ASSESSMENT AND REHABILITATION OF WEST 7TH STREET BRIDGE FULTON, MO

by

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1.0 INTRODUCTION

This report describes the superstructure conditions, sampling of materials, material characterization and recommendations for the repair and strengthening of the West 7th St. Bridge over Stinson Creek in the City of Fulton, Missouri (see Figure 1-1). The report also presents a summary of the subsurface site conditions, geotechnical data, laboratory work, and evaluation of alternatives for the bridge abutment wingwalls. The part dealing with the superstructure includes the following subjects:

- Structural analysis of bridge to determine the current and expected demands in the concrete members (i.e. arch ribs, beams and columns) with the objective to remove the load posting.
- Development of rehabilitation strategy which includes the repair of concrete and strengthening of the superstructure with Fiber Reinforced Polymer (FRP) composites.

The objective of the strengthening of the superstructure is to remove the 15 ton load posting that has been imposed on it

The analysis, conclusions, and recommendations for the rehabilitation of the substructure were based on site conditions existing at the time of the investigation and on the assumption that the information obtained from the borings is representative of the subsurface conditions throughout the site. Unanticipated conditions may be encountered during construction because of variations not detected during the investigation program. If, during construction, conditions differ due to natural or manmade causes, this report should be reviewed by qualified professionals to determine the applicability of the conclusions and recommendations concerning the differences in conditions. The objective of the substructure study is to determine the possible cause of the wing walls' failure and propose a conceptual design alternative for rehabilitation or replacement of the wing walls. This part includes the following information:

- Details of the subsurface investigation program
- Results of laboratory tests on soil samples
- Subsurface characterization, including boring logs
- Evaluation of bridge wingwall design alternatives

Figure 1-1. West 7th Street Bridge

2.0 PROJECT DESCRIPTION

The bridge was built in the 1910’s. The bridge structure has a span of 64.2 ft and a rise of 15 ft and it has an east-west orientation. The site location is shown on Figure 2-1. With regard to the boundary conditions, the bridge can be classified as a two-hingeless RC arch rib, whereas in accordance with the method of supporting the structure, it can be classified as an open spandrel arch, where the loads from the deck are transmitted to the arch by means of transversal beams and columns. The original deck was replaced in the 1970’s. The new concrete was cast on trapezoidal-type corrugated steel sheets running in the direction of traffic.

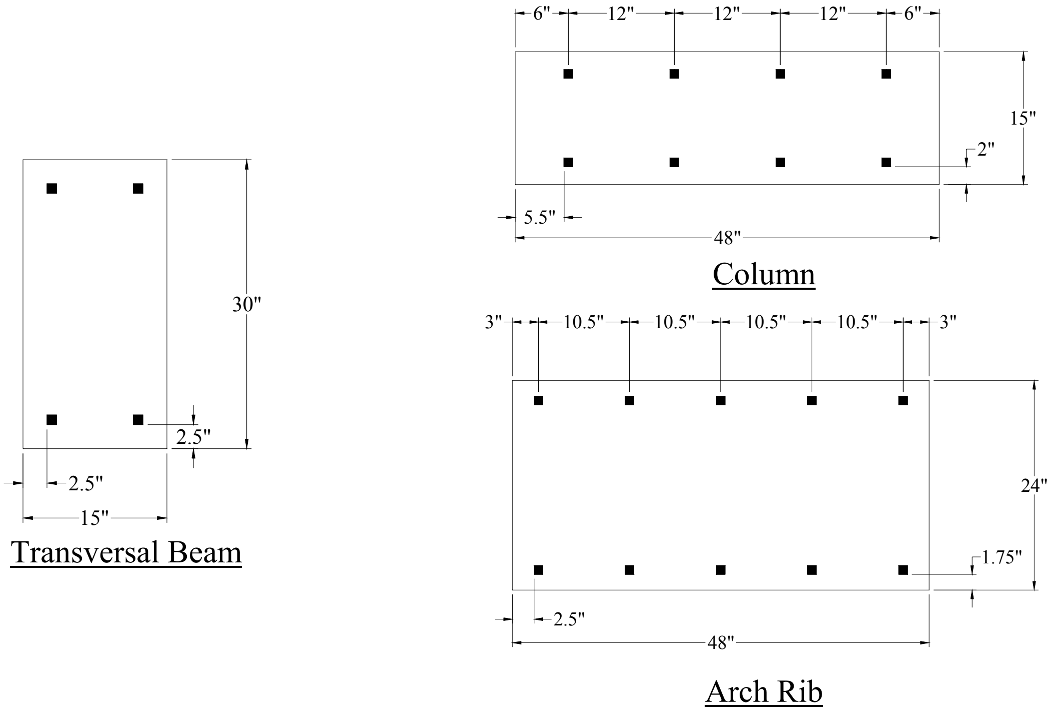
The abutments transmit the reaction from the superstructure to the foundation and retain the earth embankment of the approach roadway. The abutments are typical gravity abutments with wingwalls. The wing walls and the abutments are not structurally connected. Furthermore, it appears that the wing walls are unreinforced, consisting of a plain concrete section. The wingwalls on all four sides of the bridge abutments show extensive cracking and lateral displacement. Apparently, steel tieback anchors were installed in three of the four wingwalls in an attempt to stop the displacement and cracking of the wingwalls. However, the tiebacks have failed as indicated by their pulling out of the wingwall face. In general, the approach embankment slopes down from the bridge and road level at 2:1 to 3:1 slopes and is constrained by the bridge abutment wingwalls. The embankments are typically covered with grass and small brush, and a few trees are scattered throughout. A concrete sidewalk runs north-south underneath the eastern end of the bridge. Three 5-foot diameter culverts are located below the sidewalk north of the bridge, where the sidewalk crosses the creek. A wooden plank boardwalk runs parallel to the bridge along its northern side. This boardwalk is supported by a steel structure which is connected to the bridge deck.

Figure 2-1. Site Location Map

3.0 REHABILITATION OF SUPERSTRUCTURE

3.1 Bridge Inspection

Original drawings showing the internal reinforcement of the bridge were not available. A field survey was then conducted by personnel of the University of Missouri-Rolla (UMR). The survey included inspection of superstructure and substructure systems. The survey of the superstructure included the evaluation of the concrete condition, coring of concrete samples and location of steel reinforcement in the arch ribs, columns, and transversal beams. It was found that the structural elements were internally reinforced with square 1x1 in. steel rebars and lacked shear reinforcement (i.e. no stirrups/ties were detected). Details of the steel reinforcement are shown in 3-1.



Column

Transversal Beam

Arch Rib

Figure 3-1. Internal Steel Reinforcement

Concrete was cored from the east side of both arch ribs, close to the abutment. Visual inspection of the concrete in the north arch rib showed material soundness; whereas, concrete in the south arch rib allowed to observe some deficiencies, basically presence of air pockets and honeycombs (see 3-2a). During the coring, the water used to operate the coring machine, did flow out of the rib indicating that air pockets were spread throughout the region (see 3-2b). The area exhibiting honeycombs will require to be pressure-injected with a fine grout. The extension of the air pockets and honeycombing can only be determined with a thorough inspection of the entire bridge.


(a) Air Pockets (b) Water flowing out

Figure 3-2. Concrete Condition in South Rib

Some areas of the bridge exhbited concrete spalling and delamination caused by corrosion of steel reinforcement, as a consequence the reinforcement is exposed in some regions, mainly in the bridge crown (see 3-3). The repair work will include the replacement of the affected parts and restoration of the cross section with a no-shrinkage cementitious mortar.


Figure 3-3. Reinforcement Exposed in the Bridge Crown

Standard tests to determine the engineering properties of concrete (ASTM C39) and steel reinforcement (ASTM A370) were conducted. The results indicated that the concrete compressive strength, f'_c, is 3978 psi and the yielding strength of steel reinforcement, f_y, is 36 ksi.

3.2 Bridge Analysis

A preliminary analysis was conducted using a commercially available finite element software. The modeling was based on the dimensions provided by the City of Fulton (see 3-4).

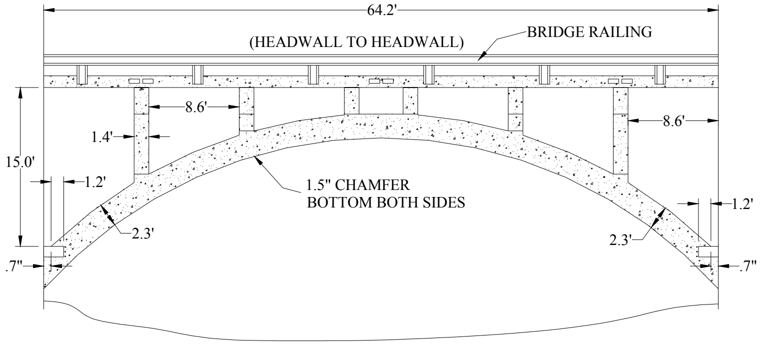
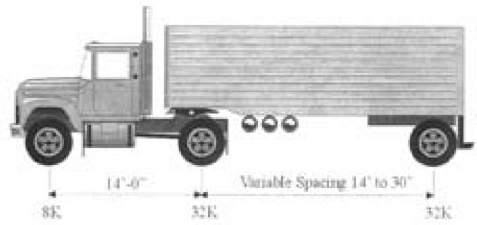


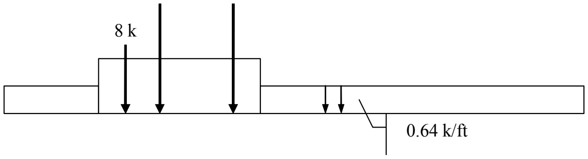
Figure 3-4. Bridge Geometry

The following considerations were taken into account for the analysis:

- Boundary conditions: the deck was assumed as simply supported by the transverse beams (i.e. no composite action). The arch was assumed fixed on both sides (hingeless).
- Loading conditions: According to ASSHTO provisions only one lane load is needed for the bridge analysis. This load configuration corresponds to a HS20 truck plus a uniform 0.64 kips/ft live load simulating smaller vehicles (see Figure 3-5). A uniform live load pressure of 150 psf was assumed for pedestrian traffic. The structural analysis takes into consideration both loading conditions acting non-simultaneously.



(a) HS20 Truck



(b) Load Lane Representing a HS20 Truck Plus a Uniform Live Load

Figure 3-5. Vehicle Loading Conditions

Figure 3-6 shows an idealization of the bridge used for analysis. Table 3-1 summarizes maximum values for axial load, bending moment, and shear force for beam, column, and arch element identified in Figure 3-6.

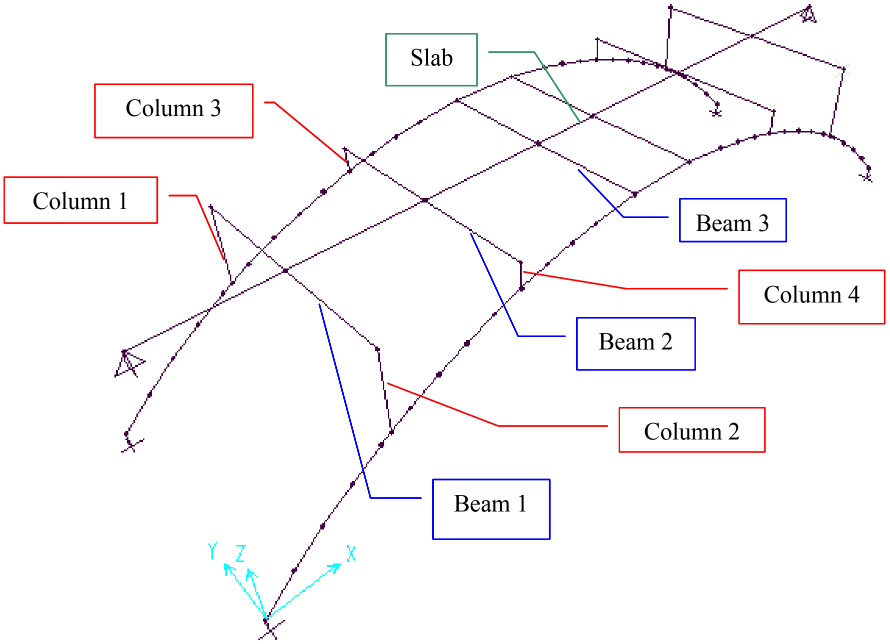


Figure 3-6. Bridge Idealization for Analysis

Table 3-1. Results of Bridge Analysis.

Superstructure Element	**Axial Load Pu (kips)**	**Bending Moment Mu (ft-kips)**		**Shear Force Vu (kips)**
Beam		Positive	Negative	
1	---	254	180	50
2	---	290	124	48
3	---	258	48	36
Slab	---	79	70	46
Column		Top	Bottom	
1	56	180	2	29
2	56	179	10	27
3	50	118	80	20
4	50	124	83	21
Arch Rib	315	241		34

3.3 Rehabilitation Strategy

3.3.1 Concrete Repair

The repair work of concrete will include the replacement of the affected parts and restoration of the cross sections. The procedure to follow can be summarized as:

- Location and marking of delaminated concrete areas using a hammer sounding technique.
- Removal of delaminated concrete until a minimum depth of ¾" under the corroded steel bars.
- Sawcutting of concrete in the periphery of the affected area to prevent feather edged conditions.
- Sandblasting and exposing of steel reinforcement to remove rust and scale. Surface cleaning is required to achieve an adequate bonding between the repair and the existing concrete.
- Impregnate an epoxy bonding agent to exposed areas. The material must meet the requirements specified by ASTM C881 (Epoxy-Resin Based Bonding Systems for Concrete)
- Gunite back using a design mix having a compressive strength of 5000 psi and finishing of surface.

The areas having honeycombs or large voids will be repaired by internal grouting of a hydraulic cement-based material.

3.3.2 Strengthening Strategy

The ultimate strength design criterion states that the design flexural capacity (or design shear capacity) of a member must exceed the flexural demand (or shear demand). Thus:

$$\phi M_n \geq M_u \ (\phi V_n \geq V_u)$$

if this condition is not satisfied the member needs to be strengthened.

Based on the results of the structural analysis, a strengthening strategy using CFRP laminates has been developed. The selected system, CF130, has a tensile strength of 550 ksi and a modulus of elasticity equal to 3300 ksi. The proposed strategy is in compliance with the "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures" reported by ACI Committee 440 and the "Building Code Requirements for Reinforced Concrete and Commentary" reported by ACI Committee 318. The flexural strengthening strategy is summarized in Table 3-2; whereas, the shear strengthening is summarized in Table -3. Details of the strengthening are presented in the Appendix A.

Table 3-2. Design of Flexural Strengthening

Superstructure Element		ϕMn (ft-kips) Before Strengthening	Mu (ft -kips)	FRP Reinforcement	ϕMn (ft –kips) After Strengthening
Beam					
1	$^{+}$M(1)	146.2	254	2 Bottom Plies CF 130, 15" wide	277.7
	$^{-}$M(1)	146.2	180	2 Lateral Plies CF 130, 9" wide	182.9
2	$^{+}$M	146.2	290	3 Bottom Plies CF 130, 15" wide	321.8
	$^{-}$M	146.2	124	---	---
3	$^{+}$M	146.2	258	2 Bottom Plies CF 130, 15" wide	277.7
	$^{-}$M	146.2	48	----	---
Slab (2)					
$^{+}$M		29.3	4.4=79/18.1	---	---
$^{-}$M		28.1	3.9=70/18.1	---	---

(1) M^{+} = positive moment, M^{-} = negative moment

(2) Moments in slab are expressed in ft-kips/ft. Slab width is equal to18.1 ft.

Table 3-3. Design of Shear Strengthening

	ϕVn (kips) Before Strengthening	Vu (kips)	FRP Reinforcement (a)	ϕVn (kips) After Strengthening
Beam				
1	44.4	50	1 Ply CF 130 U-wrap, 9" wide, 24” o.c	57.1
2	44.4	48	1 Ply CF 130 U-wrap, 9" wide, 24” o.c	57.1
3	44.4	36	---	---
Slab (1)	4.9	2.5=46/18.1	---	---
Column				
1	76.2	29	1 Ply CF 130 fully wrapped (2)	---
2	76.2	27		---
3	76.2	20		---
4	76.2	21		---
Arch Rib	133.5	34	1 Ply CF 130 fully wrapped, 24” wide, 36” o.c. (3)	---

(1) Shear forces in slab are expressed in kips/ft. Slab width is equal to18.1 ft.

(2) Even though ϕVn>Vu, ACI-318 (Sections 7.10.5.1 and 7.10.5.2) specifies that for compression members a minimum confining reinforcement should be provided.

(3) Only between the abutment and first column

4.0 REHABILITATION OF SUBSTRUCTURE

4.1 Subsurface Investigation

The subsurface exploration was performed at the W. 7th Street bridge in Fulton, Missouri in May 2002, to aid in determining the site subsurface conditions to be used in the feasibility study for wing wall rehabilitation.

4.1.1 Field Testing Program

The field investigation included drilling and sampling of two soil borings. The borings were located in the field by a B&V representative at each end of the bridge span, near the center of the approach roadway, by measuring from existing site structures. The test borings were drilled by Geotechnology, Inc. of St. Louis, Missouri, and were advanced to depths ranging from 26 feet to 34.5 feet below ground surface (bgs). Soil borings were advanced using 6-$^{3}/_{4}$ inch outside diameter (OD) hollow stem augers. Rock coring was performed in test boring BV-2, using a 2-inch diameter core barrel and water as drilling fluid. A truck-mounted drill rig was used to drill the borings. A B&V geotechnical engineer was present throughout the field work to observe drilling, assist in obtaining samples, and prepare descriptive logs of the test borings. Upon completion of drilling, all borings were backfilled to ground level with cement-bentonite grout.

Split spoon samples were obtained via the Standard Penetration Test (SPT), using a 2-inch split spoon sampler driven with a 140-pound automatic safety hammer. Relatively undisturbed samples of the cohesive soils were obtained by hydraulically pushing 3-inch OD thin-walled Shelby tubes into the soil at selected depths and locations. All samples were secured, sealed, and sent to the geotechnical laboratory at UMR for further testing. The sampling intervals, soil descriptions, SPT results, and other pertinent field data are summarized on individual boring logs presented in Appendix B.

4.1.2 Laboratory Testing

A laboratory test program was performed to classify the soils encountered at the site and to estimate engineering properties. The laboratory test program was developed by B&V and performed by the UMR.

The various laboratory tests performed on the soil samples recovered from the field included the following:

- Moisture content determinations of cohesive soil samples.

- Atterberg limits, including plastic and liquid limits.
- Dry density determinations on selected soil samples.
- Sieve and Hydrometer analysis to determine the fine-grained fraction of soil samples.
- Unconsolidated undrained (UU) triaxial shear strength tests on selected relatively undisturbed soil samples.

All testing was performed in accordance with established ASTM testing procedures. Laboratory test results are presented in Appendix C.

4.2 Subsurface Conditions

4.2.1 Soil Subsurface Conditions

Subsurface soils at the proposed site generally consist of 10 inches of concrete/asphalt pavement underlain by the following soil layers:

1. Fill Layer: Fill soils encountered at the site consisted of brown to dark brown and reddish brown, low plasticity silty clays, extending to depths of 18.5 to 19 feet bgs. The consistency of this layer is typically soft to medium stiff. Traces of gravel are typically encountered in this layer. The SPT N values range from 3 to 11 blows per foot (bpf), with an average of 5 bpf.
2. Layer M-1: Underlying the fill soils at the site is a black, soft to firm silt layer that extends to a depth of 20 to 20.5 feet bgs. Trace roots and decayed wood are encountered within this layer. The SPT N values range from 5 to 7 bpf, with an average of 6 bpf. Boring BV-1 encountered a half-inch thick seam of loose brown sand underlying this layer.
3. Layer L-2: Underlying the upper cohesive soils, is a 2.5 to 5 feet thick zone of weathered limestone, extending to a depth of 24.5 to 26 feet, where auger refusal was encountered.
4. Layer L-3: Fresh limestone was encountered underlying the weathered bedrock at the site. The limestone is white, fine grained, extremely strong and hard. Core recovery ranged from 50 to 96 percent, and RQD values ranged from 0 to 86 percent.

4.2.2 Groundwater Conditions

Groundwater levels were not recorded during drilling, although soils at a depth of about 18 to 19 feet bgs were wet. Groundwater levels at the site are expected to follow the level of the Stinson Creek, and are anticipated to fluctuate during high and low precipitation seasons.

4.3 Conceptual Design Evaluation

In-situ soils consist of soft to medium stiff, low plasticity silty clay fill with trace to some gravel. It appears that placement of this fill was not performed adequately, resulting in highly variable consistencies within the soil mass. The fill has continued to settle and creep under its own weight, resulting in increased lateral loads on the wingwalls. The increased lateral loads in combination with the lack of adequate reinforcement of the wing wall sections has resulted in considerable wall cracking and lateral displacement.

Based on the results of the field investigation, soil descriptions, and laboratory test results, conceptual design recommendations for the W. 7th Street Fulton bridge wingwall rehabilitation were developed. As requested by UMR, one of the options evaluated for conceptual design consists of a soil-nailed wall using fiber-reinforced polymer (FRP) ties or nails as opposed to regular steel ties. The second option chosen for evaluation consists of a gabion wall system. The evaluation of both these earth retaining wall systems is presented in the following sections.

4.3.1 Soil-Nailed Wall

Soil nailing is an in-situ soil reinforcement technique wherein passive inclusions, in this case soil nails, are placed into the natural ground at relatively close spacing to increase the strength of the soil mass. Construction is staged from top-down and, after each stage of excavation, the nails are installed, drainage systems are constructed, and shotcrete with wire reinforcement is applied to the excavation face.

Based on the in-situ soil conditions as well as the general site characteristics, we are of the opinion that the soil-nailed wall system is not the best option for rehabilitation of the wingwalls. Disadvantages of the application at this site include:

1. The nature of the in-situ soil. The highly variable consistency of the cohesive fill soils at the site makes it difficult for the soil nails to develop adequate pullout resistance capacity. Cohesive soils with low undrained shear strengths may continue to settle and creep under their own weight over a long term, thus increasing the lateral loading and facilitating nail pullout. Preliminary analysis indicates that soil nail lengths in excess of 20 feet would be required to stabilize the wingwalls.
2. Soil nails in excess of 20 feet would not only be more expensive and difficult to install, but could also interfere with underground utilities in the proximity of the bridge.

3. Lack of adequate space available for a top-down construction of soil nail walls, since the embankments are sloped. In the case of a bottom-up type nail installation, a 15 to 20 foot wide temporary embankment would have to be constructed over the creek to provide a stable base for the installation equipment and maneuvering. Construction of this embankment would have to take in consideration the fluctuating levels of the Stinson Creek which can flow full during periods of high precipitation, making construction schedule difficult.
4. Sand backfill could be used to replace the in-situ soils. However, this would not only add more cost to the construction but also lengthen the construction schedule since the backfill would have to be compacted in place, and more than likely some dewatering would be required at the base of the soil-nailed wall. The size of the overexcavation required would also be significant and excavation stabilization would need to be considered.
5. A specialized contractor is required for soil-nailed wall installation, which would add to the construction cost.

4.3.2 Gabion Wall

Gabion walls are compartmented units filled with stone that is 4 to 8 inches in size. Each unit is a rectangular basket made of galvanized steel wire. Each gabion unit is laced together on-site and filled with select stone. Select backfill is placed behind the gabion wall as required, with a filter geotextile placed between the backfill and in-situ soil if necessary.

For this project the gabion wall system is considered to be the best option for rehabilitation of the wingwalls, due to its simplicity of design and installation. The gabion wall would be placed directly on top of the weathered limestone layer to provide for an adequate bearing surface. Backfill behind the gabion wall may consist of clean gravel, with a filter geotextile between the gravel and in-situ soil. The main advantages of the the system at this site include:

1. Although overexcavation and replacement of a portion of the in-situ soils would be required, the amount of excavation would be minimal. Whereas for the soil-nailed wall a large overexcavation would be required over the whole height of the system to provide embedment of the soil nails in the granular backfill material, the gabion walls would only require the failure wedge behind the wall to be excavated and replaced. This allows for a sloped overexcavation of reduced area. Moreover, the amount of backfill required is reduced and this helps in reducing the installation cost.
2. Placing the gabion baskets on top of the weathered limestone at the site should be scheduled when the creek levels are expected to be low. However, the nature of the

installation of this system translates into a very quick installation time. It is estimated that it would take two days or less for installation of each wingwall. This makes planning of construction schedule easier to avoid times when the creek level may be high.

3. Installation of the gabion wall does not require a specialized contractor or specialized equipment. Lack of space is not an issue. Moreover, contractors that install gabion walls have been located in and near Fulton, Missouri.

Figure 4-1 presents a sketch of the conceptual design for gabion wing walls at the W. 7th street bridge site.

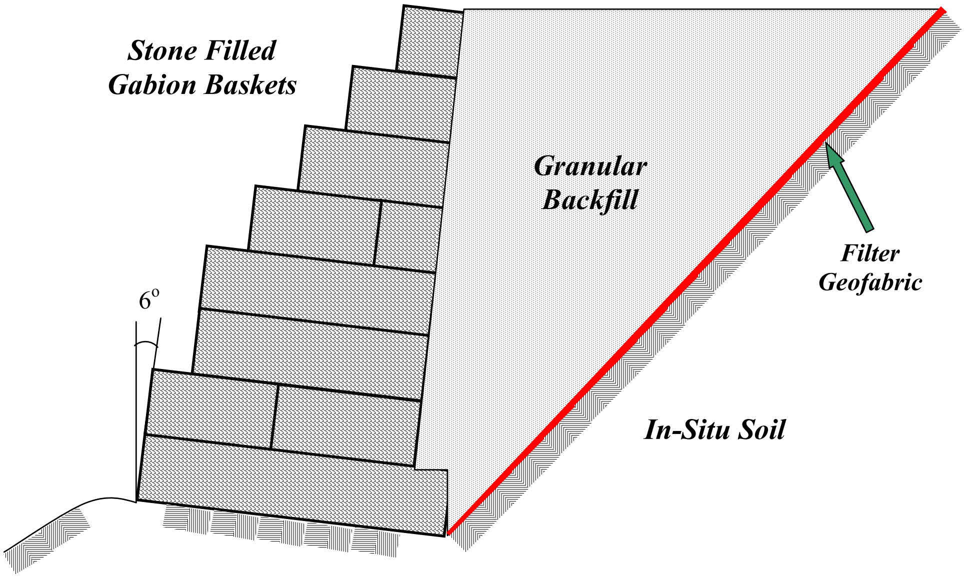


Table 4-1. Gabion Wall Conceptual Design

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Recommendations for Superstructure

The West 7th Street Bridge exhibits areas of concrete spalling, delamination caused by steel corrosion, and exposed reinforcement. In addition, to remove the load posting the structural analysis has demonstrated that the RC members (i.e. rib arches, beams and columns) are deficient in flexure and/or shear, on account of which they will need to be strengthened. To retrofit the bridge on what the superstructure concerns the actions listed below require to be executed:

- Repair of spalled areas of concrete needs to be conducted
- The concrete in the south arch rib exhibiting air pockets and honeycombs needs to be grout injected
- Shear and flexural capacities of RC members need to be upgraded using a CFRP system

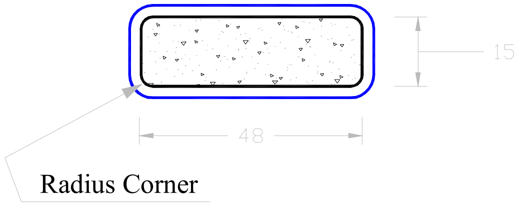
5.2 Recommendations for Substructure

In-situ soils at the Stinson creek bridge consist of soft to medium stiff, low plasticity silty clay fill with trace to some gravel, underlained by limestone bedrock which is weathered within its top 2.5 to 5.0 feet. Based on the results of the subsurface investigation and laboratory test results, it appears that placement of this fill was not performed adequately, resulting in highly variable consistencies within the soil mass. The fill has continued to settle and creep under its own weight, resulting in increased lateral loads on the wingwalls. The increased lateral loads in combination with the lack of adequate reinforcement of the wing wall sections has resulted in considerable wall cracking and lateral displacement.

It is recommended that gabion walls be used to replace the existing wing walls at the bridge. Gabion walls are deemed to be the most practical and economic option for this site. The gabion wall is to be supported on the weathered limestone layer encountered at the site. Backfill behind the gabion wall may consist of clean gravel, with a filter geotextile between the gravel and in-situ soil.

Appendix A
FRP Strengthening

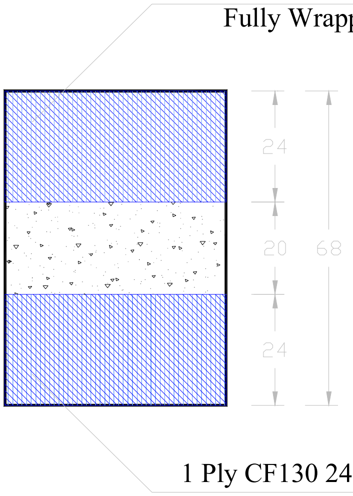
Columns 1 and 2



Radius Corner
2.5" minimum

Plan View

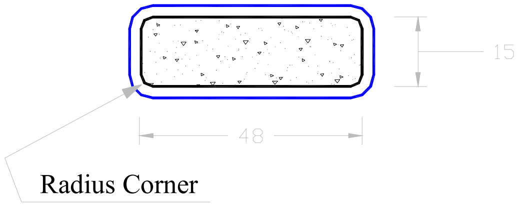
1 Ply CF130 24" wide
Fully Wrapped



1 Ply CF130 24" wide
Fully Wrapped

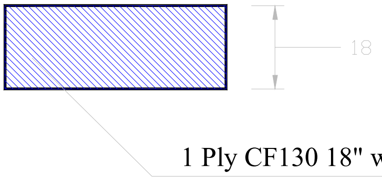
Elevation View

Columns 3 and 4



Radius Corner
2.5" minimum

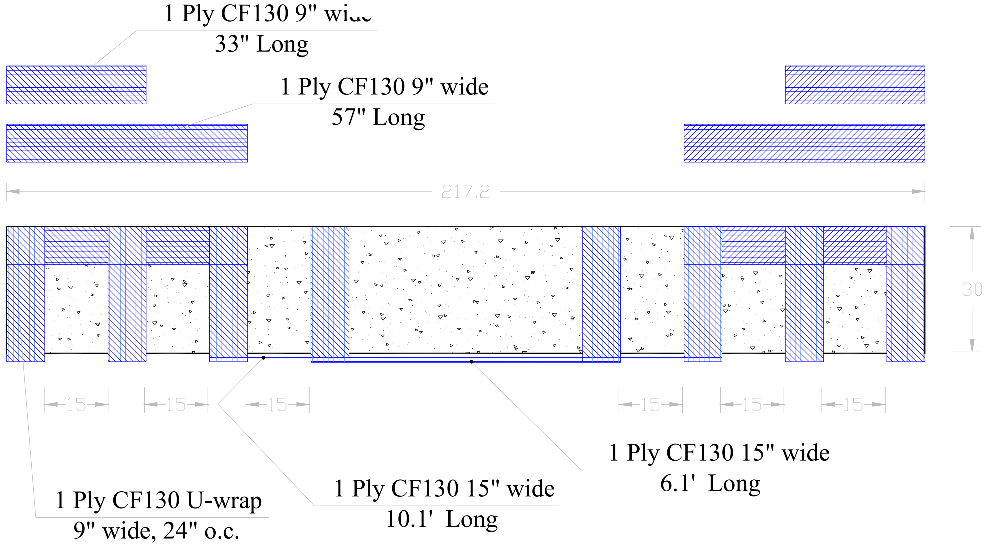
Plan View



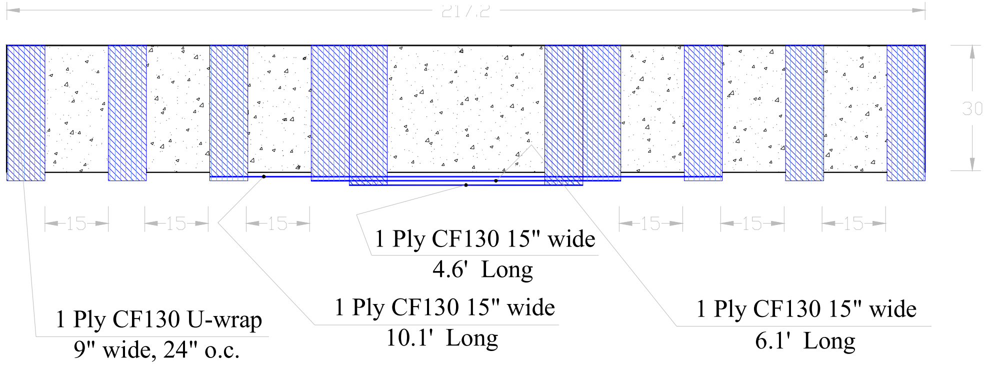
1 Ply CF130 18" wide
Fully Wrapped

Elevation View

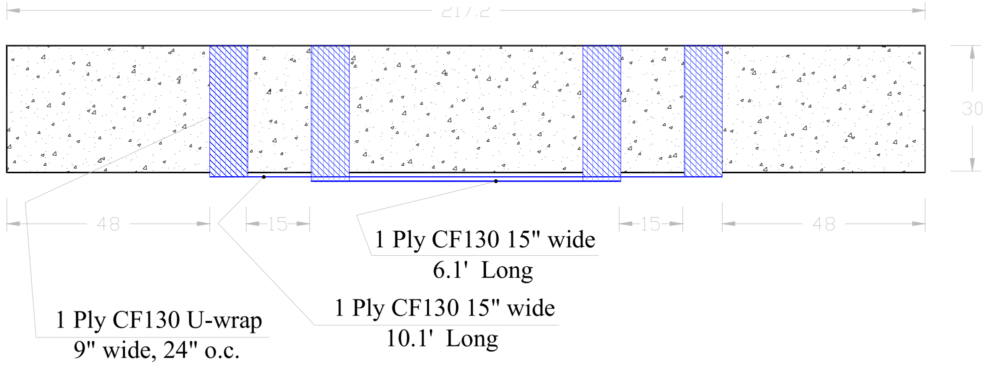
Beam 1



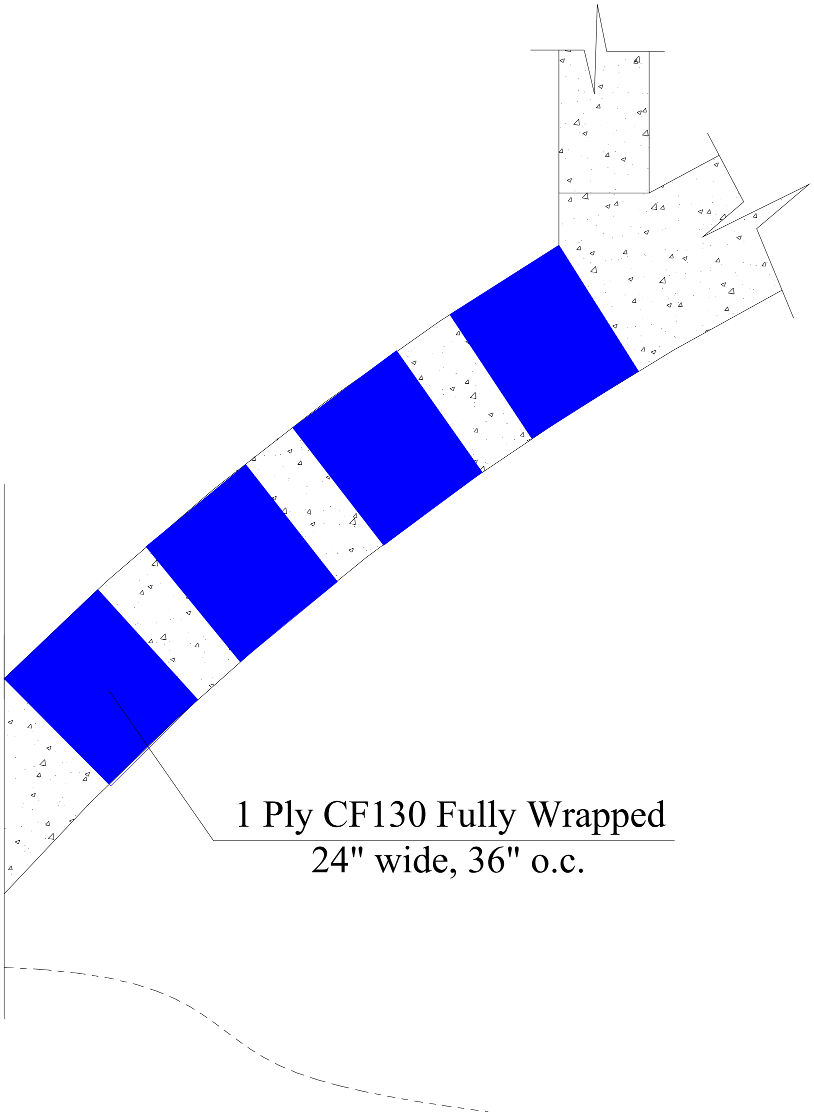
Beam 2



Beam 3



Arch Rib



Appendix B
Boring Logs

KEY TO SYMBOLS

Symbol	Description
<u>Strata symbols</u>	
▩	Concrete/Asphalt Pavement
▨	Silty <u>CLAY</u>
▥	Silt
░	Sand
▤	Weathered <u>LIMESTONE</u>
▦	Limestone
<u>Soil Samplers</u>	
◪	Standard penetration test
■	Undisturbed thin wall Shelby tube
▯	Rock core

<u>Notes:</u>

1. Exploratory borings were drilled on 05/30/02 using a 6-3/4 inch diameter hollow stem auger.

2. No free water was encountered at the time of drilling

3. Boring locations were taped from existing features

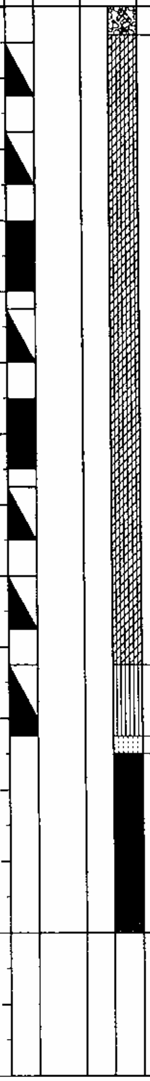
4. These logs are subject to the limitations, conclusions, and recommendations in this report.

BLACK & VEATCH LOG OF BORING

BORING NO. BV-1
SHEET 1 OF 1

CLIENT University of Missouri- Rolla	PROJECT W. 7th Street Bridge	PROJECT NO. 067322
PROJECT LOCATION Fulton, Missouri	COORDINATES	GROUND ELEVATION (DATUM)
TOTAL DEPTH 26.0 (FEET)	DATE START 05/30/02	DATE FINISHED 05/30/02
SURFACE CONDITIONS WEST ABUTMENT		
LOGGED BY Boris W Leoro	CHECKED BY GAZ Giuliana A Zelada	APPROVED BY Thomas P. Hart TH

SAMPLING: SAMPLE TYPE / CORING: CORE SIZE	SAMPLE NUMBER / RUN NUMBER	SET 6 INCHES / RUN LENGTH	2ND 6 INCHES / RUN RECOVERY	3RD 6 INCHES / RQD RECOVERY	N VALUE / PERCENT RECOVERY	SAMPLE RECOVERY / RQD	DEPTH (FEET)	CLASSIFICATION OF MATERIALS	REMARKS
							0	Concrete/Asphalt Pavement	Boring advanced with 6-3/4" O.D hollow stem augers
							0.8	Silty CLAY; brown; soft; moist; low plasticity (FILL)	
SPT	1	2	2	2	4	0.2	2		
							4	grading with trace gravel	
SPT	2	1	1	2	3	0.5			
							6		PP=1.25 to 1.75 tsf
TW	3					1.9	8		
								grading dark brown w/ reddish brown and gray mottles; firm	
SPT	4	1	3	4	7	1.2	10		
							12		Tube slightly bent at the tip PP=2.0 to 2.5 tsf
TW	5					1.6			
							14		PP=1.0 to 2.0 tsf
SPT	6	1	3	4	7	1.2			
							16		4" rock at the tip of split spoon
SPT	7	2	5	6	11	1.2			
							18		
							18.5	SILT; black; soft to firm; moist to wet; low plasticity; with trace roots and decayed wood	PP<0.25 tsf
SPT TW	8	2	3	4	7	1.0 0.5	20		
							20.5	SAND; brown; loose; wet; medium grained; well graded; rounded	difficult drilling from 21'
							21.0	Weathered bedrock (LIMESTONE)	
							22		
							24		Auger refusal at 26'
							26.0		Bottom of Boring @ 26'. Water level not recorded. Boring backfilled with cement-bentonite.
							28		
							30		

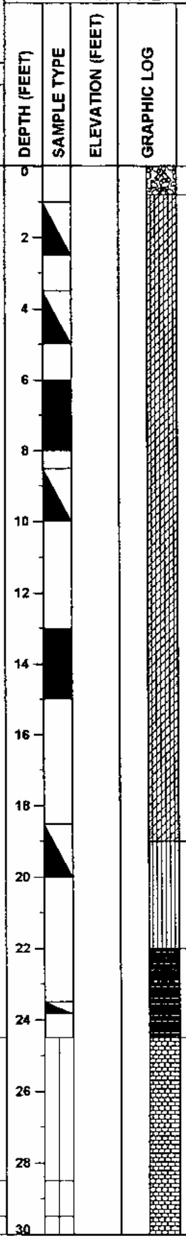


BLACK & VEATCH LOG OF BORING

BORING NO. BV-2
SHEET 1 OF 2

CLIENT: University of Missouri- Rolla
PROJECT: W. 7th Street Bridge
PROJECT NO.: 067322
PROJECT LOCATION: Fulton, Missouri
COORDINATES:
GROUND ELEVATION (DATUM):
TOTAL DEPTH: 34.5 (FEET)
SURFACE CONDITIONS: EAST ABUTMENT
DATE START: 05/30/02
DATE FINISHED: 05/30/02
LOGGED BY: Boris W Leoro
CHECKED BY: GAZ Giuliana A Zelada
APPROVED BY: Thomas P. Hart TH

SAMPLING									
SAMPLE TYPE	SAMPLE NUMBER	SET 6 INCHES	2ND 6 INCHES	3RD 6 INCHES	N VALUE	SAMPLE RECOVERY		CLASSIFICATION OF MATERIALS	REMARKS
CORING									
CORE SIZE	RUN NUMBER	RUN LENGTH	RUN RECOVERY	RQD RECOVERY	PERCENT RECOVERY	RQD	DEPTH (FEET)		
							0	Concrete/Asphalt Pavement	Boring advanced with 6-3/4" O.D hollow stem augers
							0.8	Silty CLAY; dark brown; soft to firm; moist; low plasticity ; w/ trace roots (FILL)	
SPT	1	2	2	2	4	0.5			
SPT	2	1	2	3	5	0.8		grading reddish brown	
									PP=0.75 to 1.0 tsf
								grading dark brown	
TW	3					1.0			
								grading with gray mottles; trace gravel	
SPT	4	2	2	3	5	0.8			
									Tube tip bent due to gravel
TW	5					1.0			
							19.0	SILT; black; soft to firm; wet; low plasticity; w/ trace gravel; some roots and decayed wood	
SPT	6	1	2	3	5	1.0			
							22.0	Weathered LIMESTONE	
SPT	7	50/4"	-	-	>50	0.2			Auger refusal at 24.5'
			24.5				24.5	LIMESTONE; white; fine grained; slightly weathered to fresh; extremely strong; hard; with several clay filled discontinuities up to 6" thick	
2"	1	4.0	2.0	0.0	50	0			
			28.5						Water loss @ 28'. Core plugged with clay @ 28.5'.
2"	2	1.0	0.67	0	67	0			
			29.5						



BLACK & VEATCH

LOG OF BORING

BORING NO. BV-2
SHEET 2 OF 2

CLIENT University of Missouri- Rolla
PROJECT W. 7th Street Bridge
PROJECT NO. 067322
PROJECT LOCATION Fulton, Missouri
COORDINATES
GROUND ELEVATION (DATUM)
TOTAL DEPTH 34.5 (FEET)
SURFACE CONDITIONS EAST ABUTMENT
DATE START 05/30/02
DATE FINISHED 05/30/02
LOGGED BY Boris W Leoro
CHECKED BY GAZ Giuliana A Zelada
APPROVED BY Thomas P. Hart TH

SAMPLING: SAMPLE TYPE CORING: CORE SIZE	SAMPLE NUMBER RUN NUMBER	SET 6 INCHES RUN LENGTH	2ND 6 INCHES RUN RECOVERY	3RD 6 INCHES RQD RECOVERY	N VALUE PERCENT RECOVERY	SAMPLE RECOVERY RQD	DEPTH (FEET)	SAMPLE TYPE	ELEVATION (FEET)	GRAPHIC LOG	CLASSIFICATION OF MATERIALS	REMARKS
2"	3	5	4.8	4.3	96	86	30 32 34			▨	LIMESTONE; white; fine grained; slightly weathered to fresh; extremely strong; hard; w/ few clay filled discontinuities less than 0.1" thick	
							36 38 40 42 44 46 48 50 52 54 56 58 60					Bottom of Boring @ 34.5'. Water level not recorded. Boring backfilled with cement-bentonite.

Appendix C
Laboratory Test Results

Soil Testing Of Fulton Bridge Project

EXECUTIVE SUMMARY

The lab tests including water content, Atterberg limit and hydrometer analysis were conducted for different kinds of soil samples provided by the BLACK & VEARECH company. The descriptions of the tested soils are in the log. Due to the quality of Shelby tubes, the strength tests will be conducted depending on the evaluation of the sample's integrity.

METHODOLOGY

The testing was performed in accordance with ASTM D 2216 for water content, ASTM D 4318 for Atterberg limit and ASTM D 422 for hydrometer analysis. The water content of thirteen samples was obtained. Atterberg limit testing was performed for nine soil samples. Due to the existence of sand and gravel, the samples were dealt with to pass sieve 200. Four samples were tested.

RESULTS AND ANALYSIS

The test results are shown in the following tables. The detailed test data, calculation and results are attached as appendix.

Table 1 Water content and Atterberg limit

Boring number	Sample number	Water content (%)	Liquid limit (%)	Plastic limit (%)	PI	Soil type
BV-1	SPT-2	24.2	38.95	21.32	17.63	CL
BV-1	SPT-4	20.5	40.18	23.34	16.84	CL
BV-1	SPT-6	25.4	41.48	23.60	17.88	CL
BV-1	SPT-7	26.6	46.57	24.44	22.13	CL
BV-1	SPT-8	20.6	26.77	18.20	8.57	CL
BV-1	TW-3	27.8				
BV-1	TW-5	22.9				
BV-2	SPT-1	17.8	37.99	20.93	17.06	CL
BV-2	SPT-2	21.8	43.03	23.99	19.04	CL
BV-2	SPT-4	23.4	39.73	22.34	17.39	CL
BV-2	SPT-6	25.3	23.37	18.04	5.33	CL-ML
BV-2	TW-3	24.3				
BV-2	TW-5	27.7				

Table 2 Clay fraction by hydrometer analysis

Boring number	Sample number	Clay fraction (%)
BV-1	TW-3	28.8
BV-1	TW-5	35.1
BV-2	TW-3	35.2
BV-2	TW-5	37.4

The above results are based on the soil sample passing sieve 200 and don't account for the grain size distribution of total soil samples.

Based on the Casagrande's plasticity chart, the soil samples are classified as shown in the table 1 according to USCS.

CONCLUSIONS

Based on the test results, the soils are classified as silty clay and clayed silt with some gravel and sand. In order to make sure the bearing capacity of soil for the bridge foundation, additional testing is recommended. The additional testing would include:

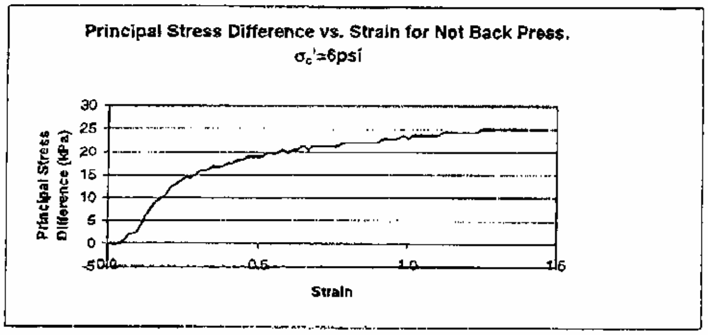
- Hydraulic conductivity tests for seepage characteristic determination
- Consolidation tests for time dependent deformation characteristics determination
- CU Triaxle tests for stress-strain determination

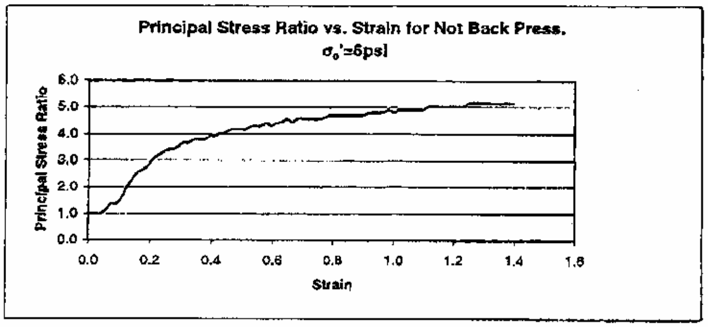
Undrained triaxial test (BV-1 TW3)

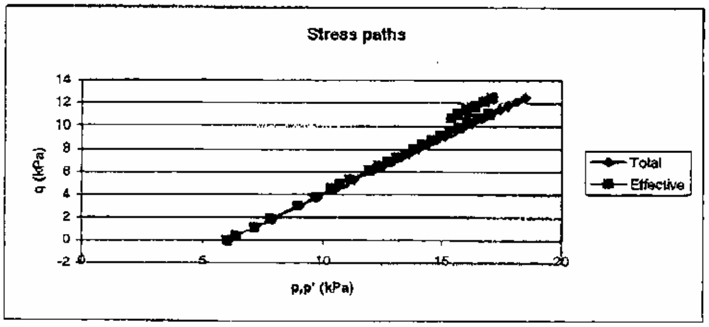
Weight of dry sample W =	2.8639 lb
Initial height of sample h_0 =	6 in
Initial sample diameter D_0 =	2.898 in
Soil specific gravity G_s =	2.65 (assume)
Confining pressure σ_3 =	6 psi
Back pressure σ_b =	0 psi
Saturation coefficient B =	0 %
Rate of loading v =	1.52 mm/min
Water content w=	20%
Initial dry unit weight γ_{d0} =	0.07 pci
Volume change during consolidation ΔV_c =	0.00 in^3
Dry unit weight after consolidation γ_d =	0.07 pci
Height after consolidation h_c =	6.00 in
Volume after consolidation V_c =	39.58 in^3
Area after consolidation A_c =	6.60 in^2
Shear modulus G =	0.85 psi
Peak friction angle ϕ_p =	57.63 deg
Residual friction angle ϕ_r =	57.61 deg
Peak undrained shear strength S_u =	25.00 psi
Residual undrained shear strength S_{ur} =	24.98 psi

Axial displacement (in)	Axial force (lbf)	Pore pressure (psi)	Axial strain (%)	Shear stress (psi)	Effective normal stress (psi)	Effective Confining Stress (psi)	Total Normal Stress (psi)	τ/σ'	Principal Stress Difference (psi)	Principal Stress ratio	q	p	p'
Δh	F	u	eps	τ	σ_1'	σ_3'	σ_1	ratio	$\Delta\sigma$	ratio	(psi)	(psi)	(psi)
0.0000	0.0000	0.00	0.00	0.0	6.0	6.00	6.0	0.00	0.00	1.00	0.00	6.00	6.00
0.0128	0.0000	0.00	0.02	0.0	6.0	6.00	6.0	0.00	0.00	1.00	0.00	6.00	6.00
0.0229	0.0000	0.00	0.04	0.0	6.0	6.00	6.0	0.00	0.00	1.00	0.00	6.00	6.00
0.0326	5.0000	0.00	0.05	0.8	6.8	6.00	6.8	0.11	0.76	1.13	0.38	6.38	6.38
0.0438	15.0000	0.00	0.07	2.3	8.3	6.00	8.3	0.27	2.27	1.38	1.14	7.14	7.14
0.0546	15.0000	0.00	0.09	2.3	8.3	6.00	8.3	0.27	2.27	1.38	1.14	7.14	7.14
0.0646	25.0000	0.10	0.11	3.8	9.7	5.90	9.8	0.39	3.79	1.63	1.89	7.89	7.79
0.0743	40.0000	0.10	0.12	6.1	12.0	5.90	12.1	0.51	6.06	2.01	3.03	9.03	8.93
0.0844	50.0000	0.10	0.14	7.8	13.5	5.90	13.6	0.56	7.57	2.26	3.78	9.78	9.68
0.0941	60.0000	0.20	0.16	9.1	14.9	5.80	15.1	0.61	9.08	2.51	4.54	10.54	10.34
0.1040	65.0000	0.20	0.17	9.8	15.6	5.80	15.8	0.63	9.84	2.64	4.92	10.92	10.72
0.1144	70.0000	0.20	0.19	10.6	16.4	5.80	16.6	0.65	10.59	2.77	5.30	11.30	11.10
0.1243	81.0000	0.20	0.21	12.3	18.1	5.80	18.3	0.68	12.25	3.04	6.13	12.13	11.93
0.1342	86.0000	0.20	0.22	13.0	18.8	5.80	19.0	0.69	13.01	3.17	6.50	12.50	12.30
0.1449	91.0000	0.20	0.24	13.8	19.6	5.80	19.6	0.70	13.76	3.29	6.88	12.88	12.68
0.1546	96.0000	0.20	0.26	14.5	20.3	5.80	20.5	0.71	14.52	3.42	7.26	13.26	13.06
0.1645	96.0000	0.20	0.27	14.5	20.3	5.80	20.5	0.71	14.51	3.42	7.26	13.26	13.06
0.1764	101.0000	0.20	0.29	15.3	21.1	5.80	21.3	0.72	15.27	3.64	7.63	13.63	13.43
0.1853	106.0000	0.20	0.31	16.0	21.8	5.80	22.0	0.73	16.02	3.67	8.01	14.01	13.81
0.1952	106.0000	0.20	0.33	16.0	21.8	5.80	22.0	0.73	16.02	3.67	8.01	14.01	13.81
0.2059	111.0000	0.20	0.34	16.8	22.6	5.80	22.8	0.74	16.77	3.80	8.39	14.39	14.19
0.2161	111.0000	0.20	0.36	16.8	22.6	5.80	22.8	0.74	16.77	3.79	8.38	14.38	14.18
0.2280	111.0000	0.20	0.38	16.8	22.6	5.80	22.8	0.74	16.76	3.79	8.38	14.38	14.18
0.2367	116.0000	0.20	0.39	17.5	23.3	5.80	23.5	0.75	17.52	3.92	8.76	14.76	14.56
0.2466	116.0000	0.20	0.41	17.5	23.3	5.80	23.5	0.75	17.51	3.92	8.76	14.76	14.56
0.2565	121.0000	0.20	0.43	18.3	24.1	5.80	24.3	0.76	18.27	4.04	9.13	15.13	14.93
0.2669	121.0000	0.20	0.44	18.3	24.1	5.80	24.3	0.76	18.26	4.04	9.13	15.13	14.93
0.2766	126.0000	0.20	0.46	19.0	24.8	5.80	25.0	0.77	19.01	4.17	9.51	15.51	15.31
0.2862	126.0000	0.20	0.48	19.0	24.8	5.80	25.0	0.77	19.01	4.17	9.51	15.51	15.31
0.2959	126.0000	0.20	0.49	19.0	24.8	5.80	25.0	0.77	19.01	4.17	9.50	15.50	15.30
0.3063	126.0000	0.20	0.51	19.0	24.8	5.80	25.0	0.77	19.00	4.17	9.50	15.50	15.30
0.3159	131.0000	0.20	0.53	19.8	25.6	5.80	25.8	0.77	19.76	4.29	9.88	15.88	15.68
0.3258	131.0000	0.20	0.54	19.8	25.6	5.80	25.8	0.77	19.75	4.29	9.88	15.88	15.68
0.3363	131.0000	0.20	0.56	19.7	25.5	5.80	25.7	0.77	19.76	4.29	9.87	15.87	15.67
0.3467	136.0000	0.20	0.58	20.5	26.3	5.80	26.5	0.78	20.50	4.42	10.25	16.25	16.05
0.3566	131.0000	0.20	0.59	19.7	25.5	5.80	25.7	0.77	19.74	4.29	9.87	15.87	15.67
0.3676	136.0000	0.20	0.61	20.6	26.3	5.80	26.6	0.78	20.49	4.42	10.25	16.25	16.05
0.3775	136.0000	0.20	0.63	20.5	26.3	5.80	26.5	0.78	20.49	4.41	10.24	16.24	16.04

0.3879	142.0000	0.20	0.65	21.4	27.2	5.80	27.4	0.79	21.39	4.56	10.69	16.69	16.49
0.3986	136.0000	0.20	0.66	20.5	26.3	5.60	26.5	0.78	20.46	4.41	10.24	16.24	16.04
0.4082	142.0000	0.20	0.68	21.4	27.2	5.80	27.4	0.79	21.38	4.56	10.69	16.69	16.49
0.4181	142.0000	0.20	0.70	21.4	27.2	5.80	27.4	0.79	21.38	4.56	10.69	16.69	16.49
0.4288	142.0000	0.20	0.71	21.4	27.2	5.80	27.4	0.78	21.37	4.56	10.69	16.69	16.49
0.4387	142.0000	0.10	0.73	21.4	27.3	5.90	27.4	0.78	21.37	4.56	10.69	16.69	16.59
0.4484	142.0000	0.10	0.75	21.4	27.3	5.90	27.4	0.78	21.37	4.56	10.68	16.68	16.58
0.4588	142.0000	1.30	0.76	21.4	28.1	4.70	27.4	0.82	21.36	4.56	10.68	16.68	15.38
0.4687	147.0000	0.10	0.78	22.1	28.0	5.90	28.1	0.79	22.11	4.69	11.06	17.06	16.96
0.4786	147.0000	0.10	0.80	22.1	28.0	5.90	28.1	0.79	22.11	4.68	11.05	17.06	16.95
0.4882	147.0000	0.10	0.81	22.1	28.0	5.90	28.1	0.79	22.10	4.68	11.05	17.05	16.95
0.4987	147.0000	0.10	0.83	22.1	28.0	5.90	28.1	0.79	22.10	4.68	11.05	17.05	16.95
0.5086	147.0000	0.10	0.85	22.1	28.0	5.90	28.1	0.79	22.10	4.68	11.05	17.05	16.95
0.5180	147.0000	1.40	0.86	22.1	26.7	4.60	28.1	0.83	22.09	4.68	11.05	17.05	15.65
0.5289	147.0000	1.40	0.88	22.1	26.7	4.60	28.1	0.83	22.09	4.68	11.04	17.04	15.64
0.5388	147.0000	1.40	0.90	22.1	26.7	4.60	28.1	0.83	22.09	4.68	11.04	17.04	15.64
0.5487	152.0000	1.40	0.91	22.8	27.4	4.60	28.8	0.83	22.83	4.81	11.42	17.42	16.02
0.5592	152.0000	1.40	0.93	22.8	27.4	4.60	28.8	0.83	22.83	4.80	11.41	17.41	16.01
0.5693	152.0000	1.40	0.95	22.8	27.4	4.60	28.8	0.83	22.83	4.80	11.41	17.41	16.01
0.5782	152.0000	1.40	0.97	22.8	27.4	4.60	28.8	0.83	22.82	4.80	11.41	17.41	16.01
0.5902	157.0000	1.40	0.98	23.6	28.2	4.60	29.6	0.84	23.57	4.93	11.78	17.78	16.38
0.6001	152.0000	1.40	1.00	22.8	27.4	4.60	28.8	0.83	22.81	4.80	11.41	17.41	16.01
0.6103	157.0000	1.40	1.02	23.6	28.2	4.60	29.6	0.84	23.56	4.93	11.78	17.78	16.38
0.6209	157.0000	1.40	1.03	23.6	28.2	4.60	29.6	0.84	23.56	4.93	11.78	17.78	16.38
0.6306	157.0000	1.40	1.05	23.6	28.2	4.60	29.6	0.84	23.55	4.93	11.78	17.78	16.38
0.6402	157.0000	1.40	1.07	23.5	28.1	4.60	29.5	0.84	23.55	4.92	11.77	17.77	16.37
0.6501	157.0000	1.40	1.08	23.5	28.1	4.60	29.5	0.84	23.54	4.92	11.77	17.77	16.37
0.6603	157.0000	1.40	1.10	23.5	28.1	4.00	29.5	0.84	23.54	4.92	11.77	17.77	16.37
0.6702	162.0000	1.40	1.12	24.3	28.9	4.60	30.3	0.84	24.29	5.05	12.14	18.14	16.74
0.6799	162.0000	1.40	1.13	24.3	28.9	4.60	30.3	0.84	24.28	5.05	12.14	18.14	16.74
0.6903	162.0000	1.40	1.15	24.3	28.9	4.60	30.3	0.84	24.28	5.05	12.14	18.14	16.74
0.6997	162.0000	1.40	1.17	24.3	28.9	4.60	30.3	0.84	24.27	5.05	12.14	18.14	16.74
0.7093	162.0000	1.40	1.18	24.3	28.9	4.60	30.3	0.84	24.27	5.04	12.13	18.13	16.73
0.7200	162.0000	1.40	1.20	24.3	28.9	4.60	30.3	0.84	24.27	5.04	12.13	18.13	16.73
0.7302	162.0000	1.40	1.22	24.3	28.9	4.60	30.3	0.84	24.26	5.04	12.13	18.13	16.73
0.7401	162.0000	1.40	1.23	24.3	28.9	4.60	30.3	0.84	24.26	5.04	12.13	18.13	16.73
0.7506	167.0000	1.40	1.25	25.0	29.6	4.60	31.0	0.84	25.00	5.17	12.50	18.50	17.10
0.7607	167.0000	1.40	1.27	25.0	29.6	4.60	31.0	0.84	25.00	5.17	12.50	18.50	17.10
0.7706	167.0000	1.40	1.28	25.0	29.6	4.60	31.0	0.84	24.99	5.17	12.50	18.50	17.10
0.7813	167.0000	1.40	1.30	25.0	29.6	4.60	31.0	0.84	24.99	5.16	12.49	18.49	17.09
0.7914	167.0000	1.40	1.32	25.0	29.6	4.60	31.0	0.84	24.98	5.16	12.49	18.49	17.09
0.8019	167.0000	1.40	1.34	25.0	29.6	4.60	31.0	0.84	24.98	5.16	12.49	18.49	17.09
0.8118	167.0000	1.40	1.35	25.0	29.6	4.60	31.0	0.84	24.98	5.16	12.49	18.49	17.09
0.8217	167.0000	1.40	1.37	25.0	29.6	4.60	31.0	0.84	24.97	5.16	12.49	18.49	17.09
0.8313	167.0000	1.40	1.39	25.0	29.6	4.60	31.0	0.84	24.97	5.16	12.48	18.48	17.08
0.8384	167.0000	1.40	1.40	25.0	29.6	4.60	31.0	0.84	24.98	5.16	12.48	18.48	17.08
0.8384	167.0000	1.40	1.40	25.0	29.6	4.60	31.0	0.84	24.98	5.16	12.48	18.48	17.08


Principal Stress Difference vs. Strain for Not Back Press.
σc'=6psi
Principal Stress Difference (kPa)
30
25
20
15
10
5
0
-5
0.0
0.5
1.0
1.5
Strain


Principal Stress Ratio vs. Strain for Not Back Press.
σ0'=6psi
Principal Stress Ratio
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
Strain

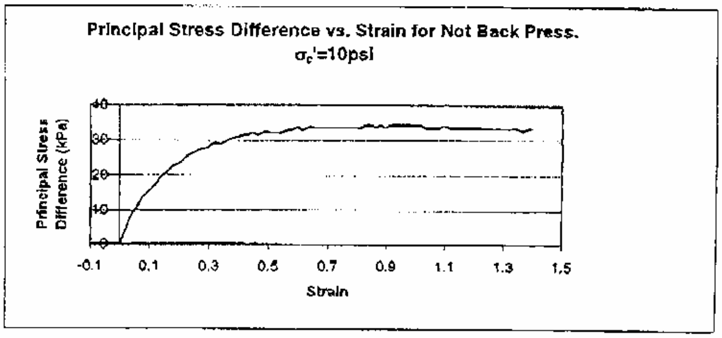

Stress paths
q (kPa)
14
12
10
8
6
4
2
0
-2
0
5
10
15
20
p,p' (kPa)
Total
Effective

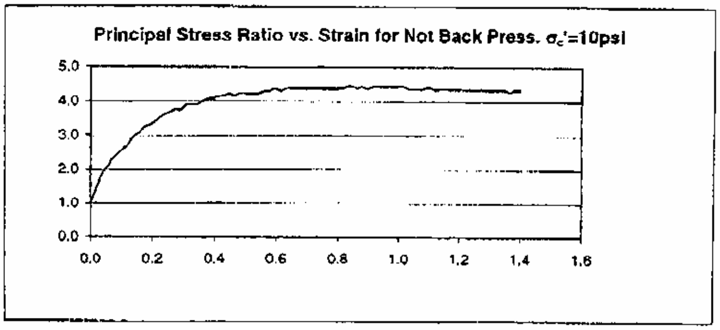
Undrained triaxial test (BV-1 TW5)

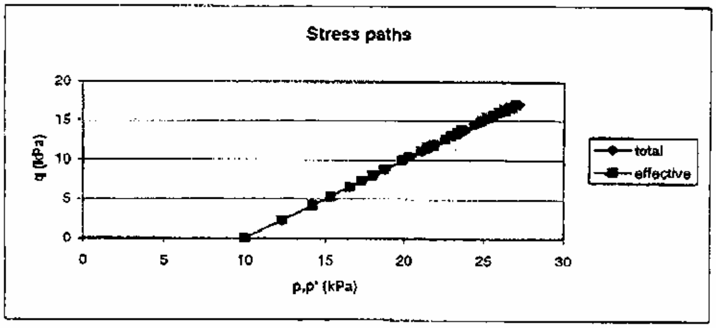
Weight of dry sample W = 2.84765 lb
Initial height of sample h_0 = 6 in
Initial sample diameter D_0 = 2.893 in
Soil specific gravity G_s = 2.65 (assume)
Confining pressure σ_3 = 10 psi
Back pressure σ_b = 0 psi
Saturation coefficient B = 0 %
Rate of loading v = 1.52 mm/min
Water content w= 19%
Initial dry unit weight γ_{d0} = 0.07 pci
Volume change during consolidation ΔV_c = 0.00 in^3
Dry unit weight after consolidation γ_d = 0.07 pci
Height after consolidation h_c = 6.00 in
Volume after consolidation V_c = 39.44 in^3
Area after consolidation A_c = 6.57 in^2
Shear modulus G = 2.83 psi
Peak friction angle ϕ_p = 51.10 deg
Residual friction angle ϕ_r = 50.34 deg
Peak undrained shear strength S_u = 34.40 psi
Residual undrained shear strength S_{ur} = 33.45 psi

Axial displacement (in)	Axial force (lbf)	Pore pressure (psi)	Axial strain (%)	Shear stress (psi)	Effective normal stress (psi)	Effective Confining Stress (psi)	Total Normal Stress (psi)	τ/σ'	Principal Stress Difference (psi)	Principal Stress ratio	q	p	p'
Δh	F	u	eps	τ	σ_1'	σ_3'	σ_1	ratio	$\Delta\sigma$	ratio	(psi)	(psi)	(psi)
0.0000	0.0000	0.00	0.00	0.0	10.0	10.00	10.0	0.00	0.00	1.00	0.00	10.00	10.00
0.0126	30.0000	0.00	0.02	4.6	14.6	10.00	14.6	0.31	4.56	1.46	2.28	12.28	12.28
0.0225	55.0000	0.00	0.04	8.4	18.4	10.00	18.4	0.46	8.36	1.84	4.18	14.18	14.18
0.0339	70.0000	0.00	0.06	10.6	20.6	10.00	20.6	0.52	10.64	2.06	5.32	15.32	15.32
0.0438	85.0000	0.00	0.07	13.1	23.1	10.00	23.1	0.57	13.07	2.31	6.54	16.54	16.54
0.0537	96.0000	0.00	0.09	14.6	24.6	10.00	24.6	0.59	14.59	2.46	7.30	17.30	17.30
0.0644	106.0000	0.10	0.11	16.1	26.0	9.90	26.1	0.62	16.11	2.61	8.05	18.05	17.95
0.0743	116.0000	0.10	0.12	17.6	27.5	9.90	27.6	0.64	17.63	2.76	8.81	18.81	18.71
0.0842	131.0000	0.10	0.14	19.9	29.8	9.90	29.9	0.67	19.90	2.99	9.95	19.95	19.85
0.0949	137.0000	0.10	0.16	20.8	30.7	9.90	30.8	0.68	20.81	3.08	10.40	20.40	20.30
0.1045	147.0000	0.10	0.17	22.3	32.2	9.90	32.3	0.69	22.32	3.23	11.16	21.16	21.06
0.1137	152.0000	0.10	0.19	23.1	33.0	9.90	33.1	0.70	23.08	3.31	11.54	21.54	21.44
0.1244	157.0000	0.10	0.21	23.8	33.7	9.90	33.8	0.71	23.83	3.38	11.92	21.92	21.82
0.1343	167.0000	0.10	0.22	25.3	35.2	9.90	35.3	0.72	25.35	3.55	12.67	22.67	22.57
0.1442	172.0000	0.10	0.24	26.1	36.0	9.90	36.1	0.73	26.10	3.61	13.05	23.05	22.95
0.1652	177.0000	0.10	0.26	26.9	36.8	9.90	36.9	0.73	26.86	3.69	13.43	23.43	23.33
0.1651	182.0000	0.20	0.28	27.6	37.4	9.80	37.6	0.74	27.61	3.76	13.81	23.81	23.61
0.1755	182.0000	0.20	0.29	27.6	37.4	9.80	37.6	0.74	27.61	3.76	13.80	23.80	23.60
0.1864	192.0000	0.20	0.31	29.1	38.9	9.80	39.1	0.75	29.12	3.91	14.56	24.56	24.36
0.1969	192.0000	0.20	0.33	29.1	38.9	9.80	39.1	0.75	29.11	3.91	14.56	24.56	24.36
0.2073	192.0000	0.20	0.35	29.1	38.9	9.80	39.1	0.75	29.11	3.91	14.55	24.55	24.35
0.2175	198.0000	0.20	0.36	30.0	39.8	9.80	40.0	0.76	30.01	4.00	15.01	25.01	24.81
0.2284	203.0000	0.20	0.38	30.8	40.6	9.80	40.8	0.76	30.76	4.08	15.38	25.38	25.18
0.2378	205.0000	0.20	0.40	30.8	40.6	9.80	40.8	0.76	30.76	4.08	15.38	25.58	25.18
0.2477	208.0000	0.20	0.41	31.5	41.3	9.80	41.5	0.76	31.51	4.15	15.76	25.76	25.56
0.2581	208.0000	0.20	0.43	31.5	41.3	9.80	41.5	0.76	31.51	4.15	15.75	25.75	25.56
0.2678	213.0000	0.20	0.45	32.3	42.1	9.80	42.3	0.77	32.26	4.23	16.13	26.13	25.93
0.2772	208.0000	0.20	0.46	31.5	41.3	9.80	41.5	0.76	31.50	4.15	15.75	25.75	25.55
0.2878	213.0000	0.20	0.48	32.2	42.0	9.80	42.2	0.77	32.26	4.22	16.12	26.12	25.92
0.2975	213.0000	0.20	0.50	32.2	42.0	9.80	42.2	0.77	32.24	4.22	16.12	26.12	25.92
0.3069	213.0000	0.20	0.51	32.2	42.0	9.80	42.2	0.77	32.24	4.22	16.12	26.12	25.92
0.3176	213.0000	0.20	0.53	32.2	42.0	9.80	42.2	0.77	32.23	4.22	16.12	26.12	25.92
0.3272	213.0000	0.20	0.55	32.2	42.0	9.80	42.2	0.77	32.23	4.22	16.11	26.11	25.91
0.3371	218.0000	0.20	0.56	33.0	42.8	9.80	43.0	0.77	32.96	4.30	16.49	26.49	26.29
0.3475	218.0000	0.20	0.58	33.0	42.8	9.80	43.0	0.77	32.97	4.30	16.49	26.49	26.29
0.3574	223.0000	0.20	0.60	33.7	43.5	9.80	43.7	0.77	33.72	4.37	16.85	26.86	26.66
0.3676	218.0000	0.20	0.61	33.0	42.8	9.80	43.0	0.77	32.96	4.30	16.48	26.48	26.28
0.3783	223.0000	0.20	0.63	33.7	43.5	9.80	43.7	0.77	33.71	4.37	16.86	26.86	26.66

0.3885	223.0000	0.20	0.65	33.7	43.6	9.80	43.7	0.77	33.71	4.37	16.85	26.85	26.65
0.3984	223.0000	0.20	0.66	33.7	43.5	9.80	43.7	0.77	33.70	4.37	16.85	26.85	26.65
0.4085	223.0000	0.20	0.68	33.7	43.5	9.80	43.7	0.77	33.69	4.37	16.85	26.85	26.65
0.4195	223.0000	0.20	0.70	33.7	43.5	9.80	43.7	0.77	33.69	4.37	16.84	26.84	26.64
0.4291	223.0000	0.20	0.72	33.7	43.5	9.80	43.7	0.77	33.68	4.37	16.84	26.84	26.64
0.4390	223.0000	0.20	0.73	33.7	43.5	9.80	43.7	0.77	33.68	4.37	16.84	26.84	26.64
0.4495	223.0000	0.20	0.75	33.7	43.5	9.80	43.7	0.77	33.67	4.37	16.84	26.84	26.64
0.4591	223.0000	0.20	0.77	33.7	43.5	9.80	43.7	0.77	33.67	4.37	16.83	26.83	26.63
0.4688	223.0000	0.20	0.78	33.7	43.5	9.80	43.7	0.77	33.66	4.37	16.83	26.83	26.63
0.4792	223.0000	0.20	0.80	33.7	43.5	9.80	43.7	0.77	33.65	4.37	16.83	26.83	26.63
0.4888	223.0000	0.20	0.81	33.6	43.4	9.80	43.6	0.77	33.65	4.36	16.82	26.82	26.62
0.4982	228.0000	0.20	0.83	34.4	44.2	9.80	44.4	0.78	34.40	4.44	17.20	27.20	27.00
0.5084	228.0000	0.20	0.85	34.4	44.2	9.80	44.4	0.78	34.39	4.44	17.20	27.20	27.00
0.5178	223.0000	0.20	0.86	33.6	43.4	9.80	43.6	0.77	33.63	4.36	16.82	26.82	26.62
0.5272	228.0000	0.20	0.88	34.4	44.2	9.80	44.4	0.78	34.38	4.44	17.19	27.19	26.99
0.5373	223.0000	0.20	0.90	33.6	43.4	9.80	43.6	0.77	33.62	4.36	16.81	26.81	26.81
0.5470	228.0000	0.20	0.91	34.4	44.2	9.80	44.4	0.78	34.37	4.44	17.18	27.18	26.98
0.5571	228.0000	0.20	0.93	34.4	44.2	9.80	44.4	0.78	34.36	4.44	17.18	27.18	26.98
0.5681	228.0000	0.20	0.95	34.4	44.2	9.80	44.4	0.78	34.36	4.44	17.18	27.18	26.98
0.5783	228.0000	0.20	0.96	34.4	44.2	9.80	44.4	0.78	34.35	4.44	17.18	27.18	26.98
0.5887	228.0000	0.20	0.98	34.3	44.1	9.80	44.3	0.78	34.35	4.43	17.17	27.17	26.97
0.5988	228.0000	0.20	1.00	34.3	44.1	9.80	44.3	0.78	34.34	4.43	17.17	27.17	26.97
0.6098	228.0000	0.20	1.02	34.3	44.1	9.80	44.3	0.78	34.33	4.43	17.17	27.17	26.97
0.6192	223.0000	0.20	1.03	33.6	43.4	9.80	43.6	0.77	33.57	4.36	16.79	26.79	26.59
0.6291	223.0000	0.20	1.05	33.6	43.4	9.80	43.6	0.77	33.57	4.36	16.78	26.78	26.58
0.6395	223.0000	0.20	1.07	33.6	43.4	9.80	43.6	0.77	33.66	4.36	16.78	26.78	26.58
0.6489	223.0000	0.20	1.08	33.6	43.4	9.80	43.6	0.77	33.56	4.36	16.78	26.78	26.58
0.6585	228.0000	0.20	1.10	34.3	44.1	9.80	44.3	0.78	34.30	4.43	17.15	27.15	26.95
0.6690	223.0000	0.20	1.12	33.5	43.3	9.80	43.5	0.77	33.55	4.35	16.77	26.77	26.57
0.6786	223.0000	0.20	1.13	33.5	43.3	9.80	43.5	0.77	33.64	4.35	16.77	26.77	26.57
0.6880	223.0000	0.10	1.15	33.5	43.4	9.90	43.5	0.77	33.54	4.36	16.77	26.77	26.67
0.6987	223.0000	0.10	1.16	33.5	43.4	9.90	43.5	0.77	33.53	4.35	16.76	26.76	26.66
0.7083	223.0000	0.10	1.18	33.5	43.4	9.90	43.5	0.77	33.52	4.35	16.76	26.76	26.66
0.7177	223.0000	0.10	1.20	33.5	43.4	9.90	43.5	0.77	33.52	4.35	16.76	26.76	26.66
0.7279	223.0000	0.10	1.21	33.6	43.4	9.90	43.5	0.77	33.51	4.35	16.76	26.76	26.66
0.7373	223.0000	0.10	1.23	33.6	43.4	9.90	43.5	0.77	33.51	4.35	16.75	26.75	26.65
0.7469	223.0000	0.10	1.24	33.5	43.4	9.90	43.5	0.77	33.50	4.35	16.75	26.75	26.65
0.7579	223.0000	0.10	1.26	33.5	43.4	9.90	43.5	0.77	33.50	4.35	16.75	26.75	26.65
0.7676	223.0000	0.10	1.28	33.5	43.4	9.90	43.5	0.77	33.49	4.35	16.75	26.75	26.65
0.7777	223.0000	0.10	1.30	33.6	43.4	9.90	43.5	0.77	33.49	4.35	16.74	26.74	26.64
0.7878	223.0000	0.10	1.31	33.5	43.4	9.90	43.5	0.77	33.48	4.35	16.74	26.74	26.64
0.7983	223.0000	0.10	1.33	33.5	43.4	9.90	43.5	0.77	33.47	4.35	16.74	26.74	26.64
0.8084	223.0000	0.00	1.36	33.5	43.5	10.00	43.5	0.77	33.47	4.35	16.73	26.73	26.73
0.8183	218.0000	0.00	1.36	32.7	42.7	10.00	42.7	0.77	32.71	4.27	16.96	26.36	26.36
0.8290	223.0000	0.00	1.38	33.5	43.5	10.00	43.5	0.77	33.46	4.35	16.73	26.73	26.73
0.8387	223.0000	0.00	1.40	33.5	43.5	10.00	43.5	0.77	33.45	4.36	16.73	26.73	26.73
0.8382	223.0000	0.00	1.40	33.5	43.5	10.00	43.5	0.77	33.45	4.35	16.73	26.73	26.73


Principal Stress Difference vs. Strain for Not Back Press.
σ_c'=10psi
Principal Stress Difference (kPa)
40
30
20
10
0
-0.1
0.1
0.3
0.5
0.7
0.9
1.1
1.3
1.5
Strain


Principal Stress Ratio vs. Strain for Not Back Press. σ_c'=10psi
5.0
4.0
3.0
2.0
1.0
0.0
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6


Stress paths
q (kPa)
20
15
10
5
0
0
5
10
15
20
25
30
p,p' (kPa)
total
effective