MKM & Associates
structural engineering

Larry Miyano ▪ Bud Malmanis ▪ Eric Kreager ▪ John Cook

May 22, 2015 *MKM File # 140328*

STRUCTURAL CALCULATIONS

for

Water Fence

6194 San Mateo Ct

Rohnert Park, Ca 94928

OWNER:

Ken McDowell

REGISTERED PROFESSIONAL ENGINEER
ERIC N. KREAGER
No. SE 3266
Exp 3-31-17
STRUCTURAL
STATE OF CALIFORNIA

5/22/2015

CS052215JW.DOC

441 College Avenue ▪ Santa Rosa, CA 95401 ▪ (707)578-8185 ▪ Fax: (707)578-7153 ▪ Email: info@mkmassociates.com

Code(s) Used: 2013 CBC, ASCE 7-10

Risk Category =	II		(CBC, Tables 1604.5)
Importance Factor, I =	1.00		(ASCE 7, Table 1.5-2)
Response Modification Coefficient, R =	1.25		(ASCE 7, Table 12.2-1)
Max ground acceleration for 0.2 spectral response, S_s =	1.794g		(CBC, Figure 1613.3.1(1))
Site Class =	D		(ASCE 7, Table 20.3-1)
Site Coefficient, $F_a = S_{MS} / S_S$ =	1.00		(ASCE 7, Table 11.4-1)
S_{MS} =	1.794g		(ASCE 7, Eq. 11.4-1)
$S_{DS} = 2/3(S_{MS})$ =	1.196g		(ASCE 7, Eq. 11.4-3)
Seismic Response Coefficient, $C_s = S_{DS}/(R/I)$ =	0.957		(ASCE 7, Eq. 12.8-2)
S_1 =	0.716g		(CBC, Figure 1613.3.1(2))
$F_v = S_{M1} / S_1$ =	1.50		(ASCE 7, Table 11.4-2)
S_{M1} =	1.073g		(ASCE 7, Eq. 11.4-2)
$S_{D1} = 2/3(S_{M1})$ =	0.715g		(ASCE 7, Eq. 11.4-4)
h_n =	6	ft	
C_t =	0.020		(ASCE 7, Table 12.8-2)
x =	0.75		(ASCE 7, Table 12.8-2)
$T_a = C_t h_n^x$ =	0.077	sec	(ASCE 7, Eq. 12.8-7)
T_L =	8	sec	(ASCE 7, Figure 22-12)
Cs need not exceed the following			(ASCE 7, 12.8.1.1)
$C_s = S_{D1}/(T^*R/I)$ (for $Ta \le T_L$) =	7.46		(ASCE 7, Eq. 12.8-3)
$C_s = (S_{D1}^*T_L)/(T^{2*}R/I)$ (for $T_a > T_L$) =			(ASCE 7, Eq. 12.8-4)
C_s shall not be less than the following			
$C_s = 0.044 S_{DS} I \ge 0.01$ =	0.05		(ASCE 7, Eq. 12.8-5)
$C_s = 0.5^*S_1/(R/I)$ (for $S_1 \ge 0.6g$) =	0.2864		(ASCE 7, Eq. 12.8-6)
Base Shear			Strength Design Level
$V = C_s * W$ =	0.957	W	(ASCE 7, Eq. 12.8-1)
Vertical Seismic Load Effect			Strength Design Level
$E_v = 0.2 S_{DS} D$ =	0.239	D	(ASCE 7, Eq. 12.4-4)
Seismic Design Category (CBC)	D		(ASCE 7, Tables 11.6)
Load to column=	2126	lbs	USD (1516# Allowable)
weight=	2222	lbs	

Section 29.4.1 ASCE 10 Wind Loads on Solid Freestanding Walls

Risk Category =	II	
$V_{(asd)}$=	85	mph
Kd=	0.85	
I=	1	
Exposure=	C	
K_{zt}=	1.0	

h=	15	ft
K_Z=	0.849	

q_h=	13.3	psf	(6-15)

G=	0.85	section 26.9
C_f=	1.53	(fig. 29.4-1)
A_s=	48	Sq. ft

$F=q_hGC_fA_s$	(29.4-1)	
F=	833	lbs.

W=	121	plf
post spacing=	7	ft o.c.

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Steel Column

Description : typical column

Code References

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Steel Section Name : **HSS6x3x1/8**
Analysis Method : Allowable Strength
Steel Stress Grade
Fy : Steel Yield 36.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi
Load Combination : ASCE 7-10

Overall Column Height 6.0 ft
Top & Bottom Fixity Top Free, Bottom Fixed
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 6.0 ft, K = 2.1
Y-Y (depth) axis :
Unbraced Length for Y-Y Axis buckling = 6.0 ft, K = 2.1

Applied Loads

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

Column self weight included : 43.790 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 6.0 ft, D = 2.20 k
BENDING LOADS . . .
Lat. Point Load at 3.0 ft creating My-y, E = 2.126 k
Lat. Point Load at 3.0 ft creating My-y, W = 0.8330 k

DESIGN SUMMARY

Bending & Shear Check Results

Max. Axial+Bending Stress Ratio = **1.092** : 1
Load Combination +D+0.70E+H
Location of max.above base 0.0 ft
At maximum location values are . . .
Pa : Axial 2.244 k
Pn / Omega : Allowable 19.982 k
Ma-x : Applied 0.0 k-ft
Mn-x / Omega : Allowable 6.952 k-ft
Ma-y : Applied -4.465 k-ft
Mn-y / Omega : Allowable 4.311 k-ft

PASS Maximum Shear Stress Ratio = **0.1870** : 1
Load Combination +D+0.70E+H
Location of max.above base 0.0 ft
At maximum location values are . . .
Va : Applied 1.488 k
Vn / Omega : Allowable 7.958 k

Maximum SERVICE Load Reactions . .
Top along X-X 0.0 k
Bottom along X-X 2.126 k
Top along Y-Y 0.0 k
Bottom along Y-Y 0.0 k

Maximum SERVICE Load Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination :
Along X-X 0.8789 in at 6.0 ft above base
for load combination : E Only

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+H	0.112	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+L+H	0.112	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr+H	0.112	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+S+H	0.112	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750Lr+0.750L+H	0.112	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.750L+0.750S+H	0.112	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.60W+H	0.404	PASS	0.00 ft	0.063	PASS	0.00 ft
+D+0.70E+H	1.092	FAIL !	0.00 ft	0.187	PASS	0.00 ft
+D+0.750Lr+0.750L+0.450W+H	0.317	PASS	0.00 ft	0.047	PASS	0.00 ft
+D+0.750L+0.750S+0.450W+H	0.317	PASS	0.00 ft	0.047	PASS	0.00 ft
+D+0.750L+0.750S+0.5250E+H	0.833	PASS	0.00 ft	0.140	PASS	0.00 ft
+0.60D+0.60W+0.60H	0.381	PASS	0.00 ft	0.063	PASS	0.00 ft
+0.60D+0.70E+0.60H	1.069	FAIL !	0.00 ft	0.187	PASS	0.00 ft

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Project Title: Water Fence
Engineer: JW
Project Descr:

Project ID: 140328

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Printed: 22 MAY 2015, 3:57PM

Steel Column

File = U:\2014\Water Fence 140328\ENG\CALCS\Owners house calcs\sc052215jw.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.26, Ver:6.14.1.26

Lic. # : KW-06005908 | Licensee : mkm associates

Description : typical column

Maximum Reactions - Unfactored

Note: Only non-zero reactions are listed.

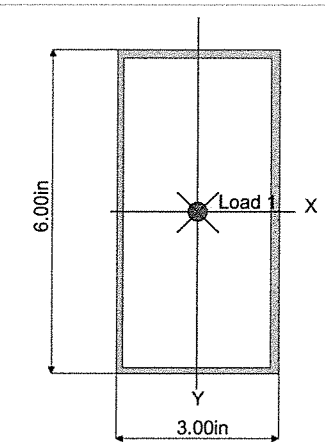
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base
D Only		k		k	2.244 k
W Only	-0.833	k		k	k
E Only	-2.126	k		k	k
D+W	-0.833	k		k	2.244 k
D+E	-2.126	k		k	2.244 k

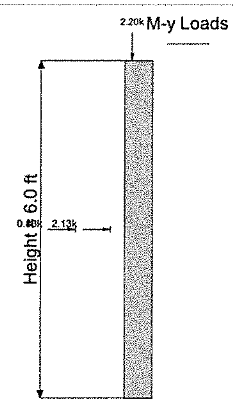
Maximum Deflections for Load Combinations - Unfactored Loads

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
W Only	0.3444	in	6.000	ft	0.000	in	0.000	ft
E Only	0.8789	in	6.000	ft	0.000	in	0.000	ft
D+W	0.3444	in	6.000	ft	0.000	in	0.000	ft
D+E	0.8718	in	5.960	ft	0.000	in	0.000	ft

Steel Section Properties : HSS6x3x1/8

Depth	=	6.000 in	I xx	=	9.43 in^4	J	=	7.730 in^4
			S xx	=	3.14 in^3	Cw	=	3.93 in^6
Width	=	3.000 in	R xx	=	2.170 in			
Wall Thick	=	0.125 in	Zx	=	3.870 in^3			
Area	=	2.000 in^2	I yy	=	3.230 in^4	C	=	3.930 in^3
Weight	=	7.298 plf	S yy	=	2.150 in^3			
			R yy	=	1.270 in			
			Zy	=	2.400 in^3			
Ycg	=	0.000 in						





Loads are total entered value. Arrows do not reflect absolute direction.

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Water Fence 6194 San Mateo Ct., Rohnert Park, Ca	Job 140328 Date 5/22/2015 PE MKM 5 of

PIER CALCULATOR

Pier and Loading Properties		
Lateral Load Type	Seismic	lbs
Lateral Load	1516	lbs
Lateral Load Height Above Top of Pier	3.5	ft
Vertical Load	2222	lbs
Uplift	0	lbs
Minimum Embedment into Competent Soil	0	ft
Minimum Pier Depth	0	ft
Duration Factor	1.3	-
Soil Height Neglected (S) (Vertical)	0	ft
Soil Height Neglected (S) (Lateral)	0	ft
Pier Diameter (D)	14	in
Passive Pressure	200	pcf
Applied Pier Diameters **(Passive)**	2	-
Max Passive Pressure	1500	psf
Pier Creep Pressure	0	pcf
Applied Pier Diameters **(Creep)**	0	-
Skin Friction (Downward Loading)	250	psf
Skin Friction (Uplift)	125	psf

PIER DESIGN SUMMARY		
Min. Embed. into competent soil (lateral)	**5.5**	ft
Min. Embed. into competent soil (vertical)	**2.4**	ft
Min. Embed. into competent soil (uplift)	**0.0**	ft
Total Pier Depth Required	**5.5**	ft
Moment at Pier (M_W)	**8.1**	kip*ft
Moment at Pier (M_U)	**11.3**	kip*ft
Shear in Pier (V_W)	**1.5**	kips
Shear in Pier (V_U)	**2.1**	kips

Pier Type = 14" /4-#5

HOLDOWN ANCHORAGE ACI 2005 APPENDIX D		
Anchor Type	**A.T.R. with double nut**	
Equations		
a	steel strength in tension	$N_{SA} = n * A_{SE} * f_{UTA}$
c	concrete breakout anchor in tension	$N_{CB} = A_{nc} / A_{nco} * \Psi_{ed,n} * \Psi_{c,N} * \Psi_{CP,N} * N_b$
e	pullout strength (tension)	$N_{PN} = \Psi_{c,P} * N_P$
f	concrete side-face blowout strength anchor in tension	$N_{SB} = 160 * C_{a1} *$ sqrt (Abrg) * sqrt (F'c)

FOUNDATION PROPERTIES		
Concrete Compressive Strength	2500.00	psi
Anchor Diameter	3/4	in
Anchor Type	A36	
Condition Type	A	
Anchor Area	0.334	in^2
f_{UTA}	58.00	ksi
F_{ya}	36	ksi
Footing width	14.00	in
Embedment	18.00	in
Edge Clearance (ca)	3.00	in
Uplift Capacity (USD)	**10912**	**lbs**
Uplift Capacity (ASD) Earthquake	**7638**	**lbs**
Uplift Capacity (ASD) Wind	**9093**	**lbs**

$T = \frac{1.5^k \times 36''}{5''} = 10.1^k \div 2 \text{ bolts} = 5.45^k < 7.6^k$ ok