Fontaine, Lisa

From: Benito, Brian
Sent: Tuesday, November 15, 2016 2:40 PM
To: Mathews, Lisa A
Cc: Bachman, Melanie; Cunliffe, Fred; Fontaine, Lisa; Mercier, Robert; Mulcahy, Carriann; Perrone, Michael; Walsh, Christina
Subject: RE: PE 1130 Decision Letter
Attachments: #52 Tower drawing Mt. Parnassus E. Haddam 2016-06-16.pdf; #52 Tower Foundation Design Calcs 10-2016.pdf

Follow up on EM 1130:
Prior to construction, DESPP shall file the final structural design drawings for the tower and foundation.
Attached: Tower Drawing
Foundation design calculations

Existing tower has been removed : January 2016, no operational communication equipment existing at time of removal.

Notice will be sent once telecommunications site is operational.

Regards
Brian Benito
Planning Specialist
Connecticut Department of Emergency Services and Public Protection
Division of Statewide Emergency Telecommunications - CTS Unit
860-539-4960 cell
860-685-8280 office

From: Mathews, Lisa A
Sent: Monday, February 23, 2015 1:04 PM
To: Benito, Brian; admin@easthaddam.org; landuse@easthaddam.org; 'ehaddamemgmt@easthaddam.org'
Cc: Bachman, Melanie; Cunliffe, Fred; Fontaine, Lisa; Mathews, Lisa A; Mercier, Robert; Mulcahy, Carriann; Perrone, Michael; Walsh, Christina
Subject: PE 1130 Decision Letter

Please see the attached correspondence.

Lisa A. Mathews
Connecticut Siting Council
10 Franklin Square
New Britain, CT 06051
Lisa.A.Mathews@ct.gov
(860) 827-2935

Submittal Cover Sheet

SCOPE CONSTRUCTION COMPANY INC.

416 Slater Road
PO Box 1466
New Britain, CT 06050

Date: 06/12/2016

Project Name : CTS East Haddam Remote Site Enhancement
Scope Project Number : 1606

Submittal Description : Tower Foundation Design Calcs
Submittal Number : 16 71 00 - 02
Specification Section : 16 71 00
Specification Subparagraph :

Applicable Drawing Reference:

Subcontractor: Northeast Tower
Vendor : Valmont

Scope Construction Review Stamp:

Scope Construction Co., Inc.
416 Slater Rd., P.O. Box 1466, New Britain, CT 06050
(X)Reviewed
( )Reviewed as Noted
( )Correct and Re-submit

Date: 06/12/2016 By:

Review is for general arrangement and does not release the supplier and/or trade contractor from responsibilities for quantities, dimensions performance or other requirements of the contract document.

Design Team Review Stamp:

UNIT BASE FOUNDATION SUMMARY

Pyramid Network Svcs.
East Haddam, CT

U- 22.0 180 v 2.2
A- 265482

Foundation Dimensions

Pad width, W:	36.0	ft
Depth, D:	5.0	ft
Ext. above grade, E:	0.5	ft
Pier diameter, d_i:	4.0	ft
Pad thickness, T:	2.25	ft
Depth neglected, N:	5.0	ft
Volume, V_o:	112.54	cy

Reinforcement Design

pad rebar qty., m_p:	48	bars *
size, s_p:	10	
pier vertical qty, m_c:	23	verticals/pier
size, s_c:	8	3.5' cage
pier tie qty., m_t:	5	ties/pier
size, s_t:	4	w/ overlap

* Rebar to be equally spaced, both ways, top & bottom, for a total of 192 bars
* Use standees to support top rebar above bottom rebar in mat

Soil Information Per:

Terracon, Dated: 7/7/14 (File: J2145151)

Soil Parameters

Soil unit weight, γ:	100	pcf
Ultimate Bearing, B_c:	12.000	ksf
Cohesion, C_o:	0.000	ksf
Friction angle, φ:	0.0	degrees
Ult. Passive P., P_p:	0.100	pcf
Base sliding, μ:	0.20	
Seismic Design Cat.:	A	
Water at:	5	ft

Anchor Steel Selection

Part Number, P/N:	111965	Dia = 1.25" Length = 42"

Material Properties

Steel tensile str, F_y:	60000	psi
Conc. Comp. str, F'_c:	4500	psi
Conc. Density, δ:	150	pcf
Clear cover, cc:	3.00	in

Backfill Compaction

Lift thickness:	8	in
Compaction:	95	%
Modified Proctor:	ASTM	D1557

Tower design conforms to the following:

* *2012 International Building Code (IBC)*
* *ANSI TIA-222-G*
* *Building Code Requirements for Reinforced Concrete (ACI 318-05)*

Note: The centroid of the tower is offset from the centroid of the foundation

Foundation Loading

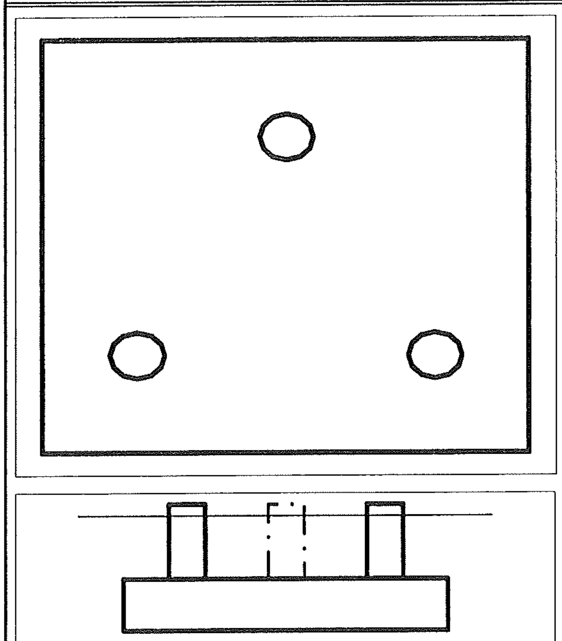
Load Case 1

Load Case 2 — stress ratio: 97.0% — mark up: 3.1%

Shear (total), S:	98.00 kips	x 1.031 =	101.04 kips
Moment, M:	10485.00 ft-kips	x 1.031 =	10810.04 ft-kips
Compression/Leg, C:	572.00 kips	x 1.031 =	589.73 kips
Uplift/Leg, U:	504.00 kips	x 1.031 =	519.62 kips
Tower Weight, W_t:	67.00 kips	=	67.00 kips

Views of Foundation

(not to scale)



STATE OF CONNECTICUT WILLIAM HEIDEN III No. 23038 LICENSED PROFESSIONAL ENGINEER

JUN 10 2016

Additional Notes:

* No foundation modifications listed.
* See attached "Foundation Notes" for further information.

FOUNDATION NOTES

1 PROVISIONS SHALL BE MADE TO PROTECT THE SUBGRADE FROM EXCESS MOISTURE.

2 A SUMP PUMP OR OTHER DEWATERING SYSTEM MAY BE REQUIRED TO LOWER THE WATER TABLE TO FACILITATE THE INSTALLATION OF THE FOUNDATION.

3 DIFFICULTIES DURING EXCAVATION MAY ARISE DUE TO THE PRESENCE OF BOULDERS, COBBLES, AND/OR SHALLOW BEDROCK.

4 ANY SOFT OR UNSTABLE SUBGRADE SOILS DETECTED DURING THE EXCAVATION SHOULD BE REMOVED AND REPLACED WITH COMPACTED FILL AS PER THE SPECIFICATIONS PROVIDED IN THE GEOTECHNICAL REPORT.

5 OVER-EXCAVATIONS BELOW FOUNDATION BEARING LEVEL MUST EXTEND LATERALLY BY A MIN. OF 8IN ON ALL SIDES OF THE FOUNDATIONS FOR EVERY FOOT OF OVER-EXCAVATION DEPTH BELOW FOUNDATIONS.

6 BACKFILL PLACED ABOVE THE FOUNDATIONS IS USED TO RESIST OVERTURNING. THE BACKFILL MUST HAVE A MIN. MOIST UNIT WEIGHT OF 100PCF.

UNIT BASE FOUNDATION (Load Case 2)

Pyramid Network Svcs.
East Haddam, CT

U- 22.0 180
A- 265482

v 2.2

Reactions	stress ratio 97.0%		mark up: 3.1%
Shear, S:	98.00 kips	x 1.031 =	101.04 kips
Moment, M:	10485.00 ft-kips	x 1.031 =	10810.04 ft-kips
Compression / leg, C:	572.00 kips	x 1.031 =	589.73 kips
Uplift / leg, U:	504.00 kips	x 1.031 =	519.62 kips
Tower weight, W_t:	67.00 kips	=	67.00 kips

Soil per: Terracon, Dated: 7/7/14 (File: J2145151)

Ultimate bearing: 12.000 ksf
Ultimate Pp: 0.100 kcf

Load Case 2 = 0.9*D + 1.0*Dg + 1.6*Wo

Physical Parameters:

Concrete volume:	$V = T \cdot W^2 + 3 \cdot (di^2/4 \cdot \pi) \cdot (D + E - T)$	$V =$	112.5	cy
Concrete weight:	$W_c = V \cdot \delta$	$W_c =$	455.8	kips
Soil weight:	$W_s = (D - T) \cdot (W^2 - 3 \cdot (di^2/4 \cdot \pi)) \cdot \gamma$	$W_s =$	346.0	kips
Total weight:	$P = Wc + Ws + Wt$	$P =$	868.81	kips

Passive Pressure:

Pp coefficient:	$K_p = TAN(45 + \varphi/2)^2$	$K_p =$	1.000	
	$P_{pn} = Kp \cdot \gamma \cdot N + 2 \cdot Co \cdot \sqrt{(Kp)}$	$P_{pn} =$	0.500	ksf
	$P_{pt} = Kp \cdot \gamma \cdot (D - T) + 2 \cdot Co \cdot \sqrt{(Kp)}$	$P_{pt} =$	0.275	ksf
	$P_{pb} = Kp \cdot \gamma \cdot D + 2 \cdot Co \cdot \sqrt{(Kp)}$	$P_{pb} =$	0.500	ksf
	P_{ptop} = IF(N < (D - T), Ppt, Ppn)	$P_{ptop} =$	0.5	ksf
	$Pp' = (Pptop + Ppb)/2$	$Pp' =$	0.500	ksf
Shear area:	$T_{pp} = 0$	$T_{pp} =$	0.0	ft
	$A_{pp} = Tpp \cdot W$	$A_{pp} =$	0.00	ft^2
Shear Capacity: *φr = 0.75*	$S_{actual} = (Pp' \cdot App + \mu \cdot P) \cdot \varphi r$	$S_{actual} =$	130.322	kips

Check	$S_{actual} =$ 130.32 kips	>=	$S =$ 101.04 kips	OK

Overturning Moment Resistance at Toe:

Wt of soil wedge:	$W_{sw} = D \cdot (D \cdot TAN(\varphi))/2 \cdot W \cdot \gamma$	$W_{sw} =$	0.0	kips
Dist. from leg to edge:	$O = (W - 0.866 \cdot w')/2$	$O =$	8.474	ft
Additional offset of Wt:	$O_a = (2/3 \cdot 0.866 \cdot w' + O) - W/2$	$O_a =$	3.175	ft
Resisting moments:	$M_{rwt} = P \cdot W/2 - Wt \cdot Oa$	$M_{rwt} =$	15425.85	ft-kips
	$M_{rp} = Pp' \cdot App \cdot (D - N)/3$	$M_{rp} =$	0.00	ft-kips
	$M_{rsw} = Wsw \cdot (W + D \cdot TAN(\varphi)/3)$	$M_{rsw} =$	0.00	ft-kips
Total resisting: *φr = 0.75*	$M_{rt} = (Mrwt + Mrp + Mrsw) \cdot \varphi r$	$M_{rt} =$	11569.39	ft-kips
Total overturning:	$M_o = M + S \cdot (D + E)$	$M_o =$	11365.74	ft-kips

Check	$M_{rt} =$ 11569.39 ft-kips	>=	$M_o =$ 11365.74 ft-kips	OK

Bearing Resistance due to Pressure Distribution:

Area of mat:	$area = W^2$	area =	1296.0	ft^2
Section modulus:	$SM = W^3/6$	SM =	7776.0	ft^3
Factored total weight:	$P' = Wt + 0.9 \cdot (Wc + Ws)$	$P' =$	788.6	kip
Pressure exerted:	$P_{pos} = P'/area + Mo/SM$	$P_{pos} =$	2.070	ksf
	$P_{neg} = P'/area - Mo/SM$	$P_{neg} =$	-0.853	ksf

Note: The stress resultant is NOT within the kern. Bearing area has been adjusted below.

Load eccentricity:	$e_c = Mo/P'$	$e_c =$	14.41	ft
	$P_{adj} = 2 \cdot P'/(3 \cdot W \cdot (W/2 - ec))$	$P_{adj} =$	4.1	ksf
Adj. applied pressure: *φr = 0.75*	q_a = IF(Pneg >= 0, Ppos, Padj)	$q_a =$	4.070	ksf

Check	$q_a =$ 4.070 ksf	<=	$B_c \cdot \varphi r =$ 9.000 ksf	OK

Concrete Shear Strength:

One way beam action at d_t from tower

Effective depth:	$d_c = T - cc - db\_p/2$	$d_c =$	23.365	in
Factored intensity:	$q_s = C/area$	$q_s =$	0.455	ksf
Required shear: *φs = 0.75 [ACI 9.3.2.3]*	$V_{n1} = qs \cdot (O - di/2 - dc) \cdot W/\varphi s$	$V_{n1} =$	98.88	kips
Available shear: *[ACI 11.2.1.1]*	$V_{c1} = 2 \cdot \sqrt{(F'c)} \cdot W \cdot dc$	$V_{c1} =$	1354.21	kips

Check	$V_{c1} =$ 1354.21 kips	>=	$V_{n1} =$ 98.88 kips	OK

Two way beam action at d_l / 2 from tower

Perimeter:	Po = (di + dc) * π	P_o =	18.68	ft
Required shear: φs = 0.75 [ACI 9.3.2.3]	V_{n2} = qs / φs * (area - (di + dc)² * π / 4)	V_{n2} =	769.46	kips
Available shear: [ACI 11.11.2.1]	V_{c2} = 4 * √(F'c) * Po * dc	V_{c2} =	1405.62	kips

Check	V_{c2} =	1405.62	kips	>=	V_{n2} =	769.46	kips	OK

Column Compression Capacity:

Compression reaction: φc = 0.65 [ACI 9.3.2.2]	P_c = φc * 0.8 * F'c * (di² / 4 * π)	P_c =	4234.4	kips

Check	P_c =	4234.36	kips	>=	C =	589.73	kips	OK

Pier Reinforcement:

Cross-sectional area:	A_g = di² * π / 4	A_g =	1809.56	in²
Min. area of steel (pier): [ACI 10.9.1] & [ACI 10.8.4]	A_{st_c} Ag * 0.005	A_{st_c} =	9.05	in²
Cage circle:	d_o = di - 2 * cc	d_o =	42.00	in
Rebar:	s_c = 8; d_{b_c} = 1 in			
	m_c = 23; A_{b_c} = 0.79 in²			
	A_{s_c} = Ab_c * m_c	A_{s_c} =	18.17	in²

Check	A_{s_c} =	18.17	in²	>=	A_{st_c} =	9.05	in²	OK

Actual moment:	M_{max} = (D - T + E) * S / 2	M_{max} =	164.19	ft-kips
Pier moment capacity:	M_{allow} *per Maxmomnt.xls (see attached)*	M_{allow} =	339.88	ft-kips

Check	M_{allow} =	339.88	ft-kips	>=	M_{max} =	164.19	ft-kips	OK

Bar separation:	B_{s_c} = (do * π) / m_c - db_c	B_{s_c} =	4.74	in

Check	11	>=	B_{s_c} =	4.74	in	>=	4.5"	OK

Vertical Rebar Development Length:

Reinforcement location: [ACI 12.2.4]	ψ_{t_c} = if the space under the rebar > 12 in, use 1.3, else use 1.0	ψ_{t_c} =	1.3	
Epoxy coating: [ACI 12.2.4]	ψ_{e_c} = if epoxy-coated bars are not used, use 1.0; but if epoxy-coated bars are used, then if Bs < 6 * db or cc < 3 * db, use 1.5, else 1.2	ψ_{e_c} =	1.0	
Max term: [ACI 12.2.4]	$\psi_t\psi_{e\_c}$ = the product of ψt & ψe, need not be taken larger than 1.7	$\psi_t\psi_{e\_c}$ =	1.3	
Reinforcement size: [ACI 12.2.4]	ψ_{s_c} = if the bar size is 6 or less, then use 0.8, else use 1.0	ψ_{s_c} =	1	
Light weight concrete: [ACI 12.2.4]	$\lambda_{\_c}$ = if lightwieght concrete is used, 1.3, else use 1.0	$\lambda_{\_c}$ =	1.0	
Spacing/cover: [ACI 12.2.4]	$c_{\_c}$ the smaller of: half the bar spacing or the concrete edge distace	$c_{\_c}$ =	3.37	in
Transverse bars: [ACI 12.2.3]	k_{tr_c} = 0 in (per simplification)	k_{tr_c} =	0	in
Max term: [ACI 12.2.3]	$c_{\_c}'$ = MIN(2.5, (c_c + ktr_c) / db_c)	$c_{\_c}'$ =	2.500	
Excess reinforcement: [ACI 12.2.5]	R_c = Mmax / Mallow	$R_{\_c}$ =	0.48	
Development (tensile): [ACI 12.2.2]	$L_{dt}'_{\_c}$ = (3 / 40) * (Fy / √(F'c)) * (ψtψe_c * ψs_c * λ_c * R_c / c_c') * db_c	$L_{dt}'_{\_c}$ =	16.85	in
Minimum length: [ACI 12.2.1]	L_{d_min} = 12 inches	L_{d_min} =	12.0	in
Development length:	L_{dt_c} = MAX(Ld_min, Ldt'_c)	L_{dt_c} =	16.85	in
Development (comp.): [ACI 12.3.2]	$L_{dc}'_{\_c}$ = 0.02 * db_c * Fy * R_c / √(F'c)	$L_{dc}'_{\_c}$ =	8.64	in
	$L_{dc}''_{\_c}$ = 0.0003 * db_c * Fy * R_c	$L_{dc}''_{\_c}$ =	8.70	in
Development length:	L_{dc_c} = MAX(8, Ldc'_c, Ldc"_c)	L_{dc_c} =	8.70	in
Length available in pier:	L_{vc} = D - T + E - cc	L_{vc} =	36.0	in

Check	L_{vc} =	36.0	in	>=	L_{dt_c} =	16.9	in	OK
Check	L_{vc} =	36.0	in	>=	L_{dc_c} =	8.7	in	OK

Length available in pad:	L_{vp} = T - cc	L_{vp} =	24.0	in

Check	L_{vp} =	24.0	in	>=	L_{dt_c} =	16.9	in	OK
Check	L_{vp} =	24.0	in	>=	L_{dc_c} =	8.7	in	OK

Vertical Rebar Hook Ending:

Bar size & clear cover: [ACI 12.5.3]	α_h if the bar size <= 11 and side cc >= 2.5", use 0.7, else use 1.0	ψ_{t_h} =	0.7		
Epoxy coating: [ACI 12.5.2]	β_h if epoxy-coated bars are used, use 1.2, else use 1.0	ψ_{e_h} =	1.0		
Light weight concrete: [ACI 12.5.2]	λ_h if lightwieght concrete is used, 1.3, else use 1.0	$\lambda_{\_h}$ =	1.0		
Development (hook): [ACI 12.5.2]	L_{dh}' = 0.02 * ψt_h * ψe_h * λ_h * Fy / √(F'c) * db_c	L_{dh}' =	12.5	in	
Minimum length: [ACI 12.5.1]	L_{dh_min} the larger of: 8 * db or 6 in	L_{dh_min} =	8.0	in	
Development length:	L_{dh} = MAX(Ldh_min, Ldh')	L_{dh} =	12.5	in	
	Check L_{vp} = 24.0 in >=	L_{dh} =	12.5	in	OK
Hook tail length:	L_{h_tail} 12 * db beyond the bend radius	L_{h_tail} =	16.0	in	
Length available in pad:	L_{h_pad} = (W - w' - di) / 2	L_{h_pad} =	60	in	
	Check L_{h_pad} = 60.0 in >=	L_{dh_tail} =	16.0	in	OK

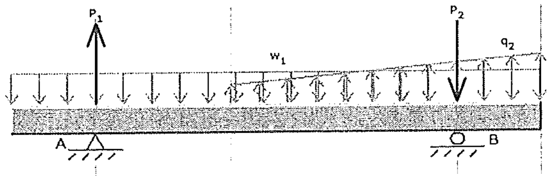
Pier Ties:

Minimum size: [ACI 7.10.5.1]	$s_{\_t\_min}$ =IF(s_c <= 10, 3, 4)	$s_{\_t\_min}$ =	3		
z factor:	z = 0.5 if the seismic zone is less than 2, else 1.0	z =	0.5		
Tie parameters:	$s_{\_t}$ = 4 d_{b_t} = 0.5 in				
	$m_{\_t}$ = 5 A_{b_t} = 0.2 in^2				
Allowable tie spacing:					
per vertical rebar [ACI 7.10.5.2] & [ACI 21.3.3.2]	$B_{s\_t\_max1}$ = 8 / z * db_c	$B_{s\_t\_max1}$ =	16	in	
per tie size [ACI 7.10.5.2] & [ACI 21.3.3.2]	$B_{s\_t\_max2}$ = 24 / z * db_t	$B_{s\_t\_max2}$ =	24	in	
per pier diameter [ACI 7.10.5.2] & [ACI 21.3.3.2]	$B_{s\_t\_max3}$ = di / (4 * z²)	$B_{s\_t\_max3}$ =	48	in	
per seismic zone [ACI 7.10.5.2] & [ACI 21.3.3.2]	$B_{s\_t\_max4}$ = 12" in active seismic zones, else 18"	$B_{s\_t\_max4}$ =	18	in	
	$B_{s\_t\_max}$ = MIN(Bs_t_max1, Bs_t_max2, Bs_t_max3, Bs_t_max4)	$B_{s\_t\_max}$ =	16	in	
	$m_{\_t\_min}$ = (D - T + E) / Bs_t_max + 2	$m_{\_t\_min}$ =	4.4		
	Check $m_{\_t}$ = 5.0 >=	$m_{\_t\_min}$ =	4.4		OK

Anchor Steel:

A/S parameters:	P_{as} = 111965 L_{as} = 42 in				
	d_{as} = 1.25 in E_{as} = 33.50 in				
Development available:	L_{das} *per Anchor Bolts (see attached)*	L_{das} =	20.00	in	
Required development:	L_{das_min} *per Anchor Bolts (see attached)*	L_{das_min} =	16.85	in	
	Check L_{das} = 20.00 in >=	L_{das_min} =	16.85	in	OK
To bottom rebar grid:	E_{as_max} =D + E - cc - 2 * db_p	E_{as_max} =	60.46	in	
	Check E_{as} = 33.50 in <=	E_{as_max} =	60.46	in	OK
To top rebar grid:	rebar @ = D + E - T + cc	rebar @	42.00	in	
	Check 42 + 6 in >= E_{as} = 33.50	in or	<=	42 in	OK
Min. cage dia:	d_{o_min} *per ancsteel.xls (see attached)*	d_{o_min} =	30.82	in	
	Check d_o = 42.00 in >=	d_{o_min} =	30.82	in	OK

Pad Reactions:



MDSolids Geometry Input (Option 1)				
Total Beam Length:	B_{L2_1} =W	B_{L2_1}=	36	ft
Location of Left Support:	S_{L2_1} =O	S_{L2_1}=	8.474	ft
Location of Right Support:	S_{R2_1} =W-O	S_{R2_1}=	27.53	ft
MDSolids Geometry Input (Option 2)				
Total Beam Length:	B_{L2_2} =W	B_{L2_2}=	36.0	ft
Location of Left Support:	S_{L2_2} =(W - w\) / 2	S_{L2_2}=	7.00	ft
Location of Right Support:	S_{R2_2} =S_{L1_2} + w\	S_{R2_2}=	29.00	ft
MDSolids Load Input (Option 1 & Option 2)				
Uplift:	P_{2_1} =U	P_{2_1}=	519.6	kips
Compression:	P_{2_2} =C	P_{2_2}=	589.73	kips
Weight of Overburden: (Distributed)	w_{2_1} =0.9 * (W_c + W_s) / W	w_{2_1}=	20.05	klf
		Applied over the beam starting at 0' and ending at W=36ft.		
Distributed Soil Pressure: (Linearly Increasing)	q_{2_2L} =0	q_{2_2L}=	0.00	klf
	q_{2_2R} =q_a * W	q_{2_2R}=	146.53	klf
		This linearly increasing load is applied from e=14.41ft to W=36ft		
MDSolids Design Result				
Option 1:	M_{max2_1} =M_{max2_1} (Max. Moment calculated from MDsolids for Option 1)	M_{max2_1}=	3860.00	ft*kips
Option 2:	M_{max2_2} =M_{max2_2} (Max. Moment calculated from MDsolids for Option 2)	M_{max2_2}=	2720.00	ft*kips
Max moment:	M_{maxp} =Max(Mmax2_1,Mmax2_2)	M_{maxp}=	3860.00	ft*kips
Required moment: φt = 0.9 [ACI 9.3.2.1]	M_n = Mmaxp / φt	M_n =	4288.89	ft*kips

Pad Reinforcement:

	β = IF(F'c <= 4000, 0.85, IF(F'c >= 8000, 0.65, 0.85 - (F'c - 4000) * 0.05))	β = 0.825	
Effective width:	W_e = w' * 0.866 + di	W_e = 23.052	ft
	A_{st_p}' = Mn / (0.9 * Fy * dc)	A_{st_p}' = 40.791	in^2
	a_p = Ast_p' * Fy / (β * F'c * We)	a_p = 2.38	in
Required steel:	$A_{st\_p\_st}$ = Mn / (Fy * (dc - ap / 2)) * (W / We)	$A_{st\_p\_st}$ = 60.414	in^2
Shrinkage:	ρ_{sh} = IF(Fy >= 60000, 0.0018, 0.002)	ρ_{sh} = 0.0018	
	$A_{st\_p\_sh}$ = ρsh * W * T / 2	$A_{st\_p\_sh}$ = 10.498	in^2
	A_{st_p} = MAX(Ast_p_st, Ast_p_sh)	A_{st_p} = 60.414	in^2
Rebar:	$s_{\_p}$ = 10 *Equally spaced, top and* d_{b_p} = 1.27 in		
	$m_{\_p}$ = 48 *bottom, both directions.* A_{b_p} = 1.27 in^2		
	A_{s_p} = Ab_p * m_p	A_{s_p} = 60.96	in^2
	Check A_{s_p} = 60.96 in^2 >=	A_{st_p} = 60.41	in^2 OK
Bar separation:	B_{s_p} = (W - 2 * cc - db_p) / (m_p - 1) - db_p	B_{s_p} = 7.77	in
	Check 10.73 >= B_{s_p} = 7.77 in	>=	4.5" OK

Pad Development Length:

Reinforcement location: [ACI 12.2.4]	ψ_{t_p} = if the space under the rebar > 12 in, use 1.3, else use 1.0	ψ_{t_p} = 1.3	
Epoxy coating: [ACI 12.2.4]	ψ_{e_p} = if epoxy-coated bars are not used, use 1.0; but if epoxy-coated bars are used, then if Bs < 6 * db or cc < 3 * db, use 1.5, else 1.2	ψ_{e_p} = 1.0	
Max term: [ACI 12.2.4]	$\psi_t\psi_{e\_p}$ = the product of ψt & ψe, need not be taken larger than 1.7	$\psi_t\psi_{e\_p}$ = 1.3	
Reinforcement size: [ACI 12.2.4]	ψ_{s_p} = if the bar size is 6 or less, then use 0.8, else use 1.0	ψ_{s_p} = 1	
Light weight concrete: [ACI 12.2.4]	$\lambda_{\_p}$ = if lightwieght concrete is used, 1.3, else use 1.0	$\lambda_{\_p}$ = 1.0	
Spacing/cover: [ACI 12.2.4]	$c_{\_p}$ = the smaller of: half the bar spacing or the concrete edge distace	$c_{\_p}$ = 3.64	in
Transverse bars: [ACI 12.2.3]	k_{tr_p} = 0 in (per simplification)	k_{tr_p} = 0	in
Max term: [ACI 12.2.3]	c_p' = MIN(2.5, (c_p + ktr_p) / db_p)	$c_{\_p}'$ = 2.500	
Excess reinforcement: [ACI 12.2.5]	R_p = Ast_p / As_p	$R_{\_p}$ = 0.99	
Development (tensile): [ACI 12.2.2]	L_d = (3 / 40) * (Fy / √(F'c)) * ψtψe_p * ψs_p * λ_p * R_p * db_p / c_p'	L_{dp}' = 43.9	in
Minimum length: [ACI 12.2.1]	L_{d_min} = 12 inches	L_{d_min} = 12.0	in
Development length:	L_{dp} = MAX(Ld_min, Ldp')	L_{dp} = 43.9	in
Length available in pad:	L_{pad} = (W / 2 - w' / 2) - cc	L_{pad} = 81.0	in
	Check L_{pad} = 81.00 in >=	L_{dp} = 43.90	in OK

THIS SPREADSHEET IS SET UP FOR A MAXIMUM OF 56 BARS.
MAXIMUM FACTORED MOMENT OF A CIRCULAR SECTION

Loading (negative for compression)		
Axial load =	519.62	kips

Foundation		
Concrete		
Pier diameter =	4.00	ft
Pier area =	1809.6	in^2
Reinforcement		
Clear cover =	3.00	in
Cage diameter =	3.42	ft
Bar size =	8	
Bar diameter =	1.000	in
Bar area =	0.785	in^2
Number of bars =	23	

Material Strengths			
Concrete compressive strength =	4500	psi	
Reinforcement yield strength =	60000	psi	
Modulus of elasticity =	29000	ksi	
Reinforcement yield strain =	0.00207		(per ACI 10.3.5 - OK)
Limiting compressive strain =	0.003		

Seismic	
Seismic Zone =	1
Are hooks required?	no

Minimum Area of Steel

Required area of steel =	9.05	in^2	
Actual area of steel =	18.06	in^2	OK
Bar spacing =	4.74	in	

Axial Loading

Load factor =	1.00		
Reduction factor =	0.65575		(per ACI 9.3.1 & 2)
Factored axial load =	792.42	kips	

Neutral Axis

Distance from extreme edge to neutral axis =	3.56	in	
Equivalent compression zone factor =	0.825		(per ACI 10.2.7.3)
Distance from extreme edge to Equivalent compression zone factor =	2.94	in	
Distance from centroid to neutral axis =	20.44	in	

Compression Zone

Area of steel in compression zone =	0.00	in^2	
Angle from centroid of pier to intersection of equivalent compression zone and edge of pier =	28.64	deg	
Area of concrete in compression =	45.61	in^2	
Force in concrete = 0.85 * f'c * Acc =	174.45	kips	(per ACI 10.3.6.2)
Total reinforcement forces =	-966.86	kips	
Factored axial load =	792.42	kips	
Force in concrete =	-174.45	kips	
Sum of the forces in concrete =	0.00	kips	OK

Maximum Moment

First moment of the concrete area in compression about the centoid =	1014.52	in^3
Distance between centroid of concrete in compression and centroid of pier =	22.24	in

Moment of concrete in compression =	3880.52	in-kips		
Total reinforcement moment =	2339.17	in-kips		
Nominal moment strength of column =	6219.69	in-kips		
Factored moment strength of column =	4078.55	in-kips	339.88	ft-kips

Maximum allowable moment of the pier =	339.88	ft-kips

Individual Bars

Bar #	Angle from first bar (deg)	Distance to centroid (in)	Distance to neutral axis (in)	Distance to equivalent comp. zone (in)	Strain	Area of steel in compression (in^2)	Axial force (kips)	Moment (in-kips)
1	0.00	0.00	-20.44	-21.06	-0.01723	0.00	-47.12	0.00
2	15.65	5.53	-14.91	-15.53	-0.01257	0.00	-47.12	-260.63
3	31.30	10.65	-9.79	-10.41	-0.00825	0.00	-47.12	-501.94
4	46.96	14.98	-5.46	-6.08	-0.0046	0.00	-47.12	-706.02
5	62.61	18.20	-2.24	-2.86	-0.00189	0.00	-43.01	-782.77
6	78.26	20.07	-0.37	-0.99	-0.00031	0.00	-7.11	-142.66
7	93.91	20.45	0.01	-0.61	9.1E-06	0.00	0.21	4.25
8	109.57	19.32	-1.13	-1.75	-0.00095	0.00	-21.60	-417.27
9	125.22	16.75	-3.69	-4.32	-0.00311	0.00	-47.12	-789.23
10	140.87	12.94	-7.50	-8.13	-0.00633	0.00	-47.12	-609.66
11	156.52	8.17	-12.27	-12.90	-0.01035	0.00	-47.12	-384.87
12	172.17	2.79	-17.65	-18.27	-0.01488	0.00	-47.12	-131.54
13	187.83	-2.79	-23.23	-23.86	-0.01959	0.00	-47.12	131.54
14	203.48	-8.17	-28.61	-29.23	-0.02412	0.00	-47.12	384.87
15	219.13	-12.94	-33.38	-34.00	-0.02814	0.00	-47.12	609.66
16	234.78	-16.75	-37.19	-37.81	-0.03135	0.00	-47.12	789.23
17	250.43	-19.32	-39.76	-40.38	-0.03352	0.00	-47.12	910.26
18	266.09	-20.45	-40.89	-41.52	-0.03447	0.00	-47.12	963.79
19	281.74	-20.07	-40.51	-41.14	-0.03415	0.00	-47.12	945.83
20	297.39	-18.20	-38.64	-39.27	-0.03258	0.00	-47.12	857.73
21	313.04	-14.98	-35.42	-36.05	-0.02986	0.00	-47.12	706.02
22	328.70	-10.65	-31.09	-31.72	-0.02621	0.00	-47.12	501.94
23	344.35	-5.53	-25.97	-26.59	-0.0219	0.00	-47.12	260.63

DEVELOPMENT LENGTH CHECK OF PIER REINFORCEMENT

Foundation:	Pier diameter =	4.0	ft	Cover between side of pier and cage =	3.00	in.
	Cage diameter =	3.5	ft	Cover between top of pier and cage =	3.00	in.
	Rebar size =	8		Compressive strength of concrete =	4500	psi
	Number of bars =	23		Rebar yield strength =	60000	psi
	Clear spacing =	7.77	in.			
	Are there hooks?	n				
	Check Compression?	n				
Anchor Steel:	Part number:	111965		Actual Bending Moment =	164.19	ft-kips
	Embedment length =	33.5	in.	Allowable Bending Moment =	339.88	ft-kips
	Bolt Diameter =	1.25"		Excess Reinforcement Ratio =	0.483	
Anchor Plate:	Part number:	217970				
	Plate width =	21	in.			

Required development length (compression) =	999.00	in.	
Required development length (tension) =	34.88	in.	
Required development length (tension) =	16.85	in.	(reduced)
Available development length =	20.000	in.	

OK

The length available in the pier for the development of the vertical reinforcement exceeds the required length (ACI 318-02, section 12.2).

CHECK EMBEDMENT PLATE CLEARANCE IN THE PIER

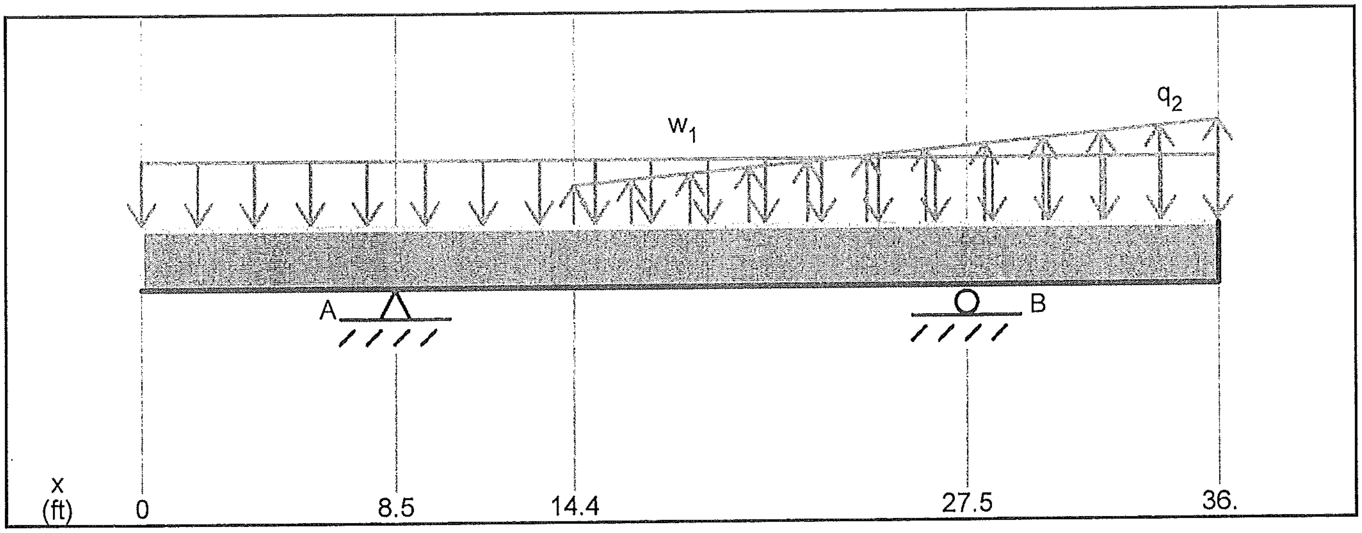
Foundation:	Pier diameter =	4.0	ft	Cover between side of pier and cage =	3.00	in.
	Cage diameter =	3.5	ft	Minimum cover between A/S and cage =	3.00	in.
Anchor Steel:	Part number:	111965		Angle of anchor steel in foundation =	3.3	degrees
	Embedment length =	33.5	in.			
Anchor Plate:	Part number:	217971				
	Largest plate width =	21.00	in.			
	Bolt Diameter =	1.25	in.			

Minimum cage diameter =	30.82	in.
Actual cage diameter =	42	in.

OK

The available space exceeds the minimum cage diameter required for anchor steel installed in the pier at an angle.

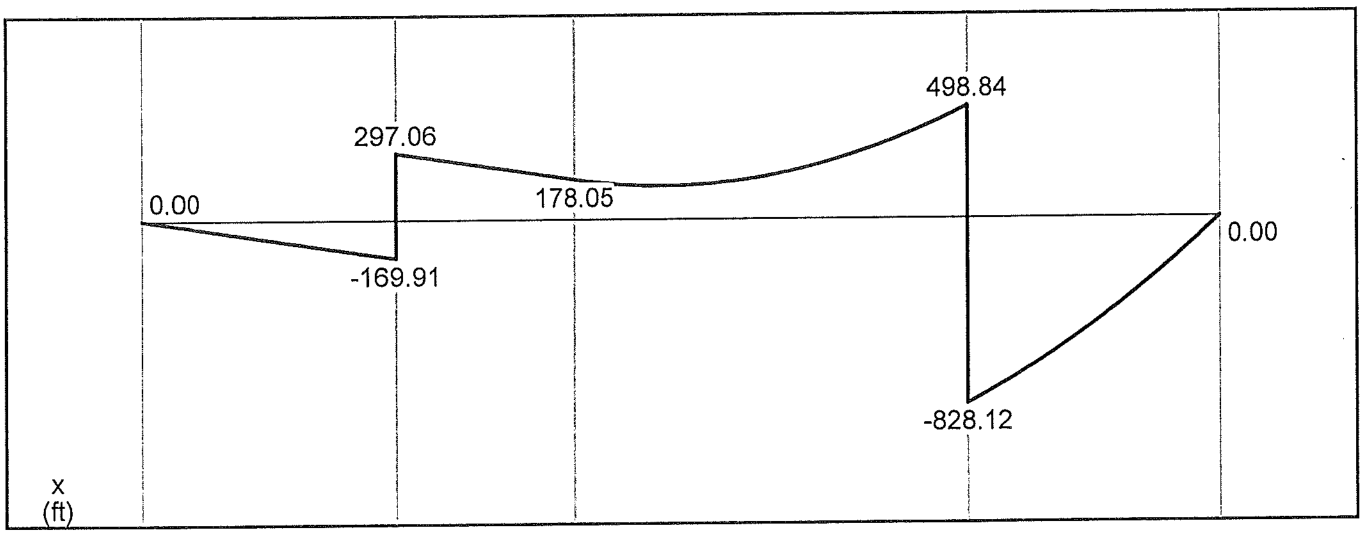
LC2 - OPTION 1



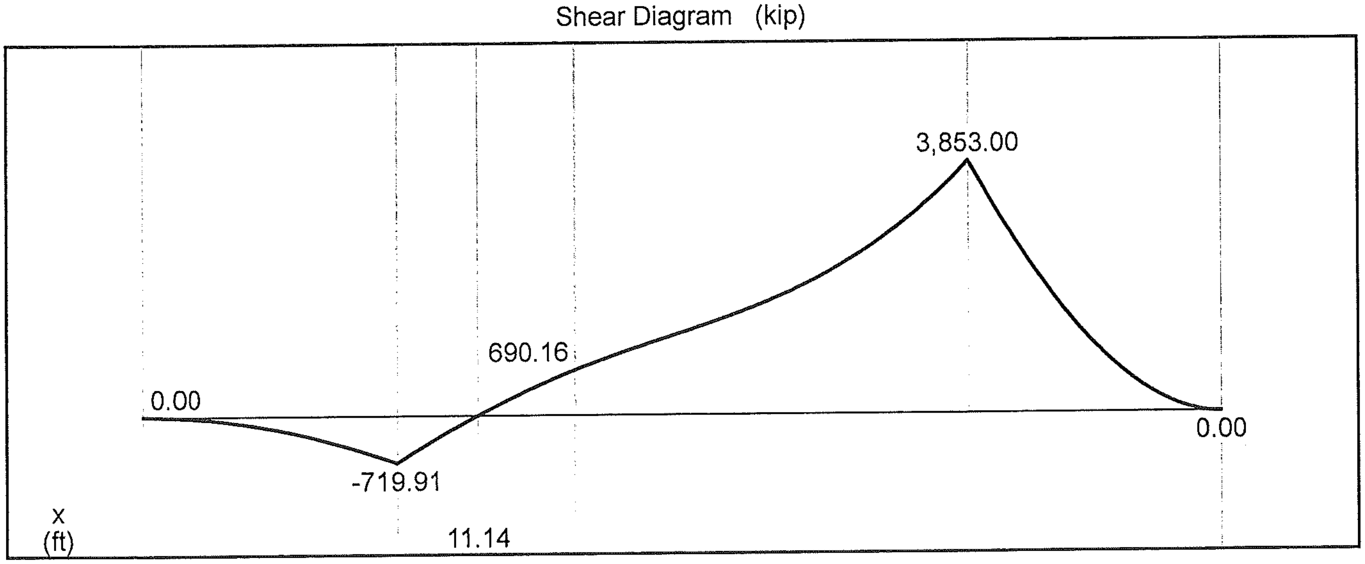
Load Diagram

w_1 = 20.05 kip/ft (down)
q_2 = 0.0 to 146.53 kip/ft (up)

A_y = 466.97 kip (up)
B_y = 1,326.96 kip (down)



Shear Diagram (kip)



Moment Diagram (kip-ft)

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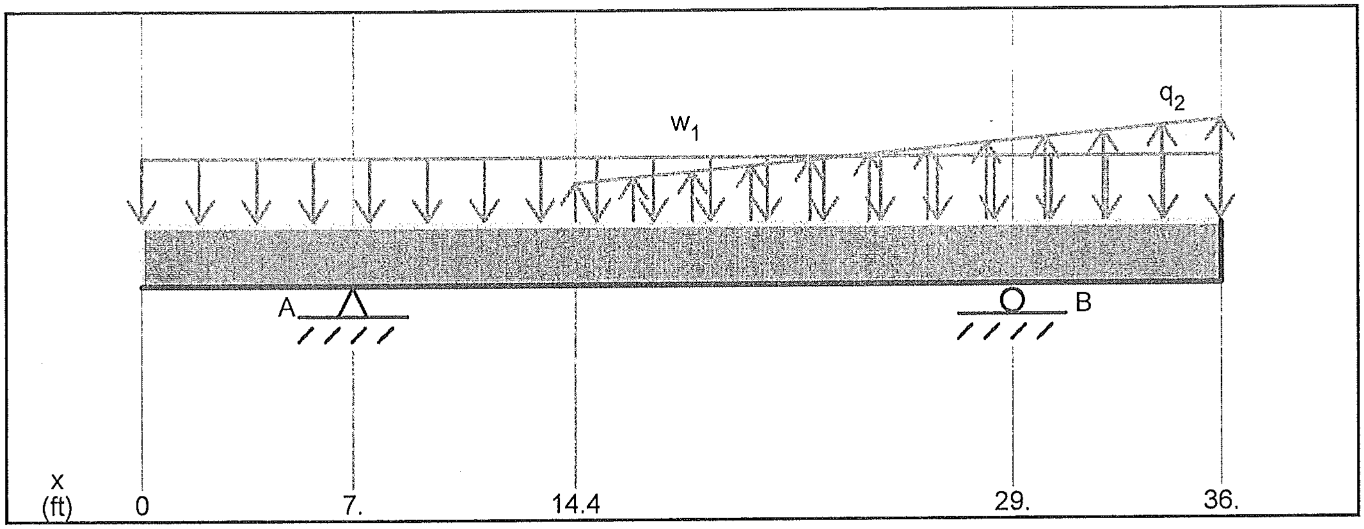
Friday, June 10, 2016

11:22:21 AM

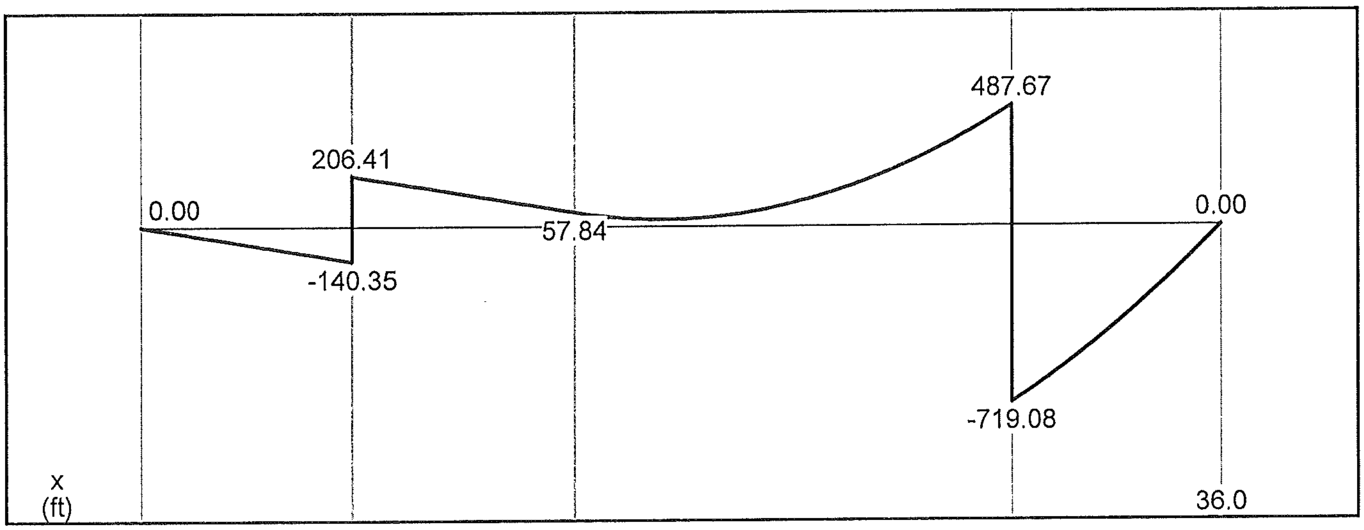
LC2 - OPTION 2



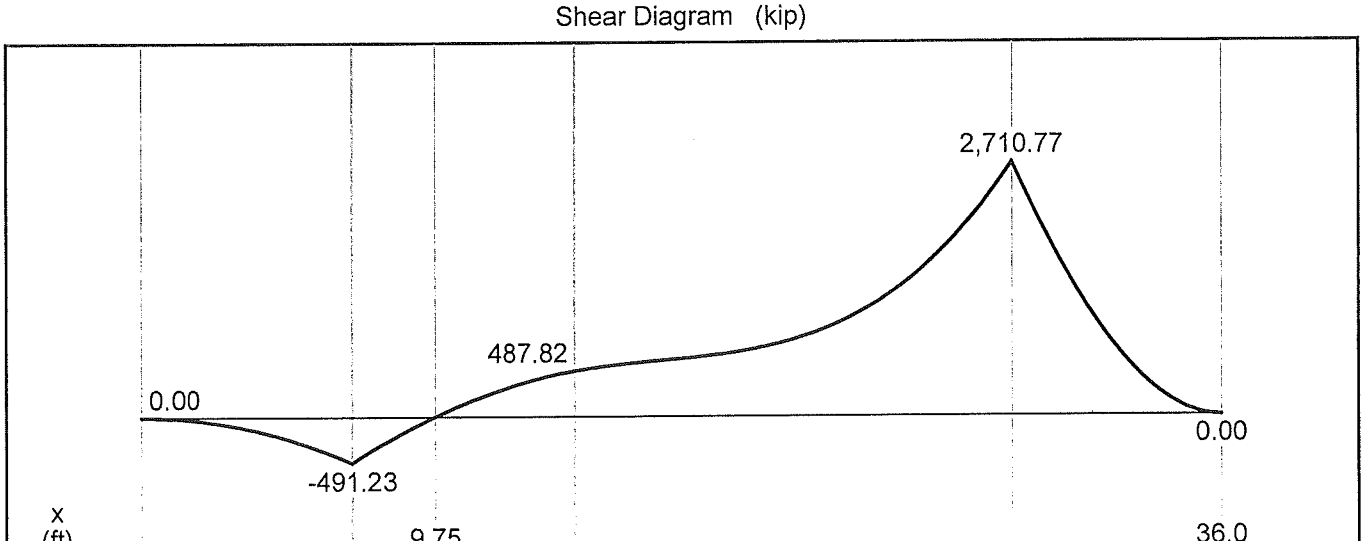
Load Diagram

w_1 = 20.05 kip/ft (down)
q_2 = 0.0 to 146.53 kip/ft (up)

A_y = 346.76 kip (up)
B_y = 1,206.75 kip (down)



Shear Diagram (kip)



Moment Diagram (kip-ft)