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March 8, 2024

The Honorable Trevor Bunde
City of Colton
PO BOX 66
Colton, SD 57018

RE: Colton Phase 4 & 5 Sanitary Sewer CIPP Improvements – Manhole Rehabilitation

Dear Mr. Bunde and Council:

This letter is in response to the Manhole Rehabilitation that was performed on the Phase 4 & 5 Sanitary Sewer CIPP Improvements Project.

HK Solutions is a sub-contractor under Hulstein Excavating Inc. (General Contractor). They had performed the sanitary sewer lining rehabilitation work as well as the manhole lining rehabilitation scope of work. The manhole lining was completed in June 2023. Required testing on the liners was completed on November 7, 2023. The testing results indicate non-compliance with required adhesion and required liner thickness.

Following testing of the Manholes, a meeting with HK Solutions and Banner Associates, Inc. was conducted on January 3, 2024. During this meeting, HK Solutions reviewed the testing results and stated that the preparation work on the existing manholes was not completed properly. HK Solutions had also stated that their crews did not complete liner thickness measurements adequately.

Follow up meetings took place on January 17, 2024 and February 19, 2024. Attendees at these meetings included representatives from HK Solutions, Banner Associates and City of Colton. The follow up meetings resulted in discussions regarding the method of correcting the manhole liner and providing a product complying with the specifications. HK Solutions is proposing to install a new 1” thick liner in each manhole to provide a structurally independent manhole. This proposal would require surface preparation, re-lining of the manhole and testing to verify thickness compliance of 1”.

Through discussion and review of the current conditions and HK Solutions proposal for repair, Banner recommends that the City of Colton move forward with accepting the repair method of installing a new liner in each manhole structure. We have attached applicable coordination regarding the manhole rehabilitation to this letter.

Please contact me if you have any questions or comments regarding the manhole rehabilitation.

Sincerely,

Weston J. Blasius
Banner Associates, Inc.
Attachments: (12/4/2023) Report of Testing Results; Contact Report (1/3/2024); Meeting Minutes (1/17/2024); (2/14/2024)Memo from Vortex Companies; Meeting Minutes (2/19/2024); (2/19/2024) Letter from QUADEX; (3/6/2024) Email from Vortex Companies

C1583: Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method)

Colton Manhole Adhesion Testing
Colton, South Dakota

AET Report No. P-0027945

Date:
December 4, 2023

Prepared for:

Monica Ede
HK Solutions Group
1809 N. Terin Circle
Sioux Falls, SD 57107

Geotechnical • Materials
Forensic • Environmental
Building Performance
Petrography/Chemistry

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St. Paul, MN 55114-1804
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C1583: Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direction Tension (Pull-off Method)

SCOPE OF WORK

AET was asked to assess the bond of GeoKrete liner material to five existing concrete manholes in Colton, South Dakota in accordance with ASTM C1583-20: *Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength of Tensile Strength of Concrete Repair and Overlay Materials by Direction Tension (Pull-Off Method).*

Our services were requested by Monica Ede, Structure Rehab Program Manager at Hydro-Klean Solutions Group.

According to the Product Data Sheet, the GeoKrete geopolymer product should be applied by low-pressure spraying or the spin cast application process on horizontal or vertical surfaces to a monolithic minimum thickness of ½-inch for a protective layer to a new or non-corroded infrastructure and 1.0-inch for structural restoration of existing infrastructure.

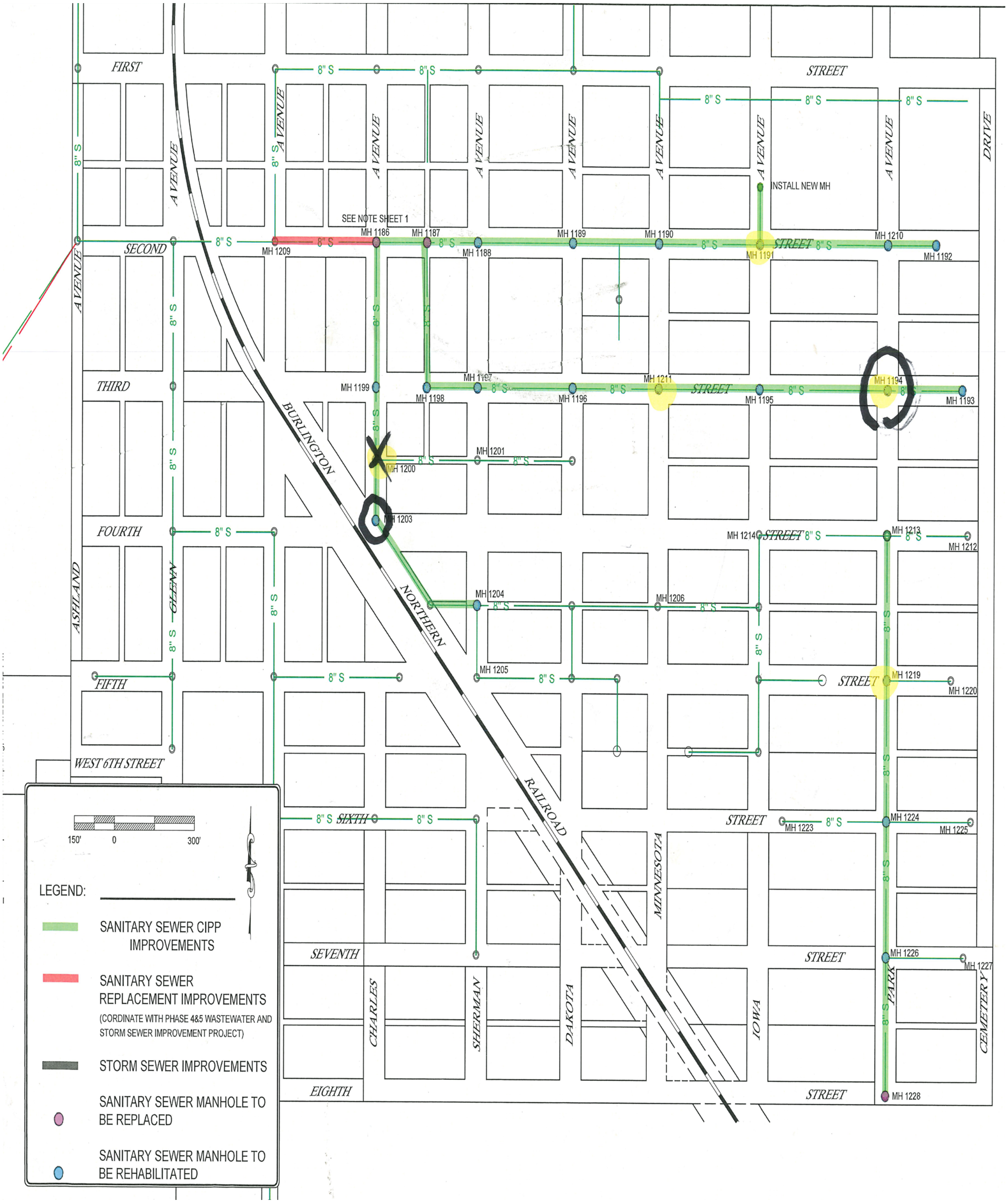
PROJECT INFORMATION

Hydro-Klean hired a subcontractor to apply the GeoKrete. Banner was the Engineer of Record for the project. AET does not know the date the GeoKrete was applied in each manhole. AET requested the testing be scheduled after the product specification cure date.

AET tested the following manholes on November 7, 2023: MH 1191, MH 1211, MH 1203, MH 1219, and MH 1194. MH 1200 replaced MH 1203 due to safety concerns. This change was approved by Reece Poppen from Banner. AET recorded an ambient temperature of 47° and cloudy, a surface temperature of 50°, a relative humidity reading of 77%, and a dew point reading of 40°. This was consistent within a few degrees across all manholes.

METHOD AND EQUIPMENT

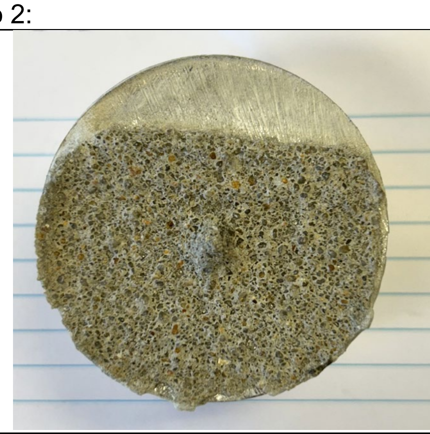
A 2-inch core barrel was used to core through the GeoKrete and roughly a half-inch into the substrate. The test location was then lightly ground down to clean and scarify the surface. J-B Weld Clear Epoxy Resin was then used to adhere the load fixture to the test area and a hold-down bracket was installed to allow the epoxy to cure for 24 hours. The HILTI Puller 28 was used to measure the adhesion of the GeoKrete to the substrate.

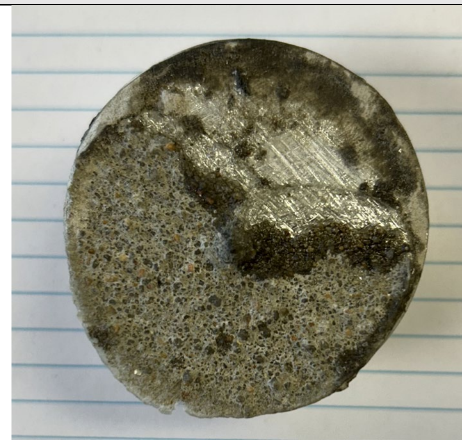

FIRST
SECOND
THIRD
FOURTH
FIFTH
WEST 6TH STREET
SIXTH
SEVENTH
EIGHTH
STREET
AVENUE
ASHLAND
GLENN
CHARLES
SHERMAN
DAKOTA
MINNESOTA
IOWA
PARK
DRIVE
CEMETERY
BURLINGTON
NORTHERN
RAILROAD
8" S
SEE NOTE SHEET 1
INSTALL NEW MH
MH 1186
MH 1187
MH 1188
MH 1189
MH 1190
MH 1191
MH 1192
MH 1193
MH 1194
MH 1195
MH 1196
MH 1198
MH 1199
MH 1200
MH 1201
MH 1203
MH 1204
MH 1205
MH 1206
MH 1209
MH 1210
MH 1211
MH 1212
MH 1213
MH 1214
MH 1219
MH 1220
MH 1223
MH 1224
MH 1225
MH 1226
MH 1227
MH 1228
150'
0
300'
LEGEND:
SANITARY SEWER CIPP IMPROVEMENTS
SANITARY SEWER REPLACEMENT IMPROVEMENTS
(CORDINATE WITH PHASE 4&5 WASTEWATER AND STORM SEWER IMPROVEMENT PROJECT)
STORM SEWER IMPROVEMENTS
SANITARY SEWER MANHOLE TO BE REPLACED
SANITARY SEWER MANHOLE TO BE REHABILITATED

TEST RESULTS

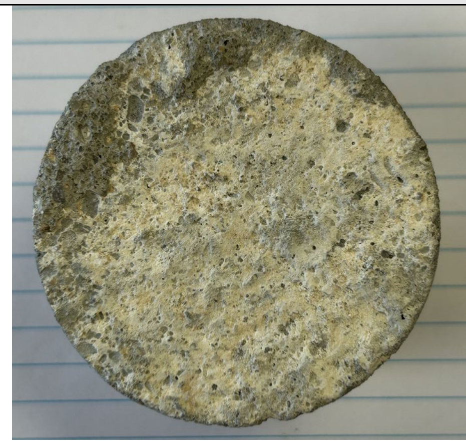
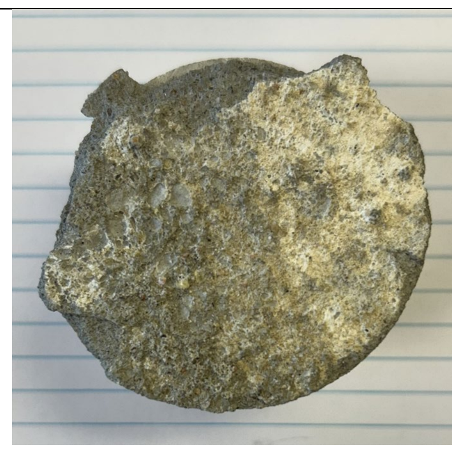
The results of our bond strength testing on November 7, 2023, are as follows:

Manhole 1191				
Location	**Load Fixture**	**Bond Strength (PSI)**	**Failure Mode**	**Notes**
MH 1191	A	31	50% Bond failure at epoxy/GeoKrete interface 50% Bond failure in GeoKrete	1. Undetermined thickness 2. Epoxy covered approximately 2/3 of the load fixture surface 3. Photo #2
	B	63	100% Failure in GeoKrete	4. Undetermined thickness. 5. Epoxy covered approximately 80% of the load fixture surface. 6. Photo #4
	C	28	90% Failure in GeoKrete 10% Bond failure at epoxy/GeoKrete interface	7. Undetermined thickness. 8. Epoxy covered approximately 70% of the load fixture surface. 9. Photo #6

MH 1191 Photographs		
MH 1191 Load Fixture A	 Photo 1:	 Photo 2:
MH 1191 Load Fixture B	 Photo 3:	 Photo 4:

MH 1191 Photographs		
MH 1191 Load Fixture C		
	Photo 5:	Photo 6:

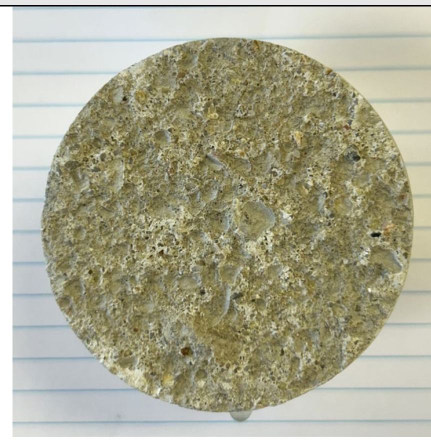
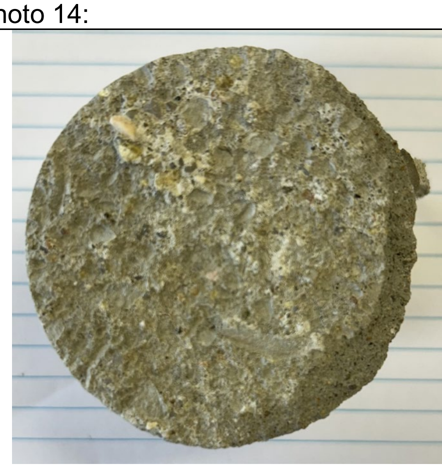
Manhole 1211				
Location	**Load Fixture**	**Bond Strength (PSI)**	**Failure Mode**	**Notes**
MH 1211	A	0	90% Bond failure at GeoKrete/substrate interface 10% Substrate failure	1. Under ½” recommended thickness 2. Possible substrate surface contamination (white substance) 3. Photo #8
	B	103	100% Bond failure in GeoKrete	1. Under ½” recommended thickness 2. Epoxy covered approximately 90% of the load fixture surface 3. Photo #10
	C	0	95% Bond failure at GeoKrete/substrate interface 5% GeoKrete failure	1. Under ½” recommended thickness 2. Possible substrate surface contamination (white substance) 3. Photo #12

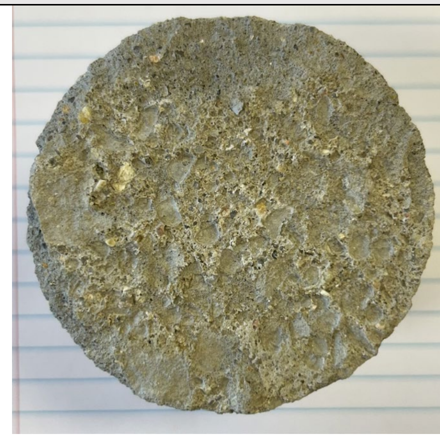
<table>
<tr><th colspan="3">MH 1211 Photographs</th></tr>
<tr><td rowspan="2">MH 1211
Load Fixture A</td><td></td><td></td></tr>
<tr><td>Photo 7:</td><td>Photo 8:</td></tr>
<tr><td rowspan="2">MH 1211
Load Fixture B</td><td></td><td></td></tr>
<tr><td>Photo 9:</td><td>Photo 10:</td></tr>
<tr><td rowspan="2">MH 1211
Load Fixture
C</td><td></td><td></td></tr>
<tr><td>Photo 11:</td><td>Photo 12:</td></tr>
</table>

Manhole 1194

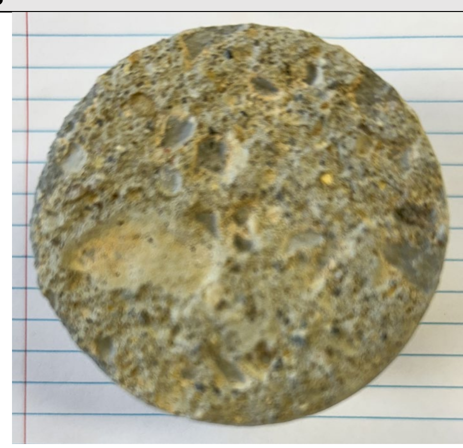
Location	Load Fixture	Bond Strength (PSI)	Failure Mode	Notes
MH 1194	A	15	95% Bond failure at the GeoKrete/substrate interface 25% Failure in substrate	1. Under ½” recommended thickness 2. Photo #14
	B	60	60% Bond failure at the GeoKrete/substrate interface 20% Failure in GeoKrete 20% Failure in Substrate	1. Under ½” recommended thickness 2. Photo #16
	C	31	85% Bond failure at the GeoKrete/substrate interface 15% Failure in substrate	1. Under ½” recommended thickness 2. Photo #18

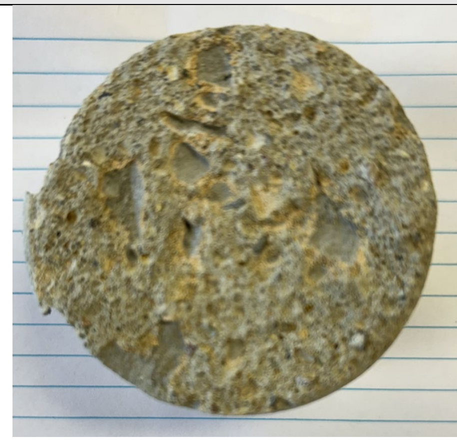
MH 1194 Photographs

MH 1194 Load Fixture A	 Photo 13:	 Photo 14:
MH 1194 Load Fixture B	 Photo 15:	 Photo 16:

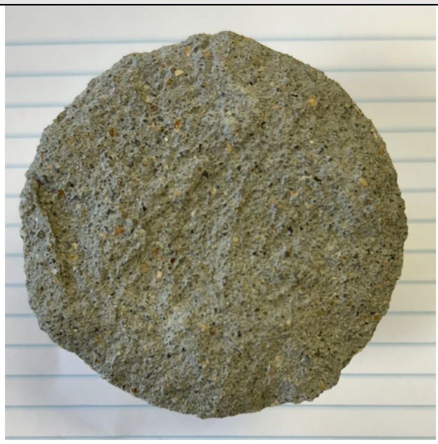
MH 1194 Photographs		
MH 1194 Load Fixture C		
	Photo 17:	Photo 18:

Manhole 1219				
Location	**Load Fixture**	**Bond Strength (PSI)**	**Failure Mode**	**Notes**
MH 1219	A	0	90% Failure of substrate 10% Bond failure at GeoKrete/substrate interface	1. Met recommended thickness. 2. Orange color in substrate from H2S contamination. 3. Photo #20
	B	0	85% Failure of substrate 15% Bond failure at GeoKrete/substrate interface	1. Under ½" recommended thickness 2. Orange color in substrate from H2S contamination. 3. Photo #22
	C	8	55% Failure in substrate 45% Bond failure at GeoKrete/substrate interface	1. Under ½" recommended thickness 2. Orange color in substrate from H2S contamination. 3. Photo #24

MH 1219 Photographs		
MH 1219 Load Fixture A		
	Photo 19:	Photo 20:

MH 1219 Photographs		
MH 1219 Load Fixture B		
	Photo 21:	Photo 22:
MH 1219 Load Fixture C		
	Photo 23:	Photo 24:

Manhole 1203				
Location	**Load Fixture**	**Bond Strength (PSI)**	**Failure Mode**	**Notes**
MH 1203	A	143	100% Bond failure in GeoKrete	1. Met recommended thickness. 2. Photo #26
	B	0	100% Bond failure in GeoKrete	1. Undetermined thickness 2. Epoxy covered approximately 65% of the load fixture surface. 3. Photo #28
	C	63	100% Bond failure in GeoKrete	1. Undetermined thickness 2. Epoxy covered approximately 90% of the load fixture surface. 3. Photo #30

<table>
<tr><th colspan="3">MH 1203 Photographs</th></tr>
<tr><td rowspan="2">MH 1203
Load Fixture A</td><td></td><td></td></tr>
<tr><td>Photo 25:</td><td>Photo 26:</td></tr>
<tr><td rowspan="2">MH 1203
Load Fixture B</td><td></td><td></td></tr>
<tr><td>Photo 27:</td><td>Photo 28:</td></tr>
<tr><td rowspan="2">MH 1203
Load Fixture C</td><td></td><td></td></tr>
<tr><td>Photo 29:</td><td>Photo 30:</td></tr>
</table>

OBSERVATIONS

Item of concern in MH 1194; a large section of uncoated substrate.

MH 1191-Pucks A, B, and C have less than 1/16th of H_2S-affected concrete adhered to the disc of the repair mortar. The substrate had not been cleaned properly to remove all the affected concrete.

MH 1211-Pucks A and C have less than 1/16th of the H_2S-affected concrete adhered to the disc of the repair mortar.

MH 1194- Pucks A, B, and C have less than 1/16th of H_2S affected concrete adhered to the disc of the repair mortar. The substrate had not been cleaned properly to remove all the affected concrete.

MH 1219-pucks A, B, and C have less than 1/16th of H_2S-affected concrete on the disc of the repair mortar. The substrate had not been cleaned properly to remove all the affected concrete.

Thank you for the opportunity to partner with you. Please contact David or Christine with comments or questions.

Sincerely,

AET Representatives:

Christine M. Snider
Project Manager
csnider@teamAET.com
651-659-1376

David Montgomery
Technician III
dmontgomery@teamAET.com
651-206-0292

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CONTACT REPORT

DATE: **1/3/2024**

CONTACT			
FIRM	HK SOLUTIONS	TEL	
NAME	MICHAEL INGHAM, MATT HUSTON, TIMOTHY BUZIC, MONICA EDE, ADAM VALENZUELA JOHN BLUNLACH,	EMAIL	
FIRM	BANNER ASSOCIATES, INC		
	WESTON BLASIUS, REECE POPPEN		

SUBJECT	COLTON SANITARY SEWER IMPROVEMENTS – MANHOLE REHABILITATION: LINING		
PROJECT	PHASE 4 AND 5 - COLTON SANITARY SEWER IMPROVEMENTS	BAI NO.	23872.00

TEAMS CALL BETWEEN HK SOLUTIONS AND BANNER ASSOCIATES, INC. TO DISCUSS ADHESION TESTING RESULTS OF THE CEMENTITIOUS LINER INSTALLED IN MANHOLES FOR THE PHASE 4 AND 5 SANITARY SEWER IMPROVEMENTS PROJECT.

HK SOLUTIONS (MICHAEL INGHAM) STATED THAT THE ADHESION TESTING RESULTS SHOWED THAT THE INSTALLED CEMENTITIOUS LINER FOR THE 5 MANHOLES TESTED DOES NOT MEET THE PROJECT SPECIFICATIONS.

HK SOLUTIONS (MATT HUSTON) STATED THAT THE LIKELY CAUSE WAS INADEQUATE PREPARATION OF THE MANHOLE. HE ALSO STATED THAT THE LINER THICKNESS WAS LESS THAN WHAT IS REQUIRED IN THE SPECIFICATIONS.

BANNER ASSOCIATES (WESTON BLASIUS) ASKED FOR A PROPOSED CORRECTIVE ACTION.

HK SOLUTIONS (MATT HUSTON) PROPOSED INSTALLING ADDITIONAL LINER MATERIAL TO PROVIDE MORE STRUCTURAL INTEGRITY.

BANNER ASSOCIATES (WESTON BLASIUS) BROUGHT UP CONCERN WITH THE ADDITIONAL LINER MATERIAL NOT SOLVING THE INADEQUATE ADHESION.

HK SOLUTIONS (MICHAEL INGHAM) PROPOSED THAT HK PROVIDE A 5 YEAR WARRANTY ON THE WORK WITH A POST LINER INSPECTION AT 30 MONTHS.

BANNER ASSOCIATES (WESTON BLASIUS) ASKED IF REMOVAL AND REPLACEMENT OF THE LINER WAS AN OPTION.

HK SOLUTIONS (MATT HUSTON) SAID THERE WOULD BE CONCERNS WITH STRUCTURE DAMAGE DUE TO THE MECHANICAL REMOVAL PROCESS.

BANNER ASSOCIATES (REECE POPPEN) QUESTIONED THE LINER INTEGRITY ISSUE AT MANHOLE #1194.

HK SOLUTIONS (MATT HUSTON) STATED THAT THIS IS DUE TO A PINHOLE LEAK AND SHOULD HAVE BEEN REPARIED PRIOR TO LEAVING THE PROJECT.

BANNER ASSOCIATES (REECE POPPEN) INQUIRED ABOUT THE FAILURE MODE IDENTIFICATION REGARDING SUBSTRATE FAILURE AND BOND FAILURE IN GEOKRETE.

HK SOLUTIONS (MICHAEL INGHAM) SAID THAT HK WILL REACH OUT TO THE TESTING FIRM AND REQUEST AN EXPLANATION ON THE TYPES OF FAILURE MODE.

AT THE END OF THE CONVERSATION, BANNER REQUESTED THAT HK SOLUTIONS SEND A PROPOSED REPAIR FOR THE ADHESION ISSUES. BANNER WILL DISCUSS THE TESTING REPORT WITH THE CITY OF COLTON AND THEN REVIEW HK'S PROPOSED REPAIR. BANNER REQUESTED THAT CORRESPONDENCE GO THROUGH THE GENERAL CONTRACTOR.

PREPARED BY

WESTON J. BLASIUS

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MEETING MINUTES

DATE	17 January, 2024	
PROJECT	Colton Phase 4&5 Sanitary Sewer Improvements	BAI No.23872.00
SUBJECT	Unacceptable Manhole Rehabilitation Lining – Meeting No. 02	
LOCATION	Teams	
ATTENDEES	HK Solutions Group – Matt Huston, Michael Ingham	
	City of Colton – Jerrit Pedersen	
	Banner Associates – Weston Blasius, Reece Poppen	

PURPOSE OF MEETING

TO DISCUSS THE UNACCEPTABLE MANHOLE REHABILIATION WORK AND WORK TOWARDS A RESOLUTION.

Weston started the meeting by reviewing notes from the previous meeting by stating that there was adhesion failure and a deficiency in the thickness that was applied. He also stated that Banner has reached out to some industry professionals as well as some other Banner co-workers that have experience in manhole rehabilitation.

Weston stated the goal of the rehabilitation efforts was to update the lifespan of this area to match the rest of the system.

Banner's recommendation to resolve the issue of unacceptable work is to remove and replace the current liner with a liner that is acceptable. Matt (HK) stated that testing each structure to determine which structures would need additional work would be expensive. Weston asked Matt (HK) if the areas of low bond strength would delaminate easily once the process had been started; Matt (HK) agreed.

Matt (HK) stated that HK wants to make this right and HK proposes building an additional 1" of GeoKrete to the existing liner to make this more of a structural liner. Matt (HK) stated he would be concerned about damaging the host structure if the existing liner was to be removed.

Weston asked if HK had other communities where this proposal was completed that could be used as a "case study". Matt (HK) stated that this proposal was used in the city of Volga, with Banner as well.

Matt (HK) expressed concern with the test process. He stated he thinks that when the core is being performed, the cutting action weakens the bonds within the material and the bond to the host structure.

Weston asked Matt (HK) if a good bond is necessary, and Matt (HK) replied that he did not know.

Matt (HK) stated that HK's concern is the thickness of the lining material, not the lack of bond strength. He also said they would reach out to the manufacturer for their recommendations to resolve this issue.

Michael (HK) stated that they had bad project management on their end. He said there should have been pictures of the number of bags used on each manhole to ensure the correct thickness was applied, and this was not completed.

Weston stated that whatever the course of action is determined to be between Banner and the Contractor, it will need to be presented and approved by the City Council. Michael (HK) stated he would be willing and intends to attend the City Council Meeting where this is discussed to present their proposal.

PREPARED BY Reece Poppen

REECE POPPEN, STAFF ENGINEER

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MEETING MINUTES

DATE	2/19/2024	
PROJECT	Colton Phase 4&5 Sanitary Sewer Improvements	BAI No.23872.00
SUBJECT	Unacceptable Manhole Rehabilitation Lining – Meeting No. 03	
LOCATION	Teams	
ATTENDEES	HK Solutions Group – Matt Huston, Tim Buzick, John Bluntach, Michael Ingham, Adam Valenzuela	
	Hulstein Excavating Inc – Jeff Haas	
	Banner Associates – Weston Blasius, Beth Neimeyer, Reece Poppen	
	City of Colton – Jerrit Pedersen	

PURPOSE OF MEETING

TO DISCUSS THE UNACCEPTABLE MANHOLE REHABILIATION WORK AND WORK TOWARDS A RESOLUTION.

DISCUSS THE REPORT FROM VORTEX REGARDING THE MANHOLE REHAB ISSUES.

Matt (HK) walked through the memo from Vortex to discuss the results. Vortex reached out to a third-party structural engineer to check the design calculations. The structural engineer used the following conditions for the calculation check: HS-25 Single Wheel Loading and Ground Water at the surface (full depth of the manhole). The calculations determined the 1" thick liner will satisfy the standards. This includes a minimum bond strength between liner layers of 35 psi and a CSR profile of 5. This would also include the proper surface prep of the existing liner to address the H_2S environment.

Weston Asked what it would take for the existing liner to meet the CSR Profile 5? Matt (HK) stated that this would be discussed with Vortex to see what processes they would recommend meeting this requirement. Matt guessed this would be using a higher-pressure pressure washer or abrasive media.

Weston asked how the minimum bond strength between the existing liner and the additional liner Matt suggested some mock samples would be created to simulate this bond and that could be evaluated. This would also be discussed with Vortex to gain their insight as to how this could be evaluated.

Matt offered his opinion based on the calculations that the additional 1" of lining material would be a structural component to the point where he thinks the bond strength to the host structure is more irrelevant.

Weston asked for clarification of the work plan if HK intends to add 1" of lining material or if they intend to reach a total thickness of 1". Matt responded by stating that a total of 1" would be the goal, but an internal meeting would be held to determine if HK would aim for a total of 1" or if they would add an additional 1" of material. Michael further clarified that the work plan would consist of adding an additional 1" of GeoKrete.

Weston asked that the manufacturer's intent would be for this product, if they would be intending for 1" with no bond to the host structure? Weston also asked why the manufacturer states in the application procedure "This will provide a clean, damp surface to allow for a good bond". Matt offered the thought if there is a suitable structural component with a 1" thick liner, why would they spend the time with the prep work to begin with. John stated the bond considerations are for the bond of the material during application to ensure the material does not slough off before it is cured. Matt offered the prep would be to ensure there is not a contamination issue with the added lining material. Weston further asked what a "good bond" is and how that could be measured? Matt stated he was unsure but was willing to facilitate introductions between Banner and Vortex to address this question/concern.

Reece asked what method would be used to ensure there would be 1" of material added. He also noted that the field staff onsite during the initial application noted the installer used their finger to measure the thickness. Matt responded by stating that there are many ways the installation crew can achieve this measurement. They can use a wet-film thickness gauge. The crew could install pins along the manhole that are 1" long to ensure they are installing 1" of material to cover the pins.

Weston asked Beth if she had been a part of a similar remediation in the past. Beth stated she has not. Matt jumped into state that there was a similar issue in Volga that was resolved using the same approach of adding more material.

Weston stated that the City Council will look to Banner for their recommendations. He stated that he was trying to think of the questions that the council might have to have answers for them before presenting this option. He stated he could think of the following: if it is a 1" liner, does this turn structural; would there be a reduced service life of the lining if there is no bond to the host structure? Matt responded by stating that the report from Vortex stated that this approach will work, and they claim to have a 50-year design life.

Jerrit asked what the design life would be for a new pre-cast manhole? Weston said that would be dependent on the type of system it is included in. Weston asked when the current pre-cast structures were installed and Jerrit replied this was back in the 60s and 70s, so the current lining could be comparable to the system that is being rehabilitated.

Reece asked Matt if there would be consideration to additional adhesion testing on the bench or lower in the manhole. Matt responded by asking if the existing structure was evaluated to assess the soundness of the structure before the lining was applied/assessed. This was not done. Matt stated the leaky manhole would be sealed with grout or additional material before the additional 1" would be applied. Matt stated that he has seen small leaks not appear until after the cure process. Matt also stated this additional testing would be expensive and he is not sure this would provide any information that is not already available.

Weston summarized the options into two categories: HK recommending adding 1" of material, and the other option being removing the material that is not bonded and replacing to get the bond. Matt added that removal could result in damaging the host structure.

Weston summarized some concerns that the council could have such as: are we accepting a product that is less than specified; and what would deducts look like? Matt responded by stating they could have this discussion for an extended warranty. Matt stated that the same design calculations are used for brick structures, where they are not as round as a pre-cast structure.

Weston asked Reece to confirm if there was a manhole that was evaluated that had H_2S present. Reece responded by stating he was not positive if there was more than one, but there was one that had some discoloration on the substrate that is due to H_2S. Matt also confirmed this. Weston continued to ask how much of an issue this is to have this deterioration behind the liner. Matt stated this is not an issue. The deterioration would be isolated from the new lining so it should not be an issue for applying the additional lining material.

Weston asked if HK would be willing to have a representative present for the Council Meeting to present this to the Council. Matt stated that he can reach out to Vortex, and he is sure they would be willing to attend as well. Matt asked when the next council meeting is scheduled. Weston and Jerrit discussed it would be the second Monday in March, so the 11th of March. Mike stated he plans to attend with Vortex.

Action Items:

Matt stated he will reach out to Vortex to discuss the CSP 5 requirements and test methods for determining the existing material bond strength to the new material.

Weston stated he would attend the Council Meeting and consult with some individuals involved in Volga to determine if this is a viable option.

Weston asked for a contact with Vortex and Matt responded by saying he will reach out to make sure he is introducing the correct person, and he will include everyone in the meeting and the invitation is open to others that should be looped in.

Weston stated that the service life will be a large factor in determining what the option will be moving forward. He asked if Vortex would be able to stand in the Council Meeting and state this will have a 50-year life? Matt stated that he believes this is possible and that he will add this to the list to discuss with Vortex.

Jerrit stated that he feels comfortable with these steps that have been discussed so far.

The meeting ended at 2:13pm.

PREPARED BY ______________________________

REECE POPPEN, STAFF ENGINEER

February 14, 2024
File No.: 220103-001

Vortex Companies
18150 Imperial Valley Dr.
Houston TX, 77060

Attention: Mr. Josh Marazzini
Technical Director

Subject: Thickness Design for 48" Manhole Hole - Opp- 2428251
Colton, SD

Mr. Josh Marazzini,

As requested by Vortex Companies, we have estimated the minimum liner thickness for the referenced rehabilitation project using utilizing GeoKrete. GeoKrete is a high-strength geopolymer designed to provide corrosion-resistant protection in a high hydrogen sulfide environment, increase structural integrity, and stop groundwater infiltration in deteriorated structures. The GeoKrete geopolymer reaction mechanism is polymerization, which yields superior strength and chemical/abrasion resistance without producing high hydration heat, thus eliminating shrinkage. In addition, GeoKrete has excellent bonding performance to materials commonly found in wastewater and stormwater systems. The datasheet for GeoKrete is presented in Attachment (1). All mechanical properties of GeoKrete are presented in Attachment (1).

Design Approach

As requested, we have reviewed the data provided by Vortex Companies to determine the required liner thickness to meet structural rehabilitation requirements. The approach is based on using the modified failure theories provided in ASTM F1216-16, while considering the deflection limit of 1% for GeoKrete in addition to using tunnel analysis to examine the liner response to anticipated loading. Our review includes independent calculations for verification of the minimum thickness results.

Our review was based on the following parameters

- The information, and documents provided to us (through e-mail) by your firm.
- Data in Table 1.

We have assumed:

a) Data provided by Vortex Companies and data provided by the Owner are appropriate for the Owner's design.
b) The provided height of soil and groundwater depth, and MH dimensions for the design calculations are correct.

c) The installed liner parameters would meet or exceed the design parameters used in the analyses that are presented in Table 1.

We have reviewed the design parameters, and in our opinion, the calculated minimum design thickness of 0.5” meets the requirements to resist the assumed loads per the attached design calculations. The proposed 0.5” liner is sufficient up to depths 15 VF depending upon the horizontal stress along the perimeter of the MH and meets the design requirements for the deepest MH on the subject project. The detailed calculations are presented in Attachment (2).

Where applicable, we have assumed that the liner will experience live loads resulting from an HS-25 truck. Should it be anticipated that this will be exceeded during the liner service life, we recommend being notified so that we can review the design accordingly.

For circular MH, it is anticipated that the pressure along the perimeter of the MH, at any given depth, is assumed to be uniform. Thus, no moment should be anticipated; however, to be conservative, it was assumed that the lateral pressure at two perpendicular directions can vary by 5%. The induced moment was calculated using the flexibility ratio approach presented by Peck et al. (1972), which is included in Attachment (3).

It is worth noting the following:

1. Once the 1.0-inch total installation of GeoKrete is installed, adhesion properties to the host will not matter, as the GeoKrete will take on all future loading to the asset once cured.
2. Preparation between layers of GeoKrete should be sufficient so as to achieve a CSP5 or greater.
3. Adhesion between the existing GeoKrete installation and the new installation is recommended to be 50 psi but shall not be less than 35 psi.

For the Circular MHs, the finished installed thickness must be equal to or thicker than the design minimum thickness for the stated assumptions.

We recommend that design assumptions and dimensions such as depth of cover, depth of water table, shape of the MH, mechanical properties, and water tightness of the installed liner be checked in the field during construction and that we be notified accordingly if any assumptions or design parameters are different than those used in our design so we can modify the design accordingly.

All manufacturer guidelines and instructions for the use and application of the product shall be followed.

Table 1: Design Parameter Values

Design Parameter	Value
Maintenance Holes Condition – Circular	Fully Deteriorated
• Depth Of Manhole	14.1 ft
• Depth of Groundwater	At surface
• Flexural Modulus (Long-term/Short-term)	4,000,000 psi (34,473.79 MPa)
• Flexural Strength (Long-term/Short-term)	800 psi (6.89 MPa)
• Compression Strength (Long-term/Short-term)	8000 psi (62.05 MPa)
Soil Density	130 pcf (1,922 kg/m3)
Soil Modulus, E's	1,000 psi (6.9 MPa)
Ovality - Circular	1.0%
Poisson's Ratio	0.2
Live Load (AASHTO Method)	Highway HS-25
K Factor	7.0
Coefficient of at rest soil pressure (k_0)	0.67

We appreciate the opportunity to be of service to you on this project. Please call with any questions.

Sincerely,
TU GEOSTRUCTURAL, LLC
Mohamed Gamal, PhD, PE
Geo-Structure Engineer

Attachments –
Attachment 1 – GeoKrete MH Liner Datasheet
Attachment 2 – Design Calculations
Attachment 3 – Peck et all (1972) Paper

ATTACHMENT 1

GeoKrete Datasheet

GEOKRETE® GEOPOLYMER

TECHNICAL DATA SHEET

Rev. 09-2020

QUADEX® A VORTEX COMPANY | REPAIR MATERIALS

Typical Performance Characteristics*

- Compressive Strength (ASTM C39 & C109)
  28-days >8,000 psi | 55.1 MPa
- Flexural Strength (ASTM C78)
  28-days >800 psi | 5.5 MPa
- Bond Strength (ASTM C882)
  28-days >3,000 psi | 20.7 MPa
- Modulus of Elasticity (ASTM C469)
  28-days = 5.49 x 10^6 psi | 37.8 GPa
- Chemical Resistance (ASTM C267)
  0% mass loss in 12 week
  sulfuric acid @ pH 1.0 immersion
- Chloride Ion Penetration Resistance
  (ASTM C1202)
  28-Day < 250 Coulombs (very low)
- Split Tensile Strength (ASTM C496)
  28-days >900 psi | 6.2 MPa
- Shrinkage (ASTM C1090)
  28-days ≤ 0.02%
- Freeze Thaw (ASTM C666)
  No visible damage after 300 cycles
- Abrasion Resistance (ASTM C1138)
  6 Cycles at 28 Day – loss <1.0%

** The values stated in inch-pound units are to be regarded as the standard. The values given in International System are for information only.*

CERTIFIED REDUCED CARBON FOOTPRINT™ SCS GLOBAL SERVICES

Compared to Baseline for Trenchless Repair Systems for Structural Rehabilitation of Civil Infrastructure

STRUCTURAL REHABILITATION MORTAR

DESCRIPTION

Quadex® GeoKrete® geopolymer is formulated to provide corrosion resistant protection in a high hydrogen sulfide environment, restore structural integrity and eliminate the infiltration of groundwater in deteriorated structures. GeoKrete is a factory blended, one-component (just add water), eco-friendly, micro-fiber reinforced geopolymer mortar synthesized from reactive SiO_2 and Al_2O from industrial byproducts, enhanced with monocrystalline quartz aggregate. The GeoKrete geopolymer reaction mechanism is alkali-activated polycondensation which yields superior physical properties and chemical resistance. It can be applied in one pass up to several inches thick on horizontal or vertical surfaces by low pressure spraying or spin cast application process.

RECOMMENDED FOR

Structural restoration of large diameter pipes, culverts and tunnels, including raw, storm and wastewater, consisting of metal, concrete, stone, masonry and others. Other structures such as manholes, wet-wells, and treatment plant structures also benefit from the superior strength and corrosion resistance properties of this advanced geopolymer material.

FEATURES AND BENEFITS

- Quality controlled, one-component blend for uniform results.
- High early and ultimate compressive, flexural and bond strengths.
- Resistant to acid attack in wastewater streams with pH as low as 1.0 and temperature exceeding 212°F | 100°C for industrial effluent.
- Extremely low permeability.

PACKAGING

GeoKrete geopolymer is supplied in 60 lb. | 27.2 kg. poly-lined bags or 1,000 lb. | 454kg super sacks.

YIELD

One 60 lb. | 27.2 kg bag of GeoKrete geopolymer will yield approximately 0.45 cu. ft. | 0.013 cu. m. and will cover 10.8 sq. ft. | 1.0 sq. m. at a 1/2-inch | 12.7 mm thickness.

PROCEDURE

Prepare surface to be patched by removing unsound concrete, dirt, dust, oil and other debris using high pressure (3,500 PSI | 241.3 bar) water blasting. Stop active infiltration. Then rinse with potable water to remove all remaining dirt, sand and loose debris. This will provide a clean, damp surface to allow for a good bond.

Use approximately 0.48 to 0.57 gallons | 1.82 to 2.16 liters of potable water per 60 lb. | 27.2 kg bag of GeoKrete geopolymer. For 1,000 lb. | 454 kg. supersack use approximately 8.0 to 9.5 gallons | 30.3 to 36 liters of potable water. First add water to mixer, start the mixer and add GeoKrete geopolymer until mortar is completely mixed. Once all geopolymer material and water has been added to mixer, it needs to mix for approximately five (5) minutes prior to being transfered into the material hopper. Once fully mixed, additional water may be added, as approved by Quadex, should it be necessary for proper consistency.

Apply GeoKrete geopolymer by low pressure spraying or the spin cast application process on horizontal or vertical surfaces to a monolithic minimum thickness of 1/2-inch | 12.7 mm for a protective layer to new or non-corroded infrastructure and 1.0-inch | 25.4 mm for structural restoration of existing infrastructure.

CURING

Cure in accordance with manufacturer's recommendations.

WARRANTY

Quadex warrants its products to be free of defects in material and workmanship. Quadex will replace any product proved to be defective when applied in accordance with manufacturer's instructions. Quadex's obligation shall be limited solely to such replacement. There are no other warranties by Quadex, expressed or implied.

PRECAUTIONS

Avoid eye contact or prolonged contact with skin. Wash thoroughly after use. Persons using Quadex GeoKrete geopolymer should wear necessary PPE consisting at minimum of eye protection, dusk mask and rubber gloves. Read all product labels and technical literature prior to use.

ATTCHMENT 2

Design Calculations

INPUT DATA

Thickness Design For Manhole Rehabilitation

Manhole Inner diameter (ID)	=	$D_{ID} := 4 \cdot ft$
Surcharge Pressure due to traffic, crane load etc	=	$p_{surcharge} := 0 \cdot psi$
Depth to bottom of Manhole	=	$H_m := 14.1 \cdot ft$
Water Depth Below ground surface	=	$d\_w := 0 \cdot ft$
Ovality (%)	=	$Ov := 1\%$
Long term Liner Modulus	=	$E_{liner\_i} := 4000 \cdot ksi$
Soil/rock Elastic Modulus Modulus	=	$E_m := 3000 \cdot psi$
Soil / rock Modulus of Horizontal Reaction	=	$E_{prime} := 1000 \cdot psi$
soil / rock Poisson's ratio	=	$\nu_m := 0.33$
Flexural Strength	=	$f_{flex} := 800 \cdot psi$
Compression Strength	=	$f_{compression} := 8000 \cdot psi$
Factor of Safety	=	$FS := 2.0$
Confinement enhancement factor	=	$K_e := 9.0$
Liner Poisson's Ratio	=	$\upsilon_{liner} := 0.18$
Unit weight of fluid	=	$\gamma_{water} := 62.4 \cdot pcf$
Total Unit weight of soil	=	$\gamma_m := 130 \cdot pcf$
ratio of horizontal pressure along two perpendicular directions	=	$Ko := 0.95$
Ratio of vertical to horizontal pressure	=	$K_m := 0.5$
Load factor	=	$Ld_{factor} := 1.4$
Strength reduction factor (compression)	=	$\phi := 0.9$
Strength reduction factor (bearing)	=	$\phi_b := 0.9$
Contac width of axle wheels	=	$L_{contact} := 20 \cdot in$

Allowable diametric deformation = $\varepsilon_{diametric} := 1\%$

Strength Reduction Factor for Plain Concrete Compression = $\phi c := 0.6$

Allowable Deflection ratio = $\Delta := 1\%$

Unit weight of Liner = $\gamma_{liner} := 135 \cdot pcf$

Maximum diametric deformation for CIPP used in ASTM F1216 = $\Delta_{cipp} := 5\%$

Show Radial and Tangential Stresses Considering:

- (●) Both Hoirzontal and Vertical In-Situ Stresses
- ( ) Vertical In-Situe Stress Only Ignroing In-Situ Horizontal Stress
- ( ) Horizontal In_Situ Stress Only Ignroing Vertical In-Situ Stress

Choose Soil/liner Slippage Condition

- ( ) No Slip Condition - Stiff Soil
- (●) Full Slip Condition - Soft Soils

Is the existing host fully or partially deteriorated

- (●) Fully Deteriorated or No Host or New Casing
- ( ) Partially Deteriorated

INPUT DATA

Water height above bottom of Manhole = $H\_w := H_m - d\_w = 14.1\ ft$

Long Term Liner Modulus = $E_{liner} := E_{liner\_i} = 4000 \cdot ksi$

Determine Different Pressures Acting on Liner :

Let $N_{sf} := FS$

Let C_{ovrf} = Ovality reduction factor where: $C_{ovrf} := \left[\frac{(1 - Ov)}{(1 + Ov)^2}\right]^3 = 0.91$

Thickness Design

Height of soil above bottom of Manhole = $H_m = 14.1 \cdot ft$

Height of water above bottom of Manhole = $H\_w = 14.1\ ft$

Determine Minimum Thickness Assuming Partially Deteriorated Case

Water buoyancy factor = $R_w := 1 - \frac{H\_w}{H_m} \cdot 0.33 = 0.67$ (Minimum is 0.67)

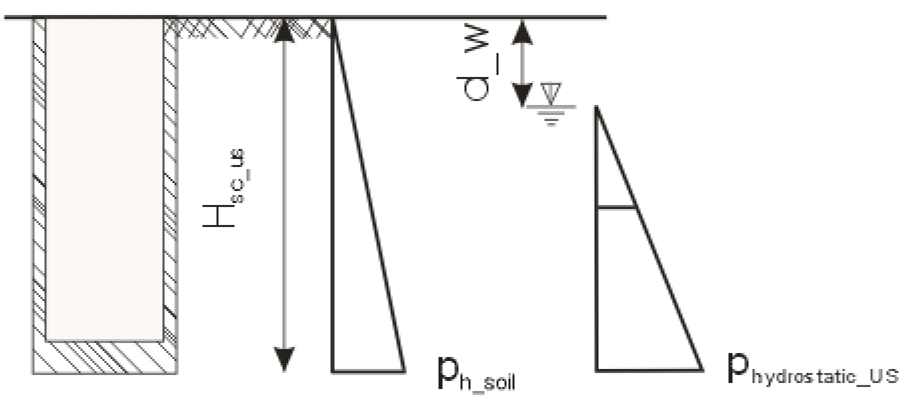
Summary of Verified Variables Values

Horizontal Hydrostatic pressure at bottom Liner= $P_{Hydrostatic} := (\gamma_{water} \cdot H\_w) = 879.84 \cdot psf$

Soil horizontal pressure at bottom of Liner= $P_{h\_soil} := K_m \cdot \gamma_m \cdot d\_w + K_m \cdot (\gamma_m - \gamma_{water}) \cdot (H_m - d\_w)$

$P_{h\_soil} = 476.58 \cdot psf$

Total pressure for at bottom of Liner = $q_{t\_US} = 9.42\ psi$



Manhole

Soil horizontal pressure

Water horizontal pressure

Determine minimum thickness using the controlling restrained buckling equation under hydrostatic pressures is given as

$$t_{liner\_X1.1\_US} := \frac{D_{ID}}{\left[\frac{2 \cdot K_e \cdot E_{liner} \cdot C_{ovrf}}{P_{Hydrostatic} \cdot N_{sf} \cdot \left(1 - \upsilon_{liner}^{2}\right)} \cdot \frac{\Delta}{\Delta_{cipp}}\right]^{.33333333} + 1} = 11.65 \cdot mm$$

(Equation X1.1 - ASTM F1216)

Determine the Thickness for Fully Deteriorated Case or New Liner

Coefficient of elastic support = $B_{prime} := \left(\frac{1}{1 + 4 \cdot e^{-0.065ft^{-1} \cdot H_m}}\right) = 0.38$

$$q_{t\_US} = \frac{1}{N_{sf}} \left(32 \cdot R_{W\_US} \cdot B_{prime} \cdot E_{soil} \cdot C_{ovrf} \cdot \frac{E_{liner} \cdot t_{liner\_X1.3}^{3}}{12 \cdot D_{ID}^{3}}\right)^{0.5}$$

(Equation X1.3 - ASTM F1216)

$$t_{liner\_X1.3\_US} := \left[\frac{\left(q_{t\_US} \cdot N_{sf}\right)^{2.0} \cdot \left(12 \cdot D_{ID}^{3}\right)}{\left(32 \cdot R_w \cdot B_{prime} \cdot E_m \cdot C_{ovrf} \cdot E_{liner}\right)} \cdot \frac{\Delta_{cipp}}{\Delta}\right]^{\frac{1}{3}} = 7.53 \cdot mm$$

$\frac{E_{liner\_i} \cdot t_{liner\_X1.4}^{3}}{12 \cdot D^{3}}$ should be greater than 0.093 ***(Equation X1.4 - ASTM F1216)***

$$t_{liner\_X1.4\_US} := \left(\frac{0.093 \cdot D_{ID}^{3} \cdot 12}{E_{liner\_i} \cdot 1 \cdot \frac{in^2}{lbf}}\right)^{\frac{1}{3}} = 7.97 \cdot mm$$

(Equation X1.4 - ASTM F1216)

For Fully Deteriorated Condition the new liner is to support all soil, surcharge and hydrostatic loads

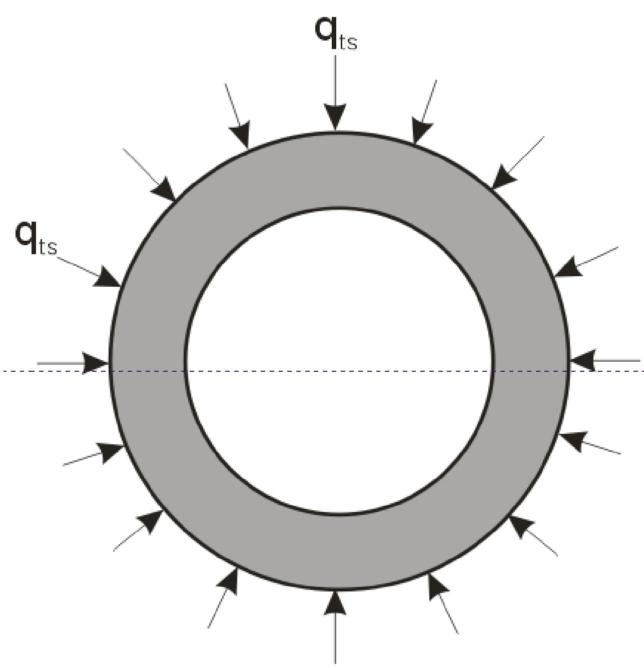
The maximum thickness from above = $t_{desing\_US} = 11.65 \cdot mm$

Overburden contribution factor = $\eta_{ob} = 1$

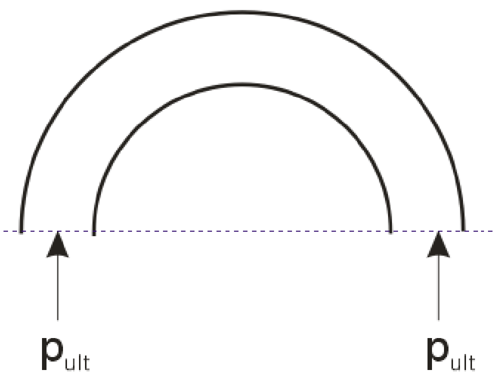
Determine the Thickness Ignoring Soil/rock confinement

Thickness based on compressive strength

Ultimate axial force (hoop force) = $P_{ult} := Ld_{factor} \cdot \left(\frac{D_{ID}}{2} + t_{desing\_US}\right) \cdot q_{t\_US} \cdot 1.ft = 3.87 \cdot kip$



Plan view of horizontal section in vertical Manhole



Plan view of hoop force per unit length in vertical direction acting on the liner

Minimum thickness required based on compression Strength= $t_{cs} := \frac{P_{ult}}{(\phi \cdot f_{compression} \cdot 1ft)} = 1.14 \cdot mm$

INPUT 2

Selected thickness = $t_{selected} := 1.0 \cdot in$ (This the thickness planned to be installed)

Flexure Strength of Liner = $f_{flex} = 800\,psi$

Factor of safety against flexure failure = $F_s := 2$

Factor of safety against Buckling failure = $FS_{buckling} := 2$

INPUT 2

Check Induced flexural Stresses

inner radius = $a\_in := \frac{D_{ID}}{2}$

Average radius = $a := a\_in + \frac{t_{selected}}{2} = 2.042\,ft$

Constrained Modulus = $M\_cons := \frac{E_m \cdot (1 - \nu_m)}{(1 + \nu_m) \cdot (1 - 2 \cdot \nu_m)} = 4444.94\,psi$

Flexibility Ratio = $F := \frac{1}{4} \cdot \frac{(1 - 2 \cdot \nu_m)}{(1 - \nu_m)} \cdot \left(1 - \upsilon_{liner}^2\right) \cdot \frac{M\_cons}{E_{liner}} \cdot \frac{(2 \cdot a)^3}{t_{selected}^3} = 16.05$

Compressibility ratio = $C := \frac{1}{2} \cdot \frac{\left(1 - \upsilon_{liner}^2\right)}{(1 - \nu_m)} \cdot \frac{M\_cons}{E_{liner}} \cdot \frac{(2 \cdot a)}{t_{selected}} = 0.04$

$$a1 := \frac{(1 - 2 \cdot \nu_m) \cdot (C - 1)}{\left[(1 - 2 \cdot \nu_m) \cdot C + 1\right]}$$

No Slip Condition Parameter:

$$a2 := \frac{(1 - 2 \cdot \nu_m) \cdot (1 - C) \cdot F - 0.5 \cdot (1 - 2 \cdot \nu_m)^2 \cdot C + 2}{F \cdot \left[(3 - 2 \cdot \nu_m) + (1 - 2 \cdot \nu_m) \cdot C\right] + \left(2.5 - 8 \cdot \nu_m + 6 \cdot \nu_m^2\right) \cdot C + 6 - 8 \cdot \nu_m}$$

$$a3 := \frac{\left[\left[1 + (1 - 2 \cdot \nu_m)\right] \cdot F - 0.5 \cdot (1 - 2 \cdot \nu_m) \cdot C - 2\right]}{\left[(3 - 2 \cdot \nu_m) + (1 - 2 \cdot \nu_m) \cdot C\right] \cdot F + \left[\left(2.5 - 8 \cdot \nu_m + 6 \cdot \nu_m^2\right) \cdot C + 6 - 8 \cdot \nu_m\right]}$$

Full Slip Condition Parameter:

$$a2 := \frac{(2 \cdot F + 1 - 2 \cdot \nu_m)}{(2 \cdot F + 5 - 6 \cdot \nu_m)}$$

$$a3 := \frac{(2 \cdot F - 1)}{(2 \cdot F + 5 - 6 \cdot \nu_m)}$$

For the selected full slip condition

$a1 = -0.322$ $a2 = 0.92$ $a3 = 0.89$ Let $\zeta := K_m \cdot 0.5 \cdot a \cdot 1 \cdot ft$ $\zeta_1 := (1 + Ko)$

$b1 := 1 - a1 = 1.32$ Let $\zeta_2 := 1 - Ko$

$b2 := 1 + 3 \cdot a2 - 4 \cdot a3 = 0.23$

$\gamma\_eff := (\gamma_m - \gamma_{water})$ Let $\zeta_3 := \frac{(1 - \nu_m)}{(1 - 2 \cdot \nu_m)}$

$\theta 1 := 90 \cdot deg$

A. Thrust Calculation:

1. Thrust Calculations due to soil/rock Medium

$$Thrust\_m := \eta_{ob} \cdot \zeta \cdot \left[\left[\gamma_m \cdot (H_m - H\_w) + p_{surcharge}\right] + \gamma\_eff \cdot H\_w\right] \cdot \left[\left[(\zeta_1) \cdot b1 - \frac{1}{3} \cdot (\zeta_2) \cdot b2 \cdot \cos(2 \cdot \theta 1)\right]\right]$$

$Thrust\_m = 1.26 \cdot kip$

2. Thrust due to water column

$Thrust\_water := \gamma_{water} \cdot H\_w \cdot a \cdot 1 \cdot ft = 1.8 \cdot kip$

3. Total Thrust per foot

Total Thrust= $T\_total := Thrust\_m + Thrust\_water = 3.05 \cdot kip$ $Ko = 0.95$

B. Moment Calculation:

1. Moment due to weight above water table

$$Moment\_m := \frac{1}{6} \cdot (1 - Ko) \cdot b2 \cdot \eta_{ob} \cdot \left[\gamma_m \cdot (H_m - H\_w) + p_{surcharge}\right] \cdot a^2 \cdot \cos(2 \cdot \theta 1) \cdot 1 \cdot ft = 0\,ft \cdot kip$$

2. Moment due to weight under water table

$$Moment\_eff := \frac{1}{6} \cdot (1 - Ko) \cdot b2 \cdot \gamma\_eff \cdot H\_w \cdot a^2 \cdot \cos(2 \cdot \theta 1) \cdot 1 \cdot ft = -0.008\,ft \cdot kip$$

3. Total Moment per foot

Total Moment = $M\_total := Moment\_m + Moment\_eff = -0.0076 \cdot kip \cdot ft$

Moment Capacity of the liner per foot = $M_{liner} := \frac{12 \cdot f_{flex} \cdot t_{selected}^3}{FS \cdot 6} = 0.067 \cdot ft \cdot kip$

The induced maximum moment is less than moment capacity - (OK)

C. Deflection Calculation:

1. Deflection due to soil/rock weight above water table

$$Deflection\_1 := 0.5 \cdot \eta_{ob} \cdot \frac{\gamma_m \cdot (H_m - H\_w) \cdot a}{M\_cons} \cdot \left[(1 - \nu_m) \cdot (\zeta_1) \cdot b1 \cdot C + \frac{2}{3} \cdot \zeta_3 \cdot (\zeta_2) \cdot b2 \cdot F \cdot \cos(2 \cdot \theta 1) \right]$$

$Deflection\_1 = 0 \cdot in$

2. Deflection due to soil/rock weight below water table

$$Deflection\_2 := 0.5 \cdot \eta_{ob} \cdot \frac{(\gamma\_eff) \cdot H\_w \cdot a}{M\_cons} \cdot \left[(1 - \nu_m) \cdot (1 + Ko) \cdot b1 \cdot C + \frac{2}{3} \cdot \zeta_3 \cdot (1 - Ko) \cdot b2 \cdot F \cdot \cos(2 \cdot \theta 1) \right]$$

$Deflection\_2 = -0 \cdot in$

3. Deflection due to water pressure

$$Deflection\_3 := 0.5 \cdot \frac{(\gamma_{water}) \cdot H\_w \cdot a}{M\_cons} \cdot \left[(1 - \nu_m) \cdot (2) \cdot b1 \cdot C \right] = 0 \cdot in$$

Total Deflection = $\Delta\_t := |Deflection\_1| + |Deflection\_2| + |Deflection\_3| = 0.11 \cdot mm$

$$Deflection := 0.5 \cdot \frac{\gamma_m \cdot H_m \cdot a}{M\_cons} \cdot \left[(1 - \nu_m) \cdot (1 + Ko) \cdot b1 \cdot C + \frac{2}{3} \cdot \zeta_3 \cdot (1 - Ko) \cdot b2 \cdot F \cdot \cos(2 \cdot \theta 1) \right] = -0.01 \cdot in$$

Diametric Strain = $D\_strain := \frac{\Delta\_t}{a} = 0.02 \cdot \%$

allowable Strain % = $\varepsilon_{diametric} = 1 \cdot \%$

The calculated diametric strain is less than allowable diametric strain - OK

Second moment of inertia per inch $I_{sec} := \frac{t_{selected}^3}{12} = 0.08 \cdot in^3$

Water Buoyancy factor = $R_w = 0.67$

Allowable Flexure Stress = $f_{allow} := \frac{f_{flex}}{F_s} = 400\,psi$

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Maximum stress = $\sigma_{max} := \frac{P_{ult}}{t_{selected} \cdot 1ft} - \frac{M\_total \cdot (0.5 \cdot t_{selected})}{I_{sec} \cdot 1ft} = 368.03 \cdot psi$

Minimum stress = $\sigma_{min} := \frac{P_{ult}}{t_{selected} \cdot 1ft} + \frac{M\_total \cdot (0.5 \cdot t_{selected})}{I_{sec} \cdot 1ft} = 277.06 \cdot psi$

(Negative indicates tension)

The induced maximum compression is less than (ϕc*0.85*fcompression) - (OK)

The induced minimum stress is less than allwable tensile flexure - (OK)

Check Critical Buckling Pressure Using ASTM F1216 Equation

Back calculate induced pressure

$P_{induced} := \frac{T\_total}{a \cdot 1ft} = 10.38 \cdot psi$

Coefficient of elastic support = $B := \frac{1}{\left(1 + 4e^{-.065 \cdot H_m \cdot 1in^{-1}}\right)} = 1$

Ovality reduction factor = $C_{factor} := \left[\frac{(1 - Ov)}{(1 + Ov)^2}\right]^3 = 0.91$ $\qquad R_w = 0.67$

$D_L := 2 \cdot a\_in = 4\,ft$

$P_{critical} := \frac{1}{FS_{buckling}}\left(32 \cdot R_w \cdot B \cdot C_{factor} \cdot E_m \cdot E_{liner} \cdot \frac{I_{sec}}{D_L^{\,3}}\right)^{0.5} = 210.47\,psi$ (ASTM F1216 Equation)

the induced compressive stress = $P_{induced} = 10.38\,psi$

The induced compressive stress is less than critical buckling stress using ASTM F1216 Formula - OK

Check Critical Buckling Pressure Using Luscher (1966) Equation (Does not account for Ovality)

$$P_{critical\_Luscher} := \frac{5.65}{FS_{buckling}} \left(R_w \cdot B \cdot E_m \cdot E_{liner} \cdot \frac{I_{sec}}{D_L^{\,3}} \right)^{0.5} = 219.88\,psi$$

The induced compressive stress = $P_{induced} = 10.38\,psi$

The induced compressive stress is less than critical buckling stress using Luscher Formula - OK

Check Critical Buckling Pressure Using Glock (1977) Equation

$$P_{critical\_glock} := \frac{E_{liner}}{\left(1 - \upsilon_{liner}^{\,2}\right)} \cdot \left(\frac{t_{selected}}{2 \cdot a} \right)^{2.2} \cdot \frac{1}{FS_{buckling}} = 395.28\,psi$$

the induced compressive stress = $P_{induced} = 10.38\,psi$

The induced compressive stress is less than critical buckling stress using Glock Formula - OK

Check Bearing Load Induced by Half Axle Load

for Hs25 truck the axle load $F_{axle} := 40000 \cdot lbf$

Half axle load acting on top of Manhole = $F_{tandem} := \frac{F_{axle}}{2} = 20000\,lbf$

Width of tandem contact area = $W_{contact} := 20 \cdot in$

Bearing support Capacity = $F_{bearing} := 0.85 \cdot \phi c \cdot f_{compression} \cdot t_{selected} \cdot W_{contact} = 81600\,lbf$

The Bearing capacity is larger than applied wheels load - OK

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Design Summary

Final Design Thickness for Manhole= $t_{selected} = 25.4 \cdot mm$ $t_{selected} = 1 \cdot in$

Flexural Stresses

Maximum stress = $\sigma_{max} = 368.03\,psi$ (Negative indicates tension)

Minimum stress = $\sigma_{min} = 277.06\,psi$

The induced maximum compression is less than (ϕc*0.85*fcompression) - (OK)

The induced minimum stress is less than allwable tensile flexure - (OK)

Stability Analysis

Critical Buckling Pressure Using ASTM F1216 Equation

Critical pressure = $P_{critical} = 210.47\,psi$

The induced compressive stress = $P_{induced} = 10.38\,psi$

The induced compressive stress is less than critical buckling stress using ASTM F1216 Formula - OK

Check Critical Buckling Pressure Using Luscher Equation (Does not account for Ovality)

Critical pressure = $P_{critical\_Luscher} = 219.88\,psi$

The induced compressive stress = $P_{induced} = 10.38\,psi$

The induced compressive stress is less than critical buckling stress using Luscher Formula - OK

Check Critical Buckling Pressure Using Glock (1977) Equation

Critical pressure = $P_{critical\_glock} = 395.28\,psi$

the induced compressive stress = $P_{induced} = 10.38\,psi$

The induced compressive stress is less than critical buckling stress using Glock Formula - OK

Check Bearing Load Induced by Half Axle Load

The Bearing capacity is larger than applied wheels load - OK

GEOSTRUCTURAL

ATTCHMENT 3

Peck et all (1972) Paper

5

Soft Ground Tunneling

Chairmen: Dewayne L. Misterek
Sam Taradash

Denver Federal Center, Denver, Colo.
Commercial Shearing & Stamping Co., Youngstown, Ohio

Chapter 19

STATE OF THE ART OF SOFT-GROUND TUNNELING

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INTRODUCTION

The state of the art of soft-ground tunneling was discussed in detail by the senior author at the 7th International Conference on Soil Mechanics and Foundation Engineering held in Mexico City in 1969. Little can be gained by repeating the information assembled at that time. Hence, in this report, only the briefest summary will be given of the overall state of the art and attention will be concentrated on trends and developments since 1969.

A characteristic of recent developments is the continued trend toward use of shields and excavating machines. The implications of this trend with respect to design and construction, and particularly with respect to the treatment of unfavorable ground conditions, will be examined.

Settlement associated with lost ground, a subject as old as tunneling itself, will be reviewed and methods will be considered for its reduction.

Design of tunnel supports was discussed in detail in 1969 with respect to essentially rigid or essentially flexible types of linings. In this report, a definition of flexibility will be considered and design procedures suggested for linings of flexibility intermediate between the two extremes.

SHIELDS AND MACHINES

Today, almost all circular and some horseshoe tunnels are excavated within the protection of shields. Safety is the prime consideration; the possibility of collapse of an unprotected crown or face is no longer considered tolerable. The trend is understandable and justifiable, but it has its undesirable features. Today's workman has lost much of his skill in hand mining, and so has his supervisor.

This aspect of the state of soft-ground tunneling is unfortunate because complex modern underground systems involve many geometrical forms not adaptable to shield tunneling. These include junctions of rapid transit lines, stations formed by excavating between shield-driven tubes, escalator and stairway passageways and their junctions with driven tunnels, and a host of auxiliary structures for various purposes.

It is a truism in tunneling that the beginning of a job is almost always fraught with lost ground, slow progress, and even accidents, until the crew and supervisory staff become acquainted with the necessary steps of the work to be done. The period of learning may easily be several weeks to a few months. If hand excavation is regarded as a minor adjunct to shield tunneling, and if most of the planning is devoted to economical and rapid progress of the running tunnels, whatever hand work is necessary becomes a fruitful source of delays, accidents, and excessive loss of ground.

The lore of hand mining in unfavorable ground is almost forgotten; men with a variety of experience in directing hand mining are a vanishing breed; skilled, soft-ground hand miners are rare. Unsatisfactory and inept methods, discarded and replaced by better ones many years ago, are being revived through ignorance of the lessons of the past. It is a matter of concern to all interested in soft-ground tunneling that our emphasis on progress and mechanization is causing us to lose an important and useful heritage.

To remedy this situation, the tried and true techniques need to be restated and brought up to date for the benefit of those who have a job to do, who wish to do it well, but who have neither the time nor the opportunity to study old and somewhat obscure descriptions of difficulties and how they were overcome.

No significant improvement in shields as such can be noted in the past few years. They still tend to roll, are difficult to steer, and are difficult to keep on grade. Shapes other than circular are occasionally attempted but, except for roof shields, have found little application. Most of the comments about shields are associated with loss of ground and its prevention and will be deferred until that topic is discussed.

Excavating machines are becoming increasingly popular. The excavating equipment itself remains undesirably sensitive to changes in the nature of the ground; the greatest advances appear to have been in the systems for removing and handling the muck and installing the lining. The difficulty of controlling the face under adverse ground conditions has led to increased use of methods to improve or homogenize the properties of the soil so that progress will not be impeded and so that changes in the type of excavating equipment will not be necessary. The same methods of improvement also aid in reducing lost ground and will be discussed further under that heading.

The investment in an excavating machine is so great that rapid progress is essential for recovery of profit. Choice of the best machine for given conditions depends in part on the experience of the constructor and in part on the accuracy with which the significant characteristics of the soil deposits are portrayed to the bidders. The two conditions that appear to have given the greatest difficulty in recent years are the presence of groundwater in pervious zones and the presence of larger sizes and greater quantities of boulders than anticipated. Both conditions have led to litigation, a sure indicator of an unsatisfactory state of the art.

With respect to boulders, the limitations of test borings should be appreciated. For example, in excavating a tunnel of 10-ft diameter by machine, two 8-inch boulders per foot of tunnel would usually be considered a large number. Yet, statistically, it is likely that, if the boulders were uniformly distributed throughout the deposit, only one boulder would be encountered in a boring 100 ft long. The actual influence of the boulders depends on several factors in addition to their frequency. If they are large compared to the size of openings or slits in the excavating machine, they may be troublesome. If, in addition, they are embedded in a hard cohesive matrix they may greatly impede the progress of even a hand-mined shield and may render completely impotent a mechanical excavator of almost any type.

The detrimental effects of groundwater in impervious zones depends to a great extent on the type of the geological formation and the details of its structure. Whereas a waterbearing lens may drain almost harmlessly into the heading, a waterbearing seam connected to a source of supply may lead to instability and a run. Investigations of groundwater conditions should include an assessment of the geologic implications. There is need for far better understanding and cooperation in this respect among the engineer who conducts the subsurface exploration, the engineering geologist who can develop the implications of the structure of the deposit, and the prospective builder of the tunnel who should have the background to appreciate the implications.

Indeed, one of the outstanding shortcomings in the state of the art of soft-ground tunneling at the present time is the manner in which subsurface information is obtained, presented, made available to bidders, and related to the contract documents. The engineer or owner, fearing claims, is strongly tempted to place no conclusions regarding the behavior of the soil in the contract documents, although he and his advisors are probably the only ones having the time and facilities to make an adequate assessment of the subsurface conditions. The bidders, on the other hand, are tempted to be optimistic to enhance their likelihood of being the lowest bidder, and to look for every apparent deviation, significant or otherwise, from the conditions they say they have assumed on the basis of the contract documents. This mutually antagonistic relationship is unhappily growing worse and threatens to overshadow many of the technical improvements that potentially decrease the cost of tunneling.

LOSS OF GROUND

An approximation of the settlement that must be anticipated above a single shield-driven tunnel, executed with proper techniques and good workmanship, can be made by the procedure advocated by Schmidt (1969). The shape of the settlement curve is that of the probability function; the significant parameters are shown in Fig. 1. The maximum settlement can be estimated on the assumption that the volume of the settlement trough will be about 1 per cent of the volume of the tunnel. Under exceptionally good conditions and workmanship, the settlement may be as little as half this amount; in contrast, volumes of settlement of up to 40 or 50 per cent of the volume of the tunnel are not unknown. Such settlements represent, of course, the results of extremely poor practices.

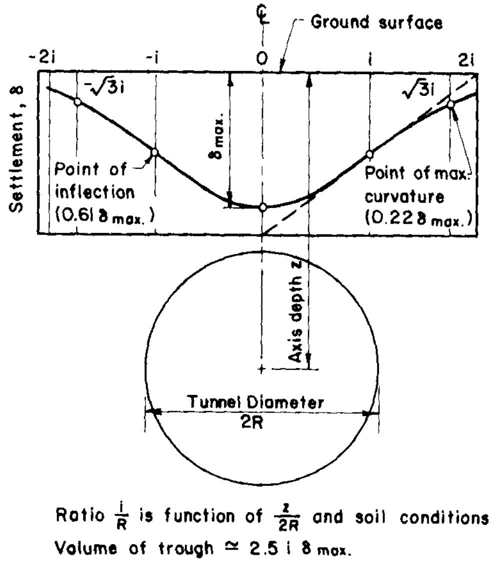


Fig. 1. Settlement curve above shield-driven tunnel as predicted by Schmidt (1969).

The settlements immediately associated with construction (exclusive of long-time consolidation) may be conveniently separated into those associated with movement toward the working face, invasion of the surrounding soil into the annular space left by the tailpiece clearance and such similar features as poling plates, and inflow of material with groundwater entering the tunnel at unprotected places. The movements may be accentuated by yawing, diving or nosing of the shield and by the necessity for negotiating curves. The various sources of settlement are well illustrated in the paper by Hansmire and Cording.

The movement of the soil toward the working face and the invasion of the annular spaces surrounding the tunnel lining are caused by the reduction or removal of the original stresses within the soil mass. Fundamentally, two ways are available for preventing or reducing the movement. Either the ground must be so stiff and strong, or must be converted into a medium so stiff and strong, that the reduction of stress causes negligible deformations; or else the reduction of stress must be eliminated or restricted until the tunnel lining is capable of sustaining the earth loads without significant deformation.

The strength and stiffness of a granular soil below water table may, of course, be increased drastically by drainage. The drainage is necessary to prevent instability; the increase in stiffness is a valuable byproduct. Other than by drainage, improvement of the properties of granular soils is being accomplished by the injection of cement or chemical grouts; the choice depends on the nature of the formation. Although expensive, grouting may be particularly attractive if its use in short intervals of particularly bad ground can eliminate the necessity for compressed air. Grouting may also reduce or eliminate the flow of groundwater.

Many misconceptions still exist concerning the benefits of grouting and, particularly, the manner in which grout penetrates or permeates the soil and serves its useful purpose. It can be taken for granted that the voids of a granular soil are rarely filled completely or uniformly by any kind of grout. Grout of a given consistency preferentially enters the voids of the coarsest material from which, if it does not set too quickly, it slowly permeates the less pervious materials. Compressible materials such as fine sand or coarse silt, or laminated silts, sands and clays, are often split by the grout. The fluid grout takes the form of a lens or sheet from which it may penetrate remaining portions of the soil. Often the principal influence of the sills and dikes of grout is the compression or consolidation of the intervening material while the grout pressure is still being maintained. The peculiarities of grout penetration are well known in some quarters, but often unappreciated in others. A series of investigations in the 1940's and 1950's, of grout patterns found in railroad roadbeds, is particularly enlightening and would be worth contemplation by those who wish to improve their feel for the manner in which grout carries out its function. They are published in the Proceedings of the American Railway Engineering Association, a journal not widely read by tunnel designers or constructors.

The art of chemical grouting has improved to the point that massive or varved silts have been successfully impregnated, as indicated in the paper by Anderson and McCusker. Far more grouting is done in Europe than in the United States. It can be anticipated that the practice will grow in this country with increasing use of mining machines, because grouting offers the greatest potential for selective improvements of specific zones likely to be the seat of trouble in otherwise unsatisfactory ground. The most effective grouting, under these circumstances, is accomplished from the ground surface ahead of tunneling, or perhaps from pilot drifts, because attempts to grout from inside the heading during the use of tunnel driving machines seriously impedes progress of the machines.

Dewatering remains a fundamental procedure for general improvements of granular materials. Although the techniques for dewatering are well established, close enough spacing of deep or eductor wells and sufficient time for adequate dewatering are still not always provided. Stabilization of granular material by dewatering can substantially reduce the loss of ground at the working face and can, to some extent, increase the time available for expansion of lining against the soil behind the tailpiece or for filling the tailpiece void. It can rarely eliminate the latter source of lost ground unless sufficient apparent cohesion is developed to increase the stand-up time appreciably.

In plastic cohesive soils, no satisfactory way is available for increasing the strength of the material, but air pressure may be used to decrease the reduction in stress due to excavation until the permanent lining is placed. The high cost and physiological effects of air pressure place serious limitations on its utility. It provides a positive means in plastic soils, however, for preventing the inward movement of the soil behind the tailpiece of the shield until appropriate measures can be taken. Hence, where appreciable loss of ground is intolerable, compressed air still represents the most effective method of control.

Today's state of the art includes at least one demonstrably successful tunneling machine, in Japan, in which fluid pressure is held against the working face while the workmen can erect the lining in free air. The principle is sound, and progress is being made. Similarly, the use of slip forms and an exotic quick-setting strong material holds promise for being able not only to fill the annular space behind the tailpiece promptly, but to provide the permanent lining as well. Such a blue-sky device, described in the paper by Parker and Semple, seems to be practicable in principle, and the various components of the equipment have been tested. Successful application may not be too far away. Nevertheless, although the development of such semi-automated tunneling machines is a desirable step forward, under some circumstances less automated procedures, and even hand mining, may be economically preferable. The best ultimate development of the state of the art is likely to include improved methods for hand mining as well as for machine mining, and the ability of engineers and constructors to chose the system most suitable for the circumstances.

DESIGN OF TUNNEL LININGS

The design procedures summarized in 1969 were divided into two categories: those to be used in proportioning flexible and rigid tunnel liners. A liner is said to be "flexible" if it interacts with the surrounding soil in such a way that the pressure distribution on the liner and the corresponding deflected shape result in negligible bending moments at all points in the lining. A "rigid" liner is one which deflects insignificantly under the loads imposed by the soil; thus there is very little soil-structure interaction. Real linings, however, are neither perfectly flexible nor perfectly rigid.

In present practice there is no quantitative method to classify the stiffness of a tunnel liner in terms of both the structural properties of the liner and the stress-strain characteristics of the surrounding soil. A tunnel liner which may be stiff with respect to a soft clay may behave as a flexible liner in a very stiff clay. Thus, there is a need to account for both the stress-strain properties of the soil and the flexibility of the tunnel liner. In this section a method will be presented for quantitatively determining the relative flexibility of tunnel liners of stiffness intermediate between essentially "flexible" and essentially "rigid."

The structural engineer designs a tunnel lining for certain combinations of thrust and moment. The magnitude of the thrust and moment is dependent upon the stiffness of the lining relative to that of the medium and to the depth of the tunnel. In order to appreciate the factors affecting the structural design of liners of intermediate flexibility, the design procedures presently used by structural engineers for both "flexible" and "rigid" tunnel liners are reviewed briefly.

Flexible liners, which interact fully with the soil in such a way that a nearly uniform pressure distribution ultimately acts on them, do not have to be designed for moments consistent with the initial stress distribution in the soil. But the liner must be designed to accomodate the diameter changes necessary to develop a uniform pressure distribution on the liner. These diameter changes can be estimated from experience and are usually in the range of about half a per cent. The structural section must be designed to withstand the bending moments induced by the estimated diameter changes. In addition, it must be designed to prevent buckling. In soft clays this is usually accomplished by insuring that the overburden stress, γH, is less than 3 EI/R^3, where EI and R are the flexural stiffness and the mean radius of the liner, respectively.

For rigid liners the coefficient of earth pressure at rest is usually estimated, and the moments and thrusts are calculated on the assumption of no interaction between the soil and the liner. Thus the soil is assumed to produce a load on the lining as shown in Fig. 2, where the maximum moment is given by

Loads causing moment is the differential between horizontal and vertical pressures

$$M = \pm 1/4 \; \gamma H \; (K_o - 1) \; R^2$$

The thrust at the springline is given by

Thrust load is caused by total overburden pressure

$$T_s = \gamma H R \qquad (2)$$

and the thrusts at the invert and crown are given by

$$T_{CI} = \gamma H \; K_o \; R \qquad (3)$$

It should be noted that the moment (eq. 1) is given by a constant, $\bar{K}$, times $\gamma H R^2$; $\bar{K}$ is commonly referred to as the moment coefficient. For values of coefficient of earth pressure at rest equal to 1/2 and 2, the moment coefficients are 12.5 per cent and 25 per cent respectively. These moment coefficients are too high because of the assumption of no interaction between the liner and

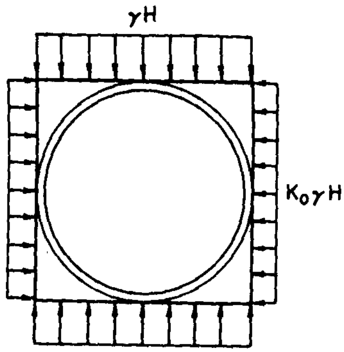


Fig. 2. Pressure distribution on a rigid liner assuming no interaction between the soil and the liner

the soil. Therefore, a more general procedure for proportioning tunnel liners of intermediate stiffness must take into account the soil-structure interaction and must yield design moments and design thrusts as functions of liner stiffness. The structural engineer then needs only to compare the expected moments and thrusts at any point in the section with the limiting values of thrust and moment which can be determined for a given structural section from an interaction diagram.

Definition of Stiffness Ratio for Tunnel Liners

The stiffness of a tunnel liner-soil system is conveniently considered as being divided into two separate and distinct types. The first is extensional stiffness, which is a measure of the equal all-around uniform pressure necessary to cause a unit diametral strain of the liner with no change in shape. The second is flexural stiffness, which is a measure of the magnitude of the non-uniform pressures necessary to cause a unit diametral strain which results in a change in shape or an ovaling of the liner.

Recent analytical work by Burns and Richards (1964) and Höeg (1968), in soil-structure interaction, can be used to assess quantitatively the stiffness of a liner relative to a soil medium. In these studies the relative stiffness of the liner and surrounding medium is characterized by two ratios designated as the compressibility ratio and the flexibility ratio. A definition of and a physical interpretation of these ratios are given below.

The compressibility ratio is a measure of the extensional stiffness of the medium relative to that of the liner. The extensional stiffness of the medium can be obtained by considering a portion of the medium subjected to a uniform external pressure, p, as shown in Fig. 3a.

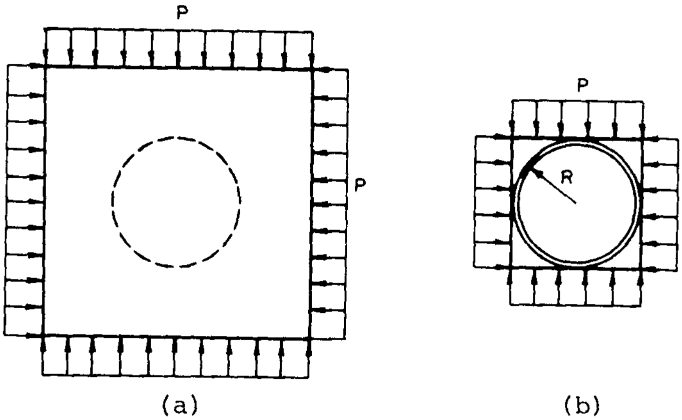


Fig. 3. Medium and liner under a state of uniform compression.

The diametral strain across an imaginary circular tunnel (shown by the dotted line) is given by

$$\frac{\Delta D}{D} = \varepsilon_m = \frac{p}{E}(1+\nu)(1-2\nu) \tag{4}$$

and the extensional stiffness is given by

$$\frac{p}{\Delta D/D} = \frac{E}{(1+\nu)(1-2\nu)} \tag{5}$$

where E and ν are the Young's modulus and the Poisson's ratio of the medium. The extensional stiffness of the liner, which replaces the cylinder of material within the imaginary circle shown in Fig. 3a, can be obtained by considering a ring subjected to a uniform pressure, p, as shown in Fig. 3b. The diametral strain is given by

$$\frac{\Delta D}{D} = \frac{pR}{E_\ell t} \tag{6}$$

where E_ℓ, R, and t are respectively the modulus of elasticity, the radius, and the thickness of the ring. The extensional stiffness of the liner in plane strain is obtained from eq. 6 by replacing E_ℓ by $E_\ell/(1-\nu_\ell^2)$ where ν_ℓ is the Poisson's ratio of the liner material. Thus, the extensional stiffness of the liner is given by

$$\frac{p}{\Delta D/D} = \frac{\frac{E_\ell t}{R}}{(1 - \nu_\ell^2)} \tag{7}$$

The compressibility ratio, C, is obtained by dividing eq. 6 by eq. 7.

$$C = \frac{\frac{E}{(1+\nu)(1-2\nu)}}{\frac{E_\ell t}{(1-\nu_\ell^2)} \quad \frac{1}{R}} \tag{8}$$

The above expression is for a liner of uniform cross-sectional thickness, t. Since most tunnel liners are composed of built-up sections with non-uniform thickness, similar to that shown in Fig. 4, eq. 8 is modified by taking the thickness, t, as the cross-sectional area, A, of a typical element divided by the length, L, of the element; that is, t = A/L.

The flexibility ratio is a measure of the flexural stiffness of the medium relative to that of the liner. The flexural stiffnesses of both the medium and the liner, as defined here, are essentially measures of the resistance of each to a change in shape under a state of pure shear. The flexural stiffness of the medium can be obtained by considering the diametral strain of the imaginary circle shown in Fig. 5a. The diametral strain is given by

$$\frac{\Delta D}{D} = \frac{p}{E} (1+\nu) \tag{9}$$

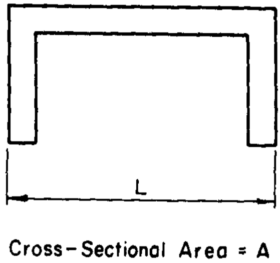
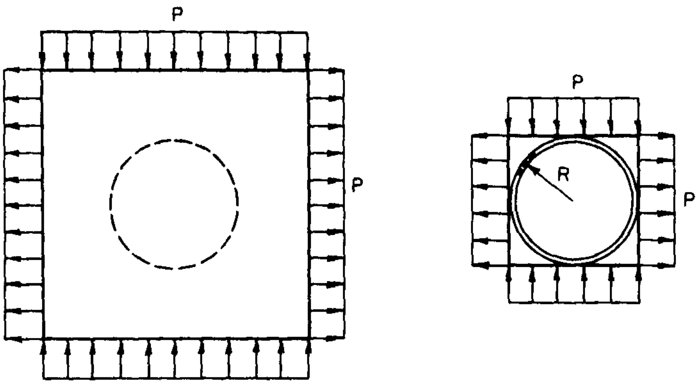


Fig. 4. A typical built-up liner section



(a) (b)

Fig. 5. Medium and liner under a state of pure shear

The flexural stiffness of the medium is taken as a ratio of the pressure, p, to the corresponding unit diametral strain across the cylinder. The resulting expression for the flexural stiffness of the medium is

$$\frac{P}{\Delta D/D} = E\ (1+\nu) \tag{10}$$

The diametral strain of a ring subjected to the pressure distribution shown in Fig. 5b is

$$\frac{\Delta D}{D} = \frac{pR^3}{6\ E_\ell I_\ell} \tag{11}$$

where I_ℓ is the moment of inertia of the cross-sectional area of the ring. If $E_\ell I_\ell$ is replaced by $E_\ell I_\ell/(1-\nu_\ell^2)$ to account for the plane strain effect in the liner, the liner stiffness is given by

$$\frac{p}{\Delta D/D} = \frac{\dfrac{6\ E_\ell I_\ell}{R^3}}{(1 - \nu_\ell^2)} \tag{12}$$

The flexibility ratio is obtained by dividing eq. 11 by eq. 12. Thus,

$$F = \frac{\dfrac{E}{(1+\nu)}}{\dfrac{6\ E_\ell I_\ell}{(1 - \nu_\ell^2)}\ \dfrac{1}{R^3}} \tag{13}$$

In all the expressions above, I_ℓ is the moment of inertia of the cross-section per unit length along the axis of the tunnel. Thus, for the section shown in Fig. 4, I_ℓ is the moment of inertia of the entire cross-section divided by the length L.

Burns and Richard (1964) have shown that, on account of the interaction between the soil and the structure, the resulting thrusts and moments are affected by

(1) the compressibility ratio

(2) the flexibility ratio

(3) the slippage which takes place at the interface between the structural liner and the medium.

Various solutions are given for both full and no slippage between the medium and the liner. Because of the existence of high shear stresses at the interface between the liner and the medium for most cases, the condition of full slippage is believed to approximate more nearly the behavior of soft-ground tunnel liners. Although the expressions developed by Burns and Richard are for the case of a one-dimensional air-blast loading for protective structures, the expressions may easily be modified to give the thrusts, moments, and displacements for various initial values of the coefficient of earth pressure at rest as shown in Figs. 6 to 9 inclusive. The equations are given below; they are valid only for a deeply buried tunnel.

For crown and invert:

$$T = \frac{1}{2}\left[(1 + K_o)\ b_1 - \frac{1}{3}(1 - K_o)\ b_2\right] \gamma H\ R \qquad (14)$$

$$M = \frac{1}{6}(1 - K_o)\ b_2\ \gamma H\ R^2 \qquad (15)$$

$$W = \frac{1}{2}\ \frac{\gamma HR}{M_c}\left[(1-\nu)(1 + K_o)\ b_1 C + \frac{2}{3}\ \frac{1-\nu}{1-2\nu}(1 - K_o)\ b_2\ F\right] \qquad (16)$$

and for springline

$$T = \frac{1}{2}\left[(1 + K_o)\ b_1 + \frac{1}{3}(1 - K_o)\ b_2\right] \gamma H\ R \qquad (17)$$

$$M = -\frac{1}{6}(1 - K_o)\ b_2\ \gamma H\ R^2 \qquad (18)$$

$$W = \frac{1}{2}\ \frac{\gamma HR}{M_c}\left[(1-\nu)(1 + K_o)\ b_1 C - \frac{2}{3}\ \frac{1-\nu}{1-2\nu}(1 - K_o)\ b_2\ F\right] \qquad (19)$$

where

$$b_1 = 1 - a_1$$

$$b_2 = 1 + 3\,a_2 - 4\,a_3$$

and

$$a_1 = \frac{(1-2\nu)(C-1)}{(1-2\nu)C + 1}$$

$$a_2 = \frac{2F + 1 - 2\nu}{2F + 5 - 6\nu}$$

$$a_3 = \frac{2F - 1}{2F + 5 - 6\nu}$$

γ = unit weight of soil,

H = height to center of the tunnel,

R = mean radius of the liner,

C = compressibility ratio,

F = flexibility ratio,

and M_c is the constrained modulus of the soil which is given by

$$M_c = \frac{E\,(1-\nu)}{(1+\nu)(1-2\nu)}$$

The displacements given by eqs. 16 and 19 refer to a portion of the medium containing the liner and loaded by external pressures. The expressions include the displacements which have already taken place due to the free-field stresses.

The expressions for moments, eqs. 15 and 18, indicate that the moment is proportional to $(1 - K_o)$, which is a measure of the difference between the major and minor principal stresses in the free field. The moment expressions also indicate that the moment is a function of the flexibility ratio and not of the compressibility ratio (see the expression for b_2).

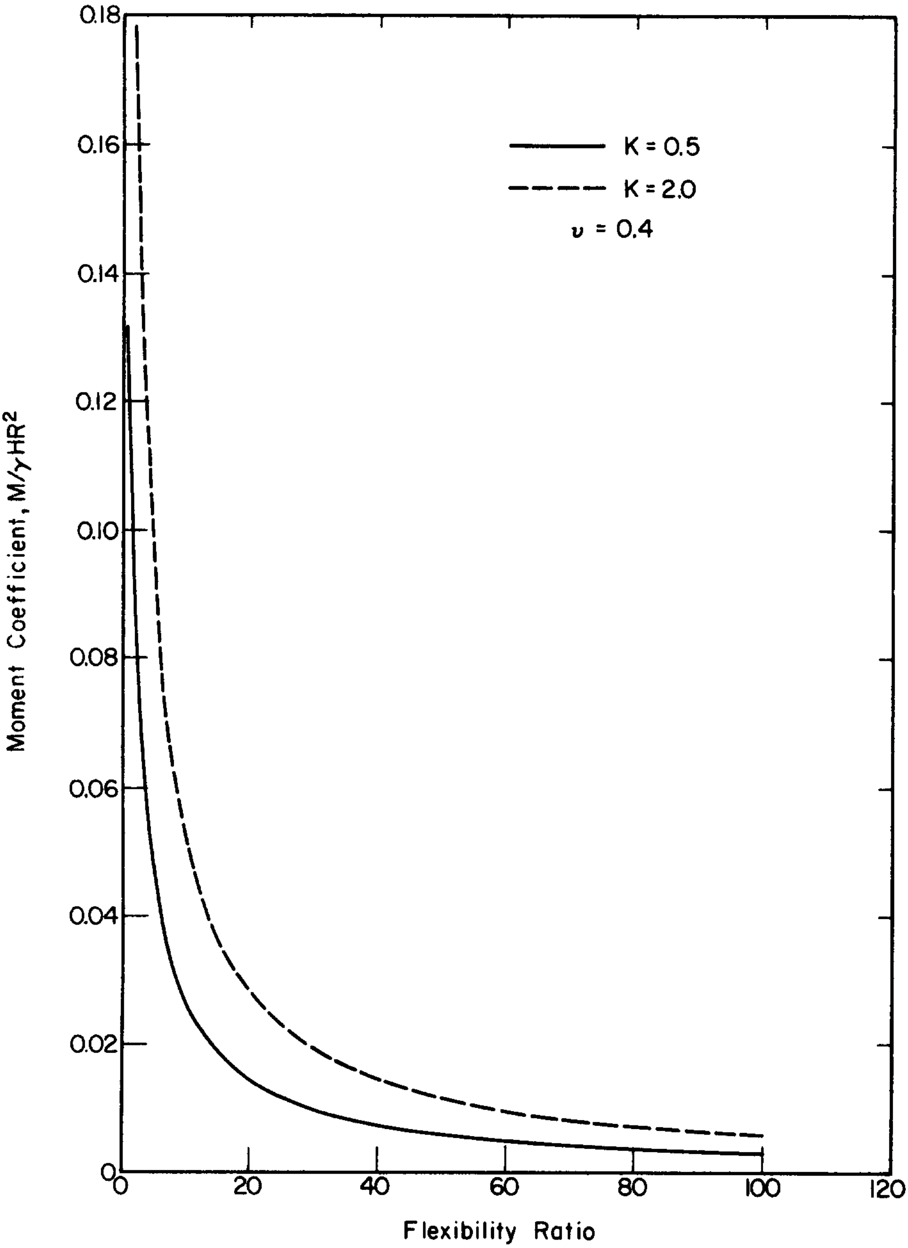


Fig. 6. Variation of moment coefficient with flexibility ratio

In Fig. 6, the dimensionless moment, or the moment coefficient, $(M/\gamma HR^2)$, is given as a function of the flexibility ratio for two values of K_o. As the flexibility ratio increases, the moment coefficient decreases. The decrease is very substantial for a flexibility ratio less than 10 and, thereafter, the moment coefficient is less than 5 per cent. Thus, for flexibility ratios greater than about 10, the curves indicate that the liner behaves relatively flexibly with respect to the medium. The moment coefficients for design could be obtained from Fig. 6 or the moment expressions.

Figures 7 and 8 give the thrust coefficients $(T/\gamma HR)$ as functions of flexibility and compressibility, respectively. The thrust coefficient is a function of both the flexibility and the compressibility ratios, and also of the coefficient of earth pressure at rest. Fig. 7 shows that the thrust coefficient is relatively insensitive to the flexibility ratio but is sensitive to the initial value of K_o. This plot shows, however, that the thrust coefficient is practically the same for all flexibility ratios greater than 10; this indicates that tunnel liners with flexibility ratios greater than 10 behave as flexible liners. For flexibility ratios greater than 10, the thrust coefficient is nearly $1/2\ (1 + K_o)$ as given by Peck (1969) for a compressibility ratio of 1.0. This simplified expression is conservative for compressibility ratios greater than 1.0; for compressibility ratios less than 1.0, the expression may be modified as follows to give a simple relation which approximates the more detailed calculations given by eqs. 14 and 17 for flexibility ratios greater than 10:

$$\frac{T}{\gamma HR} = \frac{1}{2}\,(1 + K_o)\ [1.2 - .2C] \qquad (20)$$

In Fig. 8 the thrust coefficient is shown as a function of compressibility ratio for two values of K_o and two flexibility ratios. This plot indicates that the thrust coefficient decreases as the compressibility ratio increases. Moreover, for a given compressibility ratio, the thrust at the crown and springline are somewhat different for low values of the flexibility ratio, but approach each other as the flexibility ratio increases. The same effect is shown in Fig. 7. It is suggested that eqs. 14 and 17 may be used for preliminary design.

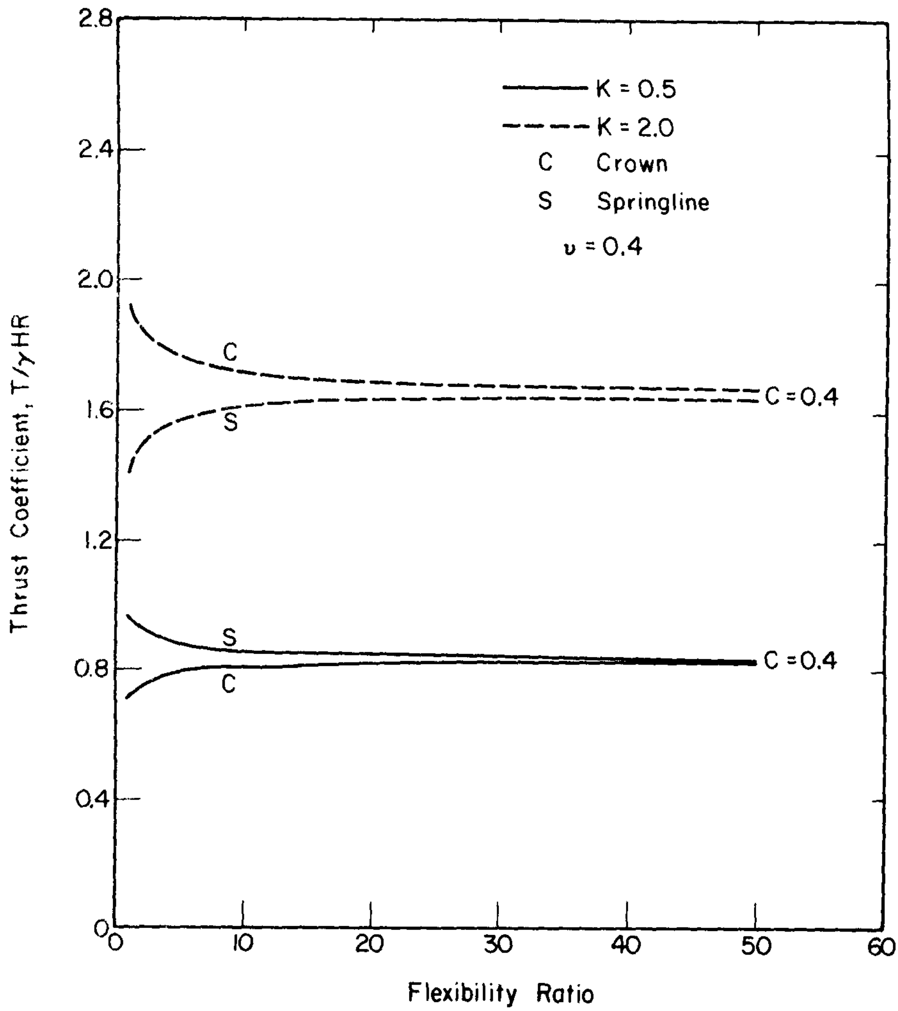


Fig. 7. Variation of thrust coefficient with flexibility ratio

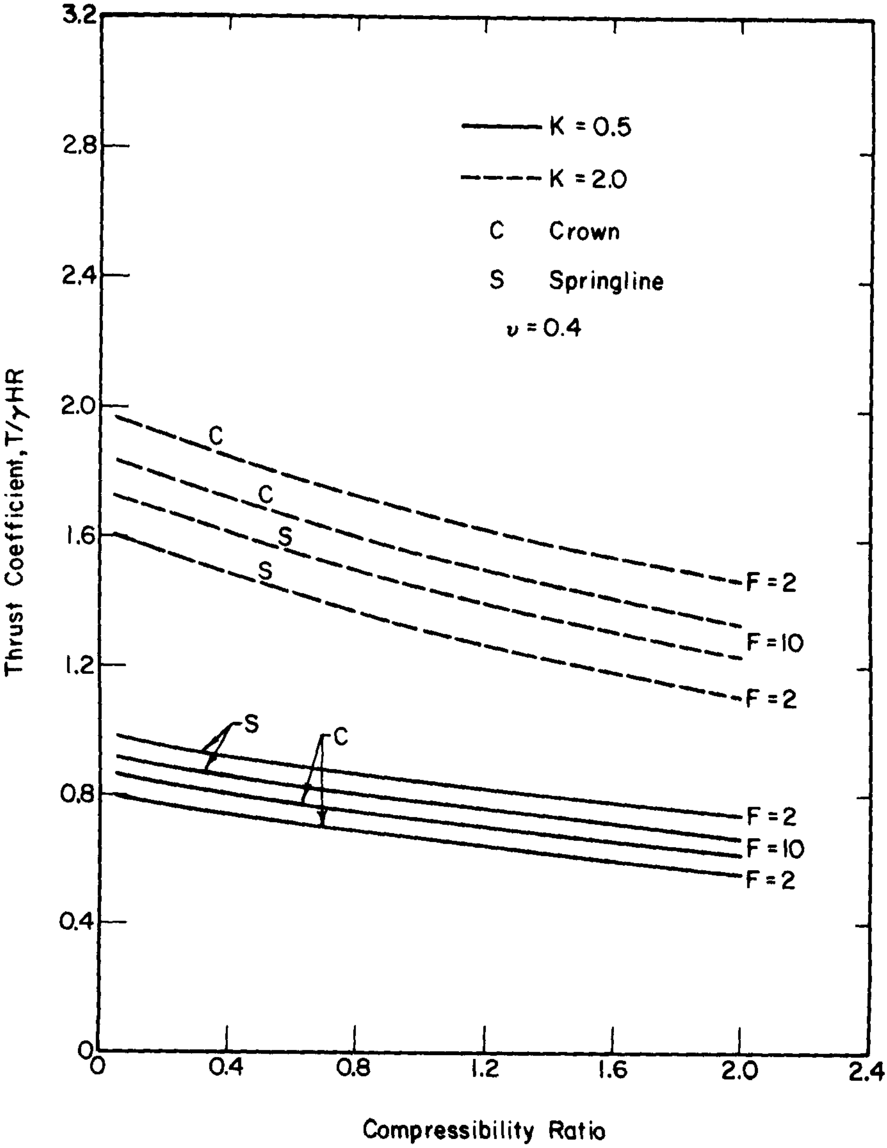


Fig. 8. Variation of thrust coefficient with compressibility ratio

Equations 16 and 19 show the per cent diameter change as a function of the soil stiffness, depth of tunnel, flexibility ratio, and compressibility ratio. Although the diameter change is influenced to a minor extent by the compressibility ratio and the average all-around stress given by $1/2\ (1 + K_o)\gamma H$, the primary factors affecting the diameter changes are those which tend to distort or oval the tunnel lining. Thus, the diameter changes of the tunnel are primarily a function of the flexibility ratio, the difference in the free-field principal stresses $(1 - K_o)(\gamma H)$, and the modulus of the soil medium. Eqs. 16 and 19 can be simplified in terms of a dimensionless diameter change, $(\Delta D/D)/(\gamma H/M_c)$. Fig. 9 shows the dimensionless diameter change plotted as a function of the flexibility ratio for a compressibility ratio of 0.4. The curves in Fig. 9 show that the diameter changes approach constant values when the flexibility ratio exceeds 10. Thus, for flexibility ratios greater than 10, the deformations are primarily governed by the soil and are very little influenced by the structural liner. That is, for flexibility ratios greater than 10, the liner behaves as a flexible liner. For flexibility ratios less than 10, the behavior is affected by both the liner and the soil. For other coefficients of earth pressure at rest, eqs. 16 and 19 can be used to give an estimate of the diameter changes. As mentioned previously, the equations do not account for deformations which have already taken place due to the free-field stresses.

Application of Finite-Element Techniques to the Design of Tunnel Liners

Although closed-form solutions from the theory of elasticity, such as those presented above, are available for deep tunnel liners, closed-form solutions are not available for lined tunnels near the surface where surface displacements and changes in the state of stress with depth significantly affect the behavior of the tunnel. For these more complex conditions a discrete method of analysis such as the finite-element method can be employed to obtain the desired information. Even if this approach is adopted, however, the closed-form solutions and the information obtained from them identify significant dimensionless parameters. If the results of the finite-element calculation are plotted in terms of these dimensionless parameters, fewer finite-element solutions are needed to cover the range of variables desired.

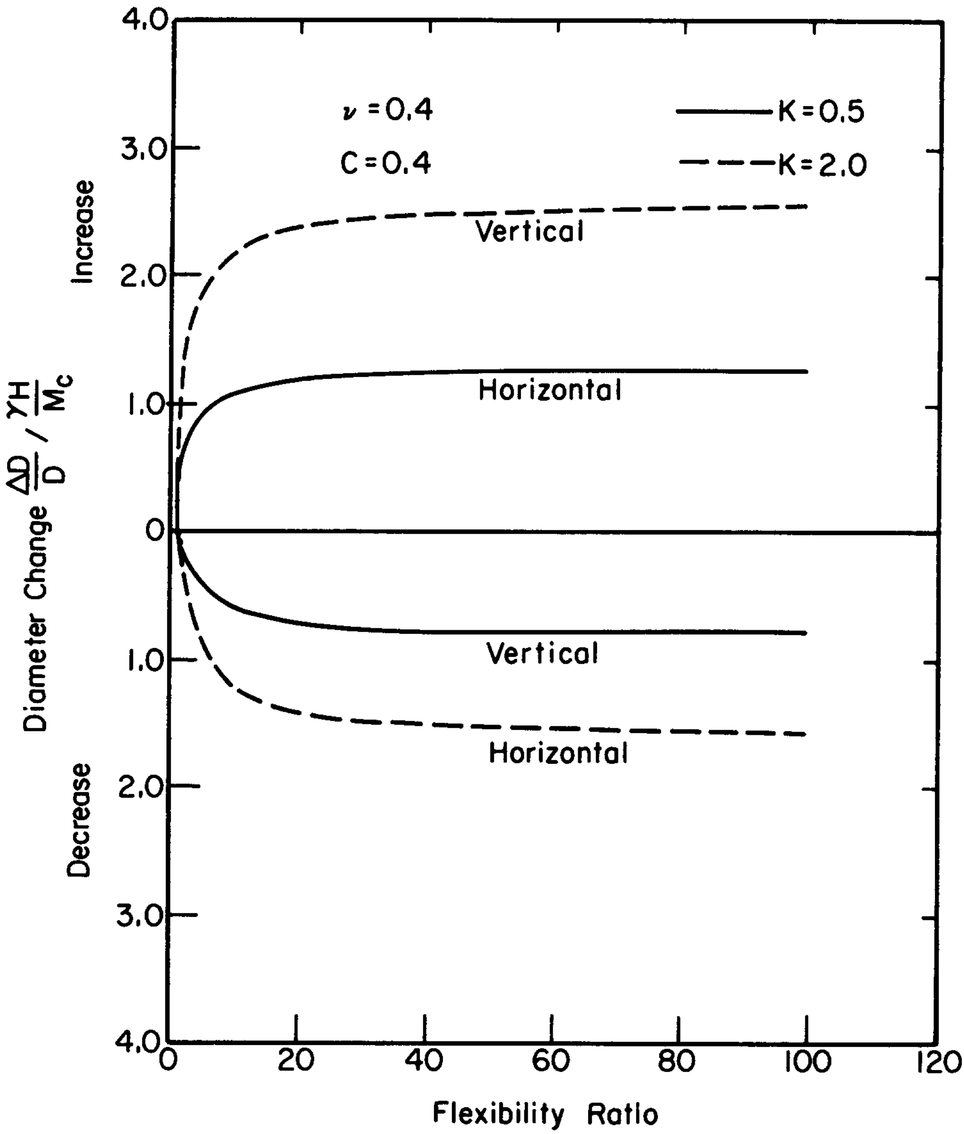


Fig. 9. Variation of diameter change with flexibility ratio (values are corrected for free-field deformation)

To determine the effect of shallow depths of cover on various design parameters for tunnel liners, finite-element solutions were obtained for three different lining stiffnesses, for three different depths of burial in terms of the diameter, and for two values of the initial coefficient of earth pressure at rest. The solutions were obtained by unloading the stresses which exist at the boundaries of the tunnel before excavation. The computations considered the crown of the tunnel to be buried at depths corresponding approximately to one-third, one, two, and three times the diameter of the tunnel lining. The ranges of stiffnesses corresponded to a range in compressibility ratio from about 0.3 to 0.6 and a range in flexibility ratio from 2.3 to 15.3. These values in general cover the range for both steel and economically proportioned concrete liners. The computations were carried out for coefficients of earth pressures at rest of 0.5 and 2. These values were chosen because in normally consolidated soils one should design the liner for moments consistent with a coefficient at least as low as 0.5, whereas for tunnels in overconsolidated clays the coefficient could be as high as 2.0. Tunnel liners designed for overconsolidated clays should take into account the potential for high bending moments and high thrusts at the crown and invert associated with a high horizontal stress.

The thrust coefficients from these calculations versus the dimensionless depth of burial, H/D, are shown in Fig. 10. Only the maximum thrust coefficients governing the design are shown. For a K_o value of 0.5 the maximum thrust coefficient occurs at the springline and is relatively insensitive to the flexibility and compressibility ratios. Moreover, the thrust coefficient at the springline, for a K_o value of 0.5, increases as the depth of burial increases and approaches the fully buried condition at a dimensionless depth of burial of 1.5. For a K_o value of 2.0 the maximum thrust coefficient generally decreases as the dimensionless depth of burial increases. The fully buried condition occurs at a depth of about 1.5 diameters. The thrust coefficient at the invert for a coefficient of earth pressure at rest of 2.0 is sensitive to the stiffness of the tunnel lining as shown by the upper three curves in Fig. 10.

A plot of the dimensionless moment versus the dimensionless depth of burial for the same tunnel liners is shown in Fig. 11. For a coefficient of earth pressure at rest equal to 0.5 the moment coefficients are a function of the flexibility of the tunnel liner, but for all the tunnel liners the moment coefficient is less than

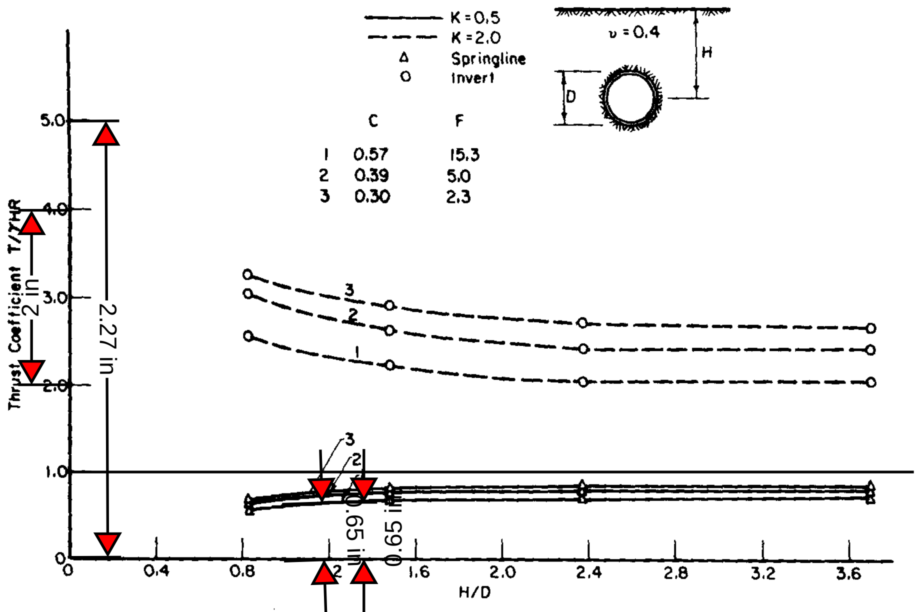


Fig. 10. Variation of thrust coefficient with depth of burial

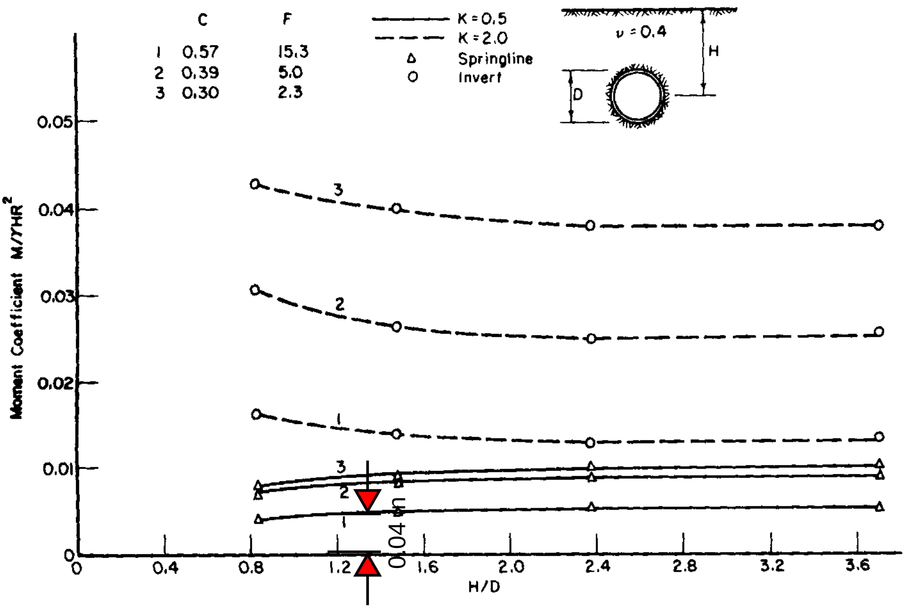


Fig. 11. Variation of moment coefficient with depth of burial

1 per cent. The moment coefficient increases with dimensionless depth of burial and the fully buried condition occurs at a depth of burial of about 1.5 diameters. For a coefficient of earth pressure at rest of 2.0, the moment coefficient is more sensitive to the stiffness of the tunnel liner. The moment coefficient generally decreases as the dimensionless depth of burial increases and reaches the fully buried condition at a dimensionless depth of burial of about 1.5. Although the moment coefficient is sensitive to the flexibility of the tunnel liner, for all the liners the values are below about 4 per cent. This value is considerably below the coefficient of 25 per cent which would result if the design were based upon a coefficient of earth pressure at rest of 2.0 for a rigid cylinder.

The dimensionless displacement from the finite-element solutions is shown versus the dimensionless depth of burial in Fig. 12. For a value of K_o of 0.5, the largest diameter change takes place on the horizontal diameter and decreases with increasing dimensionless depth of burial to a constant value of about 0.75 for dimensionless depths of burial greater than 1.5 diameters. For $K_o = 2.0$, the largest diameter change takes place on the vertical diameter and the dimensionless displacement decreases as the dimensionless depth of burial increases. The maximum dimensionless deflection for $K_o = 0.5$ for a liner with a flexibility ratio of 10 is about 1. Thus a simplified equation for liners with flexibility ratios greater than 10 may be written as follows

$$\frac{\Delta D}{D} = 1 \frac{\gamma H}{M_c} \frac{|1 - K_o|}{0.5} \qquad (21)$$

The value of Young's modulus for a clay may be approximated as being equal to 300 times the unconfined compressive strength. For reasonable values of Poisson's ratio corresponding to equilibrium conditions of drainage in clays, the value of the constrained modulus, M_c, is about 5/3 times the value of Young's modulus.

Equation 21 then becomes

$$\frac{\Delta D}{D} = \frac{1}{500} \frac{\gamma H}{q_u} \frac{|1 - K_o|}{0.5} \qquad (22)$$

Thus, when the total overburden stress is about 5 times the unconfined compressive strength of the soil, the diameter change should be about 1 per cent; when the ratio of the total overburden stress to the unconfined compression strength is about 2.5, the diameter change is about 0.5 per cent.

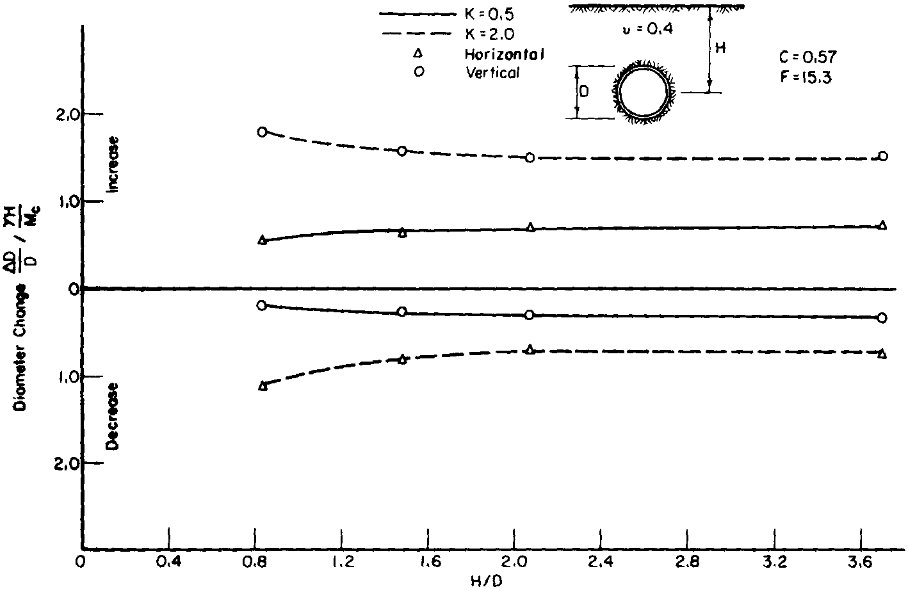


Fig. 12. Variation of diameter change with depth of burial

SUMMARY AND CONCLUSIONS

In this paper some of the recent trends and developments in the state of the art of practical tunneling have been reviewed. In addition, information has been presented to close the gap in current design procedures between tunnel liners that can be considered either perfectly flexible or perfectly rigid.

The results of a closed-form elastic-interaction solution have been presented which suggest the use of the compressibility ratio and the flexibility ratio as a means of defining tunnel liner stiffness. It has been shown that tunnels may for practical purposes be considered as flexible if the flexibility ratio is greater than about 10. Since the flexibility ratio is given as

$$F = \frac{\dfrac{E}{(1 + \nu)}}{\dfrac{6\, E_\ell\, I_\ell}{(1 - \nu_\ell^{\,2})}\ \dfrac{1}{R^3}}$$

and the Young's modulus of a clay is approximately 300 times the unconfined compressive strength, the tunnel liner behaves essentially as a flexible liner if its value of EI/R^3 is less than about 5 times the unconfined compressive strength of the soil. For liners more flexible than this, the deformations are dictated almost entirely by the properties of the soil and the depth of the tunnel. For liners with flexibility ratios greater than 10, it has also been shown that the diameter changes may be approximated by the following expression in terms of the tunnel depth and unconfined compressive strength of the soil.

$$\frac{\Delta D}{D} = \frac{1}{500}\ \frac{\gamma H}{q_u}\ \frac{|1 - K_o|}{0.5}$$

The thrust in a tunnel liner with a flexibility ratio greater than 10 can be expressed approximately by

$$\frac{T}{\gamma HR} = \frac{1}{2}\ (1 + K_o)\ [1.2 - 0.2C]$$

For the design of tunnel liners with flexibility ratios less than 10, moment and thrust coefficients should be taken from Figs. 10 and 11 (or similar figures obtained for a greater range in tunnel liner stiffness) for preliminary design. Accounting for the interaction between the soil and liner in this manner will result in a significant saving of reinforcement in concrete tunnel liners. This saving will become increasingly important as the state of the art permits the construction of larger diameter tunnels in soft ground.

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(2) Höeg, K., "Stresses Against Underground Structural Cylinders," Journal of the Soil Mechanics and Foundation Division, Vol. 94, No. SM4, 1968, pp. 833-858.

(3) Peck, R. B., "Deep Excavations and Tunneling in Soft Ground," Proceedings, 7th International Conference on Soil Mechanics, Mexico, State-of-the-art Volume, 1969, pp. 225-290.

(4) Schmidt, B., "Settlements and Ground Movements Associated with Tunneling in Soil," Ph. D. Thesis, University of Illinois, Urbana, 1969, 224 pp.

QUADEX™ A VORTEX COMPANY | STRUCTURAL ASSET RENEWAL

02/19/2024

Matt Huston | Business Development Director - Structure Rehabilitation
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RE: 50-Year Service Life Discussion for GeoKrete™ Geopolymer in Pipe Lining – Colton, SD

Mr. Huston,

I am submitting this information to you that substantiates the 50-year service life of GeoKrete Geopolymer applied by the Quadex Lining System (QLS). In making this engineering determination, the following discussion addresses how this result was arrived at and the supporting documentation that drove this determination. Extensive laboratory testing, along with field performance testing was performed.

The US Army Corps of Engineers (USACoE) address the question of expected service life of cementitious materials in their Manual No. 1110-2-2902 entitled: **Engineering and Design – Conduits, Culverts and Pipes**

1-4. Life Cycle Design

a. *General*. During the design process, selection of materials or products for conduits, culverts or pipes should be based on engineering requirements and life cycle performance. This balances the need to minimize first costs with the need for reliable long-term performance and reasonable future maintenance cost.

b. *Product Service Life*. Product made from different materials or with different protective coatings may exhibit markedly different useful lives. The service life of many products will be less than the product service life, and this must be considered in the life cycle design process. A literature search (Civil Engineering Research Foundation 1992) reported the following on product service lives for pipe materials. In general, concrete pipe can be expected to provide a product service life approximately two times that of steel or aluminum. However, each project has a unique environment, which may either increase or decrease product service life. Significant factors include soil pH, resistivity, water pH, presence of salts or other corrosive compounds, erosion sediment, and flow velocity. The designer should investigate and document key environmental factors and use them to select an appropriate service life.

Manual No. 1110-2-2902 entitled Engineering and Design – Conduits, Culverts and Pipes:
Continued

1. *Concrete*. Most studies estimated product service life for concrete pipe to be between 70 and 100 years. Of nine state highway departments, three listed the life as 100 years, five states stated between 70 and 100 years and one state gave 50 years.

Therefore, the US Army Corps of Engineers assumes a design life of 70-100 years for new concrete pipe, and there are countless examples of installations that surpass those numbers.

Because our GeoKrete lining material is a geopolymer, we offer the following specific properties which are superior to those of Ordinary Portland Cement: Highly increased acid and sulfate resistance, increased abrasion performance, low shrinkage, highly reduced footprint, high compressive strength, faster strength gain rate, increased heat resistance and a stable low-water mix ratio. Other material property advantages experienced in the field include improved pumpability, better bonding, very low rebound, and the ability to apply multiple layers without cold joints.

It is clear to see that the advantages of the geopolymer product warrant the 50-year design life status. While the US Corps of Engineers readily agrees many ordinary Portland cement products have lasted 50 years or more, Quadex advertises 50-75 years for the sake of conservatisms and acknowledges that the QLS produces pipes in a field environment and not a manufacturing setting.

The remaining item for consideration is engineer design, more specifically liner design thickness of the material to meet the service conditions as specified or determined by the asset owner. The design thickness will be a function of the service loads, environmental conditions, local regulations, material properties, hydraulic flow requirements and other factors specific to each project. This remains the responsibility of the project engineer to determine the final thickness required based on the factors provided by the asset owner in the advertisement documents. This ensures that the specific requirements for the service life of the culvert or pipe are considered. Certain project conditions may pose consideration for a service life well beyond 50 years and this may be considered by the third-party engineering providing the project's stamped liner design.

For our project designs we utilize third party registered engineers to perform and stamp the required design calculations and provide the thickness requirements, utilizing product and site-specific information. While ASTM designs are often useful, our design approach takes consideration of factors far beyond a standard calculator and provides a custom approach with project specific variables and appropriate safety factors.

Please do not hesitate to contact me with questions or further discussion regarding product service life and engineer considerations. We appreciate this opportunity to be of service to you and look forward to working with you in the very near future.

Sincerely,

Josh Marazzini
Vortex Companies - Technical Director
Josh.Marazzini@vortexcompanies.com
C 210.323.6997

Weston Blasius

From:	Josh Marazzini <Josh.Marazzini@vortexcompanies.com>
Sent:	Wednesday, March 6, 2024 1:53 PM
To:	Matt Huston; Weston Blasius
Cc:	Jerrit Pedersen; Reece Poppen; Beth Niemeyer; Michael Ingham; Adam Valenzuela; Tim Buzick; John Bluntach; Ryan Eakin; Matthew Peterson; Jeff Haas; Derek Offutt
Subject:	Re: Introduction - Vortex Companies / Banner Associates / City of Colton, SD
Attachments:	GeoKrete Geopolymer 50-year Service Life Letter - Colton, SD - HK Solutions Group.pdf

Matt,

Thank you for the introduction.

Team,

Please see discussion on your inquiries in green below. Should additional questions/clarifications come up as you review or anything further be needed, please don't hesitate to reach out.

Regards,

Josh Marazzini | Technical Director
Vortex Companies | 210.323.6997

From: Matt Huston <mhuston@HKSolutionsGroup.com>
Sent: Monday, February 19, 2024 4:07 PM
To: Josh Marazzini <Josh.Marazzini@vortexcompanies.com>; Weston Blasius <westonb@bannerassociates.com>
Cc: Jerrit Pedersen <jerritpedersen.coltonsd@gmail.com>; Reece Poppen <reecep@bannerassociates.com>; bethn@bannerassociates.com <bethn@bannerassociates.com>; Michael Ingham <MIngham@HKSolutionsGroup.com>; Adam Valenzuela <AValenzuela@hksolutionsgroup.com>; Tim Buzick <tbuzick@HKSolutionsGroup.com>; John Bluntach <jbluntach@HKSolutionsGroup.com>; Ryan Eakin <ryan.eakin@quadexonline.com>; Matthew Peterson <matt.peterson@vortexcompanies.com>; Jeff Haas <Jeff@hulexinc.com>
Subject: Introduction - Vortex Companies / Banner Associates / City of Colton, SD

Hi Josh,

Thank you for taking my call earlier. As mentioned, we had an opportunity to present the structural engineering memo and design calculations at our meeting today with Banner Associates and the City of Colton, SD. Overall, I feel the meeting went well, but there were a few questions asked which I think would be best answered directly by Vortex. Some of the questions and concerns include:

(1) What preparation method is recommended to achieve a CSP5 profile prior to the application of additional GeoKrete lining material?

Given the previously installed liner has likely achieved a compressive strength of 8,000 psi or better, we would assume either high water pressure blasting or media blasting would be necessary to achieve the desired profile. Alternatively, a surface profiler such as the LINKED can be used.

(2) What test method is recommended to check the bond between subsequent layers of GeoKrete lining?

Provided the surface is prepared as instructed the specified 50 psi, but no less than 35 psi discussed in the design, are conservative values we would not expect the product to have issue achieving. As we don't typically recommend coring and pull testing new infrastructure, we would instead recommend the focus of any inspection to be performed be on review of the profile prior to lining as opposed to testing following. We'd not recommend testing the bond after install as this introduced holes in new liners that then must be patched. With that said, if insisting that this be done for confidence in the product/installation, we'd recommend starting with a small random sampling of the MHs and pausing to review results and determine whether curiosities are satisfied before making a call whether or not to move on to testing additional MHs.

(3) If the second liner application is installed at a 1” minimum, is testing for bond to the initial liner required?

No, a 1.0-inch liner thickness is standalone structural and does not require additional material or host to be structural.

(4) What documentation is available supporting the 50-year liner design life.

Please find attached memo on this topic. While it will be noted that this memo focuses on storm installations in pipe, we would not expect any deviation in long term performance when used in MHs found with traditional sewer collection systems such as those proposed for this project. It is our understanding that the conditions the MHs are subjected to fall well within the advertised parameters of the GeoKrete material. As can be seen with review of the testing found within the GeoKrete Submittal Package, corrosive and abrasive testing has been performed to simulate decades of use in aggressive environments far in excess of the conditions anticipated for this project.

(5) What concerns would there be with removing the existing liner previously installed?

Complete destruction and failure of the MH to include collapse. It is incredibly difficult to remove materials such as these once installed. While we have never had anyone instruct us to remove a GeoKrete application, we have received that direction with other mortar lining materials in the past. While we understand the concern with the results as they have been presented, our experience has been that even on projects where, like this one, adhesion results for the liner weren't as high as specified/expected, during removal, liner was actually found to be performing much better than adhesion testing had suggested making liner removal incredibly difficult, especially without damage to the host structure.

With that, I’d like to make an introduction and provide the following contact info to start some dialog. Josh & Weston, feel free to loop-in any others within your organization as you see fit. It is proposed that recommendations will be presented at the City of Colton common council meeting on March 11, 2024. Please let me know if there is anything else needed at this time.

We understand from Matt that you may wish to have a member of the Vortex team virtually available for the March 11th meeting? if details on the timing can be provided, we will do our best to accommodate. In the meantime, should there be additional questions or a need for further discussion, please do not hesitate to reach out.

Thank you,

Matt Huston

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Structure Rehabilitation
HK Solutions Group
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Mobile: (612) 963-0643