



SERRI Report 70015-003

USE OF GEOTEXTILE AND GEOMEMBRANE TUBES TO CONSTRUCT TEMPORARY WALLS IN A FLOODED AREA



SERRI Project: *Increasing Community
Disaster Resilience Through Targeted
Strengthening of Critical Infrastructure*

Project Principal Investigator:

Isaac L. Howard, PhD

Report Written and Performed By:

Isaac L. Howard - Mississippi State University

Ed Trainer-TenCate™ Geotube®



This material is based upon work supported by the U.S. Department of Homeland Security under U.S. Department of Energy Interagency Agreement 43WT10301. The views and conclusions contained in this document are those of the authors and should not be interpreted as necessarily representing the official policies, either expressed or implied, of the U.S. Department of Homeland Security.

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Through Targeted Strengthening of Critical Infrastructure

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TEMPORARY WALLS IN A FLOODED AREA**

Performing Organization Report No. CMRC-09-04

Written By:

*Isaac L. Howard, PhD, Assistant Professor, Mississippi State University
Ed Trainer, Market Manager-Marine & Coastline, TenCate™ Geotube®*

Date Published:

May 2011

Prepared for
U.S. Department of Homeland Security
Under U.S. Department of Energy Interagency Agreement 43WT10301

Prepared by
OAK RIDGE NATIONAL LABORATORY
Oak Ridge, Tennessee 37831-6283
managed by
UT-BATTELLE, LLC
for the
U.S. DEPARTMENT OF ENERGY
under contract DE-AC05-00OR22725

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ACKNOWLEDGEMENTS

Thanks are due to many for the successful completion of this report. The authors are especially grateful for the financial support provided by the *Southeast Region Research Initiative (SERRI)*. In addition, due gratitude is extended to everyone employed at the *US Department of Homeland Security (DHS)* and *Oak Ridge National Laboratory (ORNL)* who worked diligently with the authors to make this project a success. A great deal of the success of this research can be attributed to the efforts of *DHS* and *ORNL* personnel.

All participants of the *2008 Geotextile Tubes Workshop* provided valuable guidance that was critical to the development of this report. The participants are named in the workshop report. This project would have been of much lower quality in absence of the input of these individuals who represented consultants, agencies, universities, and private companies. The workshop was conducted as part of this research effort and provided very useful information and guidance to the authors.

David Doodledge of *Water Structures Unlimited®* in Carlotta, CA provided guidance, material properties, case studies, photos, and a variety of other information pertaining to *AquaDam®* capabilities. Mr. Doodledge's contributions had a direct and positive impact to the work. Alan Juncker, Mark Gunzenhauser, and Mark Torre of *TenCate™ Geotube®* headquartered in Commerce, GA and Jody Dendurent formerly of *Tencate™ Geosynthetics* headquartered in Pendergrass, GA provided guidance related to geosynthetics and geotextile tubes. Assistance was also provided in scheduling and visiting the *Matagorda Ship Channel* and *Grand Isle*. Mike Groh of *Geo-Synthetics, Inc* and Tim Groh of *Integrated Water Solutions, LLC* are owed thanks for many items, notably assistance in visiting *Peoria Island*.

Site visits were also assisted by other entities who are owed thanks. Douglas Gaffney and Brent Cooper of *Ocean and Coastal Consultants* were helpful in securing information for the *Grand Isle* site visit, alongside the contractor who filled the geotextile tubes and allowed the research team's site visit; *Infrastructure Alternatives*. Doug Bryan of *Midwest Foundation Corporation* is owed thanks for assistance with *Peoria Island* and allowing the research team's visit. Similarly, Randy Boyd of *RLB Contracting, Inc.* is owed thanks for assistance with the *Matagorda Ship Channel* and allowing the research team's visit.

All five US fabricators of geotextile tubes identified by the research team are owed thanks for providing information relative to their products. They are: 1) Tony Bradley of *Bradley Industrial Textiles*; 2) Bill Smallwood of *Flint Industries*; 3) Mike Groh of *Geo-Synthetics, Inc*; 4) April Kiskis of *Industrial Fabrics*; and 5) Mark Gunzenhauser of *TenCate™ Geotube®*.

Thanks are especially due to those who reviewed the document at various stages and provided feedback. Dr. Thomas D. White of Mississippi State University and Mark Gunzenhauser of *TenCate™ Geotube®* are owed thanks for review of the report and providing technical guidance. Review of select portions of this report was performed by Douglas Gaffney, Mike Groh, and Doug Bryan.

LIST OF SYMBOLS

A	cross sectional area of geotextile tube
AOS	apparent opening size
$A T III$	Type III portland cement from Holcim plant in Artesia, MS
B	geotextile tube base contact width
β	angle of settlement that includes tension in geomembrane
C	geotextile tube circumference
c_{a-p}	peak adhesion during <i>ASTM D 5321</i> (kPa)
c_{a-r}	residual adhesion during <i>ASTM D 5321</i> (kPa)
CD	cross direction for geotextiles
CEE	Civil and Environmental Engineering
CIKR	Critical Infrastructure and Key Resources
CLSM	controlled low strength material
cov	statistical parameter coefficient of variation
D	dosage rate (kg/m ³)
DHS	Department of Homeland Security
DOD	Department of Defense
D_{10}	soil effective grain size corresponding to 10% finer on grain size curve
D_{50}	soil mean grain size corresponding to 50% finer on grain size curve
D_{60}	soil particle size corresponding to 60% finer on grain size curve
δ_p	peak angle of friction during <i>ASTM D 5321</i> (°)
δ_r	residual angle of friction during <i>ASTM D 5321</i> (°)
δ_U	interface friction angle on upper side of geomembrane
δ_L	interface friction angle on lower side of geomembrane
E	elastic modulus
ERDC	Engineer Research and Development Center
EOC	Emergency Operations Center
ESF	Emergency Support Function
ε_{max}	maximum strain
FEMA	Federal Emergency Management Agency
FM	fineness modulus
FS	factor of safety against geotextile tube rupture
FS_s	factor of safety against slope stability
FS_{slide}	factor of safety against sliding failure
FS_{Tip}	factor of safety against tipping failure
F_{R-W}	resultant force acting on the wall due to water
F_T	friction resistance between the top and bottom stacked geotextile tubes
F_{V-T}	vertical downward force induced by geomembrane tube
F_{V-W}	vertical upward force induced by pore water pressure
GDT	Geotube [®] dewatering test
G_s	specific gravity of soil solids by <i>ASTM D 854</i>
γ	unit weight of structure
γ_s	unit weight of slurry
γ_w	unit weight of water
H	total height of geotextile tube wall

h	geotextile or geomembrane tube height
HDPE	high density polyethylene
<i>HLT</i>	Hurricane Liaison Team
<i>HPO</i>	Hurricane Protection Office
H_{res}	depth of water in reservoir
H_w	height of water being resisted when water is present on one side of tube
h_{eff}	effective height of geotextile tube that is stacked on other geotextile tubes
<i>ICS</i>	Incident Command System
<i>IFAI</i>	Industrial Fabrics Association International
k	soil modulus
L	length of standard geotextile or geomembrane tube
l	unit length of geotextile tube wall
LL	liquid limit by <i>ASTM D 4318</i>
LL_{OD}	liquid limit on oven dry basis by <i>ASTM D 4318</i>
L_R	interior length of geotextile tube water reservoir at base
<i>MARV</i>	minimum average roll values
MD	machine direction for geotextiles
<i>MSU</i>	Mississippi State University
<i>NIMS</i>	National Incident Management System
<i>NRF</i>	National Response Framework
<i>OMC</i>	optimum moisture content (%)
<i>ORNL</i>	Oak Ridge National Laboratory
ω_p	static friction coefficient
ω_{p-min}	minimum static friction coefficient that will prevent sliding
P_{base}	pressure at base of geotextile tube
PI	plasticity index by <i>ASTM D 4318</i>
PL	plastic limit by <i>ASTM D 4318</i>
PVC	polyvinyl chloride
ψ	contact angle between geotextile tubes in a geotextile tube wall
pwp	pore water pressure
QA	quality assurance
R	radius of curvature for geomembrane tube reservoir
<i>SB-HB</i>	ground granulated blast furnace slag from Hayward Baker project
<i>SC1</i>	specialty portland cement from Holcim plant in Artesia, MS
<i>SC6</i>	specialty Portland cement from Holcim plant in Theodore, AL
<i>SERRI</i>	Southeast Region Research Initiative
SG_{Int}	specific gravity of fill material
s_u	undrained shear strength (kg/cm ²)
σ_n	stress applied due to hydrostatic pressure
σ_{allow}	allowable geomembrane stress
T	maximum tensile force in geotextile
$TS\%$	total solids (%)
t	thickness of geomembrane liner
UC	unconfined compression
<i>USACE</i>	U.S. Army Corps of Engineers
<i>USCS</i>	unified soil classification system

UVI	ultraviolet light infiltration rating
V	volume per unit length of geotextile tube
V_{13}	volume of tube 13 in geomembrane tube reservoir
V_R	volume of water reservoir in millions of liters
W	filled width of geotextile or geomembrane tube
W_R	interior width of geotextile tube water reservoir at base
W_{T-A}	weight of tube A in geotextile tube wall
w/c	water to cement ratio
w/cm	water to cementitious material ratio
$w\%$	moisture content (%)
$w\%_{mixed}$	moisture content during mixing (%)
x	distance required to mobilize geomembrane deformation

CHAPTER 1 - INTRODUCTION

1.1 General and Background Information

The work presented in this report was developed in partial fulfillment of the requirements of Task Order 4000064719 sponsored by the *Department of Homeland Security (DHS)* through its *Southeast Region Research Initiative (SERRI)* program administered by *UT-Battelle* at the *Oak Ridge National Laboratory (ORNL)* in Oak Ridge, Tennessee. The research was proposed by members of the *Department of Civil and Environmental Engineering (CEE)* at *Mississippi State University (MSU)* to *SERRI* in a document dated 1 June 2007. The proposed research was authorized by *UT-Battelle* in its task order dated 10 December 2007. This task order included a scope of work defined through joint discussions between *MSU* and *SERRI*. Work on the project was initiated on 1 January 2008. A modification of Task Order 4000064719 was proposed on 9 September 2008 and agreed upon on 29 September 2008. A second Task Order modification dated 22 June 2010 was also performed, which is the Task Order used to generate this report.

The scope of work associated with Task Order 4000064719 included several related components. The general objectives of the project were to investigate means for rapidly using on-site materials and methods in ways that would most effectively enable local communities to rebuild in the wake of a flooding disaster. Within this general framework, several key work components were associated with Task Order 4000064719. Specifically, the scope of work dated 22 June 2010 includes research efforts in the following six task groups:

Task 1: Erosion Control-Erosion Protection for Earthen Levees.

Task 2: Bridge Stability-Lateral & Uplift Stability of Gravity-Supported Bridge Decks.

Task 3: Levee Breach Repair-Closure of Breaches in Flood Protection Systems.

Task 4: Pavement Characterization and Repair.

Task 5: Emergency Construction Material Development-Staging Platform Construction.

Task 6: Fresh Water Reservoir-Restoration of Fresh Water Supplies.

The division of the research effort allowed the work to be broken into manageable portions so that key components could be reported in separate volumes to allow readers to obtain only the work related to their needs. The work contained herein was associated mostly with Task 6 and to a lesser extent with Task 5. The report of this work was the third deliverable item of the research project, hence the designation of the report as *SERRI Report 70015-003* of Task Order 4000064719. Work related to Task 6 was also submitted in *SERRI Report 70015-002*; these two reports represent full completion of Task 6.

1.2 Objectives

The general objective of Task Order 4000064719 was to investigate several specific means by which local communities may best use available resources in an effort to rapidly

recover from a flooding disaster. In the wake of a flooding disaster, this broad objective would include rebuilding a community with the efforts of a variety of professionals practicing within the physical and social sciences. The research conducted was much more narrowly focused upon certain recovery efforts typically associated with Civil Engineering.

A key component of this research was to develop solutions which may be rapidly deployed to achieve maximum benefit to the community, typically through the use of on-site materials, pre-engineered components, and innovative construction materials and techniques. This research aimed to develop solutions for protecting and/or expeditiously reconstituting critical civil infrastructure components. The research emphasized rapid constructability where existing on-site materials are used to strengthen selected infrastructure components. In this context, the specific objective of the total effort of Task Order 4000064719 was to develop specialty materials and design and construction procedures which may be rapidly deployed to protect and restore selected key civil infrastructure components.

The primary objective of this report is to develop methods to construct stable walls within a flooded area using geotextile and/or geomembrane tubes. The end use of these walls could be far reaching. The primary focus of the walls for this research was to allow construction of a freshwater reservoir. A secondary objective for the walls was construction of the exterior walls of a platform or other emergency structure. To accomplish the objectives, a series of tasks were undertaken. They are listed in the bullets that follow.

- Conduct literature and practice review aimed at evaluation of constructability, stability analysis, material availability, alternate technologies, and similar.
- Utilize information from the *2008 Geotextile Tubes Workshop* (Howard et al. 2009) alongside literature and practice review to further construction of freshwater reservoir and platform walls.
- Select and visit construction and manufacturing sites based on the information from literature review, practice review, and *2008 Geotextile Tubes Workshop*.
- Investigate stability of geotextile tube walls with water on only one side.
- Recommend construction practices based on all information obtained.

1.3 Scope

This report was performed under the revised Statement of Work dated 22 June 2010 and fulfills deliverable i) from Task 5 and fulfills deliverables a), b), c), and e) from Task 6. All deliverables from Task 5 and Task 6 are shown below. Item d) of Task 6 was fully addressed in *SERRI Report 70015-002*, while the remaining items from Task 5 were addressed in reports *70015-006*, *70015-007*, and *70015-008*.

Task 5:

- a) Acquire representative material for testing from locations that would be candidates for flooding (e.g. New Orleans and Mobile). The origin of the material will vary from dredging operations to native soils in these types of areas, and will be used throughout testing. Where applicable in-situ moisture contents will be obtained to provide a baseline of properties. Large quantities of three soils will be obtained with varying plasticity and organic content.
- b) Characterize basic properties of materials. Testing will be performed to measure: 1) Activity (ASTM D 422), 2) Organic Content (ASTM D 2974 or equivalent), 3)

- Atterberg Limits (ASTM D 4318), 4) Specific Gravity (ASTM D 854), 5) USCS Classification (ASTM D 2487), 6) Particle Size Distribution (ASTM D 422), 7) XRF, and 8) pH.
- c) Develop a comprehensive suite of load response properties with time for the soils described in a) using bench scale testing. The testing protocol will consist of shear strength testing of prepared stabilized slurry slabs and unconfined compression testing as appropriate. Very thin membranes will also be tested in conjunction with the materials. Both types of testing will be intended to simulate shear strength of the stabilized slurries with time over a period of seven days. The aforementioned test protocol was selected for two reasons. The slab testing method will be developed in a manner that will be applicable to on site responders, which makes it highly desirable. The stabilization materials to be blended with the candidate soils include: 1) *Type I* portland cement from both the major types of cement plants, 2) *Type III* portland cement from both major types of cement plants, 3) commercially available rapid set cement, 4) six specialty cements produced specifically from this research (four by interrupting normal production at both major types of portland cement plants and two blended calcium sulfoaluminate cements), 5) ground granulated blast furnace slag, and 6) two types of polymer fibers. This materials protocol includes 14 different stabilization additives encompassing a wide variety of properties. Development of the specialty cements will be performed using laboratory testing including semi-adiabatic calorimetry.
 - d) Investigate dewatering equipment and materials for applicability in disaster environments, in particular to assist in development of emergency construction materials with secondary emphasis in handling contaminated sediments. The investigation will focus on the use of polymers for dewatering a soil mass and also investigate geotextile tubes. A test environment will be developed where a series of potentially applicable polymers will be tested (in conjunction with scaled geotextile tubes in some instances as appropriate) to determine if the technology can produce sufficient material at an acceptable moisture content for large scale emergency construction material needs. Moisture content variability conditions will also be investigated in the context of dewatering. The effect of dewatering polymers on shear strength in the presence of multiple cements will also be investigated via slab and unconfined compression techniques.
 - e) Select cementitious materials investigated in the bench scale study c) will be further investigated in a mixing (or blending) study to evaluate effect of key parameters. Examples of key parameters would be cementitious sulfate content and its effect on shear strength and the effect of blending ground granulated blast furnace slag with portland cement in high moisture content fine grained soils.
 - f) Test the behavior of multiple cement blends (selected from the 14 original blends previously mentioned) in the presence of brackish water and seawater. Testing will be performed via slab and unconfined compression techniques. The bench and mixing studies only incorporate fresh (tap) water. A final blend will be selected for each soil type and set of conditions at the conclusion of this subtask considering all knowledge gained from subtasks a) to f).
 - g) Develop design and construction guidance (e.g. identifying suitable applications and providing placement and mixing approach) for using the emergency construction

- material blends developed at the conclusion of subtask f). Use of the material for the purpose of developing a staging platform will be highlighted. Strength and stiffness of the materials developed will be incorporated into the staging platform guidance (e.g. ability of staging platform to support helicopter loads and/or support freight lowered onto platform from a helicopter).
- h) Design and construction procedures using the emergency material will be highly dependent upon the stabilized soil blend achieving a given set of properties with time. For this reason, hand held field shear strength measurement devices will be evaluated statistically for the purpose of assessing risk associated with strength gain measurement over time (precision, accuracy, and repeatability are envisioned to be the focus of the assessment). The results of the hand held gage assessment could be used on site to quantify the impacts of equipment malfunctions, lack of personnel, or other events on the stability of the constructed platform or other structure.
 - i) Test material obtained from construction site visits in unconfined compression to provide a comparison of the properties of the stabilized blends made from materials obtained in subtask a). It is anticipated that test results will be obtained from three to five sites.

Task 6:

- a) Conduct stability analysis of geotextile and/or geomembrane tube configurations to assess the stability of the wall during the critical condition when water is only on one side of the wall.
- b) Identify specific geotextile and/or geomembrane tube properties and construction techniques of value to support the development of stable walls for a fresh water reservoir.
- c) Observe construction sites where valuable techniques are being employed (e.g. use of fine grained soils, construction in water, large scale construction activities).
- d) Organize a roundtable workshop to discuss ideas and experiences related to rapid development of walls using geotextile tubes, as well as dewatering soil with geotextile tubes. The goal of the workshop is to obtain information that can be used to further the research and disseminate the knowledge in an effective manner.
- e) Develop rapid design and construction procedures for geotextile and/or geomembrane tube walls, with the intent of complete constructability of a freshwater reservoir in a minimum amount of time.

There are existing solutions for obtaining water in a disaster environment (e.g. the Red Cross has performed water treatment in many countries for some time). This portion of the overall research does not aim to develop a unique concept. Rather, the scope of the project aims to provide information that adds to the knowledge base of using existing materials, technologies, and methods in a manner that provides additional options to the responders. In order to provide responders as much flexibility as possible, the research team investigated multiple options to allow the materials available, needs of the disaster, materials and equipment available, and similar to dictate the path taken. This approach was taken with all six tasks.

While a fresh water reservoir was the primary objective of the research, the solutions investigated have the potential to be applicable in other facets of recovery from a flooding disaster. Temporary walls have value for multiple construction applications. Examples could include a staging platform, diversion of water to or away from an area of interest, or encapsulating an area to allow water removal and subsequent construction within the encapsulated area.

For purposes of this research, geotextile tubes as they have historically been used can be classified into two categories: 1) dewatering; and 2) marine environments and shoreline protection. These applications vary in their objectives to the extent that different materials are commonly used for each. The work presented in this report is more aligned with the marine environments and shoreline protection category, and the work performed was limited to geotextile tubes pertaining to structural applications. Dewatering applications using geotextile tubes are addressed in *SERRI Report 70015-007*.

A safe water supply is central to survival and recovery of flooded communities. A default value of 2 L is used as the water ingestion per capita per day by the *Environmental Protection Agency* and *World Health Organization* (EPA 2004). When water for cooking, first aid, and sanitary needs are combined with the ingestion requirements, the amount of fresh water required in the community can easily reach 40 L per capita per day. For a community of stranded residents (lasting days to weeks) and aid workers (lasting days to months), the reservoir required to store an adequate fresh water supply would have to be quite large. In addition, distribution of the water may be challenging, so placement of the fresh water supply to optimize community needs would be desirable.

The goal of this report was to develop means of constructing a reservoir with large storage capacity. Ten million liters was selected as an initial targeted capacity. The research assumes the facility will be built in still to slowly moving water but not in flowing water. Highly uniform, fine grained sediment that is not stabilized can pipe through a geotextile tube under dynamic loads such as wave action in some conditions. This could become an important consideration for construction of a water reservoir in moving (especially turbulent) waters, and no efforts were made in this research to quantify or address this issue.

1.4 Incorporation into the *National Response Framework*

The *National Response Framework (NRF)* is a document that guides the United States when conducting all-hazards response (response refers to immediate actions to save lives, protect property and the environment, and meet basic human needs). This framework is entailed in *NRF* (2008), which has complimentary material found in print and online. The *NRF* is a continuation of previous federal level planning documents (e.g. *Federal Response Plan* of 1992), and serves as the state of the art in responding to disaster events. The following paragraphs summarize how the research conducted in this report could be applicable to the *NRF* and in what manner. The tone of the paragraphs assumes the reader is at least casually familiar with the *NRF* and supporting documentation.

The *Stafford Act* is a key piece of legislation regarding disaster response and recovery. Specifically, the *Stafford Act Public Assistance Program* provides disaster assistance to key responding units (e.g. states, local governments). Figure 1.1 was taken from *NRF* (2008) to illustrate the overall disaster funding flowchart that summarizes *Stafford Act* support.

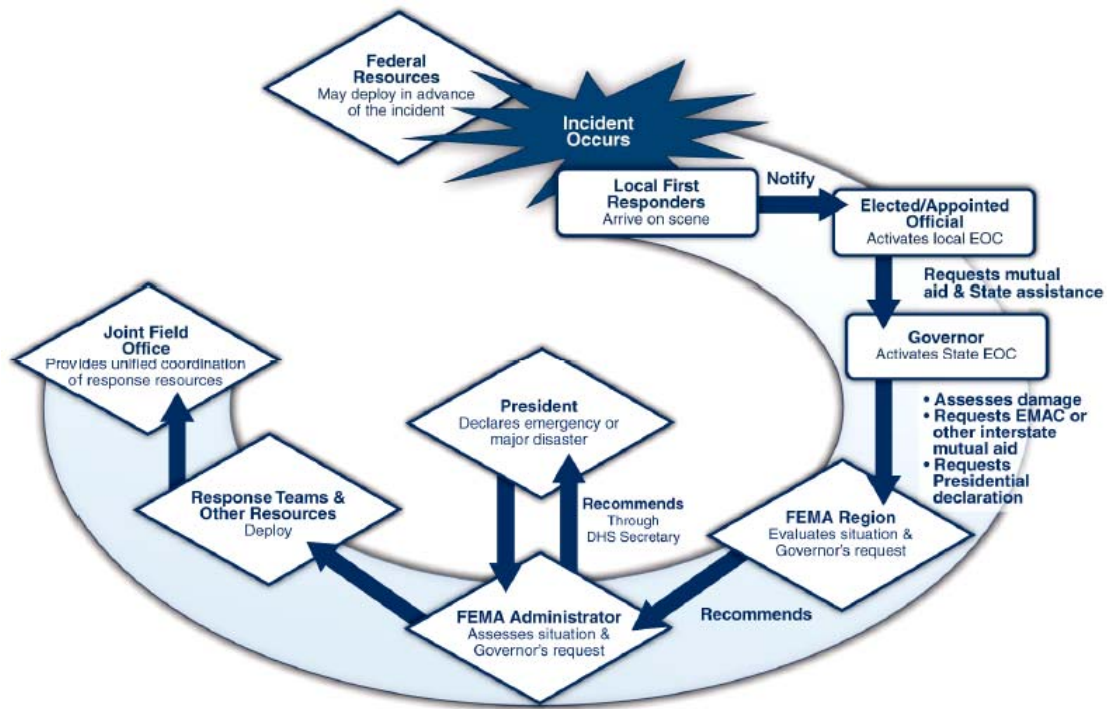


Figure 1.1. Stafford Act Summary From NRF (2008)

According to NRF (2008), “Resilient communities begin with prepared individuals and depend on the leadership and engagement of local government, nongovernmental organizations, and the private sector.” The word “prepared” in the previous sentence is very powerful and could refer to numerous components. The current state of practice in emergency strengthening prior to a water based catastrophe and post emergency construction where the authors feel the United States is not fully “prepared”. To approach a state of readiness where the United States is “prepared” for these types of events, concepts need to be developed that are studied to reasonable resolution where design methods and materials are developed (primarily laboratory scale and analytical studies). These methods and materials then need to be demonstrated at full scale, and thereafter training needs to be performed to ensure construction responders can perform the needed tasks. In present day, this level of preparedness does not exist.

The *NRF* is primarily oriented toward implementing nationwide response policy and operational coordination for any domestic event. *NRF* (2008) focuses on responding to and recovering from incidents that do occur, which is one of four major parts of a larger *National Strategy for Homeland Security*. *NRF* (2008) states that although some risk may be unavoidable, first responders can effectively anticipate and manage risk through proper training and planning. An entire chapter of *NRF* (2008) addresses planning. One of the three principal benefits that is listed for planning is “it contributes to unity of effort by providing a common blueprint for activity in the event of an emergency. Planning is a foundational element of both preparedness and response and thus is an essential homeland security activity.”

Neither training nor planning appears to be performed to any significant extent related to emergency design and construction for the purpose of rapidly strengthening and/or repairing civil infrastructure. Training programs that result in certifications to perform certain activities would expedite selection of qualified groups in the highly time sensitive environment of a disaster. Having known quantities of certified contractors in place would also be valuable during planning exercises. The end products of this report (in particular walls constructed with geotextile or geomembrane tubes) would need to be further developed into full scale demonstrations. Contractors and design firms could then be certified to perform the tasks.

The response structure of NRF (2008) is based on the *National Incident Management System (NIMS)*. Several key concepts are presented in the *NIMS*. First, leaders and staff are said to require initial and ongoing training on response principles. Second, classifying types of resources is said to be essential to ensure effectiveness. During a crisis it is stated that there will not be time to determine staff qualifications, therefore all stakeholders should regularly exercise incident management and response capabilities. A system similar to this for emergency construction activities incorporating geotextile and geomembrane tubes could prove useful.

The goals of the research conducted under Task Order 4000064719 align with the needs of the *Hurricane Liaison Team (HLT)*, whose goal is to enhance hurricane disaster response. Response was stated earlier to refer to immediate actions to save lives, protect property and the environment, and meet basic human needs. Task 6 is directly aligned with the stated mission of the *HLT*. All the aforementioned discussion also aligns with *Scenario 10: National Disaster-Major Hurricane* of the National Planning Scenarios that have been established in NRF (2008).

Response at the local level is organized within an *Incident Command System (ICS)*. At the field level local responders use the *ICS*, which is led by an Incident Commander who has overall authority and responsibility at the incident site. An *Emergency Operations Center (EOC)* is a physical location established at the incident site. The *EOC* can be organized by discipline (e.g. transportation), jurisdiction (e.g. city), Emergency Support Function (e.g. engineering), or a combination. A key *EOC* function is to ensure on scene responders have needed resources. The design recommendations and construction methods produced from this study will identify needed resources by the Incident Commander.

Repeatedly, preparedness is stated (directly or indirectly) as an essential precursor to response. The *RESPONSE ACTIONS* chapter of NRF (2008) shows a preparedness cycle consisting of the following four categories: 1) plan; 2) organize, train, and equip; 3) exercise; and 4) evaluate and improve. The organize category consists of assembling well-qualified teams of paid and volunteer staff for essential response and recovery tasks. Also under the organize category is discussion of *Pre-Scripted Mission Assignments*. They are used to assist in planning for and reduction in time necessary to deploy resources that can be tailored for training, development, and to exercise rosters of deployable resources. These assignments would need to be developed for effective use of geotextile and geomembrane tubes.

Advanced Readiness Contracting is used to ensure contracts are in place before an incident for often needed commodities (a list is provided that does not include construction materials). Geosynthetics, fabricated geotextile tubes, specialty cements, and dewatering polymers are construction materials that would need to be included in *Advanced Readiness Contracting*. This could be an essential step for some commodities (e.g. dewatering

polymers). For Task 6, geosynthetics and fabricated geotextile tubes would be of highest priority.

Under the *RESPOND* heading of the *RESPONSE ACTIONS* chapter of NRF (2008), the response process is divided into three categories: 1) gain and maintain situational awareness; 2) activate and deploy resources and capabilities; 3) coordinate response actions. Providing the right information at the right time is critical to gaining and maintaining situational awareness. With regard to activating and deploying resources, the text in the following paragraph is included in NRF (2008).

“Identifying needs and pre-positioning resources. When planning for heightened threats or in anticipation of large-scale incidents, local or tribal jurisdictions, states, or the Federal Government should anticipate resources and capabilities that may be needed. Based on asset availability, resources should be pre-positioned and resource teams and other support resources may be placed on alert or deployed to a staging area. As noted above, mobilization and deployment will be most effective when supported by planning that includes pre-scripted mission assignments, advance readiness contracting, and staged resources.” This level of detail would be appropriate for the methods investigated in this report, but currently they are not in place. Pre-positioning of geotextile tubes could be needed based on availability and production data discussed later in this report.

As stated in NRF (2008), the emphasis on response will gradually transition to an emphasis on recovery. Short-term recovery is defined as immediate, it overlaps with response, and it lasts up to a few weeks. Long-term recovery is beyond the scope of NRF (2008). Long-term recovery can last for months to years, and includes some of the actions involved in short-term recovery. Quoting NRF (2008): “In the short term, recovery is an extension of the response phase in which basic services and functions are restored. In the long term, recovery is a restoration of both the personal lives of individuals and the livelihood of the community.” The majority of this research is on short term recovery. A water reservoir would be useful for long term recovery but is a short term project.

Fifteen Emergency Support Functions (ESF’s) have been established under *FEMA* coordination. Of the fifteen ESF’s, *ESF #3-Public Works and Engineering* is applicable to this report. This research effort is primarily applicable to Regions IV (Atlanta headquarters) and VI (Denton headquarters) of the ten *FEMA* regions. *ESF #3* includes: 1) conducting pre-incident and post-incident public works and infrastructure assessments; 2) providing technical and engineering expertise including repair of damaged public infrastructure; 3) construction management; and 4) other scenarios outside the scope of this research.

State, tribal, and local governments are responsible for their own public works and infrastructures. Private sector entities, though, either own or operate a significant portion of the nation’s infrastructure and must be included in response and recovery. *DHS/FEMA* are the leads for providing ESF #3 recovery resources, which includes assistance under the *Stafford Act Public Assistance Program*. The US Army Corps of Engineers (*USACE*) and Department of Defense (*DOD*) are ESF #3 coordinators, and are the primary agencies for response. Response and short term recovery overlap in very early stages, thereafter recovery becomes an extension of response.

There are several support agencies identified within ESF #3 tasked with functions applicable to the current research are discussed as follows. **“Unified Coordination Group:** For a flooding event or other incident where *DOD/USACE* has jurisdictional authority and/or responsibilities for directing or managing major aspects of the response. The *DOD/USACE*

may be requested to provide a senior official to participate in the Unified Coordination Group.” The Unified Coordination Group is field level support for ESF #3. Activities within ESF #3 include but are not limited to: 1) coordination and support of infrastructure risk and vulnerability assessments; 2) participation in pre-incident activities such as assessment team positioning and deploying advance support elements; 3) participation in post-incident assessments of infrastructure; and 4) execution of emergency contracting support for life-saving services that include providing potable drinking water. Providing potable drinking water is a key thrust of Task 6 of the current research.

Responsibility to respond to natural events (e.g. hurricane) is initiated at the local level, particularly with elected officials. Key responsibilities of these officials include: 1) establishing strong working relationships with vital public and private sector entities; 2) training with local partners in advance of an incident; 3) leading and encouraging local leaders to focus on preparedness by participating in planning, training, and exercises. With regard to coordinating response actions, catastrophic events with little to no notice are a precedent for state and federal governments to take proactive measures to mobilize and deploy assets in anticipation of formal requests for assistance. During this period, manufacture of geosynthetics, fabrication of geotextile tubes, and mobilization of construction equipment could be performed.

As stated in NRF (2008), government works with private sector groups as partners in emergency management; examples include businesses involved in transportation and civil infrastructure. A thrust is to clearly define the role of private sector in response to disasters, alongside the effect of the disaster on the private sector and government. *Critical Infrastructure and Key Resources (CIKR)* are grouped into 17 sections that provide essential functions and services. Task 6 involved private sector members.

1.5 Alternate Methods of Fresh Water in a Disaster Area

As mentioned in the *Scope* (Section 1.3), this report is not intended to develop a unique concept for providing fresh water in a disaster area. Some existing methods of providing potable water are discussed in this section. Alternatives other than those presented in this report may exist.

Fabric bladders have been utilized as storage containers of potable water. According to Koerner (1998) super bags were available at the time of publication that were available up to $5(10^6)$ liters, and that larger sizes were available by special order. A 5MT nitrile rubber bag was said to weigh 55 kN and Koerner (1998) shows an example photo of a container filled with air that was manufactured by *Firestone Coated Fabrics Co* and *Fabritank Co*. Attempts to locate such a device were unsuccessful; a number of phone calls initiating with the aforementioned company did not locate such a product. A bladder on the order of $0.43(10^6)$ liters was available from *Western Environmental Liner Co-A Division of Western Ag Enterprises, Inc*.

Smaller containers with potential applicability were found on a frequent basis. *Husky Model BT-1000/XR-3PW* is a $3.8(10^3)$ liter bladder that is commercially available. *Amfuel Fabritank* produces truck mounted liquid handling membranes up to $24(10^3)$ liters. *Army Technologies* can supply an $18(10^3)$ liter tank (model NSN: 8145-12-346-8311).

Water Structures Unlimited® markets 38 to $56(10^3)$ liter water storage containers that when packaged can fit into a passenger truck. The same company also manufactures water

filled tubes that could be used for water storage. Tubes less than 0.30 m high when inflated could be used for water supply in large structures (e.g. sports arena housing stranded people). These tubes could be placed underneath the rows of seating and provide water via hydraulic pressure. Four sizes were available as of Jan 2008 with heights up to 0.30 m (storage capacity of 149 liters per meter) and at a cost \$11.48 per meter.

Water Structures Unlimited[®] also produces larger geomembrane tubes (*AquaDam*[®] units). While these containers are not recommended by the manufacturer to contain drinking water (largely because after manufacture and handling they are not tested based on acceptable methods), they could be adapted for drinking water storage with relative ease. They are made from material similar to that in household plastics. The primary use of *AquaDam*[®] units as it pertains to this research, though, are for water reservoir walls that will be discussed later in this report.

1.6 Overview and Applications of Geotextile and Geomembrane Tubes

Geotextile tubes, geotextile bags, geotextile containers, and geomembrane tubes are a family of products used in a variety of structural (marine) applications. The general goal is to contain a material (e.g. sand, slurry, water) within a thin membrane for the purpose of developing a resistance to applied loads. Trademarked names for the aforementioned family of products include *Aqua Barrier*[™], *AquaDam*[®], *Geotube*[®], *Geobag*[®], *Geocontainer*[®], *MacTube*[™], *SiltTex Geotextile Tubes*[™], *SPIRAL*[®], *TitanTubes*[®], *WaterStructures*[®], and *WIPP*[™]. Geotextile and geomembrane tubes are of primary interest to this research, but information related to geotextile bags and geotextile containers is presented in this report as it pertains to the project and provides valuable information.

Geotextile tubes are manufactured by sewing together multiple sheets of high-strength polyester or polypropylene fabrics (typically woven fabrics) to create an enclosed tube. Geotextile tubes may be filled with untreated soils, stabilized soils, or even concrete. As an example, high slump concrete was pumped into *Geotube*[®] units at the Chivor Dam in Columbia. *Geotube*[®] units are specialty geotextile tubes and are a registered trademark of *TenCate*[™]. *Geotube*[®] technology was developed in the early 1990's through partnership of *TenCate*[™] and the *US Army Corp of Engineers (USACE)*. *Geotube*[®] units are one of the primary materials evaluated in this research.

Geomembrane tubes are manufactured by placing two independent watertight membrane tubes inside a larger tube (or sleeve) that is typically made from a woven geotextile. Geomembrane tubes are filled with water. *AquaDam*[®] units are a specialty geomembrane tube and are a registered trademark of *Water Structures Unlimited*[®]. They are a rapidly deployable barrier when filled with water that has been successfully used for multiple applications. *AquaDam*[®] units are one of the primary products evaluated in this research.

Geotextile containers are designed to be filled while outside the water, sewn/tied shut, and subsequently dropped into position from a split bottom barge. Geotextile bags are similar in concept to conventional sand bags, though they are typically larger. These two materials are only considered in this research in the context of literature and practice review and were not considered when developing water reservoir designs.

The remainder of this section summarizes applications of geotextile and geomembrane tubes with respect to their potential utility. Many details of geotextile tube

applications are subsequently presented in the portion of this report dedicated to site visits. In turn, they are not as well represented in this section as are geomembrane tubes. Multiple cases of geomembrane tube use are presented to provide context and demonstrate their applicability, which is also one of the key purposes of the geotextile tube site visits presented in Chapter 3. Multiple photos are presented in the remainder of this section which were provided by material suppliers from their personal collections.

Unique fabric applications are discussed by Koerner and Welsh (1980). Examples include: 1) underpinning of scoured bridge piers by pumping cement grout into the tubes; 2) protection of pipelines by using divers to place tubes 9.1 m underwater, stationing them with cables, and inserting a sand grout at a water to cement (w/c) ratio of 0.67; 3) shoring up abandoned mines and other cavities by wrapping a fabric around the grout pipe, injecting grout into the fabric that acts as formwork, and removing the pipe without having to enter the mine; and 4) erosion prevention applications where concrete filled fabric tubes are placed along slopes.

The first geotextile tube that would later be developed into a *Geotube*[®] unit was installed in 1962 and as of 2008 over 240 km of shoreline have been protected using this product. Geotextile tubes gained prominence in the early 1990's and have proven effective for shoreline erosion control (e.g. Gibeaut et al. 2003) and in numerous environmental applications (e.g. Fowler et al. 2007). Additionally, geotextile tubes have been used to solve several difficult engineering problems: e.g. dike construction in wetlands, underwater stability berms, and island construction (Fowler and Sprague 1993). Furthermore, geotextile tubes have been used in a variety of structural applications including: 1) dikes or breakwaters for the prevention of beach erosion and the protection of coastal infrastructure; 2) protection of slopes, bridge piers (scour protection), tunnels, and under water pipelines; 3) cores of sand dunes, rip rap breakwaters, and rip rap jetties; 4) underwater structures; 5) diversion dikes; and 6) dredge material containment walls. Marlin (2002) describes use of geotextile tubes filled with silt and/or peat using multiple types of equipment including hydraulic dredges. Two projects mentioned specifically were: 1) Drake's Creek Project in the USACE Nashville District (this project was also discussed in the 2008 *Geotextile Tubes Workshop* (Howard et al. 2009); and 2) a restored island in northern Illinois by the Fox Waterway Agency.

Figure 1.2 shows a project in Israel incorporating *SPIRAL*[®] geotextile tube units where conveyor belt armoring was used in a shoreline protection application. The armoring was used as protection from high wave action. Figure 1.3 shows a portion of the *SPIRAL*[®] geotextile tube units used to protect the shoreline of the Galveston Ship Channel. On the order of 4,200 m of geotextile tube were required for this project. Figure 1.4 shows *SPIRAL*[®] geotextile tube units blocking a waterway in California while the water below the tube is simultaneously being pumped out. Figure 1.5 shows a construction project at a naval base in Norfolk, VA where *SPIRAL*[®] geotextile tube units were designed to support a 4.57 m water depth after the tubes were in place and the interior water pumped from an area where construction was to be performed. *Water Structures Unlimited*[®] was involved in the construction at Norfolk, VA, and purchased the *SPIRAL*[®] geotextile tube units to allow the 4.57 m height to be achieved.



Figure 1.2. Bank Protection in Israel using Conveyor Belt Armor Cover



Figure 1.3. Protection of Galveston Ship Channel



Figure 1.4. Temporary Blocking of Waterway



Figure 1.5. SPIRAL® Geotextile Tube Units Restraining Atlantic Coast Waters

Alternatives to sandbags began to emerge in the late 1980's in the form of *Water Structures*® (Landis 2000). Since that time applications of geomembrane tubes have included submerged sites, levee extensions, canal projects, levee toppings, cofferdams, river diversions, water removal for construction sites, building protection during floods, and water storage. Landis (2000) provides several case studies documenting superior performance of *WaterStructures*® (initial trade name of *AquaDam*® units) relative to traditional methods of flood protection such as sandbags.

As discussed by Landis (2000), a single tube filled with water could not support water, but frictional forces in the outer geotextile allow water to be supported with two interior water tubes. In the year 2000, *Water Structures*® were available from 0.3 to 5.5 m high in lengths of 15.2, 30.5, and 61.0 m. Recommendations were to keep water at or below 75% of the tube height. The tubes were reported to be compact enough for handling by laborers by Landis (2000). The research team coordinated with the owner and sole proprietor of *Water Structures Unlimited*® regarding the potential use of *AquaDam*® units to meet disaster recovery needs. Three applicable US patents are held by the company owner who was enthusiastic about the work and was very interested in assisting the research team.

A variety of applications have made use of *AquaDam*® units. Many examples are provided in *AquaDam Applications* (2002), which is a document compiled by the manufacturers. Select case studies and applications have been provided in the paragraphs and figures that follow, many of which can be found in the aforementioned reference.

Figure 1.6 shows application in flood control in Clear Lake, CA using 0.91 to 1.22 m dams. Figure 1.6a is protection of a residential dwelling, while Figure 1.6b shows protection of a hotel. Figure 1.7 shows 4.88 m *AquaDam*® units used to provide a barrier for pumping an area to allow boat ramp construction and repair in Norfolk, VA. Note the unit was placed in a non-straight configuration with relative ease, and also note this is the same project shown in Figure 1.5. The project combined geotextile and geomembrane tubes and was thus used as an example of both applications. Figure 1.8 shows a 1.83 m dam backed with a 1.22 m dam used to remove water for boat ramp repair on Lake Erie in Ohio. Figure 1.9 shows a 2.44 m *AquaDam*® unit placed into a tight circle to isolate a work area for pier construction in Philadelphia, PA.



(a) Residential Dwelling Protection



(b) Hotel Protection

Figure 1.6. AquaDam® Units Used for Flood Control



(a) View of Dam and Dewatered Area



(b) Working Conditions

Figure 1.7. AquaDam® Units Used for Boat Ramp Construction and Repair

Figure 1.10 shows a 3.05 m AquaDam® unit stopping water flow in a canal to allow construction of a bridge pier. Figure 1.11 shows a 1.83 m dam backed with a 1.22 m dam to allow repair of a section of a pond liner in Kingman, AZ. A 0.46 m AquaDam® unit was used to isolate and collect seepage to ensure adequate working conditions to repair the liner.



Figure 1.8. Backed AquaDam® Units



Figure 1.9. AquaDam® Units Used for Pier Construction



Figure 1.10. AquaDam® Units Used for Bridge Pier Construction



Figure 1.11. AquaDam® Units Used for Pond Liner Repair

A site for construction of a shoreline amphitheatre in Foster City, CA is shown in Figure 1.12. A 2.44 m unit was used for a 1.83 m water depth. A 1.52 m unit was placed behind the primary unit to seal and as secondary support. *Water Structures Unlimited®* teamed with the *Bureau of Reclamation* on the Umatilla River in Echo, OR to repair a submerged weir and install a fish ladder. To complete the project, the entire flow was diverted to one side of the river. *AquaDam®* units (0.91 and 2.44 m) were used during the October 2008 construction. A demonstration video was produced and is available from *Water Structures Unlimited®*. Figure 1.13 shows photographs of the project.



(a) Site Prior to Construction



(b) First AquaDam[®] Unit Placed



(c) Second AquaDam[®] Unit Placed



(d) Water Pumped From Site

Figure 1.12. Amphitheatre Construction in Foster City, CA



(a) Diverting Umatilla River



(b) Flow Diverted to One Side of the Weir

Figure 1.13. Diverting Flow of Umatilla River in Echo, OR

CHAPTER 2 - FABRICATION AND AVAILABILITY OF GEOTEXTILE AND GEOMEMBRANE TUBES

2.1 Geotextile Tubes

Geotextile tube availability was a concern that was addressed by contacting manufacturers. The *Industrial Fabrics Association International (IFAI) Specifier's Guide* (2009) was used as the initial reference for companies manufacturing geotextile tubes. All companies listed under the headings: 1) *Geotextile Tubes*; 2) *Fabricator, Geotextile Tubes*; and 3) *Dewatering* were contacted by phone and/or email. A small number of additional companies, found through communication with other manufacturers, were also contacted. In total, fourteen companies were contacted. Through this effort the industries capability to respond in the context of a disaster was determined.

A standard set of questions was asked of all fabricators, and the companies contacted were separated into two groups: fabricators and distributors. Five fabricators were identified by the research team between March to June of 2009. The five fabricators identified were: 1) *Bradley Industrial Textiles*; 2) *Flint Industries Inc*; 3) *Geo-Synthetics Inc*; 4) *Industrial Fabrics*; and 5) *TenCate™*. Some additional companies indicated that they possessed the capabilities to produce geotextile tubes. These groups were not included in the current work since only those companies that routinely manufacture geotextile tubes were deemed appropriate for disaster assessment purposes. The remainder of this section provides the information obtained from the companies during the aforementioned time period.

2.1.1 Fabrication of Geotextile Tubes

Four categories of geotextile tubes were identified as defined by fabrication methods: 1) longitudinal seamed, 2) circumferential seamed, 3) helical rib supported, and 4) uni-axial. Smaller circumference tubes (e.g. 10.4 m) are typically manufactured using longitudinal seaming, while larger circumference tubes (in excess of 13.7 m) are typically manufactured using circumferential seaming. To fabricate a geotextile tube using longitudinal seaming, two geotextile sections each slightly wider than half the desired circumference and equal to the desired length are placed on top of one another. A seam is sewn along both sides and along the end of the geotextiles to produce the tube. Members of the research team visited *TenCate™* in the summer of 2008, and Figure 2.1 shows photos of a longitudinal seamed tube being manufactured. Once manufactured the tube is rolled onto a pipe and wrapped with a protective covering. Figure 2.2 is a schematic of a typical longitudinal seamed tube (10.4 m circumference) manufactured by *TenCate™*.

To fabricate a geotextile tube using circumferential seaming, multiple geotextile sections are sewn together. The width of the geotextile is placed in the longitudinal direction of the tube and the length of the geotextile is cut slightly longer than the desired circumference of the tube. The width of geotextiles used is typically on the order of 4.57 m. Figures 2.3 and 2.4 are schematics of typical 30.5 m and 61.0 m long circumferential seamed tubes, respectively, that are 13.7 m circumference and manufactured by *TenCate™*. The only longitudinal seam created in the process is along the bottom of the tube, and all ports are placed 180 degrees from this seam on top of the tube.



(a) Geotextiles Prior to Fabrication



(b) Sewing of Initial Seams



(c) Inserting Fill Ports



(d) Completed Fill Port



(e) Handling Straps



(f) Sewing of Final Seams



(g) Rolling Geotextile Tube Onto Pipe



(h) Final Product Ready for Shipment

Figure 2.1. TenCate™ Geotextile Tube Manufacturing Photos, Commerce, GA

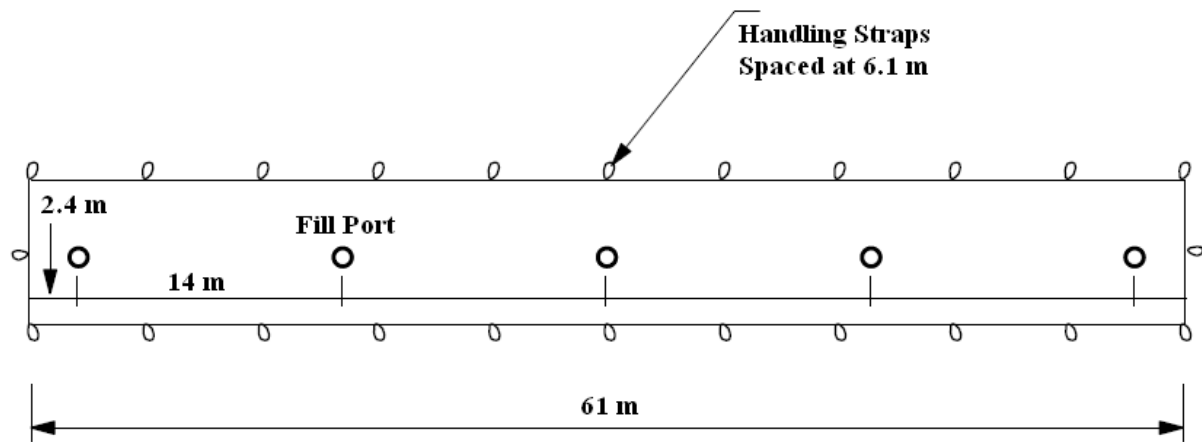


Figure 2.2. Schematic of 10.4 m Circumference Longitudinally Seamed Geotextile Tube

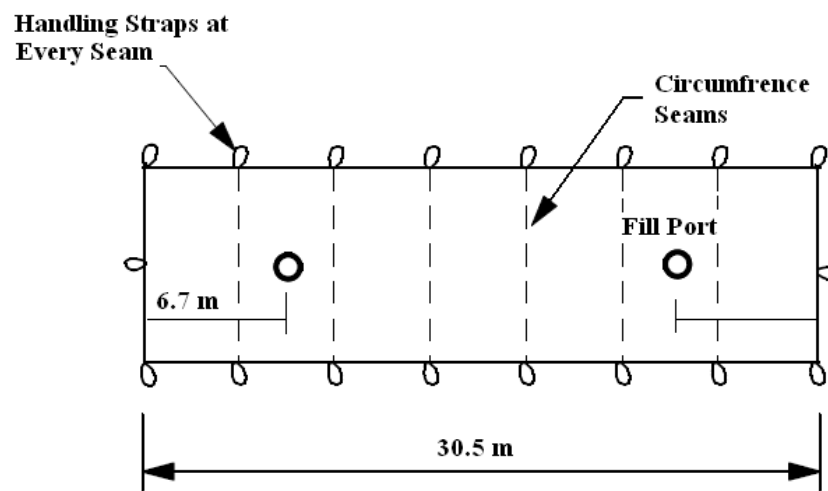


Figure 2.3. Schematic of 30.5 m Long Geotextile Tube Circumferentially Seamed

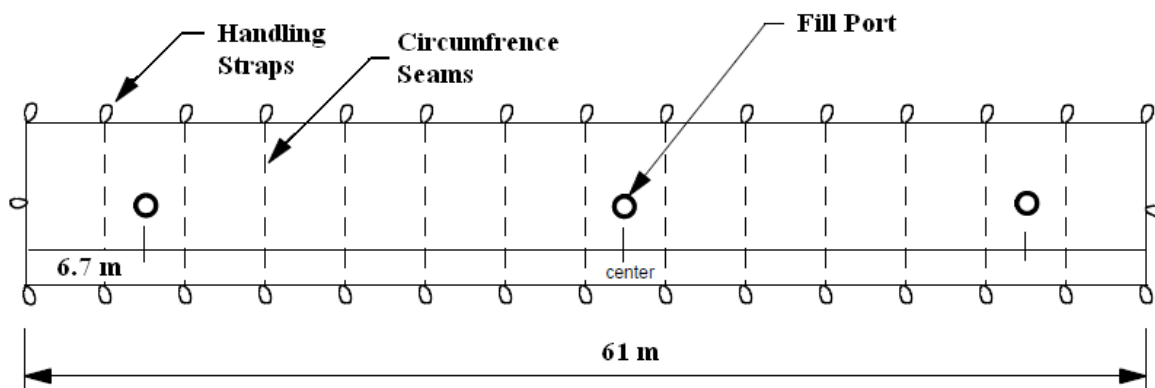


Figure 2.4. Schematic of 61.0 m Long Geotextile Tube Circumferentially Seamed

Helical rib supported geotextile tubes are manufactured by *Bradley Industrial Textiles*. Figure 2.5 is a view of the inside of one of the tubes when inflated using pressurized air. The manufacturer states that these ribs have four times the strength of the outside shell of the fabric, which allow essentially any fill material (e.g. silt, clay, organic matter) to be used.

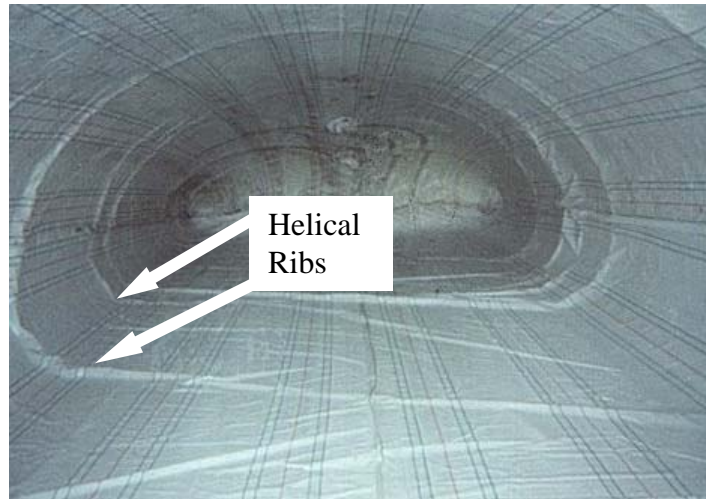


Figure 2.5. Helical Ribs Inside SPIRAL® Geotextile Tube

Flint Industries manufactures geotextile tubes that are described as uni-axial. The concept of the uni-axial tube according to the manufacturer is to keep seams in lower stressed areas. Key seams are perpendicular to the circumferential closure seam along the bottom of the tube. The concept of the seaming method is to allow production of a geotextile tube with the same strength as other methods using a lower strength fabric.

2.1.2 Availability of Geotextile Tubes

Availability of geotextile tubes in the wake of a disaster was significant to the efforts of the research team since it must be considered when designing solutions that incorporate the technology. Five geotextile tube manufacturing facilities are located in four states, but are concentrated in the southeastern US. Locations are: 1) *Bradley Industrial Textiles* in Valparaiso, FL; 2) *Flint Industries Inc* in Metter, GA; 3) *Geo-Synthetics Inc* in Waukesha, WI; 4) *Industrial Fabrics* in Baton Rouge, LA; and 5) *TenCate™* in Commerce, GA.

All US fabricators provided maximum production capacity of 13.72 m geotextile tubes with their current capabilities provided this was the only type of tube being produced at the time. According to *TenCate™* representatives, the circumference of the geotextile tube does not drastically affect production capacity, though production capacity does decrease somewhat as the circumference increases. For this reason, a circumference within the middle of the range of conventionally manufactured tubes was selected for analysis of the manufacturing times and costs.

To protect commercial interests of all fabricators, production capacity information was reported in order of highest capacity to lowest capacity. The information provided was reported in terms of an 8 hour production cycle with company capabilities provided to the

authors during the months of March to June of 2009. Production capacities of geotextile tubes, or any manufactured product, are subject to change due to a variety of factors. The capacities of length of tube per 8 hour production cycle were: 1) 200 to 240 m; 2) 180 to 240 m; 3) 120 to 240 m (stated to be a function of the type of tube manufactured, quantity desired, and notice provided); 4) 180 m (plus or minus depending on multiple parameters); and 5) 120 to 150 m.

Flint Industries indicated geotextile tube production was a peaks and valleys process, and that a single line of production could produce on the order of 61 m of geotextile tube in an 8 hour day. They also noted that capacity could be increased relatively quickly by: adding laborers with moderate skill levels that would be managed by full time and fully trained personnel; renting warehouses; and other activities that do not require significant time. Along these lines, *Industrial Fabrics* indicated they have another warehouse that could be temporarily used as a manufacturing facility. All five companies indicated they would do everything within their ability to assist in disaster recovery, while also being mindful of the needs of their current customers. It was noted that in the event of a disaster, most construction projects would be willing to have their shipments sent to the disaster area.

Availability of geotextile to fabricate the tubes was mentioned more than once as a concern for maximum production over a sustained period. It is the understanding of the authors that most polyester geotextile used in fabrication of geotextile tubes within the US is imported from outside the US while polypropylene geotextile for geotextile tube fabrication is almost entirely manufactured within the US. As of 2009, *TenCate*TM was the world's leading provider of woven and non-woven geotextiles. They are also the only company manufacturing both geotextiles and geotextile tubes. Investigation of geotextile supply patterns to address this concern was beyond the scope of this research. For a sustained need of maximum production of geotextile tubes, this matter should be investigated.

Flint Industries estimated that 50 to 75 geotextile tubes 30.5 m long and 13.72 m circumference would be a reasonable assessment of inventory from all US fabricators, in addition to any tubes in production that have already been allocated to a project. They estimated their typical inventory on the order of 20, 30, and 15 geotextile tubes 30.5 m long that are 9.14, 13.72, and 18.29 m circumference, respectively. Note this is not a guaranteed quantity by any means, rather a typical value based on yearly capacities and experience.

*TenCate*TM provided a snapshot of their inventory in March of 2009, which consisted of 400 m of 9.14 m circumference, 1,500 m of 13.72 m circumference, and 2,000 m of 18.29 m circumference *Geotube*[®] units, all of which incorporated *GT 500* geotextile. March is typically the start of the construction season. The company noted that this is a large stock and that quantities stocked vary throughout a year as well as from day to day.

Geo-Synthetics estimated their average daily inventory of 9.14 m circumference tubes at 150 to 225 m and 13.72 m circumference tubes at 300 to 450 m, respectively. *Bradley Industrial Textiles* and *Industrial Fabrics* indicated little inventoried material. Geotextile tubes that have been pre-positioned and allocated for disaster recovery was mentioned as the only guarantee sufficient tubes could be made available in the event of a disaster.

2.1.3 Geotextile Tube Properties

This section discusses geotextile tube properties but omits design height and fill volume as they are left for the next chapter pertaining to analysis and design of geotextile

tubes. Standard lengths of geotextile tubes are 30.5 and 61.0 m, though other lengths can be obtained. The types of geotextiles used for manufacture of typical geotextile tubes are less than 4 mm thick, classifying them as a thin membrane.

Geotextile tube circumferences were found to be available from 4.57 m to 36.58 m, though both the smaller and larger tubes did not appear readily available. Circumferences of 9.14 m and 13.72 m were readily available and are the primary focus of this investigation; 18.29 m circumferences are also readily available and are a secondary focus of this investigation. All fabricators produced these three circumferences on a routine basis. Shoreline products were found to be on the order of 9.14 m circumference on a frequent basis, with a fair amount of 13.72 m circumferences used in these and similar applications. Other sizes appear to be less common.

For all materials investigated, durability in terms of sunlight exposure is of no major concern for the short durations of interest; Koerner et al. (2008) provides ultraviolet light durability data of many geosynthetics. Commercial products can withstand hundreds of hours of UV exposure (e.g. *TenCate™ GT* series). All five geotextile tube fabricators used mostly polypropylene geotextiles. Estimates of 90% use of polypropylene and 10% use of polyester were common. Tables 2.1 and 2.2 provide properties of geotextiles routinely used to manufacture geotextile tubes. As noted by Leshchinsky et al. (1996), geotextiles are commonly manufactured as an anisotropic material. Their strength in the machine (warp) direction is different than the cross (fill) direction. The geotextiles shown in Table 2.1 are isotropic, while those shown in Table 2.2 are anisotropic.

Table 2.1. Geotextiles Used in Fabrication of Geotextile Tubes (Table 1 of 2)

	Product Name	<i>GT1000M^a</i>	<i>GT1000MPET</i>
	Producer	<i>TenCate™</i>	<i>TenCate™</i>
Specified Geotextile Property	ASTM Method	Value	Value
Material	---	Polypropylene	Polyester
Biological Degradation	---	Inert	Inert
Chemical, Alkali, and Acids	---	Resistant	Resistant
Wide Width Tensile Strength-MD (kN/m) ^b	D 4595	175	175
Wide Width Tensile Strength-CD (kN/m) ^c	D 4595	175	175
Wide Width Tensile Elongation-MD (%) ^d	D 4595	20	15
Wide Width Tensile Elongation-CD (%) ^d	D 4595	20	15
Factory Seam Strength (kN/m)	D 4884	87.6	88.0
Apparent Opening Size-AOS (mm/US)	D 4751	0.60/30	0.15/100
Water Flow Rate (L/min/m ²)	D 4491	813	240
Mass per Unit Area (g/m ²)	D 5261	1117	813
UV Resistance at 500 hr (% Strength Ret.)	D 4355	85	65

Note: Properties shown are Minimum Average Roll Values (MARV).

a) Equivalent to GC1000, PP200S, and GC1000M.

b) Properties in Machine Direction.

c) Properties in Cross Direction.

d) Maximum Value.

Flint Industries focuses on operational strength, which they define as the minimum strength anywhere on the geotextile. They do not support reporting of geotextile properties, rather focusing on performance properties as manufactured. As a result, the properties of the geotextiles used to manufacture their geotextile tubes have not been included in this report.

Table 2.2. Geotextiles Used in Fabrication of Geotextile Tubes (Table 2 of 2)

Property	Product Name	GT 500	GEOTEX [®] 4x6
	Producer	TenCate [™]	Propex [®]
	ASTM Method	Value	Value
Material	---	Polypropylene	Polypropylene
Biological Degradation	---	Inert	Inert
Chemical, Alkali, and Acids	---	Resistant	Resistant
Wide Width Tensile Strength-MD (kN/m) ^a	D 4595	70	70
Wide Width Tensile Strength-CD (kN/m) ^b	D 4595	96	105
Wide Width Tensile Elongation-MD (%) ^c	D 4595	20	14
Wide Width Tensile Elongation-CD (%) ^c	D 4595	20	9
Factory Seam Strength (kN/m)	D 4884	70	70
Apparent Opening Size-AOS (mm/US)	D 4751	0.43/40	0.43/40
Water Flow Rate (L/min/m ²)	D 4491	813	810
Mass per Unit Area (g/m ²)	D 5261	585	525
UV Resistance at 500 hr (% Strength Ret.)	D 4355	80	80

Note: Properties shown are Minimum Average Roll Values (MARV).

a) Properties in Machine Direction

b) Properties in Cross Direction

c) Maximum Value

Fill ports are a key geotextile tube feature. The aforementioned discussion related to availability of geotextile tubes (in particular inventories discussed in Section 2.1.2) should be considered in conjunction with the tube port spacing. Industrial (dewatering) tubes, in general, have wide port spacing's where coastal (shoreline) tubes have shorter port spacing's (e.g. ± 10 m). Inventories will typically include both dewatering and shoreline products so the full inventory would not likely be available for any given application.

Fill ports are placed along the length of the tube to allow simultaneous filling within multiple ports (if desired), to force water out of one port by pressurizing another, and to provide pressure relief. Inlet sleeves of 305 to 457 mm diameter and 914 to 1524 mm length are typical on fill ports. Filling ports also typically consist of a robust design. An example of a fill port design would be 38 mm thick inner and outer flange rings that encases the geotextile surface between rubber gasket material 3.2 mm thick and the entire assembly secured with 19 mm diameter bolts.

2.1.4 Economic Data for Geotextile Tubes

All US fabricators were queried regarding pricing information of 13.72 m geotextile tubes (excluding freight). Two of the fabricators did not have published price lists. To protect commercial interests of the three fabricators who provided pricing information, the information has been provided as a range of values, and it was obtained during the months of March to June of 2009. Geotextile tube pricing varies and is subject to change due to a variety of factors. The pricing information obtained was: 1) \$62 to \$98 per meter for 9.14 m circumference; 2) \$85 to \$154 per meter for 13.72 m circumference; and 3) \$148 to \$213 per meter for 18.29 m circumference. These prices do not include any specialty features such as armored coatings, project specific packaging, non-standard ports or port spacing, and similar.

Representatives from the Galveston District of the USACE estimated geotextile tube projects cost between \$164 to \$492 per meter depending on multiple parameters in a workshop discussed in Davis and Landin (1997) that is discussed in more detail in subsequent chapters. Adjusting these values for inflation from the summer of 1995 to the

summer of 2009 assuming an annually compounded rate of 3% resulted in adjusted costs of \$248 to \$744 per meter. Long tubes constructed in accessible areas with active dredging were the least expensive and as any of these parameters change the cost was reported to increase. A cost of \$245 to \$825 per meter was given by a participant in the workshop held in 2008 and discussed in Howard et al. (2009) for traditional applications but this cost was noted to vary substantially for non-conventional applications.

Chapter 3 discusses site visits conducted by the research team related to geotextile tubes. Details are left for those chapters with geotextile tube cost information repeated in this section. At the *Matagorda Ship Channel*, the total cost of the 9.14 m circumference geotextile tubes, anchor tubes, scour apron, and shroud were \$261 per m including freight. The total project cost was \$1,956 per m; the project was conducted in water. At *Grand Isle*, the total cost of the 9.14 m circumference polyurea coated (form of armoring) geotextile tubes, anchor tubes, and scour apron were \$460 per m including freight. The total project cost was \$2,850 per m; the project was conducted on the beach and included significant items not directly associated with placement of geotextile tubes. At *Peoria Island*, the total cost of the 13.72 m circumference geotextile tubes were \$65 per m to \$80 per m including freight. The total project cost was \$537 per m; the project was conducted in water.

2.2 Geomembrane Tubes

Availability of geomembrane tubes was given consideration, but not to the level of geotextile tubes. Geomembrane tube efforts were related primarily to a single supplier due to many parameters including time constraints of the research team. The remainder of this section summarizes the information obtained related to geomembrane tubes.

AquaDam[®] refers to the same product as that discussed in Landis (2000), which was first referenced in Section 1.6. In the original reference, *WaterStructures*[®] was used to identify these specialty geomembrane tubes. The terminology was changed by the fabricator to allow the product to be more identifiable. *AquaDam*[®] units are the focus of this section, and unless it is specifically stated otherwise they are the product being discussed.

2.2.1 Fabrication of Geomembrane Tubes

AquaDam[®] units are based on a patented triple tube philosophy. Two independent geomembrane tubes are housed inside a larger master tube made of high strength geotextile. The concept of geomembrane tubes (specifically *AquaDam*[®] units) is illustrated in Figure 2.6. They consist of two basic components: 1) outer *master tube* made of woven polypropylene geotextile (indicated by C in Figure 2.6); and 2) inner geomembrane tubes that come into contact when filled (indicated by A and B in Figure 2.6).

Multiple styles of *AquaDam*[®] units are commercially available depending on the needs of the project. Two styles were identified as candidates for the needs of the current project in terms of their fabrication. A *closed-ended AquaDam*[®] unit has an open end that must be elevated, while the other end is closed. The closed end is often fitted with a collar to allow a watertight attachment of a second *AquaDam*[®] unit. A *double closed-ended AquaDam*[®] unit has both ends closed with pre-inserted fill hoses, and collars can be used to extend these units in both directions.

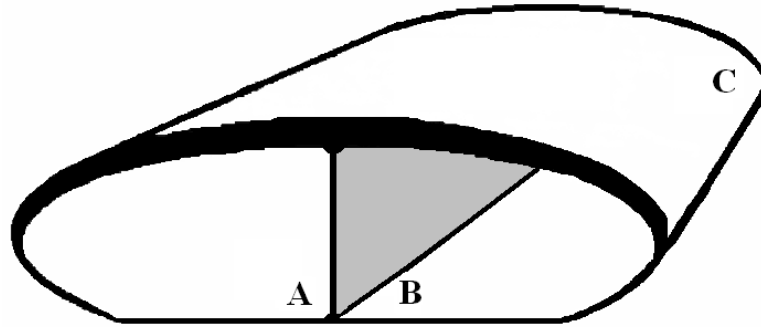


Figure 2.6. Concept of Water Filled Geomembrane Tubes

The single closed-ended *AquaDam*[®] as shown in Figure 2.7a is the standard configuration. As seen, the end nearest in Figure 2.7a is closed, while the far end remains open. Figure 2.7b is a double-closed ended unit, which is often used as a plug. Both ends are closed, the unit is symmetric, and the fill tubes extend out the top rather than the end. A baffled *AquaDam*[®] unit is the third style of potential applicability to the current needs. Figure 2.8 shows an example of a baffled unit. *AquaDam*[®] units are rolled onto a solid core (packaged similar to a carpet roll) and the inner and master tubes unroll perpendicular to the core due to water pressure as they are filled with water.



(a) Closed-Ended



(b) Double Closed-Ended

Figure 2.7. Photos of Standard AquaDam[®] Units



Figure 2.8. Baffled AquaDam[®] Unit

2.2.2 Availability of Geomembrane Tubes

The availability of geomembrane tubes was not explored to the extent of geotextile tubes due to time considerations. *Bradley Industrial Textiles* has developed geomembrane tubes (i.e. bladders) that fit inside their *SPIRAL*[®] geotextile tube units, which allows full fabrication of a geomembrane tube capable of supporting load. *Bradley Industrial Textiles*, though, noted that their geomembrane tubes (bladders) would probably need inventoried by the agency. *Hydrological Solutions Inc* provides *WIPP*[™], and *Aqua Barrier*[™] units. Both of these geomembrane tubes are available with inflated heights up to 2.44 m.

AquaDam[®] units are available in heights from 0.31 to 4.88 m in standard lengths of 15.2 and 30.5 m. Other lengths are available with notice, and attachment collars are available to connect as many *AquaDam*[®] units as are needed for the application. *Vantage Partners LLC* manufactures *AquaDam*[®] units for *Water Structures Unlimited*[®]. As of January 2009, *Water Structures Unlimited*[®] had a readily available stock of at least 305 m of each size *AquaDam*[®] unit in addition to what could be fabricated. The combination of maximum size and availability compelled the research team to consider *AquaDam*[®] units as the preferred option for applications requiring a geomembrane tube.

2.2.3 Properties of Geomembrane Tubes

Table 2.3 provides fundamental properties of *AquaDam*[®] units, including thicknesses of the impervious *inner tube*. Table 2.4 provides properties of the geotextile used to fabricate the *AquaDam*[®] *master tube*. The *inner tube* material is polyethylene and the *master tube* material is polypropylene. Since a preferred geomembrane tube was identified early in the evaluation, additional effort was not expended to catalog all properties of the geomembrane tubes. With exception of the properties in Tables 2.3 and 2.4, few other key parameters were identified. It is important to note that geomembrane tubes can be drained and re-used.

Table 2.3. Fundamental *AquaDam*[®] Properties

Inflated Height (m)	Empty Weight (kg/m)	Inner Tube Thickness ¹ (µm)	Master Tube ² (---)
0.30	0.5	254	LP 300
0.46	0.6	254	LP 300
0.61	0.8	254	LP 300
0.91	1.7	305	LP 300
1.22	2.9	305	LP 300
1.52	3.4	305	LP 300
1.83	5.7	305	LP 300
2.44	8.7	356	LP 300(d)
3.05	26.8	406	LP300(2)
3.66	33.5	406	LP 300 (d-2)
4.88	53.7	762	LP 300 (d-2)

1: Material used was polyethylene except for 4.88 m tube which was vinyl. The 3.66 and 4.88 m dams also had an LP 300 woven inner tube.

2: A material ending with “d” or a “2” indicates the material was used in a double layer, or two ply, respectively. Both used together indicate double 2-ply. LP 300 is a woven polypropylene fabric. *TenCate*[™] (*Mirafi 600-x*) is equivalent product.

Table 2.4. Properties of *Master Tube* Portion of an *AquaDam*®

Property	ASTM Test Method	Typical Value
Fabric Weight	---	211 g/m ²
UVI Ratings	---	90% at 3,500 hr
Breaking Load	D 4632	1,600 N
Apparent Elongation	D 4632	20 %
Puncture Resistance	D 4833	580 N
Trap Tear Resistance	D 4533	580 N
Mullen Burst Strength	D 3786 ¹	4.14 MPa
AOS	D 4751	0.40 mm (No 40)
Permittivity	D 4491	0.06 sec ⁻¹
Flow Rate	D 4491	163 L/min/m ²
Permeability	D 4491	0.003 cm/s

1: Modified Method.

2.2.4 Economic Data for Geomembrane Tubes

Table 2.5 provides economic information related to *AquaDam*® units. As seen, both purchasing and rental information is available. For a disaster environment, the option to rent the geomembrane tubes could be appealing provided the stocked quantities were sufficient and/or provided satisfactory arrangements with the company could be made. All three styles of geomembrane tube described in this report are included in Table 2.5.

Table 2.5. Cost Estimates of *AquaDam*® Units as of Jan 2009

Acquisition Method	Inflated Ht (m)	Closed End (\$/m)	Closed End Baffled (\$/m)	Double Closed End ¹ (\$/m)	Attachment Collar ²
Purchase	0.30	\$11.48	\$14.76	\$13.12	\$20.00
	0.46	\$19.69	\$24.61	\$22.97	\$25.00
	0.61	\$31.17	\$39.37	\$36.09	\$30.00
	0.76	\$32.81	\$41.01	\$37.73	\$35.00
	0.91	\$60.70	\$68.90	\$65.62	\$40.00
	1.22	\$127.95	\$150.92	\$149.28	\$80.00
	1.52	\$177.17	\$218.18	\$213.25	\$120.00
	1.83	\$223.10	\$262.47	\$255.91	\$200.00
	2.44	\$328.08	\$387.14	\$360.89	\$400.00
	3.05	\$820.21	Quoted	Quoted	\$700.00
	3.66	\$1,148.29	Quoted	Quoted	\$1,200.00
	4.88	\$1,476.38	Quoted	Quoted	\$2,400.00
Rental	0.30	---	---	---	---
	0.46	---	---	---	---
	0.61	---	---	---	---
	0.76	---	---	---	---
	0.91	\$36.42	\$41.34	\$39.37	\$24.00
	1.22	\$76.77	\$90.55	\$89.57	\$48.00
	1.52	\$106.30	\$130.91	\$127.95	\$72.00
	1.83	\$133.86	\$157.48	\$153.54	\$120.00
	2.44	\$196.85	\$232.28	\$216.54	\$240.00
	3.05	\$492.13	Quoted	Quoted	\$420.00
	3.66	\$688.98	Quoted	Quoted	\$720.00
	4.88	\$885.83	Quoted	Quoted	\$1,440.00

1: Cost Includes Hoses and Fittings.

2: Cost of a Single Attachment Collar of the Indicated Size.

CHAPTER 3 - GEOTEXTILE TUBE CONSTRUCTION SITE VISITS

3.1 Overview of Site Visits

Three projects were visited by the research team related to geotextile tubes and corresponding applications. The projects visited were: 1) *Matagorda Ship Channel* where marine construction was of primary interest; 2) *Grand Isle* where coated geotextile tubes and simultaneous filling from multiple ports were the primary interests; and 3) *Peoria Island* where using fine grained materials as fill was the primary interest. Two additional projects on the Gulf Coast (Deer Island and Pascagoula Beach) were discussed with the Mobile District of the US Army Corps of Engineers (*USACE*), but schedules did not permit visiting of these sites. The Pascagoula Beach project was of interest since sand was to be pumped on the order of 11 km on a project employing approximately 2.3 km of geotextile tube.

3.2 *Matagorda Ship Channel*

The research team visited a geotextile tube construction project in the vicinity of Port Lavaca and Port Comfort Texas on April 14 and 15, 2009. The project was adjacent to the *Matagorda Ship Channel*, a waterway approximately 35.4 km long and over 11 m deep. The channel is routinely dredged to maintain adequate depth; the waters adjacent to the channel are shallow (on the order of 1 to 2 m depending on the tidal conditions). The purpose of the wall constructed out of geotextile tubes was to retain sediment dredged from the channel to prevent it from washing back into the channel over time with movement of tidal waters. The channel is part of the Gulf Inner Coastal Waterway and a primary shipping route connecting major industrial facilities with the Gulf of Mexico. Figure 3.1 is an example of industrial activity within a kilometer of the construction site.



Figure 3.1. Industrial Activity Adjacent to Construction Site

3.2.1 *Matagorda Ship Channel* Project Parameters

The total project contained on the order of three million cubic meters of dredging, for which the majority was related to dredging the ship channel. A subcontract was issued for geotextile tube placement. *RLB Contractors* of Port Lavaca, Texas was subcontracted as an add-on pay item by *McQuay Dredging* to perform the construction. Larger dredging companies are typically more focused on large volume dredging projects and do not typically

own the smaller and more specialized equipment associated with geotextile tube construction. As a result, geotextile tube construction is usually issued as a subcontract from a large scale company (e.g. *McQuay Dredging*) to groups with specialized experience in the area (e.g. *RLB Contractors*). For perspective, typical mass dredging crews for large scale operations were estimated by *McQuay Dredging* to be 20 personnel per ship working and 10 personnel idle. A typical day shift was estimated at 10 to 12 personnel and the night shift was estimated at 8 to 10 personnel. Each shift lasts 12 hr.

The total project length was on the order of 460 m. The goal of the project was to replace existing geotextile tubes that, according to the contractor, were originally filled with fine grained material. Over time the fine grained materials had pumped out of the geotextile tubes, marine equipment had torn the geotextile tubes, or the tubes had otherwise suffered damage. The client was the Galveston District of the *USACE*.

3.2.2 Matagorda Ship Channel Equipment

The equipment used during construction was: 1) *Flexi Float* spud barges (nine total) and tug; 2) *Ellicott Series 370* cutterhead dredge with a 250 mm pump; 3) polyethylene dredge pipe with 250 mm inner diameter and 25 mm wall thickness; 4) *CAT 330 L* trackhoe; 5) *CAT D6R* dozer; 6) *CAT 322 CL* amphibious trackhoe; 7) three small operations boats; and 8) incidental equipment such as passenger vehicles. Figure 3.2 shows the spud barges (connected together in figure) and tug, Figure 3.3 shows the dredge and dredge pipe laid out between the dredge and geotextile tubes, and Figure 3.4 shows the amphibious trackhoe.



(a) *Flexi Float Spud Barge*



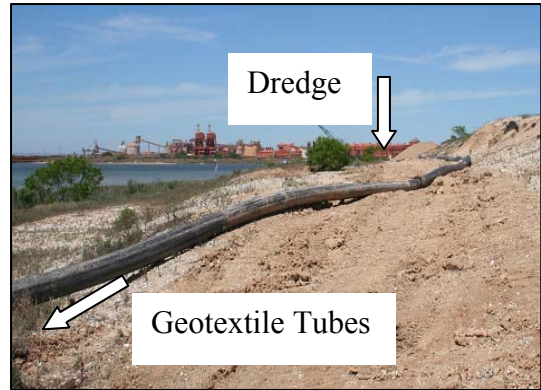
(b) *Tug*

Figure 3.2. *Flexi Float Barge and Tug Used at the Matagorda Ship Channel*

Note cutter head dredges are supplemented with water jets when dredging sands and silts, but are typically not used with clays. *McQuay Dredging* representatives noted that D_{50} was used qualitatively when considering the necessity of water jets, though experience gained in the early part of the project often dictates the addition of water jets. Also note sands do not usually flow well unless cut on a slope. Clays often stand on vertical submerged cuts and the cutter head progresses in a column pattern; the dredge is lifted from the water to start the cut in some instances. Silts are usually fairly easy to get into suspension and into the dredge pipe. Water jets are also used on soils with trapped gases to prevent them from damaging the equipment via vibrations generated by gas cavitations in the pipeline.



(a) Dredge



(b) Dredge Pipe

Figure 3.3. Dredge and Dredge Pipe Used at the Matagorda Ship Channel



Figure 3.4. Amphibious Trackhoe

3.2.3 Matagorda Ship Channel Materials Incorporated

Materials needs for construction of geotextile tubes can be grouped into: 1) geosynthetics used to manufacture the geotextile tubes, anchor tubes, scour apron and/or UV shroud; and 2) soil used to fill geotextile tubes and/or anchor tubes. Figure 3.5 is a profile view of all geosynthetic materials used at the *Matagorda Ship Channel*.

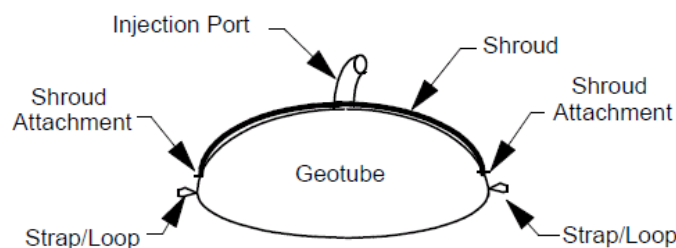


Figure 3.5. Profile View of Geotube[®] Units Used at the Matagorda Ship Channel

A woven polypropylene geotextile (GT 1000M) was used for the tubes that had an Apparent Opening Size (AOS) of 30; complete properties shown in Table 2.1. Also shown in

Table 2.1 are the properties of the geotextile (*GT 1000MPET*) used on a previous project at the *Chocolate Bay* in Texas involving the same contractor and material supplier. The *AOS* of 100 (*GT 1000MPET*) would more easily retain particles than would the *AOS* of 30 (*GT 1000M*). The scour apron and shroud at the *Matagorda Ship Channel* were manufactured using *GT 500* geotextile; complete properties can be found in Table 2.2.

The contractor indicated that a key to success on this project was to minimize the amount of material (sand in this case) exiting the geotextile tube during filling since it was a purchased commodity. Coarser sand was used for the project that was located 56 km from the site in favor of finer sand located only 16 km from the site. The material used to fill the geotextile tubes was tailings from a sand and gravel operation. On the order of 4,500 metric tons of sand was purchased to fill the geotextile tubes. A sample of the fill material was taken and tested as described later in this chapter.

Discussion was held with the contractor and geotextile tube supplier related to selection of coarse sand. Relatively fine sand was used at the *Chocolate Bay* and the contractor felt it could have been problematic with the more porous fabrics used at the *Matagorda Ship Channel*. Noticeable portions of the sand selected for this project could be seen escaping the apertures in the geotextile tube during filling. This was especially true when pressure was applied to the outside of the geotextile tube (e.g. walking on tube when filling). See Figure 3.6 for photos of water escaping the tube with portions of the fill.

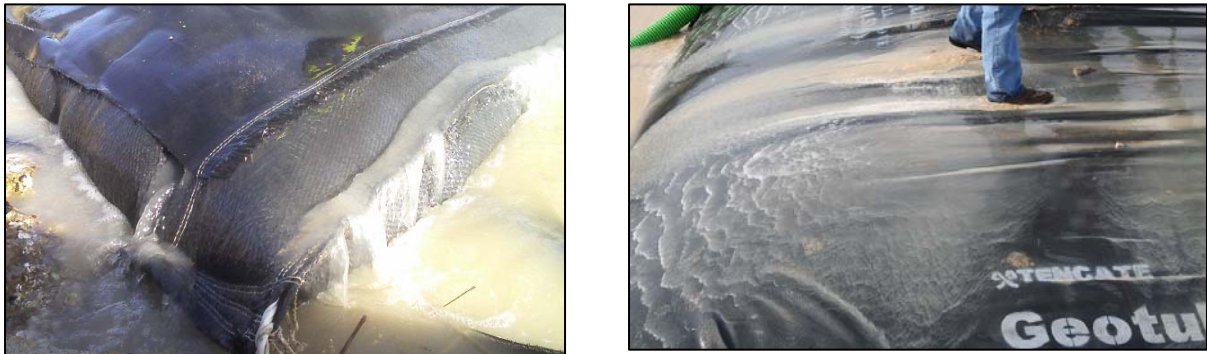


Figure 3.6. Water Escaping Geotextile Tubes During Filling

The contractor indicated the filling methods incorporated into a project varied depending on the type of material. The material supplier discussed past experiences where geotextile tubes made of polyester were found to wear near the inlets in some conditions due to the abrasive nature of many sands. The *GT 1000MPET* geotextile with an *AOS* of 100 used at the *Chocolate Bay* was made of polyester and is being phased out by *TenCate*™.

Five geotextile tubes were used for the construction. Four of the tubes were 91.4 m long with the final tube being 106.7 m long. For the 91.4 m tubes, fill ports were placed 3.05 m from either end of the geotextile tube and 12.20 m center to center throughout the interior of the tube (dimensioning was similar for the 106.7 m tube). All geotextile tubes were manufactured from longitudinal seaming using two 5.19 m geotextile sheets, which produced tubes with a circumference on the order of 9.14 m and a design height of 1.52 m. The scour apron was 6.1 m wide and anchored by 0.91 m circumference tubes on either side. Figure 3.7 is a photo of geotextile and anchor tubes on the barge awaiting placement.



Figure 3.7. Geotextile and Anchor Tubes Used at Matagorda Ship Channel

3.2.4 Matagorda Ship Channel Construction Procedures

Figure 3.8 is an overall view of the construction and key equipment. Figures 3.9 and 3.10 are close up views near the sand stockpile and dredge. Figure 3.11 is a photographed view of the geotextile tube wall and island from atop the reclaimed land, while Figure 3.12 is a view of the geotextile tube wall from the island looking toward the reclaimed land just in front of the geotextile tube deployment. Figure 3.13 has photos showing the sluice box as sand was loaded to be slurried. The sand slurry was pumped on the order of 450 m. There was not a target slurry percent solids; it was loosely controlled based on inlet feed rate and estimated by the contractor to be 10% or more. The dredge pump operated at approximately 11,000 liters per minute at 1,450 rpm. At the time of construction the water depth was approximately 0.9 m (note, though, that construction personnel were working in mud up to 0.3 m beneath the brackish water surface).

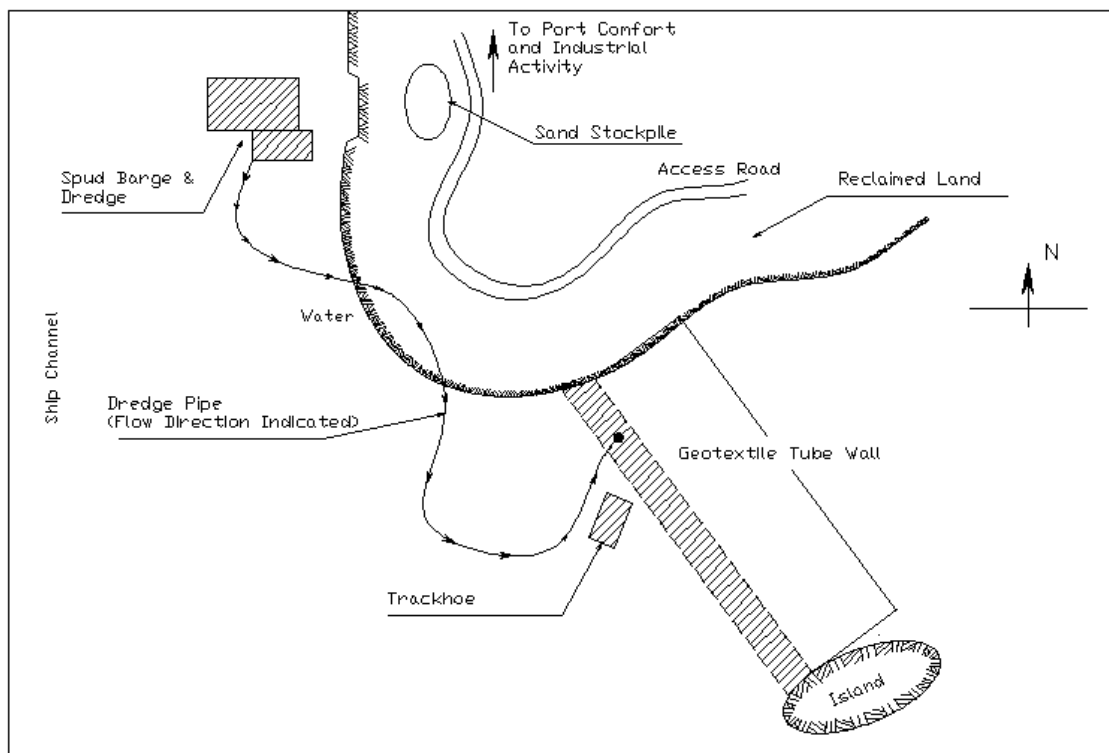


Figure 3.8. Plan View of Entire Matagorda Ship Channel Construction

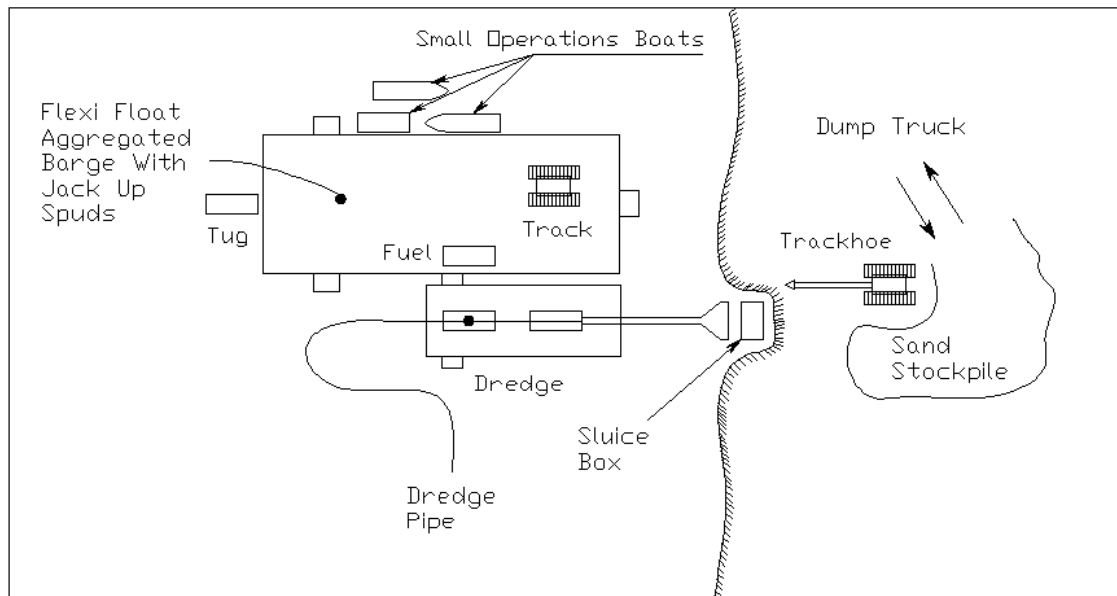


Figure 3.9. Plan View of Matagorda Ship Channel Construction at Sand Stockpile

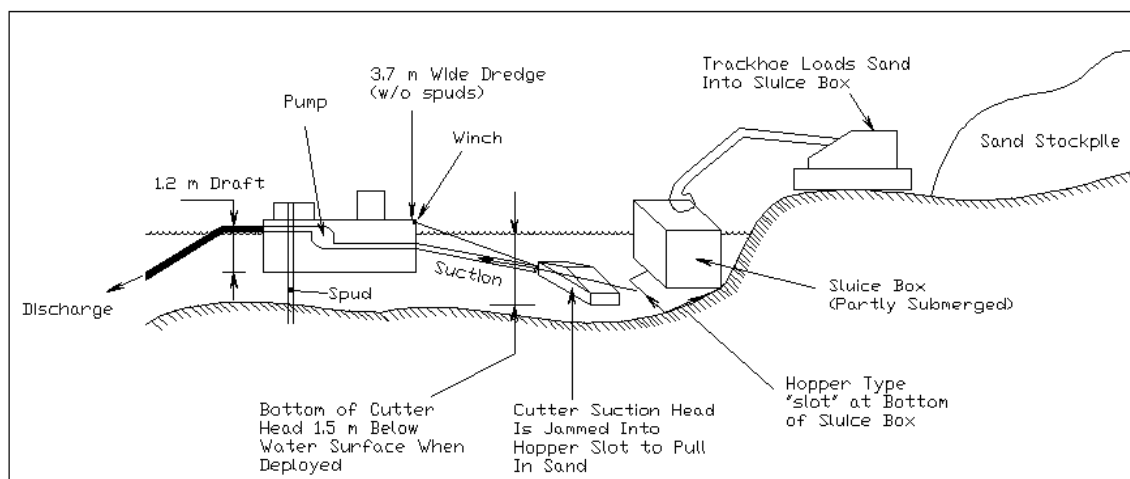


Figure 3.10. Profile View of Matagorda Ship Channel Construction at Dredge



Figure 3.11. View of Matagorda Ship Channel Geotextile Tube Wall from Reclaimed Land



(a) View from Island to Reclaimed Land



(b) Geotextile Tube on Pole Being Deployed

Figure 3.12. View of Matagorda Ship Channel Geotextile Tube Wall from Island

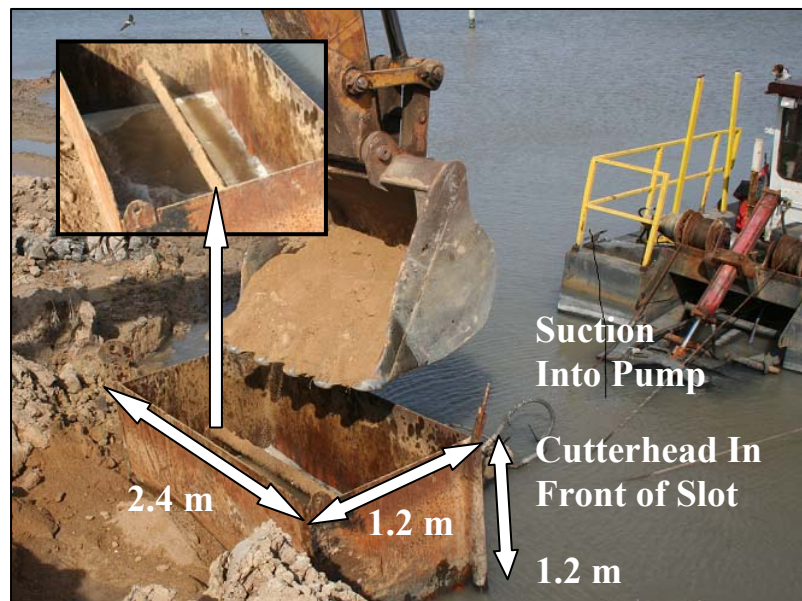


Figure 3.13. Sluice Box Used to Feed Dredge

In general, the tubes were filled from one port (Figures 3.8 and 3.14). They were topped off as necessary from multiple ports using 100 mm diameter flexible line running from the primary dredge pipe. The 100 mm flexible line was also used to fill the anchor tubes (Figure 3.14b). Note the trackhoe supported the discharge pipe to reduce stress on the port (especially the port seams).

The major steps in construction of the geotextile tube wall at each geotextile tube are shown in the seven steps outlined below. It was noted while on site that some contractors pump the anchor tubes first while others pump the geotextile tubes first to allow settlement prior to pumping the anchor tubes. The order of operations within step 7 varies considerably from project to project and between contractors. Specifications were noted to affect the order of pumping in many instances.

- 1) Determine placement location using standard surveying equipment
- 2) Drive lines of poles with the bucket of the amphibious trackhoe
- 3) Pin one end of the scour apron

- 4) Unroll scour apron with amphibious trackhoe
- 5) Unroll geotextile tube with amphibious trackhoe while tying it to poles as unrolled
- 6) Unroll anchor tubes
- 7) Pump geotextile tube partially, pump anchor tubes, finish pumping geotextile tube



(a) Filling Geotextile Tube



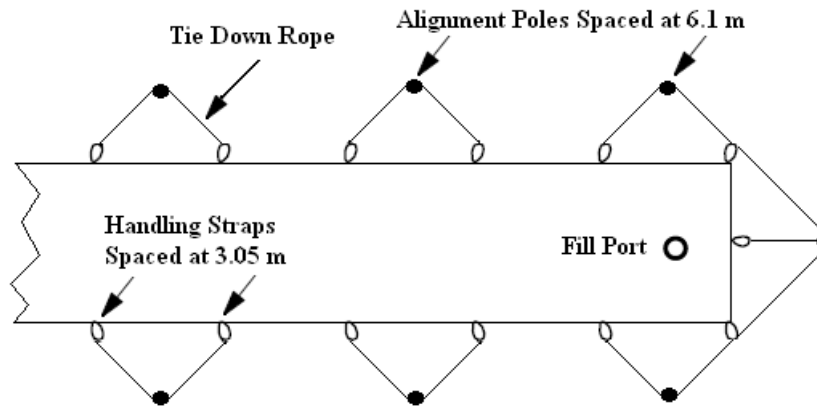
(b) Filling Geotextile and Anchor Tubes

Figure 3.14. Filling Matagorda Ship Channel Geotextile Tubes from one End

Wind was blowing 10 to 15 knots during the site visit, which made small waves in the area where the wall was being constructed. Material suppliers estimated that a wind exceeding 25 knots might delay such a project during the phases where the tubes were being placed and that the effect of the wind was described to be a function of the water depth and amount of geotextile tube to place.

The support poles (steel pipe) were offset a moderate distance from the edge of the geotextile tubes (Figure 3.15). They were 64 mm diameter poles with 6.25 mm wall thickness. The poles were 4.9 m long and 50 to 60% of their depth was pushed below the mudline. Handling straps were placed at 3.05 m centers for this project rather than the typical manufacturer standard of 6.1 m. Figure 3.15 shows alignment of the handling straps and alignment poles.

During filling of the geotextile tubes, the dredge was paused to replace bearings and when re-started the dredge pipe was clogged. The contractor envisioned servicing to go quickly and did not flush the dredge pipeline before shutting down for repair. The sand in the pipe fell out of suspension and developed a plug approximately halfway between the dredge and tubes. To remove the plug, the polyethylene pipe was cut, the plug removed, and the pipe re-attached using a fusion welder. The dredge pipe was 250 mm inner diameter with a 25 mm wall thickness. Approximately 8 hours of working time were lost to the clogged pipe. The polyethylene dredge pipe floats when filled with water but not when plugged (Figure 3.16 shows the pipe while floating). The contractors tracked the plug in the pipe by observing the flotation of the dredge pipe.



(a) Schematic of Alignment Poles and Handling Straps



(b) Photo of Alignment Poles Connected to Handling Straps

Figure 3.15. Alignment Poles and Handling Straps Used at Matagorda Ship Channel



Figure 3.16. Dredge Pipe Floating in Water

3.2.5 Matagorda Ship Channel Construction Costs

The sand used as fill material was purchased for \$9.09 per metric ton (\$10 per short ton), with 25% of the cost being the material and 75% of the cost being transport to the site. A total of \$50,000 was spent on fill material, a total of \$120,000 was spent on geosynthetic materials (geotextile tubes, anchor tubes, scour apron, and shroud), and the bid price was \$900,000.

Table 3.1 summarizes equipment and daily labor costs for the *Matagorda Ship Channel* obtained from the contractor. As seen, the cost can easily exceed \$20,000 per day per construction group excluding mobilization. Mobilization information is very difficult to fully quantify for purposes of this project due to the number of variables involved. Mobilization and/or demobilization for the barge and dredge from Port Lavaca to Houston, Texas was estimated at \$40,000 and travel was estimated at 3 to 5 knots.

Table 3.1. Matagorda Equipment and Labor Daily Cost Estimates From Contractor

Item	Type	Quantity	Unit Cost	Total Cost
Spud Barge and Tug	Flexi Float	1	\$5,000	\$5,000
Dredge	Ellicott Series 370	1	\$4,500	\$4,500
Dredge Pipe	Polyethylene	450	\$2.67	\$1,200
Trackhoe	CAT 330 L	1	\$1,200	\$1,200
Dozer	CAT D6R	1	\$1,000	\$1,000
Amphibious Trackhoe	CAT 322 CL	1	\$2,500	\$2,500
Small Operation Boats	---	3	\$167	\$500
Labor	---	3 to 10	\$300	\$900 to \$3,000
Total Daily Cost of Key Items:			\$16,800 to \$18,900	

Notes: Dredge pipe quantity expressed in meters of pipe and the seven laborers were the number on site during the visit by the research team. Other costs such as passenger vehicles, small equipment, commodities, and similar were not considered since they are difficult to quantify and could vary substantially from project to project but one should note they would add additional expense.

Ten construction personnel were required to place the poles, scour apron, anchor tubes, and geotextile tube for one section of the wall and begin pumping into it the same day (estimated to take 8 to 10 hours). The goal of this project was to fill one of the 91.4 m tubes (already placed) in one day (estimated at 12 hours or more). Once pumping of the geotextile tubes was underway two or three construction personnel were often sufficient. Note the same contractor and material supplier worked on a project in the *Chocolate Bay* in Texas where finer sand was used in conjunction with a less porous geotextile tube. In general it took 15 to 20 hours to pump an equivalent circumference tube that was 61 m long; the time would have shortened provided more of the fine sand could have been retained by the tube.

3.3 Grand Isle Hurricane Protection

The research team visited a geotextile tube construction project in *Grand Isle*, LA on May 26, 2009. *Grand Isle* is located on Louisiana's southernmost inhabited barrier island. The project incorporated large numbers of geotextile tubes that were coated with a protective medium. Significant findings from the site visit are provided in the remainder of this section, while Bygness (2009) and Cardno (2009) also provide details of the entire project.

3.3.1 Grand Isle Project Parameters

The *USACE* designed the project out of the St. Louis District office with assistance from *Ocean and Coastal Consultants* (on site consultants for St. Louis District). The *Hurricane Protection Office (HPO)* out of New Orleans was responsible for on site Quality Assurance (QA). The prime contractor was *Weeks Marine*. They performed all dredging and

project management. *Infrastructure Alternatives* was subcontracted to place and fill all geotextile tubes. *Bertucci* was subcontracted for grubbing and vegetation parameters. The same trend of a larger entity (*Weeks Marine*) subcontracting placement and filling of the geotextile tubes to a specialized contractor (*Infrastructure Alternatives*) was observed on *Grand Isle* just as with the *Matagorda Ship Channel*. *Infrastructure Alternatives* had 13 people on the project during typical operation.

The total length of the project where the research team visited was on the order of 9.5 km; Bygness (2009) indicates a total storm surge barrier of 11.25 km. The objective of the construction was hurricane protection for the community of *Grand Isle*. The project was to be completed in just over three months (late May to late August of 2009). Figure 3.17 is a photo from behind the geotextile tube wall facing the shoreline.



Figure 3.17. View of Grand Isle Shoreline from Behind Geotextile Tube Wall

3.3.2 *Grand Isle* Equipment

Table 3.2 lists all major equipment used at *Grand Isle*. Two locations were used on the project that incorporated practically identical equipment with exception of the screener which was used to process all dune sand for both construction locations (i.e. there were two of every item listed with exception of the lone screener). Photos of selected equipment are shown in Figure 3.18.

Table 3.2. Equipment Used at *Grand Isle*

Equipment	Make	Model
Trackhoe	Komatsu	PC 200 LE
Slurry Box	---	---
Generator	Aggreko	---
Dozer	Komatsu	KM060084
Water Pump	BPS	C 300-120001
Pipe	SS 200	EDPM S&D
Forklift	GEHL	GH070374
Slurry Pump	---	100 mm
Slurry Pump	---	200 mm
Screener	McCloskey	621



(a) Slurry Box



(b) Generator



(c) Screener and Conveyor Belt



(d) Waste Material Screened from Fill

Figure 3.18. Select Equipment Used at Grand Isle

3.3.3 Grand Isle Materials Incorporated

Polyurea coated geotextile tubes were used on the entire project. The material was applied at the *Tencate™* facility. According to the producers, the polyurea felt somewhat solid to the touch at approximately fifteen seconds, and was fully cured within one hour. Ten tubes were coated per day with a nominal minimum polyurea thickness of 760 μm (30 mils). Figure 3.19a shows an overall view of the coating, while Figure 3.19b shows a close up view of the coating. The coating was applied to the upper half of the geotextile tube.



(a) Global View



(b) Local View

Figure 3.19. Polyurea Coating on Geotextile Tubes

The polyurea coating was specified to be tan in color due to aesthetic parameters. The purpose of the polyurea coating was to provide additional protection against debris damage during storms (i.e. armoring). Note the tubes were to be covered with sand as part of the construction. The project called for slightly over 150 geotextile tubes that were each 61 m long, in addition to a small number of specialty length geotextile tubes near the end of construction to complete the hurricane protection wall. Figure 3.20 shows a considerable number of the geotextile tubes on site and awaiting placement.



Figure 3.20. Geotextile Tubes at Grand Isle Awaiting Placement

Each tube contained five fill ports with uniform spacing; fill ports began on the order of 3 m from each end and were spaced 15 m apart in the interior. The circumference of the geotextile tubes was 9.14 m, and the design height of the geotextile tubes was 1.68 m. This combination of circumference and height resulted in an internal factor of safety on the order of 3, which is on the low end of internal factors of safety preferred by *Tencate*[™] representatives. The anchor tubes were 2.14 m circumference, which is on the larger end of circumferences typically used as anchor tubes. *GT 500* was the geotextile used for the entire project (geotextile tubes, anchor tubes, and scour apron). The anchor tubes were integrated into the scour apron. Table 2.2 contains fundamental properties of the *GT 500* material.

Dune sand was used to fill the geotextile tubes after it was screened for impurities and mixed with salt water. A maximum of 10% fines was allowed in the fill material. Sand dredged from off the coastline was used to cover the geotextile tubes. A sample of the dune sand was taken for testing after the material went through the screener and was ready for use.

3.3.4 Grand Isle Construction Procedures

Figure 3.21 is the overall construction schematic incorporated at *Grand Isle*. The contractor filled the geotextile tubes by first pumping simultaneously into ports 1 and 3 before moving the hoses and pumping into ports 2 and 4. Port 5 remained open the entire time. Once the tube was essentially filled, a 100 mm flexible pipe was used to bring the entire tube to an even profile. Note that the anchor tube for the next geotextile tube was being filled in unison. Construction was simultaneously conducted at two sites that were both oriented as shown in Figure 3.21. The total length of the project was on the order of 9.5 km. Figure 3.22 shows representative portions of the construction.

The scour apron and anchor tubes were first placed onto the beach and the geotextile tube placed on top of the scour apron. Initially, the specifications for the project required pumping the anchor tubes before pumping the geotextile tubes. A meeting was held with *USACE*, *Tencate*[™], and contractors on May 26, 2009 related to a few logistical construction

parameters (e.g. sequence of filling anchor and geotextile tubes, light vehicle traffic on scour apron, and similar). The discussion focused on the sequence of filling tubes.

In the initial stages of the project the contractor was trying to fill one to two tubes per day; it took on the order of 8 hr to completely fill a tube once pumping initiated. Cardno (2009) indicated 183 m of tube was placed per 12 hr shift. The first geotextile tube was curved to tie into a previously installed *burrito* tube from a company other than *TenCate*TM. A *burrito* tube is a technique where a geotextile is wrapped around fill material. Figure 3.23 shows the curving of the geotextile tube wall to overlap the previous installation. Environmental parameters placed the geotextile tubes very close to the housing running adjacent to the beach, as is visible in Figure 3.23.

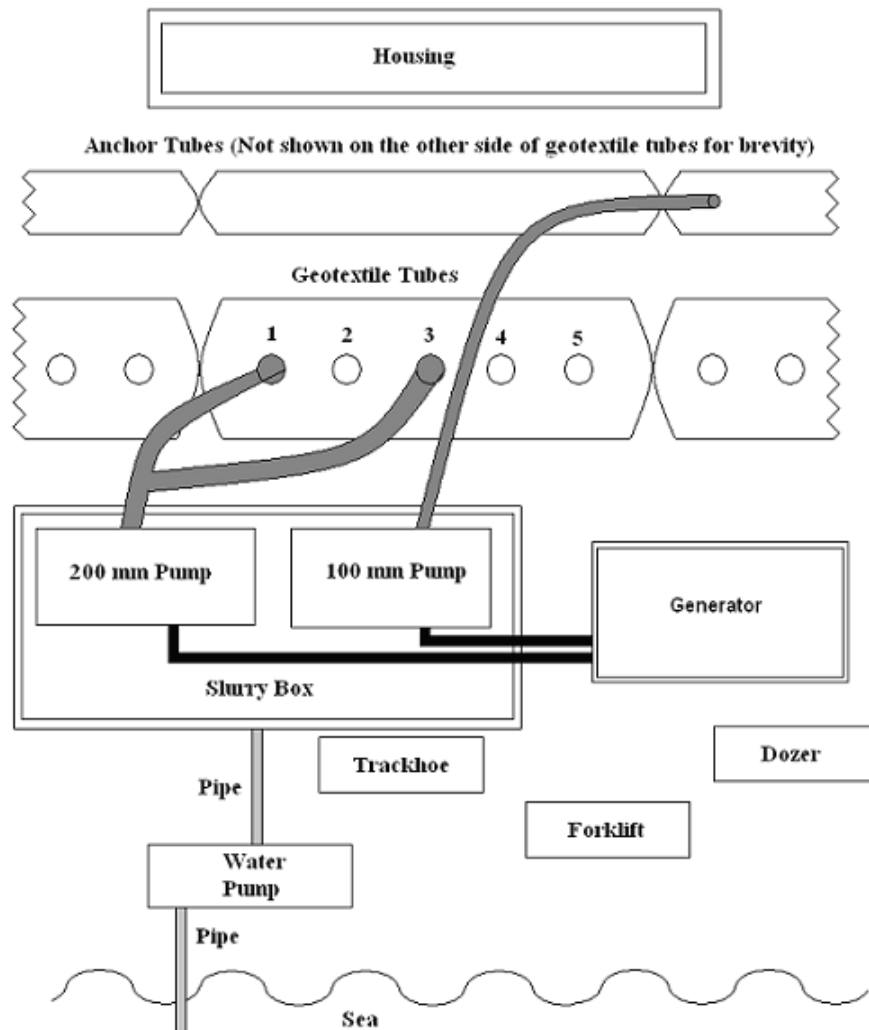


Figure 3.21. Schematic of Grand Isle Construction Layout



(a) Placed and Ready for Filling



(b) After Filling

Figure 3.22. Geotextile Tubes, Anchor Tubes, and Scour Apron Pre and Post Filling



Figure 3.23. Curved Portion of Geotextile Tube Wall

The polyurea coating significantly changes how water escapes from the geotextile tube during filling. Instead of much of the water escaping through the pores of the fabric, the water is released through the open fill ports along the top. During initial filling of the tubes, though, significant water escaped as seen in the trench shown in Figure 3.24 where water was channeled back into the sea.



Figure 3.24. Trench Capturing Water Escaping Tubes During Filling

The percent solids were believed to be less than 20% during filling but were not rigorously controlled. Introduction of sand into the slurry box caused variability in the percent solids, with the solids content being relatively low just before a new bucket of sand

was introduced and relatively high just after a new bucket of sand was introduced. The slurry box (Figure 3.21 and Figure 3.25a) was used to create the slurry by pumping water from the sea and dumping processed sand into the slurry box with a trackhoe. At peak operation, on the order of 1 m^3 of sand was placed into the geotextile tube by the 200 mm pump every minute. When filled the tubes held 294 m^3 and weighed approximately 600 metric tons.

The tubes were filled with a manifold style system using pinch clamps (Figure 3.25). This style of clamp is somewhat common in geotextile tube construction projects. The clamps are used to regulate the flow of slurry into the tube as the filling progresses.

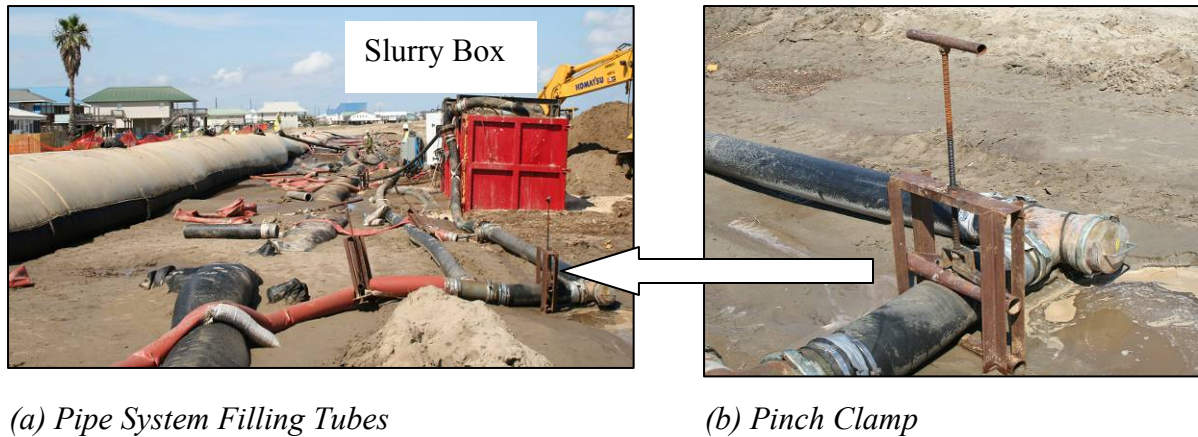


Figure 3.25. Manifold System and Pinch Clamps

Figure 3.26 shows the ends of two tubes that were completely filled. As seen, the profile at the ends of the tubes can be somewhat lower than the majority of the tube. Adapters are available to restore the uniform profile across the butting of geotextile tubes. It is noteworthy that the 100 mm flexible pipe was clogged during the project visit. It is also noteworthy that the cover required over the geotextile tubes was 0.91 m.



Figure 3.26. Ends of Two Geotextile Tubes After Filling

3.3.5 *Grand Isle Construction Costs*

The total cost of the project was \$25.7 million according to representatives on the job site and Cardno (2009). Bygness (2009) indicates a \$50 million budget for re-construction of *Grand Isle* flood barriers decimated by previous hurricanes. The geosynthetic materials were purchased for on the order of \$4.2 million, or approximately \$460 per m. More detailed economic information was obtained for the *Matagorda Ship Channel* project since the project more closely resembled the type of work that would be performed in a disaster environment within a flooded area.

3.4 *Peoria Island*

The research team visited a geotextile tube construction project referred to herein as *Peoria Island* on August 4 to 5 of 2009. The research team was taken to the site by boat. The project was on the *Illinois River* in Peoria, IL at a widening of the river referred to as *Lower Peoria Lake*. Construction was adjacent to the *McClugage Bridge*. Significant findings from the site visit are provided in the remainder of this section, and Putzmeister America (2009) also documented the project and has additional information.

3.4.1 *Peoria Island Project Parameters*

The *Rock Island District* of the *USACE* designed the project and referred to it as: *Peoria Riverfront Ecosystem Development Project-Stage 1*. Both internal and external stability calculations were said to be performed on the project. High solids dredging was a notable component of the project whose goals were stated to be: restore depth diversity; provide habitat improvements; and improve water quality. Conventional hydraulic dredging was not allowed on the project. *Midwest Foundation Corporation* was the prime contractor and did not employ notable subcontractors. The island covered 85,000 m², and the geotextile tube fill material was obtained by dredging a channel around the project whose ultimate purpose was a fish habitat. Figure 3.27 is an overall view of the project. The project timeline was January 15 to December 15 of 2009.



Figure 3.27. Overall View of Peoria Island Project

3.4.2 Peoria Island Equipment

Equipment used to construct *Peoria Island* were: crane with clamshell bucket, tugboat, mud hopper, modified *Putzmeister 14000 HPD* positive displacement pump with a capacity of 200 m³ per hour, *BobCat*® 337 amphibious trackhoe, large barge that had small miscellaneous equipment in addition to that listed, small float where geotextile tubes were stored, and small motor boats. Of primary significance to this research was the use of a positive displacement pump. Photos of the equipment can be seen later in this section when construction procedures are discussed.

3.4.3 Peoria Island Materials Incorporated

Maccaferri Inc supplied the geotextile tubes (*MacTube*™), which were packaged in a very compact manner as seen in Figure 3.28 relative to the other site visits. Approximately 165 geotextile tubes were used for the project, with 30.5 m lengths making up nearly all of the tubes. Tubes 18.3 m long were used for select needs such as connection of portions of the wall. Two 18.3 m tubes were used as replacement of one 30.5 m tube to allow for adequate overlap. The total length of geotextile tubes was on the order of 5,000 m. As a reference, the maximum allowable length of the geotextile tubes for this project was 61 m. No anchor tubes, shrouds, or scour apron were used on this project in conjunction with geotextile tubes. Geotextile tubes were placed directly on the riverbed sediment.



Figure 3.28. Peoria Island Geotextile Tubes Packaged and Awaiting Placement

Geotextile tubes of 13.72 m circumference were used throughout. Straps were placed every 6.1 m along the sides of the tube. Fill ports were spaced at 6.10 m. The maximum allowable fill port spacing for this project was 15.24 m. Geotextile tube heights of 1.83 ± 0.31 m were required 30 days after placement. The height required was specified for the tube and not for finished grades; it was noted that final elevations could vary. Construction drawings showed geotextile tube widths of 5.8 m, heights of 1.83 m, and an increase in ground line elevation at the top of the tube of approximately 1.52 m (i.e. 0.31 m of settlement was shown). Figure 3.29 shows settlement of one of the tubes at *Peoria Island*.



Figure 3.29. Settlement of Geotextile Tube

ASTM D 4873 governed delivery, storage, and handling of the tubes. Geotextiles were specified to be a woven pervious sheet of polypropylene yarn (slit film geotextiles were not allowed). The geotextile was required to be a supplier stock item and have been previously used in at least two projects. Required properties as per original plans are shown in Table 3.3; information from the contractor indicated some properties were allowed to be modified at the approval of all interested parties.

Table 3.3. Required Geotextile Properties for Peoria Island

Property	Test Method	Acceptable Value
Apparent Opening Size	ASTM D 4751	No. 40
Flow Rate	ASTM D 4491	815 L/m ² /min
Permeability	ASTM D 4491	0.04 cm/sec
Permittivity	ASTM D 4491	0.26 sec ⁻¹
Puncture	ASTM D 4833	1,246 N
Wide Width Strength (MD)	ASTM D 4595	70 kN/m
Wide Width Strength (CD)	ASTM D 4595	96 kN/m
Wide Width Elongation (MD)	ASTM D 4595	20%
Wide Width Elongation (CD)	ASTM D 4595	20%
Mullen Strength	ASTM D 3786	8.27 MPa
Trapezoid Tear (MD)	ASTM D 4533	801 N
Trapezoid Tear (CD)	ASTM D 4533	1,223 N
UV Degradation at 500 hr	ASTM D 4355	80% Retained
Factory Seam Strength	ASTM D 4884	81.7 kN/m
Percent Open Area	Specifications	4%

The material used to fill the geotextile tubes was fine grained river sediment with an in-situ density of 1.59 g/cm³. Sixteen boring logs were compiled between 2000 and 2004. Logs of these borings were provided by the *Rock Island District* of the *USACE* in their design drawings (Figure 3.30). The predominant material within a moderate distance of the surface was fine grained (mostly *CH* material with noticeable organics in some areas). Liquid limits from the boring samples ranged from 56 to 72, plastic limits from 24 to 33, and fines contents of 71 to 99%.

Sediments were sampled daily with the weekly average total solids (*TS*_%) required to be a minimum of 50%, and the weekly minimum *TS*_% value required to be 40%. Repeated

filling was allowed to achieve the design height. The contractor indicated $TS_{\%}$ values on the order of 60% were typically being pumped into the tubes during construction. Ten samples taken by the research team from the geotextile tube inlet while on site produced maximum, minimum, and average $TS_{\%}$ values of 59.9%, 57.3%, and 58.7%, respectively, which aligns well with contractor findings. A sample of material was obtained from the inlet of one of the geotextile tubes and was taken for testing described later in this chapter.

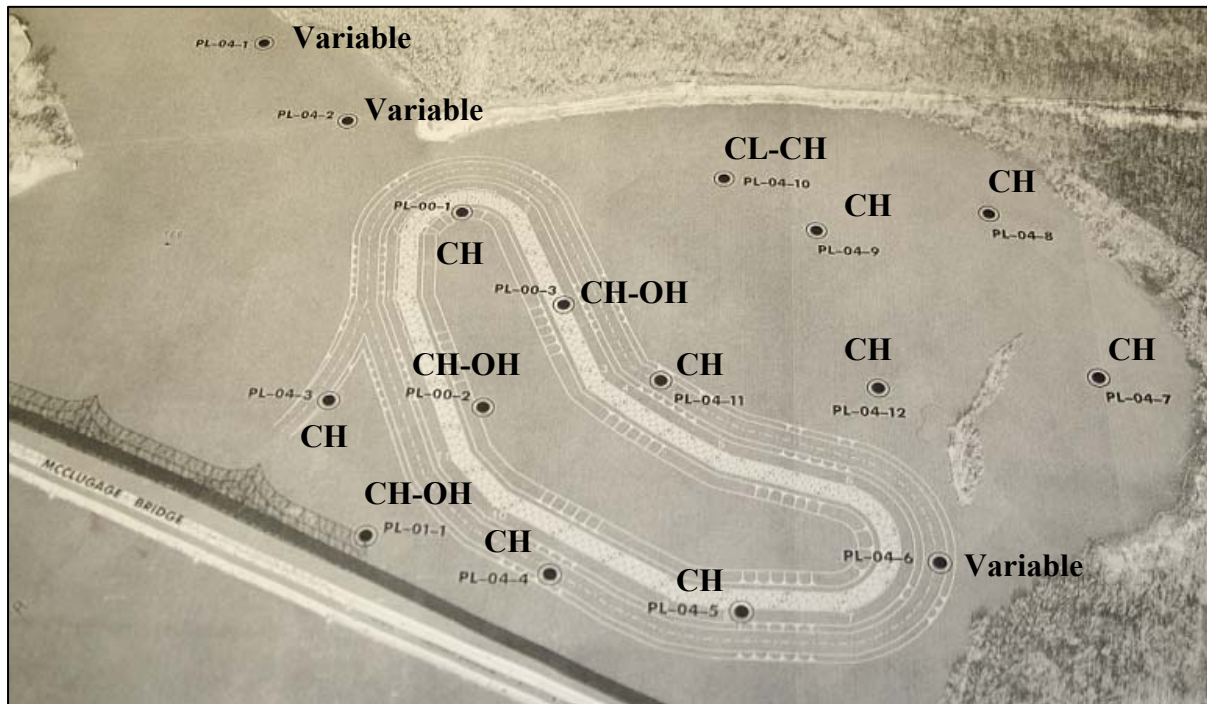


Figure 3.30. Schematic of Peoria Island and Boring Log Locations

3.4.4 Peoria Island Construction Procedures

Figure 3.31 shows the barge containing a substantial amount of the equipment used. Access from the navigation channel to the project site required dredging. As seen, the clamshell bucket (specialty patented bucket) removed the sediment from adjacent to the barge, placed it onto a mud hopper, and once screened the material was placed into the geotextile tubes (not shown in Figure 3.31) using a positive displacement pump. Construction logistics were to dredge a 1V:4H channel on the order of 1.68 m deep and 25 m wide at a minimum distance of 14.5 m from the outer edge of the geotextile tubes and use the dredged sediment to fill the tubes and to construct a berm around the inside of the geotextile tube wall.

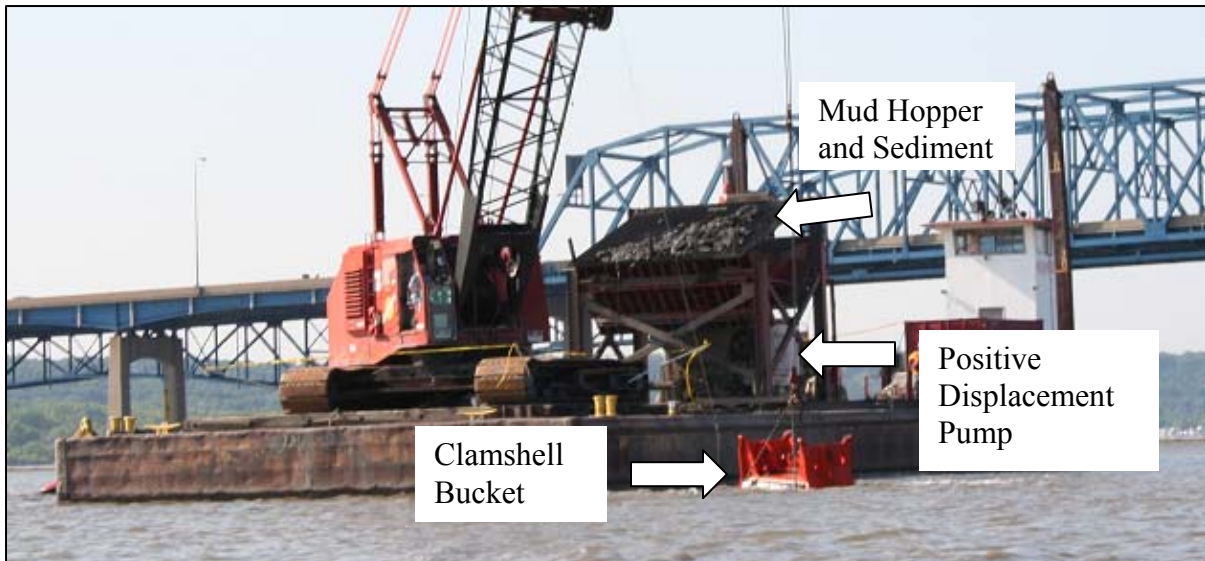


Figure 3.31. Barge Containing Major Equipment at Peoria Island

The geotextile tubes were filled with a 200 mm diameter HDPE pipe running from the positive displacement pump. A small amphibious trackhoe (Figure 3.32) was used to support and maneuver the pipe during construction. In Figure 3.32 the water was on the order of 0.6 m deep; anticipated water depths were 0.3 to 1.5 m during construction.



Figure 3.32. Amphibious Trackhoe Used at Peoria Island

The 4.6 m³ environmental clamshell bucket was said by construction personnel to be able to keep pace with the positive displacement pump. The rate of dredging varied within the project; no accurate assessment of pumping rate was available to the research team. The total quantity of geotextile tube fill material was on the order of 38,000(+) m³. Material quantities for dredging were obtained by surveying the channel.

The surface to support the geotextile tubes was required to be leveled and prepared to a relatively smooth condition. Three rows of tubes were placed side by side to create the wall with the ends of the tubes in the various rows staggered a minimum of 25% of their length. Butt joints were used and the height requirements for the joints were the same as in the remainder of the tube. The required fill height of the tubes had to be maintained for 30

days after filling. Figure 3.33 shows the outer rows of tubes in place with the third row yet to be placed. As seen, plastic pipe was used for alignment poles. A fourth row of geotextile tube was to be placed as an experimental section stacked between two of the three original tubes along a short portion of the island to investigate feasibility of increasing wall height.



Figure 3.33. Geotextile Tubes Placed at Peoria Island

Stone was placed outside portions of the geotextile tube wall where erosion was a potential concern. During construction a turbidity curtain (Figure 3.34) was used downstream of the island to minimize environmental impact on adjacent waters. Turbidity, moisture content, and specific gravity samples were taken on a routine basis during the project.



Figure 3.34. Turbidity Curtain Used During Peoria Island Construction

3.4.5 Peoria Island Construction Costs

The request for proposals estimated the bid price range between \$1,000,000 and \$5,000,000. The bid for geotextile tubes was to be conducted on the first 3,650 m and then over 3,650 m. Payment was for the materials, placement, and everything else associated with the geotextile tubes. The total bid price of *Peoria Island* was \$2,700,000. The cost of the geotextile tubes was on the order of \$65 to \$80 per meter, or \$325,000 to \$400,000 total cost.

The *Unified Soil Classification System (USCS)* was utilized to classify the soils as described in *ASTM D 2487*. Organics/volatiles content was determined according to *ASTM D 2974* using a muffle furnace at 750 C. Atterberg Limits were performed according to *ASTM D 4318*, and specific gravity of solids (G_s) was performed according to *ASTM D 854*. Soil pH was measured on two random soil samples of 10 g added to 20 ml of deionized water. The mixture was allowed to sit for 15 to 30 minutes, stirred to uniformity, and the pH measured using a hand held meter. Index property test results are provided in Table 3.4.

Table 3.4. Geotextile Tube Fill Index Property Test Results

	<i>Matagorda</i>	<i>Grand Isle</i>	<i>Peoria Island</i>
<i>LL</i>	---	---	52
<i>LL_{OD}</i>	---	---	43
<i>PL</i>	---	---	22
<i>PI</i>	---	---	30
<i>USCS Classification</i>	SW ^b	SP ^b	CH
Organics/Volatiles	---	---	4.1
pH ^a	8.83	9.45	7.97
G_s	---	---	2.63
<i>FM</i>	2.13 to 2.20	0.49 to 0.68	---
Fines (%)	12	8	82
Clay (%)	---	---	26
Silt (%)	---	---	56
Sand (%)	88	92	18
Activity	---	---	1.15

Note: the higher the fineness modulus (*FM*) the coarser the material.

a: pH of dried solids and the average of two tests.

b: Insufficient data for full classification; well graded (*SW*) and poorly graded (*SP*) sands could be determined.

3.6 Testing of Geotextile Tube Fill Materials with Cementitious Stabilization

Many of the practices used in this report were also used in *SERRI Report 70015-006*. Notable differences were in the methods used to process material and also in procedures used to create specimens. Key details pertaining to the geotextile tube site visits material is presented in the remainder of this section.

Cementitious material was added in terms of a dosage rate (*D*) defined as kg of cementitious material per cubic meter treated. Three combinations were tested. Specifically: 1) *Type III* portland cement from Holcim Artesia (*A T III*); 2) specialty grind portland cement from Holcim Artesia (*SCI*); and 3) a blend of 25% *Type I* portland cement and 75% ground granulated blast furnace slag taken from a project built by Hayward Baker (*SB-HB*).

Batch quantities for *Grand Isle* and the *Matagorda Ship Channel* sands were selected to provide reasonable proportions for flowable fill (aka controlled-low-strength material or *CLSM*) with high moisture contents. If sand were stabilized and used to fill a geotextile tube, creating a material resembling *CLSM* would be a practical solution as the material could be delivered with positive displacement pumps rather than a hydraulic dredge. Typical *CLSM* batch quantities presented in *SERRI Report 70015-006* were used as a reference, and blends with higher water to cement (*w/c*) or water to cementitious material (*w/cm*) ratios were created since delivery distances could be increased as a result of the additional water.

Sand, cementitious material, and water were mixed to uniformity with a hand drill and off the shelf attachment for *Grand Isle* and the *Matagorda Ship Channel*. A constant amount of sand and cementitious material was used per batch with the water varied to produce a moisture content during mixing ($w\%$ mixed) of 25 to 100% depending on the test. After mixing, all free water drained within 30 minutes of cement addition, which was monitored and used to calculate the dosage rate based on the moisture remaining. Specimens with less than 38% moisture did not lose any water. Once the free water was removed, unconfined compression (UC) specimens were produced. Flowable fill placed in a geotextile tube would experience water loss due to drainage through the geotextile provided the tube was not fully submerged. Water loss would also occur due to settling and be pushed out the tube once the tube were completely full. Excess water would not be kept within sand.

Peoria Island specimens were prepared by taking material at the in situ moisture condition and mixing cementitious material to uniformity. Mixing was performed with a hand drill and off the shelf attachment. The material was placed with a positive displacement pump during the site visit so the in situ condition was an appropriate reference.

A total of eighty-one UC specimens 76 mm diameter and 152 mm tall were tested; three replicates for each combination were tested after curing in a submerged condition with a porous stone at each end of the specimen at room temperature (approximately 20 C) for 72 or 168 hr. The test matrix for each project was developed considering the available sample size as they varied between projects. Eighteen, thirty, and thirty-three tests were performed on material from the *Matagorda Ship Channel*, *Grand Isle*, and *Peoria Island*, respectively. The loading rate was 2.29 mm/min.

3.7 Cementitiously Stabilized Fill Material Test Results

Table 3.5 contains maximum strain (ϵ_{max}), unit weight (γ_s), and percent solids ($TS\%$) test results. Both sands had higher strain to failure, higher unit weight, and higher percent solids than did the fine grained *Peoria Island* material. The large amount of water in the *Peoria Island* specimens could have resulted in the lower strain to failure.

Table 3.5. Site Visits Test Results Summary

Site	ϵ_{max} (%) ¹	γ_s (g/cm ³)	$TS\%$ (%) ²
<i>Matagorda Ship Channel</i>	3.0	1.89 to 1.94	69 to 73
<i>Grand Isle</i>	2.9	1.82 to 1.95	64 to 80
<i>Peoria Island</i>	1.9	1.56 to 1.74	59

1: Average maximum strain (ϵ_{max}) values reported

2: Total solids prior to cement addition

Figure 3.37 plots shear strength as a function of w/c ratio for the *Matagorda Ship Channel* and *Grand Isle* sands. Strength decreases with increased w/c ratio as expected. The data provided in Figure 3.37 could be used to estimate strength loss as water is added to improve the ability of the material to be pumped. Insufficient material was obtained to characterize pumpability in terms of a specific distance; multiple variables affect pump distances including the equipment used. *SERRI Report 70015-008* provides additional information related to pumping.

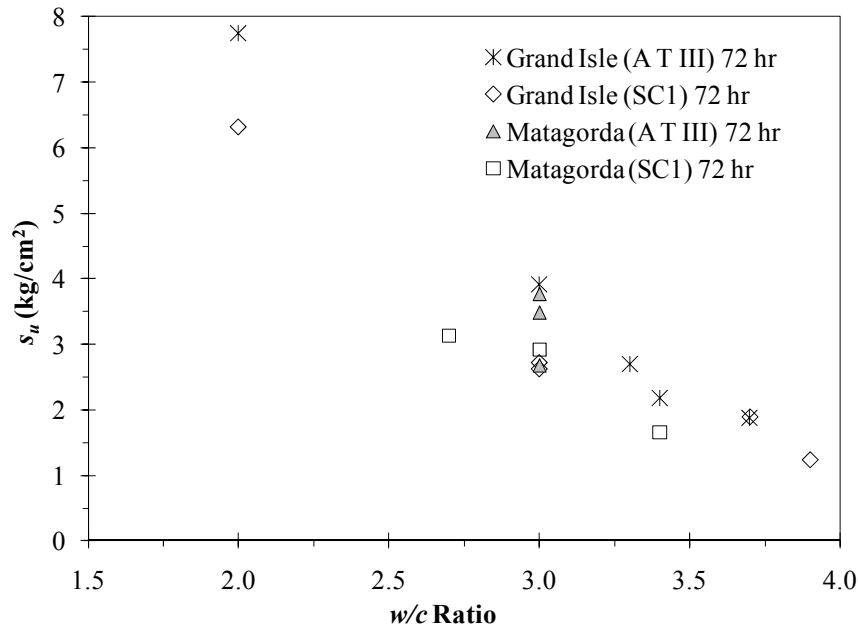


Figure 3.37. Shear Strength versus w/c Ratio for Sands

Figure 3.38 plots shear strength test results as a function of dosage rate. *A T III* out performed *SC1* in both sands, though by small amounts in some cases (Figure 3.38a). *SC1* and *A T III* strengths were practically identical for fine grained material from *Peoria Island*. Comparing *SC1* to *SB-HB* at 168 hr from *Peoria Island* showed *SC1* to be the superior performer at all three dosage rates (Figure 3.38b). At a 75 kg/m^3 dosage rate the performance difference was significant for this research as the portland cement produced a material of useable quality while *SB-HB* did not. At a dosage rate of 150 kg/m^3 *SC1* out performed *SB-HB* by 22% and at 250 kg/m^3 *SC1* had just over twice the strength of *SB-HB*.

At 72 hr, the shear strength of the fine grained material (Figure 3.38b) was 2.62 kg/cm^2 at D of 150 kg/m^3 . This level of shear strength was practically identical to the sand (Figure 3.38a) with *SC1* at the same test time and dosage rate. However, 6.32 kg/cm^2 shear strength was developed from a dosage rate of 174 with *SC1* at 72 hr with the sand while it took a dosage rate of 250 to produce a shear strength of 6.86 kg/cm^2 with the fine grained material. The data indicates that a fine grained material at high moisture can produce early shear strength comparable to sand, though it also appears that sand will gain strength and produce higher shear strength at longer cure times.

Figure 3.39 plots elastic modulus test results as a function of shear strength. The trendlines are for all data plotted as one series. Multiple series are plotted on the figures for clarity. The *Grand Isle* and *Matagorda Ship Channel* sands (Figure 3.39a) were reasonably well correlated to shear strength provided the equation contained an intercept and a slope; there was no correlation with only an intercept. A specimen with zero shear strength should have a corresponding modulus of zero indicating the shear strength to modulus relationship of these materials is probably non-linear as the intercept was a considerable value of 15 MPa. It would be expected that at low shear strengths a small increase in shear strength would produce a relatively large increase in modulus. The data in Figure 3.39a should not be used to predict the modulus of sands below 1 kg/cm^2 .

Figure 3.39b plots the shear strength to modulus relationship for the fine grained material obtained from *Peoria Island*. The correlation is strong with the intercept set to zero indicating the relationship is linear over a wide range of shear strengths. The modulus of the fine grained *Peoria Island* material is considerably higher than the *Grand Isle* and *Matagorda Ship Channel* sands.

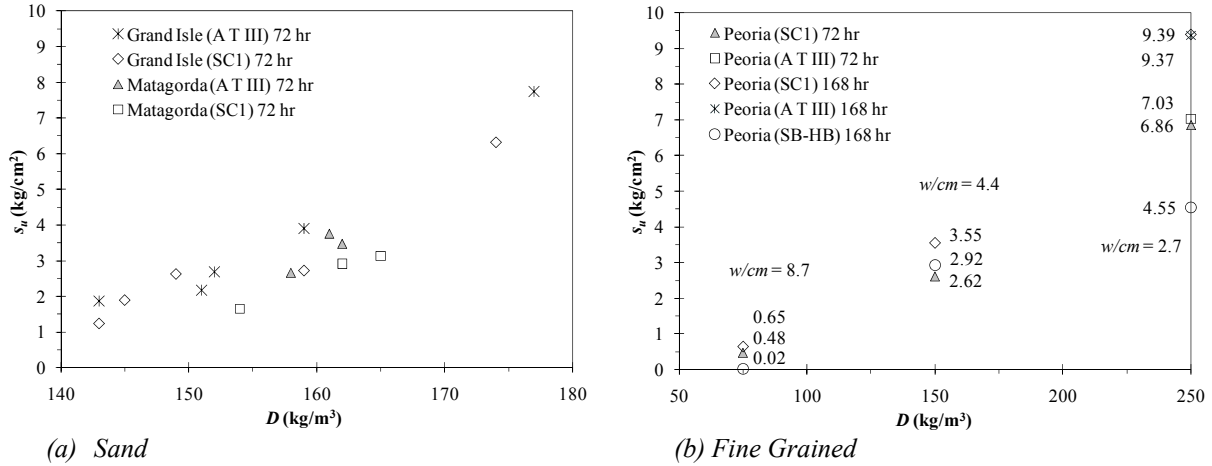


Figure 3.38. Shear Strength as a Function of Dosage Rate

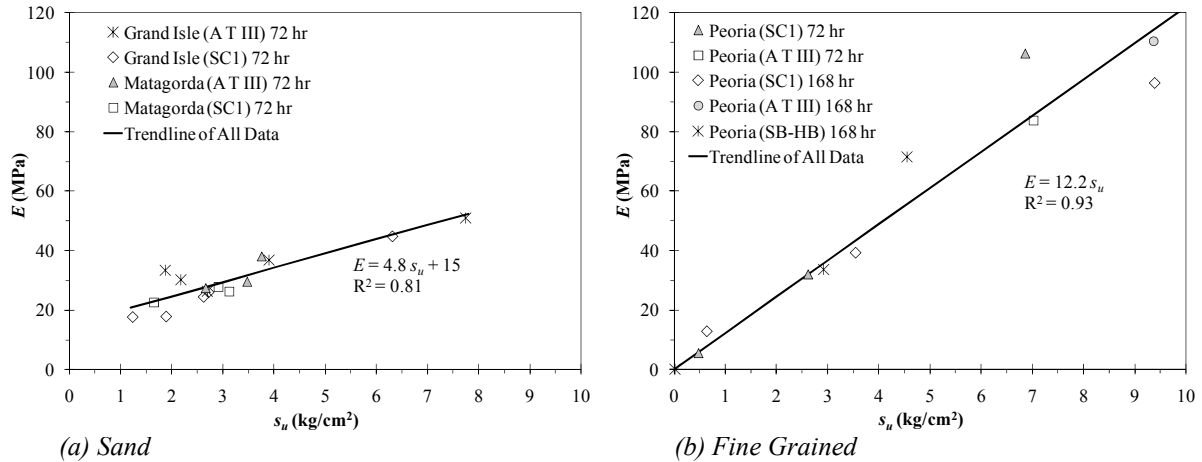


Figure 3.39. Elastic Modulus Test Results

3.8 Summary of Geotextile Tube Site Visits

During the site visits it was observed that multiple materials can be used to fill geotextile tubes (two different sands and river sediment were observed). Geotextiles with AOS values of 30 to 40 appear to be standard. Circumferences of 9.14 and 13.72 m were observed during the visits. Geotextile tubes with flat ends and multiple loops appear to be an efficient method of securing and connecting the units for construction of a water reservoir in a disaster environment.

Geotextile tube construction appears to be commonly performed as a subcontract by specialty contractors. Key construction procedures did not appear to be drastically different between projects with the notable exception of the positive displacement pump used at *Peoria Island*. The positive displacement pump was shown effective at *Peoria Island* and would be the preferred method of filling geotextile tubes in a disaster environment. Clogging of pipes was observed to be a potential concern when conventional dredging methods were used in conjunction with sand.

Only shallow water construction (e.g. construction in 1.5 m of water or less) was observed during the site visits. Deep water construction such as that of interest to this research would be expected to be more problematic. Based on information from the site visits, a construction rate of one 61 m of tube per 12 hr per construction crew is the best estimate available of a maximum construction time in a disaster. If one pro-rated the length of *Peoria Island* to the amount of geotextile tube needed to construct a reservoir as discussed later in this chapter, it would be allotted 45 calendar days as a routine project. When compared to the sites visited, emergency construction would require much higher production rates to provide significant value to recovery efforts.

For a water reservoir requiring 732 m of geotextile tube (12 tubes each 61 m long), the total cost would be approximately \$400,000 if the cost per unit length at *Peoria Island* (the site visit project closest to construction of a water reservoir in a disaster environment) were used at a reference. Provided a polyurea coating were desired, an added cost measured in thousands of dollars (estimated at less than \$50,000) would be incurred. Cementitious material for a 9.14 m circumference tube wall could be estimated at \$10,000 per percent of cement added by total weight. The upper end cost per unit length pro-rated into 2009 dollars based on literature review for routine construction would result in a reservoir cost on the order of \$600,000, or \$825 per m. The cost in a disaster environment would be higher than in routine construction, making the best estimate of cost based on available data of a single water reservoir with 732 m of geotextile to be \$1,000,000 (+).

Cementitious stabilization of fill materials was investigated. Test results indicated stabilization was viable whether the fill was sand or fine grained. Use of fine grained materials in a disaster environment would likely prove more useful as they would be plentiful in most areas.

CHAPTER 4 - TESTING CEMENT STABILIZED SLURRIES

4.1 Overview and Goals of Testing

The data presented in this chapter was used to evaluate the properties of cementitiously stabilized fine grained material that could be pumped into a geotextile tube for the purpose of creating a rapidly deployable structural element. Testing focused on shear strength and volume change. Shear strength was investigated with specimens cured in laboratory molds and tested in unconfined compression (*UC*) as well as specimens cured inside a geotextile tube and tested with hand held gages. Volume change associated with filling a geotextile tube with cement stabilized slurry was investigated by monitoring change in height of the geotextile tube with time. Shear strength is directly related to the stability of the wall, while volume change is critical in that it will dictate the final height of the wall and has many construction implications.

4.2 Materials Tested

One geotextile (*GT 500*), two soils (*Soil 1* and *Soil 3*), and one portland cement (*SC6*) were tested. *GT 500* properties were provided in Table 2.2 and were used to fabricate the small-scale geotextile tubes. Both soils and the portland cement were also tested in *SERRI Report 70015-006*, and the majority of the associated details of these materials were not repeated in this report. In summary, *Soil 1* was classified as *CL* to *CH*, and *Soil 3* was classified as *CH* to *OH*. *SC6* is a specialty grind *Type III* portland cement from Holcim's Theodore, AL facility. The product was produced by interrupting normal production at the full-scale facility and targeting normal *Type III* fineness and finish mill temperatures, while lowering the SO_3 content.

4.3 Test Methods

4.3.1 Unconfined Compression Testing

Unconfined compression testing was performed in the manner outlined in *SERRI Report 70015-006*. Protocol 2 was followed for the testing where two suites of tests were performed with *Soil 1* and two suites of tests were performed with *Soil 3*. Curing occurred at room temperature with the specimens submerged. Half of the testing was performed on specimens produced at a moisture content ($w\%$) of 233 ($TS\%$ of 30) and total cementitious content of 15; this combination is denoted (15, 233) hereafter. The other half of the testing was performed on specimens in a (5, 100) condition. The water to cement (w/c) ratios of (15, 233) and (5, 100) are 4.7 and 10, respectively. The purpose of the testing was to measure the undrained shear strength (s_u) and elastic modulus (E) over time. Thousands of additional data points are presented in *SERRI Report 70015-006* that encompasses a broader range of soils, cements, and test combinations.

4.3.2 Testing of Geotextile Tube Fill

Soil slurry was first blended to achieve 233% moisture, allowed to sit at a minimum overnight, re-mixed with a hand drill and off the shelf attachment, and then 15% SC6 portland cement by total soil slurry mass was added and mixed into the slurry using the same mixing technique. Approximately 41 kg of soil slurry was divided equally in three plastic buckets and the portland cement was introduced into each bucket and mixed. The total mass of portland cement was approximately 6 kg. The general procedure was the same as in *SERRI Report 70015-006*. The slurry was in a fluid like state even after addition of the portland cement.

A modified version of the *Geotube*® Dewatering Test (*GDT*) was used to test the stabilized slurries. The equipment and geotextile tube were not modified, rather they were used with a different test protocol. A small-scale geotextile tube was used to conduct the experiment, which has dimensions of approximately 53 cm by 53 cm and holds approximately 28,000 cm³. *GT 500* geotextile is used along with conventional seams. The small-scale tube was often referred to as a *pillow*.

Two types of tests were conducted: emerged (Figure 4.1a) and submerged (Figure 4.1b). The emerged test was to evaluate characteristics when the geotextile tube was completely out of the water, while the submerged tube was to evaluate characteristics when the tube was completely covered with water. Four tests were conducted emerged and one test was conducted submerged.

For both methods, the stand was leveled before testing, and the standpipe was secured to the *pillow*. Two dial gage stands and pieces of string were used to measure the change in height of the *pillow* over time. The lower string was leveled and positioned beneath the *pillow* to serve as the datum for measurements while the upper string was leveled and positioned at the top of the *pillow*. The distance measured between the two strings was considered to be the height of the *pillow*. Change in height is not equivalent to volume change on a percentage basis since the *pillow* is curved when filled with stabilized soil slurry.

All tests were performed at temperatures on the order of 24 C. One test was commenced outdoors (Figure 4.1a) and moved indoors 15 minutes after filling with slurry. The remaining tests were commenced and conducted indoors. Moving the first test indoors after 15 minutes would have no effect on test results as the stand only had to be lifted and carried a short distance.

The slurry mixed with cement was poured into the top of the standpipe (Figure 4.1c) while the standpipe and *pillow* were held slightly above the stand as the first bucket was being poured to ensure proper filling. Immediately after the *pillow* was filled, the initial height of the bag was measured and recorded using the string apparatus (Figure 4.1d). The total amount of material that could be placed into the *pillow* was recorded. All *Soil 1* tests had 47 kg in the *pillow*, while the *Soil 3* test had 43 kg in the *pillow* at the commencement of the dewatering phase.

After curing for 24 or 72 hours, the final height of the *pillow* was recorded and it was cut open. The *Pocket Geotester (Dial)* gage was used to record 20 readings on the top surface of the specimen (Figure 4.1e). The specimen was then flipped over and 20 more *Dial* gage readings were taken on the bottom surface (Figure 4.1f).



(a) *Emergent Test Set-Up*



(b) *Submerged Test Set-Up*



(c) *Slurry Entering Tube*



(d) *Measuring Initial Height*



(e) *Testing Top Surface Post Dewatering*



(f) *Bottom Surface After Testing*

Figure 4.1. Testing of Cement Stabilized Soil Placed in Geotextile Tube

4.4 Test Results

UC test results are plotted in Figure 4.2 and Figure 4.3. At (15, 233), *Soil 3* resulted in higher strengths, likely because it has a much higher liquid limit and can absorb more water than *Soil 1*. At (5, 100) *Soil 1* was stronger than *Soil 3*. To provide context to the strength of the materials tested, the categories described by Terzaghi et al. (1996) are

provided: *Very Soft* clay has an s_u of 0 to 0.13 kg/cm²; *Soft* clay has a s_u of 0.13 to 0.25 kg/cm²; *Medium* clay has an s_u of 0.25 to 0.5 kg/cm²; *Stiff* clay has an s_u of 0.5 to 1.0 kg/cm²; *Very Stiff* clay has an s_u of 1.0 to 2.0 kg/cm²; and *Hard* clay has an s_u in excess of 2.0 kg/cm². *Hard* clay is an excellent bearing material.

The modulus of the (5, 100) material was much higher than the (15, 233) material. Such a drastic difference in modulus would make (5, 100) slurry more appealing than (15, 233) slurry. Assuming placement costs were the same, a considerable savings would also be observed with (5, 100) slurry as much less cement would be required.

The (5, 100) mixture would be more desirable than the (15, 233) mixture unless a very fluid material was required for pumping. The (5, 100) materials investigated are both pumpable using positive displacement pumps according to equipment manufacturers. The distance will vary based on equipment, though *Soil 1* at (5, 100) was stated to be pumpable long distances (additional detail provided in *SERRI Report 70015-008*).

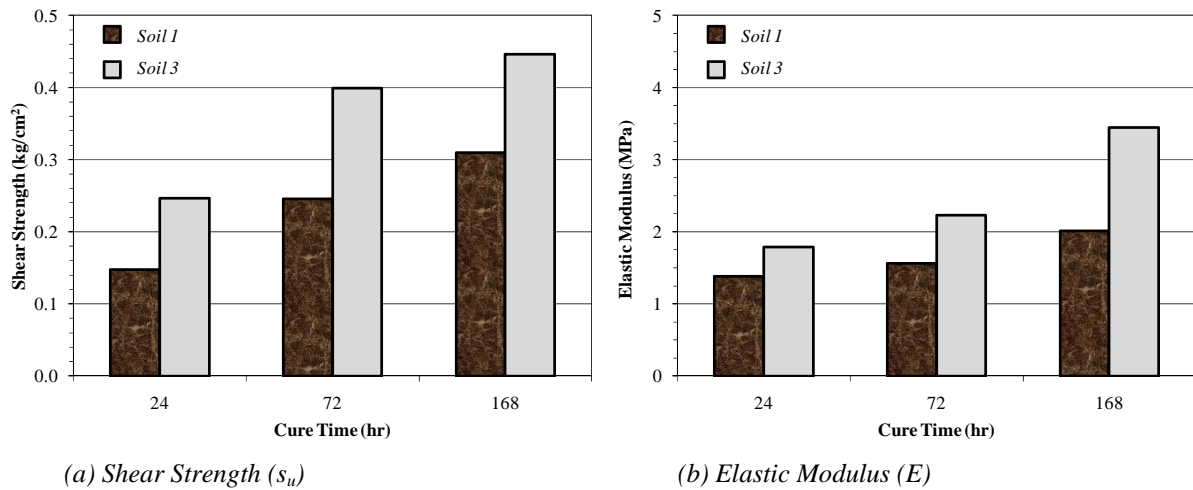


Figure 4.2. UC Test Results (15, 233)

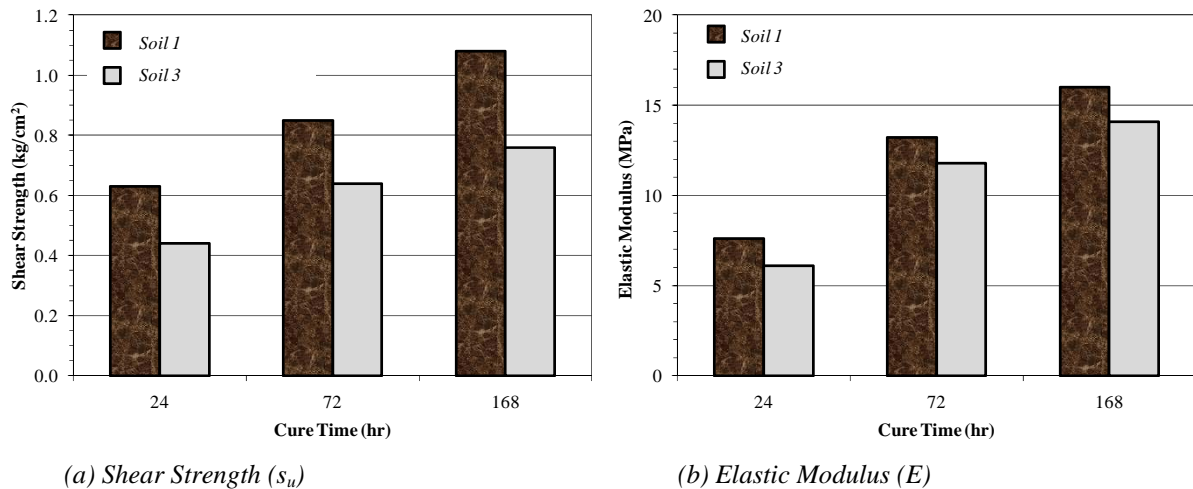


Figure 4.3. UC Test Results (5, 100)

Table 4.1 provides geotextile tube test results. Height change ranged from 10 to 24% and nearly all the height change occurred during the first four hours after placing the material into the tube. This amount of volume change is tolerable and would not cause significant problems during construction of a geotextile tube wall. Some of this volume change is believed to be release of entrapped air, while the rest would be water expelled through the geotextile. Construction sequencing could allow a wall to be built to a predictable design height with this material assuming settlement beneath the wall were properly addressed.

Table 4.1. Results of Modified GDT Tests Using Dial Gage

Soil	Type	Time (hr) ¹	s_u top (s_u bottom)		Height		
			Mean (kg/cm ²)	cov (%)	Initial (cm)	Final (cm)	Change (%)
1	Submerged	24	0.6(1.0)	17(11)	25.4	22.9	10
1	Emerged	24	1.3(2.9)	17(4)	25.4	20.3	20
1	Emerged	24	1.8(2.7)	14(11)	26.7	22.2	17
1	Emerged	72	2.5(3.0)	8(0)	25.4	22.9	10
3	Emerged	24	2.6(3.0)	14(0)	21.6	16.5	24

1: Temperature-Time Factor at 24 hr was 575 C-hr and at 72 hr was 1,725 C-hr.

Table 4.1 contains strength results measured on top and bottom of the material cured in a geotextile tube. Shear strength measured by the *Dial* gage was shown to be greater than *UC* measured strength in *SERRI Report 70015-006*. The average extent of over-prediction for (15, 233) was 2.24 times in *Soil 1* and 1.93 times in *Soil 3*. When adjusted to correspond to *UC* measured strength, the submerged data from *Soil 1* exceeded the data in Figure 4.2. The adjusted submerged strength at the top and bottom of the material in the geotextile tube was 0.27 kg/cm² and 0.45 kg/cm², respectively. These values exceed Figure 4.2 data and indicate submerged *UC* testing may be conservative in representing the shear strength of material within a geotextile tube when the tube is submerged.

Two 24 hr tests were performed with *Soil 1* with the tube emerged. The data was reasonably repeatable. Shear strength increased by a factor of 2.5 to 3 when the material was allowed to drain. It is likely that part of the geotextile tube wall would be out of the water, and this data shows it would be much stronger than the submerged portion of the tube.

The fourth *Soil 1* test was at 72 hr, and the strength on top increased approximately 60% with respect to the average 24 hr strength in the same conditions. The bottom readings were maximum gage values and have no quantifiable meaning. At 24 hr, *Soil 3* was nearly identical to *Soil 1* at 72 hr as evidenced by the data collected in the fifth and final test.

The data collected indicates that a high moisture content cement stabilized slurry has potential for rapid construction of a wall when pumped into geotextile tubes. The data presented in this chapter will be used as a reference for analysis and design procedures provided in the next chapter. The material tested in this chapter is intended to represent a relatively easily to achieve material that is easily pumped. Much stronger fill materials could be produced with the same amount of cement by lowering the moisture content of the soil. The data collected is probably at cooler temperatures than will be encountered in a hurricane response environment, indicating that strength gain in the field will likely occur faster than that shown in this section. Additionally, the large mass in the field will produce much more heat than the laboratory specimens, which will allow the mass to gain strength at a faster rate.

CHAPTER 5 - ANALYSIS AND DESIGN OF GEOTEXTILE AND GEOMEMBRANE TUBE WALLS

5.1 Overview of Analysis and Design

The analysis and design procedures presented in this chapter are intended to establish feasibility of geotextile and geomembrane tube use in construction of walls in a disaster area to support lateral water forces. The problem is approached from a fundamental perspective and does not employ techniques capable of optimizing performance prediction. The decision to approach the problem from a fundamental perspective was made considering test data presented in Chapter 4 and literature review presented later in this chapter.

Failure mechanisms that would initiate with the foundation were not considered in any detail in this analysis; i.e. consolidation, seepage, and bearing capacity. Key structural considerations were failure of the fill materials and frictional resistance of the configuration. Multiple designs were investigated where water storage volume was a key consideration. Geotextile tube designs focused on stacked configurations as they are more complicated than using a single tube as a wall. Use of a single tube should be employed if water depths are shallow enough where this approach is feasible.

5.2 Literature Review Pertaining to Analysis and Design

5.2.1 Geotextile Tubes Literature Review

In the 1990's the *USACE* increased use of geotextile tubes. However, they found little design and construction guidance. Custom made geotextile tubes were initiated in the early 1990's after two decades of varying levels of geotextile use in wetlands. The *USACE* sponsored a workshop in August of 1995 in Galveston, TX. There were fifty-five attendees from multiple *USACE* offices, dredging contractors, geotextile tube fabricators, consultants, port directors, and professors (Davis and Landin 1997). The objective was to assimilate the state of knowledge in geotextile tube design and construction. Design related issues are presented in this chapter while construction related issues are provided in Chapter 6.

Multiple causes were listed in the workshop for failure of geotextile tubes: overturning, sliding, deformation due to local scour, and multiple types of wave forces. It was stated that: 1) sliding resistance could be determined by evaluating the normal force produced by gravity using the friction factor between the tube and supporting medium; 2) little to no research had been conducted in the US or abroad on the forces on geotextile tubes; 3) forces on geotextile tubes had never been measured in the field or in the laboratory; and 4) stacking geotextile tubes was not well established and that tying stacked tubes together could be beneficial.

The workshop summary concluded that geotextile tubes were only useful as longer term breakwaters when filled with sand, and that use with fine grained materials was primarily limited to contaminants storage and isolation. Recently, the *USACE* put forth a set of guidelines for contract specifications using geotextile tubes (Jones et al. 2006). Therein, fill material was generally required to have less than 15% fines; no stabilization of fill materials was mentioned.

Fine grained materials in absence of chemical treatment can take considerable time to develop any shear strength (e.g. 1 week as in Shin and Oh 2003). Design and placement were topics of discussion during the workshop and were identified as areas needing additional study. The workshop summary concluded geotextile tube technology was in the experimental stage in most cases. Unanswered questions included calculation of stresses on geotextile tubes in a routine manner and availability of geotextile tubes from various manufacturers.

5.2.1.1 Geotextile Tube Internal Stability

Howard et al. (2009) describes internal stability analysis and design methods with corresponding commentary. Internal stability methodologies and software appear to be fairly well developed. Cantre (2002) indicated the methods used to calculate the loads and geometry during the filling process of a geotextile tube lead to comparable results.

SOFTWIN was developed for simulation of fluid filled tubes several years ago (a program called *Geosynthetics Applications Program-GAP* was developed by same individual). The program cannot simulate different fluid depths on different sides of the tube. The primary purpose of the program was for jetties with a given depth of water. The required inputs of the program are: 1) bulk specific gravity of the slurry; and 2) overall factor of safety for seams. The program works by calculating any four of the following six parameters when the other two are selected: 1) tensile strength of the tube fabric and seams; 2) excess pressure above the top of the tube; 3) tube circumference; 4) height of inflated tube; 5) ratio of tube area to tube volume; and 6) filled tube volume.

Carroll (1994a) refers to the encapsulating geotextile tube as an isotensile surface. Essentially, the two fabric stress components are equal to each other at any point on the surface. A constant fabric stress results in no shear stress at any point on the surface. A near isotensile pancake bag (as it is referred to by Carroll 1994a) can be fabricated by sewing together five pieces of fabric, while closer approximations to the theoretical isotensile surface can be achieved by using more than five fabric pieces. Carroll (1994a) investigated the possibility of a symmetrical pancake-shaped design that would have equal tensile stresses at any point throughout the surface (i.e. no shear stress).

A numerical derivation and solution of an isotensile surface was presented in the paper, which was developed into a computer program. Four inputs are part of the program: 1) specific gravity of internal fluid; 2) specific gravity of external fluid; 3) tensile stress throughout pancake surface; and 4) pressure differential at top of surface. The outputs of the program include diameter of flat base, height, and volume of the isotensile surface.

Carroll (1994b) provides theoretical computations for non-circular cylindrical members filled with one or more denser fluids that is partially to fully submerged in air or water. The tensile stress in the circumferential direction is constant while it varies in the axial direction. The analysis focuses on techniques where the fabric cylinder is attached to rigid flat ends, but curved fabric surfaces were also considered.

According to Carroll (1994b) the simplest (and yet practically significant) mathematical form of geotextile tubes is a non-circular cross section that is constant along its length. The solution derived incorporates no internal shear stress in the circumferential direction, and it has shear stress acting in all other directions within the surface. A computer

program was written that incorporates specific gravity, tensile stress in the geotextile, and top pressure differential.

Leshchinsky et al. (1996) makes the following assumptions in derivation of formulations used to analyze filled geotextile tubes:

- The problem is plane strain, neglects pressure loss due to tube drainage, and sets the pumping pressure at the inlet as the basis for analysis.
- The geotextile tube material itself is thin, flexible, and has negligible weight.
- The fill material is a fluid slurry that causes internal hydrostatic stresses.
- No shear stress is developed between the slurry and the geotextile tube, making the tensile force in the tube constant along its circumference.
- The geotextile tube rests on a flat base.

The solution was developed so that once a specified circumference was entered, two of the three following parameters could be entered and the third parameter determined: 1) circumferential tension in the geotextile tube; 2) fill height of the geotextile tube; and 3) inlet pumping pressure. The solution was said to be tedious and required a trial and error procedure. A computer program (*GeoCoPS*) was developed by Leshchinsky and Leshchinsky (1996) to perform the aforementioned calculations. *GeoCoPS* also computes the axial tension in the geotextile tube, which is usually less than the circumferential tension. The program was verified using multiple studies.

Leshchinsky et al. (1996) presents analysis that calculates stress and geometry of a single geotextile tube. The most critical factor needed to assure successful construction was said to be the pumping pressure. Slight pumping pressure increases can significantly increase the stress in the geotextile tube. Additionally, pressure increase beyond a threshold level does not provide significant increase in storage capacity. If the fill material is sand, inlet ports on geotextile tubes were recommended to be placed close together (10 m was given as an example) to allow uniform filling of the tubes. If the fill material is a clay slurry, inlet ports on geotextile tubes were recommended to be placed far apart (up to 150 m was given as an example).

A parametric study with *GeoCoPS* by Leshchinsky et al. (1996) was performed on a 9 m circumference tube containing a slurry with a specific gravity of 1.2. In general, the calculations were performed with air surrounding the geotextile tube. The maximum theoretical predicted height for this condition was 2.87 m. At a tensile stress of 14.6 kN/m, 63% of the aforementioned 2.87 m can be achieved (i.e. 1.8 m) where an infinite tensile stress would be required to achieve 2.87 m. Tensile stress of 87.9 kN/m is required to produce 89% of the maximum height, or 2.6 m. The cross sectional area changes little as the height changes. A design height of 0.9 m can be achieved with no pumping pressure, which results in tensile stress of 2.6 kN/m. To achieve a design height of 2.7 m, the pumping pressure is 122.8 kPa, which results in a tensile stress of 190 kN/m. Height increase beyond a pumping pressure of 35 kPa was insignificant; 87% of the maximum height was achieved at this pressure. Leshchinsky et al. (1996) noted that dredge line pressures can easily be 300 kPa and that this condition at the inlet could cause major problems if not properly controlled. The parametric study also showed that submerging the tube in water increased the height of a 9 m tube when all other parameters were held constant, indicating a reduced stress when submerged. Leshchinsky et al. (1996) recommended the working stress in the geotextile tube

be at least a factor of 3.9 below the ultimate strength of the geotextile as measured by *ASTM D 4545-94* (Wide-Width Strip Method).

Geotube® Simulator™ (*TenCate™* propriety software) provides a cross section for a given circumference tube filled to a given height and the associated factor of safety against tube rupture. The primary use of the software is to design individual geotextile tubes for filling and installation. The software requires five inputs and produces seven outputs. Inputs are: 1) water level (H_w), 2) tube height (h), 3) geotextile tube circumference (C), 4) specific gravity of fill material (SG_{int}), and 5) geotextile type. Outputs are: 1) maximum tensile force in geotextile (T), 2) tube base contact width (B), 3) filled width (W), 4) cross sectional area (A), 5) volume per unit length (V), 6) pressure at base of tube (P_{base}), and 7) factor of safety (FS). The software always assumes an elliptical shape, calculations can be performed for emerged and submerged conditions (no partially submerged condition), and only one tube can be evaluated at a time. The software does not calculate the height or shape of a tube placed on top of other tubes.

5.2.1.2 Geotextile Tube External Stability

Howard et al. (2009) describes external stability analysis and design methods with corresponding commentary. It was emphasized that there is a significant lack of well documented design procedures for geotextile tubes in the literature. External hydrodynamical stability is the parameter that appears to be the least understood for this application in terms of geotextile tube stability in structural applications. Rigid bodies are often assumed, friction between geotextile tubes is often neglected in analysis (e.g. Plaut and Klusman 1999), and two dimensional (2-D) models are common.

Tyagi and Mandal (2004) performed 2-D equilibrium and vibration analysis with air and water as internal geotextile materials with three different foundation types. The geotextile was assumed to be an inextensible membrane, bending resistance was neglected, and friction between the tube and supporting surface was neglected. They reported anchored tubes provided the same deformation patterns as freestanding tubes, but with additional stability.

Cantre (2002) used Abaqus to simulate stacking of geosynthetic tubes. Three rows of tubes were stacked in a pyramid with three tubes in the bottom row. Soil inside the tubes was modeled using the Mohr-Coulomb model and a friction coefficient of 0.5 was taken between the geotextile tube and foundation. A linear distributed load was applied to represent stacked tubes and the load was not transferred by friction. The stack of tubes was assumed to be symmetrical.

The geotextile shell and fill of Cantre (2002) were modeled with eight node plane strain CPE8 solid elements, and B22 beam elements were used to calculate circumferential tension in the geotextile and were fixed at the solid elements of the geotextile. This approach made for a relatively rigid tube, which would not occur in actual geotextile tubes.

Miki et al. (1996) tested two woven geotextile tube configurations: 1) 51 kN/m strength 157 cm circumference tube filled to a center depth of 19 cm and loaded with a 20 cm diameter plate; 2) 72 kN/m strength 565 cm circumference tube filled to a center depth of 75 cm and loaded with a 50 cm diameter plate. An 84% clay soil with a liquid limit of 233, plastic limit of 56, and loss on ignition of 21% was placed inside the tubes. An unspecified hardening agent was used to adjust cohesion. Plate bearing testing of the 157 cm

circumference resulted in a reaction value of 400 kN/m³ with no cohesion in the fill and in 3,000 kN/m³ when the fill had 0.10 kg/cm² cohesion. The pressure deflection relationship with a 0.08 kg/cm² fill was approximately linear up to a deflection of approximately 1/6th of the 19 cm depth. Plate bearing testing of the 565 cm circumference tube resulted in a reaction value of 200 kN/m³ with no cohesion in the fill and in 1,150 kN/m³ when the fill had 0.08 kg/cm² cohesion. The pressure deflection relationship with a 0.08 kg/cm² fill was approximately linear up to a deflection of approximately 1/3rd of the 75 cm depth.

A larger experiment was also conducted by Miki et al. (1996) where three 8 m circumference tubes were tied together containing a 0.04 kg/cm² fill material. Two additional tubes were placed onto the bottom row of tubes and no stability problems resulted. The bottom row of tubes was 33 to 48 cm tall, and the second row of tubes added approximately 100 cm to the height of the wall.

The most compelling information obtained during review of literature related to external stability of stacked geotextile tubes was a project recently constructed in Morocco using *Geotube*[®] units. A temporary dam was constructed using three 15.7 m circumference tubes made of GC-1000 geotextile (Table 2.1) having a 160 kN/m seam strength stacked in a pyramid with one tube resting in the middle of the bottom two tubes. The width of each bottom tube was 6.25 m (12.5 m total wall width) and the tubes were each pumped to a height of 3 m making the wall on the order of 6 m tall and 12.5 m wide at the base. The water depth was 3.6 to 4.2 m, or 60 to 70% of the wall height. The tubes were filled with sand and the only stability calculations performed were to ensure the wall would not slide. This application demonstrates feasibility of the concept being investigated, and construction details of this project are provided in Chapter 6.

5.2.2 Geomembrane Tubes Literature Review

Some geomembrane tube failures occurred due to slipping and rolling during early uses of the technology. Turk and Torrey (1993) documented use of water-filled barriers during the 1993 Midwest Flood. Single tubes being manufactured at the time were effective at resisting high water levels for a period of time, but ultimately deflected to an extent they were not effective in resisting headwaters. The barriers used were designed for containing potable water, and were able to resist headwaters up to approximately 2/3 of the height of the dam. Enis (1997) investigated the use of geomembrane tubes for use in flood control for protecting buildings in Mississippi. The barriers were successful in holding back sediment contaminated water; barriers were mostly 0.9 m high, with occasional heights of 1.8 m.

5.2.2.1 Geomembrane Tube Internal Stability

A single water filled tube cannot support lateral forces. Two tubes inside an outer *master tube* is a stable configuration due to internal friction forces provided the outer geotextile has sufficient tensile strength to carry the induced forces. Friction between the walls of the inner tubes maintain internal stability. Visualization of this behavior can be seen in Figure 2.6, and Landis (2000) provides additional description of the behavior. Onsite water is pumped into the inner tubes. The geotextile *master tube* confines the inner geomembrane tubes while counter friction resulting largely from hydraulic pressure maintains equilibrium of the inner tubes which are sometimes referred to as bladders.

Seay and Plaut (1998) present three-dimensional modeling using Abaqus of a single tube resting on a tensionless elastic foundation. The slurry is modeled as a fluid yet the shapes did not resemble actual geosynthetic tubes. The work dealt with filling of a tube and the resulting three-dimensional shape. The tube was assumed to be linearly elastic with a 3 mm thickness, E of 7.035 GPa, Poisson's Ratio of 0.45, and density of 75 kg/m³. Density of fill was 1.5 g/cm³. Shell elements (S4R) were used for the tube with four elements traversing the curved edge.

5.2.2.2 Geomembrane Tube External Stability

Broadly speaking, two types of analysis were identified during literature review. The first type uses numerical analysis to evaluate stability. Plaut and Suherman (1998) performed a two-dimensional analysis to evaluate the shape of a single geosynthetic tube. Huong et al. (2002) used *Fast Lagrangian Analysis of Continua (FLAC[®])* to investigate a single geomembrane tube on a soil foundation with a downstream wedge.

Plaut and Klusman (1999) consider stacked geosynthetic tubes containing hydrostatic dense fluid resting on a deformable foundation. The tube is considered to be an inextensible membrane. Friction was neglected between the tubes and the foundation interface. Two configurations were considered: 1) two tubes stacked vertically onto each other; and 2) three tubes in a pyramid. When stacked in a pyramid, rigid blocks were anchored to the ground to prevent sliding, and external water was applied to the pyramid configuration. External water levels tend to increase the total height, yet decrease tension in the bottom tube impinged by the water while having little effect on the tensile stresses in the other tubes.

Kim et al. (2004) investigated attachment of an apron on the water side of a geomembrane tube to provide additional friction resistance and inhibit large deformations. The authors performed experimental and numerical investigation and found the apron to be very effective. An apron with sufficient length was observed to allow a single geomembrane tube to restrain external water up to the deformed height of the tube.

Kim et al. (2005) states stacked geomembrane tubes have been applied in place of sandbags for flood control. The authors investigated the geomembrane tubes stacked in a pyramid and strapped together via large scale experiments and two-dimensional numerical analysis using *FLAC[®]*. Failure mechanisms that were noted were under seepage and piping of the soil foundation, rolling of the individual tubes, and sliding of all tubes due to increases in headwater.

Experiments were performed by Kim et al. (2005) in a 3.25 m wide by 1.22 m deep box with poorly graded compacted sand with silt that was 50 cm deep. Three tubes with 1.49 m circumference were used in conjunction with discretely spaced straps (strapping system posed difficulty during testing). When filled the dam was 65 cm tall and 115 cm wide. Motion was observed at headwater depths of 25 cm and sliding failure occurred at headwater depths of 43 cm; i.e. maximum headwater that could be resisted was approximately 2/3 the height of the dam. *FLAC[®]* modeling was able to provide good agreement with experimental results.

Kim et al. (2005a) studied multi-chambered geomembrane tubes of two types: 1) perforated internal baffle (i.e. diaphragm); and 2) two tubes surrounded by outer tube (i.e. sleeve). The sleeved dam was tested in a 3.25 m wide and 1.22 m deep box over poorly graded sand. The inner tubes were 1.44 m circumference, which were tested in conjunction

with 2.31 m and 2.46 m sleeves. The height of the dam was 35 cm, and failure of the 2.31 m and 2.46 m dams occurred at headwater levels of 26 cm and 29 cm, respectively (74 to 83% of the height of the dam). Failure occurred relatively quickly at the critical headwater level.

FLAC[®] predicted behavior of the 2.31 m circumference reasonably well, but predicted about half the headwater at failure that was measured in the experiments. Peak versus residual friction angles were attributed to the differences. *FLAC*[®] predicted critical headwater levels between 80 to 92% of the dam height. Behaviors of the two configurations were reported to be different (sleeved dam was governed by interfacial characteristics of tubes and diaphragm dam relied on interfacial characteristics with the foundation).

The second type of analysis identified in literature takes a fundamental approach to stability calculations rather than using numerical techniques. Rigid body equilibrium calculations for a tipping condition were performed by Landis (2000) and the results are shown in Eq. 5.1 and 5.2. To derive Eq. 5.1 one must assume that the geomembrane tube is the shape of a pyramid with maximum height (h) with a base width (W). If a rectangular shape is assumed, the 0.82 constant reduces to 0.58. Landis (2000) also performed rigid body sliding calculations where a smooth flat surface was assumed, and the interior tubes were assumed to be rectangular when filled. Pore water pressure was not considered, which could be problematic for some foundations. Eq. 5.3 is the final form of the equation.

$$W > 0.82h \quad (5.1)$$

$$FS_{Tip} = \frac{W}{(0.82)H_w} \quad (5.2)$$

$$\omega_{p-min} = \frac{H_w}{2W} \quad (5.3)$$

Where,

h = height of geomembrane tube

FS_{Tip} = factor of safety against tipping failure

W = inflated width of geomembrane tube

H_w = height of water being resisted

ω_{p-min} = minimum static friction coefficient that will prevent sliding

Table 5.1 shows inflated dimensions of commercially available *AquaDam*[®] units, with corresponding heights of water (H_w) that can be controlled with respect to tipping for a range of factors of safety (FS_{Tip}) calculated according to Eq. 5.2. Therein, a single *AquaDam*[®] unit with a 9.76 m width is capable of supporting the maximum depth of water considered in this research (3.05 m) with a FS_{Tip} of 3.9 according to Eq. 5.2. Provided the worst case scenario of the depth of water (H_w) being equal to the inflated height (H) of the 9.76 m wide unit, the FS_{Tip} remains at 2.4 to 2.9 for the tube heights displayed in Table 5.1.

With a constant FS_{Tip} , the minimum required coefficient of friction is constant regardless of the water depth according to Eq. 5.2 and 5.3. The minimum coefficient of friction is obtained by combining Eq. 5.2 and 5.3 which results in Eq. 5.4. The analysis conducted does not consider stacked geomembrane tubes, rather considers a single

geomembrane tube used to construct the wall. Pore water pressure was not considered in Eq. 5.3 and the shape of the wall differed in Eq. 5.1 and Eq. 5.3.

$$\omega_{p-\min} = \frac{0.61}{FS_{Tip}} \quad (5.4)$$

The values of $\omega_{p-\min}$ for the three cases considered in Table 5.1 are 0.20, 0.15, and 0.12 for FS_{Tip} values of 3, 4, and 5, respectively. These values when represented as friction angles are 11.3, 8.5, and 6.8°, respectively. Any friction coefficient exceeding these numbers will prevent sliding provided no pore water pressure is present and the conditions represented by Eqs. 5.1 through 5.3 are satisfied.

Biggar and Masala (1998) indicated the factor of safety against overturning for a flexible structure such as a water filled geomembrane tube was very difficult to determine. The authors also provided equations to evaluate the factor of safety against sliding and overturning. As a general rule, the height of restrained water was taken to be on the order of 80% of the height of the structure. Eq. 5.5 provides the factor of safety against sliding (pore water pressure was considered) and Eq. 5.6 provides the factor of safety against overturning provided by Biggar and Masala (1998).

Table 5.1. Water Depths to be Controlled with *AquaDam*® Units per Eq. 5.2

<i>h</i> (m)	<i>W</i> (m)	<i>H_w</i> (<i>FS</i> _{<i>Tip</i>} of 3)	<i>H_w</i> (<i>FS</i> _{<i>Tip</i>} of 4)	<i>H_w</i> (<i>FS</i> _{<i>Tip</i>} of 5)
0.30	0.61	0.25	0.19	0.15
0.46	0.91	0.37	0.28	0.22
0.61	1.22	0.50	0.37	0.30
0.91	2.13	0.87	0.65	0.52
1.22	2.90	1.18	0.88	0.71
1.52	3.35	1.36	1.02	0.82
1.83	4.27	1.74	1.30	1.04
2.44	5.49	2.23	1.67	1.34
3.05	6.71	2.73	2.05	1.64
3.66	7.32	2.98	2.23	1.79
4.88	9.76	3.97	2.98	2.38

Note: The values shown are for flat surfaces. Large slopes should be taken into consideration.

$$FS_{slide} = \frac{[F_{V-T} - F_{V-W}]\tan(\delta_p)}{0.5\gamma_w H_w^2} \quad (5.5)$$

$$FS_{Tip} = \frac{3 \frac{h}{H_w} \left(\frac{\gamma}{\gamma_w} \right)}{\left(\frac{H_w}{W} \right)^2 + 2 \left(\frac{W}{W+L} \right)} \quad (5.6)$$

Where,

FS_{slide} = factor of safety against sliding failure

F_{V-T} = vertical downward force induced by geomembrane tube

F_{V-W} = vertical upward force induced by pore water pressure

δ_p = peak angle of friction

γ = unit weight of structure

γ_w = unit weight of water

H_w = height of water being resisted when water is present on one side of the tube

FS_{Tip} = factor of safety against tipping failure

h = geomembrane tube height

W = filled width of geotextile or geomembrane tube

L = length of impermeable blanket (if used)

For FS_{slide} equal to 1, the peak friction angle required to maintain equilibrium ranged from 8 to 13° for a variety of products and manufacturers. For FS_{slide} equal to 1.5, the peak friction angle was from 11 to 19°.

In current practice, a static water height on the order of 75% of the height of a fully inflated geomembrane tube seems practical. The recommendation for *WIPP*TM and *Aqua Barrier*TM units is not to exceed 75% of the properly filed height of the units. Baffled *AquaDam*[®] units were developed using the same philosophy as the closed-ended and double closed-ended units. They have been tested with water depths near the dam height (very little freeboard). At present the manufacturer recommends them for the same water depths (on the order of 75% of the height of the dam) as closed-ended and double closed-ended products though they would likely provide additional factors of safety for a given water depth.

5.2.3 Interface Testing

ASTM D 5321 is a test method for determining the coefficient of friction for geosynthetics in contact with each other or for geosynthetics in contact with other materials. The method defines the slope of the limiting values of shear stress plotted versus applied normal stress as the coefficient of friction (ω_p). Eq. 5.7 and Eq. 5.8 define the relationship between the peak angle of friction (δ_p) and ω_p . Similar relationships could also be developed for other friction angles (e.g. residual friction or δ_r). In cases where the slope of the shear stress versus applied normal stress does not pass through zero, an adhesion intercept is reported; c_{a-p} for the peak condition and c_{a-r} for the residual condition.

$$\delta_p = \tan^{-1}(\omega_p) \quad (5.7)$$

$$\omega_p = \tan(\delta_p) \quad (5.8)$$

Kim et al. (2004) and Kim et al. (2005) performed interface testing according to *ASTM D 5321* except a 10 cm square test box was utilized. Test results are provided in Table 5.2. Shearing occurred at 0.9 mm/min to displacements of 1.0 to 1.3 cm. Kim et al. (2004) noted the materials were immediately inundated and then sheared after placement into the test box, while Kim et al. (2005) made no mention of whether the test specimens were

inundated during testing. Kim et al. (2004) noted sand was placed below the *PVC Tube* during testing, while four straps were attached to a plywood block for testing of *Strap* to *PVC Tube* interface properties. *PVC Tube* to *PVC Tube* testing incorporated normal weights of 45 to 260 N, while *Strap* to *PVC Tube* testing incorporated 145 to 260 N normal weights. Residual displacement was reached at approximately 0.4 cm deformation. The high value of peak friction (54°) between two tube sections was noted to be likely due to interlocking of the polyester reinforcing strands since the material was textured.

Table 5.2. Interface Friction Test Data

Source	Interface Material Description	c_{a-p} (kPa)	δ_p ($^\circ$)	c_{a-r} (kPa)	δ_r ($^\circ$)
Kim et al. (2004)	Sand: 6% fines, D_{60} of 0.16 mm, D_{10} of 0.08 mm, 15.5 kN/m ³ test density, 15% compaction moisture	0.00	34	0.00	32
	Monofilament: woven polypropylene (AOS of 0.21 mm)	1.24	17	0.00	17
	Silty Sand: 48% fines, 17.9 kN/m ³ test density, 12.5% OMC	2.76	32	0.00	32
	Silt: 77% fines, LL of 51, PI of 16, 17.9 kN/m ³ test density, 25.0% OMC	3.45	30	0.00	27
	Grass: Healthy fine-bladed <i>festuca sp.</i>	0.00	22	0.00	22
Kim et al. (2005)	Strap: Woven nylon 2.5 cm wide and 1.6 mm thick	---	25	---	18
	PVC Tube: PVC with 7.5 parallel interwoven reinforcement strands per cm (0.5 mm) (610 g/m ² coated vinyl)	---	54	---	14

Note: Interface material tested relative to the *PVC geomembrane tube* listed in the last row.

Test listed in the last row was *PVC Tube* interfacing with *PVC Tube*.

Davis and Landin (1997) reported δ_p on the order of 18° between two geotextile tubes and on the order of 25° between geotextile tubes and sand. Martin et al. (1984) reported friction angles for a variety of materials. A summary of the data provided is provided below.

- Sand to geotextile: 23 to 30°
- Sand to geomembrane: 17 to 27°
- Geomembrane to woven monofilament geotextile: 6 to 11°
- Geomembrane to woven slit film geotextile: 13 to 28°
- Geomembrane to non-woven geotextile: 8 to 23°

Table 5.3 provides friction data for a variety of materials as a reference. The majority of the data was taken from Pestel and Thomson (1969). The data provided in Table 5.3 should only be taken as an estimate that may vary depending on the specific materials tested.

Table 5.3. Friction Coefficients of Select Materials

Material	ω_p (---)		δ_p ($^\circ$)	
	Dry	Greased	Dry	Greased
Teflon on Teflon	0.04	---	2.3	---
Steel on Steel	0.15	0.10	8.5	5.7
Steel on Cast Iron	0.25	0.10	14.0	5.7
Wood on Wood	0.40 to 0.60	0.20	21.8 to 31.0	11.3
Wood on Metal	0.60 to 0.70	0.10	31.0 to 35.0	5.7
Rubber on Asphalt	0.70 to 1.00	---	35.0 to 45.0	---

5.3 Analysis of Geotextile Tubes

Modeling flexible structural elements encapsulated in a thin membrane is a complicated problem in absence of assumptions. Internal and external stability must be maintained. Review of literature and practice revealed internal stability could be calculated with reasonable precision. External stability, however, was found to require many assumptions and was not nearly as well developed as internal stability. Since external stability calculations are not well established for permanent applications, the decision was made to perform analysis using fundamental principles coupled with designs shown to be stable based on those principles; i.e. no attempts were made to refine designs.

There is not enough information available on the tensile behavior of geotextile tubes to allow more refined calculations in absence of detailed finite element modeling that is out of the scope of this effort. For example, when slurry is pumped into the bottom row of geotextile tubes, tension would increase. When the third tube is stacked on the first tubes tension in the bottom geotextile tubes would likely decrease in some areas and increase in other areas. The tensile state of the tubes would also change as the cementitious stabilized slurry cured, and the frictional behavior between the tubes would also affect tensile forces in any one location within the tube.

5.3.1 Calculation of Geotextile Tube Internal Properties

*TenCate*TM staff members calculated fill volume, fill dimensions, and internal stability with *Geotube*[®] *Simulator*TM software for a variety of conditions. The results are shown in Table 5.4. All calculations were to achieve a factor of safety against tube rupture of 3, which is the company recommended minimum value for many applications.

The data provided in Table 5.4 encompassed the range of fill density that would be available. Fill density represented by $SG_{Int} \approx 1.2$ would be slightly lighter than expected for any stabilized slurry material whereas $SG_{Int} \approx 1.6$ would be slightly heavier than expected for any stabilized slurry material. Sand is typically represented by $SG_{Int} \approx 1.8$ when placing geotextile tubes. Fill volumes are needed for construction time estimates (the smaller the volume the faster the construction time). Also, filled dimensions are required for layout of the water reservoir and to estimate the height a reservoir wall would be when constructed from geotextile tubes.

Table 5.4. Internal Properties of Geotextile Tubes using *Geotube® Simulator™*

<i>C</i> (m)	Type (---)	<i>h</i> (m)	<i>SG_{Int}</i> (---)	<i>H_w</i> (m)	<i>B</i> (m)	<i>W</i> (m)	<i>V</i> (L/m)
9.14	<i>GT 500</i>	2.07	1.2	0.00	2.01	3.46	5940
		2.67	1.2	2.67	0.73	3.05	6600
		1.92	1.6	0.00	2.29	3.56	5693
		2.38	1.6	2.38	1.39	3.26	6345
		1.86	1.8	0.00	2.39	3.60	5593
		2.26	1.8	2.26	1.65	3.34	6195
	<i>GT 1000M</i>	2.18	1.2	0.00	1.80	3.39	6090
		2.71	1.2	2.71	0.63	3.02	6630
		2.04	1.6	0.00	2.06	3.48	5894
		2.44	1.6	2.44	1.26	3.21	6395
		1.98	1.8	0.00	2.18	3.52	5793
		2.35	1.8	2.35	1.46	3.28	6295
13.72	<i>GT 500</i>	2.48	1.2	0.00	4.10	5.59	11720
		3.70	1.2	3.70	1.80	4.79	14480
		2.23	1.6	0.00	4.48	5.74	10910
		3.02	1.6	3.02	3.18	5.25	13167
		2.13	1.8	0.00	4.61	5.79	10584
		2.80	1.8	2.80	3.57	5.39	12640
	<i>GT 1000M</i>	2.65	1.2	0.00	3.83	5.49	12220
		3.77	1.2	3.77	1.64	4.74	14580
		2.44	1.6	0.00	4.16	5.61	11587
		3.20	1.6	3.20	2.84	5.13	13568
		2.32	1.8	0.00	4.35	5.69	11211
		2.96	1.8	2.96	3.30	5.29	13017
18.29	<i>GT 500</i>	2.64	1.2	0.00	6.42	7.84	17860
		4.53	1.2	4.53	3.26	6.67	24830
		2.35	1.6	0.00	6.78	8.00	16380
		3.44	1.6	3.44	5.26	7.37	21360
		2.22	1.8	0.00	6.93	8.07	15690
		3.11	1.8	3.11	5.77	7.57	20010
	<i>GT 1000M</i>	2.90	1.2	0.00	6.07	7.69	19080
		4.70	1.2	4.70	2.90	6.55	25230
		2.57	1.6	0.00	6.51	7.88	17510
		3.66	1.6	3.66	4.89	7.23	22180
		2.45	1.8	0.00	6.65	7.95	16900
		3.36	1.8	3.36	5.38	7.42	21050

Note: Values shown are for FS of 3 to slightly above 3.

Note: Fill volumes for standard Geo-Synthetics, Inc 9.14, 13.72, and 18.29 m circumference tubes are reported by the manufacturer to be 5300, 9800, and 15300 L/m, respectively.

Note: *SG_{Int}* of 1.8 is the typical value used by TenCate™ when sand is the fill material.

5.3.2 Calculation of Geotextile Tube External Stability

Figure 5.1 is a schematic of a geotextile tube wall as analyzed in this report with each tube given a letter designation (A, B, or C). The height of Tube A was estimated using judgment as no information is available. The shape of Tube A is arbitrarily shown in the figure as no information on its shape is available. The analysis condition of interest is when the interior water has been removed and water is only on one side of the wall of total height (*H*) to height *H_w*. The water applies lateral forces on the wall, which was the focus of the analysis.

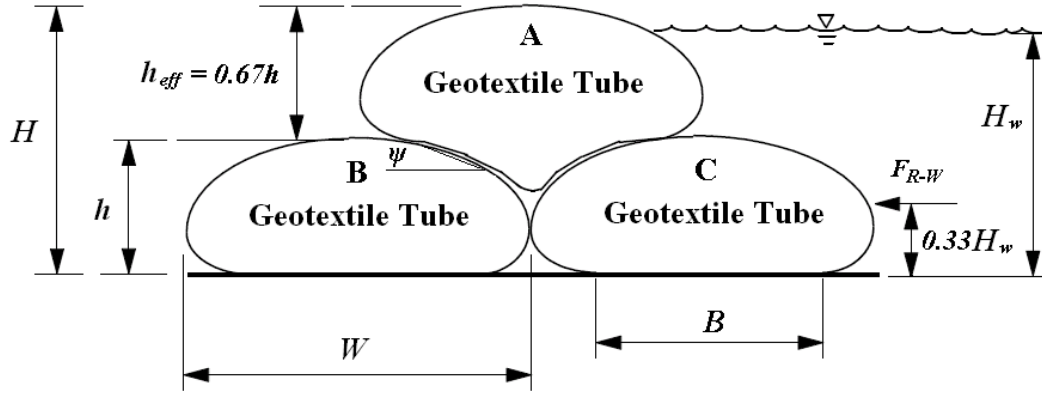


Figure 5.1. Schematic of Stacked Geotextile Tubes

The resultant horizontal force acting on the wall due to the water can be calculated according to Eq. 5.9. The resultant force is located as shown in Figure 5.1. For H_w equal to 3.05 m and considering seawater, F_{R-W} is equal to 4,800 kg per meter of wall.

$$F_{R-W} = 0.5\gamma_w(H_w)^2 l \quad (5.9)$$

Where,

F_{R-W} = resultant horizontal force acting on the wall due to water

l = unit length of wall

A wall constructed of three 9.14 m circumference tubes made from *GT 1000M* geotextile and *ST_{Int}* slurry of 1.6 is the primary case considered for design. The height of the wall was estimated at 3.8 m using the data in Table 5.4, and the total volume required to fill all tubes was estimated at 18,700 L/m or 18.7 m³/m. A 3.8 m wall height in conjunction with a 3.05 m water depth (water 80% of wall height) is less conservative than the Morocco project discussed previously (water 60 to 70% of wall height). The most expedient solution would be to use small circumference tubes. For water depths less than 3.05 m a single tube of larger circumference could be the most practical solution. Single geotextile tube walls are a simplified version of the three tube configurations considered in this report that could be analyzed with methods similar to those shown.

The critical shear area for the wall is equal to $2Bl$, or 2.52 m². The maximum horizontal shear stress is thus 4,800 kg/2.52 m², or 0.19 kg/cm². Either of the (5, 100) slurries shown in Figure 4.3 would have more than twice this shear strength (even when unconfined as in Figure 4.3-strength increases with confinement) at 24 hr without mobilizing any strength from the geotextile tubes and as a result, horizontal shear failure is not perceived to be a problem with 9.14 m circumference tubes.

To further investigate shear failure within the fill, the sand within the tubes used for the Morocco wall was estimated at 30°, which should be an upper end friction angle for sand that would typically be used to fill a geotextile tube. The tubes were covered with an impervious tarp, which should limit pore water pressure (*pwp*) development within the tubes after a period of time, so a total stress assessment of the maximum shear strength was

performed. Using ST_{Int} slurry of 1.8 for sand, and a 6 m wall height, the total vertical stress acting near the base of the wall would be approximately $10,800 \text{ kg/m}^2$. Multiplying the total vertical stress by $\tan(30^\circ)$ and converting units results in a maximum shear strength in the sand near the base of the wall constructed in Morocco to be approximately 0.62 kg/cm^2 . If the same sand were used to fill the three 9.14 m circumference tubes being investigated herein, the maximum total vertical stress and shear strength would reduce to $6,800 \text{ kg/m}^2$ and 0.40 kg/cm^2 , respectively. These values should be considered crude estimates as some pwp could be present that would reduce the values, though confinement from the geotextile tubes would increase these values. The key point is that the shear strengths are in the range of values provided in Figure 4.3 for cement stabilized soil slurry in a (5, 100) condition.

The ability of the bottom tubes to support the top tube was investigated by conservatively neglecting the strength of the geotextile. For ST_{Int} slurry of 1.6 and a 3.8 m wall, the maximum vertical stress near the base of the wall would be approximately $6,100 \text{ kg/m}^2$, which translates to shear stress on the order of 0.31 kg/cm^2 . The Figure 4.3 slurries are capable of supporting this stress at 24 hr when unconfined. This material would be confined and have the support of the geotextile, so the bottom tubes should be capable of supporting the top tube 24 hr after the slurry is placed into the bottom tubes.

Sliding of the wall was calculated with the same concept shown in Eq. 5.5 where a geotextile tube wall replaces a single geomembrane tube. Pore water pressure (pwp) would exert more uplift force on Tube C than Tube B and therefore stability calculations were performed in a stepwise manner. Assuming a linear dissipation of pwp from a maximum value of $\gamma_w * H_w$ at the outer edge of Tube C to zero at the outer edge of Tube B, Tube C would experience approximately 7,500 kg/m uplift and Tube B would experience approximately 2,500 kg/m uplift. The total weight of the $18.7 \text{ m}^3/\text{m}$ wall made from ST_{Int} slurry of 1.6 would be on the order of 30,000 kg/m, or 15,000 kg/m resting on Tube B and on Tube C. The net downward force at the wall/foundation interface for friction calculations is thus 12,500 kg/m and 7,500 kg/m for Tube B and Tube C, respectively.

Data from Section 5.2.3 was used to select a range of friction coefficients to consider for sliding calculations. From the perspective of the foundation, placement of a geotextile under the wall would be desirable to limit stress on the foundation and minimize settlement. The geotextile to geotextile friction characteristics, however, are lower than when the geotextile is in contact with actual foundation materials. For this analysis, δ_p was taken to be 17° , 22° , and 25° for geotextile, grass, and sand foundation materials, respectively. Superposition of the net downward force from each tube (20,000 kg) and multiplying by $\tan(\delta_p)$ results in the maximum frictional force resisting the 4,800 kg/m driving force from lateral water forces. The resulting resisting forces are 6,100, 8,000, and 9,300 kg/m for geotextile, grass, and sand foundation materials, respectively. The resulting factors of safety against sliding (FS_{slide}) are 1.27, 1.66, and 1.93, respectively.

Theoretically any FS_{slide} value exceeding 1.0 would prevent sliding. Sliding resistance would also be improved if the wall deformed the foundation, which would occur for any of the materials considered during calculation. In a disaster environment, however, higher factors of safety should be employed unless site specific decisions warrant otherwise. If a wall is to be constructed with 9.14 m circumference tubes, the interface friction angle should be at or greater than 25° . Ideally, the reservoir would be constructed on a paved parking lot to eliminate bearing capacity issues and provide adequate friction resistance as a paved surface should have a friction angle with geotextile at or greater than 25° .

Sliding of Tube A off Tube B and Tube C was also considered. To do so, the contact angle relative to the horizontal between Tube A and either of the other tubes (ψ) was estimated based largely on judgment as no data was available. To estimate a reasonable value of ψ , an ellipse was assumed since that assumption is used within the *Geotube[®] Simulator[™]* software to calculate the shape of filled geotextile tubes. A straight line was fit through the coordinates of an ellipse at 0° and 90° and taken as ψ , which results in Eq. 5.10. For the case considered herein, ψ is 37°, which is crudely approximated.

$$\psi = \tan^{-1} \left(\frac{h}{W} \right) \quad (5.10)$$

The lateral water force acting on Tube A would be at to slightly less than 200 kg as the water height would be on the order of 0.6 m. Allowing frictional resistance to be mobilized on only one side of Tube A, coordinate geometry would approximate the frictional resistance with Eq. 5.11, which should also be taken as a crude approximation.

$$F_T = 0.5W_{T-A} (\cos \psi)^2 \tan \delta_p \quad (5.11)$$

Where,

F_T = frictional resistance between the top and bottom stacked geotextile tubes

W_{T-A} = weight of Tube A

Tube A weighs approximately 9,500 kg and δ_p was taken as 18°, which results in a frictional resistance of 984 kg. The factor of safety against the top tube sliding would be on the order of 5 based on these approximate calculations. While the true factor of safety isn't known due to the assumptions made due to lack of data, sliding of the top tube is not perceived to be a problem. The top tube will likely rest between the bottom tubes, which will provide a substantial resistance to sliding not considered in these calculations.

The final calculations for the geotextile tube wall was slope stability performed by *WinStabl*. Calculations were performed with zero degree direction limits in the clockwise and counterclockwise directions, and each combination incorporated 50 initiation points and 50 surfaces generated. Circular failure surfaces were considered with *Janbu* and *Modified Bishop* methods.

The wall was idealized as 6.4 m wide at the base, 3.8 m tall, 2 m wide at the top, with slopes inclined 60° from the horizontal, and with 3.05 m of water behind the wall. The base width was $2W$ based on submerged tubes, the height was estimated previously, the width at top was taken as the base width (B) of the top tube emerged, and the slopes were calculated based on the remaining dimensions, which were probably steeper than they would be in the actual wall making the analysis conservative in that regard. Calculations did not consider geotextiles to be part of the wall, rather determined if the cementitious stabilized slurry would be stable on its own. The density of the wall was 1,600 kg/m³ and the density of the foundation was 2,000 kg/m³. Calculation parameters (e.g. initiation and termination dimensions, segment lengths, minimum failure surface elevation) were varied to determine the factor of safety (FS_s) for slope stability for given foundation and wall shear strengths.

Table 5.5 presents slope stability test results. Strength of the foundation did not have an effect for 0.1 or 0.2 kg/cm² wall strengths but did have an effect at 0.3 kg/cm². Shear strength of the (5, 100) material in Figure 4.3 provides adequate strength at 24 hr to alleviate problems with slope stability. A factor of safety in excess of 2 neglecting geotextile strength should be adequate for the geotextile tube wall, indicating a minimum of 0.3 kg/cm² wall shear strength and 0.4 kg/cm² foundation shear strength is needed for slope stability.

Table 5.5. WinStabl Slope Stability Calculation Results

Foundation s_u (kg/cm ²)	Wall s_u (kg/cm ²)	FS_s	Failure Description
0.2	0.1	0.7	Toe to back slope-above foundation
	0.2	1.5	Toe to back slope-above foundation
	0.3	1.7	Almost entirely through foundation
0.4	0.1	0.7	Toe to back slope-above foundation
	0.2	1.5	Toe to back slope-above foundation
	0.3	2.3	Toe to back slope-above foundation

5.4 Analysis of Geomembrane Tubes

5.4.1 Calculation of Geomembrane Tube Internal Properties

Dimensions and volumes of the geomembrane tubes under consideration were determined by the manufacturer and provided to the research team (Table 5.6). The inflated properties were taken partially from the Users Guide (2004), but updated with assistance from the fabricator to reflect minor modifications since publication of the aforementioned document. With water being the only material filling the tubes, the needed information reduces to the inflated dimensions and the volume of fluid necessary to fill the tubes.

Table 5.6. Inflated AquaDam[®] Dimensions and Volumes

h (m)	W (m)	Water Storage (L/m)
0.30	0.61	149
0.46	0.91	311
0.61	1.22	683
0.91	2.13	1,490
1.22	2.90	2,981
1.52	3.35	4,037
1.83	4.27	4,968
2.44	5.49	7,452
3.05	6.71	9,936
3.66	7.32	11,178
4.88	9.76	15,525

Note: A 2.44 m tall Aqua Barrier[™] provided by another supplier is also 5.49 m wide when fully inflated.

5.4.2 Calculation of Geomembrane Tube External Stability

Review of literature indicated a 3.05 m water depth could be resisted with a geomembrane tube 4.1 m or higher. Literature review also indicated friction at or greater

than grass provided in Table 5.2 (22 degrees) would prevent sliding. The required friction between the dam and surface is reasonable indicating sliding is not problematic. A reasonable design is to construct the water reservoir out of a wall of 4.88 m *AquaDam*[®] units placed without stacking.

5.5 Reservoir Liner Analysis

Geomembrane reservoir liners date back to pre 1930 to the 1940's depending on the properties one selects to qualify a geomembrane. Geomembranes have contained everything from freshwater to hazardous liquids. Polymeric geomembranes have been successfully used to waterproof over 200 dams (Koerner and Wilkes 2007). Geosynthetic clay liners (GCL's) and geomembranes have been placed in live (i.e. flowing) canals successfully (Koerner et al. 2008). They were placed with a canal spanning truss system. Reservoir liners typically found in practice perform in liquid depths of 2 to 8 m and on slopes of 14 to 45 degrees from the horizontal (Koerner et al. 2008). Durability of the exposed (above liquid level) geomembrane is usually the primary concern, though for a short duration project it shouldn't be a difficulty. In routine practice, at least 300 mm of soil is generally placed over the membranes for protection.

Ten materials are actively used as geomembranes for containment of all types of liquids. A geotextile should be placed below the geomembrane liner to facilitate field seaming, improve puncture resistance, and minimize stresses on the geomembrane (Koerner et al. 1984). Koerner (1998) notes the specific type of geomembrane material isn't critical though polyvinyl chloride (PVC) is often easier to place but must be covered for extended use and high density polyethylene (HDPE) does not require covering. Koerner (1998) proposed an analysis for geomembranes that was used herein and is summarized by Eq. 5.12.

$$t = \frac{\sigma_n(x)(\tan \delta_U + \tan \delta_L)}{\sigma_{allow}(\cos \beta - \sin \beta \tan \delta_L)} \quad (5.12)$$

Where,

σ_n = stress applied due to hydrostatic pressure

x = distance required to mobilize geomembrane deformation

δ_U = interface friction angle on upper side of geomembrane

δ_L = interface friction angle on lower side of geomembrane

σ_{allow} = allowable geomembrane stress

β = angle of settlement that induces tension in geomembrane

Appropriate inputs for Eq. 5.12 are unknown in absence of specific disaster parameters. As a result, a conservative approach was taken to estimate a thickness for the reservoir liner. Hydrostatic stress was calculated using 5 m of water, which is more than the reservoir will hold and would easily account for any minor depressions in the reservoir floor. The value used was (σ_n) of 50 kPa. The distance required for mobilization was taken from data provided in Koerner (1998). A range of values related to anchorage was given of 50 to 300 mm, so (x) was conservatively chosen at 300 mm. With water above the geomembrane, δ_U is zero. Below the geomembrane, δ_L was taken as 11 degrees since this was the highest

value in the data from Martin et al. (1984) in Section 5.2.3 for a geotextile under a geomembrane and would thus induce the highest geomembrane stress. The maximum geomembrane stress was reported for HDPE by Koerner (1998) to be on the order of 15,900 kPa, so the allowable geomembrane stress (σ_{allow}) was taken as 5,300 kPa after applying a factor of safety of 3. The angle of settlement (β) was taken as a high end value of 45 degrees.

The required thickness using Eq. 5.12 and the aforementioned inputs was 0.97 mm. If all parameters are held constant and concrete sand replaced the geotextile (δ_L of 18 degrees) the required thickness would increase to 1.93 mm. A rough material that is also prone to settlement is unlikely. A fine grained material (e.g. silt or clay) is more likely. Frictional behavior of geomembranes placed on clay has been investigated (e.g. Koerner et al. 1986) but the result is sensitive to project specific properties so testing is recommended in the literature rather than using values from other experimental programs for design. Due to the uncertainty of the environment and the calculation results, a 1.5 mm thick HDPE liner is a reasonable estimate pending project specific conditions.

5.6 Design of Geotextile Tube Water Reservoir

Wall design of an expedient water reservoir using geotextile tubes focused on a configuration using three tubes as shown in Figures 5.2 and 5.3. A key issue of the design was to determine a stacking configuration to minimize construction complexity by using typical geotextile tube lengths. Other issues were reservoir capacity and construction time. Figure 5.3 shows the wall and dimensions for volume calculations. The wall height (H) was taken as $1.67h$.

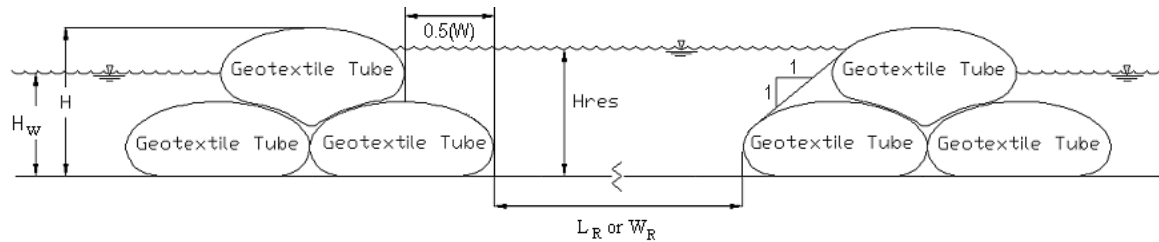


Figure 5.2. Profile View of Geotextile Tube Water Reservoir

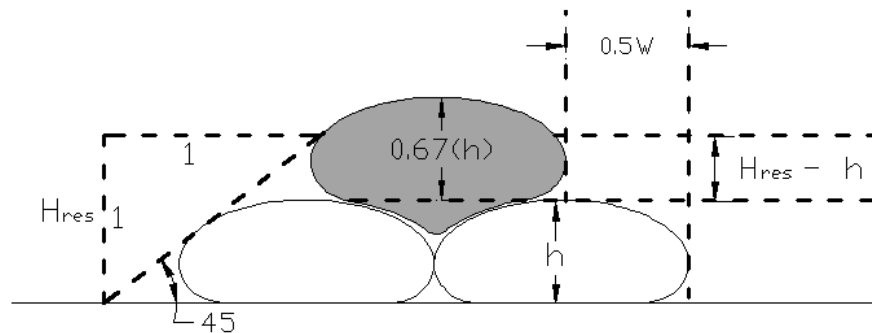


Figure 5.3. Dimensioned Profile View of Geotextile Tube Wall

The targeted reservoir capacity was 10 million liters. Three designs were considered that are shown in Figures 5.4 to 5.6. All designs incorporated 9.14 m circumference tubes

since construction time is reduced with smaller tubes. For the slurries investigated in complimentary research SG_{Int} was bounded by 1.2 to 1.6; 1.6 was used for calculations. Properties of the tubes were taken from Section 5.4 using *GT 1000M*. The height of water in the reservoir (H_{res}) was 3.05 m for all calculations. Calculations incorporated submerged tubes on the bottom row, and to be conservative assumed the top tube (Tube A) was emerged. Tube A was centered over tubes B and C in all designs.

All three design options incorporate twelve geotextile tubes with flat ends. The standard geotextile tube length (L) used for the designs was 61 m. The total height of the geotextile tube walls for all design options was 3.8 m, giving 0.75 m of freeboard. The volume of the water reservoirs was estimated using Eq. 5.13.

$$V_R = [(L_R)(W_R)(H_{res}) + (W)(H_{res} - h)(W_R + L_R)](1/1000) \quad (5.13)$$

Where,

V_R = volume of water reservoir in millions of liters

L_R = interior length of geotextile tube water reservoir at base

W_R = interior width of geotextile tube water reservoir at base

H_{res} = depth of water in reservoir

W = filled width of geotextile tube

h = height of geotextile tubes on bottom row

In design option 1 (Figure 5.4) tubes 1 to 4 and tubes 9 to 12 are 61 m long. Tubes 5 to 8 are $L-2W$, or 54.6 m long for the dimensions considered. Dimensions L_R and W_R are $L-3W$, or 51.4 m long for the dimension considered. The estimated capacity of this reservoir is 8.3 million liters.

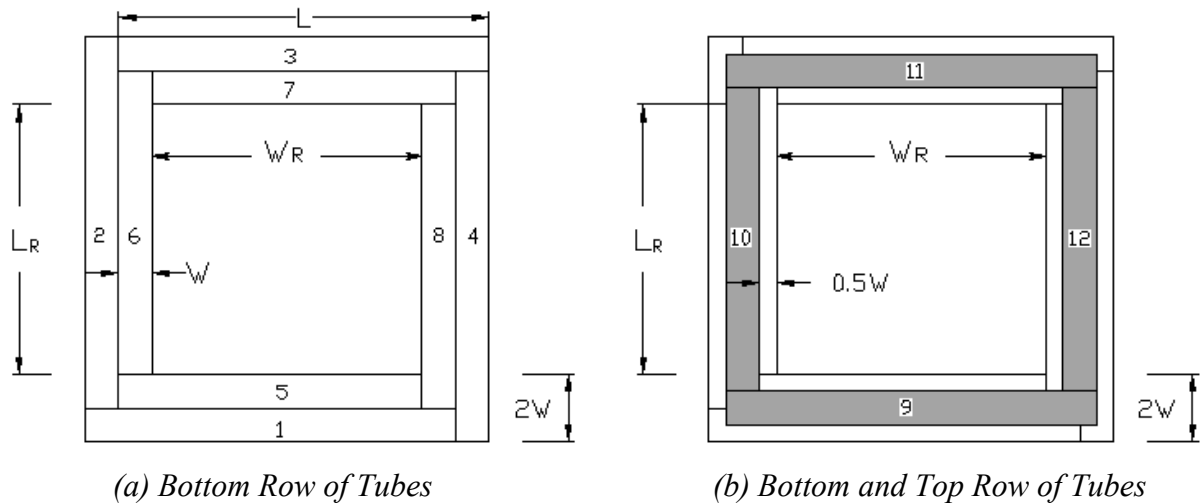


Figure 5.4. Geotextile Tube Water Reservoir Design Option 1

In design option 2 (Figure 5.5) tubes 1 to 9 and tube 11 are 61 m long. Tubes 10 and 12 are $L+W$, or 64.2 m long for the dimensions considered. It might be desirable to make these tubes somewhat longer than 64.2 m, but no specific statement can be made at present.

Dimension L_R is equal to L of the standard length tubes, or 61 m. Dimension W_R is equal to $L-4W$, or 48.2 m long for the dimensions considered. The estimated capacity of this reservoir is 9.2 million liters.

In design option 3 (Figure 5.6) all twelve tubes are 61 m. Dimensions L_R and W_R are $L-2W$, or 54.6 m long for the dimensions considered. The estimated capacity of this reservoir is 9.3 million liters. Design option 3 appears to be the best design in that it uses tubes of the same length, has the highest volume capacity, and does not appear to pose any additional construction challenges relative to the other designs.

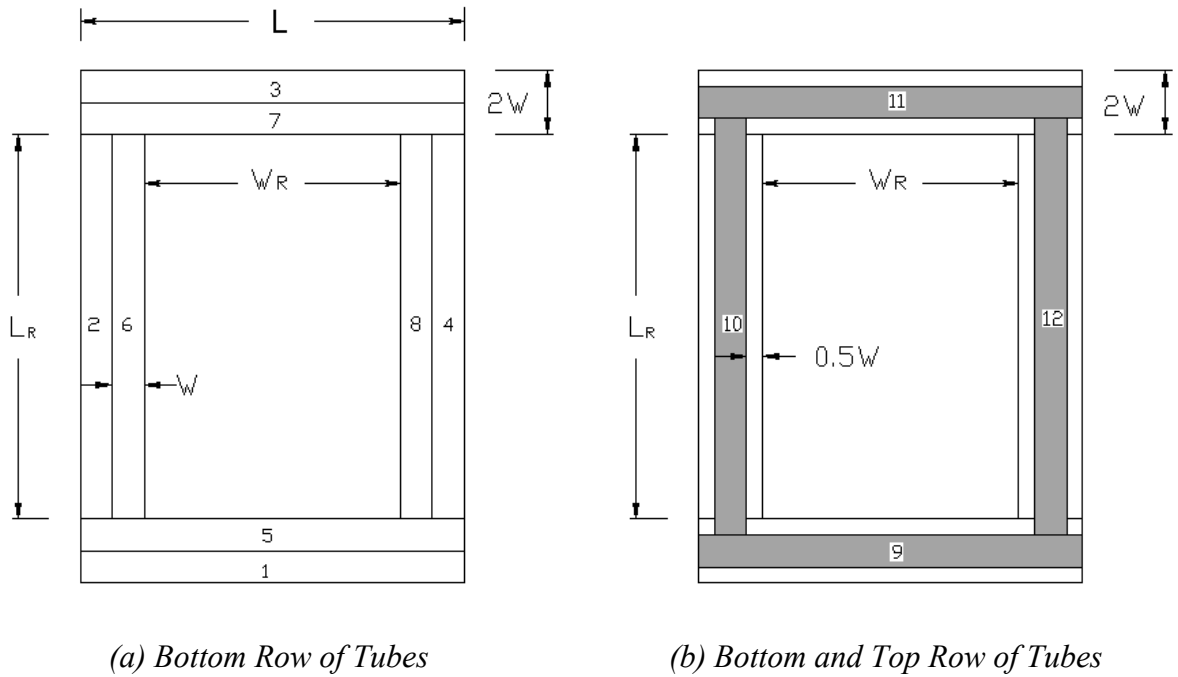


Figure 5.5. Geotextile Tube Water Reservoir Design Option 2

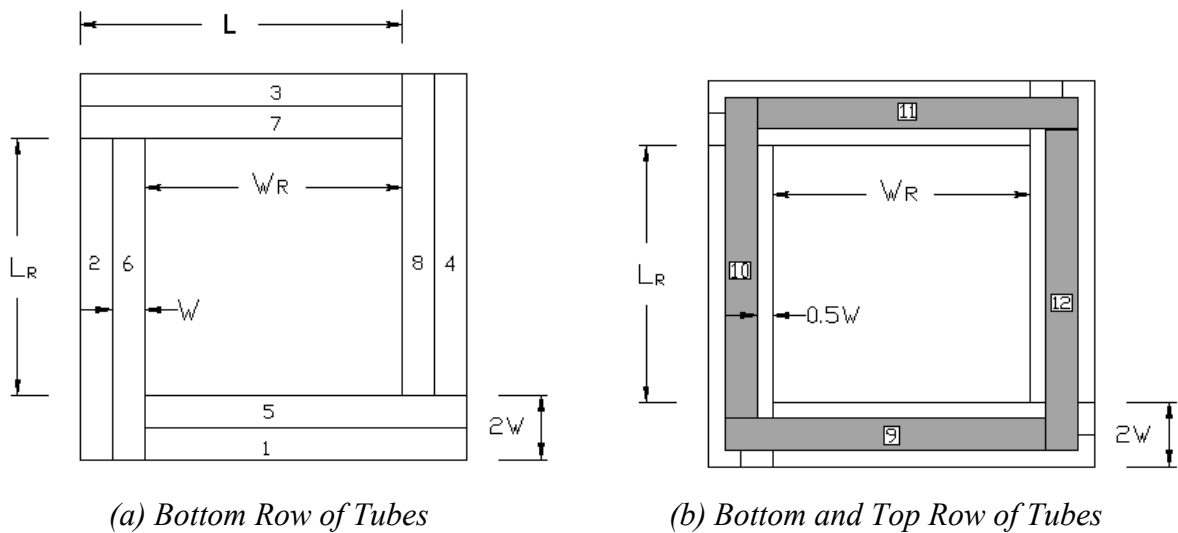


Figure 5.6. Geotextile Tube Water Reservoir Design Option 3

The wall should be placed on a surface providing an angle of friction of 25° or more. A high strength geotextile under the wall would reduce settlement, but probably would decrease the angle of friction below 25 degrees. A parking lot would be a good location for the reservoir. A liner as designed in Section 5.5 should be placed in the reservoir.

Completely closing the reservoir was an option entertained by the research team but was beyond the scope of this project. Koerner (1998) provides information related to geomembranes and also provides information on floating coverings which could be used if a covered reservoir were desired. Covering the water reservoir could reduce the required freeboard height (i.e. $H-H_w$).

5.7 Design of Geomembrane Tube Water Reservoir

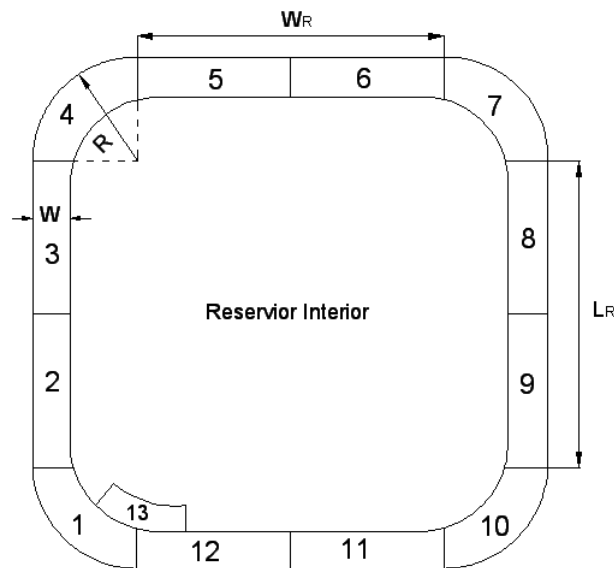
A single geomembrane tube (4.88 m tall and 9.76 m wide) is commercially available. This tube can withstand a 3.05 m water depth and is stable against sliding with a friction angle less than that of the 9.14 m stacked geotextile tube wall. The highest required value reported in Section 5.2.2.2 was 19 degrees, which is lower than the 25 degrees recommendation for the 9.14 m circumference stacked geotextile tube wall. A logical shape of the reservoir is an oval as shown in the plan view of Figure 5.7. Reservoir volume calculations were performed according to Eq. 5.14, where all dimensions are in meters.

$$V_R = \left(\left[\pi(R - W)^2 + L_R W_R + 2W_R(R - W) + 2L_R(R - W) \right] H_{Res} - V_{T13} \right) (1/1000) \quad (5.14)$$

Where,

V_R = Volume of water within reservoir in millions of liters

V_{T13} = Volume of tube 13 (m³)



Not To Scale

Figure 5.7. Plan View of Geomembrane Tube Water Reservoir

The reservoir has a wall length of 305 m since this was the minimum quantity the fabricator maintains in stock in routine operation. Tubes 1, 4, 7, and 10 were selected to be 4.88 m tall and 15.2 m long, tube 13 is 1.52 m tall and 15.2 m long, and the remaining tubes are 4.88 m tall and 30.5 m long. Inputs to Eq. 5.14 would thus be: R of 9.68 m, W of 9.76 m, L_R of 61 m, W_R of 61 m, H_{res} of 3.05 m, and V_{T13} of 62 m³. Note that the term $R-W$ would be negative yet on the order of unity; for calculation purposes the value was taken as 0. Essentially, the corner tubes would need to be bent at a near 90 degree angle to make the connection with 15.2 m long units; longer corner tubes would provide more curvature. Figures 1.6 and 1.7 show *AquaDam*[®] units with sharp bends (note a full scale demonstration of closing the geomembrane tubes into a perimeter would be needed prior to deployment). The storage capacity of the geomembrane tube reservoir was calculated to be approximately 11.3 million liters. As a reference, the tubes themselves would contain on the order of 4.8 million liters, or 42% of the water within the reservoir itself.

The wall should be placed on a surface providing an angle of friction of 22 degrees or more. A high strength geotextile under the wall would reduce settlement, but probably would decrease the angle of friction below 22 degrees. A parking lot would be a good location for the reservoir. A liner as designed in Section 5.5 should be placed in the reservoir.

CHAPTER 6 - CONSTRUCTION OF WATER RESERVOIR WITH GEOTEXTILE AND GEOMEMBRANE TUBES

6.1 Overview of Construction

This chapter provides geotextile and geomembrane tube water reservoir construction information. The site visits data presented in Chapter 3 was a key resource relied upon in this chapter. Literature and practice information pertinent to water reservoir construction is presented first, followed by construction procedures for geotextile and geomembrane tube water reservoirs. Installation of an impervious liner is presented at the end of the chapter.

6.2 Literature and Practice Review of Construction

6.2.1 Geotextile Tubes Construction Literature and Practice Review

At the *USACE* workshop (Davis and Landin 1997) first discussed in Section 5.2.1, it was estimated to take 6 to 18 hr to fill 150 m of geotextile tube with sand. In another portion of the workshop, it was said to take 11 hr to fill 150 m of geotextile tube. A case study was presented from Barren Island where 36 geotextile tubes each 61 m long and 9.14 to 11.43 m circumference were used to form a retaining dike where a soil with 20% sand, 8% silt, and 72% clay was used. The presenter noted they did not feel this was a high quality material for filling the tubes, but made no mention of any performance problems while in service. The material was pumped between 1.8 to 3.6 km with a 1,000-hp pump and a 300 mm discharge line. The tubes were filled at 6.8 m³/min, which resulted in filling times of 6 to 8 hr. The tubes were installed in 1.52 m deep water. Initially, the labor was reported to be intensive, as up to six workers were required in the water to unfold, hold, and secure the geotextile tubes with the entire 61 m being deployed at once. During construction the contractor incorporated 61 m floating PVC pipe to assist in placement of the tubes, which required fewer workers to hold the tube in place. During the project attempts to stack two tubes on top of one another were unsuccessful; the top tube either slid or rolled off the bottom tube.

In 1997, the area of construction and implementation related to geotextile tubes was so new that the *USACE* had no list of qualified contractors. As far as construction techniques, the participants had used different techniques under different circumstances, and most participants had been involved with enough construction projects related to geotextile tubes to know items that would routinely be effective. The workshop noted that applications could increase greatly after stacking technology was more ensured. Sand was identified as the preferred fill material in almost every possible situation.

Duarte et al. (1995) described design, construction, and results of using geotextile bags and containers for construction of dikes on the Mississippi River south of Baton Rouge, LA within the New Orleans District of the *USACE*. The motivation of the work was to reduce the amount of dredging required. One question to be answered by the work was whether or not construction contractors in the US were qualified for this type of work.

At the time of publication, a concern of Duarte et al. (1995) was lack of published information regarding effects of water velocity, container drift, and impact forces being constructed in actively-flowing channels. The project dropped the majority of the geotextile bags and geotextile containers in 9.2 to 12.2 m of water, with select drops being as deep as

21.4 m. River velocities were on the order of 1.5 m/s. Calculations resulted in a drop velocity of 5 m/sec, momentum of 80 metric tons, and a required working strength of each longitudinal seam on the order of 30 kN/m (the ultimate value of longitudinal seam strength was required to be at least twice this value). Each geotextile container had a 13.72 m circumference, while lengths varied from 12 to 35 m.

The construction group for the geotextile bags worked from barges with sand hoppers that were used to fill the geotextile bags that had a 2.3 m³ capacity. The filled bags were transferred in a cage using a front end loader and dropped into the water from a height not exceeding 3.7 m; the drop height was typically 2.3 m. Geotextile bags were not completely filled to allow for some internal energy dissipation. Average production during the peak period was 373 geotextile bags per day with a standard deviation of 58 bags.

Geotextile containers were placed with split hopper barge. On these barges the containers were filled with moist sand using a backhoe having a 7 m³ scoop capacity. The barge with the geotextile container was then tied to an empty barge positioned to account for drift. The geotextile containers were dropped on 6.1 m centers, and alternate layers were offset by 3 m to fill gaps remaining between them. On average, 8 geotextile containers were dropped per day, and 556 were placed during the project. The largest geotextile containers placed had a volume of 422 m³.

Instrumentation on geotextile containers measured maximum strains of 8 to 12%, which occurred while sliding out of the barge bin. Strains of 3 to 4% were measured on impact. Geotextile container terminal velocities were 3.5 to 4.5 m/sec, and drift distances were 1.2 to 2.4 m. Maximum geotextile bag strain was 3 to 8% and occurred upon impact of the water surface. They drifted on approximately a 1H:1V slope. The project was successful and revealed the US had qualified construction contractors for work of this nature.

A temporary dam approximately 70 m long was recently constructed in Morocco using *Geotube*[®] units (Figure 6.1). This project was also discussed in Chapter 5. The tubes were covered with an impervious liner with properties shown in Table 6.1. To construct the walls shown in Figure 6.1, the rock walls were first smoothed by placing concrete. The first *Geotube*[®] unit was placed, and then the second was placed 3 m from the first. The gap between the bottom *Geotube*[®] units was filled with sand (Figure 6.2) and then covered with a geotextile. The third and final *Geotube*[®] unit was installed on top of the other two units and intermediate sand. The impervious liner was then placed over the entire structure. Thereafter, the water was pumped out from the one side (Figure 6.1b).



(b) Site Location



(c) Final Product

Figure 6.1. Construction of Temporary Dam in Morocco Using *Geotube*[®] Units

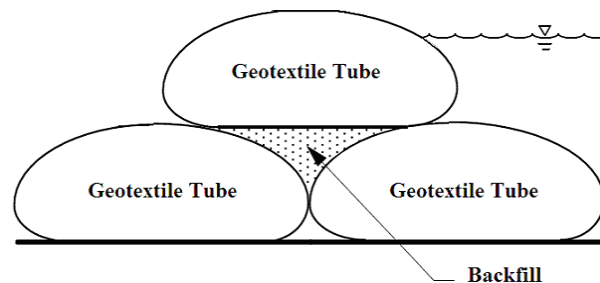
Table 6.1. Properties of Impervious Liner Used in Morocco Project

Property	ASTM Method	Result
Grab Tensile Strength-kN ^a	D 4632-91	1.29
Grab Tensile Strength-kN ^b	D 4632-91	1.00
Grab Tensile Elongation-% ^a	D 4632-91	31
Grab Tensile Elongation-% ^b	D 4632-91	40
Trapezoid Tear Strength-kN ^a	D 4533-91	0.30
Trapezoid Tear Strength-kN ^b	D 4533-91	0.20
Puncture Strength-kN	D 4833-00	0.53
Permeability-cm/sec	D 4491-99A	< 1E ⁻¹⁴
Abrasion Resistance-% Str Ret.	D 4886-88 ^c	90
500 hr UV Resistance-% Str Ret.	D 4355-02	> 70
Mass/Unit Area-g/m ²	D 5261-92	287
Thickness-mm	D 5199-01	0.43
Roll Width-m	----	4
Roll Length-m	----	100
Roll Area-m ²	----	400
Estimated Roll Weight-kg	----	120

a) Properties in Machine Direction

b) Properties in Cross Direction

c) Sliding Block

**Figure 6.2. Sand Backfill Used in Morocco Project**

A Netherlands company (*ACZ Marine Contractors BV*) used a steel cradle to position pre-filled tubes into place (Fowler and Sprague 1993). These types of applications typically require a crane, as opposed to a continuous position and fill approach. Steel frames are available to hold tubes in place while they are being filled.

As of June 2009, *Flint Industries* maintained a certification system to qualify contractors that all their distributors had access to. They maintain a list of those contractors, which is divided into coastal contractors and dewatering contractors. According to company representatives there are significantly more dewatering contractors than there are coastal contractors, and the coastal contractors on their list are concentrated in Florida, New York (and surrounding northeastern states), Michigan, Texas, and Utah.

SPIRAL[®] geotextile tube units have been successfully filled using one inlet while the inlet at the other end remained open for discharge. Suspension time of solid particles in the slurry can be increased by pumping out water from the far end port to create vacuum forces within the tube according to the manufacturer. Soils with lower apparent specific gravity (e.g. 1.8) will remain in suspension longer than soils with higher specific gravity (e.g. 2.7).

Under the proper conditions, a 61 m long tube can be filled with clay slurry from only one port, but sand does not fill nearly as easily. Keeping sand in suspension poses some difficulty in filling the tubes, which is a rationale for needing low percent solids sands in routine practice. Placement of the material into the geotextile tubes requires careful control of water in that too much water reduces the efficiency of the process but so does too little water. The amount of water is dependent on material type. Optimum results are obtained when a medium to fine sand is the dredge material.

Hydraulic dredges are one of the most common (if not the most common) methods of filling geotextile tubes. Fill time for geotextile tubes in conventional applications (i.e. those in which sand is predominant material) is typically on the order of hours as discussed previously in this section. During tube filling, fluid is pumped into the tubes under pressure and if fluid pressure is too high, seam failure may occur. Geotextile tubes can also be filled with hoppers. They have been filled with sand from hoppers for shoreline protection. In these applications, water is flushed through the tubes to distribute the material. Hand held filling equipment maneuverable by divers is also available for geotextile tubes.

Pumps from 100 mm to 300 mm diameter are routinely used to fill the geotextile tubes, though pumps of 250 mm diameter or greater often require assembly and are not easily transferable. The discharge line of the dredge pump is often fitted with a valve/manifold system to control the rate of tube filling, and connection to the fill ports can be performed with standard plumbing fittings. Pinch valves placed at input ports or on flexible sections of piping are often used to regulate flow rate and pressure. A pump rate of 11,000 liters per minute is reasonable for a 250 mm discharge.

Overlap of 1.5 m is common for scour aprons underneath a geotextile tube. Scour aprons are placed for long term erosion prevention so that foundation stability is maintained. The foundation for the tubes is important and in some cases, a foundation must first be constructed. A shallow trench or “cradle” in the foundation is occasionally constructed to prevent rollover of the tubes. A perfectly horizontal placement of a geotextile tube is preferable. These and many other site preparation parameters are used in typical applications, though in a flooded disaster area they would not be options.

Two distinct philosophies regarding structural applications were found to exist among the participants of the workshop discussed in Howard et al. (2009) depending on the material filling the tubes: 1) sand, i.e. select material, or 2) fine grained material such as silt or clay, i.e. non-select material. The use of select materials to fill geotextile tubes for structural applications was, in general, strongly preferred. The use of non-select material (e.g. silt and clay) was a point of contention. Specific details regarding non-select material use were discussed without producing directed or immediately applicable end products. Some participants were of the opinion that non-select materials with very low initial percent solids were worth investigation while other participants were less optimistic and in turn less supportive of the concept. Non-select material applications were presented during presentations of invited participants but they were not rapid projects (at least not rapid based on the needs of a disaster environment). Rapid dewatering and/or cementitious stabilization of non-select materials were also discussed and felt to be potentially viable options by some participants.

Poor quality materials (e.g. silts and clays) have been used within geotextile tubes for structural applications. A U-shaped dike-contained channel was constructed using 13.72 m circumference geotextile tubes that were filled with dredged material ranging from organics to

silty sand to 125 mm stone. The project was constructed in 2000 and as of 2008 the geotextile tube dikes remained in place and were functional. This project was presented by a participant of the workshop summarized in Howard et al. (2009). Polyurea coated tubes could provide some benefits to a freshwater reservoir. This coating has been sprayed onto geotextile tubes before and after deployment (*Grand Isle* is an example). The top portions of the tubes could be sprayed with the coating to increase puncture resistance and make them more impermeable while the bottoms of the tubes remain untreated.

6.2.2 Geomembrane Tubes Construction Literature and Practice Review

Using water as the fill material has many conceptual advantages in a flooding disaster. The water filled dams discussed in Landis (2000) were constructed faster and with less construction personnel compared to other techniques. As an example, the *USACE* used *WaterStructures*® in 1997 to protect the levees of the Sutter Bypass just north of Sacramento, CA. A 100 year flood had just damaged the levee, and five to ten million dollars had been spent to repair it. A second flood threatened to destroy the levee in its vulnerable state. In less than seven hours 240 m of 0.91 m high *WaterStructures*® material was installed, which prevented damage to the earthen levee. The dams were dismantled in three hours after the water receded. According to company literature, the *USACE* performed an installation time study comparing sandbag dams and *AquaDam*® units. A 1.22 by 30.5 m dam of sandbags was constructed in 240 minutes by a group of trained individuals while two personnel installed the same size *AquaDam*® unit in 30 minutes.

A level surface with no large holes, roots, or other obstruction is needed for proper use of geomembrane tubes. A depression, regardless of footprint, increased localized stresses and can lead to global failure if not properly accounted for. The best method of accounting for depressions is to take the height of water in the deepest depression and use this value as the water height for the entire structure.

In some applications a second but smaller geomembrane tube is placed behind the primary unit to provide increased stability, as well as to assist in sealing the flow of water underneath the dam. The additional stability is especially significant in flowing water, whereas for the applications of interest in this research minimizing the flow of water under the dam due to seepage or imperfect sealing would be highly valuable. Essentially complete (i.e. near 100%) sealing has been reported on mud surfaces, though less than but close to 100% sealing is estimated for gravel or rock surfaces.

If damaged during construction, geomembrane tubes can be patched. To patch geomembrane tubes, a pair of pliers, razor, small propane torch, and patch tape are needed. A small slit is made in the outer tube, and the area is heated. The patch tape is applied to an area larger than the puncture. Patch tape is also placed over the outer geotextile.

6.3 Geotextile Tube Water Reservoir Construction

Construction of a fresh water reservoir in a disaster environment using geotextile tubes is an extension of existing technologies yet has not been conducted at any scale to the knowledge of the authors. No specialty equipment is envisioned during construction. A full scale demonstration is needed before any of the construction procedures provided in the remainder of this section can be deemed feasible.

It is recommended to attach adjacent bottom tubes and deploy simultaneously to provide continuous contact and improve the benefit of the top tube. When the bottom tubes are tied to and in contact with each other, the third tube will provide additional wall height versus the un-tied case where the tube will sink deeper into the space between the bottom tubes and thus reduce wall height. Typical handling straps on *Geotube*® units can safely support on the order of 90 kg. The straps can be used to tie the tubes (other manufacturers provide similar handling straps). Lateral stability is increased with tied bottom tubes.

Large shards of glass, metal, large sharp stones, and similar must be removed from the footprint of the reservoir to prevent puncture. The site of the reservoir should also be relatively flat (a surfaced parking lot if possible). Design option 3 (Figure 6.3) was the favored approach and it has been used for development of construction procedures. Tubes 1 and 5 would first be tied together and sank in unison by pumping them full of cementitious stabilized fine grained material. Tubes should be filled using y-splitters or manifolds with pinch clamps to fill multiple tubes in unison. Tubes 2 and 6 would be tied together, the side of tube 6 tied to the end of tubes 1 and 5, and then the tubes would be sank in unison. This process would be repeated for all eight tubes on the bottom row. Tube 9 would then be centered over tubes 1 and 5 and filling commenced. Ideally, tubes 10 through 12 would be aligned while tube 9 is being filled to ensure all connections can be made. Once all walls are in place, the water within the reservoir would be removed and on site modifications made to the reservoir in locations where it was not sealed. The reservoir liner would then be placed and lastly potable water would be pumped into the reservoir.

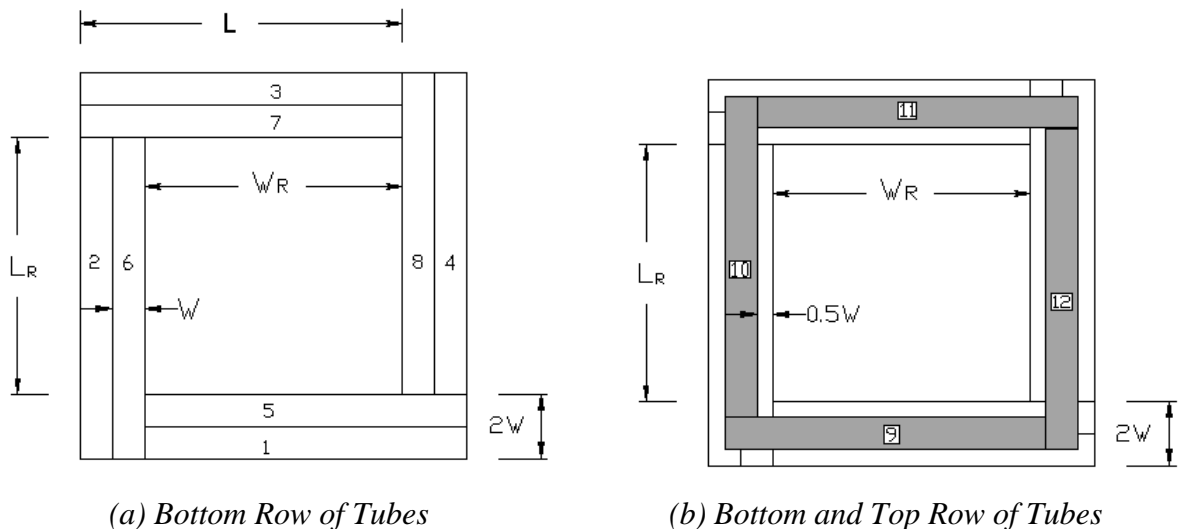


Figure 6.3. Geotextile Tube Water Reservoir Design Option 3

Based on the best information available (largely from Section 3.2), 61 m of tube can be placed per 12 hr with one crew in normal conditions. Two crews working simultaneously (12 hr days) for six days would be the absolute minimum required for construction of the geotextile tube water reservoir walls. Pumping out the interior water and placement of the impervious liner would require additional time. A minimum of one to two weeks appears to be needed for construction of a geotextile tube water reservoir, and this amount of time does not consider any difficulty getting construction equipment to the site, unforeseen equipment

malfunctions, or similar. Data presented in Section 3.8 estimated the cost of the reservoir in a disaster environment to exceed \$1,000,000.

6.4 Geomembrane Tube Water Reservoir Construction

Large shards of glass, metal, large sharp stones, and similar must be removed from the footprint of the reservoir to prevent puncture. The site of the reservoir should also be relatively flat (surfaced parking lot if possible). Specific construction procedures are as follows for *AquaDam*[®] units with heights greater than 1.22 m (construction with smaller tubes is a simplified version of the procedures shown). Additional information regarding placement of *AquaDam*[®] units can be found in *AquaDam*[®] User's Guide (2004).

Labor requirements to install large *AquaDam*[®] units in standing water are three to six personnel. At least three and as many as five laborers are required to place the dam, while one laborer is required to remove rocks and other puncture risks from in front of the dam as it is placed. Minimal equipment needed includes a discharge and a suction hose for each water pump, two high capacity water pumps, duct tape, and a utility knife for each laborer.

Two pumps are required to prevent switching from one tube to the next. The fill tubes can be opened to accommodate any size discharge hose, so the responder has flexibility to use the best available pumps. Four 12.7 mm diameter ropes are needed (Figure 6.4) to guide the tubes into position and maintain pressure within the tubes as they are filled. The ropes are tied to *AquaDam*[®] units and are either held by construction personnel or secured to a rigid object such as a tree or post. The ropes circle under the *AquaDam*[®] and back to the anchoring location, and are placed under the *AquaDam*[®] unit before pumping. The free end must be held in a manner that allows the rope to be let out as the unit unrolls due to filling. The rope should be twice as long as the unit plus an additional 15 m for incidental needs.



Figure 6.4. Ropes Used to Deploy Large *AquaDam*[®] Units

Typically *AquaDam*[®] units are packaged similar to a carpet roll. In general, the dam is unrolled slightly and filled. Pressure should be kept on the dam at all times during filling; i.e. do not unroll the full length and pump into it. This process is repeated throughout the filling of the dam. When unrolling, 0.3 to 0.9 m should be unrolled at a time and at least 0.3 to 0.6 m elevation should be maintained above the adjacent water. Always fill the

AquaDam[®] unit to the recommended height. When filling first begins, start slowly. It is possible that the hoses will be kinked, and must be massaged until the kink comes out and water starts flowing. Provided it is beneficial, the fill hoses can be pulled out up to 1.6 m. It is important to understand that air has difficulty escaping the fully enclosed *AquaDam*[®] units without assistance. Construction personnel should walk down the unit toward the fill hoses at approximate internal water depths of 0.3 and 0.6 m.

The installation process of multiple *AquaDam*[®] units is illustrated in Figure 6.5. Figure 6.5a shows one filled *AquaDam*[®] unit with a coupling collar attached (the collar is attached while filling). The other end of this unit would contain the fill tubes. The second unit is placed directly behind the first then approximately 3 m of the dam and the fill tubes are unrolled (Figure 6.5b). The fill tubes are individually bunched together and wrapped with tape to facilitate insertion through the holes cut in the top of the coupling collar. Once the fill tubes are inserted through the collar, pumps can be placed on the previously filled unit (Figure 6.5c) on the order of 6 m behind the fill tubes. If desired, the pumps can be placed on plywood (or equivalent) to prevent damage to the previously installed unit. This is not essential but is an additional safety precaution. The final step prior to pumping the unit full in the same manner as the previous dams is to remove the tape from the previously bunched fill tubes and insert the discharge hoses into them. The hoses should be taped to the hoses to prevent them from slipping out (Figure 6.5d).



(a) One Filled Unit and Collar



(b) Second Unit Awaiting Attachment



(c) Pumps Sitting on Filled *AquaDam*[®]



(d) Discharge Hoses Taped to Fill Tubes

Figure 6.5. Installation of *AquaDam*[®] Units

Construction of enclosed walls for a water reservoir (or similar) is straightforward in concept when *AquaDam*[®] units are used. A schematic of the construction process is shown in Figure 6.6. The units are placed in the numbered order of Figure 6.6; thirteen units are needed for the configuration shown, but more or less could easily be used to meet a desired storage capacity. Unit 1 is placed with a collar on only one end and is a double closed-ended unit. Units 2 through 12 are subsequently placed with a collar on the closed end to allow connection of the next unit and are closed-ended units. Units 1 and 12 cannot be easily connected using standard construction practices and available collars, so they should be butted against each other and that location sealed with unit 13. Unit 13 could be a smaller height unit to allow more capacity in the reservoir since it only needs to seal the area between the structural units to prevent water inflow from outside the reservoir.

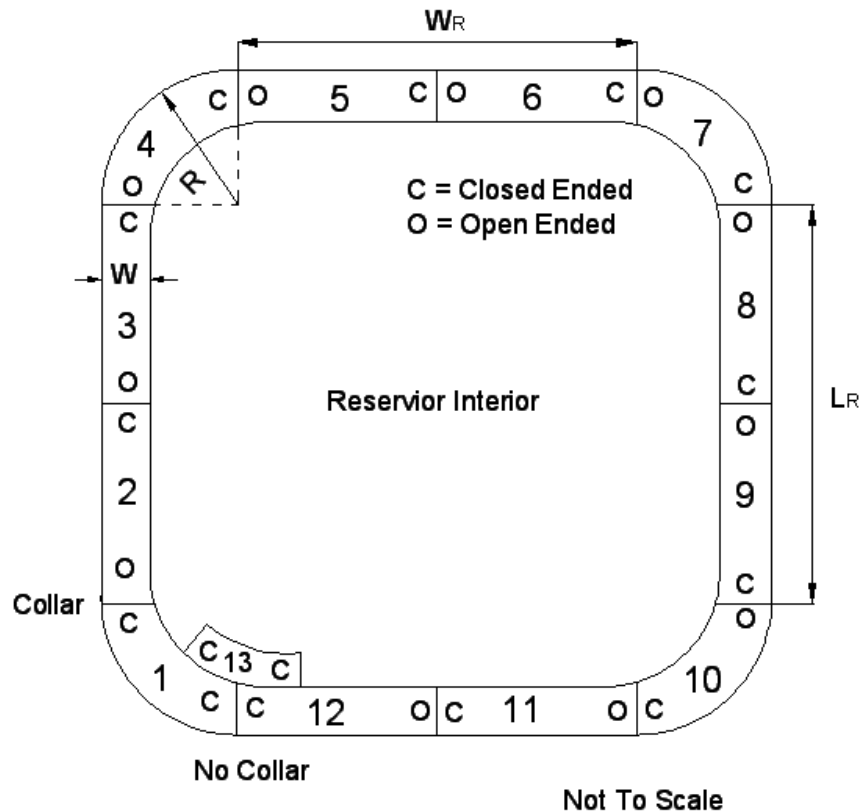


Figure 6.6. Plan View of Reservoir Constructed of AquaDam[®] Units

The radius of curvature of the corner *AquaDam*[®] units is on the order of 9.7 m measured from the exterior of the wall when 15.2 m lengths are incorporated. To place the corner units, tighten the inner rope and allow the outer rope to extend with little to no resistance while adjusting the pumping rates into the two inner tubes of the unit as necessary. There will not be a drastic difference in volume of water between the inner and outer tubes in the corner units, although it might appear so in Figure 6.6. In reality, the inner curve of the unit will not be smooth as shown, rather will resemble the photos shown in Chapter 1. Loss of stability due to less water in the inner tube is not expected to be significant, especially

since the arch shape should provide additional stability likely offsetting (at a minimum) the reduction in water volume of the inner tube.

A single pump with 11,000 L/min ($\approx 3,000$ gal/min) was used for construction time estimates to provide comparisons to geotextile tubes. It would take approximately 8 hr for a single pump of this size to fill all tubes needed for a reservoir of desired size. It would take approximately 18 hr for the same pump to empty the interior of the reservoir. As a practical comparison, the literature review provided in Section 6.2.2 showed that 240 m of 0.91 m geomembrane tube was installed in less than 7 hours. Full installation of a geomembrane tube water reservoir in well under a week appears feasible.

Material cost estimates were taken from Table 2.5; no pertinent construction cost information could be found. For 15 m of double closed end material 4.88 m tall, 290 m of closed end material 4.88 m tall, 15 m of double closed end material 1.52 m tall, and eleven attachment collars, purchase price was estimated at approximately \$500,000. Rental price would be approximately \$300,000. As minimal large equipment is needed for filling tubes with water, the cost for construction is believed to be less than the purchase price for materials. The water reservoir made from geomembrane tubes should be able to be constructed for less than \$1,000,000 based on the best available data.

6.5 Installation of Impervious Liners

Deployment of an impervious liner large enough to cover the water reservoir without seams is relatively straight forward. The liner is unfolded into place by two or more construction personnel and can be transported by a passenger vehicle and trailer. If desired, a small anchoring system can be placed outside the walls of the reservoir by driving anchors through the liner or mounding material over the edges of the liner. Figure 6.7 is an example of an impervious liner. Provided seams are necessary, they can either be performed prior to deployment or at the site of the reservoir. Seaming of a liner was beyond the scope of this research but is an existing and established technology.



Figure 6.7. Example Waterproof Liner After Deployment

CHAPTER 7 - CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The primary conclusions of the research are that: 1) geotextile and geomembrane tube walls appear to be feasible; 2) geomembrane tube walls are preferred to geotextile tube walls for construction of a fresh water reservoir in a disaster environment if geomembrane tubes are available; and 3) construction time estimates are not well established as truly comparable projects were not identified during the research. Relative construction time and cost are the two factors that led to the recommendation of geomembrane tubes over geotextile tubes for the temporary water reservoir application. Some applications in the realm of disaster recovery would favor use of geotextile tubes.

To effectively use geotextile tubes in a disaster environment, planning, training, and demonstration exercises are needed beyond that currently in existence according to the participants of the workshop discussed in Howard et al. (2009) and the research presented in this report. There is not a consensus among the engineering community regarding the best method to proceed with these approaches. Construction time of comparable projects in normal conditions could take a few weeks, a time that could be shortened with multiple crews working simultaneously. The amount the time reduction is not fully understood at present. Other conclusions from the research are provided in the bullets that follow.

- The best geotextile tube water reservoir consisted of twelve tubes each 61 m long (732 m total length) with a storage capacity of 9.3 million liters. The best estimate of cost in a disaster environment was over \$1,000,000, and the best estimate of a minimum construction time was one to two weeks.
- The best geomembrane tube water reservoir consisted of thirteen tubes of varying length (305 m total length) with a storage capacity of 11.3 million liters. The best estimate of cost in a disaster environment was less than \$1,000,000, and the best estimate of a minimum construction time was less than one week.
- Geotextile tube walls constructed by stacking three 9.14 m circumference tubes in a pyramid that have been filled with high moisture content fine grained cementitiously stabilized material appears to be a stable configuration with a wall height of 3.8 m. Horizontal shear failure, the ability of the bottom tubes to support the top tube, sliding of the entire wall, sliding of the top tube off the bottom tubes, and slope stability were investigated in the presence of lateral forces from water 3.05 m deep. A friction angle with the foundation at or greater than 25° , 0.3 kg/cm^2 or higher shear strength in the wall, and 0.4 kg/cm^2 or higher shear strength in the foundation were observed to provide a stable configuration based on the analysis conducted. The foundation was not analyzed for bearing capacity or seepage.
- Geomembrane tube walls constructed using one tube 9.76 m wide and 4.88 m tall was shown to be stable provided a friction angle resembling that of grass to a PVC geomembrane tube (i.e. 22°) was present with the foundation.
- A 1.5 mm thick HDPE liner is reasonable for use in the reservoir pending project specific details.

- No specialty equipment is required for routine construction in shallow water (e.g. 0.9 m) as at the *Matagorda Ship Channel*. Deep water (e.g. 3 m) construction procedures applicable to temporary water reservoirs does not appear to be well established.
- Use of a positive displacement pump and a clamshell bucket as at *Peoria Island* was an effective means for filling geotextile tubes with high moisture content fine grained material. Minor modifications (e.g. pugmill mixer) would allow this approach to fill geotextile tubes with cementitious stabilized slurry.
- Site visit test data indicated fine grained material at high moisture content can produce early strength comparable to sand, though it appears that sand will gain strength and produce higher shear strength at longer cure times.
- Short term volume change of high moisture content cementitious stabilized slurries was tolerable and should not be problematic during geotextile tube wall construction.
- Availability of geotextile tubes per 8 hr fabrication shift was obtained from all US manufacturers identified and was 800 to 1,050 m. One reservoir would require 732 m, so a moderate number of reservoirs could easily be produced.
- Availability of geomembrane tubes was given less consideration than geotextile tubes, though a cursory assessment is they would be less available. Sufficient quantities, though, are available to construct one or more reservoirs.
- The body of knowledge related to modeling geotextile tubes is less than that of geomembrane tubes in terms of external stability.
- The techniques investigated in this report could be useful to developing countries, or in offshore applications such as an island where the need for the reservoir may be longer term. The usefulness in the continental US will probably be limited by construction time.
- Geotextile and geomembrane tubes are very versatile as evidenced by project documentation from producers and through visit to select construction sites. The tubes have many applications in disaster recovery in addition to that shown in this report. Exterior walls of temporary emergency structures, exterior walls to facilitate dewatering an area for construction, and containment of problematic sediments are candidate applications.

7.2 Recommendations

The primary recommendations of this research are to: 1) develop a pre-certification system for geotextile and geomembrane tube installers who can be placed on a list for use in emergency response applications; and 2) increase the body of knowledge related to external stability of stacked geotextile tube walls. At least one geotextile tube fabricator maintains a list of pre-qualified contractors for routine applications. Other recommendations from the research are provided in the bullets that follow.

- Perform finite element modeling (or equivalent) to characterize the behavior of geotextile tubes when filled with cement stabilized slurry and stacked in a pyramid. This effort would require test data that was not found during review of literature. Instrumentation of geotextile tubes during construction of walls such as those seen at *Peoria Island* would be beneficial.

- Use the full scale facility recently constructed at the *Engineer Research and Development Center (ERDC)* that was funded by the *Department of Homeland Security (DHS)* to test stability of stacked geotextile tubes.
- Investigate alternative emergency construction options for geotextile tubes filled with cement stabilized slurry.
- Consider use of geotextile tubes to contain problematic sediments by stabilizing with cementitious materials.
- Perform full scale demonstrations of geotextile and/or geomembrane tube reservoirs prior to considering use in disaster applications. Stability and construction procedures should be monitored during the demonstration.

CHAPTER 8 - REFERENCES

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