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May 3, 2022

Monica Redhouse
Project Control Engineer/Estimator
Navajo Engineering & Construction Authority (NECA)
#1 Uranium Blvd, P.O. Box 969
Shiprock, NM 87420

Re: Ft. Defiance Sewer Line Arroyo Crossing Rehabilitation

Bohannan Huston, Inc. was asked to assess the existing gravity sewer line crossing and arroyo approximately 950 feet south and 875 feet west of the intersection of Kit Carson Drive and Indian Route 112 in Ft. Defiance, AZ. The existing elevated pipeline has been damaged and pier supports have failed in several locations.

We recommend the existing piping and pier supports be removed and replaced with new concrete piers and tamper-resistant, adjustable galvanized steel pipe support stanchions at approximately 30'-0" O.C. and new 316SS 8" diameter, schedule 40 pipe. The new piers should be 1'-2" x 1'-2" square columns, reinforced with (4) #7 vertical bars (at corners), and #3 ties at 8" O.C. TYP, bearing on a minimum 3'-0" x 3'-0" x 1'-0" thick foundation reinforced with (4) #4 bars each way top and bottom. See Detail 5 and structural calculations.

At the bank, the new pipe should bear on a new concrete block foundation. The foundation should be installed in the soil bank currently supporting the existing pipe, be 2'-0" x 2'-0" x 1'-0" thick minimum, and bear a minimum of 2'-6" below grade. The foundation should be reinforced with (3) #4 bars each way top and bottom. See Detail 6.

Installation of new piers should occur prior to pipeline section replacement, to minimize downtime for the pipeline. Where necessary for new pier installation, temporary shoring of the pipeline may be implemented to allow for removal of existing piers.

The new pipe should be connected to the existing pipe with a rigid connection such as a flanged connection (Class 150). A flanged connection can be installed to the existing pipe by field cutting and installing a Mega Flange Adaptor to the pipe end, this is the preferred method of connection. Alternately, the connection may also be made with the use of a dresser coupling as long as the two material types are electrically isolated from each other. This installation would still require the existing pipe to be field cut, but would also allow for field cutting of the new 316SS pipe, if necessary. Cradle support should be provided on both pipe end sections so as not to allow any deflection or stress at the connection between the two pipe ends.

Sincerely,

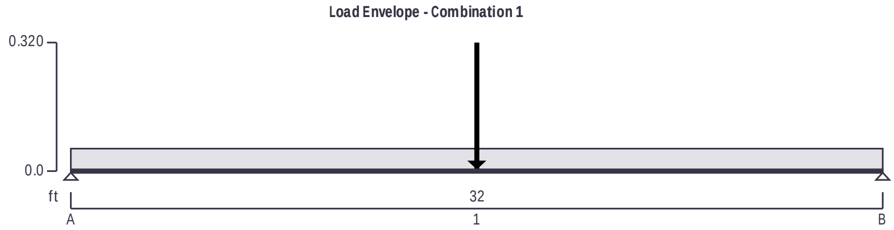
Nisa Rascon, PE	Matt Bean, PE	Cody MacLake, EI
Water Systems	Structural Engineering	Structural Engineering

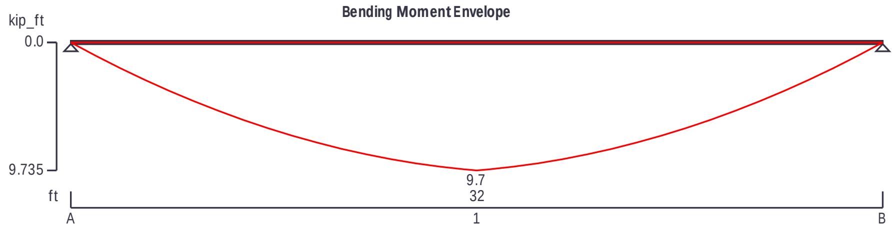
NR/mjb/cm
Enclosures

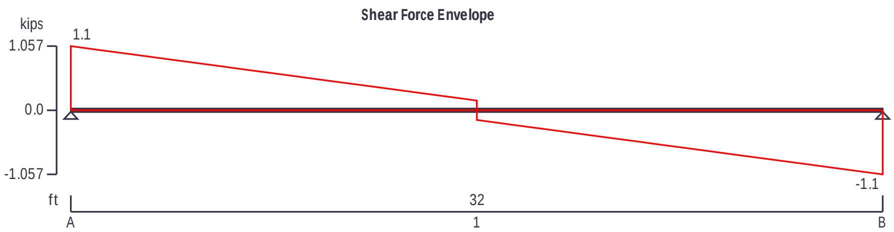
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

In accordance with AISC360-10 using the LRFD method

Tedds calculation version 3.0.15







Support conditions

Support A	Vertically restrained Rotationally free
Support B	Vertically restrained Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1 water - Dead full UDL 0.02 kips/ft live - Live point load 0.2 kips at 192.00 in

Load combinations

Load combination 1	Support A	Dead × 1.20 Live × 1.60 Roof live × 1.60 Snow × 1.60

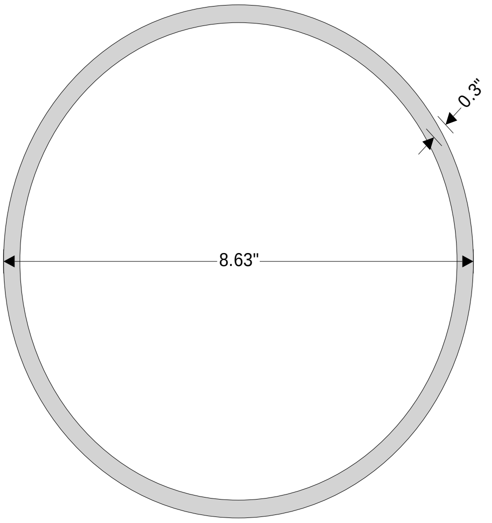
		Dead × 1.20
		Live × 1.60
		Roof live × 1.60
		Snow × 1.60
	Support B	Dead × 1.20
		Live × 1.60
		Roof live × 1.60
		Snow × 1.60

Analysis results

Maximum moment	M_{max} = **9.7** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **1.1** kips	V_{min} = **-1.1** kips
Deflection	δ_{max} = **0.7** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **1.1** kips	R_{A_min} = **1.1** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **0.7** kips	
Unfactored live load reaction at support A	R_{A_Live} = **0.1** kips	
Maximum reaction at support B	R_{B_max} = **1.1** kips	R_{B_min} = **1.1** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **0.7** kips	
Unfactored live load reaction at support B	R_{B_Live} = **0.1** kips	

Section details

Section type	**Pipe STD x8 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A53 Gr.B**
Steel yield stress	F_y = **35** ksi
Steel tensile stress	F_u = **60** ksi
Modulus of elasticity	E = **29000** ksi



Resistance factors

Resistance factor for tensile yielding	ϕ_{ty} = **0.90**
Resistance factor for tensile rupture	ϕ_{tr} = **0.75**
Resistance factor for compression	ϕ_c = **0.90**
Resistance factor for flexure	ϕ_b = **0.90**

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

Classification of section in flexure - Table B4.1b (case 20)

Width to thickness ratio $D_o / t =$ **28.75**

Limiting ratio for compact section $\lambda_{pff} = 0.07 \times E / F_y =$ **58.00**

Limiting ratio for non-compact section $\lambda_{rff} = 0.31 \times E / F_y =$ **256.86** Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **1.057** kips

Nominal shear strength - eq G6-1 $V_n = 0.6 \times F_y \times A / 2 =$ **82.425** kips

Resistance factor for shear $\phi_v =$ **0.90**

Design shear strength $V_c = \phi_v \times V_n =$ **74.183** kips

PASS - Design shear strength exceeds required shear strength

Design of members for flexure in the major axis - Chapter F

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **9.735** kips_ft

Yielding - Section F8.1

Nominal flexural strength for yielding - eq F8-1 $M_{nyld} = M_p = F_y \times Z =$ **60.667** kips_ft

Nominal flexural strength $M_n = M_{nyld} =$ **60.667** kips_ft

Design flexural strength $M_c = \phi_b \times M_n =$ **54.600** kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, roof live and snow loads

Limiting deflection $\delta_{lim} = L_{s1} / 180 =$ **2.133** in

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0.678** in

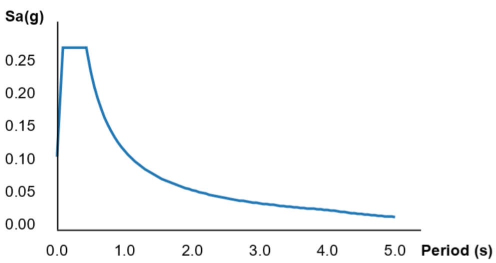
PASS - Maximum deflection does not exceed deflection limit

ATC Hazards by Location

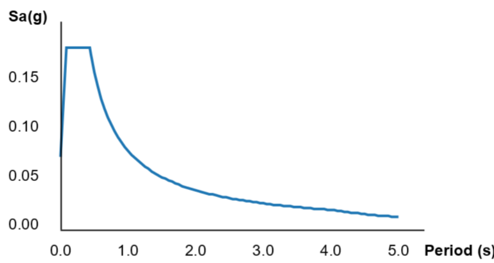
Search Information

Coordinates:	35.736034, -109.069116
Elevation:	6782 ft
Timestamp:	2022-03-24T20:46:44.840Z
Hazard Type:	Seismic
Reference Document:	ASCE7-10
Risk Category:	III
Site Class:	D

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.173	MCE_R ground motion (period=0.2s)
S_1	0.05	MCE_R ground motion (period=1.0s)
S_{MS}	0.277	Site-modified spectral acceleration value
S_{M1}	0.12	Site-modified spectral acceleration value
S_{DS}	0.185	Numeric seismic design value at 0.2s SA
S_{D1}	0.08	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	B	Seismic design category
F_a	1.6	Site amplification factor at 0.2s
F_v	2.4	Site amplification factor at 1.0s
CR_S	0.902	Coefficient of risk (0.2s)
CR_1	0.93	Coefficient of risk (1.0s)
PGA	0.094	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.15	Site modified peak ground acceleration
T_L	4	Long-period transition period (s)
SsRT	0.173	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.192	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.05	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.054	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

SEISMIC FORCES (ASCE 7-10)

Tedds calculation version 3.1.00

Site parameters

Site class D

Mapped acceleration parameters (Section 11.4.1)

at short period S_S = **0.173**

at 1 sec period S_1 = **0.05**

Site coefficientat short period (Table 11.4-1) F_a = **1.600**

at 1 sec period (Table 11.4-2) F_v = **2.400**

Spectral response acceleration parameters

at short period (Eq. 11.4-1) $S_{MS} = F_a \times S_S$ = **0.277**

at 1 sec period (Eq. 11.4-2) $S_{M1} = F_v \times S_1$ = **0.120**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3) $S_{DS} = 2 / 3 \times S_{MS}$ = **0.185**

at 1 sec period (Eq. 11.4-4) $S_{D1} = 2 / 3 \times S_{M1}$ = **0.080**

Seismic design category

Risk category (Table 1.5-1) III

Seismic design category based on short period response acceleration (Table 11.6-1)

B

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)

B

Seismic design category B

Approximate fundamental period

Height above base to highest level of building h_n = **8** ft

From Table 12.8-2:

Structure type All other systems

Building period parameter C_t C_t = **0.02**

Building period parameter x x = **0.75**

Approximate fundamental period (Eq 12.8-7) $T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x$ = **0.095** sec

Building fundamental period (Sect 12.8.2) $T = T_a$ = **0.095** sec

Long-period transition period T_L = **4** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1) A. Bearing_Wall_Systems

2. Ordinary reinforced concrete shear walls

Response modification factor (Table 12.2-1) R = **4**

Seismic importance factor (Table 1.5-2) I_e = **1.250**

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-3) $C_{s\_calc} = S_{DS} / (R / I_e)$ = **0.0577**

Maximum (Eq 12.8-3) $C_{s\_max} = S_{D1} / ((T / 1\text{ sec}) \times (R / I_e))$ = **0.2628**

Minimum (Eq 12.8-5) $C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01)$ = **0.0101**

Seismic response coefficient C_s = **0.0577**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure W = **1.0** kips

Seismic response coefficient C_s = **0.0577**

Seismic base shear (Eq 12.8-1) $V = C_s \times W$ = **0.1** kips

FT. DEFIANCE - COLUMN WIND LOADS CODY MACLAKE 03/29/2022

RISK CAT. = III (ASCE 7-10 TABLE 1.5-1)

BASIC WIND SPEED = 120 MPH (ASCE 7-10 FIG. 26.5-1B)

DIRECTIONALITY FACTOR, K_d = 0.85 (ASCE 7-10 TABLE 26.6-1; SOLID FREESTANDING WALLS)

EXPOSURE CATEGORY = C (ASCE 7-10 26.7)

TOPOGRAPHIC FACTOR, K_{zt} = 1.0 (ASCE 7-10 26.8)

ENCLOSURE CLASSIFICATION = ENCLOSED (ASCE 7-10 26.10)

VELOCITY PRESSURE EXPOSURE COEFFICIENT, K_z = 0.85 (ASCE 7-10 TABLE 29.3-1, 0-15 FT, EXP. C)

VELOCITY PRESSURE, $q_z = 0.00256\, K_z K_{zt} K_d V^2$ (LB/FT²) (ASCE 7-10 29.3.2)

$= 0.00256(0.85)(1.0)(0.85)(120) = 0.222$ (LB/FT²)

DESIGN WIND FORCE, $F = q_h G C_f A_s$ (LB) (ASCE 7-10 29.4.1)

$G = 0.85$ (ASCE 7-10 26.9.1)

$B = 2'$

$h = s = 8'$

CLEARANCE RATIO, $s/h = 1.0$

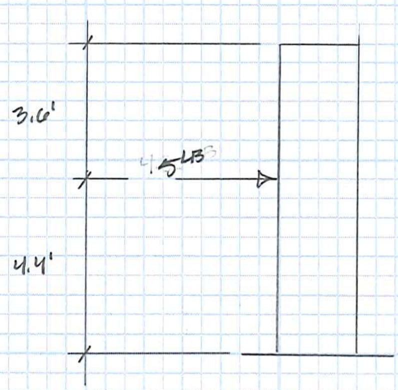
ASPECT RATIO, $B/s = 0.25 \approx 0.2$

$C_f = 1.65$ (CASE A & B) (ASCE 7-10 FIG. 29.4.1)

DISREGARD CASE C.

$F = q_h G C_f A_s$ (LB)

$= 0.222(0.85)(1.65)(16)$ $\boxed{= 4.98 \text{ LB} \quad @\ 4'\text{-}0" + 0.05(8) = 4.4' = 4'\text{-}5"}$



ENGINEERING ▲ SPATIAL DATA ▲ ADVANCED TECHNOLOGIES

Bohannan ▲ Huston

PROJECT NAME ______ SHEET ______ OF ______

PROJECT NO. ______ BY ______ DATE ______

SUBJECT ______ CH'D ______ DATE ______

Bohannan Huston
Courtyard I
7500 Jefferson St. NE
Albuquerque, NM
87109-4335

Project:	FT DEFIANCE SEWER REPLACEMENT		
Subject:	LOADS		
Client:	NECA	Job Number:	20220396
Prepared By:	SCM	Date:	4/1/2022
Checked By:	MJB	Date:	4/1/2021

Dead Load

	8	ft	Height of Column
	2.700	k	Weight of Column
	0.857	k	Weight of Pipe
	0.800	k	Weight of Fluid
	3.557	k	Total

Live Load

	0.200	k	Weight of LL

Wind Load

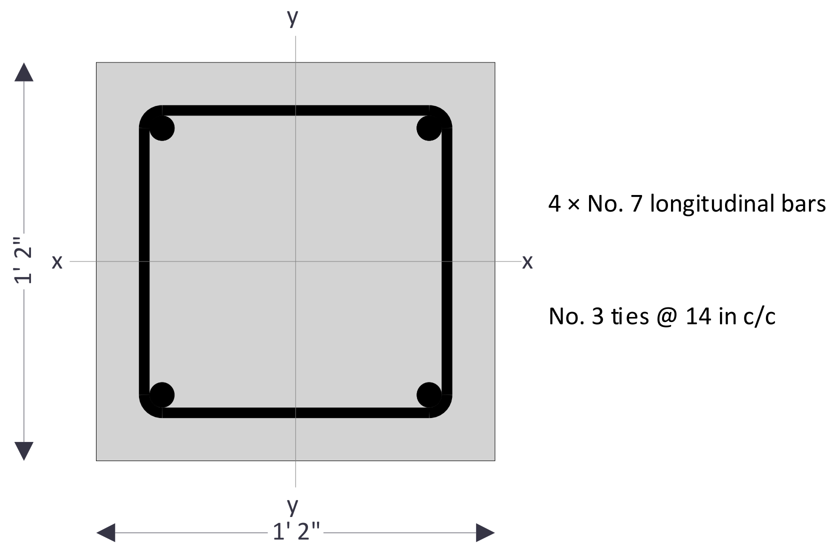
F	0.005	k	Wind Load
h/2+0.05h	4.400	ft	ASCE 7-10 Fig. 29.4-1
	0.022	k-ft	Moment due to Wind

Seismic Load

C_s	0.058		
W	3.557	k	
V	0.20521	k	Seismic Base Shear
V*h/2	0.82084		Seismic Moment

RC RECTANGULAR COLUMN DESIGN (ACI318-14)

Tedds calculation version 2.2.02



Applied loads

Ultimate axial force acting on column P_{u_act} = **10** kips

Ultimate moment about major (X) axis M_{ux_act} = **10** kips_ft

Geometry of column

Depth of column (larger dimension of column) h = **14.0** in

Width of column (smaller dimension of column) b = **14.0** in

Clear cover to reinforcement (both sides) c_c = **1.5** in

Unsupported height of column about x axis l_{ux} = **7.0** ft

Effective height factor about x axis k_x = **1.00**

Column state about the x axis Unbraced

Unsupported height of column about y axis l_{uy} = **7.0** ft

Effective height factor about y axis k_y = **1.00**

Column state about the y axis Unbraced

Check on overall column dimensions

Column dimensions are OK - h < 4b

Reinforcement of column

Numbers of bars of longitudinal steel N = **4**

Longitudinal steel bar diameter number D_{bar_num} = **7**

Diameter of longitudinal bar D_{long} = **0.875** in

Stirrup bar diameter number D_{stir_num} = **3**

Diameter of stirrup bar D_{stir} = **0.375** in

Specified yield strength of reinforcement f_y = **60000** psi

Specified compressive strength of concrete f'_c = **4000** psi

Modulus of elasticity of bar reinforcement E_s = **29** × **10⁶** psi

Modulus of elasticity of concrete $E_c = 57000 \times f'_c{}^{1/2} \times (1psi)^{1/2}$ = **3604997** psi

Yield strain $\varepsilon_y = f_y / E_s =$ **0.00207**

Ultimate design strain $\varepsilon_c =$ **0.003** in/in

Check for minimum area of steel - 10.6.1.1

Gross area of column $A_g = h \times b =$ **196.000** in^2

Area of steel $A_{st} = N \times (\pi \times D_{long}^2) / 4 =$ **2.405** in^2

Minimum area of steel required $A_{st\_min} = 0.01 \times A_g =$ **1.960** in^2

$A_{st} > A_{st\_min}$, PASS- Minimum steel check

Check for maximum area of steel - 10.6.1.1

Permissible maximum area of steel $A_{st\_max} = 0.08 \times A_g =$ **15.680** in^2

$A_{st} < A_{st\_max}$, PASS - Maximum steel check

Slenderness check about x axis

Radius of gyration $r_x = 0.3 \times h =$ **4.2** in

Actual slenderness ratio $s_{rx\_act} = k_x \times l_{ux} / r_x =$ **20**

Slenderness ratio is less than 22, slenderness effects may be neglected

Slenderness check about y axis

Radius of gyration $r_y = 0.3 \times b =$ **4.2** in

Actual slenderness ratio $s_{ry\_act} = k_y \times l_{uy} / r_y =$ **20**

Slenderness ratio is less than 22, slenderness effects may be neglected

Axial load capacity of axially loaded column

Strength reduction factor $\phi = 0.65$

Area of steel on compression face $A'_s = A_{st} / 2 =$ **1.203** in^2

Area of steel on tension face $A_s = A_{st} / 2 =$ **1.203** in^2

Net axial load capacity of column $P_n = 0.8 \times (0.85 \times f'_c \times (A_g - A_{st}) + f_y \times A_{st}) =$ **642.031** kips

Ultimate axial load capacity of column $P_u = \phi \times P_n =$ **417.320** kips

PASS : Column is safe in axial loading

Uniaxially loaded column about major axis

Details of column cross-section

c/d_t ratio $r_{xb} =$ **0.188**

Effective cover to reinforcement $d' = c_c + D_{stir} + (D_{long}/2) =$ **2.313** in

Spacing between bars $s = ((h - (2 \times d')))/((N/2)-1) =$ **9.375** in

Depth of tension steel $d_t = h - d' =$ **11.687** in

Depth of NA from extreme compression face $c_x = r_{xb} \times d_t =$ **2.196** in

Factor of depth of compressive stress block $\beta_1 =$ **0.850**

Depth of equivalent rectangular stress block $a_x = \min((\beta_1 \times c_x), h) =$ **1.866** in

Yield strain in steel $\varepsilon_{sx} = f_y / E_s =$ **0.002**

Strength reduction factor $\phi_x =$ **0.900**

Details of concrete block

Force carried by concrete

Forces carried by concrete $P_{xcon} = 0.85 \times f'_c \times b \times a_x =$ **88.840** kips

Moment carried by concrete

Moment carried by concrete — $M_{xcon} = P_{xcon} \times ((h/2) - (a_x/2))$ = **44.914** kip_ft

Details of steel layer 1

Depth of layer — x_{x1} = **2.313** in

Strain of layer — $\varepsilon_{x1} = \varepsilon_c \times (1 - x_{x1} / c_x)$ = **-0.00016**

Stress in layer — $\sigma_{x1} = \max(-1 \times f_y, E_s \times \varepsilon_{x1})$ = **-4626.39** psi

Force carried by layer — $P_{x1} = N_x \times A_{bar} \times \sigma_{x1}$ = **-5.564** kips

Moment carried by steel layer — $M_{x1} = P_{x1} \times ((h / 2) - x_{x1})$ = **-2.173** kip_ft

Details of steel layer 2

Depth of layer — x_{x2} = **11.688** in

Strain of layer — $\varepsilon_{x2} = \varepsilon_c \times (1 - x_{x2} / c_x)$ = **-0.01297**

Stress in layer — $\sigma_{x2} = \max(-1 \times f_y, E_s \times \varepsilon_{x2})$ = **-60000.00** psi

Force carried by layer — $P_{x2} = N_x \times A_{bar} \times \sigma_{x2}$ = **-72.158** kips

Moment carried by steel layer — $M_{x2} = P_{x2} \times ((h / 2) - x_{x2})$ = **28.187** kip_ft

Force carried by steel

Sum of forces by steel — P_{xs} = **-77.7** kips

Total force carried by column

Nominal axial load strength — P_{nx} = **11.117** kips

Strength reduction factor — ϕ_x = **0.900**

Ultimate axial load carrying capacity of column — $P_{ux} = \phi_x \times P_{nx}$ = **10.005** kips

Total moment carried by column

Total moment carried by column — M_{ox} = **70.928** kip_ft

Ultimate moment strength capacity of column — $M_{ux} = \phi_x \times M_{ox}$ = **63.835**kip_ft

Check load capacity for uniaxial loads about the x axis

Factored axial load — P_{u_act} = **10** kips

Ultimate axial capacity — P_{ux} = **10** kips

PASS - Ultimate axial capacity exceeds factored axial load

Factored moment about x axis — M_{ux_act} = **10** kip_ft

Ultimate moment capacity about the x axis — M_{ux} = **63.8** kip_ft

PASS - Ultimate moment capacity exceeds factored moment about x axis

Design of column ties - 25.7.2

Spacing of lateral ties — s_{v_ties} = **14.000** in

16 times longitudinal bar diameter — $s_{v1} = 16 \times D_{long}$ = **14.000** in

48 times tie bar diameter — $s_{v2} = 48 \times D_{stir}$ = **18.000** in

Least column dimension — $s_{v3} = \min(h,b)$ = **14.000** in

Required tie spacing — $s = \min(s_{v1}, s_{v2}, s_{v3})$ = **14.000** in

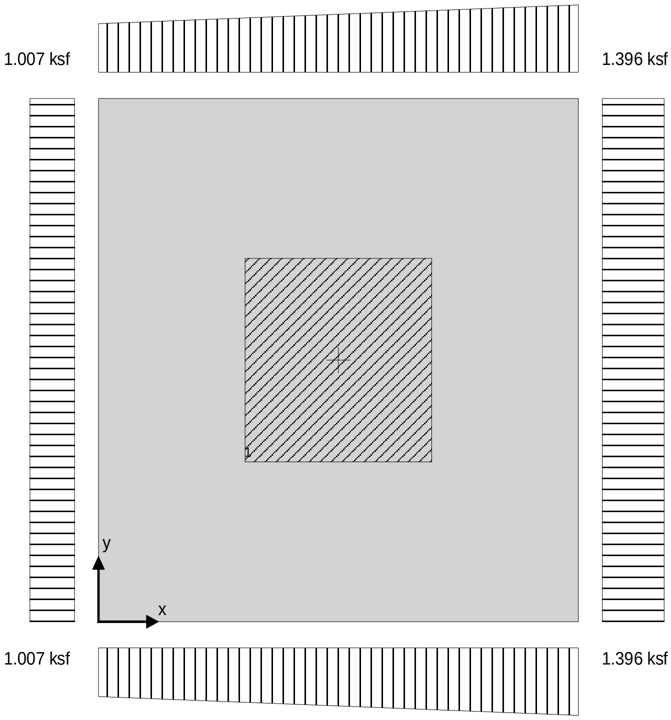
s_{v_ties} < s PASS

Foundation analysis & design (ACI318) in accordance with ACI318-14

Tedds calculation version 3.2.09

FOOTING ANALYSIS

Length of foundation	L_x = **3** ft
Width of foundation	L_y = **3** ft
Foundation area	$A = L_x \times L_y$ = **9** ft^2
Depth of foundation	h = **12** in
Depth of soil over foundation	h_{soil} = **30** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **14.00** in
Width of column	l_{y1} = **14.00** in
position in x-axis	x_1 = **18.00** in
position in y-axis	y_1 = **18.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **1.5** ksf
Density of soil	γ_{soil} = **95.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **21.8** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.400**
Design wall friction angle	δ_b = **15.0** deg

Horizontal acceleration factor K_h = **0.4**

Vertical acceleration factor K_v = **0**

Acceleration coefficient $\theta = \text{atan}(K_h / (1 - K_v))$ = **21.801**

Passive pressure coefficient (Coulomb) $K_P = \sin(90 - \phi_b)^2 / (\sin(90 + \delta_b) \times [1 - \sqrt{[\sin(\phi_b + \delta_b) \times \sin(\phi_b) / (\sin(90 + \delta_b))]}]^2)$ = **4.977**

Passive dynamic pressure coefficient (M-O) $K_{PE} = 0$ = **0**

Self weight $F_{swt} = h \times \gamma_{conc}$ = **150** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **237.5** psf

Column no.1 loads

Dead load in z F_{Dz1} = **6.0** kips

Live load in z F_{Lz1} = **0.5** kips

Wind load in x F_{Wx1} = **0.0** kips

Seismic load in x F_{Ex1} = **0.3** kips

Wind load moment in x M_{Wx1} = **0.0** kip_ft

Seismic load moment in x M_{Ex1} = **1.0** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.703)

1.0D + 1.0L (0.740)

1.0D + 1.0Lr (0.703)

1.0D + 1.0S (0.703)

1.0D + 1.0R (0.703)

1.0D + 0.75L + 0.75Lr (0.731)

1.0D + 0.75L + 0.75S (0.731)

1.0D + 0.75L + 0.75R (0.731)

1.0D + 0.6W (0.706)

$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.931)

1.0D + 0.75L + 0.75Lr + 0.45W (0.733)

1.0D + 0.75L + 0.75S + 0.45W (0.733)

1.0D + 0.75L + 0.75R + 0.45W (0.733)

$(1.0 + 0.10 \times S_{DS})$D + 0.75L + 0.75S + 0.525E (0.898)

0.6D + 0.6W (0.425)

$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.453)

Combination 10 results: $(1.0 + 0.14 \times S_{DS})$D + 0.7E

Forces on foundation

Force in x-axis $F_{dx} = \gamma_E \times F_{Ex1}$ = **0.2** kips

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1}$ = **10.8** kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_E \times (M_{Ex1} + F_{Ex1} \times h)$ = **17.1** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1)$ = **16.2** kip_ft

Uplift verification

Vertical force F_{dz} = **10.816** kips

PASS - Foundation is not subject to uplift

Stability against overturning in x direction, moment about x is L_x

Overturning moment $M_{OTxL} = \gamma_E \times (M_{Ex1}+F_{Ex1} \times h)$ = **0.87** kip_ft

Resisting moment $M_{RxL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (F_{Dz1} \times (x_1 - L_x))$ = **-16.22** kip_ft

Factor of safety $abs(M_{RxL} / M_{OTxL})$ = **18.541**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against sliding

Resistance due to base friction $F_{RFriction} = \max(F_{dz}, 0 \text{ kN}) \times \tan(\delta_{bb})$ = **4.326** kips

Stability against sliding in x direction

Resistance from passive soil pressure $F_{RxPass} = 0.5 \times K_{PE} \times (h^2 + 2 \times h \times h_{soil}) \times L_y \times \gamma_{soil}$ = **0.614** kips

Total sliding resistance $F_{Rx} = F_{RFriction} + F_{RxPass}$ = **4.94** kips

Factor of safety $abs(F_{Rx} / F_{dx})$ = **28.23**

PASS - Sliding factor of safety exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0.971** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.007** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.007** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.396** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.396** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **1.007** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **1.396** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross}$ = **1.5** ksf

q_{max} / q_{allow} = **0.931**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete f'_c = **4000** psi

Yield strength of reinforcement f_y = **60000** psi

Cover to reinforcement c_{nom} = **3** in

Concrete type Normal weight

Concrete modification factor λ = **1.00**

Column type Concrete

Analysis and design of concrete footing

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	Calc. by SCM	Date 4/1/2022	Chk'd by MJB	Date 4/1/2022	App'd by	Date

Load combinations per ASCE 7-10

1.4D (0.038)

1.2D + 1.6L + 0.5Lr (0.039)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on foundation

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} = \mathbf{12.2}$ kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{18.3}$ kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{18.3}$ kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2 = \mathbf{0}$ in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{0}$ in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.354}$ ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.354}$ ksf

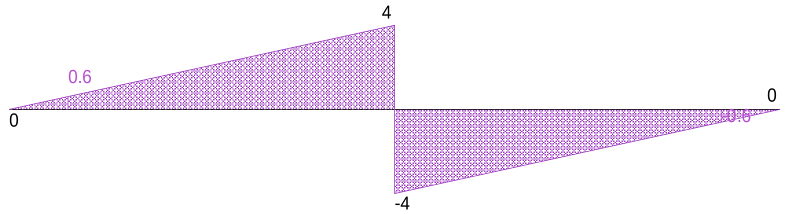
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.354}$ ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.354}$ ksf

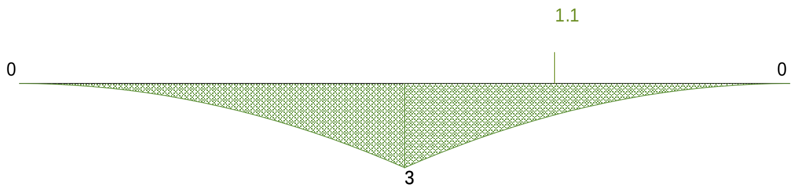
Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{1.354}$ ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{1.354}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max} = \mathbf{1.12}$ kip_ft

Tension reinforcement provided 4 No.4 bottom bars (9.8 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov} = \mathbf{0.8}$ in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_y \times h = \mathbf{0.778}$ in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) s_{max} = min(2 × h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement d = h - c_{nom} - $\phi_{x.bot}$ / 2 = **8.750** in

Depth of compression block a = $A_{sx.bot.prov}$ × f_y / (0.85 × f'_c × L_y) = **0.392** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis c = a / β_1 = **0.461** in

Strain in tensile reinforcement (8.3.3.1) ε_t = 0.003 × d / c - 0.003 = **0.05390**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity M_n = $A_{sx.bot.prov}$ × f_y × (d - a / 2) = **34.216** kip_ft

Flexural strength reduction factor ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.900**

Design moment capacity ϕM_n = ϕ_f × M_n = **30.794** kip_ft

$M_{u.x.max}$ / ϕM_n = **0.036**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x}$ = **0.611** kips

Depth to reinforcement d_v = h - c_{nom} - $\phi_{x.bot}$ / 2 = **8.75** in

Shear strength reduction factor ϕ_v = **0.75**

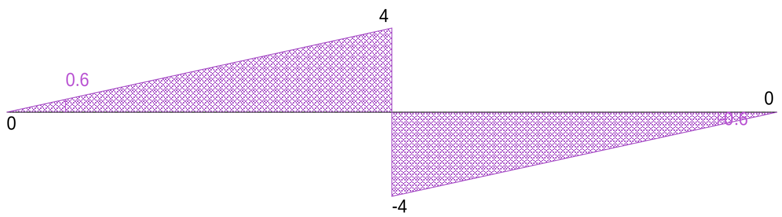
Nominal shear capacity (Eq. 22.5.5.1) V_n = 2 × λ × √(f'_c × 1 psi) × L_y × d_v = **39.845** kips

Design shear capacity ϕV_n = ϕ_v × V_n = **29.884** kips

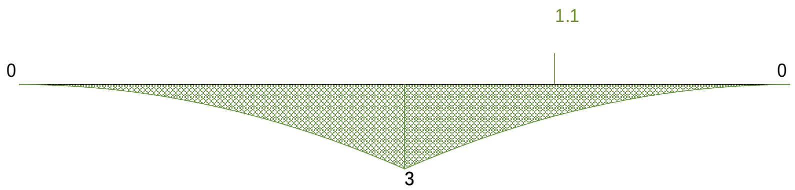
$V_{u.x}$ / ϕV_n = **0.020**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **1.12** kip_ft

Tension reinforcement provided 4 No.4 bottom bars (9.8 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **0.8** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min}$ = 0.0018 × L_x × h = **0.778** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) s_{max} = min(2 × h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2 = \mathbf{8.250}$ in

Depth of compression block — $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x) = \mathbf{0.392}$ in

Neutral axis factor — $\beta_1 = \mathbf{0.85}$

Depth to neutral axis — $c = a / \beta_1 = \mathbf{0.461}$ in

Strain in tensile reinforcement (8.3.3.1) — $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.05065}$

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity — $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) = \mathbf{32.216}$ kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity — $\phi M_n = \phi_f \times M_n = \mathbf{28.994}$ kip_ft

$M_{u.y.max} / \phi M_n = \mathbf{0.039}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force — $V_{u.y} = \mathbf{0.611}$ kips

Depth to reinforcement — $d_v = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2 = \mathbf{8.25}$ in

Shear strength reduction factor — $\phi_v = \mathbf{0.75}$

Nominal shear capacity (Eq. 22.5.5.1) — $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v = \mathbf{37.568}$ kips

Design shear capacity — $\phi V_n = \phi_v \times V_n = \mathbf{28.176}$ kips

$V_{u.y} / \phi V_n = \mathbf{0.022}$

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement — $d_{v2} = \mathbf{8.5}$ in

Shear perimeter length (22.6.4) — $l_{xp} = \mathbf{22.500}$ in

Shear perimeter width (22.6.4) — $l_{yp} = \mathbf{22.500}$ in

Shear perimeter (22.6.4) — $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = \mathbf{90.000}$ in

Shear area — $A_p = l_{x,perim} \times l_{y,perim} = \mathbf{506.250}$ in^2

Surcharge loaded area — $A_{sur} = A_p - l_{x1} \times l_{y1} = \mathbf{310.250}$ in^2

Ultimate bearing pressure at center of shear area — $q_{up.avg} = \mathbf{1.354}$ ksf

Ultimate shear load — $F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p = \mathbf{4.487}$ kips

Ultimate shear stress from vertical load — $v_{ug} = \max(F_{up} / (b_o \times d_{v2}),0 \text{ psi}) = \mathbf{5.865}$ psi

Column geometry factor (Table 22.6.5.2) — $\beta = l_{y1} / l_{x1} = \mathbf{1.00}$

Column location factor (22.6.5.3) — $\alpha_s = \mathbf{40}$

Concrete shear strength (22.6.5.2) — $v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{379.473}$ psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{365.419}$ psi

$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{252.982}$ psi

$v_{cp} = \min(v_{cpa},v_{cpb},v_{cpc}) = \mathbf{252.982}$ psi

Shear strength reduction factor — $\phi_v = \mathbf{0.75}$

Nominal shear stress capacity (Eq. 22.6.1.2) — $v_n = v_{cp} = \mathbf{252.982}$ psi

Design shear stress capacity (8.5.1.1(d)) — $\phi v_n = \phi_v \times v_n = \mathbf{189.737}$ psi

$v_{ug} / \phi v_n = \mathbf{0.031}$

PASS - Design shear stress capacity exceeds ultimate shear stress load

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