ADDENDUM # 1

CITY OF TORRANCE
3031 Torrance Blvd.
Torrance, CA 90503

BID NO. B2013-40

Bid for 555 Maple Avenue Recreational Sports Field

ADDENDUM # 1- Issued 5-3-13

THE FOLLOWING CHANGES ARE HEREBY INCORPORATED INTO AND MADE A MANDATORY PART OF SUBJECT BID:

The bid opening remains **Thursday, September 19, 2013 by 2:00 PM** in the City Clerk's office.

ATTACHMENTS:

1. Structural Calculations
2. Geotechnical Investigations
3. Standard Urban Stormwater Mitigation Plan (SUSMP)
4. Percolation/Infiltration Testing Report
5. Hydrology Calculations
6. Electrical Plans, revision date 8-29-13, sheets 1 through 4 inclusive
7. Specification Section 03350– Precast Restroom

Please return this addendum with your bid proposal.
I hereby acknowledge receipt of this addendum.

____
Name of Company

____
Address

____
City State Zip Code

One

E S I / F M E
I N C.
S T R U C T U R A L   E N G I N E E R S

PROJECT:	Structural calculations for LAND CONCERN on **"SPORTS FIELD FENCE AND WALLS "** to be built at Torrance, California
	SPORTS FIELD
	555 MAPLE AVE, TORRANCE, CA
	(PER 2010 CBC)

Date: May 13, 2013	**Client:** LAND CONCERN
Revisions:	**Client Job No.**
Shipped:	**Job No.** 0 5 1 3 - C 8 7 9

1800 E. 16th Street, Unit B, Santa Ana, CA 92701 / Tel: (714) 835-2800 / Fax: (714) 835-2819

ESI / FME Inc.
STRUCTURAL ENGINEERS

Client: **Land Concern**

Project Name: Sports Field @ Torrance, CA

Plan #: Sports Field

Soils Information:		Seismic:	SDC D	Wind: 85 MPH / EXPOSURE C			
Soils Report:	Petra	S1=	0.6	Kd =	0.85	Cf =	1.46
P.N.	13-248	R=	1.25	Kzt =	1.00	G =	0.85
Date:	6/3/2013	I =	1	Kz =	0.85	I =	1
f'c =	**2500**	**Type II or V**		V =	85	MPH (3-sec)	
SBP:	2000 psf						
Passive:	150 pcf	**Cs=**	**.8 S1 / (R/I) = 0.384**	**qz = .00256 Kz Kzt Kd V^2 I**			
Active:	40 pcf			**qz =**	**13.36 psf**		
Friction:	0.25						
Sulfates:	negligible			**F = qz G Cf**			
Site Class:	D			**F =**	**16.58 psf**		

STANDARD SPECIFICATIONS FOR STRUCTURAL CALCULATIONS

1. Sketches of details in calculations are not to scale and may not represent true conditions on plans. Architect or designer is responsible for drawing details in plans which represent true framing conditions and scale. Enclosed details are intended to complement standard construction practice to be used by experienced and qualified contractors.

2. The structural calculations included here are for the analysis and design of primary structural system. The attachment of non-structural elements is the responsibility of the architect or designer, unless specifically shown otherwise.

3. The drawings, calculations, specifications and reproductions are instruments of service to be used only for the specific project covered by agreement and cover sheet. Any other use is solely prohibited.

4. All changes made to the subject project shall be submitted to E S I / F M E, Inc. in writing for their review and comment. These calculations are meant to be used by a design professional, omissions are intended.

PROJECT DESCRIPTION:

Job Name	**Sports Field**
City	**Torrance, California**
Client	**Land Concern**

PROJECT ENGINEER:	**R.R.W.**	
CALCS BY:	**T.L.W.**	DATE: **5/13/2013**
ASSOC. CHECK:		DATE:
BACK CHECK:		DATE:
ROOF TRUSS Rev.:		DATE:
FLR. TRUSS Rev.:		DATE:
P/T FOUND. Rev.:		DATE:
PLAN CHECK:		DATE:

REVISIONS:

A	SHTS:	DATE:
		Init.:
B	SHTS:	DATE:
		Init.:
C	SHTS:	DATE:
		Init.:
D	SHTS:	DATE:
		Init.:
E	SHTS:	DATE:
		Init.:
F	SHTS:	DATE:
		Init.:
G	SHTS:	DATE:
		Init.:

E S I / F M E, Inc. - *Structural Engineers*
(This signature is to be a wet signature, not a copy.)
APPROVED BY:

REGISTERED PROFESSIONAL ENGINEER
RAMON R. WONG, II
S2345
STRUCTURAL
STATE OF CALIFORNIA

DATE:

		Page: 3
	ESI / FME Inc. *STRUCTURAL ENGINEERS*	Date: 5/13/2013
		Job #: C879

Client: **LAND CONCERN**

Project Name: **SPORTS FIELD 555 MAPLE AVE. TORRANCE CA** Plan #: **LANDSCAPE**

DESIGN CRITERIA SHEET
FOR RESIDENTIAL CONSTRUCTION

CODE: 2010 CALIFORNIA BUILDING CODE REV. 12/22/2010
In all cases calculations will supersede this design criteria sheet.

TIMBER

Douglas Fir-Larch - 19% max. moisture content

2x4	#2:	Fb =	1315/1552	psi; fv=180 psi; E=1.6	4x6,8	#2/#1:	Fb =	1170/1300	psi; fv=180 psi; E=1.6/1.7
2x6	#2:	Fb =	1170/1345	psi; fv=180 psi; E=1.6	4x10	#2/#1:	Fb =	1080/1200	psi; fv=180 psi; E=1.6/1.7
2x8	#2:	Fb =	1080/1242	psi; fv=180 psi; E=1.6	4x12	#2/#1:	Fb =	990/1100	psi; fv=180 psi; E=1.6/1.7
2x10	#2:	Fb =	990/1138	psi; fv=180 psi; E=1.6	4x14	#2/#1:	Fb =	900/1000	psi; fv=180 psi; E=1.6/1.7
2x12	#2:	Fb =	900/1150	psi; fv=180 psi; E=1.6	4x16	#2/#1:	Fb =	900/1000	psi; fv=180 psi; E=1.6/1.7
2x14	#2:	Fb =	810/931	psi; fv=180 psi; E=1.6	6x10	#1/SS:	Fb =	1350/1600	psi; fv=170 psi; E=1.6
					6x12	#1/SS:	Fb =	1350/1600	psi; fv=170 psi; E=1.6

It is recommended that lumber be free of heart center.
Glued Laminated Beams: Douglas Fir-Larch
Ind. App. Grade: Fb=2400 psi;Fv=240 psi;E=1.8E6 psi

PARALLAM PSL 2.0E
fb=2900.psi; fv=290.psi; E=2.0
MICROLLAM LVL: Fb=2600psi;Fv=285psi;E=1.9

CONCRETE

1. Drypack shall be composed of one part Portland Cement to not more than three parts sand.
2. All structural concrete .... f'c = 3000 psi w/ inspection.
   All slab-on-grade/continuous footings/pads .... f'c = 2500 psi w/o inspection.
   All concrete shall reach minimum compressive strength at 28 days.

REINFORCING STEEL

1. All reinforcing shall be A.S.T.M. A-615-40 for #4 bars and smaller, A-615-60 for #5 bars and larger.
   Welded wire fabric to be A.S.T.M. A-185, lap 1-1/2 spaces, 9" min.
2. Development length of Tension Bars shall be calculated per ACI318-08 Section 12.2.2. . Class B Splice = 1.3 x l_d.
   Splice Lengths for 2500 psi concrete is: #4 Bars (40K) = 21", #5 Bars (60K) = 39", #6 Bars (60K) = 47"
   (30 dia. for compression).
   Masonry reinforcement shall have lappings of 48 dia. or 2'-0". This is in all cases U.N.O.
3. All reinforcing bars shall be accurately and securely placed before pouring concrete, or grouting masonry.
4. Concrete protection for reinforcement shall be at least equal to the diameter of the bars.
   Cover for cast-in-place concrete shall be as follows, U.N.O.:
   A. Concrete cast against & permanently exposed to earth.......................... 3"
   B. Concrete exposed to earth or weather < = #5 Bars.......................... 1 1/2" #6 => #18 Bars 2"
   C. Concrete not exposed to weather or in contact with ground
      Slabs, walls, joists, < = #11 Bars .... 3/4"
      Beams & Columns: Primary reinforcement, ties, stirups, spirals.......................... 1 1/2"

STRUCTURAL STEEL

1. Fabrication and erection of structural steel shall be in accordance with "Specifications for the Design, Fabrication and Erection of Structural Steel Buildings", AISC, current edition. Steel to conform to ASTM A992. Round pipe columns shall conform to ASTM A53, Grade B. Square/Rectangular steel tubes ASTM A500, Grade B.
2. All welding shall be performed by certified welders, using the Electric Shielded Arc Process at licensed shops or otherwise approved by the Bldg. Dept. Continuous inspection required for all field welding.
3. All steel exposed to weather shall be hot-dip galvanized after fabrication, or other approved weatherproofing method.
4. Where finish is attached to structural steel, provide 1/2"ø bolt holes @ 4'-0" o.c. for attachment of nailers, U.N.O. See architectural drawings for finishes (Nelson studs 1/2" x 3" CPL may replace bolts).

MASONRY

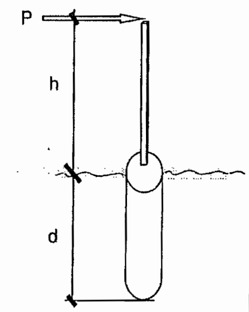
1. Concrete block shall be of sizes shown on architectural drawings and/or called for in specifications and conform to ASTM C-90-09, grade A normal weight units with max. linear shrinkage of 0.06%
2. All vertical reinforcing in masonry walls not retaining earth shall be located in the center of the wall (U.N.O.), retaining walls are to be as shown in details.
3. All cells with steel are to be solid grouted (except retaining walls where all cells are to be solid grouted).

ESI/FME, Inc.
Structural Engineers
1800 E. 16th Street, Unit B
Santa Ana, CA. 92701

Flagpole Footing Design Based on Chapter 18 of IBC & CBC

INPUT DATA & DESIGN SUMMARY

IS FOOTING RESTRAINED @ GRADE LEVEL ? (1=YES,0=NO)		0	no
LATERAL FORCE @ TOP OF POLE	P =	1.44	k
HEIGHT OF POLE ABOVE GRADE	h =	8	ft
DIAMETER OF POLE FOOTING	b =	1.5	ft
LATERAL SOIL BEARING CAPACITY	S =	0.15	ksf / ft
ISOLATED POLE FACTOR (2012 IBC 1806.3.4)	F =	2	
FIRST TRIAL DEPTH ===>	d =	8	ft



Use 1.5 ft dia x 7.08 ft deep footing unrestrained @ ground level

ANALYSIS

LATERAL BEARING @ BOTTOM : $S_3 = FS\ \ Min(d\ \ ,\ 12')$

LATERAL BEARING @ d/3 : $S_1 = FS\ \ Min\left(\frac{d}{3}\ \ ,\ 12'\right)$

$$A = \frac{2.34P}{bS_1}$$

REQUIRD DEPTH :

$$d = \begin{cases} \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] & \text{, FOR NONCONSTRAINED} \\ \sqrt{\frac{4.25Ph}{bS_3}} & \text{, FOR CONSTRAINED} \end{cases}$$

		NONCONSTRAINED		CONSTRAINED	
LATERAL FORCE @ TOP OF POLE	P =>	1.44	k	1.44	k
HEIGHT OF POLE ABOVE GRADE	h =>	8.0	ft	8.0	ft
DIAMETER OF POLE FOOTING	b =>	1.50	ft	1.50	ft
LATERAL SOIL BEARING CAPACITY	FS =>	0.30	ksf / ft	0.30	ksf / ft
1ST TRIAL	TRY d_1 =>	8.00	ft	8.00	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	0.80	ksf	0.80	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	2.40	ksf	2.40	ksf
CONSTANT 2.34P/(bS_1)	A =>	2.81	-		-
REQD FOOTING DEPTH	RQRD d =>	6.55	ft	3.69	ft
2ND TRIAL :	TRY d_2=>	7.27	ft	5.84	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	0.73	ksf	0.58	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	2.18	ksf	1.75	ksf
CONSTANT 2.34P/(bS_1)	A =>	3.09	-		-
REQD FOOTING DEPTH	RQRD d =>	6.96	ft	4.31	ft
3RD TRIAL :	TRY d_3=>	7.12	ft	5.08	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	0.71	ksf	0.51	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	2.13	ksf	1.52	ksf
CONSTANT 2.34P/(bS_1)	A =>	3.16	-		-
REQD FOOTING DEPTH	RQRD d =>	7.06	ft	4.63	ft

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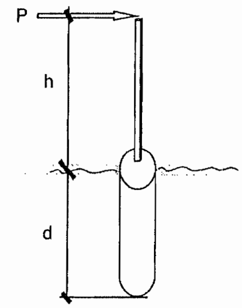
Flagpole Footing Design Based on Chapter 18 of IBC & CBC

INPUT DATA & DESIGN SUMMARY

IS FOOTING RESTRAINED @ GRADE LEVEL ? (1=YES,0=NO)		0	no
LATERAL FORCE @ TOP OF POLE	P =	1.44	k
HEIGHT OF POLE ABOVE GRADE	h =	8	ft
DIAMETER OF POLE FOOTING	b =	1.5	ft
LATERAL SOIL BEARING CAPACITY	S =	0.15	ksf / ft
ISOLATED POLE FACTOR (2012 IBC 1806.3.4)	F =	2	
FIRST TRIAL DEPTH ===>	d =	8	ft



Use 1.5 ft dia x 7.08 ft deep footing unrestrained @ ground level

ANALYSIS

LATERAL BEARING @ BOTTOM : $S_3 = FS \ Min(d \ , \ 12')$

LATERAL BEARING @ d/3 : $S_1 = FS \ Min\left(\frac{d}{3} \ , \ 12'\right)$

$$A = \frac{2.34P}{bS_1}$$

REQUIRD DEPTH :

$$d = \begin{cases} \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] & \text{, FOR NONCONSTRAINED} \\ \sqrt{\frac{4.25Ph}{bS_3}} & \text{, FOR CONSTRAINED} \end{cases}$$

		NONCONSTRAINED		CONSTRAINED	
LATERAL FORCE @ TOP OF POLE	P =>	1.44	k	1.44	k
HEIGHT OF POLE ABOVE GRADE	h =>	8.0	ft	8.0	ft
DIAMETER OF POLE FOOTING	b =>	1.50	ft	1.50	ft
LATERAL SOIL BEARING CAPACITY	FS =>	0.30	ksf / ft	0.30	ksf / ft
1ST TRIAL	TRY d_1 =>	8.00	ft	8.00	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	0.80	ksf	0.80	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	2.40	ksf	2.40	ksf
CONSTANT 2.34P/(bS_1)	A =>	2.81	-		-
REQD FOOTING DEPTH	RQRD d =>	6.55	ft	3.69	ft
2ND TRIAL :	TRY d_2=>	7.27	ft	5.84	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	0.73	ksf	0.58	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	2.18	ksf	1.75	ksf
CONSTANT 2.34P/(bS_1)	A =>	3.09	-		-
REQD FOOTING DEPTH	RQRD d =>	6.96	ft	4.31	ft
3RD TRIAL :	TRY d_3=>	7.12	ft	5.08	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	0.71	ksf	0.51	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	2.13	ksf	1.52	ksf
CONSTANT 2.34P/(bS_1)	A =>	3.16	-		-
REQD FOOTING DEPTH	RQRD d =>	7.06	ft	4.63	ft

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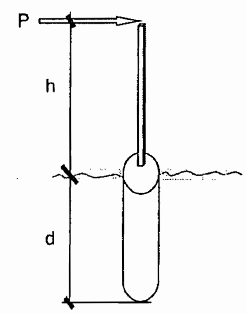
ESI/FME, Inc. *Structural Engineers* 1800 E. 16th Street, Unit B Santa Ana, CA. 92701	PROJECT : Sport Fields @ 555 Maple Avenue, Torrance,CA		PAGE : Detail C-Gate
	CLIENT : City Of Torrance (Land Concern)		DESIGN BY : R.R.W.
	JOB NO. : 513-C879	DATE : May 13th, 2013.	REVIEW BY : R.R.W.

Flagpole Footing Design Based on Chapter 18 of IBC & CBC

INPUT DATA & DESIGN SUMMARY

IS FOOTING RESTRAINED @ GRADE LEVEL ? (1=YES,0=NO)		0	no
LATERAL FORCE @ TOP OF POLE	P =	0.6	k
HEIGHT OF POLE ABOVE GRADE	h =	8	ft
DIAMETER OF POLE FOOTING	b =	12	ft
LATERAL SOIL BEARING CAPACITY	S =	0.15	ksf / ft
ISOLATED POLE FACTOR (2012 IBC 1806.3.4)	F =	2	
FIRST TRIAL DEPTH	===> d =	4	ft



Use 12 ft dia x 2.34 ft deep footing unrestrained @ ground level

ANALYSIS

LATERAL BEARING @ BOTTOM : $S_3 = FS\ Min(d\ ,\ 12')$

LATERAL BEARING @ d/3 : $S_1 = FS\ Min\left(\frac{d}{3}\ ,\ 12'\right)$

$$A = \frac{2.34P}{bS_1}$$

REQUIRD DEPTH :

$$d = \begin{cases} \frac{A}{2}\left[1+\sqrt{1+\frac{4.36h}{A}}\right] & \text{, FOR NONCONSTRAINED} \\ \sqrt{\frac{4.25Ph}{bS_3}} & \text{, FOR CONSTRAINED} \end{cases}$$

		NONCONSTRAINED		CONSTRAINED	
LATERAL FORCE @ TOP OF POLE	P =>	0.60	k	0.60	k
HEIGHT OF POLE ABOVE GRADE	h =>	8.0	ft	8.0	ft
DIAMETER OF POLE FOOTING	b =>	12.00	ft	12.00	ft
LATERAL SOIL BEARING CAPACITY	FS =>	0.30	ksf / ft	0.30	ksf / ft
1ST TRIAL	TRY d_1 =>	4.00	ft	4.00	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	0.40	ksf	0.40	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	1.20	ksf	1.20	ksf
CONSTANT 2.34P/(bS_1)	A =>	0.29	-		-
REQD FOOTING DEPTH	RQRD d =>	1.75	ft	1.19	ft
2ND TRIAL :	TRY d_2=>	2.87	ft	2.60	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	0.29	ksf	0.26	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	0.86	ksf	0.78	ksf
CONSTANT 2.34P/(bS_1)	A =>	0.41	-		-
REQD FOOTING DEPTH	RQRD d =>	2.10	ft	1.48	ft
3RD TRIAL :	TRY d_3=>	2.49	ft	2.04	ft
LAT SOIL BEARING @ 1/3 d	S_1 =>	0.25	ksf	0.20	ksf
LAT SOIL BEARING @ 1.0 d	S_3 =>	0.75	ksf	0.61	ksf
CONSTANT 2.34P/(bS_1)	A =>	0.47	-		-
REQD FOOTING DEPTH	RQRD d =>	2.27	ft	1.67	ft

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ESI/FME, Inc. *Structural Engineers* 1800 E. 16th Street, Unit B Santa Ana, CA. 92701	PROJECT : Sports Field @ 555 Maple Avenue, Torrance, CA CLIENT : City Of Torrance JOB NO. : 513-C879 DATE : May 13th, 2013.	PAGE : Detail D-Pilaster DESIGN BY : RRW REVIEW BY : RRW

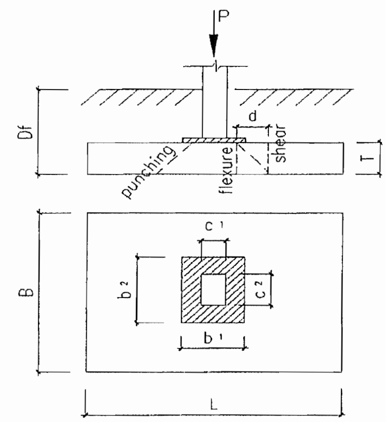
Pad Footing Design Based on ACI 318-11

INPUT DATA

COLUMN WIDTH	c_1	=	24	in
COLUMN DEPTH	c_2	=	24	in
BASE PLATE WIDTH	b_1	=	48	in
BASE PLATE DEPTH	b_2	=	48	in
FOOTING CONCRETE STRENGTH	f_c'	=	2.5	ksi
REBAR YIELD STRESS	f_y	=	40	ksi
AXIAL DEAD LOAD	P_{DL}	=	4.8	k
AXIAL LIVE LOAD	P_{LL}	=	0	k
LATERAL LOAD (0=WIND, 1=SEISMIC)		=	0	Wind,SD
WIND AXIAL LOAD	P_{LAT}	=	0.4	k, SD
SURCHARGE	q_s	=	0	ksf
SOIL WEIGHT	w_s	=	0.11	kcf
FOOTING EMBEDMENT DEPTH	D_f	=	2	ft
FOOTING THICKNESS	T	=	18	in
ALLOW SOIL PRESSURE	Q_a	=	1	ksf
FOOTING WIDTH	B	=	4	ft
FOOTING LENGTH	L	=	4	ft
BOTTOM REINFORCING		#	4	

DESIGN SUMMARY

FOOTING WIDTH		B	=	4.00	ft		
FOOTING LENGTH		L	=	4.00	ft		
FOOTING THICKNESS		T	=	18	in		
LONGITUDINAL REINF.	4	#	4	@	14	in o.c.	
TRANSVERSE REINF.	4	#	4	@	14	in o.c.	



THE PAD DESIGN IS ADEQUATE.

ANALYSIS

DESIGN LOADS (IBC 1605.3.2 & ACI 318 9.2.1)

CASE 1:	DL + LL	P	=	5	kips	1.2 DL + 1.6 LL	Pu	=	6	kips
CASE 2:	DL + LL +0.6(1.3) W	P	=	5	kips	1.2 DL + LL + 1.0 W	Pu	=	6	kips
CASE 3:	DL + LL + 0.6(0.65) W	P	=	5	kips	0.9 DL+ 1.0 W	Pu	=	5	kips

CHECK SOIL BEARING CAPACITY (ACI 318 15.2.2)

$$q_{MAX} = \frac{P}{BL} + q_s + (0.15 - w_s)T =$$

CASE 1	CASE 2	CASE 3
0.36 ksf,	0.38 ksf,	0.37 ksf

q_{MAX} < kQ_a, **[Satisfactory]**

where k = 1 for gravity loads, 4/3 for lateral loads.

DESIGN FOR FLEXURE (ACI 318 15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho = \frac{0.85 f_c'\left(1-\sqrt{1-\frac{M_u}{0.383 b d^2 f_c'}}\right)}{f_y} \qquad \rho_{MAX} = \frac{0.85\beta_1 f_c'}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} \qquad \rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right)$$

	LONGITUDINAL	TRANSVERSE
d	14.75	14.50
b	48	48
$q_{u,max}$	0.39	0.39
M_u	0.19	0.19
ρ	0.000	0.000
ρ_{min}	0.000	0.000
A_s	0.01	0.01
ReqD	1 # 4	1 # 4
Max. Spacing	18 in o.c.	18 in o.c.
USE	4 # 4 @ 14 in o.c.	4 # 4 @ 14 in o.c.
ρ_{max}	0.019	0.019
Check $\rho_{prod} < \rho_{max}$	**[Satisfactory]**	**[Satisfactory]**

(cont'd)

CHECK FLEXURE SHEAR (ACI 318 9.3.2.3, 15.5.2, 11.1.3.1, & 11.2)

$$\phi V_n = 2\phi bd\sqrt{f'_c}$$

	LONGITUDINAL	TRANSVERSE
V_u	-1.12	-1.09
ϕ	0.75	0.75
ϕV_n	53.1	52.2
Check $V_u < \phi V_n$	**[Satisfactory]**	**[Satisfactory]**

CHECK PUNCHING SHEAR (ACI 318 15.5.2, 11.11.1.2, 11.11.6, & 13.5.3.2)

$\phi V_n = (2+y)\phi\sqrt{f'_c}A_p$ = 444.23 kips

where ϕ = 0.75 (ACI 318 9.3.2.3)

β_c = ratio of long side to short side of concentrated load = 1.00

b_0 = $c_1 + c_2 + b_1 + b_2 + 4d$ = 202.5 in

A_p = $b_0\, d$ = 2961.6 in^2

y = MIN(2 , $4/\beta_c$, $40\, d / b_0$) = 2.0

$V_u = P_{u,\max}\left[1 - \frac{1}{BL}\left(\frac{b_1+c_1}{2}+d\right)\left(\frac{b_2+c_2}{2}+d\right)\right] =$ -0.692 kips < ϕV_n **[Satisfactory]**

ESI/FME, Inc.
1800 E. 16th Street, Unit B
Santa Ana, CA 92603
PH: 714-835-2800
FX: 714-935-2819

Title : SPORTS FIELD
Engineer:
Project Desc.: 555 Maple Ave. Torrance, CA 90503

Job # C879

9

Printed: 13 AUG 2013, 11:45AM

Masonry Column

ENERCALC, INC. 1983-2012, Build:6.12.8.2, Ver:6.12.8.2

Lic. # : KW-06000077

Licensee : ESI/FME INC.

Description : 8' Pilaster at Entry Wall

Code References

Calculations per ACI 530-08, IBC 2009, CBC 2010, ASCE 7-05
Load Combinations Used : ASCE 7-05

General Information

Material Properties

F'm	=	1,500.0 psi
Fr - Rupture	=	75.0 psi
Em = f'm *	=	900.0
Column Density	=	130.0 pcf
Rebar Grade	=	Grade 60
Fy - Yield	=	60000 psi
Fs - Allowable	=	24000 psi
E - Rebar	=	29,000.0 ksi
Load Combination	=	ASCE 7-05

Column Data

Column width along X-X	=	23.625 in
Column depth along Y-Y	=	23.625 in
Longitudinal Bar Size	=	# 4.0
Bars per side at +Y & -Y	=	3
Bars per side at +X & -X	=	3
Cover from ties	=	3.50 in
Actual Edge to Bar Center	=	4.125 in

Analysis Settings

Analysis Method	=	**Working Stress Design**
End Fixity Condition	=	Top Pinned, Bottom Pinned
Overall Column Height	=	8.0 ft
Construction Type		Solid Grouted Hollow Concrete Masonry
Tie Bar Size	=	# 3
Tie Bar Spacing	=	8.0 in

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 8 ft, K = 2.1
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 8 ft, K = 2.1

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 4,031.02 lbs * Dead Load Factor
AXIAL LOADS . . .
Signage & Steel: Axial Load at 8.0 ft, D = 0.50 k
BENDING LOADS . . .
Seismic Pilaster: Lat. Point Load at 8.0 ft creating My-y, E = 0.9830 k
Wind Signage: Lat. Point Load at 8.0 ft creating My-y, W = 0.50 k
Wind Pilaster: Lat. Point Load at 8.0 ft creating My-y, W = 0.30 k
Seismic Signage: Lat. Point Load at 8.0 ft creating My-y, E = 0.1920 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS **Maximum Bending Stress Ratio =** **0.020** : 1
Load Combination +D
Location of max.above base 0.000 ft
At maximum location values are . . .
Axial - Applied 4.531 k
Axial - Allowable 223.241 k
Moment - Applied 0.000 k-ft
Moment - Allowable 27.905 k-ft

PASS **Reinforcing Area Check** *(ACI 530-08, Sec 3.3.4.*
As : Actual Reinforcement 1.600
Min: 0.0025 * An 1.395
Max: 0.04 * An 22.326

Dimensional Checks
Min. Width/Depth >= 8" *(ACI 530-08, Sec 3.4.4.*
PASS Overall Height / Min Dim <= 25 *(ACI 530-08, Sec 3.4.4.*

Maximum SERVICE Load Reactions . .
Top along X-X 0.000 k
Bottom along X-X 0.000 k

Maximum SERVICE Load Deflections . . .
Along x-x 0.000 in at 0.000 ft above base
for load combination :

Compressive Strength 223.269 k *(ACI 530-08, Sec 3.3.4.*
Pa = (0.25 f'm An + 0.65 Ast Fs) * [1-(h/(140*r))^2]

PASS **Check Column Ties** *(ACI 530-08, Sec 2.1.6.*
Min. Tie Dia. = 1/4", # 3 bar provided
Max Tie Spacing = 8.00 in, Provided = 8.00 in

Load Combination Results

Load Combination	Maximum Bending Stress Ratios Stress Ratio	Status	Location	Maximum Axial Load Actual	Allow	Maximum Moments Actual	Allow
+D	0.02014	PASS	0.0 ft	4.531 k	223.241 k	0.0 k-ft	27.905 k-ft
+D+W+H	0.02014	PASS	0.0 ft	4.531 k	223.241 k	0.0 k-ft	27.905 k-ft
+D+0.70E+H	0.02014	PASS	0.0 ft	4.531 k	223.241 k	0.0 k-ft	27.905 k-ft
+D+0.750Lr+0.750L+0.750W+H	0.02014	PASS	0.0 ft	4.531 k	223.241 k	0.0 k-ft	27.905 k-ft
+D+0.750L+0.750S+0.750W+H	0.02014	PASS	0.0 ft	4.531 k	223.241 k	0.0 k-ft	27.905 k-ft
+D+0.750Lr+0.750L+0.5250E+H	0.02014	PASS	0.0 ft	4.531 k	223.241 k	0.0 k-ft	27.905 k-ft

ESI/FME, Inc.
1800 E. 16th Street, Unit B
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Title : SPORTS FIELD
Engineer:
Project Desc.: 555 Maple Ave. Torrance, CA 90503

Job # C879

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Masonry Column

ENERCALC, INC. 1983-2012, Build:6.12.8.2, Ver:6.12.8.2

Lic. # : KW-06000077

Licensee : ESI/FME INC.

Description : 8' Pilaster at Entry Wall

Load Combination Results

Load Combination	Maximum Bending Stress Ratios Stress Ratio	Status	Location	Maximum Axial Load Actual	Allow	Maximum Moments Actual	Allow
+D+0.750L+0.750S+0.5250E+H	0.02014	PASS	0.0 ft	4.531 k	223.241 k	0.0 k-ft	27.905 k-ft
+0.60D+W+H	0.01208	PASS	0.0 ft	2.719 k	223.241 k	0.0 k-ft	27.905 k-ft
+0.60D+0.70E+H	0.01208	PASS	0.0 ft	2.719 k	223.241 k	0.0 k-ft	27.905 k-ft

Maximum Reactions - Unfactored

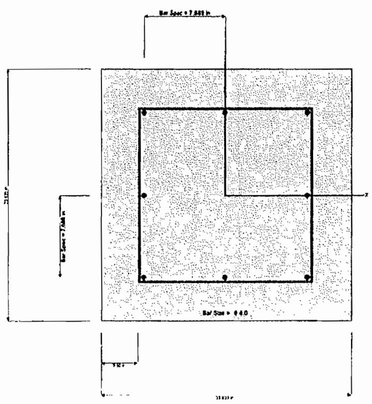
Note: Only non-zero reactions are listed.

Load Combination	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base
D Only	k	k	4.531 k
W Only	k	k	k
E Only	k	k	k
D+W	k	k	4.531 k
D+E	k	k	4.531 k

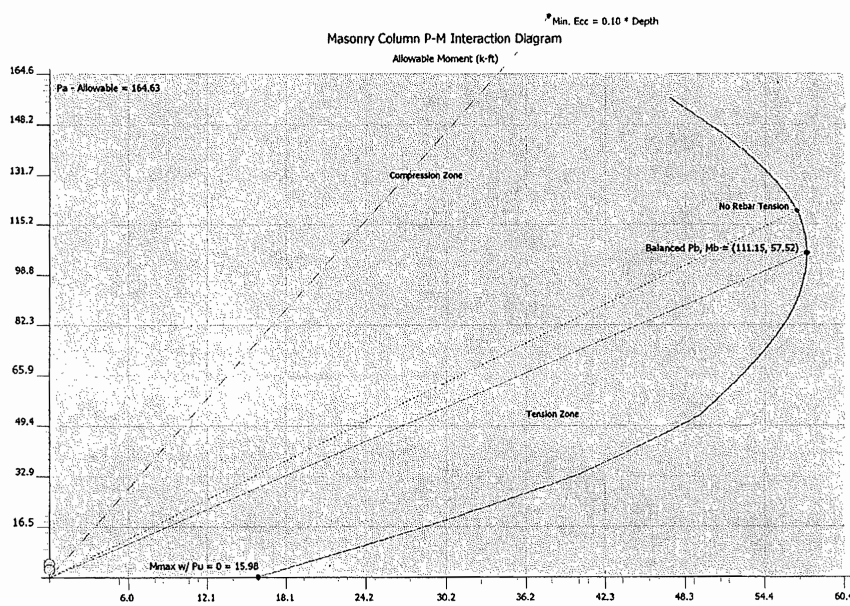
Maximum Deflections for Load Combinations - Unfactored Loads

Load Combination	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft
W Only	0.0000 in	0.000 ft
E Only	0.0000 in	0.000 ft
D+W	0.0000 in	0.000 ft
D+E	0.0000 in	0.000 ft

Cross Section



Interaction Diagram



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Project Desc.: 555 Maple Ave. Torrance, CA 90503

Job # C879 11

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General Footing

File: E:\Files\C879-Sports\Excel Files\calcs.ec6
ENERCALC, INC. 1983-2012, Build:6.12.8.2, Ver:6.12.8.2

Lic. # : KW-06000077 Licensee : ESI/FME INC.

Description : Footing under 8' Pilaster

Code References

Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05
Load Combinations Used : ASCE 7-05

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	0.00140
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	2.0 ksf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	150.0 pcf
Soil/Concrete Friction Coeff.	=	0.250

Increases based on footing Depth

Footing base depth below soil surface	=	2.0 ft
Allowable pressure increase per foot of deptl	=	0.20 ksf
when footing base is below	=	2.0 ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depl	=	0.20 ksf
when maximum length or width is greater	=	2.0 ft

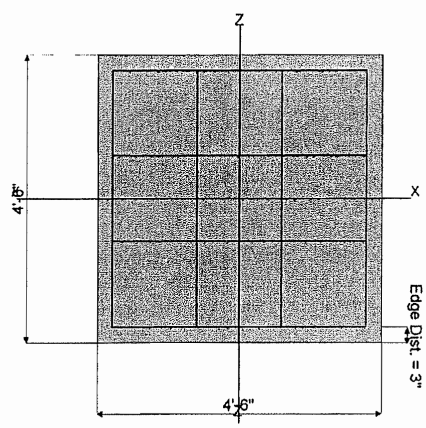
Dimensions

Width parallel to X-X Axis	=	4.50 ft
Length parallel to Z-Z Axis	=	4.50 ft
Footing Thicknes	=	18.0 in

Pedestal dimensions...

px : parallel to X-X Axis	=	in
pz : parallel to Z-Z Axis	=	in
Height	=	in

Rebar Centerline to Edge of Concrete..
at Bottom of footing = 3.0 in



Reinforcing

Bars parallel to X-X Axis

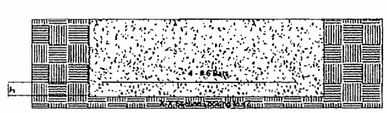
Number of Bars	=	4.0
Reinforcing Bar Size	=	# 6

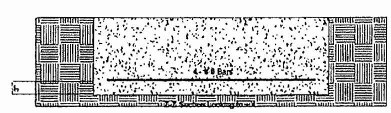
Bars parallel to Z-Z Axis

Number of Bars	=	4.0
Reinforcing Bar Size	=	# 6

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	3.50							k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=					7.20	9.30		k-ft
V-x	=					0.80	1.090		k
V-z	=								k

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Job # C879 12

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General Footing

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ENERCALC, INC. 1983-2012, Build:6.12.8.2, Ver:6.12.8.2

Lic. # : KW-06000077 Licensee : ESI/FME INC.

Description : Footing under 8' Pilaster

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.5748	Soil Bearing	1.437 ksf	2.50 ksf	+0.60D+W+H
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	1.270	Overturning - Z-Z	8.40 k-ft	10.671 k-ft	0.6D+W
PASS	3.064	Sliding - X-X	0.80 k	2.451 k	0.6D+W
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.07519	Z Flexure (+X)	1.934 k-ft	25.725 k-ft	+0.90D+E+1.60H
PASS	0.01926	Z Flexure (-X)	0.4954 k-ft	25.725 k-ft	+0.90D+E+1.60H
PASS	0.02381	X Flexure (+Z)	0.6124 k-ft	25.725 k-ft	+1.40D
PASS	0.01530	X Flexure (-Z)	0.3937 k-ft	25.725 k-ft	+0.90D+E+1.60H
PASS	0.01669	1-way Shear (+X)	1.371 psi	82.158 psi	+1.40D
PASS	0.01669	1-way Shear (-X)	1.371 psi	82.158 psi	+1.40D
PASS	0.01669	1-way Shear (+Z)	1.371 psi	82.158 psi	+1.40D
PASS	0.01669	1-way Shear (-Z)	1.371 psi	82.158 psi	+1.40D
PASS	0.03054	2-way Punching	5.018 psi	164.317 psi	+1.40D

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc	Zecc	Actual Soil Bearing Stress +Z	+Z	-X	-X	Actual / Allowable Ratio
X-X, +D	2.50	n/a	0.0	0.3903	0.3903	n/a	n/a	0.156
X-X, +D+W+H	2.50	n/a	0.0	0.3903	0.3903	n/a	n/a	0.156
X-X, +D+0.70E+H	2.50	n/a	0.0	0.3903	0.3903	n/a	n/a	0.156
X-X, +D+0.750Lr+0.750L+0.750W+H	2.50	n/a	0.0	0.3903	0.3903	n/a	n/a	0.156
X-X, +D+0.750L+0.750S+0.750W+H	2.50	n/a	0.0	0.3903	0.3903	n/a	n/a	0.156
X-X, +D+0.750Lr+0.750L+0.5250E+H	2.50	n/a	0.0	0.3903	0.3903	n/a	n/a	0.156
X-X, +D+0.750L+0.750S+0.5250E+H	2.50	n/a	0.0	0.3903	0.3903	n/a	n/a	0.156
X-X, +0.60D+W+H	2.50	n/a	0.0	0.2342	0.2342	n/a	n/a	0.094
X-X, +0.60D+0.70E+H	2.50	n/a	0.0	0.2342	0.2342	n/a	n/a	0.094
Z-Z, +D	2.50	0.0	n/a	n/a	n/a	0.3903	0.3903	0.156
Z-Z, +D+W+H	2.50	12.752	n/a	n/a	n/a	0.0	0.9780	0.391
Z-Z, +D+0.70E+H	2.50	11.621	n/a	n/a	n/a	0.0	0.9066	0.363
Z-Z, +D+0.750Lr+0.750L+0.750W+H	2.50	9.564	n/a	n/a	n/a	0.0	0.8004	0.320
Z-Z, +D+0.750L+0.750S+0.750W+H	2.50	9.564	n/a	n/a	n/a	0.0	0.8004	0.320
Z-Z, +D+0.750Lr+0.750L+0.5250E+H	2.50	8.715	n/a	n/a	n/a	0.01738	0.7633	0.305
Z-Z, +D+0.750L+0.750S+0.5250E+H	2.50	8.715	n/a	n/a	n/a	0.01738	0.7633	0.305
Z-Z, +0.60D+W+H	2.50	21.254	n/a	n/a	n/a	0.0	1.437	0.575
Z-Z, +0.60D+0.70E+H	2.50	19.368	n/a	n/a	n/a	0.0	1.087	0.435

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, D	None	0.0 k-ft	Infinity	OK
X-X, 0.6D+W	None	0.0 k-ft	Infinity	OK
X-X, 0.6D+0.7E	None	0.0 k-ft	Infinity	OK
Z-Z, D	None	0.0 k-ft	Infinity	OK
Z-Z, 0.6D+W	8.40 k-ft	10.671 k-ft	1.270	OK
Z-Z, 0.6D+0.7E	7.655 k-ft	10.671 k-ft	1.394	OK

Sliding Stability

All units k

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio	Status
X-X, D	0.0 k	3.242 k	No Sliding	OK
X-X, 0.6D+W	0.80 k	2.451 k	3.064	OK
X-X, 0.6D+0.7E	0.7630 k	2.451 k	3.213	OK
Z-Z, D	0.0 k	3.242 k	No Sliding	OK
Z-Z, 0.6D+W	0.0 k	2.451 k	No Sliding	OK
Z-Z, 0.6D+0.7E	0.0 k	2.451 k	No Sliding	OK

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Job # C879

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ENERCALC, INC. 1983-2012, Build:6.12.8.2, Ver:6.12.8.2

General Footing

Lic. # : KW-06000077 — Licensee : ESI/FME INC.

Description : Footing under 8' Pilaster

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Which Side ?	Tension @ Bot. or Top ?	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	6123911111111	+Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +1.40D	6123911111111	-Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +1.20D+1.60Lr+0.80W	5249066666666	+Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +1.20D+1.60Lr+0.80W	5249066666666	-Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +1.20D+1.60S+0.80W	5249066666666	+Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +1.20D+1.60S+0.80W	5249066666666	-Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +1.20D+0.50Lr+0.50L+1.60W	5249066666666	+Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +1.20D+0.50Lr+0.50L+1.60W	5249066666666	-Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +1.20D+0.50L+0.50S+1.60W	5249066666666	+Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +1.20D+0.50L+0.50S+1.60W	5249066666666	-Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +1.20D+0.50L+0.20S+E	5249066666666	+Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +1.20D+0.50L+0.20S+E	5249066666666	-Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +0.90D+1.60W+1.60H	3936799999999	+Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +0.90D+1.60W+1.60H	3936799999999	-Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +0.90D+E+1.60H	3936799999999	+Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
X-X, +0.90D+E+1.60H	3936799999999	-Z	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +1.40D	6123911111111	-X	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +1.40D	6123911111111	+X	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +1.20D+1.60Lr+0.80W	2216272592592	-X	Top	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +1.20D+1.60Lr+0.80W	2714405925925	+X	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +1.20D+1.60S+0.80W	2216272592592	-X	Top	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +1.20D+1.60S+0.80W	2714405925925	+X	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +1.20D+0.50Lr+0.50L+1.60W	6588576333500	-X	Top	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +1.20D+0.50Lr+0.50L+1.60W	3273372224223	+X	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +1.20D+0.50L+0.50S+1.60W	6588576333500	-X	Top	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +1.20D+0.50L+0.50S+1.60W	3273372224223	+X	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +1.20D+0.50L+0.20S+E	5873515798322	-X	Top	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +1.20D+0.50L+0.20S+E	8422780053958	+X	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +0.90D+1.60W+1.60H	4954040999999	-X	Top	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +0.90D+1.60W+1.60H	4903799465750	+X	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +0.90D+E+1.60H	4954040999999	-X	Top	0.3888	Min Temp %	0.391111111111125.725141612200		OK
Z-Z, +0.90D+E+1.60H	9341326171268	+X	Bottom	0.3888	Min Temp %	0.391111111111125.725141612200		OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	1.371 psi	1.371 psi	1.371 psi	1.371 psi	1.371 psi	82.158 psi	0.01669	OK
+1.20D+1.60Lr+0.80W	1.175 psi	1.175 psi	1.175 psi	1.175 psi	1.175 psi	82.158 psi	0.01431	OK
+1.20D+1.60S+0.80W	1.175 psi	1.175 psi	1.175 psi	1.175 psi	1.175 psi	82.158 psi	0.01431	OK
+1.20D+0.50Lr+0.50L+1.60W	1.175 psi	1.175 psi	1.175 psi	1.175 psi	1.175 psi	82.158 psi	0.01431	OK
+1.20D+0.50L+0.50S+1.60W	1.175 psi	1.175 psi	1.175 psi	1.175 psi	1.175 psi	82.158 psi	0.01431	OK
+1.20D+0.50L+0.20S+E	1.175 psi	1.175 psi	1.175 psi	1.175 psi	1.175 psi	82.158 psi	0.01431	OK
+0.90D+1.60W+1.60H	0.8815 psi	0.8815 psi	0.8815 psi	0.8815 psi	0.8815 psi	82.158 psi	0.01073	OK
+0.90D+E+1.60H	0.8813 psi	0.8813 psi	0.8815 psi	0.8815 psi	0.8815 psi	82.158 psi	0.01073	OK

Punching Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	5.018 psi	164.317 psi	0.03054	OK
+1.20D+1.60Lr+0.80W	4.301 psi	164.317 psi	0.02617	OK
+1.20D+1.60S+0.80W	4.301 psi	164.317 psi	0.02617	OK
+1.20D+0.50Lr+0.50L+1.60W	4.761 psi	164.317 psi	0.02898	OK
+1.20D+0.50L+0.50S+1.60W	4.761 psi	164.317 psi	0.02898	OK
+1.20D+0.50L+0.20S+E	4.412 psi	164.317 psi	0.02685	OK
+0.90D+1.60W+1.60H	3.845 psi	164.317 psi	0.0234	OK
+0.90D+E+1.60H	3.715 psi	164.317 psi	0.02261	OK

ESI/FME, Inc.
1800 E. 16th Street, Unit B
Santa Ana, CA 92603
PH: 714-835-2800
FX: 714-935-2819

Title : SPORTS FIELD
Engineer:
Project Desc.: 555 Maple Ave. Torrance, CA 90503

Job # C879 14

Printed: 13 AUG 2013, 2:05PM

Masonry Lintel

File: E:\Files\C879-Sports\Excel Files\calcs.ec6
ENERCALC, INC. 1983-2012, Build:6.12.8.2, Ver:6.12.8.2

Lic. # : KW-06000077 | Licensee : ESI/FME INC.

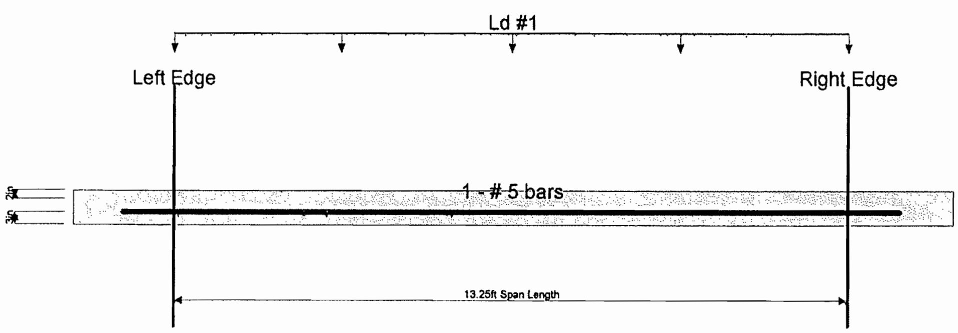
Description : Masonry Lintel

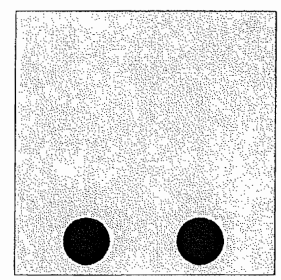
Code References

Calculations per ACI 530-08, IBC 2009, CBC 2010, ASCE 7-05
Load Combinations Used : ASCE 7-05

General Information

f'm	3,000.0 psi	Clear Span	13.250 ft	Rebar Size	5.0
Fs	24,000.0 psi	Lintel Depth	0.670 ft	# Bars E/F	1
Em = f'm *	750.0	Thickness	8 in	Top Clear	2.0 in
Wall Wt Mult	1.0	End Fixity	Pin-Pin	Btm Clear	3.0 in
Block Type	Normal Wt	Equiv. Solid Thickness	7.60 in	# Bar Sets	1
Lateral Wind Load	16.580 psf	Wall Weight	84.0 psf	Bar Spacing	3.50 in
Lateral Wall Weight Seismic Factor	0.3840	E	2,250.0 ksi		
Calculate & include vertical lintel weight ?	Yes	n	12.889		





Uniform Loads

	Start X	End X	Dead Load	Lf : Floor Live	Lr : Roof Live	S : Snow	W : Wind	E : Earthquake	
#1	ft	13.250 ft	0.070						k/ft
#2	ft	ft							k/ft
#3	ft	ft							k/ft
#4	ft	ft							k/ft

DESIGN SUMMARY

Design OK

Maximum Stress Ratios...	Vertical	Lateral	Combined
fb/Fb	0.8753	0.1086	**0.8820** : 1.00
fv/Fv	0.4354	0.04482	**0.4377** : 1.00

Maximum Moment	Actual	Allowable k-ft
Vertical Loads for Load Combination : +D	2.770 k-ft	3.165 k-ft
Lateral Loads for Load Combination : +D+0.70E+H	0.3319 k-ft	3.055

Maximum Shear	Actual	Allowable
Vertical Loads for Load Combination : +D	21.770 psi	50.0 psi
Lateral Loads for Load Combination : +D+0.70E+H	2.241 psi	50.0 psi

Vertical Strength

As	0.620 in^2
rho	0.01613
np	0.2079
k : ((np)^2+2np)^.5-np	0.4696
j = 1 - k/3	0.8435
M:mas=Fb k j b d^2/2	3.165 k-ft
M:Stl = Fs As j d	5.271 k-ft

Lateral Strength (Checking lateral bending for span)

As	0.310 in^2
rho	0.006932
np	0.08934
k : (np^2+2np)^.5-np	0.3427
j = 1 - k/3	0.8858
M:mas=Fb k j b d^2/2	3.115 k-ft
M:Stl = Fs As j d	3.055 k-ft

ESI/FME, Inc.
1800 E. 16th Street, Unit B
Santa Ana, CA 92603
PH: 714-835-2800
FX: 714-935-2819

Title : SPORTS FIELD
Engineer:
Project Desc.: 555 Maple Ave. Torrance, CA 90503

Job # C879

15

Printed: 13 AUG 2013, 2:05PM

Masonry Lintel

File: E:\Files\C879-Sports\Excel Files\calcs.ec6
ENERCALC, INC. 1983-2012, Build:6.12.8.2, Ver:6.12.8.2

Lic. # : KW-06000077

Licensee : ESI/FME INC.

Description : Masonry Lintel

Detailed Load Combination Results

Load Combination	Vertical				Lateral			
	Mmax k-ft	Mallow k-ft	fv : Vert psi	Fv : Vert psi	Mactual k-ft	Mallow k-ft	fv psi	Fv psi
+D	2.77	3.16	21.77	50.00	0.00	3.05	0.00	50.00
+D+W+H	2.77	3.16	21.77	50.00	0.24	3.05	1.65	50.00
+D-1.0W+H	2.77	3.16	21.77	50.00	0.24	3.05	1.65	50.00
+D+0.70E+H	2.77	3.16	21.77	50.00	0.33	3.05	2.24	50.00
+D-0.70E+H	2.77	3.16	21.77	50.00	0.33	3.05	2.24	50.00
+D+0.750Lr+0.750L+0.750W+H	2.77	3.16	21.77	50.00	0.18	3.05	1.23	50.00
+D+0.750Lr+0.750L-0.750W+H	2.77	3.16	21.77	50.00	0.18	3.05	1.23	50.00
+D+0.750L+0.750S+0.750W+H	2.77	3.16	21.77	50.00	0.18	3.05	1.23	50.00
+D+0.750L+0.750S-0.750W+H	2.77	3.16	21.77	50.00	0.18	3.05	1.23	50.00
+D+0.750Lr+0.750L+0.5250E+H	2.77	3.16	21.77	50.00	0.25	3.05	1.68	50.00
+D+0.750Lr+0.750L-0.5250E+H	2.77	3.16	21.77	50.00	0.25	3.05	1.68	50.00
+D+0.750L+0.750S+0.5250E+H	2.77	3.16	21.77	50.00	0.25	3.05	1.68	50.00
+D+0.750L+0.750S-0.5250E+H	2.77	3.16	21.77	50.00	0.25	3.05	1.68	50.00
+0.60D+W+H	1.66	3.16	13.06	50.00	0.24	3.05	1.65	50.00
+0.60D-1.0W+H	1.66	3.16	13.06	50.00	0.24	3.05	1.65	50.00
+0.60D+0.70E+H	1.66	3.16	13.06	50.00	0.33	3.05	2.24	50.00
+0.60D-0.70E+H	1.66	3.16	13.06	50.00	0.33	3.05	2.24	50.00

ESI/FME, INC.
STRUCTURAL ENGINEERS
1800 E. 16th Street, Unit 8
Santa Ana, CA 92701
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Cantilevered Retaining Wall Design

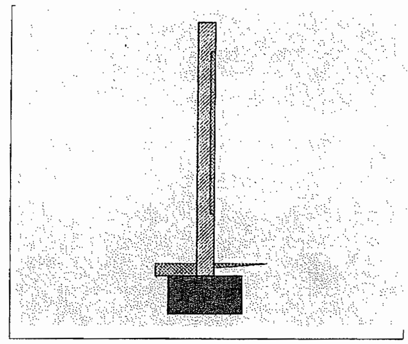
Code: CBC 2010,ACI 318-08,ACI 530-08

Criteria

Retained Height	=	0.50 ft
Wall height above soil	=	9.67 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	6.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	40.0 psf/ft
	=	
Passive Pressure	=	150.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	0.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
The above lateral load has been increased by a factor of		1.00
Wind on Exposed Stem	=	0.0 psf

Wind acts left-to-right toward retention side.

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Stem Weight Seismic Load

F_p / W_p Weight Multiplier = 0.384 g

Added seismic base force -196.9 lbs

Seismic Self-Weight acts left-to-right toward retention side.

Design Summary

Wall Stability Ratios

Overturning	=	1.70 OK
Sliding	=	6.84 OK
Total Bearing Load	=	1,436 lbs
...resultant ecc.	=	10.00 in
Soil Pressure @ Toe	=	0 psf OK
Soil Pressure @ Heel	=	1,532 psf OK
Allowable	=	2,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	0 psf
ACI Factored @ Heel	=	1,839 psf
Footing Shear @ Toe	=	2.3 psi OK
Footing Shear @ Heel	=	5.8 psi OK
Allowable	=	75.0 psi

Sliding Calcs (Vertical Component NOT Used)

Lateral Sliding Force	=	116.9 lbs
less 100% Passive Force	= -	225.0 lbs
less 100% Friction Force	= -	574.4 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	0.0 lbs OK

Load Factors

Building Code	CBC 2010,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Top Stem	2nd	3rd
		Stem OK	Stem OK	Stem OK
Design Height Above Ftg	ft =	9.00	2.50	0.00
Wall Material Above "Ht"	=	Masonry	Masonry	Masonry
Thickness	=	8.00	6.00	8.00
Rebar Size	=	# 4	# 4	# 4
Rebar Spacing	=	32.00	32.00	32.00
Rebar Placed at	=	Center	Center	Center
Design Data				
fb/FB + fa/Fa	=	-0.036	-1.776	-2.257
Total Force @ Section	lbs =	27.0	139.3	191.9
Moment....Actual	ft-# =	15.8	556.0	975.4
Moment.....Allowable	ft-# =	432.2	313.1	432.2
Shear.....Actual	psi =	0.6	4.2	4.3
Shear.....Allowable	psi =	38.7	38.7	38.7
Wall Weight	psf =	84.0	63.0	84.0
Rebar Depth 'd'	in =	3.75	2.75	3.75
LAP SPLICE IF ABOVE	in =	24.00	24.00	24.00
LAP SPLICE IF BELOW	in =	24.00	24.00	
HOOK EMBED INTO FTG	in =			6.00
Masonry Data				
f'm	psi =	1,500	1,500	1,500
Fs	psi =	20,000	20,000	20,000
Solid Grouting	=	Yes	Yes	Yes
Use Half Stresses	=	n/a	No	No
Modular Ratio 'n'	=	21.48	21.48	21.48
Short Term Factor	=	1.000	1.000	1.000
Equiv. Solid Thick.	in =	7.60	5.60	7.60
Masonry Block Type	=	Normal Weight		
Masonry Design Method	=	ASD		
Concrete Data				
f'c	psi =			
Fy	psi =			

ESI/FME, INC.
STRUCTURAL ENGINEERS
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RetainPro 10 (c) 1987-2011, Build 10.12.1.9
License : KW-06053839
License To : ESI/FME, INC.

Cantilevered Retaining Wall Design

Code: CBC 2010,ACI 318-08,ACI 530-08

Footing Dimensions & Strengths

Toe Width	=	1.13 ft
Heel Width	=	1.79
Total Footing Width	=	2.92
Footing Thickness	=	18.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.00 ft

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	1,839 psf
Mu' : Upward	=	3	1,181 ft-#
Mu' : Downward	=	280	0 ft-#
Mu: Design	=	-278	-1,181 ft-#
Actual 1-Way Shear	=	2.30	5.75 psi
Allow 1-Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	None Spec'd	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings
Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

Item		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	80.0	0.67	53.3
Surcharge over Heel	=			
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
	=			
Seismic Stem Self Wt		-196.9	6.46	-1,271.5
Total		-116.9	O.T.M.	-1,218.2
	=		=	

Resisting/Overturning Ratio = **1.70**
Vertical Loads used for Soil Pressure = 1,435.9 lbs

If seismic included the min. OTM and sliding ratios may be 1.1 per IBC '09, 1807.2.3.

Item		.....RESISTING..... Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	61.9	0.56	34.8
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	61.9	2.35	
Surcharge Over Toe	=			
Stem Weight(s)	=	717.8	1.51	1,080.9
Earth @ Stem Transitions	=			
Footing Weight	=	656.3	1.46	957.0
Key Weight	=		2.00	
Vert. Component	=			
Total =		1,435.9	lbs R.M.=	2,072.7

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

DESIGNER NOTES:

ESI/FME, INC.
STRUCTURAL ENGINEERS
1800 E. 16th Street, Unit 8
Santa Ana, CA 92701
PHONE: 714-835-2800 FAX: 714-835-2819

Cantilevered Retaining Wall Design

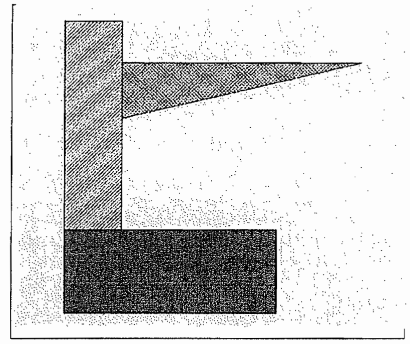
Code: CBC 2010,ACI 318-08,ACI 530-08

Criteria

Retained Height	=	2.00 ft
Wall height above soil	=	0.50 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	40.0 psf/ft
	=	
Passive Pressure	=	150.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	0.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
The above lateral load has been increased by a factor of		1.00
Wind on Exposed Stem	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Stem Weight Seismic Load

F_p / W_p Weight Multiplier = 0.384 g

Added seismic base force 57.6 lbs

Design Summary

Wall Stability Ratios

Overturning	=	3.80 OK
Sliding	=	1.66 OK
Total Bearing Load	=	988 lbs
...resultant ecc.	=	4.46 in
Soil Pressure @ Toe	=	748 psf OK
Soil Pressure @ Heel	=	42 psf OK
Allowable	=	2,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	898 psf
ACI Factored @ Heel	=	51 psf
Footing Shear @ Toe	=	0.0 psi OK
Footing Shear @ Heel	=	1.3 psi OK
Allowable	=	75.0 psi

Sliding Calcs (Vertical Component NOT Used)

Lateral Sliding Force	=	237.6 lbs
less 100% Passive Force	= -	0.0 lbs
less 100% Friction Force	= -	395.3 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	0.0 lbs OK

Load Factors

Building Code	CBC 2010,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

			Top Stem
			Stem OK
Design Height Above Ftg	ft	=	0.00
Wall Material Above "Ht"		=	Masonry
Thickness		=	8.00
Rebar Size		=	# 4
Rebar Spacing		=	32.00
Rebar Placed at		=	Edge
Design Data			
fb/FB + fa/Fa		=	0.205
Total Force @ Section	lbs	=	137.6
Moment....Actual	ft-#	=	125.3
Moment.....Allowable		=	612.1
Shear.....Actual	psi	=	2.2
Shear.....Allowable	psi	=	38.7
Wall Weight		=	84.0
Rebar Depth 'd'	in	=	5.25
LAP SPLICE IF ABOVE	in	=	24.00
LAP SPLICE IF BELOW	in	=	
HOOK EMBED INTO FTG	in	=	6.00
Masonry Data			
f'm	psi	=	1,500
Fs	psi	=	20,000
Solid Grouting		=	Yes
Use Half Stresses		=	n/a
Modular Ratio 'n'		=	21.48
Short Term Factor		=	1.000
Equiv. Solid Thick.	in	=	7.60
Masonry Block Type		=	Normal Weight
Masonry Design Method		=	ASD
Concrete Data			
f'c	psi	=	
Fy	psi	=	

ESI/FME, INC.
STRUCTURAL ENGINEERS
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Cantilevered Retaining Wall Design

Code: CBC 2010,ACI 318-08,ACI 530-08

Footing Dimensions & Strengths

Toe Width	=	0.00 ft
Heel Width	=	2.50
Total Footing Width	=	2.50
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.00 ft

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	898	51 psf
Mu' : Upward	=	0	553 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	-553 ft-#
Actual 1-Way Shear	=	0.00	1.33 psi
Allow 1-Way Shear	=	0.00	75.00 psi
Toe Reinforcing	=	None Spec'd	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	180.0	1.00	180.0
Surcharge over Heel	=			
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
	=			
Seismic Stem Self Wt		57.6	2.25	129.6
Total		237.6	**O.T.M.**	309.6
	=		=	

Resisting/Overturning Ratio = **3.80**
Vertical Loads used for Soil Pressure = 988.3 lbs

If seismic included the min. OTM and sliding ratios may be 1.1 per IBC '09, 1807.2.3.

.....RESISTING.....

		Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	403.3	1.58	638.6
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	403.3		
Surcharge Over Toe	=			
Stem Weight(s)	=	210.0	0.33	70.0
Earth @ Stem Transitions	=			
Footing Weight	=	375.0	1.25	468.8
Key Weight	=		2.00	
Vert. Component	=			
Total =		988.3 lbs	R.M.=	1,177.4

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

DESIGNER NOTES:

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STRUCTURAL ENGINEERS
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Cantilevered Retaining Wall Design

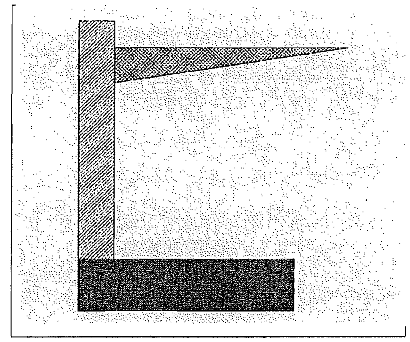
Code: CBC 2010,ACI 318-08,ACI 530-08

Criteria

Retained Height	=	4.00 ft
Wall height above soil	=	0.50 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	40.0 psf/ft
	=	
Passive Pressure	=	150.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	0.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
The above lateral load has been increased by a factor of		1.00
Wind on Exposed Stem	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Stem Weight Seismic Load

F_p / W_p Weight Multiplier = 0.384 g

Added seismic base force 103.7 lbs

Design Summary

Wall Stability Ratios

Overturning	=	4.06	OK
Sliding	=	1.62	OK
Total Bearing Load	=	2,445 lbs	
...resultant ecc.	=	6.44 in	
Soil Pressure @ Toe	=	1,103 psf	OK
Soil Pressure @ Heel	=	119 psf	OK
Allowable	=	2,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,324 psf	
ACI Factored @ Heel	=	143 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	2.1 psi	OK
Allowable	=	75.0 psi	

Sliding Calcs (Vertical Component NOT Used)

Lateral Sliding Force	=	603.7 lbs	
less 100% Passive Force	= -	0.0 lbs	
less 100% Friction Force	= -	977.9 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	0.0 lbs	OK

Load Factors

Building Code	CBC 2010,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

			Top Stem
			Stem OK
Design Height Above Ftg	ft	=	0.00
Wall Material Above "Ht"		=	Masonry
Thickness		=	8.00
Rebar Size		=	# 5
Rebar Spacing		=	32.00
Rebar Placed at		=	Edge
Design Data			
fb/FB + fa/Fa		=	0.706
Total Force @ Section	lbs	=	423.7
Moment....Actual	ft-#	=	659.9
Moment.....Allowable		=	934.2
Shear.....Actual	psi	=	6.7
Shear.....Allowable	psi	=	38.7
Wall Weight		=	84.0
Rebar Depth 'd'	in	=	5.25
LAP SPLICE IF ABOVE	in	=	30.00
LAP SPLICE IF BELOW	in	=	
HOOK EMBED INTO FTG	in	=	7.00
Masonry Data			
f'm	psi	=	1,500
Fs	psi	=	20,000
Solid Grouting		=	Yes
Use Half Stresses		=	n/a
Modular Ratio 'n'		=	21.48
Short Term Factor		=	1.000
Equiv. Solid Thick.	in	=	7.60
Masonry Block Type		=	Normal Weight
Masonry Design Method		=	ASD
Concrete Data			
f'c	psi	=	
Fy	psi	=	

Cantilevered Retaining Wall Design

Code: CBC 2010,ACI 318-08,ACI 530-08

Footing Dimensions & Strengths

Toe Width	=	0.00 ft
Heel Width	=	4.00
Total Footing Width	=	4.00
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.00 ft

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,324	143 psf
Mu' : Upward	=	0	2,986 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	-2,986 ft-#
Actual 1-Way Shear	=	0.00	2.13 psi
Allow 1-Way Shear	=	0.00	75.00 psi

Toe Reinforcing = None Spec'd
Heel Reinforcing = # 4 @ 8.00 in
Key Reinforcing = None Spec'd

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: #4@ 9.50 in, #5@ 14.50 in, #6@ 20.50 in, #7@ 28.00 in, #8@ 36.75 in, #9@ 46
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING..... Item		Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	500.0	1.67	833.3
Surcharge over Heel	=			
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
	=			
Seismic Stem Self Wt		103.7	3.25	337.0
Total		603.7	**O.T.M.**	1,170.3

= =

Resisting/Overturning Ratio = **4.06**
Vertical Loads used for Soil Pressure = 2,444.7 lbs

If seismic included the min. OTM and sliding ratios may be 1.1 per IBC '09, 1807.2.3.

.....RESISTING.....		Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	1,466.7	2.33	3,422.2
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	1,466.7		
Surcharge Over Toe	=			
Stem Weight(s)	=	378.0	0.33	126.0
Earth @ Stem Transitions	=			
Footing Weight	=	600.0	2.00	1,200.0
Key Weight	=		2.00	
Vert. Component	=			
Total =		2,444.7 lbs	R.M.=	4,748.2

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

DESIGNER NOTES:

Two

GEOTECHNICAL INVESTIGATION, PROPOSED COMMUNITY SPORTS FIELD, 555 MAPLE AVENUE, TORRANCE CALIFORNIA

LAND CONCERN
C/O PHILIP STEVENS

JUNE 3, 2013
J.N. 13-248

Orange County / Environmental / Corporate

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PETRA

past + present + future
it's in our science

Engineers, Geologists
Environmental Scientists

June 3, 2013
J.N. 13-248

LAND CONCERN
1750 East Deere Avenue
Santa Ana, CA 92705

Attention: Mr. Philip Stevens

Subject: Summary Report of Geotechnical Field Investigation for Proposed Community Sports Fields, 555 Maple Avenue, Torrance, California

We are pleased to submit herewith a summary report of our geotechnical field investigation for the proposed community sports field located at 555 Maple Avenue, Torrance, California. This work was performed in accordance with the scope of work outlined in our proposal dated April 25, 2013. This report presents the results of our field investigation and laboratory testing, and our engineering judgment, opinions, conclusions and recommendations pertaining to geotechnical design aspects of the proposed development.

It has been a pleasure to be of service to you on this project. Should you have any questions regarding the contents of this report, or should you require additional information, please do not hesitate to contact us.

Respectfully submitted,

PETRA GEOTECHNICAL, INC.

Theodore M. Wolfe
Senior Associate Geologist

GEOTECHNICAL INVESTIGATION, PROPOSED COMMUNITY SPORTS FIELDS, 555 MAPLE AVENUE, TORRANCE, CALIFORNIA

INTRODUCTION

This report presents the results of our geotechnical investigation of the subject property. The purposes of this study were to determine the nature of subsurface soils, to evaluate their in-place characteristics, and to then provide geotechnical recommendations with respect to site preparation, and for design and construction of foundations.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The site is a vacant lot which is bordered by parking lots on the west and east, Maple Avenue to the east and Civic Center Drive to the south. The site is essentially flat and is covered by a light growth of grasses. A row of large eucalyptus trees is located adjacent to the easterly property boundary. Small to moderate size trees are scattered along the north and south property lines. A site map/aerial photo is presented as Figure 1 Chain link fencing encompasses the site.

Based on a review of a preliminary site plan and conversations with the project team, it is proposed to construct several sports (soccer) fields on the site. Sports lights are proposed along the periphery of the site and prefabricated restroom facilities are planned along with attendant hardscape/concrete improvements. It is our understanding that storm water drainage facilities will also be a part of the project requirements.

SUBSURFACE EXPLORATION

Our subsurface exploration was performed on May 7, 2013 and consisted of excavating six backhoe test pits to depths ranging from 7 to 12 feet below the existing ground surfaces. Soils encountered were visually classified and logged in accordance with the Unified Soil Classification System. Representative samples of soil were also obtained for laboratory testing. Relatively undisturbed samples of subsurface soil were obtained with a hand-driven, split-spoon sampler. The approximate locations of the borings are shown on the enclosed Site Plan (Figure 1) and the exploration logs are presented in Appendix A.

LABORATORY TESTING

To evaluate the engineering properties of site soils, several laboratory tests are currently being performed on selected samples considered representative of those encountered during our investigation of the site. Laboratory tests include the determination of expansion potential, soluble sulfate contents, chloride content, pH, shear strength and in-place moisture and density. The results of the laboratory testing are presented in the Appendix B.

FINDINGS

Regional Geology

The subject site is located in the Los Angeles Basin which is a large, alluvial filled basin that was formed during the later stages of the Pleistocene Epoch. The basin is characterized by thick accumulations of alluvial soils that vary in thickness from hundreds to thousands of feet.

Geology and Subsurface Conditions

Based on our exploratory borings, the site is underlain by alluvium that is typically a very fine grained sand with some secondary silt and clay fractions. The upper 6 inches to 1 foot of alluvium is weathered with rootlets and minor porosity. The alluvium becomes dense to very dense with depth. Occasional sandy layers were noted. Finer grained materials were difficult to excavate. Minor occurrences of fill were noted in 3 of the test pits. The fill was on the order of one foot in thickness and consisted of gravel pockets and silty sands with asphalt pieces and minor construction debris.

Faulting

Based on our review of the referenced geologic maps and literature, no active or potentially active faults are known to project through the property. Furthermore, the site does not lie within the boundaries of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act (Hart and Bryant, 1997). The Alquist-Priolo Earthquake Fault Zoning Act (AP Act) defines an *active fault* as one that has had surface displacement within Holocene time (about the last 11,000 years). The primary objective of the AP Act is to prevent the construction of structures for human occupancy over the traces of active faults that could displace the ground surface and result in loss of life and property.

Seismic Hazard Zones

Through the Seismic Hazards Mapping Act, the CGS, formerly known as the California Division of Mines and Geology (CDMG), has established Seismic Hazard Zones that depict areas considered susceptible to earthquake-induced soil liquefaction and landsliding within the more densely populated portions of southern and northern California. According to the published Seismic Hazard Zone map for the Torrance 7.5-minute quadrangle (CDMG, 1998), the subject site does not lie within a zone that is susceptible to either earthquake-induced liquefaction or landsliding. Since the site is underlain by dense alluvial deposits and is not underlain by shallow groundwater, we consider this zonation to be appropriate.

Seismic-Induced Flooding

Seismically induced flooding which might be considered a potential hazard to a particular site normally includes flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major reservoir retention structure upstream of the site. The site lies approximately 3 miles inland from the Pacific Ocean at an approximate elevation of 100 feet, and does not lie in close proximity of an enclosed body of water or downstream of a major reservoir retention structure, the probability of flooding from a tsunami, seiche or dam-break is considered to be negligible.

CONCLUSIONS AND RECOMMENDATIONS

General

From a soils engineering and engineering geologic point of view, the subject property is considered suitable for the proposed construction provided the following conclusions and recommendations are incorporated into the design criteria and project specifications. Furthermore, provided that grading and construction within the site are performed in accordance with the recommendations of this report, the proposed grading and improvements are not expected to adversely impact the stability of the adjacent properties.

Earthwork

General Earthwork and Grading Specifications

All earthwork and grading should be performed in accordance with the Municipal Code of the City of Torrance, and in accordance with the following recommendations prepared by this firm.

Site Clearing

Prior to site grading, all unsuitable materials including grasses and weeds are to be removed and hauled from the site. The project geotechnical consultant should be notified at the appropriate times to provide observation and testing services during clearing operations. In addition, should any buried structures or unusual or adverse soil conditions be encountered during clearing operations or during grading that are not described or anticipated herein, these conditions should be brought to the immediate attention of the geotechnical consultant.

Excavation Characteristics

Based on our subsurface exploration, existing topsoil and native terrace deposit materials will be easily excavatable with conventional earthmoving equipment.

Ground Preparation – Concrete Hardscape Improvements and Prefabricated Restroom Faciilities

In order to provide proper support for the concrete hardscape surfaces and pads for prefabricated restroom facilityies, it is recommended that the subgrade to a depth of 8 to 12 inches be scarified, moisture conditioned and compacted to a minimum relative compaction of at least 90 percent. It should be noted that the recommended depth of reprocessing is an estimate based upon observed conditions at the time of our field study. The required depth may vary depending upon weather conditions and should be based on observations and testing conducted during site development.

Ground Preparation- Sports Fields

Given the observed near surface conditions, specific ground preparation for turf sports field areas are not required. Manufacturer requirements should be followed if an artificial turf product is utilized.

Fill Compaction

It is anticipated that minor cuts and fills will be required to attain proposed grades. New fill should be placed in 6- to 8-inch-thick maximum lifts, watered or air-dried as necessary to achieve near optimum moisture conditions, and then compacted in place to a minimum relative compaction of 90 percent. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D 1557-07. A representative of the project geotechnical consultant should be present on-site during grading operations to verify proper compaction for the subgrade in hardscape and restroom facility.

Geotechnical Observations

The bottom of the footing excavations should be observed by a representative of the project geotechnical consultant prior to placing steel reinforcement. It may be necessary to deepen localized areas of the footings to remove unsuitable bearing materials. A representative of the project geotechnical consultant should also be present on site during grading operations to verify depth of scarification and moisture conditioning below hardscape improvements and proper compaction of all fill, as well as to verify compliance with the other recommendations presented herein.

Hardscape Concrete Flatwork

General

Near-surface compacted fill soils within the site are variable in expansion behavior and are expected to exhibit very low to low expansion potential. For this reason, we recommend that all concrete flatwork be designed by the project architect and/or structural engineer with consideration given to mitigating the potential cracking and uplift that can develop in soils exhibiting expansion index values that fall in the low category.

The guidelines that follow should be considered as minimums and are subject to review and revision by the project architect, structural engineer and/or landscape consultant as deemed appropriate. If sufficient time will be allowed in the project schedule for verification sampling and testing prior to the concrete pour, the test results generated may dictate that a somewhat less conservative design could be used.

Thickness and Joint Spacing

To reduce the potential of unsightly cracking, concrete walkways, large decorative slabs and concrete subslabs to be covered with decorative pavers should be at least 4 inches thick and provided with construction joints or expansion joints every 6 feet or less.

Reinforcement

All concrete flatwork having their largest plan-view panel dimension exceeding 5 feet should be reinforced with a minimum of No. 3 bars spaced 24 inches on centers, both ways. Alternatively, the slab reinforcement may consist of welded wire mesh of the sheet type (not rolled) with 6x6/W1.4xW1.4 designation in accordance with the Wire Reinforcement Institute (WRI). The reinforcement should be properly positioned near the middle of the slabs.

> *The reinforcement recommendations provided herein are intended as guidelines to achieve adequate performance for anticipated soil conditions. The project architect, civil and/or structural engineer should make appropriate adjustments in reinforcement type, size and spacing to account for concrete internal (e.g., shrinkage and thermal) and external (e.g., applied loads) forces as deemed necessary.*

Edge Beams (Optional)

Where the outer edges of concrete flatwork are to be bordered by landscaping, it is recommended that consideration be given to the use of edge beams (thickened edges) to prevent excessive infiltration and accumulation of water under the slabs. Edge beams, if used, should be 6 to 8 inches wide, extend 8 inches below the tops of the finish slab surfaces. Edge beams are not mandatory; however, their inclusion in flatwork construction adjacent to landscaped areas is intended to reduce the potential for vertical and horizontal movement and subsequent cracking of the flatwork related to uplift forces that can develop in expansive soils.

Subgrade Preparation

Compaction

To reduce the potential for distress to concrete flatwork, the subgrade soils below concrete flatwork areas to a minimum depth of 6 inches (or deeper, as either prescribed elsewhere in this report or determined in the field) should be moisture conditioned to at least equal to, or slightly greater than, the optimum moisture content and then compacted to a minimum relative compaction of 90 percent.

Pre-Moistening

As a further measure to reduce the potential for concrete flatwork cracking, subgrade soils should be thoroughly moistened prior to placing concrete. The moisture content of the soils should be at least 1.2 times the optimum moisture content and penetrate to a minimum depth of 12 inches into the subgrade. Flooding or ponding of the subgrade is not considered feasible to achieve the above moisture conditions since this method would likely require construction of numerous earth berms to contain the water. Therefore, moisture conditioning should be achieved with sprinklers or a light spray applied to the subgrade over a period of few to several days just prior to pouring concrete. Pre-watering of the soils is intended to promote uniform curing of the concrete, reduce the development of shrinkage cracks and reduce the potential for differential expansion pressure on freshly poured flatwork. A representative of the project geotechnical consultant should observe and verify the density and moisture content of the soils, and the depth of moisture penetration prior to pouring concrete.

Drainage

Drainage from patios and other flatwork areas should be directed to local area drains and/or graded earth swales designed to carry runoff water to the adjacent streets or other approved drainage structures. The concrete flatwork should be sloped at a minimum gradient of one percent, or as prescribed by project civil engineer or local codes, away from building foundations, retaining walls, masonry garden walls and slope areas.

Tree Wells

Tree wells are not recommended in concrete flatwork areas since they introduce excessive water into the subgrade soils and allow root invasion, both of which can cause heaving and cracking of the flatwork.

Light Pole Foundation Design Recommendations

Light Poles will be placed around the perimeter of the site. It is our understanding that the poles will be precast and will be founded in cast-in-place drilled hole (CIDH) piles. Based on the field investigation, the site soils are suitable for the support of the light pole foundations. Presented below are recommendations for design of the light poles.

Earthquake Loads

Structures within the site should be designed and constructed to resist the effects of seismic ground motions as provided in Section 1613 of the 2010 California Building Code (CBC). The method of design is dependent on the seismic zoning, site characteristics, occupancy category, building configuration, type of structural system and on the building height.

For structural design in accordance with the 2010 CBC, a computer program, Earthquake Ground Motion Parameters Version 5.07, developed by the United States Geological Survey (USGS, 2007) was utilized to provide ground motion parameters for the subject site. The program includes hazard curves, uniform hazard response spectra and design parameters for sites in the 50 United States, Puerto Rico and the United States Virgin Islands. Based on the latitude, longitude and site classification, seismic design parameters and spectral response for both short periods and 1-second periods are calculated including Mapped Spectral Response Acceleration Parameter, Site Coefficient, Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter and Design Spectral Response Acceleration Parameter. The program is based on USGS research and publications in cooperation with the California Geological Survey for evaluation of California faulting and seismicity (USGS, 1996a; 1996b; 2002; 2007).

The Palos Verdes Fault (approximately 5.3 miles or 8.8 kilometers to the west of the site) should be considered to be the causative fault for the subject site and is expected to generate the most severe site ground motions with an anticipated maximum moment magnitude (Mw) of 6.9 and an anticipated slip rate of 3 mm/year (CGS, 2002).

The following 2010 CBC seismic design coefficients should be used for the proposed light poles. These criteria are based on the site class as determined by existing subsurface geologic conditions, on the proximity of the site to the nearby fault and on the maximum moment magnitude and slip rate of the nearby fault.

2010 CBC Section 1613, Earthquake Loads	
Mapped Spectral Response Acceleration Parameter, S_s (Figure 1613.5(3) for 0.2 second)	1.335
Mapped Spectral Response Acceleration Parameter, S_1 (Figure 1613.5(4) for 1.0 second)	0.600
Site Class Definition (Table 1613.5.2)	D
Site Coefficient, F_a (Table 1613.5.3 (1) short period)	1.0
Site Coefficient, F_v (Table 1613.5.3 (2) 1-second period)	1.5
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS} (Eq. 16-36)	1.335
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1} (Eq. 16-37)	0.900
Design Spectral Response Acceleration Parameter, S_{DS} (Eq. 16-38)	0.890
Design Spectral Response Acceleration Parameter, S_{D1} (Eq. 16-39)	0.600

Allowable Bearing Values

An allowable soil bearing capacity of 2,000 pounds per square foot may be used for design. This value may be increased by 200lbs./sq.ft. for each additional foot of depth to a maximum allowable value of 4,000 lbs./sq.ft.

Recommended allowable bearing values include both dead and live loads, and may be increased by one-third when designing for short duration wind and seismic forces.

Lateral Resistance

A passive earth pressure of 150 pounds per square foot per foot of depth, to a maximum value of 2,250 pounds per square foot, may be used to determine lateral bearing. A coefficient of friction of 0.25 times the dead load forces may also be used between concrete and the supporting soils to determine lateral sliding resistance. An increase of one-third of the above values may also be used when designing for short duration wind and seismic forces. The above values are based on footings placed directly against competent undisturbed native soil. In cases where footing sides are formed, all backfill against footings should be compacted to at least 90 percent of maximum density.

CIDH Design

The following values summarize the allowable "End-Bearing" values for 18-inch and 24-inch diameter drilled holes.

Allowable End-Bearing Capacity

Depth (ft)	Caisson Diameter 18"	24"
5	3.6 Kips	6.5 Kips
10	7.3 Kips	13.0 Kips
15	11.0 Kips	26.1 Kips

In addition, a pile capacity versus diameter (P/d) curve, Plate C, may be utilized in the design of caissons using skin friction. A value of 0.4 may be used for the coefficient of friction between soil and concrete/slurry.

Construction/Observations

All pile excavations should be observed by a representative of the project geotechnical consultant to verify that they have been excavated into competent bearing soils prior to the placement of precast light poles and concrete. All loose, sloughed or moisture-softened soils should be removed from the caisson excavation prior to placing of concrete.

The caisson walls and base should be observed for anomalies, unexpected soft soil conditions, obstructions or caving.

No borings should be allowed to remain open overnight.

The annular space between light pole standards and the bored caisson hole should be filled with a 4-sack slurry/concrete mix.

For caissons deeper than 5 feet concrete should not be allowed to "freefall" during placement.

Soluble Sulfate Analysis and General Corrosivity Screening

The following sections represent an interpretation of current codes and specifications that are commonly used in our industry as they relate to the adverse impact of chemical components of the site soils on various components of the proposed structures. As a screening level study, limited chemical testing was performed on representative samples of onsite soils to identify potential corrosive characteristics of these soils. The testing procedure referred to herein is considered to be typical for our industry and has been adopted and/or approved by many public or private agencies

Petra does not practice corrosion engineering; therefore, the opinion and engineering judgment provided herein should be considered as general guidelines only. Further analyses would be warranted for cases where buried metallic building materials such as copper and ductile iron are planned for the project. For these conditions, we recommend that the project design professionals (i.e., the architect and/or structural engineer) consider recommending a qualified corrosion engineer to conduct additional sampling and testing of near-surface soils during the final stages of site grading to provide a complete assessment of soil corrosivity. Recommendations to mitigate the detrimental effects of corrosive soils on buried metallic and other building materials that may be exposed to corrosive soils should be provided by the corrosion engineer as deemed appropriate.

Concrete in Contact with Site Soils

Soils containing soluble sulfates beyond certain threshold levels are considered to be detrimental to integrity of concrete placed in contact with such soils. For the purpose of this study, soluble sulfates concentration in soils were determined in accordance with California Test Method No. 417.

The results of our laboratory tests indicate that on-site soils within the subject site contain a water soluble sulfate content of less than 0.10 percent. Based on Section 1904.3 of the 2010 CBC, concrete that will be exposed to sulfate-containing soils should comply with the provisions of Section 4.3 of ACI 318.

According to Table 4.2.1 of ACI 318-08 (a precursor to Section 4.3), an exposure class of S0 is appropriate for onsite soils. As such, a **Not Applicable** exposure to sulfate may be expected for concrete placed in contact with the onsite soil materials. As dictated by Table 4.3.1 of ACI 318-08, no restriction for cement or maximum water-cement ratio for the fresh concrete would be required for this condition. However, the concrete minimum unconfined compressive strength should not be less than 2,500 psi.

Metallic Elements in Contact with Site Soils

Elevated concentrations of soluble salts in soils tend to induce low level electrical currents in metallic objects in contact with such soils. This process promotes metal corrosion and can lead to distress to components that are in contact with site soils. The minimum electrical resistivity indicates the relative concentration of soluble salts in the soil and, therefore, can be used to estimate soil corrosivity with regard to metals. For the purpose of this investigation, the minimum resistivity in soils is measured in accordance with California Test Method No. 643.

The minimum electrical resistivity for onsite soils was found to be 2,600 ohm-cm based on limited testing. This result indicates that on-site soils are **moderately corrosive** to ferrous metals and copper. As such, any ferrous metal or copper components of the subject buildings (such as cast iron or ductile iron piping, copper tubing, etc.) that are expected to be placed in direct contact with site soils should be protected against detrimental effects of moderately corrosive soils.

FUTURE IMPROVEMENTS

Should any new structures or improvements be proposed at any time in the future other than those described herein, our firm should be notified so that we may provide design recommendations.

REPORT LIMITATIONS

This report presents a summary of the geotechnical investigation conducted on the subject site and the results of our research of the existing site seismic considerations. The materials encountered on the project site, described in other literature, and utilized in our laboratory investigation are believed representative of the project area, and the conclusions contained in this report are presented on that basis. However, soil and bedrock materials can vary in characteristics between points of exploration, both laterally and vertically, and those variations could affect site development/design considerations. As such, observation and testing by a geotechnical consultant during the grading and construction phases of the project are essential to confirming the basis of this report. To provide the greatest degree of continuity between the design and construction phases, consideration should be given to retaining Petra Geotechnical, Inc., for construction services.

This report has been prepared consistent with the level of care being provided by other professionals providing similar services at the same locale and time period. As previously described, the purpose of this report is to provide a summary of the site conditions as observed in our recent field investigation. Results of laboratory testing and recommendations concerning design and construction of the improvements will be presented in an addendum report.

This opportunity to be of service is sincerely appreciated. Please call if you have any questions pertaining to this report.

Respectfully submitted,
PETRA GEOTECHNICAL, INC.

Theodore M. Wolfe
Senior Associate Geologist
CEG 1626

Distribution: (1) Land Concern- Philip Stevens (electronic copy)

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LITERATURE REVIEWED

California Division of Mines and Geology, 1997, Seismic Hazard Zone Report for the Venice 7.5-Minute Quadrangle, Los Angeles County, California: California Department of Conservation Division of Mines and Geology, Seismic Hazard Zone Report 036 (official map Released April 15, 1998).

California Geological Survey, 2002, Probabilistic Seismic Hazard Assessment for the State of California, Open-File Report 96-08, Revised 2002 California Seismic Shaking Analysis, , Appendix A.

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______, 2007, Preliminary Documentation for the 2007 Update of the United States National Seismic Hazard Maps, Seismic Hazards Mapping Project, Open-File Report 2007-June Draft.

PETRA

APPENDIX A

EXPLORATION LOGS

PETRA

(14)

LOG OF TEST PITS
JN-13-248 DATE EXCAVATED 5/7/13

Test Pit 1 (TP-1)
0-6 inches- Fill (af)- gravel pocket on east side of hole- probable base material
6 inches- 7 feet- Alluvium (Qal)- very fine grained sand (SM)- light orange to brown orange, dry becoming slightly moist below 2 feet, medium dense to dense, occasional roots in upper 2 feet,
@5 feet- becomes dense to very dense
Total Depth 7 feet
Bulk Sample- 5 feet

Test Pit 2 (TP-2)
0-1-1/2 feet- Fill (af)- silty very fine grained sand- some construction debris (concrete/tar paper), rootlets, dry, medium dense,
@ 1 foot- 6 inch gravel layer- probable base material
1-1/2 - 9 feet- Alluvium (Qal)- very fine grained sand (SM)- light orange to brown orange with staining, dry becoming slightly moist below 2 feet, medium dense to dense, occasional roots in upper 2 feet,
@6 feet- increase in fines- becomes clayey very fine to fine grained sand- slightly moist to moist, dense to very dense
Total Depth 9 feet
Ring Sample- 6 feet
Dry Density- 93.8 pcf, Moisture Content- 13.6%

Test Pit 3 (TP-3)
0-7-1/2 feet- Alluvium (Qal)- silty, very fine grained sand (SM)- brown orange to grey, dry becoming slightly moist below 3 feet, medium dense to dense, abundant rootlets in upper 1 foot, larger eucalyptus roots in upper 4 feet
@5 feet- increase in fines- becomes silty/clayey, very fine to fine grained sand- slightly moist dense to very dense
Difficult digging-refusal
Total Depth 7-1/2 feet
Bag Sample- 1 foot

Test Pit 4 (TP-4)
0-9 feet- Alluvium (Qal)- very fine grained sand (SM)- grey, dry, loose to medium dense
@3 feet- slightly clayey very fine grained sand (SM)- dry very hard
@6 feet- decrease in fines- very fine grained sand (SP)- orange/brown, hard, dry to slightly moist.
Total Depth 9 feet
Ring Sample- 2 feet
Bag Sample 4 to 7 feet

PETRA

Test Pit 5 (TP-5)
0-12 feet- Alluvium (Qal)- very fine grained sand (SM)- grey, dry, loose to medium dense, occasional roots in upper 4 feet
@5 feet- slightly clayey very fine grained sand (SM)- dry very hard
@8 feet- decrease in fines- very fine grained sand (SP)- orange/brown, hard, dry to slightly moist.
Total Depth 12 feet
Ring Sample- 11-1/2 feet
Dry Density- 83.9 pcf, Moisture Content- 4.6%

Test Pit 6 (TP-6)
0-6 inches- Fill (af)- silty fine grained sand- dry, rootlets, some construction debris (asphalt/tar paper, gravel), rootlets, dry, medium dense,
6 inches - 10 feet- Alluvium (Qal)- very fine grained sand (SM)- light orange to brown orange, dry becoming slightly moist below 2 feet, medium dense to dense, occasional roots in upper 2 feet,
@5 feet- becomes dense to very dense
Total Depth10 feet

APPENDIX B

LABORATORY TEST PROCEDURES

LABORATORY TEST DATA

PETRA

17

LABORATORY TEST PROCEDURES

Soil Classification

Soil materials encountered within the property were classified and described using the visual-manual procedures of the Unified Soil Classification System and the Engineering Geology Field Manual by the U.S. Department of the Interior, Bureau of Reclamation, respectively, and in general accordance with Test Method ASTM D 2488. The assigned group symbols are presented in the exploration logs, Appendix A.

In Situ Moisture and Density

Moisture content and unit dry density of the in place soil materials were determined in representative strata. Test data are presented in the exploration logs, Appendix A.

Expansion Potential

Expansion index tests were performed on representative samples of the on-site soils in accordance with test method ASTM D 4829. The test results are presented on Plate B-1.

Chemical Analysis

Chemical analyses were performed on selected bulk samples of onsite soils to determine their soluble sulfate and chloride contents. These tests were performed in accordance with the current versions of California Test Method Nos. CTM 417 and CTM 422, respectively. The results of these tests are included on Plate B-1.

pH and Minimum Resistivity

pH and resistivity tests were performed on selected bulk samples of onsite soils to determine their acidity and electrical resistance. These tests were performed in accordance with the current versions of California Test Method Nos. CTM 532 and CTM 643, respectively. The results of these tests are included in Plate B-1.

Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for relatively undisturbed samples from onsite soil. These tests were performed in general accordance with Test Method No. ASTM D 3080-11. One specimen was prepared for each test. The test specimens were artificially saturated, and then sheared under a normal load at a constant rate of strain of 0.05 inches per minute. Results are graphically presented on Plate SH-1.

LABORATORY DATA SUMMARY

Boring Number	Sample Depth (ft)	Soil Description	Max. Dry Density [1] (pcf)	Optimum Moisture[1] (%)	Expansion Index[2]	Expansion Potential	Sulfate Content[3] (%)	Chloride Content[4] (ppm)	PH[5]	Minimum Resistivity[6] (Ohm-cm)
T-3	5	Silty Sand, SM			2	Very Low				
T-3	5	Silty Sand, SM					0.03			
T-4	4-7	Silty Sand, SM							7.4	2,600

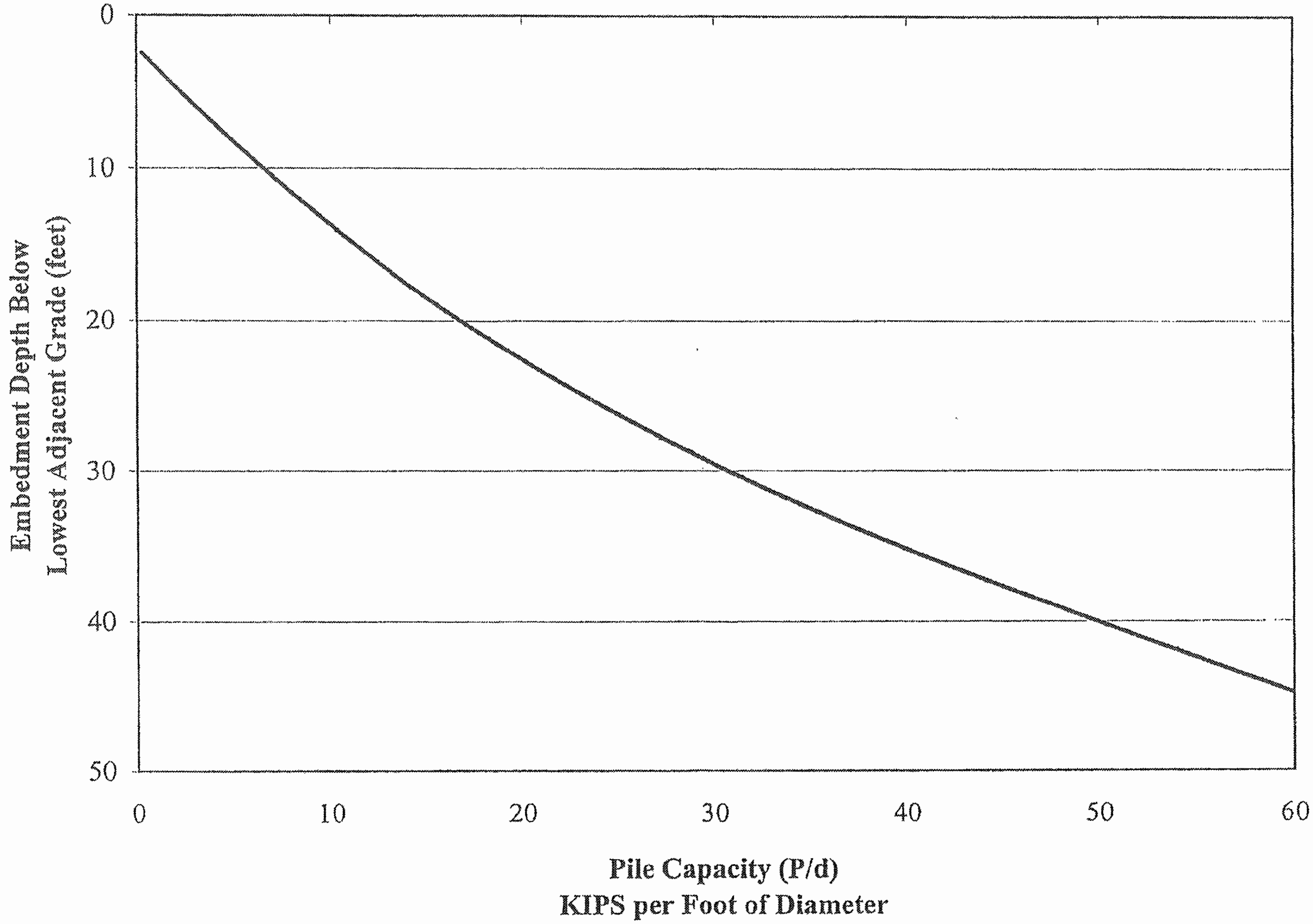
Test Procedures:

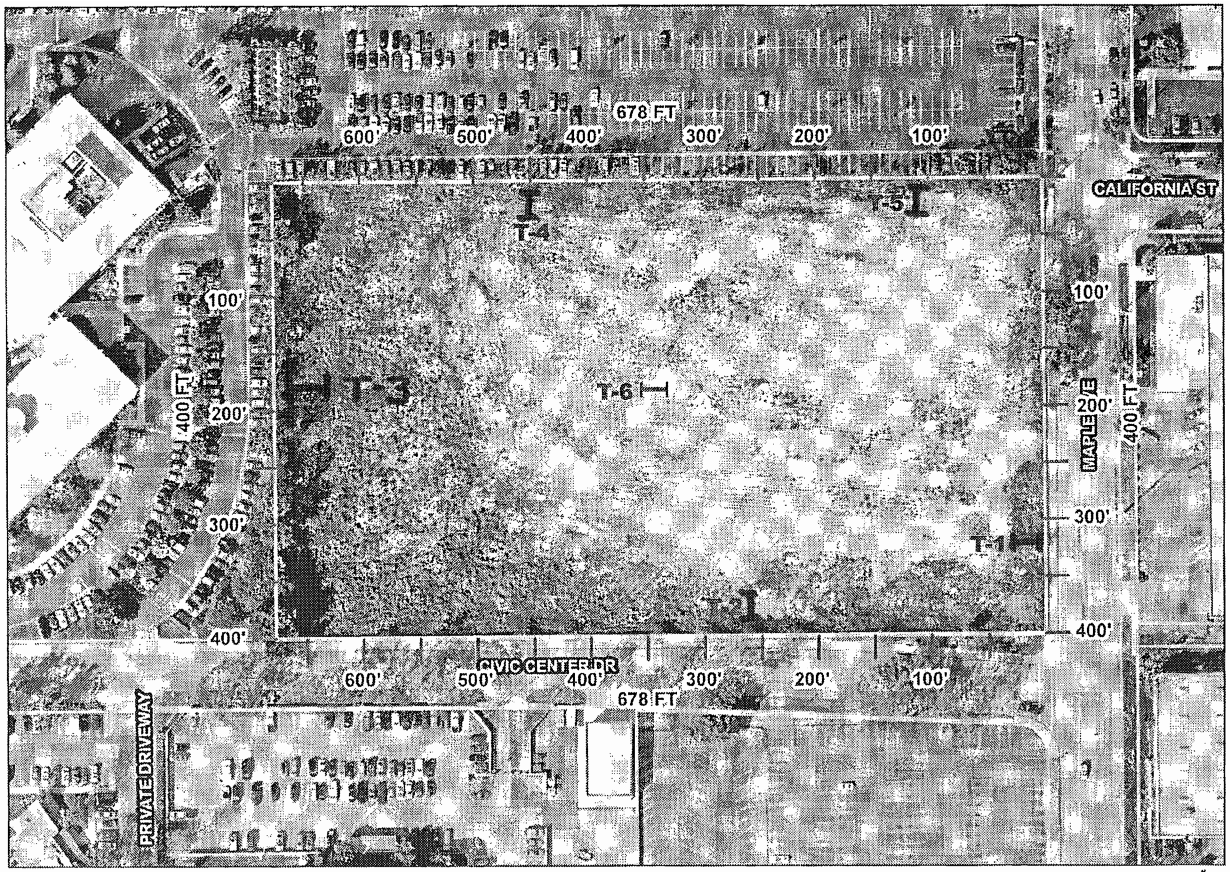
1 Per California Test Method 417
2 Per ASTM D 4839
3 Per California Test Method No. 417
4 Per California Test Method No. 622
5 Per California Test Method No. 532
6 Per California Test Method No. 643

May, 2013
J.N. 13-248

Plate B-1

Pile Capacity per Depth of Embedment





Lines and photos are approximate, not to be used for establishing absolute or relative positions

0 25 50 100 Feet

SCALE 1" : 100'

555 MAPLE AVE, TORRANCE
AREA: 271,037 SF

EXPLANATION

T-6 ├─┤ TEST PIT LOCATION

PETRA GEOTECHNICAL, INC.
3185-A AIRWAY AVENUE
COSTA MESA, CALIFORNIA 92626
PHONE: (714) 549-8921
COSTA MESA MURRIETA PALM DESERT SANTA CLARITA

SITE PLAN

COMMUNITY SPORTS FIELD
555 MAPLE AVENUE, TORRANCE, CALIFORNIA

DATE: May, 2013	J.N.: 13-248	Figure 1
DWG BY: SSR	SCALE: 1"=100'	

(21)

Three

TORRANCE COMMUNITY SPORTS FIELD
City of Torrance, CA

STANDARD URBAN STORMWATER MITIGATION PLAN (SUSMP)
August 7, 2013

Prepared For:

City of Torrance
3350 Civic Center Drive
Torrance, CA 90503

Prepared By:

UR

URBAN RESOURCE
CONSULTING CIVIL ENGINEERS

Urban Resource Corporation
23 Mauchly, Suite 110
Irvine, CA 92618

ATTACHMENT 3, PG 1-29

TORRANCE COMMUNITY SPORTS FIELD
City of Torrance, CA

STANDARD URBAN STORMWATER MITIGATION PLAN (SUSMP)

Prepared For:

City of Torrance
3350 Civic Center Drive
Torrance, CA 90503

Prepared By:

URBAN RESOURCE
CONSULTING CIVIL ENGINEERS

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23 Mauchly, Suite 110
Irvine, CA 92618

Terry P. Au, P.E.
State of California No. 68466

II. REFERENCES

1. A Manual for the Standard Urban Stormwater Mitigation Plan (SUSMP), Los Angeles County Department of Public Works, September 2002.
2. County of Los Angeles Low Impact Development Standards Manual, January 2009.
3. County of Los Angeles, Department of Public Works, Administrative Manual, GS200.1, June 1, 2011.

TABLE OF CONTENTS

SITE DESCRIPTION

The following site is located in the City of Torrance, California, at 555 Maple Drive, at the northwest corner of the intersection of Civic Center Drive and Maple Drive. The total area within the property boundary is 6.42 acres.

Existing Condition

The site is currently an existing field with fair to poor vegetative cover consisting of grass and weeds. The existing site is 100% pervious. Existing site topography is relatively flat, and drainage runs southerly and easterly towards towards the intersection of Maple Drive and Civic Center Drive. All drainage patterns flow via surface.

The Los Angeles County Hydrology Manual Soil Type for the site is 010.

Proposed Condition

The proposed site will consist of grass sports fields, bathrooms, fencing, gates, and storage areas. No onsite parking is provided.

Perviousness for this project is approximately 100%, but for the purpose of this analysis, an imperviousness of 10% is used for the "Local Parks" designation per Appendix D of the County of Los Angeles Hydrology Manual.

Drainage from the site will be collected via the proposed subsurface drainage system located below the field. Any flows not collected by the system will naturally flows to existing streets, and enter the existing catch basin located at the southeast corner of the site, in Maple Drive.

STANDARD URBAN STORMWATER MITIGATION PLAN

The following site is a redevelopment project, and per Standard Urban Stormwater Mitigation Plan guidelines, will require a SUSMP. The project will incorporate infiltration to meet the Low Impact Development criteria per the County of Los Angeles Low Impact Development Standards Manual.

SITE DESIGN BMPs

Site design BMPs for Torrance Community Sports Field include the grass fields located upstream of the proposed bio-swales.

LID/TREATMENT CONTROL BMPs

LID/Treatment Control BMP options are provided in the following table. Water quality calculations are provided herewith.

Name	Included?		If not applicable, state brief reason
	Yes	No	
Vegetated (Grass) Strips		x	N/A; Infiltration Trench and Filter Proposed
Vegetated (Grass) Swales		x	N/A; Infiltration Trench and Filter Proposed
Proprietary Control Measures		x	N/A; Infiltration Trench and Filter Proposed
Dry Detention Basin		x	N/A; Infiltration Trench and Filter Proposed
Wet Detention Basin		x	N/A; Infiltration Trench and Filter Proposed
Constructed Wetland		x	N/A; Infiltration Trench and Filter Proposed
Detention Basin/Sand Filter		x	N/A; Infiltration Trench and Filter Proposed
Name	Included?		If not applicable, state brief reason
	Yes	No	
Porous Pavement Detention		x	N/A; Infiltration Trench and Filter Proposed
Porous Landscape Detention		x	N/A; Infiltration Trench and Filter Proposed
Infiltration Basin		x	N/A; Infiltration Trench and Filter Proposed
Infiltration Trench	x		
Media Filter	x		
Other		x	N/A; Infiltration Trench and Filter Proposed

CONCLUSION

Water quality treatment to meet SUSMP and LID requirements for Torrance Community Sports Field will be provided by an infiltration trench and Pretreatment Triton Filter located at the bottom of a 36” Brooks Box. Sizing and details of the proposed system can be found in the Appendix and in the Precise Grading Plan.

The proposed system will infiltrate the required volume, and provide pretreatment for infiltrated flows. Calculations are provided herewith in Appendix A.

APPENDIX A

WATER QUALITY CALCULATIONS

TORRANCE SPORTS PARK
LID VOLUME CALCULATIONS

PARAMETERS

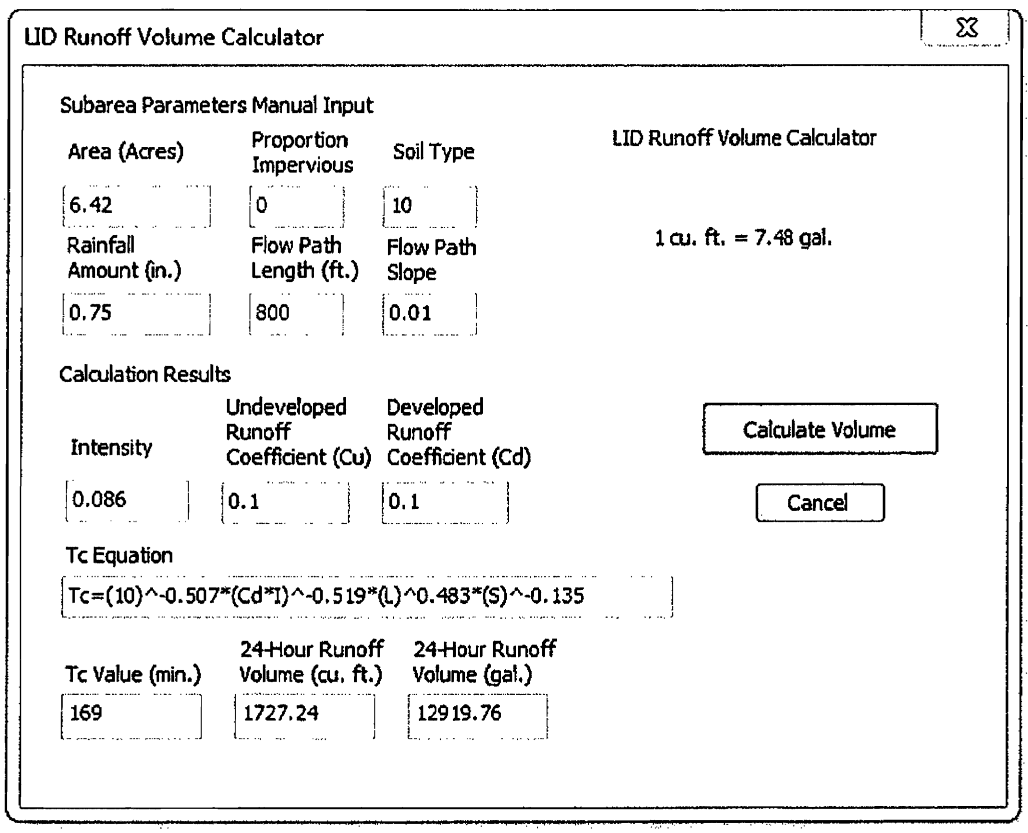
STORM EVENT: 2 YR 24 HOUR
RAINFALL AMOUNT: 0.75"
FLOW PATH LENGTH=800' (EXIST. AND PROPOSED CONDITION)

IMPERVIOUSNESS (EXIST.) = 0%
IMPERVIOUSNESS (PROPOSED) = 10%*

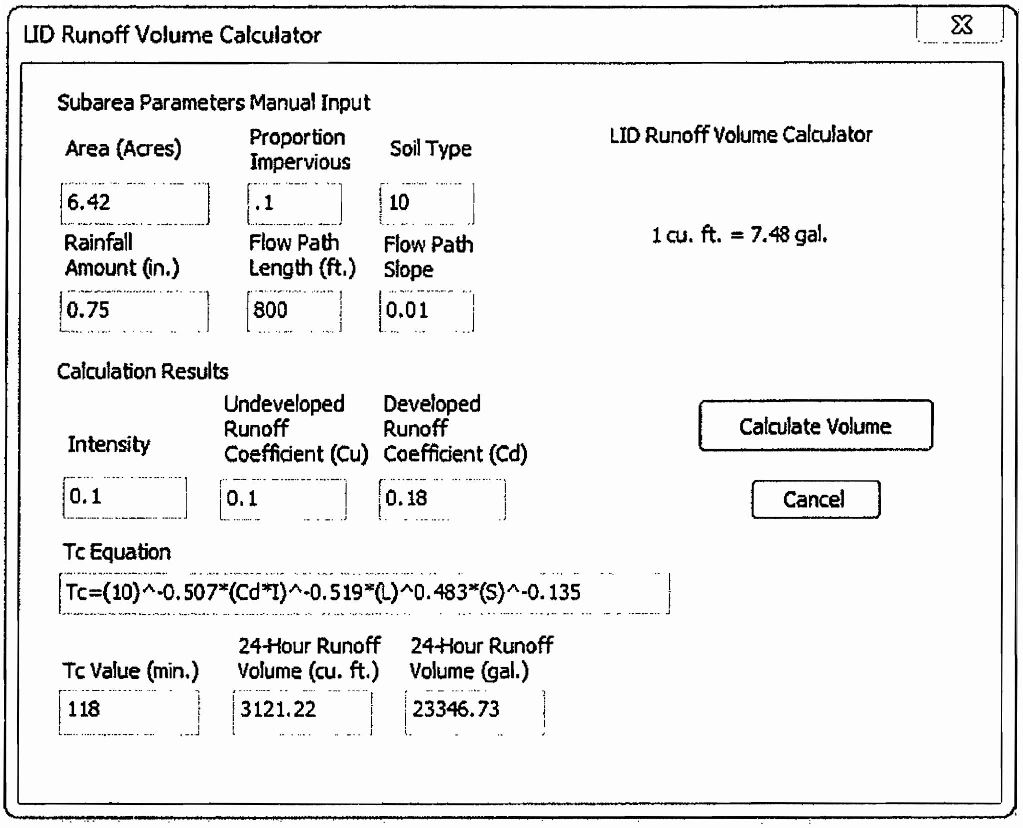
*Proposed Condition Imperviousness per LA County Hydrology Manual for "Local Parks"

CALCULATIONS

Undeveloped Runoff Volume:


LID Runoff Volume Calculator
Subarea Parameters Manual Input
Area (Acres): 6.42
Proportion Impervious: 0
Soil Type: 10
Rainfall Amount (in.): 0.75
Flow Path Length (ft.): 800
Flow Path Slope: 0.01
LID Runoff Volume Calculator
1 cu. ft. = 7.48 gal.
Calculation Results
Intensity: 0.086
Undeveloped Runoff Coefficient (Cu): 0.1
Developed Runoff Coefficient (Cd): 0.1
Calculate Volume
Cancel
Tc Equation
Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Tc Value (min.): 169
24-Hour Runoff Volume (cu. ft.): 1727.24
24-Hour Runoff Volume (gal.): 12919.76

Developed Runoff Volume:



LID Runoff Volume Calculator

Subarea Parameters Manual Input

Area (Acres)	Proportion Impervious	Soil Type
6.42	.1	10

Rainfall Amount (in.)	Flow Path Length (ft.)	Flow Path Slope
0.75	800	0.01

LID Runoff Volume Calculator

1 cu. ft. = 7.48 gal.

Calculation Results

Intensity	Undeveloped Runoff Coefficient (Cu)	Developed Runoff Coefficient (Cd)
0.1	0.1	0.18

Calculate Volume

Cancel

Tc Equation

Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135

Tc Value (min.)	24-Hour Runoff Volume (cu. ft.)	24-Hour Runoff Volume (gal.)
118	3121.22	23346.73

Excess Volume (ΔV):

Developed – Undeveloped = 3121cf – 1727 cf = <u>1394 cf (To be Infiltrated)</u>

Infiltration Trench Calculations:

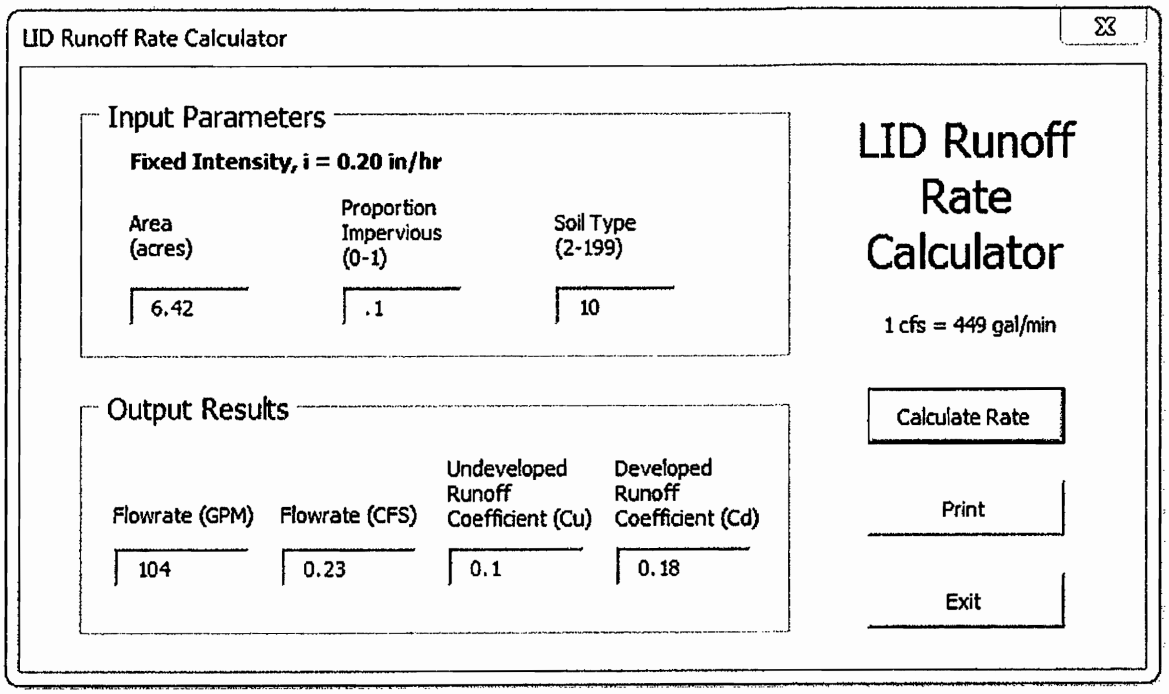
Proposed System Dimensions = 35' x 35'
Depth = 3'
Porosity = 0.40

Volume Storage Provided = 35 x 35 x 3 x 0.40 = **<u>1470 cf</u>**

Field Measured Infiltration Rate = 0.666 in/hr to 0.819 in/hr (report provided herewith)
Factor of Safety = 2
Design Infiltration Rate = 0.33 in/hr

Drawdown Time for a Full Trench = [(3ft x 0.40) x 1ft/12in]/(0.33in/hr) = **<u>44 hours</u>**

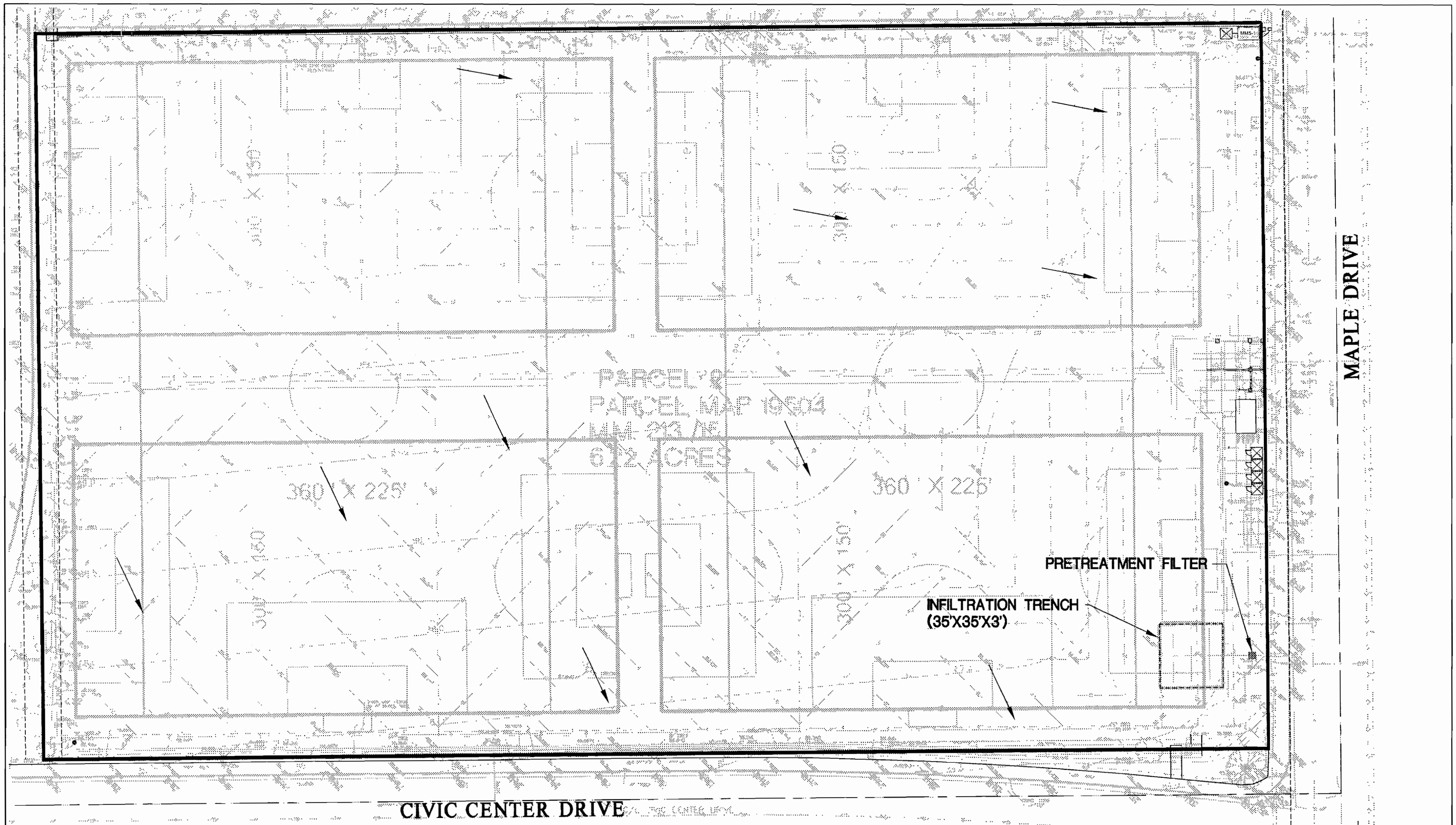
Flowrate Calculation (Pretreatment):



- ⇨ **The proposed Triton 180 Filter, Model No. TR14-180 (32" HT) has a filtration treatment capacity of 1.45cfs and has more than enough capacity to pretreat flow before infiltration.**
- ⇨ **The Triton Filter Insert will be located at the bottom of a proposed 36" Brooks Box per the Precise Grading Plan.**

APPENDIX B

SUSMP



LEGEND

- PROJECT BOUNDARY
- SURFACE OR STORM DRAIN FLOW DIRECTION
- PROPOSED INFILTRATION TRENCH
- 36" BROOKS BOX/TRITON TR14-180 (32")

NO.	DATE	REVISIONS	APP.	DATE
6				
5				
4				
3				
2				
1				

ENGINEER'S SEAL

PREPARED BY:
URBANRESOURCE
CONSULTING CIVIL ENGINEERS
23 Mauchly, Suite 110
Irvine, CA 92618
Phone: 949-727-9095
Fax: 949-727-9096

PREPARED FOR:
CITY OF TORRANCE

COMMUNITY SPORTS FIELDS
555 MAPLE AVENUE
SUSMP

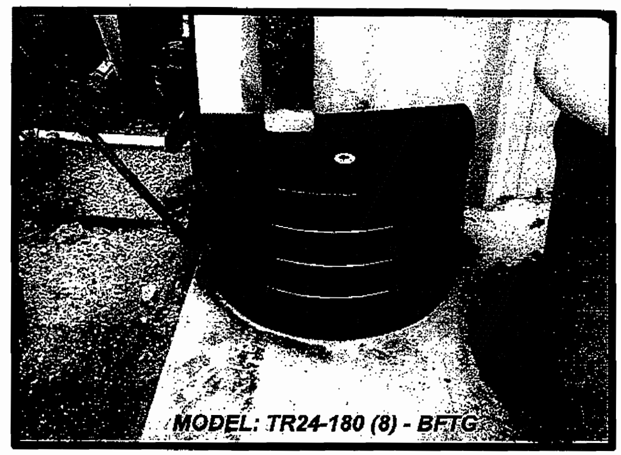
CITY OF TORRANCE

SHEET __ OF __

PLAN SET: A
DATE: 5-14-13
PLAN CONTROL
CITY OF TORRANCE

TRITON 180 FILTRATION SERIES for Downspouts

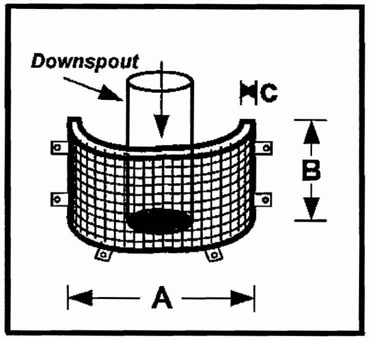
REM's TRITON 180 Filter Series are universally designed for downspout applications of all types and sizes.

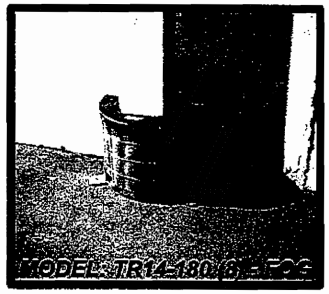


STANDARD DIMENSIONS
(CUSTOM SIZES AVAILABLE UPON REQUEST)

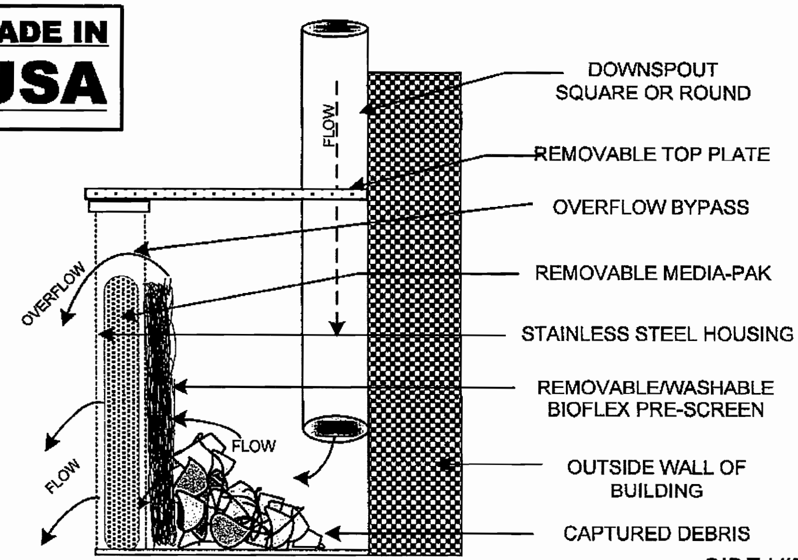
MODEL NO.	A	B	C
TR14-180 (8")	14"	9"	1.5"
TR14-180 (16")	14"	17"	1.5"
TR14-180 (32")	14"	33"	1.5"
TR24-180 (17")	24"	17"	2"
TR24-180 (34")	24"	34"	2"

Overflow Bypass
Top Sections of the TRITON 180 Downspout Filters are designed so that they are able to release the larger storm events when needed.





MADE IN USA



SIDE VIEW

Notes:

- TRITON 180 Series Filters are universally designed for downspouts of all types and sizes.
- The TRITON 180 Series Filters can be equipped with different media and prescreen combinations to address specific flow rates and removal requirements.
- For larger flows and debris capacities please see our TRITON 360 Series.
- Filter Media-Pak's can be equipped with media that capture: fats, oil, grease, metals, bacterial growth and many other runoff pollutants.
- Filters configured with FOG media polishes flow to remove TSS's including but not limited to fine sediment, silt and debris. Heavy metals may be reduced and/or removed from flow through agglomeration.
- FOG media encapsulates hydrocarbons including Fats, Oil and Grease.
- Filters configured with Biofex Media and BFTG Prescreen are designed for increased flow and capture 100% of particles that are 5mm or greater in size.
- Filters are constructed using Non-reactive High Density Polyethylene Plastic (HDPE) with U.V. inhibitors added.
- The interior & exterior cage of the cartridges are made from Type 304 Stainless Steel configured with 12 gauge welded 1.75" X 2" square openings.
- REM will help develop custom sizes to fit most applications.
- Bioflex Prescreens are constructed using Polyester Fiber Mesh (Netting), a blend of coir fibers (fiber), and water based latex (as the binding agent).
- BioFlex shall be biodegradable and may often be washed and re-used.
- Bioflex Prescreen UV Resistance (ASTM D 4355 – 500 hour exposure)
- Bioflex Tensile Properties (ASTM D 5035/ECTC).
- Sediment Control (ASTM D 5141).
- Please refer to manufacture's recommendations for an approved maintenance program.

Patent Pending

PH: (888) 526-4736

DIMENSIONS ARE IN INCHES UNLESS OTHERWISE NOTED.

REM Inc.

TRITON 180 DOWNSPOUT FILTRATION SERIES
(Multiple Applications)

SIZE	DRAWN BY:	FOR:	REV
	C.F.	Downspout Filtering	
SCA 1/4 : 1	DATE: 3/22/2009	SHE 1 OF 1	

SPECIFIER CHART TRITON 180 & 360 SERIES FILTERS	FILTERING OF DOWNSPOUTS (1" THRU 8" Dia.) (2"SQ TO 8"SQ)	MEDIA REM - BFTG* FILTERED Flow Rate	MEDIA REM - FOG** FILTERED Flow Rate	MEDIA REM - FOG & BFTG*** FILTERED Flow Rate	TOTAL FILTER BYPASS Flow Rate	DEBRIS HOLDING CAPACITY
MODEL:	*(inch)*	CFS	CFS	CFS	CFS	CUBIC FEET
			Standard			
14" - 180 DOWNSPOUT FILTER	1" TO 10" Dia.					
TR14-180 (8" HT.)		1.3	0.35	0.35	6.58	0.19
TR14-180 (16" HT.)		2.59	0.72	0.72	6.58	0.39
TR14-180 (32" HT.)		5.18	1.45	1.45	6.58	0.83
24" - 180 DOWNSPOUT FILTER	1" TO 10" Dia.					
TR24-180 (17" HT.)		5.38	1.21	1.21	25.65	1.37
TR24-180 (34" HT.)		10.75	2.42	2.42	25.65	2.91
14" - 360 DOWNSPOUT FILTER	1" TO 10" Dia.					
TR14-360 (8" HT.)		2.59	0.69	0.69	13.15	0.385
TR14-360 (16" HT.)		5.18	1.43	1.43	13.15	0.78
TR14-360 (32" HT.)		10.36	2.9	2.9	13.15	1.66
24" - 360 DOWNSPOUT FILTER	1" TO 10" Dia.					
TR24-360 (17" HT.)		10.75	2.42	2.42	51.29	2.74
TR24-360 (34" HT.)		21.5	4.84	4.84	51.29	5.82

Notes:

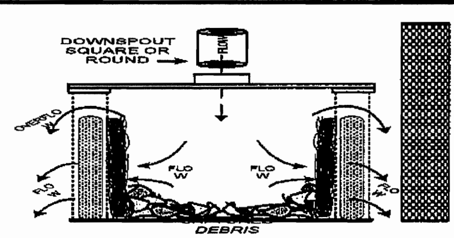
§ Standard cartridge configuration for the TRITON 180 & 360 Downspout Series filter. Please specify if larger capacity cartridges are required. The cartridge heights are listed in the ( ) of the model number. Depending upon available space and configuration of downspout, height adjustments can be made.

* REM - BFTG: Bioflex (BFTG) Media is designed to capture debris, trash and sediment while sustaining very high treatment rates. Mesh density of 3.5 ounces per square foot minimizes occlusion and blinding while capturing 100% of particles at 5mm or greater in size.

** REM - FOG: The FOG media is housed in a mono-filament weaved geotextile containment pack. FOG media effectively encapsulates liquefied petroleum hydrocarbons (Fats, Oils & Grease including animal fats). It's highly hydrophobic characteristic allows for increased polish of flow resulting in the reduction of Total Suspended Solids (TSS). TSS reduction includes (but is not limited to) debris, trash, silt, sediment and agglomerated heavy metals. REM- FOG is the standard media that is configured for Top Hats. Media options for other pollutants and particulate sizes are also available.

*** REM - FOG -BFTG: Media configuration utilizes both BFTG and FOG media strategies. The BFTG Media serves as a pre-screen to treat for larger debris, such as trash, leaves etc... The FOG media pack captures finer suspended solids and liquefied hydrocarbons.

REM technical support is available to assist with TRITON Series filter configurations, media strategies and customization of models.



REM
www.remfilters.com

Revel Environmental Manufacturing Inc.
sales@remfilters.com (888) 526-4736 Lic. No. 857410

Northern California
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F: (925) 676-8676

Southern California
2110 South Grand Avenue
Santa Ana, California 92705
P: (714) 557-2676
F: (714) 557-2679

Filter Series	Drawing No.	Date	Sheet
TRITON 180 & 360 FILTER	TDSF -0001	3/3/2013	1 of 1

Maintenance

The Triton catch basin inserts should be inspected at regular intervals and maintained when necessary to ensure optimum performance.

The useful life of the media-pak is based on the quantity of pollutants it collects. The media-pak with XSORB® media has the ability to repel water and absorb up to 500 percent of its own weight in oil when fully saturated. A typical TR24SR cartridge has the ability to absorb between 1 and 2 quarts of contaminants when fully saturated. On average we suggest three cleanouts of the cartridge basin and one change out or replacement of the media-pak per year. For Triton catch basin inserts placed in areas of greater pollutant loading the maintenance and change out frequency will be increase.

Life expectancy of the Triton catch basin inserts is anticipated to be excellent because of the materials of construction. The non-reactive high density polystyrene plastic with U.V. inhibitors and fiberglass housings provide excellent protection against damp and corrosive environments. The cartridge components including the stainless steel wire cage, geotextile polypropylene fabric are all considered durable and have good longevity in the environment.

On-site Procedures for Inspection and Maintenance

- Secure traffic and pedestrian traffic with cones, barrels, etc.
- Clean surface area immediate around each catch basin
- Remove grates and set aside
- Clean grates, remove litter and debris that may be trapped within the grate
- Inspect perimeter gasket system of the cartridge making sure no flows are bypassing the cartridge, repair as needed.
- Remove by vactor hose the debris that has been trapped in the trough area. Dispose of in accordance with local, state and federal regulatory agency requirements. Most debris that is captured in the trough or sump area will fall into the non-hazardous waste category.
- Visually inspect and chech the condition of the trough area.
- Inspect the media-pak condition in the wire mesh cartridge. When the normally white colored media turns black, the media should be changed. When service requires replacement of the cartridge media-pak please contact your local CONTECH Stormwater Solutions office.
- Replace grate and lockdown as needed.
- Secure and date weatherproof lock out tags.
- Report any concerns or improvements regarding the Triton insert on a service report.
- Un-secure traffic control area.
- Complete service report and submit to facility owner.

13

Infiltration Facility Operations and Maintenance

General Requirements

Infiltration facility maintenance should include frequent inspections to ensure that water infiltrates into the subsurface completely within the recommended infiltration time of 72 hours or less after a storm (see Appendix E for guidance on facility inspection and Appendix F for an infiltration inspection and maintenance checklist).

Maintenance and regular inspections are of primary importance if infiltration basins and trenches are to continue to function as originally designed. A specific maintenance plan shall be developed specific to each facility outlining the schedule and scope of maintenance operations, as well as the documentation and reporting requirements. The following are general maintenance requirements:

1. Regular inspection should determine if the sediment pretreatment structures require routine maintenance.

2. If water is noticed in the basin more than 72 hours after a major storm or in the observation well of the infiltration trench more than 48 hours after a major storm, the infiltration facility may be clogged. Maintenance activities triggered by a potentially clogged facility include:

   - Check for debris/sediment accumulation, rake surface and remove sediment (if any) and evaluate potential sources of sediment and vegetative or other debris (e.g., embankment erosion, channel scour, overhanging trees, etc). If suspected upland sources are outside of the County's jurisdiction, additional pretreatment operations (e.g., trash racks, vegetated swales, etc.) may be necessary.

   - For basins, removal of the top layer of native soil may be required to restore infiltrative capacity.

   - For trenches, assess the condition of the top aggregate layer for sediment buildup and crusting. Remove top layer of pea gravel and replace. If slow draining conditions persist, entire trench may need to be excavated and replaced.

3. Any debris or algae growth located on top of the infiltration facility should be removed and disposed of properly.

4. Facilities should be inspected annually. Trash and debris should be removed as needed, but at least annually prior to the beginning of the wet season.

5. Site vegetation should be maintained as frequently as necessary to maintain the aesthetic appearance of the site, and as follows:

   - Vegetation, large shrubs, or trees that limit access or interfere with basin operation should be pruned or removed.

- Slope areas that have become bare should be revegetated and eroded areas should be regraded prior to being revegetated.
- Grass should be mowed to 4"-9" high and grass clippings should be removed.
- Fallen leaves and debris from deciduous plant foliage should be raked and removed.
- Invasive vegetation, such as Alligatorweed (*Alternanthera philoxeroides*), Halogeton (*Halogeton glomeratus*), Spotted Knapweed (*Centaurea maculosa*), Giant Reed (*Arundo donax*), Castor Bean (*Ricinus communis*), Perennial Pepperweed (*Lepidium latifolium*), and Yellow Starthistle (*Centaurea solstitalis*) must be removed and replaced with non-invasive species. Invasive species should never contribute more than 25% of the vegetated area. For more information on invasive weeds, including biology and control of listed weeds, look at the "encycloweedia" located at the California Department of Food and Agriculture website at http://www.cdfa.ca.gov/wma or the California Invasive Plant Council website at http://portal.cal-ipc.org/weedlist. .
- Dead vegetation should be removed if it exceeds 10% of area coverage. Vegetation should be replaced immediately to maintain cover density and control erosion where soils are exposed.

6. For infiltration basins, sediment buildup exceeding 50% of the forebay sediment storage capacity, as indicated by the steel markers, should be removed. Sediment from the remainder of the basin should be removed when 6 inches of sediment accumulates. Sediments should be tested for toxic substance accumulation in compliance with current disposal requirements if visual or olfactory indications of pollution are noticed. If toxic substances are encountered at concentrations exceeding thresholds of Title 22, Section 66261 of the California Code of Regulations, the sediment must be disposed of in a hazardous waste landfill and the source of the contaminated sediments should be investigated and mitigated to the extent possible.

7. Following sediment removal activities, replanting and/or reseeding of vegetation may be required for reestablishment.

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Maintenance Standards

A summary of the routine and major maintenance activities recommended for infiltration facilities is shown in Table 6-1. Detailed routine and major maintenance standards are listed in Tables 6-2 and 6-3.

Table 6-1: Infiltration Facility Routine and Major Maintenance Quick Guide

	Inspection and Maintenance Activities Summary
Routine Maintenance	• Remove trash and debris as required • Repair and reseed erosion near inlet if necessary • Remove any visual evidence of contamination from floatables such as oil and grease • Clean under-drain (if present) and outlet piping to alleviate ponding and restore infiltrative capacity. • Remove minor sediment accumulation, debris and obstructions near inlet and outlet structures as needed • Mow routinely to maintain ideal grass height and to suppress weeds • Periodically observe function under wet weather conditions • Take photographs before and after maintenance (encouraged)
Major Maintenance	• Clean out under-drains if present to alleviate ponding. Replace media if ponding or loss of infiltrative capacity persists and revegetate • Repair structural damage to flow control structures including inlet, outlet and overflow structures • De-thatch grass to remove accumulated sediment and aerate compacted areas to promote infiltration

Table 6-2: Routine Maintenance – Infiltration Facilities

<table>
<tr><th>Defect</th><th>Conditions When Maintenance Is Needed</th><th>Results Expected When Maintenance Is Performed</th><th>Frequency</th></tr>
<tr><td>Trash & Debris</td><td>Any trash and debris which exceed 5 cubic feet per 1,000 square feet (one standard garbage can). In general, there should be no visual evidence of dumping.
If less than threshold, all trash and debris will be removed as part of next scheduled maintenance.</td><td>Trash and debris cleared from site.</td><td rowspan="5">Annually prior to wet season.
After major storm events (>0.75 in/24 hrs) if spot checks indicate widespread damage/ maintenance needs.
Litter removal is dependent on site conditions and desired aesthetics and should be done at a frequency to meet those objectives.</td></tr>
<tr><td>Inlet Erosion</td><td>Visible evidence of erosion occurring near inlet structures.</td><td>Eroded areas repaired/reseeded</td></tr>
<tr><td>Visual Contaminants and Pollution</td><td>Any evidence of oil, gasoline, contaminants or other pollutants.</td><td>No contaminants or pollutants present.</td></tr>
<tr><td>Slow Drain Time</td><td>Standing water long after storm has passed (after 48 to 72 hours), or visual inspection of wells (if available) indicates that design drain times are not being achieved.</td><td>Water drains within 48 to 72 hours. Drainage pipe is cleared, accumulated litter on surface is removed, and top 1-2" of soil is raked or replaced.</td></tr>
<tr><td>Inlets Blocked</td><td>Trash and debris or sediment blocking inlet structures.</td><td>Inlets clear and free of trash and debris.</td></tr>
<tr><td>Appearance of Poisonous, Noxious or Nuisance Vegetation</td><td>Excessive grass and weed growth. Noxious weeds, woody vegetation establishing, Turf growing over rock filter.</td><td>Vegetation is mowed or trimmed to restore function. Weeds are removed to prevent noxious and nuisance plants from becoming established.</td><td>Monthly (or as dictated by agreement between County and landscape contractor).</td></tr>
</table>

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Table 6-3: Major Maintenance – Infiltration Facilities

Defect	**Conditions When Maintenance Is Needed**	**Results Expected When Maintenance Is Performed**	**Frequency**
Standing Water	Standing water long after storm has passed (after 24 to 48 hours), or visual inspection of wells (if available) indicates that design drain times are not being achieved	Design infiltration rate restored, either through excavation and filter media replacement or surface sediment removal. If applicable, underdrain cleaned, reset or replaced.	As needed

APPENDIX C

RESULTS OF PERCOLATION/INFILTRATION TESTING

LOS ANGELES COUNTY

25050 Avenue Kearny, Suite 110A
Los Angeles, CA 91355
T: 661.255.5790 F: 661.255.5242

August 7, 2013
J.N. 13-248

LAND CONCERN
1750 East Deere Avenue
Santa Ana, CA 92705

Attention: Mr. Philip Stevens

Subject: Results of Percolation/Infiltration Testing, Proposed Community Sports Fields, 555 Maple Avenue, Torrance, California

Dear Mr. Stevens:

At your request, **Petra Geotechnical, Inc. (Petra)** has completed percolation/infiltration testing along the eastern boundary of the proposed community sports fields at 555 Maple Avenue in Torrance, California. The purpose of this study was to evaluate the feasibility of using infiltration in the design of the stormwater system. Two percolation tests borings were performed. The locations and depths of the percolation tests were based on communications with the project Civil Engineer.

BACKGROUND

The subject site, which is a vacant dirt lot, is proposed to be developed as a sports field complex. This firm conducted a geotechnical study in June of 2013 (see References). This study included the excavation of 6 backhoe test pits. Information presented in that study was used in the site evaluation for the percolation testing. As per communication with the project civil engineer, Urban Resource, it is our understanding that infiltration of surface waters/runoff is being considered for the design of the stormwater system. The proposed infiltration areas are located along the southeastern property boundary. Two possible locations were investigated by the drilling of two hollow stem borings. The approximate locations of the exploratory borings (PT-1 and PT-2) are indicated on the attached Site Map (Figure 1).

FIELD INVESTIGATION

The field exploration for this study, performed August 2, 2013, included drilling two percolation test borings (PT-1 and PT-2) and conducting percolation tests as per County of Los Angeles Department of Public Works, Geotechnical and Materials Engineering Division guidelines. The percolation test borings were drilled to an approximate depth of 12 feet below the existing ground surface with a CME 55 truck-mounted drilling rig using an eight-inch diameter continuous flight hollow stem auger. The logs of the percolation test borings are included in Appendix A.

Based on the percolation test borings, and the information from the previous study, the site is mantled by 1 to 2 feet of artificial fill which is underlain by native alluvial soil. The alluvial soils are typically a very fine grained sand with some secondary silt and clay fractions. The upper 6 inches to 1 foot of alluvium is weathered with rootlets and minor porosity. The alluvium becomes dense to very dense with depth. Occasional sandy layers were noted. No groundwater was encountered to the depths explored (12+ feet). Based on review of published groundwater data, the depth to groundwater in this area of Torrance is greater than 50 feet.

PERCOLATION TESTING

Following the drilling and logging of the 8 inch hollowstem auger borings, a three inch (I. D.) perforated P. V. C. pipe was placed in the test holes. Based on communication from the Civil Engineer, the depth of the proposed infiltration zone is approximate 8 to 10 feet below existing grades. Therefore, the perforated zone extended from 7 to 12 feet. Gravel ($^3/_8$ to ¾ inch) was then placed within the annular space between the pipe and boring wall. Relatively clean water was then added to the borings to pre-soak the adjacent soils prior to performance of the percolation test. As per County guidelines the water level for the presoak was a minimum of one foot above the bottom of the borings. Water seeped out of the holes within 30 minutes on two consecutive pre-soak procedures. The holes were then filled to approximately 8 feet. The water level dropped to the bottom of the hole within 10 minutes. With this being the case and per County guidelines, the percolation tests were then conducted by filling the test holes with clean water to approximately 8 feet of the surface. The water level was measured at approximately 4 to 10-minute intervals and then readjusted to approximately the initial water elevation. From these readings, the percolation characteristics of the underlying alluvial soils were estimated. Percolation test results are included in Appendix A and are summarized below:

Table 1 –Percolation Test Results

Test No.	Soil Type (USCS)	Depth of Hole (Feet)	Infiltration Rate (Inches/Hour)
PT-1	SM	12	0.666
PT-2	SM	12	0.819

It should be noted that the absorption/infiltration rate incorporates the required Reduction Factor for nonvertical flow and is based on the lowest rate observed.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based upon the data obtained from the field exploration and percolation testing conducted on the project site and on **Petra's** understanding of the proposed construction.

1. Two infiltration tests (Borings PT-1 and PT-2) were conducted for this study to provide information on the feasibility of using infiltration for stormwater disposal. Absorption/infiltration rates varying from 0.666 to 0.819 inches/hour were determined. These values are greater than the minimum acceptable infiltration rate of 0.5 inches per hour.

2. It should be noted that the percolation test was conducted with relatively clean water. Nuisance water, which contains sediments and other impurities, may reduce the soil absorption rate.

3. The stormwater system should be designed according to the standards set by the City of Torrance, or other jurisdictional agencies. An appropriate safety factor should be used for preliminary design calculations.

REPORT LIMITATIONS

The conclusions and recommendations submitted in this report relative to the proposed development are based, in part, upon the data obtained from one percolation test boring, site observations during the field exploration operations, and past experience. Variations between observed conditions may become evident during construction. If variations are observed, it will be necessary to re-evaluate the recommendations of this report.

This report has been prepared consistent with the level of care being provided by other professionals providing similar services at the same locale and in the same time period. This report provides our professional opinions and, as such, they are not to be considered a guaranty or warranty.

This report has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

We sincerely appreciate this opportunity to be of service. Please do not hesitate to call the undersigned if you have any questions regarding this report.

Respectfully submitted,
PETRA GEOTECHNICAL, INC.

Theo M. Wolfe

Theodore M. Wolfe
Associate Geologist
CEG 1626

TMW/kg

Attachments: Percolation Test Location Map – Figure 1

Appendix A –

Boring Logs
Percolation Test Summary

REFERENCES

California Division of Mines and Geology, 1997, Seismic Hazard Zone Report for the Torrance 7.5-Minute Quadrangle, Los Angeles County, California: California Department of Conservation Division of Mines and Geology, Seismic Hazard Zone Report 035 (official map Released March 25, 1999).

California Geological Survey, 2002, Probabilistic Seismic Hazard Assessment for the State of California, Open-File Report 96-08, Revised 2002 California Seismic Shaking Analysis, , Appendix A.

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Cao, T., et al., 2003, Revised 2002 California Probabilistic Seismic Hazard Maps, June 2003: California Geological Survey.

Petra Geotechnical, Inc., 2013 Summary Report of Geotechnical Field Investigation for Proposed Community Sports Fields, 555 Maple Avenue, Torrance, California, June 3, Project 13-248.

United States Geologic Survey (U.S.G.S.), 1996a, Probabilistic Seismic Hazard Assessment for the State of California, Open-File Report 96-706.

______, 1996b, National Seismic-Hazards Maps, Open-File Report 96-532.

______, 2002, Documentations for the 2002 Update of the National Seismic Hazard Maps, Open-File Report 02-20.

______, 2007, Preliminary Documentation for the 2007 Update of the United States National Seismic Hazard Maps, Seismic Hazards Mapping Project, Open-File Report 2007-June Draft.

APPENDIX A

BORING LOGS

PERCOLATION TEST SUMMARIES

Boring PT-1

Depth (ft.)	Sample	Blow Count (per foot)	Dry Density (pcf	Moisture Content (%)	Description
0-2					**ARTIFICIAL FILL** Silty SAND (SM) - yellow brown, fine-grained, moist, medium dense
2-12					**ALLUVIUM (Qal)** Silty SAND (SM) - yellow brown, fine-grained, moist, medium dense
					End boring at 15 feet. No caving. No groundwater.

Boring PT-2

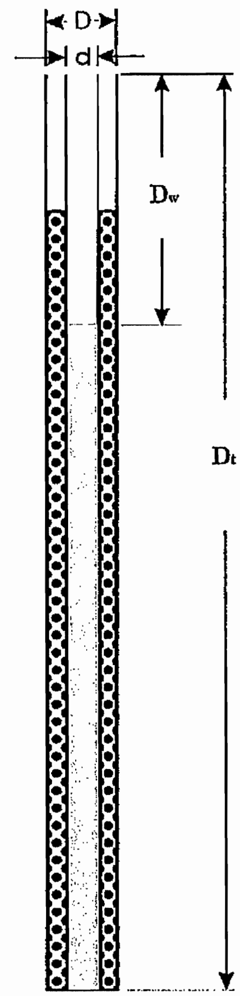
Depth (ft.)	Sample	Blow Count (per foot)	Dry Density (pcf	Moisture Content (%)	Description
0-1					**ARTIFICIAL FILL** Silty SAND (SM) - yellow brown, fine-grained, moist, medium dense
1-12					**ALLUVIUM (Qal)** Silty SAND (SM) - yellow brown, fine-grained, moist, medium dense
					End boring at 15 feet. No caving. No groundwater.

PERCOLATION TEST SUMMARY

Job No. 13-248
Project Name: Torrance Sports Fields- 555 Maple Avenue
Date: 8/1/13

Test Number: PT-1

Depth to Bottom,ft (D_t): 12
Diameter of Hole, in (D): 8
Diameter of Pipe, in (d): 3
Agg. Correction (% Voids): 45.00



Time Interval (min)	Depth to Water Surface Dw (ft)		Change in Head (in)	Perc Rate gal/day/ft^2
	1st Reading	2nd Reading		
5	9.70	11.25	18.60	173.53
5	8.90	10.78	22.50	152.46
8	8.85	10.85	24.00	102.19
5	8.08	10.00	23.10	116.51
8	8.63	10.13	18.00	63.60
7	8.08	9.93	22.20	79.03
10	8.43	9.95	18.30	48.47
9	8.43	9.90	17.70	51.66

Preadjuested
Infiltration Rate:

48.47 gal/day/ft^2

3.24 Inches/Hour

Reduction Factor:

4.86

Infiltration Rate:

0.666 Inches/Hour

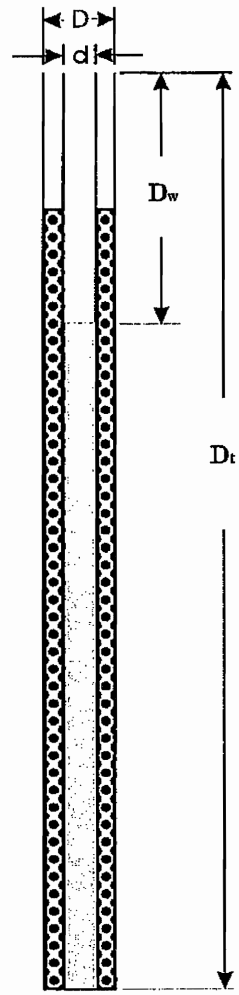
PETRA GEOTECHNICAL, INC.

PERCOLATION TEST SUMMARY

Job No. 13-248
Project Name: Torrance Sports Fields- 555 Maple Avenue
Date: 8/1/13

Test Number: PT-2

Depth to Bottom,ft (D_t): 12
Diameter of Hole, in (D): 8
Diameter of Pipe, in (d): 3
Agg. Correction (% Voids): 45.00



Time Interval (min)	Depth to Water Surface Dw (ft)		Change in Head (in)	Perc Rate gal/day/ft^2
	1st Reading	2nd Reading		
4	9.50	11.00	18.00	185.28
4	9.30	11.00	20.40	199.57
4	9.55	11.00	17.40	181.47
5	9.70	11.23	18.30	169.48
5	9.00	10.83	21.90	153.34
6	7.93	10.25	27.90	119.17

Preadjuested
Infiltration Rate:

119.17 gal/day/ft^2

7.97 Inches/Hour

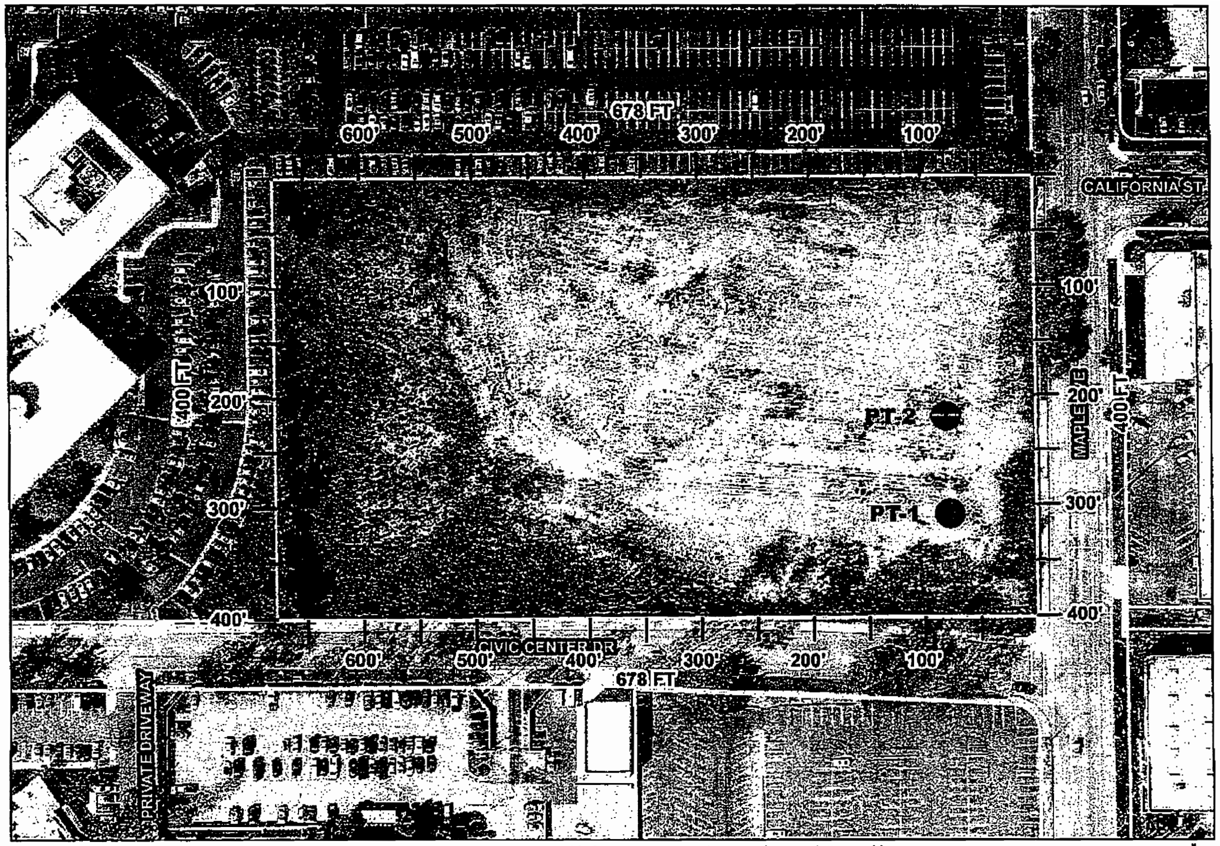
Reduction Factor:

9.72

Infiltration Rate:

0.819 Inches/Hour

PETRA GEOTECHNICAL, INC.



Lines and photos are approximate, not to be used for establishing absolute or relative positions

0 25 50 100 Feet
SCALE 1" : 100'

555 MAPLE AVE, TORRANCE
AREA: 271,037 SF

EXPLANATION

PT-2 ● PERCOLATION TEST LOCATION

PETRA GEOTECHNICAL, INC.
3185-A AIRWAY AVENUE
COSTA MESA, CALIFORNIA 92626
PHONE: (714) 549-8921
COSTA MESA MURRIETA PALM DESERT SANTA CLARITA

SITE PLAN

COMMUNITY SPORTS FIELD
555 MAPLE AVENUE, TORRANCE, CALIFORNIA

DATE: August, 2013	J.N.: 13-248	**Figure 1**
DWG BY: SSR	SCALE: 1"=100'	

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Four

LOS ANGELES COUNTY

25050 Avenue Kearny, Suite 110A
Los Angeles, CA 91355
T: 661.255.5790 F: 661.255.5242

PETRA

past + present + future
it's in our science

Engineers, Geologists
Environmental Scientists

August 7, 2013
J.N. 13-248

LAND CONCERN
1750 East Deere Avenue
Santa Ana, CA 92705

Attention: Mr. Philip Stevens

Subject: Results of Percolation/Infiltration Testing, Proposed Community Sports Fields, 555 Maple Avenue, Torrance, California

Dear Mr. Stevens:

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BACKGROUND

The subject site, which is a vacant dirt lot, is proposed to be developed as a sports field complex. This firm conducted a geotechnical study in June of 2013 (see References). This study included the excavation of 6 backhoe test pits. Information presented in that study was used in the site evaluation for the percolation testing. As per communication with the project civil engineer, Urban Resource, it is our understanding that infiltration of surface waters/runoff is being considered for the design of the stormwater system. The proposed infiltration areas are located along the southeastern property boundary. Two possible locations were investigated by the drilling of two hollow stem borings. The approximate locations of the exploratory borings (PT-1 and PT-2) are indicated on the attached Site Map (Figure 1).

ATTACHMENT 4, PG 1-10

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Based on the percolation test borings, and the information from the previous study, the site is mantled by 1 to 2 feet of artificial fill which is underlain by native alluvial soil. The alluvial soils are typically a very fine grained sand with some secondary silt and clay fractions. The upper 6 inches to 1 foot of alluvium is weathered with rootlets and minor porosity. The alluvium becomes dense to very dense with depth. Occasional sandy layers were noted. No groundwater was encountered to the depths explored (12+ feet). Based on review of published groundwater data, the depth to groundwater in this area of Torrance is greater than 50 feet.

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CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based upon the data obtained from the field exploration and percolation testing conducted on the project site and on **Petra's** understanding of the proposed construction.

1. Two infiltration tests (Borings PT-1 and PT-2) were conducted for this study to provide information on the feasibility of using infiltration for stormwater disposal. Absorption/infiltration rates varying from 0.666 to 0.819 inches/hour were determined. These values are greater than the minimum acceptable infiltration rate of 0.5 inches per hour.

2. It should be noted that the percolation test was conducted with relatively clean water. Nuisance water, which contains sediments and other impurities, may reduce the soil absorption rate.

3. The stormwater system should be designed according to the standards set by the City of Torrance, or other jurisdictional agencies. An appropriate safety factor should be used for preliminary design calculations.

REPORT LIMITATIONS

The conclusions and recommendations submitted in this report relative to the proposed development are based, in part, upon the data obtained from one percolation test boring, site observations during the field exploration operations, and past experience. Variations between observed conditions may become evident during construction. If variations are observed, it will be necessary to re-evaluate the recommendations of this report.

This report has been prepared consistent with the level of care being provided by other professionals providing similar services at the same locale and in the same time period. This report provides our professional opinions and, as such, they are not to be considered a guaranty or warranty.

This report has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

We sincerely appreciate this opportunity to be of service. Please do not hesitate to call the undersigned if you have any questions regarding this report.

Respectfully submitted,
PETRA GEOTECHNICAL, INC.

Theodore M. Wolfe
Associate Geologist
CEG 1626

TMW/kg

Attachments: Percolation Test Location Map – Figure 1

Appendix A –

Boring Logs
Percolation Test Summary

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APPENDIX A

BORING LOGS

PERCOLATION TEST SUMMARIES

Boring PT-1

Depth (ft.)	Sample	*Blow Count (per foot)*	*Dry Density (pcf*	*Moisture Content (%)*	*Description*
0-2					**ARTIFICIAL FILL** Silty SAND (SM) - yellow brown, fine-grained, moist, medium dense
2-12					**ALLUVIUM (Qal)** Silty SAND (SM) - yellow brown, fine-grained, moist, medium dense
					End boring at 15 feet. No caving. No groundwater.

Boring PT-2

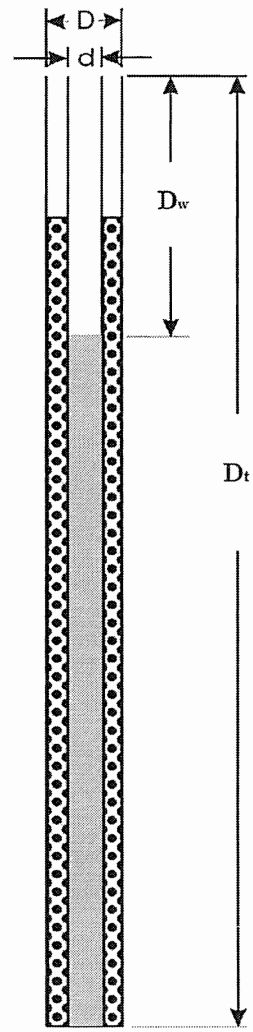
Depth (ft.)	Sample	*Blow Count (per foot)*	*Dry Density (pcf*	*Moisture Content (%)*	*Description*
0-1					**ARTIFICIAL FILL** Silty SAND (SM) - yellow brown, fine-grained, moist, medium dense
1-12					**ALLUVIUM (Qal)** Silty SAND (SM) - yellow brown, fine-grained, moist, medium dense
					End boring at 15 feet. No caving. No groundwater.

PERCOLATION TEST SUMMARY

Job No. 13-248
Project Name: Torrance Sports Fields- 555 Maple Avenue
Date: 8/1/13

Test Number: PT-1

Depth to Bottom,ft (D_t): 12
Diameter of Hole, in (D): 8
Diameter of Pipe, in (d): 3
Agg. Correction (% Voids): 45.00



Time Interval (min)	Depth to Water Surface Dw (ft)		Change in Head (in)	Perc Rate gal/day/ft^2
	1st Reading	2nd Reading		
5	9.70	11.25	18.60	173.53
5	8.90	10.78	22.50	152.46
8	8.85	10.85	24.00	102.19
5	8.08	10.00	23.10	116.51
8	8.63	10.13	18.00	63.60
7	8.08	9.93	22.20	79.03
10	8.43	9.95	18.30	48.47
9	8.43	9.90	17.70	51.66

Preadjuested
Infiltration Rate:

48.47 gal/day/ft^2

3.24 Inches/Hour

Reduction Factor:

4.86

Infiltration Rate:

0.666 Inches/Hour

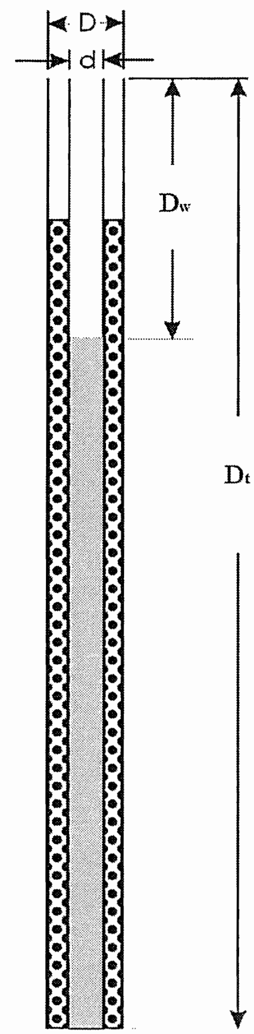
PETRA GEOTECHNICAL, INC.

PERCOLATION TEST SUMMARY

Job No. 13-248
Project Name: Torrance Sports Fields- 555 Maple Avenue
Date: 8/1/13

Test Number: PT-2

Depth to Bottom,ft (D_t): 12
Diameter of Hole, in (D): 8
Diameter of Pipe, in (d): 3
Agg. Correction (% Voids): 45.00



Time Interval (min)	Depth to Water Surface Dw (ft)		Change in Head (in)	Perc Rate gal/day/ft^2
	1st Reading	2nd Reading		
4	9.50	11.00	18.00	185.28
4	9.30	11.00	20.40	199.57
4	9.55	11.00	17.40	181.47
5	9.70	11.23	18.30	169.48
5	9.00	10.83	21.90	153.34
6	7.93	10.25	27.90	119.17

Preadjuested
Infiltration Rate:

119.17 gal/day/ft^2

7.97 Inches/Hour

Reduction Factor:

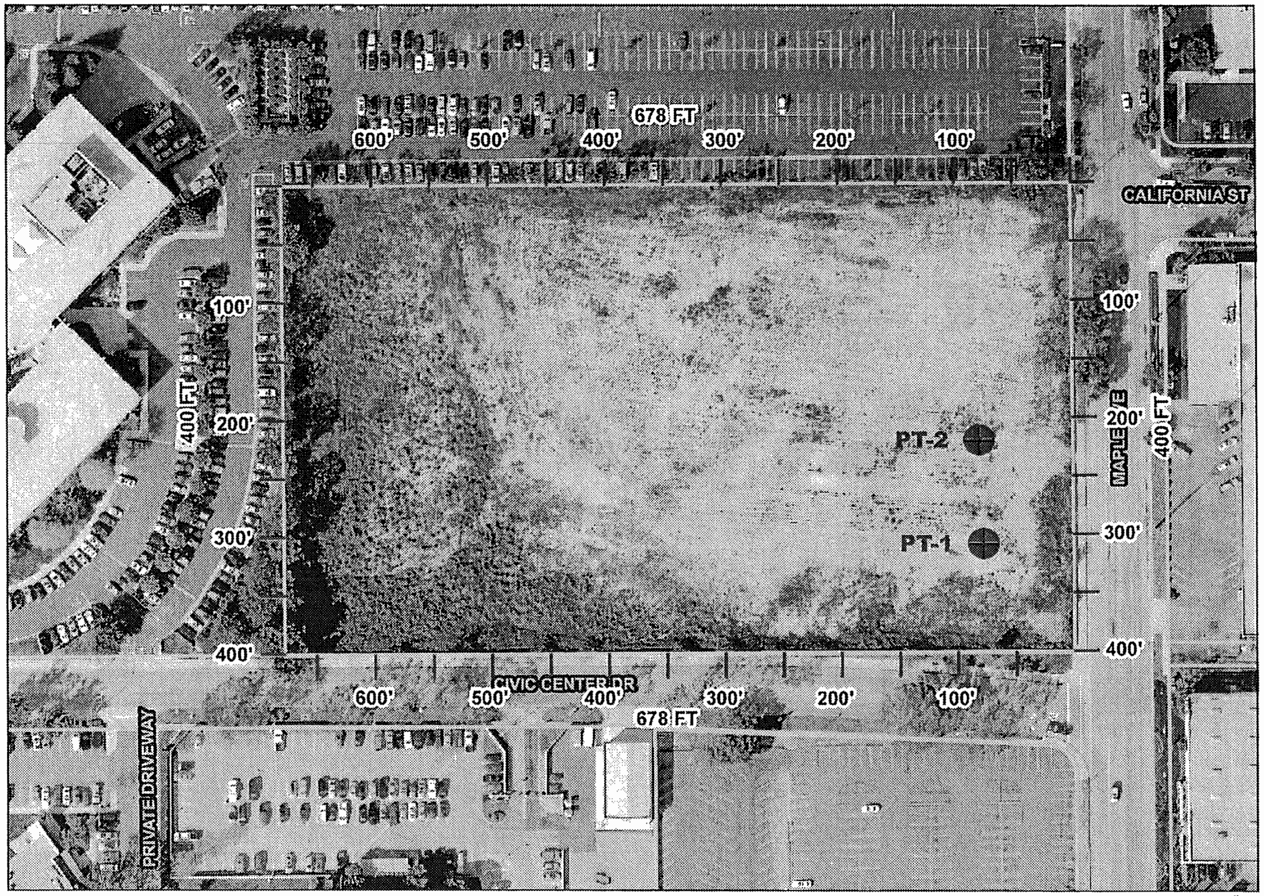
9.72

Infiltration Rate:

0.819 Inches/Hour

PETRA GEOTECHNICAL, INC.

9



Lines and photos are approximate, not to be used for establishing absolute or relative positions

555 MAPLE AVE, TORRANCE
AREA: 271,037 SF

EXPLANATION

PT-2 PERCOLATION TEST LOCATION

PETRA GEOTECHNICAL, INC.
3185-A AIRWAY AVENUE
COSTA MESA, CALIFORNIA 92626
PHONE: (714) 549-8921

COSTA MESA MURRIETA PALM DESERT SANTA CLARITA

SITE PLAN

COMMUNITY SPORTS FIELD
555 MAPLE AVENUE, TORRANCE, CALIFORNIA

DATE: August, 2013	J.N.: 13-248	**Figure 1**
DWG BY: SSR	SCALE: 1"=100'	

Five

TORRANCE SPORTS PARK
HYDROLOGY CALCULATIONS

SOIL TYPE: 010
TOTAL AREA=6.42ac

PROJECT IMPERVIOUSNESS=10% ("LOCAL PARKS" PER LA COUNTY HYDROLOGY MANUAL)

50 YR 24 HR ISOHYET=5.8"

25 YR 24 HR REDUCTION FACTOR=0.878

25 YR 24 HR RAINFALL=5.09"

Tc Calculator

Subarea Parameters Manual Input

Subarea Number		
1a		
Area (Acres)	Proportion Impervious	Soil Type
6.42	.1	10
Rainfall Isohyet (in.)	Flow Path Length (ft.)	Flow Path Slope
5.09	800	.01

Subarea Parameters Selected

Subarea Number		
1a		
Area (Acres)	Proportion Impervious	Soil Type
6.42	0.1	10
Rainfall Isohyet (in.)	Flow Path Length (ft.)	Flow Path Slope
5.09	800	0.01

Input File

Check Here If Subarea Parameters Are Defined In An Input File

Import "tcdata.xls" File

Calculation Results

Subarea Number	Intensity	Undeveloped Runoff Coefficient (Cu)	Developed Runoff Coefficient (Cd)
1a	1.62	0.32	0.38

Tc Equation

Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135

Tc Value (min.)	Flowrate (cfs)
19	3.95

Calculate T

Cancel

ATTACHMENT 5, PG 1-3

PIPE HYDRAULIC ANALYSIS (CONSERVATIVE)

****

>>>>PIPEFLOW HYDRAULIC INPUT INFORMATION<<<<

PIPE DIAMETER(FEET) = 1.000
PIPE SLOPE(FEET/FEET) = 0.0050
PIPEFLOW(CFS) = 3.95
MANNINGS FRICTION FACTOR = 0.009000

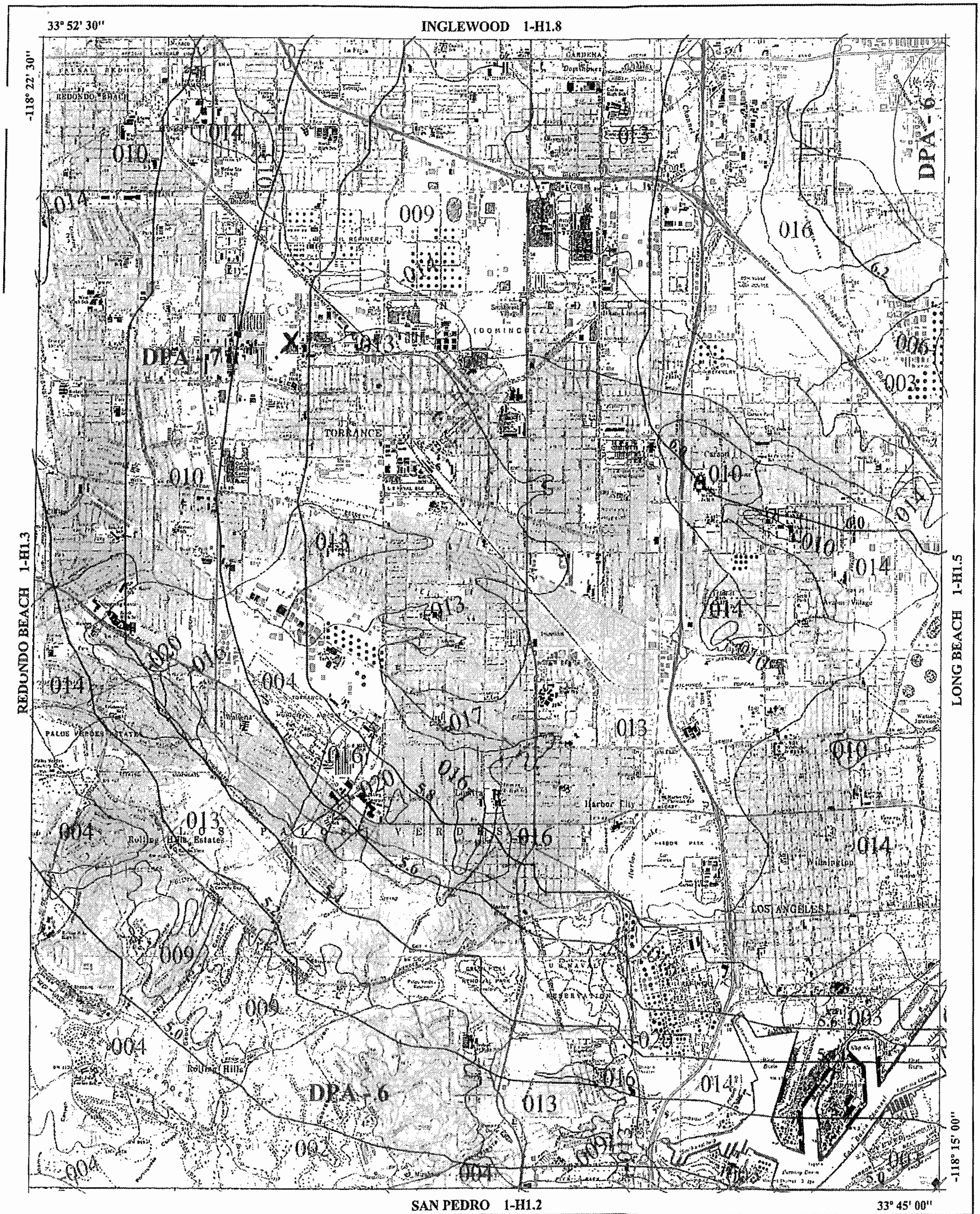
===

CRITICAL-DEPTH FLOW INFORMATION:

CRITICAL DEPTH(FEET) = 0.84
CRITICAL FLOW AREA(SQUARE FEET) = 0.706
CRITICAL FLOW TOP-WIDTH(FEET) = 0.728
CRITICAL FLOW PRESSURE + MOMENTUM(POUNDS) = 59.96
CRITICAL FLOW VELOCITY(FEET/SEC.) = 5.591
CRITICAL FLOW VELOCITY HEAD(FEET) = 0.49
CRITICAL FLOW HYDRAULIC DEPTH(FEET) = 0.97
CRITICAL FLOW SPECIFIC ENERGY(FEET) = 1.33
==>NORMAL PIPEFLOW IS PRESSURE FLOW

===

=>THE **TWO** 12" PVC STORM DRAIN LINES OUTLETTING TO THE EXISTING CATCH BASIN HAVE SUFFICIENT CAPACITY.



N
W E
S

016 SOIL CLASSIFICATION AREA

7.2 INCHES OF RAINFALL

DPA - 6 DEBRIS POTENTIAL AREA

1 0 1 2 Miles

25-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.878

10-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.714

TORRANCE

1-H1.4

50-YEAR 24-HOUR ISOHYET

Six

City of Torrance - 555 Maple Avenue Recreational Sports Field

ATTACHMENT 6

Electrical Plans sheets 1-4 inclusive, with revisions as dated PLAN REV. 8-29-13.

Please note:

a. Current Electrical Plans are those with the last revision listed as PLAN REV. 8-29-13.

b. Include current electrical plans with the last revision listed as PLAN REV. 8-29-13 for all bidding and construction purposes. Plans include all electrical revisions through 8-29-13.

c. See attached Bulletin No. E1 dated 08/29/13 summarizing revision 4.

PLAN NOTES

1. SCE TRANSFORMER. SEE SCE WORK ORDER FOR INSTALLATION REQUIREMENTS.
2. 277/480V, 400A, 3Ø, 4W SWITCHGEAR MMS1. SEE DETAIL A/EL-3.
3. PROVIDE DB CONDUIT ONLY WITH 3/16" POLY-PROPYLENE PULL ROPE. INSTALL PER SCE WORK ORDER.
4. PROVIDE CONCRETE UNDERGOUND PULLBOX WITH EVERY LIGHT POLE. SEE SPEC. 1 AND DETAIL B/EL-03.
5. MUSCO SPORTS LIGHTING CONTROL CABINET. SEE 1-LINE DIAGRAM DETAIL A & D/EL-3 AND CONTROLS SUMMARY ON SHEET EL-03.
6. 480-120/240V STEP-DOWN TRANSFORMER 'T1' SEE 1-LINE DIAGRAM DETAIL A & F/EL-O.
7. EXISTING 120/240V PANEL 'A' SUPPLIED AND INSTALLED BY BUILDING MANUFACTURER. CONTRACTOR TO MAKE ELECTRICAL CONNECTION IN PULL BOX.
8. IRRIGATION CONTROLLER. VERY EXACT LOCATION WITH LANDSCAPE PLANS
9. PROVIDE 2"C-14#8 (MMS1-13,15.17 (2 SETS OF SWITCH-LEGS FOR POLE S4),19,21,23,25,27,29,31), 1#8 GND. BURY 24" BELOW GRADE.
10. PROVIDE 2"C-11#8 (MMS1-13,15.17 (2 SETS OF SWITCH-LEGS FOR POLE S4),19,21,23,31), 1#8 GND. BURY 24" BELOW GRADE.
11. PROVIDE 2"C-3#8 (MMS1-13,15,17 (2 SETS OF SWITCH-LEGS FOR POLE S4), 31), 1#8 GND. BURY 24" BELOW GRADE.

4

12. PROVIDE 2"C-9#6 (MMS1-1,3,5,7,9,11,2,4,6), 3#8 (MMS1-31,33),1#8 GND. BURY 24" BELOW GRADE.
13. PROVIDE 2"C-6#6 (MMS1-1,3,5,7,2,4,6), 3#8 (MMS1-31,33),1#8 GND. BURY 24" BELOW GRADE.
14. PROVIDE 2"C-3#6 (MMS1-2,4,6), 2#8 (MMS1-33),1#8 GND. BURY 24" BELOW GRADE.
15. PROVIDE 2"C-3#6 (MMS1-8,10,12),1#8 GND. BURY 24" BELOW GRADE.

4

16. PROVIDE 2"C-9#6 (MMS1-8,10,12,14,16 (2 SETS OF SWITCH-LEGS FOR POLE S9),18). 2#8 (MMS1-33),1#8 GND. BURY 24" BELOW GRADE.
17. PROVIDE 2"C-3#6 (MMS1-20,22,24),1#8 GND. BURY 24" BELOW GRADE.
18. PROVIDE 1 1/4"C-3#4, 1#8 GND. BURY 24" BELOW GRADE.
19. PROVIDE 2"C-6#6 (MMS1-20,22,24). 5#8 (MMS1-26,28,30.33),1#8 GND. BURY 24" BELOW GRADE.
20. PROVIDE 1"C-2#8, 1#10 GND. BURY 24" BELOW GRADE.
21. PROVIDE 3/4"C-2#10, 1#10 GND. BURY 24" BELOW GRADE. FUTURE SIGN POWER. JUNCTION BOX SHALL BE RECESSED INTO SIDE OF COLUMN, WITH 1/2" PVC STUBBED TO TOP OF COLUMN, EXPOSED AND CAPPED.

555 Maple Sports park

Lighting & Electrical

Description: Added two switch legs to poles S4 and S9.
See plan notes 9, 10, 11 AND 16.

Issued By: P. Stevens

Bulletin No.

E1

Date:08/29/13

LAND CONCERN •• P.949.250.4822 •• F.949.752.2469

Seven

City of Torrance - 555 Maple Avenue Recreational Sports Field

ATTACHMENT 7

SECTION 03350 – PRECAST CONCRETE RESTROOM BUILDING (PRE-ASSEMBLED)

a. Alternate building designs to the pre-engineered EASI-SET Sierra Double must be preapproved by the owner 10 days prior to the bid date.
b. The contractor shall present plans, structural calculations and other required specifications for precast restroom to the building department for approval and permits. The point of contact for the EASI-SET Sierra Double includes, but may not be limited to:

Larry Turpin
Precast Sales Manager
Structure Cast
8261 McCutchen Rd.
Bakersfield, CA. 93311
ph: 661-833-4490
Fax: 661-280-5626
Cell: 661-742-3919
larry@structurecast.com
www.structurecast.com