SHORING NOTES & PLAN

GENERAL

1. THE EARTH RETENTION DESIGNER OR SHORING ENGINEER IS DEFINED AS THE REGISTERED CIVIL ENGINEER WHOSE STAMP IS SHOWN ON THESE PLANS.
2. THE EARTH RETENTION CONTRACTOR IS DEFINED AS THE ORGANIZATION(S) OR PERSON(S) WHO INSTALLS THE EARTH SUPPORT SYSTEMS INDICATED ON THESE PLANS.
3. THE GENERAL CONTRACTOR IS AS DEFINED AS THE ORGANIZATION(S) OR PERSON(S) WITH OVERALL CONTROL OF THE SITE CONSTRUCTION.
4. THE PROJECT GEOTECHNICAL ENGINEER IS DEFINED AS THE ORGANIZATION OR PERSON WHO AUTHORED THE APPROVED SOILS/GEOLOGY REPORT ASSOCIATED WITH THIS PROJECT.
5. THESE EARTH RETENTION PLANS SHALL BE REVIEWED BY THE PROJECT GEOTECHNICAL ENGINEER PRIOR TO BEGINNING WORK.
6. LOCATION OF ALL EXISTING FACILITIES, ADJACENT STRUCTURES AND UNDERGROUND AND OVERHEAD UTILITIES SHOWN ARE AS INTERPRETED FROM INFORMATION PROVIDED TO FOUNDATION ENGINEERING CONSULTANTS DURING THE PROPOSAL & DESIGN STAGES. THE ACTUAL LOCATIONS OF ALL SUCH ITEMS SHALL BE FIELD VERIFIED PRIOR TO COMMENCING CONSTRUCTION OF THE EARTH RETENTION. ANY DISCREPANCIES OR POTENTIAL CONFLICTS BETWEEN EXISTING FACILITIES AND THIS WORK SHALL IMMEDIATELY BE BROUGHT TO THE ATTENTION OF THE EARTH RETENTION DESIGNER. THE EARTH RETENTION DESIGNER SHALL NOT BE RESPONSIBLE FOR DAMAGE TO EXISTING FACILITIES, STRUCTURES OR UTILITIES DUE TO INCORRECT INTERPRETATION OF EXISTING DRAWINGS OR DATA OR MISSING LOCATION INFORMATION.
7. AN UNDERGROUND SERVICE ALERT INQUIRY IDENTIFICATION NUMBER MUST BE OBTAINED AT LEAST TWO WORKING DAYS BEFORE STARTING WORK WITH THIS PERMIT.
8. EXCAVATION SHALL BE CO-ORDINATED WITH THE INSTALLATION OF THE EARTH RETENTION SYSTEM SO AS TO PREVENT LOSS OF GROUND AND/OR SETTLEMENT OF ADJACENT STRUCTURES. HEAVY EQUIPMENT OR CRANES SHALL NOT BE LOCATED ADJACENT TO THE EARTH RETENTION SYSTEM EXCEPT WHERE SPECIFICALLY PROVIDED FOR IN THE DESIGN.
9. THE EARTH RETENTION CONTRACTOR SHALL BE RESPONSIBLE TO ENSURE THAT THE EARTH RETENTION IS BUILT IN ACCORDANCE WITH THESE PLANS. IF THEREARE ANY QUESTIONS REGARDING THESE PLANS, THE CONTRACTOR SHALL REQUEST AN INTERPRETATION BEFORE DOING ANY WORK BY CONTACTING THE EARTH RETENTION DESIGNER IN WRITING.
10. SITE DRAINAGE AND SITE DEWATERING TO BE PROVIDED BY THE EARTH RETENTION CONTRACTOR OR BY THE GENERAL CONTRACTOR. THE GENERAL CONTRACTOR SHALL ALSO TAKE THE NECESSARY STEPS TO PROTECT THE PROJECT AND ADJACENT PROPERTIES FROM ANY EROSION AND SILTATION THAT RESULT FROM HIS OPERATIONS BY APPROPRIATE MEANS (SAND BAGS, HAY BALES, TEMPORARY DESILTING BASINS, DIKES, EARTH RETENTION, ETC.) UNTIL SUCH TIME THAT THE PROJECT IS COMPLETED AND ACCEPTED FOR MAINTENANCE BY THE OWNER.
11. ALL CONSTRUCTION SHALL CONFORM TO ALL LOCAL CODES, ORDINANCES, RESTRICTIONS AND OSHA REQUIREMENTS.
12. NO EXCAVATION OR GRADING SHALL COMMENCE UNTIL 10 DAYS AFTER ADJOINING PROPERTY OWNERS HAVE BEEN NOTIFIED IN WRITING AS REQUIRED BY SECTION 3307.1 OF THE CALIFORNIA BUILDING CODE 2013.
13. EXCAVATION VOLUMES (CUTS AND FILLS) TO BE CALCULATED BY THE PROJECT'S CIVIL ENGINEER.

GENERAL PROCEDURES

1. ALL NECESSARY PERMITS SHALL BE OBTAINED (BY OTHERS) PRIOR TO COMMENCEMENT OF INSTALLATION OF THE EARTH RETENTION SYSTEM.
2. GENERAL CONTRACTOR AND EARTH RETENTION CONTRACTOR WILL FOLLOW ALL APPLICABLE CAL-OSHA REQUIREMENTS IN THE EXECUTION OF THE WORK DESCRIBED HEREIN.
3. PRIOR TO THE START OF WORK THE EXISTING GRADE ELEVATIONS AND THE LOCATION AND DEPTH OF ALL EXISTING AND NEW UTILITIES SHALL BE DETERMINED BY THE GENERAL CONTRACTOR AND PROVIDED IN WRITING TO THE EARTH RETENTION CONTRACTOR.
4. LAYOUT OF EARTH RETENTION LINE AND ELEVATIONS SHALL BE PROVIDED BY GENERAL CONTRACTOR. WHERE APPLICABLE, ALL BUILDING OUTSIDE CORNERS SHALL BE FIELD STAKED AND EARTH RETENTION PILES LOCATED IN RELATIONSHIP TO STAKED BUILDING CORNERS TAKING INTO ACCOUNT ANTICIPATED PILE INSTALLATION TOLERANCES, PILE DEFLECTION DURING EXCAVATION AND ANY REQUIRED WATERPROOFING AND DRAINAGE. PILES MAY BE MOVED UP TO ONE FOOT IN ANY DIRECTION FROM LOCATIONS SHOWN THESE PLANS TO ACCOMMODATE AS STAKED BUILDING LOCATIONS. PILE LOCATIONS SHALL BE VERIFIED BY OTHERS IMMEDIATELY AFTER INSTALLATION OF SOLDIER BEAMS.
5. A SAFETY RAILING AROUND THE PERIMETER OF THE EXCAVATION, BUILT PER CAL-OSHA STANDARDS, WILL BE PROVIDED PRIOR TO CREATING FALL HAZARDS OVER 5 FEET.
6. SOILS ENGINEER/ENGINEERING GEOLOGIST SHALL REVIEW THESE PLANS AND CONFIRM THAT THE RECOMMENDATIONS OF THE GEOTECHNICAL ENGINEERING REPORT HAVE BEEN USED.

DEWATERING

THE SHORING DESIGN ASSUMES SITE GROUNDWATER IS AT THE BOTTOM OF THE SHORING PILE TIP. FOR PRACTICAL REASONS, HOWEVER, SHORING CONTRACTOR SHALL KEEP THE GROUNDWATER AT LEAST 2 FT BELOW BOTTOM OF THE SHORING PILE TIP.

PRE-EXCAVATION NOTES

1. ALL NECESSARY PERMITS SHALL BE OBTAINED PRIOR TO COMMENCEMENT OF SHORING INSTALLATION.
2. LOCATION OF ALL DRILLED SHAFTS TO BE APPROVED BY CIVIL ENGINEER/SURVEYOR PRIOR TO DRILLING.
3. GENERAL CONTRACTOR AND EARTH RETENTION CONTRACTOR SHALL FOLLOW ALL APPLICABLE CAL-OSHA REQUIREMENTS IN THE EXECUTION OF THE WORK DESCRIBED HEREIN.
4. PRIOR TO THE START OF WORK THE EXISTING GRADE ELEVATIONS AND THE LOCATION AND DEPTH OF ALL EXISTING AND NEW UTILITIES SHALL BE DETERMINED BY THE GENERAL CONTRACTOR AND PROVIDED IN WRITING TO THE EARTH RETENTION CONTRACTOR.
5. LAYOUT OF TEMPORARY EARTH RETENTION LINE AND ELEVATIONS SHALL BE PROVIDED TO SHORING CONTRACTOR. WHERE APPLICABLE. PILES MAY BE MOVED UP TO 6 INCHES IN ANY DIRECTION FROM LOCATIONS SHOWN ON THESE PLANS TO ACCOMMODATE AS STAKED BUILDING LOCATIONS. PILE LOCATIONS SHALL BE VERIFIED IMMEDIATELY AFTER INSTALLATION OF SOLDIER BEAMS.
6. A SAFETY RAILING AROUND THE PERIMETER OF THE EXCAVATION, BUILT PER CAL-OSHA STANDARDS, SHALL BE PROVIDED PRIOR TO CREATING FALL HAZARDS OVER 3 FEET.
7. THE SHORING CONTRACTOR SHALL NOT RELY ON SOIL BORINGS AS INDICATION OF DRILLING DIFFICULTY. THE SHORING CONTRACTOR SHALL PERFORM HIS OWN TEST EXCAVATIONS PRIOR TO BIDDING.

SPECIAL INSPECTIONS

SPECIAL INSPECTION BY CITY APPROVED DEPUTY IS REQUIRED.

CONCRETE: f'c (28 days) = 2,500psi

REINFORCING STEEL: fy=60ksi, (PERIODIC INSPECTION PRIOR TO CONCRETE PLACEMENT)

THE PROJECT INVOLVES UNUSUAL HAZARDS OR EARTH RETENTION WORK (INCLUDING SLOT CUTS WHEN REQUIRED). CONTINUOUS INSPECTION BY THE SOILS ENGINEER/GEOLOGIST IS REQUIRED FOR DRILLED PIER EXCAVATION AND SLOT CUT EXCAVATION.

THIS SPECIAL INSPECTION IS NOT A SUBSTITUTE FOR INSPECTION BY A CITY INSPECTOR. SPECIALLY INSPECTED WORK INSTALLED OR COVERED WITHOUT THE APPROVAL OF THE CITY INSPECTOR IS SUBJECT TO REMOVAL OR EXPOSURE.

THE DEPUTY INSPECTOR MUST BE APPROVED BY RELEVANT AGENCY IN ADVANCE IN ORDER TO PERFORM THE TYPES OF INSPECTION SPECIFIED.

IT IS THE RESPONSIBILITY OF THE CONTRACTOR TO SCHEDULE THE TIME WITH THE DEPUTY INSPECTOR OR INSPECTION AGENCY & SHORING AND GEOTECHNICAL FIRMS PRIOR TO PERFORMING ANY WORK THAT REQUIRES SPECIAL INSPECTION.

SPECIAL INSPECTION REPORTS SHALL BE SUBMITTED TO THE BUILDING DIVISION FOR APPROVAL PRIOR TO CITY INSPECTOR APPROVAL OF THAT WORK.

DRILLED SOLDIER BEAM PIERS

1. DRILL VERTICAL SOLDIER BEAM SHAFTS TO DEPTHS INDICATED ON THESE PLANS AND WITHIN ACCEPTABLE TOLERANCES FOR LOCATION AND ALIGNMENT. WHERE DRILLED SHAFTS WILL NOT STAY OPEN, PROVIDE PROTECTION AGAINST SLOUGHING OR CAVING AS REQUIRED.
2. PLACE SOLDIER PILES IN DRILLED SHAFTS AND POUR 2,500psi CONCRETE WITHIN EMBEDDED PORTION OF THE SOLDIER PILE. MAINTAIN VERTICALITY OF SOLDIER PILES DURING CONCRETE POUR. FILL SHORED HEIGHT OF SHAFT WITH A COURSE GRAVEL BACKFILL.
3. IF GROUNDWATER IS ENCOUNTERED, CONCRETE FOR PILE TOES SHOULD BE TREMIED INTO POSITION WITH APPROVED DEVICES, AND COMPRESSIVE STRENGTH OF CONCRETE SHALL BE INCREASED BY 1,000 PSI.
4. SHAFTS CLOSER THAN TWO SHAFT DIAMETERS ON CENTER SHALL BE DRILLED AND POURED IN AN ALTERNATING SEQUENCE SUCH THAT THE ALTERNATE SHAFT IS NOT DRILLED UNTIL THE ADJACENT PILE HAS CURED FOR AT LEAST 24 HOURS.

LAGGING

1. BEGIN EXCAVATION IN LIFTS NOT EXCEEDING 4 FT. UNLESS APPROVED BY THE GRADING INSPECTOR, THE GEOTECHNICAL ENGINEER AND THE EARTH RETENTION DESIGNER.
2. CLEAN SOLDIER PILES AS REQUIRED. PLACE LAGGING AS INDICATED ON THE EARTH RETENTION PLAN.
3. FOR CANTILEVER PILES, CONTINUE THIS SEQUENCE TO THE BOTTOM OF EXCAVATION.

MONITORING

1. PHOTOGRAPHS, VIDEOS, SURVEY MEASUREMENTS AND OTHER METHODS SHALL BE EMPLOYED TO DOCUMENT ADJACENT RESIDENCES. THIS DOCUMENTATION SHOULD BE DONE PRIOR TO STARTING SHORING EXCAVATIONS. IF POSSIBLE, THE INTERIOR CONDITION OF THE NEIGHBORING STRUCTURES SHOULD BE DOCUMENTED. THIS BECOMES THE BASIS OF AS IS CONDITION PRIOR TO SHORING & OPERATIONS.
2. COMMENCING PRIOR TO EXCAVATION, EVERY FOURTH EARTH RETENTION PILE SHALL BE SURVEY MONITORED WEEKLY BY A LICENSED SURVEYOR FOR LATERAL AND VERTICAL MOVEMENT. MOVEMENT DATA SHALL BE PROVIDED TO THE SHORING DESIGNER ON A BI-WEEKLY BASIS.
3. THE MOVEMENT DATA SHOULD BE UTILIZED BY THE GENERAL CONTRACTOR AND THE EARTH RETENTION CONTRACTOR TO DETERMINE IF LAGGING POSITIONS NEED TO BE ADJUSTED TO MAINTAIN THE LAGGING TOLERANCES AGREED FOR THE PROJECT.
4. ONCE EXCAVATION SUBGRADE IS REACHED AND THE PILE MOVEMENT DATA INDICATES THE EARTH RETENTION SYSTEM IS STABLE THEN MONITORING INTERVALS MAY BE INCREASED TO MONTHLY UNTIL THE EARTH SUPPORT SYSTEM IS NO LONGER REQUIRED. ANY MOVEMENT SHALL BE ANALYZED BY THE SOILS ENGINEER AND AN APPROVED REMEDIAL EARTH RETENTION PLAN PREPARED.

PROCEDURE FOR INSTALLATION OF DRILLED PIERS/SHORING

1. STRUCTURAL OBSERVATION BY DESIGN ENGINEER IS REQUIRED.
2. DRILL THE SPECIFIED DRILLED PIER SHAFT TO THE BOTTOM OF THE DESIGN PILE TIP.
3. REMOVE ALL LOOSE MATERIALS. CASING IS NOT ANTICIPATED ON THIS SITE.
4. PLACE DESIGN WIDE FLANGE SOLDIER BEAM IN ITS STRONG AXIS.
5. POUR FIRST, 2,500 PSI CONCRETE WITH TYPE II CEMENT FROM DRILLED PIER TIP TO BASE OF EXCAVATION. POUR 1 SACK SLURRY MIX FROM BASE OF EXCAVATION TO TOP OF SHORING. SLUMP TO BE 5" MAXIMUM.
6. ALLOW MAXIMUM OF 4 FT VERTICAL CUT AT ANY GIVEN TIME.
7. INSTALL 3X12 PRESSURE TREATED WOOD LAGGING FROM THE TOP DOWN. ENSURE 3/4" VERTICAL SEPARATION BETWEEN LAGGING BOARDS (LOUVRE).
8. DRILLING, STEEL BEAM PLACEMENT, AND CONCRETE/SLURRY BACKFILL PLACEMENT SHALL BE OBSERVED BY THE GEOTECHNICAL FIRM CONTINUOUSLY.
9. EXCAVATION CAN BEGIN 5 DAYS AFTER DRILLED PIER SHAFTS ARE POURED.
10. WIDE FLANGE, PIER DIAMETER, EMBEDMENT DEPTH, AND PIER SPACING SHALL BE APPROVED BY THIS FIRM PRIOR TO PLACING CONCRETE.

POST-GROUTED TIEBACK INSTALLATION

PRIOR TO START OF THE WORK, VERIFY LOCATION OF UTILITIES. LOCATE TIEBACKS TO ASSURE CLEARANCE OF EXISTING OR NEW LINES.

A SUITABLE FIRM, DRY AND LEVEL BENCH SHALL BE PROVIDED "BY EXCAVATOR" FOR TIEBACK INSTALLATION AT EACH TIEBACK LEVEL. ALL BENCHING AND SLOPING SHALL CONFORM TO OSHA REQUIREMENTS.

TIEBACK ANGLE AND LOCATIONS MAY BE ALTERED, PROVIDED THE DESIGN LOADS ARE ADJUSTED ACCORDINGLY BY THE EARTH RETENTION DESIGNER. A TOLERANCE OF FIVE DEGREES ON THE ANGLE OF INCLINATION, AND 12" ± ON LOCATION WILL BE PERMITTED.

MACHINE DRILL THE TIEBACK SHAFT (WITH A TEMPORARY CASING WHERE REQUIRED) TO PREVENT SLOUGHING OR CAVING OF MATERIAL INTO THE HOLE. BOREHOLE DIAMETERS SHALL BE AS INDICATED ON THESE PLANS.

WHEN THE SHAFT HAS BEEN ADVANCED TO TIP, ENSURE ADEQUATE CLEANING OF THE BOREHOLE WITH AIR OR WATER FLUSHING AND INSTALL THE PREFABRICATED ANCHOR WITH ATTACHED CENTRALIZING DEVICES INTO THE BOREHOLE. THE PREFABRICATED ANCHOR SHALL CONSIST OF: HIGH-STRENGTH STEEL TENDONS OR RODS, SPACERS TO SEPARATE THE INDIVIDUAL STRANDS, CENTRALIZERS TO CENTRALIZE THE ASSEMBLY, TREMIE GROUT TUBE, POST-GROUT TUBE(S) WITH GROUT VALVES AND A PVC SHEATHING IN THE FREE STRESSING LENGTH.

FILL THE BOREHOLE THROUGH A MIN. 1/2" I.D. POLYETHYLENE TREMIE PIPE WITH APPROVED HIGH-STRENGTH TERMINATE TREMIE GROUTING WHEN THE BOREHOLE IS COMPLETELY FILLED.

ONCE THE TREMIE GROUT HAS ATTAINED ITS INITIAL SET, PERFORM POST-GROUTING OF THE ANCHOR BOND ZONE THROUGH THE ATTACHED POST-GROUT LINE AND VALVES. THE POST-GROUT LINE CONSISTS OF A 1/2" SCH 40 PVC PIPE WITH RUBBER VALVES AT FOUR FEET O.C. IN THE POST-GROUT ZONE. FRACTURE THE INITIALLY SET BOND ZONE GROUT BY INJECTING WATER OR CEMENT GROUT UNDER PRESSURE. AFTER INITIAL FRACTURE, PUMP NEAT GROUT UNTIL A MINIMUM PRESSURE OF 300 PSI IS ACHIEVED. FLUSH THE POST-GROUT LINE WITH WATER FOR REUSE. ACTUAL REQUIRED INJECTION PRESSURES AND GROUT VOLUMES WILL VARY DEPENDING ON GROUND CONDITIONS AND HOLDING CAPACITIES REQUIRED.

THE ANCHOR SHALL REMAIN UNDISTURBED UNTIL THE GROUT HAS CURED A MINIMUM OF THREE DAYS.

AFTER TESTING THE TIEBACKS, LOCK OFF TO TIEBACK DESIGN LOAD INDICATED ON THESE PLANS.

CUT OFF EXCESS TENDON LEAVING A MINIMUM OF FOUR INCHES BEYOND THE ANCHOR PLATE.

TIEBACK TESTING

EACH ANCHOR SHALL BE SATISFACTORILY PROOF TESTED TO A MINIMUM OF 133% OF THE DESIGN LOAD. PROOF TESTING LOADING SHALL BE APPLIED INCREMENTS OF 25% OF DESIGN LOAD WITH THE INCREMENTAL LOAD HELD FOR 1 MINUTE. THE PROOF LOAD OF 133% OF DESIGN LOAD SHALL BE HELD FOR 15 MINUTES. THE ANCHOR TENDON INCLUDING TENDON STRETCH SHALL NOT MOVE MORE THAN 12" TOTAL DURING APPLICATION OF THE TEST LOAD FROM 0% TO 133%. AT 133% TEST LOAD THE ANCHOR MOVEMENT SHALL NOT EXCEED 0.1 INCH DURING A 15 MINUTE TEST PERIOD.

AFTER TESTING ALL ANCHORS SHOULD BE LOCKED OFF THE THE SPECIFIED DESIGN LOAD AND THE LOCK-OFF LOAD SHALL BE VERIFIED BY MEANS OF A LIFT OFF TEST. IF THE LOCK-OFF LOAD VARIES BY MORE THAN 10% FROM THE DESIGN LOAD, THE LOAD SHOULD BE RESET UNTIL THE LOCK-OFF LOAD IS WITHIN 10% OF THE DESIGN LOAD.

CERTIFICATION FROM AN APPROVED TESTING LABORATORY IS REQUIRED FOR THE CALIBRATION OF THE ANCHOR LOADING DEVICES AT THE START OF EACH JOB AND AT 60 DAY INTERVALS THEREAFTER.

NO TEST SHALL BE PERFORMED UNTIL THE CONCRETE HAS ATTAINED A MINIMUM UNCONFINED COMPRESSIVE STRENGTH OF 2500psi (TYPICALLY 5 DAYS AFTER FINAL GROUTING).

THE DEPUTY INSPECTOR SHALL INSPECT AND APPROVE THE TESTING OF ALL ANCHORS. HE SHALL KEEP A RECORD OF ALL TEST LOADS AND TOTAL ANCHOR MOVEMENTS AND CERTIFY TO THEIR ACCURACY. THIS RECORD SHALL BE KEPT ON THE JOB SITE AND SHALL BE AVAILABLE FOR INSPECTION BY THE BUILDING INSPECTOR.

UPON COMPLETION OF THE ANCHOR INSTALLATION AND TESTING THE DEPUTY INSPECTOR SHALL SUBMIT A REPORT TO THE DEPARTMENT STATING THAT THE INSTALLATION AND TESTING OF ALL ANCHORS IS IN CONFORMANCE WITH THE RECOMMENDATIONS OF THE FOUNDATION REPORT AND ANY SUPPLEMENTS.

ANCHORS FAILING THE TEST CRITERIA SHALL BE DISAPPROVED EXCEPT THAT REDUCED CAPACITIES MAY BE ASSIGNED TO SUCH ANCHORS UPON RECOMMENDATION OF THE EARTH RETENTION ENGINEER AND APPROVAL BY THE DEPARTMENT.

CODES, SPECIFICATIONS AND MATERIALS

CALIFORNIA BUILDING CODE – 2019 EDITION
STEEL DESIGN – AISC 14th EDITION
CONCRETE DESIGN – ACI 318-14

STEEL WIDE FLANGE STEEL : ASTM A-572 OR A-992 GR-50
(NEW CONDITION WITH APPLICABLE CERTIFICATES)
WELDING ELECTRODES: E-70 KSI

WOOD LAGGING: PRESSURE TREATED REDWOOD, HEM FIR, OR DOUGLAS FIR:
MINIMUM F_b = 1300 PSI OR BETTER

CEMENTITIOUS– CEMENT: ASTM C-150, TYPE II
DRILLED PILE BACKFILL: 2,500 psi CONCRETE IN PILE TOES.
DESIGN COMPRESSIVE STRENGTH AT 28 DAYS = 2,500psi
NEAT CEMENT SHALL BE 2,500 PSI MINIMUM COMPRESSIVE STRENGTH AT 28 DAYS WITH 5 GALLONS OF WATER TO 94 POUND BAG OF CEMENT PROPORTION.

CANTILEVER PILE SCHEDULE

PILE #	SHORED HEIGHT [ft]	EMBEDMENT [ft]	TOTAL PILE LENGTH [ft]	PIER Ø [ft]	BEAM SIZE
P1 : P14	12.00	17.0	29.00	2.0	W14x53
P15 : P16	19.00	25.0	44.00	2.5	W16x100
P17 : P61	12.00	17.0	29.00	2.0	W14x53

NOTES
1. ALL WOOD LAGGING SHALL BE P.T. 3X12; Fb=1300 psi

RAKER PILE SCHEDULE

PILE #	SHORED HEIGHT [ft]	EMBEDMENT [ft]	TOTAL PILE LENGTH [ft]	PIER Ø [ft]	BEAM SIZE	RAKER SIZE	RAKER BASE PLATE
R1 : R3	19 (A = 7' ; B = 12')	12.0	31.00	2.0	W14x53	8" SCHD 80	12"x12"x1"

NOTES
1. ALL WOOD LAGGING SHALL BE P.T. 3X12; Fb=1300 psi

TIEBACK S C H E D U L E

SOLDIER BEAMS				ANCHORS							
SOLDIER BEAM NUMBER & SIZE	MAX. VERT. HT. ABOVE BOTT OF EXCAV. 'H'	EMBED BELOW BOTT OF EXCAV.	PIER DIA	DESIGN LOAD (KIPS)	133% TEST LOAD (KIPS)	# OF Ø0.6" x 7-WIRE STRANDS ASTM A416 (270 KSI)	ANCHOR ANGLE (DEG)	ANCHOR DIAMETER	MIN UNBONDED LENGTH (FEET)	MIN BOND LENGTH (FEET)	TOTAL SHAFT LENGTH (FEET)
TB1 : TB28 W14x53	19'	12'	24"	146.6	195	5	20	6"	15	39*	54

* ACTUAL LENGTH SHALL BE CALCULATED BY THE SHORING SUBCONTRACTOR TO MEET THE TEST LOAD AND ELONGATION REQUIREMENTS.

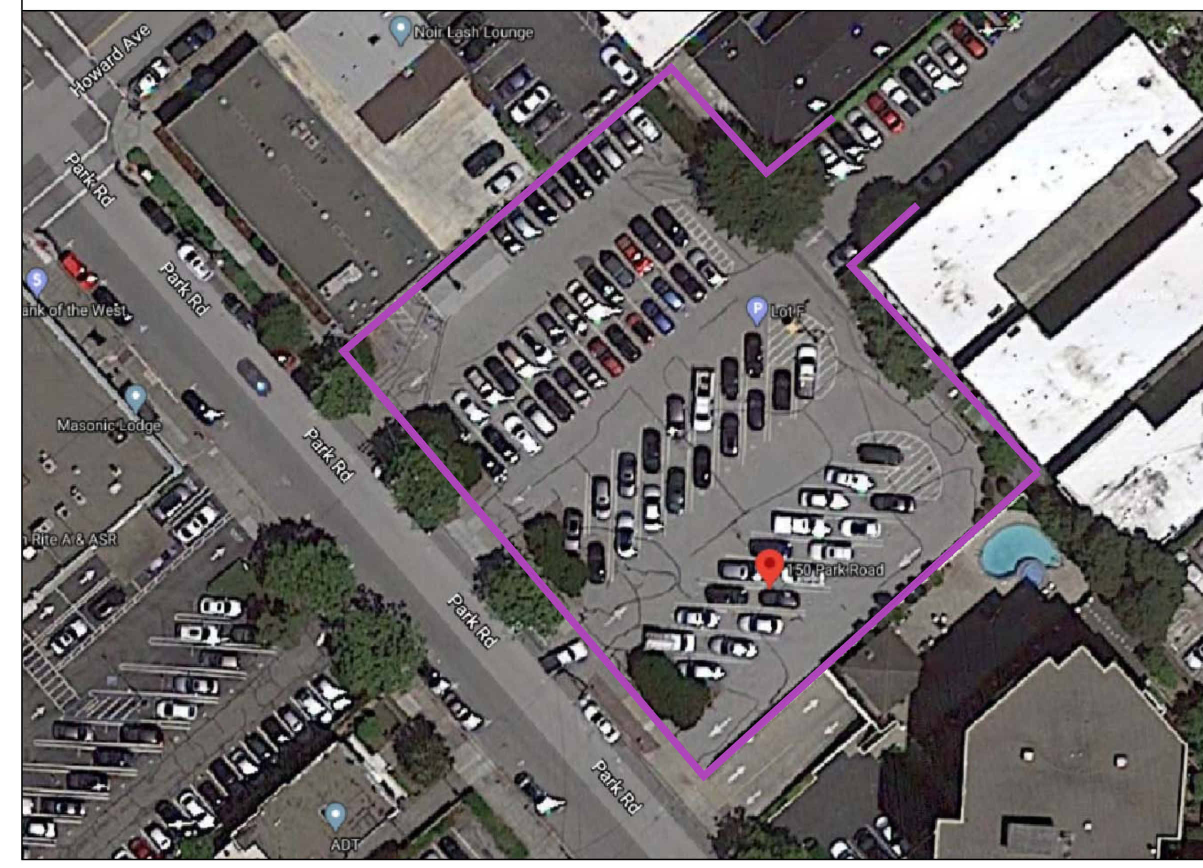
ELONGATION CALCULATIONS

T_L = 195 KIP

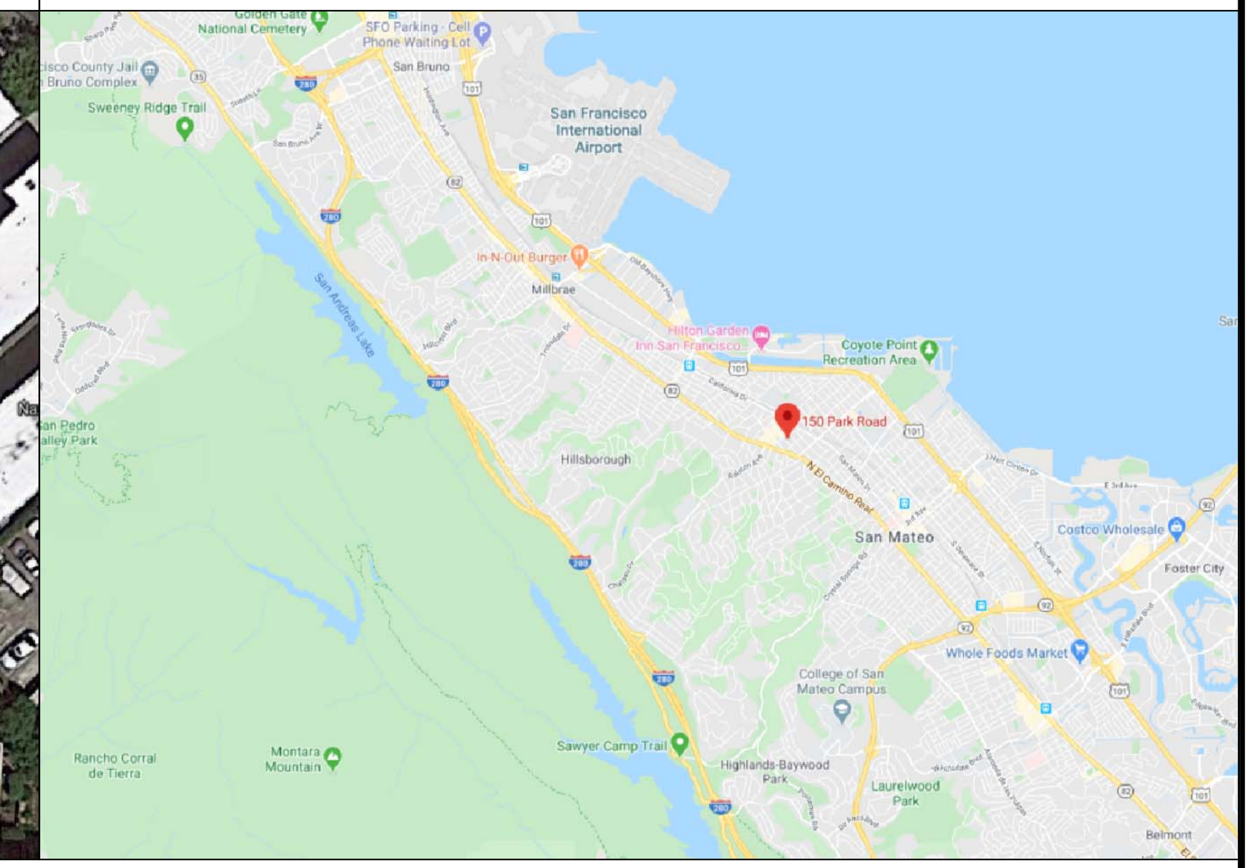
$$\Delta = \frac{0.9 * T_L * L}{A * E} = \frac{0.9*195\ k*19'}{1.09\ in^2 * 29{,}000\ ksi} = 1.27"$$

MIN REQ'D ELONGATION = 0.8*Δ = 1.01"

AERIAL PHOTO



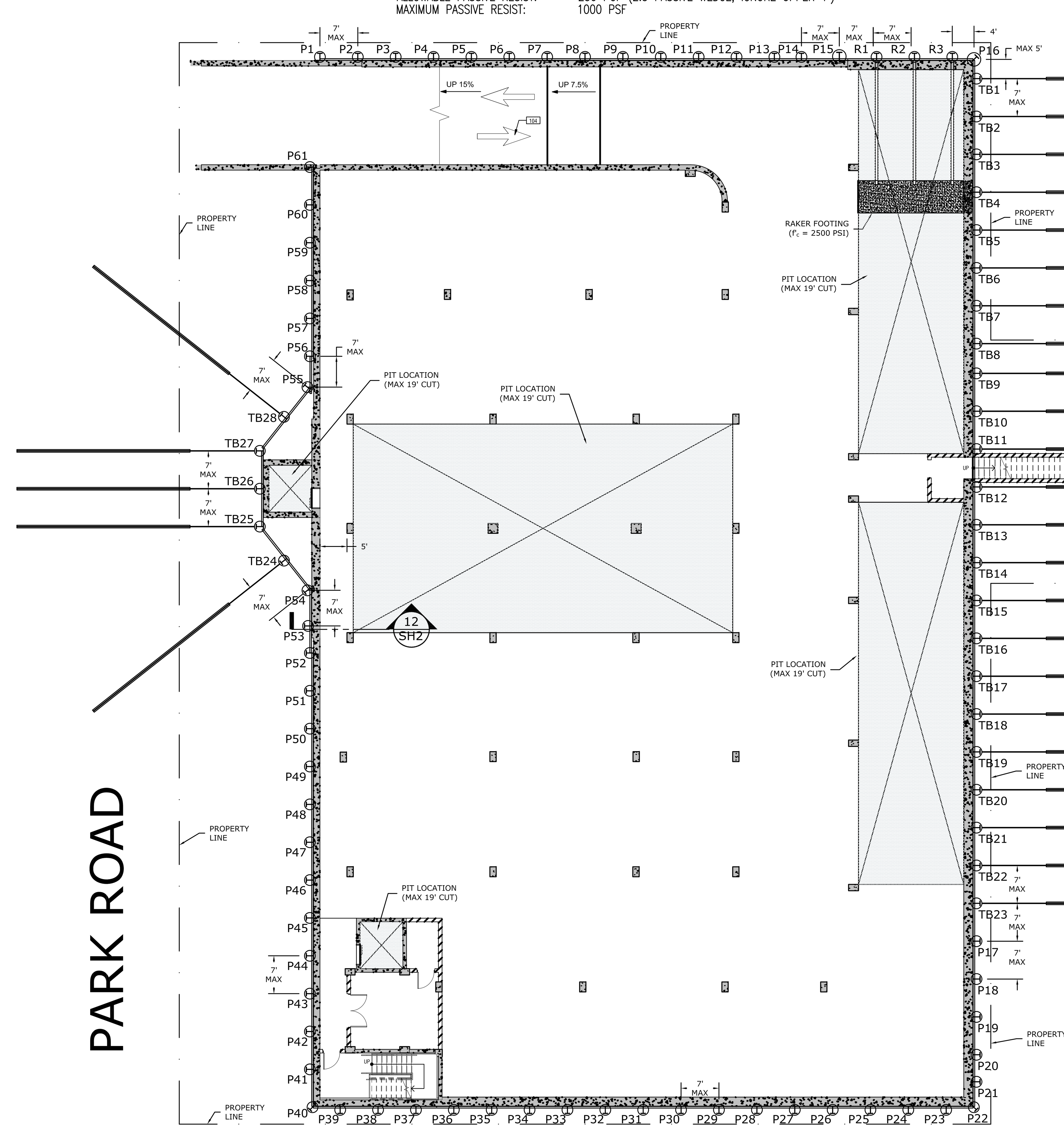
VICINITY MAP



DESIGN PARAMETERS

REFERENCE:
BASED ON GEOTECHNICAL INVESTIGATION BY ACE QUALITY CONTROL, DATED 1/9/19, FILE #10-16013GF.

BACKFILL PRESSURE:	40 PCF
SOIL UNIT WEIGHT:	125 PCF
ALLOWABLE PASSIVE RESIST:	250 PCF (2.5 PASSIVE WEDGE; IGNORE UPPER 1')
MAXIMUM PASSIVE RESIST:	1000 PSF



SHORING PLAN
SCALE: 1/16" = 1'

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a/e stamp

client

agency approval

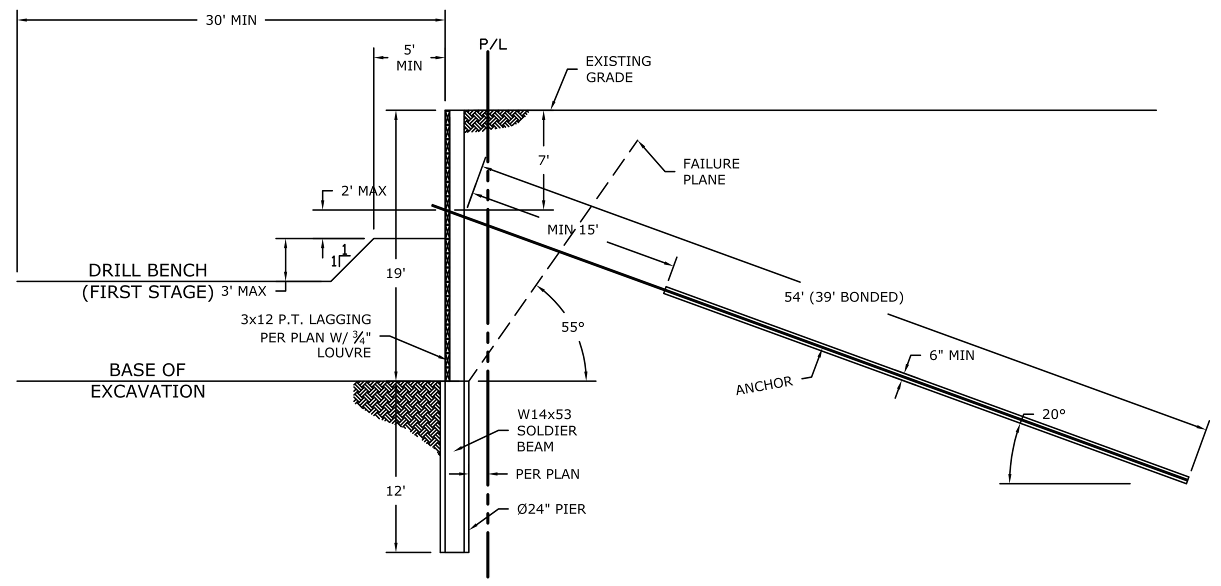
project info

drawing name
SHORING PLAN

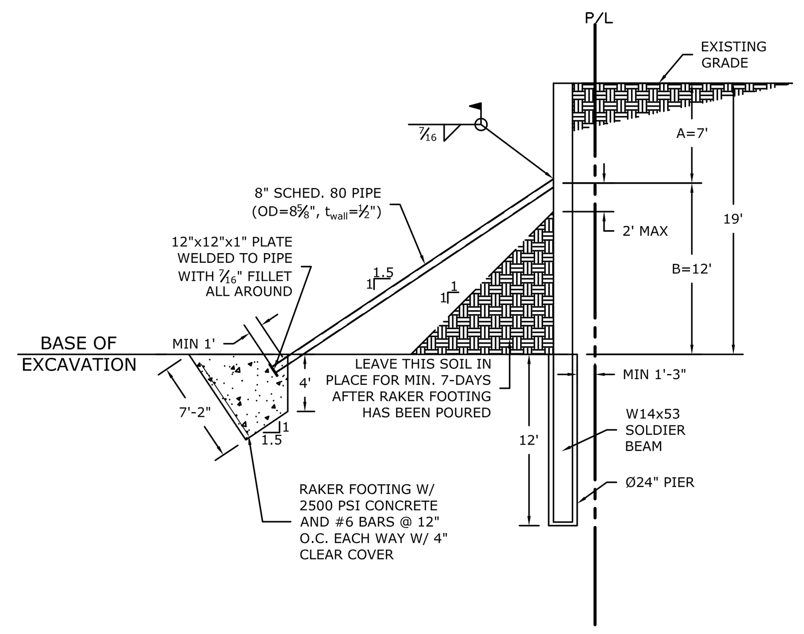
issue	date	
CITY SUBMIT	--	project no. 1718
		drawn by JWF
		checked by LA
		date March 12, 2020

drawing no.
SH1

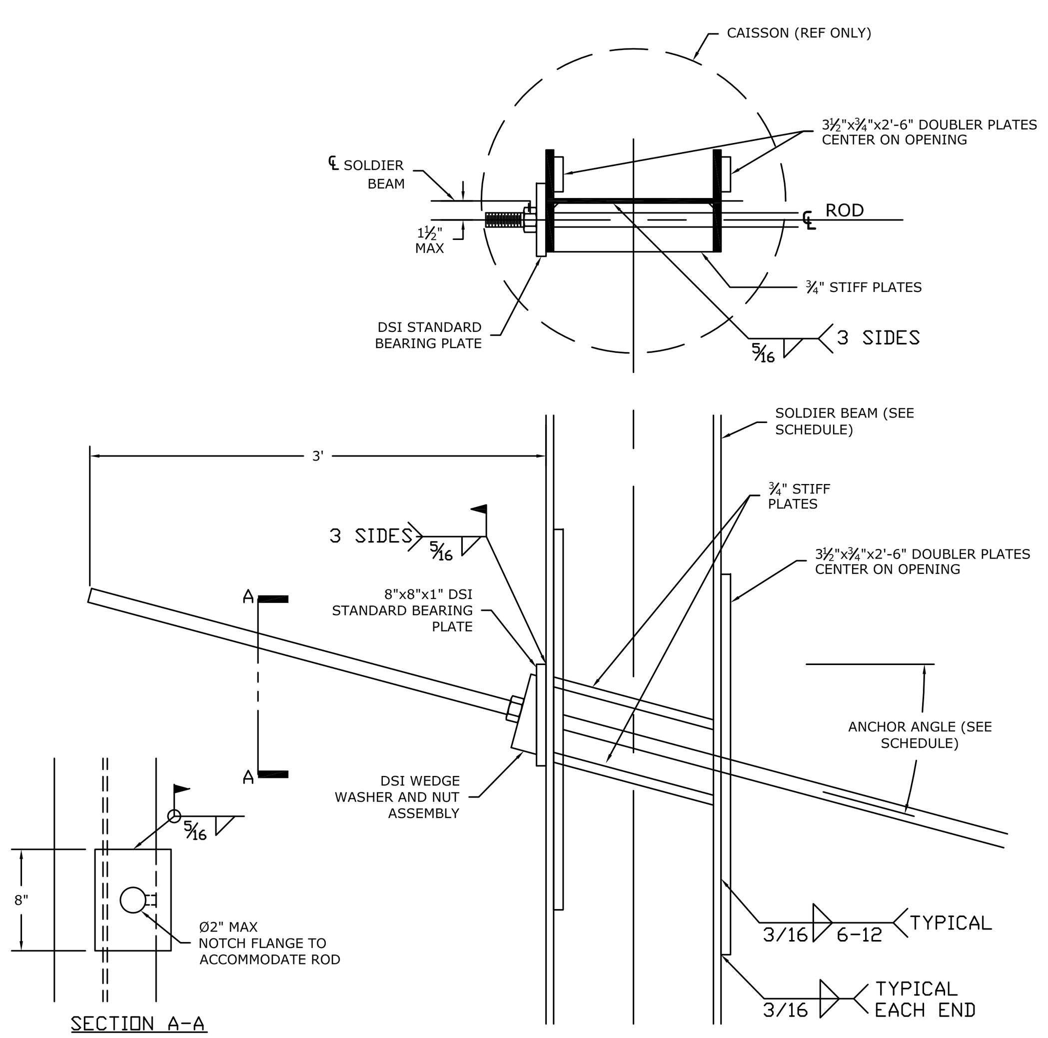
TIEBACK, RAKER, & CANTILEVER DETAILS



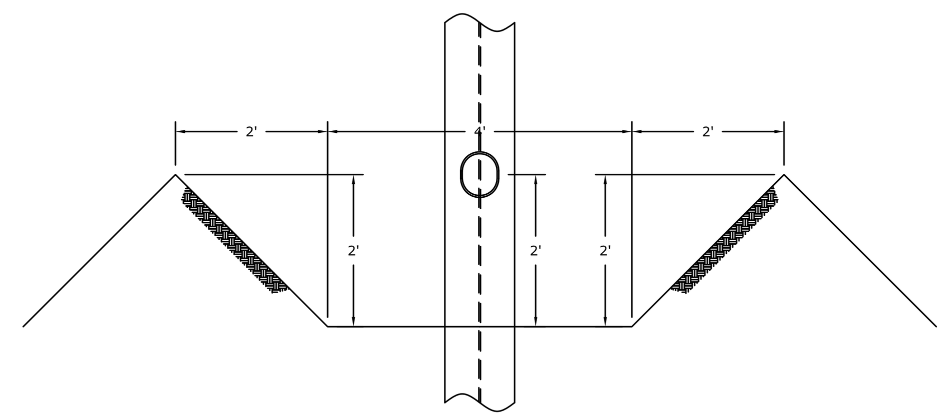
1 SH2 TYPICAL TIEBACK SECTION VIEW
SCALE: 3/32" = 1'-0"



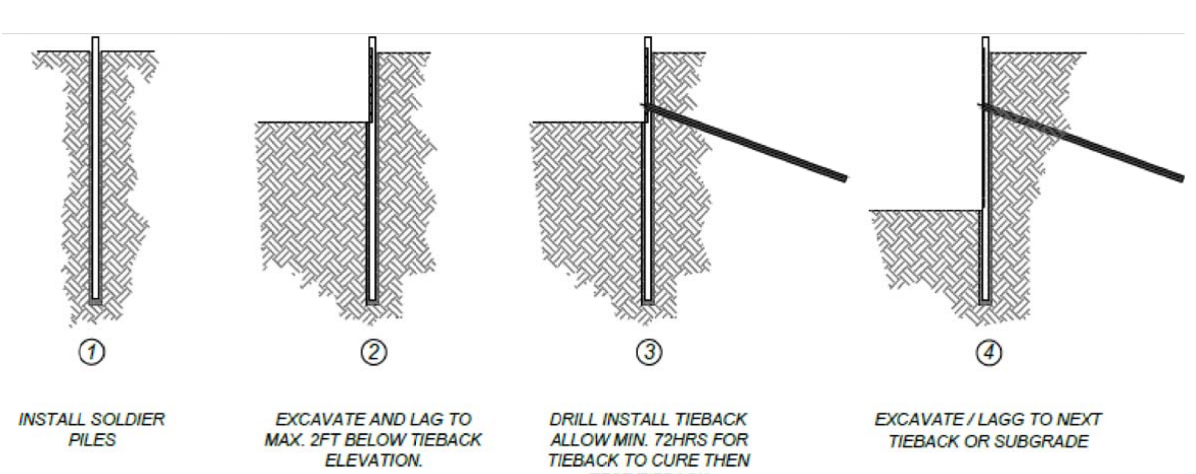
4 SH2 TYPICAL RAKER SECTION VIEW
SCALE: 3/32" = 1'-0"



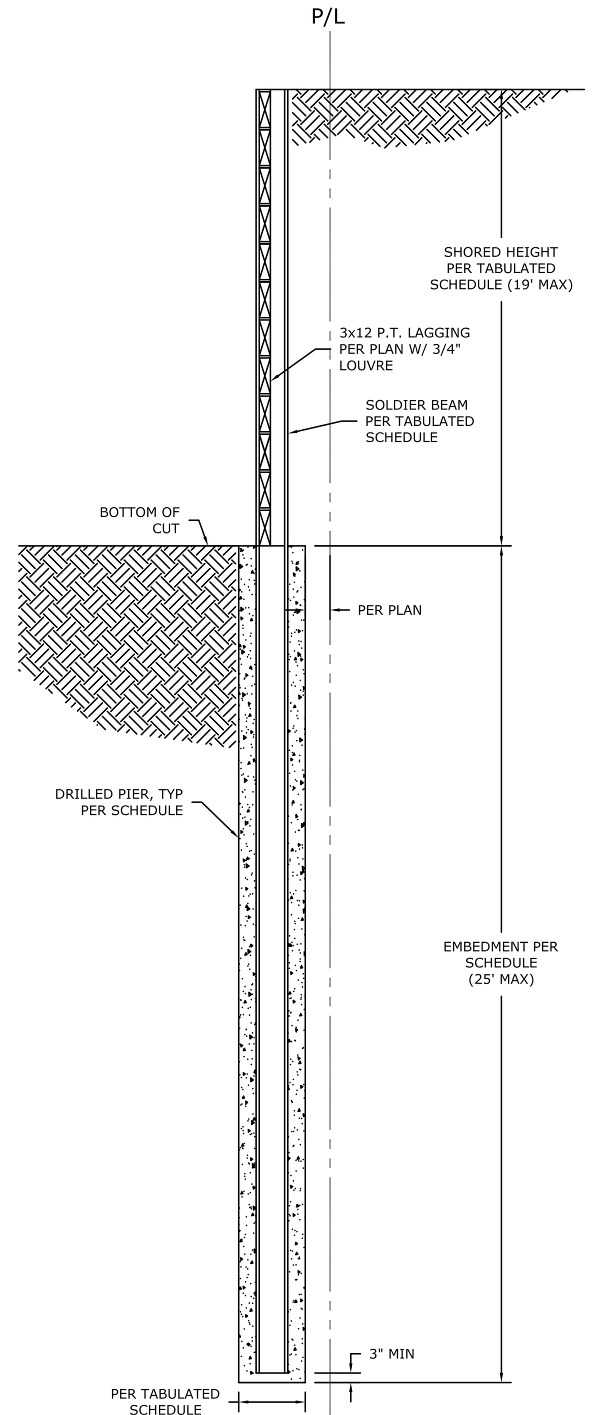
2 SH2 SOLDIER BEAM POCKET FOR ANCHOR
SCALE: NTS



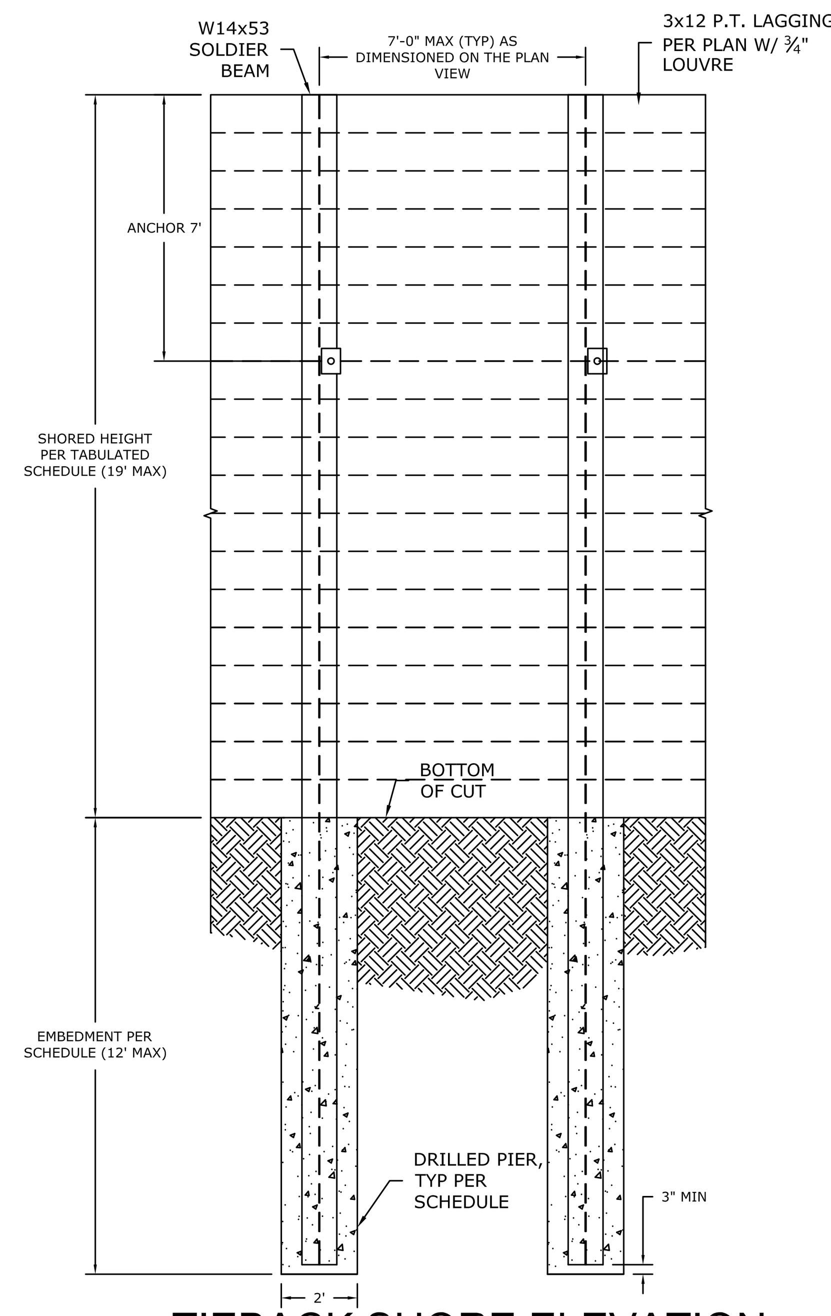
5 SH2 CUT BENCH AT RAKER
SCALE: 1/2" = 1'-0"



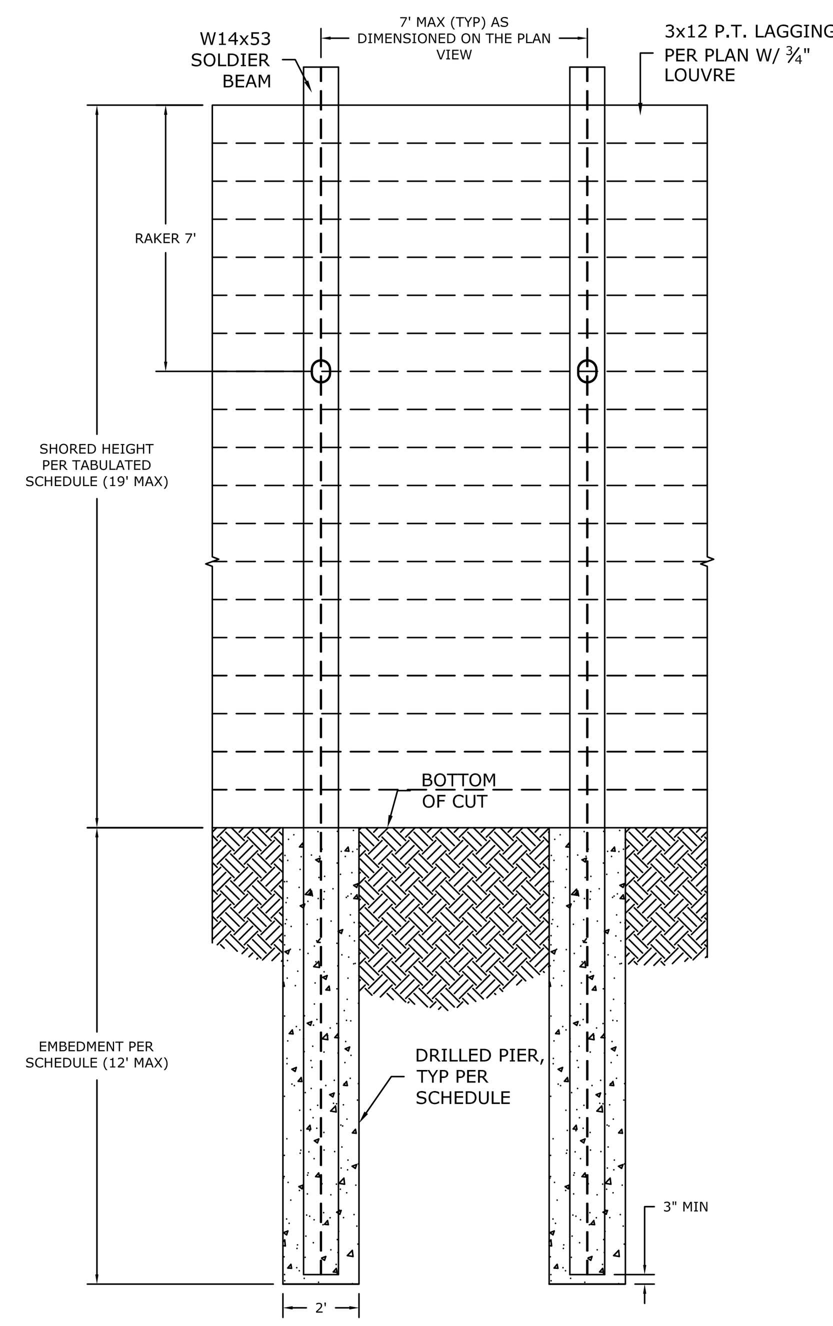
3 SH2 SEQUENCE OF TIEBACK CONSTRUCTION
SCALE: NTS



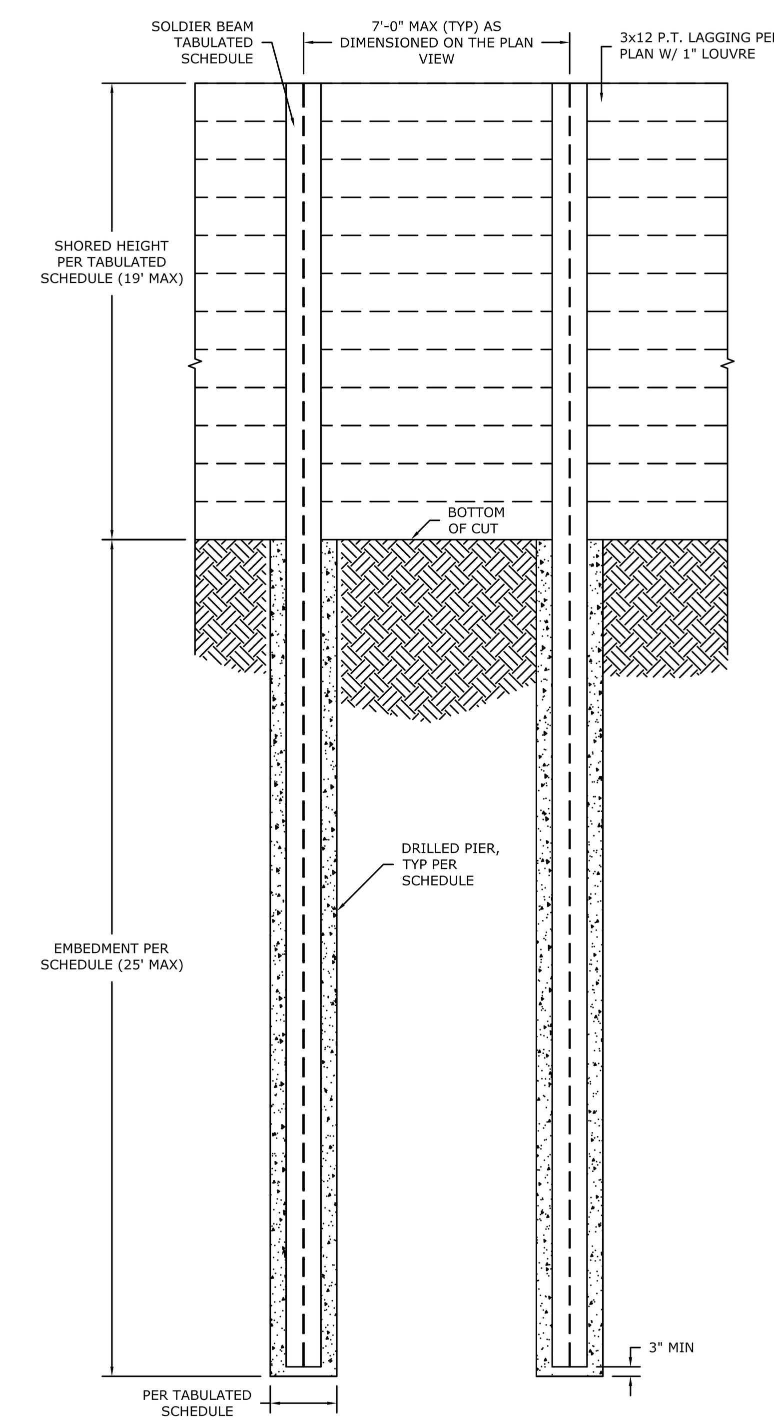
6 SH2 CANTILEVER SHORING SECTION
SCALE: NTS



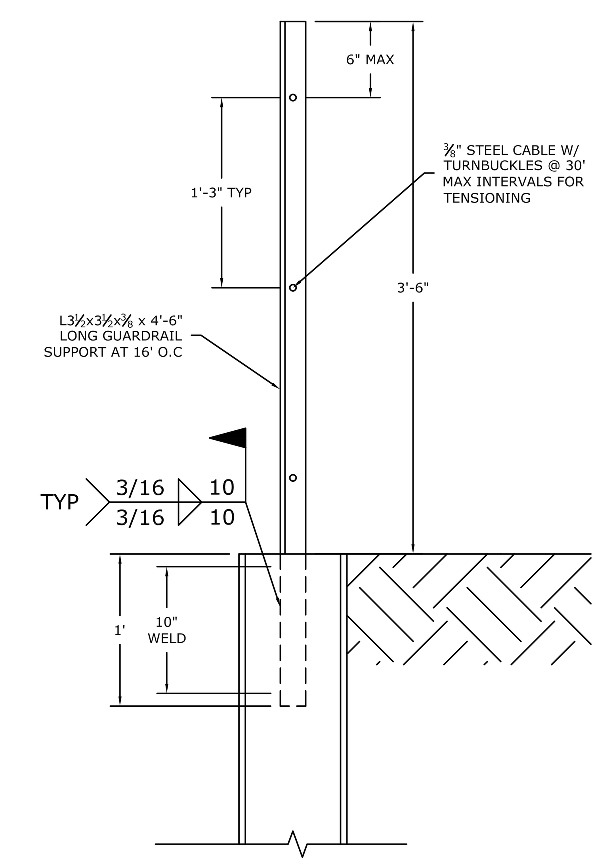
7 SH2 TIEBACK SHORE ELEVATION
SCALE: 1/4" = 1'-0"



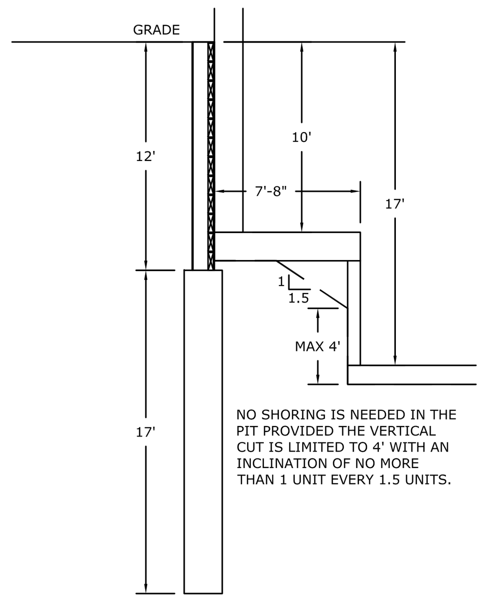
8 SH2 RAKER SHORE ELEVATION
SCALE: 1/4" = 1'-0"



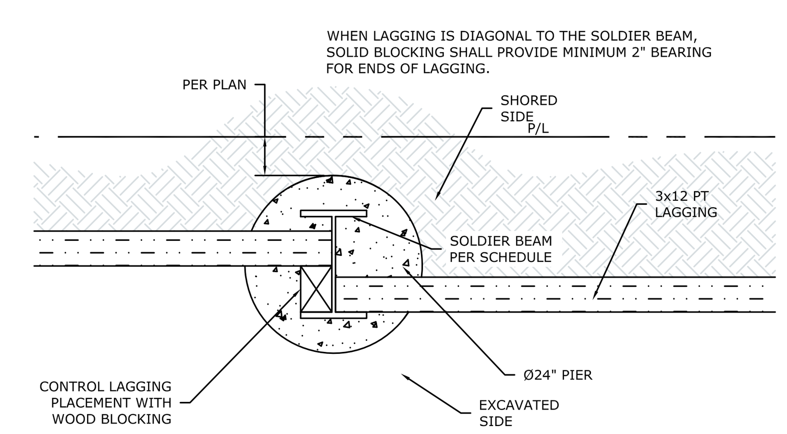
9 SH2 CANTILEVER SHORING ELEVATION
SCALE: NTS



11 SH2 TYPICAL GUARD RAIL
SCALE: 1" = 1'-0"

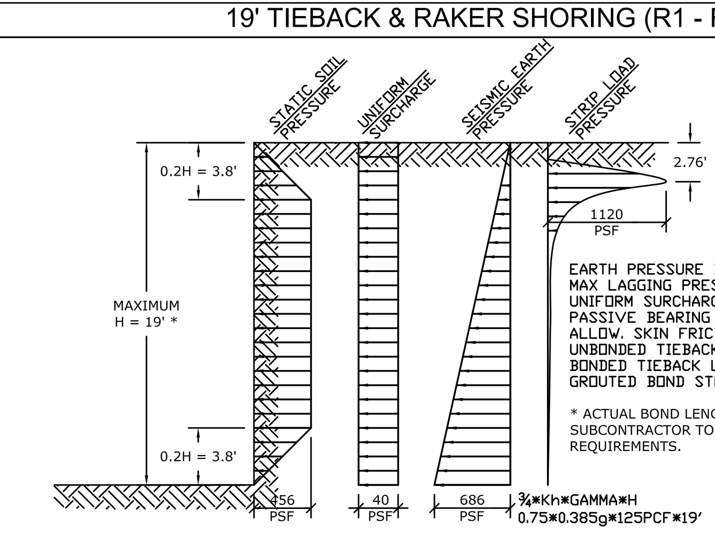


12 SH2 SECTION @ PIT
SCALE: 1/8" = 1'



10 SH2 WOOD LAGGING PLAN VIEW
SCALE: NTS

19' TIEBACK & RAKER SHORING (R1 - R3; TB1 - TB28)



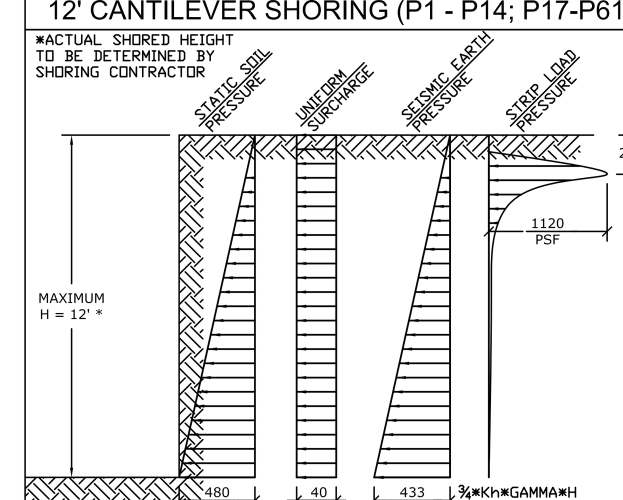
EARTH PRESSURE DISTRIBUTION AS NOTED TO THE LEFT

MAX LAGGING PRESSURE	= 280 PSF
UNIFORM SURCHARGE (Ka*100PSF)	= 40 PSF
PASSIVE BEARING PRESSURE	= 250 PCF
ALLOW. SKIN FRICTION	= 650 PSF
UNBONDED TIEBACK LENGTH	= 15'
BONDED TIEBACK LENGTH*	= MIN 39'
GROUTED BOND STRENGTH	= 5 KIP/FT

* ACTUAL BOND LENGTH SHALL BE CALCULATED BY THE SHORING SUBCONTRACTOR TO MEET THE TEST LOAD AND ELONGATION REQUIREMENTS.

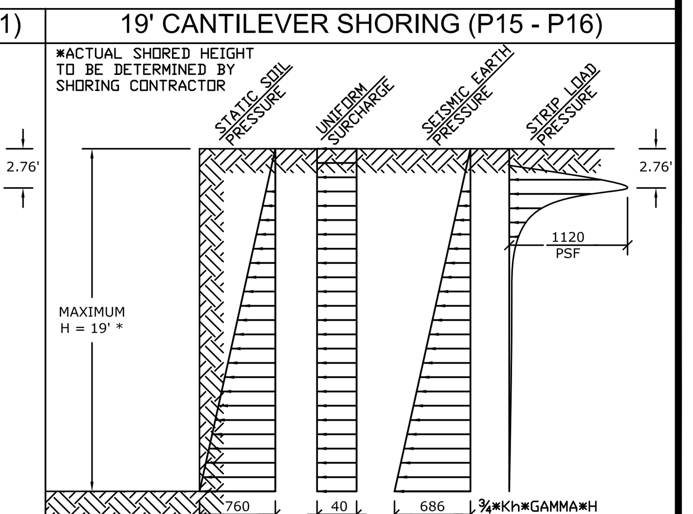
12' CANTILEVER SHORING (P1 - P14; P17-P61)

*ACTUAL SHORED HEIGHT TO BE DETERMINED BY SHORING CONTRACTOR



19' CANTILEVER SHORING (P15 - P16)

*ACTUAL SHORED HEIGHT TO BE DETERMINED BY SHORING CONTRACTOR



NOTES

1. ALL DIMENSIONS ARE APPROXIMATE AND TO BE FIELD VERIFIED BY THE CONTRACTOR. SEE CIVIL DWGS FOR EXACT ELEVATIONS AND LAYOUT.
2. NOTIFY SHORING ENGINEER OF RECORD OF ANY DISCREPANCIES BETWEEN THE PLANS AND THE FIELD CONDITIONS.
3. CONTRACTOR TO TAKE NECESSARY MEASURE TO MAINTAIN STRUCTURAL STABILITY BEFORE ANY DEMOLITION.

CONTACT 811 BEFORE YOU DIG
811
SAFE DIGGING PARTNER

Foundation Engineering Consultants, Inc.
3984 Washington Blvd
#236
Fremont, CA 94538
phone: (510) 371-5019
email: la@foundeng.com
www.foundeng.com

a/e stamp

client

agency approval

project info

drawing name
SHORING DETAILS

issue	date	
CITY SUBMIT		project no. 1718
		drawn by JWF
		checked by LA
		date March 12, 2020

drawing no.
SH2

Foundation Engineering Consultants, Inc.
3984 Washington Blvd., #236, Fremont, CA 94538
TEL 510-371-5019 http://www.FoundEng.com FAX 510-619-7230

SHORING DESIGN CALCULATIONS

PROPOSED TEMPORAY SHORING

Burlingame, CA 94010

PREPARED FOR:

FOUNDATION ENGINEERING CONSULTANTS, Inc.
Job # 1718
March 12th, 2020

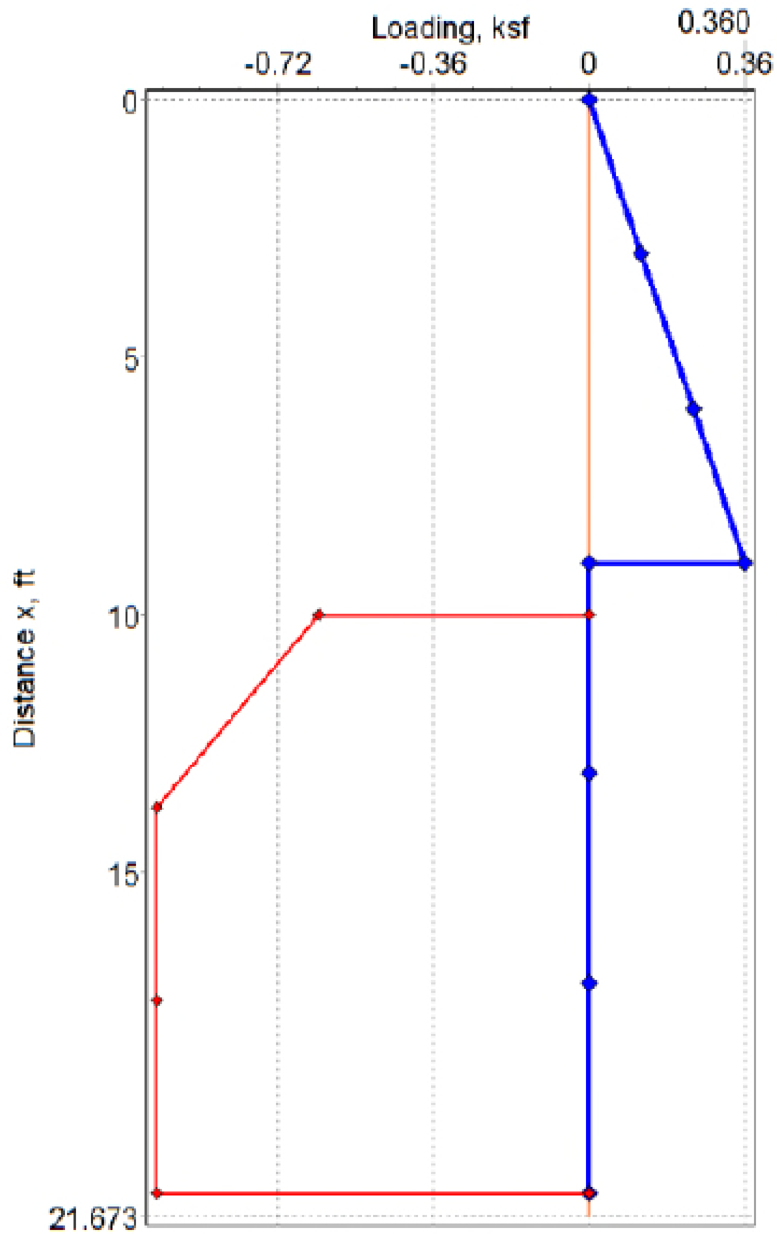
CANTILEVER SHORING – LOADING DIAGRAM:

DESIGN PARAMETERS

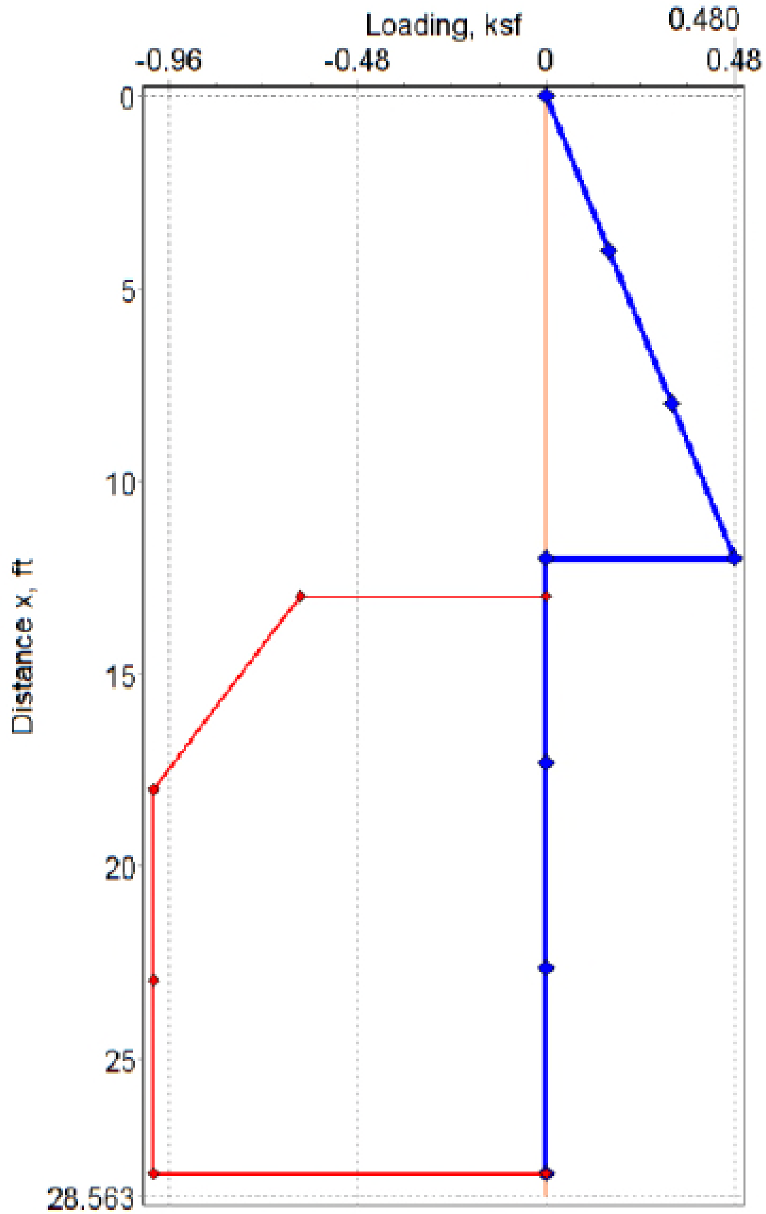
REFERENCE:
BASED ON GEOTECHNICAL INVESTIGATION BY ACE QUALITY CONTROL, DATED 1/9/19, FILE #10-16013GF.

BACKFILL PRESSURE:	40 PCF
SOIL UNIT WEIGHT:	125 PCF
ALLOWABLE PASSIVE RESIST:	250 PCF (2.5 PASSIVE WEDGE; IGNORE UPPER 1')
MAXIMUM PASSIVE RESIST:	1000 PSF

Cantilever Shoring (9 ft Cut [7 ft Cantilever + 2 ft Overex]- Tieback/Raker Shoring prior to Anchor Stressing)



Cantilever Shoring (12 ft Cut)



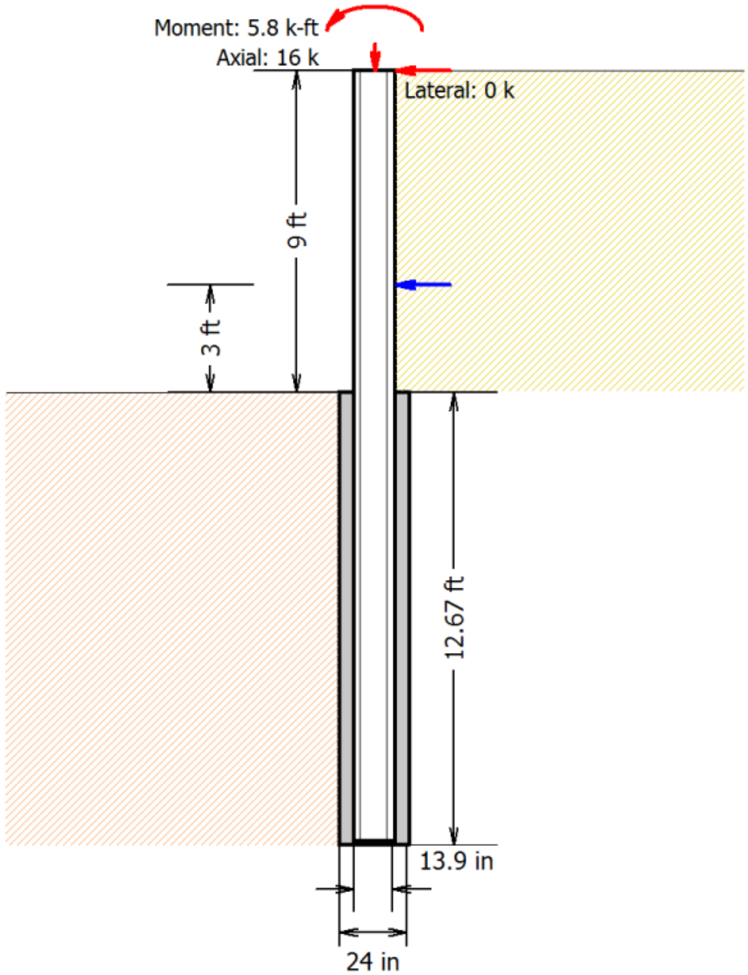
TIEBACK & RAKER SHORING –LOADING TABLE:

	DEPTH [ft]	STATIC PRESSURE [plf]	SEISMIC [plf]	SURCHARGE [plf]	TRAFFIC [plf]	STRIP LOAD [plf]	TOTAL LOADING [Lbf/Span]
0*H	0	0	0	280	420	0	700
	4.2	3511	1061	280	420	7000	11852
	16.7	3511	4219	280	420	0	8431
1.1*H	20.9	0	5281	280	420	0	5981

Cantilever Shoring Analysis

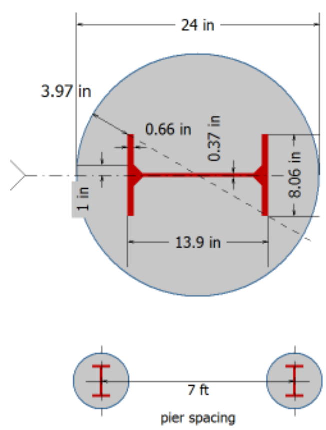
Cantilever Shoring (9 ft Cut [7 ft Cantilever + 2 ft Overex]

Organization:	**Foundation Engineering Cons.**
Project Name:	**150 Park Ave, Lot F, Burlingame**
Design by:	**Liiban Affi, P.E.**
Job #:	**1718**
Date:	**3/10/2020**



Backfill Soils
Equiv. Backfill Pressure: 40.0 psf/ft
Backfill Slope Angle: 0.0 degrees
Backfill Soil Unit Weight: 125.0 pcf
Vertical Uniform Surcharge: 0.00 psf
Seismic Load: 1949 lb/ft @ 3 ft depth

Passive Soils (Allowable)
Equiv. Passive Resistance: 250.0 psf/ft
Max. Passive Resistance: 1000 psf/ft
Passive Slope Angle: 0.0 degrees
F.S. on Passive: 1.00
Ignore Passive Height: 1.00 ft
Passive Soil Unit Weight: 125.0 pcf



Inputs

General Data	
Units	English
Analysis Method	Net Pressure
Installation Method	Drilled
Pile Type	Soldier Beam (King Pile)
Reinforcement	I-Beam
Shored Height, H	9.00 ft
Pile Spacing, S	7.00 ft
Pile Width or Pier Diameter, B	2.00 ft

Backfill Soils	
Equiv. Backfill Pressure	40.0 psf/ft
Backfill Slope Angle	0.0 degrees
Backfill Soil Unit Weight	125.0 pcf
Vertical Uniform Surcharge	0.00 psf

Passive Soils	
Allowable Passive	
Equiv. Passive Resistance	250.0 psf/ft
Max. Passive Resistance	1000 psf/ft
Passive Slope Angle	0.0 degrees
Passive Soil Unit Weight	125.0 pcf
Cohesion	0 psf
Ignore Passive Height	1.00 ft
Passive Wedge Multiplier	2.50

Structural data	
I-Beam	
Beam Type	North American
Beam Size	W14X53
Beam Diagonal Length (<24 in - 4 in, O.K.)	16.07 in
Pipe Filled with Concrete	No
Elastic Pile Modulus	29000 ksi
Yield Strength, Fy	50 ksi
Allow. Top of Pile Defl.	0.60 in
Conc. Compress. Str, f'c	4.00 ksi

Loads Applied to the Pile	
Strip Load	**Yes**
Strip Pressure, q	0 psf
Strip Setback, a	0.00 ft
Strip Width, b	0.00 ft
Strip Depth, d	0.00 ft
Mom. due to Strip, Mstrip	0.00 k-ft/ft
Seismic Load	**Yes**
Seismic Thrust, Pseis	1949 lb/ft
Loc. from Base of Excav, d	3.00 ft
Mom. due to Seismic, Mseis	5.85 k-ft/ft
Loads per Pile	
Axial Load	16.00 k
Lateral Load @ Top	0.00 k
Other Moment	0.00 k-ft
Total Moment	5.85 k-ft

Results

Pressure Data (ASD) per ft Length of Wall

	Loading Side		Passive Side	
	x, ft	w, ksf	x, ft	w, ksf
1	0.00	0.000	10.00	-0.625
2	3.00	0.120	13.74	-1.000
3	6.00	0.240	17.49	-1.000
4	9.00	0.360	21.22	-1.000
5	9.00	0.000		
6	13.07	0.000		
7	17.15	0.000		
8	21.22	0.000		

Output Data			
Total Beam Length			21.67 ft
Unbraced Length, Lr			22.21 ft
Unbraced Length, Lp			6.78 ft
Max. Shear	-28 k	@	15.48 ft
Max. Moment Above Grade	39.9 k-ft	@	9.00 ft
Max. Moment Below Grade	56.7 k-ft	@	11.44 ft
Max. Deflection	0.183 in	@	0.00 ft

Checks	Capacity	Utilization
Moment Above Grade	206 k-ft	
Moment Below Grade	217 k-ft	27 %
Shear	103 k	27 %
Axial	168 k	10 %
P-M interaction	32 %	
Slenderness Ratio, kL/r	118	

Guardrail Design

Guardrail Design	
Shape	L3-1/2X3-1/2X3/8
Length of the Guardrail	3.50 ft
Lateral Bracing	3.50 ft
Max. Spacing of Guardrail	7.00 ft
Dist. Load in Any Dir.	50 plf
Point Load in Any Dir.	200 lbf
ASD Safety Factor of Comp.	1.67
ASD Safety Factor of Bend.	1.67

Guardrail Design Results	Allowable	Applied
Compression in the Post	31.29	0.35 k
Yield Mom. Axis of Bending		5.175 k-ft
Elastic lat-tors. Buckling Mom.		5.552 k-ft
Bending moment in the post	3.099	1.225 k-ft
Welding Stress, y-axis	31.49	9.46 ksi
Welding Stress, x-axis	30.31	15.66 ksi

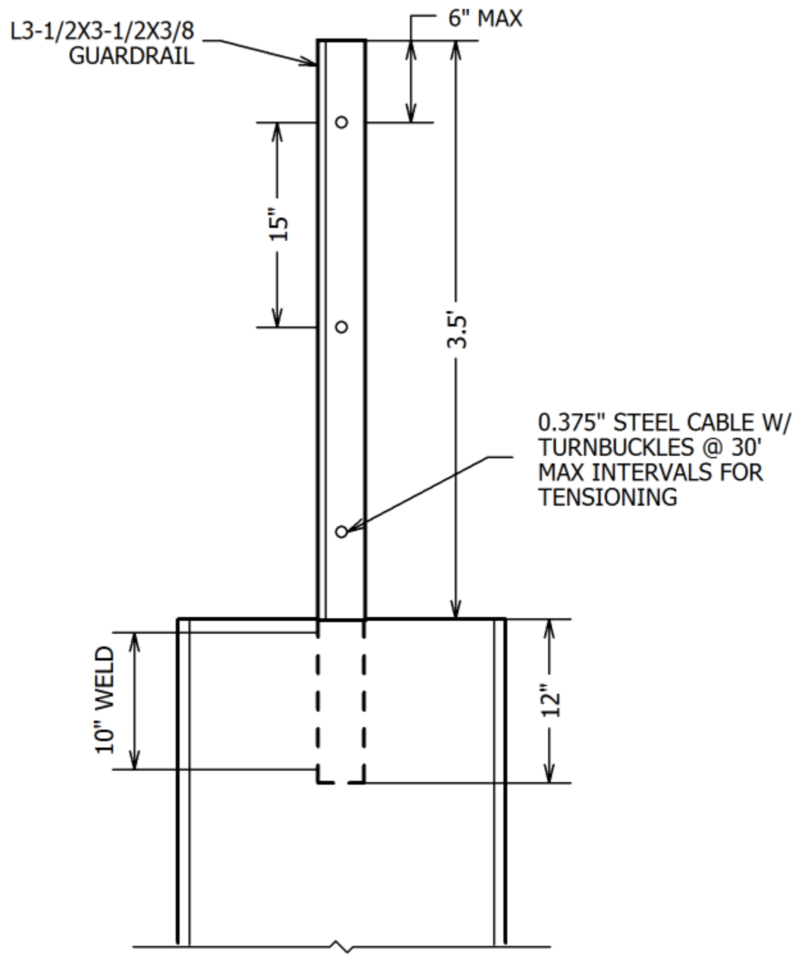
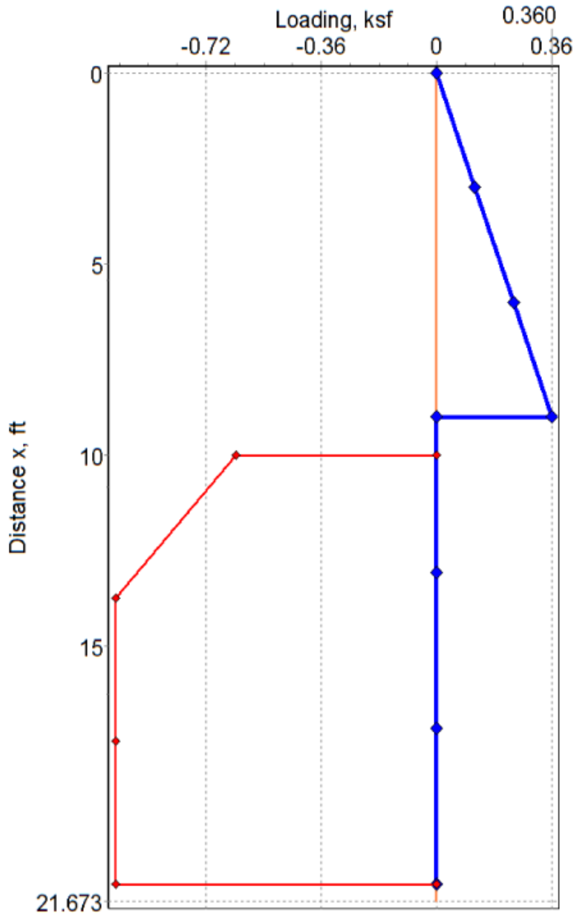


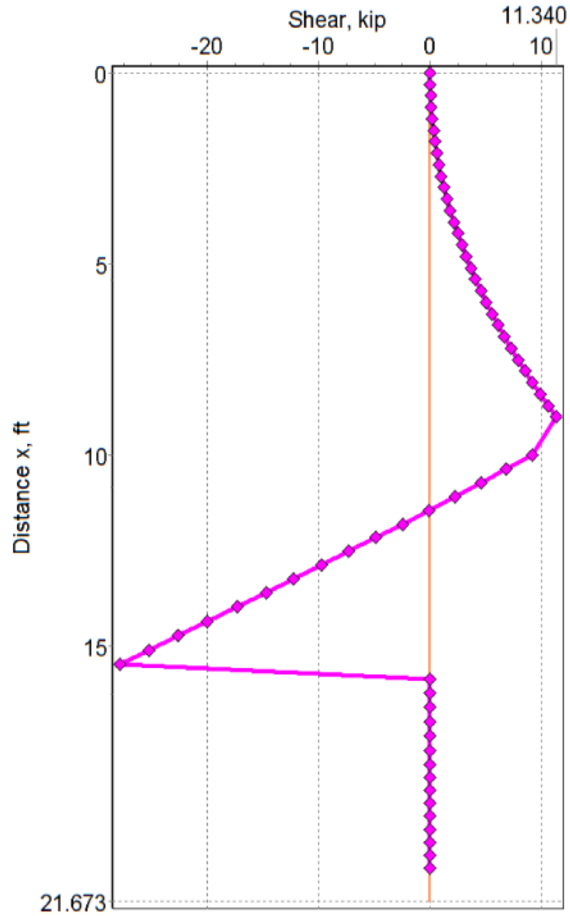
Table of Test Results

Node #	Depth, ft	Shear, k	Moment, k-ft	Slope, rad.	Defl, inch
0	0.00	0.00	5.85	0.00187	-0.183
1	0.30	0.01	5.85	0.00185	-0.176
2	0.60	0.05	5.86	0.00184	-0.170
3	0.90	0.11	5.88	0.00182	-0.163
4	1.20	0.20	5.93	0.00180	-0.157
5	1.50	0.32	6.01	0.00179	-0.150
6	1.80	0.45	6.12	0.00177	-0.144
7	2.10	0.62	6.28	0.00175	-0.137
8	2.40	0.81	6.50	0.00174	-0.131
9	2.70	1.02	6.77	0.00172	-0.125
10	3.00	1.26	7.11	0.00170	-0.119
11	3.30	1.52	7.53	0.00168	-0.113
12	3.60	1.81	8.03	0.00166	-0.107
13	3.90	2.13	8.62	0.00163	-0.101
14	4.20	2.47	9.31	0.00161	-0.095
15	4.50	2.84	10.11	0.00158	-0.089
16	4.80	3.23	11.02	0.00155	-0.084
17	5.10	3.64	12.05	0.00152	-0.078
18	5.40	4.08	13.21	0.00149	-0.073
19	5.70	4.55	14.50	0.00145	-0.067
20	6.00	5.04	15.94	0.00141	-0.062
21	6.30	5.56	17.53	0.00136	-0.057
22	6.60	6.10	19.28	0.00131	-0.052
23	6.90	6.67	21.19	0.00125	-0.048
24	7.20	7.26	23.28	0.00119	-0.043
25	7.50	7.88	25.55	0.00113	-0.039
26	7.80	8.52	28.01	0.00105	-0.035
27	8.10	9.19	30.66	0.00097	-0.032
28	8.40	9.88	33.52	0.00088	-0.028
29	8.70	10.60	36.60	0.00079	-0.025
30	9.00	11.34	39.89	0.00068	-0.023
31	9.00	11.34	39.89	0.00068	-0.023
32	10.00	9.15	50.13	0.00057	-0.015
33	10.00	9.15	50.13	0.00057	-0.015
34	10.36	6.88	53.02	0.00052	-0.013
35	10.72	4.58	55.08	0.00048	-0.011
36	11.08	2.25	56.31	0.00043	-0.009
37	11.44	-0.10	56.70	0.00038	-0.007
38	11.80	-2.47	56.23	0.00033	-0.005
39	12.16	-4.87	54.91	0.00028	-0.004
40	12.52	-7.29	52.72	0.00023	-0.003
41	12.88	-9.74	49.66	0.00019	-0.002
42	13.24	-12.22	45.70	0.00014	-0.001
43	13.60	-14.71	40.86	0.00011	-0.001
44	13.98	-17.33	34.85	0.00007	0.000
45	14.35	-19.96	27.85	0.00004	0.000
46	14.73	-22.58	19.88	0.00002	0.000
47	15.10	-25.21	10.91	0.00001	0.000
48	15.48	-27.83	0.97	0.00000	0.000
49	15.85	0.00	0.00	0.00000	0.000
50	16.23	0.00	0.00	0.00000	0.000
51	16.60	0.00	0.00	0.00000	0.000
52	16.98	0.00	0.00	0.00000	0.000
53	17.35	0.00	0.00	0.00000	0.000
54	17.73	0.00	0.00	0.00000	0.000
55	18.10	0.00	0.00	0.00000	0.000
56	18.44	0.00	0.00	0.00000	0.000
57	18.78	0.00	0.00	0.00000	0.000
58	19.11	0.00	0.00	0.00000	0.000
59	19.45	0.00	0.00	0.00000	0.000
60	19.79	0.00	0.00	0.00000	0.000
61	20.13	0.00	0.00	0.00000	0.000
62	20.46	0.00	0.00	0.00000	0.000
63	20.80	0.00	0.00	0.00000	0.000

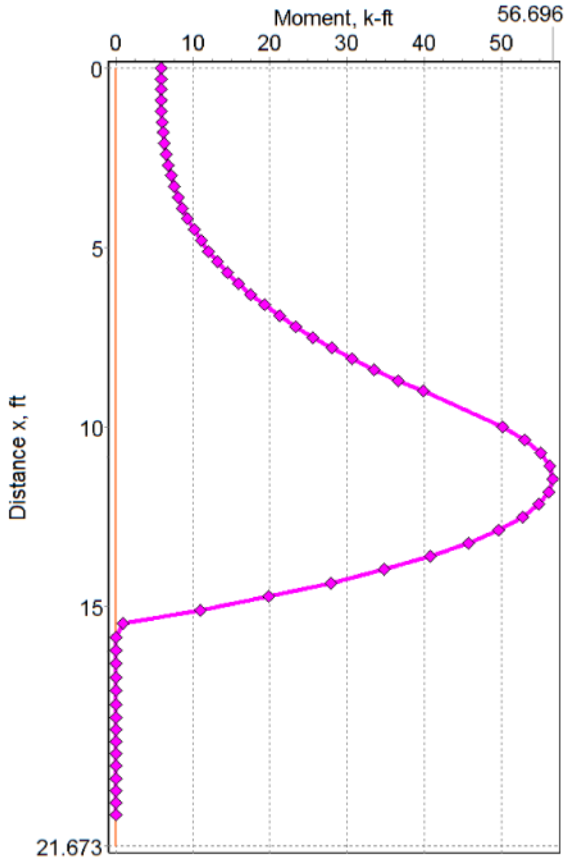
Charts



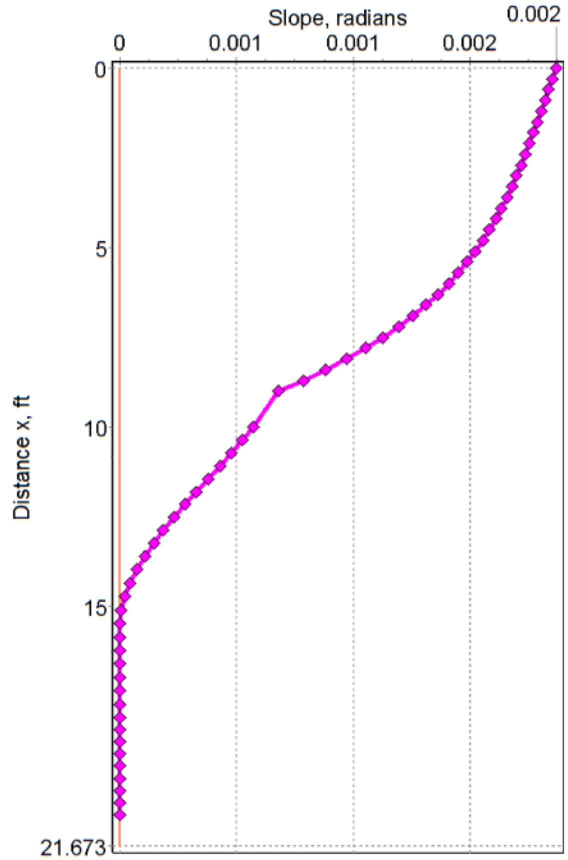
Loading, ksf
0.360
-0.72
-0.36
0
0.36
Distance x, ft
0
5
10
15
21.673



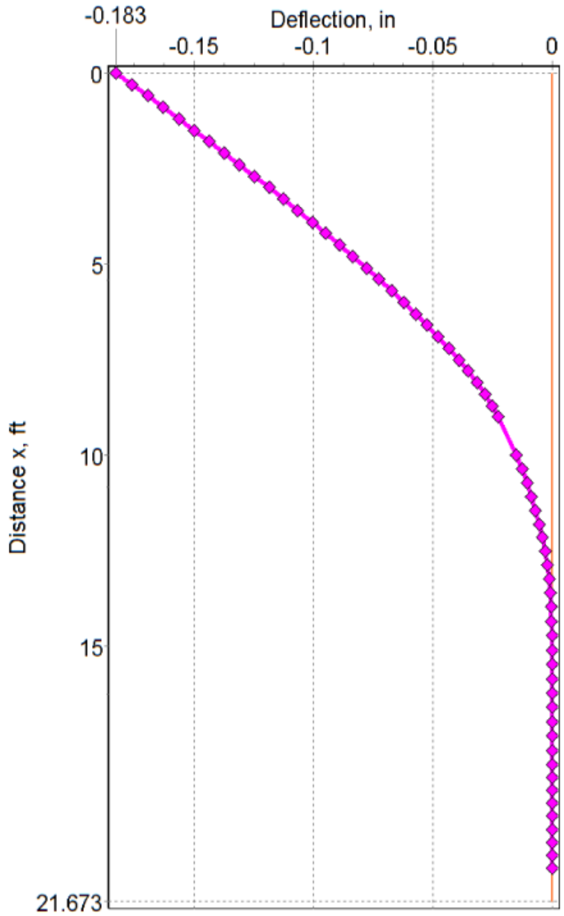
Shear, kip
11.340
-20
-10
0
10
Distance x, ft
0
5
10
15
21.673



Moment, k-ft
56.696
0
10
20
30
40
50
Distance x, ft
0
5
10
15
21.673



Slope, radians
0.002
0
0.001
0.001
0.002
Distance x, ft
0
5
10
15
21.673



References:

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3. Foundation Engineering, A.R. Jumikis, 2nd Ed., 1987
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7. AISC Steel Construction Manual, 14th Ed., 2010
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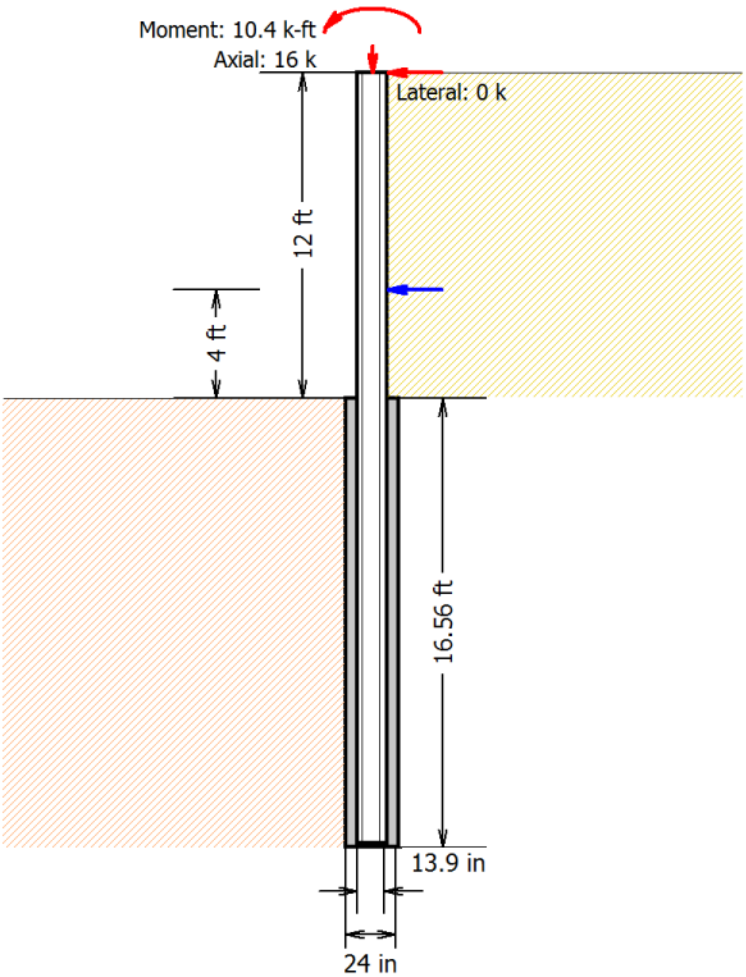
9. Guide to Design of Slabs on Ground - ACI 360R-10, American Concrete Institute, 2010
10. Practical Design of Sheet Pile Bulkheads, Arbed, 1991
11. Lateral Pressure on Sheet Pile Walls due to Strip Load, Georgiadis & Anagnostospoulos, ASCE, 1998
12. SoilStructure Software: Cantilever Shoring v1.3.2

Cantilever Shoring Analysis

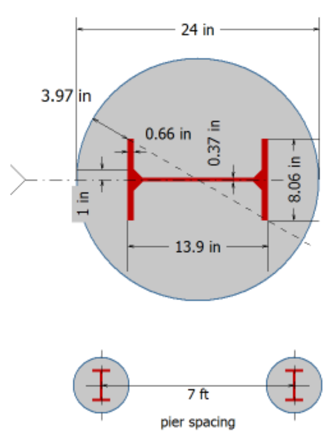
Cantilever Shoring (12 ft Cut)

Organization: **Foundation Engineering Cons.**
Project Name: **150 Park Ave, Lot F, Burlingame**
Design by: **Liiban Affi, P.E.**
Job #: **1718**
Date: **3/10/2020**



Backfill Soils
Equiv. Backfill Pressure: 40.0 psf/ft
Backfill Slope Angle: 0.0 degrees
Backfill Soil Unit Weight: 125.0 pcf
Vertical Uniform Surcharge: 0.00 psf
Seismic Load: 2598 lb/ft @ 4 ft depth

Passive Soils (Allowable)
Equiv. Passive Resistance: 250.0 psf/ft
Max. Passive Resistance: 1000 psf/ft
Passive Slope Angle: 0.0 degrees
F.S. on Passive: 1.00
Ignore Passive Height: 1.00 ft
Passive Soil Unit Weight: 125.0 pcf



Inputs

General Data	
Units	English
Analysis Method	Net Pressure
Installation Method	Drilled
Pile Type	Soldier Beam (King Pile)
Reinforcement	I-Beam
Shored Height, H	12.00 ft
Pile Spacing, S	7.00 ft
Pile Width or Pier Diameter, B	2.00 ft

Backfill Soils	
Equiv. Backfill Pressure	40.0 psf/ft
Backfill Slope Angle	0.0 degrees
Backfill Soil Unit Weight	125.0 pcf
Vertical Uniform Surcharge	0.00 psf

Passive Soils	
Allowable Passive	
Equiv. Passive Resistance	250.0 psf/ft
Max. Passive Resistance	1000 psf/ft
Passive Slope Angle	0.0 degrees
Passive Soil Unit Weight	125.0 pcf
Cohesion	0 psf
Ignore Passive Height	1.00 ft
Passive Wedge Multiplier	2.50

Structural data	
I-Beam	
Beam Type	North American
Beam Size	W14X53
Beam Diagonal Length (<24 in - 4 in, O.K.)	16.07 in
Pipe Filled with Concrete	No
Elastic Pile Modulus	29000 ksi
Yield Strength, Fy	50 ksi
Allow. Top of Pile Defl.	0.90 in
Conc. Compress. Str, f'c	4.00 ksi

Loads Applied to the Pile	
Strip Load	**Yes**
Strip Pressure, q	0 psf
Strip Setback, a	0.00 ft
Strip Width, b	0.00 ft
Strip Depth, d	0.00 ft
Mom. due to Strip, Mstrip	0.00 k-ft/ft
Seismic Load	**Yes**
Seismic Thrust, Pseis	2598 lb/ft
Loc. from Base of Excav, d	4.00 ft
Mom. due to Seismic, Mseis	10.39 k-ft/ft
Loads per Pile	
Axial Load	16.00 k
Lateral Load @ Top	0.00 k
Other Moment	0.00 k-ft
Total Moment	10.39 k-ft

Results

Pressure Data (ASD) per ft Length of Wall

	Loading Side		Passive Side	
	x, ft	w, ksf	x, ft	w, ksf
1	0.00	0.000	13.00	-0.625
2	4.00	0.160	17.98	-1.000
3	8.00	0.320	22.98	-1.000
4	12.00	0.480	27.96	-1.000
5	12.00	0.000		
6	17.32	0.000		
7	22.65	0.000		
8	27.96	0.000		

Output Data			
Total Beam Length			28.56 ft
Unbraced Length, Lr			22.21 ft
Unbraced Length, Lp			6.78 ft
Max. Shear	-42 k	@	21.80 ft
Max. Moment Above Grade	91.1 k-ft	@	12.00 ft
Max. Moment Below Grade	135.3 k-ft	@	15.88 ft
Max. Deflection	0.828 in	@	0.00 ft

Checks	Capacity	Utilization
Moment Above Grade	190 k-ft	
Moment Below Grade	217 k-ft	68 %
Shear	103 k	41 %
Axial	95 k	17 %
P-M interaction	77 %	
Slenderness Ratio, kL/r	158	

Guardrail Design

Guardrail Design	
Shape	L3-1/2X3-1/2X3/8
Length of the Guardrail	3.50 ft
Lateral Bracing	3.50 ft
Max. Spacing of Guardrail	7.00 ft
Dist. Load in Any Dir.	50 plf
Point Load in Any Dir.	200 lbf
ASD Safety Factor of Comp.	1.67
ASD Safety Factor of Bend.	1.67

Guardrail Design Results	Allowable	Applied
Compression in the Post	31.29	0.35 k
Yield Mom. Axis of Bending		5.175 k-ft
Elastic lat-tors. Buckling Mom.		5.552 k-ft
Bending moment in the post	3.099	1.225 k-ft
Welding Stress, y-axis	31.49	9.46 ksi
Welding Stress, x-axis	30.31	15.66 ksi

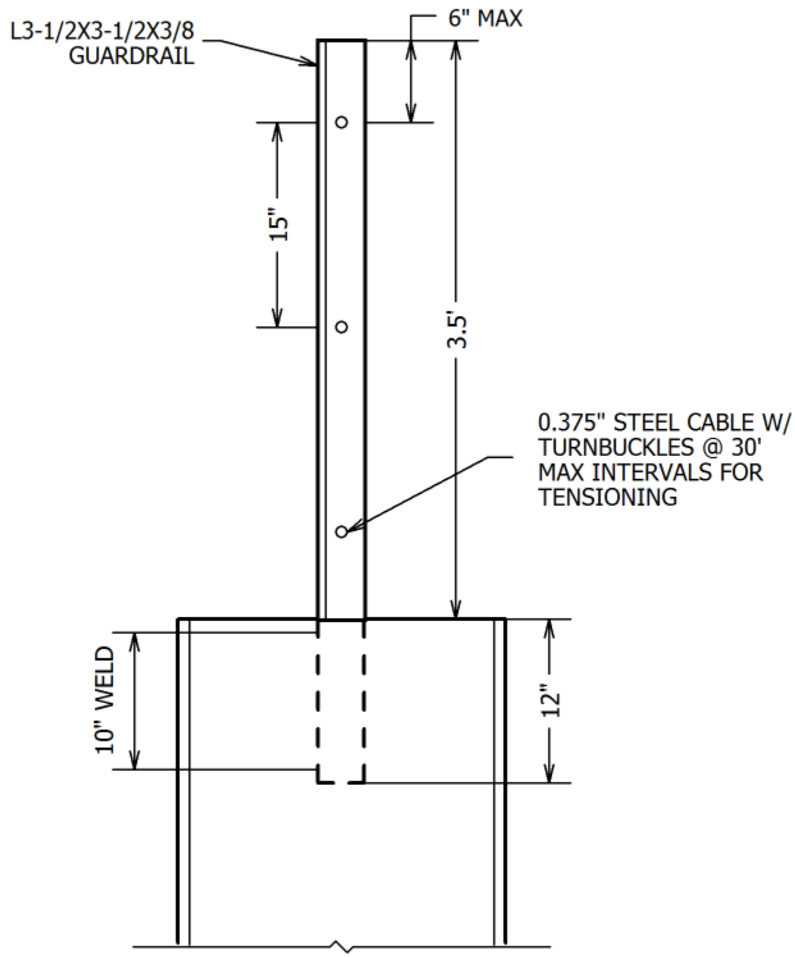
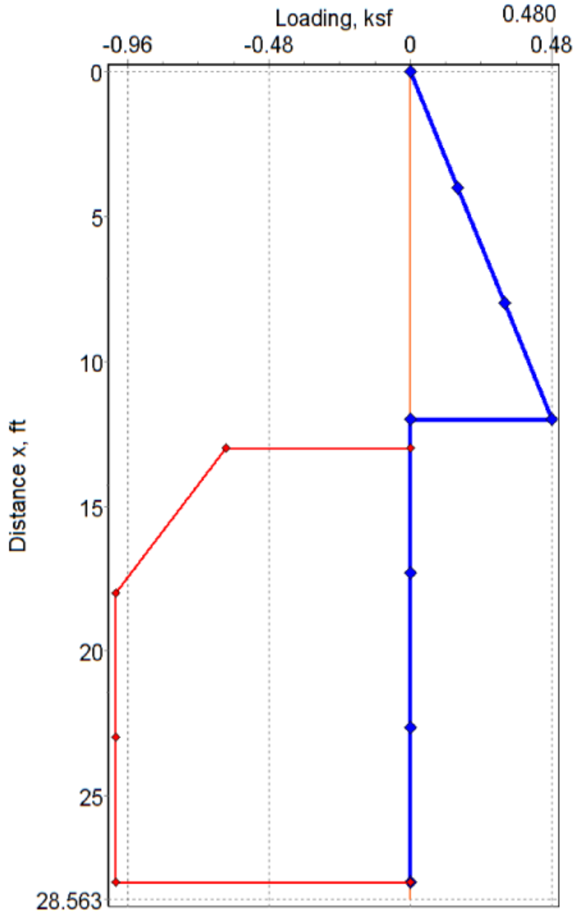
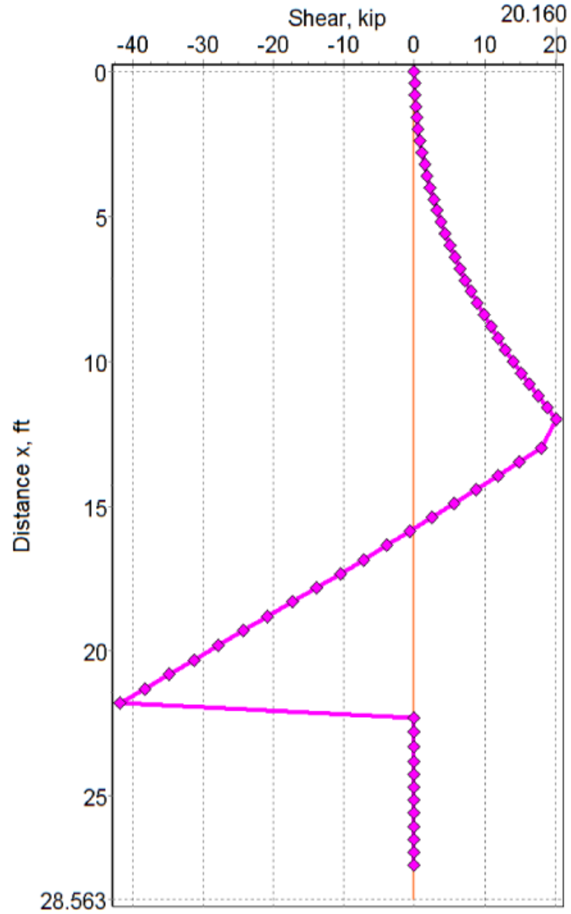


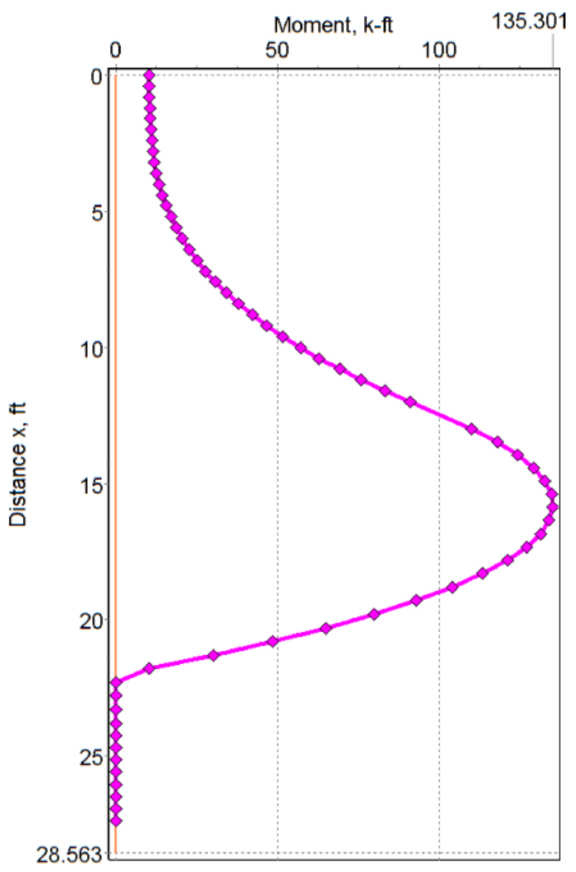
Table of Test Results

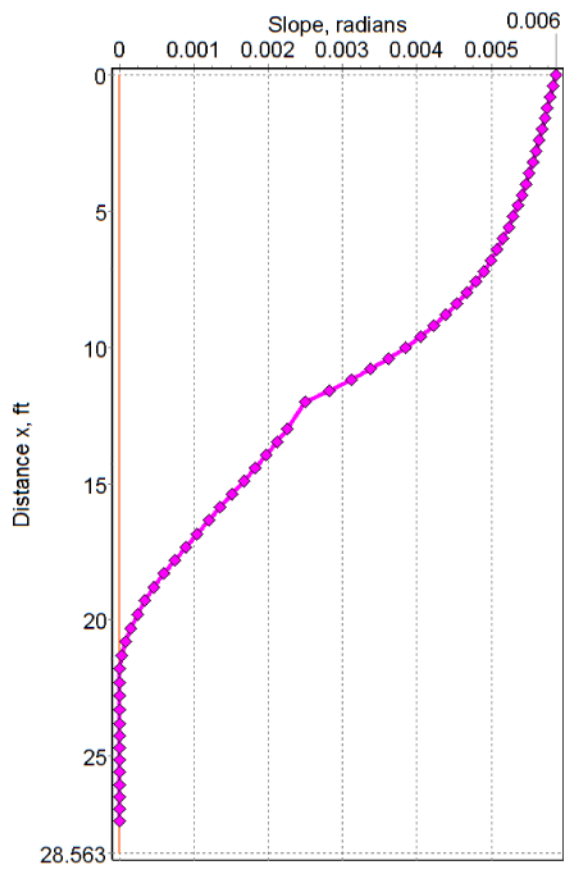
Node #	Depth, ft	Shear, k	Moment, k-ft	Slope, rad.	Defl, inch
0	0.00	0.00	10.39	0.00587	-0.828
1	0.40	0.02	10.40	0.00583	-0.800
2	0.80	0.09	10.42	0.00579	-0.772
3	1.20	0.20	10.48	0.00576	-0.744
4	1.60	0.36	10.59	0.00572	-0.717
5	2.00	0.56	10.77	0.00568	-0.689
6	2.40	0.81	11.05	0.00564	-0.662
7	2.80	1.10	11.43	0.00560	-0.635
8	3.20	1.43	11.93	0.00555	-0.608
9	3.60	1.81	12.58	0.00551	-0.582
10	4.00	2.24	13.39	0.00546	-0.555
11	4.40	2.71	14.38	0.00541	-0.529
12	4.80	3.23	15.57	0.00535	-0.503
13	5.20	3.79	16.97	0.00530	-0.478
14	5.60	4.39	18.61	0.00523	-0.453
15	6.00	5.04	20.49	0.00516	-0.428
16	6.40	5.73	22.65	0.00508	-0.403
17	6.80	6.47	25.09	0.00499	-0.379
18	7.20	7.26	27.84	0.00489	-0.355
19	7.60	8.09	30.91	0.00479	-0.332
20	8.00	8.96	34.32	0.00467	-0.309
21	8.40	9.88	38.08	0.00453	-0.287
22	8.80	10.84	42.23	0.00439	-0.266
23	9.20	11.85	46.77	0.00422	-0.245
24	9.60	12.90	51.72	0.00404	-0.225
25	10.00	14.00	57.10	0.00384	-0.206
26	10.40	15.14	62.92	0.00362	-0.189
27	10.80	16.33	69.22	0.00338	-0.172
28	11.20	17.56	76.00	0.00311	-0.156
29	11.60	18.84	83.28	0.00282	-0.142
30	12.00	20.16	91.08	0.00250	-0.129
31	12.00	20.16	91.08	0.00250	-0.129
32	13.00	17.97	110.14	0.00225	-0.101
33	13.00	17.97	110.14	0.00225	-0.101
34	13.48	14.94	118.04	0.00212	-0.088
35	13.96	11.88	124.48	0.00198	-0.076
36	14.44	8.79	129.44	0.00183	-0.065
37	14.92	5.66	132.91	0.00167	-0.055
38	15.40	2.50	134.87	0.00151	-0.046
39	15.88	-0.70	135.30	0.00136	-0.038
40	16.36	-3.93	134.19	0.00120	-0.030
41	16.84	-7.19	131.52	0.00104	-0.024
42	17.32	-10.49	127.28	0.00089	-0.018
43	17.80	-13.82	121.45	0.00074	-0.014
44	18.30	-17.31	113.66	0.00060	-0.010
45	18.80	-20.81	104.13	0.00046	-0.006
46	19.30	-24.31	92.85	0.00034	-0.004
47	19.80	-27.81	79.82	0.00024	-0.002
48	20.30	-31.31	65.04	0.00015	-0.001
49	20.80	-34.81	48.51	0.00008	0.000
50	21.30	-38.31	30.23	0.00003	0.000
51	21.80	-41.81	10.20	0.00001	0.000
52	22.30	0.00	0.00	0.00000	0.000
53	22.80	0.00	0.00	0.00000	0.000
54	23.30	0.00	0.00	0.00000	0.000
55	23.80	0.00	0.00	0.00000	0.000
56	24.25	0.00	0.00	0.00000	0.000
57	24.70	0.00	0.00	0.00000	0.000
58	25.15	0.00	0.00	0.00000	0.000
59	25.60	0.00	0.00	0.00000	0.000
60	26.05	0.00	0.00	0.00000	0.000
61	26.50	0.00	0.00	0.00000	0.000
62	26.95	0.00	0.00	0.00000	0.000
63	27.40	0.00	0.00	0.00000	0.000

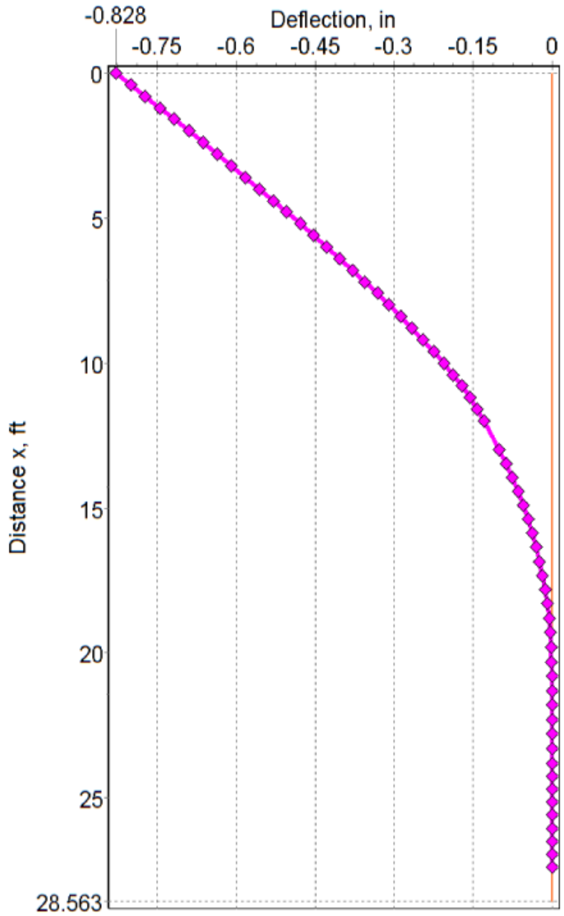
Charts


Loading, ksf
0.480
-0.96
-0.48
0
0.48
Distance x, ft
0
5
10
15
20
25
28.563


Shear, kip
20.160
-40
-30
-20
-10
0
10
20
Distance x, ft
0
5
10
15
20
25
28.563


Moment, k-ft
135.301
0
50
100
Distance x, ft
0
5
10
15
20
25
28.563


Slope, radians
0.006
0
0.001
0.002
0.003
0.004
0.005
Distance x, ft
0
5
10
15
20
25
28.563

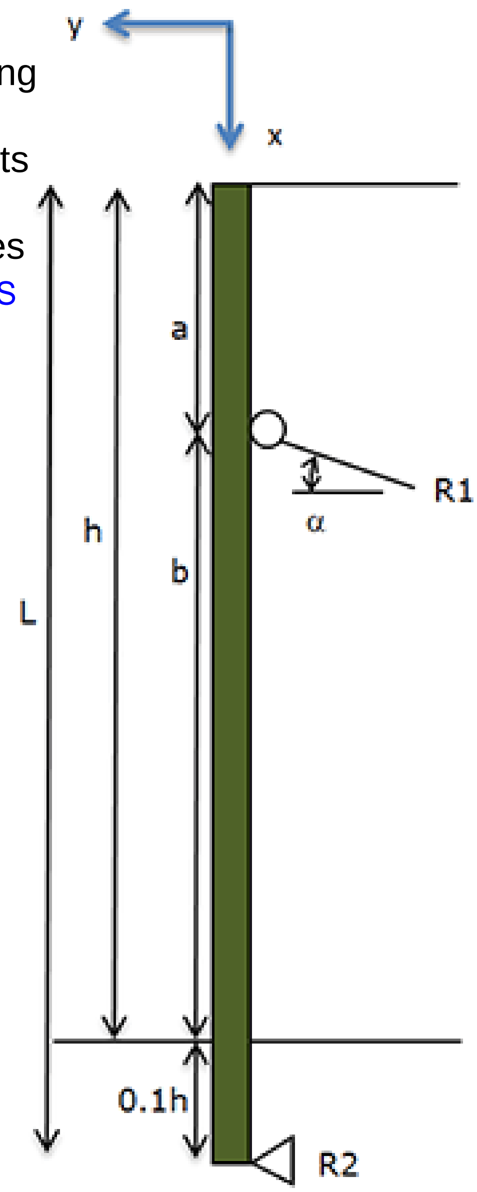


References:

1. EM 1110-2-2502, Retaining Walls, Corps of Engineers, 1961
2. Foundation Design, W.C. Teng, 1962
3. Foundation Engineering, A.R. Jumikis, 2nd Ed., 1987
4. Foundation Analysis & Design, J. E. Bowles, 5th Ed., 1996
5. 2015 IBC, International Code Council
6. Recommendations on Excavations, DGGT, 3rd Ed., 2014
7. AISC Steel Construction Manual, 14th Ed., 2010
8. Hot Rolled & Structural Steel Products - 7th Ed, OneSteel Manufacturing, Australia, 2014
9. Guide to Design of Slabs on Ground - ACI 360R-10, American Concrete Institute, 2010
10. Practical Design of Sheet Pile Bulkheads, Arbed, 1991
11. Lateral Pressure on Sheet Pile Walls due to Strip Load, Georgiadis & Anagnostospoulos, ASCE, 1998
12. SoilStructure Software: Cantilever Shoring v1.3.2

TBWall Report

Tieback Calcs



Project Information

Designed By	Liiban Affi, P.E.
Organization	Foundation Engineering
Date	3/10/2020
Project	The Village Apartments
Job #	1718
Client	The Pacific Companies

RAKER SHORING CALCS

Number of Tieback Levels **One**

Units System **ft**

Geometry

a	7.0 ft
b	12.0 ft
h	19.0 ft
L	20.9 ft

Properties

E	29000 ksi
fy	50 ksi

Max. Deflection **0.5 in**

Beam Shape **W14X53**

Tieback Data

Angle1	20

Design Philosophy

The analysis is based on "Equivalent Beam Method" first proposed by Blum and explained in detail in "Foundation Design" Teng, 1962, 1st & only edition or in "Foundation Engineering" Jumikis, 1987 2nd ed.

The design is based on classical structural analysis:

* This program uses classic-beam-theory beam elements to solve the multispan tieback design.

* The equivalent nodal loads for each span are determined by numerical integration of the beam equations to allow for the non uniform loads.

* The equivalent nodal loads, the stiffness matrix, and the support conditions are used to solve for the support reactions and the support rotations.

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* The deflection output is based on structural analysis but an independent check should be made by Finite Element method or by site surveying.

Reaction 1	Reaction 2
-137.79 kips	-45.57 kips

Maximum Shear	-79.3 kip at 7.00 ft
Maximum Moment	158.2 kip-at 7.00 ft
Maximum Deflection	-0.2785 in at 14.51 ft

Required Aw	3.97 in2	Adequate for Shear
Required Zx	63.41 in3	Adequate for Bending
Utilized Ix	56%	Adequate for Deflection

	R1
Tieback Force	146.6 kips
Unbonded Tieback Length	15.0 ft
Test Load	195.0 kips

Lateral Torsional Buckling Check		Axially-Loaded Member Check	
Lb	144 in	P	14 kips
Cb	1	L	12 ft
ry	1.92 in	K	0.8
Iy	57.70 in4	A	15.6 in2
h0	13.24 in	KL/r	60.0
J	1.94 in4	Fe	80 ksi
rts	2.2 in	Fcr	38 ksi
Lp	81.4 in	Pn/Q	359 kips
Lr	266.5 in		
Fcr	86 ksi		
Mn/Q	190 kip-ft		

Required Embedment	11.59 ft
Tschebotarioff Check	10.07 ft

Combined Forces Utilization 85%

Organization	Foundation Engineering	**Client**	The Pacific Companies
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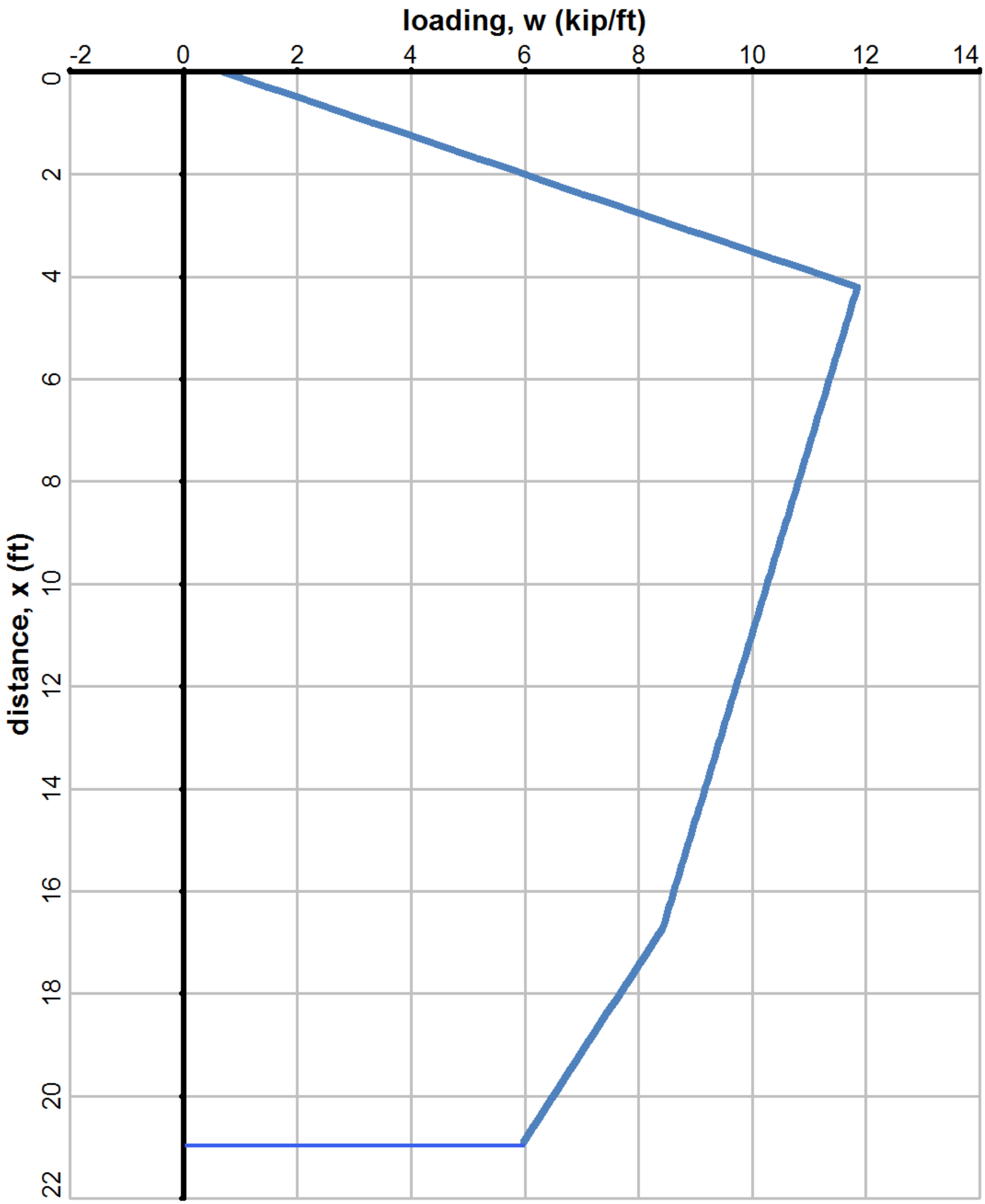

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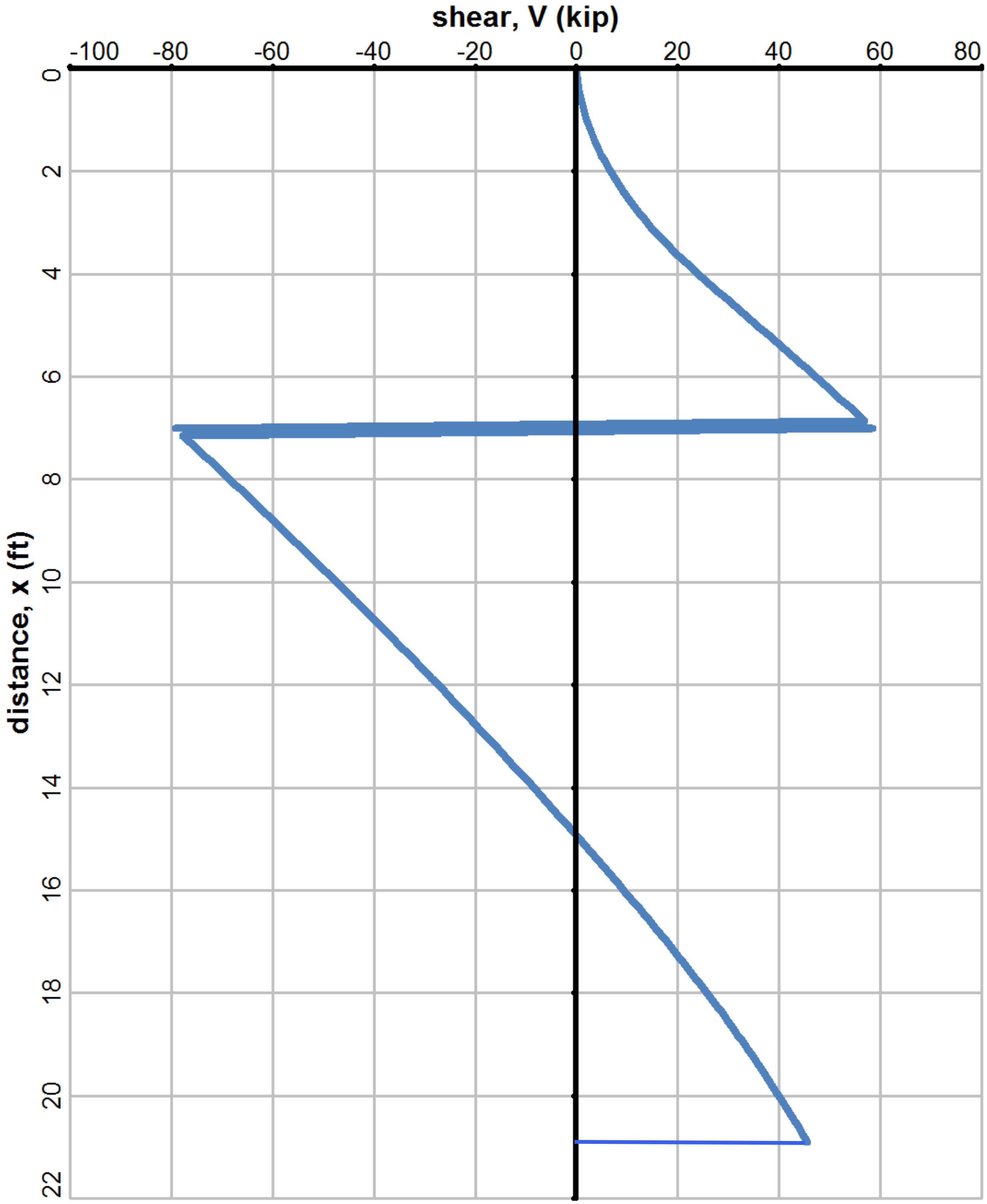
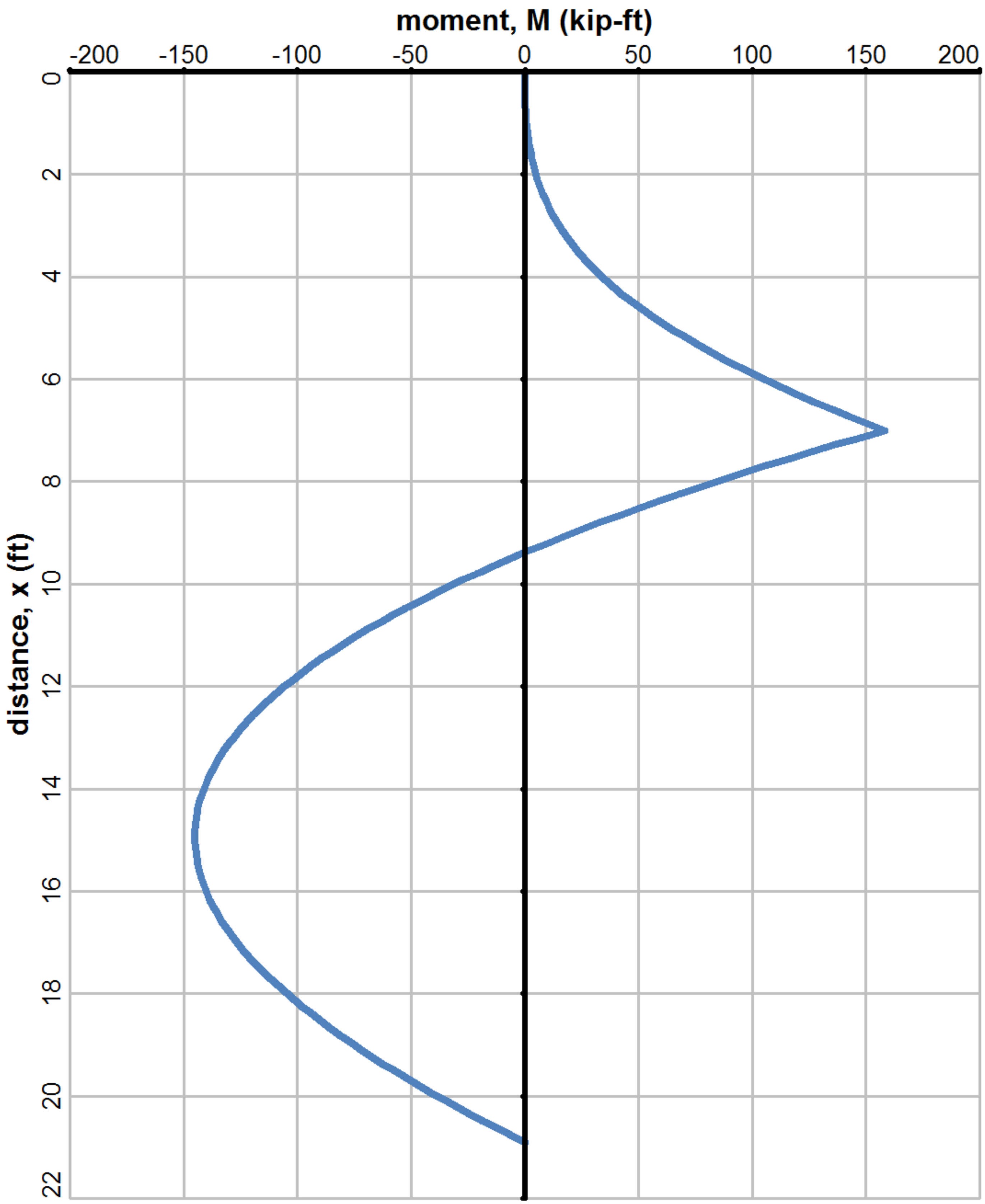
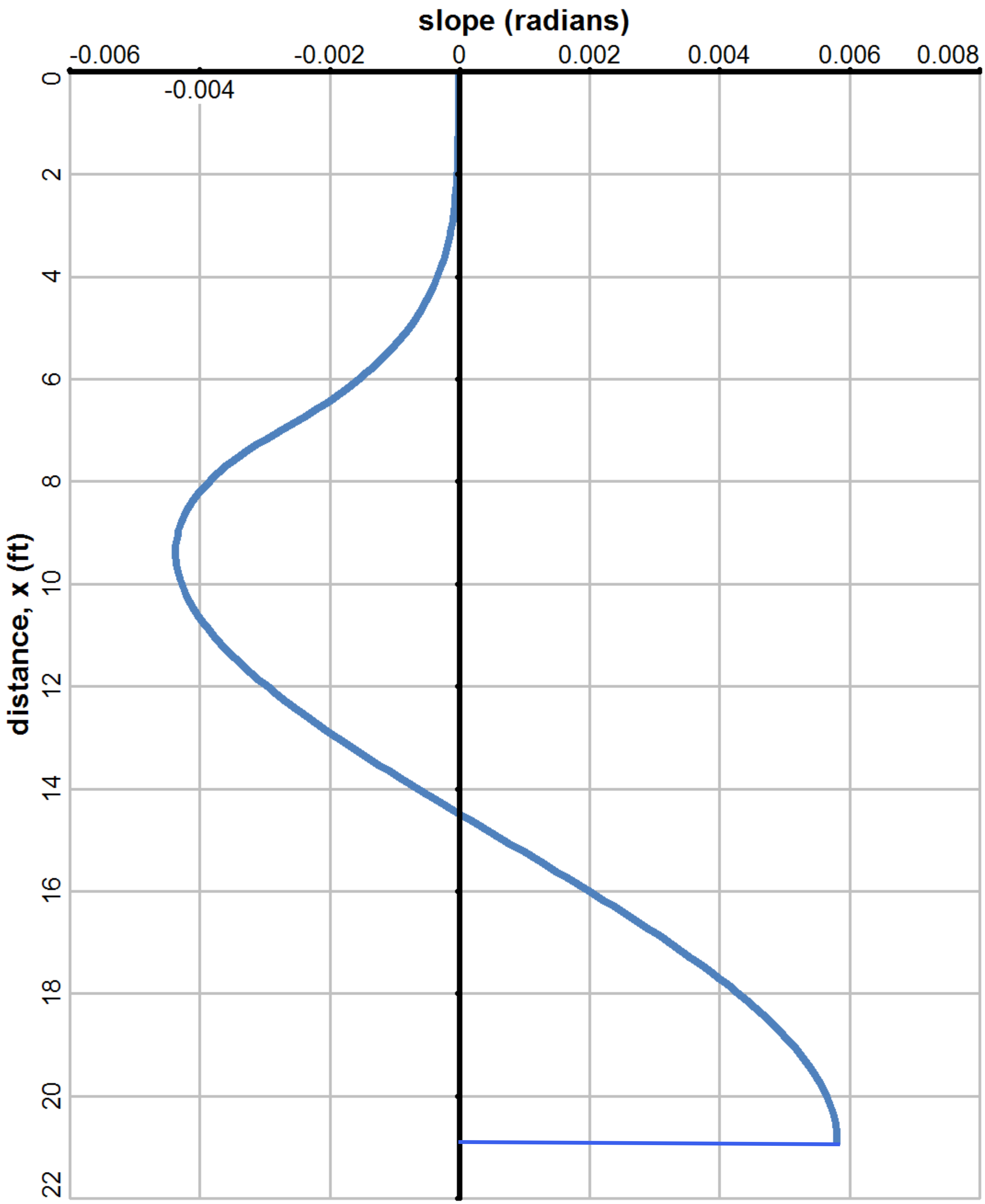
Designed By	Liiban Affi, P.E.	**Project**	The Village Apartments
Date	3/10/2020	**Job #**	1718





moment, M (kip-ft)
-200
-150
-100
-50
0
50
100
150
200
distance, x (ft)
0
2
4
6
8
10
12
14
16
18
20
22

Organization	Foundation Engineering	**Client**	The Pacific Companies
Designed By	Liiban Affi, P.E.	**Project**	The Village Apartments
Date	3/10/2020	**Job #**	1718



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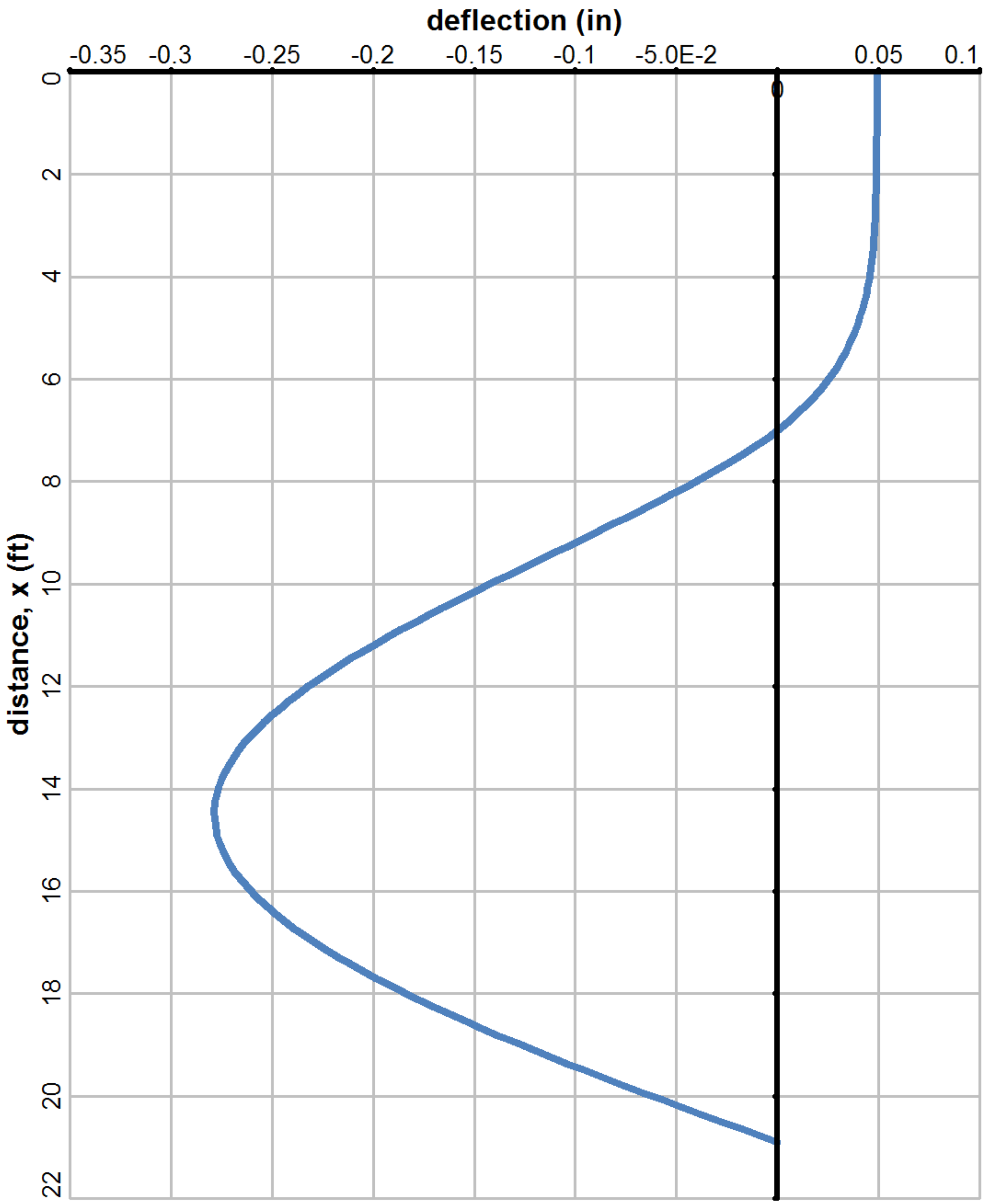
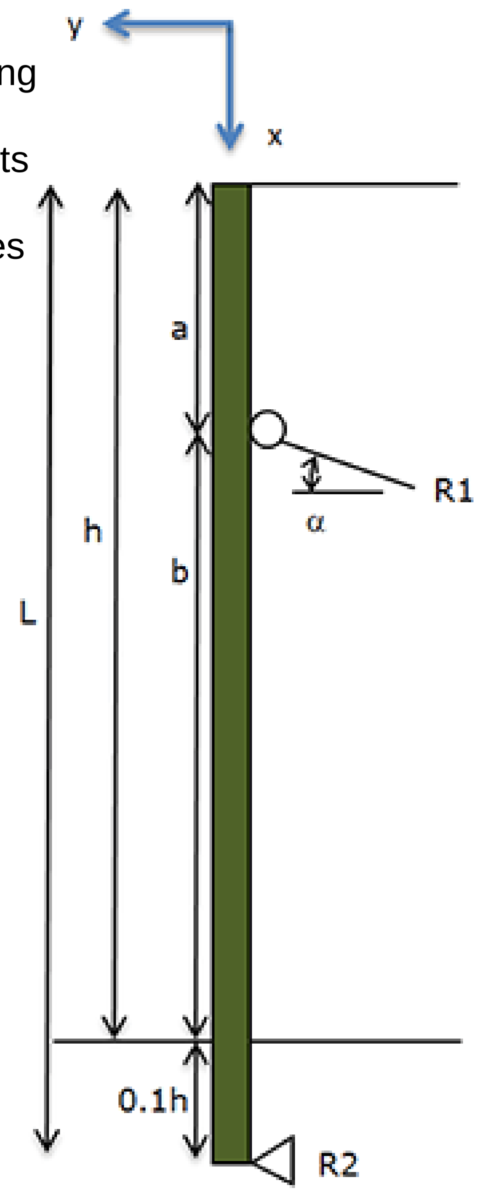
Designed By	Liiban Affi, P.E.	**Project**	The Village Apartments
Date	3/10/2020	**Job #**	1718



TBWall Report

RAKER SHORING CALCS



Project Information

Designed By	Liiban Affi, P.E.
Organization	Foundation Engineering
Date	3/10/2020
Project	The Village Apartments
Job #	1718
Client	The Pacific Companies

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Units System **ft**

Geometry

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b	12.0 ft
h	19.0 ft
L	20.9 ft

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fy	50 ksi

Max. Deflection **0.5 in**

Beam Shape **W14X53**

Tieback Data

Angle1	34

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Utilized Ix	56%	Adequate for Deflection

	R1
Tieback Force	166.2 kips
Unbonded Tieback Length	15.0 ft
Test Load	221.1 kips

Lateral Torsional Buckling Check		Axially-Loaded Member Check	
Lb	144 in	P	14 kips
Cb	1	L	12 ft
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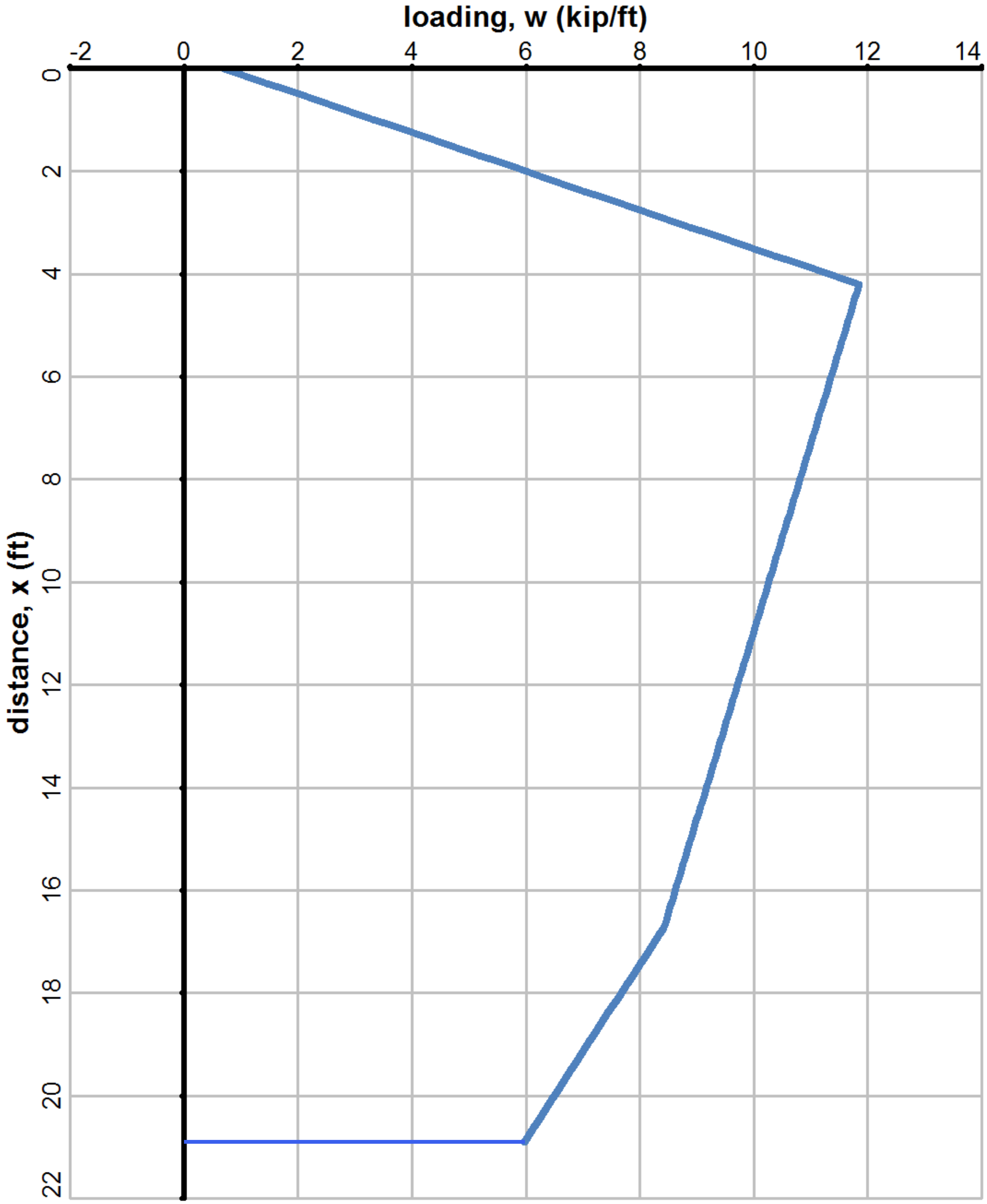
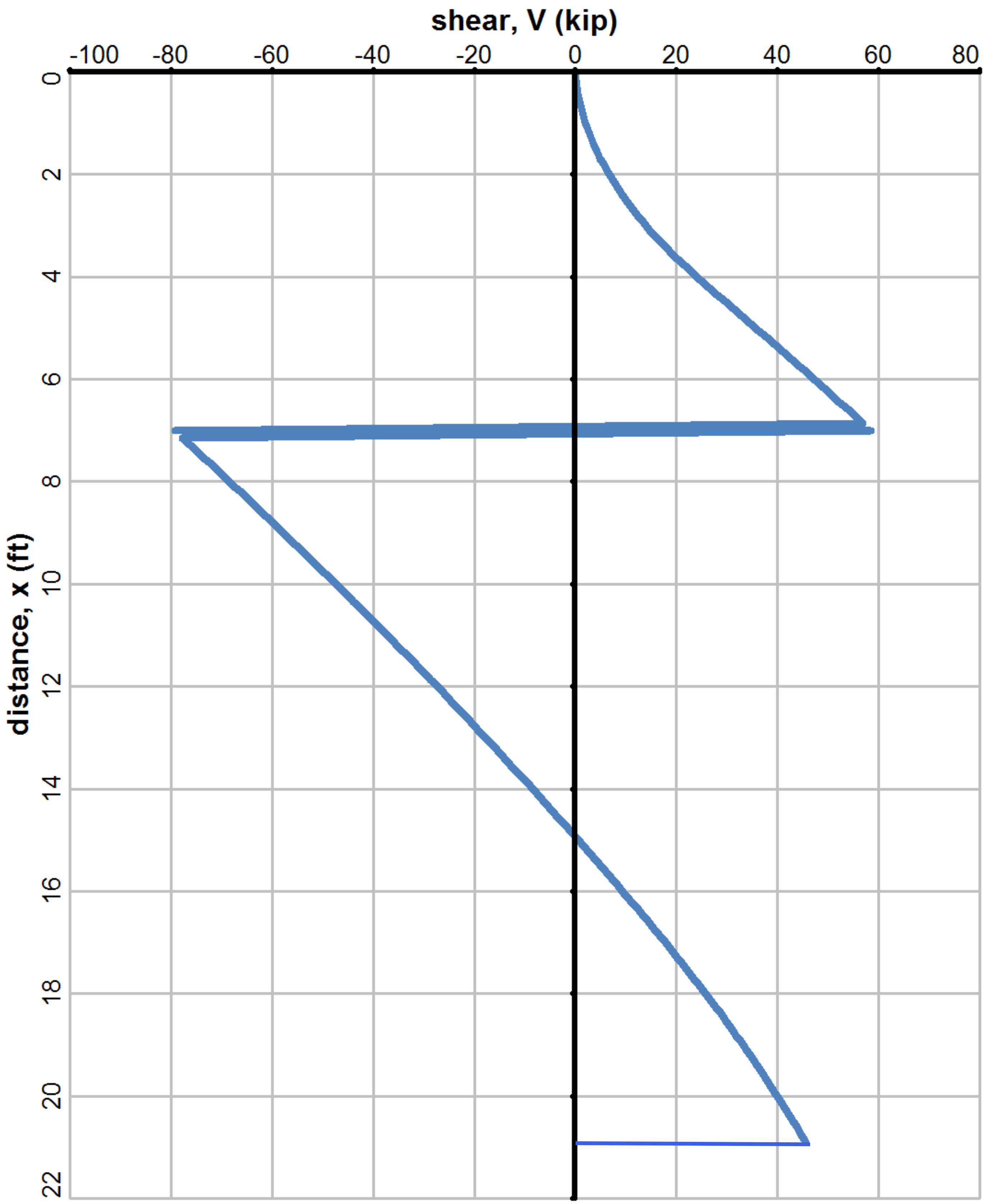
Iy	57.70 in4	A	15.6 in2
h0	13.24 in	KL/r	60.0
J	1.94 in4	Fe	80 ksi
rts	2.2 in	Fcr	38 ksi
Lp	81.4 in	Pn/Q	359 kips
Lr	266.5 in		
Fcr	86 ksi		
Mn/Q	190 kip-ft		

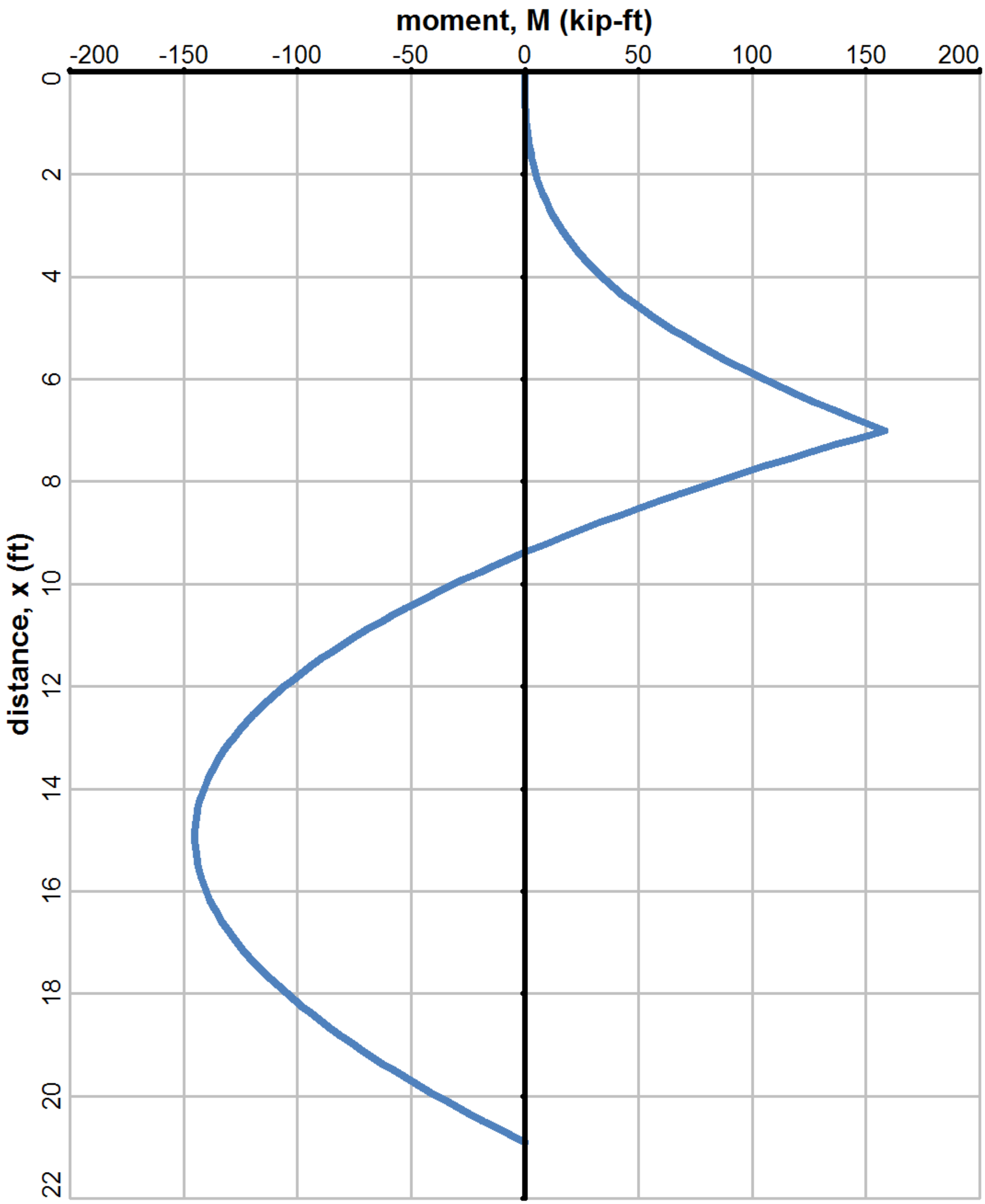
Required Embedment	11.59 ft
Tschebotarioff Check	10.07 ft
Combined Forces Utilization	85%


loading, w (kip/ft)
-2
0
2
4
6
8
10
12
14
distance, x (ft)
0
2
4
6
8
10
12
14
16
18
20
22



shear, V (kip)
-100
-80
-60
-40
-20
0
20
40
60
80
distance, x (ft)
0
2
4
6
8
10
12
14
16
18
20
22

Organization	Foundation Engineering	**Client**	The Pacific Companies
Designed By	Liiban Affi, P.E.	**Project**	The Village Apartments
Date	3/10/2020	**Job #**	1718



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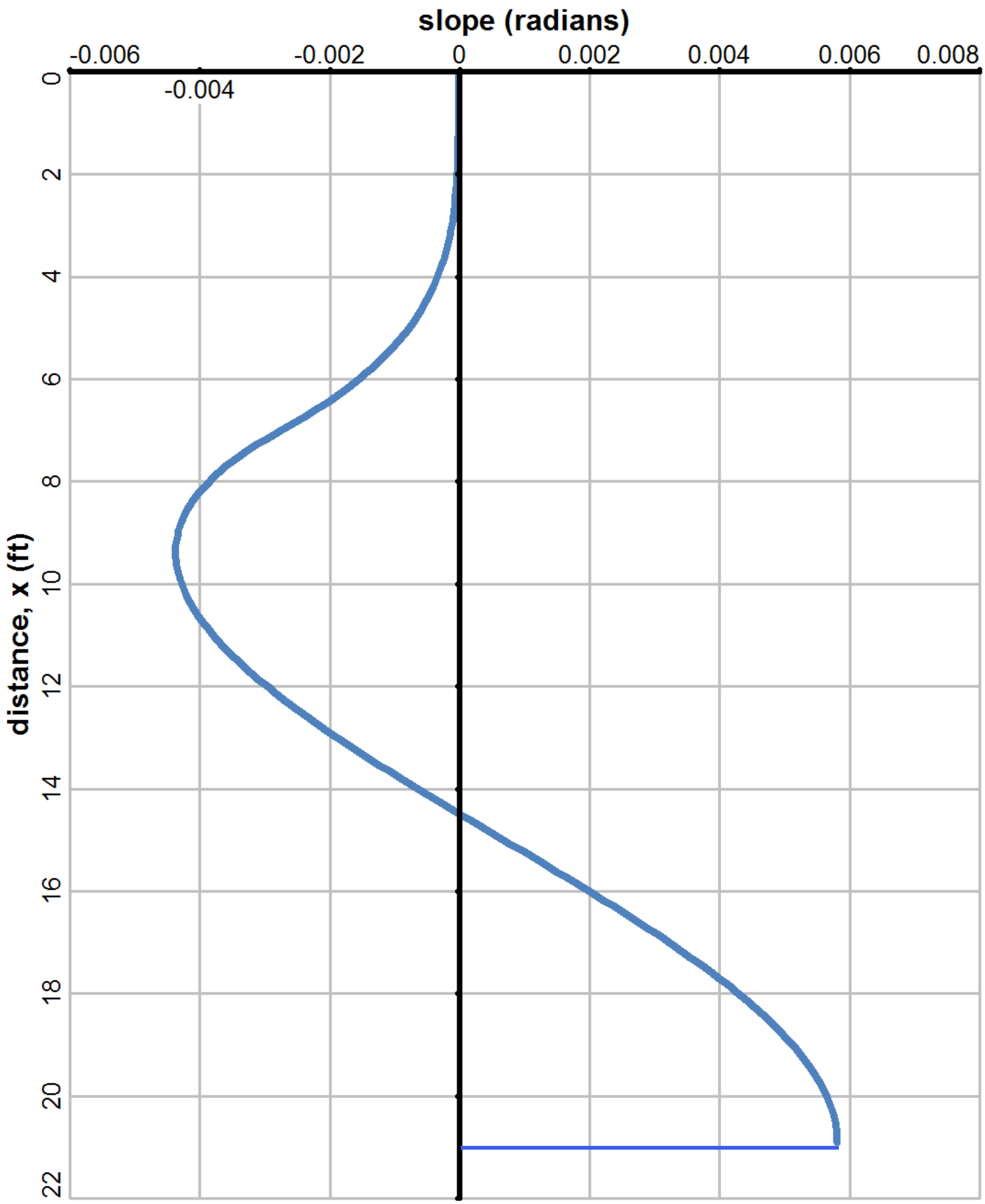
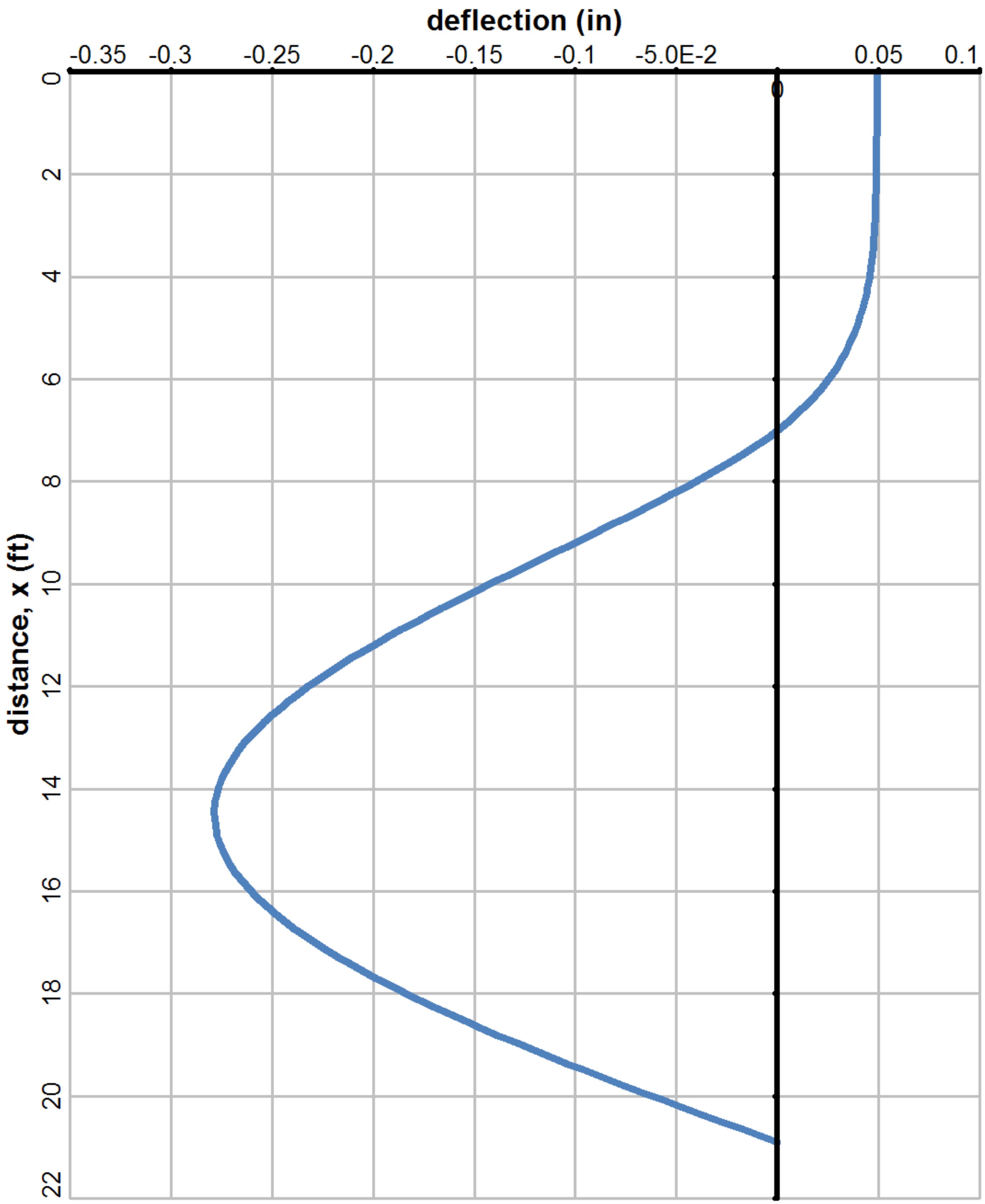
Date	3/10/2020	**Job #**	1718




deflection (in)
-0.35
-0.3
-0.25
-0.2
-0.15
-0.1
-5.0E-2
0
0.05
0.1
distance, x (ft)
0
2
4
6
8
10
12
14
16
18
20
22

▶ Useful Functions

AXIAL CAPACITY OF ROUND HOT ROLLED SECTION

Shape := "Pipe8XS" Enter shape name (Enter fractions of inches following the example: 3-1/2 for 3.5")

Member Configuration

$L := 23.5 \cdot ft$ Length of the member

$L_x := L$ $K_x := 1$ Strong axis laterally unbraced length and effective length factor

$L_y := L$ $K_y := 1$ Weak axis laterally unbraced length and effective length factor

▶ Material and Section Properties Lookup

▶ Allowable Tension, Compression, Bending, Shear, Torsion, and Deflection Calculations: Round HSS & Pipe

Compression (E3, E7 & E7.2)

$\phi := 0.9$ Load factor for compression

Pipe Properties:

$OD = 8.625 \cdot in$

$A_g = 11.9 \cdot in^2$

$t_{nom} = 0.5 \cdot in$

$t_{des} = 0.465 \cdot in$

$r_x = 2.89 \cdot in$

$r_y = 2.89 \cdot in$

$F_y = 35 \cdot ksi$

$E = 29000 \cdot ksi$

$$KL/r := \max\left(\frac{K_x \cdot L_x}{r_x}, \frac{K_y \cdot L_y}{r_y}\right)$$

$KL/r = 97.6$ preferably should not exceed 200

$$Q_a := \text{if}\left[0.11 \cdot \frac{E}{F_y} < \frac{OD}{t_{des}} < 0.45 \cdot \frac{E}{F_y}, \frac{0.038 \cdot E}{F_y \cdot \left(\frac{OD}{t_{des}}\right)} + \frac{2}{3}, \text{if}\left(\frac{OD}{t_{des}} \le 0.11 \cdot \frac{E}{F_y}, 1, 0\right)\right]$$

$Q_a = 1$

$$F_e := \frac{\pi^2 \cdot E}{KL/r^2}$$

$F_e = 30.1 \cdot ksi$

$$F_{cr} := \begin{cases} Q \leftarrow Q_a \\ Q \cdot \left(0.658^{\frac{Q \cdot F_y}{F_e}}\right) \cdot F_y & \text{if } KL/r \le 4.71 \cdot \sqrt{\frac{E}{Q \cdot F_y}} \\ 0.877 \cdot F_e & \text{otherwise} \end{cases}$$

$F_{cr} = 21.5 \cdot ksi$

$C_n := F_{cr} \cdot A_g$

$C_a := \phi \cdot C_n$ $C_a = 230.26 \cdot kip$ **Design Axial Compression ≥ Required Strength = 221 kip**

Weld Length Required To Develop Pipe Strength

$F_{e70} := 70 \cdot ksi$ Electrode ultimate strength

$d_w := \frac{7}{16} \cdot in$ Leg length of the fillet weld

$\phi := 0.75$ Load factor on welds

$$L_{req} := C_a \cdot \frac{\sqrt{2}}{\phi \cdot 0.6 \cdot F_{e70} \cdot d_w}$$

$L_{req} = 23.629 \cdot in$ ≤ $\pi \cdot OD = 27.096 \cdot in$ **Because of the angle of the raker, the all-around weld will exceed the circumfrence of the round, so the comparison to the left is conservative.**