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STANDARDS FOR TIRE-BALE EROSION CONTROL AND BANK STABILIZATION PROJECTS: VALIDATION OF EXISTING PRACTICE AND IMPLEMENTATION

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16. Abstract

In an effort to promote the use of increasing stockpiles of waste tires and a growing demand for adequate backfill material in highway construction, NMDOT has embarked on a move to use compressed tire-bales as a means to reduce cost of construction and to recycle used tires which would otherwise occupy much larger space in landfills or be improperly disposed. Compressing the tires into bales has prompted unique environmental, technical, and economic opportunities. This is due to the significant volume reduction obtained when using tire-bales (approximately 100 auto tires with a volume of 20 cubic yards can be compressed to 2 cubic yard blocks, i.e. a tenfold reduction in landfill space). Lighter unit weight, (37 pcf dry), results in lower earth pressure with lesser possibility for foundation failure.
The objective of this project is to address the question, "Can tire-bales be used as a cost effective alternate fill material for erosion control and bank stabilization projects?"

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STANDARDS FOR TIRE-BALE EROSION CONTROL AND BANK STABILIZATION PROJECTS: ENGINEERING VALIDATION OF EXISTING PRACTICE AND IMPLEMENTATION

Final Report

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PREFACE

The research reported herein explores the issue of using tire-bales as a cost effective alternate fill material for erosion control and bank stabilization projects. This project will also conduct an in-depth literature search and establish contacts with manufacturers, contractors & other government agencies; pertaining to design and application of tire-bales. This project will also formalize/validate existing construction standard drawings and specifications prepared by NMDOT Maintenance and Design staff.

NOTICE

DISCLAIMER

ABSTRACT

In an effort to promote the use of increasing stockpiles of waste tires and a growing demand for adequate backfill material in highway construction, NMDOT has embarked on a move to use compressed tire bales as a means to reduce cost of construction and to recycle used tires which would otherwise occupy much larger space in landfills or be improperly disposed of. Compressing the tires into bales has prompted unique environmental, technical, and economic opportunities. This is due to the significant volume reduction obtained when using tire bales (approximately 100 auto tires with a volume of 20 cubic yards (15.3 cu m) can be compressed to 2 cubic yard (1.53 cu. m) blocks, i.e. a tenfold reduction in landfill space). Lighter unit weight, 37 pcf dry, (592 kg/cu. m), results in lower earth pressure with lesser possibility for foundation failure.

The objective of this project is to address the question, “Can tire-bales be used as a cost effective alternate fill material for erosion control and bank stabilization projects?”

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INTRODUCTION

In an effort to promote the use of increasing stockpiles of waste tires and a growing demand for adequate backfill material in highway construction, NMDOT embarked on a move to use compressed tire-bales as a means to reduce cost of construction and to recycle used tires which would otherwise occupy much larger space in landfills or be improperly disposed. Compressing the tires into bales prompted unique environmental, technical, and economic opportunities. This is due to the significant volume reduction obtained when using tire-bales (approximately 100 auto tires with a volume of 20 cubic yards is compressed to 2 cubic yard blocks, i.e. a tenfold reduction in landfill space). Lighter unit weight, (37 pcf dry), results in lower earth pressure with lesser possibility for foundation failure.

Over the course of the project the research team worked to investigate state of the art tire-bale utilization potential in the USA and internationally. Information was gathered from 50 states including published sources and telephone surveys of different state DOTs. Appendix A lists the questions formulated for the telephone survey and details of the survey are provided in Appendix B. Significant progress was made over the past few years and the research looks promising. The final report gives a summarized overview of various aspects on the technology of tire-bale construction, as well as an in-depth analysis of the failure of the tire-bale structure located on NM 143 near Deming, NM, along with the construction details of the Field Demonstration Facility (FDF) and current conditions of the in-field tire-bale structures The final report is organized by the tasks lined out in the contract between New Mexico Department of Transportation and New Mexico Institute of Mining and Technology. Thus the opening paragraph for each task restates the contracted paragraph(s) for clarity and the intent, for the report's continuity. Contractor has been changed to research team and other additions of pertinent information learned in the course of the research or removal of the unnecessary information such as legal terms or statements are present

1.0 TASK 1:

"The research team assessed the current state of practice (both in the US and internationally) in the use of tire-bales in erosion control and bank stabilization. Information from 50 states was gathered from published sources and telephone surveys of different state DOTs was conducted. Key performance parameters were identified and failure or distress modes were noted and cross referenced. Existing standards that were backed by engineering analysis were identified and compared to intended uses in New Mexico to determine the applicability of the analysis. Material and performance characteristics of the tire-bales were collected, and areas in which further work was needed was identified *(1)*."

1.1 Literature Review

A large number of tire-bale projects were completed in the US and internationally for erosion control and bank stabilization. Tire-bales offer significant advantages in construction projects due to the following features:

- Permeability comparable to gravel
- High porosity
- Low bulk density and
- Good frictional response and stiffness

1.1.1 Investigation on Current State of Practice in USA

Texas Department of Transportation *(2)* Between February 2002 and August 2002, the TxDOT's Fort Worth District considered using tire-bales as partial replacement for fill soil used in slope repair projects. Initial evaluation, six years after the completion of the project, revealed that the use of tire-bales instead of the original soil slope had improved the factor of safety by 2-3 times.

Nebraska Department of Environmental Quality *(3)* Following the failure to perform as expected in two major projects, in 2003, the Nebraska legislature *(4)* revised the scrap tire statutes and eliminated the use of tire-bales in Nebraska. In one of the projects, Nebraska Game and Parks Department *(3)* dismantled part of the project and removed the failed tire-bales and replaced the space with rocks. Most of these failures were due to either improper soil cover or corrosion of the tie wire.

New Mexico Department of Transportation In 1986, the Rio Puerco was down cutting near the city of Cuba threatening a bridge across HW 44 (now NM550). A 12 foot high gabion structure was emplaced to raise the level of the stream bed and protect the bridge. In 1992 another gabion structure was constructed 200 meters downstream of the original one, raising the streambed at that point by 9 meters. By 1998 the second structure was being undercut by soil piping around and underneath the dam. Tire-bales were used for bank toe and slope stabilization around the second structure. By 2002, the tire-bales were breaking up and a large number of tires were released into the channel of the Rio Puerco. The tires were released due to corrosion of the ties in the tire-bales. As the integrity of the tire-bales was compromised a large number of them had to be removed at a considerable cost.

Status on all other states is provided in Appendix B.

1.1.2 Current Project Status in UK

A Publicly Available Specification, PAS 108, prepared by The British Standards Institution (BSI) *(5)* in collaboration with Waste & Resources Action Program (WRAP) provides a specification for producing compact tire-bales of a consistent and verifiable quality and dimension *(5, 6, and 7)*. The PAS was prepared following exhaustive consultation from a wide range of stakeholders from the secondary tire industry. PAS 108 provides a specification to be adopted by suppliers for producing tire-bales so that potential customers are assured they are procuring a construction material of consistent and verifiable quality. Thus the core of PAS 108 addresses the production, handling, storage, transport and placement of standardized tire-bales, as well as the dimensions and properties. Additionally, guidance is given on engineering properties and typical construction applications.

1.2 Recommendation

The literature survey in this section demonstrates that the permeability of tire-bale structures can change continuously and eventually become impermeable. In addition to the engineering validation of existing practice, the research team designed and constructed two tire-bale structures for field demonstration facility regarding some of the critical drainage parameters, which the literature survey showed no in-depth studies. The field demonstration facility (FDF) will provide data regarding how the structures' permeability changes with time.

2.0 TASK 2:

"As a result of Task 1; optimal configurations of tire-bales for engineering application were identified. An extensive literature review was conducted to compare available options for erosion control and bank stabilization; in addition to tire-bales, other methods evaluated included: (1) stone armors or revetments, such as riprap blanket and longitudinal stone toe; (2) self-adjusting armors, such as concrete blocks, sacks, soil-cement blocks and rubble from demolition; (3) flexible mattresses: geotextiles, gabion mattresses, and concrete block mattresses; (4) retaining structures, including those constructed with gabion baskets; and (5) vegetative methods for erosion protection. Advantages and disadvantages of each available alternative were documented, along with typical applications and suitability of each method to different site conditions. Practicality of construction of these alternatives was evaluated by comparing the following aspects: construction time, ease of construction, equipment, labor, materials required for installation, expected maintenance, initial costs and life-cycle costs. Durability was determined by evaluating resistance to debris damage, freeze-thaw cycles, fire, and corrosion abrasion, and other factors as applicable. Potential environmental impacts were considered by evaluating, where applicable, the possible effects each method has on wildlife habitat and general environmental conditions. Some aspects considered included: presence of leachates, changes in the amount and type of vegetative cover, interruption of wildlife movement during construction, as well as others. The outcome of this investigation consisted of matrices comparing the most recent alternatives as documented in the literature for erosion control and bank stabilization to the proposed tire-bale application *(1)*."

2.1 Literature Review

Review of the available literature options for erosion control and bank stabilization confirmed that numerous options were available to control bank degradation and ensure stability. Descriptions, advantages, disadvantages, and suitability to different site conditions were compiled for some of the available alternatives and are summarized below. Practicality of construction, initial cost, durability and expected maintenance are being investigated as the field demonstration facility was constructed and during continued monitoring. Appendix C presents a summary of the available alternatives.

2.1.1 Stone Armors or Revetments

This erosion control technique is the placement of loose stones or blocks directly over the sloping bank, after the surface has been properly stabilized, compacted, and smoothed (Figure 1). In some instances, a filter layer composed of approximately 6 to 8 inches of well-graded stone or a filter fabric (geotextile) is supplied to provide support to the revetment as well as to allow the movement of water through the structure (*8, 9)*. The protective material is expected to be strong, heavy, and large enough to resist hydraulic forces, remain in place to absorb the energy of moving water, and the impact from drifting objects *(10, 11)*.

The most common type of stone armor is the riprap blanket *(10, 11, and 12)*. Riprap stone selection is usually well graded so that the larger stones can resist water motion, while smaller ones prevent soil from being carried away *(10, 11)*. A standardized methodology for stone size and slope of riprap particle distribution curve is proposed in Lagasse *(13)*.

As with most protection systems, riprap blankets should be periodically inspected and maintained. Even with its widespread use, very little guidance is available for inspection and quality control of riprap during construction or for long-term monitoring *(13)*. A simple inspection method was however described by Galay *(14)* and presented in Lagasse *(13)*. It consists of a numeric ranking scheme based on the observed condition of the entire system and of the riprap particles themselves.

FIGURE 1 Riprap erosion protection of sidewalk, Campbell River, British Columbia (http://www.goodinghydrology.com).

Advantages, disadvantages, and typical applications for stone armors, and more specifically riprap, are presented below, having been compiled from the following sources: Fisher *(10)*, Biedenharn (11), Davis and Maynord *(12)*, Northwest Regional Planning Commission (*8)*, Lagasse *(13)*, and North Carolina Department of Environment and Natural Resources *(9)*.

Advantages:

Since the stone armors method has been extensively used in practice and widely researched, design guidelines are readily available. The following are the guidelines referred to as part of this project: ASTM D6825 *(15)*, U.S. Army Corps of Engineers EM 1110-2-1601 *(11)*, National Resources Conservation Service, New Mexico (NRCS - NM) Conservation Specification 580R-1 *(16)*, and the National Cooperative Highway Research Program (NCHRP) Project 24-23 *(18)*. The latter also includes construction methods and placement techniques

- Natural material is usually readily available for use in riprap.
- Experienced contractors with appropriate equipment are generally available.
- Resistant to minor damage.
- Remains functional even if some stones are lost.
- Easily repaired.
- Practical when quick response and immediate effectiveness are required.
- May provide an adequate habitat for some aquatic organisms.

- Because it provides permeability without exposing bank material, it does not require the placement of underlayment material. (However, Geotextile fabric or a filter of sand and/or gravel may be installed).

Disadvantages:

Cost can be significant if material is not locally available. According to ASTM Standard D 4992, Practice for Evaluation of Rock to Be Used for Erosion Control, briefly summarized in Fisher *(10)*, most riprap comes from sedimentary rock, mainly limestone and dolomite. Some igneous and metamorphic rock may also be used, as some sandstone has been previously weathered and case hardened. Although commercial quarries may be located near the construction site, they may not be able to produce the required gradation, either because of the blasting techniques employed (which often produces only small pieces) or because of inadequate thickness of the sedimentary rock layers (ASTM Standard D 5779, Test Method for Field Determination of Apparent Specific Gravity of Rock and Manmade Materials for Erosion Control summarized in Fisher *(10)*.

- Extensive preparation of the bank slope may be required prior to installation to provide stability, proper compaction and/or adequate smoothness. As a result, costs and disturbance to adjacent environment may be significant.
- Weakens with age.
- Not aesthetically pleasing.
- Difficult and sometimes dangerous pedestrian access to water’s edge.
- For some applications, larger stones are needed, therefore requiring the use of heavy equipment for installation.
- These structures are not recommended for steep slopes or areas with significant amount of loose soil.

- **Typical Applications** Riprap blankets may be used to reduce and control erosion of riverbanks, highway structures in riverine environments as well as bridge piers and abutments *(13)* where embankment is stable *(8)*. These structures are effective in areas with turbulent flow *(11)* and horizontal to vertical slopes of less than 2:1 since most rock cannot be stacked on horizontal to vertical banks steeper than 1.5:1 *(8, 9)*. In steeper slopes, gabion baskets may be used.

2.1.2 Self-Adjusting Armors

Armors formed by blocks that are placed individually and therefore have the ability to adjust to irregularities of the bank surface are often referred to as self-adjusting *(11)*. They are usually composed of concrete blocks, or sacks of earth, sand and/or cement (Figure 2). According to Biedenharn *(11)*, rubble from demolition of pavement, slag from steel furnaces and automobile bodies have reportedly been considered, but with no success. Sacks can be placed on steeper slopes than concrete blocks *(11)*.

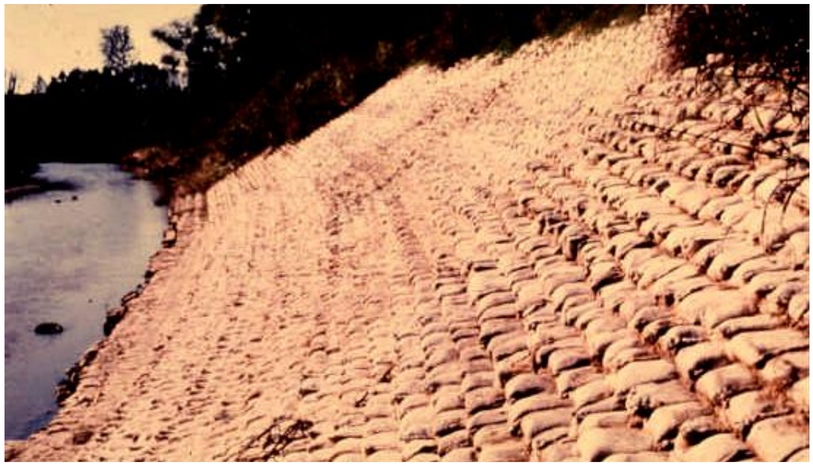


FIGURE 2 Typical Sack Revetment (Biedenharn *et al.* 1997).

Several unique shapes for concrete units have been patented and are commercially available for use in self-adjusting armor systems. Examples include: Samoa Stone™ *(19)*, Core-loc® *(20)*, both developed and patented by the U.S. Army Corps of Engineers (Figure 3), Accropode™ *(21)* and Ecopode™ *(22)*, both developed and patented by SOGREAH (Figure 4).

FIGURE 3 Core-Lock Individual Block (http://www.concretelayer.com) and Revetment (http://www.core-loc.com).

(a) **(b)**

FIGURE 4 (a) Accropode™ Armor Block and (b) Ecopode™ Armor Block (http://www.concretelayer.com).

Advantages, disadvantages, and typical applications for self-adjusting armors were gathered from the following reports: Biedenharn *(11)*, Davis and Maynord *(12)*, Turk and Melby *(19)*, and Sogreah Consultants *(20, 21, and 22)*.

Advantages:

- Concrete blocks can often be cast on site with the use of forms, hence reducing the cost of transportation.
- If sacks are to be used, filler material is often available locally, also reducing the cost of transportation.
- Blocks can be installed over uneven and irregular surfaces, hence reducing the use of heavy construction equipment and cost of surface preparation.
- Some blocks and sacks can be manually installed, making them not only cost effective, but also easy to install in urban areas or areas of difficult access for heavy equipment. Mechanized placement may also be used to reduce installation time and the amount of labor required. Larger blocks can usually be installed with the use of forklifts.
- Because concrete armor units may be used as a single-layer, these systems use 3 to 4 times less material than natural rock armors.
- Concrete blocks are durable.
- Easily maintained.
- The use of individual blocks provides for bank drainage.
- System can be covered with vegetation.
- Usually provide easy pedestrian access to water's edge.
- High porosity provides for good energy dissipation.
- Voids provide habitat for different species.
- Allows for vegetation cover (not true for sacks filled with cementious material).

Disadvantages:

- Often requires a fabric or filter.
- Since blocks (or sacks) are interlocked, displacement of one block (or sack) by water flow may lead to successive displacement of adjacent units. In the case of sacks filled

with cementious mixtures, this process can be prevented by bonding adjacent units together.
- Susceptible to weather delays if blocks are cast on site.
- Susceptible to theft or vandalism.
- Unnatural appearance.
- Sack armors often act monolithically on steeper slopes.
- Some of the materials used for sacks may be vulnerable to fire, ice, livestock traffic, floating debris and even rupturing by roots vegetation.

Typical Applications: Concrete blocks are commonly used in bank armors, ditch and spillway linings, and culvert outlets. Sacks are effective on transitions to steep slopes and where low-cost labor is available; they may be the most cost-effective alternative.

2.1.3 Rigid Armors

Armors may also be made of rigid materials such as asphalt, concrete, and grouted riprap that, after construction, will be unable to adequately conform to bank irregularities or small settlement *(11)*. An interesting application of this type of revetment was proposed by Huang and Yu *(23)* and consisted in the use of "no-fines concrete" to reduce the environmental impact of the structure. As indicated by its name, this type of concrete mixture only included coarse aggregates, cement, and water, hence yielding a strong, durable, and very permeable material. This type of concrete is also referred to as "permeable", "pervious", or "green" concrete and has been used, among other applications, to create artificial reefs, to reduce water accumulation in parking lots, residential driveways, swimming pool decks, aquatic amusement parks, and tennis courts, to reduce noise produced by tires on pavements *(24)*, and to increase thermal and acoustic efficiency of structures *(25)*. In the work of Huang and Yu *(24)*, "no-fines concrete" is proposed as an environmentally-friendly erosion control alternative. The presence of the relatively large pores will allow for the retention of seeds and sludge, thus creating a suitable environment for vegetation growth even in the presence of large and/or frequent water-level fluctuations (Figure 5). The presence of these larger voids also allows for the drainage of the pore-water, which is usually basic due to concrete hydration reactions and therefore hinder vegetation growth. If needed, larger artificial access holes may also be created to accommodate aquatic species. Vegetation will not only provide a more aesthetically-pleasing structure, but it will also provide additional resistance to erosion mechanism. Advantages, disadvantages, and typical applications for rigid armors were obtained from Biedenharn *(11)* and are presented below.

Advantages:

- Capable of withstanding large hydrostatic forces.
- Extremely resistant to damage from debris, corrosion, and other destructive agents.
- Not susceptible to vandalism.
- Where slopes are not too steep, they provide easy pedestrian access to water's edge.

- In regions where material for self-adjusting armors is not locally available, and extensive subsurface drainage is not required, rigid armors are often the most cost-effective solution.


FIGURE 5 "No-Fines" concrete as erosion control system (a) pores retain sludge and seeds, promoting vegetation growth (b) completed product *(23)*

Disadvantages:

- Require careful design and quality control.
- Construction is susceptible to weather delays.
- In the case of impermeable armors, filters or subsurface drains must be provided for draining groundwater and preventing buildup of excess positive pore-water pressures, which usually causes a significant increase in costs.

Typical Applications: These systems are usually recommended in regions of turbulent flow or high velocity flow, where self-adjusting armors are either ineffective or cost-prohibitive. They are suitable for steep slopes and for artificial channels and recommended when water must not infiltrate into the bank.

2.1.4 Flexible Mattresses

Flexible mattresses are created by combining or fastening materials to create flexible systems capable of resisting erosive forces *(11)*. Common types of flexible mattresses include gravel admixtures, fiber mattresses, biodegradable mattresses; geotextiles, gabion mattresses, and concrete block mattresses. An interesting application of gravel admixtures for erosion control in semi-arid climates was proposed by Anderson and Stormont *(26)* who combined gravel with native soil at a site in southeastern New Mexico. In this system, finer soil particles are removed by erosive forces leaving behind an "armored layer that inhibits the formation of deep rills and gullies." This method was found to be particularly attractive to the Southwest region of the United States where the high temperatures and the minimal rainfall prevent the growth of dense vegetation which is often desirable for aesthetic reasons as well as for additional protection against erosive forces. A similar approach was proposed by Gyasi-Agyei *(27)* who successfully used waste ballast mulch, that is, discarded railway ballast after it has been fouled by coal dust infiltration and by ballast breakdown into smaller pieces, as a flexible mattress.

Compost soil has also been used as flexible mattress. Xiao *(28)* showed that the application of a layer of compost soil was effective in controlling erosion of roadside embankments in addition to allowing for quicker vegetation growth. The patent pending compost was developed by Earth Solutions and consisted of cattle manure, natural soil and agricultural byproducts.

Natural or synthetic fiber mattresses are also effective alternatives *(12, 27, 29, 30, and 31)*. Jutemat (Finemat), Environmat, and Coconut Fiber blanket, although expensive for widespread application were found to be very effective by Gyasi-Agyei *(27)* who proposed laying them on sections of the slope to reduce costs. Geosynthetic rolled erosion control products (RECP) have increasingly been used due to their reduced cost and their ability to promote vegetation growth *(29)*. These systems may be made of degradable or non-degradable materials weaved to form a mat or blanket that is rolled over the slope to be protected (Figure 6). Different geosynthetic were tested by Smith *(29)* on slopes of different steepness and by de la Cruz *(30)* on steep slopes. Results showed that most systems tested were effective in protecting the banks from damage caused by rain, reducing erosion due to runoff by retaining the soil and reducing flow velocity (because of the material's roughness), maintaining close contact with the soil and hence minimizing water flow beneath the RECP *(29)*. Gabion mattresses are yet another type of flexible mattress available for erosion protection. They consist of rectangular baskets formed by a mesh of galvanized steel filled with cobbles or quarried stone *(8, 11)*.

Advantages, disadvantages, and typical applications of flexible mattresses were compiled from the following source; flexible mattresses in general and concrete block mattresses, Biedenharn (11), (1) gabion mattresses, Biedenharn *(11)*, Northwest Regional Planning Commission *(8)*, Honnigford *(31)*, (2) natural fiber mattresses, Davis and Maynord *(12)*, Gyasi-Agyei *(27)*, Smith *(29)*, (3) gravel admixtures, Gyasi-Agyei *(27)*, Anderson and Stormont *(26)*, (4) rolled erosion control products, Smith *(29)*, Honnigford *(31)*, and (5) compost layer: Xiao *(28)*.

FIGURE 6 Installation of rolled erosion control product *(31)*

Advantages:

- Flexible mattresses are capable of adjusting to settlement and remain in contact with the bank.
- Most materials for flexible mattresses are available under trade names in various configurations and can therefore be applied to numerous situations.
- The use of a layer of gravel admixtures has very little impact on local vegetation or soil-water balance.
- While vegetative covers alone may be effective in reducing erosion, Anderson and Stormont *(26)* showed that gravel admixture layers provide the mechanical stability often essential in the arid and semi-arid Southwest region of the United States.
- Gravel layers are effective in reducing erosion due to runoff and wind as well as retaining seeds in place until germination.
- As shown by Gyasi-Agyei *(27)*, flexible mattresses do not need to cover the entire slope to effectively reduce erosion. They can be laid only on the outer verge of the slope where they will spread runoff and therefore control the formation of rills or on the lowest section of the slope where it will reduce piping (or tunnel erosion) and induce sediment and seed deposition.
- Waste ballast mulch mattresses may be used in areas where fires are a potential hazard.
- Compost layer not only reduces erosion but also promotes the establishment of vegetation.

- Natural fiber mattresses effectively reduce slope erosion by reducing runoff velocity, raindrop impact and encouraging infiltration.
- Natural fiber mattresses and geosynthetic RECP offer protection during vegetation growth.
- Rolled erosion control products (RECPs) are affordable and easy to install.
- Fiber mattresses and RECPs are not very noticeable and after vegetation has grown, have an aesthetically pleasant appearance.
- Gabion mattresses allow for the establishment of vegetation.
- Provide easy pedestrian access to the water's edge.

Disadvantages:

- The use of gravel layers is not recommended for channels or areas of concentrated flow.
- Fiber mattresses may need to be anchored because their broad surfaces may experience large uplift forces.
- No design guidelines for mattress selection or for anchoring techniques are available.
- Manufacturers of rolled erosion control products provide little design guidance besides maximum slope, flow velocity and shear stress. For this reason, several products will usually meet the requirements, leading to the belief that all alternatives will behave similarly, which was found to not always be true (Smith *et al.* 2005).
- Mattresses are often susceptible to deterioration and vandalism.
- Construction of some alternatives is labor intensive and may require specialized labor force.
- Gabion mattresses require a firm soil foundation and a solid toe. These features may cause an increase in cost.
- Installation costs of gabion mattresses are often higher than those of other bank stabilization alternatives.
- Heavy machinery is often required for installation of gabion mattresses.
- The installation of filters may be required before placement of gabion mattresses if stones used to fill the basket are large.
- Gabion mattresses are not as flexible as some of the mattresses available.
- Even though wires used in gabion baskets are coated with corrosion-resistant substances, it is still susceptible to deterioration.
- Gabion mattresses create a barrier for wildlife.
- Gabion mattresses require high monitoring and maintenance as well as frequent repairs.

Typical Applications: Flexible mattresses (*27, 31)* are effective in steep slopes. Mattresses formed by concrete blocks *(11)* or RECPs *(31)* are suitable for areas where erosive forces are severe or where construction operations are difficult due to high flow velocity or great depths. Gabion mattresses are recommended for slopes or banks that are moderately steep *(31)*. Flexible mattresses are often cost-effective *(31)* especially when materials used are locally available *(11)*.

2.1.5 Retaining Structures

Gabion walls are among the most common types of retaining structure used for erosion control; they consist of stacked gabion baskets made from heavy gauge wire rectangular boxes filled with large diameter rocks (Figure 7). Two basic types of baskets are readily available; woven wire mesh and welded wire mesh. Woven wire mesh baskets consist of a double-twisted, hexagonal mesh obtained by twisting two wires together in two 180-degree turns. Welded wire mesh baskets have a uniform square or rectangular pattern and a resistance weld at each intersection *(9)*. Adjacent baskets are fastened to each other to prevent movement and failure of the structure. A firm soil foundation is required to assure stability and a filter layer composed of six inches of well-graded stone or a filter fabric (geotextile) is recommended between the slope and the baskets to provide support and allow water movement through the structure *(8)*. According to Brand *(32)*, when geogrid reinforcement is placed between the soil and the gabion wall, the geotextile adds stability to the embankment and helps anchor the wall, while the gabions prevent erosion. In addition, by properly strengthening the soil embankment, designers can reduce the number of gabion baskets required to stabilize the slope.

Gabions baskets are generally assembled at the work site and are only filled with rock after they have been placed in their appropriate location and tied to adjacent baskets. Galvanized wires are used for corrosion protection. If abrasion from stream sediments is expected, poly vinyl chloride (PVC) coated material is recommended. Although material costs may be increased, the benefits will include durability and longevity of the installation. This coating provides long term benefits for a relatively small increase in material costs *(9)*, since these structures have been widely used in the United States, detailed standards and specifications for their design and installation are readily available *(33, 34)*.

Vegetated gabion walls have been proposed to improve aesthetic and ecological impacts; for this purpose, topsoil is introduced into the voids present in the gabion structure (30 to 40 % of the structure's volume), allowing for root propagation between the stones *(9)*. A thin layer of backfill can also be spread on top of each row of baskets and live branches or cuttings 0.5 to 1 inch in diameter inserted parallel to the baskets and covered with yet another layer of soil. The length of the cuttings should reach behind the back of the gabions into the backfill. Roots will provide additional soil strength while the vegetation will hide the surface of the baskets, improving the aesthetics of the system *(8)*.

Advantages, disadvantages and typical applications for gabion basket walls were obtained from Anon *(33)*, Burroughs *(35)*, Brand *(32)*, Yoon *(36)*, and the Ohio Department of Natural Resources *(37)* and are presented below**.**

Advantages:

Since this method has been extensively used in practice and widely researched, assembly and installation guidelines such as Natural Resources Conservation Service New Mexico (NRCS - NM) Conservation Specification 580W-1 *(16)* are readily available. Assembly of double-twisted wire mesh gabions are presented in ASTM D7014 *(38)*.

FIGURE 7 Retaining Structure Composed of Gabion Baskets (http://www.geotas.com)

- The mesh construction and the loose rocks employed, permits natural adjustment (flexibility) to varying settlement without causing fracture or collapse of the structure.
- Gaps between the stones silt up naturally as time passes. Silting supports the growth of grass and plants which serve as a bonding agent for the stone.
- These structures allow water to drain through, therefore, water pressure does not build up behind them and they are not subject to hydrostatic pressure.
- The flexibility of the Gabion structure provides an inherent strength to dissipate and withstand pressures exerted by water and earth masses.
- Gabions can save up to 40% in material and construction costs compared with rigid structures.
- Filling materials are usually found on or near the site, reducing transportation costs and making them an attractive alternative in remote areas, where only the bundled baskets need to be brought to the site.
- No structural maintenance is needed and foundation work is usually unnecessary.
- The structure can be made aesthetically pleasing by use of natural stones especially when subsequent vegetation growth takes place.
- Unskilled labor can be used for quick assembly.
- Extensions of the existing structures can be done by simple adding additional units.
- Gabions can be installed without drying the creek beds.

Disadvantages:

- Wire baskets are susceptible to deterioration, especially if not galvanized or coated. Although plastic gabion baskets are sometimes available, they are more vulnerable to vandalism and deterioration due to sunlight exposure.
- Usually requires the use of heavy equipment. Hand labor is often required to avoid gaps on exposed surfaces.

Typical Applications: Gabion structures are effective along moderate slopes *(8)*, high-energy environments and where construction area is limited *(9)*. These systems can be used to stabilize the entire slope or just the toe of the embankment. They are not recommended for steep slopes or areas of loose soil *(8)*.

2.1.6 Tire-Bales

Old tires have been used in a variety of applications, for example; residential walls, sound barrier fills, animal fences on farms, barriers for road construction, pavement frost barriers, lightweight embankment fills, and erosion control *(39, 40)*. Two typical methods of recycling or disposing of discarded tires are shredding or baling *(41)*; production of powdered, ground or granulated rubber, and splitting of the tires are also commonly employed. The manufacturing of tire-bales is quite energy efficient; consuming only $^{1}/_{16}$ of the energy required to shred a similar mass of tires *(7)*. When left whole, tires were found to present no pollution problems, and when filled with soil or baled, they were found to be fire resistant and very durable in various environments *(39)*. To assure fire will not be a threat, Jones *(40)* recommends that tire-bales be coated with a thick layer of non-combustible cement-based material, earthen plaster, or stucco and that voids be grouted to further reduce the amount of oxygen present. To fill the voids in the tire-bales, compacted soil has been placed between layers, under the entire structure, and as a cover *(7, 42)*. In other applications, shotcrete was reportedly used as a protective cover for the exposed tire-bales *(42)*. Tire-bales also have been found to have low leaching potential and high chemical and physical durability *(42)*. According to Zornberg *(43)*, they have been used in civil engineering and embankment applications for more than 14 years.

Several companies have been manufacturing tire-bales by compressing whole tires in a hydraulically operated baling machine, and tying them together with galvanized or stainless steel wires *(44, 40, 42)*, as shown in Figure 8. Tire-bale sizes vary from manufacturer to manufacturer. However, each tire-bale is typically composed of approximately 100 tires, usually overlapped in each layer. Typical sizes range from 30” to 47” x 50” to 60” x 60” to 66” *(39, 41, 40, 42, 45, and 46)*. Interestingly, according to Hoenig *(39)* and to the Texas Department of Transportation *(44)*, after a year in baled condition, wires used to hold tires together may be removed and the tires will still remain in place. Zornberg *(42)* reported that Encore Systems of Minnesota may place steel pipes in two directions through the middle of the tire-bale to permit their linking in the field, by use of aircraft cable threaded through the pipe, thus preventing them from shifting, and improving their shear resistance. Tests conducted at the Colorado School of Mines *(40)* showed that tire-bales are strong in compression and that they are able to support the applied load even after one of the wires has failed, although more deformation will evidently occur. Similar results were reported by Zornberg *(42)* who describe a study conducted by Central States Tire Recycling of Nebraska, where an “Enviro Block” tire-bale was subjected to a static load equivalent to a “fully loaded semi trailer,” suffering only minimal distortion.

FIGURE 8 Tire-Bale manufactured by Encore System, Inc.

(http://www.tirebaler.com/).

According to Zornberg *(42)*, although tire-bales have been efficiently used for erosion control, little information has been published regarding these applications or their design. As an example of successful application, they mention the large restoration project along Lake Carlsbad in New Mexico, where a 1,220 m (4,000 ft) long section of the shoreline was protected against erosion with the use of tire-bales. In this project, tire-bales were placed over a wet concrete leveling pad, enclosed in shotcrete, and the top covered by backfill material which was also placed behind the structure. Although not baled, tires were also used in the construction of a small dam in an arroyo. According to Hoenig *(39)*, this structure was still performing satisfactorily five years after construction. Dr. Hoenig and some colleagues from the University of Arizona are also responsible for a tire erosion control structure built at the King's Avil Ranch near Tucson, Arizona *(47)*. This structure was built by stacking tires and tying them together with half-inch plastic straps. They were then filled with gravel and covered with chain-link mesh. Similarly, tire-bale applications have used geosynthetic or metallic reinforcement and concrete or stone fill to increase interface shear resistance and reduce movement of tire-bales in the field *(42)*.

Zornberg *(42)* also reported on a 2002 project involving the use of tire-bales as strong lightweight fill for repair of a slope along Interstate 30, near Fort Worth, Texas, that had failed due to above average rainfall. In this project, 360 tire-bales were placed at the toe of the slope, which was then covered with soil, compost, and seeds to stimulate vegetation growth and reduce erosion. Satisfactory results were observed during a subsequent site visit, and preliminary slope stability analysis indicates that the factor of safety has been improved by 2 to 3 times.

In New Mexico, tire-bales have also been successfully used in erosion control projects by the New Mexico Department of Transportation (NMDOT), as reported by Bandini *(48)*. Although at least seven structures using tire-bales have been constructed in New Mexico, Bandini's paper *(48)* focused only on two: the first one near Winston, New Mexico, along

NM 52, milepost 27, built during the fiscal year 2003-2004, and the second one in Hillsboro, New Mexico, along NM 152, milepost 48, built during the fiscal year 2004-2005. These structures used 160 to 610 tire-bales to provide a cost-effective and expectantly long-lasting solution to the recurring erosion of the arroyos' banks. Construction began by anchoring two rows of tire-bales with 4" x 10' angle irons and $^{3}/_{8}$ in steel cables, below the flow line of the channel. After the first layer was secured, it was encased in hexagonal gabion wire. The second layer of tire-bales consisted of a single row, centered over the first layer, also encased in gabion wire, and secured to the first layer with wire. The third layer was created in a manner similar to that of the second one, ensuring the presence of an offset back, approximately equal to half the width of one tire-bale, for structural stability. Since banks were significantly eroded, after the placement of the tire-bales, backfill material was compacted behind the structures. Although soil was used to cover the finished structures, they are currently exposed to rain; it was noted that structural stability was not compromised and that these projects have performed satisfactorily. Erosion problems have ceased at both sites, and after four years and several exposures to heavy runoff events, the structures remain stable and although several wires used to fasten the bales show signs of oxidation, none were broken.

Advantages, disadvantages and typical applications of tire-bales are presented below:

Advantages:

- Comparatively inexpensive to manufacture and install *(42)*.
- Lightweight compared to concrete and riprap, and therefore easier and cheaper to transport and install *(7, 44)*. They can usually be moved and placed using a forklift *(42)*.
- Comparable permeability to gravel *(5)*.
- Good load-bearing capacity *(40, 42)*.
- High chemical and physical durability *(39, 42)*.
- Fire-resistant when coated with a thick layer of non-combustible cement-based material, earthen plaster or stucco. Voids may also be grouted to further reduce the amount of oxygen present *(40)*.
- Low leaching potential *(42)*.
- After being baled for a certain time, they will retain their shape even if some of the wires used to hold the tires together break or are removed *(39, 44)*.

Disadvantages:

- Practice not yet well established, causing design to be "somewhat experimental" *(42)*.
- Although tire-bales have been successfully used for erosion control, little information has been published regarding these applications or their design *(42)*.
- Although lightweight, some contractors have found them difficult to place due to the lack of specific lifting points *(41)*.
- Long-term compression and creep rates are still unknown *(42)*.

Typical Applications: Tire-bales have been used in various applications, for example; residential walls, barn construction, sound barrier fills, animal fences on farms, barriers for road construction, pavement frost barriers, lightweight embankment fills, erosion control and slope stabilization (Hoenig 2003, Jones 2005, Winter *et al.* 2005, Zornberg *et al.* 2005).

2.2 Environmental Impact Assessment

"An environmental impact assessment and landscaping studies was performed, and the potential environmental impacts on wildlife habitat and general environmental conditions were assessed. The results of this investigation were incorporated into the matrices for the design of the tire-bale structure.*(1)*"

The Environmental Impact Assessment included a literature review, in-field documentation of erosion patterns to included tensile cracking, scouring, and upper level erosion, additionally any visual changes in the structures, vegetation growth, habitat usage, and soil retention were monitored. The tools for documentation were photo journalism, Garmin eTrex Legend Cx, a measuring tape, and a written journal. Microsoft Excel was used to design a representation of two structures so specific erosion features could be positioned on the upper level of the structure. The data used came from the Garmin eTrex Legend Cx.

2.2.1 Literature Review

A literature review on EPA guidelines *(49)* for hazardous waste brought the expected information regarding tire piles. Mainly, that tire piles are not a hazardous waste until a fire causes the material to break down into hazardous compounds, such as gases, heavy metals, and oils. The need for cleanup triggers "Superfund" procedure and policy, which is costly and time consuming. The Superfund includes tires that have been released into waterways regardless of constant flow. Along with these findings were the numerous guidelines for tire storage and tire shreds, which have set the standard for the considerations for tires as a hazardous waste.

Also found on the EPA website *(50)* was a literature review done by the University of Maine *(51)* on water quality and environmental toxicology effects of tire-derived aggregate (TDA). It was found that TDA was likely to increase concentration of iron and manganese with a migration away from the installation site. A cited report suggested the potential for toxics to leach from the tires when placed in wet soil and that the impact varied according to the local water and soil conditions, especially pH value. Although the review pertains to TDA a presumption by the research team was made to include tire-bales as a conservative approach since little information was available in 2008.

A 2005 Colorado DOT report *(42)* used ASTM D6270-98 *(52)* for exothermic reactions in stock piles of whole tires and tire shred embankments of more than 1 meter (3.3 ft) thick.

The exposure of steel (corrosion) is thought to be the major source of internal heating, and in addition the presence of organic material and exposure to air and water increase corrosion potential. The report stated; although there are limitations on the fill height the reasoning for internal heating is not conclusive. The ASTM D6270 *(52)* was adopted in 1998 after the Federal Highway Administration's (FHWA) adoption in 1997. The conservative guidelines were prepared by a committee established in 1995 comprised of individuals from industry, government, and academia with the principal author being Dr. Dana Humphrey from University of Maine.

As part of the Colorado DOT *(42)* report the U.S. Fire Administration (1998) states that there are 12 key issues with reducing the possibility and impact of tire fires. Six key issues in the report were: (1) code enforcement, (2) agency coordination, (3) equipment needs, (4) extinguishing tactics and agents, (5) disposal of burned tires, and (6) cost.

The literature appeared to make assumptions pertaining to the exothermic heating of tire-bales being reduced by very limited exposure to oxidation and smaller sizes of tire shreds. There was very little research available in 2008 to back up the theory on the tire shreds and none found on tire-bales.

There was research available regarding drainage within the tire-bale structures. Since lack of good drainage or water retention within the tire-bales can become a breeding environment for mosquitoes and can increase diseases like West Nile, the Texas DOT report from the University of Texas Austin gave us concern *(43)*. Although Texas receives more rainfall than New Mexico and the information provided in this report does not have a time frame for the structure, 2000 gallons of water poured from the tire-bale structure upon excavation. The excavation was necessary due to an unreinforced slope failure adjacent to the tire-bale structure. After communication with Dr. Jorge Zornberg, he confirmed the length of time for water accumulation was unknown. The report does recommend the usage of appropriate drainage or drainage products, such as a basal drainage blanket.

In another study by Dr. Zornberg regarding embankment built with tire shreds and nongranular soil *(53)*, soil migrated into voids of the tire shred and tire shred-soil mix caused sinkholes (in the first 120 days). By the end of 824 days, the tire shred and tire shred mix had sinkholes of 87% and 62% (respectively), larger than the maximum for the soil only area. No geotextiles were used in the testing so the extent of the sinkholes could be evaluated.

Further literature review looked into the standards for landfills since geotextiles are applied for sinkholes (or raveling) concerns. In addition, the Earthship building was researched and discussed. The structures have been successful with drainage problems as well as out gassing as shotcrete is used to cover the finished product. In addition, some discussions with New Mexico fire authorities were necessary to further determine their needs regarding the current structures and any new installations.

There was a conversation with New Mexico Fire Marshal Josh Stanford on October 3, 2008. Mr. Stanford's main concerns were for fire safety and notification for indoor and outdoor structural storage of tire-bales. Outdoor structural storage is defined as a structure that is sided and roofed on three sides that does not allow heat and gasses to escape the structure. The entrapment of heat and gasses keep the fire ignited. Additionally an indoor (four sided and roofed) structure entraps the heat and gasses keeping the fire ignited. There is still a monitored “Superfund” tire fire located in Socorro, NM and according to Mr. Stanford there was still heat under the layers of clay that bury the burnt tires. The burying of the burnt tires is the method used to smoother the gasses from further ignition. Mr. Stanford suggested several independent testing laboratories for further research. Unfortunately no testing information regarding tire-bales and fire in 2008 were available, so the International Fire Code *(54)* was used for the appropriate design and procedure of authority notification and storage during regarding tire-bales construction. Additionally, New Mexico Environmental Protection Agency *(55)* guidelines and procedures were used.

2.2.2 In-field Site Locations and Topography

The NM52 & NM152 in-field sites had ongoing observations of erosion patterns, tensile cracking, vegetation growth, and soil retention over the course of the project. Photo journalism was kept and measurements taken for a mapping comparison using a measuring tape. Measurements of erosion patterns and tensile cracking were also taken in 2008 and 2009 and the finally in 2010 with the Garmin eTrex Legend Cx. The Garmin has a DGPS accuracy of 10-16 feet (3-5 meters) 95% typical in North America *(56).* This method was used to verify the practicality of using GPS for this purpose. In the field we found that the Garmin unit was considerably more accurate than the published value, and could potentially be used by a single worker to map the in-field structures.

The measurements taken in 2010 were plotted in a spreadsheet for mapping the current visual condition of the tire-bale structures on NM52. No measurements of the NM152 site were taken as there were no visual changes in the tire-bale structures appearance. The analysis using the Garmin is merely to map the lower and upper levels of the tire-bale structures

Figure 9 show the location of NM52 & NM152 tire-bale in-field sites in relation to their topography and each other. Winston, NM is the nearest town on NM52 and Hillsboro, NM is the nearest town on NM152. The in-field NM52 sites approximate elevation is over 6300 feet but not exceeding 6400 feet, while the NM152 site is slightly over 5300 feet. Both elevation estimates came from using the Garmin and Google Earth. The in-field sites lie in the eastern foothills of the mountain range in the Gila National Forest, which has its highest elevation at Reeds Peak of 10,011 feet. Figures 10 and 11 show the location of each tire-bale structure on their respective New Mexico highways with the nearby topography, Google Earth was used to display topography of the in-field sites.

NM52 has three locations at mileposts 26.7, 27.2 and 27.75. They are located in Figure 10 with designated arrows while their configuration follows the contour of the western side of the arroyo near the roadway. Milepost 26.7 is the upstream direction of the arroyo.

NM152 has one location at milepost 47.7; the Google Earth map (Figure 11) has been rotated for better viewing ease as the structure was built against a hill thus north is to the right of the structure. The site follows the arroyo along the hillside. The two arrows depict the upstream and downstream locations of the structure.

FIGURE 9 Topography of NM52 & NM152

FIGURE 10 NM52 Topography

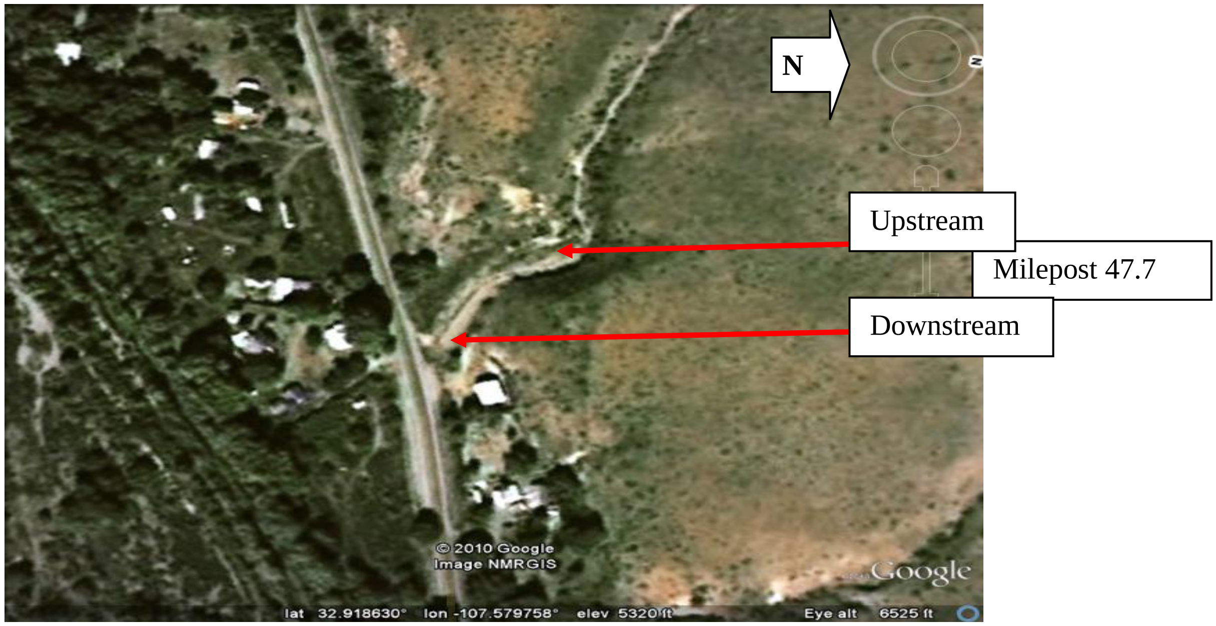


FIGURE 11 NM152 Topography

2.2.3 Erosion Patterns & Tensile Cracking

Fortunately, the visual observations of NM152 required little photo journalism or details regarding erosion features or patterns. The site appears unchanged to the observing eye. For the purpose of comparison and documentation two photos are shown below in Figures 12 and 13. The arrows indicate the same area in both figures. As a frame of reference the rock in Figure 12 is still present and in the same location in Figure 13. On the July 2008 in-field site visit there was some sediment displacement within the tire-bales accompanied by some voids. In the May 2010 visit it appears settlement of the sediment has slowed or ceased.

FIGURE 12 July 3 2008 NM152

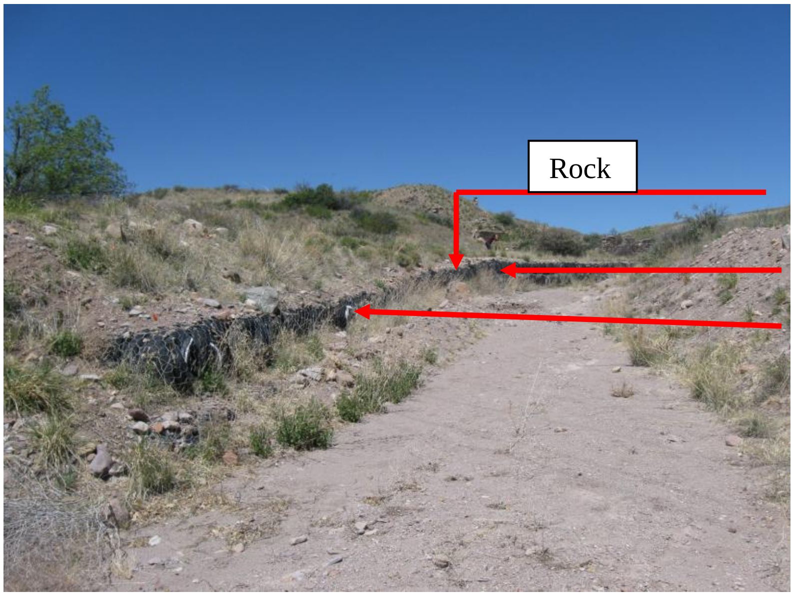


FIGURE 13 May 4, 2010 NM152

However, NM52 had measurable visual observations of concern:

1. On December 20, 2008 milepost 27.2 had approximately an additional 129 feet of exposed tire-bales that were not visible on June/July 2008 when Dr. Budek-Schmeisser and students placed survey points in the area. The exposed structure measured approximately 228 feet when the survey points were installed in June/July 2009. On December 20, 2008 the exposed structure measured approximately 352 feet. Figure 14

indicates the new exposed area. New Mexico had a wet monsoon season in 2009 which was only one contributor to the exposure of the tire-bales.

- On July 5, 2010 two years following the initial survey point installation, the full structure measured approximately 379 feet. Taking into account the Garmin accuracy factor of a maximum of 16 feet, the minimum additional exposure could have been 5 feet or a maximum exposure of 59 feet. The observed additional tire-bale exposure was at the beginning of the structure and measured less than 5 feet. This additional 5 feet at the beginning of the structure appears to be the remaining length (Figure 15).

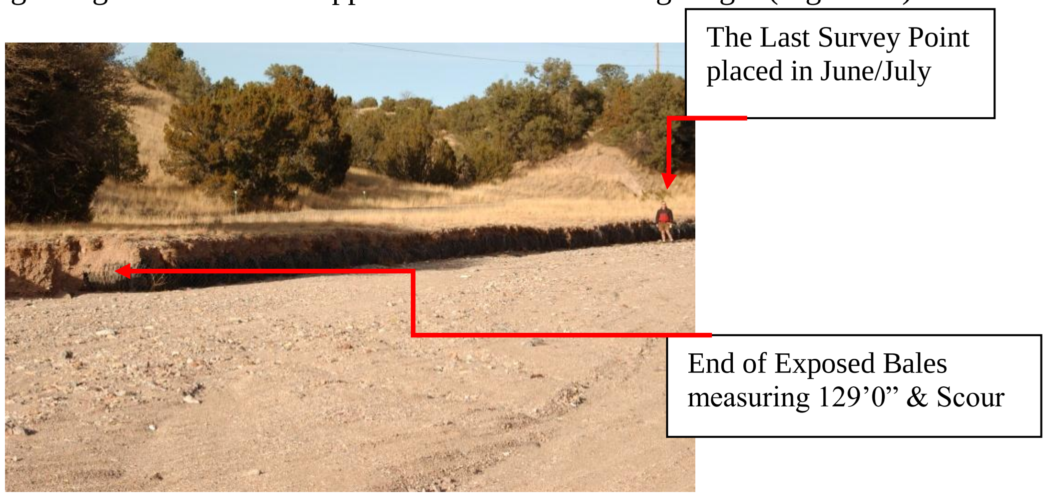


FIGURE 14 Newly Exposed Bales @ NM 52 Mile Post 27.2

2. In addition to the exposed tire-bale structure at milepost 27.2 on December 20, 2008, a scour feature at the end of the structure measured 6 feet by 6 feet by 5 feet and tensile cracks on the upper level of the newly exposed structure followed the back side of the tire-bale structure. The scour appeared to be formed from water flowing over the structure, although the flow was not witnessed. The tensile cracks measured 12 inches long.
   - On July 5, 2010 the scour feature had changed (Figure 16). Tree debris had filled in the scour at the end of the tire-bale structure while increased scour occurred beyond the end of structure scour. This second scour was also present at each visit but also had increased in size. The tree debris appeared to come from the opposite side of the roadway where some clearing against the road way had occurred. Whether it was placed intentionally or by natural forces was unclear.
   - On July 5, 2010 the tensile cracking had lengthen while the width appears unchanged. The tensile cracking is not a continuous feature in length or depth, while there are obvious breaks in the feature there are also subtle erosion patterns in those breaks. Figure 18 shows the photographic representation of the tensile cracks while Figure 17 shows the analytical representation of the tensile crack location above the tire-bale structure. The measurements taken with the Garmin estimate the tensile cracking length at 88 feet long with a maximum depth of 13 inches. The cracking begins

approximately 14 feet from the end of the tire-bale structure and varying in width from the tire-bale edge to approximately 34 feet.

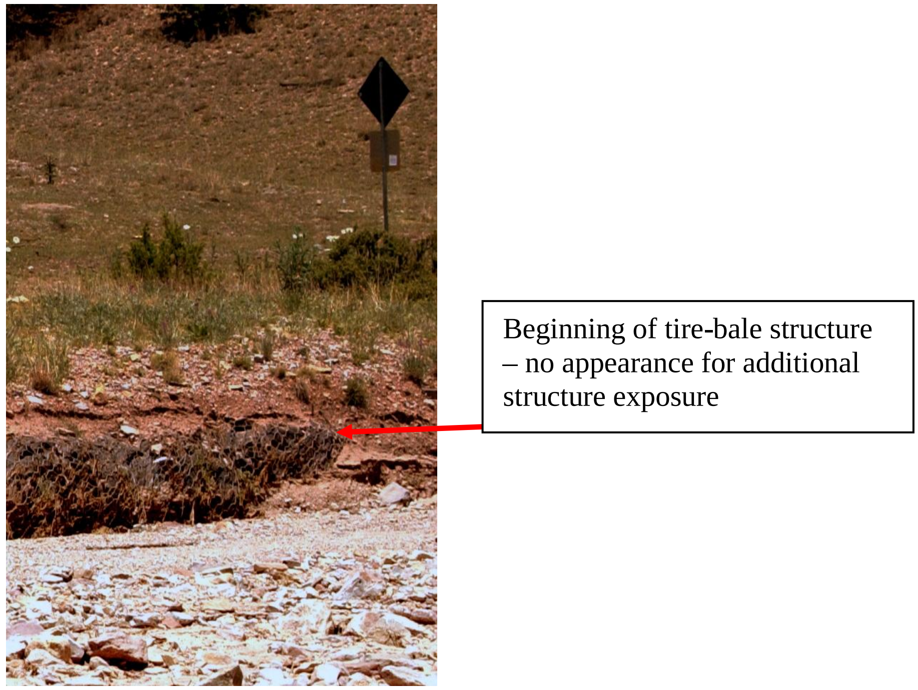


FIGURE 15 Additional 5 feet (NM52 MP27.2)

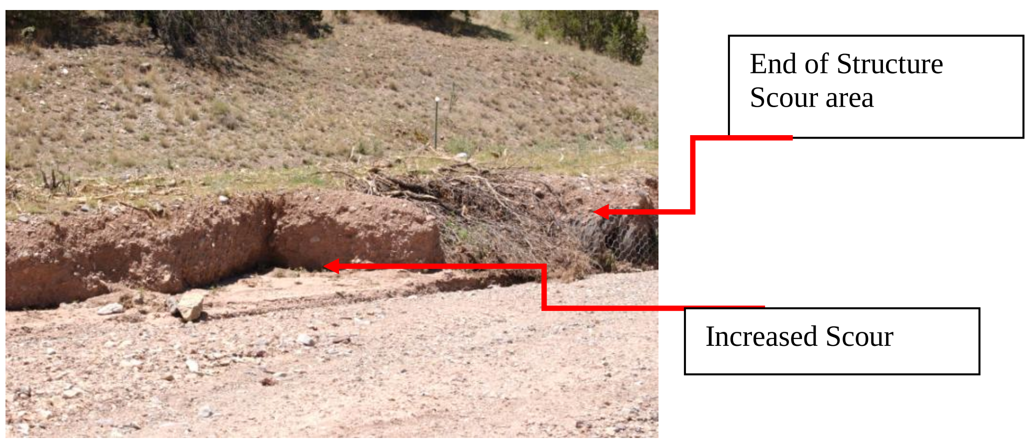


FIGURE 16 Scour Change MP 27.2

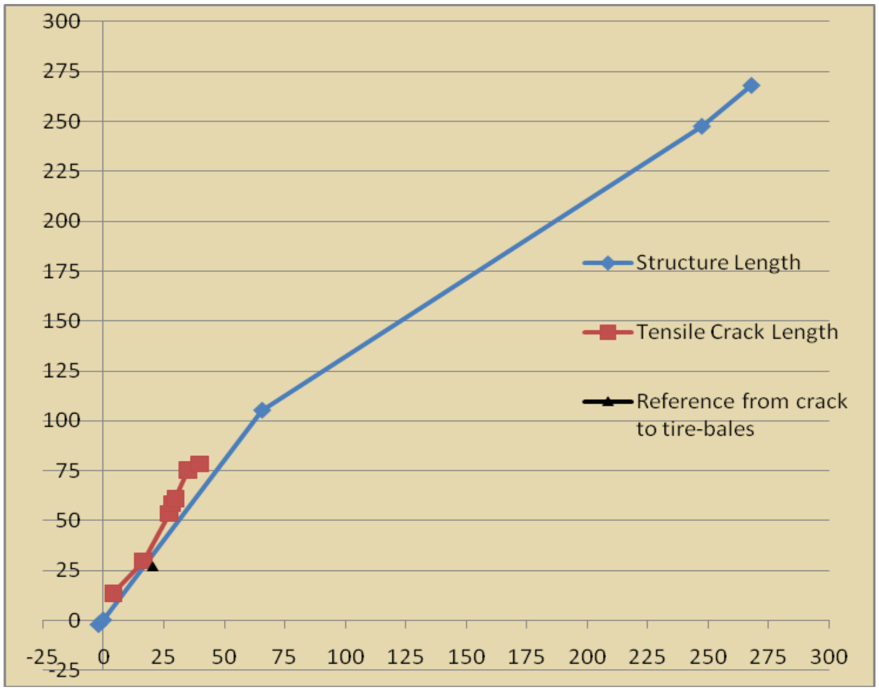


FIGURE 17 Representation of Tensile Cracks above Tire-bale Structure MP 27.2

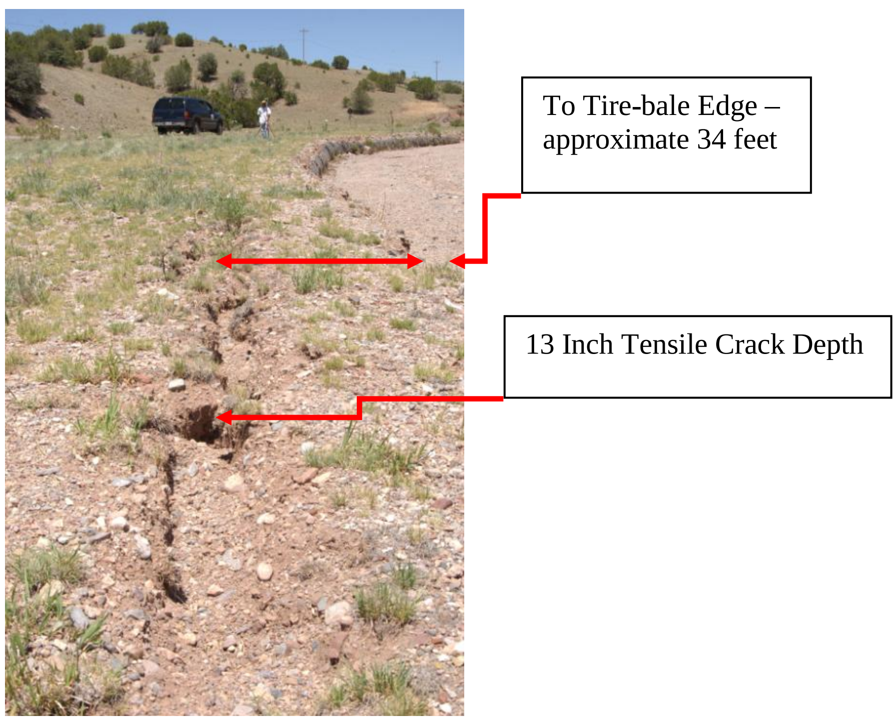


FIGURE 18 Tensile Crack with 13 inch Depth MP 27.2

3. On December 20, 2008 milepost 26.7 had measurable areas of erosion at both ends of the structure with tire-bale exposure.
   - At the beginning of the structure an erosion feature measured 47 feet long and 4 feet 6 inches deep with a varying width (Figure 19). This feature appeared to be formed as water flowed over the structure and the streambed. Two (2) tire-bales on the upstream side of the structure which are also located on the north end of the culvert are entirely exposed.
   - At the end of the structure a scour feature measured 12 feet long with varying width and depth (Figure 20). Again appears to be formed by water flow over the structure.

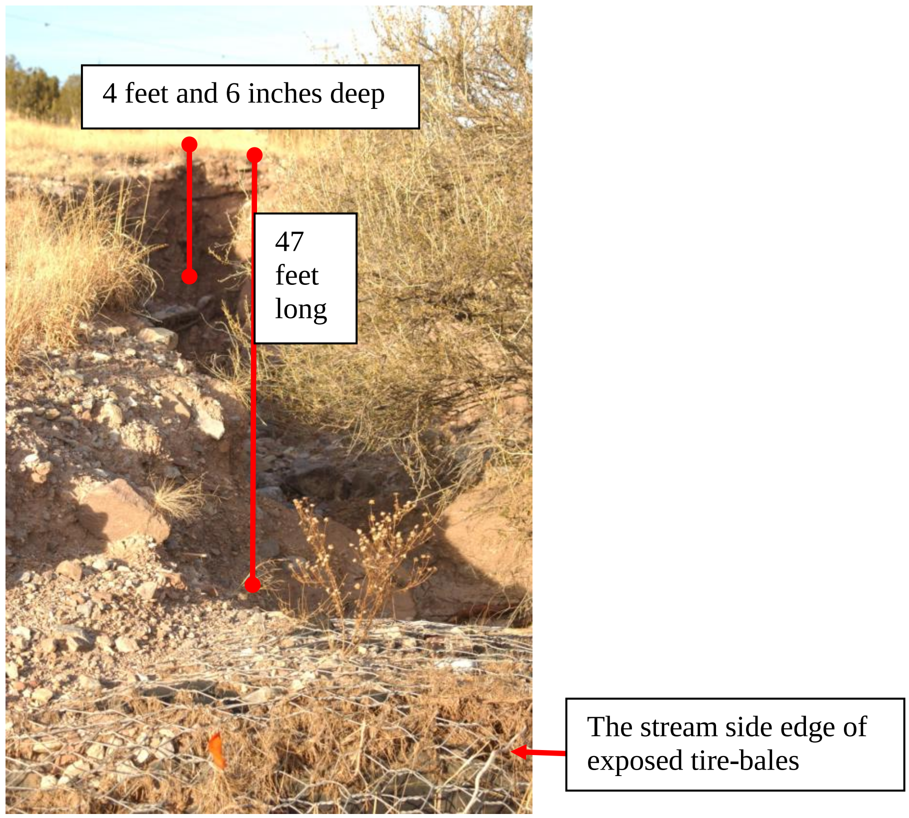


FIGURE 19 Erosion Feature 47 feet long MP 26.7

FIGURE 20 Scour at End of the Structure Mile Post 26.7

- Additionally, the structure has a culvert with tire-bales lining the bottom of the outflow area of the culvert and walls on either side of the outflow lining. The culvert had erosion on either side that stretched behind the culvert toward the roadway. The erosion features measured 8 feet 2 inches from the mouth of the culvert toward the road (Figure 21).
- Finally, 29 feet from the end of the structure there was 40 feet of exposed upper level tire-bales (Figure 22).
- On July 5, 2010 the erosion feature that appeared most changed was the one at the beginning of the structure. The length is approximately 93 feet long however the beginning of the depth of the feature begins at 47 feet. The length change had similarity to that of the tensile cracking occurring at milepost 27.5. The length change of the feature shows shallow erosion pattern moving toward the drop off into the 47 feet long feature. Capturing the photographic intricacies was not possible.

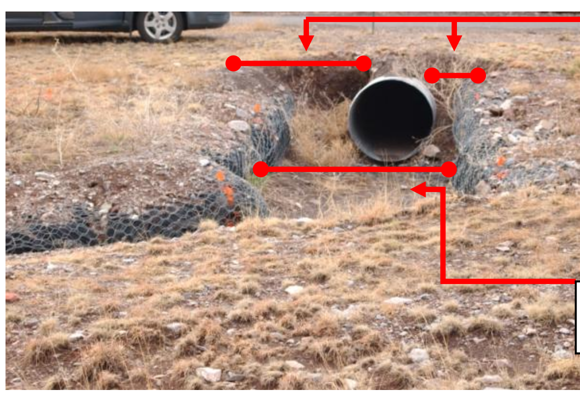


FIGURE 21 Culvert at MP 26.7

FIGURE 22 40 feet of upper level exposure MP 26.7

4. On December 2008 at milepost 27.75, had two observed erosion features.
   - One was observed erosion on June/July 2008 above a culvert. The erosion had continued on the upper level of the culvert measuring 22 feet 6 inches by 22 feet 6 inches. It had formed on either side of the culvert and was beginning to join behind the tire-bale structure (Figure 23) Arrows 1 and 2 indicate the two erosion patterns on either side of the culvert.
     - It is noteworthy to compare the culvert design between milepost 27.75 and milepost 26.7. In the case of milepost 27.75 the culvert is at the base of the tire-bale structure. In the case of milepost 26.7 (Figure 21) the culvert is on the upper level of the tire-bale structure. It appears neither design has improved the water flow enough to prevent erosion around the culvert.
   - An additional 21 feet of exposed tire-bales were visible at the beginning of the structure that was not exposed on the June/July 2008 visit and survey point placements (Figure 25).
   - On July 5, 2010 the culvert erosion appeared visually to have varied slightly. Figure 26 is an analytical representation of the culverts position on the upper level of the tire-bale structure. The erosion pattern measured 21 feet 6 inches by 16 feet (Figure 24). Again the measurements were taken with the Garmin so the accuracy is within limits, so it can be conclude that the culvert erosion was unchanged. It should be stated that latitude and longitude were not used in the analysis.
   - Additionally, it was observed that the culvert is beginning to fill-in with sediment (Figure 24), a potential maintenance necessary for continued usefulness of the culvert.

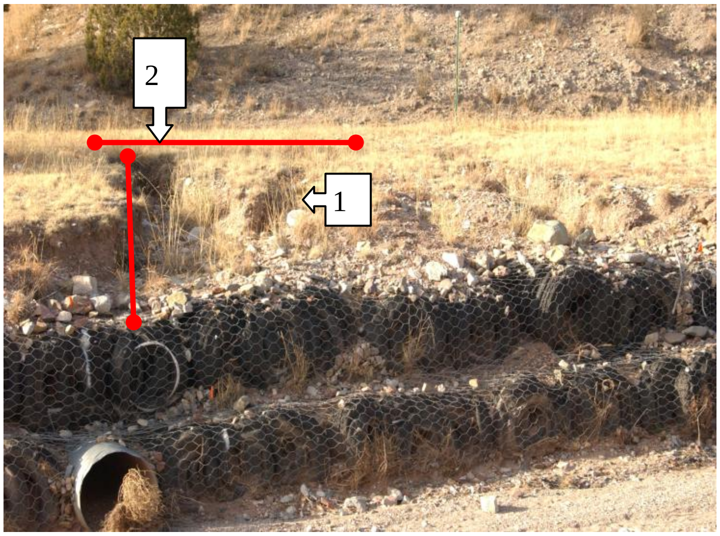


FIGURE 23 Culvert Upper Level Erosion – December 2008 MP 27.75

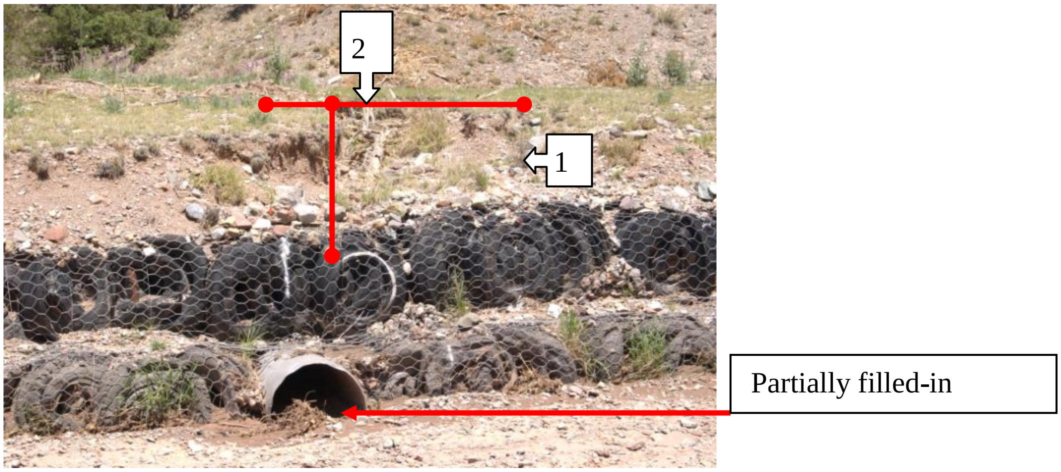


FIGURE 24 Culvert Upper Level Erosion - July 5, 2010 at MP 27.75

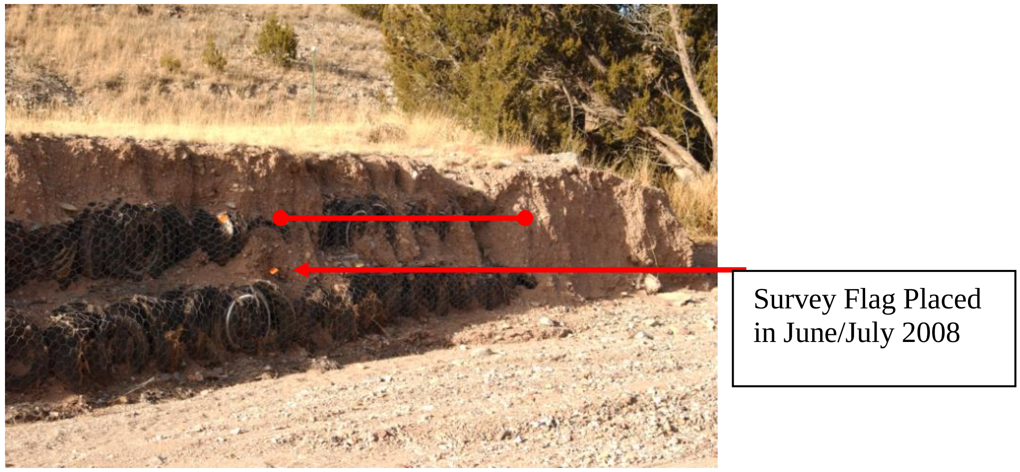


FIGURE 25 21 Feet of Newly Exposed Tire-bales December 2008 MP 27.75

2.2.4 Soil Retention

As was stated in the above section, NM52 at milepost 27.75 does have soil retention around the culvert; although there was substantial soil retention at the culvert when the last survey done on July 5, 2010 all lower level survey points were located. As Figure 24 shows the lower level tire-bales are partially covered approximately half their depth in comparison to their exposure in Figure 23 on December 2008. It needs to be noted that milepost 27.75 is on an incline so the last 125 feet (Figure 26) has no the lower level tire-bales with soil retention.

At NM52 milepost 27.2 the opposite observation occurred. The lower level tire-bales have complete soil retention, so much so that the survey team was only able to obtain two rows of readings in comparison to milepost 27.75 where all three rows were obtained. The difference between the two sites is milepost 27.2 has no incline (Figure 27).

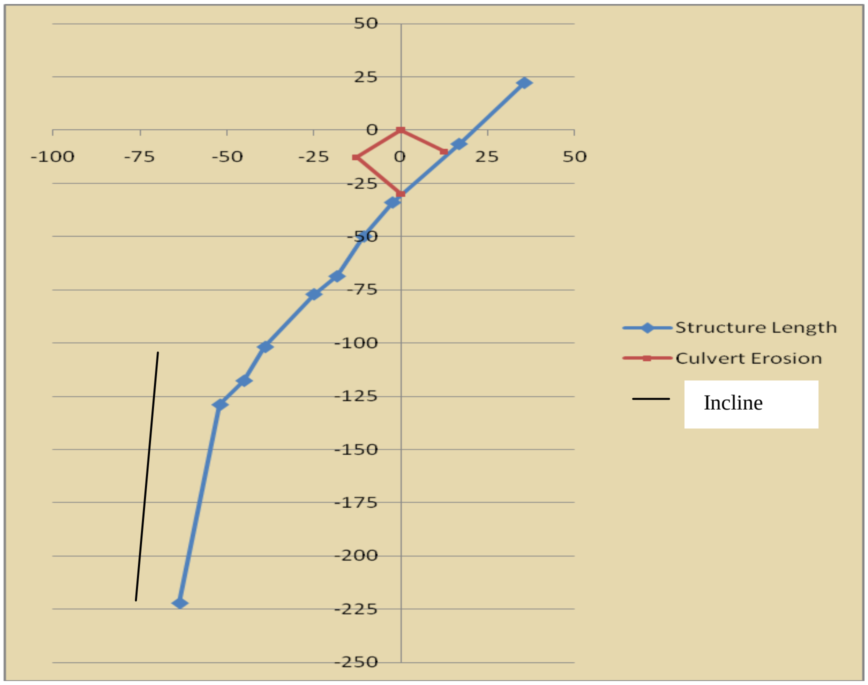


FIGURE 26 Representation of Culvert Erosion above Tire-bale Structure MP 27.75

The NM52 milepost 26.7 appeared to have some soil retention at the beginning of the structure near the culvert outflow into the arroyo. While at the end and middle of the structure a decrease in soil retention seemed to occur. Similar to NM52 milepost 27.75, milepost 26.2 is on an incline although milepost 26.2 is a less steep incline. So the soil retention accumulated at the upstream end of the structure. The configuration of milepost 26.7 site makes it difficult to get the entire structure in the photographic frame. However, the area of soil retention is located on the south end of the culvert (indicated by the arrow in Figure 28). The distance is unknown since there was no precise information or data to determine the location of the tire-bales. Parts of the structure were not surveyed since the tire-bales were not visible during survey point placement.

In regard to the NM152 site, still have similar amounts of soil on the downstream side of the structure as was present at the beginning of the project. Mostly the soil settles on the opposite of the road after it passes through a culvert. The addition of sediment was not monitored. Soil retention at the upstream side is unchanged.

FIGURE 27 NM52 MP 27.2 Lower Level Covered

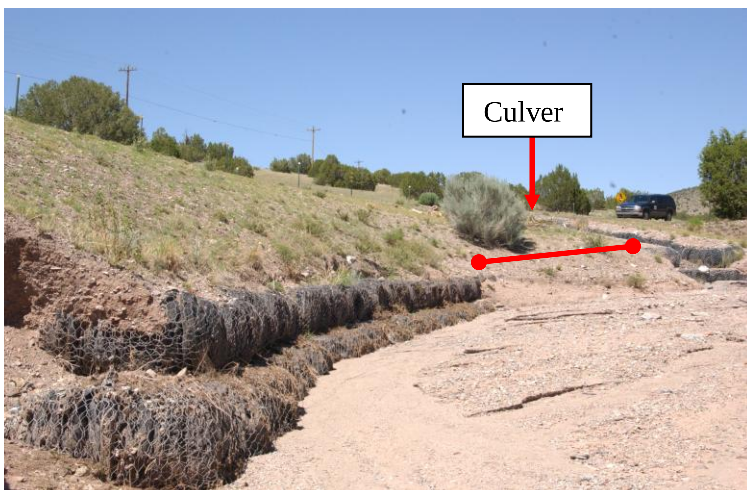


FIGURE 28 NM52 MP 26.7 Soil Retention

2.2.5 Vegetation and Habitat

As can be seen in all the above figures of the in-field sites vegetation growth has increased over the last two years. There are even larger rooting plants taking hold of the soil in the erosion areas but mostly ground cover vegetation has taken root (i.e. grasses and some flowering plants). Additionally, lizards and spiders have been observed moving in and out of the structures. There appeared to be no large burrows or larger animals or reptiles living in the structures. There was evidence of cows, rabbits, deer or antelope moving in the vicinity of the structures.

On May 4, 2010 the State of New Mexico Department of Game and Fish visited all four sites. After the visit the Project Manager at NMDOT, Virgil Valdez received a letter from Matt Wunder, Ph.D.; the Chief of Conservation Services Division dated May 12, 2010. The document is presented in Appendix D of this report.

The following is a summary of the document that states concerns of the Department and request for future involvement.

1. "The Department supports the research being conducted to determine the appropriateness and effectiveness of compress used tire bales for bank stabilization and erosion control in ephemeral arroyo situations, but not in intermittent or perennial stream situations."

2. There is concern for "the long-term reliability" as there was a failure on the Gila River at one time.

3. Another concern is for "the potential for downstream sedimentation from construction activities."

4. There is a request for future involvement in the planning process even though "NMDOT installs these structures using a categorical exclusion under NEPA." The reasoning for the request is to "provide NMDOT with site specific information regarding potential effects on state-listed species…and recommendations for possible mitigation strategies."

The letter made no specific references to the conditions of the in-field sites.

2.2.6 Recommendations

Environmental recommendations based on the literature review are provided in the Best Practices Table 1 below. The Best Practices Table is a general use table; however the design used at the field demonstration facility (FDF) utilizes some changes listed below.

1. The overall size of the structure was decreased. It is recommended that one structure be no more than 20 feet long and 3 tire-bales high.

2. The use of a Geomembrane to divert water from the top level and sides of the each structure with a length of 20 feet.

3. Treated soil was placed on the top level of the structure. Doing so omitted the need for the soil layering and the filtration bed. Due to New Mexico's arid conditions and the location of the FDF structures in ephemeral arroyos, our design methodology can be used. Other states may need to use soil layering and/or a filtration bed.

4. It is suggested that berms be placed to direct water flow. The placement of the berms could assist in reduce erosion from behind or on the edges of the structure. The berm type can be earthen or man-made materials depending on the allowable space.

Additionally, a maintenance plan for the tire-bale structures was developed based on monitoring of the field demonstration facility over time. However, preliminary recommendations are presented below based on observations of the in-field sites (Table 2). These will evidently be revisited as data resulting from the field demonstration facility become available.

To address problems as soon as they occur and prevent further damage to the structures and the environment, site visits for evaluation of existing structures are recommended as follows:

1. Monthly visits should be conducted from the beginning of the snowmelt period (usually April) to the end of the raining season (usually September);
2. Site visits are also recommended after each severe rain event;
3. Finally, if possible, a site visit should be conducted before the snowmelt period to address any damage that may have happened during the fall and winter months.

During these site visits, evaluators should conduct visual inspections of the entire structure, documenting its condition and paying special attention to the following modes of distress: loose tires, corrosion of baling wires and/or gabion wires, undercutting of tire-bales at the beginning, in middle or the end of the wall, and the presence of tension cracks behind the walls (usually on the upper level of the structure). Probable causes and recommended actions for each mode of distress mentioned are presented in Table 2.

3.0 TASK 3:

"The research team conducted a life cycle cost analysis (LCCA) of alternative methods using generally accepted practice for performing LCCA in accordance with FHWA guidelines. LCCA will be performed in consultation with the LCCA Program Manager in FHWA's Office of Asset Management *(1)*."

3.1 Life Cycle Cost Analysis of Alternative Methods

A preliminary life cycle cost analysis (LCCA) was performed on alternative methods. As the field demonstration facility is constructed and monitored, additional information will be collected regarding initial costs, maintenance costs, and rehabilitation costs to be incorporated in a more detailed analysis. The purpose of this preliminary cost analysis was to obtain a rough idea of the difference in costs between the more traditional gabion baskets approach and the tire-bales alternative. Although several contractors were contacted via email and phone, no information could be obtained on initial costs, rehabilitation or maintenance requirements of the different alternatives; these will be further investigated in the field demonstration facility.

TABLE 1 Best Practices for Tire-Bale.

Best Practice	Avoid	Why	Recommendations
Create a filtration bed	Placing tire-bales on native soil	Toxics may leach from wet soil with certain levels of pH value. It has also been observed that iron and manganese concentrations increase and migrate away from the installation site - Humphrey, Swett, 2006.	Create a filtration bed below the tire-bale structure. In addition extend that filtration bed into the stream bed and downstream from the tire-bale structure. This will help insure water filtration from the tire-bale structure for downstream water absorption. This could include small stone gabion baskets placed at the end of the structure and at the base level of the structure.
Create appropriate drainage for the area and soil conditions	Placing tire-bale structure without consideration of upstream contributors, soil conditions, erosion pathways, etc.	Water can accumulate behind the structure and cause non-reinforced areas to fail. It can also cause the structure itself to fail.	Use geosynthetic products including geodrains, geosynthetic-wrap around revetments, and drainage products to divert water from eroding soil from the back side of the tire-bale structure or allowing water to accumulate behind the structure.
Create Soil Layering between tire-bale rows	Placing one tire-bale row on top of another with no soil layering	Minimizes the flow of oxygen and water through the structure and decreases exothermic reactions and water retention - Zornberg, LaRocque, 2005. As well decreasing upper level erosion that causes tensile cracks.	Use a 12 inch thick [layer of] a cohesionless material such as sand or manufactured stone sand to cushion between successive layers of tire-bales and a spacing of at least 6 inches [between each bale side by side] for the placement of a soil filler material. The purpose is to increase compaction using vibratory rollers. - Zornberg, LaRocque, 2005. The soil layering fills the voids in the tire-bales and creates an interlocking affect between the rows of tire-bales.

TABLE 1 (Continued) Best Practices for Tire-Bale.

Best Practice	Avoid	Why	Recommendations
Place Tire-Bales in Open Air Storage	Placing Tire-Bales in Closed Air or Partial Closed Air Storage	Fire out-gassing gets caught in any roofed or completely closed structure thus contributing to combustion and increasing fire personnel safety. In 2008 – 2009 it was speculated that open air storage allows the gases to dissipate from the combustion area.	If it is necessary to store tire-bales in a closed structure or partially closed air structure local and state fire authorities need to be notified with location of the storage and how much is being stored. Additionally, the same authorities need to be notified when the storage doesn't exist.
Follow New Mexico Recycle, Illegal Dumping and Scrap Tire Management Rule and Follow International Fire Code Chapters 5 & 25	The cost in fees and time for noncompliance.	Public and employee safety, as well as environmental and emergency response personnel..	New Mexico Recycle, Illegal Dumping and Scrap Tire Management Rule sections 20.9.20.36, 20.9.20.37, 20.9.20.38, 20.9.20.40, 20.9.20.41, 20.9.20.42, 20.9.20.48(B, E, F, G). International Fire Code Chapters 5 & 25 where applicable to the installation.

TABLE 2 Distress Modes of Tire Bale Structures used for Erosion Control.

Observed Distress	Probable Cause	Recommended Actions
Loose tires in drainage basin	Loss of integrity of bales due to breakage of baling wire and gabion wire wrap (where present)	Collect loose tires, 'fix' ruptured bales by repairing wire wrap or pouring high w/c concrete into bale, perform local repair on gabion wire. Covering of tire-bale structure with geomembrane and treated soil may also be considered.
Broken baling wire on individual tire-bale	Corrosion of baling wire; shifting in structure creating local stress	Replace baling wire if possible; stabilize bale with high w/c concrete if necessary; monitor structure in region of bale for possible deformation. Covering of tire-bale structure with geomembrane and treated soil may also be considered.
Broken gabion wire	Corrosion of gabion wire; debris impact from flood event; deformation of tire-bale structure	Repair gabion wire, and monitor structure in that area for possible deformation. Covering of tire-bale structure with geomembrane and treated soil may also be considered.
Undercutting of bales in middle of wall	High-velocity stream flow	If undercutting is minor, fill with native soil/gravel. If it is serious, excavate and place another row of tire-bales. Placement of riprap at the toe of the system may also be necessary to prevent future problems.
Undercutting of bales at upstream end of wall	Poor choice of location/method of tire-bale structure termination	Rebuild upstream end of wall to incorporate deeper layer of bales and wing wall tying into original grade.
Tension cracking in soil behind wall	Settling of poorly-compacted fill; translation of wall	Excavate fill, and re-compact. Monitor wall in that area for possible movement. Covering of tire-bale structure with geomembrane and treated soil may also be considered.
Shifting of one row of bales with respect to the row immediately below	Rotation of wall due to either undercutting below base course of bales, or buildup of hydrostatic pressure behind structure.	Investigate the condition of the base course; check soil behind structure for evidence of increased water flow (surface or subsurface).

In the meantime, cost estimates were obtained from the literature review and from the 2008 RS Means Site Work and Landscape Cost Data *(57)*. In this preliminary analysis, excavation, compaction, backfilling, and preparation of foundation soils were considered approximately equal for both alternatives and therefore costs associated with these activities were not considered in the comparison. The purpose of Figures 29, 30, 31, and 32 are simply to indicate the size of gabion baskets and the configuration considered in the cost analysis, hence the rudimentary sketches that did not include details on backfill or foundation. More detailed figures could have been provided, but the intent of the preliminary analysis was simply to determine whether there would be a significant difference in costs between the two alternatives, using the very limited cost data available at this time. With regards to the units used in the analysis, these can easily be converted to cost per linear foot of construction if needed.

A life-cycle cost analysis (LCCA) is a comparison of costs incurred over time during the design life of a structure. Design alternatives can be compared by setting the future costs of each alternative in terms of present-day dollars to provide a platform for comparison.

Included in a LCCA are consideration of construction, rehabilitation, and maintenance costs to the funding agency and user costs during construction procedures and normal operations. This type of analysis does not include differences in benefits to users resulting from the project or costs due to unforeseen events, such as reconstruction costs due to failure of a structure. These types of "unforeseen" costs are referred to as "externalities" by the FHWA LCCA Primer *(58)*.

When conducting a LCCA, the more specific the set of design requirements, the more accurate the results. For example, the location of the project may determine the most economical alternative, depending on local availability of construction materials. It should therefore be noted that the analysis performed here is for a general construction site in the state of New Mexico, and is not necessarily applicable to all soil stabilization projects.

There are five steps outlined in the FHWA LCCA process *(58)*:

- Establish design alternatives
- Determine activity timing
- Estimate costs (agency and user)
- Compute life-cycle costs
- Analyze the results

Each of these steps will be considered separately in this report.

3.2 Establish Design Alternatives

This analysis considers a typical section of channel bank 200-ft long. Heights representative of the two types of structures currently encountered in the field: 5 feet and 7.5 feet are analyzed. Both slopes considered will be steep (face angle > 45°), as is typical for an arroyo. The steepness of the slope severely limits the number of feasible erosion control and slope stabilization options, prohibiting the use of many channel design alternatives, such as riprap. Additionally, although it can generally be assumed that a concrete lining would be adequate for these slopes, the costs associated are much greater than either gabions or tire-bales. As such, concrete will not be included in the analysis, leaving gabion baskets and tire-bales as the most attractive alternatives for these applications. Important to this analysis is the fact that each bank will experience significant erosional forces during its lifetime; in the case of slopes not located in arroyos, a different LCCA will be necessary.

For both alternatives selected, excavation, compaction, and backfilling are considered to be approximately equal; these costs are therefore not considered.

3.3. Determine Activity Timing

Each alternative will have a different design life and will require rehabilitation and maintenance at different time intervals. For example, one design alternative may be expected to last ten years with little to no maintenance, and require maintenance every five years thereafter, whereas another alternative may not require regular maintenance, but may need to be rehabilitated after twenty years. These timing estimates are important for establishing present value differences among the alternatives.

Although several contractors were contacted via email and phone, no information regarding rehabilitation or maintenance requirements was obtained for the alternatives considered. Maintenance and rehabilitation requirements for each alternative will therefore be further investigated in the field demonstration facility.

3.4 Estimate Costs (Agency and User)

Costs are generally divided into two groups: the costs to the funding agency, and the costs to the user. User costs are generally associated with traffic delays due to lane/road closure, etc. In this analysis, it will be assumed that user cost differences among the alternatives are minimal since construction is not on the road itself, but off to the side, or in the drainage area. User benefits will also be similar for each alternative, as they all aim to accomplish the same result.

Agency costs include the initial construction costs, and any maintenance costs incurred during the design life of the structure. For each design alternative, the cost of maintaining the structure at predetermined intervals must be estimated. These costs will be discounted with a real discount rate; "prevailing rate of interest on borrowed funds, less inflation" *(58)*. Because a real discount rate will be used, it will not be necessary to adjust for inflation when estimating future maintenance costs.

It should be noted that it is not necessary to consider costs that will be necessary to all design alternatives; that is, if drafting costs for each design are approximately the same, they do not need to be included in the comparison.

Environmental impacts will not be considered in this analysis since an LCCA does not provide for user benefits resulting from a project. A benefit-cost analysis (BCA) would be necessary if user/environmental benefits were to be considered in an economic study. It is however important to note that quantification of environmental benefits for each alternative would be highly subjective.

Initial costs associated with each design alternative (gabion baskets and tire-bales) are detailed below. These are preliminary costs, and may be updated as more information becomes available. The 2008 RS Means Site Work and Landscape Cost Data *(57)* were employed to estimate initial costs.

3.5 Alternatives Gabion Baskets and Tire-Bales

3.5.1 Alternative 1: Gabion Baskets

Costs obtained from the 2008 RS Means Site Work and Landscape Cost Data *(57)* include national averages for shipping, material costs, labor costs, and equipment costs. All costs will be adjusted for the state of New Mexico using the average city cost index for the state, 88.56 *(57)*.

5-ft Gabion Wall Since the base of a typical gabion wall needs to be between 0.5 and 0.7 times its height *(59)*, the base of this 5 ft wall should be between 2.5 ft and 3.5 ft.

The 2008 RS Means Site Work and Landscape Cost Data *(57)* provided cost data for three gabion heights: 12 in, 18 in, and 36 in. All baskets are 3 ft in depths, and the most cost-efficient length listed is 6 ft. Thus, a reasonable design for a wall with no additional vertical pressure at the top of the wall (no surcharge) would be as presented in Figure 29:

Since a 200 ft bank section would require 33 gabion baskets 6-ft long, the total number of baskets required of each size will be as follows:

- 33 baskets 36” x 36” x 6’ (RS Means line item # 32 32 36.10 4490)
- 66 baskets 12” x 36” x 6’ units (RS Means line item # 32 32 36.10 4340)

Material, labor, and equipment costs (national average): $17,750.70 (R.S. Means 2008)
New Mexico adjusted cost for this structure: $15,720.02.

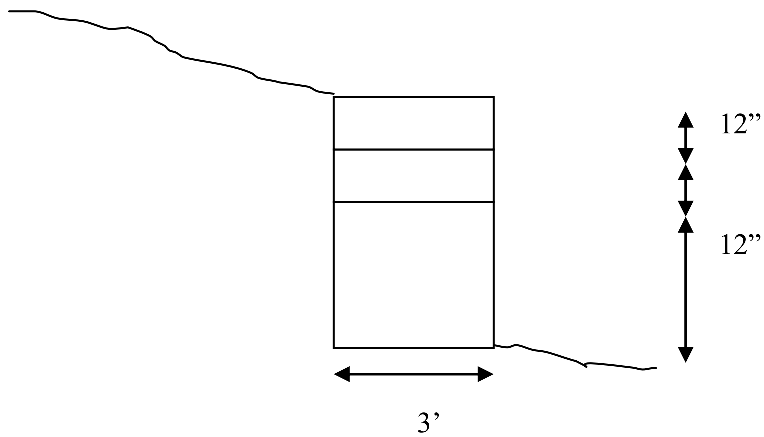


FIGURE 29 5-ft Gabion Wall

It is important to remind that in the preliminary analysis, excavation, compaction, backfilling, and preparation of foundation soils were considered approximately equal for both the gabion wall and tire-bale wall and therefore costs associated with these activities are not considered in the comparison. In addition the purpose of the figures presented in this section (Figures 29 to 32) were simply to indicate the size of gabion baskets and the configuration considered in the cost analysis, hence the rudimentary sketches do not include details on backfill or foundation.

7.5-ft Gabion Wall In this case, to maintain the required base length 0.5 and 0.7 times its height *(59)*, the base of this wall will need to be between 3.75 ft and 5.25 ft. Using the basket sizes previously specified this wall will require two baskets at its base. For gabion walls over 6 ft tall, it is typical to step the baskets by 18 in, so the configuration presented in Figure 30 was considered appropriate.

For the 200-ft bank section, the total number of baskets required is as follows:

- 99 baskets 36” x 36” x 6’ (RS Means line item # 32 32 36.10 4490)
- 33 baskets 18” x 36” x 6’ (RS Means line item # 32 32 36.10 4400)

Material, labor, and equipment costs (national average): $39,573.60 *(57)*
New Mexico adjusted cost for this structure: $35,046.38.

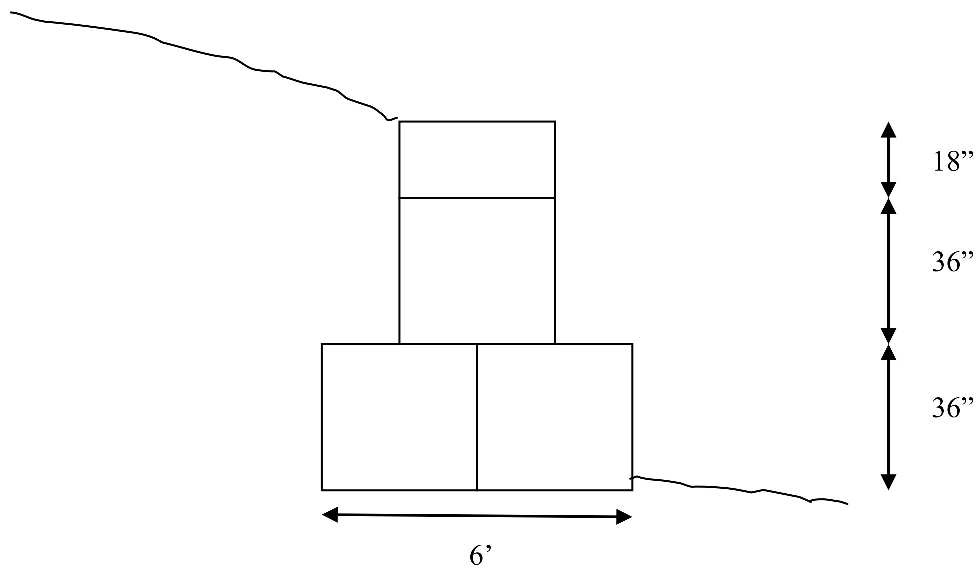


FIGURE 30 7.5-ft Gabion Wall

3.5.2 Alternative 2: Tire-Bales

The tire-bale designs considered in this analysis are similar to the ones currently encountered in New Mexico. They include one row of bales buried completely and a general pyramid shape. In the designs currently used by the New Mexico Department of Transportation, the tire-bales are typically encased in wire mesh and secured with steel stakes spaced at approximately 16-foot intervals and 3/8” steel cable. As future monitoring of the field demonstration facility continues, this section will be updated to reflect final costs.

It is assumed that the cost of scrap tires is zero, since the baling and use of tires in erosion structures may actually cost less than other disposal options for the state *(42)*. Bailing equipment and transportation of the tire-bales will be the most significant material cost. A study conducted in Colorado by Zornberg *(42)* estimated the costs presented in Table 3.

According to the R.S. Means *(57)*, the average city cost index for Colorado is 92.3; and as presented earlier, the average city cost index for New Mexico is 88.56. Also according to R.S. Means *(57)*, the actual 2003 cost index is 130, while the actual January 2008 cost index is 171. These values were used in the adjustments for time and location presented.

TABLE 3 Estimated Costs for Tire-Bale Structures

Item	Estimated Average 2003 Colorado Cost (Zornberg *et al.* 2005) ($/cy)	Estimated Average Adjusted for 2008 Colorado Cost ($/cy)	Estimated 2008 Average, Adjusted for New Mexico Cost ($/cy)
Material Cost of Scrap Tires	0	0	0
Fabrication, Handling, and Storage	11.00	14.50	13.91
Transportation (estimate for 100-mi transport)	5.00	6.58	6.31
On-site Handling, Storage	0.50	0.66	0.63
Class A geotextile between soil and tire-bales	2.00/SY	2.63/SY	2.52/SY
Wire Mesh, materials and labor	---	---	9.00 / SY *
Steel cable, 3/8", materials and labor	---	---	0.80/LF *
Steel Angle 4" x 4" x 3/8", materials and labor	---	---	11.00 / LF *

*preliminary estimate obtained from www.get-a-quote.net

5-ft Tire-Bale Wall In this analysis, each tire-bale will be assumed to be 2.5' tall, 5' wide, and 5' long (on the smaller side of the tire-bale size range). A geotextile will be placed where tire-bales are in contact with the soil; wire mesh will be wrapped around each row, and rows will be tied together. Steel angles and cables will anchor the bottom row of bales, which will be completely buried.

A 5-ft tall (Figure 31), 200-ft long wall will require 300 cubic yards of tire-bales, 28 6-foot steel angles, 400 feet of steel cable, 133 square yards of geotextile, and 350 square yards of wire mesh. Thus, the total New Mexico adjusted cost is estimated at $11,908.20.

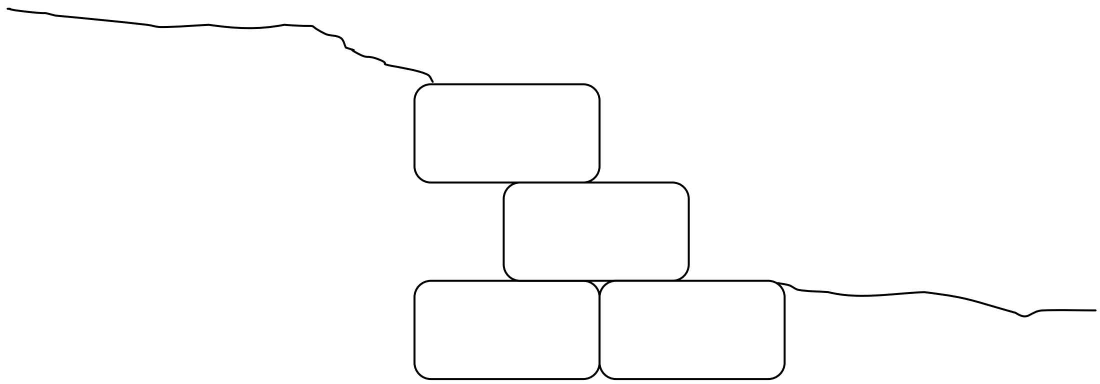


FIGURE 31 5-ft Tire-Bale Wall

7.5-ft Tire-Bale Wall If the same specifications are used for this larger wall (Figure 32), the structure will require 525 cubic yards of tire-bales, 42 6-foot steel angles, 600 feet of steel cable, 181 square yards of geotextile, and 565 square yards of wire mesh, for an estimated New Mexico adjusted cost of $19,739.40.

Preliminary cost estimates therefore indicate that for short (5 to 7ft), steep-bank erosion walls (face angle > 45°) tire-bales are more economical than gabion baskets, at least initially. More detailed initial cost data and maintenance cost data will be included in this preliminary analysis as it becomes available.

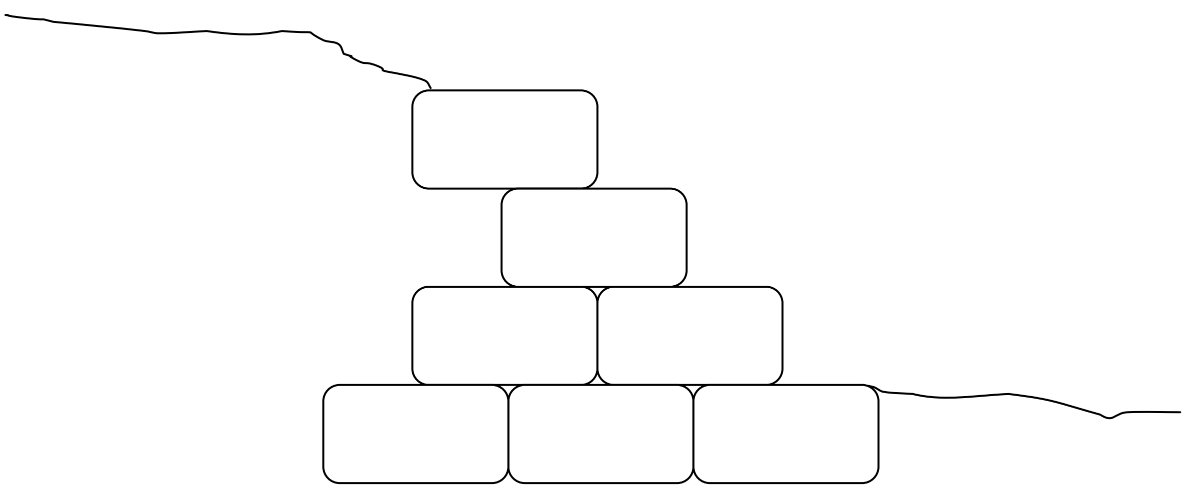


FIGURE 32 7.5-ft Tire-Bale Wall

3.6 Compute Life-Cycle Costs

Costs associated with each alternative need to be considered in terms of present value. The present value of a future cost is given by the following equation:

$$Present\ \ Value = Future\,Value \cdot \frac{1}{(1+r)^n}$$

Where *r* is the real discount rate, and *n* is the "number of years in the future when the cost will be incurred" *(58)***.**

In calculating the present value of the cost associated with each alternative, the same real discount rate should be used. Determination of this discount rate depends on the prevailing borrowing interest rates and inflation; for state and federal construction projects, the rate is typically between 3 and 5 percent.

3.7 Analyze the Results

Generally, the alternative with the lowest present cost (highest present value) is chosen. In circumstances where there are uncertain cost estimates and/or high probability of externalities, a probabilistic approach may be desirable. This type of approach weighs the uncertainty of each alternative against possible outcomes and will require special software.

4.0 TASK 4:

"The research team performed engineering analysis of existing drawings and current construction practice and monitored the performance of existing and under-construction tire-bale systems.

(1) A comprehensive qualitative site assessment was performed to identify and correlate topographical features (erosion features, vegetation patterns, etc.) to the location of the tire-bale structure. The site was mapped on a grid system using global positioning satellite (GPS) equipment..."

(2) Benchmarks and survey points were established to allow quantitative analysis of the performance of the tire-bales system over the life of the project, giving a time-history of subsidence and lateral deformation, along with relevant changes in topography adjacent to the tire-bale system; this was done using angular measurements. Retention of soil and vegetation was mapped, and monitored through the project period, as was the location and extent of any exposed tire-bales. Development of erosion features was also mapped. The data gathered during the monitoring phase was correlated to construction drawings and

photographic documentation during the construction process, to develop a general understanding of how the configuration of the tire-bale assemblages influences wall performance. A set of measurable parameters was developed which will allow rational analysis of performance (*1)*.”

4.1 Engineering Analysis of the Current Constructions

The tire-bale structures on NM-152 and NM-52 (Figure 33) were monitored by surveying to determine displacement and deformation. Several options for accomplishing this were explored; the best method is illustrated in Figure 34. A Topcon total station was placed at a permanent benchmark, and its height established. A second benchmark was then established about 20-ft (6.1 m) away, using a grade rod with a reflector. This was used as the control point for the entire survey. Each individual survey marker on the tire-bale structure is shot from the total station, and its position referenced back to the control point within an x-y-z-coordinate system, accurate to within 0.01 ft (3.05 mm).

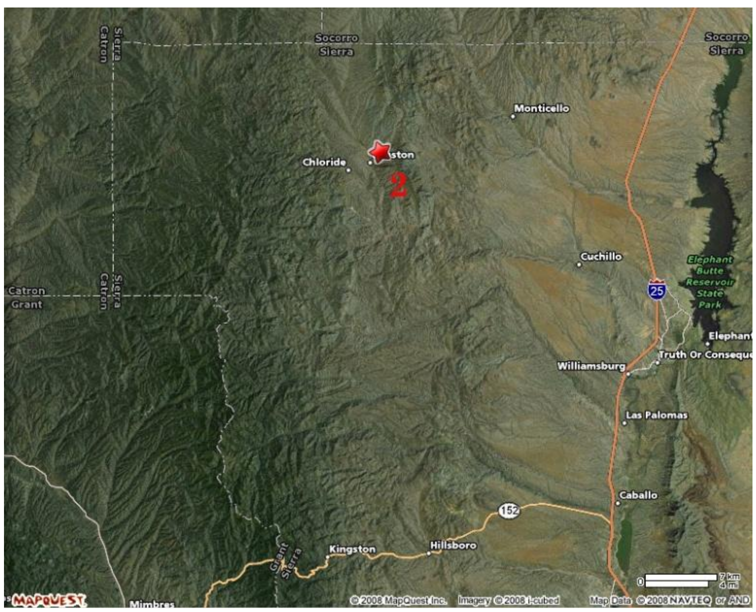


FIGURE 33 Sampling Location In Winston (NM 52) Tire-Bale Construction Site (Map By: Mapquest)

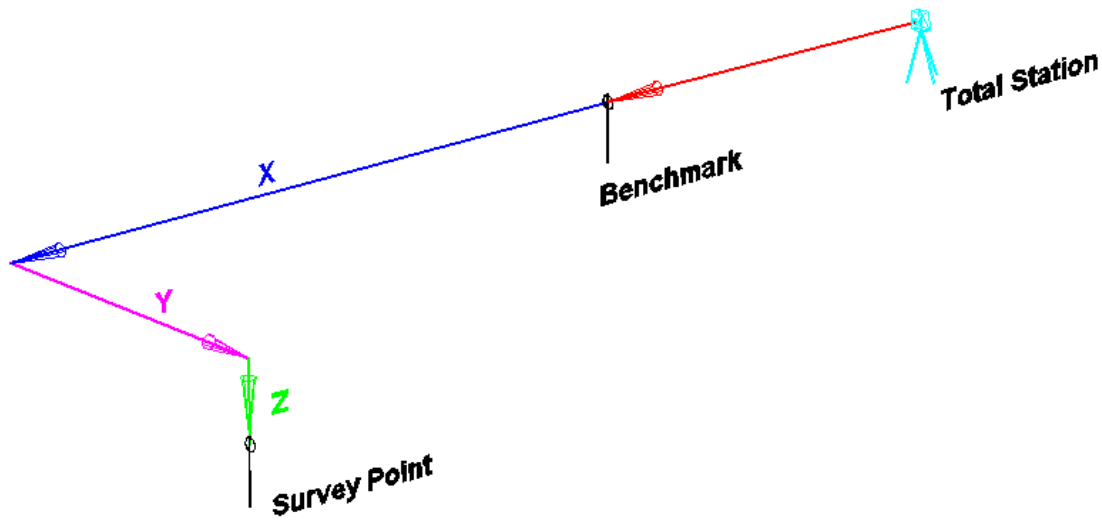


FIGURE 34 Surveying Method

The benchmarks for the total station location and the control point are 4-ft (1.22 m) lengths of #4 (D13) rebar hammered into the ground. A center punch was used to place a precise reference point on the end of the rebar. The center punch was used to locate the total station, and was distinct enough to hold the pointy end of the grade rod.

The survey points are 10-in nails with a 3/8" (9 mm) diameter hammered into holes drilled into individual tires at the desired locations. The same size drill was used to make the holes, using a heavy-duty cordless drill. This provided a snug fit, as when the rubber was drilled it both tore and deformed elastically, so that the actual hole was undersize. A drill bit 6-in long (152 mm) was found to be best. Initial attempts with a 12-in long (304 mm) bit led to rapid battery depletion and frequent bit breakage. Switching to the shorter bit prolonged battery time, and no shorter bits were broken. This was because the longer bit 'wrapped up' more, wasting energy in twisting the longer shank. The long bits also experienced excessive angles of twist within the shank, leading to fatigue failure.

The holes were drilled normal to the surface of the tire, preferably through the tread. It was also desirable to drill through several layers of rubber for stability. The nails were pounded in using a 5 lb. sledge hammer and a great deal of force. The heads were best left standing free of the gabion wire by about 1-in (25.4 mm).

The nails were also center-punched to accept the pointy end of the range pole, and were marked with orange survey flags, wired to the nail heads. This proved invaluable in relocating the nails; they were placed immediately before the monsoon season, and the rains prevented the initial surveys from being accomplished until the Fall 2008. In the intervening time some nails were buried by eroded materials from the slope above, and some were hidden by stream borne debris. It should be noted that once the initial survey was accomplished, the survey point locations were located with precision in reference to the control point, and can be found again without difficulty. Details of the survey point installation are shown in Figures 35 and 36.

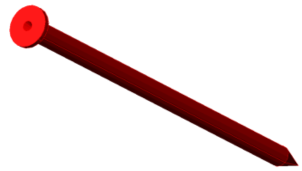


FIGURE 35 Nail Center-punched for grade rod pointy end

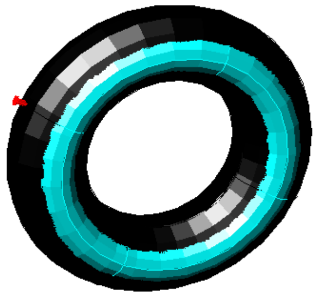


FIGURE 36 Tire-bale with nail as survey point

During the surveying process, the grade rod was held plumb using two sticks as supports. The rodman held the intersection of the rod and the sticks in one or both hands to form a tripod, with the bubble on the grade rod-indicating plumb.

The baseline surveys of the tire-bale wall sites on NM-52 were completed, and the data was used to construct three-dimensional models of the structures. A representation of what was done is shown in Figure 37 below.

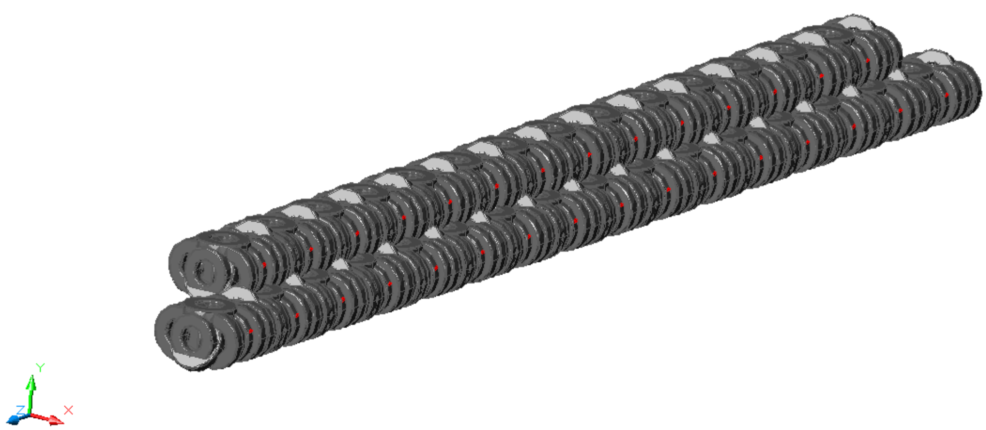


FIGURE 37 Three-dimensional model of tire-bale wall – red dots are survey points

The method used in surveying the structures consisted of first establishing two benchmarks, about 20-ft (6.1 m) apart. The total station was set up over the first benchmark, and a grade rod used to create a master reference point in space at the second benchmark. All survey points were referenced to the master point. The (XYZ) coordinate of the master point was set at (0,0,0), and the position of each point was taken in XYZ space (Figure 38).

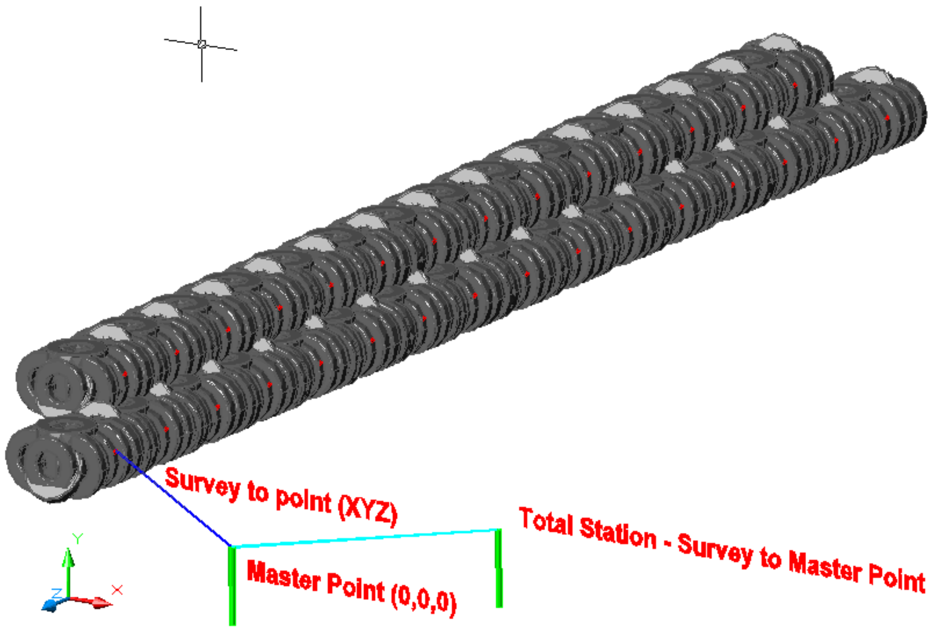


FIGURE 38 Surveying with reference to master point

This process was repeated during subsequent trips to each site; the difference in the (XYZ) position from the baseline reading surveyed for an individual point will indicate movement of the tire-bale structure. Survey points were also placed in the fill behind each structure, so that movement of the fill could be determined. Therefore, both translation and settlement for the tire-bale structures and the fill behind them would be extracted from the field data recorded. It was possible to estimate rotation of the wall in the vertical plane from the relative displacements of the survey points.

The large nails used as survey points have proven to be satisfactory. They are stable, solidly placed, and easy to find again, even when covered by stream alluvium (marking them with orange survey flags was helpful; even when the flags were lost to wind and stream flow the wire was generally easy to locate). Likewise, using #4 rebar as benchmarks worked well; each site was marked with a survey flag and a small stone cairn.

Orange fluorescent paint was used for marking each survey point position on the tire-bales as well as the rock cairns indicating the benchmark locations. Upon return to the in-field sites approximately 6 months later the orange survey flags or the wire that once held the flag could be located. The paint faded and was preserved on the tires out of direct sunlight. The paint applied to the rock cairns degraded and disappeared.

Data collection using the Total Station was achieved followed by development of the contour plots of the construction site along with the neighboring location. Because of this plot, it is tentatively estimated that a catchment area of 500 ft deep and 300 ft wide will allow for drainage through this tire-bale structure. A hilltop is present at south end of the arroyo and may affect the drainage of the arroyo. It is about 110 feet from the edge of the arroyo with an elevation change of 0-12 feet. The lowest point surveyed was approximately -8.3 feet, about 40 feet into the arroyo. The width of the arroyo varies along its length downstream, from 19 - 29 feet for the area mapped.

4.2 Different Monitoring Methods

Several methods present themselves for the monitoring of tire-bale structures. They range from the most basic visual observations to very high-tech (and high dollar) solutions. A method selected for field monitoring should meet a number of criteria:

- It must be accurate and reliable
- It must be physically and analytically robust – that is, the equipment used and its operation must be tolerant of field conditions
- It must be user-friendly, and require a 'practical minimum' of training time
- It must be portable – the most accurate and user-friendly monitoring hardware is of no use if it cannot be field-deployed
- If data is logged, it must be usable in spreadsheet programs that are commonly available.

Table 4 below summarizes the different methods and limitation.

TABLE 4 Methods for Monitoring Tire-Bale Structures used for Erosion Control

Method	Application	Principle of Operation	User Experience Required	Advantages	Limitations
Surveying (transit)	Determine translation and rotation of tire-bale structure	Set survey points and benchmarks, then measure distance from instrument, and vertical offset of points using Philadelphia rod. Change in position over time reflects structural deformation	Experience is required for accurate measurements. Data analysis requires only basic spreadsheet use.	-Use of a transit and Philadelphia rod is the most robust surveying method; there are no batteries to drain -An experienced surveyor can get very accurate results -Surveying gives the best picture of what is happening to the tire-bale structure over time	-Survey points must be placed with care; long nails placed into holes drilled into individual tires provide a stable platform but are difficult to set -Spurious indications of global structural deformation can result from the movement of individual tires within a bale -Margin for error will vary with operator experience, and according to operator, and can mask structural movement -Grade rod must typically be manually steadied by rod person; it can never be held totally steady, leading to error -For best results several readings must be taken and averaged, which is very time-consuming -Data must be manually recorded -Access to suitable benchmarks from which the structure can be surveyed can be difficult for tire-bales structures used for erosion control

TABLE 4 (continued) Methods for Monitoring Tire-Bale Structures used for Erosion Control

Method	Application	Principle of Operation	User Experience Required	Advantages	Limitations
Surveying (total station)	Determine translation and rotation of tire-bale structure	Set survey points and benchmarks, then measure distance from instrument, and vertical offset of points using grade rod with reflector. Change in position over time reflects structural deformation	-Experience is required for accurate measurements, though less than for use if a transit -Data analysis requires basic spreadsheet skills	-Measurements are automated, and repeated measurements for averaging can be taken much more quickly than with transit. -Data can be logged automatically -If permanent reflectors (or mounts) can be emplaced on the structure, the process can be expedited	-Grade rod must typically be held by rod person, and some wobble is likely -Emplacing permanent reflectors or hard mounts is difficult due to the nature of tire-bales -Battery life is limited, and it is possible to run out while surveying a large structure -Requires at least two people in the field -Time-consuming, though not as bad as using a transit -Access to suitable benchmarks from which the structure can be surveyed can be difficult

TABLE 4 (continued) Methods for Monitoring Tire-Bale Structures used for Erosion Control

Method	Application	Principle of Operation	User Experience Required	Advantages	Limitations
GPS	Determine translation in a tire-bale structure	Set markers similar to survey points, and take GPS readings at each point over time to determine displacement over time	GPS is becoming more user-friendly, so the level of experience needed is dropping. Experience needed for data analysis is low, as one merely needs to compare one set of coordinates with another taken at a different time.	GPS can provide an absolute measurement of displacement very quickly, and the method should require only one operator in the field. Access is simple, as the units are quite small.	Current GPS resolution is on the order of 12-in, which is too coarse to be of value in recording anything below extreme distress in a structure. It is claimed that 4-in resolution is coming, but this will still not be fine enough to effectively monitor the health of a structure in anything other than the 'extreme-distress' case.
Inclinometer (digital level, or 'smart level')	Determine rotation at specified 'stations' in a tire-bale structure	Set measurement points on at the same station on different tire-bale 'rows', and measure the inclination between them. Any change over time will indicate wall rotation at that station	Low operator skill required in field, low skill required for data analysis	-Very easy to use -Only one operator needed in field -Inexpensive equipment -Field readings are take little time -Wall rotation is probably the best indicator of distress	-It can be very hard to locate appropriate positions for markers such that they will remain stable over time. -Deformation (i.e., shifting of tires) within an individual bale can give spurious results

TABLE 4 (continued) Methods for Monitoring Tire-Bale Structures used for Erosion Control

Method	Application	Principle of Operation	User Experience Required	Advantages	Limitations
LIDAR (Light Detection and Ranging)	Construct a 3-D image of a tire-bale structure	Reflections of emitted energy in short wavelengths is used to map a structure. The concept is similar to radar; however, as much shorter wavelengths are used the resolution is better (theoretically comparable to the wavelength used)	High skill level needed	Potentially very accurate, and the best way to create a 'virtual database' of civil structures	Very expensive

TABLE 4 (continued) Methods for Monitoring Tire-Bale Structures used for Erosion Control

Method	Application	Principle of Operation	User Experience Required	Advantages	Limitations
GIS information	Determine translation of a tire-bale structure	Comparison of satellite imagery over time is used to monitor structures to determine distress	High skill in interpreting imagery is needed	-Theoretically no field visits are needed -Remote structures, or those whose access is limited by being on reservation land, can be monitored easily	-Resolution is not sufficient to determine low levels of distress -Time between imagery updates is not predictable enough, and is not controlled by DOT
Field distance measurement	Determine relative linear motion between points in a tire-bale structure	Set permanent markers on the structure, and use a steel tape measure to determine the distance between them, and changes in that distance over time	Careful, methodical work is needed to avoid distortion of the tape during the measurements, and temperature corrections (for tape expansion/contraction) must be used. Skill level is low, but 'care level' is high	Very intuitive, very quick to perform, can give immediate and fairly high-resolution results.	Points must be fairly close together to avoid tape distortion, so local effects will predominate, and must be compensated by taking a large number of measurements. At least two field operators needed.

TABLE 4 (continued) Methods for Monitoring Tire-Bale Structures used for Erosion Control

Method	Application	Principle of Operation	User Experience Required	Advantages	Limitations
Visual Observations	Determine distress in the structure through characteristic 'tell-tales'	Observed distress patterns are catalogued, and are applied to a given structure in the course of a walk-though	Low skill, but the operator should be very familiar with distress patterns in tire-bales structures in general, and ideally with the history of the tire-bale structure.	This method can give the most specific red flags relative to a specific distress mechanism, and provide instant feedback	Requires a high knowledge and judgment level on the part of the observer.

4.3 Analyzing Survey Data

The survey data taken from the existing tire-bale walls were analyzed to determine what deformations took place over the study period. The initial data was taken using a total station and a post-mounted reflection that was hand-steadied, using wooden ‘props’ for added stability. Use of a tripod was impractical because of the tire-bale structure configuration.

While the total station is in itself a very accurate instrument, the steadiness of the reflector was clearly an obstacle. It had to be supported by an individual standing on a very uneven (and typically highly inclined) surface, and leveled through visual observation of a bubble level. Wind was also an issue in being able to hold a steady position.

The procedure used therefore involved a number of ‘shots’ to determine the XYZ coordinates (in feet) of the original position of the survey point relative to the benchmark. At the next survey, a number of shots were taken again and the position and standard deviation obtained.

In all of the figures following, the downstream direction is to the right, and the streambed is toward the bottom. The x-axis indicates position indices; i.e., places where survey measurements were taken along the length of the structure. The y-axes indicate the ‘nominal’ position of the survey points in question. Movement of the data points in the vertical plane represents deflection of the tire-bale structure toward or away from the stream bed over the duration of the project.

This method of presentation ‘normalizes’ the structures into a straight line; clearly this is not representative of the actual structures along curving stream-beds. However, the straight-line representation provides a more direct and coherent picture of what is happening in the field.

As an example of how the points were derived. Consider the mean original XYZ position of a survey point is, in feet, (120.15, 45.33, 5.33) with a standard deviation of (.25, .12, .03). Now consider a survey at time t resulting in a mean position of (120.55, 45.4, 5.12) with a standard deviation of (0.21, 0.14, and 0.05).

For the X reading, the deflection is

$$120.55 - 120.15 = 0.4 \text{ ft}$$

The larger standard deviation is 0.25, so the maximum credible deflection in X is

$$0.4 - 0.25 = 0.15 \text{ ft}$$

For the Y reading, the deflection is

$$45.4 - 45.33 = 0.07 \text{ ft}$$

This is within the smaller standard deviation for Y of 0.12 ft, and so there is no meaningful deflection in Y. For the Z reading, the deflection is

$$5.12 – 5.33 = -0.21 \text{ ft}$$

The larger standard deviation is 0.05 ft, so the deflection in Z has a credible maximum of

$$-0.21 + 0.05 = -0.16 \text{ ft.}$$

This process must be repeated for each data point to give a maximum credible deflection for the tire-bale structure. Reducing the standard deviation of reflector position is clearly the vital link in improving accuracy.

The size of the 'final' markers closely approximates the standard deviation of the data. Therefore, when two markers representing the beginning and end of the survey period completely overlap, there is no statistically significant relative movement. If a 'diamond' appears from behind a 'square', the deflection has statistical significance

4.3.1 NM 52 Milepost 26.7

Figures 39 and 40 show lateral displacement of the top row tire-bales and backfill survey markers, respectively, as a function of position down the length of the tire-bale structure.

It should be remembered that this structure had an integral culvert outflow channel made of tire-bales in the upstream section; culvert outflow channel displacements were measured and were found to be within the standard deviation for the survey measurements, and are statistically not significant. The culvert outflow channel's position can thus be said to have remained unchanged over the period of the surveys, and need not be discussed further.

In Figures 39 and 40 downstream direction is to the right, above the 'nominal' position indicates movement toward the backfill, below the nominal position indicates movement away from the backfill.

The position index in Figures 39 and 40 relates to position in 20-ft increments, from upstream to downstream, moving left to right along the x-axis.

Figure 39 indicates that the top-row bales moved toward the streambed in the upstream portion of the structure, but that this displacement approached zero at the downstream end. (The last data point is clearly an outlier.)

Figure 40 shows the lateral movement of survey poins in the backfill, downstream from the culvert outflow channel (the presence of which is why the coverage is not complete). This data indicates that there was a slight movement of the backfill material away from the streambed, as would be expected as the backfill consolidated over time. This indicates that more thorough of the backfill was needed after the tire-bale structure was built.

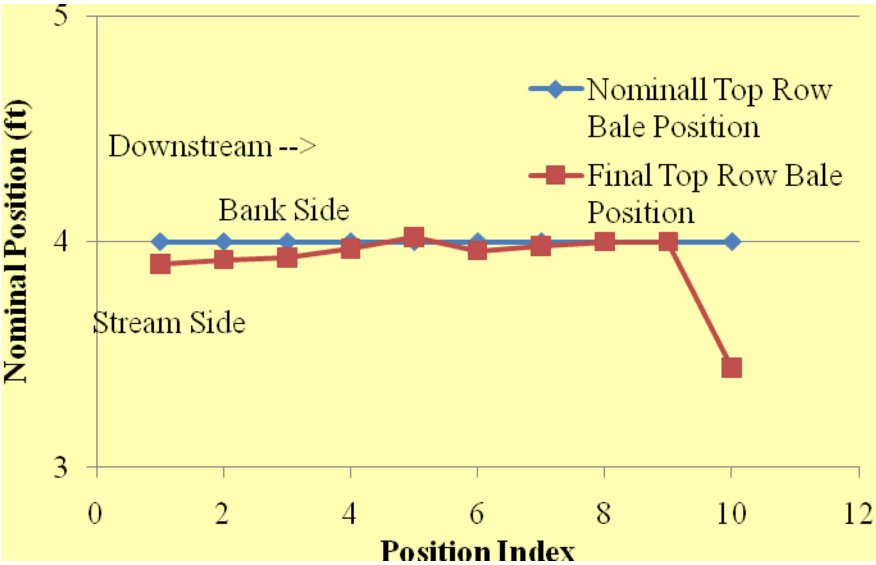


FIGURE 39 NM 52 MM 26.7: Horizontal displacement of top-row tire-bales over time

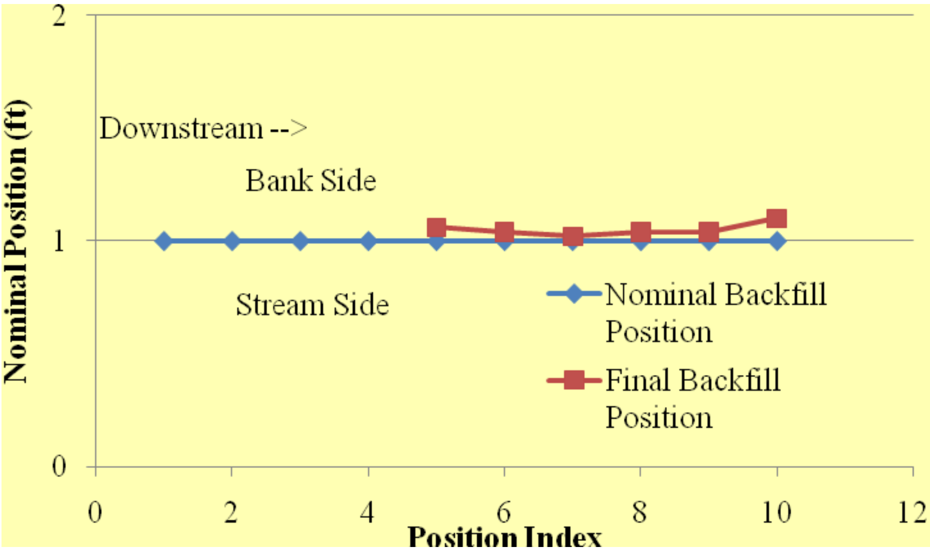


FIGURE 40 NM 52 MM 26.7: Horizontal displacement of backfill survey points over time

The vertical displacements of the top row bales (Figure 41) are consistently positive, and indicate that there was some soil swelling beneath the structure. A possible cause for this is that the backfill material below the structure had a higher proportion of native clay, which had expanded at the time of the surveys. Figure 42 shows that the vertical displacement of the backfill was very slight, and for all intents and purposes nonexistent.

In Figures 41 and 42 downstream is to the right, and ‘up’ in the field is ‘up’ on the figure. The position index in Figures 41 and 42 relates to position in 20-ft increments, from upstream to downstream, moving left to right along the x-axis.

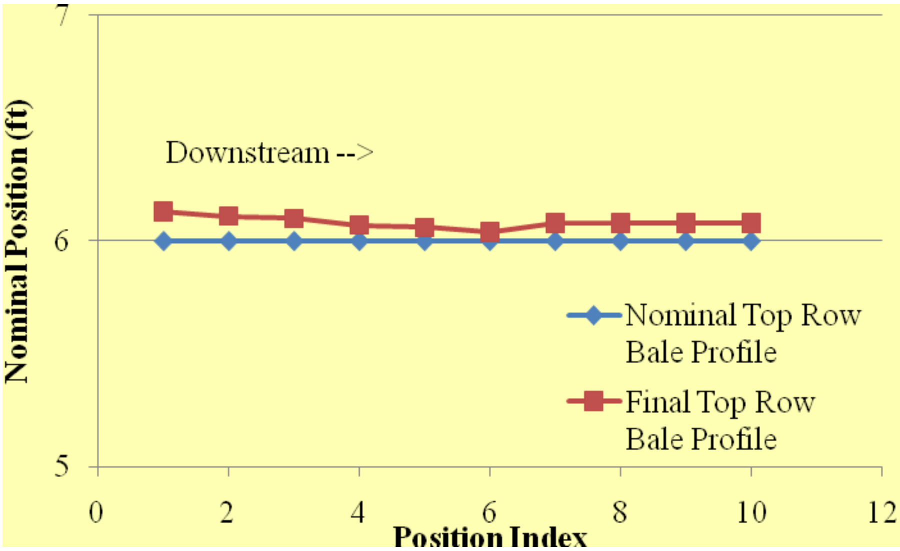


FIGURE 41 NM 52 MM 26.7: Vertical displacement profile of the top-row tire-bales

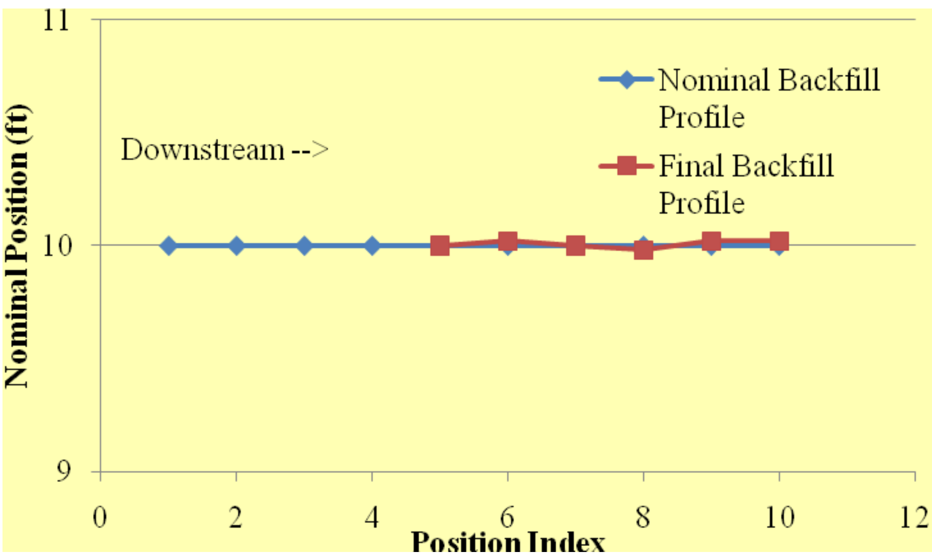


FIGURE 42 NM 52 MM 26.7: Vertical displacement profile of the backfill survey points

Appendix E figures E1-E13 display cross-sectional profile displacements along the length of the structure; E1 is the furthest upstream, and E13 is the furthest downstream. These profile views indicate that there was very little deformation within the tire-bale structure; the structural deformations that were recorded came as a result of the structure moving *en bloc* as a result of movement in the soil mass around it. The only significant profile deformation occurs at the upstream end, where observed scour has resulted in significant rotation (Figures E1 and E2)

There was also a small tire bale structure just downstream from MM 26.7, consisting of two bales wrapped in gabion wire forming the bed of an outflow channel for a pipe culvert. The bales were surveyed, and their displacements over time were within the standard deviation of the survey techniques; the displacements were statistically zero, and need not be discussed further.

4.3.2 NM 52 Milepost 27.2

In Figures 43 and 44 the lateral displacement of the tire-bale structure are presented. It should be recalled that this structure suffered from both erosion and significant tensile cracking from the midpoint in the downstream direction.

In Figures 43 and 44 downstream direction is to the right, above the ‘nominal’ position indicates movement toward the backfill, below the nominal position indicates movement away from the backfill.

The position index in Figures 43 and 44 relates to position in 20-ft increments, from upstream to downstream, moving left to right along the x-axis.

Figure 43 indicates (but for the presence of an obvious outlier) that the top row of bales has consistently shifted away from the stream bed, toward the backfill. The displacements are small but definite, and they follow a clear trend.

Figure 44 shows a similar trend, but for the presence of two probable outliers. These two data sets clearly indicate that the structure has experienced post-construction and stream-flow-induced displacement.

In figure 45 and 46 downstream is to the right, and ‘up’ in the field is ‘up’ on the figure.

The position index in Figures 45 and 46 relates to position in 20-ft increments, from upstream to downstream, moving left to right along the x-axis.

The data presented in Figures 45 and 46 is not as clearly defined. Figure 45, showing the vertical displacement of the top-row bales, indicates that the upstream bales have settled, while the downstream bales have risen (there is one obvious outlier).

Figure 46 shows an obvious settlement trend, increasing in the downstream direction. The downstream portion of the structure is that in which tensile cracking was observed.

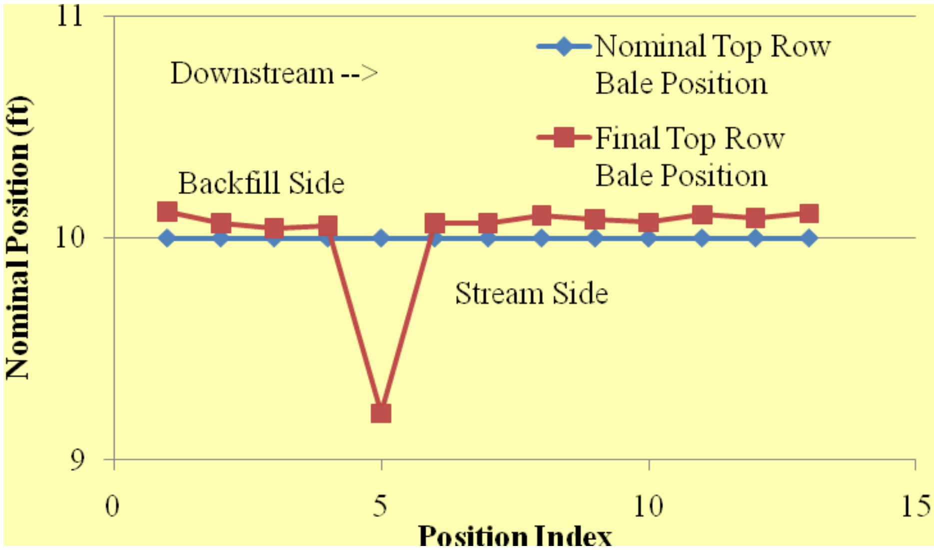


FIGURE 43 NM 52 MM 27.2: Horizontal displacement of top-row tire-bales over time

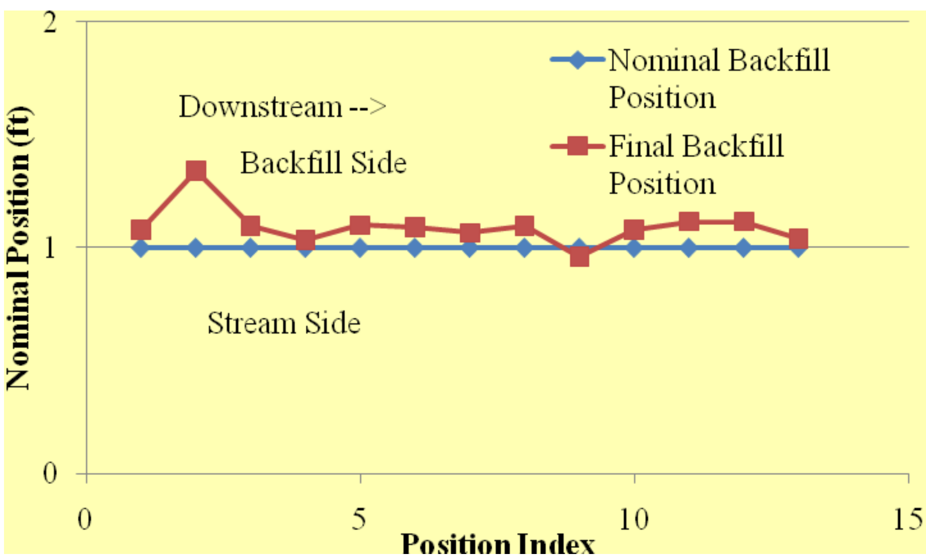


FIGURE 44 NM 52 MM 27.2: Horizontal displacement of backfill survey points over time

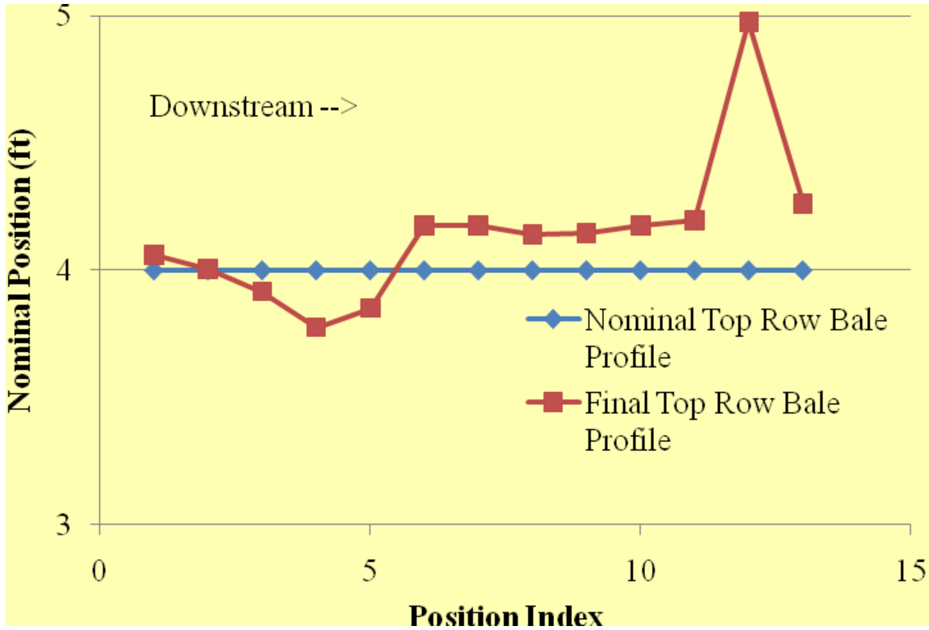


FIGURE 45 NM 52 MM 26.7: Vertical displacement profile of the top-row tire-bales

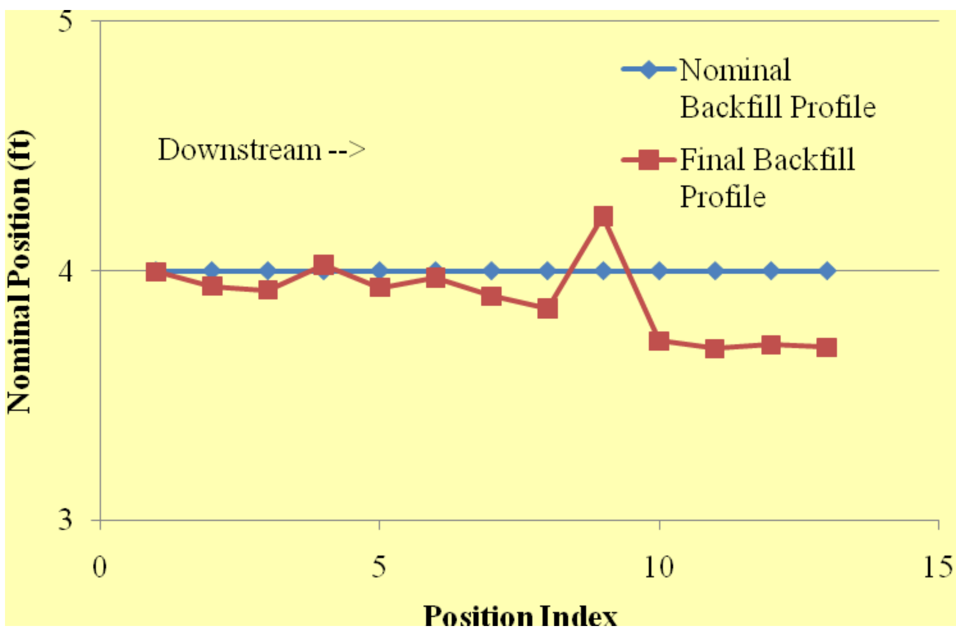


FIGURE 46 NM 52 MM 26.7: Vertical displacement profile of the backfill survey points

The interpretation of this data leads to the conclusion that the tensile cracks that developed allowed the structure to shift downward, perhaps by the scouring away of backfill material and its transmission through and into the tire-bales. The ‘lift’ in the upstream part most likely came from the swelling of clay soil mixed into the bed backfill placed when excavation for the structure was performed.

Appendix E figures E14 through E26 show that the structure is actually undergoing a rotation about its longitudinal axis; i.e., it is rotating ‘back’, away from the stream bed. This is consistent with the development of tensile cracking in the backfill behind the structure through the downstream portion of its length (particularly clear in Appendix E Figures E23-E26). While this does not presage immediate failure, as the deflections are very small, it does indicate that the structure is undergoing significant distress in this region.

4.3.3 NM 52 Milepost 27.75

Shown in Figures 47 and 48 are profiles that indicate the lateral movement of the tire-bale structure with respect to the stream bed.

In Figures 47 and 48 downstream direction is to the right, above the ‘nominal’ position indicates movement toward the backfill, below the nominal position indicates movement away from the backfill.

The position index in Figures 47 and 48 relates to position in 20-ft increments, from upstream to downstream, moving left to right along the x-axis.

Figure 47 shows lateral movement of the top row of tire bales. The first data point (on the left) is probably an outlier, as is the third-from last point on the right. These two measurements aside, this figure (and Figure 48 which follows) indicate that there has been a definite shift in the structure’s position over time; consolidation of the backfill and apparent hydrodynamic pressure from ephemeral water flow have served to ‘push’ the structure a small distance away from the stream bed. It must be emphasized that the displacements are very small, and nearly within the range of measurement error. (It should also be stated that deflections of this magnitude are to be expected for any structure, as soil consolidation takes place in the first few months-to-years after construction.)

Figure 48 shows lateral deformation in the backfill as a function of position along the length of the tire-bale structure. The results shown in Figure 39 are echoed here, though at a smaller magnitude. The displacement is away from the stream bed, again consistent with consolidation of the backfill and the influence of ephemeral-stream hydrodynamic pressure on the front of the structure.

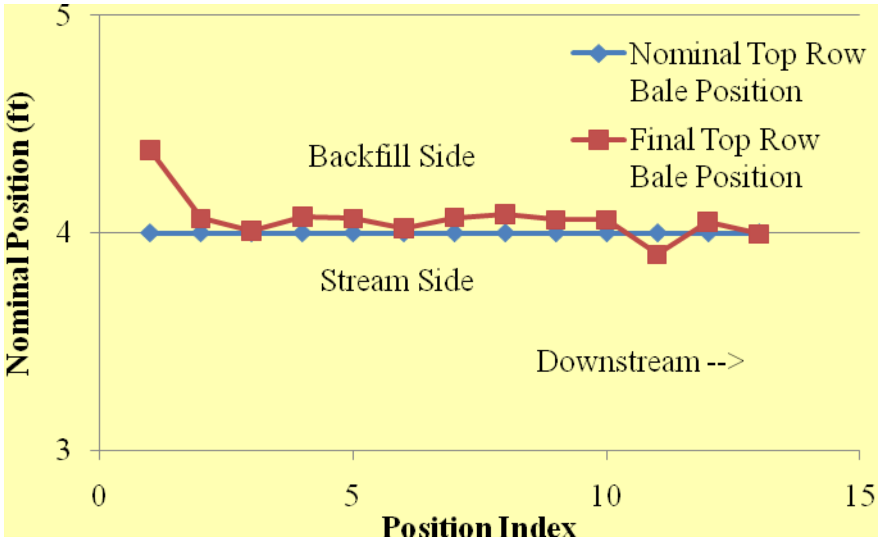


FIGURE 47 NM 52 MM 27.75: Horizontal displacement of top-row tire-bales over time

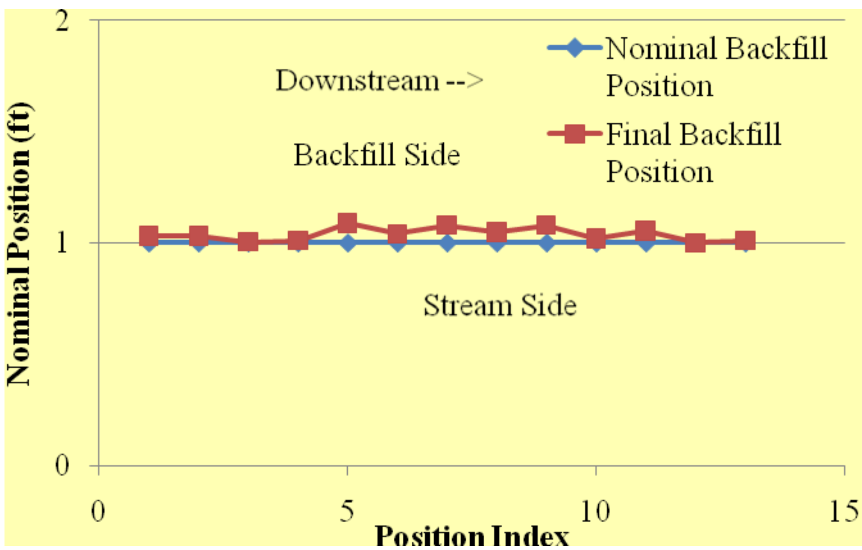


FIGURE 48 NM 52 MM 27.75: Horizontal displacement of backfill survey points

Figures 49 and 50 show the vertical profiles of the top-row tire-bales and backfill survey points, respectively, along the length of the tire-bale structure. As before, the graph moves downstream to the right, and the 'up' direction on the graph corresponds to an increase in survey point elevation.

In Figures 49 and 50 downstream is to the right, and 'up' in the field is 'up' on the figure.

The position index in Figures 49 and 50 relates to position in 20-ft increments, from upstream to downstream, moving left to right along the x-axis.

Figure 49 indicates a bit of 'waviness' is developing over time. There are three distinct settlement 'wave'; this data corresponds to the inference made above of consolidation and settlement about the tire-bale structure.

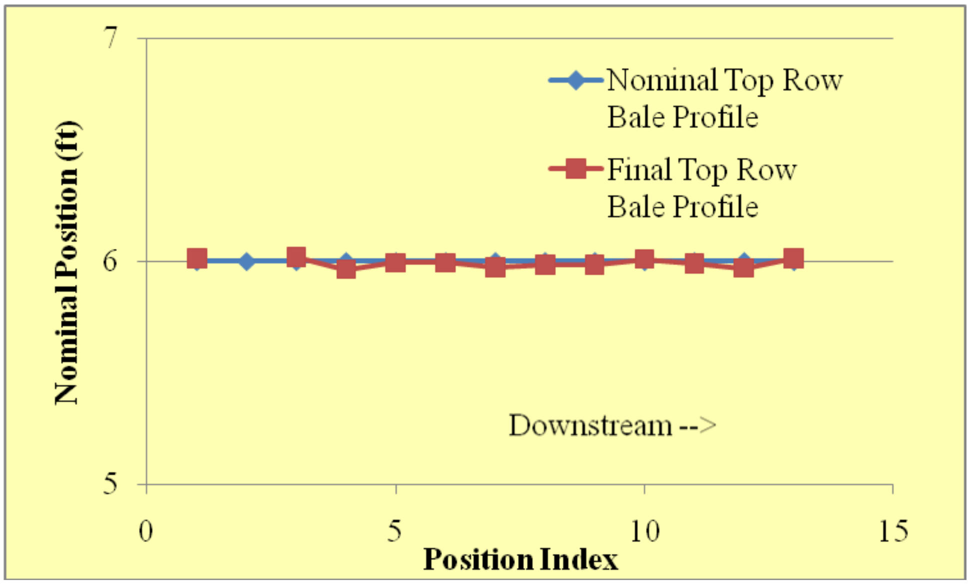


FIGURE 49 NM 52 MM 27.75: Vertical displacement profile of the top-row tire-bales

Figure 50 shows the vertical displacement profile of the backfill survey markers over the length of the tire-bale structure. A slight increase in height is seen in the upstream third of the structure, while settlement is seen in the downstream two-thirds. The uplift is very slight, and almost within the bounds of measurement error. If we assume that the uplift does exist, the best admissible explanation comes from the swelling of a local clay formation in the backfill, likely concentrated when native soil was removed for construction, and then replaced.

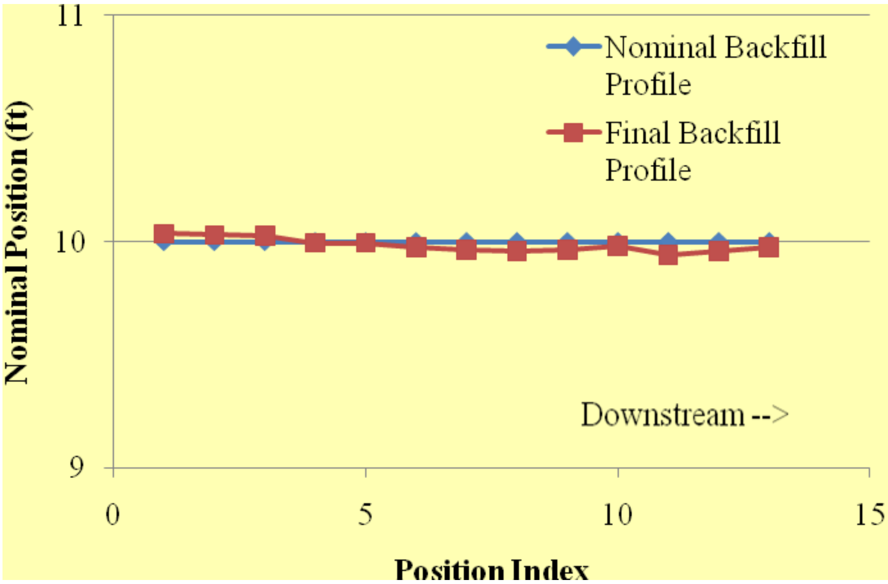


FIGURE 50 NM 52 MM 27.75: Vertical displacement profile of the backfill survey points

Appendix E E27 – E-36 shows displacement profiles for cross-sections through the structure. These cross-sectional displacement profiles indicate that the structural displacement is more a 'shift' of the whole structure rather than internal deflections within the tire-bale mass. This is consistent with the premise that the deflection is caused by settling, swelling, and consolidation of inadequately compacted soil in the backfill behind the tire-bale structure.

4.3.4 NM 152 Milepost 47.7

Figures 51 and 52 show the horizontal displacements of the top-row tire-bales and backfill survey markers, respectively, along the length of the tire-bale structure. They show a very stable structure, with some slight movement toward the stream bed. This may be expected, as the structure is 'concave' when regarded from the perspective of an observer standing on the backfill; i.e., the ends curve 'inward' rather than 'outward'. This structural configuration is unique among the tire-bale structures we surveyed; the others were either straight or convex (i.e., curving away from the backfill).

In Figures 51 and 52 downstream direction is to the right, above the 'nominal' position indicates movement toward the backfill, below the nominal position indicates movement away from the backfill.

The position index in Figures 51 and 52 relates to position in 20-ft increments, from upstream to downstream, moving left to right along the x-axis.

Our hypothesis is that this has resulted in slightly greater soil pressures from the backfill as it consolidates, pushing the structure outward toward the stream bed.

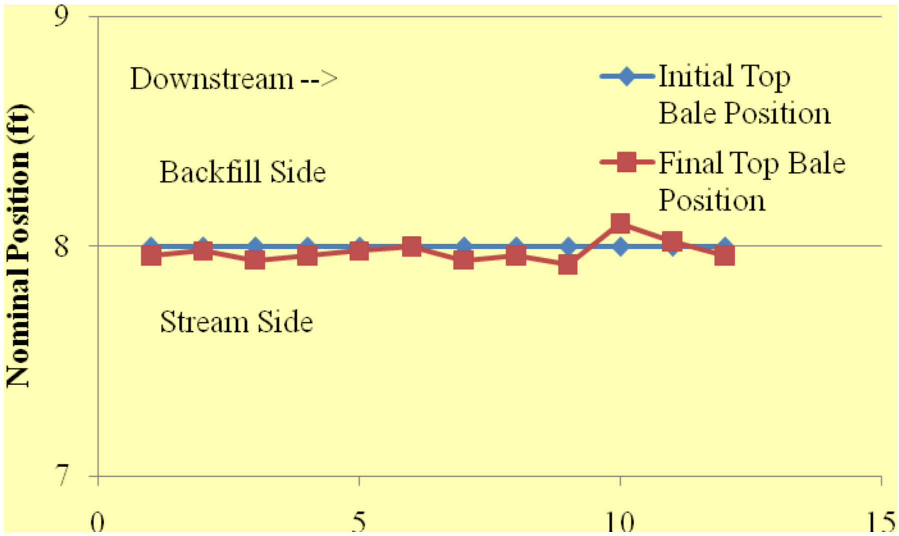


FIGURE 51 NM 152 MM 47.7: Horizontal displacement of top-row tire-bales

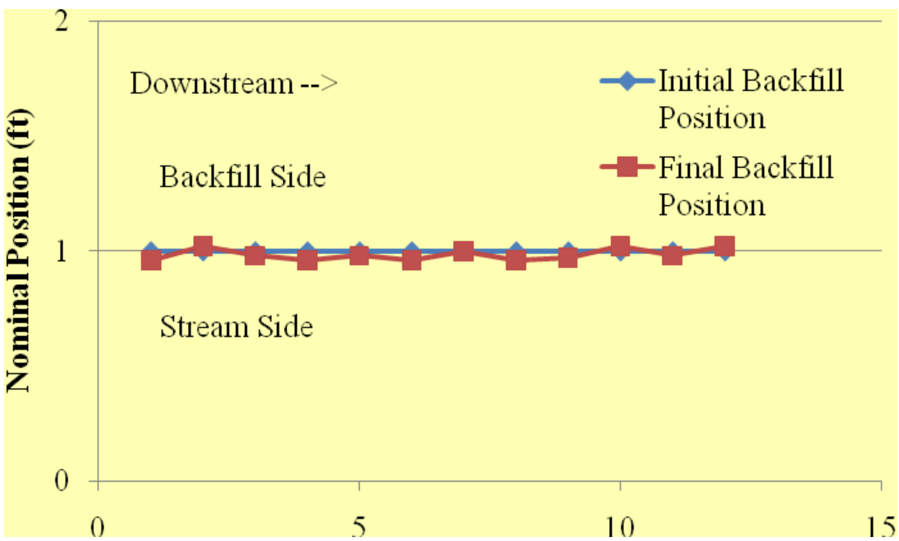


FIGURE 52 NM 152 MM 47.7: Horizontal displacement of backfill survey points

Figures 53 and 54 show the vertical deformation of the top-row tire-bales and the backfill survey markers, respectively, as a function of position along the tire-bale structure. Both Figures show a definite increase in elevation, which should be consistent with the assumption that backfill consolidation in this unique configuration has increased the soil pressure behind the tire-bale structure.

In Figures 53 and 54 downstream is to the right, and 'up' in the field is 'up' on the figure**.**

The position index in Figures 53 and 54 relates to position in 20-ft increments, from upstream to downstream, moving left to right along the x-axis.

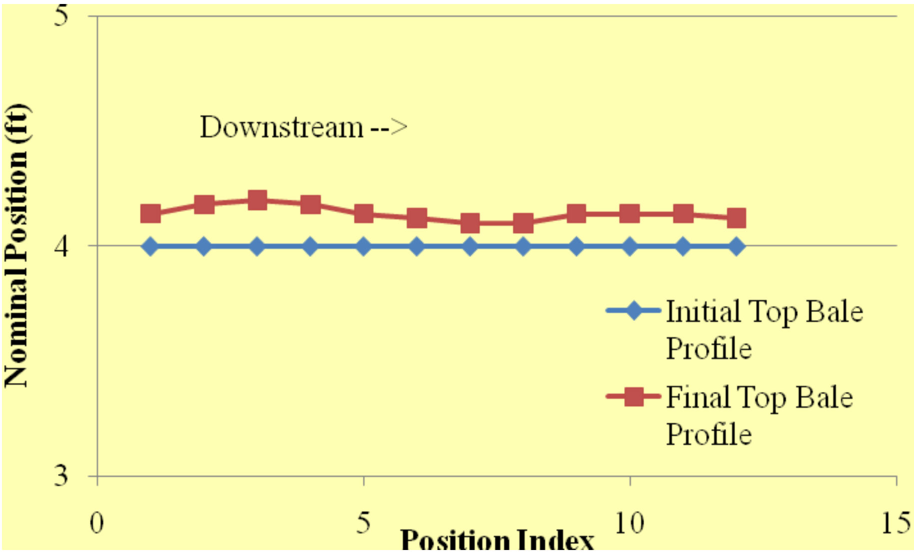


FIGURE 53 NM 152 MM 47.7: Vertical displacement profile of the top-row tire-bales

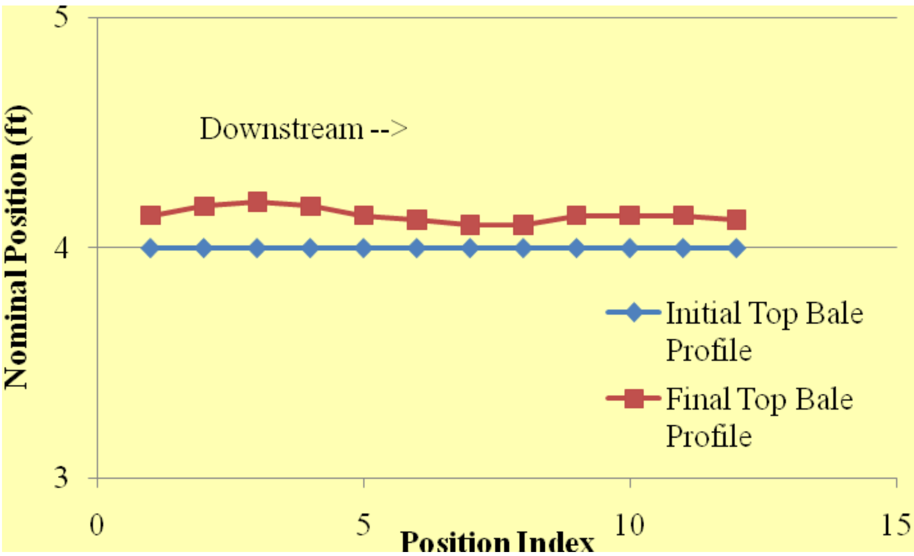


FIGURE 54 NM 152 MM 47.7: Vertical displacement profile of the backfill survey points

4.4 Analysis of Failure of NM-143 Tire-Bale Structure

At the end of July, 2008, a failure occurred at the NM-143 site, near Deming, approximately two months after installation. Figure 55, below, shows a schematic of the site. The original road design called for periodic arroyo flows to go over the road and into the natural basin on the other side. Recent increased development in the area had increased the water flow in the arroyo, leading to soil erosion on the north side of the road. The tire-bale structure was intended to stop the encroachment of the arroyo basin into the shoulder area of the north side of the road. The failure of the tire-bale structure at NM-143 was due to stream water from three directions flowing across the top of the structure and exploiting the contact between the native soil and the fill on the east side at point A on Figure 55.

Figure 56, below, shows the structure as of June 26, 2008. It appeared to be virtually brand-new, as the bottom "bed" of tire-bales was clearly visible. This was most likely the condition of the structure when the damage occurred. The original design may have called for shotcreting some areas of the structure, but this had not been completed when the damage occurred.

Figure 57, below, shows the structure after failure on July 21, 2008. The exact date of damage was unknown. It can be seen from the picture that the east wall of tire-bales had fallen into the channel, but remained intact due to the wire mesh. There was also a significant amount of debris in the basin, some of which was riprap from the western slope close to the road.

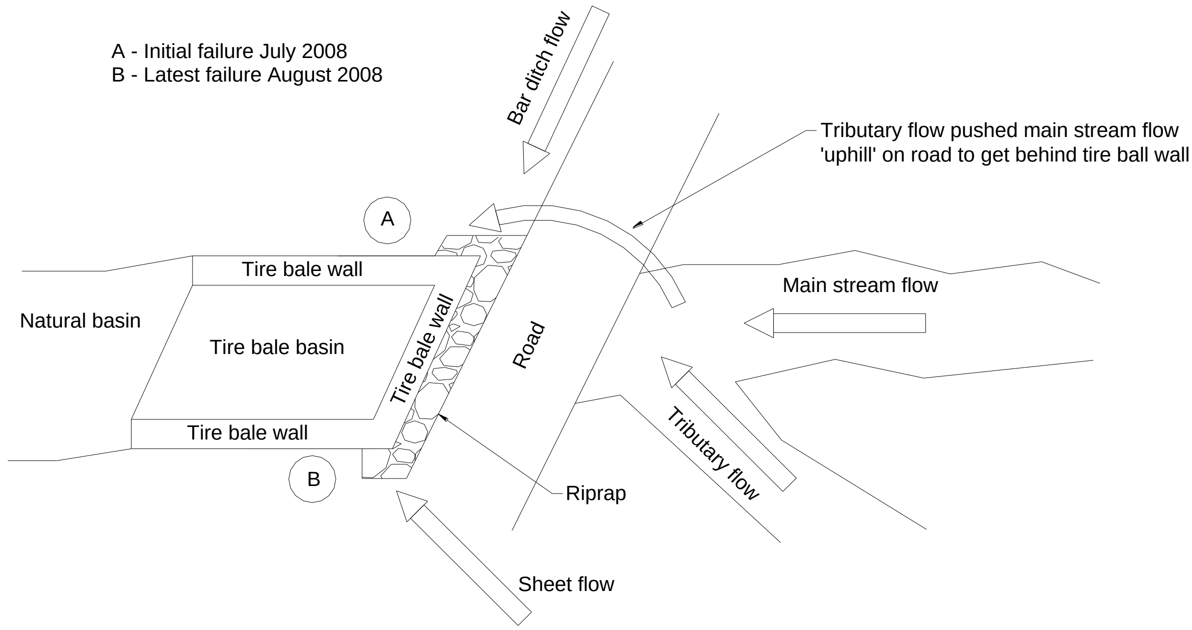


FIGURE 55 NM 143 Site Layout

FIGURE 56 NM-143 tire-bale structure as pictured on June 26, 2008

FIGURE 57 NM-143 tire-bale structure as pictured on July 21, 2008

Shortly after the damage, the fallen tire-bales from the eastern wall were removed and salvaged. These and additional tire-bales were placed in a new configuration to help protect the road until a more permanent solution could be sought. This new configuration may have cut the east slope back and replaced tire-bales in much the same way as the original design, thereby widening the basin on the north side of the road. This configuration was seen under construction in mid-August in Figure 58 and the end of August in Figure 59 below. Of particular interest in these figures was the new, reinforced corner of tire-bales on the eastern side.

The NM-143 site provided an opportunity to investigate the effects of water pressure and runoff patterns on a tire-bale stabilization design. Also of interest were the interactions of tire-bales with gabion baskets, riprap, geosynthetics, and shotcrete as they relate to drainage, build-up of water pressure, and soil stabilization.

The original drainage pattern as determined by the geometry of the road, riprap, and tire-bale structure. The hydrostatic forces acting behind the tire-bale structure and the buoyant forces from the bales themselves acted on the bales (within their 'net' of gabion wire) in the form of an outward pressure (i.e., away from the bank and into the channel). The steel cable and anchor system held the base of the tire-bale 'wall', but they acted as a pin connection; they did not provide appreciable moment resistance to resist the combination of hydrostatic pressure and buoyant forces, and the 'wall' rotated about them and overturned into the channel. The main resistance against overturning actually came from the gabion wire covering the structure, and was at its maximum at the reentrant corner where the channel wall met the head structure adjacent to the road. This is a critical failure mode for a tire-bale structure, and points to a clear requirement for lateral anchors (i.e., deadmen) to connect the structure to the backfill with horizontal tension members such as cables.

This condition is covered in more detail in Task 6, wherein is found a design example that addresses this need, and outlines the procedure for the design of lateral anchors.

The lesson of the failure of the NM 143 structure is that lateral anchors are a necessity for tire-bale structures used in erosion control applications. They cannot be built with sufficient moment resistance to resist hydrostatic pressure behind the structure, and there is no way to ensure the permanent exclusion of water through the life of the structure.

FIGURE 58 August Reconstruction – Photo courtesy of NMDOT

(date on photo is not accurate)

FIGURE 59 August Reconstruction – Photo courtesy of NMDOT

5.0 TASK 5:

"[Task 5] required construction of two representative tire-bale systems in a controlled environment so experimental and field data could be gathered. The data was to then be used to calibrate an analytical model and validate the parametric study used in developing the final design and construction guidelines. Special attention was given to the issue of corrosion of the tying wire, which appeared to be the primary cause of many of the failures in the state of Nebraska and other locations including the state of New Mexico (Rio Puerco Stabilization and Stream bank Protection Project in Cuba, NM). On the basis of this evaluation, new standards, specifications, design and methodology were formulated. Analysis on ease of construction and initial costs were performed while maintenance costs and life cycle cost are still being investigated *(1)*."

Selection of the Test Site for Field Demonstration Facility (FDF) The northern perimeter of the New Mexico Tech (NMT) campus is located on soil from Rio Puerco, easily erodible geologic formations such as the Santa Fe group or more recent stream deposits; fine sediments that are easily transported, and often contain high concentrations of salts characterize these formations. Sodium salts induced dispersion can enhance the erodibility of the sediments and may also enhance weathering of metal components of gabion structures of tire-bale-tying wires used in stabilizing streambeds. Sediment retention dams are frequently used to combat stream-induced erosion but the easily erodible sediments make designing and installation of such structures difficult. Many commonly occurring geologic formations in New Mexico have high salinities and consequently streams flowing across such formations pick up a lot of salts. In the Cuba area the Toldilto formation is predominantly gypsum and sediments from this formation and stream waters flowing across it also have high salinities. However, most of the sediments adjacent to Highway US 550 around Cuba and west of Interstate 25 from Los Lunas to Socorro are from the Rio Puerco. It was in consideration of these conditions that a site was selected for the construction of the Field Demonstration Facility (FDF); use of this land for this project was granted by the Vice President of Research and Development at New Mexico Tech.

Selection of the Tire-Bale System In the current construction, two structures were built to determine the effectiveness against head-cutting and side erosion. Each alternative were constructed adjacent to each other to establish consistent field conditions. The research team has closely monitored the installation process, recording other relevant information, time and ease of construction, and initial cost. The alternatives will continue to be monitored for the remaining duration of the project to determine effectiveness, durability, required maintenance, and life-cycle cost. Results are presented in matrix format.

Standard Laboratory Tests The following tests were performed on in-situ soil:

- Specific gravity of soil particles according to ASTM D854
- Grain size distribution according to ASTM D422 and D1140
- Liquid and plastic limits according to ASTM D4318
- Laboratory soil compaction according to ASTM D698
- Permeability according to ASTM D2434
- Unconfined compressive strength according to ASTM D2166
- Unconsolidated undrained triaxial strength according to ASTM D2850

Soil permeability and strength change with time due to consolidation, change in soil water content, and soil erosion. Any change in soil parameters will change the safety factor of the tire-bale structure for different failure modes such as overturning, sliding, and bearing capacity. Consequently, tests on time dependent properties were performed to modify the analytical model parameters for more accurate analysis on safety of the structure.

Tire-Bale Characteristics Properties of the tire-bales have a wide variation; a statistical analysis on existing literature values yielded their mechanical characteristics.

Development of Design Methodology The design standards developed from the research was rationally developed from observed performance (both the in-field and the FDF sites) and basic engineering principles. The standards were formulated in a conservative manner. Standards were developed to ensure that tire-bale structures have long-term structural and geotechnical stability, competitive life-cycle costs, and pose minimal environmental hazards.

Geotechnical stability analyses were performed for representative tire-bale systems using basic geotechnical analysis. The results were correlated to establish baseline stability data which were incorporated in user-friendly design charts or equations.

The design procedure to be developed was specifically directed toward incorporating a user-friendly approach which promotes confidence in its implementation. Design equations were developed, incorporating available and readily-measured quantities. Charts, tables and graphs were generated for use where appropriate. The formulation of a design spreadsheet based on the design standard developed in the project was anticipated. The data gathered allowed for the identification of some critical parameters in tire-bale structure performance to be used in the eventual formulation of a numerical modeling scheme to predict performance and refine the design standards.

The numerical work performed during the course of the project was intended to support the basic-principles approach to the development of tire-bale structure design standards, and to provide a basis for further refinement of these standards *(1)*."

5.1 Selection of the Site for FDF

The site is a developing arroyo situated on the edge of the La Jencia Basin, west of M Mountain (Figure 60). The arroyo is cutting through coalesced fan deposits from the Magdalena Mountains to the west. The gravels are composed primarily of rhyolytic sands and gravels with some finer sandy loam sediments. These deposits are part of the upper Santa Fe formation. Soils have developed on the surface of these deposits and are classified as part of the Millett series; a fine loamy mixed mesic Ustollic Haplargid. The soil can be described as deep well drained and moderately permeable. The degree of soil development indicates that this soil is young, probably Holocene in age indicating that the fan deposits are geologically young.

There has not been significant weathering and alteration of the deposits since deposition.

5.2 Visual Inspection of the Soil in the Field

A wide range of grain sizes was observed in both sites; from very fine particles (silt and clay) to gravel and boulders. At the time of sampling the top soil was almost dry, while the soil at a depth of 30 cm (and more) was wet. Wind and braided streams in these sites are highly erosive and carry large amount of sediments. The deposits are highly irregular in stratification as well as engineering properties.

FIGURE 60 FDF Site

The coordinates of the site are 34º06’01.7” N and 107º02’20.8”. Soil samples were taken from the streambed and the side walls of the streambed to obtain lower level disturbed samples, as core sampling was not achievable in this project. Figure 61 shows the FDF site prior to construction. The head of the structure is indicated as is the areas of soil sampling. Figure 62 shows a sample being taken.

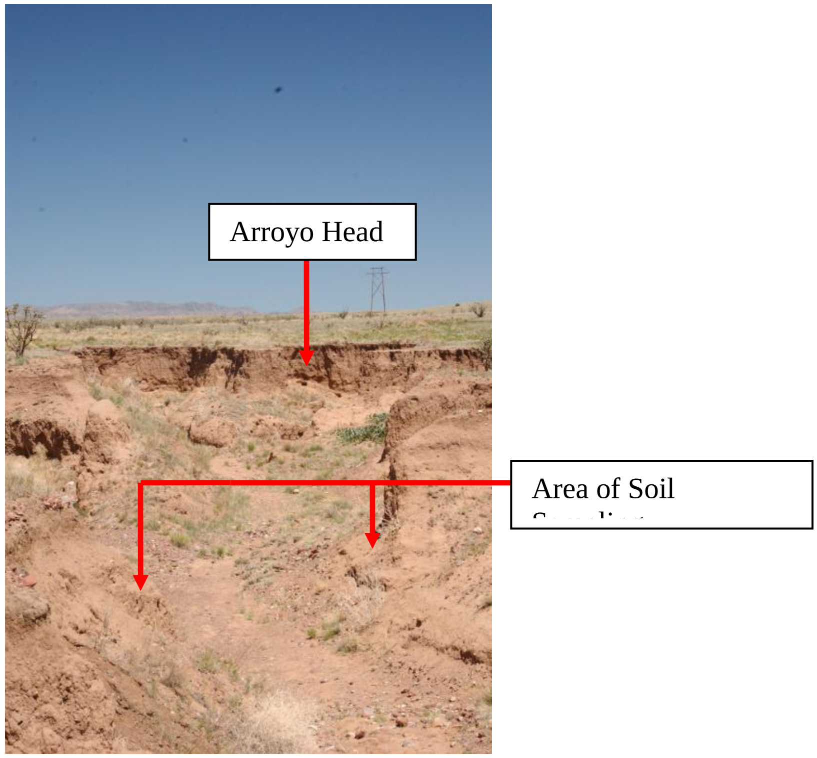


FIGURE 61 FDF Site Prior to Construction

FIGURE 62 FDF Soil Sampling

5.3 Permitting of the FDF

Permits were obtained for the construction of the tire-bale structures. The New Mexico Army Corp of Engineers and the New Mexico Environmental Department were the two entities involved. Appendix F1-F3 displays the necessary correspondence.

5.4 Laboratory Tests

Several standard soil mechanics laboratory tests have been completed or are in progress for the Energetic Materials Research and Testing Center (EMRTC) samples. Completed tests are water content, gradation, specific gravity, Atterberg limits, compaction, and permeability. Direct shear testing is in progress, the results of which will be included in Phase II (section 7.0 of this report). Also to be included in Phase II is the results of all these tests for the NM-52, NM-143, and NM-152 sites.

Water Content The average water content of the soil samples taken from the EMRTC site is given in Table 5.

TABLE 5 Average water content of the soil in EMRTC

Site	Average Water Content (%)
EMRTC	5.20

Particle Size Distribution Gradation analysis is necessary for classifying any soil; this classification is important for inferring certain basic physical and engineering soil properties. All five samples tested are coarse-grained (USCS, > 50% retained on No. 200 sieve) and well-graded.

Gradation analysis was done following ASTM D6913-04e2 using the sieves shown in Table 6. The particles found in the samples are often larger than 3/8”, and this portion of each sample was removed before sieving. A photograph was taken of each sample after this portion was removed to record the general size and quantity of the material.

TABLE 6 Sieves used for Gradations

Sieve	Maximum Grain Size (mm) – (1mm=0.03937in)
3/8”	9.51 (0.374409)
No. 4	4.76 (0.187402)
No. 16	1.19 (0.04685)
No. 40	0.42 (0.016535)
No. 100	0.149 (0.005866)
No. 200	0.074 (0.002913)

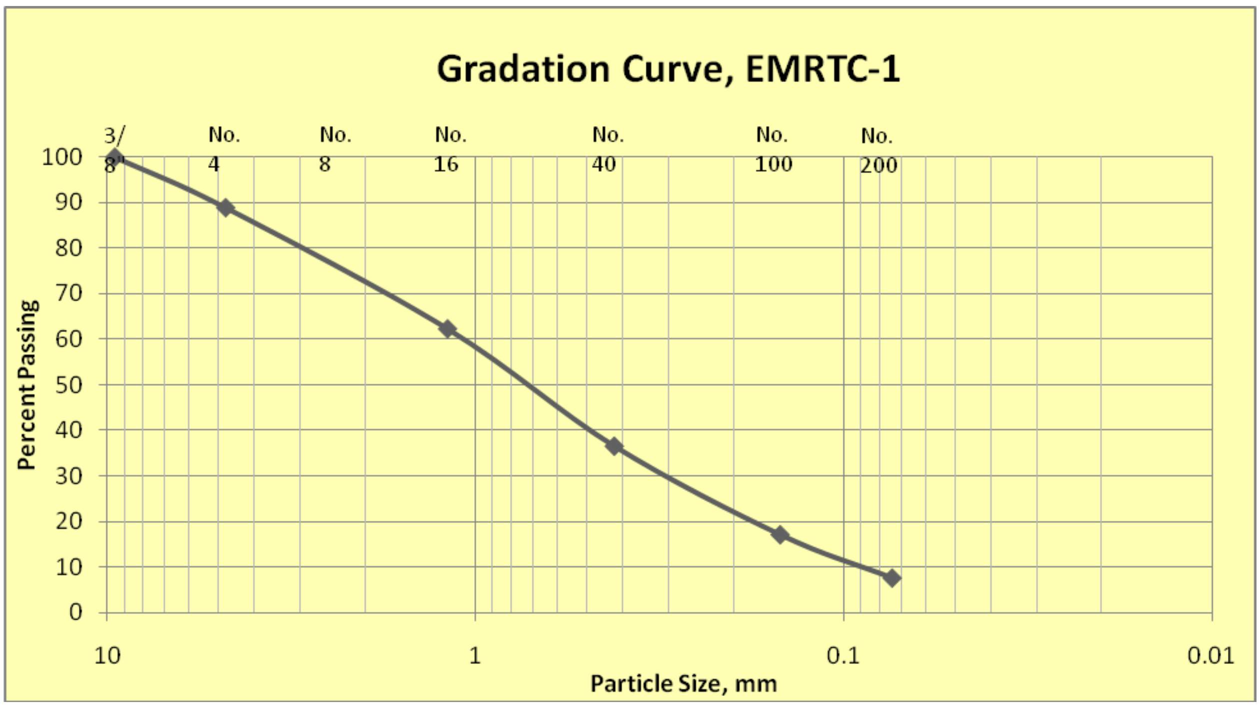


FIGURE 63 Gradation Curve for EMRTC-1 – (1mm=0.03937in)

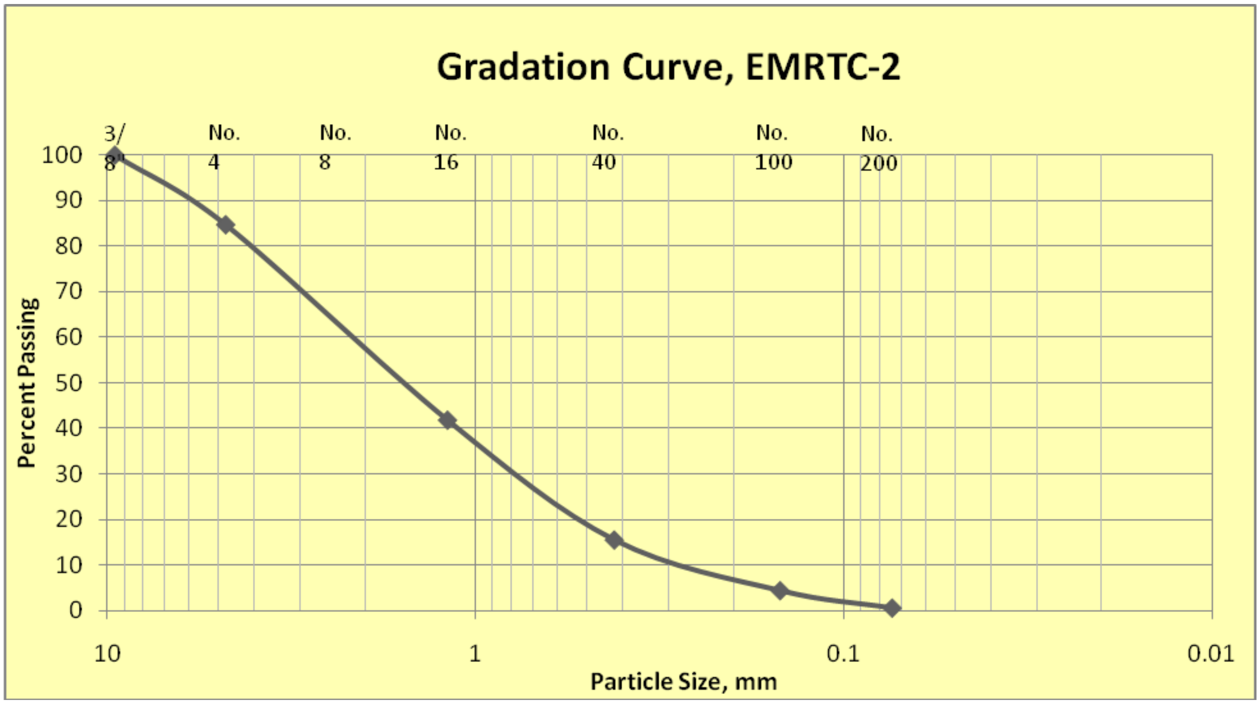


FIGURE 64 Gradation Curve for EMRTC-2 – (1mm=0.03937in)

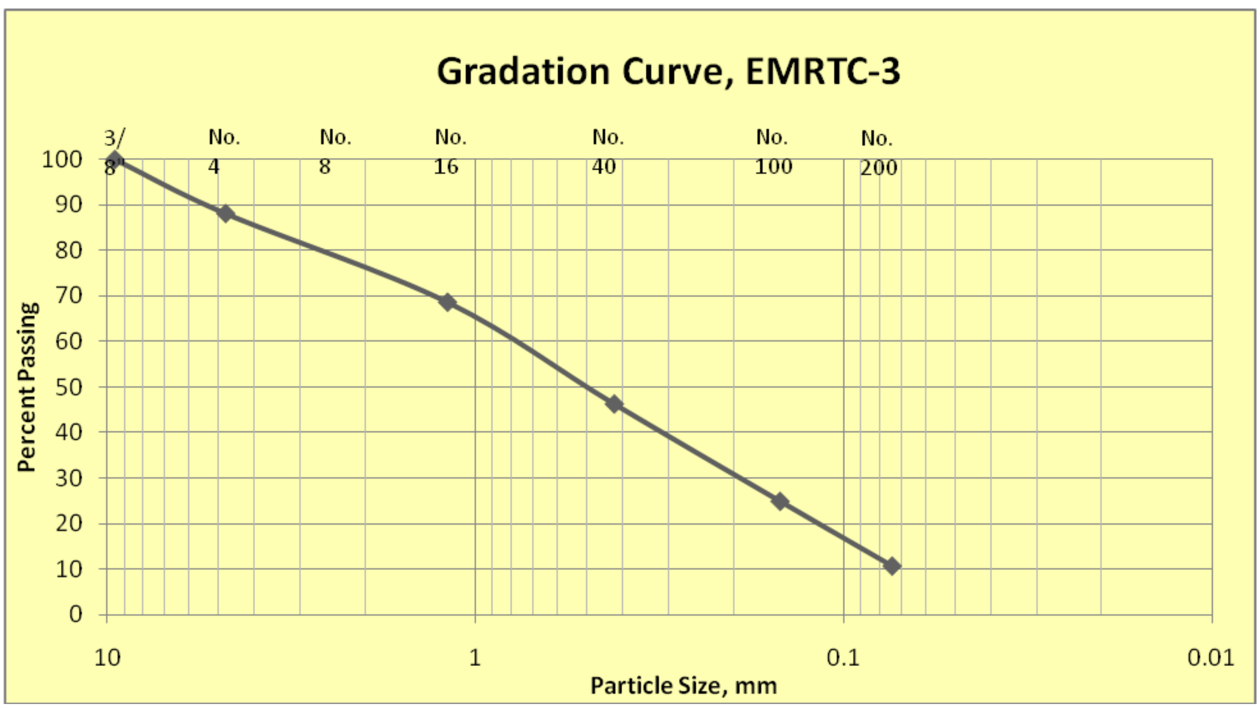


FIGURE 65 Gradation Curve for EMRTC-3 – (1mm=0.03937in)

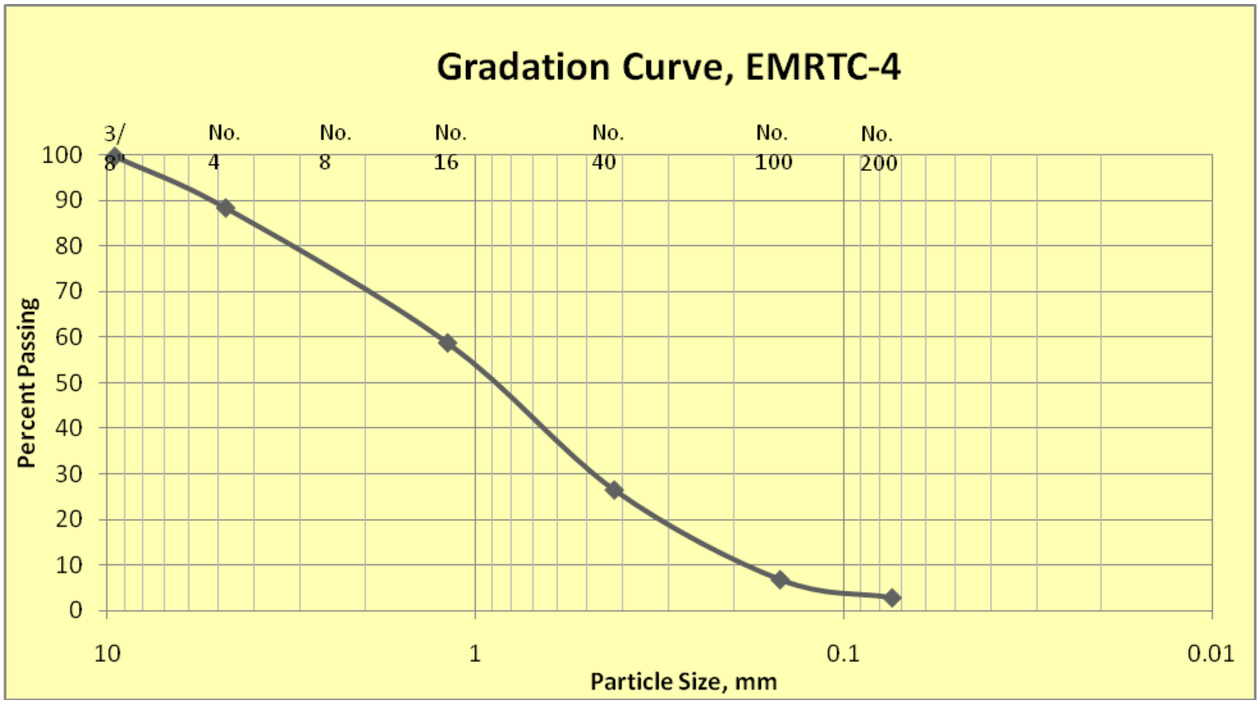


FIGURE 66 Gradation Curve for EMRTC-4 – (1mm=0.03937in)

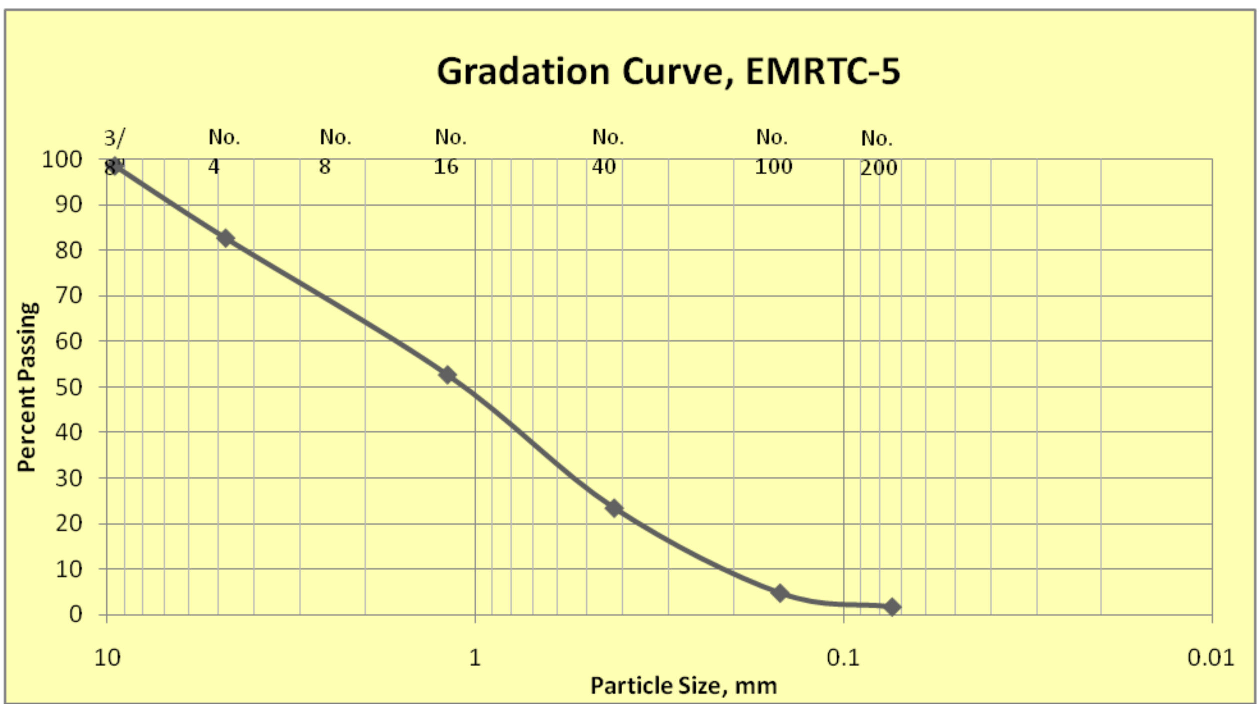


FIGURE 67 Gradation Curve for EMRTC-5 – (1mm=0.03937in)

Specific Gravity Specific gravity is useful both in its own right, and also for determining other physical soil properties, such as void ratio and degree of saturation. The test is performed only on the sample portion smaller than 4.76 mm (0.1874012in. No. 4 sieve), and starting with the field moisture content (Method A), which is preferred to oven-drying the soil before the test (Method B).

The specific gravity of each sample was found according to ASTM D854-06 ,Method A; the results are given in Table 7. The results are all within the typical range (2.64 – 2.72), but vary widely within it, from the low end of the spectrum (2.639, EMRTC-2) to the high end (2.715, EMRTC-4).

TABLE 7 Specific Gravity

Sample	Specific Gravity
EMRTC-1	2.645
EMRTC-2	2.639
EMRTC-3	2.675
EMRTC-4	2.715
EMRTC-5	2.673

Atterberg Limits The liquid and plastic limits of a soil describe the phase relationships, and are important for classifying the soil. The tests here follow ASTM D4318-05. Two of the five EMRTC samples contain too much sand to obtain liquid and plastic limits, which are considered non-plastic (NP). The results for the fine parts of EMRTC-1, 2, and 3 are plotted on a Casagrande Plasticity Chart (Figure 68), which is useful for classifying a soil. Note that all three samples fall in the hatched zone, which indicates the fine parts are silty- and/or clayey-sands (SM-SC by the USCS).

TABLE 8 Atterberg Limits

Sample	Plastic Limit	Liquid Limit	Plasticity Index
EMRTC-1	16.4	20.5	4.1
EMRTC-2	16.3	21.4	5.1
EMRTC-3	14.8	19.4	4.4
EMRTC-4	NP	NP	NP
EMRTC-5	NP	NP	NP

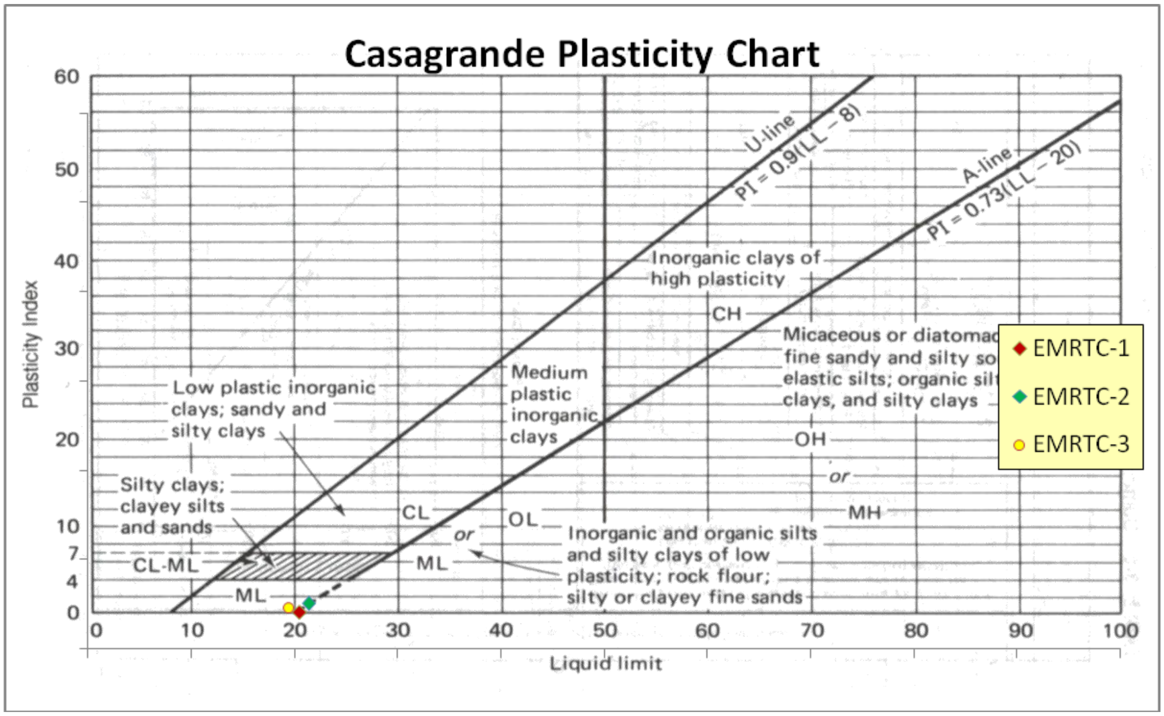


FIGURE 68 Casagrande Plasticity Chart

Standard Proctor Compaction The Standard Proctor Compaction test is important for determining the optimum water content for compaction and very useful in many construction applications additionally the maximum dry unit weight achievable for a given soil needs to be known. This test was performed according to ASTM D698-07e1, Method A, which requires that less than 25% of material passes the No. 200 sieve, and uses a 4-inch diameter mold, a 5.5-lb hammer, a 12-inch drop, three layers, and 25 blows per layer. Figure 69 shows a test in progress, and Table 9 shows a summary of the results. While Figures 70 – 74 show the curves for each of the five samples.

FIGURE 69 Performing of a Standard Proctor Compaction Test

TABLE 9 Standard Proctor Compaction Results

Sample	**Optimum Water Content (%)**	**Maximum Dry Density (kN/m^3)** **($1kN/m^3 = 6.37lb/ft^3$)**
EMRTC-1	10.0	19.4 (123.578)
EMRTC-2	11.75	19.1 (121.667)
EMRTC-3	12.3	18.7 (119.119)
EMRTC-4	13.0	19.0 (121.030)
EMRTC-5	13.3	18.5 (117.845)

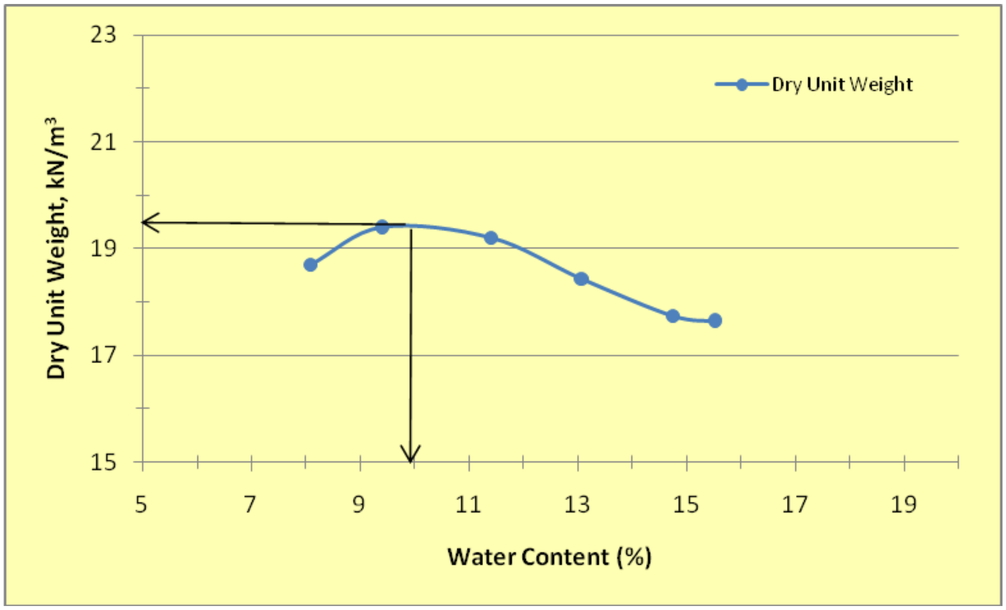


FIGURE 70 EMRTC-1 Compaction Curve

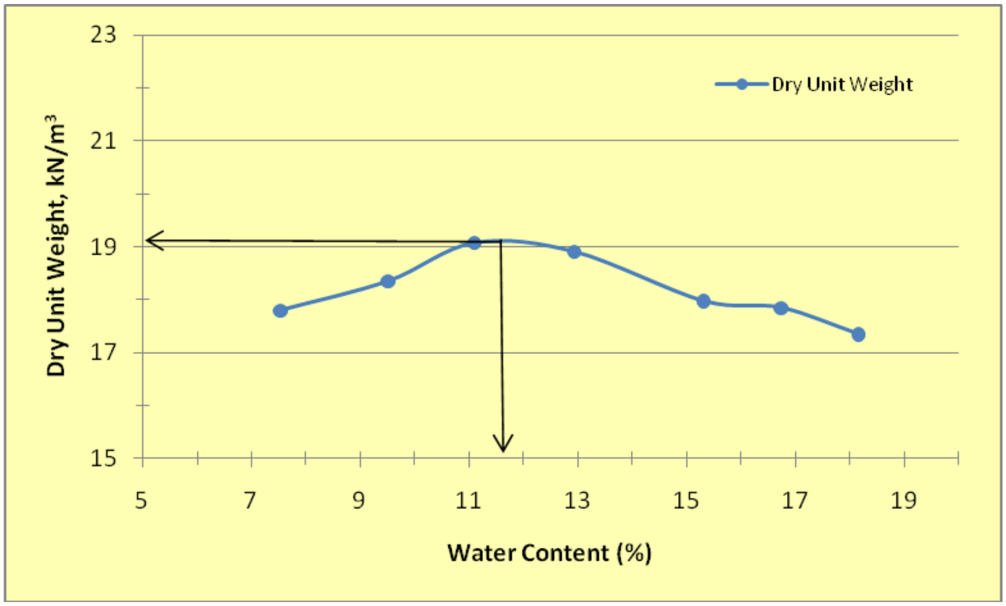


FIGURE 71 EMRTC-2 Compaction Curve

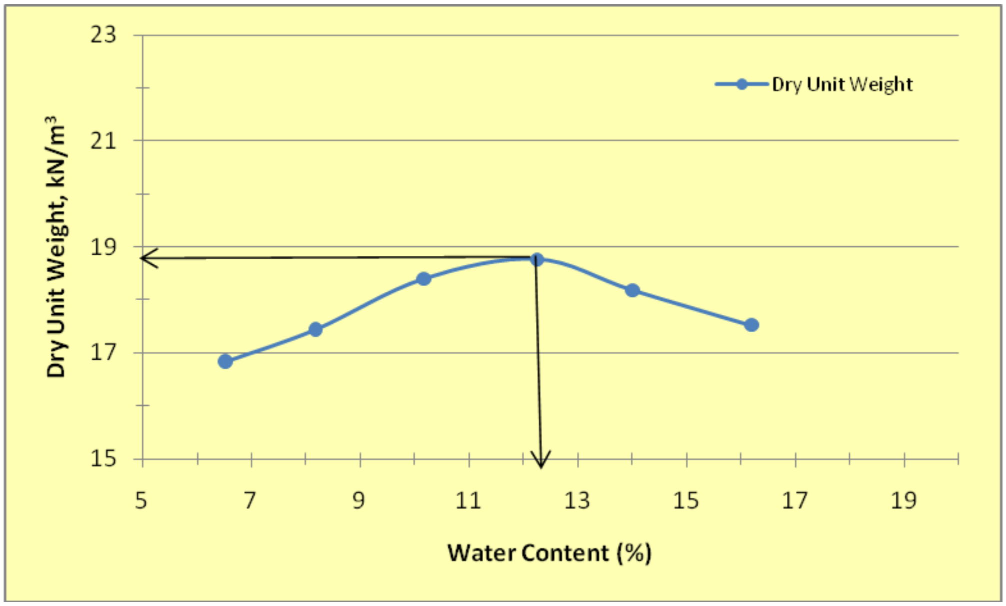


FIGURE 72 EMRTC-3 Compaction Curve

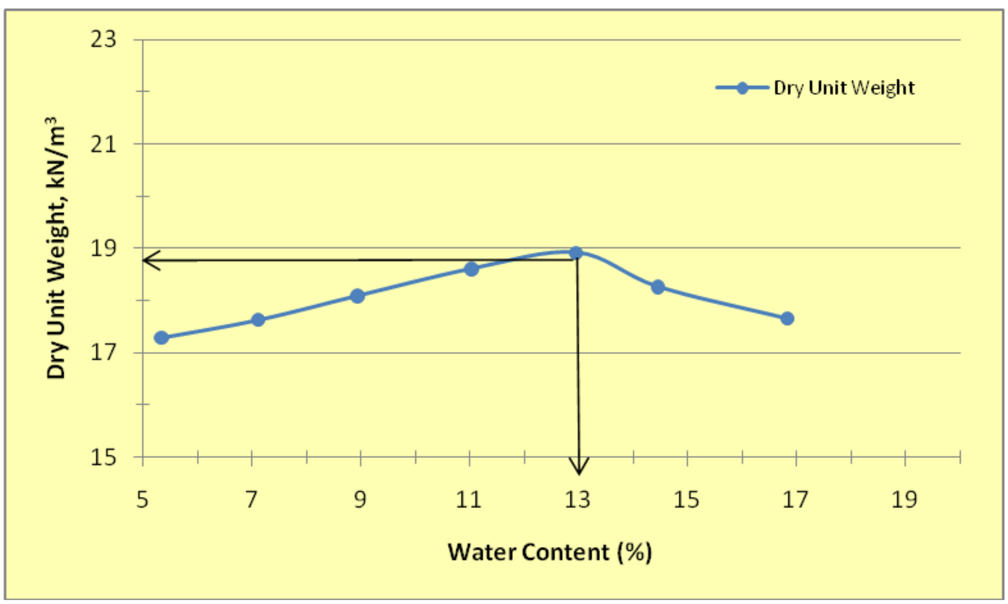


FIGURE 73 EMRTC-4 Compaction Curve

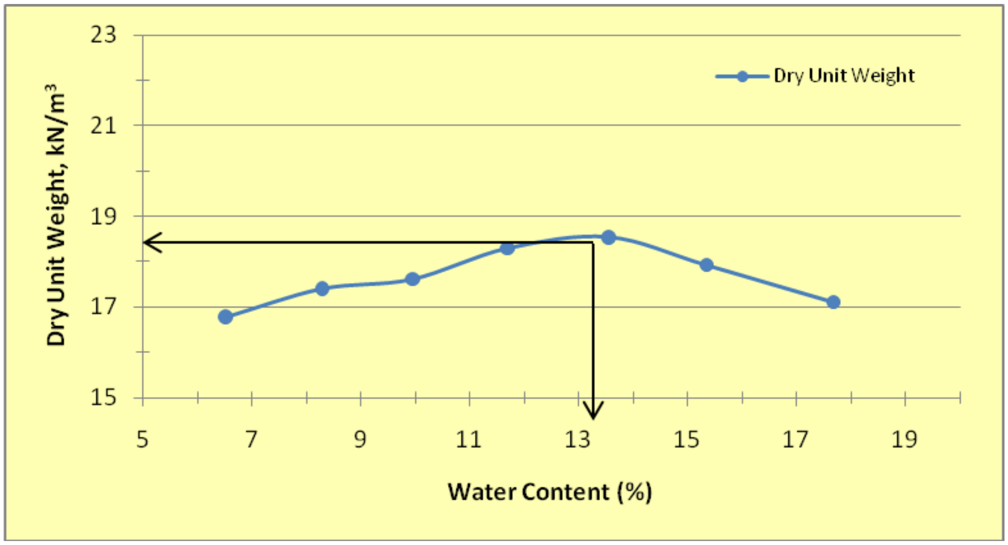


FIGURE 74 EMRTC-5 Compaction Curve

Permeability The different nature of the soils required the use of both the falling and constant head permeability techniques. The constant head technique is more suited to sandy soils with higher permeability, whereas the falling head method is better for soils with lower permeability, as it allows higher water pressure on the sample. The tests were performed with an ELE international 2-inch diameter permeability apparatus, and followed the recommendations found in chapter 12 of the Manual of Soil Laboratory Testing *(60)*. The constant head and falling head test devices are shown in Figure 75 and 76, respectively. The results are summarized in Table 10.

All the collected soil samples were sandy soils with almost zero cohesion. Therefore, it was not possible to perform unconfined compressive strength on the soil. On the other hand, the confining pressure on the collected samples was negligible due to small sampling depth; for that reason, triaxial test did not provide any additional information about the soil properties.

Soil Classification Table 11 shows the soil classification recommendations for the five samples taken from the EMRTC site. Two of the five samples were well-graded sands, and the other three were well-graded sands with some silt and clay content.

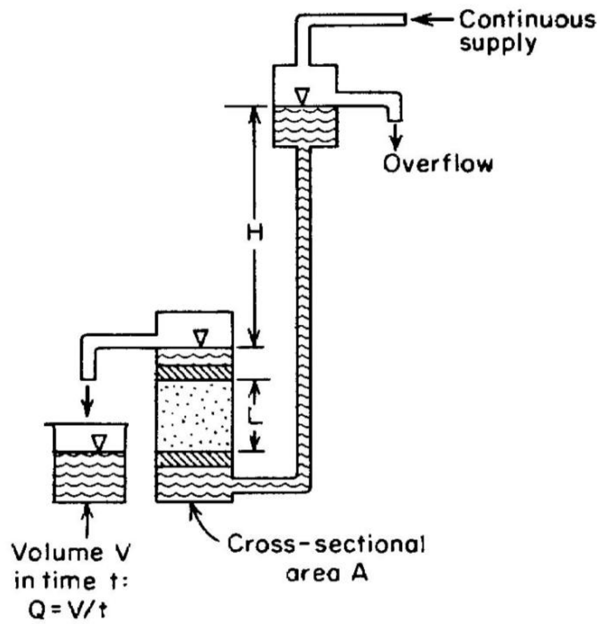


FIGURE 75 Constant Head Permeability Apparatus

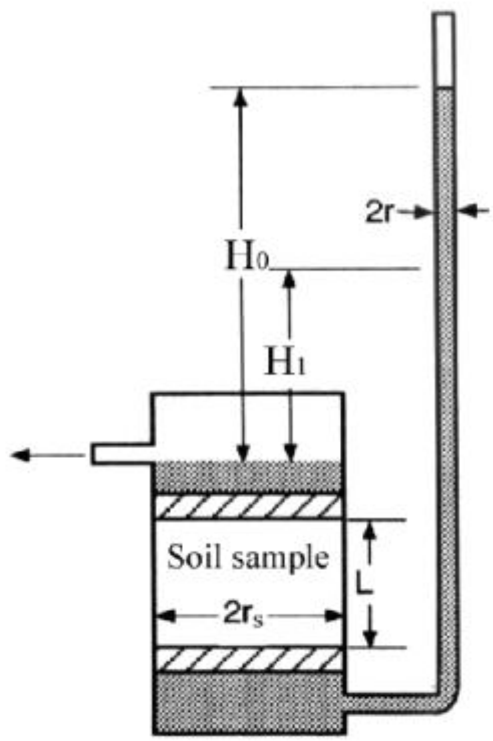


FIGURE 76 Falling Head Permeability Apparatus

TABLE 10 Coefficient of Permeability

Sample	Test	Coefficient of Permeability (cm/sec) (cm/sec = 0.393701in/sec)
EMRTC-1	Falling Head	4.23×10^{-5} (1.7×10^{-5})
EMRTC-2	Constant Head	2.33×10^{-2} (9.173×10^{-3})
EMRTC-3	Falling Head	2.44×10^{-5} (1.0×10^{-5})
EMRTC-4	Falling Head	3.64×10^{-4} (1.43×10^{-4})
EMRTC-5	Constant Head	4.76×10^{-3} (1.874×10^{-3})

5.5 Development of Design Methodology – Stability of the Tire-Bale Structure

"The design standards were developed to ensure that tire-bale structures had long-term structural and geotechnical stability, competitive life-cycle costs, and pose minimal environmental hazards *(1)*."

General description and material properties of soil at site Figure 77 and Appendix G (Construction Drawings) show schematic sections of the proposed tire-bale structures for the FDF site. The structure was built using a total number of 32 tire-bales in three rows to form a pyramid shape. Tire-bales are covered with native soil compacted in less than 12 in. thick layers of soil. A steel wire-mesh was placed on the tire-bales to increase the integrity of the structure. To avoid developing any extra pore water pressure behind and beneath the structure, a combination of an impermeable layer of geomembrane and a thin layer of non-expansive clay was used around the structure. The topsoil design to be treated with either 10% CKD or 10% cement by weight to provide more resistance against erosion (this was not achieved; only 3% was achieved). Four short concrete (or steel) piles at the four corners of the structure were designed to increase stability of the structure.

TABLE 11 Soil Classification Recommendations

Sample	% Passing No. 4 Sieve	% Passing No. 200 Sieve	Atterberg Limit Classification Recommendations	USCS Classification
EMRTC-1	88.9	7.6	Falls in the hatched zone on the Casagrande plasticity chart; sample is well-graded but contains enough clay and silt to perform liquid and plastic limit tests	SW-SM
EMRTC-2	84.7	0.6	Falls within hatched zone on Casagrande plasticity chart, but sample had very little material passing the No. 200 sieve; since enough fines were present to conduct plastic and liquid limit tests, it is possible that the fines were stuck to larger particles while sieving, thereby misleading the gradation results; since the Casagrande chart suggests the soil contains inorganic silts and very fine sands, rock flour, or silty or clayey fine sands (ML), a double symbol is most likely appropriate	SW-ML
EMRTC-3	88.1	10.6	Falls within the hatched zone on the Casagrande plasticity chart; even though this sample is very similar to EMRTC-1, a higher plasticity index indicates that this sample has more clay; the sample is well-graded	SW-SC
EMRTC-4	88.4	2.9	The sample does not contain enough fine to perform liquid and plastic limit tests; the sample is well-graded and non-plastic	SW
EMRTC-5	82.7	1.7	The sample does not contain enough fine to perform liquid and plastic limit tests; the sample is well-graded and non-plastic	SW

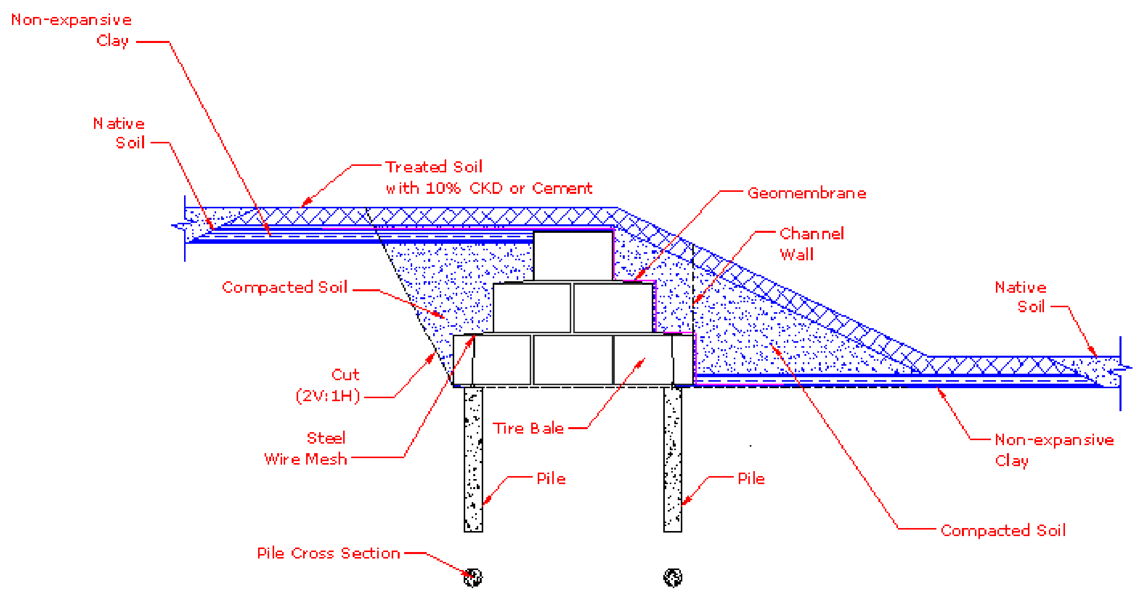


FIGURE 77 A schematic section of the proposed tire-bale structure in FDF site

Soil properties obtained or estimated from the laboratory tests performed on the collected disturbed sampled from the FDF site were used for stability analysis of the structure. However, all the soil samples were collected from a shallow depth due to limited budget and available equipment. Therefore, a large margin of safety was applied to compensate for the large error in the obtained materials properties. The engineering properties of the native soil and tire-bales used in stability analysis of the structure were as follows.

Soil

- Bulk unit weight, γ = 16 kN/m^3 (101.92lb/ft^3)
- Maximum dry unit weight, γ_{dry} = 19 kN/m^3 (121.03 lb/ft^3)
- Optimum water content, ω_{opt} = 12% ± 2%
- Buoyant unit weight, γ' = 6.2 kN/m^3 (39.494 lb/ft^3)
- Average void ratio, e = 0.60
- Average porosity, $n \approx 0.40$
- Internal friction angle, φ = 25°
- Cohesion, $c \approx 0$
- Hydraulic conductivity of the native soil, $k = 8.64\times10^{-5}$ m/min. (2.83 x 10^{-4} ft/min – 1m/min = 3.28084 ft/min)
- Coefficient of active earth pressure = 0.406

Tire-bales

- Dimensions of tire-bales ($L \times W \times H$) = 1.33 m × 1.55 m ×0.83 m (4.362ft x 5.085ft x 2.723ft – 1m = 3.28084ft)
- Nominal bulk unit weight, γ = 4.7 kN/m^3 (29.939lb/ft^3)
- Young's modulus, E = 900 MPa (130.62ksi – 1ksi = 6.89 MPa)
- Angle of inter-bale friction, φ_b = 35°
- Hydraulic conductivity through length, k_b = 0.03 m/s (0.098425 ft/s – 1m/s = 3.28084f/s)

Estimation of the geomembrane length To determine the length of the geomembrane the approximate travel distance of water is estimated by the following equation:

$$d = \frac{1}{2}\left[\frac{kt}{n} + \sqrt{\left(\frac{kt}{n}\right)^2 + \frac{4ktH}{n}}\right]$$

Where:

- d = travel distance (ft or m)
- k = hydraulic conductivity (ft/min or m/min)
- t = travel time (min)
- n = porosity (dimensionless)
- H = Head of water (ft or m)

Use consistent units for each variable.

This equation has been derived for downward flow in a saturated soil. However, the results of this equation in case of flow in an unsaturated soil will be conservative. Due to the soil in FDF site not getting saturated since the flood time is short, so steady seepage is not the case. Figure 78 shows the relationship between the travel distance and travel time of water for different water heads on the soil by using this equation.

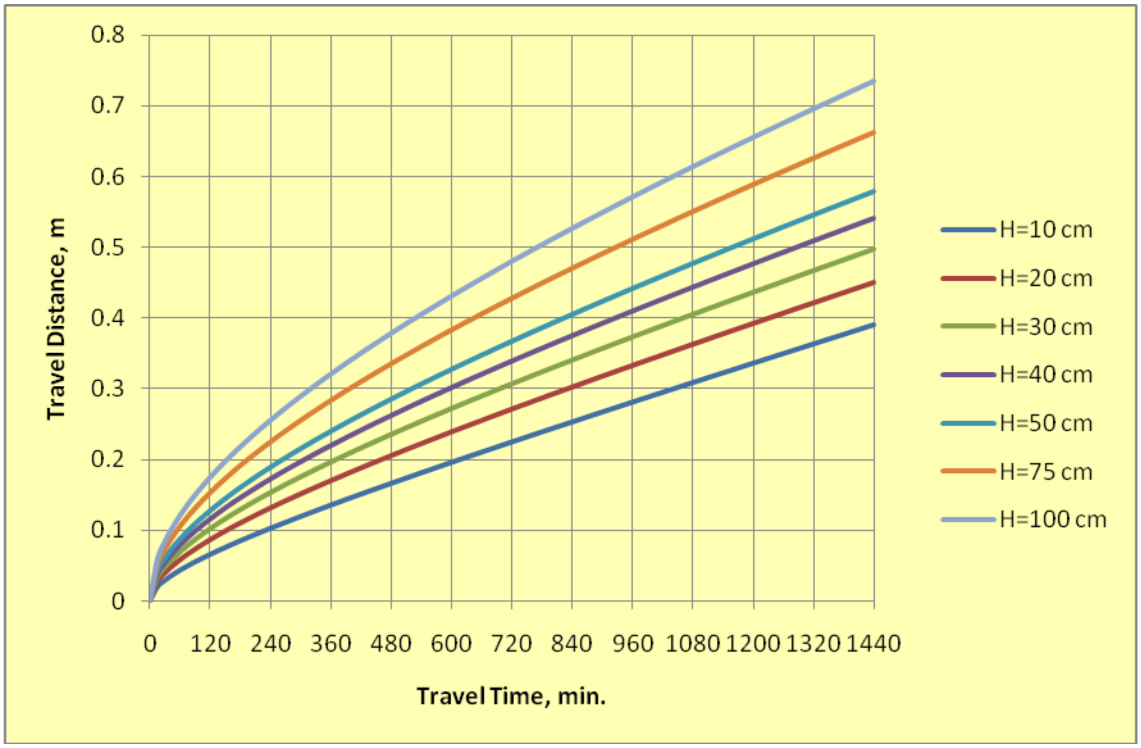


FIGURE 78 Travel distance versus travel time of water for downward flow in saturated soil

By using the results of Appendix G (Cross Sectional View of the End Structure) for H = 35 cm (13.78in.), a travel time of 12 hours, and a safety factor of 2, it is recommended to extend the geomembrane at least 70 cm (27.56in.) after the cut plane behind the structure. Length of the geomembrane in front of the structure is determined to be 155 cm (61.02in.) using a similar analysis when the channel is full. Geomembrane extension to the sides is considered to be 100 cm (39.37in.).

Overturning Factor of safety of the tire-bale structure against overturning moment due to the lateral earth pressure is determined at different levels. The results have been summarized in Table 12. In this table:

- F_a is active lateral force per unit width of the structure due to active lateral earth pressure,
- d_{Mo} is the moment arm of overturning forces,
- W_{tb} is weight of tire-bales in unit length,
- d_{Mr} is moment arm of the resisting forces,
- M_r is resisting moment, and FS is factor of safety.

In calculation of the safety factors, the weight of the soil on the tire-bales and the effect of the steel wire mesh have been ignored for more safety. All safety factors are greater than 1.50, which shows that the structure is stable against overturning.

TABLE 12 Factor of safety against overturning

Level, m*	F_a, kN/m**	d_{Mo}, m*	W_{tb}, kN/m**	d_{Mr}, m*	M_o, kN-m/m***	M_r, kN-m/m***	FS
-1.21	5.64	0.40	5.20	0.67	2.28	3.46	1.52
-2.13	17.49	0.71	15.60	1.33	12.42	20.75	1.67
-3.04	35.63	1.01	31.20	2.05	36.10	63.80	1.77

* 1m = 3.28084 ft

** 1kN/m = 68.6lb/ft

*** 1kN-m/m = 225lb-ft/ft

Sliding The active lateral earth pressure behind the structure is in equilibrium with the lateral earth pressure applied to the front face of the structure. Therefore, the structure is safe against sliding.

Bearing capacity Unit weight of the tire-bales is about 5 kN/m^3(31.85lb/ft^3), which is about one-third of the native soil unit weight, thus the load applied by the structure to its foundation is less than the weight of the native soil on the foundation. So the foundation is safe against the bearing capacity failure.

Uplift Pressure and Anchors The goal is to avoid developing extra pore water pressure behind and beneath the structure, though the weights of the structure and the soil cover are enough to resist against uplift pressure. However, the anchors (short piles) are designed to carry 50% of the uplift force when there is no soil cover on the structure. In this case, each pile is designed for a pullout force of 36 kN (8.1kips – 1 kip = 4.445kN). For the assumed soil parameters, four concrete bored piles, 30 cm (11.811in.) in diameter and 244 cm (8-ft.) in length are installed at four corners of the structure.

Slope stability Limit equilibrium analysis of the slope by using Slide computer program shows that there is no chance of slope failure for the assumed material properties and applied forces.

5.6 Finalized drawings

Appendix G shows the final design specifications for both the head-cutting and the side structures. Dimensions as well as drawing details were slightly changed during construction due to availability of materials and equipment, and different soil properties in higher depths compared to the assumed properties.

5.7 Construction and Analysis of the FDF

Construction of the tire-bale structures on the field demonstration facility at New Mexico Tech (NMT) occurred from May 10 to May 24, 2010. Two erosion control structures (tire-bale structures) were constructed according to the drawings presented in the previous quarterly reports and Figures79-82 above: a system for head-cutting and the second for side bank stabilization.

At the time of the report the head-cutting structure was complete, while the side structure still needed to be covered with either shotcrete or another type of material that would protect the geomembrane from UV (ultraviolet) radiation. As of August 2010 the total number of man-hours for both structures was approximately 137 (not including excavation for head structure or covering of the side structure's geomembrane with shotcrete). Of these, approximately 86 man-hours were spent on the head structure, and 45 on the side structure. The remaining time was spent on the construction of the flume, and the welding of the pile cap that failed during construction. The number of workers present at the construction site varied from 1 to 6 throughout the project, based on workers' availability. Construction of the flume was performed on the last construction day and required a total of 6 man-hours. The total number of actual hours spent on the construction was 32 for the head structure (not including excavation), and 18 for the side structure (not including covering the geomembrane with shotcrete or a protective material).

The following four pieces of equipment were used for the construction of the structures:

- a TH580B forklift,
- a 330C L hydraulic excavator,
- a 950G Series II wheel loader, and
- a RT 82-SC vibratory trench roller.

The forklift was only used to move the bales from their storage location closer to the construction area, while the excavator was used for excavating, backfilling, mixing soil, lifting and placing the tire bales, and driving piles once it was equipped with a jackhammer attachment. Additionally, the loader was also used for backfilling and mixing soil. While the vibratory trench roller was evidently used to compact the soil. Finally, a nuclear densitometer was brought to the site by New Mexico Department of Transportation (NMDOT) personnel to assure that 95% compaction was being achieved. Results showed that a compaction of 93% was achieved after 8 to 10 passes of the vibratory trench roller. Table 13 presents a summary of the daily construction activities that occurred between May 10 and May 24, 2010, along with their duration, the number of workers involved in the construction process, and the equipment used in each activity.

A meeting was held with both the research and the construction teams after the construction of both structures was nearly completed, so that the construction team could share their experiences, offer their recommendations, as well as point out the challenges faced during construction, and the differences between the design drawings and the actual structures built. One of the main differences was the manner used to anchor the tire-bales to the ground using cables and driven piles on either side of the structure. Because tying only the first layer with the cables was causing the tire-bales to rotate, lifting their inside edges, as shown in Figure 77, the construction crew decided to anchor the second layer instead, securing the bottom two layers of tire-bales at the same time. For this reason, two cables were used one on the near end and the second on the far ends of the second layer of tire-bales. A third cable was used to hold the top layer (Figure 78). Although L5"x5"x1/2" (12' long) angle iron was used for the piles, bending often occurred due to the slenderness of the piles and the lack of an appropriate piece of equipment. For this reason, although more expensive, solid bars, would have probably been more practical. For ease of construction, on the head-cutting structure, the piles were driven at an angle opposite to that of the design drawings, that is, instead of the outside piles being slanted under the structure towards each other, they were angled away from the structure and away from each other (Figure 78). This was rectified for the side structure, where piles were driven as specified.

Recommendations of the construction team included:

- the use of thicker (½") cable to allow the construction crew to apply tension without worrying about cable failure;
- the use of solid bars instead of angle iron for the piles to prevent bending or the use of better pile driving equipment.

Additionally, the construction team also recommended:

- alternate stitching of the gabion wires, assuring that both flaps are secured, as shown in Figure 79;
- to save time in cutting of the gabion wire, wrapping of the first and second layers of tire-bales were performed with a continuous strip of gabion wire following the pattern shown in Figure 80. Another piece was used to wrap the top row. Ties were placed on either side of the first and second rows to secure the tire layers.

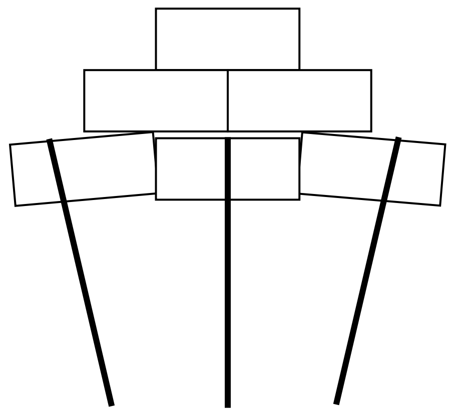
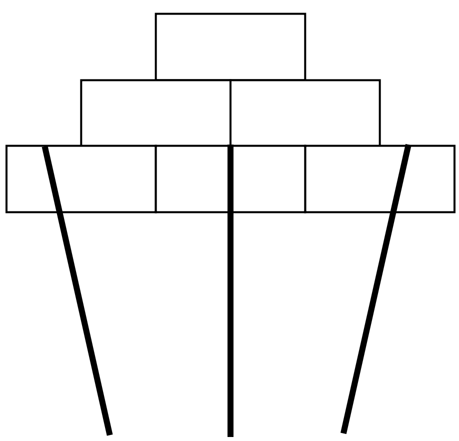
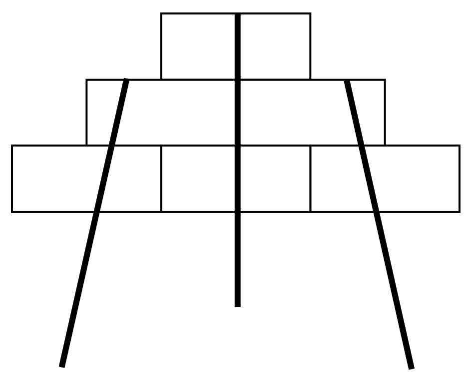


FIGURE 79 Observed Tilting of the Tire-bales Located on the Outside Edge of the First Layer



(a) Specified pile configuration



(b) Constructed pile configuration

FIGURE 80 Difference Between Specified and Constructed Pile Configuration for Head-cutting Structure

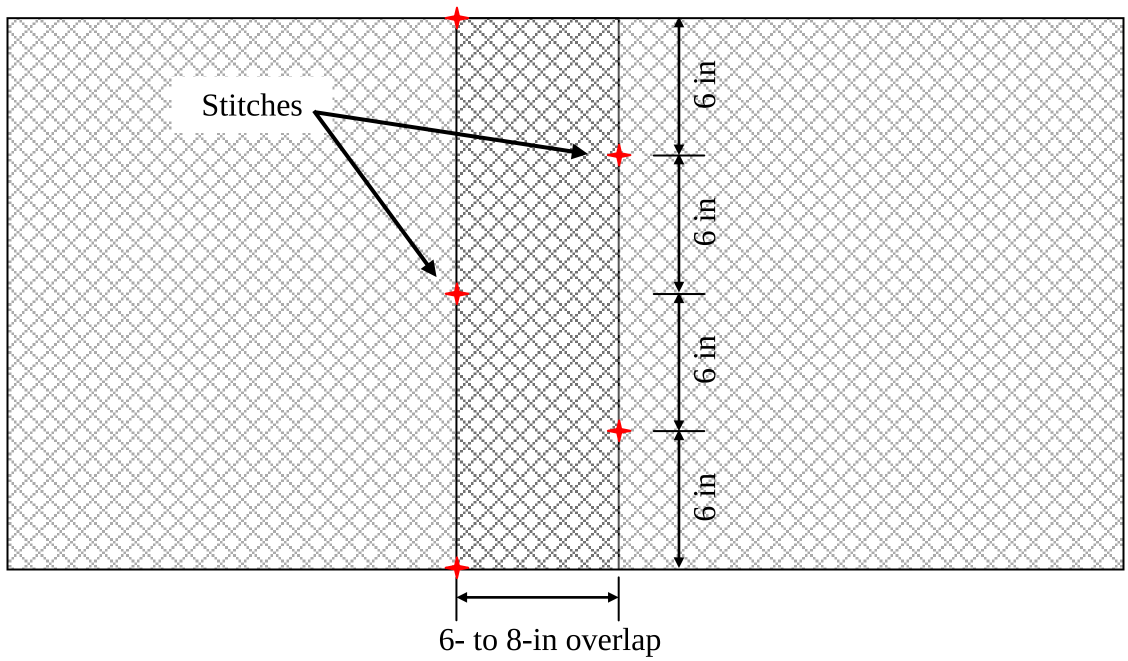


FIGURE 81 Recommended Gabion Wire Stitching (Top view)

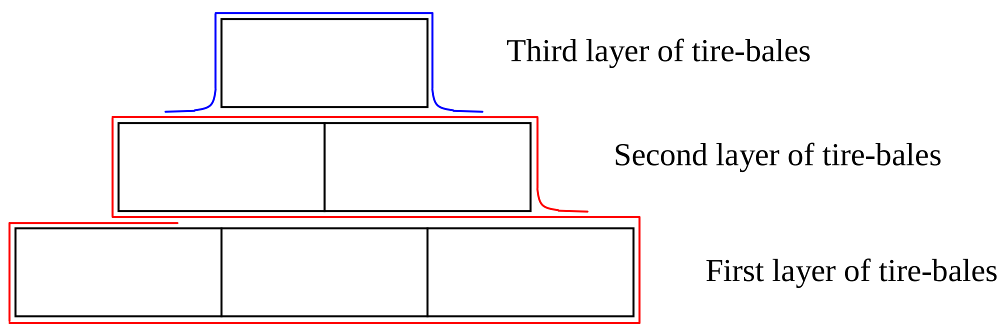


FIGURE 82 Recommended Gabion Wire Stitching (Side view)

TABLE 13 Tire-bale Construction Daily Schedule

Day	Start and End Times	No. of Hours (h)	No. of Workers	No. of Man Hours (h)	Equipment	Activity
5/10/2010	9:30 – 10:30	1	1	1	330C L hydraulic excavator	Excavation for side structure
	10:30 – 10:45	0.25	3	0.75	Water truck 330C L hydraulic excavator 950G Series II wheel loader RT 82-SC vibratory trench roller	Mixing of soil and water (with excavator) Backfill and compaction for side structure's foundation (loader and vibratory trench roller)
	10:45 – 12:00	1.25	3	3.75	Water truck 330C L hydraulic excavator RT 82-SC vibratory trench roller	Backfill and compaction of head structure's foundation (3 layers, each 6" thick)
	13:00 – 14:20	1.33	1	1.33	330C L hydraulic excavator	Excavation for channel
			2	2.67	Water truck 330C L hydraulic excavator RT 82-SC vibratory trench roller	Backfill and compaction of side structure's foundation
	15:00 – 15:30	0.5	1	0.5	Nuclear densitometer	Testing of soil compaction for head and side structures
	15:30 – 16:15	0.75	3	2.25		Preparation of gabion wire for head structure

TABLE 13 (continued) Tire-bale Construction Daily Schedule

Day	Start and End Times	No. of Hours (h)	No. of Workers	No. of Man Hours (h)	Equipment	Activity
5/11/2010	9:00 – 9:30	0.5	3	1.5		Tying of gabion wire for head structure (2 pieces, each 45 ft long)
	9:30 – 9:45	0.25	3	0.75		Placement of gabion wire for head structure
	10:00 – 10:30	0.5	2	1		Placement of tire bales for head structure (3 rows, 20 bales)
	10:30 – 11:00	0.5	3	1.5		Tying of gabion wire around 1st layer of tire bales for head structure
	11:00 – 11:30	0.5	3	1.5		Placement of 2nd layer of tire bales for head structure and tying of gabion wire around the bales
	11:30 – 12:30	1	3	3		Placement of 3rd layer of tire bales for head structure and tying of gabion wire around the bales
	12:45 – 13:30	2.75	3	8.25	330C L hydraulic excavator with jackhammer attachment	Driving of 4 piles (with jackhammer attached to excavator), approximately 6 ft deep, for the head structure
5/12/2010	9:00 – 9:30	0.5	2	1	330C L hydraulic excavator with jackhammer attachment	Driving of 2 piles (with jackhammer attached to excavator), approximately 6 ft deep, for the head structure. Trying of cables over tire bales for head structure
	9:30 – 10:45	1.25	3	3.75	Water truck 950G Series II wheel loader	Mixing of soil and water (with loader) for head structure's backfill

TABLE 13 (continued) Tire-bale Construction Daily Schedule

Day	Start and End Times	No. of Hours (h)	No. of Workers	No. of Man Hours (h)	Equipment	Activity
5/12/2010	11:00 – 12:30	1.5	3	4.5	Water truck 950G Series II wheel loader RT 82-SC vibratory trench roller	Backfill and compaction around head structure
	12:30 – 13:15	0.75				Site visit by NMDOT and NMT personnel
	13:15 – 15:30	2.25	3	6.75	Water truck 950G Series II wheel loader RT 82-SC vibratory trench roller	Backfill and compaction around head structure
5/13/2010	9:00 – 10:00	1	2	2	Water truck 330C L hydraulic excavator	Mixing of soil for head structure's backfill (with excavator)
	10:00 – 12:00	2	3	6	Water truck 330C L hydraulic excavator RT 82-SC vibratory trench roller	Backfill and compaction around head structure
	12:00 – 13:30	1.5	1	1.5	330C L hydraulic excavator	Excavation at the toe of the head structure for burying of geomembrane
	13:30 – 14:00	0.5	3	1.5		Placement of geomembrane over head structure

TABLE 13 (continued) Tire-bale Construction Daily Schedule

Day	Start and End Times	No. of Hours (h)	No. of Workers	No. of Man Hours (h)	Equipment	Activity
5/13/2010	14:15 – 14:30	0.25	2	0.5	330C L hydraulic excavator	Placement of non-expansive clay in front of head structure
	14:30 – 14:45	0.25	2	0.5	Water truck 330C L hydraulic excavator	Mixing of soil for head structure's backfill
	15:00 – 15:45	0.75	2	1.5	Water truck 330C L hydraulic excavator RT 82-SC vibratory trench roller	Backfill and compaction in front of head structure
5/14/2010	11:00 – 16:00	5	3	15	Water truck 330C L hydraulic excavator RT 82-SC vibratory trench roller	Backfill and compaction in front of head structure
5/17/2010	10:30 – 11:15	0.75	3	2.25	Water truck 330C L hydraulic excavator 950G Series II wheel loader	Mixing of soil and water (with excavator and loader) for head structure's backfill

TABLE 13 (continued) Tire-bale Construction Daily Schedule

Day	Start and End Times	No. of Hours (h)	No. of Workers	No. of Man Hours (h)	Equipment	Activity
5/17/2010	11:15 – 12:30	1.25	3	3.75	Water truck 950G Series II wheel loader RT 82-SC vibratory trench roller	Backfill and compaction in front of head structure
	13:00 – 14:00	1	1	1	330C L hydraulic excavator	Excavation for side structure
	14:15 – 15:00	0.75	3	2.25		Preparation of gabion wire for side structure (cutting and tying)
5/18/2010	9:00 – 9:30	0.5	3	1.5		Placement of gabion wire for side structure
	9:30 – 10:15	0.75	3	2.25	330C L hydraulic excavator	Placement of 1st layer of tire bales for side structure
	10:15 – 10:45	0.5	3	1.5		Tying of gabion wire around 1st layer of tire bales on side structure
	10:50 – 11:10	0.33	3	1	330C L hydraulic excavator	Placement of 2nd layer of tire bales on side structure
	11:10 – 11:50	0.67	3	2		Tying of gabion wire around 2nd layer of tire bales on side structure
	11:50 – 12:10	0.33	3	1	330C L hydraulic excavator	Placement of 3rd layer of tire bales on side structure
	12:30 – 13:30	1	2	2		Tying of gabion wire around 3rd layer of tire bales on side structure

TABLE 13 (continued) Tire-bale Construction Daily Schedule

Day	Start and End Times	No. of Hours (h)	No. of Workers	No. of Man Hours (h)	Equipment	Activity
5/18/2010	14:00 – 15:45	1.75	2	3.5	330C L hydraulic excavator with jackhammer attachment	Driving of 3 piles (with jackhammer attached to excavator), approximately 6 ft, for side structure (note: pile cap was consistently breaking)
5/19/2010	9:00 – 10:00	1	2	2	Water truck 950G Series II wheel loader	Mixing of soil for side structure's backfill
5/19/2010	10:00 – 11:00	1	1	1		Welding of pile driving cap
5/19/2010	11:00 – 12:00	1	3	3	330C L hydraulic excavator with jackhammer attachment	Driving of 3 piles (with jackhammer attached to excavator), approximately 6 ft deep, for side structure Tying of cables over tire bales for side structure
5/19/2010	13:15 – 15:15	2	4	8	Water truck 330C L hydraulic excavator RT 82-SC vibratory trench roller	Backfill and compaction around side structure
5/20/2010	9:00 – 9:20	0.33	3	1	Water truck 330C L hydraulic excavator 950G Series II wheel loader	Mixing of soil with excavator and loader for side structure's backfill

TABLE 13 (continued) Tire-bale Construction Daily Schedule

Day	Start and End Times	No. of Hours (h)	No. of Workers	No. of Man Hours (h)	Equipment	Activity
5/20/2010	9:20 – 9:45	0.42	3	1.25	Water truck 950G Series II wheel loader RT 82-SC vibratory trench roller	Backfill and compaction around side structure
	9:45 – 10:00	0.25	3	0.75		Placement of geomembrane
	10:00 – 10:45	0.75	3	2.25	Water truck 950G Series II wheel loader RT 82-SC vibratory trench roller	Backfill and compaction around side structure
	10:45 – 11:00	0.25	1	0.25	330C L hydraulic excavator	Excavation at the toe of side structure for burying of geomembrane
	11:15 – 11:30	0.25	3	0.75	Water truck 950G Series II wheel loader RT 82-SC vibratory trench roller	Placement of geomembrane. Backfill and compaction in front of side structure
	11:30 – 12:00	0.5	3	1.5	950G Series II wheel loader RT 82-SC vibratory trench roller	Placement and compaction of non-expansive clay on side structure

TABLE 13 (continued) Tire-bale Construction Daily Schedule

Day	Start and End Times	No. of Hours (h)	No. of Workers	No. of Man Hours (h)	Equipment	Activity
5/21/2010	9:00 – 9:15	0.25	1	0.25	330C L hydraulic excavator	Excavation around head structure
	9:20 – 11:00	0.67	6	4	Water truck 950G Series II wheel loader	Mixing of cement, soil and water for covering head structure
5/21/2010	9:20 – 9:45	0.42	1	0.42	Water truck 330C L hydraulic excavator RT 82-SC vibratory trench roller	Backfill and compaction of non-expansive clay on head structure
	11:00 – 12:30	1.5	3	4.5	Water truck 330C L hydraulic excavator 950G Series II wheel loader RT 82-SC vibratory trench roller	Backfill and compaction of treated soil on head structure
	12:20 – 13:00	0.67	3	2	Water truck 330C L hydraulic excavator 950G Series II wheel loader RT 82-SC vibratory trench roller	Backfill and compact treated soil on side structure

TABLE 13 (continued) Tire-bale Construction Daily Schedule

Day	Start and End Times	No. of Hours (h)	No. of Workers	No. of Man Hours (h)	Equipment	Activity
5/24/2010	9:30 – 10:30	1	2	2		Casting slab for flume
	10:30 – 12:30	2	1	2		Constructing flume walls
	12:30 – 13:30	1	2	2		Constructing flume walls

Finally, due to the high cost of cement and the large area to be covered with treated soil, the construction team was unable to obtain a treated soil mixture of 10% cement. Approximately 3% was actually used on the two sites. In addition, a top layer of only 6 in was achieved in the field, instead of the 12 in recommended in design. However, it is important to note that the design team strongly recommends that a 12 in layer of soil treated with 10% cement be used in the construction of all future structures.

Since breaks are expected every 20ft along the structure, the research team believes that the process employed in the field demonstration facility structures could be used repeatedly in the field. The anticipated tasks would follow the flowcharts presented in Figures 83 and 84. Because structures built in the field are expected to frequently extend 100ft or more, it is also expected that the construction process will be accelerated as workers gain familiarity with the procedure. However, as an estimate for initial costs, the times recorded during construction of the field demonstration structures will be used. The values obtained are considered conservative because although the workers were experienced in construction projects, this was their first time working with tire bales. Estimated costs for labor, material and equipment are presented in Tables 14 and 15. Labor rates were estimated at $21.87/hour based on the Bureau of Labor Statistics *(61)* reported average earnings of nonsupervisory construction workers in 2008 which is higher than the average hourly rates for construction equipment operators of heavy and engineering construction ($20.02/hr). The estimated number of man-hours for the head structure was 88, which includes 2 hours for excavation; unfortunately, the exact time spent on this activity was not officially recorded. Construction time was estimated at 1 week for the head structure (actual construction time was 34 hours, including 2 hours for excavation) and this value was used to estimate equipment rental cost. For the side structure, 45 man-hours were needed, not including the time required to shotcrete the geomembrane. Equipment rental was estimated at half a week since the construction only took 18 hours. Final costs were estimated at $ 14,255.24 for the 20-ft section of head-cutting structure and $ 14,065.33 for the 20-ft section of side-cutting structure.

TABLE 14 Estimated Initial Costs for a 20-ft Section of Head-cutting Tire-bale Structure.

(Conversion: 1ft = 0.3m)

Cost Category		**Unit Cost**	**Quantity**	**Total Cost**
Labor		$ 21.87/hr	88 man-hr	$ 1,924.56
Equipment				
1	330C L hydraulic excavator	$ 3,170.00/week*	1 week	$ 3,170.00
2	950G Series II wheel loader	$ 2,540.00/week*	1 week	$ 2,540.00
3	RT 82-SC vibratory trench roller	$ 1,163.00/week	1 week	$ 1,163.00
4	TH 580B forklift	$ 1330.00/week*	1 week	$ 1,330.00
Material				
1	Tire bales (transportation and loading only)			$ 450.00
2	Double twisted galvanized mesh 12ft x 150ft	$ 832.50 / roll	1 roll	$ 832.50
	Tie Wire (12-1/2 gauge galvanized wire)	$ 0.18 / lb	15 lb	$ 2.70
	Piles (angle iron L 5x5x3/4)	$ 212.40 / pile**	6 piles	$ 1,274.40
	$^{5}/_{16}$ aircraft cable	$ 0.77 / ft	100 ft	$ 77.00
	$^{5}/_{16}$ cable clamps	$ 1.09 / unit	5 units	$ 5.45
	$^{5}/_{16}$ zinc cable clips	$ 0.59 / unit	7 units	$ 4.13
	30 mil PVC geomembrane panel (50'x70')	$ 1,565.00 / roll	$^{1}/_{2}$ roll	$ 782.50
	Cement	$ 11.65 / 94 lb sack	60 sacks	$ 699.00
Total:				**$ 14,255.24**

* equipment owned by New Mexico Tech, cost estimated by construction crew

** donated material, cost estimated based on prices available online

TABLE 15 Estimated Initial Costs for a 20-ft Section of Side Tire-bale Structure

Cost Category		Unit Cost	Quantity	Total Cost
Labor		$ 21.87/hr	45 man-hr	$ 984.15
Equipment				
1	330C L hydraulic excavator	$ 3,170.00/week*	1 week	$ 3,170.00
2	950G Series II wheel loader	$ 2,540.00/week*	1 week	$ 2,540.00
3	RT 82-SC vibratory trench roller	$ 1,163.00/week	1 week	$ 1,163.00
4	TH 580B forklift	$ 1330.00/week*	1 week	$ 1,330.00
Material				
1	Tire bales (transportation and loading only)			$ 450.00
2	Double twisted galvanized mesh 12ft x 150ft	$ 832.50 / roll	1 roll	$ 832.50
3	Tie Wire (12-1/2 gauge galvanized wire)	$ 0.18 / lb	15 lb	$ 2.70
4	Piles (angle iron L 5x5x3/4)	$ 212.40 / pile**	6 piles	$ 1,274.40
5	$^{5}/_{16}$ aircraft cable	$ 0.77 / ft	100 ft	$ 77.00
6	$^{5}/_{16}$ cable clamps	$ 1.09 / unit	5 units	$ 5.45
7	$^{5}/_{16}$ zinc cable clips	$ 0.59 / unit	7 units	$ 4.13
8	30 mil PVC geomembrane panel (50'x70')	$ 1,565.00 / roll	$^{1}/_{2}$ roll	$ 782.50
9	Cement	$ 11.65 / 94 lb sack	30 sacks	$ 349.50
10	Shotcrete			$ 1,100.00***
Total:				**$ 14,065.33**

* equipment owned by New Mexico Tech, cost estimated by construction crew

** donated material, cost estimated based on prices available online

*** estimated cost

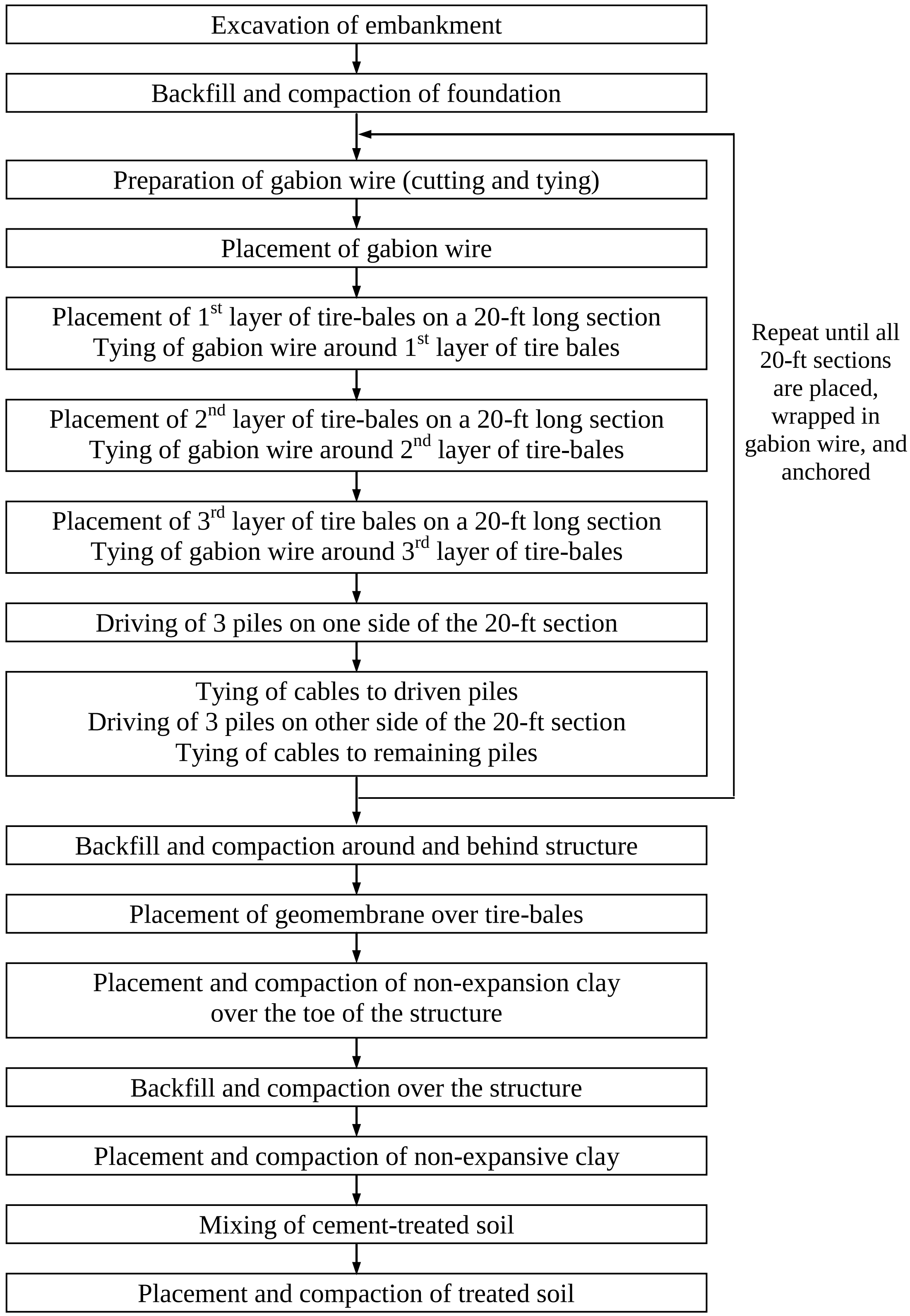


FIGURE 83 Construction Process for Head-cutting Tire-bale Structures

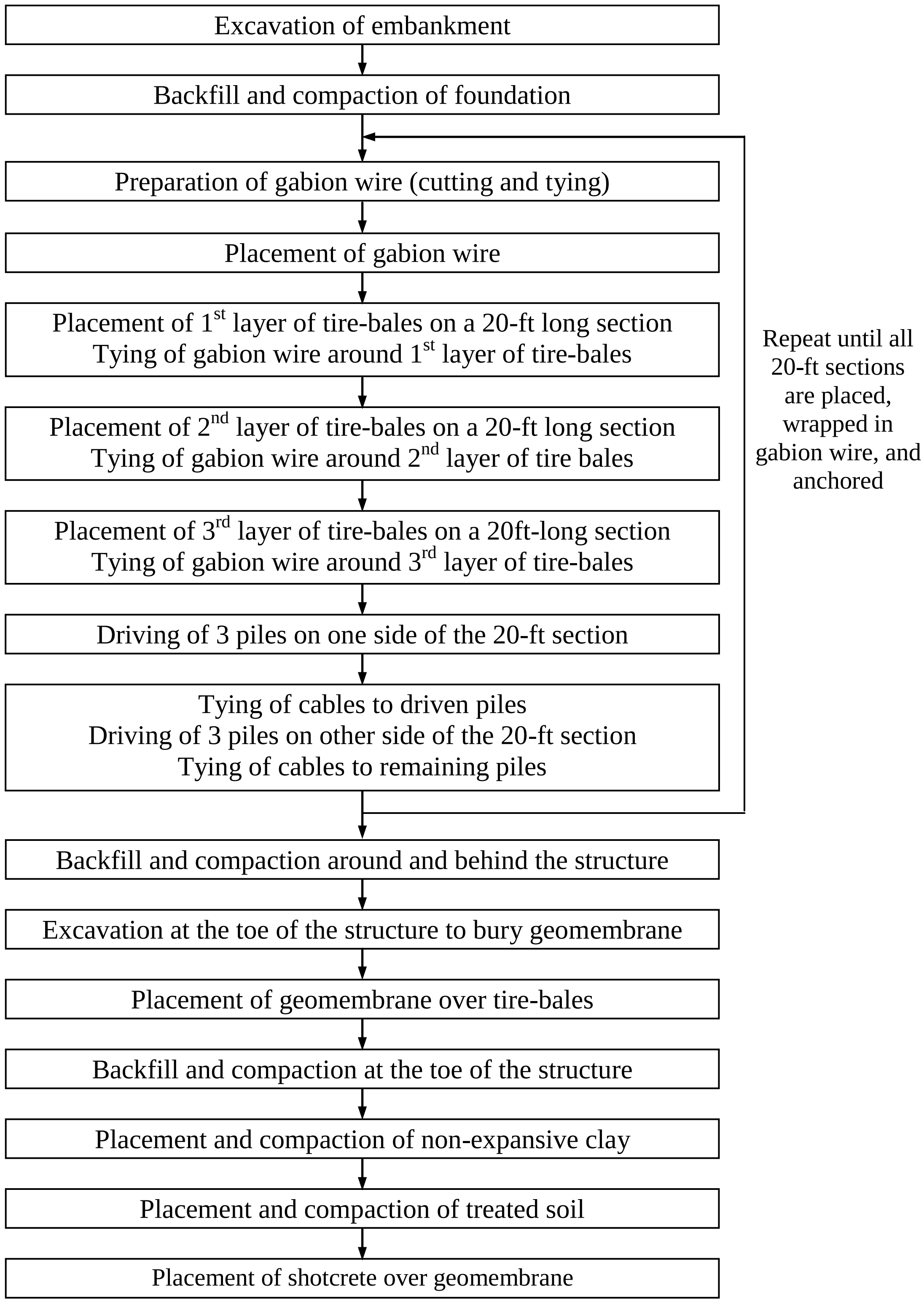


FIGURE 84 Construction Process for Side Tire-bale Structures

5.8 Instrument Calibration

The presence of water is a major concern over time in and around the tire-bale environment, therefore the relationship between soil moisture and tire-bale structure needs to be understood. The tire bale structure was designed to be impermeable to water; this was achieved by placing geomembrane and treated soil over the entire structure. To investigate the efficiency of the geomembrane and treated soil in preventing water from percolating through the structure, a total of twenty moisture sensors (Figure 85 and 86) are placed at different location on the head-cutting structure to measure the soil moisture content. The Vegetronix VG400 soil moisture sensors are used along with the LOGGER-8-USB (Figure 87). Also a WL 16U Water Level Loggers was installed in the flume to measure the amount of water flowing onto the structure. By knowing the amount of flow onto the structure and the soil moisture content, the researchers will be able to determine the effectiveness of the geomembrane and treated soil.

The VG400 moisture probes measure the dielectric constant of the soil using transmission line techniques making it insensitive to water salinity, and will not corrode as does conductivity based probes. Soil moisture sensors are highly accurate, low cost and low power probes enabling precise monitoring of soil water content (62). Some of the features of the VG400 moisture probes are:

- Not conductivity based.
- Insensitive to salinity.
- Probe does not corrode over time.
- Rugged design for long term use.
- Small size.
- Consumes less than 600µA for very low power operation.
- Precise measurement.
- Measures volumetric water content (VWC) or gravimetric water content (GWC).
- Output Voltage is proportional to moisture level.
- Wide supply voltage range
- Can be buried and is water proof.
- Probe is long and slender for wider use
- Operational Temperature of -40^{o}C to 85^{o}C

The LOGGER-8-USB logs the measurement of the VG400 sensors probes and was calibrated to measure the soil moisture content at every hour for the tire-bale structure. The frequency of measurement can be altered depending on the data needed. In the case of the FDF's remote and secure location greater frequency was deemed necessary. The data loggers are powered with a 9V DC battery. The commands for calibrating the LOGGER-8-USB are provided in Appendix H.

The data logger is low-cost and perfect for reading up to 8 Vegetronix soil moisture sensors. It relates sensor data back to a host computer via USB, or it can be a standalone system to store data to non-volatile memory to retrieve data in the future. Setup is very quick, with screwless terminal blocks. When connected via USB, the device is powered from the USB cable. It can also be powered from batteries with a wide supply range of 5 to 24V.

Figure 85 VG400 Soil Moisture Sensor

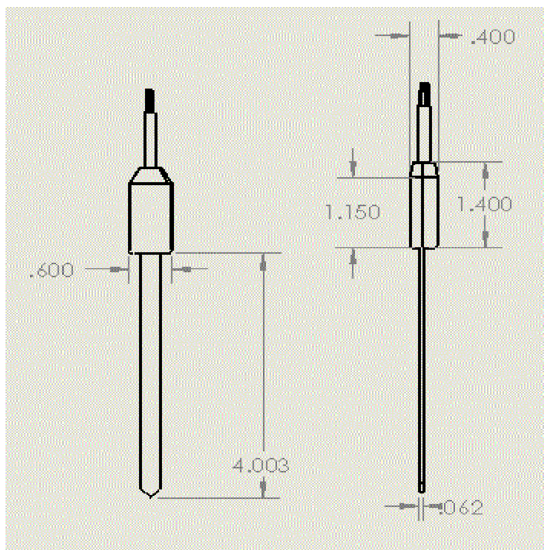


FIGURE 86 Dimensions of the VG400 Soil Moisture Sensor

FIGURE 87   LOGGER-8-USB: 8 Input Data Logger

The sensor channel uses 3 ports on the terminal block for power, ground and the sensor input. The logger will turn on power to all sensors simultaneously with a programmable setup time. The device looks like a serial port to host PC systems so any communications software program such as HyperTerminal can be used to control it and gather data. HyperTerminal is standard to all windows system, thus, no additional host software will be needed to purchase. Data is returned in a comma separated format, for easy manipulation in spreadsheet programs. Channels are easily specified and can be calibrated with a 2 to 10 point calibration curve for non-linear devices. The logger consumes only 85μA when it is not sampling data, so it is perfect for battery powered applications (63). A summary of the LOGGER-8-USB features are:

- Monitors up to 8 sensors.
- Streams data to a PC, or saves to internal non-volatile memory.
- Real time clock.
- Low power for battery powered operation.
- Low cost.
- Compact.
- Screwless terminal blocks.
- Flanged box for easy wall mounting.
- Can be powered from USB port, or from a secondary source.
- Upgradeable firmware through USB port.
- Each channel can be calibrated with up to a 10 point curve.

The WL 16U level logger (Figure 88) is a submersible pressure transducer with USB data logger, designed for highly accurate remote monitoring and data logging of water level or pressure. The water logger has four unique samples modes with programmable intervals of (1sec to a year) and it can record multiple depth ranges from 3–500 feet of water level change. The WL16, Water Level Logger, is housed in a weather-resistant cylindrical enclosure (64). The WL16 Level Logger can measure the depth of the water, the temperature, the flow rate and the velocity. Some features of the WL16 Level Logger are:

- Four sample modes: 10 times per second, timed, logarithmic, and exception
- CE Certified
- User friendly software included
- USB and Serial communication options available
- Serial version is telemetry compatible
- PDA software simplifies field data collection
- No need to remove sensor for data collection or battery change
- Highly accurate water level measurements
- User programmable start and stop alarms, engineering units, and field calibration setup
- Unique 0-3' range for shallow water
- Wet-wet transducer eliminates vent tube concerns
- Automatic barometric pressure and temperature compensation

The figures 89 and 90 are typical views of the setup screen. The WL16 level logger can be programmed to measure the depth and temperature of the water at any time interval. The flow calculations can then be determined depending on the type of the structure being monitored. Currently the Logger is set to record data at 1 hour interval and can be altered as required. The WL16 Level Logger and the LOGGER-8-USB have been calibrated and will provide valuable data for the researcher team to monitor the tire bale structure. With these tools the researcher team will have a better understanding on the relationship between the moisture and the tire bale structure. These data loggers will be housed in an all-weather protective box that prevents weather and any other factors from damaging or destroying the data loggers. Once powered, the data loggers will accumulate data which will be retrieved and analyzed by the researcher team.

FIGURE 88 Submersible Pressure Transducer and USB Data Logger

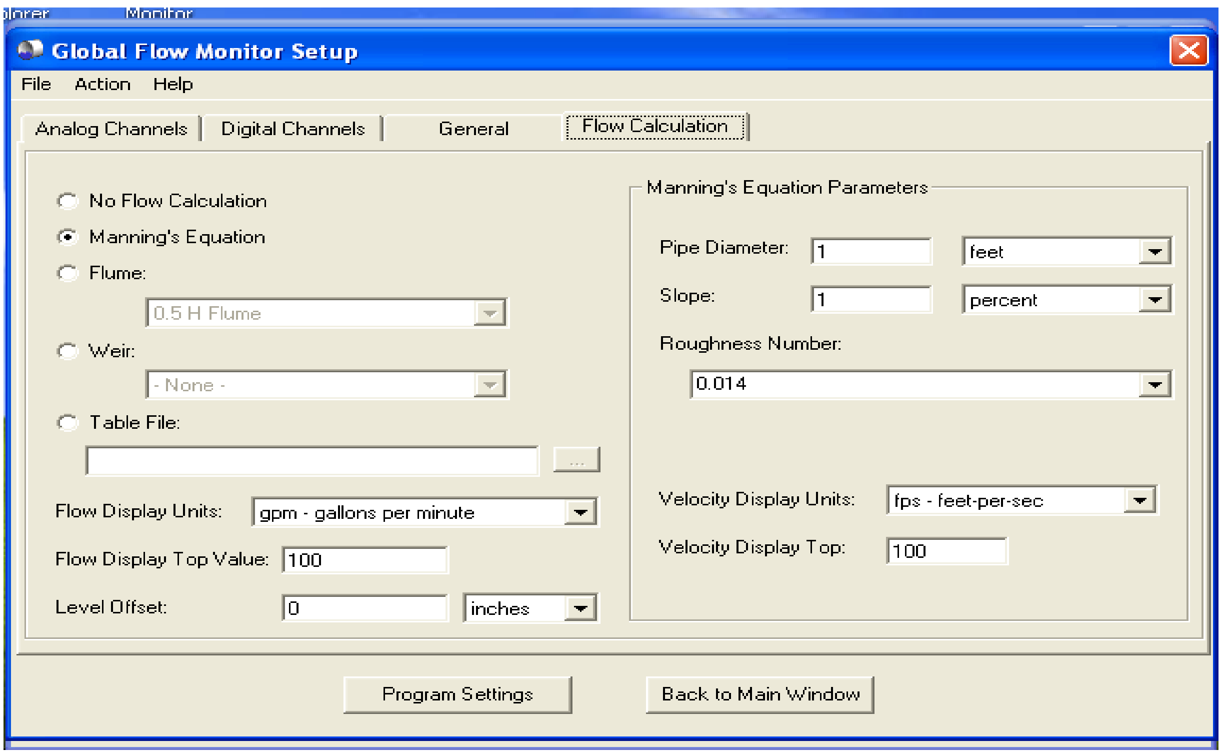


FIGURE 89 Setup Screen View 1

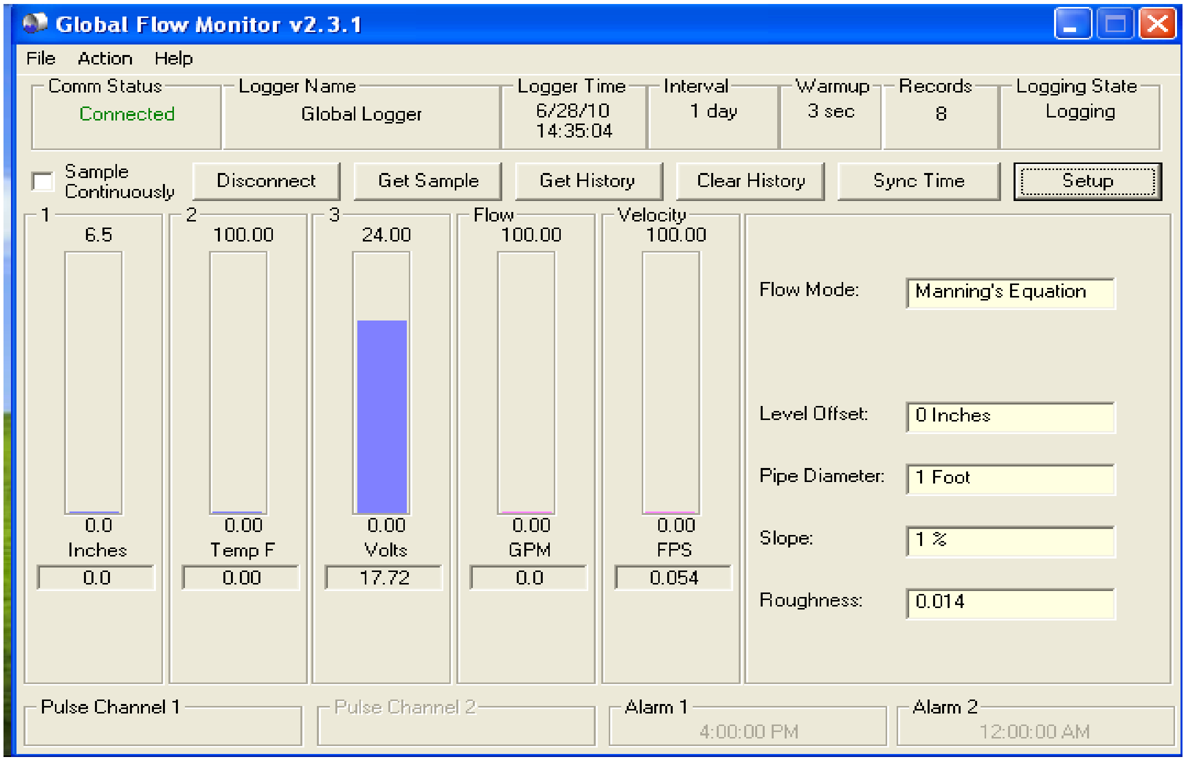


Figure 90 Setup Screen View 2

6.0 TASK 6:

"The research team shall develop a robust analytical procedure which will facilitate a parametric study of tire-bale systems *(1)*."

The following analysis reflects the potential performance of tire-bale structures similar to those currently deployed by NMDOT at the sites on NM 52 and NM 152, The analysis is also a good baseline analysis for a structure modeled after that which has been presented in the FDF associated with this project; when used to model the latter configuration it represents a worst-case scenario.

The design of a tire-bale erosion control structure parallels the design of a gravity retaining wall. It is complicated by the fact that the specific gravity of an individual tire-bale is approximately 0.51. This will severely affect the resistance to both sliding and overturning failure. Additionally, tire-bale structures are by no means monolithic; the individual bales, and rows of bales, can move with respect to one another. This must be considered in their analysis.

Before outlining a design procedure, one must first set ‘ground rules’.

- The use of tire-bale structures for erosion control in any but ephemeral streams is discouraged
- The tire-bales used should be of a uniform size, within allowable tolerances
- The tire-bale ‘assemblage’ shall be wrapped in gabion wire
- The structure will be anchored into the soil using cables connected to steel angles acting as driven piles, or their equivalent

There are additional recommendations which have not been used on previous New Mexico tire-bale projects, but which are recommended by the present research

- The tire-bale structure should be wrapped in geomembrane to prevent water intrusion.
- The tire-bale structure should be placed on a drainage bed of gravel if the soil has the potential of remaining moist.
- The tire-bale structure should be covered with reinforced soil (i.e., native soil mixed with not less than 10% Portland cement at a thickness of 12 inches)

The use of a geomembrane is intended to prevent water from collection in the void spaces of the structure, and, more importantly, from collecting in the backfill and therefore applying hydrostatic pressure to the structure. It is recognized that this may not be economically feasible in all cases; further, if the geomembrane could be compromised by mechanical damage or deterioration, water intrusion will occur. The following design methodology will therefore include analyses which assume that the soil behind the structure can become saturated.

The design of a tire-bale structure begins with the determination of the required height, and knowledge of the backfill density and slope. In most cases native soil will be used, though ideally, approved cohesionless backfill material should be chosen.

Whatever backfill material is used, proper compaction (around 95%) should be attained to prevent the development of soil distress and failure in the backfill.

In beginning the analysis, one must first consider the active soil pressure provided by the backfill. The Rankine model is most suited for ‘low’ structures’. The coefficient of active soil pressure is given as K_a, with ϕ as the soil’s angle of internal friction.

$$K_a = {}^{1-\sin\phi}\!\big/_{1+\sin\phi}$$

For most sandy soil, an internal friction angle of 30° can be assumed, giving K_a=0.333. For sloping backfill, the value of K_a decreases with increasing slope.

The effective vertical overburden pressure must now be considered. It is the effective weight of the soil above the point of consideration, and will relate to the lateral soil pressure. The effective vertical pressure from soil and groundwater is

$$\sigma_v^{'} = z\gamma + z^{'}\gamma^{'}$$

In the above equation z is the depth of soil, γ is the unit weight of soil, $z^{'}$ is the groundwater depth, and$\gamma^{'}$ is the weight of water (when the possibility of water intrusion is considered).

The lateral soil pressure at a depth z is equal to the vertical soil pressure times K_a. Therefore, the pressure in dry soil at the base of a wall of height H is given as

$$\sigma_a = K_a \gamma z$$

If a linear pressure distribution with depth is assumed, the pressure distribution for a wall with flat backfill and no surcharge loads can be modeled as a triangle with a base equal to the pressure at the base of the wall, and a pressure of zero at the top. The total force acting on the wall is equal to the integration of this distribution; i.e., the area of this triangle.

$$P_a = \frac{1}{2} K_a \gamma H^2$$

It acts through the centroid of the triangle, at a height H/3 from the base of the wall.

For a tire-bale structure, the greatest threat will come during the time an ephemeral stream is flowing. This may mean that if the geosynthetic is compromised, the backfill may become partially or fully saturated, and therefore water pressure behind the wall must be considered. Water pressure is the same in all directions, so its pressure coefficient is 1, and the pressure simply increases as the unit weight of water (γ_w = 62.4 pcf) times the depth. Therefore, the force from water pressure is

$$P_w = \frac{1}{2} \gamma_w H^2$$

Again, acts at a height of H/3 from the base of the wall.

Since the lower course of tire-bales typically have their tops flush with the level of the stream bed, passive soil pressure can also be developed as the backfill 'pushes' the tire-bales. The Rankine coefficient for passive pressure is

$$K_p = {}^{1+\sin\phi}\!\big/_{1-\sin\phi}$$

A typical value for sandy soil with ϕ=30° is K_p = 3.0. The passive force is calculated in a manner to that outlined above, with H_p being the height of soil on the 'passive' (i.e., stream-bed) face of the wall.

$$P_p = \frac{1}{2} K_p \gamma H_p^{\,2}$$

The forces are typically used in units of force per lineal foot along the length of the wall.

The active and passive pressures are used in two analyses: sliding failure, and overturning failure. Sliding failure simply balances the horizontal forces against the wall's frictional resistance to sliding; in the case of a tire-bale structure with a 'buried' bottom course of bales, a sliding failure is very unlikely unless the bottom bales are exposed by scour in the stream-bed. An overturning failure calculates the overturning moment created by the active soil force, resisted by the walls weight and the moment (usually small, and often neglected) generated by passive soil force. Rotation is assumed to occur about the toe of the structure, i.e., the lowest point of the bottom course of bales on the stream-bed side.

In the case of tire-bale structures, however, there is one more potential distress mode – that of a 'deformation failure', in which one row of bales moves relative to the row below it. The picture is complicated by the fact that the bales are wrapped *en bloc* in gabion wire, and further secured by cable anchorage to the steel-angle piles spaced about 20-ft apart. The effect of these factors cannot be easily quantified. The reason is that the nature of installation can lead to variations in effect, and that the properties will vary depending upon where one is looking; at the midpoint of the cable the anchorage system will be marginally effective in resisting local sliding and moment deformations. For this reason the structure should first be analyzed without consideration of the anchorage system.

For the conservative analysis of a tire-bale structure, the following procedure is recommended, using saturated soil in all cases:

1. Sliding failure analysis, with the assumption that the bottom bales may be exposed by scour

2. Overturning failure

3. Deformation failure, with sliding forces acting on upper rows of bales without consideration of gabion wire or cable anchorage.

In each case factors of safety are calculated. Inadequacies should be addressed by detail design of anchorages.

Design Example Consider a tire-bale structure three bales high, with sandy backfill having a unit weight of 100 pcf (Figure 91). The possibility of backfill saturation is assumed. The tire-bales have a unit weight of 32 pcf, and are nominally 2.5 ft high and 5 ft square in plan view. Anchorage and the effects of gabion wire will not be considered.

In this case, H = 7.5 ft, and therefore we have

- P_a = 928 lbs per lineal foot of wall
- P_w = 1775 lbs per lineal foot water pressure
- P_p= 938 lbs per foot of wall
- Weight of bales is 400 lbs per lineal foot per bale

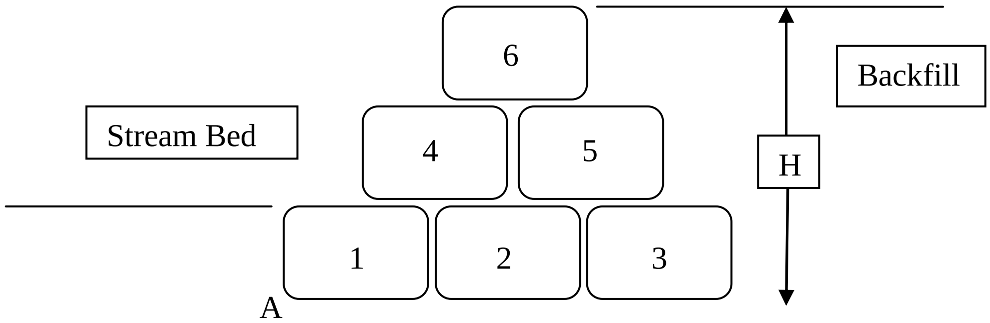


FIGURE 91 Tire-bale structure for design example

For sliding analysis of the entire wall, we can use a coefficient of friction of 0.75 between the tire-bales and the underlying soil:

- Horizontal force under dry conditions is 928 lbs
- Horizontal force under saturated-backfill conditions is 928+ 1775 lbs = 2703 lbs
- Resistance due to tire-bale friction is (400 lbs/bale) x 6 bales * 0.75 = 1800 lbs
- Passive pressure is 938 lbs

Under dry conditions, with no scour, the sliding force is 928 lbs, and the total resisting force is 2438 lbs, giving a factor of safety of 2438/928 = 2.62, which is adequate. Under dry conditions with the bottom tire-bales exposed by scour, the resisting force drops to 1800 lbs, giving a factor of safety of 1.93, which is adequate.

Under saturated-backfill conditions without scour, the sliding force is 2703 lbs, and the resisting force, including passive pressure, is 2438 lbs; the factor of safety is less than unity and the structure is in danger of sliding failure. Clearly, this situation becomes worse if the bottom bales are exposed by scour.

Overturning analysis considers overturning of the structure about its toe, point 'A' in Figure 92.

- Overturning moment of P_a under dry conditions is (7.5 ft/3) x 928 lbs = 2320 lb-ft
- Overturning moment of saturated soil is (7.5 ft/3) x 2703 lbs = 6758 lb-ft
- Resisting moment of tire-bale 1 is 400 lbs x 2.5 ft = 1000 lb-ft
- Resisting moment of tire-bale 2 is 400 lbs x 7.5 ft = 3000 lb-ft
- Resisting moment of tire-bale 3 is 400 lbs x 12.5 ft = 5000 lb-ft
- Resisting moment of tire-bale 4 is 400 lbs x 5 ft = 2000 lb-ft

- Resisting moment of tire-bale 5 is 400 lbs x 10 ft = 4000 lb-ft

- Resisting moment of tire-bale 6 is 400 lbs x 7.5 ft = 3000 lb-ft

- Total resisting moment is 1000 + 3000 + 5000 + 2000 +4000 + 3000 = 18,000 lb-ft

- Passive soil force contribution is ignored

Under dry conditions, the factor of safety against overturning is 18000/2320 = 7.75; under saturated-backfill conditions the factor of safety is 18000 / 6758 = 2.66. Both are entirely adequate.

The deformation condition of movement of individual rows of tire-bales should now be considered (Figure 92).

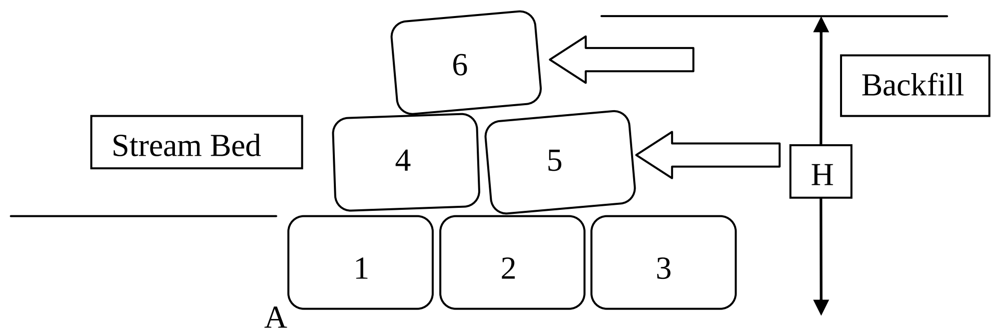


FIGURE 92 Movement of top-row tire-bales

We can calculate sliding of the top row of tire-bales (bale 6). The coefficient of friction is assumed at 0.75.

- Active soil pressure under dry conditions is 94 lbs

- Active soil pressure under saturated conditions is 94 + 195 = 289 lbs

- Resistance to sliding friction is 400 lbs x .75 = 300 lbs

The top row of tire-bales has a factor of safety against sliding of 3.19 in dry conditions, and 1.03 in saturated conditions. Under dry conditions the factor of safety is fine; it is unacceptably close to potential failure under saturated-backfill conditions

Finally we analyze sliding of the top two rows of tire-bales (bales 4-6).

- Active soil pressure under dry conditions is 375 lbs

- Active soil pressure under saturated-backfill conditions is 375 + 780 = 1155 lbs

- Resistance to sliding friction is 3 x 400 lbs x .75 = 900 lbs

Under dry conditions, the factor of safety is 2.4, which is acceptable. If the backfill is saturated, the factor of safety is 0.78, which is not acceptable.

In summary, we can see that the greatest danger to the structure comes when saturated backfill is acting on the top two rows of tire-bales. If this is the case the tire-bales *will* push forward against the restraining gabion wire. If the wire is not tight enough, and/or this is taking place at the midpoint of the cable between anchors, large deformations can develop which will 'cascade' into an overturning failure as the moment arm of the affected tire-bales (about the toe of the structure) is reduced.

To improve this design, anchorage should be specifically designed for the vulnerable failure modes identified. It should be noted that vertical anchorages (to steel-angle piles) will effectively increase the frictional force *only* if they are tensioned to apply compressive force to the tire-bales. A far more effective anchorage would use 'deadmen' buried in the backfill, with cables acting horizontally to restrain sliding of the tire-bales (Figure 93).

Using 3/8-in Galvanized cable (break strength 12,000 lbs) spaced at 10-ft (every two tire-bales) on the second row of tire-bales and running back to deadmen in the backfill, the additional restraint would be 12,000 lbs/10 ft = 1200 lbs per lineal ft. This would increase the resisting force to 2100 lbs, versus a sliding force of 1155 lbs, giving a factor of safety of 1.81 against sliding.

The top row of tire-bales is also vulnerable. If we space 3/8-in cable at 20 ft, this would increase the resisting force to 900 lbs, against a saturated-backfill sliding force of 289 lbs, increasing the factor of safety to more than 3 (from 1.03).

The best choice for deadmen is used 10-ft Jersey barriers. These can be spaced on 20-ft centers; for the middle row cables, attachment can be made at each end of the barrier for the required 10-ft spacing. For the top-row cables, attachment can be made at the center of each barrier for the required 20-ft spacing. This will effectively stagger the cables, which is structurally advantageous.

The maximum soil pressure that will be exerted through the cables is 36,000 lbs, which will be spread over the 30-ft flat plate area of the barrier. This will result in a lateral soil pressure of 1200 psf, which should be well within the allowable bearing capacity for well-compacted backfill.

The approximate cost of this option is $11 per lineal foot of wall (this assumes a price of $150 per used Jersey barrier and $146 per 200 ft of cable).

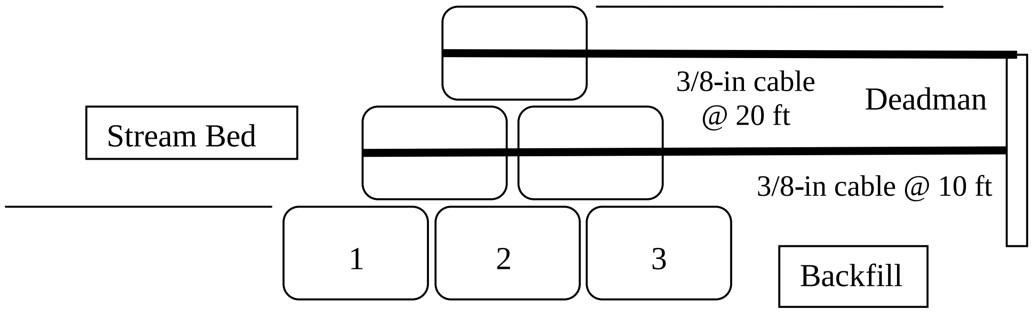


FIGURE 93 Design example final design

7.0 TASK 7:

The design of the tire-bale structure for erosion control requires that the structure be stable under credible but unanticipated loading conditions. The primary concern is the intrusion of water behind the structure causing failure. The NM 143 in-field site demonstrated such a water intrusion failure. In the case of a structure designed to prevent head-cutting of an erosion feature or in lateral structures if drainage patterns change while the structure is *in situ* water intrusion can also be of concern.

The general assumption for a tire-bale structure protecting an existing stream bank against erosion is that the base course of tire-bales will be placed such that the top of the lowest tire-bale layer will be at the mean streambed level, and the top of the upper layer will be level with the height of the original bank. It is also assumed that the wall will be 'pyramidal' in cross section; each succeeding layer will have one less tire-bale than the layer beneath. The dimensions of each tire-bale is assumed at 5-ft square in plan, with an elevation of 2.5-ft.

Additionally, the New Mexico Research Bureau Technical Panel has requested a Phase II to further analyze the FDF and in-field sites. The proposed budget and timeline are documented in the Recommendations sections 7.3.3 – 7.7 below.

Sections 7.1 and 7.2 will summarize the results of the research into three subcategories: Current State of Practice, In-Field Structural Monitoring, and Design of Tire-bale Structures for Erosion Control and Bank Stabilization, Constructability of the Field Demonstration Facility and an Alternative Design.

7.1 Summary

This section will provide the reader a brief overview of the contents of this report. Each area below is separated into the six (6) tasks above in the report. It is highly recommended that the above tasks be viewed in more detail by the reader for better understanding of the design process and knowledge necessary for understanding the behavior of the tire-bale structures.

7.1.1 Current State of Practice

A comprehensive survey of the literature was conducted to identify available options for erosion control and bank stabilization. Descriptions, advantages, disadvantages, and suitability to different site conditions were compiled for several erosion control and bank stabilization alternatives. For simplicity, these alternatives were grouped in the following categories: stone armors, self-adjusting armors, flexible mattresses, rigid armors, and retaining structures.

A phone survey was conducted with U.S. State Department of Transportation personnel to determine the use and acceptability of each state's use of tire-bale structures. For quick reference each state and the contact person are listed in Appendix B. Also listed are the questions used in the phone survey.

In accordance with the use of tires in the U.S., a best practices environmental literature review was conducted. The review consisted of the Environmental Protection Agency, International Fire Code, State Department of Transportation, and university or professional organizations. The best practices have been listed in Table 1 of this report.

7.1.2 In-Field Structural Monitoring

Four existing tire-bale structures placed by NMDOT were monitored using total-station surveying methods over the life of the project. These structures were located at

- NM 52, MM 26.7
- NM 52, MM 27.2
- NM 52, MM 27.75
- NM 152, MM 47.7

Additionally, a structure on NM 143 was examined both before and after its storm-induced failure in July 2008. This structure was examined but not surveyed.

The structures on NM 52 were placed on the west side of an ephemeral stream running parallel to the road. They were placed on the 'road' side of the streambed, at distances ranging from 50-200 ft from the right-of-way. All structures were positioned to prevent side cutting.

The structure on NM 152 was on the west side of an ephemeral stream crossing NM 152, north of the culvert. The structure followed a ‘convex’ curvature around a terrain feature; that is, the ‘side cutting’ was on the opposite side of the channel.

The structures on NM 52 and NM 152 were two to three bales high over their length (depending on stream bank configuration); the top surface of the bottom bales can be considered level with the streambed. The structure on NM 143 was four (4) bales high, with the top surface of the bottom bale consistent with streambed elevation.

Survey points were established at approximately 20-ft intervals down the length of the tire-bale structures. Where possible, three points were used per cross-section (perpendicular to the length): in the backfill, on the top-row bale, and on the bottom-most visible bale (at the streambed level).

The survey points consisted of 10-in or 12-in spikes driven into predrilled holes in the tire corresponding to the appropriate location in each bale. Specific attention was given to make sure there were enough ‘folded’ layers of rubber at each location to ensure stability. The heads of each spike were indented with a center punch to hold the point at the bottom of the grade rod. The locations were marked with orange surveying flags.

An offset benchmark was used for surveying; that is, two benchmarks were placed. The total station was placed over one, and the measurements were taken relative to the second benchmark.

Surveys were taken at three time intervals over the life of the project. During this time several of the lower-level (streambed) survey points were buried and lost. Many of the flags were lost on the other higher points but all of the affected points were relocated.

The survey of the structure at NM 52 MM 26.7 showed that the bales moved slightly toward the streambed, while the backfill experienced some settling, moving its survey points away from the streambed. The entire structure displaced slightly upward over time. Displacements corrected for measurement error were on the order of 1-in or less.

The survey of the structure at NM 52 MM 27.2 showed that the structure consistently moved away from the streambed along its length, and that it experienced uplift, while the survey points in the backfill settled. The interpretation of the data indicates that the structure actually rotated back and away from the streambed. Deformations were on the order of 3-in or less.

The survey of the structure at NM 52 MM 27.75 showed that both the structure and backfill moved away from the streambed over time, and experienced almost no vertical deformation. Deformations were on the order of 2-in or less.

The survey of the structure at NM 152 MM 47.7 showed that the structure experienced almost no lateral deformation, but that there was measureable uplift which was consistent over the length of the structure. Lateral deformations were close to zero, and vertical deformations were 1-3 inches.

The structure at NM 143 was unique among these in that it was an open-ended ‘box’ channeling an erosion feature on the downstream side of a road. The structure was an elongated rhomboid in plan, with the long sides along the north and south sides of the channel. The east side was parallel to the road, and about 8-10 ft from the pavement edge. The open end faced west. Design water flow was east-west. The structure was built per NMDOT practice, anchored with steel angles and covered with gabion wire. As such it was subject to sheet flow across the road, and down the road axis along the road's bar ditches. It failed during a storm event; one of the sides of the box experienced overturning from hydrostatic pressure that built up behind it. The bottom rows of bales remained in place, while the top rows, still incased in the gabion wire, rotated into the channel over the top of the bottom bales.

Visual observations of each in-field structure were also done. Over the time of the study significant changes in the depth of sedimentation in the channel was observed at NM 52, MP 27.75. Scouring was observed at most structures with some being located in the channel and others coming across the top of the structure especially where culverts were included in the design of the tire bale structure. Tensile cracking was observed along NM52, MP 27.2 at the contact between the native material and the fill behind the tire bale structure.

These observations indicate that erosion of sediment in the channels and water flow across the top of the structures has the potential to destabilize the tire-bale structures.

7.1.3 Design of Tire-bale Structures for Erosion Control and Bank Stabilization, Constructability of the Field Demonstration Facility and an Alternative Design

Two tire-bale structures, a head-cut structure and a side structure, were designed so that water cannot go underneath or behind the bales by using a layer of geomembrane. Required engineering soil properties for design were obtained from basic soil mechanics laboratory tests on disturbed soil samples recovered near the surface. Stability of the structures was controlled by considering different possible failure modes such as overturning, sliding, bearing capacity, and slope stability. Size of the geomembrane was determined by estimating the travel time of water in a saturated soil for the design flood. The geomembrane is covered by a layer of treated soil by using either cement or cement kiln dist (CKD). A flume, a pressure transducer on top of the structure, and a series of moisture probes at different locations within and on the bales were installed to monitor flow of water. In addition, displacements and deformations of the structure as well as the soil around the structures are monitored by visual inspection and surveying in regular time periods.

Practicality of construction of the tire-bales structures was evaluated during the construction of the field demonstration facility at the Energetic Materials Research and Testing Center (EMRTC), a research facility at New Mexico Tech. Two structures were built according to the design guidelines developed: a system for the head-cutting and another for the side cutting. Construction time, ease of construction, required equipment and material, as well as differences between the design drawings and the actual structures built were recorded for each structure. Flowcharts were used to describe the construction process and indicate the sequence of events. Since breaks are expected every 20ft along the structure and structures to be built in the field often extend for 100ft or more, it is expected that the process described will be used repeatedly. As a result, the construction of each 20ft section is likely to be accelerated as workers gain familiarity with the procedure.

An alternative design procedure for tire-bale erosion control and bank stabilization structures was presented. This procedure took the form of the well-known method used to design retaining walls, modeling the soil pressure behind the wall as hydrostatic pressure following the Rankine earth pressure laws.

To present a worst-case scenario (giving the most conservative structure, appropriate where long inspection and maintenance intervals are likely to exist), several assumptions were made:

- The gabion wire wrapping was assumed to provide no resistance to sliding or overturning
- The coefficient of friction of the bales with respect to the soil, and to each other, was 0.75
- The bales would not benefit from composite action; that is, localized failure would not be compensated by adjacent intact structure – this is consistent with retaining wall design outlined in all major design codes

The failure modes considered were:

- Sliding failure analysis, with the assumption that the bottom bales may be exposed by scour
- Overturning failure
- Deformation failure, with sliding forces acting on upper rows of bales without consideration of gabion wire or cable anchorage.

Two conditions were modeled:

- Dry backfill
- Saturated backfill

The calculations were presented, and were organized to give factors of safety against failure for the three failure modes described above. It was seen that most tire-bale structures suffer the greatest risk from deformation failure, particularly in the saturated-backfill condition.

A design example was presented, and therein was given the procedure to design lateral support for tire-bale structures in the form of deadmen buried in the backfill

7.2 Conclusion

The conclusions of the Research Team are as follows:

7.2.1 Current State of Practice

Tire-bale structures are used for erosion control in a relatively few states. The principal reason for this is that many structures that were built since the late 1990s have experienced significant distress and/or failure. The failures were almost always caused by corrosion and eventual failure of the tie wires holding individual bales together. Another failure mode that has been reported (and was seen in the structure on NM 143) was overturning of sections of tire-bale wall due to hydrostatic pressure from water infiltrating behind the structure. This failure mode has not been common, but the observations of NM 143 and the work done by Zornberg *(42, 43, and 53)* have indicated that the potential for the development of this failure mode will often exist

The reportage of such incidents has strongly influenced many state Departments of Transportation to prohibit or actively discourage the use of tire-bale structures for erosion control and bank stabilization.

It is the conclusion of the Research Team that the blanket 'banning' of the use of tire-bales for these structures is not warranted, as the development of these failure modes can be traced to inadequate structural design (i.e, the tire-bale structures which failed were neither securely anchored, nor wrapped in gabion wire with the exception of NM 143) and inadequate knowledge and control of surface and subsurface water infiltration.

Standards have been developed in the UK, and are presented in this report, to ensure that tire-bales used in erosion control and bank stabilization meet acceptable criteria both as individual structural units, and for their placement as part of a larger structure. The Research Team concludes that standardization of practice is vital for both the successful performance of tire-bale structures, and for tracking their behavior over time.

7.2.2 In-Field Structural Monitoring

The tire-bale structures on NM 52 and NM 152 were generally stable over the project period. Movement in the structures was on the order of 1-3 in, and did not seem to affect the fulfillment of their purpose. The largest observed movements, such as that on the downstream section of NM 52 MM 27.2, correlated well with observed distress in the backfill soil (tensile cracking).

The conclusion to be drawn from monitoring these tire-bale structures was that NMDOT's design practice provided stable structures for these locations under the weather conditions that prevailed during the project, and over the life span of the project.

However, the distress that was observed, in the form of streambed scour, erosion exposing additional bales, and tensile cracking in the backfill soil, indicated that there are areas of concern. The design example given in Task 6 of this research clearly shows that the development of saturated backfill, as might occur during a major storm event, would result in hydrostatic forces which could cause overturning. Additionally, scour at the toe (upstream extremity) of the structure can allow water to collect both below and behind it. Water collecting under the structure could result in uplift if the underlying soil is expansive, and water collecting behind the structure could result in both overturning hydrostatic forces, and lateral pressure from expansive backfill soil (very possible of native soil was used for backfill).

The conclusions from NM 143 are similar; there are additional lessons to be drawn. NM 143 was a fairly complex structure, with distinct reentrant corners. It was also taller than the other structures (by one bale). Its failure was clearly overturning caused by hydraulic pressure. Evaluation of this failure has two parts – structural, and hydrological.

Structurally, NM 143 failed because its base was essentially free to rotate about the steel anchors. Self-weight was the structures only significant moment resistance. A hallmark of tire-bales is that their density is about 35-40% that of soil when installed, increasing as soil is deposited in the bales' void spaces (the NM 143 structure failed soon after construction).

The failure was also caused by inadequate measures to prevent surface water infiltration behind the structure. The original drainage configuration actually channeled water behind the structure's northeast corner, and along the north side. Sheet flow from a broad, gradual upslope extending north from the north side of the structure was also a potential factor.

Thus, great care should be taken to control the movement of water across these structures. The contact between the native soil and the fill behind the structure is a weak point and every effort should be made to prevent water entering behind the structure at this point. Similar weak points are the fill associated with culverts included in the structure. These points should be covered with some impermeable barrier. Scouring at the contact between the stream and the structure has the potential for allowing water to get in behind the structure. We recommend that the tire-bale structure be pinned with angle iron at all corners not just the outside corners.

The survey methods used were appropriate for a research project, but are not seen as practical for functional surveys by Department personnel. First, they are very time-consuming. Second, considerable effort has to be made to retain the positions of the original survey points. Third, they provide more data than is required for structural health monitoring. A more reasonable approach would be an annual inspection, augmented by periodic inspections after major stream-flow events. A visual examination of the structure and backfill will reveal most of the significant signs of distress, such as tensile cracking and settling in the backfill, or scour adjacent to the structure. One possible direct measurement would be using a digital level to measure the angle of inclination between points on different rows of bales. This could show any tendency to overturn into the streambed, which is the most likely and severe failure mode.

7.2.3 Design of Tire-bale Structures for Erosion Control and Bank Stabilization, Constructability of the Field Demonstration Facility and an Alternative Design

Evaluation of the structure performance and effectiveness of the design will be done in Phase II of the project. However, the following can be concluded from this phase and knowledge of the previously built structures.

1. Basic engineering soil properties such as grain size distribution, Atterberg limits, compaction characteristics, permeability, and shear strength parameters of the soil must be obtained from standard soil mechanics laboratory tests on the representative soil samples.
2. Even if different calculations show that a tire-bale structure is stable under normal circumstances, it is necessary to tie the entire structure to a series of anchor piles. To transfer lateral and uplift forces to the anchor piles effectively, integrity of the structure

must be improved by wrapping the bales in a layer of steel wire mesh or the other applicable methods.

3. Tensile cracks will develop behind the structure at the point of separation of soil and structure. To prevent developing excess pore water pressure underneath and behind the structure due to the penetration of water through these cracks, compaction of foundation soil as well as backfill soil is necessary.

The alternative design developed from traditional retaining wall structures work well for tire-bale structures, if the unique structural properties of tire-bales are considered.

Tire-bale structures cannot economically be made monolithic; they are constructed from large elements which are not rigidly fixed to one another. Consequently, they are subject to internal sliding deformation as a result of hydrostatic pressure, either from backfill or a combination of backfill and water (saturated backfill). Additionally, they are very low-density when compared to other construction material. Tire-bales will typically have 35-40% the weight of the native soil they displace, and 20-25% of the weight of concrete.

The greatest risk to tire-bale structures comes from the pressure caused by saturated soil behind the wall causing sliding between the rows of bales. This failure mode can create instability in the structure which can lead to overturning failure (this was the likely course of events in the failure of the structure on NM 143).

The possibility of saturated backfill must be considered in the design process. These structures are designed to be used in ephemeral streambeds, and the nature of these features is that they are subject to intermittent high water flows. While designing the structure to exclude water (using surface berms or geosynthetic barriers), there is no way that water intrusion in the backfill can be prevented over the life of the project (or that once present, it can be economically removed).

This being the case, the design example includes calculations for low-cost deadmen to be buried in the backfill. These deadmen consist of used concrete Jersey barriers connected to the tire-bales by steel cables. The deadmen ensure that sliding or overturning forces are transferred from the tire-bales to the backfill, greatly reducing the risk of failure. Their use in tire-bale structures is very strongly recommended.

7.3 Recommendations

The recommendations of the research team and technical panel are in the following sections. Upon the completion of Phase II both teams expect this section to develop a more accurate understanding of the tire-bale design being monitored at the FDF site and in-field sites. The recommendations presented in section 7.3.1 are based on the findings since 2008 to 2010. Sections 7.3.3 – 7.7 are the research teams proposed research for Phase II, some of which is presented in abbreviated form.

7.3.1 Research Team

Several recommendations during the FDF site construction were made by the construction team: the use of ½" cable would allow the construction crew to apply tension without worrying about cable failure; in addition, if steel piles are to be used instead of concrete piles solid bars would be preferred over angle iron to prevent bending of the piles. The use of appropriate pile driving equipment is also recommended. Alternate stitching of the gabion wires should be used to assure that both flaps are secured. To save time in cutting of the gabion wire, wrapping of the first and second layers of tire-bales should be performed with a continuous strip of gabion wire. The top row should be secured with a separate piece. Ties should be placed on either side of the first and second rows to secure the tire layers. A 12in layer of soil treated with 10% cement should be used to cover the structures.

The research team recommends addressing problems as soon as they occur and prevent further damage to the structures, site visits for evaluation of existing structures are recommended as follows: (1) Monthly visits should be conducted from the beginning of the snowmelt period (usually April) to the end of the raining season (usually September); (2) Site visits are also recommended after each severe rain event; (3) Finally, if possible, a site visit should be conducted before the snowmelt period to address any damage that may have happened during the fall and winter months. During these site visits, evaluators should conduct visual inspections of the entire structure, documenting its condition and paying special attention to the following modes of distress: loose tires, corrosion of baling wires and/or gabion wires, undercutting of bales in middle or the end of the wall, and the presence of tension cracks behind the walls. Probable causes and recommended actions for each mode of distress mentioned above are presented in final report.

Additionally, great care should be taken to control the movement of water across these structures. The contact between the native soil and the fill behind the structure is a weak point and every effort should be made to prevent water entering behind the structure at this point. Similar weak points are the fill associated with culverts included in the structure. These points should be covered with some impermeable barrier. Scouring at the contact between the stream and the structure has the potential for allowing water to get in behind the structure. We recommend that the tire-bale structure be pinned with anchor piles at all corners not just the outside corners.

When designing tire-bale structures the following items are necessary:

1. Do not put tire-bales on organic soils. Any organic soil at foundation must be removed.
2. Foundation soil as well as backfill must be compacted at optimum water content to achieve a minimum compaction ratio of 95%. Backfill soil should be compacted in layers less 12 in. thick.
3. Over-compaction must be avoided. It is necessary to monitor compaction continuously to stop compaction at the recommended compaction ratio.
4. Use a layer of steel wire mesh on the entire structure to hold the bales together.
5. Anchor the entire structure to the ground by using at least four anchor piles at the four corners of the structure. Steel cables attached to the anchor piles and passing though the structure transfer the uplift forces to the piles.

6. Any excavation should remain stable until the end of construction.
7. Expansion index test should be done on the 1 ft clay layer beneath the geomembrane to make sure that the clay is not expansive.
8. Special care in placing geomembrane is necessary to avoid any puncture.
9. A minimum thickness of 20 mil is recommended for geomembrane. When the width of a geomembrane role is less than the structure length, several pieces of geomembrane should be attached together by overlapping these pieces and using a special tape recommended by the manufacturer.
10. Geomembrane should be extended to the back, to the sides, and to the front of the structure beyond the excavation boundaries. This extension should not be less than 25% of the structure height.
11. Geomembrane should not be exposed to sunlight after the construction period. Applying a layer of treated soil or another proper material on the geomembrane is necessary.
12. It is recommended to dissipate some part of flow energy by placing natural and manmade obstacles in the flow path.
13. In case of using native soil to cover geomembrane, the soil should be treated with a suitable additive such as lime, cement, fly ash, cements kiln dust (CKD), etc. A series of lab tests needed to be performed to determine the suitable percentage of the additive by dry weight of the soil.
14. Inspection and maintenance should be scheduled at regular intervals.
15. Follow all current federal and state regulations for the use of tire-bale products and International Fire Code regulations for storage on the job site.

7.3.2 Technical Panel Recommendations for Phase II

Technical panel members concluded that 2½ months study of the new installed FDF site is not enough to understand properly how the structures will perform in the long-term. The panel recommended that the project be extended for an additional 18 months. This additional duration would allow the research team to monitor the FDF structures and carry out necessary maintenance. Lesson learnt during Phase II would be utilized to update the design and construction specifications. The knowledge gain would also allow performing a better life cycle cost analysis.

7.3.3 Activities for Phase II

TASK 1: Continue performing engineering analysis of the existing drawings and construction practices and monitor the performance of existing and FDF tire-bale structures.

TASK 2: Select the material that will be most cost effective and durable in covering the geo-membrane. Options will be considered are: 1) shotcrete; 2) soil treated with 10% cement/fly-ash or cement kiln dust; and 3) very low density air-entrained concrete. Necessary specification will be incorporated for the selected material.

TASK 3: Soil permeability and strength change with time because of consolidation will be studied using the moisture/pressure sensor data. The design standards implemented in the construction of the FDF structures will validated or revised based on the field data collected from the FDF.

TASK 4: Flow prediction based on site drainage parameter is an important aspect of any erosion control and bank stabilization project. This task will provide guidelines to such prediction.

TASK 5: Continue performing Environmental Impact Assessment of the application of Tire-bale erosion control and bank stabilization process.

TASK 6: Acquire data on the maintenance cost and the same will be used to determine the life cycle cost (LCC) of alternate methods using the Federal Highway Administration (FHWA) general guidance for running a detailed LCC analysis.

7.3.4 Anticipated Results Based on Phase II Activities

Deliver a set of standards, specifications, and design/construction guidelines that will be based on study of FDF structures.

7.4 Implementation

7.4.1 Anticipated Results Based on Phase II Activities

Task 7: The design/construction handbook will be updated on the basis of phase II findings. Best practices for construction and maintenance will be included.

7.4.2 Production of a Multimedia Presentation

Task 8: A Web-based multimedia presentation on design, construction, and economic analysis of tire-bale systems will be produced. The economic analysis will incorporate maintenance cost for the FDF structures. The presentation will also include information for the lay public, and a Frequently Asked Question (FAQ) section.

7.4.3 Scheduling of Workshops at NMDOT Districts

Task 9: One workshop will be organized for the NMDOT personnel at a time that is convenient to the NMDOT to disseminate information on design and construction. This workshop will include visit to the FDF construction site.

7.5 Milestones

Milestones for the Phase II of the project are:

a. Formalization/validation of the existing drawings
b. Selection and placement of the cover material
c. Development of Soil permeability model
d. Development of Drainage model
e. Updating the Design/Construction Handbook and the Multimedia Presentation and
f. Development of the Life Cycle Analysis.

7.6 Time Requirements

The suggested time required to complete the project is 18 months. Schedule of the tasks/milestones is presented in the Table 16 below.

TABLE 16 Phase II Tasks/Milestones

Task	Milestone	1st Quarter	2nd Quarter	3rd Quarter	4th Quarter	5th Quarter	6th Quarter
1	A	X	X	X	X	X	
2	B	X	X				
3	C	X	X	X	X	X	
4	D	X	X				
5	See Deliverables						
6	F	X	X	X	X	X	
Deliverables	Quarterly reports	X	X	X	X	X	X
	Milestone – E						X
	Final report						X

7.7 Abbreviated Itemized Budget

TABLE 17 Phase II Abbreviated Itemized Budget

ITEM DESCRIPTION	**RATE**	**COST**	**COST**	**Total COST**
FACULTY SALARIES		**YEAR 1**	**YEAR 2**	**New Budget**
PI		$6,890	$0	$6,890
CO-PI 1		$3,544	$0	$3,544
CO-PI 2		$3,305	$0	$3,305
CO-PI 3		$3,600	$0	$3,600
CO-PI 4		$3,210	$0	$3,210
TECHNICAL SALARIES				
Environmental Specialist		$1,200	$600	$1,800
UG STUDENT SALARY / Work Study		$6,240	$6,240	$12,480
Subtotal		$27,989	$6,840	$34,829
FRINGE BENEFITS				
Subtotal		$5,235	$442	$5,677
GENERAL SUPPLIES AND EXPENSES				
Subtotal		$4,300	$2,300	$6,600
TRAVEL				
Subtotal		$1,500	$1,500	$3,000
EQUIPMENT RENTAL - NON-MTDC		$3,000	$1,000	$4,000
TOTAL DIRECT COST		**$42,024**	**$12,082**	**$54,106**
INDIRECT COST		**$7,805**	**$2,216**	**$10,021**
TOTAL BUDGET		**$49,829**	**$14,298**	**$64,127**

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APPENDIX A

Telephone Survey Questionnaire

Following lists of questions have been formulated for the telephone survey

For those officials who have used tire bales in erosion control and/or bank stabilization-

1. How many tire bales projects have been undertaken in your jurisdiction?
2. How long have you been using tire bales?
3. Under what general categories would you list these (i.e., stream bank stabilization, slope stabilization, drainage head cutting prevention)
4. What is the size range of the projects (length, height, number of bales, rows or levels of bales)
5. What other methods were considered?
6. Why did you choose tire bales?
7. What configuration of bales are you using?
8. What is the typical cost/bale (less transport)?
9. Are tire-bale structures in your jurisdiction engineered?
10. What construction configurations have you used (i.e., stacked on grade with no base or cover, stacked on gravel or geotextile, stacked with geotextile and soil between layers, secured with riprap wire)
11. How would you assess overall performance?
12. Which construction method has been most effective in terms of initial cost and maintenance cost?
13. Which method has been least effective?
14. What problems, specific to tire bales, have you observed? What remediation measures were taken?
15. What advantages have tire bales shown over other methods that could have been used in those situations?
16. Have you developed standards, or standard design drawings, for the use of tire bales?

17. Do you plan to use tire bales in the future? Will their use increase, decrease, or remain at about the same level in comparison with other methods?

For officials who have used tire bales in other transportation applications –

1. How have tire bales been used in your jurisdiction?
2. How long have bales been used in your jurisdiction?
3. Why did you select tire bales?
4. What other methods were considered?
5. What configuration of bales were used?
6. Do you have standards, or standard plans?
7. What construction methods were used?
8. How would you assess the bales' performance?
9. What problems specific to tire bales have you seen?
10. Would you consider using tire bales for erosion control/bank stabilization? Why or why not?

For officials who have used other recycled tire products –

1. Have you used recycled tire products in erosion control/bank stabilization work?
2. What kind of recycled tire product have you used?
3. If you have not used these products in erosion control/bank stabilization, but have used them in other transportation-related projects, how have you used them?
4. Has their use been engineered, or does it conform to standards or standard plans?
5. Why did you select recycled tire products?
6. What other methods were considered?
7. How would you assess the recycled tire products' performance?
8. What disadvantages have you seen specific to this material?
9. Will you use recycled tire products in the future?
10. Will you consider using tire bales in the future?

For officials in charge of erosion control/bank stabilization, in jurisdictions where recycled tire products have not been used

1. What kind of erosion control/bank stabilization methods are you currently using?
2. Have you, or would you consider using tire bales? Why or why not?

Clearly, these are basic questions, and in the course of conversations with any individual, other questions will be developed.

APPENDIX B

Telephone Survey Observations

State	Tire Bale Use	Comments	Contact
AL	Not used	Alabama has not been interested in using tire bales due to bad reports from other states, and has only baled for size reduction	Gavin Adams
AK	Not used	Tires are not recycled	Department of Environmental Conservation – Solid Waste 907-269-7802
AZ	Not currently used	Goodyear sponsored a program in the mid-90s to use individual tires for a dam on private property. Successful to date, no further work	Stuart Hoenig Professor Emeritus, University of Arizona Department of Electrical Engineering 520-887-3815 John Buross Arizona Department of Environmental Quality 520-771-4118
AR	Not used		Gary Williamson Arkansas Department of Transportation Environmental Division

State	Tire Bale Use	Comments	Contact
			501-569-2230
CA	Pilot project	TBs used as lightweight fill behind retaining wall – remained dimensionally stable through ten-year age of project	Jeff Nelson SHN Consulting Engineers 707-441-8855
CO		No reply by 9/25/08	Rich Griffin 303-767-9973
CT	Not used		James Norman Connecticut DOT – Design james.norman@po.state.ct.us
DE	Not used	Tire shreds/chips used as lightweight fill	Crystal D'Andrea Natural Resources and Environmental Control Division of Air and Waste Management 302-739-9403 crystal.dandrea@state.de.us
FL	Not used	Florida has 90% tire usage as shreds and chips, and actively discourages baling as being more difficult to handle for eventual disposal	Jan Rae Clark Florida Department of Environmental Protection 850-245-8744

State	Tire Bale Use	Comments	Contact
GA	Not used	Mats and riprap used for erosion control as GA has large sources of native rock	Georgene Geary Department of Transportation Office of Materials and Research 404-363-7512
HI	Some use	A company in Hawaii bales tires and encases them in concrete for use as walls – regulators would frown on encased TBs	Department of Health 808-586-4226 Todd Nichols
ID	Not used	Rock mulch used instead	Mike Santi Idaho Department of Transportation 208-334-8450
IL		No reply as of 9/25/08	Greg Spencer Illinois EPA 217-524-4652
IN	Not used	Shreds used as lightweight fill in peat bog	Nayyar Zia Siddiqi Indiana Department of Environmental Management 317-610-7251x228

State	Tire Bale Use	Comments	Contact
IA	Not used		Chad Stobbe Department of Natural Resources 515-242-5851 Mark Masteller Iowa DOT Department of Roadside Development 515-239-1424
KS	Some use	Bad experiences – used as windbreaks for cattle, very unstable, and baling wires easily broken	Ken Powell Kansas Department of Health and Environment 785-296-1121
KY		No reply as of 9/25/08	Rick Solomon Kentucky Department of Environmental Protection – Division of Waste Management 502-564-6716
LA		No reply as of 9/25/08	Louisiana Department of Environmental Quality – Waste Tire Program 225-219-3027

State	Tire Bale Use	Comments	Contact
MA	Not used		Nabir Hourani Dept. of Environmental Protection 617-292-5500
ME	Not used	Erosion eels with tire chips used	Peter Newkirk 207-624-3100
MD	Not used	Shreds and chips used in playgrounds and hot mix; someone in the state once bought a baler but on hearing that MD Department of the Environment required engineered design didn't pursue it.	Abby Pascual Scrap Tire Section Maryland Department of the Environment 800-633-6101
MI	Not used	Some use of chips in pavement	John Berrick Michigan Department of Transportation – Construction and Technology 517-322-1087
MN	Not used	Emphatically disapproved. Concern that TBs will leach toxic chemicals into streams. Also concerns over good installation practices	Brett Troyer DOT Environmental Services 651-366-3629

State	Tire Bale Use	Comments	Contact
MS	Not used	DEQ policy forbids tire bale use – they will not evaluate TB projects due to unbaling concerns	Bruce Laird Department of Environmental Quality 601-961-5325
MO	Used in past	Bad experiences with tires unbaling	Kirk Mitchell Missouri Department of Natural Resources 800-361-4827
MT	Plans to use	Will try project on slope stabilization	Rich Jackson Montana Department of Transportation – Geotechnical 406-444-6275
NV		Unable to reach responsible person – scrap tires were apparently used to build windbreaks and fences – ref. http://ndep.nv.gov/BWM/Docs/TirePlan.pdf	Nevada Division of Environmental Protection Bureau of Waste Management
NH	Not used		Dennis Boisvert DOT – Bureau of Materials and Research 603-271-3151

State	Tire Bale Use	Comments	Contact
NJ		No reply as of 9/25/08	Sue Gresavage New Jersey Department of Transportation Geotechnical Division 609-530-4689
NY	Not used by DOT	A tire shred embankment 650 ft long by 15 ft thick has been built as a pilot project	Donald Dwyer New York State Department of Transportation ddwyer@gw.dot.state.ny.us
NY 2	Used as road base	Used by Chautauqua County as lightweight fill under road	George Spanos County Commissioner of Public Works Chautauqua County Dept. of Public Facilities
NC	Not used		Scott Hiddem Department of Transportation 919-250-4088
ND	Not used		Brad Tolverson Department of Health 701-328-5166

State	Tire Bale Use	Comments	Contact
OH	Not used	Many concerns – not allowed	Harry Smail Ohio EPA Division of Solid and Infectious Waste Management 614-644-2621
OK		No reply as of 9/25/08	Ferrella March Department of Environmental Quality 405-702-5175
OR	Not used	One project with shredded tires as lightweight fill, several years ago	Jon Guido Oregon Department of Transportation Erosion Control 503-986-4200
PA	Not used	No TB project will be considered because of fire concern – fire under I-95 bridge near Philadelphia softened girders and collapsed bridge	Pat Gardner PennDOT Bureau of Construction and Materials 717-787-6989
RI		No reply as of 9/25/08	Emily Holland Rhode Island Department of Transportation – Environmental 401-222-2023x4100

State	Tire Bale Use	Comments	Contact
SC	Not used	Tire chips used as mulch	Mike Sander SCDOT – Engineering Division 803-737-6681
SD	Not used	Not satisfied with attempted uses – not interested in pursuing	Jim Wendte Department of Environment and Natural Resources 605-773-3153
TN	Not used	Aggressive environmental policy probably would not allow placement of tires in channel – gabion baskets used. Tire shreds used in erosion eels.	John Zirkel TennDOT Design Division 615-741-2221
TX	Pilot project	Two TB walls built along I-35 in Fort Worth district. Results mixed – TBs retained water, and were built in conditions that were highly favorable to their use, so little progress was made toward general specifications	John Delphia TxDOT Bridge Division 512-416-2359
VT	Not used	Tire shreds used as fill and in hot mix	Jennifer Fitch Vermont DOT Materials and Research 802-828-2553

State	Tire Bale Use	Comments	Contact
VA	Not used	Bad experiences	Allan Lassiter Department of Environmental Quality arlassiter@deq.virginia.gov
WA	Not used	WADOT has concerns about tire bale integrity due to wire corrosion, and does not use them	Tony Allen Division Manager Washington State DOT - Geotech (360) 709-5450
WV		No reply as of 9/25/08	Joe Hall West Virginia DOT Division of Highways – Engineering 304-558-2885
WI		No reply as of 9/25/08	Peter Kemp Wisconsin Department of Natural Resources – Waste Materials 608-246-7953
WY	One project	TB’s used to repair landslide damage – survived construction well. Size is 400 ft long, 90 ft wide, 5 ft thick	Mark Falk Wyoming Department of Transportation 307-777-4202

APPENDIX C

Summary of Some of the Available Options for Erosion Control and Bank Stabilization

	Method	Description	Advantages	Disadvantages	Applications	Estimated cost/linear foot
Stone armors	**Riprap revetment**	Placement of well-graded stones over the sloping bank after the surface has been stabilized, compacted, and smoothed. A filter layer (6-in of well-graded stone) or a filter fabric (geotextile) may be supplied to support the revetment and allow water movement through the structure. Largest rocks resist water motion, while smaller ones prevent soil from being carried away.	This method has been extensively used and proven to be successful. Material for use in riprap is usually readily and locally available. It is resistant to minor damage, can be easily repaired and remains functional even if some stones are lost.	Extensive preparation of the embankment may be required for stability. When larger stones are needed, installation requires the use of heavy equipment. Not recommended for steep slopes or areas with significant amount of loose soil. Cost can be significant if material is not locally available. Weakens with age.	Erosion control in riverbanks, highway structures, as well as bridge piers and abutments. Will only be effective when underlying soil of shorelines and embankments are stable. Recommended in stable slopes between 1.5ft and 2ft horizontal to 1ft vertical rise.	\$30 - \$55 for a structure approximately 25 - 50 ft wide. (Includes the cost for heavy machinery.)
Self-adjusting armors	**Concrete blocks**	Self-adjusting armors are formed by the placing individual concrete blocks, sacks of earth, sand and/or cement directly on the embankment, therefore allowing the structure to adjust to irregularities of the terrain. Unique shapes for concrete blocks have been patented and are commercially available.	Concrete blocks can be cast on site, reducing transportation cost. Blocks can be installed over uneven and irregular surfaces, reducing use of heavy equipment and cost of surface preparation. Some blocks can be manually installed, making them cost effective and easy to install in urban areas or areas of difficult access for heavy equipment. Mechanized placement is also possible and may reduce installation time and amount of labor required. Concrete units may be used in a single-layer, thus using 3 to 4 times less material than riprap armors. Concrete blocks are durable and easy to maintain. Individual blocks provides for bank drainage.	Due to interlocking of the blocks, displacement of one block by water flow may lead to successive displacement of adjacent units. Susceptible to weather delays if blocks are cast on site. Susceptible to theft or vandalism. Unnatural appearance.	Recommended for bank armors, ditches, spillway linings, and culvert outlets. Suitable for areas where erosive forces are severe and/or where construction is difficult due to high flow velocity or great depths.	

	Method	Description	Advantages	Disadvantages	Applications	Estimated cost/linear foot
Flexible mattresses	**Gabion mattress**	Gabion mattresses are formed by placing one layer of shallow gabion baskets (rectangular wire baskets filled with rocks) along the embankment. Adjacent baskets should be tied to each other to prevent movement of the mattress.	Adjust to settlement and remain in contact with the bank. Work well where fires are a potential hazard. Allow for the establishment of vegetation.	Labor intensive; often requires the use of heavy machinery. Requires firm soil foundation and solid toe. If large stones are used, filters are needed. Not as flexible as other flexible mattresses. Although wires are coated with corrosion-resistant substances, they are still susceptible to deterioration. Requires regular maintenance. Create a barrier for wildlife.	Recommended for slopes or banks that are moderately steep (ideal slope would be less than 2ft horizontal to 1ft vertical rise). Effective along embankments and roadways.	\$30 - \$55 (includes cost of baskets, filling of the baskets, stone fill and basket closure)
	Gravel admixture	Mattress formed by spreading a layer composed of a combination of native soil and gravel over the slope. As fine particles are removed by erosion, an armored layer is left to protect the embankment against the formation of rills and gullies.	Capable of adjusting to settlement. Very little impact on vegetation or soil-water balance. Provide mechanical stability. Reduce erosion due to runoff and wind and retain seeds in place until germination According to Gyasi-Agyei (2004), they do not need to cover the entire slope and can be laid only on the outer verge of the slope to spread runoff and control formation of rills or on the lowest section of the slope where it reduces piping (or tunnel erosion) and induce sediment and seed deposition.	Not recommended for channels or areas of concentrated flow.	Practical for use in areas where high temperatures and minimal rainfall prevent the growth of dense vegetation. Effective in steep slopes.	
	Geotextile	Geotextile mattresses are natural or synthetic fiber fabrics placed over the embankment to control erosion. These systems may be made of degradable or non-degradable materials weaved to form a mat or blanket that is rolled over the slope to be protected	Adjust to settlement. Most materials are available in various configurations and can be applied to numerous situations. Reduce slope erosion by reducing runoff velocity, raindrop impact and encouraging infiltration. Protect soil during vegetation growth. Easy to install. After vegetation growth, they have an aesthetically pleasant appearance.	Mattresses may need to be anchored to avoid large uplift forces. Manufacturers provide little design guidance besides maximum slope, flow velocity and shear stress. No design guidelines for mattress selection or for anchoring techniques are available. Susceptible to deterioration and vandalism.	Effective in protecting moderately steep banks from damage caused by raindrops and erosion due to runoff. Effective in steep slopes. Suitable for areas where erosive forces are severe and/or where construction is difficult due to high flow velocity or great depths.	Jutemat (Finemat), Environmat and Coconut Fiber blankets are quite expensive for widespread application. Geosynthetic products may be more cost effective.

	Method	Description	Advantages	Disadvantages	Applications	Estimated cost/linear foot
Rigid armors	**Concrete armor**	Rigid armors consist of the placement of a non-flexible erosion-resistant material on the embankment. Commonly used materials are concrete, asphalt, soil-cement mixtures, and grouted riprap. Permeable or pervious concrete may be used to help retain seeds and sludge, creating an environment suitable for vegetation growth.	Capable of withstanding large hydrostatic forces. Extremely resistant to damage from debris, corrosion, and other destructive agents. Not susceptible to vandalism. In regions where material for self-adjusting armors is not locally available, and extensive subsurface drainage is not required, rigid armors are often the most cost-effective solution.	Require careful design and quality control. Construction is susceptible to weather delays. In the case of impermeable armors, filters or subsurface drains must be provided for draining groundwater and preventing buildup of excess positive pore water pressures.	Recommended in regions of turbulent or high velocity flow, where self-adjusting armors are either ineffective or cost-prohibitive. Suitable for steep slopes and for artificial channels. Recommended when water must not infiltrate into the bank.	
Retaining structures	**Gabion wall**	Gabion walls are formed by stacking gabion baskets (rectangular wire baskets filled with rocks) along the embankment. A filter layer (6-in of well-graded stone) or a filter fabric (geotextile) is usually installed between the slope and the baskets. Adjacent baskets are tied to each other to prevent movement of the structure.	Capable of withstanding large hydrostatic forces. Supports vegetation.	Expensive. Labor intensive; often requires the use of heavy machinery. Although wires are coated with corrosion-resistant substances, they are still susceptible to deterioration. Requires regular maintenance. Creates barrier for wildlife.	Effective along moderate slopes. Capable of protecting high-energy environments. Practical where construction area is limited. Not recommended for steep slopes or areas of loose soil.	$50 - $100 (includes cost of baskets, filling of the baskets, stone fill and basket closure)
	Tire bales	Tire bale retaining structures are formed by stacking rectangular bales composed of approximately 100 compressed tires held together by galvanized or stainless steel cables. The first layer is typically anchored to the ground with use of angle iron and cables, and bales are usually wrapped in hexagonal gabion wire and/or covered with soil or shotcrete.	Inexpensive to manufacture and install. Lightweight, thus easier to transport. Good load-bearing capacity. High chemical and physical durability. Fire-resistant when coated with a thick layer of non-combustible material. Voids may also be grouted to further reduce the amount of oxygen present. After being baled for a certain time, they will retain their shape even if some of the wires used to hold the tires together break or are removed.	Practice not yet well established. Although lightweight, some contractors have found them difficult to place due to the lack of specific lifting points. Susceptible to fires if not properly treated or covered with non-combustible coating. Long-term compression and creep rates are still unknown.	These structures have been used to replace gabion walls and riprap – they are however still in experimental phase in several locations.	

APPENDIX D

State of New Mexico Department of Game and Fish Letter

GOVERNOR
Bill Richardson

DIRECTOR AND SECRETARY
TO THE COMMISSION
Tod Stevenson

Robert S. Jenks, Deputy Director

STATE OF NEW MEXICO
DEPARTMENT OF GAME & FISH

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For information call: 505/476-8000
To order free publications call: 1-800-862-9310

May 12, 2010

Mr. Virgil Valdez
Research Analyst
NMDOT Research Bureau
7500-B Pan American Freeway, NE
Albuquerque, NM 87199-4690

Re: Tire Bale Bank Stabilization Structures Research Project
NMDGF Doc. No. 13342

Dear Mr. Valdez:

The Department of Game and Fish (Department) appreciates the opportunity to comment on the use of compressed used tire bales for bank stabilization structures and the associated research project being conducted by Dr. Ghosh of New Mexico Tech University. Mark Watson of my staff conducted a field site visit with you of four of these structures in NMDOT District 1, Sierra County, on 4 May 2010.

The Department supports the research being conducted to determine the appropriateness and effectiveness of compressed used tire bales for bank stabilization and erosion control in ephemeral arroyo situations, but not in intermittent or perennial stream situations. However, the Department has concerns regarding the long-term reliability of these structures. Tire bale bank control structures below Wall Lake on the East Fork of the Gila River failed, strewing individual tires down the river into the Gila Wilderness Area.

Assuming that the research determines appropriate applications for the use of tire bales, due to the potential for downstream sedimentation from construction activities and potential failure of these structures over the long-term, the Department would like to be consulted regarding future placement of these structures. We recognize that NMDOT installs these structures using a categorical exclusion under NEPA, which does not necessarily require public involvement. However, we would like to be given the opportunity to provide NMDOT with site specific information regarding potential effects to state-listed species in the project area and recommendations for possible mitigation strategies.

Again, we appreciate the opportunity to comment on this research project. Should you have any questions regarding our comments, please contact Mark Watson, Habitat Specialist, of my staff at (505) 476-8115, or <mark.watson@state.nm.us>.

Sincerely,

Matt Wunder, Ph.D.
Chief, Conservation Services Division

MW/MLW

CC: Deanna Cummings (U.S. Army Corps of Engineers Albuquerque Office)
Melissa Mata (Ecological Services Biologist, USFWS)
Bob Jenks (Deputy Director, NMDGF)
Jill Wick (Aquatic Habitat Specialist, NMDGF)
Mark Watson (Conservation Services Habitat Specialist, NMDGF)

APPENDIX E

Deflection Profiles of In-field Tire-bale Structures

This appendix shows displacement profiles for cross-sections through each of the four tire-bales structures discussed in section 4.4. They are numbered sequentially from the upstream to the downstream end, and they represent cross-sectional ‘cuts’ taken at 20-ft intervals. The ‘initial’ profiles approximate a standardized representation of the profiles of the actual structure.

In each profile, the point nearest the left-hand side (i.e., nearest the y-axis) represents the survey marker placed in the backfill soil. The point at the ‘knuckle’ (i.e., the middle point) represents the survey marker in the top tow of bales, and the rightmost point represents the survey point placed in the bottom visible row. The right side of each graph represents the stream bed. For some sections, complete profiles were not obtained.

The size of the ‘final’ data points correlates to the measurement error (standard deviation), and therefore if a ‘diamond’ is visible behind a ‘square’ at a given point, statistically significant deflection has occurred.

In most cases, the deflections are both small and consistent with the settling and consolidation associated with any earth-retention structure. However, brief comments are offered to annotate significant deviations from the norm.

NM 52 MM 26.7

Displacement profiles 1 and 2 (Figures E1 and E2) represent an area in which significant scour was observed at the upstream end of this structure.

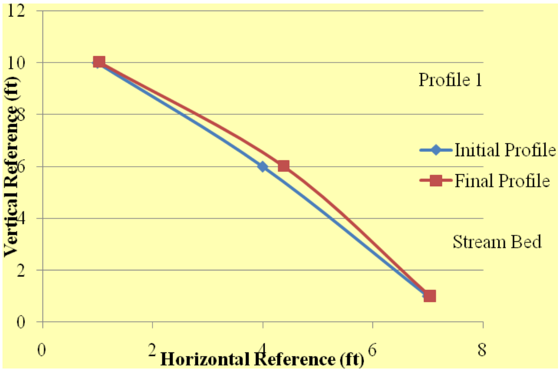


FIGURE E1 –Displacement profile 1

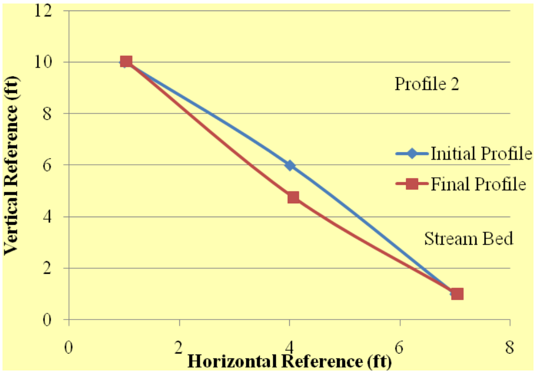


FIGURE E2 – Displacement profile 2

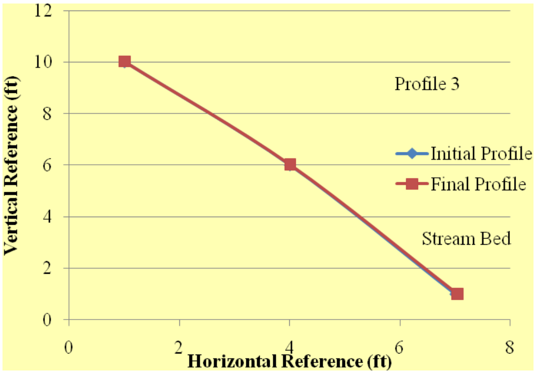


FIGURE E3 –Displacement profile 3

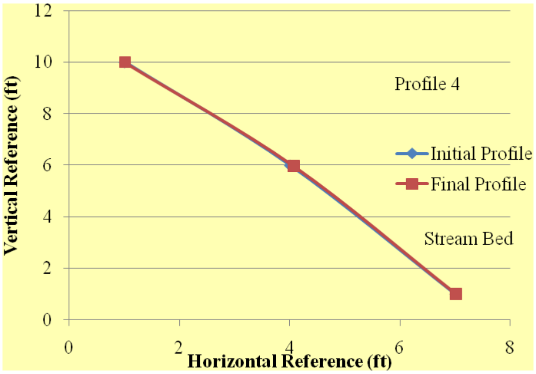


FIGURE E4 – Displacement profile 4

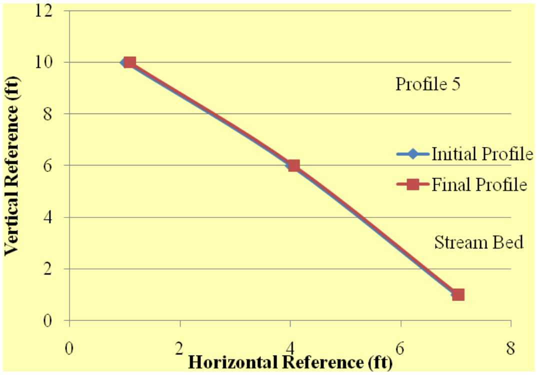


FIGURE E5 –Displacement profile 5

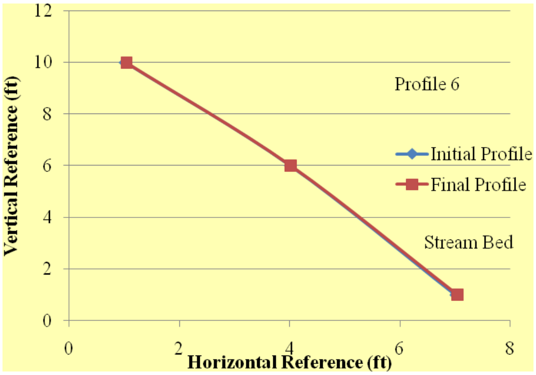


FIGURE E6 – Displacement profile 6

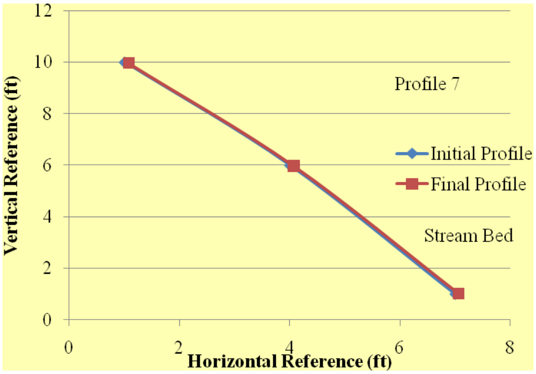


FIGURE E7 –Displacement profile 7

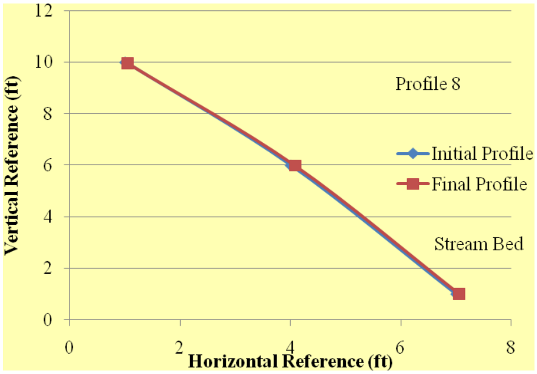


FIGURE E8 – Displacement profile 8

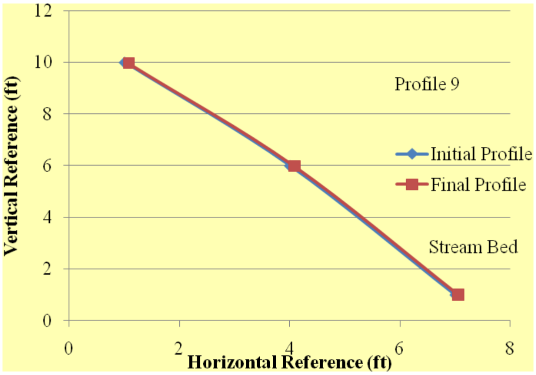


FIGURE E9 –Displacement profile 9

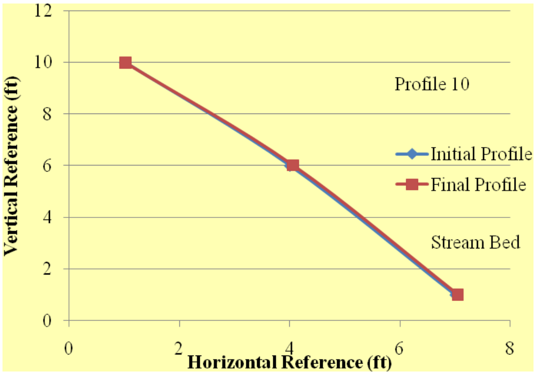


FIGURE E10 – Displacement profile 10

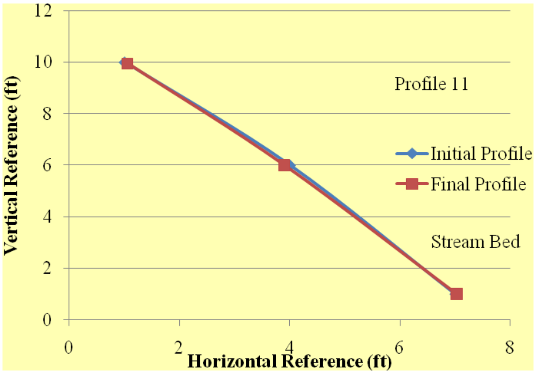


FIGURE E11 –Displacement profile 11

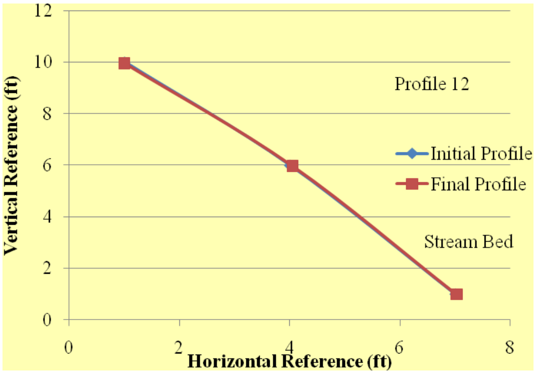


FIGURE E12 – Displacement profile 12

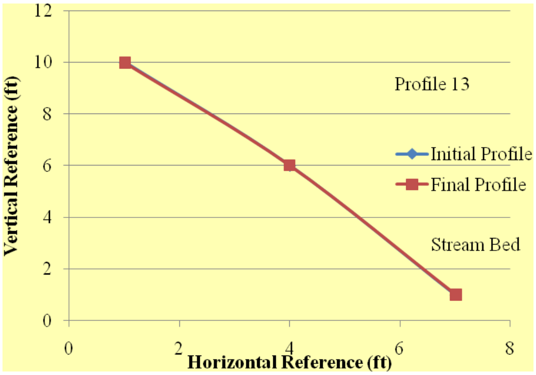


FIGURE E13 –Displacement profile 13

NM 52 MM 27.2

The most significant profiles are E23 through E26; they indicate a rotation 'backwards', and correspond to the development of tensile cracking in the backfill behind the tire-bale structure in this region

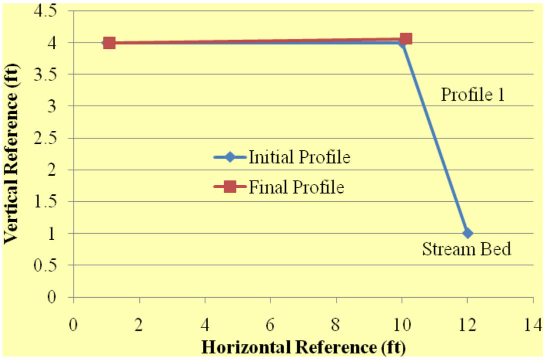


FIGURE E14 –Displacement profile 1

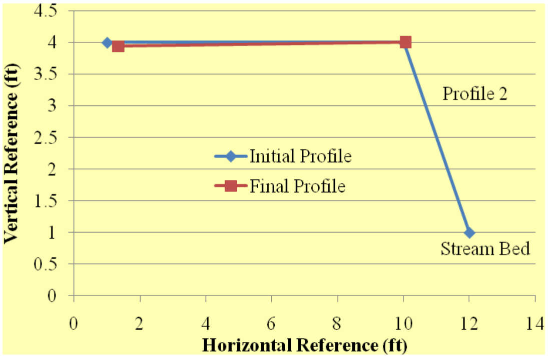


FIGURE E15 – Displacement profile 2

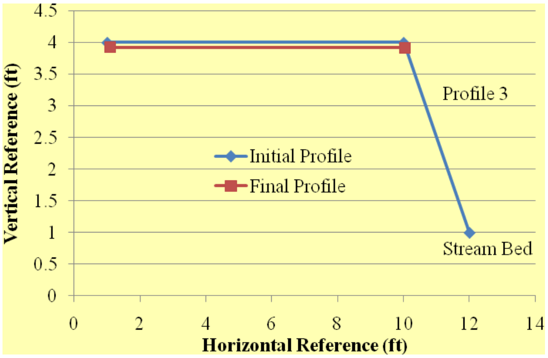


FIGURE E16 –Displacement profile 3

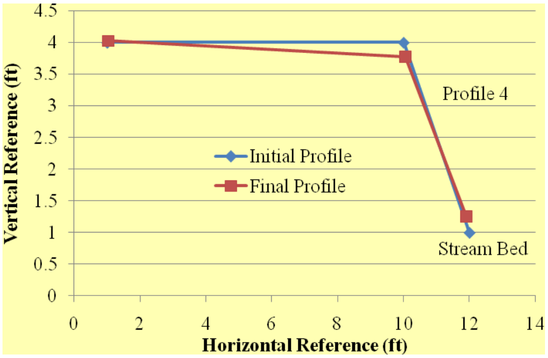


FIGURE E17 – Displacement profile 4

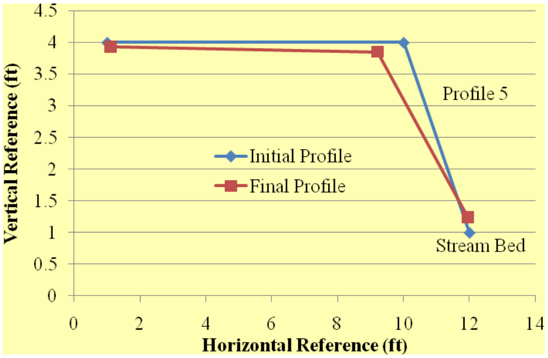


FIGURE E18 –Displacement profile 5

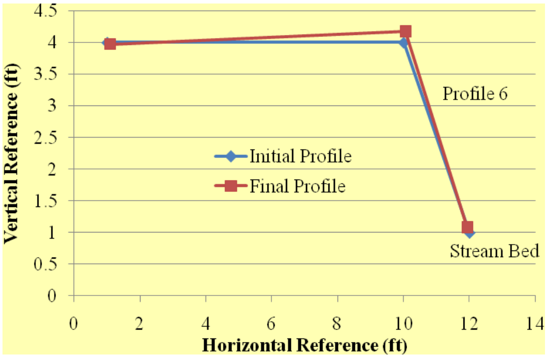


FIGURE E19 – Displacement profile 6

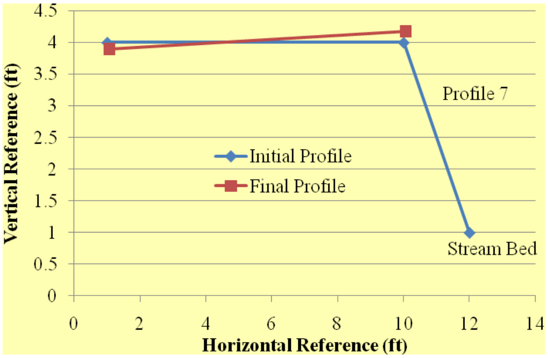


FIGURE E20 –Displacement profile 7

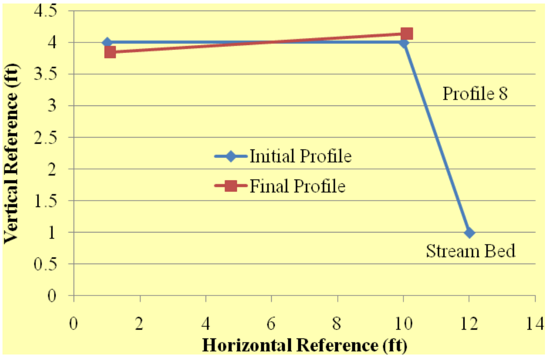


FIGURE E21 – Displacement profile 8

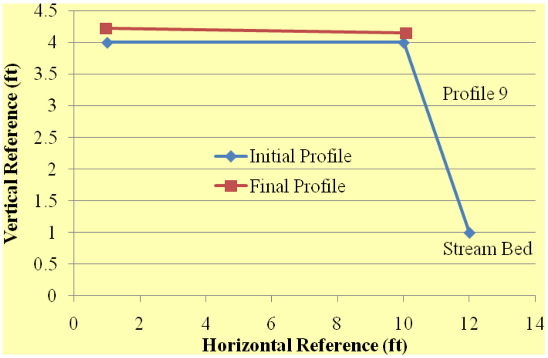


FIGURE E22 –Displacement profile 9

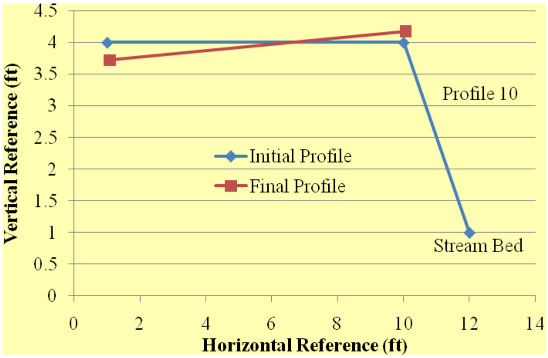


FIGURE E23 – Displacement profile 10

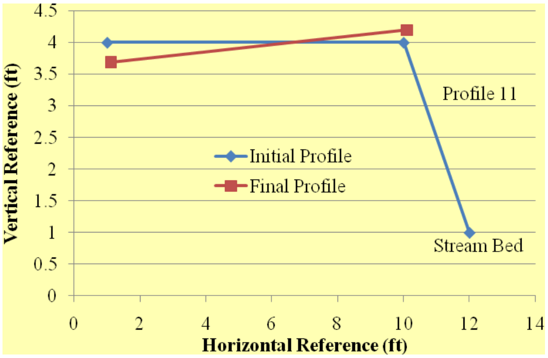


FIGURE E24 –Displacement profile 11

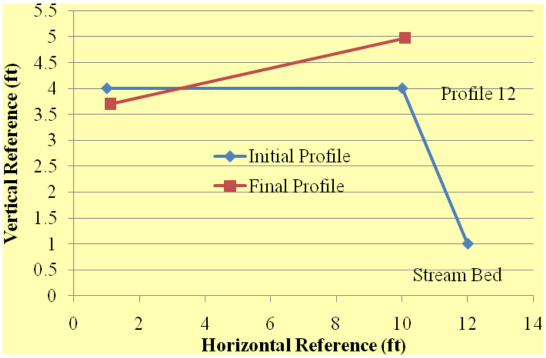


FIGURE E25 – Displacement profile 12

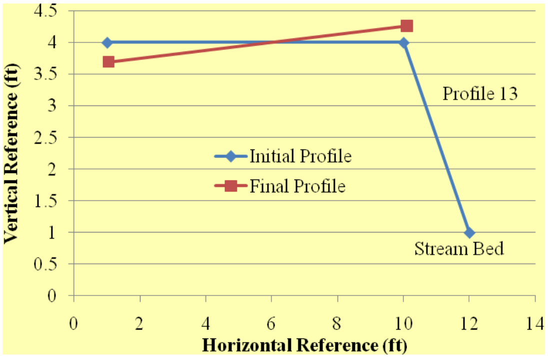


FIGURE E26 –Displacement profile 13

NM 52 MM 27.75

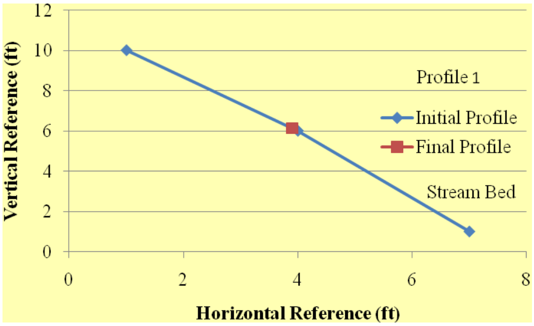


FIGURE E27 –Displacement profile 1

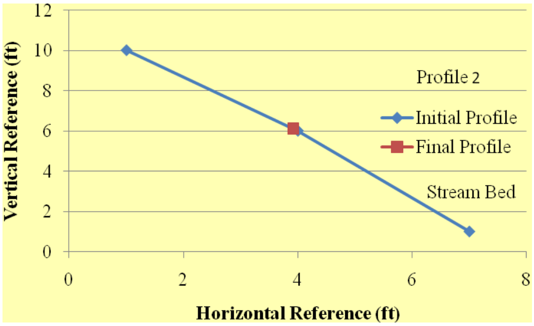


FIGURE E28 – Displacement profile 2

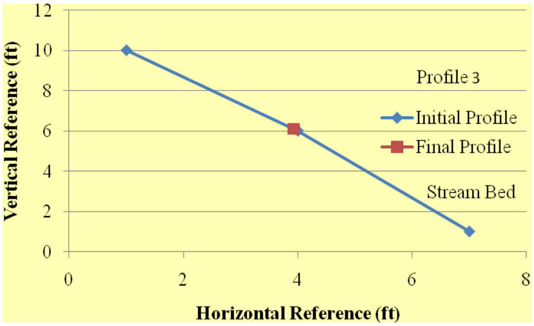


FIGURE E29 –Displacement profile 3

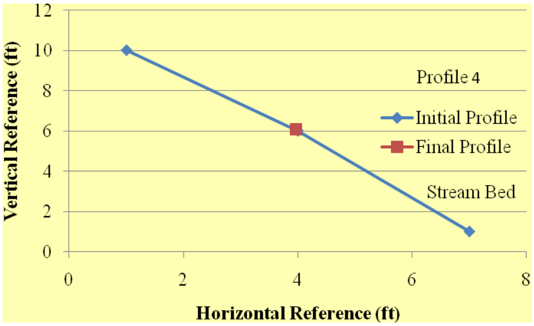


FIGURE E30 – Displacement profile 4

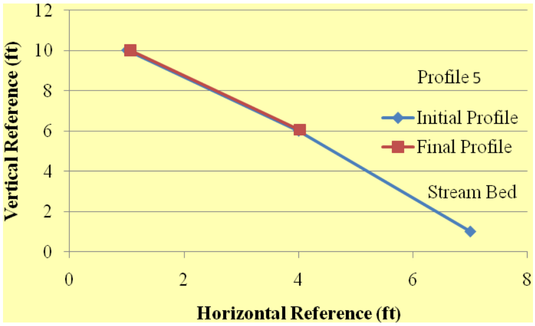


FIGURE E31 –Displacement profile 5

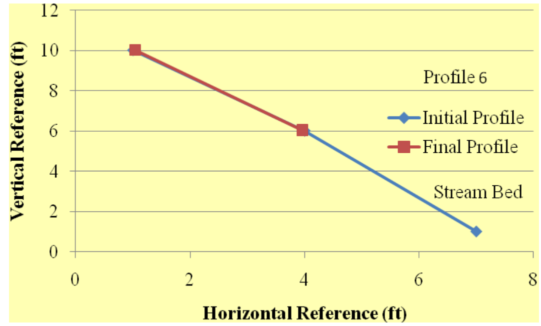


FIGURE E32 – Displacement profile 6

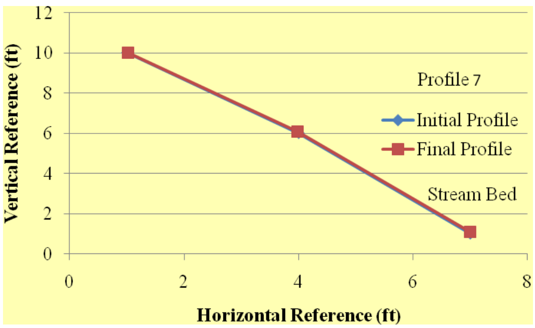


FIGURE E33 –Displacement profile 7

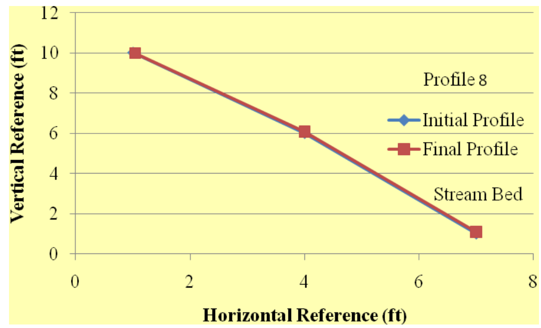


FIGURE E34 – Displacement profile 8

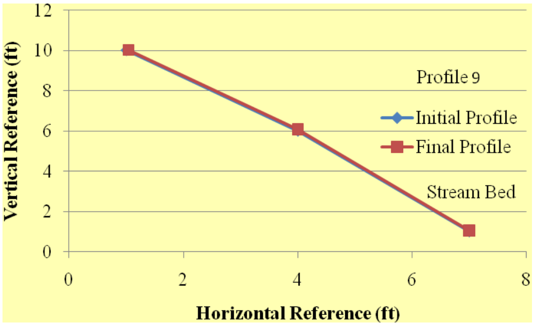


FIGURE E35 –Displacement profile 9

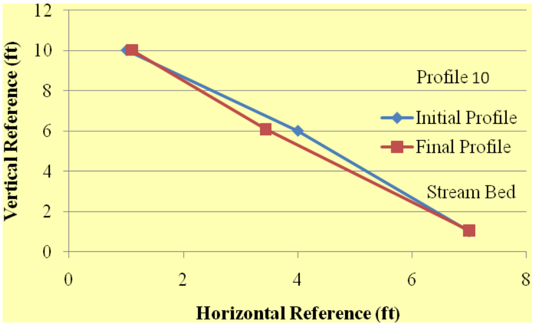


FIGURE E36 – Displacement profile 10

NM 152 MM 47.7

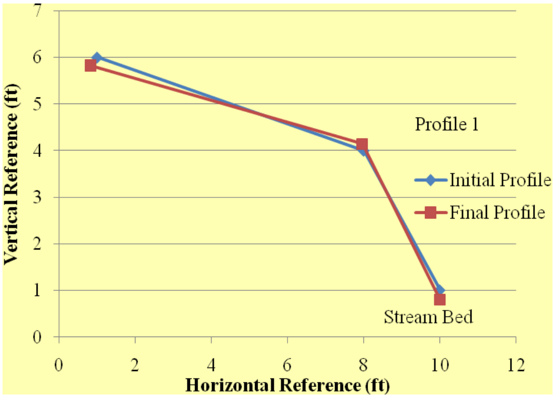


FIGURE E37 –Displacement profile 1

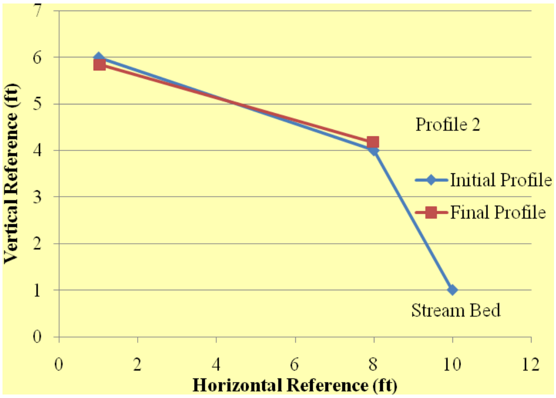


FIGURE E38 – Displacement profile 2

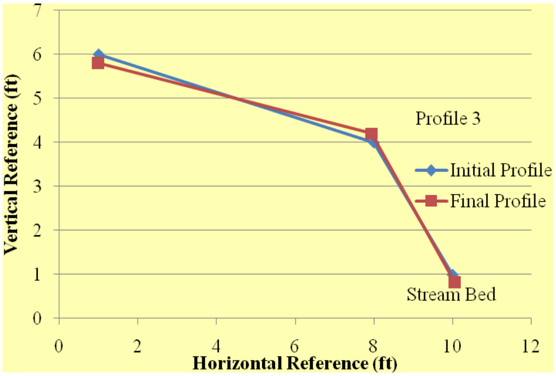


FIGURE E39 –Displacement profile 3

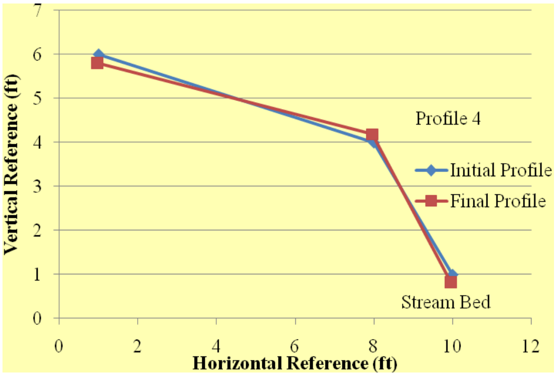


FIGURE E40 – Displacement profile 4

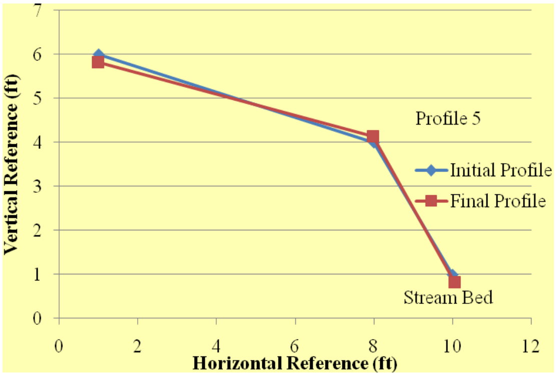


FIGURE E41 –Displacement profile 5

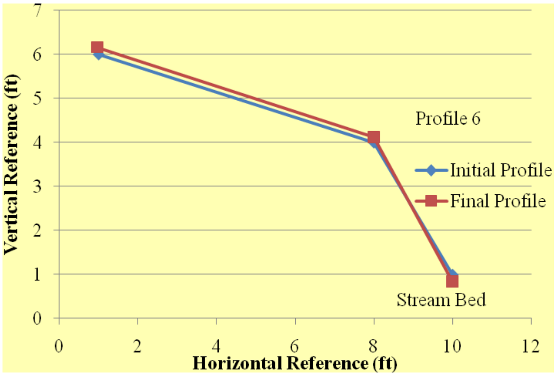


FIGURE E42 – Displacement profile 6

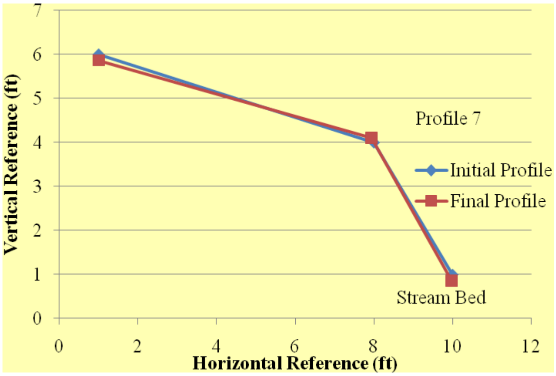


FIGURE E43–Displacement profile 7

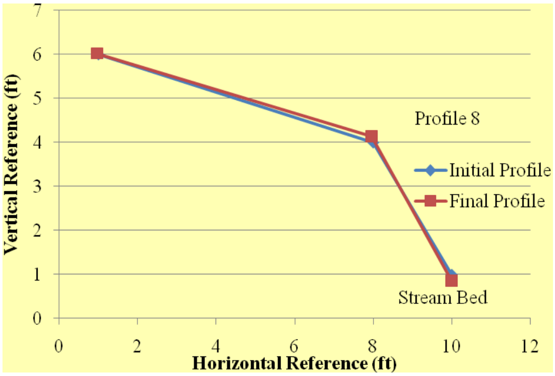


FIGURE E44 – Displacement profile 8

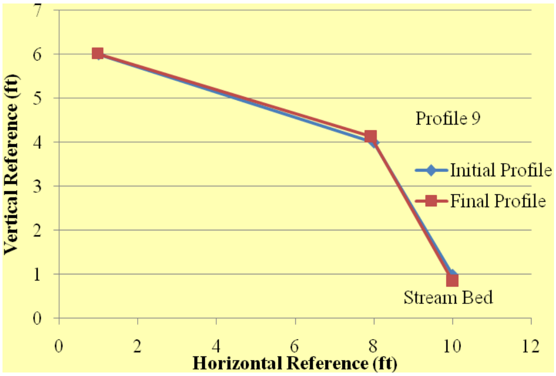


FIGURE E45 –Displacement profile 9

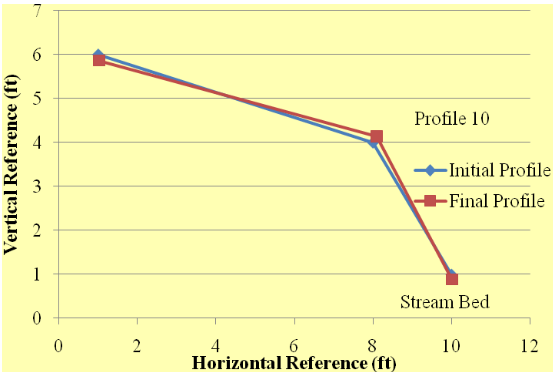


FIGURE E46 – Displacement profile 10

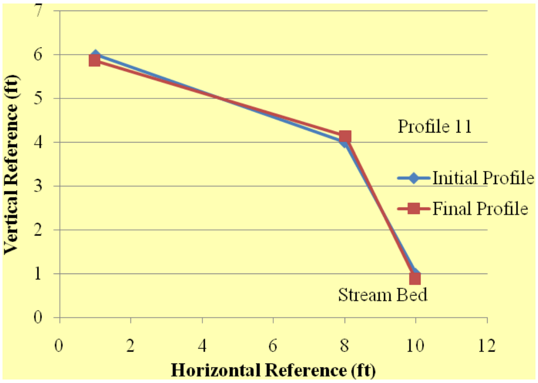


FIGURE E47 –Displacement profile 11

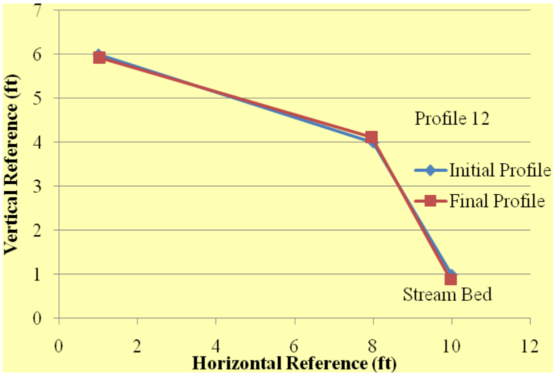


FIGURE E48 – Displacement profile1 2

APPENDIX F1

Permit Submittal

RESPONSE TO CHECKLIST FOR GENERAL (NATIONAL & REGIONAL)

PERMIT SUBMITTTALS

1. Ashok K. Ghosh, PI, Mechanical Engineering Department, 801 Leroy Place, New Mexico Tech, Socorro, NM 87801, Phone: 575-835-5505; fax: 575-835-5209; email: ashok@nmt.edu
2. Not applicable
3. July 2009
4. Latitude: N 34°6'0.1" and Longitude: W 107°2'17.2"
5. Nationwide permit 13 for bank stabilization.
6. The purpose and the need for the project is elaborated in the quarterly reports on NMDOT research project (NM08MNT-01: Standards for Tire-Bale erosion Control and Bank Stabilization Projects: Engineering Validation of Existing Practice and Implementation.
7. The preliminary jurisdictional determination form is duly filled, signed, scanned and attached for your consideration.
8. Unnamed tributary to Rio Grande River. A 50-year Flood Estimate of the construction site is provided in the **appendix A**.
9. Brief descriptions of the existing site conditions are provided in last two quarterly reports.
10. Please refer to the last two quarterly reports.
11. Following maps/figures are included in **Appendix B** to give details on the proposed construction:
    - **Map B1**: Arial map showing the construction site. There is no water body in and around this construction site.

- **Map B2**: Road map of the site showing no major or minor roads in and around the construction site.
- **Figure B1**: Photograph showing the arroyo where the tire bale structure will be constructed. **Figure B2** shows land profile around the arroyo.
- **Figure B3**: Cross section of the proposed tire bale structure.

12. Photograph of project area with explanatory labels and map showing location and direction of photos – Please refer to the figures B1-B3 of Appendix B.
13. Maps and figures sheet on 8 $^1/_2$" × 11" paper, including a vicinity map; specific project location map with project boundary indicated; jurisdictional determination map, plan and cross-section figures of all work and structures (proposed and existing) at the project site and any other map that provide information about the project and project area – Please refer to Maps and Figures provided in Appendix B.
14. Detailed description of work and materials used - Detailed description of construction methods and erosion/siltation control - Major Constructions steps are:
    1. Foundation Preparation – Appendix C has a cross section showing the locations of the moisture probes. This also given the amount of foundation preparation needed.
    2. Transporting Tire Bales – Transportation of the tire bales are to be done such that the tire bales ties are not damaged.
    3. Tire Bales Placed – Bales will be placed as per NMDOT construction specifications.
    4. Capping layer – Once all three layers are placed and soil is compacted all around the structure, a layer of geo-membrane is placed and extended as per Figure B3. A layer of treated soil will be placed to cover the geo-membrane from sun's UV.
15. Not applicable
16. Not applicable

17. Dimension of the tire-bale structure is 36 ft x 14 ft x 7 ft (approximately). Excavation of 150 cubic yard of soil will be done and the same material will be recycled to the same construction after proper treatment. There will be no need for imported soil for this construction. Total volume of soil needed to be cut and fill is around 300 cu. yd. The structure will be embedded under a layer of top soil for durability requirement.

18. The potential for impacts on endangered and threatened species is nil as there are no endangered and threatened species in the site.

19. Assessment of potential impacts to federally listed endangered and threatened species or designated critical habitat. Assessment should include endangered and threatened species list within the county the project is located in – There are no federally listed endangered and threatened species.

20. Not applicable

21. Assessment of direct and indirect adverse and/or beneficial environmental effects that would result from the project, including but not limited to: aquatic habitat; hydrology and hydraulics at the project site as well as flooding and flood plains etc…- The anticipated benefit to the site would be the control of soil erosion of the arroyo.

22. Describe short and/or long-term maintenance requirements and issues – This structure will be used as a field demonstration facility. There will be little to nil maintenance requirements to this structure.

23. Statement describing how impacts to waters of U.S. are to be avoided and minimized. Provided conceptual mitigation plan or explain why compensatory mitigation should not be required for the proposed impacts – There will be no impact and so no mitigation would be required.

24. N/A

Appendix A: A 50-year Flood Estimate of the Construction Site

This report provides an estimate of 50-year, 24 hour flood estimate of the calculated drainage area of the selected site (Lat: N 34° 6' 0.1'' and Lon: W 107° 2' 17.2''). An Estimate of the drainage area is calculated, also soil type was determined based on test laboratory test and observation which was further used to estimate; depth of runoff, volume of runoff , time of concentration, unit peak discharge, peak of flow rate, water velocity, and flow rate.

Drainage Area

The drainage area is approximately 8,573.10 acres containing the drainage channels will contribute and direct rainfall to the catchments is surveyed. The area is comprised of Group A soils that have low runoff potential. "These soils have high infiltration rates evenwhen thoroughly wetted. They consist chiefly of deep, well to excessively drained sand or gravels. This group also includes sand, loamy sand and sandy loam that have experienced urbanization but have not been significantly compacted (Ferguson, 66)", shown in Figure 1.

Group	Description
A	Group A soils have low runoff potential. They have high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sands or gravels. This group also includes sand, loamy sand and sandy loam that have experienced urbanization but have not been significantly compacted.
B	Group B soils have moderate infiltration rates when thoroughly wetted. They consist chiefly of moderately deep to deep, moderately well- to well-drained soils with moderately fine to moderately coarse textures. This group also includes silt loam and loam that have experienced urbanization but have not been significantly compacted.
C	Group C soils have low infiltration rates when thoroughly wetted. They consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. This group also includes sandy clay loam that has experienced urbanization but has not been significantly compacted.
D	Group D soils have high runoff potential. They have very low infiltration rates when thoroughly wetted. They consist chiefly of clay soils with high swelling potential, soils with permanent high water tables, soils with clay pans or clay layers at or near the surface, and shallow soils over nearly impervious material. This group also includes clay loam, silty clay loam, sandy clay, silty clay and clay that have experienced urbanization but have not been significantly compacted.
A/D	The compound classification A/D indicates that the natural soil is in group D because a high water table impedes infiltration and transmission, but following artificial drainage such as with perforated pipe underdrains, the soil's classification is changed to A.

Figure 1: Hydrologic Soils Groups (Ferguson, 66)

The Soil Conservation Service (SCS) method presently known as the Natural Resource Conservation Service (NRCS) was used to determine curve number for runoff, since it is more useful than the rational method. The surface cover is categorized as desert shrub with 30-70% ground cover thus yielding a curve number (CN) of 55 (Ferguson, 67) as found in Figure 2.

Surface Cover	Hydrologic Soil Group			
	A	B	C	D
Natural areas				
Woods				
No grazing; litter and brush cover the soil	30	55	70	77
Grazed but not burned, some forest litter covers soil	36	60	73	79
Heavy grazing or regular burning destroys litter, brush	45	66	77	83
Desert shrub (saltbush, mesquite, creosote bush, etc.)				
>70% ground cover	49	68	79	84
30 to 70% ground cover	55	72	81	86
<30% ground cover (litter, grass, and brush overstory)	63	77	85	88
Brush, grass, and weeds in arid and semiarid regions				
>70% ground cover	49	62	74	85
30 to 70% ground cover	60	71	81	89
<30% ground cover (litter, grass, and brush overstory)	71	80	87	93
Pinyon and juniper with grass understory				
>70% ground cover	21	41	61	71
30 to 70% ground cover	40	58	73	80
<30% ground cover (litter, grass, and brush overstory)	63	75	85	89
Brush, grass, and weeds in humid regions				
>75% ground cover	30	48	65	73
50 to 75% ground cover	35	56	70	77
<50% ground cover	48	67	77	83
Oak-aspen mountain brush (oak, aspen, bitter brush, etc.)				
>70% ground cover	20	30	41	48
30 to 70% ground cover	38	48	57	63
<30% ground cover (litter, grass, and brush overstory)	56	66	74	79
Sagebrush with grass understory				
>70% ground cover	22	35	47	55
30 to 70% ground cover	37	51	63	70
<30% ground cover (litter, grass, and brush overstory)	52	67	80	85

Figure 2: Curve Number Values (Ferguson, 67)

Depth of Runoff (Qd)

Using the National Oceanic and Atmospheric Administration's (NOAA) data for construction site at the given latitude and longitude for 50- year, 24 hour precipitation intensity is 0.14 in/hr. Using the basic SCS method and runoff equation:

$$Q_d = \frac{(P - I_a)^2}{(P - I_a + S)}$$

Where

P= 2.73 in, depth of rain fall in 24 hr period (in) (from NOAA)

I_a = 0.2S (in)

S= (1000/CN)-10 (in)

Therefore

Q_d = 0.13 in.

Volume of Runoff (Q_{vol})

The combination of cut-and-weigh and topographic map of Magdalena, an approximate drainage area was calculated. Using the SCS Method:

$$Q_{vol} = {(Q_d A_d)}/{12}$$

Where

A_d = 8,573.10 acres

12 = Conversion factor (in/ft)

Therefore

Q_{vol} = 92.86 acre-ft

Time of Concentration (t_c)

Using an estimated length of the longest drainage channel from the highest point to the approximate point of construction on the map, the hydraulic length (L_h) was determined. Also, the slope along the hydraulic length (G) was determined using the highest and lowest elevation of the drainage area. The SCS TP -149 method (adapted from Kent, 1968) in figure 3:

$$t_c = {L_{CN}}/{G^{0.5}}$$

Where

L_h = hydraulic length = 32808 (ft)

G =Slope along the hydraulic length = [(7200 ft- 5600 ft / L_h] * 100

L_{CN} = 117

Therefore

t_c = 31.7 minutes

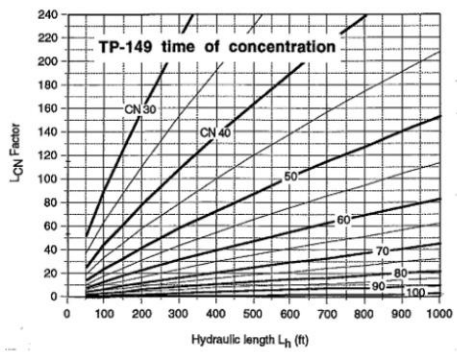


Figure 3: LCN Factor (Ferguson, 73)

Unit Peak Discharge (q_u)

The construction site is found in type II rainfall distribution, the SCS rainfall distribution in Figure 4 was used to estimate the peak discharge from the plot.

Where

t_c = 31.7 minutes

I_a/p = 0.60

q_u = 0.18 cfs /acre/in

Figure 4: Rainfall distribution (Ferguson, 75)

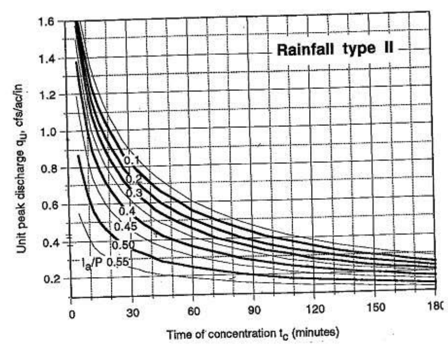


Figure 5: Unit Peak discharge for Type II (Ferguson, 73)

Peak Flow Rate (q_p)

Using the SCS Method:

$$q_p = q_u A_d Q_d F_p$$

Where

F_p = Pond and swamp factor = 1

Therefore

q_p = 3042.55 cfs For a 50 yr. flood conditions

Note

The values derived in this report are estimated and not exact. Water velocity could not be derived as area of inflow is not known at this time. A USGS map of the Magdalena area is used.

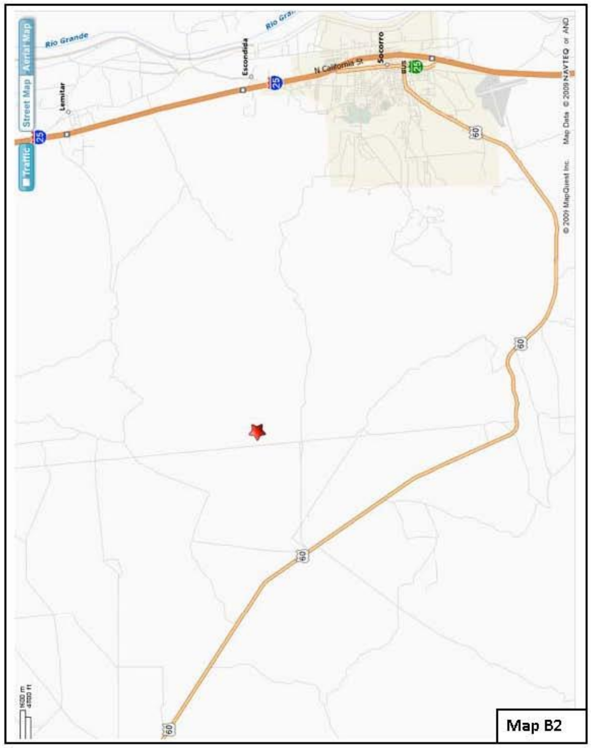
POINT PRECIPITATION FREQUENCY ESTIMATES FROM NOAA ATLAS 14

New Mexico 34.108 N 107.061 W 5830 feet

from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 1, Version 4
G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley
NOAA, National Weather Service, Silver Spring, Maryland, 2006
Extracted: Sun Jul 12 2009

Precipitation Frequency Estimates (inches)

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	0.21	0.32	0.39	0.53	0.65	0.76	0.82	0.94	1.04	1.25	1.38	1.60	1.88	2.11	2.72	3.33	4.10	4.68
2	0.27	0.41	0.51	0.68	0.84	0.97	1.05	1.20	1.32	1.58	1.74	2.00	2.35	2.64	3.40	4.16	5.10	5.82
5	0.36	0.55	0.68	0.91	1.13	1.29	1.37	1.54	1.68	1.98	2.16	2.50	2.90	3.28	4.16	5.05	6.14	7.01
10	0.43	0.65	0.81	1.09	1.35	1.55	1.64	1.81	1.97	2.30	2.52	2.90	3.35	3.79	4.75	5.73	6.91	7.89
25	0.52	0.80	0.99	1.33	1.65	1.93	2.02	2.22	2.38	2.77	3.01	3.46	3.98	4.51	5.55	6.63	7.91	9.02
50	0.60	0.91	1.12	1.51	1.88	2.25	2.35	2.56	2.73	3.14	3.41	3.92	4.50	5.09	6.17	7.32	8.63	9.84
100	0.67	1.02	1.27	1.71	2.12	2.61	2.73	2.94	3.11	3.54	3.84	4.41	5.04	5.70	6.82	8.02	9.36	10.65
200	0.75	1.15	1.42	1.91	2.37	3.01	3.14	3.35	3.54	3.98	4.30	4.93	5.62	6.36	7.47	8.72	10.07	11.44
500	0.86	1.31	1.62	2.19	2.71	3.62	3.77	3.97	4.16	4.61	4.97	5.69	6.44	7.31	8.38	9.67	11.05	12.46
1000	0.95	1.44	1.79	2.41	2.98	4.15	4.32	4.51	4.69	5.13	5.53	6.33	7.12	8.10	9.14	10.45	11.86	13.22



Map B2

11

Figure B1

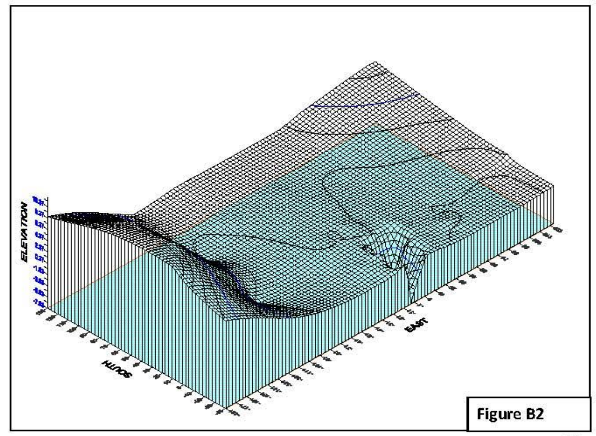


Figure B2

12

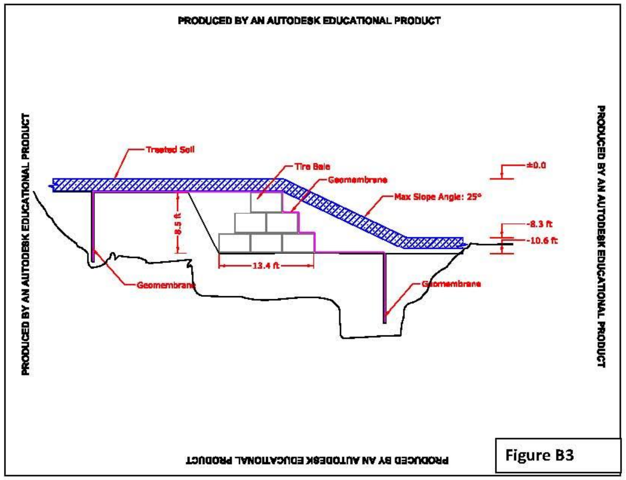
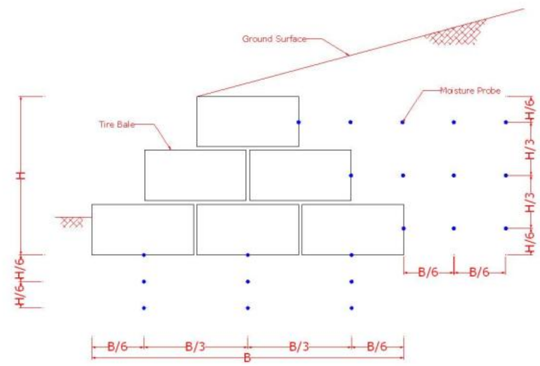


Figure B3



SECTION

PRELIMINARY JURISDICTIONAL DETERMINATION FORM

This preliminary JD finds that there *"may be"* waters of the United States on the subject project site, and identifies all aquatic features on the site that could be affected by the proposed activity, based on the following information:

District Office: Albuquerque District File/ORM #: PJD Date: August 28

State: NM City/County: Socorro, Socorro

Nearest Waterbody: Rio Grande River

Location: TRS, LatLong or UTM: Latitude: N 34o6'0.1" and Longitude: W 107o2'17.2"

Name/ Address of Person Requesting PJD: Ashok K. Ghosh, PI, Mechanical Engineering Department, 801 Leroy Place, New Mexico Tech, Socorro, NM 87801, Phone: 575-835-5505; fax: 575-835-5209; email: ashok@nmt.edu

Identify (Estimate) Amount of Waters in the Review Area:

Non-Wetland Waters: linear ft width acres Stream Flow:

Wetlands: acre(s) Cowardin Class:

Name of Any Water Bodies on the Site Identified as Section 10 Waters: Tidal: Non-Tidal:

- [ ] Office (Desk) Determination
- [ ] Field Determination: Date of Field Trip:

SUPPORTING DATA: Data reviewed for preliminary JD (check all that apply - checked items should be included in case file and, where checked and requested, appropriately reference sources below):

- [x] Maps, plans, plots or plat submitted by or on behalf of the applicant/consultant: Already submitted during quarterly reports
- [x] Data sheets prepared/submitted by or on behalf of the applicant/consultant.
  - [ ] Office concurs with data sheets/delineation report.
  - [ ] Office does not concur with data sheets/delineation report.
- [ ] Data sheets prepared by the Corps
- [ ] Corps navigable waters' study:
- [ ] U.S. Geological Survey Hydrologic Atlas:
  - [ ] USGS NHD data.
  - [ ] USGS 8 and 12 digit HUC maps.
- [ ] U.S. Geological Survey map(s). Cite quad name:
- [ ] USDA Natural Resources Conservation Service Soil Survey. Citation:
- [ ] National wetlands inventory map(s). Cite name:
- [ ] State/Local wetland inventory map(s):
- [ ] FEMA/FIRM maps:
- [ ] 100-year Floodplain Elevation is:
- [ ] Photographs: [ ] Aerial (Name & Date): [ ] Other (Name & Date):
- [ ] Previous determination(s). File no. and date of response letter:
- [ ] Other information (please specify):

IMPORTANT NOTE: The information recorded on this form has not necessarily been verified by the Corps and should not be relied upon for later jurisdictional determinations.

Signature and Date of Regulatory Project Manager (REQUIRED)

Ashok kumar Ghosh 08/28/09

Signature and Date of Person Requesting Preliminary JD (REQUIRED, unless obtaining the signature is impracticable)

EXPLANATION OF PRELIMINARY AND APPROVED JURISDICTIONAL DETERMINATIONS:

1. The Corps of Engineers believes that there may be jurisdictional waters of the United States on the subject site, and the permit applicant or other affected party who requested this preliminary JD is hereby advised of his or her option to request and obtain an approved jurisdictional determination (JD) for that site. Nevertheless, the permit applicant or other person who requested this preliminary JD has declined to exercise the option to obtain an approved JD in this instance and at this time.
2. In any circumstance where a permit applicant obtains an individual permit, or a Nationwide General Permit (NWP) or other general permit verification requiring "preconstruction notification" (PCN), or requests verification for a non-reporting NWP or other general permit, and the permit applicant has not requested an approved JD for the activity, the permit applicant is hereby made aware of the following: (1) the permit applicant has elected to seek a permit authorization based on a preliminary JD, which does not make an official determination of jurisdictional waters; (2) that the applicant has the option to request an approved JD before accepting the terms and conditions of the permit authorization, and that basing a permit authorization on an approved JD could possibly result in less compensatory mitigation being required or different special conditions; (3) that the applicant has the right to request an individual permit rather than accepting the terms and conditions of the NWP or other general permit authorization; (4) that the applicant can accept a permit authorization and thereby agree to comply with all the terms and conditions of that permit, including whatever mitigation requirements the Corps has determined to be necessary; (5) that undertaking any activity in reliance upon the subject permit authorization without requesting an approved JD constitutes the applicant's acceptance of the use of the preliminary JD, but that either form of JD will be processed as soon as is practicable; (6) accepting a permit authorization (e.g., signing a proffered individual permit) or undertaking any activity in reliance on any form of Corps permit authorization based on a preliminary JD constitutes agreement that all wetlands and other water bodies on the site affected in any way by that activity are jurisdictional waters of the United States, and precludes any challenge to such jurisdiction in any administrative or judicial compliance or enforcement action, or in any administrative appeal or in any Federal court; and (7) whether the applicant elects to use either an approved JD or a preliminary JD, that JD will be processed as soon as is practicable. Further, an approved JD, a proffered individual permit (and all terms and conditions contained therein), or individual permit denial can be administratively appealed pursuant to 33 C.F.R. Part 331, and that in any administrative appeal, jurisdictional issues can be raised (see 33 C.F.R. 331.5(a)(2)). If, during that administrative appeal, it becomes necessary to make an official determination whether CWA jurisdiction exists over a site, or to provide an official delineation of jurisdictional waters on the site, the Corps will provide an approved JD to accomplish that result, as soon as is practicable.

PRELIMINARY JURISDICTIONAL DETERMINATION FORM

This preliminary JD finds that there *"may be"* waters of the United States on the subject project site, and identifies all aquatic features on the site that could be affected by the proposed activity, based on the following information:

Appendix A - Sites

District Office | File/ORM # | PJD Date:

State | City/County | Person Requestinq PJD

Site Number	Latitude	Longitude	Cowardin Class	Est. Amount of Aquatic Resource in Review Area	Class of Aquatic Resource

Notes:

This is attached with the Check list for General (nationwide & regional) permit submittals.

APPENDIX F2

Correspondence New Mexico Army Corp of Engineers

DEPARTMENT OF THE ARMY
ALBUQUERQUE DISTRICT, CORPS OF ENGINEERS

September 9, 2009

REPLY TO
ATTENTION OF:

Regulatory Division

SUBJECT: Action Number SPA-2009-00554-ABQ, NMDOT/NMT Tire Bale Research Demonstration Project

Ashok Ghosh
New Mexico Tech
Engineering Department
801 Leroy Place
Socorro, NM 87801

Dear Mr. Ghosh:

The U.S. Army Corps of Engineers (Corps) is in receipt of your letter dated August 28, 2009 concerning your proposal to install used tire bales as bank stabilization in an ephemeral arroyo and monitor potential impacts to water quality and the bales' stability during stream flow. The activity involves the installation of approximately 350 cubic yards of compressed used tire bale with an underlying geomembrane and compacted soil cover in Nogal Arroyo, near Socorro, Soccoro County, New Mexico. We have assigned Action No. SPA-2009-00554-ABQ to this activity. To avoid delay, please include this number in all future correspondence concerning this project.

We have reviewed this project in accordance with Section 404 of the Clean Water Act and Section 10 of the Rivers and Harbors Act of 1899. Under Section 404, the Corps regulates the discharge of dredged and fill material into waters of the United States (U.S.), including wetlands. Our responsibility under Section 10 is to regulate any work in, or affecting, navigable waters of the U.S. Based on your description of the proposed work, and other information available to us, we have determined that the proposed project will involve activities subject to Section 404. Therefore, a Department of the Army permit is required.

We have determined that this project is authorized by Nationwide Permit NWP 13 Bank Stabilization.. A summary of this permit and the regional conditions for New Mexico are available on our website at www.spa.usace.army.mil/reg/. You are only authorized to conduct the work described in your submittal. Compliance with the enclosed water quality certification is a requirement of this permit. To use this permit, you must ensure that the work complies with the terms and conditions listed in the permit.

Our review of this project also addressed its effects on threatened and endangered species and historic properties in accordance with general conditions 17 and 18. Based on the information provided, we have determined that this project will not affect any species listed as threatened or endangered by the U.S. Fish and Wildlife Service within the permit area. We have also determined that this project will not affect historic properties listed, or eligible for listing, in the National Register of Historic Places. However, please note that you are responsible for meeting the requirements of general condition 17 on endangered species and general condition 18 on historic properties.

Our verification for the construction of this activity under this nationwide permit is valid for two years from the date of this letter, unless prior to that date the nationwide permit is suspended, revoked, or modified such that the activity would no longer comply with the terms and conditions of the nationwide permit regionally or nationally. The Corps will issue a public notice announcing the changes when they occur. Furthermore, activities that have commenced, or are under contract to commence, in reliance on a nationwide permit will remain authorized provided the activity is completed within 12 months of the date of the nationwide permits expiration, modification, or revocation, unless discretionary authority has been exercised on a case-by-case basis to modify, suspend, or revoke the authorization in accordance with 33 CFR 330.4(e) and 33 CFR 330.5(c) or (d). Continued confirmation that an activity complies with the terms and conditions, and any changes to the nationwide permit, is the responsibility of the permittee.

The Corps based this decision on a preliminary jurisdictional determination (JD) that there may be waters of the United States on the project site. Preliminary JDs are advisory in nature and may not be appealed. An approved JD is an official Corps determination that "waters of the U.S." and/or "navigable waters of the U.S." are either present or absent on a particular site. An approved JD precisely identifies the limits of those waters on the project site determined to be jurisdictional under the CWA or RHA. If you wish, you may request that the USACE reevaluate this case and issue an approved JD. If you request an approved JD, you may not begin work until the approved JD, which may require coordination with the Environmental Protection Agency, is completed. Please contact me if you wish to request an approved JD for this case.

You must sign and submit to us the enclosed certification that the work, including any required mitigation, was completed in compliance with the nationwide permit. You should submit your certification within 30 days of the completion of work.

This permit is not an approval of the project design features, nor does it imply that the construction is adequate for its intended purpose. This permit does not authorize any injury to property or invasion of rights or any infringement of Federal, state or local laws or regulations.

You must possess the authority, including property rights, to undertake the proposed work.

If you have any questions concerning our regulatory program, please contact me at 505-342-3280 or Deanna.L.Cummings@usace.army.mil. At your convenience, please complete a Customer Service Survey on-line available at http://per2.nwp.usace.army.mil/survey.html.

Sincerely,

Deanna L. Cummings
Regulatory Project Manager
Albuquerque District

Enclosure(s):

Blanket Ephemeral Water Quality Certification

Copies furnished (via email, without enclosures):

David Menzie, NMED (david.menzie@state.nm.us)

Certification of Compliance
with Department of the Army Nationwide Permit

Action Number: SPA-2009-00554-ABQ

Name of Permittee: Ashok Ghosh New Mexico Tech

Nationwide Permit: NWP 13 Bank Stabilization.

Upon completion of the activity authorized by this permit and any mitigation required by the permit, sign this certification and return it to the following address:

Deanna Cummings
Albuquerque District, U.S. Army Corps of Engineers
ATTN: Regulatory Division
4101 Jefferson Plaza, NE
Albuquerque, New Mexico 87109-3435

Please note that your permitted activity is subject to a compliance inspection by an U.S. Army Corps of Engineers representative. If you fail to comply with this permit, you are subject to permit suspension, modification, or revocation.

Please enclose photographs showing the completed project (if available).

I hereby certify that the work authorized by the above referenced permit has been completed in accordance with the terms and conditions of the said permit, and required mitigation was completed in accordance with the permit conditions.

Date Work Started May 10th 2010

Date Work Completed May 24th 2010

05/24/2010 — Date

Signature of Permittee

APPENDIX F3

Correspondence New Mexico Environmental Department

NEW MEXICO
ENVIRONMENT DEPARTMENT

Solid Waste Bureau

Harold Runnels Building - Suite S 2050
1190 St. Francis Drive, P. O. 5469
Santa Fe, New Mexico 87502-5469
Phone (505) 827-0197 Fax (505) 827-2902
www.nmenv.state.nm.us

BILL RICHARDSON
Governor
DIANE DENISH
Lieutenant Governor

RON CURRY
Secretary
JON GOLDSTEIN
Deputy Secretary

November 30, 2009

Dr. Ashok Kumar Ghosh
New Mexico Tech - 118 WEIR
801 Leroy Place
Socorro, NM 87801

Dear Dr. Ghosh;

Pursuant to the New Mexico Recycling, Illegal Dumping and Scrap Tire Management Rule 20.9.20.NMAC, the New Mexico Environment Department (NMED), Solid Waste Bureau, has completed a review of the Application for Civil Engineering Application Permit for the NM DOT / NMT Tire Bale Research Demonstration Project and has deemed it complete. Enclosed, please find Civil Engineering Application Permit Certificate #0128013 TCE and a copy of its Terms and Conditions.

If, at any time, you wish to modify the project, and/or storage plan outlined in the Terms and Conditions document enclosed, please submit the proposed modifications in writing to NMED, Solid Waste Bureau, so that the changes may be reviewed and new Terms and Conditions may be issued before modification begins. If I can be of any assistance, do not hesitate to contact me at (505) 827-0559.

Sincerely,

Toni J. Duggan

Toni J. Duggan
Tire Recycling Coordinator

Attachment: Civil Engineering Application Permit Certificate
Terms and Conditions

cc: Virgil Valdez, Research Bureau, NM Department of Transportation
Marco Banales, Enforcement Officer, EA-I, Solid Waste Bureau
Teri Monaghan, Enforcement Officer, EA-I, Solid Waste Bureau

NM DOT / NMT TIRE BALE RESEARCH
DEMONSTRATION PROJECT

CIVIL ENGINEERING APPLICATION PERMIT

TERMS AND CONDITIONS

In accordance with the Recycling and Illegal Dumping Act, Section 74-13-1 et seq. NMSA 1978 and the Recycling, Illegal Dumping and Scrap Tire Management Rule, Section 20.9.20.20 NMAC, a civil engineering application permit is hereby approved for NM DOT / NMT Tire Bale Research Demonstration Project, subject to the following terms and conditions.

1. PROJECT NAME: NM DOT / NMT Tire Bale Research Demonstration Project

2. FACILITY LOCATION: New Mexico Tech, Nogal Arroyo near Socorro, NM

3. FACILITY and PROPERTY OWNER: New Mexico Tech

4. CONTACT PERSONS: Ashok Kumar Ghosh

5. CONTACT ADDRESS: New Mexico Tech - 118 WEIR
801 Leroy Place
Socorro, NM 87801

6. CONTACT PHONE: (575) 835-5505

7. CONTACT E-MAIL ADDRESS: ashok@nmt.edu

8. PROJECT DESCRIPTION:This demonstration project is part of NM DOT / New Mexico Tech's research project entitled "Standards for Tire Bale Erosion Control and Bank Stabilization Projects: Engineering Validation of Existing Practice and Implementation." The structure will be built of tire-bales in three rows to form a pyramid shape. Tire-bales are covered with native soil compacted in less than 12 in. thick layers of soil. A steel wire-mesh will be placed on the tire-bales to increase the integrity of the structure. To avoid developing any extra pore water pressure behind and beneath the structure, a combination of an impermeable layer of geomembrane and a thin layer of non-expansive clay will be used around the structure. The topsoil will be treated with either 10% CKD or 10% cement by weight to provide more resistance against erosion. Four short concrete (or steel) piles at the four corners of the structure are used to increase safety of the structure. If time and funds permit, both head-cut and side cut structures will be constructed. For more details, go to the permit application and 4th Quarter FY 09 and 1st Quarter FY 10 Quarterly Reports of the NM DOT - NMT Tire Bale Erosion Control and Bank Stabilization Projects Engineering Validation of Existing Practice and Implementation.

9. APPROXIMATE NUMBER OF TIRES
TO BE USED FOR THE PROJECT: 50 BALES

10. SPECIAL PERMIT CONDITION: none

11. APPROXIMATE START DATE: December 2009

12. APPROXIMATE COMPLETION DATE: February 2010

13. FINAL REPORT: A final report shall be submitted to the department 30 days after completion and shall include:

A. as built drawings including cross section and plan view, if different from the proposed design;
B. the total number of scrap tires used for the civil engineering application;
C. the dimensions of the civil engineering application; and
D. photographs of the civil engineering application.

14. PERMIT MODIFICATION: Amendments to the permit shall be obtained from the Secretary prior to making any modifications to the Terms and Conditions specified in this document.

16. PERMIT EXPIRATION: A permit issued for civil engineering applications shall expire when the project has been completed and the final report has been submitted and approved in writing.

17. GENERAL OPERATING AND CONSTRUCTION REQUIREMENTS: Owners and operators of all tire recycling facilities and civil engineering application sites shall operate and construct the tire recycling facility or civil engineering application in a manner that:
A. does not cause a public nuisance or create a potential hazard to public health, welfare or the environment;
B. is in compliance with rules adopted by state and local fire authorities.
C. operates and maintains the facility in accordance with 20.9.20.37 NMAC.

SOCORRO COUNTY NEW MEXICO

October 20, 2009

Ashok Kumar Ghosh, Ph.D., P.E.
Associate Professor
118 WIER, 801 Leroy Place
New Mexico Tech, Socorro , NM 87801

Dear Dr. Ghosh:

This letter certifies that at this time the County of Socorro does not have any zoning or structure ordinances in effect. Therefore, there are no regulations or restrictions related to your bank stabilization project on New Mexico Tech property in Socorro County. Additionally, county zoning laws would not apply to lands owned by the state (Santa Fe County v. Milagro Wireless 130 NM 771 (Ct of App 2001))

If you require further information, please let me know.

Sincerely,

Delilah Walsh
County Manager

Pc: File: Correspondence

PO Box I, Socorro, NM 87801
Phone: 575.835.0589 Toll free: 800.727.0206
Fax: 575.835.4629
dwalsh@co.socorro.nm.us www.co.socorro.nm.us

Civil Engineering Application Permit

NM DOT / NMT TIRE BALE RESEARCH DEMONSTRATION PROJECT

has met the criteria of the New Mexico Recycling, Illegal Dumping and Scrap Tire Management Rule, Section 20.9.20.20 NMAC and is permitted as a Civil Engineering Application

Certificate #0128013 TCE

issued by

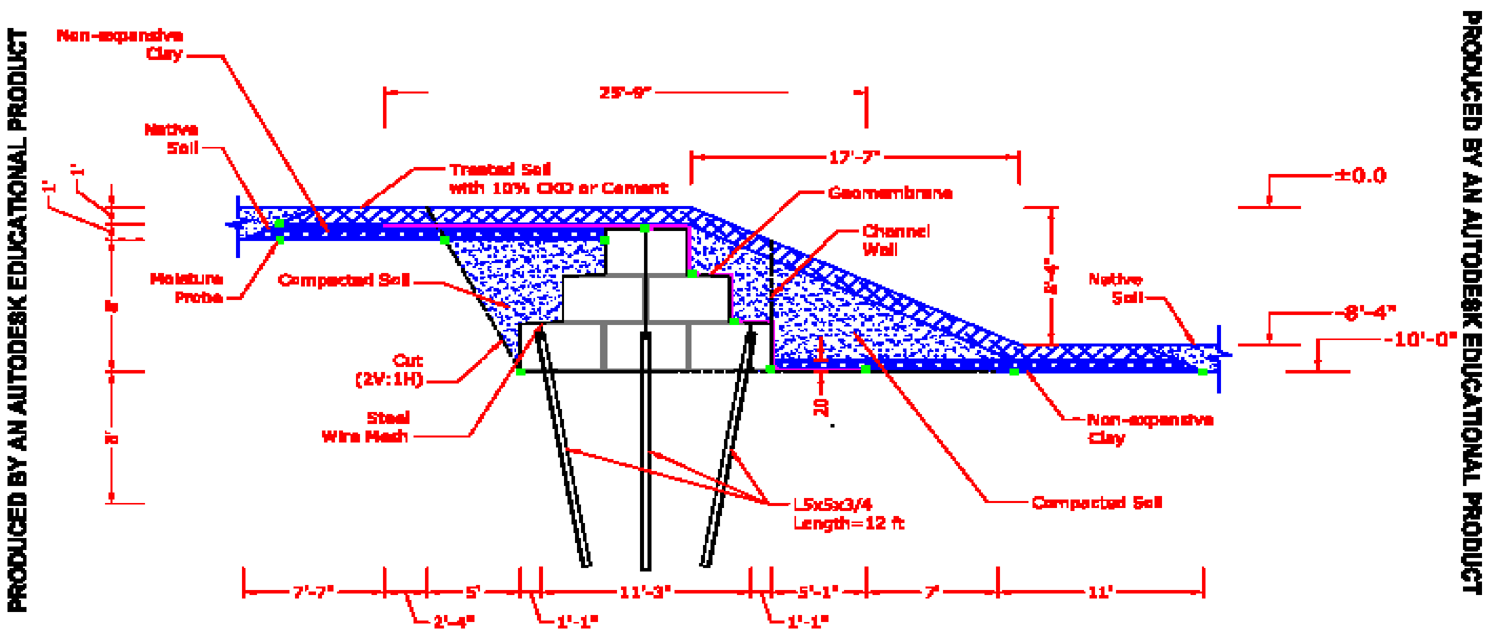
State of New Mexico Environment Department

November 30, 2009

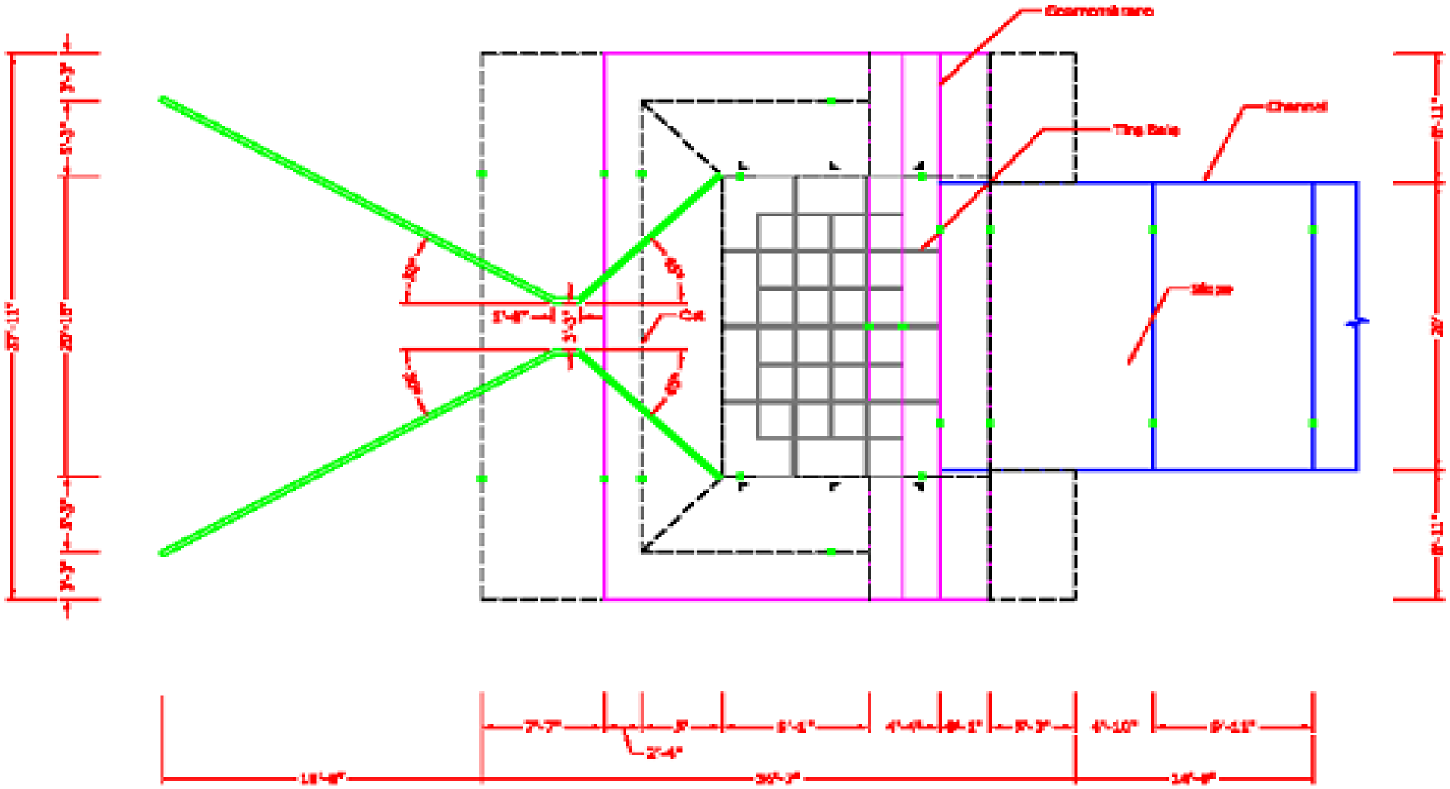
Auralie Ashley-Marx, Chief
Solid Waste Bureau

APPENDIX G

Construction Drawings

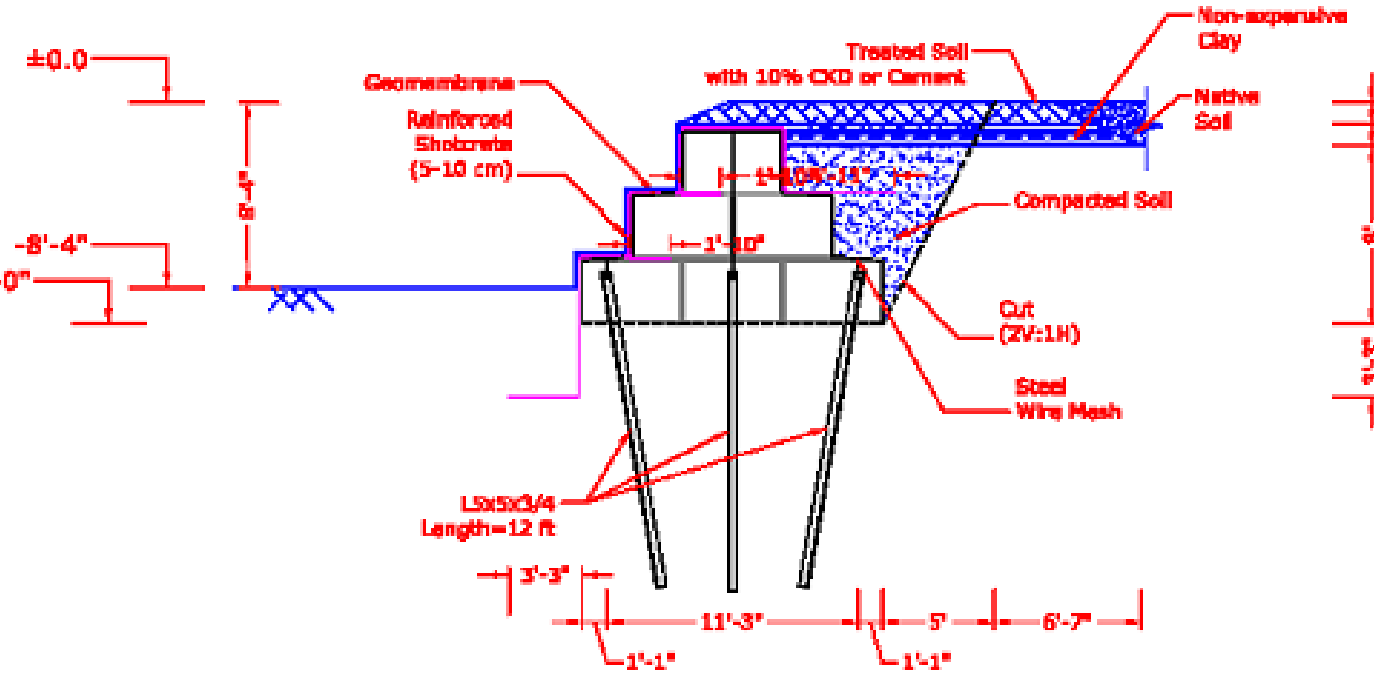


(Head-Cutting Structure – Sectional view)

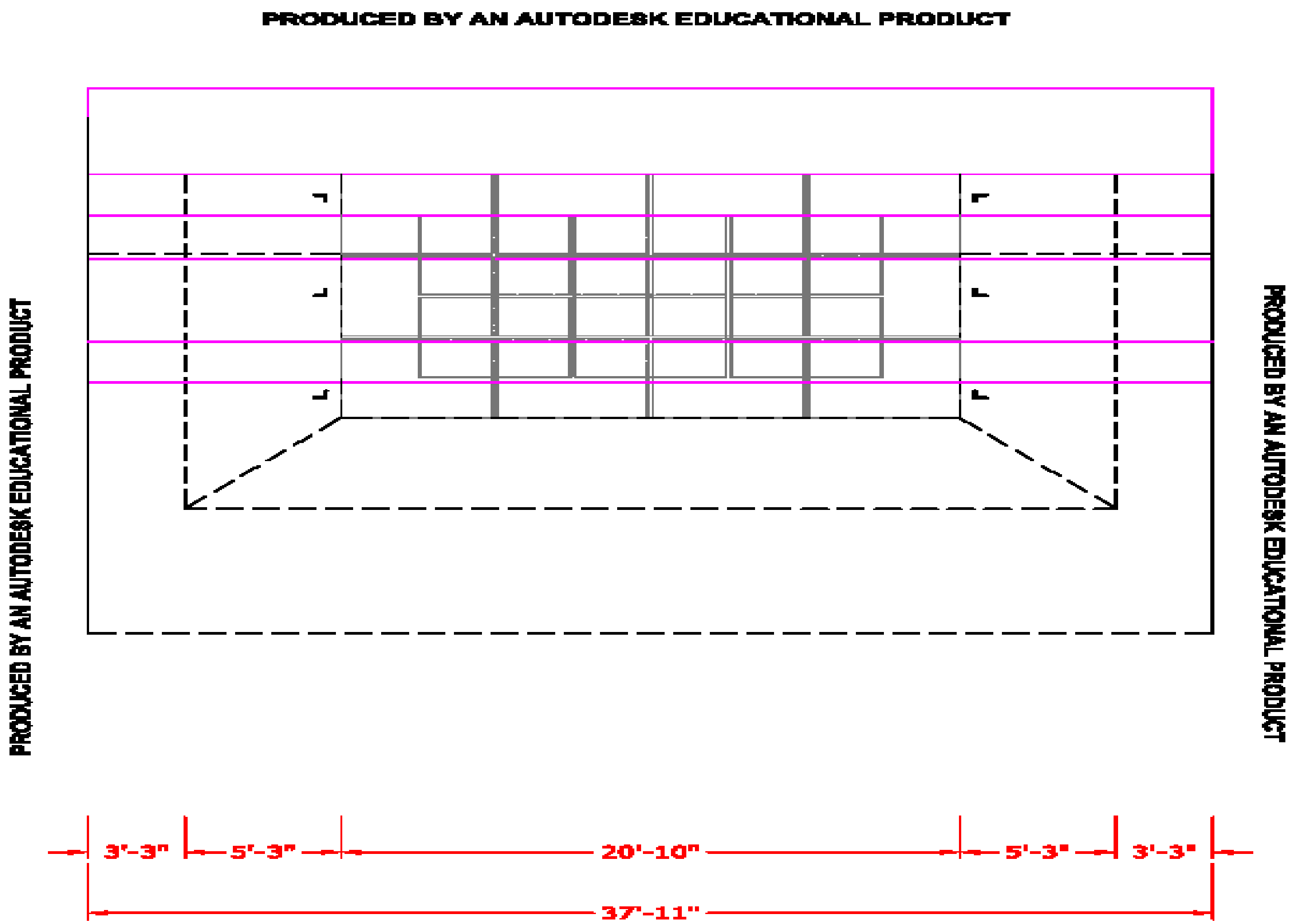


TOP VIEW

(Head-Cutting Structure – Plan view)



(Side Structure – Sectional view)



(Side Structure – Plan view)

APPENDIX H Commands LOGGER-8-USB Calabration

The controller board may be configured, controlled and monitored by using the following commands. <CR> designates a carriage return.

Open HyperTerminal - Normally under Start/Programs/Accessories/Communications in Windows.

Set up the session for the appropriate USB/comm port.   Set the comm port setting to:115200 baud, 8-N-1. (Note if you have Rev 1.00 of the software the baud rate is 9600.) The easiest way to figure out, which comm port number the USB board is being references as, is to plug it in and look at the list, then remove it and see which comm port disappears from the list.

Data in flash memory is stored as a circular queue. When the memory is full, newer data will overwrite older records.

When the logger powers on**,** it may take up to 20 seconds for it to initialize the flash memory.

The default sample rate is once per minute. Power is applied across all sensor channels, and they are sampled at the same instant.

h<CR>	Displays the help menu.
v<CR>	Displays the current software version.
o<CR>	Toggle the streaming of data output, on or off. The default is to stream data out.
d<CR>	Returns the current date.
d [DD/MM/YYYY]<CR>	Sets the current date.
t<CR>	Returns the current time.

t [HH:MM:SS]<CR>	Sets the current time.
heart<CR>	Enable or disable the heartbeat LED. The purpose of the heart beat LED is to let you know that the board is running. The reason you would turn off the LED would be to conserve battery power.
u [sensor index] 'string[5]'<CR>	Sets the units of measurement for designated probe. Probes are numbered 1 to 8. Single quotes are necessary around the string. Use an empty string to remove units. For example ''.
p [secs]<CR>	Sets or returns the sensor power on time before sampling in seconds. This allows power to stabilize before sampling the sensors. All sensors are powered together.
c [sensor index]<CR>	Returns the probe calibration table for the specified sensor.
c [sensor index] clear<CR>	Clear the probe calibration table for the specified sensor. When no table is used the raw voltage sensed is returned.
c [sensor index] (X,Y)<CR>	Sets the probe calibration table. X is sensor voltage, Y is mapped value. From 2 to 10 points may be entered into the calibration table for each sensor. To enter multiple points just re-invoke this command for each point for the selected sensor. The logger will automatically sort the points by voltage.
s<CR>	Gets the sample period in minutes and seconds.
s [MM:SS]<CR>	Sets the sample period in minutes and seconds. Use 00:00 to turn off sampling.

e [Sensor index]<CR>	Toggles enable/disable for a designated sensor.
get<CR>	Retrieve all valid data starting with the oldest valid record and ending with the newest record. We use a slow baud rate to conserve power. Retrieving all data in the internal flash memory, may take up to an hour. Use a "q<CR>" to terminate this command.
dump<CR>	Dump all valid records stored in flash, in non-sequential order. This command normally is not invoked. Because of the slow baud rate, this command may take up to 2 hours. The only way to terminate the command before it completes is to remove power from the logger. Use a "q<CR>" to terminate this command.
del<CR>	Delete all data in the internal memory.
wipe<CR>	Erase all flash memory. Use this to purge old data.

New Mexico Department of Transportation
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