GUIDELINES FOR ESTABLISHING SCOUR AND FREEBOARD FOR BRIDGES IN PIMA COUNTY

Pima County Regional Flood Control District

Pima County Department of Transportation

August, 2012

The following "*Guidelines for Establishing Scour and Freeboard for Bridges in Pima County*" are approved for use by Arizona engineering registrants and by Pima County staff preparing analyses and design documents for Pima County projects.

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1. INTRODUCTION

1.1. Purpose and Scope

The purpose of this document is to develop guidelines to standardize the procedures for establishing freeboard and scour analysis for design of bridges in Pima County. The intent of this document is also to bring bridge design in close conformity to the load and resistance factor design methodology used for bridges and as contained in current bridge design specifications prepared by the American Association of State Highway and Transportation Officials, AASHTO (2012). Preparation of this document has been jointly undertaken by an interdisciplinary team comprised of staff members from the Pima County Regional Flood Control District (PCRFCD), the Pima County Department of Transportation (PCDOT), NCS Consultants, LLC, RBF Consulting Engineers, and JE Fuller Hydrology & Geomorphology, Inc. Provisions of this document have been approved by both PCRFCD and PCDOT.

Adhering to these guidelines is recommended for the design of all new bridges, and for the evaluation of existing bridges as applicable, but it does not relieve the hydraulic design engineer from the responsibility of applying sound engineering principles during the design process. Close coordination among geotechnical, structural, and bridge hydraulics team members is strongly recommended during project development.

The initial objective of this effort was to develop policy guidelines for scour analysis only. The guidelines for bridge freeboard have been added for two reasons: first, there is no standard procedure for the amount of freeboard that needs to be provided for bridges in Pima County; and second, the amount of freeboard and computed scour depths at bridge piers and abutments are interrelated, in that lack of sufficient freeboard generally results in a pressure-flow condition during large flow events, which increases computed scour depths due to vertical contraction (and also increases hydrodynamic forces on bridge superstructures). This document therefore incorporates guidelines for both scour analysis and freeboard for bridges in Pima County.

1.2. Exceptions

The minimum requirements recommended in the following guidelines for bridge freeboard and scour analysis are not applicable to pedestrian bridges. Design of such bridges shall be based on engineering judgment using appropriate procedures as applicable, subject to practical and economic considerations. Any other exceptions to this policy require review by Pima County.

2. BRIDGE HYDRAULIC DESIGN CRITERIA

2.1. Design Flood Frequency

Total bridge scour depth shall be calculated for the design flow event and an extreme event. The design flow event for the scour depth shall be the 100-year flow event. The extreme event shall be the check flood event as defined in Appendix A. When the value of the total scour depth (Z_t) for the extreme event is smaller than that for the 100-year event, the 100-year value shall be used for the extreme event analysis.

Each project is unique, and the project team may determine it is also necessary to investigate flows smaller than the 100-year (e.g., at bankfull stage), because the latter may result in higher velocity and therefore larger scour depth than the 100-year flow, depending on the geometry of the channel cross-section. If an event other than the design or extreme event results in a larger scour depth, the largest value of the total scour depth shall be used for the applicable strength and service limit states for the bridge design as outlined in AASHTO (2012).

2.2. Bridge Design Freeboard

Freeboard is defined as the clearance between the lowest point of the bridge superstructure (bottom of girder) and the design water surface elevation immediately upstream of the bridge. Recommended freeboards for bridges are given below for major and other watercourses defined on the basis of 100-year discharges (Q_{100}) or extreme event.

2.2.1. Major Watercourses ($Q_{100} > 10,000$ cfs)

For major watercourses minimum freeboard shall be 3 feet above the 100-year water surface elevation, or one foot above the check flood water surface elevation, whichever is larger.

2.2.2. Watercourses (5,000 cfs $< Q_{100} \leq$ 10,000 cfs)

A minimum freeboard of 2 feet above the 100-year water surface elevation shall be provided.

2.2.3. Watercourses (100 cfs $\leq Q_{100} \leq$ 5,000 cfs)

A minimum freeboard of 1 foot above the 100-year water surface elevation shall be provided.

3. BRIDGE SCOUR

Estimation of scour depths at bridge crossings requires interdisciplinary knowledge and experience in hydrologic, hydraulic and geotechnical fields. Every bridge crossing is unique in regard to hydrologic, hydraulic, geomorphic and geotechnical characteristics. The following descriptions provide general guidelines for bridge scour analysis, but any specific application should utilize sound engineering principles and close coordination among hydraulic, structural and geotechnical members of the project development team. As noted throughout the rest of this document, the key publication to be used is FHWA (2012) which is the Federal Highway Administration's "Evaluating Scour at Bridges" (5th Edition, Hydraulic Engineering Circular No. 18, 2012), and is usually referred to as HEC-18. Familiarity with the detailed descriptions and observations in this publication is strongly recommended. As the knowledge and research on bridge scour and related subjects are being continually updated and improved, the project team should consult the most recent editions of relevant publications.

3.1. Scour Components

There are four scour components that need to be considered in estimating total scour at bridge crossings. These are general (or contraction) scour, local scour (at piers and abutments), long-term degradation and bend scour (if applicable). The total scour depth at a bridge crossing is expressed as:

$$Z_t = Z_{gs} + Z_{ls} + Z_{lt} + Z_{bs} \quad \text{.......(1)}$$

where:

- Z_t = Total scour depth, feet
- Z_{gs} = General or Contraction scour depth
- Z_{ls} = Local scour depth (pier or abutment)
- Z_{lt} = Degradation scour depth
- Z_{bs} = Bend scour depth

Total bridge scour depth shall be calculated for the design flow event and an extreme event.

Per Section 2.1, if an event other than the design or extreme event results in a larger scour depth, the largest value of the total scour depth shall be used for the applicable strength and service limit states for the bridge design as outlined in AASHTO (2012).

3.1.1. General or Contraction Scour

General scour depths are usually estimated from contraction scour equations included in FHWA (2012). An alternative method that is also used for estimating general scour depths is the procedure given in the City of Tucson's Manual, COT (1998). For major watercourses, application of water and sediment routing models, e.g., HEC-6, Fluvial-12, HEC-RAS (with sediment modeling capability) and other similar models is recommended for computing bed elevation changes (general scour) at the bridge location. See Chapter 6 of FHWA (2012) for details regarding the use of contraction scour equations.

3.1.1.1. Pressure Flow Effects

Pressure flow through the bridge occurs when the water surface elevation comes in contact with the bottom of the bridge structure (low chord elevation). Under this condition, it is necessary to estimate additional scour. See Section 6.10.1 and the example problems in FHWA (2012) for details on estimating the additional scour. In order to avoid this additional scour, it is recommended that adequate clearance (or freeboard) between upstream water surface elevation and bridge low chord be provided so that pressure flow does not occur.

3.1.2. Local Scour

The following subsections describe the different types of local scour and provide guidance and references for the determination of each.

3.1.2.1. Pier Scour

For details regarding the procedures for estimating local scour at piers, see Chapter 7 of FHWA (2012). Note that Section 7.1 of FHWA (2012) recommends the following maximum values for pier scour depths:

For Froude Number less than 0.8, $Y_{smax} = 2.4a$
For Froude Number more than 0.8, $Y_{smax} = 3.0a$

where:
Y_{smax} = maximum value of pier scour depth (excluding debris blockage effect), feet
a = pier diameter or width, feet

Special consideration should be given to the effects of debris blockage, presence of coarse sediments in bed layers, and pressure flow condition due to high water surface elevation, as described in the following sections.

3.1.2.1.1. Debris Blockage

To account for the effect of debris blockage on pier scour, the value of pier width or diameter (a) in Eq. 7.1 or Eq. 7.3 of FHWA (2012) is increased by 2 feet on each side of the pier (i.e., a total increase of 4 feet), as recommended by the Arizona Department of Transportation (ADOT) (and commonly used in bridge scour analysis). This guideline is in current practice in the absence of any specific research for quantifying the debris blockage effect.

Watershed and river-reach considerations are to be taken into account if there is the possibility for additional debris to accumulate on bridge piers during a flood event.

3.1.2.1.2. Scour Limitations Due to Coarse Sediments or Rock Layers

For scour limitation due to presence of coarse material or a resistant rock layer below the stream bed, see Chapter 7 of FHWA (2012). Coordination between the hydraulic and geotechnical engineers is recommended for correct application of these guidelines and modification of scour depths.

3.1.2.2. Abutment Scour

For details regarding the procedure for estimating local scour depths at bridge abutments, see Chapter 8 of FHWA (2012). If the NCHRP alternative procedure is used, the computed scour depth represents a combination of contraction scour and abutment scour. Therefore, the equation used to express total scour is expressed as:

$$Z_t = Z_{ls} + Z_{lt} + Z_{bs} \quad \text{...(2)}$$

where:

Z_t = Total scour depth, feet
Z_{ls} = Local scour depth (pier or abutment)
Z_{lt} = Degradation scour depth
Z_{bs} = Bend scour depth

3.1.3. Degradation

For procedures to estimate long-term degradation, see Chapter 5 of FHWA (2012).

3.1.4. Bend Scour

During the design process for a new bridge, every effort should be made to locate the bridge in a relatively straight reach of a river, so that the bend scour component (Z_{bs}) in Eq. (1) or (2) is generally equal to zero. However, when a bridge is located in a river bend, because of right-of-way constraint or the need to have a bridge at a particular location, bend scour depth needs to be computed and included in Eq. (1) or (2) to estimate the total scour depth at a bridge crossing. For the procedure for computing bend scour, see Chapter 6 of COT (1998).

4. REFERENCES

AASHTO (2012) ***AASHTO LRFD Bridge Design Specifications - 6th Edition***, American Association of State Highway and Transportation Officials, Washington, D.C.

COT (1998) ***Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona***, City of Tucson Department of Transportation, prepared by Simons, Li & Associates, Revised July, 1998 (first published December, 1989).

FHWA (2012) ***Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18, 5th Edition,*** Publication No. FHWA HIF-12-003, Federal Highway Administration, Washington, D. C.

APPENDIX A: DEFINITION OF TERMS

Bend Scour

Lowering of stream bed along the outer part of a channel bend caused by scouring action of transverse secondary currents across channel width. This is an additional component of general scour when a bridge is located within a meandering channel reach.

Bridge

A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, having a track or passageway for carrying traffic or other moving loads.

Check Flood for Bridge Scour

The flood resulting from storm having a flow rate in excess of the design flood for scour, but in no case a flood with a recurrence interval exceeding the typically used 500 years. The check flood for bridge scour is used in the investigation and assessment of a bridge foundation to determine whether the foundation can withstand that flow and its associated scour and remain stable. [Adapted from AASHTO (2012)]

Contraction Scour

This component of scour results from a contraction of the flow area at the bridge, which causes an increase in velocity and bed shear stress and removal of sediments from the contracted reach. The contraction can be caused by the bridge or a natural narrowing of the stream channel. [Adapted from AASHTO (2012)]

Debris

Floating or submerged materials transported by a stream or watercourse, such as trash, tree branches and vegetation removed from the stream bed and banks. During major flow events, debris often accumulates in front of bridge piers causing additional obstruction to flow and scour around piers.

Design Flow for Bridge Scour

The flood flow equal to or less than the 100-year flood that typically creates the deepest scour at bridge foundations. The highway or bridge may be inundated at the stage of the design flood for bridge scour. The worst-case scour condition may occur for the overtopping flood as a result of the potential for pressure flow. [Adapted from AASHTO (2012)]

Five Hundred Year Flow (500-Year Flow)

Estimated value of stream flow having a 0.2 percent probability of being equaled or exceeded in any given year. [Adapted from AASHTO (2012)]

General Scour

General scour is lowering of the stream bed across the channel at the bridge location, which occurs during the passage of a selected design flow event. This lowering may be uniform or non-uniform across the bed, depending on the shape of a cross-section. Scour depth is typically largest at the channel thalweg. General scour may result from contraction of the flow or other conditions such as flow around a channel bend. [Adapted from AASHTO (2012)]

Local Scour

Removal of stream bed materials from around piers, abutments and other structures causing obstruction to the flow, resulting in localized scour due to acceleration of flow and vortices induced by obstruction to the flow. [Adapted from AASHTO (2012)]

Long-Term Degradation

A general and progressive lowering of the longitudinal profile of the channel bed due to erosion over a long period of time. Long-term degradation is the result of modifications or changes to the stream or watershed, occurring over a long period, which may be due to natural processes or human activities. [Adapted from AASHTO (2012)]

One Hundred Year Flow (100-Year Flow)

Estimated value of stream flow having a one percent probability of being equaled or exceeded in any given year. [Adapted from AASHTO (2012)]

APPENDIX B: GEOTECHNICAL CONSIDERATIONS FOR SCOUR ANALYSIS

Suggested Input from Geotechnical Engineer for Scour Analysis

Background

In the past, the hydrologist provided the scour depth to the structural and geotechnical engineers, who used this information to determine the depth of borings required for the design of bridge pier and/or abutment foundations. Geotechnical recommendations for the structural design of the foundations were also influenced by the estimated scour depth. Soil data required for the analytical assessment of scour depth were often estimated by the hydrologist based on experience, visual observations of the surface soils in the stream bed, and/or the results of a limited number of soil tests performed on bag-samples taken from the surface or near-surface soils, e.g., gradation and Atterberg Limits. For the scour analysis the hydrologist generally assumed that the soil was uniform to virtually infinite depth. Input from the geotechnical engineer was seldom, if ever, sought. As a result of the uncertainty inherent in this approach, the hydrologist often increased the calculated scour depth by a factor that varied depending upon local experience before providing the value to the geotechnical engineer. This arbitrary increase in the computed scour depth often had a significant impact on the scope of the geotechnical field investigation and ultimately on the size and depth of deep foundations, e.g., drilled shafts, for piers and abutments. In many cases the impact on construction costs was also significant. This approach to the estimation of scour depth is inconsistent with current bridge design practices based on AASHTO (2012) Load and Resistance Factor Design (LRFD) methodology. LRFD considers the design process from the viewpoint of uncertainty and requires close interaction among the structural, geotechnical and hydrological engineers to reduce uncertainty. By taking steps to decrease uncertainty the design team is rewarded by reduced load factors and increased resistance factors, which generally have a favorable impact on the economics of the design. As suggested above, one of the first factors in the geotechnical design of bridge foundation elements is the depth of scour. Therefore, in the spirit of LRFD methodology it is very important that the geotechnical engineer and hydrologist work closely to develop a reliable estimate of the depth of scour. To this end, the geotechnical engineer should provide as much information to the hydrologist as possible regarding the soil profile in the watercourse being considered and the nature of the soils in that profile.

Guidelines for input from the geotechnical engineer for scour analysis

Setting guidelines for geotechnical input to hydrologic scour analysis requires a procedure which avoids a circular reference. The geotechnical engineer needs to have an estimate of the scour depth from the hydrologist in order to establish the depth of borings from which soil profiles can be determined. But the depth of scour is influenced by the type and density of the soils in the profile as determined from the borings. The following approach may provide a solution to this dilemma:

1. The hydrologist provides the geotechnical engineer with a preliminary conservative estimate of the depth of scour based on the currently used factors such as experience, visual observations of the surface soils in the stream bed, and/or the results of a limited number of soil tests performed on bag-samples taken from the surface or near-surface soils.

2. With input from the hydrologist, the geotechnical engineer will work with the structural engineer to establish the total depth of the boring and the location of borings based on the bridge layout.

3. In consultation with the hydrologist, the geotechnical engineer identifies the preferred location of the first boring so that geotechnical information can be obtained from the area

where maximum scour is expected. This location should be as close as possible to the location which corresponds to thalweg elevation. The primary purpose of this boring is to identify the geologic profile at the site and to obtain samples for laboratory testing to determine site-specific values of the soil properties that may be required for improved scour analyses and permit modification of sampling in other river borings.

4. During advancement of the boring, standard penetration tests (SPT) will be performed at 5-foot intervals throughout the boring. A soil sample will be obtained at each depth where the SPT is performed. In addition, a representative composite sample of the cuttings will be taken within each stratum encountered within the target scour depth. The representative composite sample(s) and samples from selected depths where the SPT was performed will be sent to a commercial material testing laboratory or the Pima County Materials Testing Laboratory (PCMTL) for grain size and Atterberg Limits testing. It is highly recommended that the hydrologist accompany the geotechnical engineer to the site for the first boring to observe the drilling procedures and to confirm the visual identifications of the sampled soils performed by the geotechnical engineer.

5. After the first boring is completed, the geotechnical engineer will provide the hydrologist with the following information, which the two will discuss and interpret at a review meeting:
    a. A field boring log that shows the various types of soils encountered (verbal description and Unified Soil Classification System designation) and the depths (elevations) at which each type of soil was first encountered. A typical boring log (Boring D01) is shown in Exhibit B-1. This data will allow the hydrologist to verify the soil type used in the preliminary scour analysis and to assess the validity of the assumption of a uniform soil profile. The same boring log will show the Standard Penetration Test blow counts (SPT-N) at each depth sampled. The blow counts are a measure of the in situ density of the soil and will provide the hydrologist with information to assess the scour potential of the soil at any given depth regardless of the type of soil it may be.
    b. The hydrologist's and geotechnical engineer's evaluation of the geotechnical information from the field boring log will determine whether or not a more- or less-erodible layer(s) exists within the originally estimated scour depth determined on the basis of a soil having assumed properties that is uniform with depth. This information will lead to one of two conclusions:
        - If there is enough evidence from the field boring log and the results of the laboratory tests to suggest that the soil conditions assumed by the hydrologist in the preliminary scour analysis are correct, the geotechnical engineer may consider the preliminary estimate of scour depth to be correct and continue with the rest of the field investigation as planned. Final verification will be obtained at the end of the field and laboratory investigation as discussed below under Step 6.
        - If the field boring log and the results of the laboratory tests suggest that the in situ conditions at the site are significantly different from those assumed by the hydrologist in the preliminary scour analysis, the hydrologist will have to re-analyze the scour depth by using the site-specific soil information described above. **It is essential that the hydrologist perform these additional analyses immediately because any change in the original estimate of scour depth may directly influence the rest of the geotechnical field and laboratory investigations and could have a considerable impact on the project**

costs. The hydrologist should be aware that the drilling equipment has already been mobilized and needs to be used continuously to drill additional borings, the depth of some of which may be influenced by a revised estimate of the sour depth.

c. If the re-analyzed scour depth is greater than the hydrologist's original estimate, all subsequent borings for bridge foundation elements in scour-susceptible areas will have to be advanced deeper than originally planned.

d. If the re-analyzed scour depth is less than the hydrologist's original estimate, the depths of the remaining borings for bridge foundation elements in scour-susceptible areas may be less than originally planned.

6. After the conclusion of the field investigation the geotechnical engineer will perform appropriate laboratory investigations. The geotechnical engineer will generally be responsible for reducing the lab test data and presenting the hydrologist with gradation curves for the samples tested. A typical set of gradation curves is found in Exhibit B-2 for the composite and SPT samples retrieved from Boring D01. The tables at the bottom of the plots present the geotechnical data input that may be required for many scour prediction models. The geotechnical engineer will then revise the field boring logs based on the lab test data and provide the hydrologist with a soil profile (fence diagram) developed from the final boring logs. A typical fence diagram is shown in Exhibit B-3. The profile will show the depths (elevations) and thicknesses of the various soil layers encountered in the boring. Uncorrected SPT blow counts and blow counts corrected for energy and depth, i.e., the so-called $N1_{60}$ values, will also be reported directly on the profile. The hydrologist and the geotechnical engineer will meet to discuss and interpret the soil profile and decide whether or not the conclusion made in Step 5b above is valid.

   Additional soils information may be requested by the hydrologist if project-specific constraints indicate that data not provided in the soil profile are required.

EXHIBIT B-1: TYPICAL BORING LOG

BORING LOG: D01

SHEET 1 of 2

PROJECT: La Cholla - Magee to Tangerine
CLIENT:
CLIENT PROJECT #:

CONTRACTOR:
DRILLER:
INSPECTOR:

COORDINATES N: 99,002 **E:** 78,826
LOCATION: Tucson, AZ
REF. ALIGNMENT:
STATION:
OFFSET:

RIG TYPE: Truck mt. CME 75
DRILLING METHOD: 8" ODHSA
HAMMER TYPE: Auto Hammer

COMMENTS:

SURFACE ELEV.: 2352.3
TOTAL DEPTH: 31.5

START DATE: **TIME:**
FINISH DATE: **TIME:**

	Casing	Split Spoon	Ring Sampler	Split Spoon	Cuttings	Core Barrel
Type/Symbol		S	R	S2	CU	C
I.D.		1.375"	2.5"	0.87"	--	
O.D.		2"	3"	1.5"	--	
Length		18"	18"	18"	–	
Hammer WT.	140 lbs.		Drill Rod Size			
Hammer Fall	30 in.		I.D. (O.D.)			

GROUNDWATER DATA

Date	Time	Water Depth (ft)	Casing Depth (ft)	Hole Depth (ft)	Symbol

DEPTH BELOW SURFACE (FT)	ELEVATION (FT)	GRAPHIC	SOIL SAMPLE TYPE	NUMBER	SYMBOL	DEPTH (FT) FROM	DEPTH (FT) TO	BLOWS 0-6 INCH	6-12-INCH	12-18 INCH	N-VALUE	RECOVERY (IN)	VISUAL SOIL IDENTIFICATION / DESCRIPTION AND REMARKS	MOISTURE, %	DRY DENSITY (PCF)	SAMPLES SENT TO LAB
	2350												WELL-GRADED SAND WITH GRAVEL, medium dense, dry, brown, fine to coarse SANDS, little gravel, trace of fines, weak cementation, no reaction to HCL (SW)			
5	2345		S	1		5	6.5	1	7	7	14	18		0.9		X
10	2340		S	2		10	11.5	3	4	8	12	18	Becomes no gravel	3.9		X
15	2335		R	3		15	16	10	17			12	WELL-GRADED SAND WITH SILT, medium dense, dry, brown, fine to coarse SANDS, few gravels, few low plastic fines, weak cementation, no reaction to HCL (SW-SM)	2.1 2.7		X X
20	2330		S	4		20	21.5	5	8	11	19	18	Well-graded Sand, medium dense, dry, brown, fine to coarse SANDS, trace of gravels, trace of fines, weak cementation, no reaction to HCL (SW)	3.4		X
25																

BORING LOG: D01

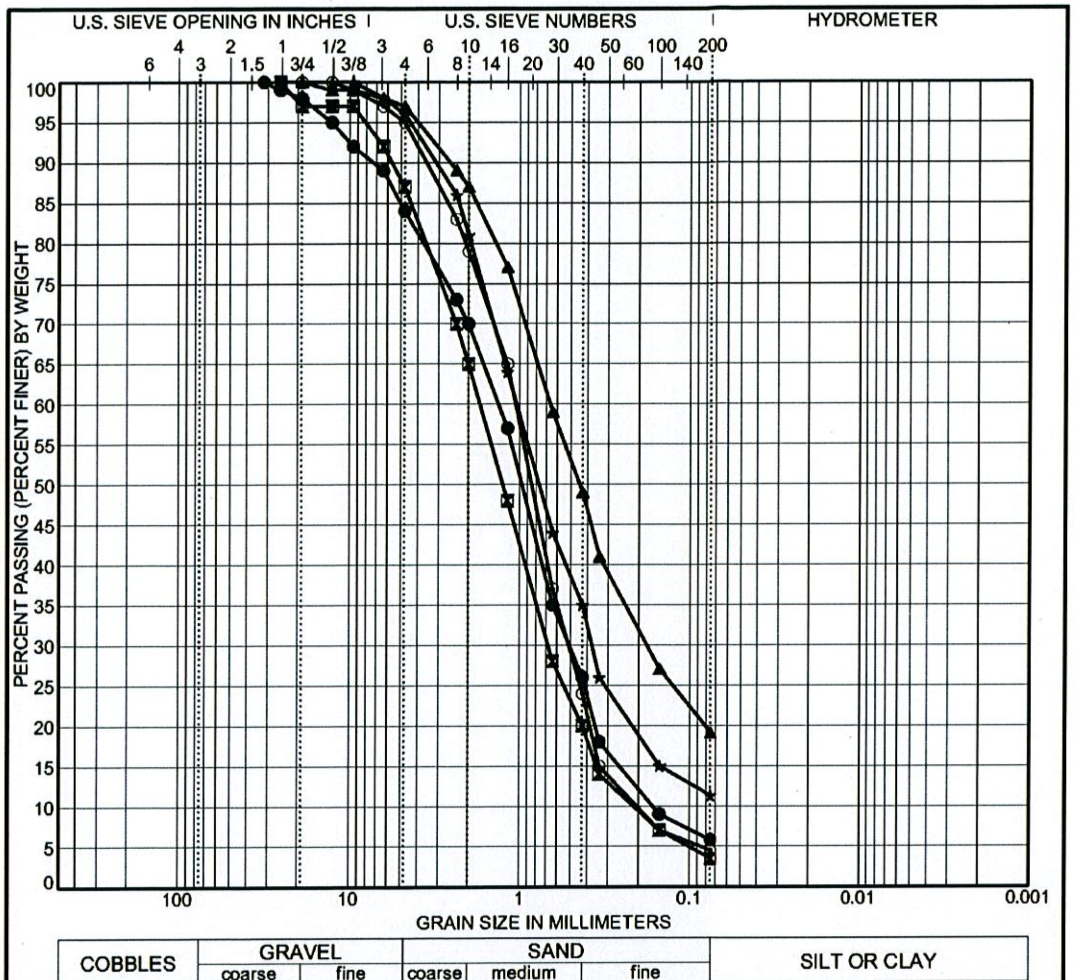
SHEET 2 of 2

PROJECT: La Cholla - Magee to Tangerine
CLIENT:
CLIENT PROJECT #:

CONTRACTOR:
DRILLER:
INSPECTOR:

DEPTH BELOW SURFACE (FT)	ELEVATION (FT)	GRAPHIC	SOIL SAMPLE					BLOWS			N-VALUE	RECOVERY (IN)	VISUAL SOIL IDENTIFICATION / DESCRIPTION AND REMARKS	MOISTURE, %	DRY DENSITY (PCF)	SAMPLES SENT TO LAB
			TYPE	NUMBER	SYMBOL	DEPTH (FT) FROM	DEPTH (FT) TO	0-6 INCH	6-12-INCH	12-18 INCH						
			S	5	X	25	26.5	5	5	8	13	18		3.1		X
	2325															
30			S	6	X	30	31.5	10	6	5	11	18	WELL-GRADED SAND WITH SILT AND GRAVEL, medium dense, dry, brown, fine to coarse SANDS, little gravel, few low plastic fines, weak cementation, no reaction to HCL (SW-SM)	4.2		X
	2320												EOB @ 30 ft, Stopped Sampler @ 31.5 ft, no water encountered, backfilled with cuttings			
35																
	2315															
40																
	2310															
45																
	2305															
50																
	2300															
55																
	2295															
60																
	2290															
65																

EXHIBIT B-2: TYPICAL GRADATION CURVES AND GEOTECHNICAL INPUT FOR SCOUR ANALYSES



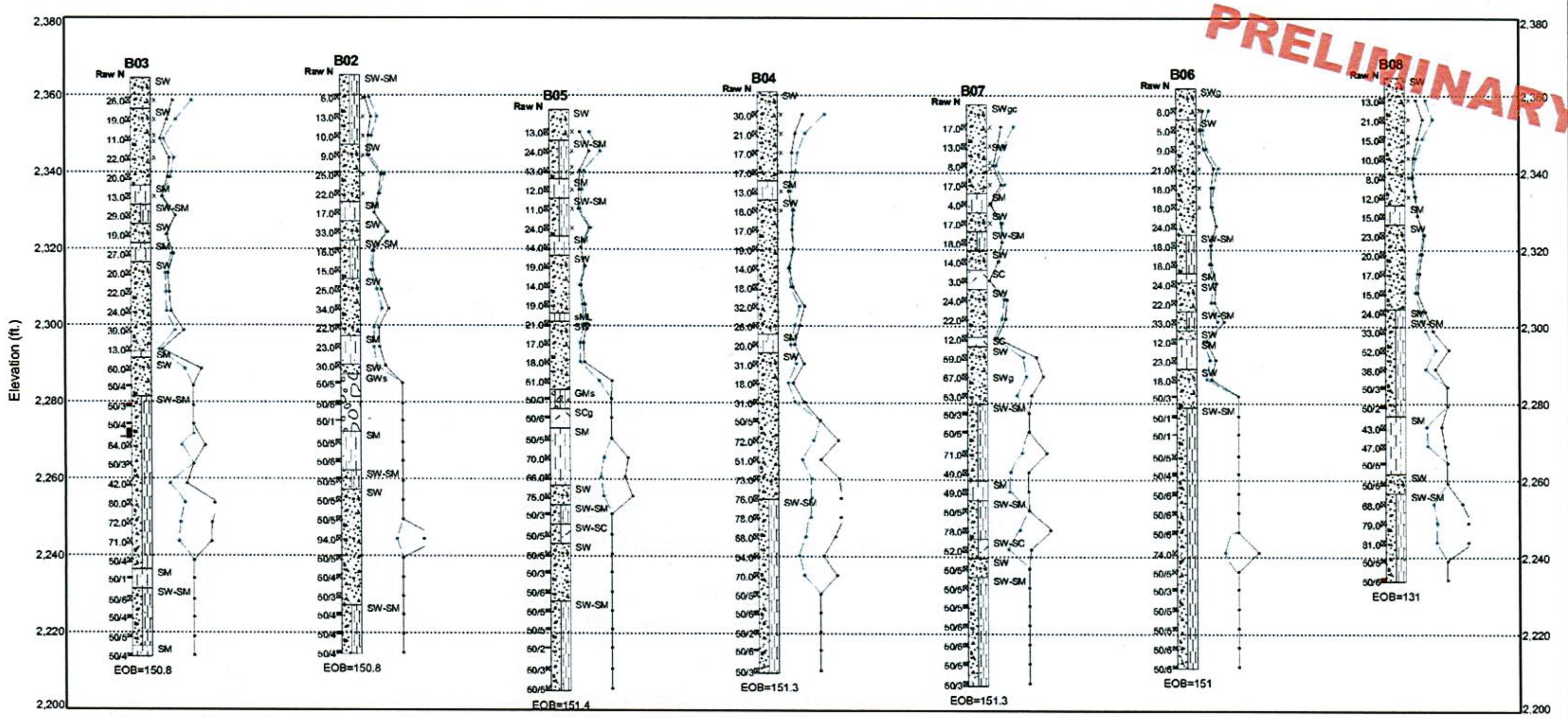
Sym.	Boring	Sample	Depth (ft.)	D100 (in)	D85 (mm)	D60 (mm)	D50 (mm)	D30 (mm)	D15 (mm)	D10 (mm)	Cc	Cu	LL	PL	PI
●	D01	Composite	0.0 - 30.0	1.25	5.077	1.385	1.034	0.528	0.289	0.164	1.227	8.445	NP	NP	NP
⊠	D01	S1	5.0 - 6.5	1.00	4.494	1.784	1.290	0.707	0.355	0.212	1.322	8.415	NP	NP	NP
▲	D01	S2	10.0 - 11.5	1.00	1.844	0.672	0.448	0.197					NP	NP	NP
★	D01	R3	15.0 - 16.0	1.00	2.290	1.087	0.823	0.380	0.150				NP	NP	NP
⊙	D01	S4	20.0 - 21.5	1.00	2.781	1.103	0.928	0.535	0.338	0.203	1.278	5.433	NP	NP	NP

Sym.	Boring	Sample	Depth (ft.)	% Cobbles & Boulders	% Gravel	% Gravel C	% Gravel F	% Sand	% Sand C	% Sand M	% Sand F	% Fines Silt | Clay	Fines USCS	Sample USCS[1]	USCS Description [1]
●	D01	Composite	0.0 - 30.0	0.0	16.0	2.0	14.0	78.2	14.0	44.0	20.2	5.8	ML	SW-SM	Well-Graded Sand With Silt And Gravel
⊠	D01	S1	5.0 - 6.5	0.0	13.0	3.0	10.0	83.5	22.0	45.0	16.5	3.5	ML	SW	Well-Graded Sand
▲	D01	S2	10.0 - 11.5	0.0	3.0	0.0	3.0	77.9	10.0	38.0	29.9	19.1	ML	SM	Silty Sand
★	D01	R3	15.0 - 16.0	0.0	4.0	0.0	4.0	84.7	15.0	46.0	23.7	11.3	ML	SW-SM	Well-Graded Sand With Silt
⊙	D01	S4	20.0 - 21.5	0.0	5.0	0.0	5.0	90.5	16.0	55.0	19.5	4.5	ML	SP	Poorly Graded Sand

GRAIN SIZE DISTRIBUTION

1. *Italicized* text indicates no plasticity tests were performed, and field classification of fines was required for USCS classification.

EXHIBIT B-3: TYPICAL SOIL PROFILE (FENCE DIAGRAM)



Borehole	Northing	Easting	Surf. Elev.	Depth
B02	98,880	79,131	2,365.2	150.8
B03	98,876	79,009	2,364.4	150.8
B04	99,088	79,131	2,360.9	151.3
B05	99,086	79,027	2,356.3	151.4
B06	99,310	79,128	2,362.0	151.0
B07	99,286	79,020	2,357.7	151.3
B08	99,485	79,131	2,364.7	131.0

GINT FENCE PARAMETERS

Position	Northing	Easting
Left, Front	98,771	79,070
Right, Front	99,571	79,059
Left, Back	98,771	79,070
Right, Back	99,571	79,059

Viewing Angle (For 3D Only)	
Horizontal	0.0
Vertical	0.0

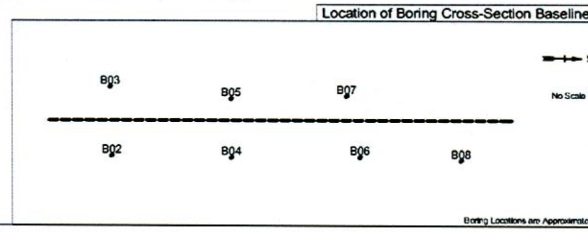
NOTES

1. The SPT values shown are measured N-Values, corrected for sampler diameter according to equation 5-1 in the Subsurface Investigation Manual (FHWA, 1997).
2. The graph of the N-values has an arbitrary maximum of 75. If an N-value is greater than 75, it is simply plotted as 75. If a value of 50/X is reported, a N of 75 is plotted as well. N-Values are plotted at a scale of 1" = 120 blows per foot.

⊻ - Indicates depth to groundwater at time of drilling

▼ - Indicates depth to groundwater measured after time of drilling (refer to boring log for details, symbol varies)

Location of Boring Cross-Section Baseline



Preliminary Profile - CDO Wash

Project: La Cholla - Magee to Tangerine
Location: Tucson, AZ
Project #: