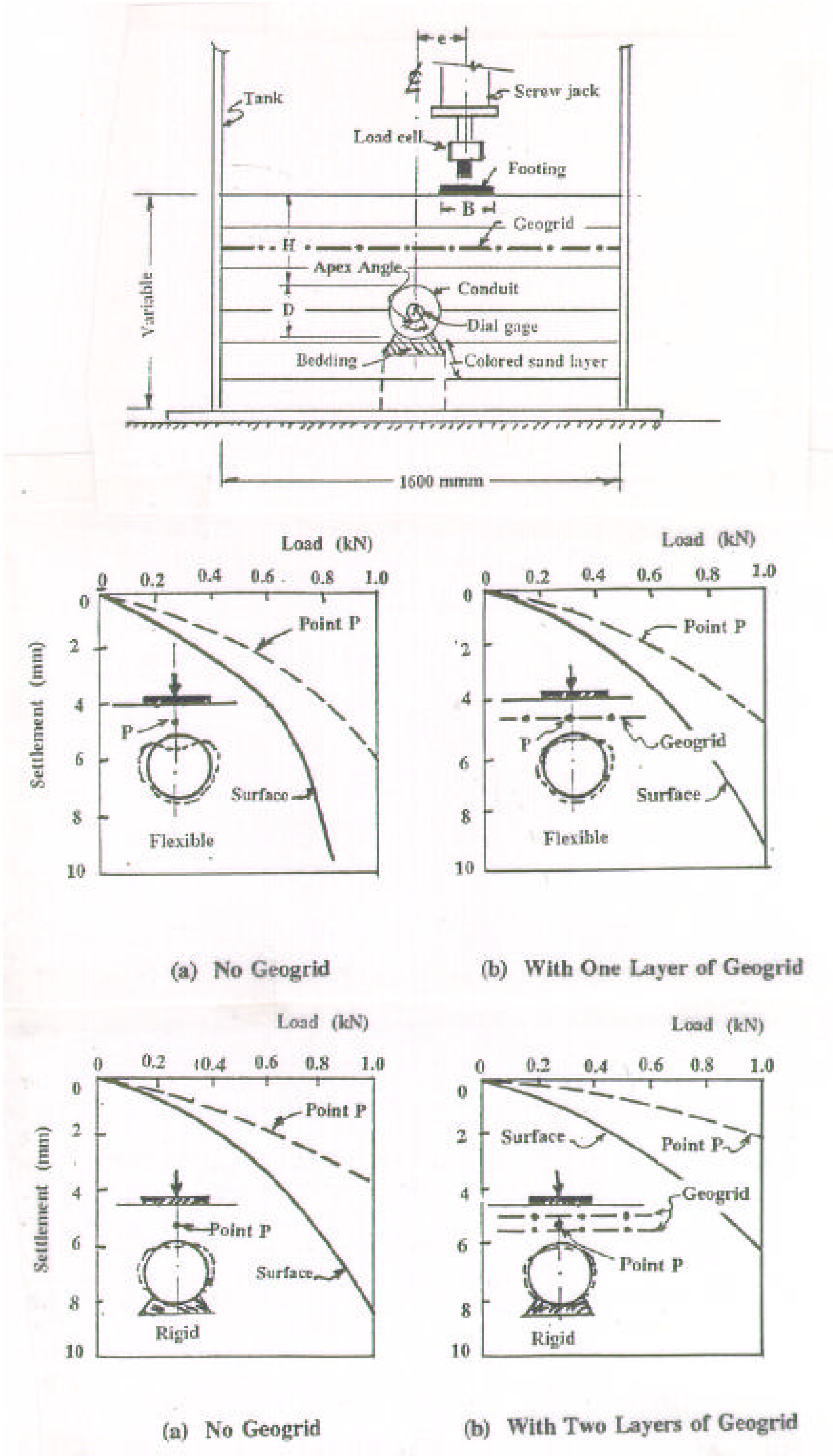

Figure 3. Test Setup and Load-Settlement Responses (Bauer 1994)

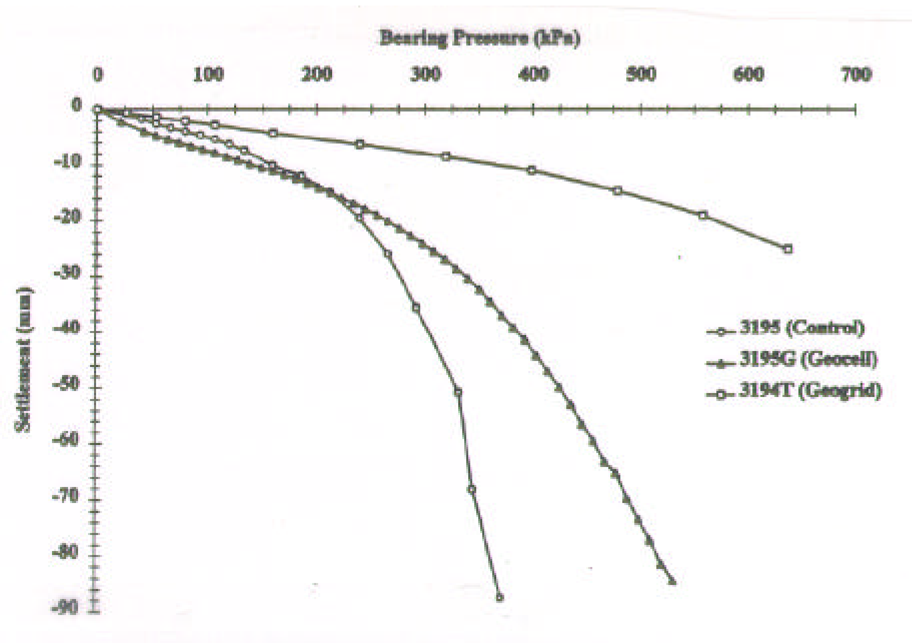

Figure 4. Typical Bearing Pressure vs. Settlement Curve (Adams and Collins 1997)

Another study by Fannin and Sigurdsson (1996) reported that the use of geotextiles and geogrids between the base course layer and subgrade soil led to a significant improvement in trafficability (quality of the roadway for passage) of an unpaved road. A geogrid tested in a 0.5 m thick base decreased the rut depth incurred by 2 cm, from 11 cm without reinforcing to 9 cm with the geogrid reinforcing at 250 load repetitions. Base or subgrade reinforcing is subject to different loading conditions than backfill reinforcing but the observed decreases in settlement suggest potential improvements in fill performance.

Controlled Low-Strength Material(CLSM)

As defined by ACI Committee 229 (1994), controlled low-strength material (CLSM) is a self-compacted, cementitious material used primarily as a backfill in lieu of compacted fill. Controlled low-strength materials are defined by "Cement and Concrete Terminology (ACI 116R)" as materials that result in a compressive strength of 8,300 kN/m^2 (1,200 psi) or less. Most current CLSM applications require unconfined compressive strengths of 2,000 kN/m^2 (300 psi) or less. The lower strength requirement is necessary to allow for future excavation of CLSM.

Controlled low-strength material should not be considered as a type of low strength concrete, but rather a self-compacted backfill material that is used in place of compacted fill. Additionally, CLSM should not be confused with compacted soil-cement. Soil cement, as defined by ACI Committee 230 on Soil-Cement, requires compaction and curing. CLSM typically requires no compaction or curing to achieve the desired strength. Although CLSM generally costs more per unit volume than most soil or granular backfill materials, its many advantages often result in lower in-place costs.

Mix Design.

A typical CLSM mix contains cement, water, fly ash, and fine aggregate. Ready mix producers can combine these components in varying proportions to meet specific performance requirements and to take advantage of locally available materials. Ranges of quantities used for different mixes are given in Table 2. As cited by Smith (1991), various organizations have recommended mix proportions including: Ohio Ready Mixed Concrete Association, Michigan Ready Mixed Concrete Association, National Ready Mixed Concrete Association, Iowa Department of Transportation, and the South Carolina Department of Highways and Public Transportation.

Table 2. Ranges for CLSM Mix Designs

Material	Quantities kg/cubic meter of fill (lb/cubic yard of fill)
Cement	30 - 166 (50 - 280)
Water	223 - 445 (375 - 750)
Fly Ash	0 - 1,187 (0 - 2,000)
Fine Aggregate	1,246 - 1,875 (2,100 - 3,160)

The principal components of a CLSM mix are cement, water, fly ash, and fine aggregates. Alternatively, chemical admixtures and non-standard materials can also be added. Chemical admixtures include water reducers, superplasticizers, and accelerators. Air-entraining agents improve flowability, reduce the density of the mix, reduce effects of freeze/thaw, and enhance the economy of the mix. Overall, however, chemical admixtures may not be cost effective unless needed for a specific requirement. Non-standard materials, if available, may be more economical depending on the project requirements. Examples of materials that may be used as aggregates include bottom ash produced in the coal combustion process, discarded foundry sand, and reclaimed crushed concrete (ACI 229 1994). As mentioned previously, aggregates that may swell due to expansive reactions or other mechanisms should be avoided. Some non-standard materials that may not be suitable for CLSM mixtures include wood chips, wood ash, or other organic materials.

According to ACI Committee 229 (1994), there are fifteen advantages to using CLSM as backfill. These include: readily available using local materials, ease of deliverance, ease of placement, versatility, strength and durability, can be excavated, less inspection, faster return to traffic, no settlement, reduction in excavation cost, increase in worker safety, all-weather construction, reduction in equipment needs, eliminates need for storage, and the use of a by-product (ACI 229 1994). Some of the potential advantages relative to this study include: versatility, strength and durability, less inspection, control of settlement, and all-weather construction.

CLSM offers the potential for eliminating damaging settlement of the roadway. If a soil backfill is not compacted properly when placed, settlement may occur after being paved. Cracks and dips may form in the roadway due to this settlement. The CLSM, on the other hand, does not form voids during construction and reportedly does not settle or rut under loading.

Versatility is important in Wyoming, as not all sites have the same requirements. The CLSM mix can be adjusted to meet the requirements at each site as needed. For instance, strength can be varied by adjusting the amount of cement or fly ash used in the mix. Other properties of the mix can also be adjusted such as flowability and rate of setting. CLSM mixes often have load-carrying capacities greater than a compacted backfill of soil or granular fill. Additionally, CLSM is less permeable and more resistant to erosion. As stated previously, CLSM can reach compressive strengths of up to 8,300 kN/m^2 which would work well as a more permanent structural fill.

Less inspection is a very desirable component of CLSM. Some of the problems with roadway settlement pointed to the possible lack of enough inspectors. CLSM is self-compacting, thereby reducing extensive field tests that must be done on soil backfill. A final advantage that seems important within the application of this study is all-weather construction. With varying conditions in Wyoming, construction schedules are always interrupted causing delays and additional costs. With the possibility of rain or snow at almost any time, trenches can fill up with water making pumping necessary. CLSM displaces standing water in trenches. In addition, materials used for CLSM can be heated, as for ready-mixed concrete, to be used in cold weather.

As backfill, CLSM can be readily placed into a trench, hole, or other cavity. As noted previously, no compaction is needed. Therefore, the trench width or size of excavation may be reduced. Granular or site excavated backfill, even if compacted properly in the required layer thickness, may not achieve the uniformity of CLSM.

A case study from Peoria, Illinois (Smith, 1991) illustrates the use of CLSM to mitigate severe settlement problems of soil backfill in utility trenches. In 1988, CLSM was used as an alternate backfill material. The CLSM was placed in trenches up to 2.7 m deep. Although fluid at the time of placement, the CLSM “hardened” to the extent that a person’s weight could be supported within 2 to 3 hours. Shrinkage cracks were very minimal. Additional tests were conducted placing a pavement patch within 3 to 4 hours. A pavement patch was successfully placed over a sewer trench immediately after the trench was backfilled. As a result of the success of the CLSM, the city of Peoria has changed its backfilling procedure to require the use of CLSM on all street openings.

CLSM has not been used widely on WYDOT projects. However, on two recent projects, the contractor requested and was given permission to use a “flowable fill” consisting of cement, sand, and water at no additional cost (Flom, personal communication). Eleven culverts were backfilled with this mix on US 191 in 1995, with fill heights ranging from 0.6 m to 4.6 m. It is highly recommended that these sites be monitored to assess the performance of CLSM.

SUMMARY

In summary, it appears roadway settlement can be caused by a number of factors. Two that were noted from the literature review include compressibility of fine compactible soils and collapse settlement due to post-construction wetting. Both of these factors could be potential causes of roadway settlement on Wyoming roads, especially the use of fine compactible soils as backfill material.

To minimize or eliminate roadway settlement, several options were found in the literature review. Options other than improved compaction include reinforced backfill and CLSM (Controlled Low-Strength Material) or flowable fill. In the literature reviewed and presented, both alternatives have been proven to reduce settlement in various test applications.

3: FIELD INVESTIGATION OF ROADWAY SETTLEMENT

This chapter describes a field investigation that was conducted to further characterize the problem of roadway settlement and damage on Wyoming highways. Mechanisms of roadway settlement above culverts were also identified.

SUMMARY OF SITE INVESTIGATIONS

The field investigation involved collecting and analyzing information on existing culvert sites in Wyoming. The information was obtained from three sources: (1) WYDOT maintenance personnel, (2) WYDOT project records, and (3) site visits and inspection. Information on culvert sites at which excess settlement has occurred was obtained initially by conducting a telephone survey of district maintenance personnel. Based on these discussions, four areas of the state involving twelve road construction projects were selected for more detailed investigation. The four areas are: (1) WYO 487 between Medicine Bow and Casper, (2) WYO 70 east of Baggs, (3) WYO 113, Pine Haven Road, and (4) various locations in District 5 near Lander, Thermopolis, Meeteetse, Cody, and Worland. All available records for each project were reviewed, including information pertaining to the soils and any available records from construction, such as field inspection reports or field test results. Figure 5 is a map showing the locations of the investigated sites.

The project team visited each of the twelve selected sites to make detailed inspections. Items which were inspected and documented included culvert type, size, and location, evidence of ground movement, damage to culverts and pipes such as buckling and excessive deflections, field identification of subgrade and backfill soils, description of drainage facilities, possible sources of water to the subgrade and backfill soils, approximate magnitude of roadway settlement, and the extent of pavement damage, if any. A typical culvert site showing roadway damage is shown in Figure 6.

For each culvert location, a qualitative damage assessment was made. Damage is described by three categories: minor, moderate, or severe. Minor damage is characterized by zero to slight settlement (0-8 cm). Moderate damage consists of settlement in the range of 8-15 cm or that required patching. A designation of severe damage indicates settlement of over 15 cm, collapsing pipes, and/or heavy maintenance requirements since the completion of construction. This classification is somewhat arbitrary and is based on crude estimates of actual settlement, but is deemed adequate for this investigation.

Following the initial site inspections, fifteen culvert crossings were chosen for subsurface drilling and testing. All of the sites chosen for drilling exhibited moderate or severe roadway damage. The WYDOT Geology Program provided a drilling rig and crew. The author and Mark Stewart, a University of Wyoming civil engineering graduate student, were present to observe drilling operations, record blow counts, and log soil samples.

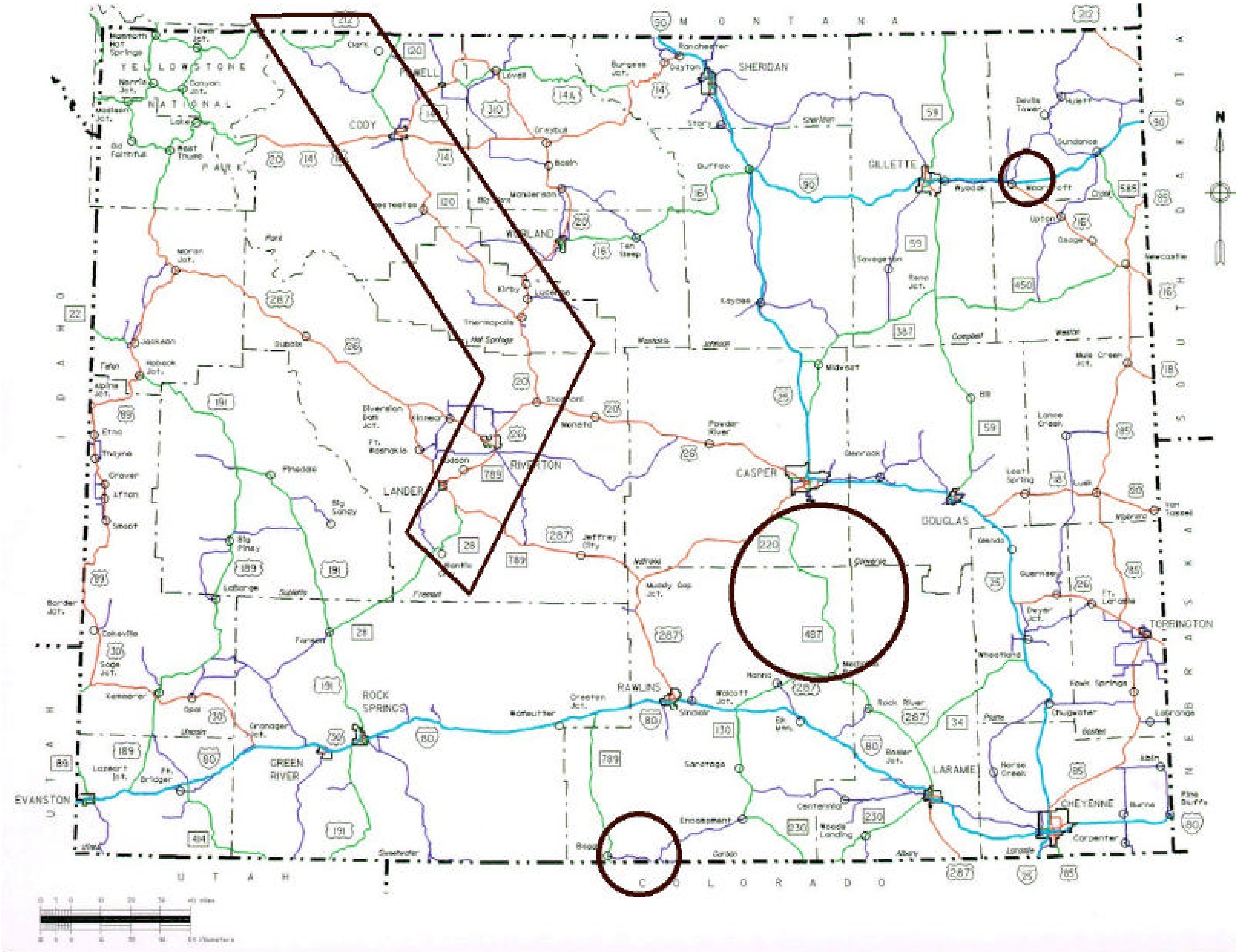

Figure 5. Investigated Site Locations

Figure 6. Photo of Investigated Site Illustrating Roadway Settlement and Recently Placed Cold Mix

Samples of subgrade and backfill soils were collected and transported back to the Geotechnical Engineering Laboratory at the University of Wyoming, where they were tested. Laboratory tests conducted on backfill and subgrade soils from the drill sites included Atterberg limits, grain size distribution, and collapse consolidometer tests .
Investigated Sites

Medicine Bow to Casper

On May 24, 1995 Dr. Thomas Edgar, Dr. John Turner, Mark Stewart, and Justin Lundvall visited the Shirley Basin area with Maintenance Foreman Danny Hobbs. This site is part of the Medicine Bow to Casper project and is Wyoming Department of Transportation project number SCP-PS-024-2(8).

General Background

Construction of this section of WYO 487 was completed during the summer of 1992. Maintenance on the roadway began almost immediately in September of 1992. Mr. Hobbs estimated the rate of construction as one mile of road per day with three inspectors on site. Mr. Hobbs mentioned that in some instances construction of one lane would be completed by morning and the other by afternoon. The north-bound lane displayed the most damage with the exception of two locations (mileposts 50.31 and 50.47) where the center or south-bound part of the road incurred the most damage.

Prior to the 1992 construction no extensive maintenance had been required on this section of road since about 1956.

Geology

The Geological Map of Wyoming (Love and Christiansen, 1985) indicates that this portion of WYO 487 is underlain by the Steele Shale formation. The Steele Shale is of Upper Cretaceous age and consists of gray soft marine shale containing several bentonite beds. Bentonite consists primarily of the clay mineral montmorillonite, and often indicates soils with high plasticity, high compressibility, low shear strength, and undesirable volume change characteristics (shrinking and swelling). Much of the soil in this area appears to consist of sandy, silty clay, and it is highly likely that these soils are derived from the Steele Shale. This suggests the possibility of poor soil conditions as a potential cause of settlement, both in the subgrade beneath culverts and in the backfill. The road also skirts alluvial deposits of Stinking Creek and it is possible that some of the fill is derived from these alluvial deposits.

Summary of Initial Findings

Table 3 lists the locations of culvert sites that exhibited moderate to severe settlement of the roadway or culvert damage. The most severe settlement was observed at mileposts 51.18, 50.47, and 47.97. At milepost 51.18 the culvert is visibly deformed. The center of the culvert appears to have settled significantly more than the ends, resulting in uniform bending along its length (referred to by maintenance personnel as "bowing"). Mr. Hobbs noted that this was the worst pipe in that section. Also at this location the road had been patched twice with 8-10 cm lifts of cold mix each time. Most of the settlement at this location was in the north-bound lane.

At milepost 50.47 settlement occurred within the first year after construction. This culvert is one of the previously mentioned locations where the south-bound lane was damaged while the north-bound lane appears undamaged. Mr. Hobbs estimated that 61 cm of cold mix had been placed at this site in the past two years.

The third location in the severe category is milepost 50.24. This location was patched two years ago with at least 31 cm of cold mix. The pipe at this site is also showing signs of collapse, with separations at three joints.

The final severe damage site was located at milepost 47.97. Prior to the 1992 construction this was a bridge crossing with no problems except for settlement associated with a sinkhole 91 m to the south. It was estimated by Mr. Hobbs that 272 metric tons of cold mix had been used to patch the 38-46 cm of settlement. The last patch was placed in September of 1993 with plans for another patch in the summer of 1995.

Table 3. Culvert Sites on WYO 487, Medicine Bow to Casper

Minor	Moderate	Severe
The remaining 53 culverts		
are classified as minor	54.45	
	54.13	
	54.07	
	53.96	
	53.69	
	53.32	
	51.61	
	51.54	
	51.35	
		51.18
		50.47
	50.31	
		50.24
	49.75	
	49.66	
	49.41	
	48.55	
		47.97
	47.70	
	47.02	

Culvert locations indicated by approximate milepost

Discussion

From this investigation it is apparent there are significant problems with this section of WYO 487. From preliminary observations and remarks made by Mr. Hobbs, poor compaction over and around culverts is a possible cause of the observed damage. Another possible cause of settlement is the presence of bentonite in soils derived from the Steele Shale, which may have been used in backfill and which may form the subgrade beneath culverts in the area. However, one would expect pavement distress in areas of the roadway other than just culvert locations if this were the case.

It was observed that the four sites exhibiting severe settlement are located in relatively shallow fills. The heights of fill covering the culverts were 3.0, 2.8, 1.4, and 1.6 meters, respectively. The ratios of fill height to culvert diameter for theses sites are 3.3, 1.9, 0.9, and 1.8, respectively. Sites exhibiting moderate damage also have relatively low cover. Sites showing only minor settlement in general had greater depths of cover and higher ratios of cover to culvert diameter.

Baggs-Encampment

On June 1, 1995 Dr. Thomas Edgar, Dr. John Turner, Mark Stewart, and Justin Lundvall visited the Baggs area. This site is part of the Baggs-Encampment Road (Baggs Section) project and is Wyoming Department of Transportation project number RS-0401(19).

General Background

Construction on this section of WYO 70 was completed in 1991. Maintenance on the roadway began in 1995, with several patches placed within the week before the site visit. The investigation showed that the road is still in fairly good shape but with a few areas of settlement beginning to appear.

Another feature that showed up in this road section was transverse cracking. The investigation team met briefly with the maintenance foreman, Mr. Buzz Myers. In his opinion the cracks every 12-18 meters are caused by the cement treated base (CTB) used in the road. Mr. Myers also offered his opinion on the cause of settlement at the culverts as being poor compaction during construction. He also feels there are not enough inspectors on the job during a construction project.

Geology

The geological map of Wyoming shows this portion of WYO 70 following the contact between the main body of the Wasatch formation and alluvial deposits of the Little Snake River. The Wasatch is described as "drab sandstone, drab to variegated claystone and siltstone" and in some locations "locally derived conglomerates" (Love and Christiansen, 1985). In general, subgrade soils and fill derived from such rocks would not necessarily indicate problem conditions, such as swelling soils, but actual engineering properties are site specific and the presence of fine-grained materials such as clay and silt raises the possibility of settlement problems.

Summary of Initial Findings

Table 4 lists the culvert sites that exhibited moderate to severe roadway settlement or culvert damage. While no sites fall into the severe category some of the worst settlement on this road section was observed at mileposts 2.68, 3.25, 3.65, 3.70, and 3.90. At milepost 2.68 a patch of cold mix approximately 8 cm thick had been placed recently. Also, the culverts appear to be too close for spacing requirements. On the north side of the road the pipes had a spacing of 0.6 meters and on the south 0.8 meters. Both pipes in this installation are 137 cm in diameter.

At milepost 3.25 settlement has occurred and a patch of cold mix has been placed. The pipe at this location is showing a good deal of bowing. Milepost 3.65 is another location where a patch has been placed recently. This is a double installation with pipe diameters of 91 cm. Both pipes show signs of damage with the eastern most pipe exhibiting more damage. Despite being patched, the middle of the road still appears to have a dip in it. Also, with the patch in place it appears that the worst settlement is in the west-bound shoulder.

Table 4. Culvert Sites on WYO 70 Baggs-Encampment (Baggs Section)

Minor	Moderate	Severe
The remaining 25 culverts are classified as minor	1.92	
	2.27	
	2.68	
	2.87	
	3.06	
	3.14	
	3.25	
	3.65	
	3.70	
	3.90	
	4.36	

Culvert locations indicated by approximate milepost

The fourth location of interest was at milepost 3.70. This location was a triple installation of 107 cm diameter pipes. The pipes all showed signs of damage. The road also showed settlement toward the middle of the roadway.

The final location showing moderate settlement or damage is milepost 3.90. This site was marked to be patched with 8 cm of cold mix. The pipe is showing signs of collapse with a bulge approximately 11 meters from the its north end.

Discussion

Roadway damage due to settlement over culverts on the Baggs-Encampment Road can be characterized as moderate. From preliminary observations and remarks made by Mr. Myers, poor compaction over and around the culverts is a possible cause of the observed damage.

It was observed again that relatively shallow cover exists at the sites exhibiting damage. The five sites exhibiting moderate settlement are located in relatively shallow fills. The fill heights were 1.1, 1.4, 1.8, 0.6, and 0.9 meters, respectively. The ratios of fill height to culvert diameter for theses sites are 1.2, 1.3, 2.0, 0.6, and 0.5, respectively. The sites showing only minor settlement were, in general, located in deeper fills.

Pine Haven Road

On June 13-14, 1995, Mark Stewart, and Justin Lundvall visited the Pine Haven area and met with Jay Gould the District Maintenance Engineer and Barry Bowersox the Maintenance Foreman. The section inspected is the Pine Haven Road project and is Wyoming Department of Transportation project number SCP-0608(1).

General Background

Construction on this section of WYO 113 was completed in 1991. Some maintenance has been conducted with several patches placed. Most of the patches have been made over stockpass locations. Mr. Bowersox estimated that the patches placed over the stockpasses were about 5 cm thick. Transverse cracking was observed in the road at the patched stockpasses. According to Mr. Gould and Mr. Bowersox, the fill height over the culverts is not adequate. Mr. Gould also felt the problems are with corrugated metal pipe (CMP) culverts rather than with concrete culverts. All of the culverts on this project are CMP. They noted that some of the pipes had been strutted during construction for support to achieve better compaction around them.

The investigation showed that the road has problems with heaving toward the west end of the project. It also appeared that many sections of this road are in worse shape than the culvert installations. This could be caused by the bentonitic soils in the area.

Geology

The Geological Map of Wyoming shows the damaged portion of Pine Haven Road as being underlain by the Pierre Shale and the Carlile Shale. Both the Pierre and Carlile are upper Cretaceous marine shales noted for containing bentonite beds. In fact much of the commercial bentonite mining industry is located in this general area. Both of these formations are notorious for causing problems with foundations, pavements, landslides, etc. because of their low shear strength, high plasticity and compressibility, and swelling characteristics. The observation that the problem areas along Pine Haven Road correspond directly with the location of these bentonitic shales suggests that geological conditions may be a contributing factor. It should be noted that Pine Haven Road east of the problem section is also underlain by Cretaceous, bentonitic shales of the Belle Fourche, Newcastle, and Cloverly formations.

Summary of Initial Findings

Table 5 lists the culvert sites that exhibited moderate to severe roadway settlement or culvert damage. No culvert sites on this section fall into the severe category. The most significant sites were observed at mileposts 0.68, 1.05, 1.57, 3.45, 3.65, and 3.92. Milepost 0.68 is a stockpass with a patch of cold mix approximately 5 cm thick. There is slight bowing of the pipe, but no observable settlement of the roadway.

Table 5. Culvert Sites on Pine Haven Road.

Minor	Moderate	Severe
The remaining 20 culverts are classified as minor	0.68	
	1.05	
	1.57	
	2.70	
	3.45	
	3.92	

Culvert locations indicated by approximate milepost

At milepost 1.05 a 5 cm patch has also been placed. Approximately 9 meters from the centerline of the pipe a transverse crack has formed through the patch. The pipe is showing signs of collapse with approximately 10 cm of bowing in the middle of the pipe. Both ends of the pipe are also showing some damage.

Milepost 1.57 is a stockpass location where a patch has been placed. There is a transverse crack in the roadway approximately 23 m east of centerline of the culvert. This crack is just beyond the reach of the patch. The pipe is also showing some damage from the middle to the north end of the pipe.

The fourth location of interest is a stockpass at milepost 2.70. There was also a 5 cm patch at this location and we observed a 1.5 meter long transverse crack from the east-bound lane shoulder to the center of the road. This crack is approximately 6 meters from the center of the culvert. The pipe at this site appears to be in good shape.

Moderate settlement and damage were observed at milepost 3.45. Transverse cracks have formed on both sides of the pipe, one at a distance of 3.7 meters to the west of the culvert and the other 4.6 meters to the east. Settlement is slight with most of it in the east-bound lane. There was also a bump in the east-bound lane on the transverse crack. The pipe exhibits slight bowing on the north end, north of the west-bound lane shoulder.

The final location showing moderate settlement and damage was at milepost 3.92. The culvert at this location was originally designated to be a 122 cm CMP but was changed to a stockpass. The culvert is a 305 cm diameter metal pipe. The east side of this pipe appears to be collapsing. A 5 cm patch has been placed and there is a transverse crack 3 meters west of the centerline of the pipe. There is also a longitudinal crack along the east-bound lane shoulder stripe. Most of the settlement appears to be in the east-bound lane.

Discussion

Problems with the Pine Haven Road can be rated as moderate to severe. However, the severe problem areas are not associated with culverts. A possible cause of heaving and settlement is bentonitic soils which are known to exist in this area, and which may undergo large volume changes when changes occur in moisture content. The observations of damage in parts of the roadway other than at culvert locations support poor soil conditions as a likely cause of roadway damage.

Again it was noted that the six sites exhibiting the greatest settlement on the project are located in relatively shallow fills. The fill heights for these six sites are 1.4, 1.1, 1.9, 1.2, 3.3, and 1.5 meters respectively. The ratios of fill height to culvert diameter for theses sites are 0.5, 0.4, 0.6, 0.4, 1.1, and 0.5, respectively. This analysis supports the comments made by Mr. Gould and Mr. Bowersox that low cover over culverts may result in settlement problems.

District Five Sites

During the period July 6-10, 1995, Mark Stewart and Justin Lundvall visited Wyoming Department of Transportation District Five. The sites visited included areas: (1) south of Lander, (2) between Thermopolis and Meeteetse, (3) south of Cody, (4) north of Cody, (5) east of Cody, and (6) south of Worland. Table 6 summarizes the roadway sections and project numbers inspected.

Table 6. Projects Inspected, District Five

Location	Project Title	WYDOT Project Number
US 287, south of Lander	Lander-Muddy Gap Road	FR-R020-2(18)
WYO 120, Thermopolis to Meeteetse	Owl Creek Section	SCP-033-1(5)
	Spring Draw	SCP-033-1(8)
	Grass Creek Section	F-033-1(7)
WYO 120, Meeteetse-Cody	Meeteetse-Cody, Park Co.	F-033-2(7)
WYO 120, north of Cody	Skull Creek	STPP-033-3(6)
North of Cody	Chief Joseph Scenic Hwy.	RSDE-0200(805)
US 14-16-20, east of Cody	Cody-Greybull, Park Co.	F-031-1(20)
US 20, south of Worland	Worland-Thermopolis, Washakie Co.	F-0171(2)

General Background

Construction on the Lander section of Wyoming 287 was completed in 1979. According to Maintenance Foreman Bill Hart, there have been problems within this section around milepost 74 with patching required every other year. During our visit we observed additional problems between mileposts 78 and 80.

The Owl Creek section of WYO 120 between Thermopolis and Meeteetse was completed in 1984. This section includes several locations with notable damage. According to Rick Wren, the Maintenance Foreman, one site has been patched every year and at the time of the visit another patch had been placed recently at milepost 4.2, just before a pull out.

The Spring Draw section also included several sites with apparent damage, although not as severe as in the Owl Creek section. The final section visited between Thermopolis and Meeteetse was the Grass Creek section. This section had severe damage at milepost 26.6. This site has been patched in the past and was in need of further maintenance at the time of the visit.

Several locations were visited in the Cody area. The first site is located south of Cody between Cody and Meeteetse on Highway 120 and was constructed in 1968. Other locations include the Skull Creek area and the Chief Joseph Scenic Highway. Resident Engineer Dave Schultz felt that problems encountered on the Chief Joseph section might be caused by poor soils used as backfill. For the section east of Cody on US 14-16-20, Gale Beers, the Maintenance Foreman, feels that problems are due to inadequate compaction.

The final area visited is south of Worland on US 20. Several locations exhibit moderate to severe damage. This project was completed in 1935 with areas just recently patched. According to Maintenance Foreman Gary Hoffman there have been some settlement problems in this area.

Geology

The general bedrock geology varies among the District Five sites. Table 7 summarizes the formations believed to underlie each of the nine sections and their descriptions as given in the Geological Map of Wyoming. It may be significant to note that, except for a single culvert installation on the Chief Joseph and a short section with three culverts between Worland and Thermopolis, all of the problem sites are underlain by Cretaceous shales of the Cody, Mowry, and Lance formations. The Cody and Mowry shales are known for containing bentonite beds.

Table 7. General Geology of District Five Sites[1]

Project	Formation	Description
Lander-Muddy Gap Road	Cody Shale	dull-gray shale, gray siltstone, and fine- grained gray sandstone (Cretaceous)
Owl Creek	Mowry Shale	silvery-gray, hard sileceous shale containing bentonite beds (Cretaceous)
Spring Draw	Cody Shale	
Grass Creek	Lance Fm.	thick-bedded buff sandstone and drab to green shale; thin conglomerate lenses (Cretaceous)
Meeteetse-Cody	Cody Shale	
Skull Creek	Cody Shale	
Chief Joseph Scenic Hwy.	Amsden	red & green shale, dolomite
Cody-Greybull	Mowry and Lance	
Worland-Thermopolis	Fort Union	brown to gray sandstone, gray to black shale; thin coal beds
	Willwood	variegated claystone, shale, and sandstone

1. General bedrock geology and descriptions from Love and Christiansen (1985)

Summary of Initial Findings

Table 8 summarizes the culvert sites south of Lander that exhibited moderate to severe roadway settlement or culvert damage. The most severe damage and settlement occurred at milepost 77.55. This is a 305 cm diameter stockpass with invert paving. The pipe is showing considerable damage with ± 20 cm of deformation, with the worst damage appearing under the south-bound lane shoulder. The roadway is also displaying 15-20 cm of settlement. This site has been treated with an overlay and it appears to have been patched also. The fill height was approximated to be 4 meters. There is also a 61 cm drainage culvert located next to and to the north of the stockpass.

Table 8. Culvert Sites on WYO 287 South of Lander

Minor	Moderate	Severe
Culverts not listed	77.27	
as moderate or severe		77.55
are classified as minor	78.20	
	79.02	

Culvert locations indicated by approximate milepost

The other culvert sites displayed only moderate damage or settlement. Milepost 77.27 is a 122 cm CMP which appears to be bowed 8-10 cm along the length of the pipe. Settlement in the road appeared to be directly over the culvert. The middle of the roadway appeared to be bowed with a hump in the north-bound lane shoulder stripe. This hump appeared to be caused by the rest of the road settling while it remained intact. The settlement was approximately 10 cm across both lanes. A transverse crack across the entire road was also noted at this location, 18 meters north of the pipe.

At milepost 78.2 there is a 183 cm diameter concrete pipe that appeared to be disjointed toward the middle of the road. Looking through the pipe it was observed that the sixth joint eastward of the flared end was coming apart, causing a disruption of water flowing in the culvert. The roadway appeared to be in fairly good condition with about 5 cm of noticeable settlement. This location also exhibited a transverse crack 11 meters north of the pipe.

The final location on this stretch of WYO 287 displaying moderate problems was at milepost 79.02. The culvert at this site is a 61 cm CMP with asphalt coating. The pipe appeared to be in good shape with the road displaying 10 cm of settlement. A transverse crack was also observed 3.7 meters south of the culvert.

Table 9 lists the culvert sites exhibiting moderate to severe damage on WYO 120 between Thermopolis and Meeteetse. Mileposts 4.2 and 4.98 (after the equation adjustment) demonstrated the most severe damage or settlement in the Owl Creek section. Milepost 4.2 is just before a pullout along WYO 120. According to Mr. Wren this section was patched in August 1994 and was nearly ready for another patch. Mr. Wren also said this had been a problem area for the past 4 years. The site exhibited close to 20 cm of patching on the north bound shoulder and approximately 15 cm of patching on the south-bound shoulder. The culvert is a 61 cm concrete pipe that was half silted in. The fill height above this culvert is in the 4.6-5.5 meter range. Transverse cracks were observed at this site 5.2 meters to the north and 6.1 meters to the south. This location also has a longitudinal crack 0.3 meters from the north shoulder stripe and the centerline stripe.

Table 9. Culvert Sites on WYO 120 Thermopolis-Meeteetse

Minor	Moderate	Severe
Culverts not listed as moderate or severe are classified as minor		
Owl Creek Section		4.2
	3.4*	
	3.75	
	4.08	
		4.98
Spring Draw Section	9.95	
Grass Creek Section		26.6

Culvert locations indicated by approximate milepost
* Note culverts are past equation 4.21 BK = 3.21 AH.

Another location in the severe category is milepost 4.98. This site had a 8 cm patch approximately 18 meters long and a moderate dip in the roadway. The culvert is an arch pipe 183 cm in height. The pipe has deflected on the west end with the north side caving in. There is approximately 13 cm of deformation in this pipe. The portion of the pipe beneath the north-bound lane seemed to show the worst settlement.

Other locations investigated on this stretch of road that showed moderate damage or settlement included mileposts 3.4, 3.75, and 4.08. Milepost 3.4 exhibited a 10-15 cm patch along with transverse and longitudinal cracks in the roadway. This was a 61 cm CMP with 3.0-3.7 meters of fill over the top of the culvert. At milepost 3.75 there is a 91 cm concrete culvert that appears to be in good shape. There is 5.5-6.1 meters of fill over this pipe. The pavement has transverse and longitudinal cracking. The south-bound lane has a noticeable dip and appears to have the worst settlement problems. The final location exhibiting moderate problems is at milepost 4.08. This location had a 8-10 cm patch with the settlement being worse in the north-bound lane. There is also a slight dip in the north-bound lane. The roadway also exhibits transverse and longitudinal cracking. The culvert at this location is 198-244 cm in diameter, is asphalt lined, and appears to be in good shape. The fill height above the culvert is 1.5 meters on the west and 2.4 meters on the east.

In the Spring Draw section one site exhibited moderate damage, at milepost 9.95. The roadway has 8-10 cm of settlement and no signs of a patch. The pavement also has transverse and longitudinal cracking. The culvert is a 122 cm CMP, asphalt coated, with bowing occurring under the south-bound lane. This culvert has 9.1+ meters of fill over the pipe.

The final project checked between Meeteetse and Thermopolis was the Grass Creek section. Milepost 26.6 in this section demonstrated severe damage. This was a double 61 cm CMP installation. Each pipe showed slight bowing with perhaps slightly more damage in the pipe on the north side. The roadway has a 10 cm patch with ± 10 cm of additional settlement in the north-bound lane. The south-bound lane appeared to be in good condition compared to the north. A 76.2 meter patch has been placed at this section covering the whole road. Also of interest at this site were tire marks to the north of the dip.

Table 10 lists the culvert locations inspected in the Cody Area. Only one site between Meeteetse and Cody exhibited notable damage, at milepost 71.3. This was a 122 cm concrete culvert with a 15 cm bow toward the middle of the pipe. This pipe is in a deep fill of approximately 12.2 meters in height. The road shows severe signs of settlement with a recent 15-23 cm patch but still leaving a gradual dip in the road.

Table 10. Culvert Sites, District Five-Cody Area

Minor	Moderate	Severe
All other culverts not listed as moderate or severe Settlement are classified Within the minor category		
Meeteetse-Cody Section Wyoming 120		71.3
Skull Creek Section	111.55 114.18*	
Chief Joseph Scenic		44.15
Cody-Greybull US 14-16-20	56.0 60.0	

Culvert locations indicated by approximate milepost

- Note old section plans to rebuild.

North of Cody on WYO 120 moderate damage was observed in the Skull Creek section. At milepost 111.55, a 305 cm stockpass displayed moderate settlement problems with 5-8 cm of settlement in the roadway. The pipe appears to be in good shape with only minimal fill of 46 cm plus surfacing to about 0.9 meters. This is a new section of road and there are no signs of patching or cracking. Another pipe just up the road at milepost 111.5 which is a 305 cm drainage pipe with approximately 1.5 meters of cover showed only slight settlement of the road. Sites were also inspected on the old road north of this new section. One site at milepost 114.18 exhibited moderate settlement of 8-10 cm. This was a 91 cm CMP with 6.1-9.1 meters of fill over the pipe. A site showing minor damage was at milepost 111.8. This was also a relatively deep fill with 6.1-7.6 meters of cover. The culvert is a 61 cm CMP, asphalt lined, with a bow in the center. The road showed no signs of damage directly over the culvert but 45-61 meters north of the culvert there was a dip in the road. The road showed approximately 5 cm of settlement in a bowl shape toward the middle of the road.

At milepost 44.15 on the Chief Joseph Scenic Highway is a 274 cm stockpass with no noticeable damage to the pipe. The road displayed 7.6+ cm of settlement directly over the culvert. There were also transverse cracks 5.5 meters to the east and 6.1 meters to the west of the culvert.
To the east of Cody on US 14-16-20 two sites were inspected, one at milepost 56 and the other at milepost 60. Each of the sites had been patched 2 to 3 times with 2.5 cm patches, according to Maintenance Foreman Gale Beers. Milepost 56 has a 198 cm CMP with approximately 1.2 meters of fill above it. The road also displayed transverse cracking. Milepost 60 was a double installation of 114 X 81 cm arch pipes. The spacing on these pipes appeared to be inadequate with about one pipe diameter between them. This site displayed transverse cracking between the pipes and also 9 meters to the east and west of the pipes.

Culvert sites between Worland and Thermopolis on US 20 are listed in Table 11. Three sites exhibited moderate to severe damage. The severe site was at milepost 152.5 where Gary Hoffman pointed out a patch has been placed every year for the past 5-6 years. At the time of the visit a recent 2.5 cm patch had been placed, with a dip still being felt in the road. It appears that the north-bound lane currently has the greater settlement problem. There is also transverse cracking under the new patch. The culvert at this location is a 244 cm CMP which appears to be in fairly good shape. The culvert is slightly out of round but in the opposite direction expected. This culvert has 9.1-10.7 meters of cover.

Table 11. Culvert Sites on US 20 Worland to Thermopolis

Minor	Moderate	Severe
Culverts not listed		152.5
as moderate or severe	152.3	
are classified as minor	152.1	

Culvert locations indicated by approximate milepost

Moderate damage or settlement was observed at milepost 152.3. This site had been patched but a dip in the road is still noticeable. The settlement appears to be more severe in the south-bound lane. The culvert at this location is a 91 cm CMP which appears to be out of round with fairly severe bowing of 10-13 cm. The fill on this culvert varies on each side of the road with approximately 2.4 meters of cover on the west side and 2.4-3.7 meters on the east.

The final location investigated was at milepost 152.1. This site had been recently patched with a 2.5-5 cm lift. There is still a slight dip in the south-bound lane. The culvert at this location is a 107 cm CMP with 1.5 meters of fill. The pipe is in relatively good shape with slight bowing occurring in the center.

Discussion

Moderate and severe problems were found on several road sections in District Five. From discussions with WYDOT maintenance engineers, inadequate compaction over and around culverts is a possible cause of the observed damage. Another possible cause of settlement, at least for several of the sections, is problematic subgrade and backfill soils associated with bentonitic Cretaceous shales.

Fill heights and the ratio of fill height to culvert diameter are tabulated in Table 12 for sites in District 5 exhibiting moderate to severe damage. In general, most of these sites have relatively low cover and low ratios of fill height to pipe diameters. There are several notable exceptions where roadway settlement has occurred over culverts with deeper fill heights.

Table 12. Fill Heights Over Culverts, Sites in District Five

Section	Milepost	Diameter (cm)	Cover Height (m)	Cover/Diameter Ratio
Lander-Muddy Gap	77.27	122	3.0	2.5
	77.55	305	4.3	1.4
	78.20	183	2.7	1.5
	79.02	46	2.7	6.0
Owl Creek	3.4	91	11.0	5.0
	3.75	61	6.1	10.0
	4.08	213	1.5	0.7
	4.2	168	4.6	2.7
	4.98	284 x 191	3.0	1.3
Spring Draw	9.95	168	8.5	5.1
Grass Creek	26.6	2 @ 61	2.1	3.5
Meeteetse-Cody	71.3	122	12.2	10.0
Skull Creek	111.55	305	0.91	0.3
	114.18	76	7.6	10.0
Chief Joseph Hwy.	44.15	274	1.3	0.5
Cody-Greybull	56.0	274	2.4	0.7
	60.0	135 x 91	1.2	1.1
Worland-Thermopolis	152.1	76	1.5	2.0
	152.3	107	2.4	2.3
	152.5	244	10.7	4.4

DRILLING PROGRAM

Subsurface borings were conducted at fourteen culvert locations. Locations of each boring are given in Table 13. The procedure at each site was as follows. A hollow stem auger containing a 1.5 meters long core sampler was used to excavate the soil in 1.5 meter increments. When the tip of the first auger section was advanced to a depth of 0.9 meters, a 5 cm OD split spoon sampler was lowered through the auger stem, and a standard penetration test (SPT) was conducted over a depth of 46 cm. The number of hammer blows required to advance the sampler over the final 31 cm of penetration was taken as the N-value. The next auger section was then used to advance the boring an additional 1.5 meters and the procedure was repeated. This was continued until relatively intact bedrock was encountered. Two types of soil samples were obtained. The first were bulk samples taken directly from the sampler within the hollow stem auger. The second type included relatively undisturbed samples taken from the split tube sampler used in conjunction with the SPT test.

Table 13. Summary of Drilling Locations

Location and Road	Milepost[1]	Culvert Size[2], cm	Cover Height, m	Cover/Diameter Ratio (C/D)	Depth of Boring, m
Medicine Bow - Casper	47.97	3@91	1.6	1.8	7.4
Shirley Basin	50.24	152	1.4	0.9	11.9
WYO 487	50.47	152	2.8	1.9	9.0
	51.18	91	3.0	3.3	7.1
	51.54	91	1.0	1.1	7.4
Baggs -Encampment	3.14	2@91	1.4	1.5	7.4
WYO 70	3.25	107	1.4	1.3	7.7
	3.65	2@91	1.8	2	6.0
	4.36	2@91	4.9	5.3	6.0
Thermopolis-Meeteetse	3.4[3]	91	4.6	5	7.4
Owl Creek	4.2	168	4.6	2.7	8.9
WYO 120	4.98	284 x 191	3.0	1.3	7.4
Thermopolis-Meeteetse Grass Creek WYO 120	26.6	2@61	1.4	3.5	2.4
Worland-Thermopolis	152.3	107	2.4	2.3	6.9
US 20	152.5	244	10.7	4.4	15.9

1. Culvert locations indicated by approximate milepost.
2. All culverts sampled were round CMP with the exception of the arch pipe with the given dimensions.
3. BK = 3.21 AH.

Using the field boring logs, subsurface profiles were constructed for each site. These profiles are presented in the thesis by Lundvall (1997) and show detailed descriptions of each soil type and SPT N-values versus depth, location and size of each culvert, and other subsurface information. Test methods and results of the laboratory tests are discussed in more detail in the next two chapters. Test results were used to evaluate compressibility, compaction characteristics, potential for swelling soils, and other soil properties which may have affected settlement.

PROBABLE CAUSES OF SETTLEMENT

Information collected by examining project records, speaking to maintenance personnel, and site inspections suggests several possible causes of roadway settlement and damage at culvert crossings. One obvious concern of all Wyoming DOT field personnel is the possibility of inadequate compaction of backfill around and above culverts. There is a perception that there are not enough inspectors on most road construction jobs to insure that the contractor conforms to compaction specifications. Several maintenance foremen mentioned that in some cases no inspector was present during the entire culvert backfilling operation. Additional inspectors are not likely to be assigned because of budget limitations and therefore this situation is not likely to improve.

A second factor which appears to be significant is the depth of cover between culvert and pavement. It was observed that a high percentage of sites with moderate to severe damage seemed to have relatively low cover depths. To evaluate this further, graphs were developed showing depth of cover (in meters) versus ratio of cover depth to culvert diameter (referred to herein as cover ratio). Data points were distinguished on the basis of minor, moderate, and severe damage, as defined in the beginning of this report. These graphs are included herein as Figure 7 through Figure 10. Figure 7 shows all of the culverts on the roadway section that was investigated on WYO 487 between Medicine Bow and Casper. For a cover height and cover ratio of 3.5 or less, 32% of the culverts show moderate or severe damage. In addition, all locations showing severe damage are within this range of cover height. For cover heights and cover ratios between 3.5 and 5, 33% of all culvert sites showed moderate damage, while above a cover depth and cover ratio of 5 no sites showed anything above minor damage.

Figure 8 and Figure 9 show similar graphs for WYO 70 between Baggs and Encampment (Figure 8) and Pine Haven Road (Figure 9). In each case, only minor or no damage was observed for cover heights over 3.5 to 5 m and for cover ratios over 3 to 6.5. Figure 10 shows a similar plot for the sites in District 5. The data collected in District 5 differs from the other three in that only locations reported as being problems were investigated, while other culvert sites on the same projects were not inspected. Therefore it was not possible to determine the percentage of sites with moderate or severe damage for a given cover height or cover ratio. District 5 includes several sites at which severe and moderate damage was observed even though cover heights and cover ratios were relatively high.

Although there are some exceptions, these data strongly suggest that culvert sites having shallow cover, say 5 m or less, are more likely to undergo damaging roadway settlement. Only two sites exhibiting severe damage have cover heights of greater than 5 m, and it may be that factors other than cover height are significant at these sites.

A third factor which emerges from the investigation is the possible influence of site geology and its influence on the types of soils used as backfill. It was found that the majority of sites at which damaging settlement has occurred are located in areas underlain by Cretaceous sedimentary rocks known to contain bentonite, which is a strong indicator of swelling, highly compressible soils. Furthermore, soil samples obtained from drilling at these sites are for the most part fine-grained soils such as clay, sandy clay, silty clay, or clayey silt. Laboratory tests on samples of these soils indicated that many of the soils used as backfill are highly plastic and compressible fine-grained soils. Standard Penetration Test N-values indicate “medium” stiff consistency for these cohesive soils. See Chapter 2 *Literature Review* for further discussion on fine compactible soils.

SUMMARY

In summary, there appear to be three factors having a high probability of resulting in damaging roadway settlement at culvert sites on Wyoming highways:

1. inadequate compaction, as a result of fewer inspection personnel
2. shallow cover of fill above culverts
3. the use of plastic, compressible subgrade soils and/or fill materials derived from bentonitic Cretaceous shales.

For further discussion of the soils and soil properties found and analyzed from the field investigation see Chapter 5 *Results of Laboratory Testing Program*.

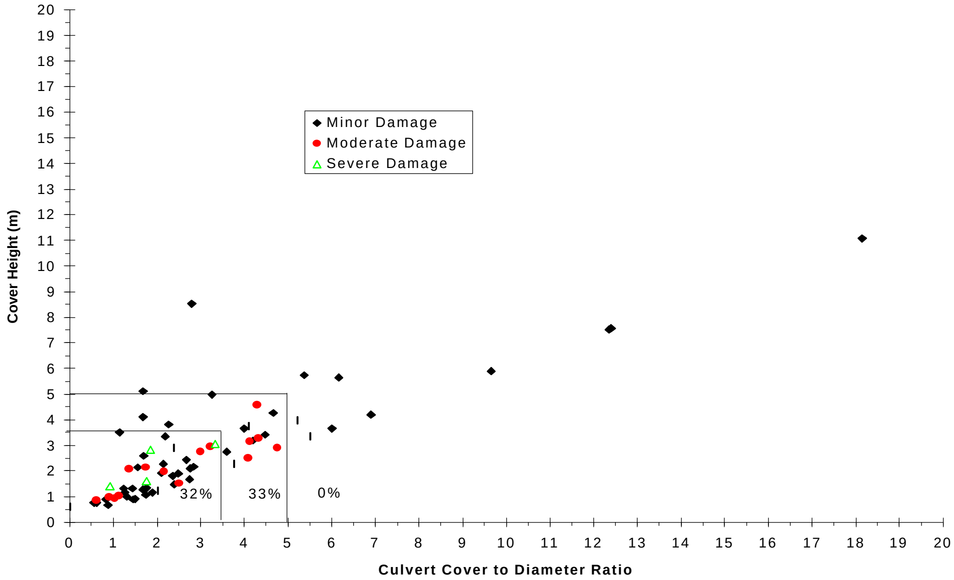

Figure 7. Cover Height versus Cover Ratio, Shirley Basin

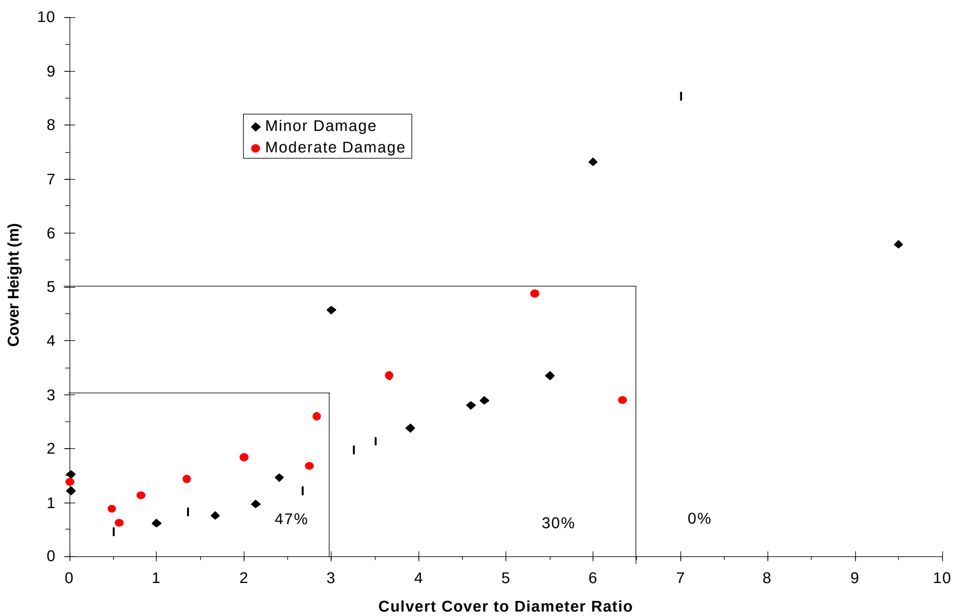

Figure 8. Cover Height versus Cover Ratio, Baggs-Encampment

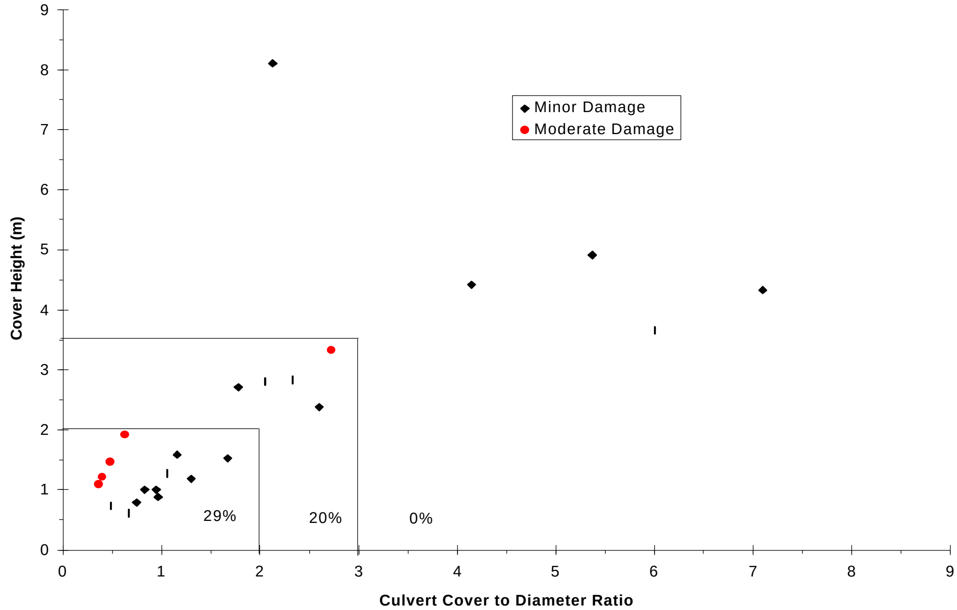

Figure 9. Cover Height versus Cover Ratio, Pine Haven Road

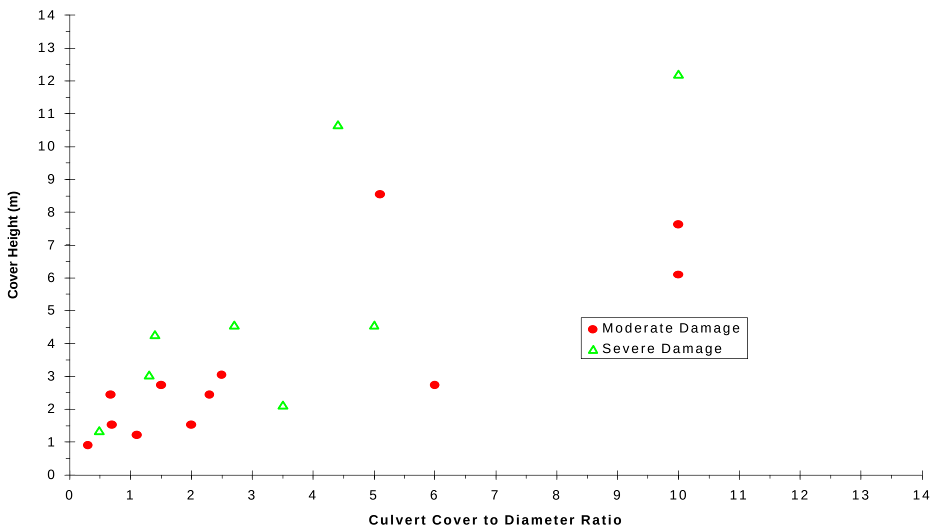

Figure 10. Cover Height versus Cover Ratio, District Five Sites

4: LABORATORY TESTING PROGRAM

This chapter describes the laboratory tests conducted to evaluate causes of settlement and to investigate methods for reducing damage to roadways.

ANALYSIS OF FIELD SAMPLES

During the field investigation and drilling program (see Chapter 3 *Field Investigation of Roadway Settlement*), soil samples were collected and brought back to the Soil Mechanics Laboratory at the University of Wyoming for analysis. Samples included water content, bulk, and tube samples. Tests conducted on the tube samples provided the primary source of information for evaluating causes of settlement. For each site that was drilled, two tube samples were tested where applicable. These included one tube sample from above the culvert and one tube sample from below the culvert.

Index Properties

Index properties were determined for the tube samples in order to classify the types of soils found above and below the culverts. Index properties that were measured included Atterberg limits, grain size distribution, and specific gravity. All test procedures followed ASTM and AASHTO standards where applicable, including the following: ASTM D 4318 "Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils," ASTM D 422 "Standard Method for Particle- Size Analysis of Soils" and ASTM C 136 "Standard Method for Sieve Analysis of Fine and Coarse Aggregates," and. ASTM D 854 "Standard Test Method for Specific Gravity of Soils".

Based on index test results, the soils were classified according to AASHTO specifications. The specific gravity was required for calculations needed for analysis of tests such as Collapse Index, which is discussed in the next section. Index test results and classifications are presented in Chapter 5 *Results of Laboratory Testing Program*.

Collapse Index

Laboratory consolidation tests on tube samples were conducted to determine the collapse index. This testing followed ASTM D 5333 "Standard Test Method for Measurement of Collapse Potential of Soils". All tests were conducted on tube samples, which are relatively undisturbed. The collapse index percent (I_e) is defined as the relative magnitude of collapse determined at 200 kN/m^2 (2 tsf) and calculated using one of three equations given in the ASTM standard. The equation used for this study is as follows:

$$I_e = \frac{\Delta e}{1+e_0} * 100 \qquad \text{(Equation 4.1)}$$

where: Δe = change in void ratio resulting from wetting, and
e_o = initial void ratio.

For this study, the collapse index was determined in order to make comparisons of the specimens tested from different sites, as well as the specimens above and below each culvert location. A summary of the test method as given by ASTM D 5333 is as follows:

> "The test method consists of placing a soil specimen at natural water content in a consolidometer, applying a predetermined applied vertical stress to the specimen and inundating the specimen with fluid to induce the potential collapse in the soil specimen. The fluid should be distilled water when evaluating the collapse index, I_e. The fluid may simulate pore water of the specimen or other field conditions as necessary when evaluating collapse potential, I_c."

To conduct the collapse index tests, two 351 Conbel consolidometers conforming to the equipment requirements of ASTM D 2435 "Standard Test Method for One-Dimensional Consolidation Properties of Soil" were used. Cutting rings and porous stones were fabricated to conform to the sample sizes of ± 5.7 cm (2.25 in) in diameter and approximately 1.9 cm (.75 in) in height, although specimen height varied slightly for each test.

The testing procedure consisted of loading the specimen at its natural water content in incremental loads corresponding to compressive stresses of 12, 25, 50, 100, 200, etc. kN/m^2 (0.12, 0.25, 0.5, 1, 2 tsf). Each load level was applied for approximately one hour. After reaching a stress of 200 kN/m^2 and allowing this load to remain constant for one hour, deaired water was added to inundate the specimen. Specimen deformation was recorded at the beginning of each load increment and at the end of the load duration. Once the specimen was inundated, readings were recorded for deformation versus time at approximate time values of 0.1, 0.25, 0.5, 1, 2, 4, 8, 15, 30 minutes and 1, 2, 4, 8, and 24 hours, as according to Test Method D 2435. In most cases, the duration of the load increment following inundation was overnight or until primary consolidation occurred according to ASTM D 2435.

Additional load was applied to the specimens after inundation, following the load increments corresponding to 400, 800, and 1,600 kN/m^2 (4, 8, and 16 tsf). Recordings of deformation versus time were made once again as described above. Also, the load was left on in the same manner as described above. Soil classifications and results of collapse index tests are discussed further in Chapter 5 *Results of Laboratory Testing Program*.

RESEARCH TO EVALUATE SETTLEMENT MITIGATION METHODS

To evaluate various methods to minimize or eliminate roadway settlement (see Chapter 2), laboratory model load tests were conducted. With the aid of University of Wyoming civil engineering graduate student Scott Riggs, a reinforced concrete testing chamber was constructed for the purpose of conducting load tests on a model culvert placed in compacted fill. The chamber was constructed with the ability to adjust the length up to 243 cm. The chamber is 165 cm wide and 152 cm in height. For this study, the length of the chamber was 137 cm.

Figure 11 illustrates the testing chamber in plan and elevation views. The two larger walls are 17.8 cm thick while the smaller walls are 7.6 cm thick. All walls have steel channels around their outer edges.

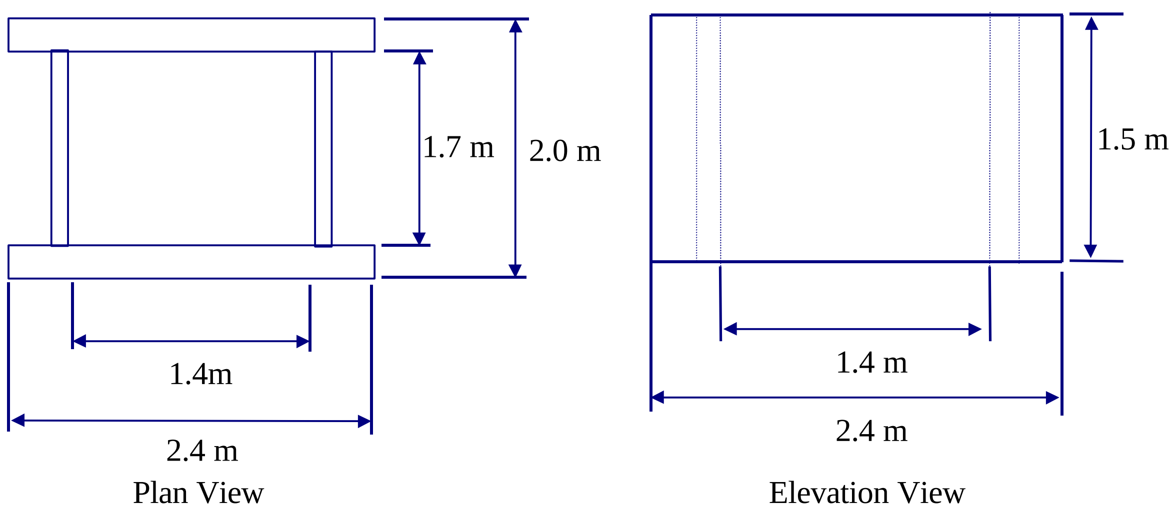

Figure 11. Dimensions of Concrete Testing Chamber

A total of seven model tests were conducted. Three different fills were used for these tests. The first was a clay soil classified as A-7-6 according to the AASHTO classification system. The soil was obtained from the Shirley Basin area on Wyoming Highway 487 near milepost 50.47. This material was obtained from a culvert site, and was used as backfill in construction of the roadway. The second soil that was used was a sand classified as A-1-b according to the AASHTO classification system. Additionally, the specific gravities (G_s) of these materials were found to be 2.71 and 2.61 for the clay and sand respectively. The final fill material was CLSM or flowable fill. Table 14 summarizes the mix design and compressive strengths attained.

Table 14. CLSM Mix Design and Attained Compressive Strengths

Material	Quantity (kg/m^3)	Time Days	Strength (kN/m^2)
Water	273.0	1	79.6
Fly Ash	148.4	2	87.8
Fine Aggregate	1661.7	7	367.6
Cement	29.7	14	628.2

Compressive strengths were measured on 10.2 cm x 20.3 cm cylinders.

Additionally pocket penetrometer tests were conducted on the CLSM. Average values were .75 kg/cm^2 at 5 hours after pouring, 3.94 kg/cm^2 after 19 hours, 4.17 kg/cm^2 after 24 hours, and 4.68 kg/cm^2 after 30 hours.

The clay soil had the following index properties: liquid limit of 46.3, plastic limit of 24.9, and a plasticity index of 21.4. The sand was non-plastic. The grain size distribution charts for these soils are given in the thesis by Lundvall (1997), Figures C-26 and C-27. Additionally, the moisture-density relationship used for field compaction of the clay was based upon the WYDOT specification for "Wyoming Modified", which follows AASHTO T-99.

The first test on the clay soil was used as a trial run to learn the operation and control of the MTS actuator. Of the remaining six tests, two were conducted on unreinforced clay from the Shirley Basin area; two more tests were performed on geogrid reinforced clay from the Shirley Basin area. One test was completed on a backfill of granular sand and the final test was conducted on CLSM backfill. All tests had a 10 cm compacted lift of the Shirley Basin area clay underneath the culvert with the remaining portion of the base (20 cm) being compacted granular sand Table 15 gives the properties of the geogrids.

The final test was conducted using the CLSM or flowable fill mixture. Initially two tests were to be conducted on the sand material. One test was done without any reinforcing or soil modification. The second test that was planned was to be with geogrid reinforcing, however after the results of the first test, it was determined that this test was not necessary.

Two tests were conducted with unreinforced clay backfill because of the procedure used to construct the test setup. To eliminate variations due to the construction of the embankment, the base below the culvert was left in place for all tests. This base was approximately 0.3 meters in thickness with 20 cm consisting of sand and then 10 cm of clay in contact with the culvert. The two tests indicated if the base was affecting the results of the tests.

Table 15. Properties of Tested Geogrids

Property	Units	Tensar BX1100	Tensar BX1300 (SS-3)
Interlock			
aperture size			
MD	in	1.0 (nom)	1.8 (nom)
CMD	in	1.3 (nom)	2.5 (nom)
open area	%	70 (min)	75 (min)
Thickness			
Ribs	in	0.03 (nom)	0.049 (nom)
Junctions	in	0.11 (nom)	0.17 (nom)
Reinforcement			
flexural rigidity	mg-cm	250,000 (min)	450,000 (min)
ultimate strength			
MD	lb/ft	855 (min)	N.G.
CMD	lb/ft	1,400 (min)	N.G.
tensile modulus	lb/ft	14,000 (min)	15,000 (min)
Junctions			
Strength	lb/ft	770 (min)	960 (min)
Efficiency	%	90 (min)	90 (min)
Material			
polypropylene	%	98 (min)	98 (min)
carbon black	%	0.5 (min)	0.5 (min)
Dimensions			
roll length	ft	164 (min)	164 (min)
roll width	ft	9.8 & 13.1	9.8 & 13.1
roll weight	lb	71 & 95	82 & 109

Notes: MD dimension is along roll length. CMD dimension is across roll width.
Maximum inside dimension in each principal direction measured by calipers.
Secant aperture stability modulus value listed is equal to the mean value less approximately one standard deviation.
N.G. = Not Given by manufacturer property data sheet.

The procedure used to construct the test setups included placing a 20 cm (8 in) diameter corrugated metal pipe (CMP) at 0.3 meters off the bottom of the concrete floor. The pipe met ASTM A444 and AASHTO M-218 specifications. Figure 12 illustrates the pipe orientation and location in the testing chamber, as well as heights of the base and fill materials. The base was prepared by compacting with a jumping jack whacker and a pogo stick type compactor. The clay was placed in approximate 10-12 cm compacted lifts at or near optimum moisture content and maximum dry density. To maintain consistency between tests, moisture contents and densities of the compacted backfill were monitored. To accomplish this, in place density tests were performed using the sand cone method.

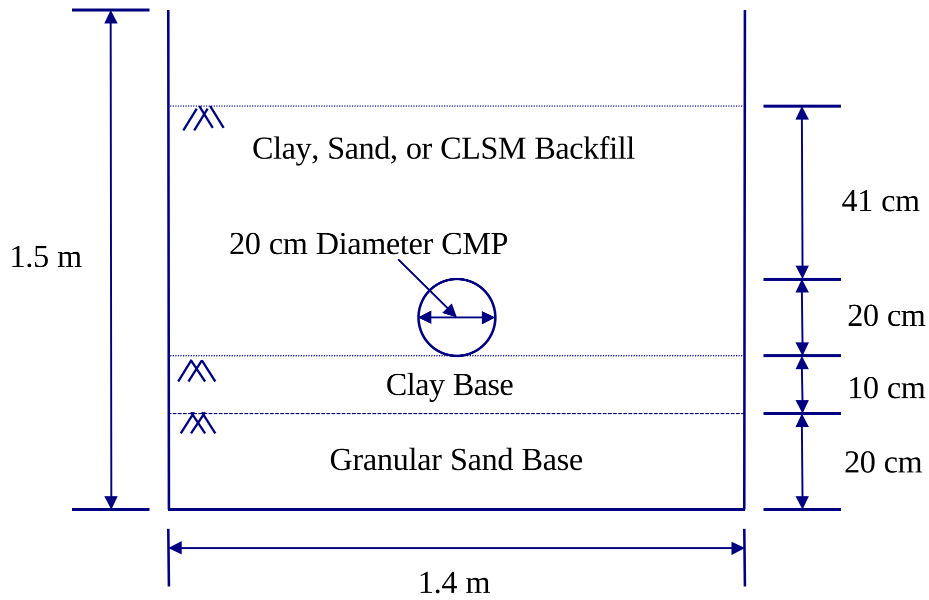

Figure 12. Culvert Orientation and Fill Heights.

The sand cone test followed ASTM D 1556 "Standard Test Method for Density of Soil In Place by the Sand- Cone Method". Additionally, upon completion of each test, bore holes were made into the culvert backfill cover to obtain final water contents. Table 16 summarizes the values obtained for initial and final water contents as well as dry densities. After each test was complete, the embankment material was removed down to the base level. Once the soil was excavated, it was then placed back in the testing chamber and recompacted in the 10 cm lifts for each new test.

Table 16. Compacted Densities and Water Contents

Test	Average γ_{dmax} (kN/m^3)	Average Initial water content	Average Final water content	% γ_{dmax}	% w_{opt} Initial	% w_{opt} Final
Under Culvert	748.6	16.6	N/A	107.2	90.2	N/A
2	720.0	18.8	19.6	103.1	107.1	106.5
3	743.5	19.1	18.1	106.5	102.2	98.4
4	706.0	3.1	18.3	101.1	103.8	99.5
5	707.9	18.6	2.8	N/A	N/A	
6	730.1	19.2	18.5	104.6	101.1	100.5
7	697.1	19.7	18.9	99.8	104.3	102.7

For the geogrid reinforced embankments, construction was identical with the exception of placing the geogrid reinforcing on a compacted lift. For both of the tests conducted using the geogrid reinforcing, the geogrid was located approximately 10.2 cm from the surface and 10.2 cm from the top of the culvert. For the CLSM test the embankment was constructed as a non-reinforced embankment. After completion of the embankment, a trench was excavated down to mid-height of the culvert. Figure 13 illustrates the configuration of the excavated trench. This trench was then backfilled using the CLSM material. The embankment was constructed in this manner to simulate field conditions.

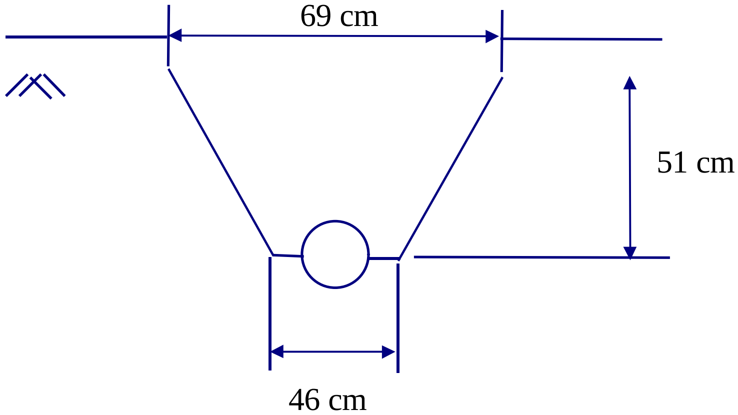

Figure 13. Excavated Trench for CLSM Backfill.

A 245 kN MTS computer-controlled actuator and two 2.5 cm thick aluminum plates were used to apply and distribute a load to the soil surface. AASHTO LRFD Design Specifications required a plate with dimensions 21.2 cm by 50.8 cm in width to model a tire patch of a truck with a constant 862 kN/m^2 pressure. After further consideration, a larger plate with dimensions 36.4 cm by 66.0 cm was constructed. The larger plate and reduced loading were used to model an embankment with asphalt covering. The asphalt would distribute the load over the surface of the culvert backfill cover, thus reducing the vertical stress applied to the subgrade. Additionally, a bearing capacity failure would likely have occurred in the clay soil if the bearing stress had not been reduced. All roads that were investigated and considered in this study were paved roads. Figure 14 illustrates the plate setup.

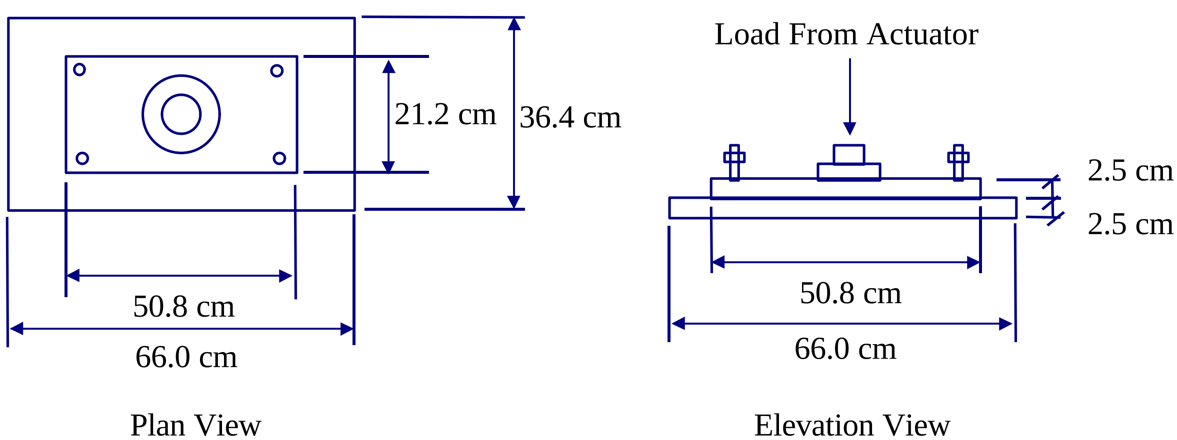

Figure 14. Illustration of Load Plates and Connections

A cyclic load of approximately 16.6 kN (3.73 kips) was applied by the actuator for a maximum compressive load which corresponds to approximately 68.9 kN/m^2 (10 psi) over the area of the larger plate. Additionally, the plates were kept in compression during the entire test by applying a minimum compressive load equal to 10 percent or less of the maximum load. The load was applied cyclically with a frequency of .204 Hz. All tests were conducted initially with at least 20,000 cycles applied using the maximum compressive load, further testing was conducted using increasing cyclic and static loads on these test setups.

To measure the vertical deformation of the plate and soil a vertical LVDT on the MTS actuator was monitored using a computer equipped with Quicklog PC data acquisition system manufactured by Strawberry Tree Incorporated. Additionally, an external LVDT was placed on top of the larger plate. The data acquisition system read the voltage output from the actuator for load and stroke (deformation) and the output voltage from the external LVDT. To analyze the data recorded by the data acquisition a Matlab program was written to convert output voltages to loads and displacements. This code and typical output generated by the Matlab program are presented as Appendix B in the thesis by Lundvall (1997).

To monitor vertical deformation of the soil surface adjacent to the plates, dial gauges were placed around the testing chamber for this purpose. Figure 15 is a photograph of the testing setup.

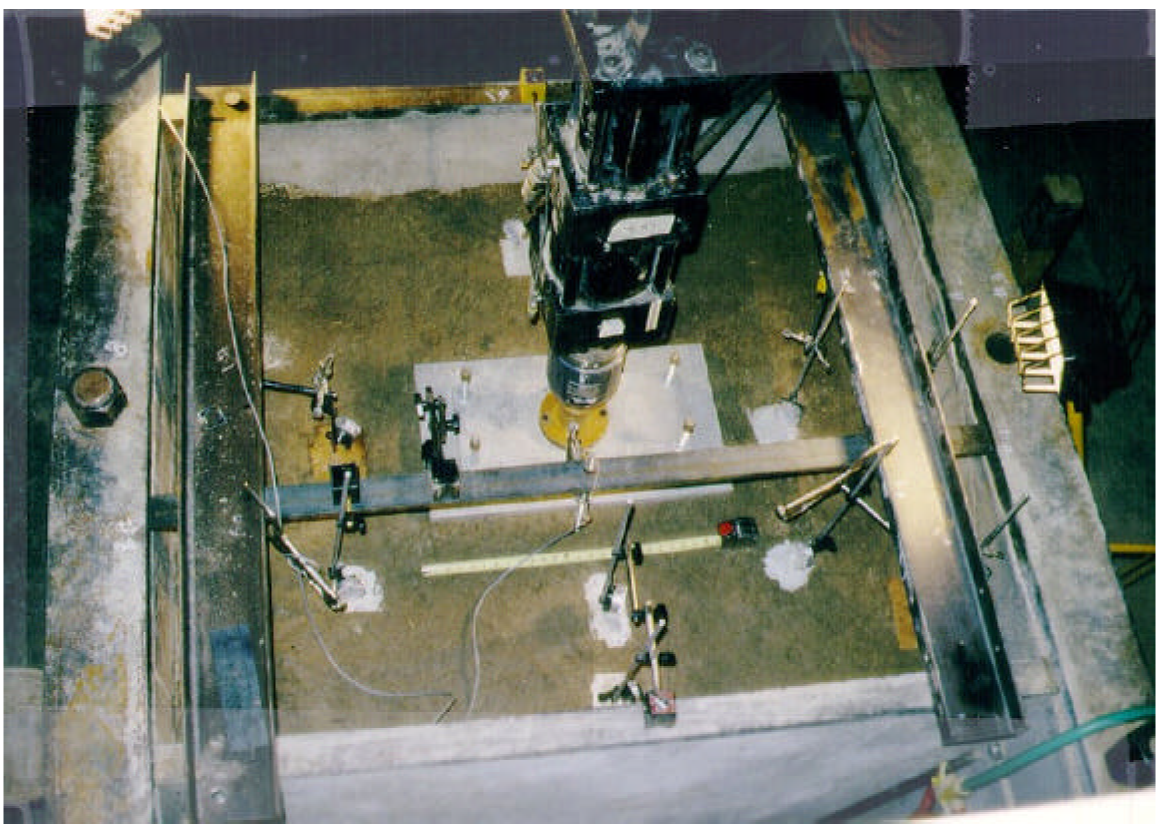

Figure 15. Concrete Testing Chamber Setup.

5: RESULTS OF LABORATORY TESTING PROGRAM

This chapter presents the results of the laboratory testing program. Two types of tests were conducted: 1) bench-scale tests to characterize the index and engineering properties of soil samples obtained from the field investigations and 2) load tests on laboratory scale models of backfilled culverts.

SOIL TESTING

Soil Classification and Index Properties

For classification purposes, Atterberg limits and grain size distribution tests were conducted. With the properties obtained from these tests, the soil samples were classified according to AASHTO specifications. Results are summarized in Table 17. Additional information regarding the specific gravity of the soils is also included. For information on the clay and sand backfills used for the model testing, see Chapter 4 *Laboratory Testing Program*. Further details pertaining to soil characteristics, including grain size distribution curves, are included in Appendix C of the thesis by Lundvall (1997).

As seen in Table 17, the majority of the soil classifications fell within the A-6 to A-7-6 categories of the AASHTO soil classification system. Spangler (1951) describes the group A-6 as follows:

> "The typical material of this group is a plastic clay soil 75 percent or more of which usually passes the No. 200 sieve. The group includes also mixtures of fine clayey soil and up to 64 percent of sand and gravel retained on the No. 200 sieve. Materials of this group usually have high volume change between wet and dry states."

McCarthy (1993) gives approximate equivalent groups using the Unified Soil Classification system. The A-6 group from the AASHTO classification has the approximate classification of CL in the Unified Classification. A brief description of terms used in the Unified Soil Classification system by Terzaghi and Peck (1967) describes a "C" as an inorganic clay. The modifier "L" means the material has a liquid limit lower than 50 percent. With this classification, typical materials are inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, and lean clays (McCarthy 1993).

Table 17. Classification and Index Properties of Field Samples.

Sample	AASHTO Classification	Unified Soil Classification	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Specific Gravity (G_s)
BE[1] 3.14 #1	A-6	CL	27.8	15.7	12.1	2.67
BE 3.25 #3	A-7-6	CL	46.0	19.9	20.7	2.74
BE 3.25 #7	A-6	CL	28.6	16.7	11.9	2.69
BE 3.65 #4	A-7-6	CL	46.8	20.1	26.7	2.72
BE 3.65 #8	A-7-6	CL	42.0	22.0	20.0	2.71
BE 4.36 #2	A-6	CL	27.5	15.7	11.8	2.75
BE 4.36 #8	A-6	CL	37.6	18.9	18.7	2.70
SB[1] 47.97 #4	A-4	ML	N.P.[2]	N.P.	N.P.	2.65
SB 47.97 #7	A-4	ML	N.P.	N.P.	N.P.	2.70
SB 50.24 #1	A-6	CL	36.3	19.4	16.9	2.66
SB 50.24 #4	A-7-6	CL	46.7	21.6	25.1	2.69
SB 50.47 #3	A-6	CL	35.5	19.7	15.8	2.69
SB 51.18 #4	A-7-6	CL	41.8	21.0	20.8	2.73
SB 51.18 #10	A-7-6	CL	49.1	22.7	26.4	2.65
SB 51.54 #2	A-6	CL	32.4	17.9	14.5	2.65
SB 51.54 #8	A-6	CL	31.6	16.9	14.7	2.77
TM[1] 3.4[3] #3	A-7-6	CL	51.2	25.8	25.4	2.75
TM 3.4[3] #8	A-7-6	CL	42.4	21.9	20.5	2.65
TM 4.2 #3	A-6	CL	38.8	19.0	19.8	2.65
TM 4.2 #12	A-6-5	CH	132.6	44.8	87.8	2.78
TM 4.98 #4	A-2-7	SC	46.1	29.6	16.5	2.71
TM 26.6 #4	A-4	ML	N.P.	N.P.	N.P.	2.68
TW[1] 152.3 #2	A-4	CL-ML	23.6	16.7	6.9	2.75
TW 152.3 #7	A-6	CL	31.8	18.7	13.1	2.73
TW 152.5 #4	A-4	CL-ML	23.9	17.9	6.0	2.71

1. BE: Baggs-Encampment, SB: Shirley Basin, TM: Thermopolis-Meeteetse, TW: Thermopolis-Worland.
2. Non-Plastic
3. 4.21 BK = 3.21 AH.

The other group in which many of the tested soils fell was A-7-6. Spangler (1951) describes the A-7 group as follows:

> "The typical material of this group is similar to that described under group A-6, but it has the high liquid limits of the A-5 group and may be elastic as well as subject to high volume change."

Spangler (1951) goes on to describe the A-7-6 sub-group:

> "Sub-group A-7-6 includes those materials which have high plasticity indexes in relation to liquid limit and which are subject to extremely high volume change."

The equivalent Unified Soil Classification for the A-7-6 group is CH or OH (McCarthy 1993). The "C" indicates clay, while "O" describes organic silts or clays. The modifier "H" further indicates the soil has a liquid limit greater than 50 percent. Typical materials found within these classifications include: CH - inorganic clays of high plasticity, fat clays, OH - organic clays of medium to high plasticity (McCarthy 1993).

The categories described above are for soils which are rated "fair to poor" as subgrade materials. Although the focus of this research is not on subgrades, in general, the materials evaluated as best for subgrades will also form the best embankments with a minimum of construction and maintenance difficulties (Spangler, 1951). The silt and clay materials in general would make poor soils for backfilling a culvert structure. This is due to the high degree of compressibility that may be exhibited even when compacted to 95 percent of maximum dry density.

Collapse Index

Additional testing that was performed on the soil samples obtained from the drilling program (see Chapter 3 *Field Investigation of Roadway Settlement*) were collapse index tests as described in Chapter 4 *Laboratory Testing Program*. Figure 16 is a plot of the void ratio versus applied vertical stress. The void ratio experiences a sudden decrease, or collapse, when the specimen is wetted at the applied vertical stress of 200 kN/m^2. This illustrates typical results obtained from the collapse index tests. However, some of the collapse index tests had the opposite result, in which the void ratio experienced a sudden increase, or swelling, when wetted. A complete set of plots showing results of all collapse index tests is given as Appendix D in the thesis by Lundvall (1997).

ASTM D 5333 "Standard Test Method for Measurement of Collapse Potential of Soils" classifies the collapse index, I_e, in the following categories: none, slight, moderate, moderately severe, and severe. The "none" category denotes a collapse index of zero percent. "Slight" indicates a collapse index of 0.1 to 2.0 percent. The "moderate" category corresponds to a collapse index of 2.1 to 6.0 percent. A specimen with a degree of collapse of "moderately severe" has a collapse index of 6.1 to 10 percent, and the final category of "severe" denotes a collapse index of over 10 percent.

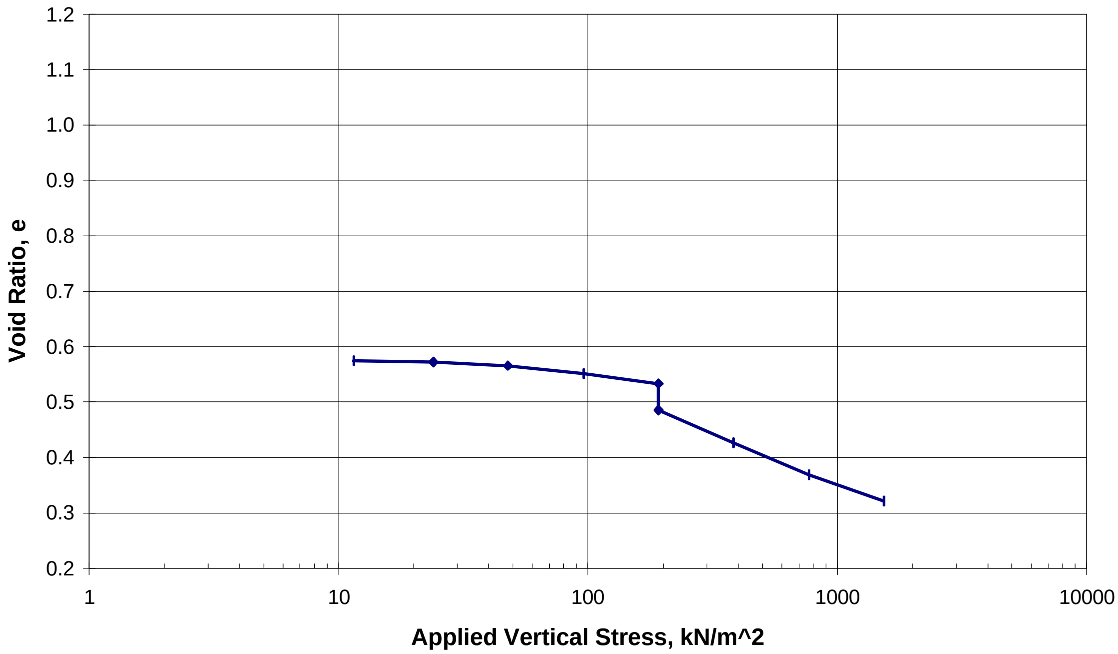

Figure 16. Typical Results From Collapse Index Tests

During this study, not all of the specimens exhibited collapse. When inundated with water, several specimens swelled. As noted by Lawton et al. (1992), swelling can predominate over collapse even at overburden pressures as great as 1,600 kN/m^2 for soils with high expansive clay contents. For these specimens, the categories for the classification of the collapse index (or swell index) were simply reversed to indicate the degree of swelling. Table 18 gives a summary of the specimens tested, the percent of collapse or swelling, and the classification of the specimens according to the categories given.

As indicated in Table 18, the specimens all had some degree of collapse or swelling. The specimens tested from the Baggs-Encampment area fell in the slight to moderate categories; the Shirley Basin area fell in the slight to moderately severe categories; the Thermopolis-Meeteetse area fell in the moderate category; and the Thermopolis-Worland area in the moderate category. The most frequent category of collapse was moderate. Several sites exhibited both swelling and collapsing soils in the same boring. This could indicate an area where problems might be likely.

Table 18. Summary of Collapse Index Tests and Results.

Samples	Collapse Index, I_e, %	Degree of Collapse
BE[1] 3.14 #1	0.84	Slight
BE 3.25 #3	-1.53[2]	Slight
BE 3.25 #7	0.73	Slight
BE 3.65 #4	-0.23	Slight
BE 3.65 #8	-2.39	Moderate
BE 4.36 #2	2.28	Moderate
BE 4.36 #8	S.U.[3]	S.U.
SB[1] 47.97 #4	3.04	Moderate
SB 47.97 #7	3.73	Moderate
SB 50.24 #1	3.04	Moderate
SB 50.24 #4	5.42	Moderate
SB 50.47 #3	0.81	Slight
SB 51.18 #4	1.92	Slight
SB 51.18 #10	-2.69	Moderate
SB 51.54 #2	-0.87	Moderate
SB 51.54 #8	9.32	Moderately Severe
TM[1] 3.4* #3	-2.19	Moderate
TM 3.4* #8	-1.24	Moderate
TM 4.2 #3	1.77	Moderate
TM 4.2 #12	-4.54	Moderate
TM 4.98 #4	S.U.	S.U.
TM 26.6 #4	S.U.	S.U.
TW[1] 152.3 #2	1.80	Moderate
TW 152.3 #7	1.27	Moderate
TW 152.5 #4	1.67	Moderate

1. BE: Baggs-Encampment, SB: Shirley Basin, TM: Thermopolis-Meeteetse, TW: Thermopolis-Worland.
2. Negative indicates swelling.
3. S.U. = Specimen Unable to be tested.

As described by ASTM D 5333, the collapse index is used to characterize a basic index property of soil. The collapse index procedure is used to describe the degree of collapse that a particular soil will exhibit under specified conditions. By using the consistent test conditions, the results obtained from different specimens can be compared directly. The collapse index procedure is not intended to duplicate any particular field conditions such as the actual loading, in-place soil structure, or soil water chemistry.

Another property that may be used to characterize collapse settlement is the collapse potential, I_c. By using "ideal" or constant conditions such as distilled water and a predetermined loading of 200 kN/m^2, a value of collapse potential may be found. These conditions are not necessarily representative of actual field conditions. However, using these conditions, the collapse potential is equivalent to the collapse index. Therefore, for the given fluid and loading, an estimate of the amount of collapse for any cover height can be estimated. The settlement is estimated by multiplying the collapse potential by the thickness of the soil layer divided by one hundred. For example, a soil layer with a thickness of 91 cm and a collapse potential of 3.0 would have an estimated settlement of 2.7 cm.

MODEL TESTING

Model tests were conducted in the testing chamber and setup as described in Chapter 4 *Laboratory Testing Program*. The model tests were conducted to evaluate possible methods to mitigate or minimize roadway settlement.

Discussion of Model Test Results

Three different backfill materials were used, including: (1) clay soil obtained from a Shirley Basin culvert site (two tests on unreinforced backfill and two tests with geogrid reinforcement), (2) compacted sand, and (3) CLSM. Results of the cyclic load tests are presented graphically by two figures for each test. These include one figure showing the vertical deformations (minimum and maximum) versus the log of the number of loading cycles and a second figure showing the load versus deformation curves for selected loading cycles. These graphs are shown as Figures 5.2 through 5.13 at the end of this chapter. In addition, three of the models were subjected to static load tests in order to compare their load-deformation behaviors at larger load magnitudes. In each case, the static loading was conducted after the cyclic load test was completed.

Tests No. 2, 3, 4, and 6 were conducted on clay backfills. As can be observed in Figures 5.2, 5.4, 5.6, and 5.10, both permanent and maximum vertical deformations increased with increasing number of load cycles. This behavior was observed for both unreinforced (Tests 2 and 3) and geogrid-reinforced (Tests 4 and 6) clay backfills. In fact, the geogrid reinforced backfills both exhibited larger maximum deformations than the two unreinforced backfills. The maximum deformation of the reinforced clay was greater beginning with the first load cycle, possibly indicating a lower overall stiffness of these deposits. However, the magnitudes of the maximum and permanent deformations are relatively small in every case (less than 5 mm), so the differences observed may not be significant and may simply represent the inherent variability of the compacted clay. In addition, upon further loading beyond the initial cycles, the load deformation curves were essentially similar for the unreinforced and reinforced tests. The conclusion is that the geogrid reinforcement apparently had little or no effect on the load-deformation relationship of the clay backfill, at least for the magnitude of load used in these tests.

A noticeable aspect of the tests with clay backfills is that the slopes of the deformation versus log number of cycles curves appear to increase when the number of load cycles is between 1,000 and approximately 10,000. This increase in slope of the maximum deformation suggests that the response would eventually become unstable and that large deformations leading to failure would occur. This behavior, which is particularly evident in Test 3 (Figure 5.4), is consistent with field observations of large settlements over culverts backfilled with this same clay soil on WYO 487 in Shirley Basin.

Results of Test 5 on sand backfill are shown in Figures 5.8 and 5.9. Both the permanent and maximum displacements appear to exhibit a constant or slightly decreasing slope up to the maximum number of load cycles (20,000). Overall, the displacements are on the same order as those of the tests on clay backfills, except that the elastic deformations (difference between maximum and permanent displacement) are smaller and appear to be decreasing with increasing load cycles, and there is no increase in slope, suggesting that the cyclic deformations would eventually reach a steady state. Table 19 summarizes the deformations observed during all of the cyclic load tests.

Test 7 was conducted with a CLSM backfill. The results, shown in Figures 5.12 and 5.13, show a stiffer response than either the clay or sand backfills. In Figure 5.12, the maximum displacement reaches a constant value of approximately 2 mm at 1,000 cycles and remains constant up to 22,500 cycles. The permanent deformation increases slightly beyond 1,000 cycles. This test indicates that CLSM offers significant resistance to settlement under low-level cyclic loading, compared to soil backfills.

Table 19. Deformation Data for Cyclic Model Tests.

Test	Minimum Cycle 1 (mm)	Maximum Cycle 1 (mm)	Minimum Cycle 20,000 (mm)	Maximum Cycle 20,000 (mm)	Δ_{max}-Δ_{min} Cycle 1 (mm)	Δ_{max}-Δ_{min} Cycle 20,000 (mm)
2	0	1.99	1.74	2.84	1.99	1.10
3	0	1.60	2.10	3.38	1.60	1.28
4	0	2.50	2.76	3.84	2.50	1.08
5	0	1.30	3.20	4.03	1.30	0.83
6	0	3.10	2.59	4.42	3.10	1.83
7	0	1.74	1.08	2.21	1.74	1.13

Note: Tests 2 and 3: Unreinforced Clay, Test 4: Tensar BX1100 geogrid, Test 5: Sand, Test 6: Tensar BX1300 (SS-3) geogrid, Test 7: CLSM. The cyclic load was approximately 16.6 kN. Δ_{min} is defined as the vertical deformation at the minimum applied load (approximately 0.75 kN). Δ_{max} is defined as the vertical deformation at the maximum applied load (approximately 17.0 kN).

The results of the cyclic load tests indicate that the magnitude of cyclic loading was not sufficient to differentiate the behavior of the various backfill materials at deformations similar to those which would cause moderate to severe damage to roadways. For the final three tests (Tests 5, 6, and 7) the investigators therefore chose to apply a gradually increasing static load to the plate, following application of the cyclic loads for approximately 20,000 cycles. Results of the static load tests are shown in Figure 5.14 and measured deformations are summarized in Table 20. The principal observation from Figure 5.14 is that both the sand and CLSM backfills show significantly higher stiffness and strength than the clay backfill. The clay backfill is reinforced with geogrid, however it is difficult to determine whether the geogrid influenced the load deformation behavior since no static load test was conducted on the unreinforced clay backfill. It is clear from these tests that a backfill constructed from either well-compacted granular soil or from CLSM would be expected to provide a much better subgrade for minimizing roadway settlement than a compacted, plastic clay.

Table 20. Load and Deformation Data for Static Loading.

Test	Axial Load Range (kN)	Deformation Range (mm)	Deformation at 100 kN (mm)
5	15.95-152.45	3.99-15.02	8.33
6	16.15-107.70	4.43-40.84	32.25
7	16.15-170.57	2.22-16.31	6.67

Note: Test 5: Sand, Test 6: Tensar BX1300 (SS-3) geogrid, Test 7: CLSM.

SUMMARY

In summary, most of the sites investigated had soils that fell within two AASHTO soil classification groups, A-6 and A-7-6. These soils contain a high quantity of fine silt and clay material. In most cases, these materials make poor backfill due to the high degree of compressibility that may be exhibited. Additionally, the collapse index test results indicated that the soils used, for the most part, had a moderate degree of collapse or swelling that occurred when wetted.

Model testing allowed for the comparison of plastic compressible clay as backfill to alternatives including reinforced clay, granular, and CLSM backfills. Results of the model testing verified that fine compressible materials should not be used as backfill around structures. Additionally, the results indicate that the geogrid reinforcing had no significant effect on the deformation of the clay backfill for the applied loading. The results do indicate that the use of a well compacted granular soil or CLSM would provide better backfill material or subgrade in minimizing roadway settlement over culverts.

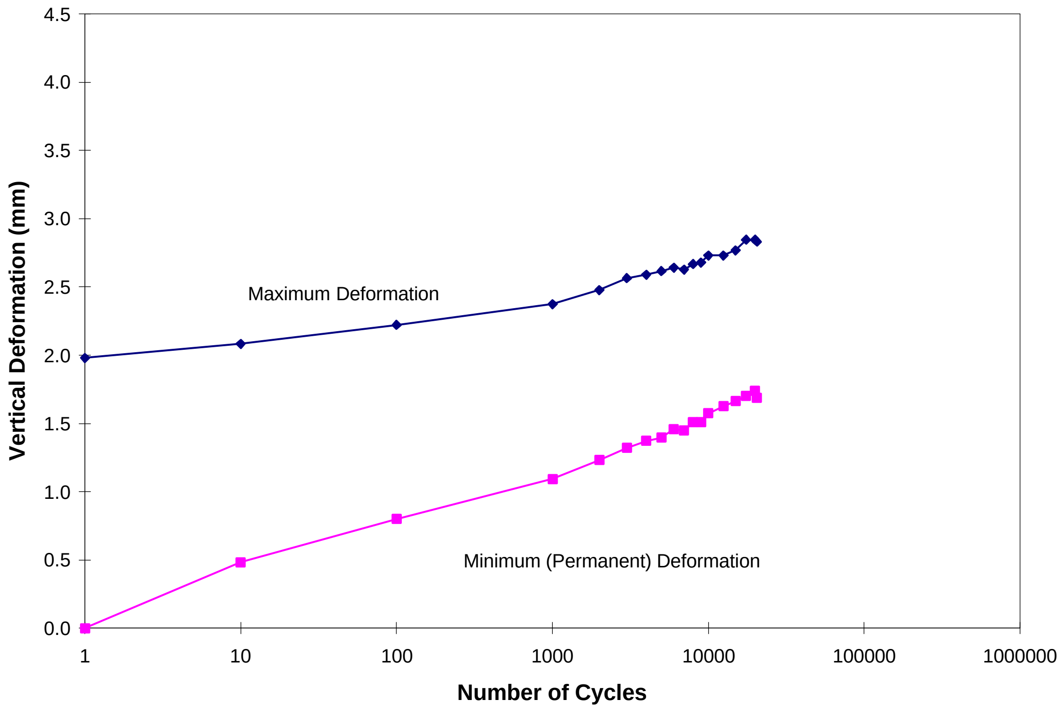

Figure 17. Test #2-Unreinforced Clay Deformation vs. Number of Cycles

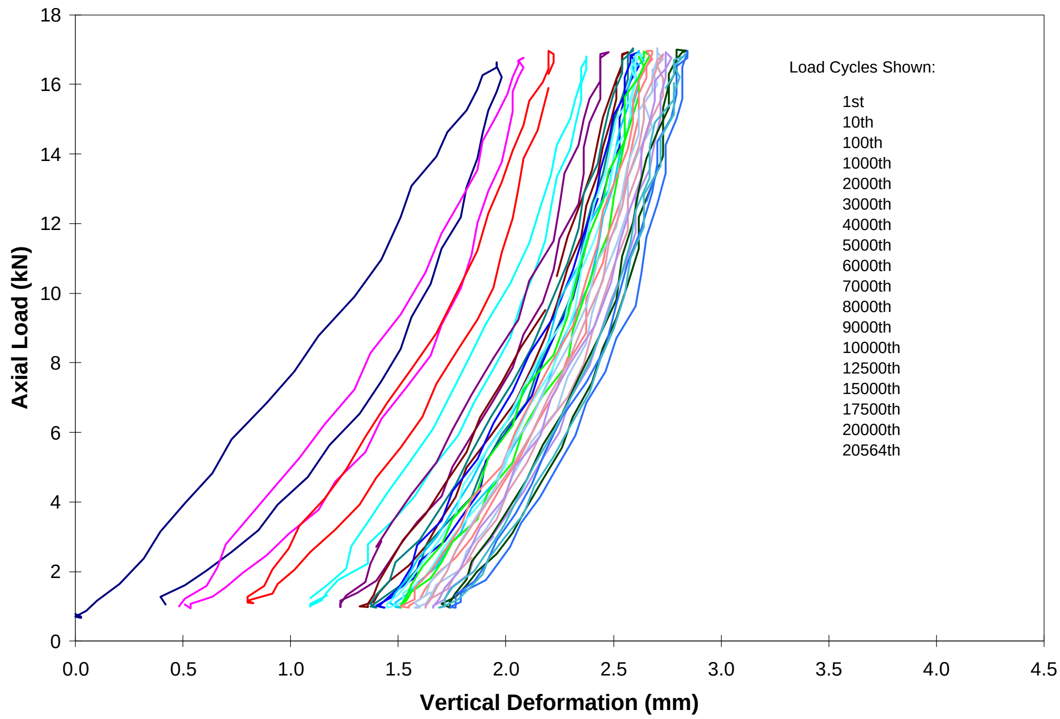

Figure 18. Test #2 Unreinforced Clay Load vs. Deformation for Increasing Cycles

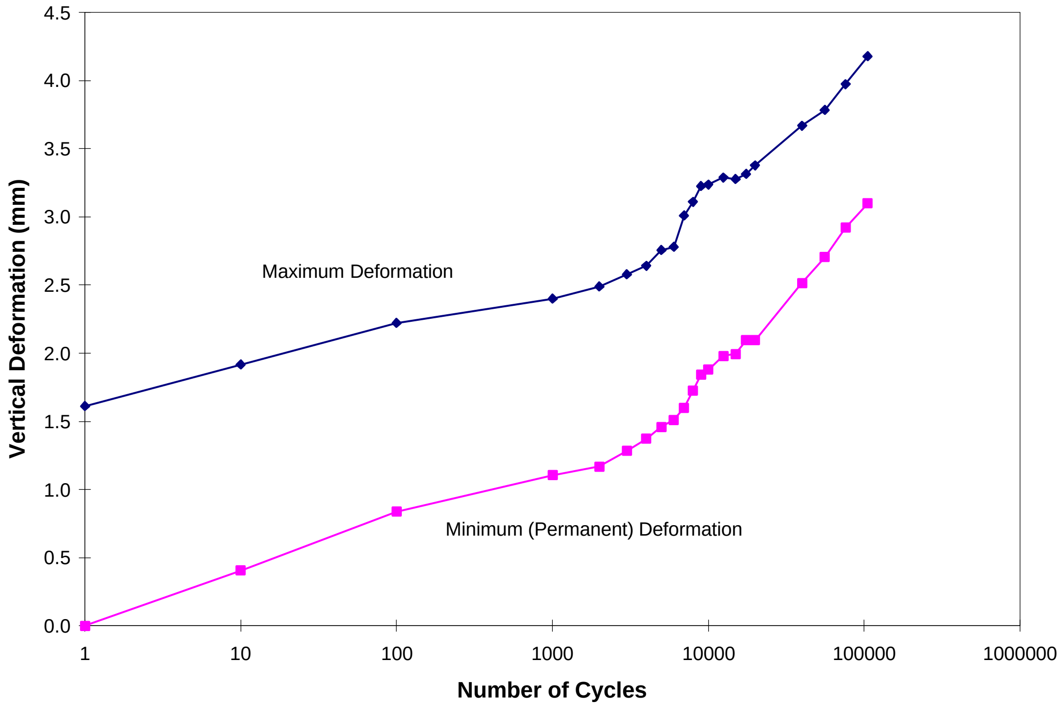

Figure 19. Test #3 Unreinforced Clay Deformation vs. Number of Cycles

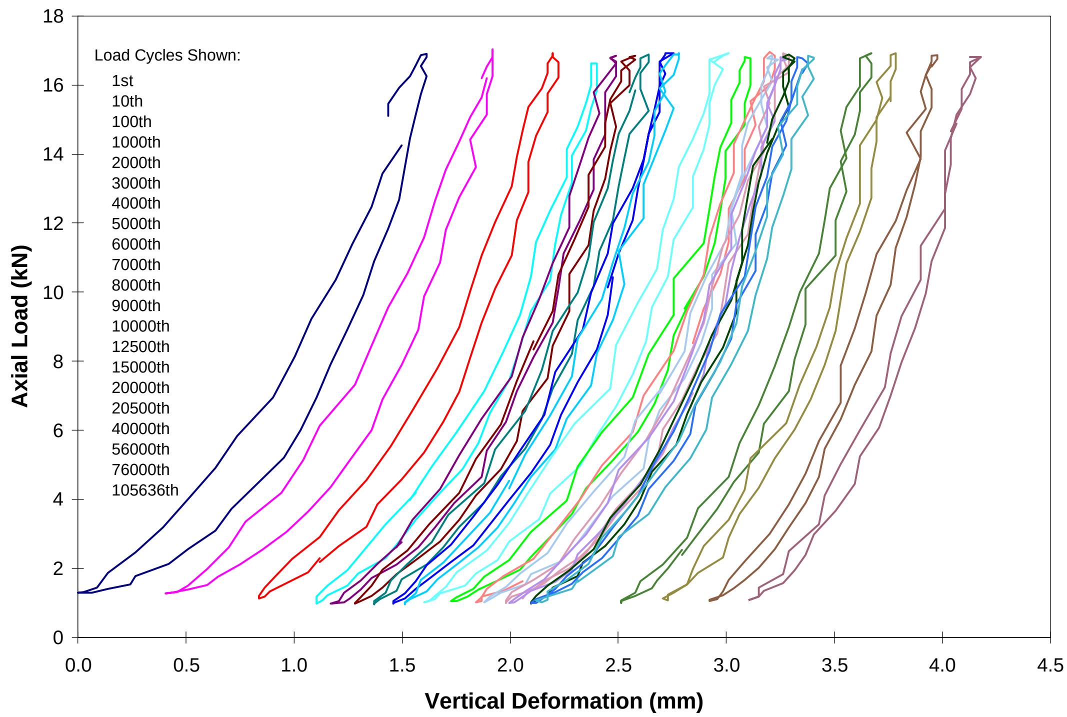

Figure 20. Test #3 Unreinforced Clay Load vs. Deformation for Increasing Cycles

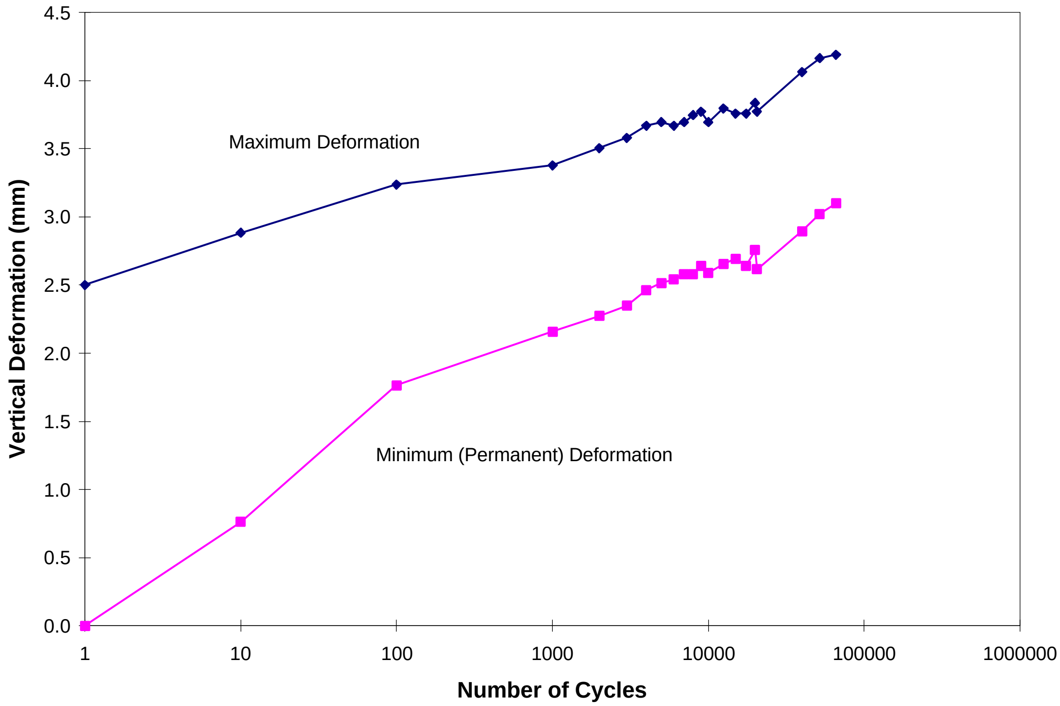

Figure 21. Test #4 Tensar BX1100 Geogrid Deformation vs. Number of Cycles

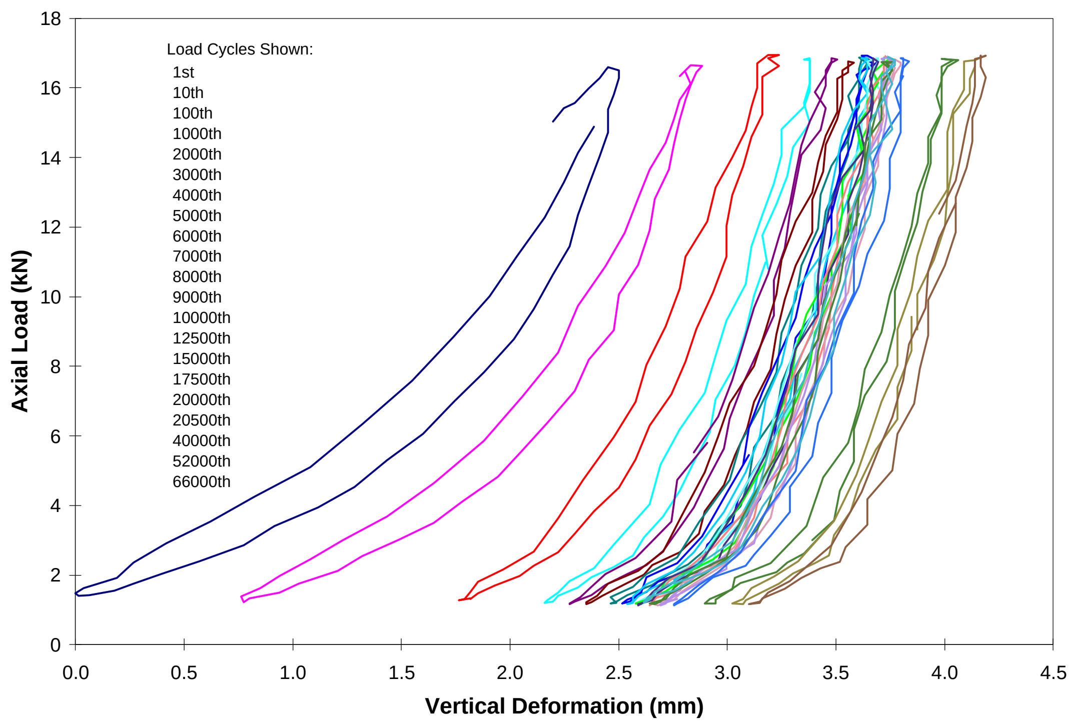

Figure 22. Test #4 Tensar BX1100 Geogrid Load vs. Deformation for Increasing Cycles

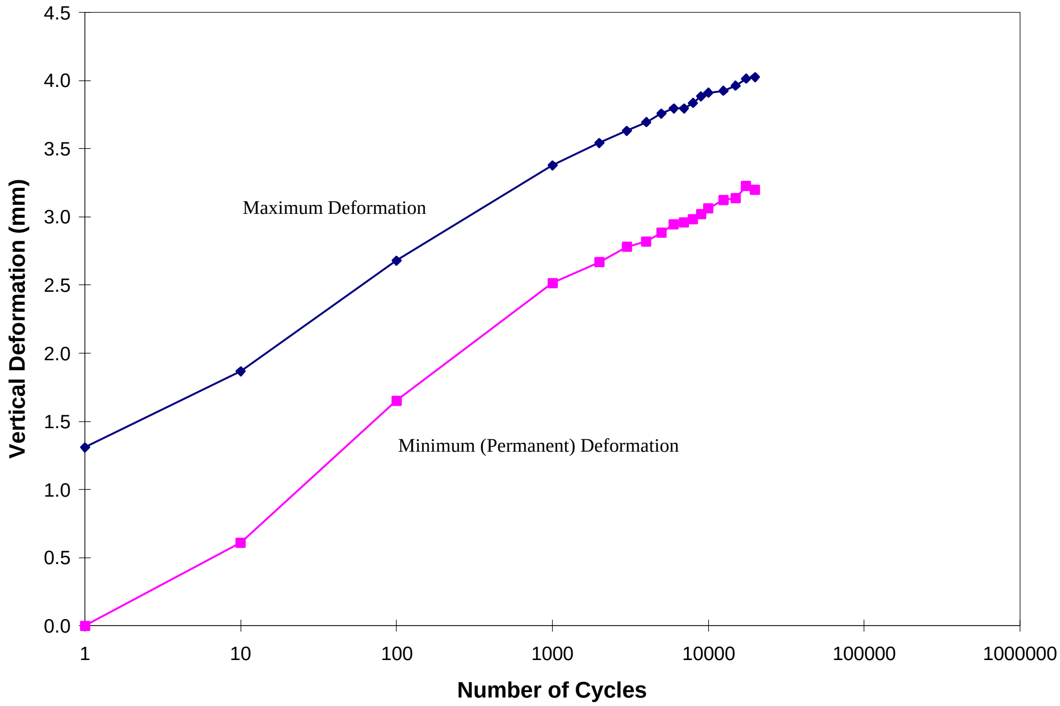

Figure 23. Test #5 Unreinforced Sand Deformation vs. Number of Cycles

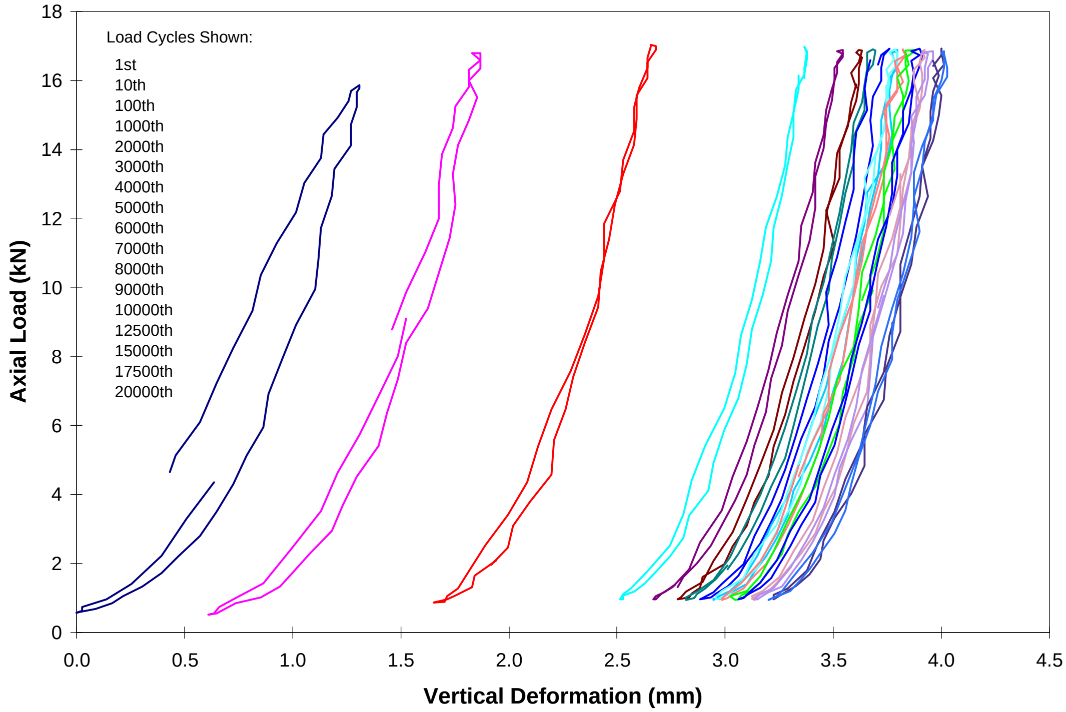

Figure 24. Test #5 Unreinforced Sand Load vs. Deformation for Increasing Cycles

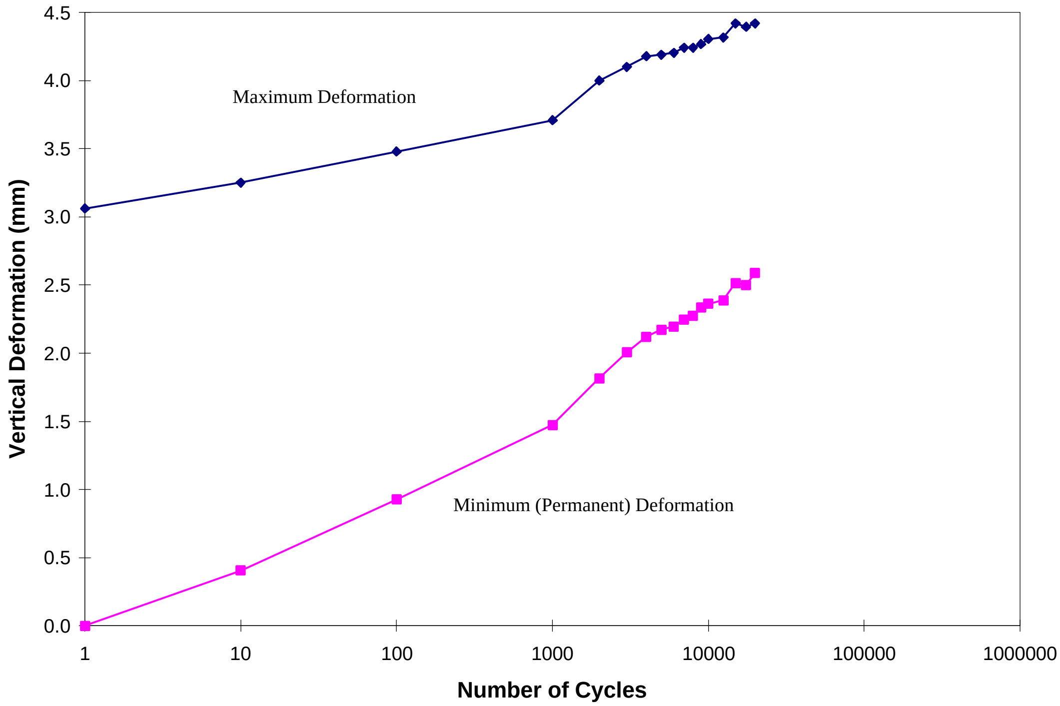

Figure 25. Test #6 Tensar BX1300 (SS-3) Geogrid Deformation vs. Number of Cycles

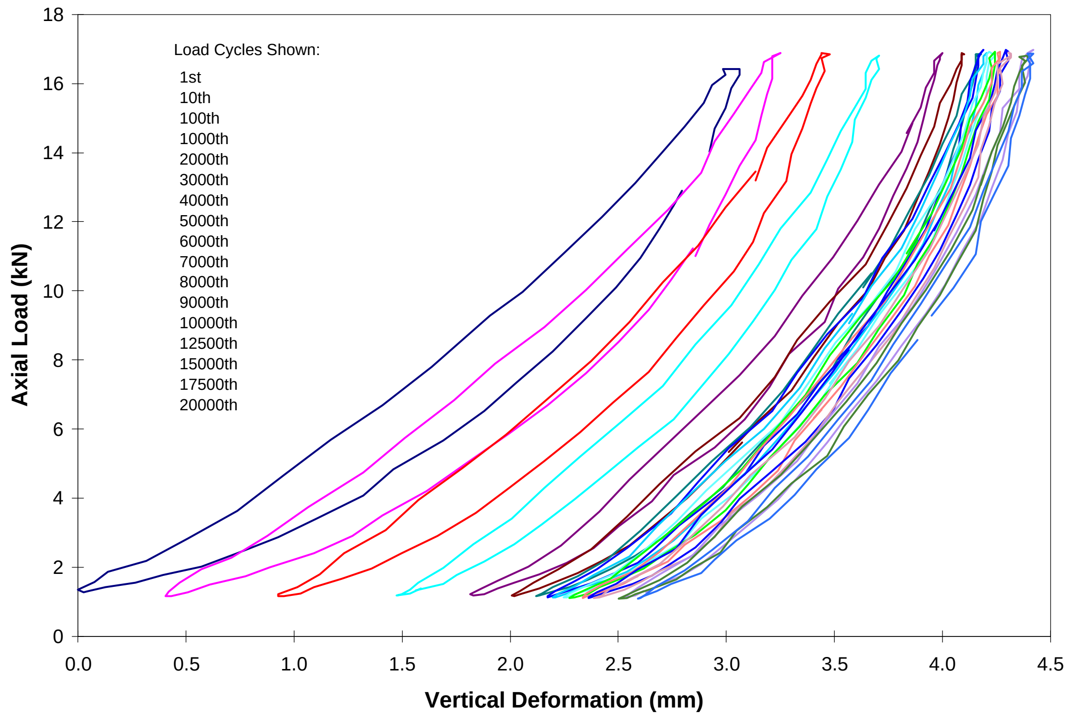

Figure 26. Test #6 Tensar BX1300 (SS-3) Geogrid Load vs. Deformation for Increasing Cycles

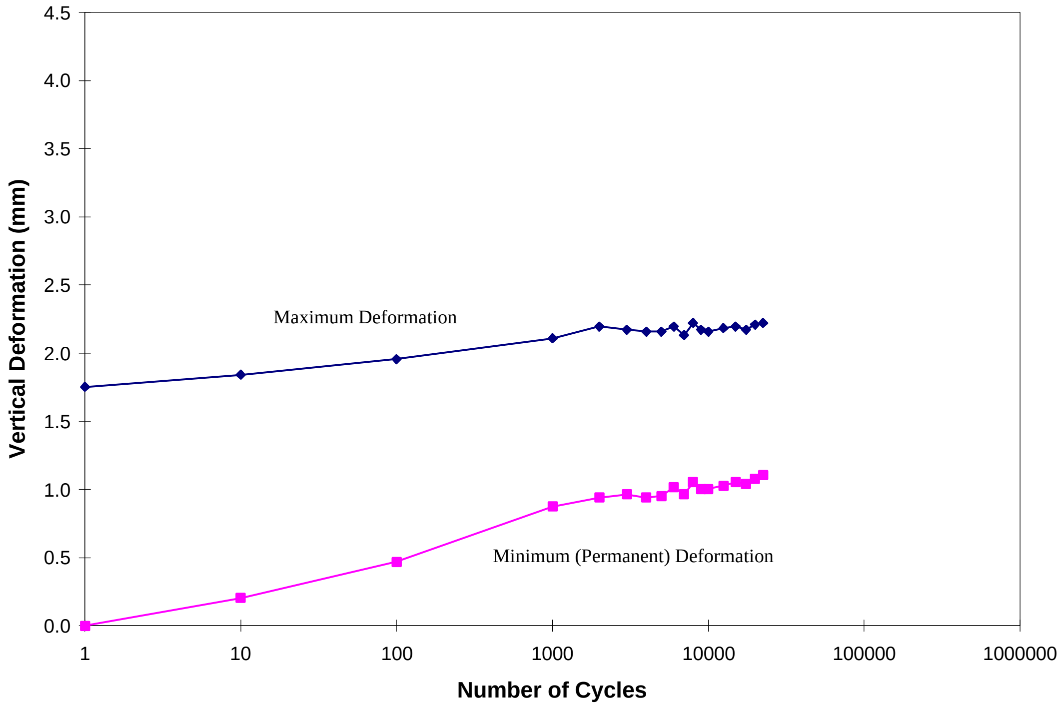

Figure 27. Test #7 CLSM Deformation vs. Number of Cycles

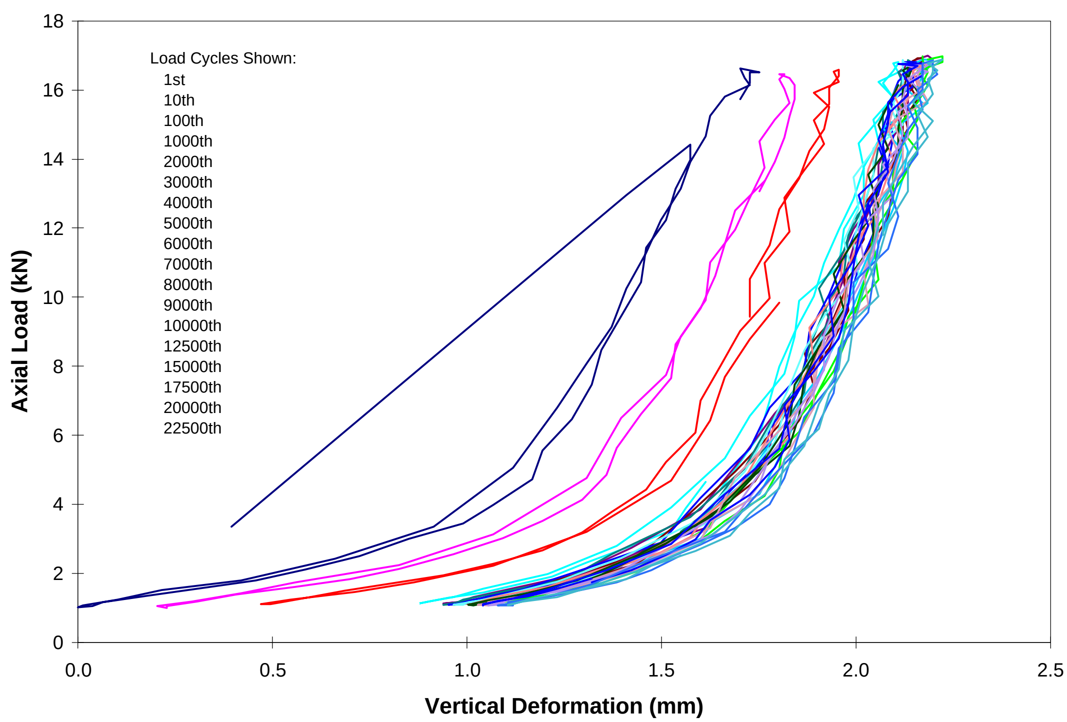

Figure 28. Test #7 CLSM Load vs. Deformation for Increasing Cycles

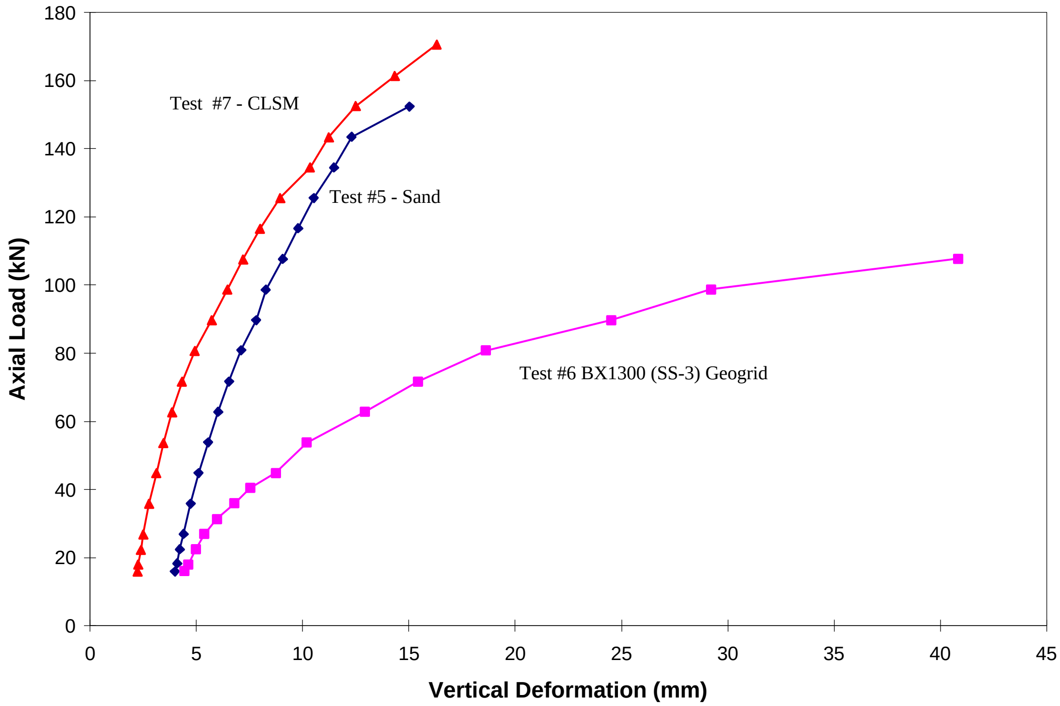

Figure 29. Combination Plot of Load vs. Deformation for Static Loading

6: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

The goal of this research is to develop recommendations for design and construction procedures to eliminate or minimize damaging roadway settlement above culvert crossings. In this study, three research objectives were addressed; these include the following:

1. Determine if roadway settlement appears to be related to culvert or pipe characteristics, geological environment, construction practices, or other factors.
2. Establish the mechanism(s) of settlement, for example, compression settlement of fill around and above culverts and pipes under traffic loads, settlement of soil beneath culverts and pipe, deformation of the culverts and pipes, or some combination of causes.
3. Evaluate design and/or construction procedures to prevent or minimize settlement for construction.

SUMMARY

Chapter 2 presents a review of the literature on possible causes of settlement in compacted backfills and a review of potential methods to improve the settlement behavior of materials used as backfill above culverts. The literature review showed that the compressibility of compacted soils is a complex topic and that field behavior is a function of many variables, including soil composition, moisture content during compaction and changes in moisture conditions during service, lift thickness, method of compaction, total fill thickness, field quality control, and others. A potentially detrimental factor for backfill over culverts beneath roadways is the presence of soils which are prone to large volume changes with changes in moisture content. Two behaviors attributed to volume change are collapse and swell. Collapsing soils undergo large volume decreases when subjected to increasing moisture content under a constant load. Fine uniform sands, wind-blown deposits, and soils containing soluble salts (e.g., gypsum) were identified as potentially collapsible. All of these soil types are found in Wyoming. Swelling soils increase in volume with increasing moisture content and decrease in volume with decreasing moisture content. Soils which contain bentonite and/or have a high plasticity index are likely to exhibit swelling behavior. Numerous locations in Wyoming are underlain by bentonitic soils and rock.

The literature review revealed that, in addition to compaction, ground modification techniques which appear to have the potential for minimizing roadway settlement include soil reinforcement and the use of controlled low strength materials (CLSM). Soil reinforcement, consisting of geosynthetic materials such as geogrid, were shown in several studies to increase the stiffness and strength of compacted soils. CLSM, consisting of sand, cement, fly ash, and water, appears to have numerous advantages from a construction viewpoint as well as the ability to limit settlement. A critical aspect of CLSM is mix design, which should be established on the basis of performance characteristics such as flowability, rate of hardening, ultimate strength, and ease of excavation.

Chapter 3 describes a field investigation to establish the most likely causes of damaging settlement at culvert sites on Wyoming highways (Objectives 1 and 2). Numerous culvert sites on twelve WYDOT road construction projects were investigated. Items which were inspected and documented included culvert type, size, and location, evidence of ground movement, level of damage to the roadway and to culverts, characterization of subgrade and backfill soils, and others, as described in Chapter 3. In addition, detailed site investigations were conducted at fifteen culvert sites selected from the twelve projects included in the field investigation. These investigations involved drilling, sampling, and testing of the backfill and subgrade soils. Laboratory tests included index tests, soil classification, and swell/collapse tests. From these investigations, three factors were identified as having a high probability of resulting in damaging roadway settlement: (1) inadequate compaction, (2) shallow cover of fill above culverts, and (3) the use of bentonitic, expansive soils as backfill.

Chapter 4 describes an experimental investigation to evaluate potential methods to mitigate roadway settlement (Objective 3). A rectangular reinforced concrete testing chamber was fabricated for conducting load tests on model culvert/backfill systems. Vertical cyclic loads were applied through a plate at the surface of the soil deposit to simulate traffic loading. Deformations of the plate and ground surface were observed and recorded. The foundation soil (below the culvert) used in these tests was obtained from a culvert site in Shirley Basin where it was used as a backfill material. A total of seven tests were conducted. Results of the first test were not used because of difficulties in adjusting the loading parameters and in setting up the data acquisition system. Of the remaining six tests, four were conducted using the same compacted clay as backfill. Two of these tests were conducted with a single layer of geogrid reinforcing in the backfill soil. One test was conducted with a compacted uniform sand as the backfill and the final test was conducted using a CLSM backfill.

Test results are presented in Chapter 5. Under low magnitude cyclic loading, all of the backfill materials exhibited relatively small displacements, in all cases less than 5 mm. However, the tests with clay backfill showed a trend of increasing rate of displacement with increasing number of cycles, suggesting potentially large deformations for larger numbers of cycles (>20,000), while the sand and CLSM backfill showed constant or decreasing rates of displacement with increasing cycles. Static load tests conducted subsequent to cyclic loading showed that the sand and CLSM backfills provided significantly higher stiffness and strength than the clay.

Results of tests in which geogrid reinforcing was used are not sufficient to make meaningful conclusions regarding the potential usefulness of geosynthetic reinforcing for reducing roadway settlement. There is a strong likelihood that the geogrids used in this study did not have sufficient stiffness (modulus) to develop enough resistance at the small displacements observed during cyclic loading. Further testing, possibly involving full-scale field tests and with higher modulus geogrids, could provide further insight into the potential benefits of this technology.

CONCLUSIONS

Based on the results of field and laboratory investigations conducted for this study, the following conclusions are drawn:

1. The most likely cause of settlement and roadway damage at culvert sites on Wyoming highways is a combination of the following factors:

    - Use of poor materials as backfill around and above culverts.
    - Inadequate compaction, in part due to difficulties in making proper observations and testing during backfill placement.
    - Low soil cover, defined as 5 m or less.

2. The technologies which show the greatest potential for reducing roadway settlement and minimizing costly damage for future culvert installations include:

    - The use of high quality granular materials compacted to a high density for backfilling culverts.
    - Avoiding the use of highly plastic, compressible fine-grained soils.
    - Use of controlled low strength materials (CLSM), commonly referred to as flowable fill, for backfilling culverts.

3. The effect of geogrid reinforcement in fine-grained backfill materials on roadway settlement was not determined conclusively. Laboratory tests conducted for this study did not suggest any significant improvement in the settlement behavior of geogrid reinforced clay backfill compared to the same material with no reinforcement. The reasons for this lack of improvement in settlement behavior may be due in part to the limited number of tests and test conditions, and further testing could show this to be a viable technology.

RECOMMENDATIONS FOR IMPLEMENTATION BY WYDOT

To incorporate the results of this study into WYDOT practice for constructing culverts beneath roadways, the authors recommend the following:

1. Whenever feasible, utilize high quality granular soils (sands and gravel) for backfilling culverts.

2. When the use of high quality backfill materials is not feasible, consider the following:

    - Use of flowable fill, which provides a stiff subgrade and is highly resistant to settlement.
    - More rigorous testing of fine-grained soils which may be available for backfilling. Not all fine-grained soils are unsuitable for culvert backfill, although soils of the AASHTO

A-6 and A-7 categories are particularly suspect. The use of fine-grained soils should be justified by adequately characterizing soil properties which are reliable indicators of settlement behavior. Recommended properties which should be determined include soil compressibility from consolidations tests, and shrink/swell or collapse potential, determined by collapse consolidometer tests. When fine-grained soils are used for backfill, excellent quality control of backfill placement and compaction is necessary. Laboratory compaction tests on the actual backfill materials are required to determine optimum moisture content and maximum dry density. Inspectors should be present during backfill placement to assure that WYDOT specifications for lift thickness and compacted density are achieved.

3. It is recommended that the Wyoming DOT consider changing the Standard Specifications for Road and Bridge Construction, subsection 206.037. Specifically, this specification currently states that "fine, compactible, excavated soil" is considered acceptable as backfill material around culverts. This specification is too broad in the sense that any soil which can be excavated and compacted (which includes virtually any soil) can be used for culvert backfill, regardless of its engineering properties and performance with regard to settlement. It is recommended that this specification be modified to state that fine-grained soils, especially those classified as A-6 or A-7, are not acceptable materials for culvert backfill unless further testing for compressibility, shrink/swell, and collapse behavior demonstrates their suitability.

4. To take full advantage of innovative ground improvement methods such as flowable fill, it is highly recommended that WYDOT continue to conduct research and development studies on CLSM materials. Specifically the following efforts should be undertaken:

   - Mix design variables and their effects on the performance of CLSM require further research aimed at establishing standard specifications for the use of flowable fill. The required performance characteristics of flowable fill include flowability, rate of hardening, ultimate strength and stiffness, and excavatibility. These characteristics affect the performance of flowable fill during construction as well as during the service life of the culvert and roadway, and the relative importance of each characteristic may vary from job to job. Furthermore, flowable fill properties can vary widely depending upon mix design variables (relative amounts of cement, water, fly ash, and sand), the source of fly ash, and environmental factors such as temperature.
   - Field studies which involve monitoring of roadway settlement at sections constructed with flowable fill and without flowable fill should be undertaken to demonstrate and verify the full-scale performance of flowable fill.
   - The use of geogrid or other geosynthetic reinforcement in poor soils used as backfill warrants further investigation. This could involve field testing or further laboratory tests combined with field tests.

5. Whenever project requirements allow, provide at least 5 m of cover over culverts.

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