CMG DRAINAGE ENGINEERING, INC.
3555 N. Mountain Ave. Tucson, Arizona 85719
Phone (520) 882-4244 Fax (520) 888-1421

Design Scour Depth C.O.T. EQTN 6.3

Prospect Wash West Bridge East Road Embankment

Client: PSOMAS
Station 662+00
Date 2/6/2012
Project #: 10-027
By: cmg

INPUTS	
Length to Hinge Point, (ft):	300
10-Year Natural Discharge, Q_n (cfs):	680
10-Year Urbanized Discharge, Q_u (cfs):	680
Natural Channel Bottom Width, b_n (ft):	160
Urbanized Channel Bottom Width, b_n (ft):	80
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.023
Reduction Factor for Sediment Supply, R_s:	0.05

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0016
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0170
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0170
Natural Channel Slope * L_h (ft):	6.99
Design Equilibrium Slope * L_h (ft):	5.11
Long Term Aggradation/Degradation (ft):	1.88

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ADWR - EQUILIBRIUM SLOPE ANALYSIS

project

Client: PSOMAS **Date** 2/6/2012
Project #: 10-027 **By:** cmg

Equilibrium Slope Analysis

$$S = S_{ex}\left(\frac{Q_{sup}}{Q_{cap}}\right)^{\left(\frac{2}{c-x}\right)} \qquad \text{Eq. 5.12}$$

Where :
S = equilibirum channel slope (ft/ft)
S_{ex} = existing channel slope (ft/ft)
Q_{sup} = upstream sediment supply (ft^3/sec)
Q_{cap} = sediment transport capacity of study reach (ft^3/sec)
x = 0.6 (0.33 * c + b)
c, b = Table 5.6b Regression Parameters

$$Q_{sup} = a\, Y_h^b\, V^c\, Tw \qquad \text{Eq. 5.8a}$$

Where :
Y_h = hydraulic depth for supply reach (ft)
V = velocity for supply reach (fps)
Tw = flow top width for supply reach (ft)
a, b, c = Table 5.6b Regression Parameters

$$Q_{cap} = a\, Y_h^b\, V^c\, Tw \qquad \text{Eq. 5.8a}$$

Where :
Y_h = hydraulic depth for study reach (ft)
V = velocity for study reach (fps)
Tw = flow top width for study reach (ft)
a, b, c = Table 5.6b Regression Parameters

Reference: *Design Manual for Engineering Analysis of Fluvial Systems,* Simons, Li & Associates. March 1985

LTD = (S_{DS} - S_{eq}) x L_R

LTD = Long term degradation downstream of bridge
S_{DS} = downstream channel slope
L_R = downstream reach length

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ADWR - EQUILIBRIUM SLOPE ANALYSIS

project

Client: PSOMAS
Project #: 10-027

Date 2/6/2012
By: cmg

Input Parameters

Calculate G:	D84 (mm)	D50 (mm)	D16 (mm)	G
	3.6	1	0.4	3.05

Reach coefficients a,b & c should not be zero(0)

L_R:	1000	Supply Reach			Study Reach		
S_{DS}:	0.02	G:	1		G:	1	
		D_{50}:	1.0mm		D_{50}:	1.0mm	
		a	b	c	a	b	c
S_{exist}:	0.022	0.0000058	-0.198	4.42	0.0000058	-0.198	4.42
Location	Discharge (cfs)	TW (ft)	Hyd. Depth (ft)	Velocity (fps)	TW (ft)	Hyd. Depth (ft)	Velocity (fps)
Prospect West	1600	278	1.1	5.4	406	0.7	6.31

Computational Results

Location	Q_{sup} (cfs)	Q_{cap} (cfs)	EQ. Slope (S) (ft)	LTD (ft)
Prospect West	2.732	8.685	0.012	-8.3

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Design Scour Depth C.O.T. EQTN 6.3

Prospect Wash East Bridge Abutments

Client: PSOMAS
Project #: 10-027

Date 2/6/2012
By: cmg

INPUTS	
General Scour	
Factor of Safety:	1.3
Discharge, Q (cfs):	1652.0
Channel Bottom Width, b (ft):	42
Average Velocity, V_m (fps):	11.82
Max Depth of Flow, Y_{max} (ft):	4.20
Hydraulic Depth of Flow, Y_h (ft):	4.20
Energy Slope, S_e (ft/ft):	0.0023
Top Width, T_w (ft):	50
Long Term Factor of Safety (not reqd):	0.0
Low-Flow Thalweg	
Thalweg Depth Required?:	no
Thalweg Depth, Z_{lft} (ft):	0.00
Bend Scour	
Bend Angle, α (deg):	15.00
Local Scour due to Pier	
Pier Width (normal to flow), b_p (ft):	0.00
Upstream Froude, F_u:	0.00
Pier Shape	Cylinder
Pier Shape Reduction Factor	0.9
Local Scour below Channel Drops	
Drop Height, h (ft):	0.00
Downstream Depth of Flow, TW (ft):	0.00
Total drop in head, H_T (ft):	0.00

Results	
General Scour	
General Scour, Z_{gs} (ft) [Eq. 6.4] :	3.03
Anti-dune Trough Depth, Z_a (ft) [6.5] :	1.91
Low Flow Thalweg Depth, Z_{lft} (ft):	0.00
Bend Scour, Z_{bs} (ft) [Eq. 6.6] :	0.00
local scour:	
Pier Scour Depth, Z_{lsp} (ft) [Eq 6.9] :	0.00
Encroachment Scour Depth, Z_{lse} (ft) [Eq 6.12] :	0.80
Vertical Drop Scour Depth, Z_{lss} (ft) [Eq. 6.14] :	0.00
Calculated Scour Depth, Z_t (ft) [Eq 6.3] :	7.47
Long Term Agg/Deg (ft) [Eq 6.26] :	1.97

Design Scour Depth (ft):	**9.44**

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Design Scour Depth C.O.T. EQTN 6.3

Prospect Wash East Bridge Abutments

Client: PSOMAS
Project #: 10-027

Date 2/6/2012
By: cmg

INPUTS	
Length to Hinge Point, (ft):	200
10-Year Natural Discharge, Q_n (cfs):	1300
10-Year Urbanized Discharge, Q_u (cfs):	1300
Natural Channel Bottom Width, b_n (ft):	180
Urbanized Channel Bottom Width, b_n (ft):	60
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.026
Reduction Factor for Sediment Supply, R_s:	0.05

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0162
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0162
Natural Channel Slope * L_h (ft):	5.20
Design Equilibrium Slope * L_h (ft):	3.23
Long Term Aggradation/Degradation (ft):	1.97

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Design Scour Depth C.O.T. EQTN 6.3

Prospect Wash East Bridge West Berm

Client: PSOMAS
Station 673+00
Date 2/6/2012
Project #: 10-027
By: cmg

INPUTS	
General Scour	
Factor of Safety:	1.3
Discharge, Q (cfs):	2537.0
Channel Bottom Width, b (ft):	308
Average Velocity, V_m (fps):	7.93
Max Depth of Flow, Y_{max} (ft):	1.76
Hydraulic Depth of Flow, Y_h (ft):	1.76
Energy Slope, S_e (ft/ft):	0.0200
Top Width, T_w (ft):	308
Long Term Factor of Safety (not reqd):	0.0
Low-Flow Thalweg	
Thalweg Depth Required?:	Yes
Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour	
Bend Angle, α (deg):	15.00
Local Scour due to Pier	
Pier Width (normal to flow), b_p (ft):	0.00
Upstream Froude, F_u:	0.00
Pier Shape	Cylinder
Pier Shape Reduction Factor	0.9
Local Scour below Channel Drops	
Drop Height, h (ft):	
Downstream Depth of Flow, TW (ft):	
Total drop in head, H_T (ft):	

Results	
General Scour	
General Scour, Z_{gs} (ft) [Eq. 6.4] :	0.00
Anti-dune Trough Depth, Z_a (ft) [6.5] :	0.86
Low Flow Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour, Z_{bs} (ft) [Eq. 6.6] :	0.00
local scour:	
Pier Scour Depth, Z_{lsp} (ft) [Eq 6.9] :	0.00
Contraction Scour Depth, Z_{lse} (ft):	0.80
Vertical Drop Scour Depth, Z_{lss} (ft) [Eq. 6.14] :	0.00
Calculated Scour Depth, Z_t (ft) [Eq 6.3] :	3.46
Long Term Agg/Deg (ft) [Eq 6.26] :	2.95

Design Scour Depth (ft):	**6.41**

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Design Scour Depth C.O.T. EQTN 6.3

Prospect Wash East Bridge/Berm

Client: PSOMAS
Project #: 10-027

Date 2/6/2012
By: cmg

INPUTS	
Length to Hinge Point, (ft):	300
10-Year Natural Discharge, Q_n (cfs):	800
10-Year Urbanized Discharge, Q_u (cfs):	800
Natural Channel Bottom Width, b_n (ft):	180
Urbanized Channel Bottom Width, b_n (ft):	60
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.026
Reduction Factor for Sediment Supply, R_s:	0.05

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0015
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0162
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0162
Natural Channel Slope * L_h (ft):	7.80
Design Equilibrium Slope * L_h (ft):	4.85
Long Term Aggradation/Degradation (ft):	2.95

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Live Bed Contraction Scour

Prospect Wash East Bridge

Client: PSOMAS **Date** 1/16/2012
Project #: 10-027 **By:** cmg

$$V_c = K_u y^{1/6} D^{1/3}$$

V_c = Critical velocity above which bed material of size D and smaller will be transported (ft/s)
y = Average depth of flow upstream of the bridge (ft)
D = Particle size for V_c (ft)
D_{50} = Particle size in mixture of which 50 percent are smaller (ft)
K_u = 11.17

ω = fall velocity of bed material based on D (figure 5.8)

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1}$$

y_1 = Average depth in the upstream main channel (ft)
y_2 = Average depth in the contracted section (ft)
Q_1 = Flow in the upstream channel transporting sediment (ft^3/s)
Q_1 = Flow in the contracted channel (ft^3/s)
W_1 = Bottom width of upstream channel transporting bed material (ft)
W_2 = Bottom width of main channel in contracted section less piers (ft)
k_1 = velocity dependent exponent

$y_s = y_2 - y_0$ = Average contraction scour depth (ft)

y_0 = Existing depth in the contracted section before scour - approx y_1 (ft)
$V_* = (g\, y_1\, S_1)^{1/2}$ = shear velocity in the upstream section (ft/s)
g = 32.2 ft/s^2
S_1 = Slope of energy grade line of main channel (ft/ft)

			Average Depth (ft)	Energy Slope (ft/ft)	Q (cfs)	Width (ft)
X-sec upstream of contraction:	50		**2.58**	**0.00199**	**2,727**	**262**
X-sec of contraction:	30		**1.58**	**0.06428**	**1,652**	**60**
Bounding X-sections of main channel upstream of contraction:	Upstream	Downstream	**y_1**			
	60.00	**20**	**1.20**		**3,552**	**318**

y_0 (ft)	D (ft)	V_* (ft/s)	ω (ft/s)	V_c (ft/s)	k_1	y_2 (ft)	y_s (ft)
1.20	0.00262	0.407	0.360333	1.80	0.64	2.00	0.80

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Design Scour Depth C.O.T. EQTN 6.3

Prospect Wash East Bridge East Road embankment

Client: PSOMAS
Project #: 10-027

Date 2/6/2012
By: cmg

INPUTS	
General Scour	
Factor of Safety:	1.3
Discharge, Q (cfs):	469.0
Channel Bottom Width, b (ft):	20
Average Velocity, V_m (fps):	2.63
Max Depth of Flow, Y_{max} (ft):	4.25
Hydraulic Depth of Flow, Y_h (ft):	4.25
Energy Slope, S_e (ft/ft):	0.0010
Top Width, T_w (ft):	20
Long Term Factor of Safety (not reqd):	0.0
Low-Flow Thalweg	
Thalweg Depth Required?:	Yes
Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour	
Bend Angle, α (deg):	45.00
Local Scour due to Pier	
Pier Width (normal to flow), b_p (ft):	0.00
Upstream Froude, F_u:	0.00
Pier Shape	Cylinder
Pier Shape Reduction Factor	0.9
Local Scour due to Embankments	
Slope Angle of Abutment Face, θ_a (deg):	33.00
Upstream Froude, F_u:	0.97
Upstream Flow Depth, Y_u (ft):	0.79
Encroachment Length, a_e (ft):	0.00
Local Scour below Channel Drops	
Drop Height, h (ft):	
Downstream Depth of Flow, TW (ft):	
Total drop in head, H_T (ft):	

Results	
General Scour	
General Scour, Z_{gs} (ft) [Eq. 6.4] :	0.00
Anti-dune Trough Depth, Z_a (ft) [6.5] :	0.09
Low Flow Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour, Z_{bs} (ft) [Eq. 6.6] :	1.48
local scour:	
Pier Scour Depth, Z_{lsp} (ft) [Eq 6.9] :	0.00
Encroachment Scour Depth, Z_{lse} (ft) [Eq 6.12] :	0.00
Vertical Drop Scour Depth, Z_{lss} (ft) [Eq. 6.14] :	0.00
Calculated Scour Depth, Z_t (ft) [Eq 6.3] :	3.35
Long Term Agg/Deg (ft) [Eq 6.26] :	2.84

Design Scour Depth (ft):	**6.19**

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Design Scour Depth C.O.T. EQTN 6.3

Prospect Wash East Road Embankment

Client: PSOMAS

Station 673+00

Date 2/6/2012

Project #: 10-027

By: cmg

INPUTS	
Length to Hinge Point, (ft):	300
10-Year Natural Discharge, Q_n (cfs):	510
10-Year Urbanized Discharge, Q_u (cfs):	510
Natural Channel Bottom Width, b_n (ft):	269
Urbanized Channel Bottom Width, b_n (ft):	80
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.023
Reduction Factor for Sediment Supply, R_s:	0.05

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0017
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0138
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0138
Natural Channel Slope * L_h (ft):	6.99
Design Equilibrium Slope * L_h (ft):	4.15
Long Term Aggradation/Degradation (ft):	2.84

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ADWR - EQUILIBRIUM SLOPE ANALYSIS

project

Client: PSOMAS **Date** 2/6/2012
Project #: 10-027 **By:** cmg

Equilibrium Slope Analysis

$$S = S_{ex}\left(\frac{Q_{sup}}{Q_{cap}}\right)^{\left(\frac{2}{c-x}\right)} \qquad \text{Eq. 5.12}$$

Where :
S = equilibirum channel slope (ft/ft)
S_{ex} = existing channel slope (ft/ft)
Q_{sup} = upstream sediment supply (ft^3/sec)
Q_{cap} = sediment transport capacity of study reach (ft^3/sec)
x = $0.6\,(0.33 * c + b)$
c, b = Table 5.6b Regression Parameters

$$Q_{sup} = a\, Y_h^b\, V^c\, Tw \qquad \text{Eq. 5.8a}$$

Where :
Y_h = hydraulic depth for supply reach (ft)
V = velocity for supply reach (fps)
Tw = flow top width for supply reach (ft)
a, b, c = Table 5.6b Regression Parameters

$$Q_{cap} = a\, Y_h^b\, V^c\, Tw \qquad \text{Eq. 5.8a}$$

Where :
Y_h = hydraulic depth for study reach (ft)
V = velocity for study reach (fps)
Tw = flow top width for study reach (ft)
a, b, c = Table 5.6b Regression Parameters

Reference: *Design Manual for Engineering Analysis of Fluvial Systems,* Simons, Li & Associates. March 1985

$LTD = (S_{DS} - S_{eq}) \times L_R$

LTD = Long term degradation downstream of bridge
S_{DS} = downstream channel slope
L_R = downstream reach length

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ADWR - EQUILIBRIUM SLOPE ANALYSIS

project

Client: PSOMAS
Project #: 10-027

Date 2/6/2012
By: cmg

Input Parameters

Calculate G:

D84 (mm)	D50 (mm)	D16 (mm)	G
3.6	1	0.4	3.05

Reach coefficients a,b & c should not be zero(0)

L_R:	1000	Supply Reach			Study Reach		
S_{DS}:	0.02	G:	1		G:	1	
		D_{50}:	1.0mm		D_{50}:	1.0mm	
		a	b	c	a	b	c
S_{exist}:	0.022	0.0000058	-0.198	4.42	0.0000058	-0.198	4.42
Location	Discharge (cfs)	TW (ft)	Hyd. Depth (ft)	Velocity (fps)	TW (ft)	Hyd. Depth (ft)	Velocity (fps)
Prospect East	510	269	0.56	5.03	112	0.69	7.74

Computational Results

Location	Q_{sup} (cfs)	Q_{cap} (cfs)	EQ. Slope (S) (ft)	LTD (ft)
Prospect East	2.208	5.926	0.013	-7.2

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Design Scour Depth C.O.T. EQTN 6.3

Bridge at Station 575+67 Cross Section 30-For Grade Control Structure

Client: PSOMAS
Project #: 10-027

Date 2012.12.053
By: wjg

INPUTS	
General Scour	
Factor of Safety:	1.3
Discharge, Q (cfs):	637.0
Channel Bottom Width, b (ft):	36
Average Velocity, V_m (fps):	7.61
Max Depth of Flow, Y_{max} (ft):	3.50
Hydraulic Depth of Flow, Y_h (ft):	1.63
Energy Slope, S_e (ft/ft):	0.0172
Top Width, T_w (ft):	110
Long Term Factor of Safety (not reqd):	1.0
Low-Flow Thalweg	
Thalweg Depth Required?:	Yes
Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour	
Bend Angle, α (deg):	0.00
Local Scour due to Pier	
Pier Width (normal to flow), b_p (ft):	
Upstream Froude, F_u:	
Pier Shape	
Pier Shape Reduction Factor	
Local Scour due to Embankments	
Slope Angle of Abutment Face, θ_a (deg):	
Upstream Froude, F_u:	
Upstream Flow Depth, Y_u (ft):	0.00
Encroachment Length, a_e (ft):	
Local Scour below Channel Drops	
Drop Height, h (ft):	5.50
Downstream Depth of Flow, TW (ft):	1.50
Total drop in head, H_T (ft):	7.50

Results	
General Scour	
General Scour, Z_{gs} (ft) [Eq. 6.4] :	0.00
Anti-dune Trough Depth, Z_a (ft) [6.5] :	0.79
Low Flow Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour, Z_{bs} (ft) [Eq. 6.6] :	0.00
local scour:	
Pier Scour Depth, Z_{lsp} (ft) [Eq 6.9] :	0.00
Encroachment Scour Depth, Z_{lse} (ft) [Eq 6.12] :	0.00
Vertical Drop Scour Depth, Z_{lss} (ft) [Eq. 6.14] :	8.30
Calculated Scour Depth, Z_t (ft) [Eq 6.3] :	2.33
Long Term Agg/Deg (ft) [Eq 6.26] :	0.00

Design Scour Depth (ft):	**3.00**

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Design Scour Depth C.O.T. EQTN 6.3

Bridge Station 661+47 Cross Section 30 - For Grade Control Structure

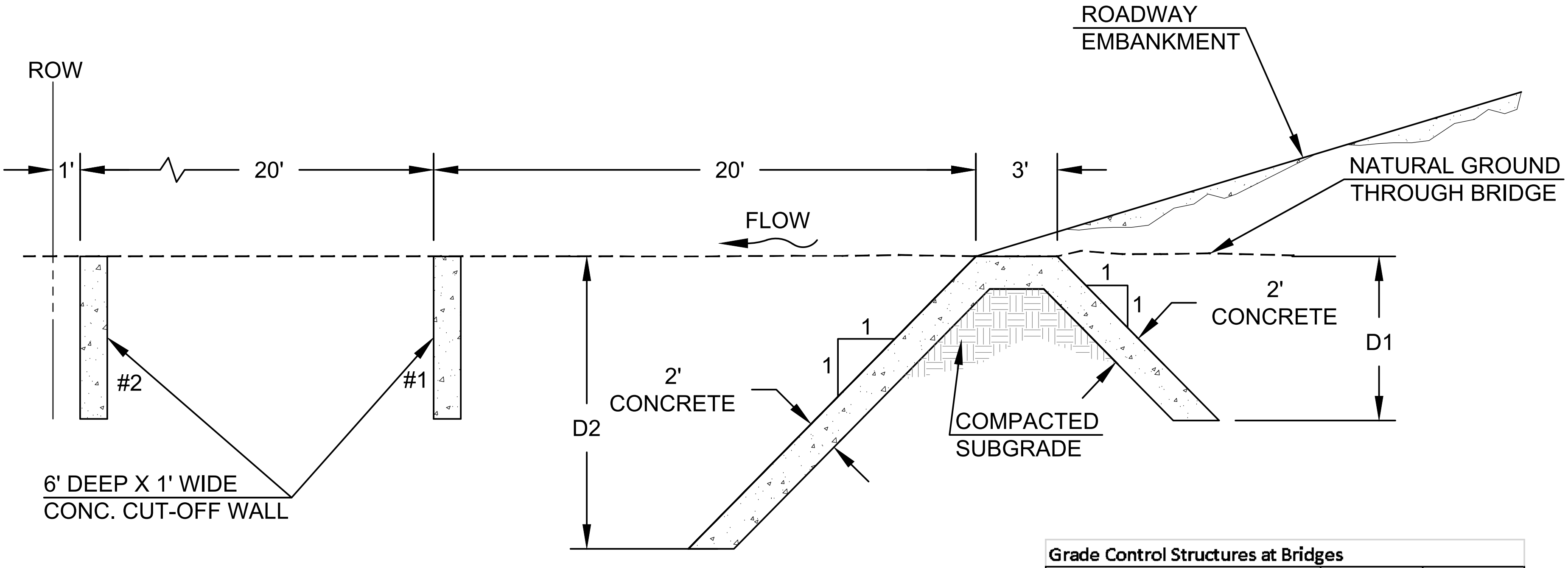
Client: PSOMAS
Project #: 10-027

Date 2012.12.053
By: wjg

INPUTS	
General Scour	
Factor of Safety:	1.3
Discharge, Q (cfs):	4059.0
Channel Bottom Width, b (ft):	76
Average Velocity, V_m (fps):	11.64
Max Depth of Flow, Y_{max} (ft):	5.99
Hydraulic Depth of Flow, Y_h (ft):	3.80
Energy Slope, S_e (ft/ft):	0.0171
Top Width, T_w (ft):	180
Long Term Factor of Safety (not reqd):	1.0
Low-Flow Thalweg	
Thalweg Depth Required?:	Yes
Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour	
Bend Angle, α (deg):	0.00
Local Scour due to Pier	
Pier Width (normal to flow), b_p (ft):	
Upstream Froude, F_u:	
Pier Shape	
Pier Shape Reduction Factor	
Local Scour due to Embankments	
Slope Angle of Abutment Face, θ_a (deg):	
Upstream Froude, F_u:	
Upstream Flow Depth, Y_u (ft):	0.00
Encroachment Length, a_e (ft):	
Local Scour below Channel Drops	
Drop Height, h (ft):	8.30
Downstream Depth of Flow, TW (ft):	3.96
Total drop in head, H_T (ft):	9.07

Results	
General Scour	
General Scour, Z_{gs} (ft) [Eq. 6.4] :	0.00
Anti-dune Trough Depth, Z_a (ft) [6.5] :	1.86
Low Flow Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour, Z_{bs} (ft) [Eq. 6.6] :	0.00
local scour:	
Pier Scour Depth, Z_{lsp} (ft) [Eq 6.9] :	0.00
Encroachment Scour Depth, Z_{lse} (ft) [Eq 6.12] :	0.00
Vertical Drop Scour Depth, Z_{lss} (ft) [Eq. 6.14] :	14.62
Calculated Scour Depth, Z_t (ft) [Eq 6.3] :	3.71
Long Term Agg/Deg (ft) [Eq 6.26] :	0.00

Design Scour Depth (ft):	**3.71**



Grade Control Structures at Bridges

Bridges	Bridge @ Sta. 661+84	Bridge @ Sta. 575+67
D1 (ft)	8	5
D2 (ft)	15	8.5
Abutement toedown depth downstream of bridge inlet (ft)	8	5
Abutement toedown depth at bridge inlet and extend 110 ft upstream (ft)	12.5	7.5
Training Berm toedown depth (ft)	8.3	7.4
Top elev. of downstream 6' deep cut-off wall #1	2461.4	2231.8
Top elev. of downstream 6' deep cut-off wall #2	2460.8	2230.8

GRADE CONTROL STRUCTURE AT BRIDGES

N.T.S

APPENDIX K

WEST END REGIONAL DRAINAGE ANALYSIS

Includes

- West End Regional Drainage Report

PRELIMIMARY (STAGE II)
WEST END REGIONAL DRAINAGE ANALYSIS
FOR
TANGERINE ROAD CORRIDOR STUDY –
INTERSTATE-10 TO LA CANADA DRIVE

Location:
Portions of Sections 25, 26, 35 & 36; Township 11 South, Range 11 East
Portions of Sections 28 – 33; Township 11 South, Range 12 East
Portions of Section 1; Township 12 South, Range 11 East
Portions of Sections 4 – 6, 8 & 9; Township 12 South, Range 12 East
Pima County, Arizona

Prepared for:
Town of Marana Department of Public Works
11555 West Civic Center Drive
Marana, Arizona 85653

Prepared by:
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December 20, 2012
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CMG Project No. 10-027

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SECTION 1.0 INTRODUCTION

The western 1.7 miles of the Tangerine Road improvement project, from I-10/Union Pacific Railroad (UPRR) to approximate road Station 526+55 (west of Concentration Point 66 near Breaker's Water Park entrance road), currently experiences widely dispersed sheet flow conditions and periodic roadway inundation. The lack of topographic relief and adequate downstream outfall channels in this area make it impractical to construct conventional culvert crossings that provide full conveyance of 100-year flood flows under the roadway. Because of this situation, an expanded "regional west end" study was added to the project scope to examine alternative drainage facilities, e.g. interceptor channel systems and stormwater detention basins, that could decrease the frequency of roadway flooding until permanent outfall channel systems to the Santa Cruz River are constructed in the future. This report presents the results and recommendations of the alternatives analysis.

The western 1.7 miles of the project (west end regional drainage study area) is located in the Town of Marana, Pima County, Arizona. Tangerine roadway alignment in this area follows along the southern boundary of Sections 31 and 32, Township 11 South, Range 12 East; and the northern boundary of Sections 5 and 6, Township 12 South, Range 12 East, G&SRB&M. A vicinity / location map for the project is presented as Figure W-1 following the text of the report. The project area has FEMA floodplain designations of Zone AO1 and AH as shown on FEMA FIRM Panel #04019C1045L (Appendix W-A).

SECTION 2.0 DRAINAGE ALTERNATIVE ANALYSES

The peak flow rates and volumes of water and sediment reaching the west end of Tangerine Road are large. As shown on Figure 1 in Appendix W-B, which is based on the 100-ft grid FLO-2D model modified for this roadway project, the cumulative discharge rates and runoff volumes approaching the road from north are 3964 cfs and 1026 cfs during the100-year and 10-year storms. The runoff volume during these storms is estimated to be 808.3 acre-feet and 231.1 acre-feet, respectively. Under existing conditions, most of the storm water drains across Tangerine Road and is significantly attenuated on farmland south of Tangerine Road. Volume exceeding the farmland storage capacity drains toward the UPRR then northwesterly along the east side of the roadway embankment eventually reaching Tangerine Road in the vicinity of Tangerine Road/UPRR crossing.

Due to the complexity of the drainage issues in this area, an alternative analysis was necessary to identify, evaluate and select a preferred alternative for reducing potential flood impacts to the proposed roadway improvements and for providing all-weather access. The drainage alternative analyses discussed in this report provide a description of identified alternatives and their pros, and cons. Some of the alternatives were not considered feasible for the purposes of the current project (primarily due to cost), but may be feasible in the future if integrated into a multi-functional regional drainage plan benefitting both public and private properties in the area. Therefore, the alternative analyses are also a valuable information source for future Tangerine Road drainage improvements and land development in the area.

In the early stages of the west end regional drainage analyses, it was determined by the project design team that a drainage system with 100-year capacity may require cost prohibitive offsite improvements. Therefore, drainage system alternatives with 10-year capacity were also included in the analysis.

The project design team identified nine preliminary drainage alternatives for the west 1.7 miles of Tangerine Road. A description of each alternative and a discussion of the evaluation process and results are provided in Appendix W-B. Evaluation criteria included cost, presence of cultural resource, property commercial values, and drainage impacts to adjoining property and flood reduction benefits derived. The Drainage Alternatives analyzed as a part of this study are summarized in Table 1. Alternative 1 involved a roadway design that would not change existing drainage patterns and conditions, Alternative 2 only considered an interceptor channel on the north side of the road, Alternative 3 proposed elevating Tangerine Road and placement of a series of 24-inch culverts beneath the roadway to convey the 100-year flood, and Alternatives 4 through 9 evaluated different detention basin locations and sizes and possible combinations with interceptor channels. Detention volume requirements for Alternative 4 through 9 were estimated to be 510 acre-feet and 153 acre-feet for 100-year and 10-year rainfall events, respectively. After completion of the evaluation process, the Tangerine Road Technical Advisory Committee (TAC) selected Alternative 2 for detailed analysis and design to the 30% level. The conclusions of the evaluation process are documented in the meeting minutes dated August 31, 2012 (Appendix W-B).

Alternative 2 proposes a channel along the north side of Tangerine Road to convey the 10-year peak flow; while runoff exceeding the 10-year event could overtop the roadway and follow existing drainage patterns. The channel would intercept overland flow arriving along this section of the road and direct it west toward the northeast corner of Union Pacific Railroad (UPRR)/Tangerine Road intersection. At the channel terminus along the UPRR, flows would weir over the channel north bank and onto adjacent properties to return to shallow overland flow along the UPRR right-of-way. Standing water in the channel below the weir will drain through a low-flow pipe to an existing swale within UPRR right-of-way. The channel is proposed to extend northward away from Tangerine Road and along the UPRR approximately 230 feet to provide additional weir length for flow dispersion. The extended channel segment onto the adjacent property is limited to 100 feet wide.

Collection and conveyance of flows along the north side of Tangerine Road will divert flows away from the farmland on the south side of the road. Hydraulic studies completed as a part of the west end study determined that the farmland on the south side of Tangerine Road functions to attenuate peak flows. The hydraulic modeling for proposed conditions determined that the peak flow rate at the northeast corner of UPRR/Tangerine Road will increase as a result of the proposed channelization. Quantitative estimates of the potential changes in flood peaks and flooding depths are discussed in the following sections of this report.

Table 1: Summary of Drainage Alternatives Analyses

Alternative ID	Description	Recommended for Further Analysis? Y/N
Alternatives without detention basins (Channelization only)		
Alternative 1	No-build/maintain existing cross drainage	No – Small grid FLO-2D showed that Tangerine Road is subject to significant flooding during 100- and 10-year rainfall event.
Alternative 2	No-basin/Channel-only	Yes – CMG to proceed with analysis and design for a 10-year capacity channel along the north side of Tangerine Road
Alternative 3	Causeway/multiple small diameter culvert system	No – Cost estimated at $16 mil+ with large adverse maintenance issues. No further analysis recommended
Alternatives with detention basins		
Alternative 4	Detention Basins east of Trico Property.	No – Known cultural resource conflicts
Alternative 5	Combined Detention Basins east of Trico Property and east of UPRR @ MSP property.	No – Known cultural resource conflicts
Alternative 6	Detention Basins on Tangerine Invest Partners LLC.	No – High commercial value of property makes unfeasible.
Alternative 7	Channelization/Basin on MSP Property.	No – High cost (estimated to be $10.2 mil) associated with this Alternative makes it unfeasible.
Alternative 8	Detention basin on Kai and other properties south of Tangerine Road	No – High cost (estimated to be $7.9 mil) associated with this Alternative makes it unfeasible.
Alternative 9	Combination of Alt. 7 and Alt. 8 with detention basins on Kai property and at downstream MSP property location connected by new channels	No – High cost (estimated to be $9.8 mil) associated with this Alternative makes it unfeasible.

SECTION 3.0 EXISTING CONDITIONS SMALL GRID FLO-2D MODELS

CMG Drainage Engineering (CMG) determined that it was necessary to obtain more accurate hydrology/hydraulics data in the vicinity of the project site to design the drainage system for the selected alternative (Alternative 2). The modified 100-ft grid FLO-2D model from the FEMA approved Town of Marana Tortolita Alluvial Fan Study was utilized to estimate 100-year discharge rates for the regional watersheds on the Tangerine Road west end. However, the 100-ft grid (which means one elevation on a 100' x 100' square grid) could not characterize the terrain accurately enough to analyze the proposed drainage facilities. Therefore, 20-foot grid models were developed to account for the roadside ditches, roadway pavement, berm, and other topographic features in the vicinity of the roadway.

The 20-foot grid FLO-2D model was built using FLO-2D, Version 2009.06. The study area was extended to 1.5 miles north of Tangerine Road to yield a more accurate estimation of the flow distribution on fan surface before it arrives at Tangerine Road. The study area was also extended 1.8 miles south of Tangerine Road to include the local high ground along east side of UPRR; the purpose being to better estimate the peak flow along the UPRR embankment in the vicinity of Tangerine Road and UPRR intersection.

The west limit of the FLO-2D model is approximately 300 feet west of Interstate 10 (I-10) to allow culverts and underpasses underneath I-10 to be included in the FLO-2D model. FLO-2D study limits are shown on Figure C-1 in Appendix W-C. The FLO-2D grid size was determined by considering the following three factors: 1) accurately depict topographic features in the vicinity of the roadway, 2) allow accurate flow distributions on the fan surface, and 3) current computer simulation capabilities. FLO-2D model parameters from the Tortolita Alluvial Fan Study were used if appropriate. The parameters for the small grid FLO-2D model are summarized in Table 2.

FLO-2D cross sections were generated to obtain peak discharge rates at certain locations. The peak discharge rates are shown on an exhibit (Figure F-1 in Appendix W-F). Maximum flow depths for the 100- and 10-year floods in the FLO-2D study limits are shown on Figure C-2 and Figure C-3. Figure C-4 and C-5 shows the areas of Tangerine Road that are subject to 100- and 10-year flooding depth of 0.1 foot or more. Maximum velocities for the 100- and 10-year floods in the FLO-2D study limits are shown on Figure C-6 and Figure C-7. These exhibits are provided in Appendix W-C. FLO-2D model input and output for existing conditions are provided in Appendix W-C and in Appendix W-I (electronic format only). Under existing conditions, Tangerine Road does not meet all-weather access criteria (flow depths on roadway less than one foot during the 100-Year storm) per Pima County Roadway Design Manual (Third Edition, 2010).

Table 2: Summary of Existing Conditions Small Grid FLO-2D Model Parameters

Small Grid FLO-2D Model Parameters	Description
FLO-2D Study Area	Approximately 8.7 square miles. See previous paragraph and Figure C-1 for details.
Grid Size	20 ft grid. The FLO-2D model has total 603,128 grids.
Topographic Data and Aerial Photos	2008 PAG LIDAR bear earth LIDAR data and 2008 1-ft aerial photos. Latest topographic data and aerial photos available at the start of the project.
Storm Frequencies Evaluated	100- and 10- year rainfall events.
Rainfall Data	Aerial reduction factor of 0.826 for Prospect Wash was used. 3-hour NOAA 14 (upper 90%) rainfall depths for 100- and 10-year are 3.10" and 1.98", respectively. Rainfall depths used in the FLO-2D model for 100- and 10-year are 2.561" (0.826 x 3.1") and 1.635" (0.826 x 1.98"), respectively.
Inflow Hydrographs	Obtained from the modified 3-hour 100-ft grid FLO-2D model for Tangerine Road project.
Infiltration	Used the combined infiltration method (both SCS Curve Number method and Green Ampt method). SCS Curve Number and Green Ampt parameters are obtained from the FLO-2D model from Tortolita Alluvial Fan Study.
Manning's Roughness Coefficient	Pavement -> 0.03; developed area ->0.05; Area between UPRR and Frontage Road -> 0.07; Overland flow area -> 0.085; FEMA Zone AH area east of UPRR -> 0.05. Channel between UPRR and MSP property ->0.035. Manning's values on some grids and channel elements are calibrated to allow stable and faster FLO-2D simulation.
Structures	Hydraulic structures (culverts under I-10, UPRR, CAP Embankment) are obtained from Tortolita Alluvial Fan Study; Added Drainage Structures: existing culverts at Trico East Driveway; channel between UPRR and MSP property was modeled as a channel component; I-10 Underpasses (at Tangerine Road and Avra Valley Road) were modeled as opening by lowering the grid elevations to be the underpass roadway elevations.

SECTION 4.0 PROPOSED REGIONAL DRAINAGE IMPROVEMENTS

4.1 Cross Culvert Near Station 515+35

In Section 2.0 it was noted that Alternative 2 is expected to increase the peak discharge rate and runoff volume at the northeast corner of UPRR/Tangerine Road due to channelization that diverts flow away from the detention storage currently provided by the farmland on the south side of Tangerine Road. To reduce potential drainage impacts in the vicinity of the UPRR/Tangerine Road crossing, the design team recommended the addition of a cross-culvert at roadway Station 515+35. Detailed analysis based on topographic data, aerial photos, and field inspection of drainage patterns has determined that Station 515+35 provides a viable cross culvert location since there is a currently a dip crossing at this location to convey flow across the road. The 100-year peak flow at this location is 785 cfs, which was determined from the 20-ft grid FLO-2D model output. The culvert size that is needed to convey 785 cfs under the road is 4-10'x4' RCBC. A stabilized training berm with a length of about 350 feet from the west abutment of the culvert is also needed to intercept and direct flow to the inlet. The top elevation of the berm either 1' above the culvert headwater elevation or 2' above the existing ground elevation, whichever is higher. Computations for the culvert hydraulics and culvert outlet basins are provided in the main hydraulic design report for Tangerine Road dated December 5th, 2012.

4.2 North Side Interceptor Channel/Culverts/Dip Crossing, Grade Controls and Outflow Weirs

The general design criteria and goals developed for West End regional drainage designs included:

1. 10-Year storm event capacity (without freeboard) for the proposed channel on the north side of Tangerine Road,
2. contain increases in flooding depths within the road right-of-way (ROW) on the north side of Tangerine Road, wherever possible, and
3. provide a roadway profile that generally matches the existing roadway profile to allow flows exceeding the 10-Year storm peak to cross the road, as under existing conditions.

The east (upstream) end of the proposed interceptor channel starts at roadway Station 504+00 and terminates at the northeast corner of UPRR/Tangerine Road (a total length of about 6,450 feet). The channel side slope on the north and south side are typically 3:1 (H:V) and 4:1 (H:V), respectively. The channel consists of concrete-lined side slopes and earthen bottom between Channel Station 211+00 and Station 258+16. The channel bottom width is 60 feet and a 5-foot wide shotcrete splash pad is provided at channel invert elevation along the toe of the north bank to prevent scour where flow spills over the north bank.

The channel consists of concrete-lined side slopes and bottom from Channel Station 210+00 to 211+00 and from Channel Station 258+16 to Station 274+53. The channel bottom width varies from 5 feet to 60 feet in the fully-lined segments based on expected variations in channel inflow.

Longitudinal slopes of the channel generally follow the roadway longitudinal slopes. The longitudinal slopes of the channel are approximately 1.3% at the east end (Roadway Station 504+00) and are gradually decreased to approximately 0.2% at the west terminus. Channel peak flow rates vary at different channel locations because the channel continuously exchanges flow with surrounding overland flow. The estimated peak flow rates conveyed within the channel prism during the 100- and 10-year storm events are summarized in Table 3.

Table 3: Summary of 100- and 10-Year Channel Peak Flow Rates

Channel Stations	Channel Peak Flow Rates (cfs)	
	100-Year	10-Year
210+00 through 213+00	470 ~ 1200	250 ~ 550
213+00 through 232+00	1200 ~ 2200	550 ~ 850
232+00 through 256+00	1700 ~ 1900	850 ~ 900
256+00 through 265+00	1100 ~ 1900	370 ~ 900
265+00 through 274+53	430 ~ 1100	30 ~ 370

The channel invert is approximately 2 to 3 feet below adjacent ground elevation at its west terminus so the channel will not have a full capacity gravity outlet until a downstream channel is extended along UPRR. Flows that are greater than the channel depth will weir out onto the adjacent overbank via the north channel bank and return to shallow overland flow conditions along the UPRR ROW. The following channel design features are proposed to facilitate re-establishing overland flow conditions downstream of the channel terminus: 1) The channel bottom widths gradually reduce from 60 feet to 10 feet. This will force more flow onto the north overbank by gradually reducing channel capacity, 2) The north channel bank will match existing ground elevations and a 5-foot wide riprap apron (D50=6”) underlain by filter fabric will be provided adjacent to the top of the north channel bank to prevent downstream erosion, 3) The channel turns northerly adjoining the UPRR for approximately 230 feet to provide additional weir length for flow dispersion. This extended channel segment also shortens the distances for the channel low-flow structure tie-in to the existing swale within the UPRR ROW. 4) A training berm (with a top of berm elevation 2043.0) is provided along the 230-foot section north of Tangerine Road to turn flows back to the historically northwesterly flow direction and to prevent flows from directly impinging on the UPRR ROW.

A 4-24-inch RCP cross-culvert is proposed beneath Tangerine Road near the UPRR crossing at roadway Station 443+10 to convey storm water emanating from areas south of the road. These culverts will outlet to the proposed interceptor channel on the north side of the road. The 4-24-inch RCP replaces the existing 2-24-inch RCP at this location, that currently drains into the existing UPRR ROW swale. The drainage capacity for the existing 2-24-inch RCP is approximately 55 cfs. At the proposed interceptor channel terminus, two low flow outlet alternatives are proposed. One alternative proposes gravity drainage structures and the other propose infiltration only structures, such as dry wells. The gravity drainage alternative was shown on the 30% plan. This alternative proposes a new 2-24-inch RCP, with a drainage capacity of approximately 52 cfs, to be installed to drain the standing storm water downstream into the existing UPRR swale. The peak flow rate from this new low-flow culvert was kept below the existing flow rate that had historically reached the UPRR swale during lesser flood events. The standing storm water volume in the interceptor channel was estimated to be 1.7 ac-ft, and is estimated to take approximately 1.5 hours to drain through the proposed 2-24-inch low-flow culverts. Computation sheets are provided in Appendix W-E. Typical channel cross sections, channel plans and profiles for the interceptor channel on the north side of the road are provided in Appendix W-D. The other low flow outlet alternative proposes to install dry wells at the channel west terminus to drain the standing storm water via infiltration. This alternative would serve to minimize the drainage easement acquisition requirements from the UPRR. However, it was not evaluated in detail and was not shown on the 30% plans.

At the east Trico Electric driveway, the driveway profile needed to be raised to accommodate a proposed 5-10’x4’ RCBC in the interceptor channel beneath the driveway. Raising the east Trico Electric driveway required the adjoining section of Tangerine Road to be raised to comply with roadway design standards. To accomplish this, the existing Tangerine Road profile was raised by up to 2.5 feet in the vicinity of east Trico Electric driveway. A 6-ft deep concrete cutoff wall will be installed at Station 258+16 at the downstream outlet of the driveway culvert, and a 65-ft long riprap apron (D50=18”) underlain by filter fabric is proposed downstream of the concrete cutoff wall to control scour.

At the west Trico Driveway, a dip section is proposed at the channel crossing. A 6-ft deep concrete cutoff wall will be installed at Station 253+39 at the downstream edge of the driveway and a 4-ft deep concrete cutoff wall will be installed at Station 253+71 at the upstream edge of the driveway to protect the pavement surface. A 65-ft long riprap apron (D50=18”) underlain by filter fabric is provided downstream of the driveway to control scour.

The CAP siphon horizontal alignment intersects the proposed channel alignment at Station 219+38. Based on survey pothole information provided by the Town of Marana, the proposed channel bottom is approximately 7 feet above the top of the CAP siphon conduit. To protect and provide adequate vertical clearance over the CAP siphon, a 6-inch thick concrete channel bottom is proposed over and adjacent to the siphon in lieu of the earthen channel bottom with greater than 4-foot deep concrete toe-down walls along the bottoms of the channel side slopes. A 6-ft deep concrete cutoff wall will be installed at Station 219+04 at the downstream edge of the concrete channel bottom, and a 4.2-ft deep concrete cutoff wall will be installed at Station 219+70 at the upstream edge of the concrete channel bottom. A 65-ft long riprap apron (D50=18") underlain by filter fabric is proposed downstream of the concrete channel bottom section to control scour.

Additional grade control structures (concrete cutoff walls) are provided at channel stations 211+00, 228+00, 238+00, 248+00, and 274+53. Additional 65-ft long riprap aprons (D50=18") underlain by filter fabric are provided downstream of these cutoff walls to mitigate vertical drop scours where the downstream channel is not armored. The location of the cutoff walls and riprap aprons are shown on the channel Plan/Profiles in Appendix W-D. The CAP pothole survey information is also provided in Appendix W-D. Hydraulic and scour computations are provided in Appendix W-E.

The total cost for the proposed regional drainage improvement on Tangerine west end was estimated to be 2.58 million. The preliminary cost estimates are provided in Appendix W-G.

4.3 Impact of Proposed Tangerine Road Design on North Side Interceptor Channel Function

Tangerine Road will be widened to a four-lane divided section. The cross section of the road is crowned with the high points at the interior edges of pavement and 2% cross-slope to the outside edges. The distance from the pavement high point to the south bank of the interceptor channel ranges from 55 feet to 110 feet so there is 1.1 feet to 2.2 feet of vertical elevation change between these points. As mentioned in previous sections, the interceptor channel is designed to convey the 10-year runoff without freeboard. However, the elevation change associated with the above described pavement cross-slope requires the flow depth to overtop the channel bank by 1.1 to 2.2 feet before flow will weir to the south side of the road.

The conditions outlined in the above paragraph describe pavement design requirements that conflict with the general design guidelines for the channel; the result being that the interceptor channel combined with elevated west bound pavement section will contain more flow on the north side of the road than the 10-year peak discharge.

4.4 Proposed Conditions FLO-2D Models

The proposed interceptor channel was modeled as a channel component in the proposed conditions FLO-2D model. Since the roadway also provides significant drainage conveyance capacity, the northern half of the roadway is simulated as a portion of the roadside channel. In FLO-2D terms, the channel cross sections (XSEC.DAT) are measured from the roadway high points to the top of the north channel bank. The dip section in the west Trico driveway is also included in the channel component. The training berm along UPRR from Tangerine Road to the channel terminus is also modeled as a portion of the channel.

The general Manning's value used for the channel was 0.022. Slightly different Manning's values were used for some channel segments to ensure stable and faster FLO-2D simulations. The culverts at the east Trico driveway (5-10'x4' RCBC) was modeled as a drainage structure (HYSTRUC.DAT). The FLO-2D modeling parameters are substantially the same as given in Table 2, except as outlined in this and the above paragraphs. The 4-24-inch RCP culverts near UPRR/Tangerine Road and the low-flow culvert (2-24-inch RCP) at the channel western terminus are not included in the proposed conditions FLO-2D models because of their relatively small conveyance capacities.

4.5 FLO-2D Modeling Results and Impacts on Tangerine Road and Adjacent Properties

FLO-2D cross sections were generated to obtain peak discharge rates and runoff volumes at certain locations. The 100- and 10-year peak discharge rates and runoff volumes for existing and proposed conditions are shown on Figure F-1 in Appendix W-F of this report. Maximum flow depths for the 100- and 10-year floods in the FLO-2D study area are shown on Figures F-2 and F-3. Figures F-4 and F-5 show 100- and 10-year maximum flooding depths in the vicinity of Tangerine Road and in the vicinity of the channel terminus. Increases in maximum flooding depths for the 100- and 10-year floods, compared to those at existing conditions, are shown on Figures F-6 and F-7, respectively. Figures F-8 and F-9 show 100- and 10-year maximum flow velocities in the FLO-2D study limit.

Under proposed conditions, the FLO-2D model results indicate that Tangerine Road is dry in the 10-year rainfall event, even in the vicinity of the UPRR and Tangerine Road intersection. During the 100-year rainfall event, storm water does not overtop Tangerine Road from roadway Station 452+00 to the east end of this project. Just east of the UPRR/Tangerine Road crossing, approximately 900 feet of the road is inundated with flow depths of up to 1.4 feet, which are measured at the inside lane of Tangerine Road. Inundation on the roadway from Station 444+50 to Station 450+00 resulted in flooding depths of 1.0 foot and above. The estimated duration for inundation with depths of 1.0 foot and above on Tangerine Road is 3 hours during the 100-year storm.

As shown on Figures F-6 and F-7, 100- and 10-year flood depths increase by approximately 0.8 and 0.9 foot respectively, in an isolated area adjacent to and just downstream of the 230 foot long channel segment adjoining the UPRR. Flooding depth increases diminish to approximately 0.6 feet from a point approximately 40 feet downstream (northwest) of the channel terminus to a point about 630 feet northwest along the UPRR right-of-way. These flood depth surcharges gradually decrease with distance from Tangerine Road. During the 100-year rainfall event, flooding depth surcharges drop to about 0.1 foot at approximately 6,700 feet downstream (northwest) of Tangerine Road. Flooding depth surcharges within the UPRR ROW are up to 0.6 foot, however, the rail tracks remain flood free during 100-year flood event, as is the case in existing conditions.

Figures F-8 and F-9 show flow velocities within the study limit. Overland flow velocities in the vicinity of Tangerine Road are generally less than 3 feet per second (fps). 100-year flow velocities within the roadside interceptor channel are up to 9 fps for the segment with earthen bottom. Grade control cutoff walls, riprap apron, bank protection, and bank protection toe wall (see channel plan/profile in Appendix W-D) are provided to mitigate scour and bank erosion within the channel.

Compared to existing conditions, flooding on Tangerine Road is greatly improved in proposed conditions. Tangerine Road is dry over the entire project limit in the 10-year rainfall event. In the 100-year rainfall event, the proposed Tangerine Road improvements limited roadway overtopping to one location, the roadway segment near Tangerine Road and the UPRR crossing. At the Tangerine/UPRR crossing, flooding depths on Tangerine Road are slightly lower than those in existing conditions. However, the duration of inundation (reduced from 5 hours in existing conditions to 3 hours in proposed conditions) and the limit of inundated roadway are greatly reduced in proposed conditions. Flooding depths and limits on Tangerine Road are shown on Figure C-4 and Figure F-4.

4.6 Sediment Yield Computations and Drainage Structure Maintenance

The proposed channel on the north side of Tangerine Road intercepts runoff from Tortolita alluvial fan areas that are subject to high soil losses. Predicted high sediment yields from the alluvial fan indicate that post-flood channel/culvert maintenance will be required at some locations to maintain design capacity. As such, a drainage structure maintenance plan needs to be in place, and implemented to ensure proper function of these drainage structures.

The majority of the soils in the upstream alluvial fan watershed are categorized as an unconsolidated sandy loam. The longitudinal slope on the fan is approximately 3.0%, which results in high flow velocities and sediment transport rates. Fan apexes are approximately 4 to 5 miles upstream of this channel and the west end of Tangerine Road lies at the toe of the fan where deposition typically occurs.

The Universal Soil Loss Equation (USLE) was used to estimate a sediment yield of 3.48 tons per acre per year. Watershed areas were estimated to be 7,617 acres (watersheds 68 and 69), which would result in an annual sediment volume of 558,415 cubic feet. This USLE generated sediment volume is abnormally high and was not considered reasonable for use. Instead, a sediment yield of 0.36 tons per acre per year was used for estimation purposes, based on a study presented at the April 2006 Federal Interagency Sedimentation Conference in Reno, Nevada (reference 15). Use of this yield value resulted in an annual sediment volume of 57,728 cubic feet for watersheds 68 and 69. Sediment yield computations are provided in Appendix W-E. Drainage structure maintenance plan are provided in Appendix W-H.

For the roadside interceptor channel, the channel segment upstream of the east Trico driveway is fully lined with concrete (or shotcrete). The longitudinal slopes range from 0.62% to 1.3%. Velocities are generally high enough to carry sediment downstream without much sediment deposition. For the channel segment downstream of the east Trico Electric driveway, the channel has an earthen bottom with longitudinal slopes ranging from 0.2% to 0.5%. Sedimentation is very likely to occur within this reach due primarily to the slope reduction and the associated decrease in flow velocity.

Assuming that the sediment is evenly distributed on the channel segment at the channel bottom, average annual sediment deposition depth is estimated to be 0.21 foot. However, sediment deposition is most likely to occur along the reach immediately downstream of the slope break (approximately Roadway Station 487+00) where the earthen channel bottom section begins. Significant single flood deposition should be anticipated in this area. It is recommended that sediment deposition monuments to be placed on the channel bank every two hundred feet along the channel to monitor deposition and to use as a guide for restoring channel grades to design elevations.

SECTION 5.0 CONCLUSIONS

The west end regional drainage analysis included two main tasks: 1) performance of a broad-based drainage alternatives analysis, and 2) preparation of a detailed drainage analysis on preferred Alternative 2, which was selected by the TAC. Alternative 2 proposed an interceptor channel along the north side of Tangerine Road to convey the 10-year peak flow. Major findings from this study are listed below:

- Nine drainage alternatives were evaluated. Alternative 2, interceptor channel along north side of Tangerine Road to convey the 10-year peak flow, was selected by the TAC for further detailed analysis.
- During the 10-year rainfall event, existing Tangerine Road is overtopped in multiple locations, with inundation depths up to 2 feet. Proposed Tangerine road will be kept dry throughout the project limits during 10-year or more frequent rainfall events.
- During the 100-year rainfall event, proposed channel and roadway improvements limit Tangerine Road overtopping to one location, approximately 900 feet of the road just east of the UPRR/Tangerine Road crossing. This segment of the road will be inundated with flow depths of up to 1.4 feet (only slightly reduced compared to existing conditions), which are measured at the inside lane of Tangerine Road. The estimated duration for inundation with depths of 1.0 foot and above on Tangerine Road is 3 hours (reduced from 5 hours in existing conditions).
- 100-Year peak discharge rates, runoff volumes, and flooding depths increase at the northeast corner of the UPRR/Tangerine Road. Flooding depths increase by up to 0.6 feet within the UPRR ROW and up to 0.8 foot on the properties adjacent to UPRR ROW. However, these flood depth surcharges gradually decrease with distance from Tangerine Road. The rail tracks remain flood free during the 100-year rainfall event, as is the case in existing conditions.

SECTION 6.0 REFERENCES

1. Natural Resources Conservation Service, Hydrologic Soils Map, Soil Survey 669 – Eastern Pima County, 1999.

2. Pima County Regional Flood Control District, *PC-HYDRO, User Guide – Pima County Hydrology Procedures*, Version 5, Arroyo Engineering, LLC March 2007.

3. Arizona Department of Water Resources, *State Standard for Hydrologic Modeling Guidelines (SS10-07, draft)*, August 2007.

4. Pima County Department of Transportation, *Roadway Design Manual*, Third Edition, 2010.

5. Pima County Regional Flood Control District, *Technical Policies*.

6. Arizona Department of Transportation, *Highway Drainage Design Manual,* March, 1993.

7. O'Brien, Jim, *FLO-2D Flood Hydrograph Package User's Manual*, Version 2009.06.

8. Town of Marana, *Tortolita Alluvial Fan Study, Volume 1~4*, CMG Drainage Engineering, Inc., 2008.

9. City of Tucson, *Standards Manual for Drainage Design & Floodplain Management*, December 1989, Revised July 1998.

10. Pima County Department of Transportation and Flood Control District, *Drainage and Channel Design Standards for Local Drainage*, June 1984.

11. Arizona Department of Water Resources, *Design Manual for Engineering Analysis of Fluvial Systems*, Simons, Li & Associates, Inc., March 1985.

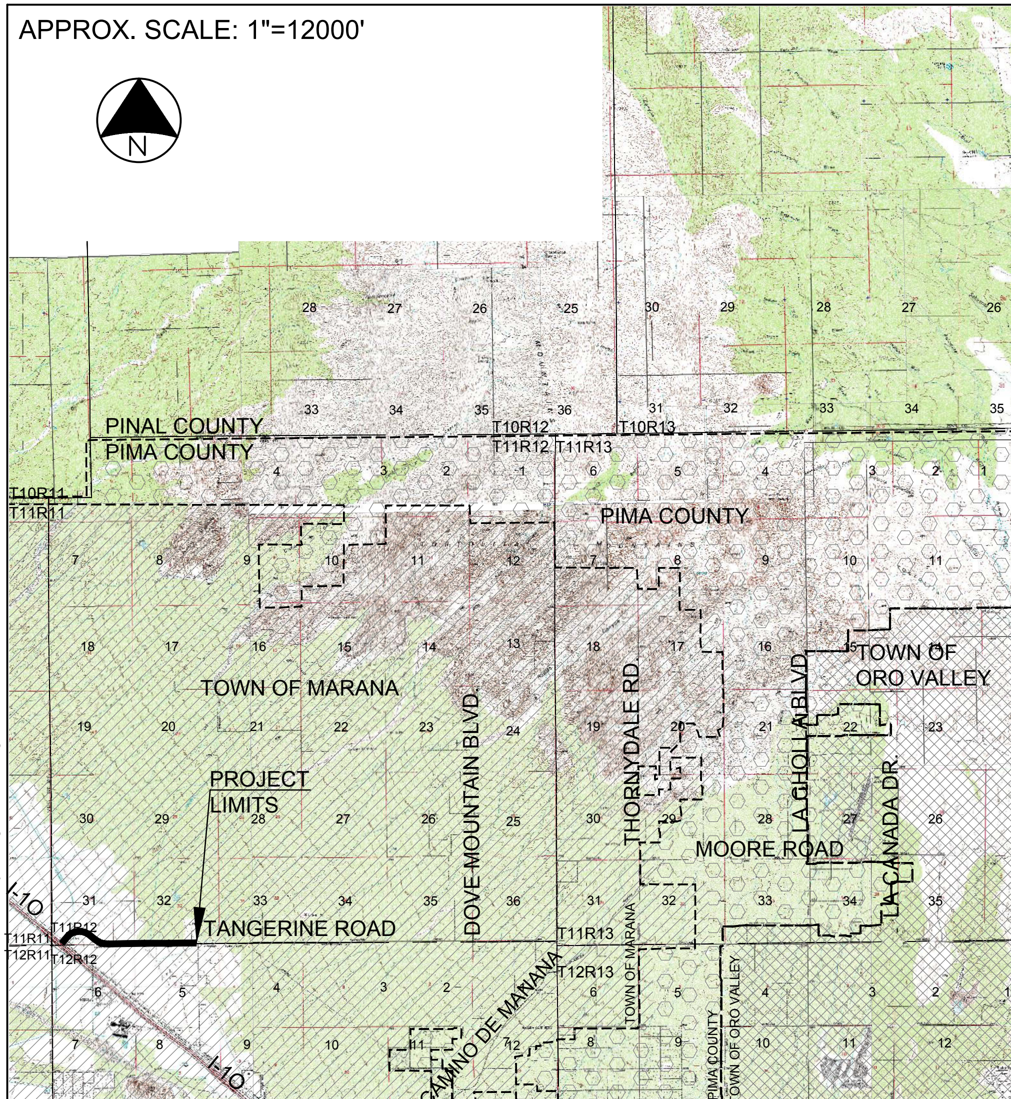
12. United States Army Corps of Engineers Hydrologic Engineering Center, *HEC-RAS River Analysis System User's Manual*, Version 4.1.0, January 2010.

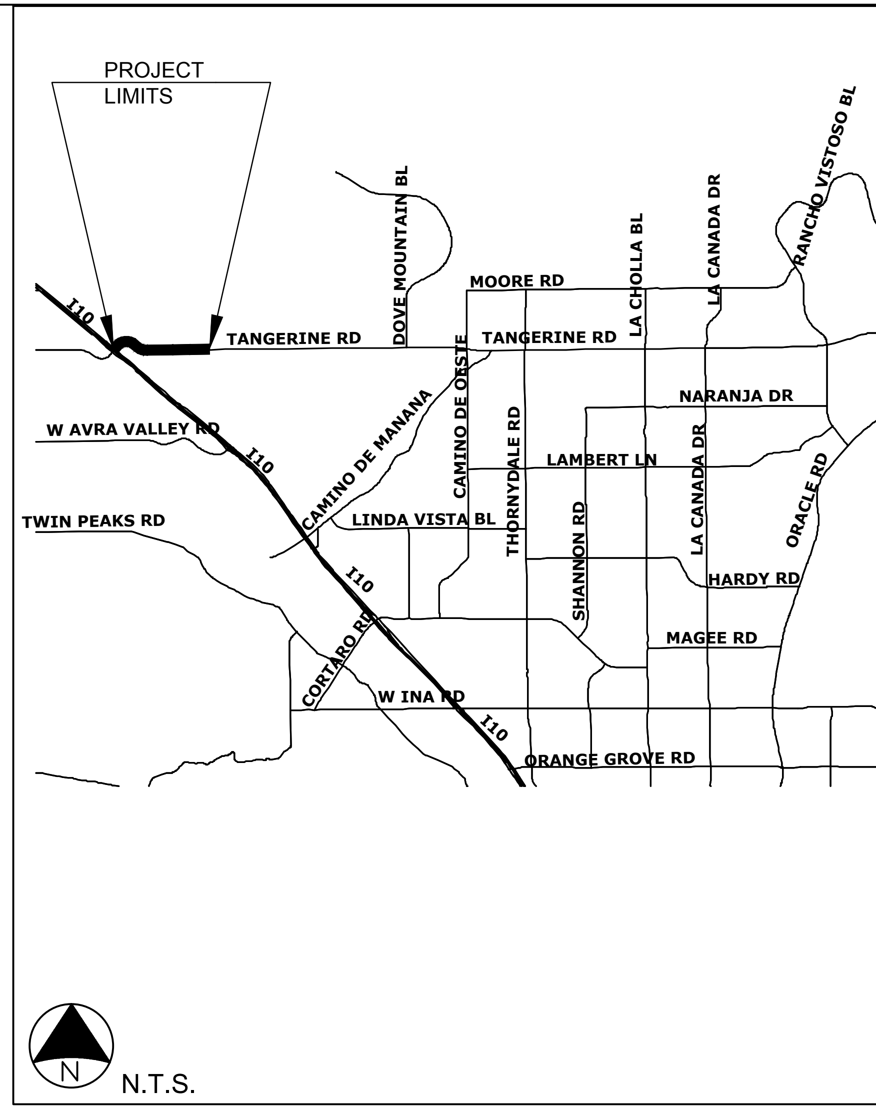
13. United States Federal Highway Administration, *HY-8 Quick Tutorial, HY-8 Version 8.7.*2, August 23 2011.

14. Ontario Ministry of Agriculture, Food and Rural Affairs, *FACTSHEET: Universal Soil Loss Equation (USLE),* May 2000.

15. Proceedings of the Eighth-Federal Interagency Sedimentation Conference, April 2006, Reno, NV, USA, *Alluvial Fan Erosion and Sedimentation Investigations Using the Hydraulic Modeling Tool FLO-2D,* April 2006.

FIGURES





LEGEND

PIMA COUNTY

TOWN OF MARANA

TOWN OF ORO VALLEY

FIGURE W-1:
TANGERINE WEST END REGIONAL DRAINAGE ANALYSES LOCATION/VICINITY MAP

T11S, R 12E, PORTIONS OF SECTIONS31, 32
T12S, R12E, PORTIONS OF SECTIONS 5,6
G.&S.R.M.,TOWN OF MARANA, PIMA COUNTY, ARIZONA

CMG JOB No. 10-027

Z:\PROJECTS\2010\10-027 Psomas-Tangerine Rd I-10 to La Canada\10-027.1 West End Analysis\dwgs\Loc Map.dwg, Loc map, 11/26/2012 2:24:18 PM

APPENDIX W-A

FEMA FLOODPLAIN MAPS

Includes

- FEMA Floodplain Maps

Z:\PROJECTS\2010\10-027 Psomas-Tangerine Rd I-10 to La Canada\Dwg\cmg_FEMA_FPL_Maps.dwg, Model, 12/27/2012 10:39:34 AM

NOTES TO USERS

This map is for use in administering the National Flood Insurance Program. It does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size. The **community map repository** should be consulted for possible updated or additional flood hazard information.

To obtain more detailed information in areas where **Base Flood Elevations** (BFEs) and/or **floodways** have been determined, users are encouraged to consult the Flood Profiles and Floodway Data and/or Summary of Stillwater Elevations tables contained within the Flood Insurance Study (FIS) report that accompanies this FIRM. Users should be aware that BFEs shown on the FIRM represent rounded whole-foot elevations. These BFEs are intended for flood insurance rating purposes only and should not be used as the sole source of flood elevation information. Accordingly, flood elevation data presented in the FIS report should be utilized in conjunction with the FIRM for purposes of construction and/or floodplain management.

Coastal Base Flood Elevations shown on this map apply only landward of 0.0' North American Vertical Datum of 1988 (NAVD 88). Users of this FIRM should be aware that coastal flood elevations are also provided in the Summary of Stillwater Elevations table in the Flood Insurance Study report for this jurisdiction. Elevations shown in the Summary of Stillwater Elevations table should be used for construction and/or floodplain management purposes when they are higher than the elevations shown on this FIRM.

Boundaries of the **floodways** were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the Flood Insurance Study report for this jurisdiction.

Certain areas not in Special Flood Hazard Areas may be protected by **flood control structures**. Refer to Section 2.4 "Flood Protection Measures" of the Flood Insurance Study report for information on flood control structures for this jurisdiction.

The **projection** used in the preparation of this map was Arizona Central State Plane zone (FIPSZONE 0202), International Feet. The **horizontal datum** was NAD 83, HPGN/HARN GRS80 spheroid. Differences in datum, spheroid, projection or State Plane zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

Flood elevations on this map are referenced to the North American Vertical Datum of 1988. These flood elevations must be compared to structure and ground elevations referenced to the same **vertical datum**. For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at http://www.ngs.noaa.gov or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, N/NGS12
National Geodetic Survey
SSMC-3, #9202
1315 East-West Highway
Silver Spring, Maryland 20910-3282
(301) 713-3242

To obtain current elevation, description, and/or location information for **bench marks** shown on this map, please contact the Information Services Branch of the National Geodetic Survey at **(301) 713-3242**, or visit its website at http://www.ngs.noaa.gov.

Base map information shown on this FIRM was derived from multiple sources. Base map imagery for eastern Pima County was provided in digital format by the Pima Association of Governments. These data were developed at 1-foot Ground Sample Distance (GSD) from color aerial photography flown in 2002. Base map imagery for western Pima County was derived from USGS Imagery available for the State of Arizona and produced at a scale of 1:12,000 from photography dated 2006 and 2007.

This map may reflect more detailed and up-to-date **stream channel configurations** than those shown on the previous FIRM for this jurisdiction. The floodplains and floodways that were transferred from the previous FIRM may have been adjusted to conform to these new stream channel configurations. As a result, the Flood Profiles and Floodway Data tables in the Flood Insurance Study Report (which contains authoritative hydraulic data) may reflect stream channel distances that differ from what is shown on this map.

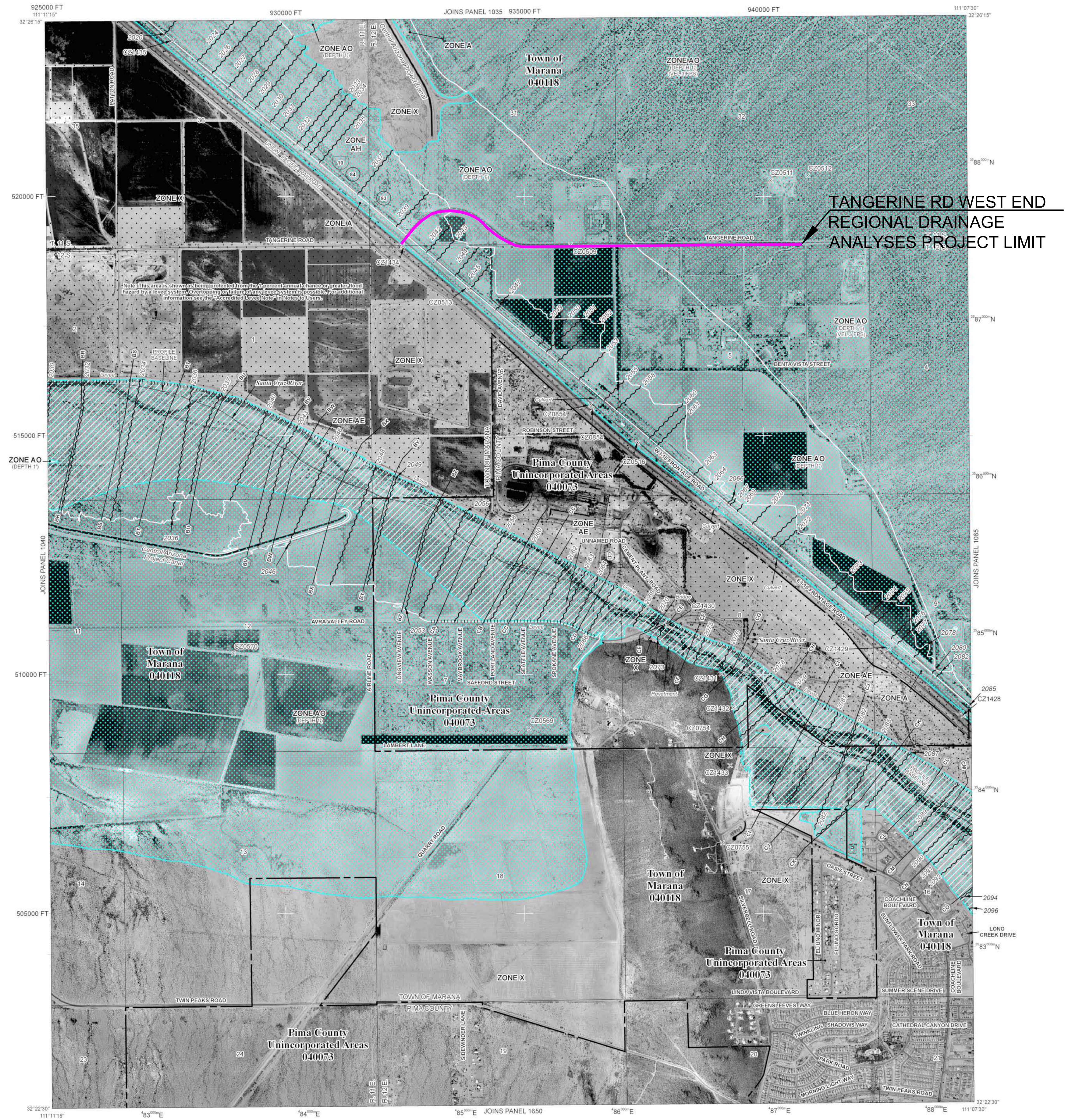
Corporate limits shown on this map are based on the best data available at the time of publication. Because changes due to annexations or de-annexations may have occurred after this map was published, map users should contact appropriate community officials to verify current corporate limit locations.

Please refer to the separately printed **Map Index** for an overview map of the county showing the layout of map panels; community map repository addresses; and a Listing of Communities table containing National Flood Insurance Program dates for each community as well as a listing of the panels on which each community is located.

For information on **available products** associated with this FIRM, visit the Map Service Center (MSC) website at http://msc.fema.gov. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. Many of these products can be ordered or obtained directly from the MSC website.

If you have **questions about this map,** how to order products, or the National Flood Insurance Program in general, please call the FEMA Map Information eXchange (FMIX) at 1-877-FEMA-MAP (1-877-336-2627) or visit the FEMA website at http://www.fema.gov/business/nfip.

Accredited Levee Notes to Users: Check with your local community to obtain more information, such as the estimated level of protection provided (which may exceed the 1-percent-annual-chance-level) and Emergency Action Plan, on the levee system(s) shown as providing protection for areas on this panel. To mitigate flood risk in residual risk areas, property owners and residents are encouraged to consider flood insurance and floodproofing or other protective measures. For more information on flood insurance, interested parties should visit the FEMA Website at http://www.fema.gov/business/nfip/index.shtm.



LEGEND

SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X — Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X — Areas determined to be outside the 0.2% annual chance floodplain.

ZONE D — Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

1% annual chance floodplain boundary

0.2% annual chance floodplain boundary

Floodway boundary

Zone D boundary

CBRS and OPA boundary

Boundary dividing Special Flood Hazard Area Zones and boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.

Limit of Moderate Wave Action

~~~ 513 ~~~ Base Flood Elevation line and value; elevation in feet*

(EL 987) Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988

(A)——(A) Cross section line

(23)- - - - -(23) Transect line

87°07'45", 32°22'30" Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere

$^{24}76^{000m}$N 1000-meter Universal Transverse Mercator grid values, zone 12

600000 FT 5000-foot grid values: Arizona State Plane coordinate system, Central zone (FIPSZONE 0202), Transverse Mercator projection

DX5510 × Bench mark (see explanation in Notes to Users section of this FIRM panel)

● M1.5 River Mile

MAP REPOSITORY
Refer to listing of Map Repositories on Map Index

EFFECTIVE DATE OF COUNTYWIDE
FLOOD INSURANCE RATE MAP
February 8, 1999

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL
June 16, 2011 - to update corporate limits, to change Base Flood Elevations and Special Flood Hazard Areas, to update map format, to add roads and road names, and to incorporate previously issued Letters of Map Revision.

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your Insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

N

REDUCED
N.T.S.

MAP SCALE 1" = 1000'

500 0 1000 2000 FEET

300 0 300 600 METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 1045L

FIRM
FLOOD INSURANCE RATE MAP

PIMA COUNTY,
ARIZONA
AND INCORPORATED AREAS

PANEL 1045 OF 4750

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
MARANA, TOWN OF	040118	1045	L
PIMA COUNTY	040073	1045	L

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

U.S. DEPARTMENT OF HOMELAND SECURITY

MAP NUMBER
04019C1045L

MAP REVISED
JUNE 16, 2011

Federal Emergency Management Agency
~~~

APPENDIX W-B

PRELIMINARY ANALYSES OF REGIONAL DRAINAGE ALTERNATIVES

Includes

- Meeting minutes from June 20, 2012 and August 31, 2012
- Detention Volume Computations
- Figure 1 – Conceptual Regional Basin Designs
- Drainage Alternatives 1 through 9 (descriptions and exhibits)
- Preliminary Cost Analyses

MEETING MINUTES

West End Regional Drainage Analysis Kick-off Meeting
Tangerine Road: I-10 to La Canada

June 20, 2012 @ 10:00 AM
PCRFCD

1. Attendees:
   Keith Brann – Town of Marana
   Scott Leska – Town of Marana
   Jennifer Christelman – Town of Marana
   Paul Baughman – Town of Marana
   Bill Zimmerman – PCRFCD
   Alejandro Angel – Psomas
   Clint Glass – CMG Drainage Engineering
   Jerry Curless – CMG Drainage Engineering

2. Review of project scope

   The project scope of work (Dec. 21, 2011 ver.) was distributed and reviewed. In summary; the scope consists of 1) a preliminary site analysis phase (guided by alternative sites chosen at this meeting); followed by 2) ranking of alternatives by CMG and another meeting to review results and choose highest ranking site for detailed H&H analysis and 30% level planning and design. Deliverables from the study will include:

   a. A separate report that includes not only drainage evaluations, but also discussions on other parameters and cost estimates;
   b. Preliminary plans for drainages facilities, e.g. basins, channels, pipes, culverts;
   c. Executive summary write up for Tangerine Road DCR.

3. Review of design parameters

   a. The project scope called for the 10-yr and 100-yr storms to be evaluated.
   b. The Town of Marana will provide additional archaeological information on known sites for informational use during evaluations, but not for inclusion in final report.
   c. The Town will also assist on archaeological mitigation cost estimates (ball park numbers only) during evaluation.
   d. Non-drainage factors, such as property ownership issues, cultural resource mitigation issues, integration into future Tangerine Rd interchange plans, etc. may cause alternatives to be dropped from evaluation list.
   e. The multi-use park facility potential for a property to be listed as a Pro- for that site, but not figured into the cost estimate.
   f. Use of a detention basin site also as a borrow source for Tangerine Rd construction should only be considered for the west half of the project (Phase 2 construction).
   g. Town of Marana to arrange talks with MSP property owner to discuss opinions on potential uses of property for west-end regional drainage facilities. Results will be used to determine inclusion or exclusion of site in preliminary evaluation list.
   h. Town of Marana would consider basins designed with retention volumes below viable gravity outfalls with use of either dry wells or possible storm water lift station to drain in 36 hr regulatory time limit.

4. Review & discussion of alternative properties for evaluation

   Preliminary write-ups and exhibits for eight alternative sites were presented by CMG and evaluated by the group. In the end, a ninth alternative was added to the list and several alternatives were dropped from the list for further analysis. Final report to outline all decision parameters considered.

   Results of alternatives review:

Alternative ID	Description	Recommended for Further Analysis? Y/N
Alternatives without detention basins		
Alternative 1	No-build/maintain existing cross drainage	Yes – add existing conditions small-grid FLO-2D modeling to the preliminary phase of the study to better define current flooding potential during various return frequency storms. Include discussion of frequency of known historical flooding.
Alternative 2	No-basin/Channel-only	Yes – Town of Marana to discuss concept with MSP property owner to determine tolerance for potential increases in 10-yr peak flows on property in interim between Tangerine Rd and new I-10 TI construction.
Alternative 3	Causeway/multiple small diameter culvert system	Cost estimated at $10 mil+ with large adverse maintenance issues. No further analysis recommended
Alternatives with detention basins		
Alternative 4	Detention Basins east of Trico Property.	No further analysis recommended due to known cultural resource conflicts
Alternative 5	Combined Detention Basins east of Trico Property and east of UPRR @ MSP property.	No further analysis recommended due to known cultural resource conflicts
Alternative 6	Detention Basins on Tangerine Invest Partners LLC.	High commercial value of property makes unfeasible. No further analysis recommended
Alternative 7	Channelization/Basin on MSP Property.	Yes – evaluate 10- and 100-yr capacity channels and maximum feasible detention basin volume (match channel and basin design parameters with Alt. 8)
Alternative 8	Detention basin ~~at southeast corner of UPRR/Tangerine Rd intersection~~ on Kai property south of Tangerine Rd	Yes – but modify to construct basin with maximum feasible volume on south portion of Kai property with 400 ft setback from Tangerine Rd to allow commercial development frontage. Evaluate 10- and 100-yr capacity channels (match evaluation parameters with Alt. 7)
Alternative 9	Combination of Alt. 7 and Alt. 8 with detention basins on Kai property and at downstream MSP property location connected by new channels	Yes – evaluate 10- and 100-yr capacity channels and maximum feasible detention basin volumes with same parameters as Alt. 7 & 8

5. Action Items

- Jennifer Christelman will provide CMG with information on known archeological sites;
- CMG to remove archeological site info from exhibits;
- CMG has confirmed that the detention requirements for MSP will be waived once channel is connected to Barnett Linear Park channel, per agreement with Gilbert Davidson. Kevin Kish, Town of Marana, plans to discuss drainage concepts with Marc Palkowitsh when he is in Denver next week;
- CMG to determine if any of the alternatives targeted for further analysis have potential conflicts with TRICO utility poles;
- CMG to evaluate whether or not channel along railroad can be redesigned to have an earthen bottom rather than concrete;

MEETING MINUTES

West End Regional Drainage Analysis Preliminary Assessment of Alternatives Meeting
Tangerine Road: I-10 to La Canada

August 31, 2012 @ 9:30 AM
Town of Marana

1. Attendees:
   Keith Brann – Town of Marana
   Scott Leska – Town of Marana
   Paul Baughman – Town of Marana
   Alejandro Angel – Psomas
   Clint Glass – CMG Drainage Engineering
   Jerry Curless – CMG Drainage Engineering
   Jiankang Wang – CMG Drainage Engineering

2. Materials distributed

   a. Tangerine Road West End Regional Drainage Analysis – Preliminary Assessment of Alternatives (including Figures);
   b. Draft summary of existing conditions small grid (20-ft grid) FLO-2D models (both 100-year and 10-year) (including Figures); The purpose is to accurately depict the flooding conditions on Tangerine Rd pavement under existing conditions;
   c. Cost Analysis for Alternatives for Tangerine Rd West End Regional Drainage Study.

3. General Overview of Results

   a. Results from small grid FLO-2D models (100- and 10-year storms) were used to evaluate Alternative 1. This modeling identified roadway overflow areas and depths during the 100- and 10-year storms.
   b. It is a general consensus that Alternative 2 would probably increase the peak flow on MSP property because runoff directed toward MSP by the proposed interceptor channel. This is because flows currently crossing Tangerine Road would not be attenuated on the broad overland areas south of Tangerine Rd. The next step will be to conduct FLO-2D modeling of this alternative to determine the magnitude of peak flow changes.
   c. Only Alternatives 8 and 9 were demonstrated to be technically feasible as a multi-use facility. The town indicated that a park is roughly 30+ acres. However, the cost of these alternatives is very high.

4. Review & discussion of alternative properties for evaluation

Preliminary write-ups and exhibits for nine alternative sites were presented by CMG and evaluated by the group. In the end, Alternative 2 (No-basin/Channel-only) was recommended by Keith Brann for further evaluation and possibly incorporated in the 30% plan.

Results of alternatives review:

Alternative ID	Description	Recommended for Further Analysis? Y/N
Alternatives without detention basins		
Alternative 1	No-build/maintain existing cross drainage	No – Small grid FLO-2D showed that Tangerine Rd is subject to significant flooding during 100- and 10-year rainfall event.
Alternative 2	No-basin/Channel-only	Yes – CMG to proceed with analysis and design for a 10-year capacity channel along the north side of Tangerine Rd
Alternative 3	Causeway/multiple small diameter culvert system	No – Cost estimated at $16 mil+ with large adverse maintenance issues. No further analysis recommended
Alternatives with detention basins		
Alternative 4	Detention Basins east of Trico Property.	No – Known cultural resource conflicts
Alternative 5	Combined Detention Basins east of Trico Property and east of UPRR @ MSP property.	No – Known cultural resource conflicts
Alternative 6	Detention Basins on Tangerine Invest Partners LLC.	No – High commercial value of property makes unfeasible.
Alternative 7	Channelization/Basin on MSP Property.	No – High cost (estimated to be $12.6 mil) associated with this Alternative makes it unfeasible.
Alternative 8	Detention basin on Kai and other properties south of Tangerine Rd	No – High cost (estimated to be $7.9 mil) associated with this Alternative makes it unfeasible.
Alternative 9	Combination of Alt. 7 and Alt. 8 with detention basins on Kai property and at downstream MSP property location connected by new channels	No – High cost (estimated to be $11.0 mil) associated with this Alternative makes it unfeasible.

5. Design parameters/considerations for Alternative 2 (No-basin/Channel-only)

   a. Design the interceptor channel along Tangerine Rd to its maximum allowable capacity within the current right of way.
   b. The interceptor channel need to satisfy CAP siphon's vertical clearance requirements.

c. CMG to evaluate whether cross culverts could be proposed in the vicinity of Sta 514+00. The purpose is to try to reduce the amount of runoff draining to MSP property via the interceptor channel.
d. Avoid worsening scour impacts within UPRR right of way in this design.
e. Avoid grading onto UPRR right of way if possible. End of the interceptor channel could encroach onto MSP property (within the TI channel Right of way) to alleviate scour damages and redistribute flow back to existing conditions, if needed.
f. Provide peak flow/runoff volume comparison between existing and proposed conditions at MSP property.
g. Document 100- and 10-year flooding conditions on proposed Tangerine Rd west end. These include peak flows that overtops the road and roadway inundation durations.

6. <u>Action Items</u>

- CMG to provide revised 100- and10-year flow depths (0.5' and above) on Tangerine Rd pavement in 20-ft grid FLO-2D models, including flow depths and durations of the flooding.
- CMG to provide Figure 1-3 (existing 10-Yr and 100-Yr Flow Depths) for 20-ft grid FLO-2D models.
- CMG to revise the basin/channel excavation cost from $8 per cubic yard to $5 per cubic yard in the cost estimates.
- CMG to credit borrow savings from the basin/channel excavation from $8 per cubic yard to $5 per cubic yard in the cost estimates.
- CMG to revise future Adonis Rd right of way from 200 feet to 150 feet.
- Move forward to evaluate Alternative 2 (No-Basin/Channel only) to 30% plan level.
- Town of Marana to find out how much recreation area are shown on MSP property Specific Plan.
- Town of Marana to coordinate with MSP property owners regarding peak flow/runoff volume increases on MSP property based detailed Alternative 2 analysis, when it is available.

CMG DRAINAGE
ENGINEERING, INC.

3555 N. Mountain Ave. Tucson, Arizona 85719
Phone (520) 882-4244 Fax (520) 888-1421

Pima County Required Detention Volume
Eq. 3.4

Project

Client: Town of Marana
Project #: 10-027

Date 7/7/2011
By: Jiankang

Per Pima County Stormwater Detention/Retention Manual Equation 3.4:

$$V_R = \left(\frac{C_{WDEV} * P_t * A}{12} \right) \left(1 - \frac{Q_0}{Q_i} \right)$$

Event (YR)	Volume (ac-ft)	Basin Outflow Channel	Basin Outflow (cfs)	Basin Inflow (cfs)	Basin Volume (ac ft)	Basin Area (acres)*	Dirt Volume (cubic yard)**
100-Year	808.3	1.5' deep	457	3,964	**715**	**172**	1,938,242
100-Year	808.3	3' deep	1,462	3,964	**510**	122	1,382,801
10-Year	231.1	1.5' deep	346	1,026	**153**	37	415,140

*Basin area was estimated by assuming average 5' ponding depth with a factor of 1.2 (account for basin berm and other features)
** Dirt volume assumes average 7' dirt over the entire basin area.
*** 10-year basin outflow is controlled by existing 10-year peak flow @ UPRR (Q10=346 cfs)

REGIONAL BASIN ANALYSIS SUMMARY TABLE

Event (YR)	Volume (ac-ft)	Basin Outflow Channel	Basin Outflow (cfs)	Basin Inflow (cfs)	Basin Volume (ac ft)	Basin Area (acres)*	Dirt Volume (cubic yard)**
100-Year	808.3	1.5' deep	457	3,964	**715**	**172**	1,938,242
100-Year	808.3	3' deep	1,462	3,964	**510**	**122**	1,382,801
10-Year	231.1	1.5' deep	346	1,026	**153**	**37**	415,140

* BASIN AREA WAS ESTIMATED BY ASSUMING AVERAGE 5' PONDING DEPTH WITH A FACTOR OF 1.2 (ACCOUNT FOR BASIN BERM AND OTHER FEATURES)

** DIRT VOLUME ASSUMES AVERAGE 7' DIRT OVER THE ENTIRE AREA.

*** 10-YEAR BASIN OUTFLOW IS CONTROLLED BY EXISTING 10-YEAR PEAK FLOW @ UPRR (Q10=346 CFS)

**** BASIN OUTFLOW CONTROLLED BY DOWNSTREAM CHANNEL CAPACITY OR EXISTING PEAK FLOWS @ UPRR
457 CFS = 1.5' DEEP CHANNEL (EST. 2~5 YR RETURN INTERVAL STORM)
1462 CFS = 3.0' DEEP CHANNEL (MORE THAN 10 YR RETURN INTERVAL STORM)

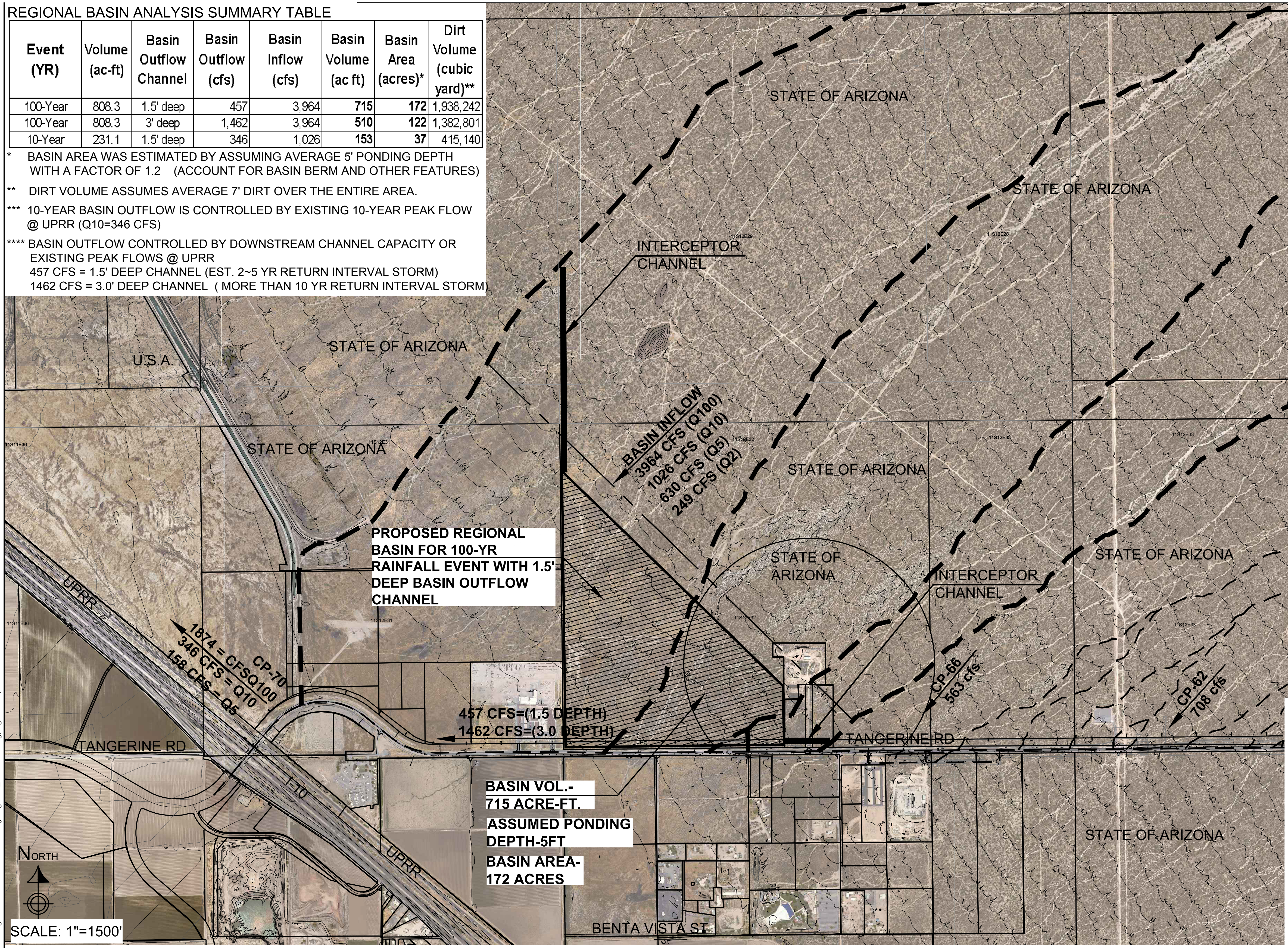
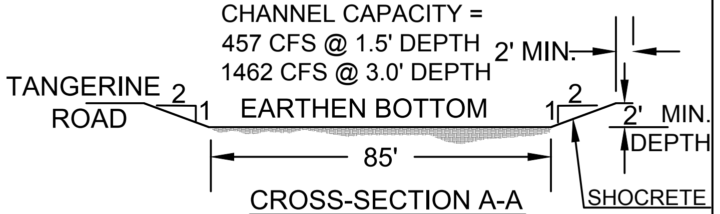


FIGURE 1:
CONCEPTUAL REGIONAL BASIN DESIGNS



CROSS-SECTION A-A
BASIN OUTLET CHANNEL

MIN. LONGITUDINAL SLOPE-0.3%

LEGEND

- REGIONAL WATERSHED BOUNDARIES
- LOCAL WATERSHED BOUNDARIES
- FLO-2D CROSS SECTION (COVERS CP-67,68 & 69)
- 38.0 (1.5' DEPTH) CHANNEL INVERT ELEVATION

NOTES:

1. HORIZONTAL PROJECTION: NAD 1983 HARN ,ARIZONA CENTRAL STATE PLANE. TOPOGRAPHY: PIMA ASSOC. OF GOV'S 2000 NAVD88, 10-FT CONTOUR INTERVAL.
2. AERIAL PHOTOGRAPHY: PIMA ASSOC. OF GOV'S 2005 DATA.
3. BASIN SHAPE & LOCATION IS FOR EXAMPLE PURPOSES ONLY. ALTERNATIVE LOCATIONS HAVE NOT BEEN EVALUATED.

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Alternative 1 – No-built/Maintain Existing Cross Drainage

Design Frequency – N/A

Description of Alternative 1 - In Alternative 1, proposed roadway profile will be kept the same as the existing one. No channel or detention basins will be built. As a result, cross drainage patterns would remains the same as that under existing conditions. The roadway will subject to periodic flooding, possibly even with storms much more frequent than the design 10-year storm event used by the project team.

Pros

- Cost saving. No channel, culverts, or detention basins need to build.

Cons/Design Issues

- Roadway will subject to periodic flooding, the same as existing conditions.
- Flooding on the roadway could result in embankment and/or pavement erosion; sediment and debris deposits on the road, either of which could require temporary road closures for maintenance and repairs
- Does not comply with established drainage design criteria

Alternative 2 – No-basin/Channel Only

Design Frequency – 10-Year

Description of Alternative 2 – Alternative 2 proposed a channel along north side of Tangerine Road to convey the 10-year peak flow, while runoff exceeding 10-year storm event could overtop the roadway and drain to the south. The channel would end at the northeast corner of UPRR/Tangerine Rd intersection without a gravity outlet. Runoff at the channel terminus would have to weir over the north bank and onto MSP Properties (Mandarina). Retention volumes (standing water in the channel below the weir) could drain to a low flow swale within UPRR right of way to purge the residual standing water.

No detention basin is proposed for this alternative. Collection and conveyance of flows along the north side of Tangerine Road will cause a bypass of the natural detention currently provided by the Kai farmland on the south side of Tangerine Rd. Previous studies have determined that this area aids to significantly decrease downstream peak flows. Therefore, the proposed peak flow (10-year: ~1026 cfs) at the northeast corner of UPRR/Tangerine Road will be much higher than that in existing conditions (10-year: ~346 cfs).

Pros

- Cost saving. No culverts or detention basins needed.
- Flooding conditions will be reduced on properties south of Tangerine for rainfall not exceeding the design storm.

Cons/Design Issues

- 10-year and 100-year peak discharges increase, along downstream reaches.
- Peak flow increases are generally not allowed per Town of Marana's floodplain ordinance.
- Grading required onto UPRR right of way to purge standing water in the channel.
- Roadway subgrade will be subject to 10-year flood and special treatment maybe needed.
- Roadway will be flooded and subject to damages during rainfall events exceeding 10-year storm.

Alternative 3 – Causeway/Multiple Small Diameter Culvert System

Design Frequency – 100-Year

Description of Alternative 3 - Alternative 3 reviews the feasibility and design issues associated with installing multiple small diameter culverts to convey the runoff beneath Tangerine Road. Because this alternative needs to raise the roadway a minimum of 3 feet (to accommodate a 24-inch culvert), 100-year design storm, instead of 10-year storm, was used to avoid acquiring a significant amount of drainage easements to mitigate the affects of extensive backwater ponding. Preliminary computations indicate that approximately 647 barrels of 24-inch diameter RCP culverts, with 1.5' headwater, would be needed to convey the 100-year flow across Tangerine Rd to the south. Culverts would be located throughout the west end section from Station 449+00 to Station 526+00; at locations most suitable for collection and release. At the east side of the UPRR, from Station 442+00 to Station 449+00, 218 barrels of 24-inch diameter RCP would be needed to convey the 100-year northerly along UPRR. Current flooding depths throughout this area are 0.5 to 1.5 feet based on the effective FEMA map.

Pros

- Existing drainage patterns are generally maintained.
- This alternative does not increase downstream peak discharges, flow velocities, and flooding depths.

Cons/Design Issues

- Requires road profile to be elevated a minimum of 3 feet to set culvert inlet and outlet on grade. A raised roadway profile means more cost.
- Elevated road profile suggests increased backwater depths and ponding outside of road right of way. Requires drop inlets, check dams, and stabilized collector and disbursal channels to lower road profile, or, ponding easements needed.
- Sediment deposition within or at the culvert inlets will be significant, requiring periodic maintenance. Culvert maintenance for small diameter structures will be difficult for culvert lengths of ±130 feet.
- Disruption of anticipated drainage patterns and flow along north side of Tangerine Road if culverts do not function as intended.
- Cost estimated to be more than $16 million.

Alternative 4 – Detention Basin East of Trico Property

Design Frequency – 10-Year

Description of Alternative 4 – Alternative 4 proposed a detention basin on State land, which is located just east of the Trico property. Detention basin volume is estimated to be 153 acre-feet for reducing the 10-year peak flow to existing conditions rates north of Tangerine Road. As shown on the attached exhibit, two interceptor channels with 10-year event runoff capacity are proposed to intercept overland flow and direct it to the detention basin. The interceptor channel that runs in the south-north direction does not extend all the way to the watershed limit which will allow a portion of the overland flow to bypass the basin. The basin outlet channel will collect the bypass runoff along Tangerine Rd and eventually convey it to the northeast corner of Tangerine Rd and UPRR, where the basin outflow channel ends. As with Alternative 2, runoff will weir to the north onto MSP Properties. Standing water in the channel below the weir will drain to a low flow swale within UPRR right of way.

Pros

- The detention basin could generate up to 415,000 cubic yards of fill material for the roadway construction, assuming an average basin depth of 7 feet.
- This alternative does not increase downstream peak discharges, flow velocities, or flooding depths.
- Potential cost saving because no culverts are needed to convey the runoff across the road.
- Flooding conditions will be reduced on properties south of Tangerine for rainfall events less than the design storm.
- Less sediment maintenance downstream of the basin outlet because the detention basin removes most of sediments.

Cons/Design issues

- Grading onto UPRR right of way is required to drain the channel, as with Alternative 2. Otherwise, 2~3 feet of standing water will present at the weir location.
- Roadway subgrade will be subject to 10-year flood and special treatment maybe needed.
- Portions of drainage structures are on known archaeology site. Further archaeology evaluations by the Town are needed.
- Sediment maintenance for interceptor channels and detention basins.
- Natural grade of this site slopes at 2%-3% which means that multiple (tiered) basins will probably be required. This design will be hydraulically less efficient than a single basin of equal volume.

Alternative 5 – Combined Detention Basins East of Trico Property and East of UPRR @ MSP Property

Design Frequency – 10-Year

Description of Alternative 5 – Alternative 5 proposed a smaller detention basin on the State land (as with Alternative 4) located east of Trico property, and, a series of small detention basins along the proposed Tangerine Road Interchange Channel along the east side of the UPRR.. The basins within the channel would be created by building a series of check dams in the channel to detain flow. Theses dams would be removed when the channel is extended and connected to the Barnett Linear Park Channel. Total detention basin volume is estimated to be 153 acre-feet for the 10-Year design frequency. The series of detention basins along UPRR would have approximately 59.0 acre-feet detention volume within the channel which is approximately 78 feet wide, 6 feet deep, and 6700 feet long (and with longitudinal slope of 0.27%). The detention basin east of Trico property would have about 94 acre-feet of detention volume. As shown on the attached exhibit, two interceptor channels with 10-year event runoff capacity are proposed to intercept overland flow and direct it to the detention basin. The interceptor channel that runs in the south-north direction does not extend all the way to the watershed limit which will allow a portion of the overland flow to bypass the basin. The basin outlet channel will collect the bypass runoff along Tangerine Rd and eventually convey it to the northeast corner of Tangerine Rd and UPRR where it then connects to the aforementioned channel (basins) along the UPRR. Runoff will then be further attenuated and ultimately weir onto downstream properties at the north end of the MSP property.

Pros

- The detention basin could generate roughly 415,000 cubic yard dirt to be used for the roadway.
- This alternative does not increase downstream peak discharges, flow velocities, and flooding depths.
- Potential cost saving because no roadway culverts are needed to convey the runoff across Tangerine Road.
- Flooding conditions will be reduced on properties south of Tangerine for rainfall less than the design storm.
- Less sediment maintenance downstream of the basin outlet (basin east of Trico) because the detention basin removes most of sediments.
- Size of basin east of Trico can be reduced by utilizing the channel along the UPRR as a basin (temporarily).

Cons/Design Issues

- Roadway subgrade will be subject to 10-year flood and special treatment maybe needed.
- Portions of drainage structures are on known archaeology site. Further archaeology evaluations are needed.
- Sediment maintenance for interceptor channels and detention basins.

Alternative 6 – Detention Basin on Tangerine Invest Partners LLC Property

Design Frequency – 10-Year

Description of Alternative 6 – Alternative 6 proposed a detention basin on Tangerine Invest Partners LLC Property. Required detention basin volume is estimated to be 153 acre-feet for the 10-year frequency storm. Depth and area of the basin have not yet been estimated. As shown on the attached exhibit, an interceptor channels with 10-year event runoff capacity are proposed to intercept overland flow and direct it to the detention basin. The 10-year storm capacity for the interceptor channel would be approximately 745 cfs. Some of the overland flow could potentially drain directly into the basin. The basin outflow would drain to the northeast corner of Tangerine Rd and UPRR, where the basin outflow channel ends. Runoff will then weir onto the north on MSP Properties as with Alternative 2. Retention volumes (standing water in the channel below the weir) could drain to a low flow swale within UPRR right of way to purge the residual standing water.

Pros

- This alternative does not increase downstream peak discharges, flow velocities, and flooding depths.
- Properties south of Tangerine would not be flooded for rainfall less than the design storm.
- Less sediment maintenance downstream of the basin outlet because the detention basin removes most of sediments

Cons/Design issues

- The natural ground slope at the basin is between 0.5% and 1.0%, which limits the basin's depth to be approximately 2 feet, with gravity drainage, unless combined with some construction of the channel/basins along the UPRR (as described in Alternative 5).
- Possible grading onto UPRR right of way to purge standing water. Otherwise, 2~3 feet of standing water may present at the weir location.
- Roadway subgrade will be subject to 10-year flood and special treatment maybe needed.
- Sediment maintenance for interceptor channels and detention basins.
- Commercial value of the property is high.

Alternative 7 – Channelization/Basin on MSP Property

Design Frequency – 10- and 100-Year

Description of Alternative 7 – Alternative 7 proposed an interceptor channel along Tangerine Rd (Cross Section A-A), a conveyance channel (Cross Section B-B, which has the same geometry and profile as the channel presented in the drainage report for the Tangerine Traffic Interchange, but revised to have earthen bottom) along UPRR on MSP Property, and a detention basin at the existing pit on the northwest corner of MSP property. The interceptor channel is along the entire west end segment from Station 450+00 to Station 526+00 to capture overland flow. The capture runoff will then drain to a 5600 feet long conveyance channel along UPRR on MSP Property and to be directed to the detention basin. A long basin outflow weir will return the flow to existing drainage patterns north of MSP property. The estimated basin area, for both 10-year and 100-year design alternatives, is of the same, 37 acres.

For 100-year drainage design alternative, approximately 3964 cfs will be collected by the interceptor channel (Cross Section A-A). The basin will have averagely 16.5' ponding water depth to generate approximately 510 acre-ft detention volume.

For 10-year drainage design alternative, approximately 1026 cfs will be collected by the interceptor channel (Cross Section A-A). The basin will have averagely 5' ponding water depth to generate approximately 153 acre-ft detention volume.

Pros

- This alternative does not increase downstream peak discharges, flow velocities, and flooding depths.
- Potential cost saving because no roadway culverts are needed to convey the runoff across Tangerine Road.
- Flooding conditions will be reduced on properties south of Tangerine for rainfall less than the design storm.
- Utilized the existing pit on MSP property to provide gravity drainage for the proposed drainage system.

Cons/Design Issues

- For the 10-year design, roadway subgrade will be subject to 10-year flood and special treatment maybe needed.
- Sediment maintenance for interceptor channels, conveyance channel, and detention basins.
- Additional costs associated with approximately 6700 feet lined channel along UPRR.
- Detention basin on MSP property is assumed to be 16.5 feet deep for 100-year design. The existing pit at this location is roughly 12 feet deep. The basin depth could be reduced by increasing the basin area. Dry wells to drain the retention volume may be needed.

Alternative 8 – Detention Basin on Properties South of Tangerine Rd

Design Frequency – 10- and 100-Year

Description of Alternative 8 – Alternative 8 proposes one or more detention basins on properties south of Tangerine Road. These properties are owned by Kai Properties, Harper Revoc Tr, and LPE 1 LLC. The estimated land area needs for 10-year and 100-year capacity basins are 31.0 acres and 102.0 acres, respectively. Most of the storm water storage within these basins would gravity drain via a conveyance channel (Cross Section B-B) along UPRR on MSP property. This channel, which revised to have earthen bottom, has the same geometry and profile as the channel presented in the drainage report for the Tangerine Traffic Interchange (TI).

For 100-year drainage design alternative, approximately 2035 cfs and 843 cfs will be collected by the interceptor channels (Cross Section A1 and A2) along Tangerine Rd and be directed to the basins south of Tangerine Rd via 7-10'x5' and 3-10'x5' RCBC culverts The basins will have averagely 6.0' ponding water depth to generate approximately 510 acre-ft detention volume. Outflow (1874 cfs) from the basin is conveyed beneath Tangerine Road through a 6-10'x5' RCBC to the channel running along the UPRR through the MSP property. This channel outlets to an existing pit on MSP property as proposed by the drainage plan for the Tangerine TI. Additional flows are received by the TI channel from interceptor channel A3 resulting in a total design flow of 1874 cfs.

For 10-year drainage design alternative, approximately 745 cfs will be collected by the interceptor channels (Cross Section A1 and A2) along Tangerine Rd and directed to the basins south of Tangerine Rd via 4-10'x4' RCBC culverts. The basins will have averagely 6.0' ponding water depth to generate approximately 153 acre-ft detention volume. Outflow (346 cfs) from the basin is conveyed beneath Tangerine Road through a 2-10'x4' RCBC to the channel running along the UPRR through the MSP property. This channel outlets to an existing pit on MSP property as proposed by the drainage plan for the Tangerine TI. Additional flows are received by the TI channel from interceptor channel A3 resulting in a total design flow of 346 cfs.

Pros

- The detention basins could generate roughly 820,000 (100-year) or 247,000 (10-year) cubic yards of fill dirt for the roadway construction (assuming average depth of 6 feet).
- This alternative does not increase downstream peak discharges, flow velocities, and flooding depths.
- Utilized the existing pit on MSP property to provide gravity drainage for the proposed drainage system.
- Flooding conditions will be reduced on properties south of Tangerine Road.

Cons/Design issues

- For the 10-year design, roadway subgrade will be subject to 10-year flood and special treatment maybe needed.
- Sediment maintenance for interceptor channels and detention basins.
- Additional costs associate with box culverts under Tangerine Road to convey runoff southerly to the basin.
- Additional costs associated with approximately 6700 feet lined channel along UPRR through MSP.

Alternative 9 – Detention Basin on MSP Property and Properties South of Tangerine Rd
Design Frequency – 10- and 100-Year

Description of Alternative 9 – Alternative 9 proposed detention basins on MSP property and properties south of Tangerine Road. These properties are owned by Kai Properties, Harper Revoc Tr, and LPE 1 LLC properties. A conveyance channel (Cross Section B-B) along UPRR on MSP property is proposed to direct basin outflow from the basin south of Tangerine Rd to the basin on MSP property. This channel, which revised to have earthen bottom, has the same geometry and profile as the channel presented in the drainage report for the Tangerine Traffic Interchange (TI). In the preliminary analysis, basins on MSP property and basins south of Tangerine Rd are assumed to provide equal amount of detention volume (50% of total volume). The estimated land area needs for 10-year and 100-year capacity basins are 34.0 acres and 70.0 acres, respectively.

For 100-year drainage design alternative, approximately 2455 cfs will be intercepted by the channel (Cross Section A1) along Tangerine Rd and be directed to the basins south of Tangerine Rd via 8-10'x5' RCBC culverts. The basins south of Tangerine Road will have averagely 6.0' ponding water depth to generate approximately 255 acre-ft detention volume. Outflow (1874 cfs) from the basin is conveyed beneath Tangerine Road through a 6-10'x5' RCBC to the channel running along the UPRR through the MSP property. This channel outlets to an existing pit on MSP property as proposed by the drainage plan for the Tangerine TI. The portion of the runoff that will be intercepted by interceptor channel A2 and A3 will be directly conveyed to downstream basin on MSP property. Both basins on MSP property and basins south of Tangerine Rd provide 255 acre-ft detention volume (total 510 acre-ft).

For 10-year drainage design alternative, approximately 745 cfs will be intercepted by the interceptor channels (Cross Section A1 and A2) along Tangerine Rd and then be direct to the basins south of Tangerine Rd via 4-10'x4' RCBC culverts. The basins south of Tangerine Road will have averagely 6.0' ponding water depth to generate approximately 77 acre-ft detention volume. Outflow is conveyed beneath Tangerine Road through a 2-10'x4' RCBC to the channel running along the UPRR through the MSP property. This channel outlets to an existing pit on MSP property as proposed by the drainage plan for the Tangerine TI. The portion of the runoff that will be intercepted by interceptor channel A3 will be directly conveyed to the basin on MSP property. Both basins on MSP property and basins south of Tangerine Rd provide 77 acre-ft detention volume (total 154 acre-ft).

Pros

- The detention basin could generate roughly 410,000 (100-year) or 124,000 (10-year) cubic yards of fill dirt for the roadway construction, assuming an average depth of 6 feet (only accounts for basins south of Tangerine Rd).
- This alternative does not increase downstream peak discharges, flow velocities, and flooding depths.
- Utilized the existing pit on MSP property to provide gravity drainage for the proposed drainage system.
- Flooding conditions will be reduced on properties south of Tangerine Road.

Cons/Design issues

- For the 10-year design, roadway subgrade will be subject to 10-year flood and special treatment maybe needed.
- Sediment maintenance for interceptor channels and detention basins.
- Additional costs associate with box culverts under Tangerine Road to convey runoff southerly to the basin.
- Additional costs associated with approximately 6700 feet lined channel along UPRR through MSP.
- Detention basin on MSP property is assumed to be 16.5 feet deep for 100-year design. The existing pit at this location is roughly 12 feet deep. The basin depth could be reduced by increasing the basin area or reduce the required detention volume.

NOTE:
- FLO-2D GRID SIZE-20 FT.
- AERIAL PHOTOGRAPHY:
  PIMA ASSOC. OF GOV'S 2008 DATA.

LEGEND

(magenta line)	0.5' ~ 1.0' FLOW DEPTHS ON TANGERINE RD PAVEMENT
(green line)	1.0' ~ 2.0' FLOW DEPTHS ON TANGERINE RD PAVEMENT
(blue line)	2.0' AND ABOVE FLOW DEPTHS ON TANGERINE RD PAVEMENT

CONCLUSION:
1. EXISTING TANGERINE RD DOES NOT MEET ALL-WEATHER ACCESS CRITERIA (FLOW DEPTHS ON ROADWAY NOT TO EXCEED 1 FT) PER PIMA COUNTY ROADWAY DESIGN MANUAL [THIRD EDITION 2010].
2. ESTIMATED DURATION FOR INUNDATION WITH DEPTHS OF 1.0 FOOT AND ABOVE ON TANGERINE RD IS 5 HOURS DURING 100-YEAR RAINFALL EVENT.

0' 300'
SCALE: 1" = 300'

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PROJECT NO.:	10-027
DESIGN:	JKW
CHECKED:	JKW
DRAWN:	BJK
DATE:	12/04/2012

ALTERNATIVE 1:
NO BUILD/MAINTAIN EXISTING CROSS-DRAINAGE

EXISTING 100-YR FLOW DEPTHS ON TANGERINE RD PAVEMENT IN 20-FT GRID FLO-2D MODEL

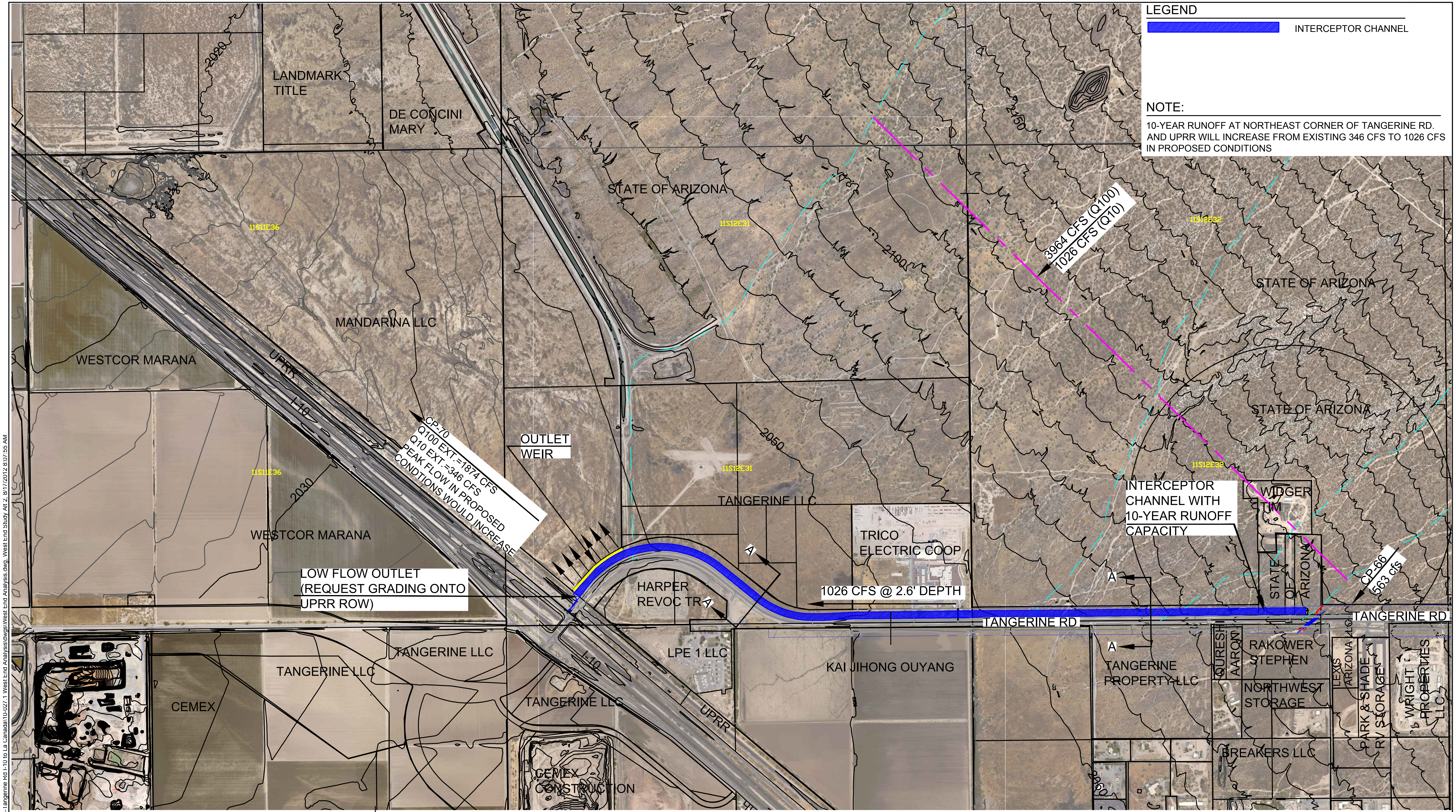
1/1

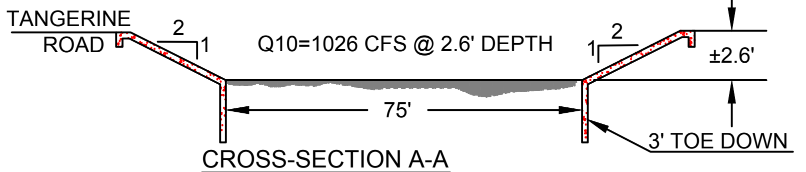
LEGEND

INTERCEPTOR CHANNEL

NOTE:

10-YEAR RUNOFF AT NORTHEAST CORNER OF TANGERINE RD. AND UPRR WILL INCREASE FROM EXISTING 346 CFS TO 1026 CFS IN PROPOSED CONDITIONS





CROSS-SECTION A-A
INTERCEPTOR CHANNEL WITH 10-YEAR RUNOFF CAPACITY
MIN. LONGITUDINAL SLOPE-0.3%
N.T.S.

0' 1000'
SCALE: 1" = 1000'

NOTES:

1. HORIZONTAL PROJECTION: NAD 1983 HARN ,ARIZONA CENTRAL STATE PLANE. TOPOGRAPHY: PIMA ASSOC. OF GOV'S 2000 NAVD88, 10-FT CONTOUR INTERVAL.
2. AERIAL PHOTOGRAPHY: PIMA ASSOC. OF GOV'S 2005 DATA.

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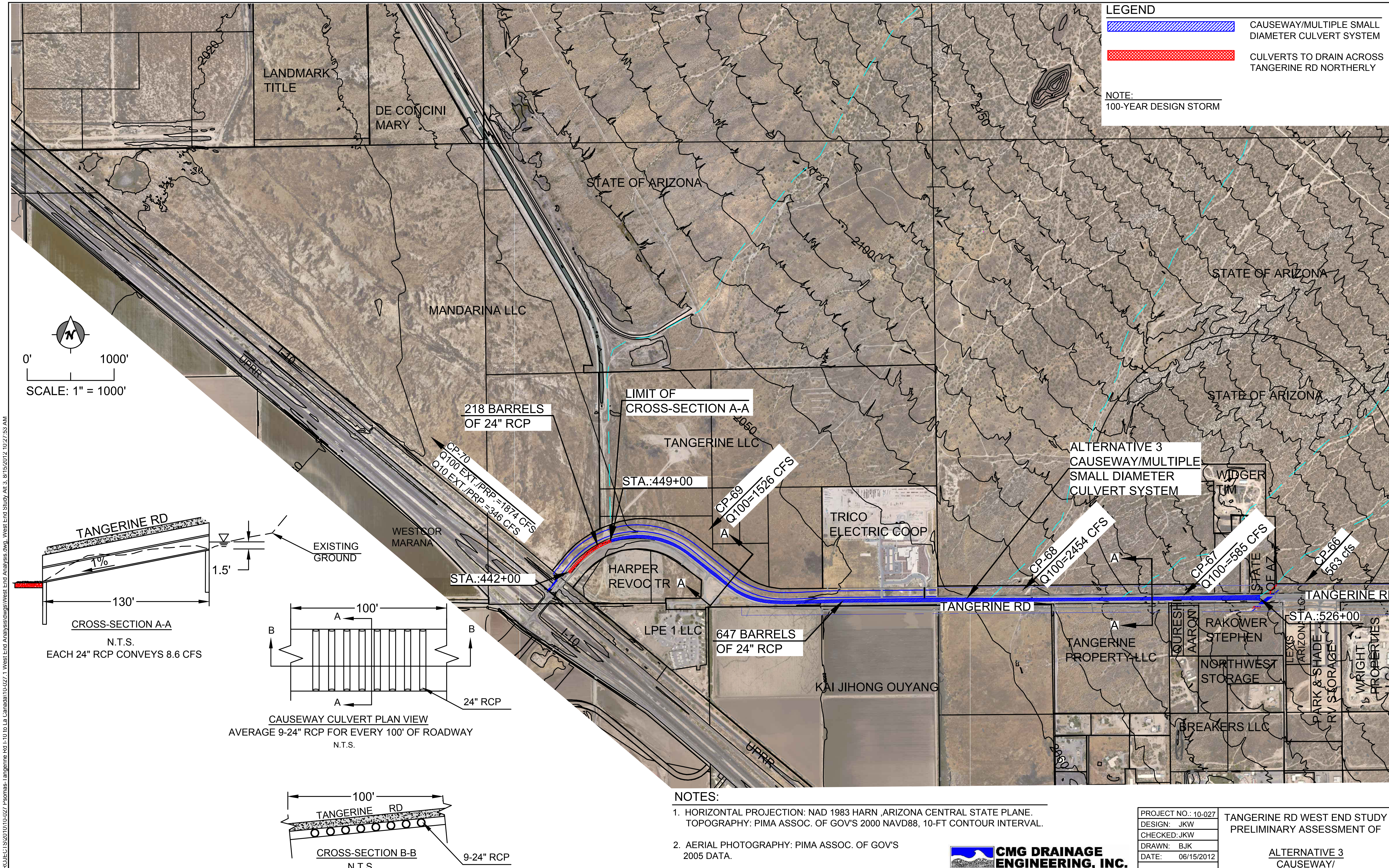
PROJECT NO.:	10-027
DESIGN:	JKW
CHECKED:	JKW
DRAWN:	BJK
DATE:	08/17/2012

TANGERINE RD WEST END STUDY
PRELIMINARY ASSESSMENT OF ALTERNATIVE

ALTERNATIVE 2 - NO-BASIN/ CHANNEL ONLY
1/1



NOTES:

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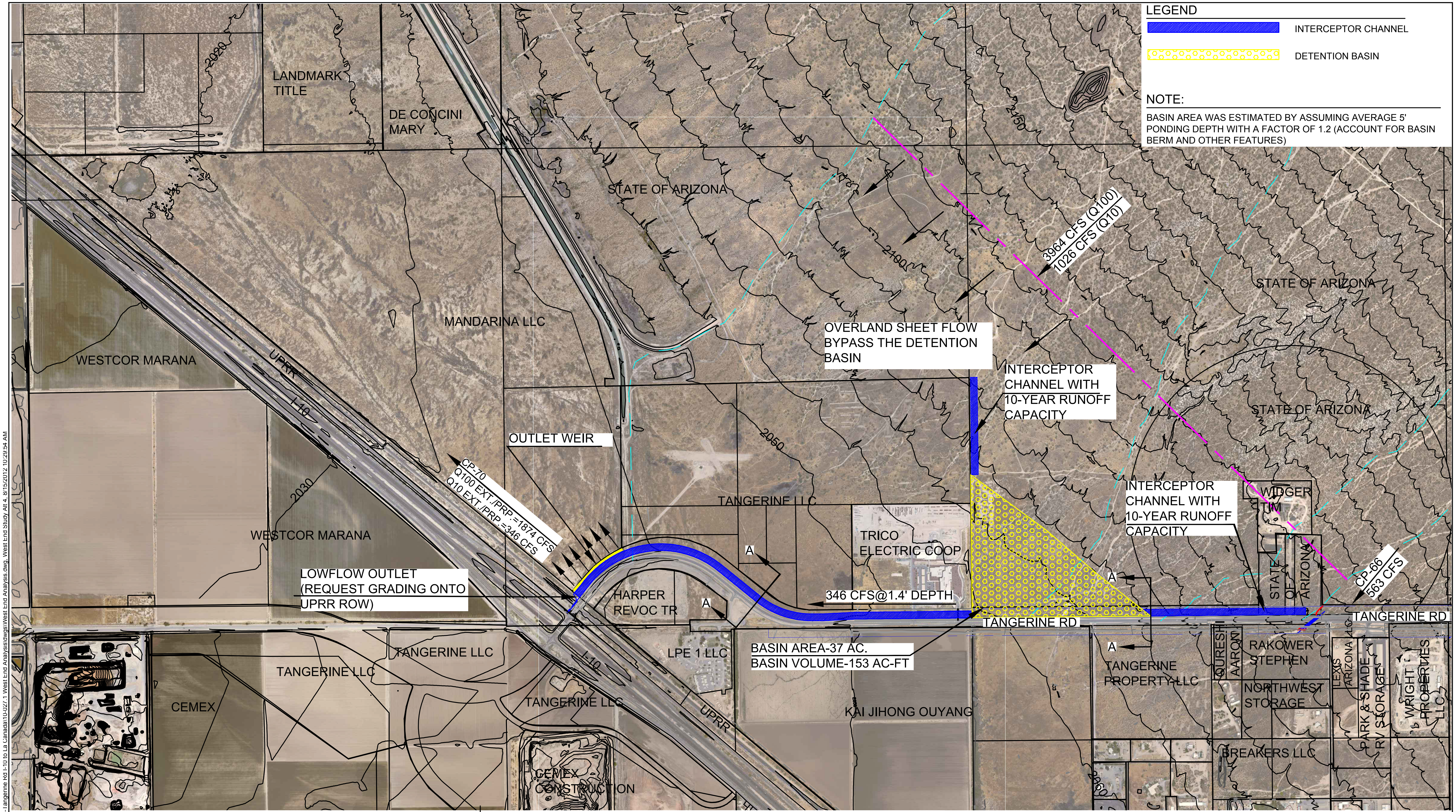
PROJECT NO.:	10-027
DESIGN:	JKW
CHECKED:	JKW
DRAWN:	BJK
DATE:	06/15/2012

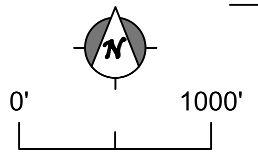
TANGERINE RD WEST END STUDY
PRELIMINARY ASSESSMENT OF

ALTERNATIVE 3
CAUSEWAY/
MULTIPLE SMALL DIAMETER
CULVERT SYSTEM

1/1

Z:\PROJECTS\2010\10-027 Psomas-Tangerine Rd I-10 to La Canada\10-027.1 West End Analysis\dwgs\West End Analysis.dwg, West End Study Alt 3, 8/15/2012 10:27:53 AM





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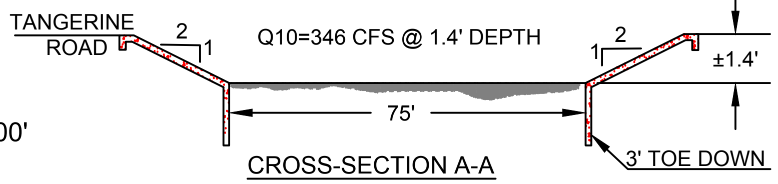

CROSS-SECTION A-A

INTERCEPTOR CHANNEL WITH 10-YEAR RUNOFF CAPACITY

MIN. LONGITUDINAL SLOPE-0.3%

N.T.S.

NOTES:

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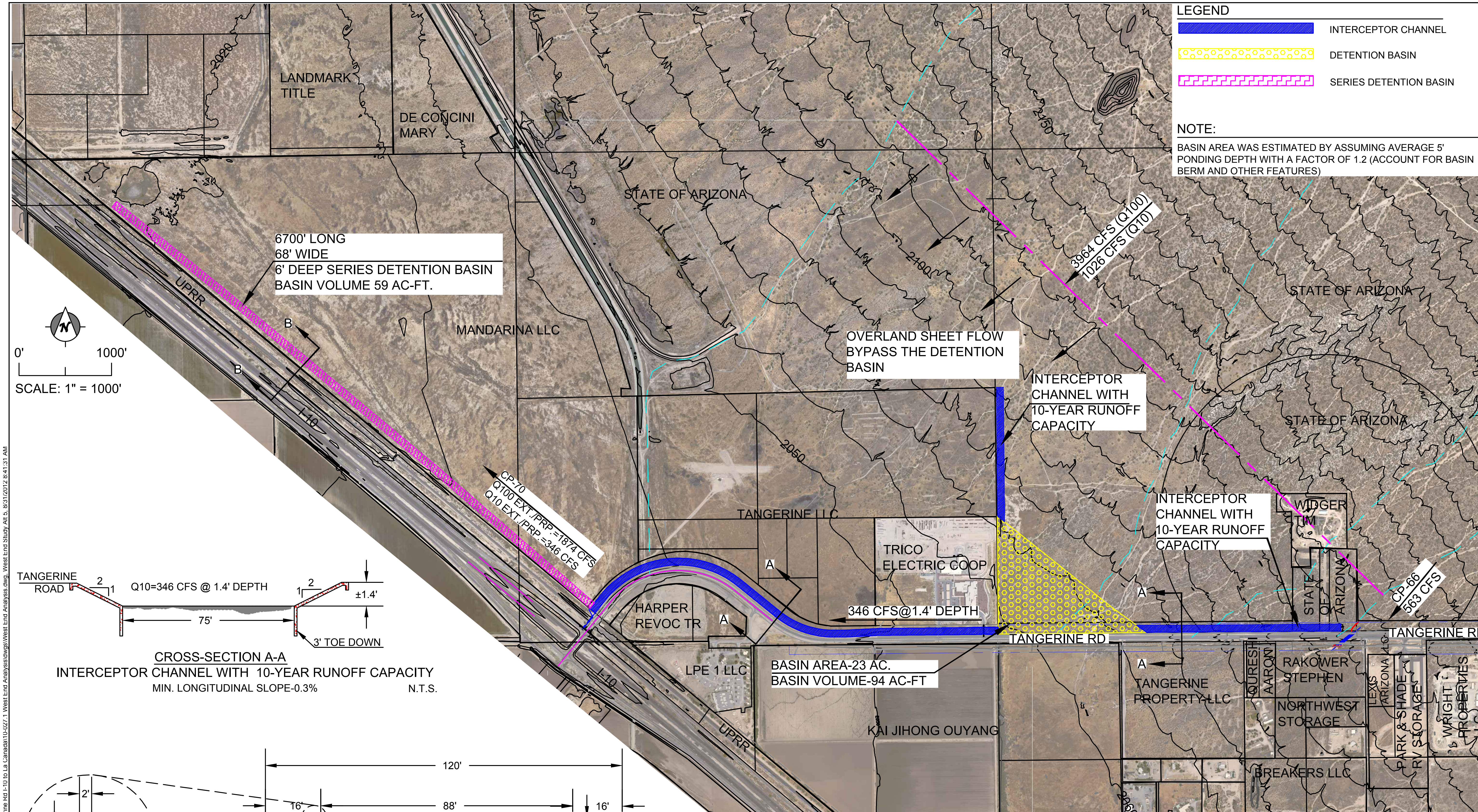
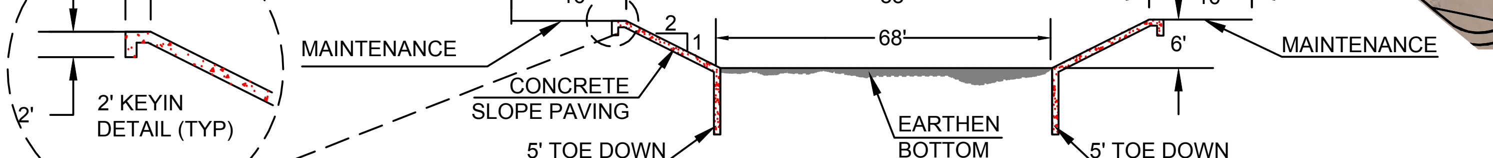
PROJECT NO.:	10-027
DESIGN:	JKW
CHECKED:	JKW
DRAWN:	BJK
DATE:	06/15/2012

TANGERINE RD WEST END STUDY
PRELIMINARY ASSESSMENT OF

ALTERNATIVE 4
DETENTION BASIN EAST OF TRICO PROPERTY



CROSS-SECTION A-A
INTERCEPTOR CHANNEL WITH 10-YEAR RUNOFF CAPACITY
MIN. LONGITUDINAL SLOPE-0.3% N.T.S.



CROSS-SECTION B-B
CONVEYANCE CHANNEL @ MSP PROPERTY

*GRADE CONTROL IS NEEDED EVERY 1500 FEET ALONG THE CHANNEL

NOTES:
1. HORIZONTAL PROJECTION: NAD 1983 HARN ,ARIZONA CENTRAL STATE PLANE. TOPOGRAPHY: PIMA ASSOC. OF GOV'S 2000 NAVD88, 10-FT CONTOUR INTERVAL.
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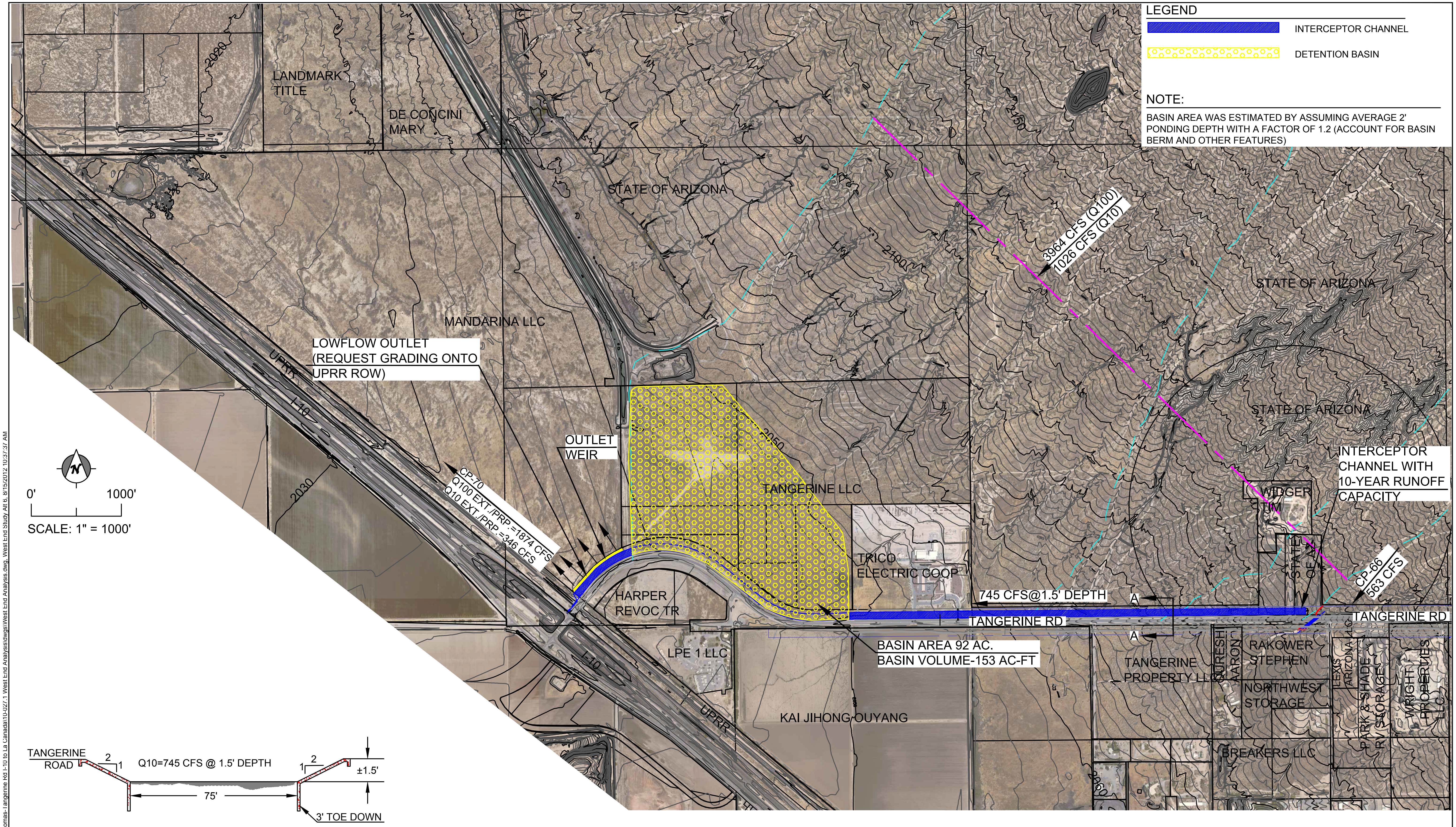
CMG DRAINAGE ENGINEERING, INC.
3555 N. Mountain Ave. Tucson, Arizona 85719
Phone (520) 882-4244 Fax (520) 888-1421

PROJECT NO.:	10-027
DESIGN:	JKW
CHECKED:	JKW
DRAWN:	BJK
DATE:	08/31/2012

TANGERINE RD WEST END STUDY
PRELIMINARY ASSESSMENT OF

ALTERNATIVE 5
COMBINED DETENTION BASINS EAST OF TRICO PROPERTY AND EAST OF UPRR @ MSP PROPERTY



CROSS-SECTION A-A
INTERCEPTOR CHANNEL WITH 10-YEAR RUNOFF CAPACITY
MIN. LONGITUDINAL SLOPE 1% N.T.S.

NOTES:
1. HORIZONTAL PROJECTION: NAD 1983 HARN ,ARIZONA CENTRAL STATE PLANE. TOPOGRAPHY: PIMA ASSOC. OF GOV'S 2000 NAVD88, 2-FT CONTOUR INTERVAL.
2. AERIAL PHOTOGRAPHY: PIMA ASSOC. OF GOV'S 2005 DATA.

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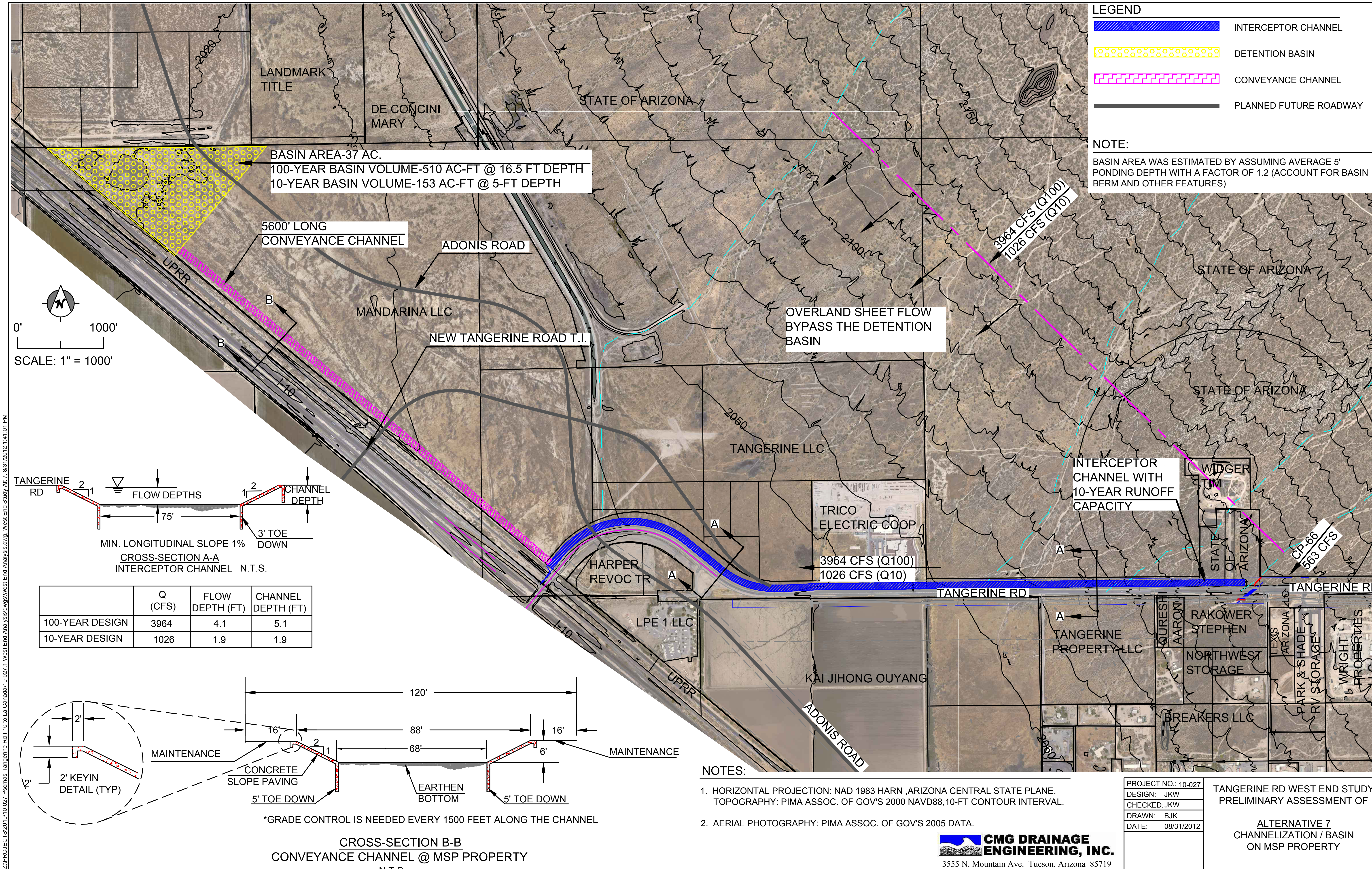
PROJECT NO.:	10-027
DESIGN:	JKW
CHECKED:	JKW
DRAWN:	BJK
DATE:	06/15/2012

TANGERINE RD WEST END STUDY
PRELIMINARY ASSESSMENT OF

ALTERNATIVE 6
DETENTION BASINS
ON TANGERINE INVEST.
PARTNERS LLC. PROPERTY



	Q (CFS)	FLOW DEPTH (FT)	CHANNEL DEPTH (FT)
100-YEAR DESIGN	3964	4.1	5.1
10-YEAR DESIGN	1026	1.9	1.9

NOTES:

1. HORIZONTAL PROJECTION: NAD 1983 HARN ,ARIZONA CENTRAL STATE PLANE. TOPOGRAPHY: PIMA ASSOC. OF GOV'S 2000 NAVD88,10-FT CONTOUR INTERVAL.
2. AERIAL PHOTOGRAPHY: PIMA ASSOC. OF GOV'S 2005 DATA.

CMG DRAINAGE ENGINEERING, INC.
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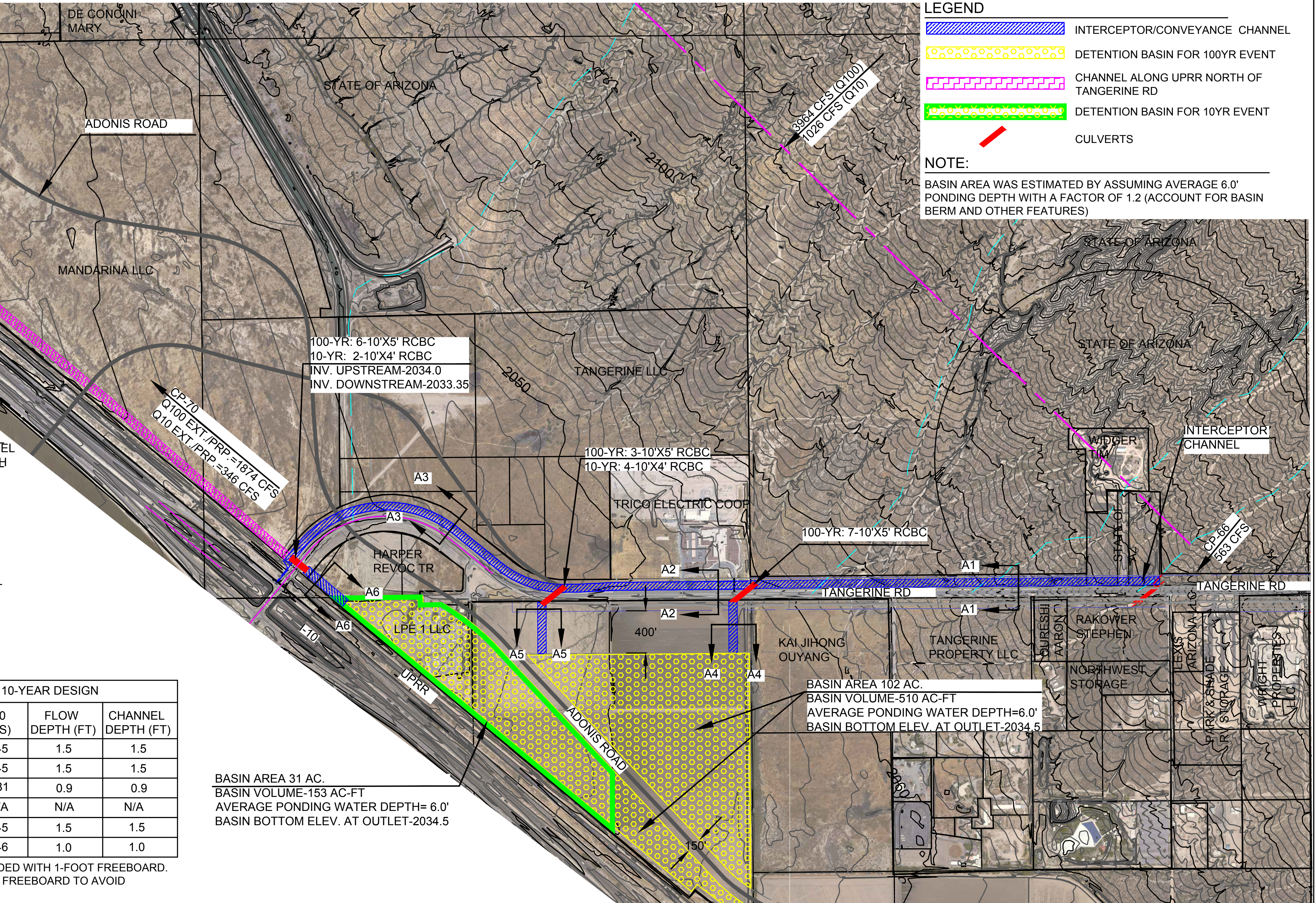
Phone (520) 882-4244 Fax (520) 888-1421

PROJECT NO.:	10-027
DESIGN:	JKW
CHECKED:	JKW
DRAWN:	BJK
DATE:	08/31/2012

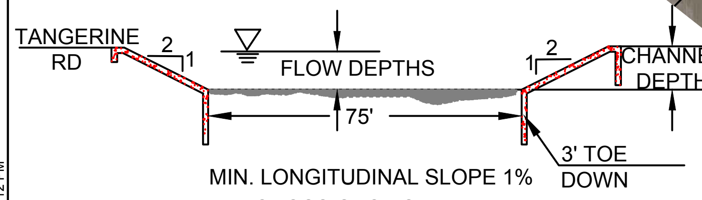
TANGERINE RD WEST END STUDY
PRELIMINARY ASSESSMENT OF

ALTERNATIVE 7
CHANNELIZATION / BASIN
ON MSP PROPERTY



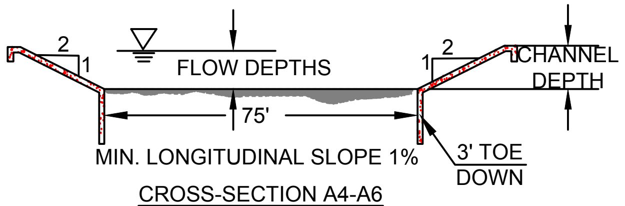
NOTE:

BASIN AREA WAS ESTIMATED BY ASSUMING AVERAGE 6.0' PONDING DEPTH WITH A FACTOR OF 1.2 (ACCOUNT FOR BASIN BERM AND OTHER FEATURES)



MIN. LONGITUDINAL SLOPE 1%

CROSS-SECTION A1-A3
INTERCEPTOR CHANNEL N.T.S.

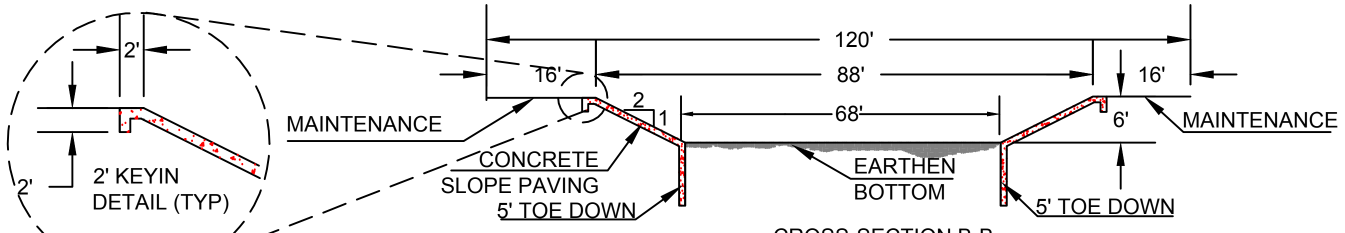


MIN. LONGITUDINAL SLOPE 1%

CROSS-SECTION A4-A6
CONVEYANCE CHANNEL N.T.S.

CROSS-SECTION ID	100-YEAR DESIGN			10-YEAR DESIGN		
	Q100 (CFS)	FLOW DEPTH (FT)	CHANNEL DEPTH (FT)	Q10 (CFS)	FLOW DEPTH (FT)	CHANNEL DEPTH (FT)
A1	2035	2.8	3.8	745	1.5	1.5
A2	843	1.6	2.6	745	1.5	1.5
A3	1086	1.9	2.9	281	0.9	0.9
A4	2035	2.8	3.8	N/A	N/A	N/A
A5	843	1.6	2.6	745	1.5	1.5
A6	1874	2.6	3.6	346	1.0	1.0

* CHANNELS DESIGNED FOR 100-YEAR RAINFALL EVENT ARE PROVIDED WITH 1-FOOT FREEBOARD. CHANNELS DESIGNED FOR 10-YEAR RAINFALL EVENT PROVIDE NO FREEBOARD TO AVOID EXCESSIVE FLOW DEPTH INCREASES OUTSIDE OF RIGHT-OF-WAY.



CROSS-SECTION B-B
CHANNEL ALONG UPRR NORTH OF TANGERINE RD WITH 100-YEAR CAPACITY N.T.S.
MIN. LONGITUDINAL SLOPE-0.3%
*GRADE CONTROL IS NEEDED EVERY 1500 FEET ALONG THE CHANNEL

NOTES:

1. HORIZONTAL PROJECTION: NAD 1983 HARN ,ARIZONA CENTRAL STATE PLANE. TOPOGRAPHY: PIMA ASSOC. OF GOV'S 2000 NAVD88, 2-FT CONTOUR INTERVAL.
2. AERIAL PHOTOGRAPHY: PIMA ASSOC. OF GOV'S 2005 DATA.

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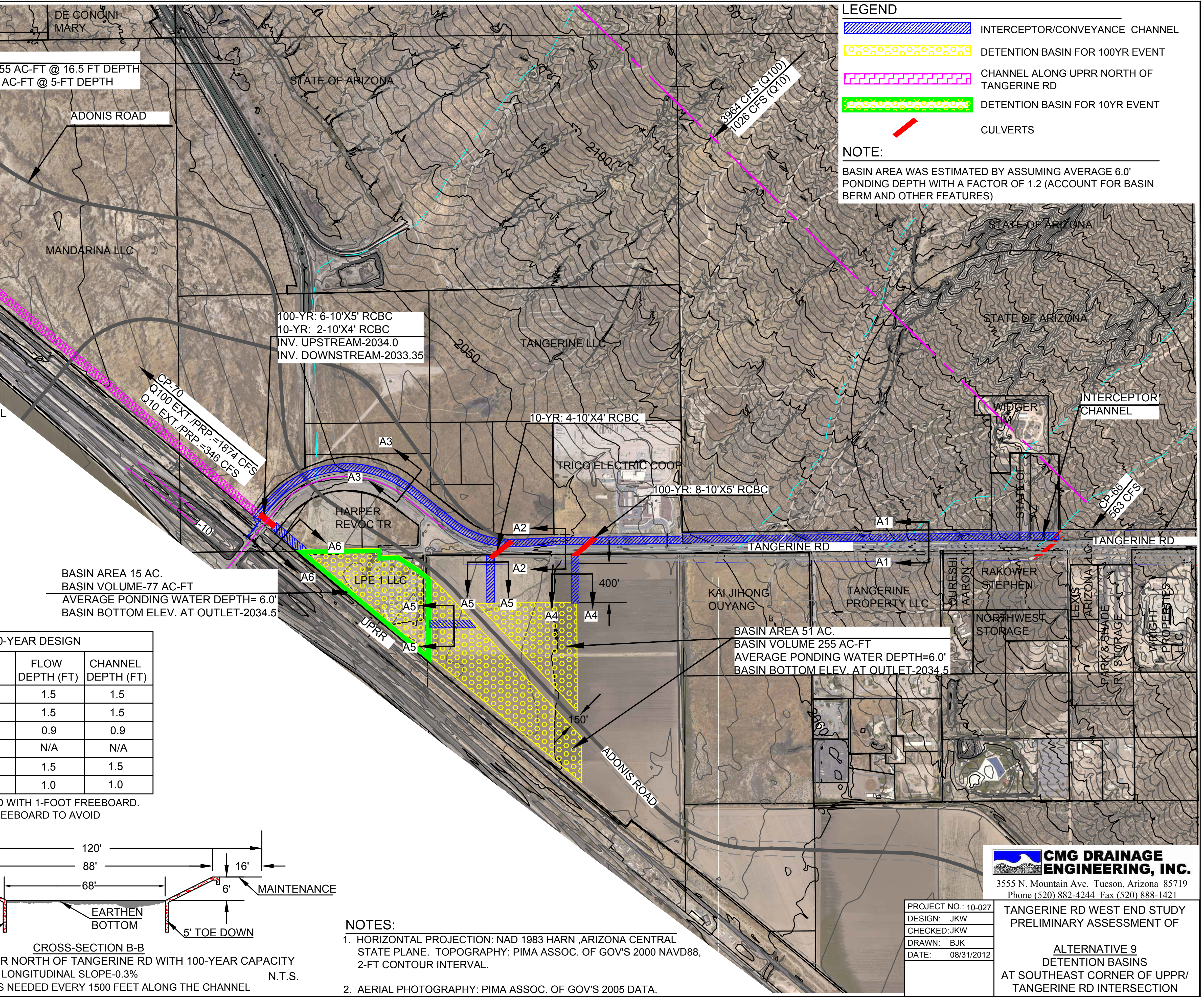
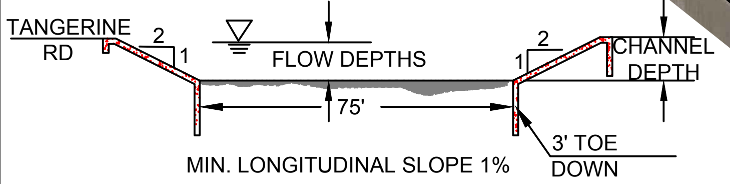
Phone (520) 882-4244 Fax (520) 888-1421

PROJECT NO.:	10-027
DESIGN:	JKW
CHECKED:	JKW
DRAWN:	BJK
DATE:	08/31/2012

TANGERINE RD WEST END STUDY
PRELIMINARY ASSESSMENT OF

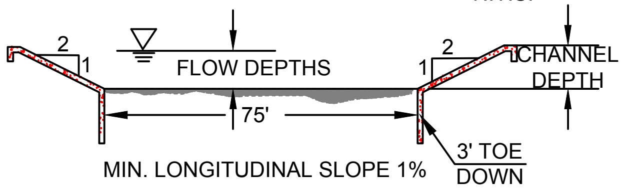
ALTERNATIVE 8
DETENTION BASINS
AT SOUTHEAST CORNER OF UPPR/
TANGERINE RD INTERSECTION





MIN. LONGITUDINAL SLOPE 1%

CROSS-SECTION A1-A3
INTERCEPTOR CHANNEL N.T.S.

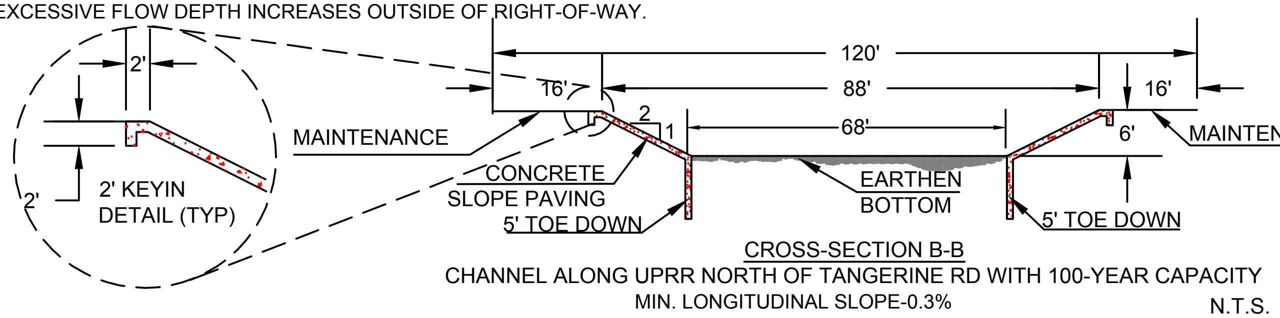


MIN. LONGITUDINAL SLOPE 1%

CROSS-SECTION A4-A6
CONVEYANCE CHANNEL N.T.S.

CROSS-SECTION ID	100-YEAR DESIGN			10-YEAR DESIGN		
	Q100 (CFS)	FLOW DEPTH (FT)	CHANNEL DEPTH (FT)	Q10 (CFS)	FLOW DEPTH (FT)	CHANNEL DEPTH (FT)
A1	2455	3.1	4.1	745	1.5	1.5
A2	1509	2.3	3.3	745	1.5	1.5
A3	1509	2.3	3.3	281	0.9	0.9
A4	2455	3.1	4.1	N/A	N/A	N/A
A5	N/A	N/A	N/A	745	1.5	1.5
A6	1874	2.6	3.6	346	1.0	1.0

* CHANNELS DESIGNED FOR 100-YEAR RAINFALL EVENT ARE PROVIDED WITH 1-FOOT FREEBOARD. CHANNELS DESIGNED FOR 10-YEAR RAINFALL EVENT PROVIDE NO FREEBOARD TO AVOID EXCESSIVE FLOW DEPTH INCREASES OUTSIDE OF RIGHT-OF-WAY.



CROSS-SECTION B-B
CHANNEL ALONG UPRR NORTH OF TANGERINE RD WITH 100-YEAR CAPACITY
MIN. LONGITUDINAL SLOPE-0.3% N.T.S.
*GRADE CONTROL IS NEEDED EVERY 1500 FEET ALONG THE CHANNEL

NOTES:
1. HORIZONTAL PROJECTION: NAD 1983 HARN ,ARIZONA CENTRAL STATE PLANE. TOPOGRAPHY: PIMA ASSOC. OF GOV'S 2000 NAVD88, 2-FT CONTOUR INTERVAL.
2. AERIAL PHOTOGRAPHY: PIMA ASSOC. OF GOV'S 2005 DATA.

CMG DRAINAGE ENGINEERING, INC.
3555 N. Mountain Ave. Tucson, Arizona 85719
Phone (520) 882-4244 Fax (520) 888-1421

PROJECT NO.:	10-027
DESIGN:	JKW
CHECKED:	JKW
DRAWN:	BJK
DATE:	08/31/2012

TANGERINE RD WEST END STUDY
PRELIMINARY ASSESSMENT OF

ALTERNATIVE 9
DETENTION BASINS
AT SOUTHEAST CORNER OF UPPR/
TANGERINE RD INTERSECTION

Z:\PROJECTS\2010\10-027 Psomas-Tangerine Rd I-10 to La Canada\10-027.1 West End Analysis\dwgs\West End Analysis.dwg, West End Study Alt 9, 8/31/2012 1:47:55 PM

Cost Analysis for Alternatives for Tangerine Rd West End Regional Drainage Study

Alternative 2, 10-year				
Item	Unit Price	Unit	Quantity of Unit	Total Item Cost
Tangerine Channel	$233	LF	8,400	$1,957,200
Barricade Railing	$20	LF	8,400	$168,000
Low Flow Outlet Structure	$5,000	/item	1	$5,000
Riprap Apron	$80	Cu Yd	259	$20,720
Channel Excavation	$5	Cu Yd	64,873	$324,364
Borrow Savings	$5	Cu Yd	64,873	($324,364)
			Alternative 2 Total:	$2,150,920

Alternative 3, 100-year				
Item	Unit Price	Unit	Quantity of Unit	Total Item Cost
Fill	$10	Cu Yd	119,467	$1,194,667
24" RCPs	$70	LF	112,450	$7,871,500
Concrete Headwall	$4,000	Pipe End	1,730	$6,920,000
Riprap Apron[4]	$80	Cu Yd	1,922	$153,778
			Alternative 3 Total:	$16,139,944

Alternative 7			10-Year		100-Year	
Item	Unit Price	Unit	Quantity of Unit	Total Item Cost	Quantity of Unit	Total Item Cost
Tangerine Channel	$208	LF	8,400	$1,747,200	-	-
Tangerine Channel	$328	LF	-	-	8400	$2,751,000
Barricade Railing	$20	LF	8,400	$168,000	8400	$168,000
Channel Excavation	$5	Cu Yd	46,580	$232,898	135,184	$675,920
Borrow Savings	$5	Cu Yd	46,580	($232,898)	135,184	($675,920)
Basin Excavation	$5	Cu Yd	246,840	$1,234,200	822,800	$4,114,000
Basin Inlet	$25,000	/item	1	$25,000	1	$25,000
Basin Outlet	$30,000	/item	1	$30,000	1	$30,000
MSP Channel	$372	LF	5,600	$2,083,200	5,600	$2,083,200
Land Acqusition Cost	$27,000	Acre	37	$999,000	37	$999,000
			Total:	$6,286,600		$10,170,200

Alternative 8			10-Year		100-Year	
Item	Unit Price	Unit	Quantity of Unit	Total Item Cost	Quantity of Unit	Total Item Cost
Tangerine Channel[1]	-	-	-	$1,798,801	-	$2,020,852
Barricade Railing	$20	LF	8,400	$168,000	8,400	$168,000
Basin Excavation	$5	Cu Yd	246,840	$1,234,200	822,800	$4,114,000
Double Barrel 10'x4' RCBC	$850	/ft	390	$331,500	-	-
Three Barrel 10'x5' RCBC	$1,300	/ft	-	-	260	$338,000
Four Barrel 10'x5' RCBC	$1,600	/ft	-	-	260	$416,000
Concrete Retaining Wall	$45	S. F.	550	$24,750	825	$37,125
Riprap (Wire-Tied)	$150	Cu Yd	60	$9,000	210	$31,500
Basin Inlet	$25,000	/item	1	$25,000	1	$25,000
Basin Outlet	$30,000	/item	1	$30,000	1	$30,000
MSP Channel	-	-	-	$2,080,908	-	$2,080,908
Land Acqusition Cost	$27,000	Acre	31	$837,000	102	$2,754,000
Borrow savings from excavation	$5	Cu Yd	246,840	($1,234,200)	822,800	($4,114,000)
			Total:	$5,304,959		$7,901,385

Alternative 9			10-Year		100-Year	
Item	Unit Price	Unit	Quantity of Unit	Total Item Cost	Quantity of Unit	Total Item Cost
Tangerine Channel[1]	-	-	-	$1,886,152	-	$2,625,846
Barricade Railing	$20	LF	8,400	$168,000	8,400	$168,000
Basin Excavation	$5	Cu Yd	246,840	$1,234,200	822,800	$4,114,000
Double Barrel 10'x4' RCBC	$850	/ft	390	$331,500	-	-
Three Barrel 10'x5' RCBC	$1,300	/ft	-	-	260	$338,000
Four Barrel 10'x5' RCBC	$1,600	/ft	-	-	260	$416,000
Concrete Retaining Wall	$45	S. F.	550	$24,750	825	$37,125
Riprap (Wire-Tied)	$150	Cu Yd	60	$9,000	210	$31,500
Basin Inlet	$25,000	/item	2	$50,000	2	$50,000
Basin Outlet	$30,000	/item	2	$60,000	2	$60,000
MSP Channel	-	-	-	$2,080,908	-	$2,080,908
Land Acqusition Cost	$27,000	Acre	34	$918,000	70	$1,890,000
Borrow savings from excavation	$5	Cu Yd	123,420	($617,100)	411,400	($2,057,000)
			Total:	$6,145,410		$9,754,380

APPENDIX W-C

EXISTING CONDITIONS FLO-2D MODELING RESULTS

Includes

- HY-8 Computations for existing culverts at Trico east driveway
- Figure C-1 through C-7

HY-8 Culvert Analysis Report

Table 1 - Culvert Summary Table: Trico Sta 487+50

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00	0.00	2056.07	0.000	0.0*	0-NF	0.000	0.000	0.000	0.000	0.000	0.000
80.00	80.00	2057.19	1.120	0.0*	1-S2n	0.701	0.703	0.702	0.922	4.749	3.000
160.00	160.00	2057.85	1.775	0.0*	1-S2n	1.110	1.116	1.115	1.399	5.980	3.891
240.00	240.00	2058.40	2.297	2.332	1-S1t	1.456	1.462	1.775	1.785	5.634	4.515
320.00	320.00	2058.90	2.757	2.825	3-M1t	1.772	1.771	2.111	2.121	6.315	5.008
400.00	400.00	2059.35	3.192	3.276	3-M1t	2.070	2.055	2.415	2.425	6.901	5.421
480.00	480.00	2059.77	3.614	3.698	3-M1t	2.352	2.321	2.695	2.705	7.421	5.779
560.00	560.00	2060.17	4.033	4.098	7-M1t	2.623	2.572	2.956	2.966	7.893	6.097
640.00	640.00	2060.55	4.459	4.478	7-M1t	2.887	2.812	3.203	3.213	8.327	6.383
720.00	720.00	2060.97	4.899	4.843	7-M1t	3.145	3.041	3.436	3.446	8.730	6.643
800.00	800.00	2061.43	5.359	5.195	7-M1t	3.396	3.263	3.660	3.670	9.108	6.884

Site Data - Trico Sta 487+50

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 2056.07 ft

Outlet Station: 54.00 ft

Outlet Elevation: 2055.92 ft

Number of Barrels: 3

Culvert Data Summary - Trico Sta 487+50

Barrel Shape: Concrete Box

Barrel Span: 8.00 ft

Barrel Rise: 4.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: 1:1 Bevel Headwall

Inlet Depression: NONE

Tailwater Channel Data - Trico RCBC

Tailwater Channel Option: Trapezoidal Channel

Bottom Width: 28.00 ft

Side Slope (H:V): 1.00 (_:1)

Channel Slope: 0.0060

Channel Manning's n: 0.0350

Channel Invert Elevation: 2055.91 ft

Roadway Data for Crossing: Trico RCBC

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 40.00 ft

Crest Elevation: 2062.40 ft

Roadway Surface: Paved

Roadway Top Width: 30.00 ft

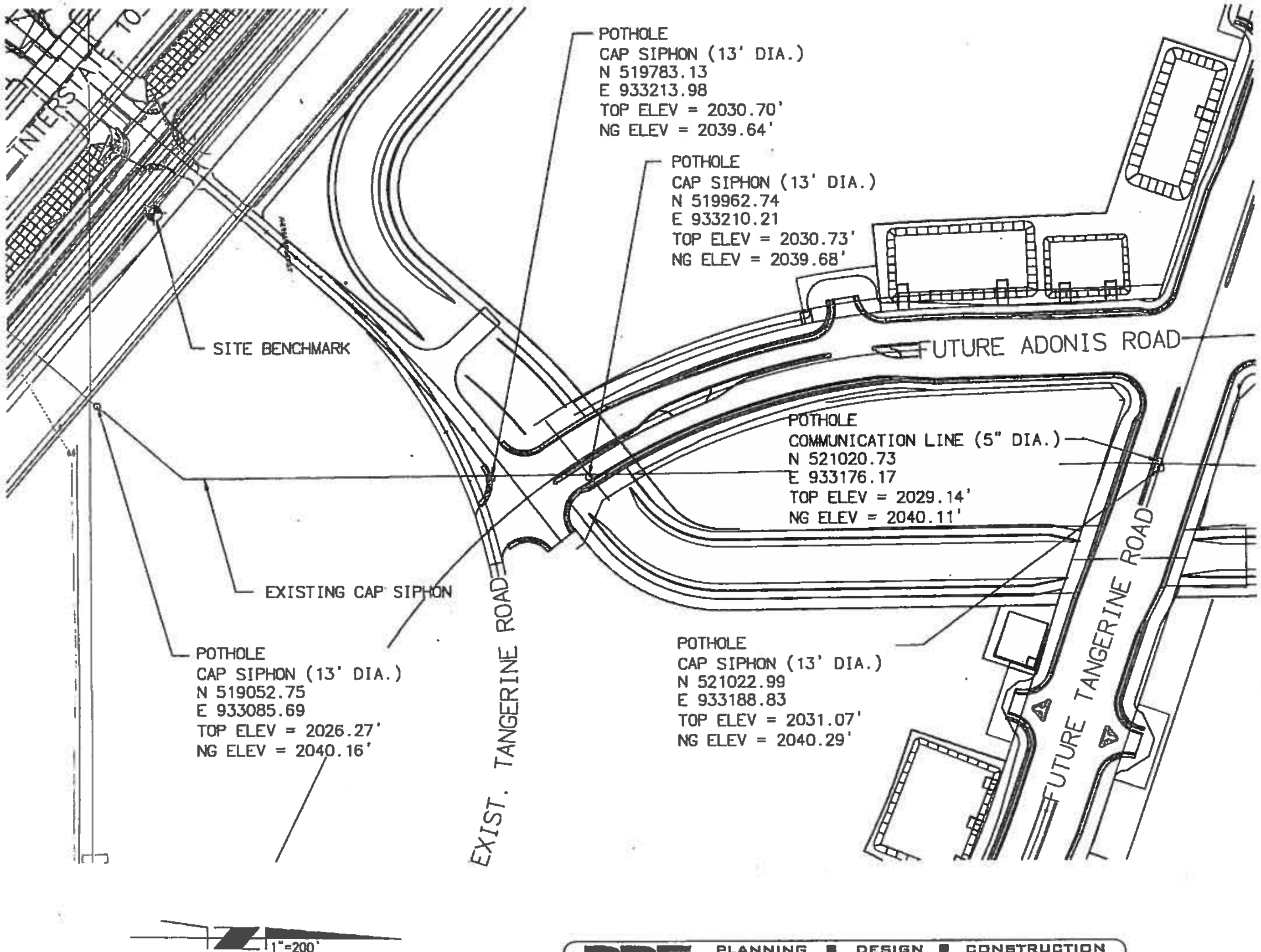
APPENDIX W-D

CHANNEL AND CULVERT PLAN/PROFILES

Includes

- Channel and Culvert Plan/Profiles (8 sheets)
- CAP Pothole Survey Data

CAP SIPHON AND COMMUNICATION LINE POTHOLE EXHIBIT
FOR FUTURE TANGERINE ROAD AND DRAINAGE IMPROVEMENTS AND SEWER ALIGNMENT FEASIBILITY STUDY



SURVEY CONTROL

FIELD MEASURED VALUES WERE LOCATED BY GPS TECHNIQUES AND WERE SCALED HORIZONTALLY BY MULTIPLYING THE GRID COORDINATE VALUES BY A COMBINED SCALE FACTOR OF 1.000133. ALL DISTANCES SHOWN IN THIS DRAWING ARE GROUND VALUES. GPS CONTROL FOR THIS PROJECT UTILIZED NGS MONUMENTS "NN 121", "LITO" AND "STEVE" WITH COORDINATES BASED ON NORTH AMERICAN DATUM 1983 STATE PLANE ARIZONA CENTRAL ZONE.

VERTICAL CONTROL FOR THIS SURVEY IS BASED ON NGS BENCHMARK "386+70", PID CZ0506 WITH AN ELEVATION OF 2034.29', NAVD 88.

SITE BENCHMARK: SET A 3 1/4" ALUMINUM CAP ON A 5/8" REBAR AT THE SE CORNER OF I-10 AND TANGERINE ROAD 25' WEST OF THE WEST RAILROAD RAIL AND 8.3' NORTH OF A METAL BUILDING, HAVING AN ELEVATION OF 2041.44', NAVD 88.

SURVEYOR CERTIFICATION

THE INFORMATION DEPICTED HEREON IS BASED ON FIELD SURVEY COMPLETED IN THE MONTH OF JULY 2009 AND CONDUCTED UNDER MY DIRECT SUPERVISION.

Dan K. Mardock

REGISTERED LAND SURVEYOR
CERTIFICATE NO.
37174
DANIEL K. MARDOCK
Date Signed 7.21.09
ARIZONA, U.S.A.
Expires: 03/31/10

RBF CONSULTING
PLANNING ■ DESIGN ■ CONSTRUCTION
3011 WEST INA ROAD, SUITE 115
TUCSON, ARIZONA 85741-2107
520.797.3235 • FAX 520.797.3236 • www.RBF.com

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APPENDIX W-E

PROPOSED HYDRAULIC/SCOUR/SEDIMENT COMPUTATIONS

Includes

- HY-8 Computations for proposed culverts at Trico east driveway and low flow culvert at the west channel terminus
- End of Channel low flow structure routing computation
- Scour computations
- Riprap rock size determination
- Sediment analyses

HY-8 Culvert Analysis Report

Table 1 - Culvert Summary Table: 5-10'x4' RCBC

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00	0.00	2054.96	0.000	0.0*	0-NF	0.000	0.000	0.000	0.000	0.000	0.000
150.60	150.60	2056.00	1.041	0.0*	1-S2n	0.452	0.657	0.456	0.366	6.606	6.690
301.20	301.20	2056.61	1.649	0.024	1-S2n	0.701	1.043	0.759	0.554	7.940	8.742
451.80	451.80	2057.11	2.145	0.175	1-S2n	0.915	1.367	1.010	0.705	8.950	10.204
602.40	602.40	2057.53	2.572	0.306	1-S2n	1.105	1.655	1.241	0.836	9.705	11.374
753.00	753.00	2057.93	2.973	0.424	1-S2n	1.280	1.921	1.461	0.954	10.308	12.364
903.60	903.60	2058.32	3.359	0.533	1-S2n	1.442	2.169	1.667	1.063	10.839	13.231
1054.20	1054.20	2058.70	3.739	0.634	1-S2n	1.603	2.404	1.867	1.164	11.293	14.004
1204.80	1204.80	2059.08	4.118	0.730	5-S2n	1.749	2.628	2.058	1.260	11.709	14.706
1355.40	1355.40	2059.46	4.504	0.820	5-S2n	1.894	2.843	2.243	1.350	12.088	15.351
1506.00	1506.00	2059.86	4.903	0.906	5-S2n	2.036	3.049	2.423	1.436	12.429	15.949

Site Data - 5-10'x4' RCBC

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 2054.96 ft

Outlet Station: 60.00 ft

Outlet Elevation: 2054.44 ft

Number of Barrels: 5

Culvert Data Summary - 5-10'x4' RCBC

Barrel Shape: Concrete Box

Barrel Span: 10.00 ft

Barrel Rise: 4.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: 1:1 Bevel Headwall

Inlet Depression: NONE

Tailwater Channel Data - Culv @ East Trico Access Rd

Tailwater Channel Option: Trapezoidal Channel

Bottom Width: 60.00 ft

Side Slope (H:V): 4.00 (_:1)

Channel Slope: 0.0500

Channel Manning's n: 0.0250

Channel Invert Elevation: 2054.43 ft

Roadway Data for Crossing: Culv @ East Trico Access Rd

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 20.00 ft

Crest Elevation: 2062.00 ft

Roadway Surface: Paved

Roadway Top Width: 35.00 ft

Table 1 - Culvert Summary Table: Channel Low Flow

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00	0.00	2035.70	0.000	0.0*	0-NF	0.000	0.000	0.000	0.000	0.000	0.000
5.50	5.50	2036.48	0.780	0.0*	1-S2n	0.430	0.574	0.437	0.589	5.498	1.957
11.00	11.00	2036.85	1.145	0.0*	1-S2n	0.617	0.825	0.618	0.840	6.673	2.373
16.50	16.50	2037.15	1.447	0.0*	1-S2n	0.764	1.021	0.770	1.026	7.386	2.647
22.00	22.00	2037.41	1.708	0.0*	1-S2n	0.896	1.188	0.904	1.178	7.973	2.857
27.50	27.50	2037.66	1.961	0.0*	1-S2n	1.022	1.331	1.075	1.310	7.992	3.030
33.00	33.00	2037.93	2.231	0.0*	5-S2n	1.143	1.459	1.201	1.427	8.373	3.177
38.50	38.50	2038.23	2.534	0.0*	5-S2n	1.267	1.575	1.328	1.532	8.702	3.307
44.00	44.00	2038.58	2.877	0.0*	5-S2n	1.392	1.664	1.397	1.630	9.391	3.423
49.50	49.50	2038.97	3.265	0.0*	5-S2n	1.539	1.745	1.580	1.720	9.302	3.528
55.00	51.89	2039.15	3.448	0.0*	5-S2n	1.605	1.780	1.605	1.804	9.596	3.625

Site Data - Channel Low Flow

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 2035.70 ft

Outlet Station: 43.00 ft

Outlet Elevation: 2035.20 ft

Number of Barrels: 2

Culvert Data Summary - Channel Low Flow

Barrel Shape: Circular

Barrel Diameter: 2.00 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Grooved End in Headwall

Inlet Depression: NONE

Tailwater Channel Data - West End Channel Lowflow

Tailwater Channel Option: Trapezoidal Channel

Bottom Width: 3.00 ft

Side Slope (H:V): 3.00 (_:1)

Channel Slope: 0.0050

Channel Manning's n: 0.0300

Channel Invert Elevation: 2035.10 ft

Roadway Data for Crossing: West End Channel Lowflow

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 100.00 ft

Crest Elevation: 2039.10 ft

Roadway Surface: Paved

Roadway Top Width: 20.00 ft

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STAGE-STORAGE-DISCHARGE TABLE

Project: Tangerine Road

Basin: **West End Lowflow Outlet**

For: **By: wjg**

CMG Job #10-027 **Date: 2012.11.1**

Outlet Summary	Q_{max} In (cfs)	Q_{max} Out (cfs)	WSE
2 Year:	-	0.00	2035.74
5 Year:	-	0.00	2035.74
10 Year:	-	0.00	2035.74
100 Year:	0.00	37.03	2038.19

Outlet Type:	User Input	
		0

Stage/Storage Data obtained from HY8 analysis included in Proposed Culverts, West End Channel Lowflow.
Inflow discharge is set to arbitraily high to fill basin.

lese See Stage Discharge Tabl

Elev	Area (ft^2)	Stage(ft)	Incremental Volume (ft^3)	Storage (ac-ft)	Discharge (cfs)	Storage (ft^3)
2035.7	0	0	0	0.000	0.0	0
2036.7	19573	1	9787	0.225	8.1	9787
2037.7	52799	2	36186	1.055	27.3	45973
2038.2	74017	2.5	31704	1.783	38.0	77677

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	Tangerine Road				Q100		
	Routing Table for West End Lowflow Outlet Pipe						
	Time		Inflow	S/dt +0/2	Outflow	Storage	Stage
Time Step	hrs.	min	(cfs)	(cfs)	(cfs)	(ft^3)	(ft)
0	0.00	0	0.00	1306.00	37.03	74029	2.49
1	0.02	1		1268.97	37.03	74029	2.49
2	0.03	2		1231.93	36.12	70487	2.44
3	0.05	3		1195.82	36.12	70487	2.44
4	0.07	4		1159.70	35.20	67051	2.39
5	0.08	5		1124.50	34.28	63722	2.34
6	0.10	6		1090.22	34.28	63722	2.34
7	0.12	7		1055.93	33.37	60498	2.29
8	0.13	8		1022.57	32.39	57381	2.24
9	0.15	9		990.18	32.39	57381	2.24
10	0.17	10		957.79	31.37	54370	2.19
11	0.18	11		926.42	31.37	54370	2.19
12	0.20	12		895.05	30.35	51465	2.15
13	0.22	13		864.70	29.33	48665	2.10
14	0.23	14		835.36	29.33	48665	2.10
15	0.25	15		806.03	28.31	45972	2.05
16	0.27	16		777.71	27.29	43374	2.00
17	0.28	17		750.43	27.29	43374	2.00
18	0.30	18		723.14	26.23	40859	1.95
19	0.32	19		696.91	26.23	40859	1.95
20	0.33	20		670.68	25.17	38426	1.90
21	0.35	21		645.50	24.12	36077	1.85
22	0.37	22		621.39	24.12	36077	1.85
23	0.38	23		597.27	23.06	33811	1.80
24	0.40	24		574.21	22.00	31628	1.75
25	0.42	25		552.21	22.00	31628	1.75
26	0.43	26		530.21	20.94	29528	1.70
27	0.45	27		509.27	20.94	29528	1.70
28	0.47	28		488.33	19.88	27511	1.65
29	0.48	29		468.45	18.83	25577	1.60
30	0.50	30		449.62	18.83	25577	1.60
31	0.52	31		430.79	17.77	23726	1.55
32	0.53	32		413.02	17.77	23726	1.55
33	0.55	33		395.25	16.71	21958	1.50
34	0.57	34		378.54	16.71	21958	1.50
35	0.58	35		361.83	15.77	20274	1.45
36	0.60	36		346.06	15.77	20274	1.45
37	0.62	37		330.30	14.85	18672	1.40
38	0.63	38		315.45	13.93	17154	1.35
39	0.65	39		301.51	13.93	17154	1.35

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Tangerine Road

Routing Table for West End Lowflow Outlet Pipe

Q100

Time Step	Time hrs.	Time min	Inflow (cfs)	S/dt +0/2 (cfs)	Outflow (cfs)	Storage (ft^3)	Stage (ft)
40	0.67	40		287.58	13.02	15718	1.30
41	0.68	41		274.56	13.02	15718	1.30
42	0.70	42		261.55	12.10	14366	1.25
43	0.72	43		249.45	12.10	14366	1.25
44	0.73	44		237.35	11.18	13096	1.20
45	0.75	45		226.16	11.18	13096	1.20
46	0.77	46		214.98	10.39	11910	1.15
47	0.78	47		204.59	10.39	11910	1.15
48	0.80	48		194.20	9.62	10807	1.10
49	0.82	49		184.58	8.86	9786	1.05
50	0.83	50		175.72	8.86	9786	1.05
51	0.85	51		166.86	8.10	8832	1.00
52	0.87	52		158.76	8.10	8832	1.00
53	0.88	53		150.66	7.33	7927	0.95
54	0.90	54		143.33	7.33	7927	0.95
55	0.92	55		135.99	7.33	7927	0.95
56	0.93	56		128.66	6.57	7071	0.90
57	0.95	57		122.09	6.57	7071	0.90
58	0.97	58		115.52	5.81	6263	0.85
59	0.98	59		109.72	5.81	6263	0.85
60	1.00	60		103.91	5.29	5505	0.80
61	1.02	61		98.62	5.29	5505	0.80
62	1.03	62		93.33	4.94	4795	0.75
63	1.05	63		88.40	4.94	4795	0.75
64	1.07	64		83.46	4.94	4795	0.75
65	1.08	65		78.53	4.58	4135	0.70
66	1.10	66		73.94	4.58	4135	0.70
67	1.12	67		69.36	4.23	3523	0.64
68	1.13	68		65.13	4.23	3523	0.64
69	1.15	69		60.90	4.23	3523	0.64
70	1.17	70		56.67	3.88	2960	0.59
71	1.18	71		52.79	3.88	2960	0.59
72	1.20	72		48.91	3.53	2447	0.54
73	1.22	73		45.39	3.53	2447	0.54
74	1.23	74		41.86	3.17	1982	0.49
75	1.25	75		38.69	3.17	1982	0.49
76	1.27	76		35.51	3.17	1982	0.49
77	1.28	77		32.34	2.82	1566	0.44
78	1.30	78		29.52	2.82	1566	0.44
79	1.32	79		26.70	2.47	1199	0.39

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	Tangerine Road Routing Table for West End Lowflow Outlet Pipe				Q100		
	Time		Inflow	S/dt +0/2	Outflow	Storage	Stage
Time Step	hrs.	min	(cfs)	(cfs)	(cfs)	(ft^3)	(ft)
80	1.33	80		24.23	2.47	1199	0.39
81	1.35	81		21.76	2.47	1199	0.39
82	1.37	82		19.30	2.12	881	0.34
83	1.38	83		17.18	2.12	881	0.34
84	1.40	84		15.06	1.76	612	0.29
85	1.42	85		13.30	1.76	612	0.29
86	1.43	86		11.54	1.76	612	0.29
87	1.45	87		9.78	1.41	391	0.24
88	1.47	88		8.37	1.41	391	0.24
89	1.48	89		6.96	1.06	220	0.19
90	1.50	90		5.90	1.06	220	0.19
91	1.52	91		4.84	1.06	220	0.19
92	1.53	92		3.78	0.71	98	0.15

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Design Scour Depth C.O.T. EQTN 6.3

West End Proposed Channel - 448+00 - downstream

Client: PSOMAS
Project #: 10-027

Date 2012.11.29
By: wjg

INPUTS	
General Scour	
Factor of Safety:	1.3
Discharge, Q (cfs):	2200.0
Channel Bottom Width, b (ft):	60
Average Velocity, V_m (fps):	8.21
Max Depth of Flow, Y_{max} (ft):	3.66
Hydraulic Depth of Flow, Y_h (ft):	3.11
Energy Slope, S_e (ft/ft):	0.0030
Top Width, T_w (ft):	86
Long Term Factor of Safety (not reqd):	1.0
Low-Flow Thalweg	
Thalweg Depth Required?:	Yes
Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour	
Bend Angle, α (deg):	0.00
Local Scour due to Pier	
Pier Width (normal to flow), b_p (ft):	
Upstream Froude, F_u:	
Pier Shape	
Pier Shape Reduction Factor	
Local Scour due to Embankments	
Slope Angle of Abutment Face, θ_a (deg):	
Upstream Froude, F_u:	
Upstream Flow Depth, Y_u (ft):	0.00
Encroachment Length, a_e (ft):	
Local Scour below Channel Drops	
Drop Height, h (ft):	1.49
Downstream Depth of Flow, TW (ft):	3.66
Total drop in head, H_T (ft):	0.93

Results	
General Scour	
General Scour, Z_{gs} (ft) [Eq. 6.4] :	1.24
Anti-dune Trough Depth, Z_a (ft) [6.5] :	0.92
Low Flow Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour, Z_{bs} (ft) [Eq. 6.6] :	0.00
local scour:	
Pier Scour Depth, Z_{lsp} (ft) [Eq 6.9] :	0.00
Encroachment Scour Depth, Z_{lse} (ft) [Eq 6.12] :	0.00
Vertical Drop Scour Depth, Z_{lss} (ft) [Eq. 6.14] :	4.72
Calculated Scour Depth, Z_t (ft) [Eq 6.3] :	4.12
Long Term Agg/Deg (ft) [Eq 6.26] :	1.49

Design Scour Depth (ft):	**5.61**

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Design Scour Depth C.O.T. EQTN 6.3

West End Proposed Channel - 448+00 - downstream

Client: PSOMAS
Project #: 10-027

Date 2012.11.29
By: wjg

INPUTS	
Length to Hinge Point, (ft):	750
10-Year Natural Discharge, Q_n (cfs):	1100
10-Year Urbanized Discharge, Q_u (cfs):	1100
Natural Channel Bottom Width, b_n (ft):	30
Urbanized Channel Bottom Width, b_n (ft):	60
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.022
Natural Channel Slope, S_n (ft/ft):	0.004
Reduction Factor for Sediment Supply, R_s:	0.05

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0005
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0020
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0020
Natural Channel Slope * L_h (ft):	3.00
Design Equilibrium Slope * L_h (ft):	1.51
Long Term Aggradation/Degradation (ft):	1.49

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Normal Depth Computation

Proposed West End Channel - 448+00 - downstream

Client: PSOMAS
Project #: 10-027

Date 2012.11.29
By: wjg

Design Depth (ft)	3.6615
Design Q (cfs)	2200
Slope	0.003

Depth	3.66		
Water Surface	3.66		
Q (cfs)	2189.7		
V_{avg} (fps)	8.21		
Channel Bottom	0.00		
Area (ft^2)	266.61		
Top Width (ft^2)	85.6	C.O.T. Freeboard	0.8
Hydraulic Radius	3.1	Froude	0.8

Location: 448+00 to downstrean

x	y	n	Pi	Ai	Ri	Vi	Qi	Twi
0	6							
18	0	0.022	11.58	20.11	1.74	5.36	107.79	10.98
78	0	0.022	60.00	219.69	3.66	8.81	1936.01	60.00
102	6	0.022	15.10	26.81	1.78	5.44	145.88	14.65

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Design Scour Depth C.O.T. EQTN 6.3

West End Proposed Channel - 477+00 - 448+00

Client: PSOMAS
Project #: 10-027
Date 2012.11.29
By: wjg

INPUTS	
General Scour	
Factor of Safety:	1.3
Discharge, Q (cfs):	1900.0
Channel Bottom Width, b (ft):	60
Average Velocity, V_m (fps):	8.58
Max Depth of Flow, Y_{max} (ft):	3.11
Hydraulic Depth of Flow, Y_h (ft):	2.69
Energy Slope, S_e (ft/ft):	0.0040
Top Width, T_w (ft):	82
Long Term Factor of Safety (not reqd):	1.0
Low-Flow Thalweg	
Thalweg Depth Required?:	Yes
Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour	
Bend Angle, α (deg):	0.00
Local Scour due to Pier	
Pier Width (normal to flow), b_p (ft):	
Upstream Froude, F_u:	
Pier Shape	
Pier Shape Reduction Factor	
Local Scour due to Embankments	
Slope Angle of Abutment Face, θ_a (deg):	
Upstream Froude, F_u:	
Upstream Flow Depth, Y_u (ft):	0.00
Encroachment Length, a_e (ft):	
Local Scour below Channel Drops	
Drop Height, h (ft):	2.00
Downstream Depth of Flow, TW (ft):	3.11
Total drop in head, H_T (ft):	1.70

Results	
General Scour	
General Scour, Z_{gs} (ft) [Eq. 6.4] :	1.08
Anti-dune Trough Depth, Z_a (ft) [6.5] :	1.01
Low Flow Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour, Z_{bs} (ft) [Eq. 6.6] :	0.00
local scour:	
Pier Scour Depth, Z_{lsp} (ft) [Eq 6.9] :	0.00
Encroachment Scour Depth, Z_{lse} (ft) [Eq 6.12] :	0.00
Vertical Drop Scour Depth, Z_{lss} (ft) [Eq. 6.14] :	5.48
Calculated Scour Depth, Z_t (ft) [Eq 6.3] :	4.02
Long Term Agg/Deg (ft) [Eq 6.26] :	1.99

Design Scour Depth (ft):	**6.01**

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Design Scour Depth C.O.T. EQTN 6.3

West End Proposed Channel - 477+00 - 448+00

Client: PSOMAS
Project #: 10-027

Date 2012.11.29
By: wjg

INPUTS	
Length to Hinge Point, (ft):	1000
10-Year Natural Discharge, Q_n (cfs):	950
10-Year Urbanized Discharge, Q_u (cfs):	950
Natural Channel Bottom Width, b_n (ft):	30
Urbanized Channel Bottom Width, b_n (ft):	60
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.022
Natural Channel Slope, S_n (ft/ft):	0.004
Reduction Factor for Sediment Supply, R_s:	0.05

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0006
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0020
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0020
Natural Channel Slope * L_h (ft):	4.00
Design Equilibrium Slope * L_h (ft):	2.01
Long Term Aggradation/Degradation (ft):	1.99

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Normal Depth Computation

Proposed West End Channel - 477+00 - 448+00

Client: PSOMAS
Project #: 10-027

Date 2012.11.29
By: wjg

Design Depth (ft)	3.109
Design Q (cfs)	1900
Slope	0.004

Location: 477+00 - 448+00

Depth	3.11		
Water Surface	3.11		
Q (cfs)	1891.4		
V_{avg} (fps)	8.58		
Channel Bottom	0.00		
Area (ft^2)	220.37		
Top Width (ft^2)	81.8	C.O.T. Freeboard	0.7
Hydraulic Radius	2.7	Froude	0.9

x	y	n	Pi	Ai	Ri	Vi	Qi	Twi
0	6							
18	0	0.022	9.83	14.50	1.47	5.55	80.46	9.33
78	0	0.022	60.00	186.54	3.11	9.12	1702.08	60.00
102	6	0.022	12.82	19.33	1.51	5.63	108.90	12.44

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Design Scour Depth C.O.T. EQTN 6.3

West End Proposed Channel - 487+00 - 477+00

Client: PSOMAS
Project #: 10-027

Date 2012.11.29
By: wjg

INPUTS	
General Scour	
Factor of Safety:	1.3
Discharge, Q (cfs):	1900.0
Channel Bottom Width, b (ft):	60
Average Velocity, V_m (fps):	9.23
Max Depth of Flow, Y_{max} (ft):	2.92
Hydraulic Depth of Flow, Y_h (ft):	2.56
Energy Slope, S_e (ft/ft):	0.0050
Top Width, T_w (ft):	80
Long Term Factor of Safety (not reqd):	1.0
Low-Flow Thalweg	
Thalweg Depth Required?:	Yes
Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour	
Bend Angle, α (deg):	0.00
Local Scour due to Pier	
Pier Width (normal to flow), b_p (ft):	
Upstream Froude, F_u:	
Pier Shape	
Pier Shape Reduction Factor	
Local Scour due to Embankments	
Slope Angle of Abutment Face, θ_a (deg):	
Upstream Froude, F_u:	
Upstream Flow Depth, Y_u (ft):	0.00
Encroachment Length, a_e (ft):	
Local Scour below Channel Drops	
Drop Height, h (ft):	1.84
Downstream Depth of Flow, TW (ft):	2.92
Total drop in head, H_T (ft):	1.75

Results	
General Scour	
General Scour, Z_{gs} (ft) [Eq. 6.4] :	1.06
Anti-dune Trough Depth, Z_a (ft) [6.5] :	1.17
Low Flow Thalweg Depth, Z_{lft} (ft):	1.00
Bend Scour, Z_{bs} (ft) [Eq. 6.6] :	0.00
local scour:	
Pier Scour Depth, Z_{lsp} (ft) [Eq 6.9] :	0.00
Encroachment Scour Depth, Z_{lse} (ft) [Eq 6.12] :	0.00
Vertical Drop Scour Depth, Z_{lss} (ft) [Eq. 6.14] :	5.42
Calculated Scour Depth, Z_t (ft) [Eq 6.3] :	4.20
Long Term Agg/Deg (ft) [Eq 6.26] :	1.84

Design Scour Depth (ft):	**6.04**

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Design Scour Depth C.O.T. EQTN 6.3

West End Proposed Channel - 487+00 - 477+00

Client: PSOMAS
Project #: 10-027

Date 2012.11.29
By: wjg

INPUTS	
Length to Hinge Point, (ft):	530
10-Year Natural Discharge, Q_n (cfs):	950
10-Year Urbanized Discharge, Q_u (cfs):	950
Natural Channel Bottom Width, b_n (ft):	30
Urbanized Channel Bottom Width, b_n (ft):	60
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.022
Natural Channel Slope, S_n (ft/ft):	0.007
Reduction Factor for Sediment Supply, R_s:	0.05

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0006
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0035
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0035
Natural Channel Slope * L_h (ft):	3.71
Design Equilibrium Slope * L_h (ft):	1.87
Long Term Aggradation/Degradation (ft):	1.84

CMG DRAINAGE ENGINEERING, INC.
3555 N. Mountain Ave. Tucson, Arizona 85719
Phone (520) 882-4244 Fax (520) 888-1421

Normal Depth Computation

Proposed West End Channel - 487+00 - 477+00

Client: PSOMAS
Project #: 10-027

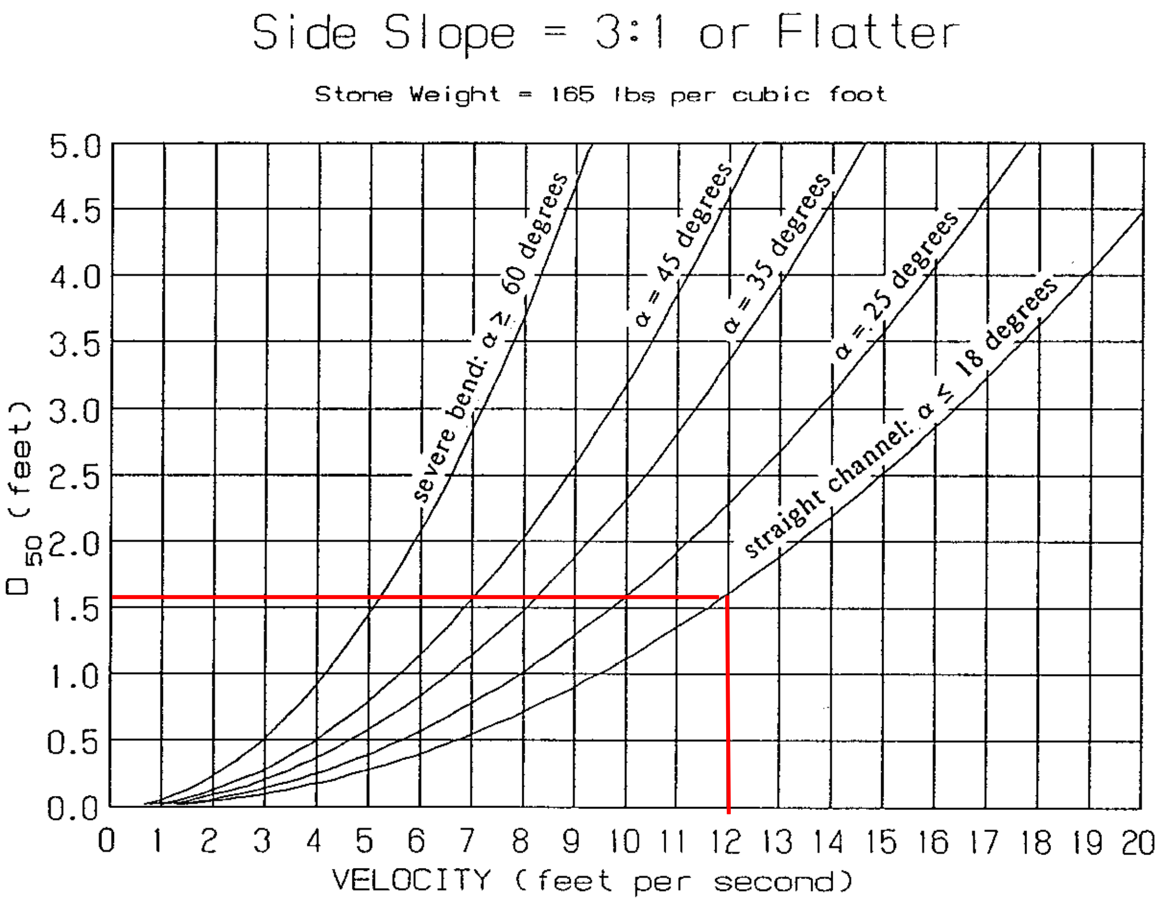
Date 2012.11.29
By: wjg

Design Depth (ft)	2.9185
Design Q (cfs)	1900
Slope	0.005

Depth	2.92		
Water Surface	2.92		
Q (cfs)	1891.5		
V_{avg} (fps)	9.23		
Channel Bottom	0.00		
Area (ft^2)	204.92		
Top Width (ft^2)	80.4	C.O.T. Freeboard	0.7
Hydraulic Radius	2.5	Froude	1.0

Location: 487+00 - 477+00

x	y	n	Pi	Ai	Ri	Vi	Qi	Twi
0	6							
18	0	0.022	9.23	12.78	1.38	5.95	76.00	8.76
78	0	0.022	60.00	175.11	2.92	9.78	1712.64	60.00
102	6	0.022	12.03	17.04	1.42	6.04	102.86	11.67



SOURCE : SIMONS, LI & ASSOCIATES, INC. (1988)

Riprap sizing for West End Channel.
Average velocity range 8 fps - 9 fps.
Conservative upper end velocity of 12 fps.

FIGURE 9.1

RIPRAP DESIGN CHART

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Sediment Delivery Estimate

Tangerine Rd West End Regional Channel

Client: Psomas
Project #: 10-027

Date 12/2/2012
By: Jiankang

Universal Soil Loss Equation:

(Wischmeier and Smith)

A = RKLSCP

A = Computed soil loss per unit area (tons per acre)

R = Rainfall factor

# of erosion index units in a normal year's rain. Erosion index is a measure of the erosive force of a specific rainfall

K = Soil erodability factor

erosion rate/unit of erosion index for a specific soil in cultivated continuous fallow on a 9% slope 72.6 feet long

L = Slope length factor

ratio of soil loss from the filed slope length to that from a 72.6-foot length on the same soil type and gradient

S = Slope gradient factor

ratio of soil loss from the field gradient to that from a 9% slope

C = Cropping management factor

ratio of soil loss from a field with specified cropping and management to that from the fallow condition on which the factor K is evaluated

P = Erosion control management factor

ratio of soil loss with contouring, strip cropping or terracing to that with straight-row farming, up and down slope

R	K	TOPOGRAPHIC FACTOR (LS)	C	P	A (tons/ac/yr)
55	0.18	1.41	0.5	0.5	3.48

WSHED AREA (ACRES)	WSHED SEDIMENT (TONS/YR)
7616.9	26525

*SOIL UNIT WIEGHT (LBS/FT³)	VOLUME (FT³/YR)	AVERAGE ANNUAL SEDIMENT DEPOSIT DEPTHS (FT)**
95	558,415	2.07

* Assumed Sandy Loam

**Assumed sediment will deposit evenly on 4500 feet long 60 feet wide channel

Note: This sediment computation was not used to estimate channel maintenance effort.

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Sediment Delivery Estimate

Tangerine Rd West End Regional Channel

Client: Psomas
Project #: 10-027

Date 12/2/2012
By: Jiankang

Soil Loss Rate(tons/ac/yr)	WSHED AREA (ACRES)	WSHED SEDIMENT (TONS/YR)
0.36	7616.9	2742

*SOIL UNIT WIEGHT (LBS/FT³)	VOLUME (FT³/YR)	AVERAGE ANNUAL SEDIMENT DEPOSIT DEPTHS (FT)**
95	57,728	0.21

* Assumed Sandy Loam

*Assumed sediment will deposit evenly on 4500 feet long 60 feet wide channel

Reference: Proceedings of the Eighth-Federal Interagency Sedimentation Conference, April 2006, Reno, NV, USA, *Alluvial Fan Erosion and Sedimentation Investigations Using the Hydraulic Modeling Tool FLO-2D* , April 2006.

Note: This sediment computation was used to estimate channel maintenance effort.

ALLUVIAL FAN EROSION AND SEDIMENTATION INVESTIGATIONS USING THE HYDRAULIC MODELING TOOL FLO-2D

Joseph Gasperi, Geologist, USDA-Natural Resources Conservation Service, 316 W. Boone Avenue, Spokane, Washington 99201, joe.gasperi@wa.usda.gov; John McClung, Hydraulic Engineer, USDA-Natural Resources Conservation Service, 101 South Main, Temple, Texas 76501, john.mcclung@tx.usda.gov

Abstract: FLO-2D offers a useful planning and evaluation tool for addressing sediment related resource concerns by providing information on the spatial distribution of erosion and deposition of sediment. This poster presents an example of how FLO-2D may be used for watershed scale evaluations of erosion and soil loss on alluvial fans. The model has been applied to four scenarios with different soil types and vegetative cover conditions to represent a range of conditions. Each scenario was evaluated using six different storm runoff events. Two-dimensional plots of the model output identify the spatial distribution of overland flow, maximum flow velocities, scour, and deposition. Processing of the model output permits the development of sediment-frequency curves and the determination of average annual soil loss rates. The soil loss rates have been compared to demonstrate the sensitivity of the watershed to differences in vegetative cover conditions and soil type.

INTRODUCTION

The Natural Resources Conservation Service (NRCS) has a long history of using physical process models to estimate erosion and transport of sediment by wind and water. FLO-2D, developed by James S. O'Brien of FLO-2D Software, Inc., continues this tradition by adding to the options available to NRCS and its partners for evaluating the impact of overland flow on erosion and deposition of sediment on alluvial fans.

FLO-2D is a two-dimensional watershed model with a sediment transport component. Model simulations describe the spatial distribution of erosion and deposition within the modeled area. Processing of the model output provides information on the relative severity of erosion in terms of average annual soil loss. O'Brien (2001) defined the sediment transport component in this way:

> FLO-2D can compute sediment transport in channels, streets and overland flow. A multiple regression sediment transport equation for sand bed channels or alluvial floodplains is used in the model. This empirical equation is a computer generated solution of the Meyer-Peter, Muller bed-load equation applied in conjunction with Einstein's suspended load integration (Zeller and Fullerton, 1983). The bed material discharge, q_s, is calculated in cfs per unit width as follows:
>
> $$q_s = 0.0064\, n^{1.77}\, V^{4.32}\, G^{0.45}\, d^{-0.30}\, D_{50}^{-0.61}$$
>
> where n is Manning's roughness coefficient, V is mean velocity, G is the gradation coefficient, d is the hydraulic depth and D_{50} is the median sediment diameter. All units in this equation are in the ft-lb-sec system except D_{50}, which is in millimeters.

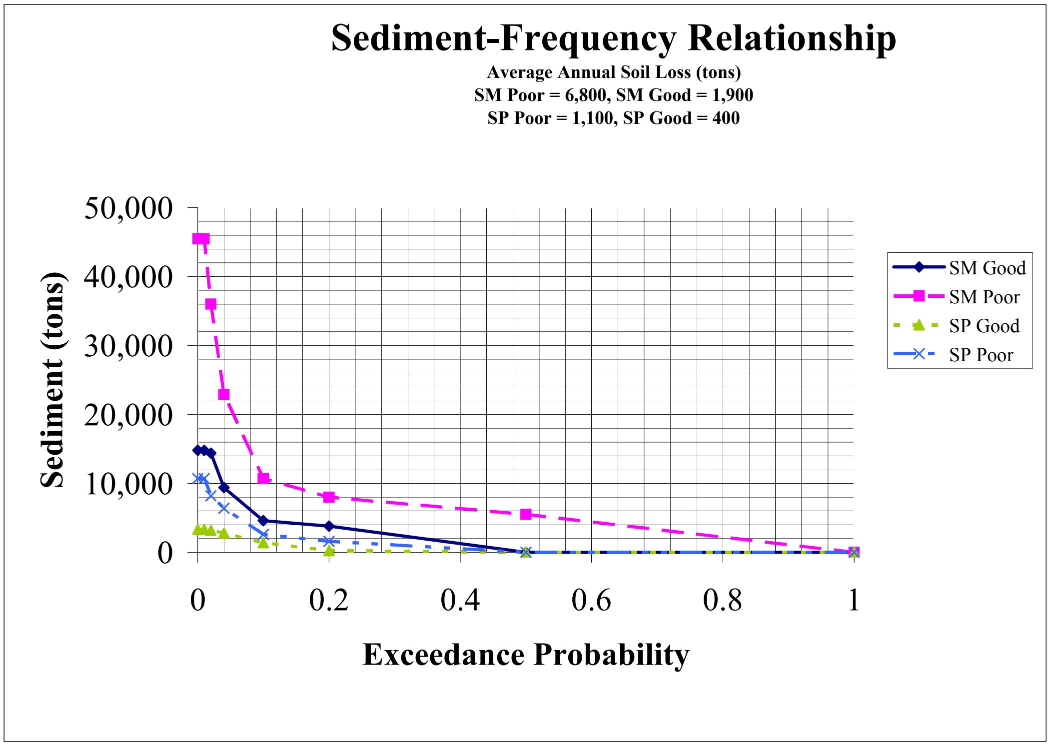

Figure 4 Sediment-frequency curves.

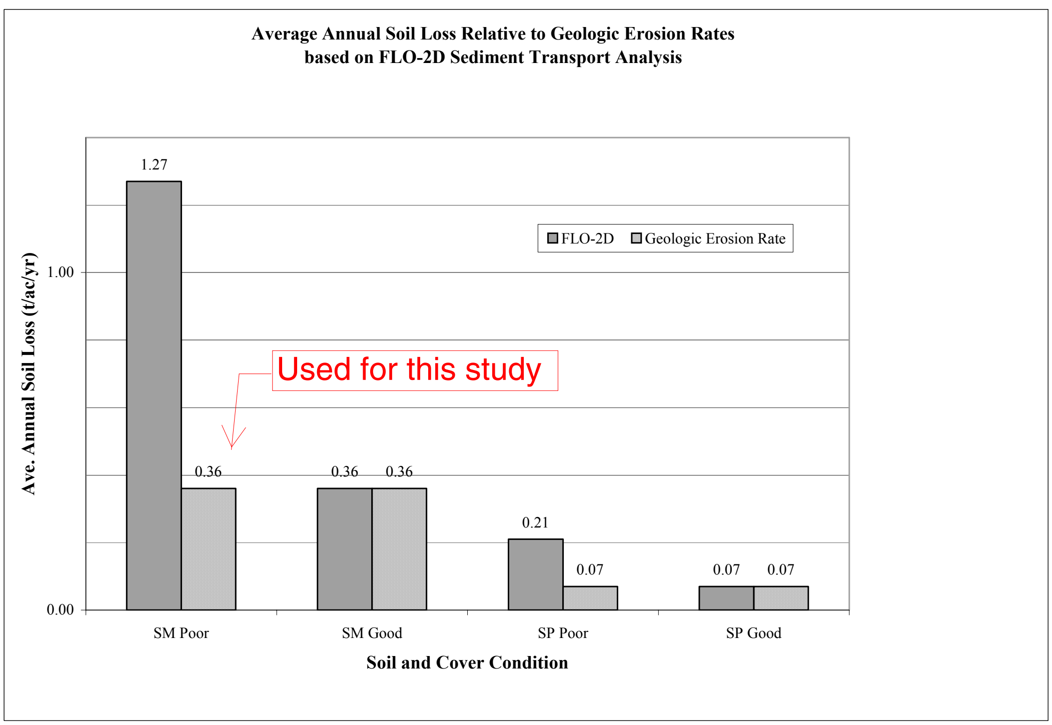

Figure 5 Average annual soil loss vs. geologi rates of erosion.

APPENDIX W-F

PROPOSED CONDITIONS FLO-2D MODELING RESULTS

Includes

- Figure F-1 through F-9
- Cross Section A-A and B-B for the channel along UPRR

Cross-Section A-A [located at about 1250 feet north of Tangerine Rd]