EXAMPLE 10 - SIGN STRUCTURE FOUNDATION DESIGN

GENERAL INFORMATION

Example Statement: Example 10 demonstrates a design procedure for a drilled shaft foundation for a cantilever sign structure. The cantilever supports a sign panel attached to the horizontal support. The example is only for the design of the shaft foundation. It does not discuss cover design of the members and attachment.

The design follows the LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, First Edition 2015, with 2017 updates (AASHTO LTS), with references to AASHTO LRFD Bridge Design Specifications, 8th Edition (AASHTO). Example 10 was designed with a geotechnical investigation performed on the soil. If one does not have geotechnical data, it is CDOT's preference to use the Brom's method in Section 13 of the AASHTO LTS to determine shaft embedment.

MATERIAL PROPERTIES

Concrete: CDOT Concrete Class BZ

Concrete Compressive Strength	f'c =	4	ksi
Concrete Unit Weight	γ_c =	150	pcf

Steel: Reinforcing Steel

Grade 60 Reinforcing Steel	fy =	60	ksi

Steel: Steel Members

Steel Density	γ_{steel} =	490	pcf

Aluminum: Sign Panels

Aluminum Density	$\gamma_{aluminum}$ =	175	pcf

SIGN STRUCTURE GEOMETRY INFORMATION (Refer to Figure 1)

Pole Length	L_{pole} =	22.00	ft.
Pole Base Diameter (outside diameter, o.d.)	$\phi_{pole\text{-}B}$ =	15.50	in.
Pole Top Diameter (o.d.)	$\phi_{pole\text{-}T}$ =	12.50	in.
Pole Wall Thickness	t_{pole} =	0.1875	in.
Depth to Arm	D_{arm} =	1.50	ft.
Arm Length	L_{arm} =	16.00	ft.
Arm Base Diameter (o.d.)	$\phi_{arm\text{-}B}$ =	10.00	in.
Arm End Diameter (o.d.)	$\phi_{arm\text{-}E}$ =	6.25	in.
Arm Wall Thickness	t_{arm} =	0.1875	in.
Shaft Depth	D_{shaft} =	13.00	ft.
Shaft Diameter	ϕ_{shaft} =	36	in.
Number of Sign Panels		1	

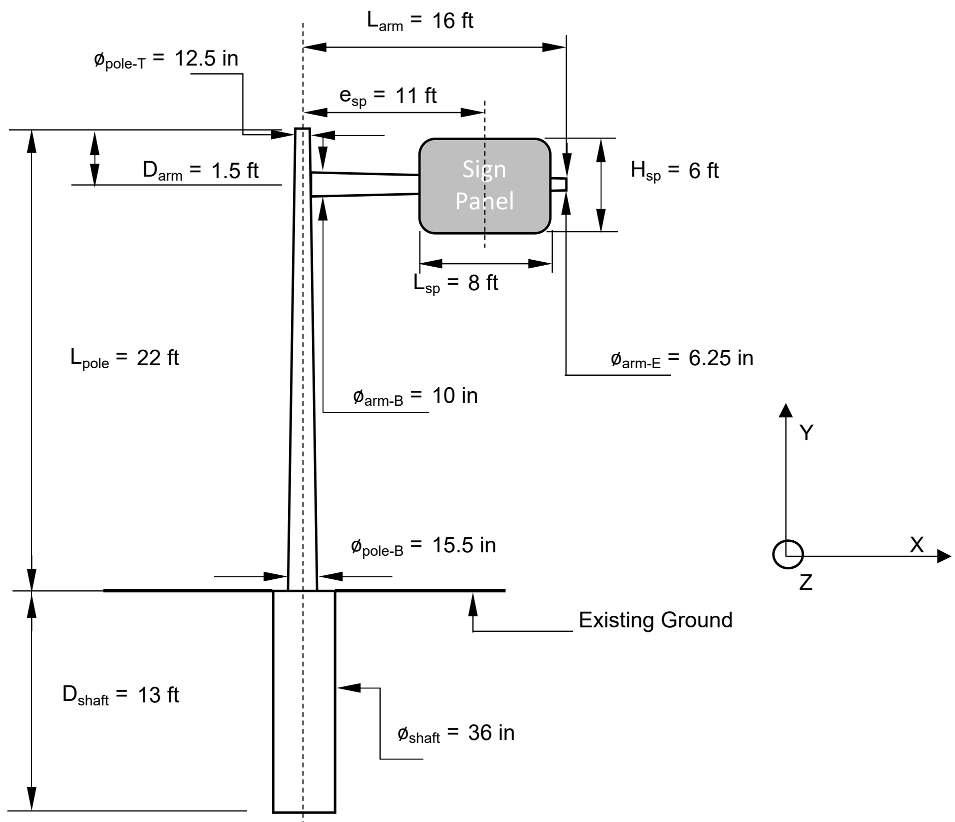


Figure 1 - Sign Structure Geometry Information

SIGN PANEL GEOMETRY INFORMATION

	Length	Height	e_{sp}	Area
Sign Panel 1	8.00 ft.	6.00 ft.	11.00 ft.	48.00 $ft.^2$

1. LOAD CALCULATION

Use the load combinations and factors from AASHTO LTS T3.4-1 for all loads acting on the sign structure. Determine the loads at the top of the shaft foundation:

APPLIED LOADS

AASHTO LTS 3

(Other loads not listed here may be applicable for different design cases.)

DC - dead load of structural components and nonstructural attachments

LL - live load is considered for designing members for walkways and service platforms

ICE - ice and wind on ice do not practically control and have been removed from the specifications

W - wind load is based on the pressure of the wind acting horizontally on all components

Dead Loads (DC) AASHTO LTS 3.5

*Weight is based on the typical weight of steel and aluminum

Pole Weight	DC_1 =	0.61	kip	
Arm Weight	DC_2 =	0.25	kip	
Sign Weight	DC_3 =	0.15	kip	*Assumed 7/32" Sign Thickness
Misc. Weight (Anchors and Sign Support)	DC_4 =	0.08	kip	*Assumed to be 50% of Sign Weight

Live Loads (LL) AASHTO LTS 3.6

Is LL applicable? no

Ice Loads (ICE) AASHTO LTS 3.7

Is ICE applicable? no

Wind Loads (W) AASHTO LTS 3.8

Mean Recurrence Interval	MRI =	1700		BDM 32.3.1.3
Basic Wind Speed	V =	120.00	mph	BDM 32.3.1.3
Height and Exposure Factor for Signs and Arm	K_z =	0.90		AASHTO LTS Eq. 3.8.4-1
Height and Exposure Factor for Pole	K_z =	0.86		AASHTO LTS Eq. 3.8.4-1
Directionality Factor	K_d =	0.85		AASHTO LTS 3.8.5
Gust Effect Factor	G =	1.14		AASHTO LTS 3.8.6
Velocity Conversion Factor - Ext Event	$C_{v\text{-Ext}}$ =	0.80		AASHTO LTS 3.8.7
	$C_v V d = C_v V ø_{pole\text{-}avg}$ =	112.00		
Velocity Conversion Factor	C_v =	1.00		AASHTO LTS 3.8.7
	$C_v V d = C_v V ø_{pole\text{-}avg}$ =	140.00		
Drag Coefficient for Members	$C_{d\text{-members}}$ =	0.45		AASHTO LTS 3.8.7
Drag Coefficient for Sign Panels	$C_{d\text{-sp}}$ =	1.19	*rounded up	AASHTO LTS 3.8.7

Wind Pressure on Members

$P_z = 0.00256\, K_z K_d G V^2 C_d =$ 14.50 psf AASHTO LTS Eq. 3.8.1-1

Wind Pressure on Sign Panels

$P_z = 0.00256\, K_z K_d G V^2 C_d =$ 38.35 psf AASHTO LTS Eq. 3.8.1-1

Pole Surface Area (along x axis)	A_{1x} =	25.67	$ft.^2$	
Pole Surface Area (along z axis)	A_{1z} =	25.67	$ft.^2$	
Arm Surface Area (along x axis)	A_{2x} =	10.83	$ft.^2$	
Sign Panels Surface Area (along x axis)	A_{3x} =	48.00	$ft.^2$	
Wind Load (x-direction)	$W = \Sigma A * P_z =$ W_x =	0.37	kip	$= A_{1z} * P_{z\text{-members}}$
Wind Load on Signs (z-direction)	$W_{z\text{-sign}}$ =	1.84	kip	$= A_{3x} * P_{z\text{-sign panels}}$
Wind Load on Arm (z-direction)	$W_{z\text{-arm}}$ =	0.16	kip	$= A_{2x} * P_{z\text{-members}}$
Wind Load on Pole (z-direction)	$W_{z\text{-pole}}$ =	0.37	kip	$= A_{1x} * P_{z\text{-members}}$

UNFACTORED LOADS AND MOMENTS AT TOP OF SHAFT

Moments taken about the centerline of the shaft

Load	Description	Load Direction (x,y,z)	Load (kip)	Moment Arm (ft.)	Moment Direction (x,y,z)	Moment at the Top of the Caisson (kip-ft.)
DC_1	Pole Weight	Y	0.61	0.00	Z	0.00
DC_2	Arm Weight	Y	0.25	4.31	Z	1.10
DC_3	Sign Weight	Y	0.15	11.00	Z	1.68
DC_4	Misc. Weight	Y	0.08	11.00	Z	0.84
LL	Live Load	Y	0.00	0.00	Z	0.00
$W_{x\text{-pole}}$	Wind on Pole	X	0.37	6.73	Z	2.51
$W_{z\text{-sign/arm}}$	Wind on Signs & Arm	Z	2.00	20.50	X	40.95
$W_{z\text{-sign}}$	Wind on Signs	Z	1.84	11.00	Y	20.25
$W_{z\text{-arm}}$	Wind on Arm	Z	0.16	4.31	Y	0.68
$W_{z\text{-pole}}$	Wind on Pole	Z	0.37	11.00	X	4.09

LOAD COMBINATIONS

AASHTO LTS T3.4-1

Load Combination	γ_{DC}	γ_{LL}	γ_W	Application
Strength I	1.25	1.60	-	Gravity
Extreme Ia	1.10	-	1.00	Wind max
Extreme Ib	0.90	-	1.00	Wind min
Service I	1.00	-	1.00	Translation

SUMMARY OF FACTORED LOADS AND MOMENTS AT TOP OF SHAFT

Moments taken about the centerline of the shaft

$$U = \gamma_{DC}DC + \gamma_{LL}LL + \gamma_W W$$

Load Combination	Axial (kip)	Moment about x-axis (kip-ft.)	Moment about y-axis* (kip-ft.)	Moment about z-axis (kip-ft.)	Shear in the x-axis (kip)	Shear in the z-axis (kip)
Strength I	1.37	-	-	4.53	-	-
Extreme Ia	1.20	45.05	20.92	6.49	0.37	2.37
Extreme Ib	0.98	45.05	20.92	5.77	0.37	2.37
Service I	1.09	45.05	20.92	6.13	0.37	2.37

*M_y to be used for torsion calculation

2. SHAFT CAPACITY

Run static L-PILE analysis with parameters from geotechnical report and calculated factored loads.

L-PILE INPUT

Soil Properties

*From Geotechnical Report

Top of Boring Elevation	$El_{boring\ top}$ =	5297.00
Bottom of Boring Elevation	$El_{boring\ bot}$ =	5270.00
Top of Shaft Elevation	$El_{caisson\ top}$ =	5297.50
Bottom of Shaft Elevation	$El_{caisson\ bot}$ =	5284.50

Top of Soil Elev.	Soil Type	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)	ε_{50}	k (pci)
5297.00	Stiff Clay w/o free water using k	120.00	0.00	2000.00	0.006	500.00
5290.00	Stiff Clay w/o free water using k	130.00	0.00	2500.00	0.005	1000.00

Shaft Section Properties

Section		Round Concrete Shaft		
Length of Section	D_{shaft} =	13.00	ft.	
Length of Section in Bedrock	D_{rock} =	5.50	ft.	
Section Diameter	ϕ_{shaft} =	36	in.	
Longitudinal Rebar Size	#	8		
Longitudinal Rebar Count		13		
Concrete Cover to Inside Edge of Stirrup Bar		3.625	in.	BDM 5.4.3
Stirrup Size	#	5		
Stirrup Spacing		12	in.	

INPUT LOADS

L-Pile models in only one plane, therefore:

Shear in the X Direction is paired with Moment in the Z Direction

Shear in the Z Direction is paired with Moment in the X Direction

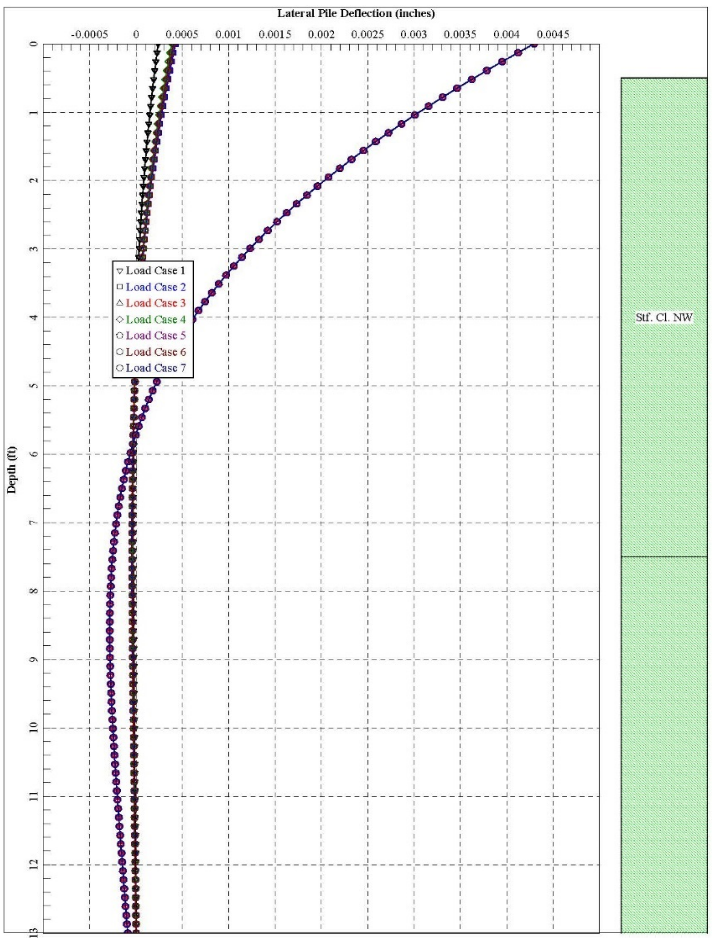
Load Case	Pile-Head Loading Condition	Shear (lb)	Moment (lb-in)	Axial (lb)
1	1	0	54,347	1,367
2	1	372	77,892	1,203
3	1	2,370	540,557	1,203
4	1	372	69,196	984
5	1	2,370	540,557	984
6	1	372	73,544	1,093
7	1	2,370	540,557	1,093

L-PILE OUTPUT

*Agg size assumed to be 0.75"

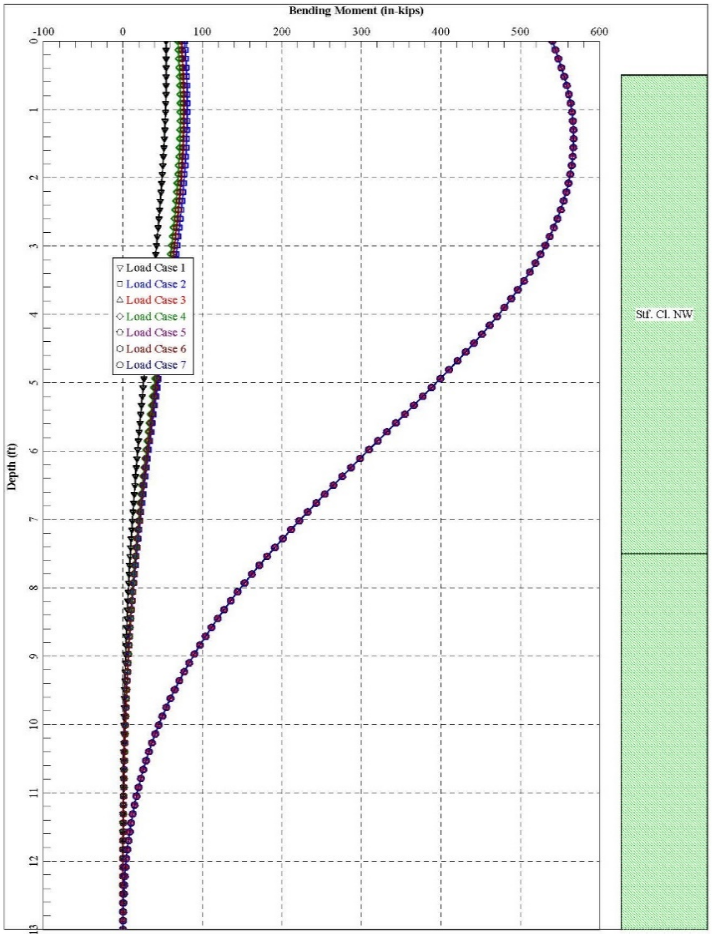
Reinforcement	13 #8		
Clear Distance Between Bars	5.64	in.	
Spacing Check for Min Spacing	>		
Min Clear Allowed, Max(1.5d_b, 1.5*Agg Size, 1.5") =	1.50	in.	AASHTO 5.10.3.1.1
Min Clear Allowed, Max(5*Agg Size, 5") =	5.00	in.	AASHTO 5.12.9.5.2
Area of Steel	10.27	$in.^2$	
Percentage of Steel	1.01%		
	>		
	0.80%		AASHTO 5.12.9.5.2

Maximum Pile-Head Deflection	0.0043	in.
Maximum Shear Force	7,261	lbs
Maximum Bending Moment	567,170	lb-in
Axial Thrust at Max Moment Case	1,203	lbs



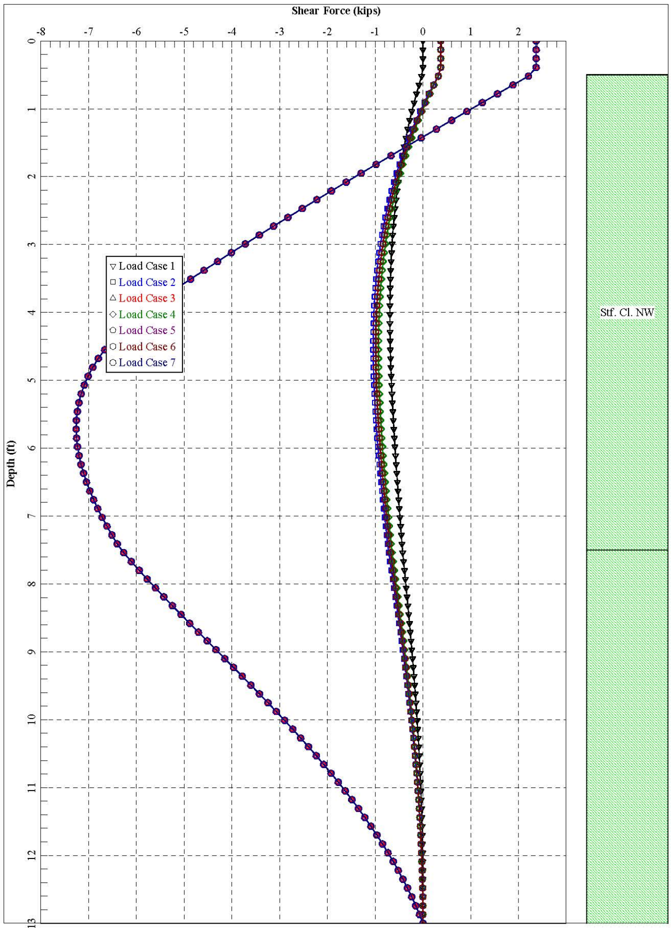
Lateral Pile Deflection (in.) vs Depth (ft.)

The maximum deflection, at the top of the caisson is 0.0043", which is considered zero; therefore, the shaft is deemed stable for the length used per the Engineer's judgment.



Bending Moment (in-kip) vs Depth (ft.)

The maximum factored moment is less than the maximum resistance moment. The shaft is considered stable per the reinforcement and size.



Shear Force (kips) vs Depth (ft.)

AXIAL RESISTANCE

Unit End Bearing Resistance	q_p =	18.00	ksf	Geotechnical Report
Unit Side Resistance	q_s =	1.00	ksf	Geotechnical Report
End Bearing Factor	ϕ_{qp} =	0.40		Geotechnical Report
Side Resistance Factor	ϕ_{qs} =	0.45		Geotechnical Report
Shaft End Bearing Area	$A_{shaft} = \pi d^2/4$ = A_{shaft} =	7.07	ft.^{2}	
Shaft Perimeter	$P_{shaft} = \pi d$ = P_{shaft} =	9.42	ft.	
Depth in Bedrock	D_{rock} =	5.50	ft.	
End Bearing Resistance	$\phi_{qp}q_pA_{shaft} = \phi_{qp}R_p$ =	50.89	kip	AASHTO Eq. 10.8.3.5-2
Side Shear Resistance	$\phi_{qs}q_sP_{shaft}D_{rock} = \phi_{qs}R_s$ =	23.33	kip	AASHTO Eq. 10.8.3.5-3
Ultimate Shaft Resistance	$R_R = \varphi R_n = \varphi_{qp}R_p + \varphi_{qs}R_s$	74.22	kip	AASHTO Eq. 10.8.3.5-1
Applied Vertical Load		15.15	kip	F_y max plus DL of shaft
		<		
		74.22	kip	
		OK!		

BENDING RESISTANCE

L-Pile provides Nominal Moment Resistance for each axial value.

The maximum factored applied moment from each L-Pile case with varying axial is compared to the nominal moment resistance provided by L-Pile.

$$\varphi M_n = M_u \geq M_{applied}$$

ϕ = 0.75 AASHTO 5.5.4.2

Load Case	Axial (lb)	Nominal Moment Resistance, M_n (kip-in.)	ϕ	Ultimate Moment Resistance, M_u (kip-in.)	Factored Applied Moment, $M_{applied}$ (kip-in.)	Check
5	984	8,472.87	0.75	6,354.65	540.56	OK!
7	1,093	8,474.12	0.75	6,355.59	540.56	OK!
3	1,203	8,475.37	0.75	6,356.53	540.56	OK!
1	1,367	8,477.25	0.75	6,357.94	54.35	OK!

SHEAR AND TORSION RESISTANCE

*The side shear resistance of soil for torsion effects is checked at the end of this example.

Shear Force	V_u =	7.26	kip	
Torsion	$M_y = T_u$ =	20.92	k-ft.	
Flexure	M_u =	45.05	k-ft.	
Tension	N_u =	15.15	kip	
Phi for Shear and Torsion	ϕ =	0.90		AASHTO 5.5.4.2

Concrete Cover to Reinforcing & Bar Size:

Side Cover	clr =	3.00	in.
Stirrup Bar Diameter	$d_{stirrup}$ =	0.63	in.

Nominal Resistance	M_n =	706.07	k-ft.	L-Pile Output
Area of Flexural Reinforcement	A_f =	5.14	in.^{2}	Half of the reinforcement in shaft
Dia of Circle Passing Through Long. Reinf	D_r =	27.75	in.^{3}	
Depth to Flexural Reinforcement	d_s =	26.83	in.	$= D_{shaft}/2 + D_r/\pi$

Torsional Cracking Moment AASHTO 5.7.2.1

Area of Concrete Perimeter	A_{cp} =	1,018	in.^{2}
Concrete Perimeter	p_c =	113.10	in.
Compressive Stress at Centroid of Section	f_{pc} =	0.00	ksi

$$T_{cr} = 0.126K\lambda\sqrt{f_c'}\frac{A_{cp}^2}{p_c}$$ AASHTO Eq.5.7.2.1-4

$$K = \sqrt{1 + \frac{f_{pc}}{0.126\lambda\sqrt{f_c'}}} \leq 2.0$$ AASHTO Eq.5.7.2.1-6

	K =	1.00		
Torsional Cracking Moment	T_{cr} =	2,308.54	k-in.	
	$0.25\varphi T_{cr}$ =	519.42	k-in.	
		>		AASHTO Eq. 5.7.2.1-3
	T_u =	251.08	k-in.	

Torsional effects can be neglected

Design Factored Shear Force	V_u =	7.26	kip

Shear Stress on Concrete AASHTO 5.7.2.8

Effective Shear Depth d_v = max of $\begin{cases} \frac{M_n}{A_s f_y} \\ 0.9 * d_s \\ 0.72 * h \end{cases}$

$M_n / A_s f_y$ =	27.50	in.	Maximum
$0.9*d_s$ =	24.15	in.	
$0.72*h$ =	25.92	in.	

d_v = 27.50 in.

Shear Stress $v_u = \frac{|V_u|}{\phi b_v d_v}$ = v_u = 0.0081 ksi AASHTO Eq. 5.7.2.8-1

Transverse Reinforcement

Transverse Reinforcement is required where: $V_u > 0.5\phi V_c$ AASHTO Eq. 5.7.2.3-1

V_u = 7.26 kip

<

$0.5\phi V_c$ = 115.15 kip

Transverse reinforcement not necessary

Minimum Transverse Reinforcement $A_{v,\,min} \geq 0.0316\lambda\sqrt{f'_c}\dfrac{b_v s}{f_y}$ AASHTO Eq. 5.7.2.5-1

$A_{v,\,min} \geq$ 0.46 in.^{2}

<

$A_{v,\,prov'd}$ = 0.62 in.^{2}

OK!

Maximum Spacing of Transverse Reinforcement AASHTO 5.7.2.6

v_u = 0.008 ksi

<

$0.125f'_c$ = 0.500 ksi

If v_u < 0.125f'c, then: $s_{max} = 0.8d_v \leq 24.0$ AASHTO Eq. 5.7.2.6-1

If v_u ≥ 0.125f'c, then: $s_{max} = 0.4d_v \leq 12.0$ AASHTO Eq. 5.7.2.6-2

s_{max} = 22.00 in.

>

$s_{v,\,prov'd}$ = 12.00 in.^{2}

OK!

Maximum Nominal Shear Resistance AASHTO 5.7.3.3

Nominal Shear Resistance $0.25*f'_c*b_v*d_v = V_n$ = 990.01 kip AASHTO Eq. 5.7.3.3-2

ϕV_n = 891.01 kip

>

V_u = 7.26 kip

OK!

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u|\right)}{E_s A_s}$$ AASHTO Eq. 5.7.3.4.2-4

Net Longitudinal Tensile Strain ε_s = 0.0002

For sections containing at least the minimum amount of transverse reinforcement specified in Art. 5.7.2.5, the value of β may be determined by the following equation:

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)}$$ AASHTO Eq. 5.7.3.4.2-1

β = 4.09

θ = 29.81 AASHTO Eq. 5.7.3.4.2-3

Nominal Shear Resistance of Concrete $V_c = 0.0316\beta\lambda\sqrt{f_c'}b_v d_v$ AASHTO Eq. 5.7.3.3-3

V_c = 255.88 kip

>

V_u = 7.26 kip

OK!

Side Shear Resistance of Soil in Torsion

Per CDOT's experience, the soil torsion capacity may control the shaft length. If the drilled shaft sees torsion, the following applicable checks should be completed. Refer to Report No. CDOT-DTD-R-2004-8 for equations and procedure.

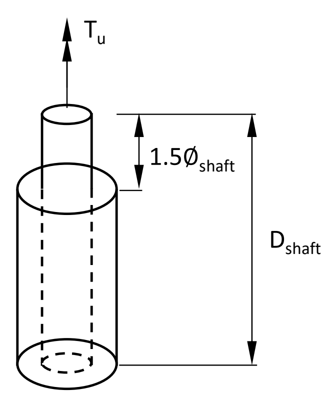
<u>Cohesive Soil Resistance</u>

It is CDOT's approach that the soil resistance to torsion in cohesive soils is based on the drilled shaft embedment area into the soil, neglecting the top 1.5' of section length. Perform the following check if the drilled shaft is in cohesive soil.

Torsion	T_u =	20.92	k-ft.
Soil profile used for example	cohesion, s_u =	2000	psf
Assumed Phi for Torsion, per SF = 1.25	ϕ =	0.80	
Section Diameter	$\emptyset_{shaft}$ =	36	in
Length of Section	D_{shaft} =	13.00	ft.

Drilled shaft side resistance $$T_s = \frac{\pi \cdot \emptyset_{shaft}^2}{2}(D_{shaft} - 1.5\emptyset_{shaft}) \cdot s_u$$

Drilled shaft toe resistance $$T_t = \frac{\pi \cdot \emptyset_{shaft}^3}{12} s_u$$



T_s = 240.33 k-ft.

T_t = 14.14 k-ft.

Nominal Total Torsion Resistance $T_n = (T_s + T_t) =$ 254.47 k-ft.

ϕT_n = 203.58 k-ft.

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T_u = 20.92 k-ft.

OK!

Cohesionless Soil Resistance

It is CDOT's approach that the soil resistance to torsion in cohesionless soils is based on the drilled shaft embedment into the soil. Perform the following check if the drilled shaft is in cohesionless soil.

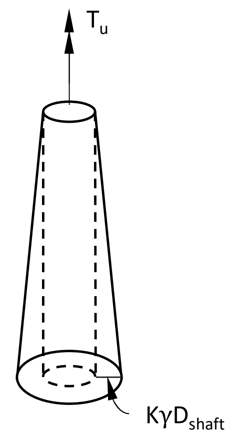
Torsion	T_u =	20.92	k-ft.
Soil profile used for example	unit weight, γ =	120	pcf
Soil profile used for example	friction angle, φ =	30.00	degrees
Assumed Phi for Torsion, per SF = 1.25	ϕ =	0.80	
Section Diameter	$\emptyset_{shaft}$ =	36.00	in
Length of Section	D_{shaft} =	13.00	ft.
Weight of Section	W =	13.78	kip

Drilled shaft side resistance $$T_s = \frac{\pi \cdot \emptyset_{shaft}^2}{2} D_{shaft} \cdot r_s$$

Drilled shaft toe resistance $$T_t = \frac{\emptyset_{shaft}}{3} W \cdot tan\varphi$$

Coefficient of lateral earth pressure $$K = \frac{2D_{shaft}}{3\emptyset_{shaft}}(1 - sin\varphi) = 1.44$$

Unit shaft side resistance $$r_s = K\gamma \frac{D_{shaft}}{2} tan\varphi = 0.65 \text{ ksf}$$



	T_s =	119.55	k-ft.
	T_t =	7.96	k-ft.
Nominal Total Torsion Resistance	$T_n = (T_s + T_t) =$	127.51	k-ft.
	ϕT_n =	102.00	k-ft.
		>	
	T_u =	20.92	k-ft.
		OK!	