Kansas Citys, Missouri and Kansas
Flood Risk Management Feasibility Study
Engineering Appendix to the Final Feasibility Report

CHAPTER A-12

GENERAL STRUCTURAL ANALYSIS

ENGINEERING APPENDIX
CHAPTER A-12
GENERAL STRUCTURAL

A-12.1 Structural Analysis Methodology 1
A-12.1.1 Introduction 1
A-12.1.2 Deterministic Design Criteria 1
A-12.1.2.1 General Assumptions 2
A-12.1.2.2 Stability and Pile Capacity Requirements 2
A-12.1.2.3 Strength Requirements 3
A-12.1.3 Structural Reliability Methodology (existing structures only) 4
A-12.1.4 Deterministic Criteria (existing structures only) 4
A-12.1.5 Reliability Analysis (existing conditions only) 5
A-12.1.5.1 Risk Calculation 5
A-12.1.5.2 Structural Material Properties 6
A-12.1.5.3 Soil Material Properties 6
A-12.1.6 Structural Analysis 6
A-12.1.6.1 Floodwalls on Spread Footings 7
A-12.1.6.2 Floodwalls on Piles 7
A-12.1.6.3 Stoplog and Sandbag Closure Structures 8
A-12.1.6.4 Pump Stations 8
A-12.1.6.5 Gatewells, Reinforced Concrete Boxes, and Drainage Structures 8
A-12.2 Structural Considerations in Raise Alternatives 9
A-12.3 Example Calculations 10

Chapter A-12
General Structural

A-12.1 Structural Analysis Methodology

A-12.1.1 Introduction

The structural features of the levee units included in this study consist of floodwalls, pump stations, closure structures for openings in levees and floodwalls, gatewells, reinforced box culverts, drainage structures, and retaining walls integral to the integrity of the levee system. The evaluation of each unit's structures includes the assessment of existing conditions and formulation of several project alternatives. These alternatives include raising the level unit to meet the elevation of a Nominal 500 year flood (N500), a N500 plus three additional feet (N500+3'), and a N500 plus five additional feet (N500+5'). The findings then provide input to the HEC-FDA economics model used to develop benefit to cost (B/C) ratios and an economic assessment.

For existing conditions, the features were analyzed at various water levels to establish reliability as required by the HEC-FDA program model. This work was based on visual observation, dated construction plans, historical data, discussions with the Corps of Engineers and Levee District personnel (those familiar with and involved in the inspection, operation, and maintenance of the levee units), detailed engineering analysis, and engineering judgment.

Alternatives for creating flood control protection above the current level of protection included removal and replacement of floodwall with earthen levee, modification, and new construction. For modifications, similar work to that stated above for existing structures was required including assumptions and engineering judgment. Analysis of the N500, N500+3' and N500+5' focused on the N500+3' event and those findings were extrapolated to estimate requirements for the N500 and N500+5' where possible. The B/C ratios for future conditions were also used in the development of the economic curve.

Note, the work contained herein is to provide Feasibility Analysis results only; it does not replace a deterministic design analysis, or answer the questions that only a deterministic design analysis can. The structures were analyzed, without factors of safety and with consistent assumptions, in order to evaluate the relative risk and consequences for economic and risk-informed decision-making purposes. Risk and Reliability studies do not replace deterministic analyses, nor do such studies confirm the satisfaction of any design criteria, past or present. They simply provide additional information for the decision-maker with respect to the possible performance of the structure for the loads under consideration. This provides a risk-informed decision with respect to project repairs or improvements.

A-12.1.2 Deterministic Design Criteria

A series of screening criteria are used to determine if a probabilistic analysis is necessary for a given structure. Summarized below are some general assumptions used to analyze structural components as well as the strength and stability criterion from the

current design standards. If the analysis shows the existing structural component meets the below criteria, it is assumed reliable and a 99.8% reliability is assigned. If the structural component does meet the criteria, a reliability analysis is performed.

A-12.1.2.1 General Assumptions

The following lists major assumptions in the feasibility study. Other assumptions specific to a feature are noted in respective levee system appendix.

a. Some structural components were not analyzed. Only components judged to be critical based upon engineering experience were analyzed for feasibility.
b. The structural components were analyzed based on dimensions, quantities, and conditions represented by record drawings. Deviations from plans cannot be verified per scope and budget. This is a consistent assumption for relative risk and reliability assessment for Kansas City's levee feasibility studies.
c. Parts of the components being analyzed that were not evaluated with this analysis include, but may not be limited to, minimum rebar embedment lengths, structure capacity at rebar cutoff locations, axial tension in heels due to resisting pressures on the key, various cut-off wall efficiencies, and various soil resistances.
d. Secondary and incremental load effects are not considered for the feasibility analysis. Permanent deformation or damages from any less than extreme floods that occur prior to the extreme flood are expected to have been repaired to minimum USACE criteria.
e. Construction practices are considered good, and all specifications noted on plans satisfied.
f. Structural materials, such as reinforcing bar and concrete, are in good condition – without voids or other significant defect.
g. Soil is adequately compacted and fill type and strength parameters supplied by Geotechnical Section is correct.
h. Cut-off walls are 50% efficient, and are of adequate strength and good condition. The upstream face of structural wedges was analyzed with a Line of Creep, reduction beginning at top of ground. This is the default method in CTWALL. Toe drains are considered inoperable.

A-12.1.2.2 Stability and Pile Capacity Requirements

Pile capacity requirements are based on EM 1110-2-2906 *Design of Pile Foundations*. Structural stability criterion can be seen in Table A-12-1. It is based upon EM 1110-2-2100 *Stability Analysis of Concrete Structures* (01 Dec 2005), with the exception of the extreme load condition. The Missouri River L-142 Design Criteria Issue Resolution Paper (2002) addressed a concern with the extreme load condition categories as specified in EM 1110-2-2100 and put forth more stringent guidelines for recommended extreme load condition stability criteria. That criterion is used herein.

Recommended Sliding Stability Factor of Safety		
Load Condition Category	Return Period	Factor of Safety
Usual	10 yrs	2
Unusual	300 yrs	1.5
Extreme	Top of Protection	1.3*

Recommended Rotational Stability Percent of Base in Compression		
Load Condition Category	Return Period	Percent of Base in Compression
Usual	10 yrs	100%
Unusual	300 yrs	75%
Extreme	Top of Protection	25% *

Recommended Maximum Allowable Bearing Capacity % Increase in Allowable Bearing Capacity		
Load Condition Category	Return Period	% Increase in Allowable Bearing Capacity
Usual	10 yrs	0%
Unusual	300 yrs	15%
Extreme	Top of Protection	50%
Recommended Flotation Stability Factor of Safety		
Load Condition Category	Return Period	Factor of Safety
Usual	10 yrs	1.3
Unusual	300 yrs	1.2
Extreme	Top of Protection	1.1

* Stability requirements increased from value in EM 1110-2-2100.

Table A-12-1 Stability Criterion

A-12.1.2.3 Strength Requirements

a. Strength requirements are based on the Strength Design Method as outlined by EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures.*

b. Dead and live load factor (LF) of 1.7 and a hydraulic factor (HF) of 1.3 (when applicable).

c. The respective ACI Strength Reduction Factors are used with the corresponding Load Factors above (see ACI 350 Appendix C). The strength reduction factor for

tension-controlled sections (ϕ)is 0.90; the strength reduction factor for shear and torsion is 0.85. Other strength reduction factors can be found in ACI 350-06, Appendix C.

A-12.1.3 Structural Reliability Methodology (existing structures only)

The following structural methodology was developed by the Kansas City District during the course of the Phase 1 – Kansas Citys Levees Feasibility Study. The subsequent criterion was accepted by representatives of the U.S. Army Corps of Engineers Headquarters in the Fall of 2005. The approved structural reliability methodology referred to below, can be found in the MFR "Kansas Citys Structural Summit held 01 Dec 05", Memorandum dated 13 Jan 06.

A-12.1.4 Deterministic Criteria (existing structures only)

Typical strength reduction factors and load factors were not used in the analysis of these structures. Load factors and reduced strengths are used in design, but are considered inappropriate for a probability of failure analysis. If an existing structure has a calculated factor of safety of less than 1.0 (Capacity/Demand), then it implies failure of that structure.

A high enough Factor of Safety (FS) in the strength analysis will provide 99.8% reliability because any variance in coefficients is too low to overcome the safety factor. There is a limit where the FS is still above 1, yet the probability of failure (POF) will begin to increase due to statistical possibilities presented by the coefficient of variance. To prevent unnecessary POF analyses, it is desirable to determine this FS threshold. Two reasons are given to set this FS threshold at 1.5. First, it is possible to calculate the maximum range of FS based upon the coefficients of variation used in the analysis. This is performed by likening the analysis to measurement and instrumentation. Coefficients of Variation (COV) are then treated like uncertainty of a measurement (FS) based upon the mean values. For a system with N^{th} order of uncertainty, a 95% confidence estimate of total uncertainty can be computed by the square root of the sum of the squares of each coefficient of variation. Considering the Coefficient of Variation for concrete compressive strength, steel yield strength, unit weight of soil, seepage pressures, and the angle of internal friction yields a probable maximum range in FS of +/- .28. Failure is not attained until FS < 1. Therefore, by this method, the FS should not reduce the POF unless the FS is near or below 1.28. A FS threshold of 1.5 would guarantee capturing any change to the POF. A second reason why a threshold FS of 1.5 is sufficient is based upon historical results. Historical results from Phase 1 – Kansas City Levees Feasibility Analysis have shown that for a POF analysis with FS above 1.3, the reliability results were still the maximum (99.8% Reliability). Historical analyses have also shown that POF results did not vary appreciably unless the FS was lower than 1.2. This was largely because the Standard Variation used in analysis was small compared to 0.5, and there were only two variables in the majority of the analyses. Using FS threshold of 1.5 has been shown reliable, theoretically and historically.

A-12.1.5 Reliability Analysis (existing conditions only)

a. For structural features meeting the deterministic strength or stability criteria listed above, a reliability of 99.8% was assigned. For structural features not meeting deterministic strength and stability criterion established above, a risk and reliability analysis was performed. The method adopted for calculating a probability of failure is that outlined for geotechnical engineering in "Factors of Safety and Reliability in Geotechnical Engineering", by J. Michael Duncan, published in the Journal of Geotechnical and Geoenvironmental Engineering, April 2000. The use of this method provided consistency between the structural and geotechnical analyses.

b. To produce a probability of failure curve, the critical section of each feature not meeting criteria was analyzed (factor of safety determined) using mean material strengths and/or mean soil properties. Next, each of the parameters was varied to plus and minus one standard deviation from the mean one at a time and the factor of safety was recomputed. The reliability index equation from EM 1110-2-547 was used to determine the reliability of the feature not meeting the factor of safety. The Reliability Index, β, assumes a lognormal distribution, and is relative to a FS equal to 1.0. Assuming the feature started as 100% reliable, the probability of failure was determined by subtracting the reliability from the starting reliability. A 0.2% probability of failure was used as an appropriate non-failure threshold. If a probability of failure greater than 0.2% resulted, then the water elevation was lowered in 1-foot increments and the feature was reanalyzed until the probability of failure obtained was less than 0.2%.

c. The methods used are appropriate when data is normally distributed, when parameters display a linear relationship, and when degradation over time is not a consideration. Because of the limited availability of data and with no information to suggest otherwise, an assumption of normal distributions for input data is reasonable and consistent with guidance provided in ETL 1110-2-547 (paragraph B-6.c). Examples of non-linear behavior for which the methodology should not be used include overturning stability analysis when the resultant is outside the kern of the base. Examples of degradation over time would include scour around piles, reactive concrete, sliding movement, and deteriorating drainage systems that affect uplift. All available historic data, limited site inspections, and engineering judgment do not show time dependent deterioration of structures to be a concern for the Kansas Citys Levee Systems.

A-12.1.5.1 Risk Calculation

a. For strength calculations, mean and standard deviation were qualified for the following: concrete and steel strengths. The selected mean and normal standard deviation were based on engineering judgment and information published in Reliability Based Design in Civil Engineering by Milton E. Harr and ETL 1110-2-556.

b. For stability calculations, mean and standard deviation were qualified for the following: soil unit weight and shear strength. The values were provided by the geotechnical engineers. Varying concrete density had only a minor effect on the factor of safety and therefore was not considered for the risk calculation for this feasibility study.

A-12.1.5.2 Structural Material Properties

a. For the screening portion of the Kansas Citys Levee Systems feasibility study the following structural properties were used. The American Concrete Institute recommended the use of a 3,000 psi concrete design compressive strength around the 1940's through 1960's, the typical timeframe of construction for most of the levee structures in the study. For earlier concrete strengths little information exists, and 2000 psi concrete was assumed.

b. Based upon the construction time period (~1940's – 1960's) and the Portland Cement Association's pamphlet, *Engineered Concrete Structures*, 1997, an assumed reinforcing steel minimum design yield strength, Fy, of 40 ksi is used for most computations, unless known or stated otherwise. For earlier structures (1900-1940), the Concrete Reinforcing Steel Institute's *Engineering Data Report 48* suggests 33 ksi steel is typical.

c. Based on FEMA 310, the mean strength (or expected strength) for Risk and Reliability calculations shall be taken as 125% of the design strength. For reinforced concrete structures Harr suggests a 14% standard deviation.

Concrete Strength Variation (14%)

1940's-1960's: $-\sigma$ = 3225psi, μ = 3750psi, $+\sigma$ = 4275psi (3000 psi min)

1900's-1930's: $-\sigma$ = 2150psi, μ = 2500psi, $+\sigma$ = 2850psi (2000 psi min)

Steel Strength Variation (14%)

1940's-1960's: $-\sigma$ = 43ksi, μ = 50ksi, $+\sigma$ = 57ksi (40 ksi min)

1900's-1930's: $-\sigma$ = 35.5ksi, μ = 41.25ksi, $+\sigma$ = 47.0ksi (33 ksi min)

A-12.1.5.3 Soil Material Properties

The soil properties used for the Kansas Citys Feasibility study structural calculations can be found in the specific levee unit chapter. In general, the soil material properties used came from historical documentation on the specific levee unit. Soil to structure friction and cohesion interaction was neglected for stability and strength calculations for pile founded floodwalls. However, for gatewells and for spread footing founded floodwalls, this behavior was considered under geotechnical guidance. For material variation, according to ETL 1110-2-561 and ETL 1110-2-556, the following standard deviations are appropriate:

- Soil unit weight: 8%
- Angle of Internal Friction: 10%

For pile capacity, according to ETL 1110-2-561, the following standard deviations were used:

- Compression 25%
- Tension 18%

A-12.1.6 Structural Analysis

The following structural features were analyzed for the Kansas Citys feasibility study. Features specific to only one levee unit are mentioned below briefly with feature specifics given in the unit chapters. Features unique to a levee unit and analyzed in a manner different than described below are also more thoroughly discussed in the related levee unit section.

A-12.1.6.1 Floodwalls on Spread Footings

a. Spread footing floodwalls were analyzed for sliding, bearing capacity and overturning stability, along with wall stem and foundation strengths. Each floodwall cross-section was analyzed using the Corps of Engineers' Computer-Aided Structural Engineering (CASE) project program CTWALL. CTWALL analyzes floodwalls and retaining walls based on *EM 1110-2-2502 Retaining and Flood Walls* (Sep 89). To estimate at-rest pressures using Coulomb's active earth pressure equation, the SMF value in the program was set at 2/3 (0.6667) as indicated by EM 2502 resulting in developed shear strength values assumed to be operative in equilibrium conditions. CTWALL computed a sliding factor of safety, percent base in compression, and maximum bearing pressure. Sliding factors of safety and percent base in compression were then compared to required design minimums. The ratio of bearing pressure to allowable soil bearing capacity (supplied by geotechnical team members) was compared to allowable maximums.

b. CTWALL output includes a free body diagram detailing the horizontal and vertical forces acting on the wall cross section. These forces were entered into a MathCAD worksheet developed by the Kansas City District to check shear and flexural strengths. The failure of floodwall stems or foundations was based on a capacity/demand ratio of less than one.

c. To report existing conditions of a floodwall not meeting the minimum strength and stability screening criteria, a reliability analysis was conducted for the floodwall cross section displaying the lowest (controlling) factor of safety and largest COV. Sometimes more than one component can control for development of a POF curve at various water surface elevations. The resulting critical cross section reliability curve was then assigned as the representative curve for the entire reach of floodwall. For example, if a floodwall had 5 different cross sections, Sections A through E, all having the same varied parameters, and section C had the lowest factor of safety, the resulting reliability curve for Section C was used to define the reliability of the entire floodwall.

A-12.1.6.2 Floodwalls on Piles

A Mathcad worksheet was used to perform the static analysis of the floodwalls on piles. The loads applied to the walls were based on EM 1110-2-2502 *Retaining and Flood Walls* (29 Sep 89). This sheet was used to generate the axial, lateral, and bending loads required for input into the Corps' CASE computer program, Pile Group Analysis (CPGA). In addition to the pile cap loading, CPGA required input of pile properties such as type of material (concrete, timber or steel), the shape (square or circular), strength of the material, cross-section and length, fixity of the piles, and soil properties. CPGA in turn determined the combined axial bending forces on the piles. This Program assumes a perfectly rigid pile cap. See closure's structural analysis calculations for levee units with

closure to discern how this program's assumption was managed, as well as analysis of torsion in the pile cap.

The individual pile loads output from CPGA were then used to check the pile capacity based on soil and material strength. This included the assessment of the load against the concrete strength capacity by calculating the pile's interaction diagram and also comparing the load with the soil based capacity. The governing factor of safety was then reported.

A-12.1.6.3 Stoplog and Sandbag Closure Structures

a. For stability, stoplog closure structures were analyzed in a manner similar to floodwalls (outlined previously). Included in the strength analysis were the stoplog strength, the slots for the stoplogs and the posts (when applicable).

b. Routine levee inspections of sandbag gaps have revealed no foundation slab issues for the Kansas Citys units. Strength and stability calculations were not performed for sandbag closure structures. If strength or uplift concerns are experienced during flood events, it can reasonably be assumed that flood fighting efforts (additional sandbags) would be successful in addressing any uplift problems.

A-12.1.6.4 Pump Stations

See pump station specific chapters.

A-12.1.6.5 Gatewells, Reinforced Concrete Boxes, and Drainage Structures

Gatewells and other drainage closure structures were all analyzed in a manner similar to the pump station evaluations. MathCAD worksheets evaluated floatation stability and structural component strengths. Because of the length to width aspect ratios of these structures, plate mechanics were not used. Instead wall and floor component capacities were assessed using one-way beam analysis. The CENWK Local Protection Guidance and EM 1110-2-3104, Appendix B were used to determine the uplift forces for these structures with the exception of vertical resistance mobilized by friction along the exterior face of the structure. Side friction was considered. An effective lateral load that contributes to side friction was calculated and was used to determine an assumed side friction resistance to uplift. Geotechnical engineers provided the hydraulic grade lines and bottom of blanket elevations at the structure location so that the uplift forces could be calculated. For structures not meeting the screening factors of safety for strength and stability, reliability analysis was conducted.

Strength analysis of each gatewell began with a generalized conservative approach to expedite the process. This initial check considered the load at the base to be acting on the typical section just above the opening for the pipe. This is conservative because at the base an opening typically exists in two walls and all four walls are detailed specially for that reason. Therefore, the load in the initial analysis on the section just above the opening is actually too high. Also, the walls just above the base slab will be supported by that slab and will act in a two-way mode rather than a one-way. If this first check gave acceptable results, no further refinement was necessary.

If the initial check did not produce desirable results, the analysis for the typical box section (just above the opening) was reanalyzed for the actual load acting at that

elevation. The walls without openings were then checked as simply supported beams near the base. In cases with large openings, this refinement was often required as the large openings require long walls and resulting gatewells with high aspect ratios. The long walls have openings at the base and do not exist until above the opening while the shorter perpendicular walls form at the base. In such a case, the short walls typically have special reinforcement below the top of the opening in the long walls because they behave more like a simply supported member.

Reinforced concrete boxes were analyzed using the Corps' CASE computer program, for design or investigation of orthogonal culverts (OCRTCUL).

For pipes associated with gatewells, such as RCP, DIP, and CIP, available information and research was used to make recommendations. Generally, the pipe material, the invert elevation, and the size were known, but often little else. In that case, recommendations were made using available information and engineering judgment. Specifics are given in the unit chapters. In addition to a proposed action, detailed inspections will be recommended for all pipes during PED.

A-12.2 Structural Considerations in Raise Alternatives

Below is a list of the primary raise alternatives with corresponding structural implications. The list is in order of preference at locations with an existing levee and an existing floodwall.

Existing Levee:

1. Levee Raise.
    a. Raises soil and water loads on gatewells and pipes.
    b. May require a retaining wall at landside toe.
2. T-wall on existing levee.
    a. Used to lessen footprint as required by site constraints.
    b. Impacts gatewells and pipes by water load only (no additional soil).
3. Floodwall.
    a. Required when T-wall on levee requires a stability berm but the real estate is unavailable (reduces global stability concerns by removing some or all of existing levee material).
    b. Will have negligible impact on pipes as existing soil is removed at the maximum fill location.

Existing Floodwall:

1. Remove and replace floodwall with levee.
    a. Unlikely due to site constraints.
    b. Will have significant soil load increase on pipes.
2. Use existing floodwall as retaining wall to lessen footprint of landside levee.
    a. Reduces impact of a landside levee raise.
    b. Floodwalls are not typically designed to be loaded from the landside and have little ability to retain landside soil.
    c. Requires removal of stem above landside soil to allow for water movement.
    d. May have large impact on pipes (large increase in soil loads)

3. Raise existing floodwall.
    a. Previous experience has shown that this is more economical than replacement when work is not required on the foundation.
    b. Impacts pipes and gatewells due to raisedwater load.
4. Replace existing floodwall with a new floodwall
    a. Nearly equivalent cost to major modification (those requiring foundation work in addition to stem modification) based on Phase 1 estimates.

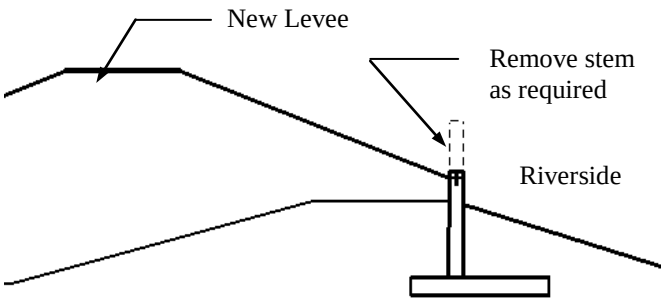


Figure A-12-1: Floodwall Acting as Retaining Wall

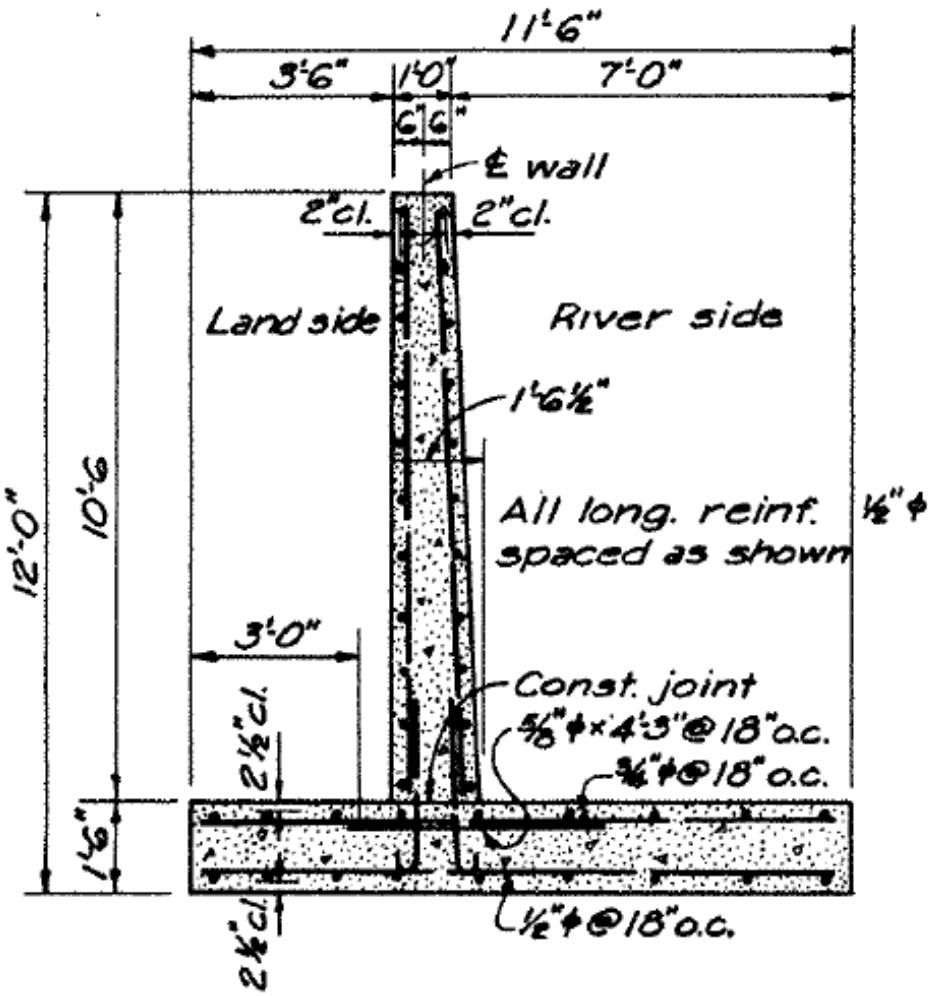
A-12.3 Example Calculations

Example calculations can be found in the General Structural Exhibits accompanying this chapter.

General Structural Exhibits

Exhibit Number	Description
1	Spread Footing Floodwall Sample Calculations
2	Pile Founded Floodwall Sample Calculations
3	Gatewell Sample Calculations
4	Stoplog Gap Sample Calculations

General Structural Exhibit 1:
Spread Footing Floodwall Sample Calculations



Typical Section from 1946 Record Drawings

Variables

$kip := 1000lb \quad plf := \frac{lb}{ft} \quad psf \equiv \frac{lb}{ft^2} \quad ksf \equiv \frac{1000lb}{ft^2} \quad psi \equiv \frac{lb}{in^2} \quad ksi \equiv \frac{1000lb}{in^2} \quad pcf := \frac{lb}{ft^3}$

CTWALL INPUT FILE NAME: CIDMO12.OUT

Elevation of top of stem (ELTS).....	$ELTS := 12ft$
Height of stem (HTS).....	$HTS := 10.5ft$
Thickness of stem (TTS).....	$TTS := 1ft$
Thickness bottom of stem (TBS).....	$TBS := 1.541667ft$
Dist. of batter at bot.of stem (TBSR).....	$TBSR := 0.541667ft$
Depth of heel (THEEL).....	$THEEL := 1.5ft$
Distance of batter for heel (BTRH).....	$BTRH := 0ft$
Depth of toe (TTOE).....	$TTOE := 1.5ft$
Width of toe (TWIDTH).....	$TWIDTH := 3.5ft$
Distance of batter for toe (BTRT).....	$BTRT := 0ft$
Width of base (BWIDTH).....	$BWIDTH := 11.5ft$
Depth of key (HK).....	$HK := 0ft$
Width of bottom of key (TK).....	$TK := 0ft$
Dist. of batter at bot. of key (BTRK).....	$BTRK := 0ft$
Driving side soil elevation (ELSTDS).....	$ELSTDS := 5.25ft$
Resisting side soil elevation (ELSTRS).....	$ELSTRS := 6.25ft$
Driving side water elevation (WATELD).....	$WATELD := 12ft$
Resisting side water elevation (WATELR).....	$WATELR := 6.25ft$

$A := WATELD - ELSTDS$

$B := HTS - (ELTS - ELSTDS)$

$C := THEEL$

$D := TK$

$E := BTRK$

$F := BWIDTH - (TWIDTH + TBS + BTRK + TK)$

$G := TBS$

$H := TWIDTH$

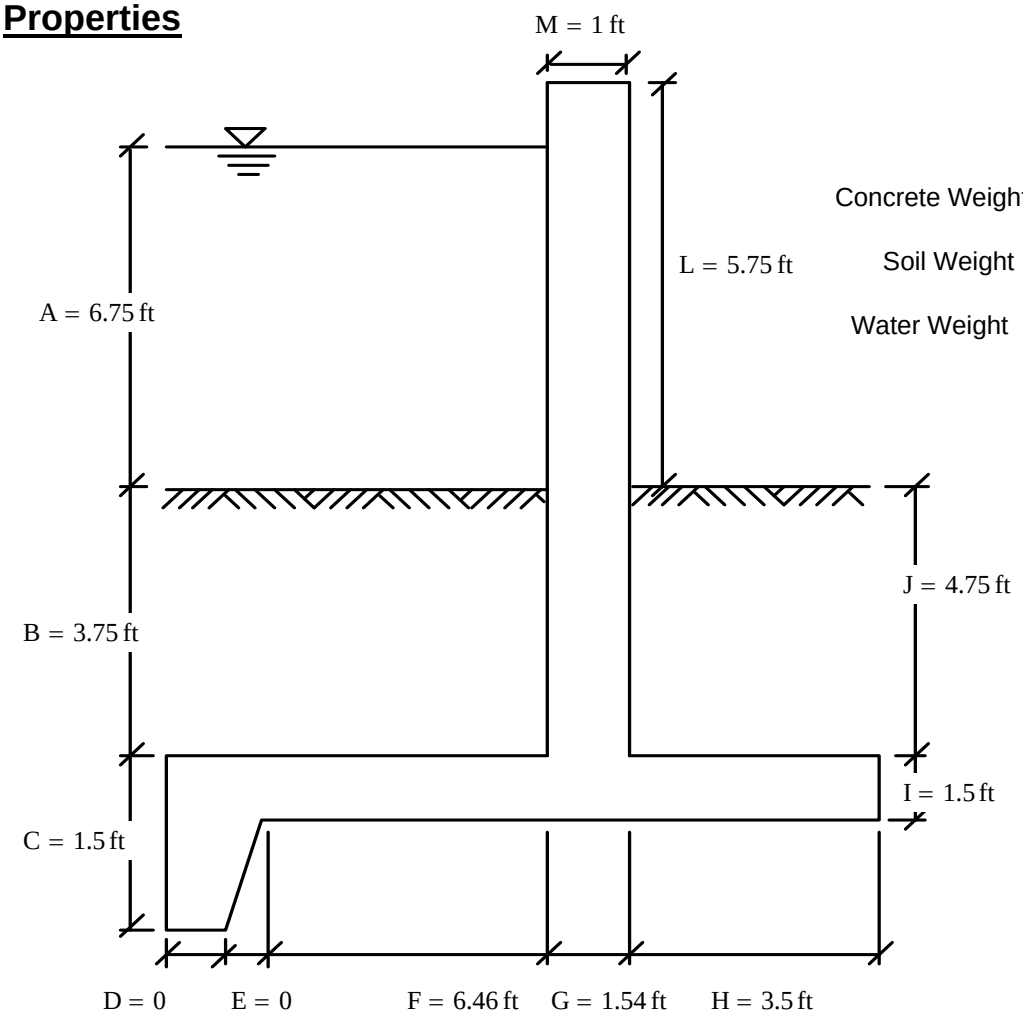
$I := TTOE + BTRT$

$J := HTS - (ELTS - ELSTRS)$

$L := ELTS - ELSTRS$

$M := TTS$

Properties



Concrete Weight $\gamma_c := 150pcf$

Soil Weight $\gamma := 115pcf$

Water Weight $\gamma_w := 62.4pcf$

********** Echoprint of Input Data **********

Date: **/08/02 Time: 14.33.26

JANUARY 11, 2008
CIDMO12.DAT

Company name:
  USACE
Project name:
  KANSAS CITY LEVEES FEASIBILITY STUDY PHASE 2
Project location:
  CID-MISSOURI
Wall location:
  12-FOOT WALL
Computed by: KSM

Structural geometry data:

Elevation of top of stem (ELTS)	=	12.00 ft
Height of stem (HTS)	=	10.50 ft
Thickness top of stem (TTS)	=	1.00 ft
Thickness bottom of stem (TBS)	=	1.54 ft
Dist. of batter at bot. of stem (TBSR)	=	.54 ft
Depth of heel (THEEL)	=	1.50 ft
Distance of batter for heel (BTRH)	=	.00 ft
Depth of toe (TTOE)	=	1.50 ft
Width of toe (TWIDTH)	=	3.50 ft
Distance of batter for toe (BTRT)	=	.00 ft
Width of base (BWIDTH)	=	11.50 ft
Depth of key (HK)	=	.00 ft
Width of bottom of key (TK)	=	.00 ft
Dist. of batter at bot. of key (BTRK)	=	.00 ft

Structure coordinates:

x (ft)	y (ft)
.00	.00
.00	1.50
6.46	1.50
6.46	12.00
7.46	12.00
8.00	1.50
11.50	1.50
11.50	.00

NOTE: X=0 is located at the left-hand side
      of the structure. The Y values correspond
      to the actual elevation used.

Structural property data:
  Unit weight of concrete = .150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
22.00	.000	.110	.115	.00	5.25

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	.00	500.00
2	.00	.00
3	.00	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1493.54	5.25
2	6.46	5.25

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)

```
   22.00     .000      .110      .115     6.25     .00

Resisting side soil profile:

  Soil        x          y
  point      (ft)       (ft)
  ============================
   1         7.76      6.25
   2       507.76      6.25

Foundation property data:
  phi for soil-structure interface =    22.00 (deg)
  c for soil-structure interface   =     .000 (ksf)
  phi for soil-soil interface      =    22.00 (deg)
  c for soil-soil interface        =     .000 (ksf)

Water data:
  Driving side elevation   =    12.00 ft
  Resisting side elevation =     6.25 ft
  Unit weight of water     =    .0624 kcf
  Seepage pressures computed by Line of Creep method.

Minimum required factors of safety:
  Sliding FS   =    1.30
  Overturning  =   25.00% base in compression

Crack options:
  o  Crack *is* down to bottom of heel
  o  Computed cracks *will* be filled with water

User input failure angle data:
  Failure angle of wedge  2 =     .00 deg

Strength mobilization factor =    .6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

*Do* iterate in overturning analysis.



***** Summary of Results *****

JANUARY 11, 2008

Project name: KANSAS CITY LEVEES FEASIBILITY STUDY PHASE 2

***************    *** Satisfied ***
* Overturning *    Required base in comp. =  25.00 %
***************    Actual base in comp.   =  71.56 %
                   Overturning ratio      =   1.19

Xr (measured from toe) =   2.74 ft
Resultant ratio        =  .2385
Stem ratio             =  .3043
Base pressure at x=   8.23 ft from toe =     .0000 ksf
Base pressure at toe                   =    1.0739 ksf

*** Warning ***  The maximum available shear along the
base of the structure has been exceeded!

***********    *** Not Satisfied ***
* Sliding *    Min. Required =   1.30
***********    Actual FS     =   1.12

  To increase stability try one or a combination
  of the following:
    1.  Increase the base width
    2.  Slope the base of the structure
    3.  Lower the wall base
    4.  Add a key


********************** Output Results **********************


Date: **/08/02                                Time: 14.33.26

JANUARY 11, 2008
CIDMO12.DAT

Company name:
  USACE
```

Project name:
KANSAS CITY LEVEES FEASIBILITY STUDY PHASE 2
Project location:
CID-MISSOURI
Wall location:
12-FOOT WALL
Computed by: KSM

****
** Overturning Results **
****

Solution converged in 6 iterations.

SMF used to calculate K's = .6667
Alpha for the SMF = .0000
Calculated earth pressure coefficients:
Driving side at rest K = .0000
Driving side at rest Kc = .0000
Resisting side at rest K = .6254
Resisting side at rest Kc = .7908
At-rest K's for resisting side calculated.

Depth of cracking = 5.25 ft
Crack extends to bottom of base of structure.

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
12.00	.0000
.00	.7488

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
6.25	.0000
.00	.5163

Earth pressures:

Elevation (ft)	Pressure (ksf)
6.25	.0000
.00	.1266

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
.00	.7488
3.27	.7488
11.50	.5163

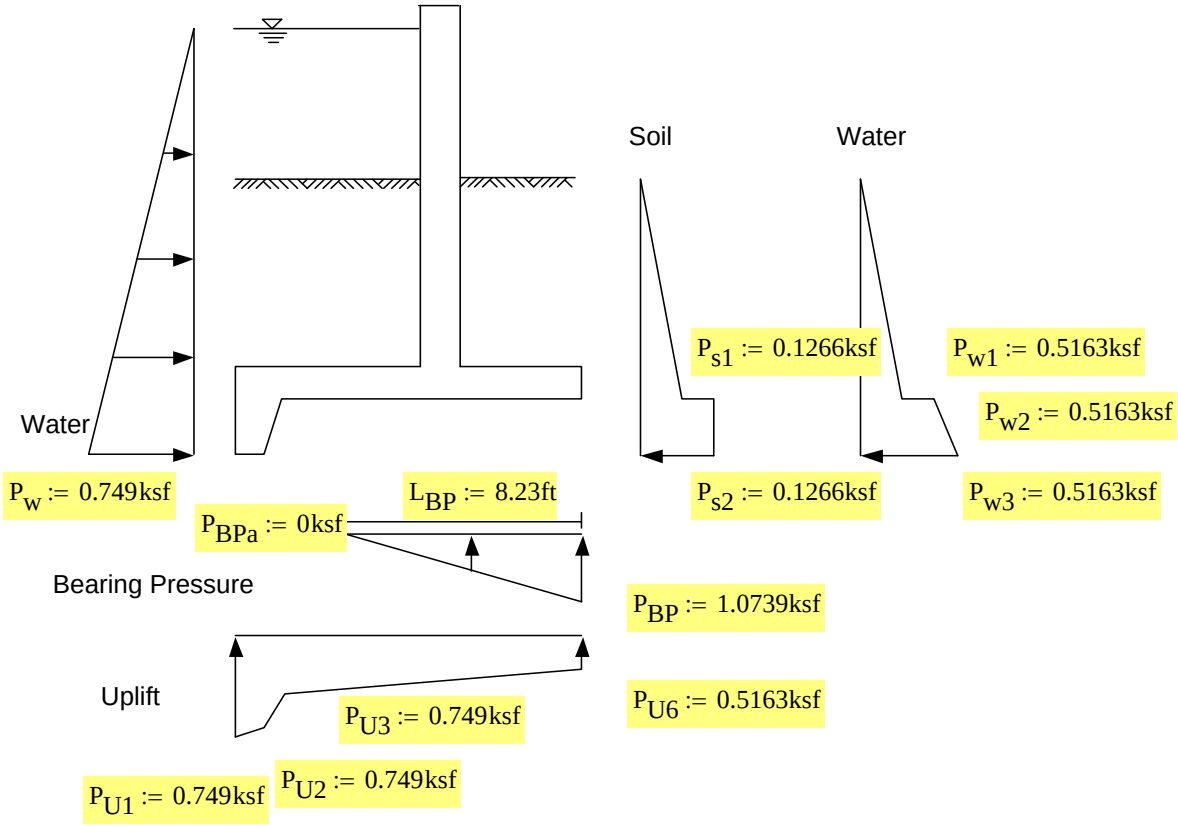
** Forces and moments **

Part	Force (kips) Vert.	Force (kips) Horiz.	Mom. Arm (ft)	Moment (ft-k)
Structure:				
Structure weight.....	4.588		-5.16	-23.67
Structure, driving side:				
Moist soil.....	.000		.00	.00
Saturated soil.....	2.786		-8.27	-23.04
Water above structure.....	.000		.00	.00
Water above soil.....	2.721		-8.27	-22.50
External vertical loads....	.000		.00	.00
Ext. horz. pressure loads..		.000	.00	.00
Ext. horz. line loads.....		.000	.00	.00
Structure, resisting side:				
Moist soil.....	.000		.00	.00
Saturated soil.....	1.979		-1.81	-3.58
Water above structure.....	.000		.00	.00
Water above soil.....	.000		.00	.00
Driving side:				
Effective earth loads.....		.000	.00	.00
Shear (due to delta).....	.000		.00	.00
Horiz. surcharge effects...		.000	.00	.00
Water loads.....		4.493	4.00	17.97
Resisting side:				

```
  Effective earth loads......              -.396       2.08      -.82
  Water loads...............             -1.614       2.08     -3.36
Foundation:
  Vertical force on base.....   -4.419                -2.74     12.12
  Shear on base.............             -2.484        .00       .00
  Uplift....................   -7.654               -6.13     46.89
=================================================================
** Statics Check **   SUMS =      .000       .000                 .00

Angle of base        =      .00 degrees
Normal force on base =    4.419 kips
Shear force on base  =    2.484 kips
Max. available shear force =    1.785 kips

*** Warning ***  The maximum available shear along the
base of the structure has been exceeded!

Base pressure at x=   8.23 ft from toe =    .0000 ksf
Base pressure at toe                   =   1.0739 ksf

Xr (measured from toe) =     2.74 ft
Resultant ratio        =    .2385
Stem ratio             =    .3043
Base in compression    =    71.56 %
Overturning ratio      =     1.19

Volume of concrete =     1.13 cubic yds/ft of wall

NOTE:  The engineer shall verify that the computed
bearing pressures below the wall do not exceed the
allowable foundation bearing pressure, or, perform a
bearing capacity analysis using the program CBEAR.
Also, the engineer shall verify that the base pressures
do not result in excessive differential settlement of
the wall foundation.



***********************
**  Sliding Results  **
***********************

Solution converged.  Summation of forces = 0.

           Horizontal   Vertical
  Wedge      Loads       Loads
  Number    (kips)       (kips)
  ==================================
     1        .000        .000
     2       4.493       2.721
     3        .000        .000

  Water pressures on wedges:

             Top      Bottom
    Wedge    press.   press.   x-coord.    press.
    number   (ksf)    (ksf)     (ft)       (ksf)
  ================================================
      1     .0000     .0000
      2                          .0000      .7488
      2                         3.2703      .7488
      2                        11.5000      .5163
      3     .0000     .5163

Points of sliding plane:
  Point 1 (left),  x =      .00 ft,   y =      .00 ft
  Point 2 (right), x =    11.50 ft,   y =      .00 ft

Depth of cracking =     5.25 ft
Crack extends to bottom of base of structure.

          Failure   Total    Weight    Submerged   Uplift
  Wedge    angle    length  of wedge    length     force
  number   (deg)     (ft)    (kips)      (ft)      (kips)
  ========================================================
     1      .000      .000       .000      .000      .000
     2      .000    11.500      9.352   11.500     7.655
     3    35.090    10.872      3.197   10.872     2.807

  Wedge     Net force
  number     (kips)
  ===================
     1         .000
     2       -2.897
     3        2.897
  ===================
     SUM =     .000
```

```
+----------------------------+
| Factor of safety =    1.119 |
+----------------------------+
```

Loading from CTWALL Output



Assumptions

- Concrete and reinforcing strengths were not specified in the documents found. However, modifications to the CID-KS unit was under construction/design at the same time as the construction/design of the CID-MO unit. Therefore, it is a reasonable assumption to make that the same material strengths would be specified for CID-MO. The CID-KS design memorandum specifies the concrete strength and reinforcing steel properties as listed here:

Concrete Properties $f'_c := 3000 \cdot psi$

Steel Properties $F_y := 36ksi$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$

Hydraulic Load Factor $\gamma_H := 1.0$

Extreme Case Factor $\gamma_X := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.

Load Multiplication Factor EM 1110-2-2104 (3-1)

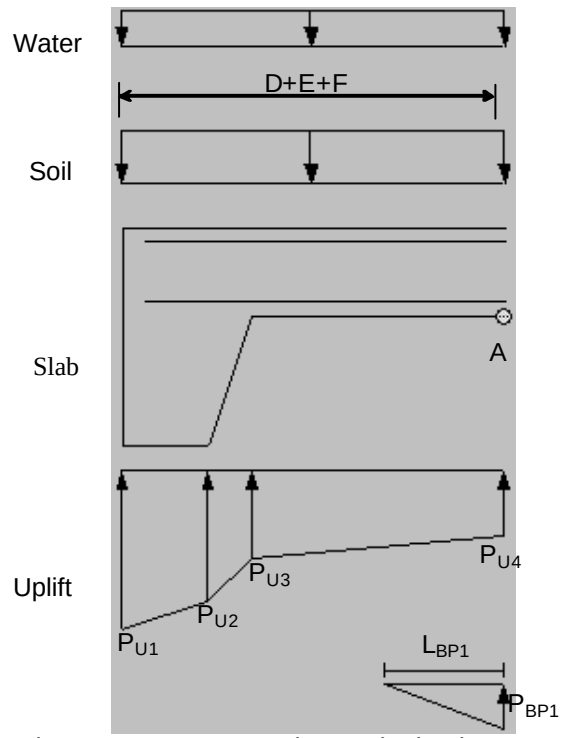
Hydraulic Factor EM 1110-2-2104 (3-2)

Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Reinforcement Checks

Location where moment is taken about.

- HEEL



$W_w := \gamma_w \cdot A \cdot (D + E + F)$

$W_w = 2720 \frac{lb}{ft}$

$W_s := \gamma \cdot B \cdot (D + E + F)$

$W_s = 2785.16 \frac{lb}{ft}$

$A_{Heel1} := 0.49 \frac{in^2}{ft}$ $C_{CH1} := 3.375in$

$d_{H1} := I - C_{CH1}$

$A_{Heel2} := 0.133 \frac{in^2}{ft}$ $C_{CH2} := 3.25in$

$d_{H2} := I - C_{CH2}$

$P_{U4} := (P_{U3} - P_{U6}) \cdot \left(\frac{G + H}{F + G + H}\right) + P_{U6}$

$P_{U4} = 618.32 \frac{lb}{ft^2}$

$L_{BP1} := if\left[\left[L_{BP} - (G + H)\right] \geq 0ft, \left[L_{BP} - (G + H)\right], 0\right]$

$L_{BP1} = 3.19\,ft$

Bearing Pressure, P_{BP1} acting on the heel at location "A"

$P_{BP1} := if\left[L_{BP} < (D + E + F + G + H), \frac{L_{BP1}}{L_{BP}} \cdot P_{BP}, P_{BPa} + \frac{L_{BP1} \cdot (P_{BP} - P_{BPa})}{L_{BP}}\right]$

$P_{BP1} = 416 \frac{lb}{ft^2}$

Slab Centroid

$$X_H := \frac{(D + E + F) \cdot I \cdot \left(\frac{D + E + F}{2}\right) + (C - I) \cdot D \cdot \left(E + F + \frac{D}{2}\right) + \left[\frac{(C - I) \cdot E}{2}\right] \cdot \left(F + \frac{2 \cdot E}{3}\right)}{(D + E + F) \cdot I + (C - I) \cdot D + \frac{(C - I) \cdot E}{2}}$$

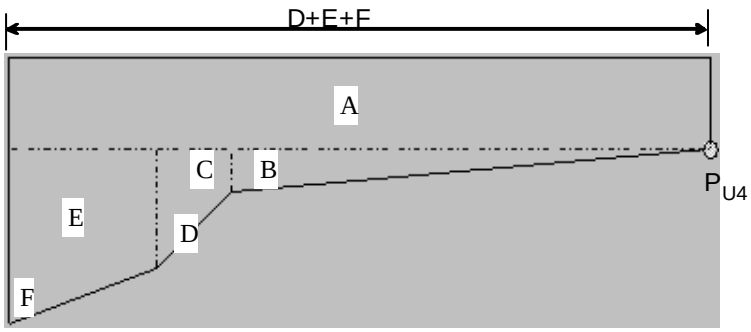
$X_H = 3.23\,ft$

- HEEL check (cont'd)

Slab Weight

$$W_H := \left[(D + E + F)\cdot I + (C - I)\cdot D + \frac{(C - I)\cdot E}{2}\right]\cdot 150pcf \qquad W_H = 1453\,\frac{lb}{ft}$$

Uplift Centroid



AREA

$$A_A := (D + E + F)\cdot P_{U4}$$

$$A_B := \frac{F\cdot(P_{U3} - P_{U4})}{2}$$

$$A_C := E\cdot(P_{U3} - P_{U4})$$

$$A_D := \frac{E\cdot(P_{U2} - P_{U3})}{2}$$

$$A_E := D\cdot(P_{U2} - P_{U4})$$

$$A_F := \frac{D\cdot(P_{U1} - P_{U2})}{2}$$

CENTROID

$$C_A := \frac{D + E + F}{2}$$

$$C_B := \frac{2\cdot F}{3}$$

$$C_C := F + \frac{E}{2}$$

$$C_D := F + \frac{2\cdot E}{3}$$

$$C_E := \left(\frac{D}{2} + E + F\right)$$

$$C_F := \left(\frac{2\cdot D}{3} + E + F\right)$$

$$X_U := \frac{A_A\cdot C_A + A_B\cdot C_B + A_C\cdot C_C + A_D\cdot C_D + A_E\cdot C_E + A_F\cdot C_F}{A_A + A_B + A_C + A_D + A_E + A_F} \qquad X_U = 3.33\,ft$$

Uplift on Heel

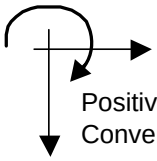
$$W_U := A_A + A_B + A_C + A_D + A_E + A_F \qquad W_U = 4415\,\frac{lb}{ft}$$

- HEEL check (cont'd)

Loading:

Bending

$$M_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[W_U \cdot X_U + \frac{P_{BP1} \cdot L_{BP1}}{2} \cdot \frac{L_{BP1}}{3} - \left[\left(W_w + W_s \right) \cdot \left(\frac{D + E + F}{2} \right) + W_H \cdot X_H \right] \right]$$



Positive Sign Convention

$$M_H = -7053.44 \cdot \frac{ft \cdot lb}{ft} \qquad M_{uH} := \left| M_H \right|$$

Note:
IF: M_H is < 0
THEN: Steel in top of heel is in tension

Shear

$$V_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left(W_w + W_s + W_H - W_U - \frac{P_{BP1} \cdot L_{BP1}}{2} \right) \qquad M_{uH} = 7.1 \cdot \frac{kip \cdot ft}{ft}$$

$$V_H = 1880.01 \frac{lb}{ft} \qquad V_{uH} := \left| V_H \right| \qquad V_{uH} = 1.9 \cdot \frac{kip}{ft}$$

Capacity:

Flexural Capacity

$$A_{s1} := A_{Heel1} \qquad A_{s2} := A_{Heel2}$$

$$b := 12 \frac{in}{ft} \qquad b := 12 \frac{in}{ft}$$

$$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a1 = 0.58 \cdot in \qquad a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b} \qquad a2 = 0.16 \cdot in$$

$$\phi M_{H1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{H1} - \frac{a1}{2} \right) \qquad \phi M_{H2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{H2} - \frac{a2}{2} \right)$$

$$\phi M_{H1} = 21.08 \cdot \frac{kip \cdot ft}{ft} \qquad \phi M_{H2} = 5.85 \cdot \frac{kip \cdot ft}{ft}$$

$$\phi M_H := \text{if}\left(M_H > 0, \phi M_{H2}, \phi M_{H1} \right) \qquad \phi M_H = 21.08 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{H1} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{H2} \cdot \sqrt{f'_c \cdot psi}$$

$$\phi V_{c1} = 19225 \frac{lb}{ft} \qquad \phi V_{c2} = 19389 \frac{lb}{ft}$$

$$\phi V_H := \text{if}\left(M_H > 0, \phi V_{c2}, \phi V_{c1} \right) \qquad \phi V_H = 19.23 \cdot \frac{kip}{ft}$$

- HEEL check (cont'd)

Factors of Safety

Bending

$\phi M_H = 21.08 \cdot \frac{kip \cdot ft}{ft}$

$M_{uH} = 7.05 \cdot \frac{kip \cdot ft}{ft}$

$FS_1 := \frac{\phi M_H}{M_{uH}}$

$FS_1 = 2.99$

$Check1 := if\left(\phi M_H > 1.5 M_{uH}, "OKAY", "NO\ GOOD"\right)$

$Check1 = "OKAY"$

Shear

$\phi V_H = 19.23 \cdot \frac{kip}{ft}$

$V_{uH} = 1.88 \cdot \frac{kip}{ft}$

$FS_2 := \frac{\phi V_H}{V_{uH}}$

$FS_2 = 10.23$

$Check2 := if\left(\phi V_H > 1.5 V_{uH}, "OKAY", "NO\ GOOD"\right)$

$Check2 = "OKAY"$

Controlling Factor of Safety

$FS_H := \min\left(FS_1, FS_2\right)$

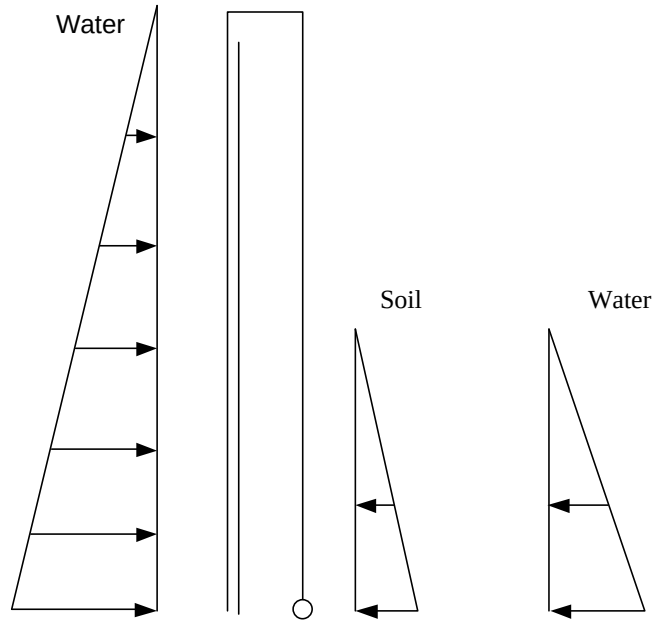
$FS_H = 2.99$

Controlling Mechanism

$Comment_H := if\left(FS_1 > FS_2, "Shear\ in\ Heel", if\left(M_H > 0, "Flexural\ Bottom\ Steel\ in\ Heel", "Flexural\ Top\ Steel\ in\ Heel"\right)\right)$

$Comment_H = "Flexural\ Top\ Steel\ in\ Heel"$

- STEM Check



Note:
P_{ss} = Soil pressure present at base of stem
P_{ws2} = Water pressure present at base of stem

$P_{ws1} := (A + B)\cdot\gamma_w$

$P_{ws1} = 655\,\frac{lb}{ft^2}$

$A_{Stem1} := 0.35\,\frac{in^2}{ft}$ $A_{Stem2} := 0.2\,\frac{in^2}{ft}$

$P_{ss} := \frac{J}{I + J}\cdot P_{s1}$ $P_{ws2} := \frac{J}{I + J}\cdot P_{w1}$

$C_{CS1} := 2.8125in$ $C_{CS2} := 2.75in$

$P_{ss} = 96.22\,\frac{lb}{ft^2}$ $P_{ws2} = 392.39\,\frac{lb}{ft^2}$

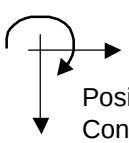
$d_{S1} := G - C_{CS1}$ $d_{S2} := G - C_{CS2}$

Loading:

Bending

$$M_S := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A + B)}{2}\cdot\frac{A + B}{3} - \frac{(P_{ss} + P_{ws2})\cdot(J)}{2}\cdot\frac{J}{3}\right]$$

Note:
IF: M_S is > 0
THEN: Riverside steel is in tension



Positive Sign Convention

$M_S = 10201.9\cdot\frac{ft\cdot lb}{ft}$ $M_{uS} := |M_S|$ $M_{uS} = 10.2\cdot\frac{kip\cdot ft}{ft}$

Shear

$$V_S := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A + B)}{2} - \frac{(P_{ss} + P_{ws2})\cdot(J)}{2}\right]$$

$V_S = 2.28\cdot\frac{kip}{ft}$ $V_{uS} := |V_S|$ $V_{uS} = 2.3\cdot\frac{kip}{ft}$

- STEM Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Stem1}$ $\qquad$ $A_{s2} := A_{Stem2}$

$b := 12\frac{in}{ft}$ $\qquad$ $b := 12\frac{in}{ft}$

$a1 := \frac{A_{s1}\cdot F_y}{0.85f'_c\cdot b}$ $\qquad a1 = 0.41\cdot in$ $\qquad$ $a2 := \frac{A_{s2}\cdot F_y}{0.85f'_c\cdot b}$ $\qquad a2 = 0.24\cdot in$

$\phi M_{S1} := \phi_B A_{s1}\cdot F_y\cdot\left(d_{S1} - \frac{a1}{2}\right)$ $\qquad$ $\phi M_{S2} := \phi_B A_{s2}\cdot F_y\cdot\left(d_{S2} - \frac{a2}{2}\right)$

$\phi M_{S1} = 16.26\cdot\frac{kip\cdot ft}{ft}$ $\qquad$ $\phi M_{S2} = 9.38\cdot\frac{kip\cdot ft}{ft}$

$\phi M_S := if\left(M_S > 0, \phi M_{S1}, \phi M_{S2}\right)$ $\qquad$ $\phi M_S = 16.26\cdot\frac{kip\cdot ft}{ft}$

Shear Capacity

$\phi V_{c1} := \phi_V\cdot 2\cdot b\cdot d_{S1}\cdot\sqrt{f'_c\cdot psi}$ $\qquad$ $\phi V_{c2} := \phi_V\cdot 2\cdot b\cdot d_{S2}\cdot\sqrt{f'_c\cdot psi}$

$\phi V_{c1} = 20622\frac{lb}{ft}$ $\qquad$ $\phi V_{c2} = 20704\frac{lb}{ft}$

$\phi V_S := if\left(M_S > 0, \phi V_{c1}, \phi V_{c2}\right)$ $\qquad$ $\phi V_S = 20.62\cdot\frac{kip}{ft}$

Factors of Safety

Bending

$\phi M_S = 16.26\cdot\frac{kip\cdot ft}{ft}$ $\qquad$ $FS_3 := \frac{\phi M_S}{M_{uS}}$

$M_{uS} = 10.2\cdot\frac{kip\cdot ft}{ft}$ $\qquad$ $FS_3 = 1.59$

$Check3 := if\left(\phi M_S > 1.5M_{uS}, "OKAY", "NO\ GOOD"\right)$ $\qquad$ $Check3 = "OKAY"$

Shear

$\phi V_S = 20.62\cdot\frac{kip}{ft}$ $\qquad$ $FS_4 := \frac{\phi V_S}{V_{uS}}$

$V_{uS} = 2.28\cdot\frac{kip}{ft}$ $\qquad$ $FS_4 = 9.05$

$Check4 := if\left(\phi V_S > 1.5V_{uS}, "OKAY", "NO\ GOOD"\right)$ $\qquad$ $Check4 = "OKAY"$

Controlling Factor of Safety

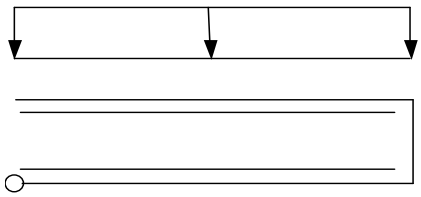
$FS_S := \min\left(FS_3, FS_4\right)$ $\qquad$ $FS_S = 1.59$

Controlling Mechanism

- TOE Check

Soil

Slab



$W_s := \gamma \cdot H \cdot (J)$ $\qquad W_s = 1911.87 \frac{lb}{ft}$

$A_{Toe1} := 0.29 \frac{in^2}{ft}$ $\qquad C_{CT1} := 3.375in$

$d_{T1} := I - C_{CT1}$

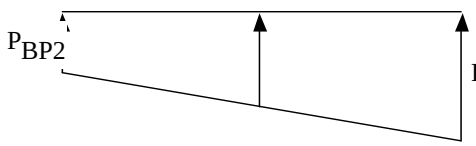
$A_{Toe2} := 0.133 \frac{in^2}{ft}$ $\qquad C_{CT2} := 3.25in$ $\qquad d_{T1} = 14.63 \cdot in$

$L_{BP2} := if(H \le L_{BP}, H, L_{BP})$

$L_{BP2} = 3.5\,ft$

$d_{T2} := I - C_{CT2}$

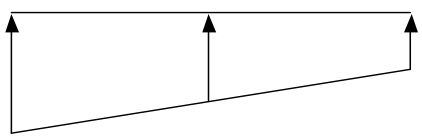
Bearing Pressure



$P_{BP} = 1073.9 \frac{lb}{ft^2}$ $\qquad d_{T2} = 14.75 \cdot in$

$$P_{BP2} := if\left[L_{BP} < (D + E + F + G + H), P_{BP} - P_{BP} \cdot \frac{L_{BP2}}{L_{BP}}, P_{BP} - L_{BP2} \cdot \left(\frac{P_{BP} - P_{BPa}}{L_{BP}}\right)\right] = 617.2 \frac{lb}{ft^2}$$

Uplift



$P_{U6} = 516.3 \frac{lb}{ft^2}$

$$P_{U5} := \left[\left[(P_{U3} - P_{U6}) \cdot \frac{H}{F + G + H} + P_{U6}\right]\right]$$

$$P_{U5} = 587.12 \frac{lb}{ft^2}$$

Bearing Pressure

$$L_{bp} := \frac{(L_{BP2} \cdot P_{BP2}) \cdot \left(\frac{L_{BP2}}{2}\right) + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2} \cdot \left(\frac{2L_{BP}}{3}\right)}{L_{BP2} \cdot P_{BP2} + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2}} \qquad L_{bp} = 2.76\,ft$$

$$W_{bp} := L_{BP2} \cdot P_{BP2} + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2} \qquad W_{bp} = 2959 \frac{lb}{ft}$$

Uplift

$$L_u := \frac{H \cdot P_{U6} \cdot \frac{H}{2} + \frac{(P_{U5} - P_{U6}) \cdot H}{2} \cdot \frac{H}{3}}{H \cdot P_{U6} + \frac{(P_{U5} - P_{U6}) \cdot H}{2}} \qquad L_u = 1.71\,ft$$

$$W_u := H \cdot P_{U6} + \frac{(P_{U5} - P_{U6}) \cdot H}{2} \qquad W_u = 1930.99 \frac{lb}{ft}$$

Toe Check (continued)

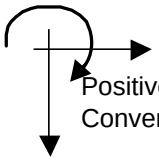
Loading:

Bending

$$M_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[\left[W_s + (H \cdot I) \cdot \gamma_c \right] \cdot \frac{H}{2} - \left(W_{bp} \cdot L_{bp} + W_u \cdot L_u \right) \right]$$

Note:
IF: $M_T < 0$
THEN: Bottom steel is in tension

$$M_T = -6.7 \cdot \frac{kip \cdot ft}{ft} \qquad M_{uT} := \left| M_T \right| \qquad M_{uT} = 6.75 \cdot \frac{kip \cdot ft}{ft}$$



Shear

$$V_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[W_s + (H \cdot I) \cdot \gamma_c - W_{bp} - W_u \right]$$

$$V_T = -2.19 \cdot \frac{kip}{ft} \qquad V_{uT} := \left| V_T \right| \qquad V_{uT} = 2.19 \cdot \frac{kip}{ft}$$

Capacity:

Flexural Capacity

$$A_{s1} := A_{Toe1} \qquad A_{s2} := A_{Toe2}$$

$$b := 12 \frac{in}{ft} \qquad b := 12 \frac{in}{ft}$$

$$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a1 = 0.34 \cdot in \qquad a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b} \qquad a2 = 0.16 \cdot in$$

$$\phi M_{T1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{T1} - \frac{a1}{2} \right) \qquad \phi M_{T2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{T2} - \frac{a2}{2} \right)$$

$$\phi M_{T1} = 12.58 \cdot \frac{kip \cdot ft}{ft} \qquad \phi M_{T2} = 5.85 \cdot \frac{kip \cdot ft}{ft}$$

$$\phi M_T := \text{if}\left(M_T > 0, \phi M_{T1}, \phi M_{T2} \right) \qquad \phi M_T = 5.85 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{T1} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{T2} \cdot \sqrt{f'_c \cdot psi}$$

$$\phi V_{c1} = 19225 \frac{lb}{ft} \qquad \phi V_{c2} = 19389 \frac{lb}{ft}$$

$$\phi V_T := \text{if}\left(M_{uT} > 0, \phi V_{c1}, \phi V_{c2} \right) \qquad \phi V_T = 19.23 \cdot \frac{kip}{ft}$$

Toe Check (continued)

Factors of Safety

Bending

$\phi M_T = 5.85 \cdot \frac{kip \cdot ft}{ft}$

$M_{uT} = 6.75 \cdot \frac{kip \cdot ft}{ft}$

$FS_5 := \frac{\phi M_T}{M_{uT}}$

$FS_5 = 0.87$

$Check5 := if\left(\phi M_T > 1.5 \cdot M_{uT}, "OKAY", "NO\ GOOD"\right)$

$Check5 = "NO\ GOOD"$

Shear

$\phi V_T = 19.23 \cdot \frac{kip}{ft}$

$V_{uT} = 2.19 \cdot \frac{kip}{ft}$

$FS_6 := \frac{\phi V_T}{V_{uT}}$

$FS_6 = 8.77$

$Check6 := if\left(\phi V_T > 1.5 V_{uT}, "OKAY", "NO\ GOOD"\right)$

$Check6 = "OKAY"$

Controlling Factor of Safety

$FS_T := \min\left(FS_5, FS_6\right)$

$FS_T = 0.87$

Controlling Mechanism

$Comment_T := if\left(FS_5 > FS_6, "Shear\ in\ Heel", if\left(M_T > 0, "Flexural\ Top\ Steel\ in\ Toe", "Flexural\ Bottom\ Steel\ in\ Toe"\right)\right)$

$Comment_T = "Flexural\ Bottom\ Steel\ in\ Toe"$

Overall Factor of Safety (without considering multiple layers of steel)

Factor of Safety

$$FoS := \min(FS_H, FS_S, FS_T)$$

Limiting Mechanism

$$Mechanism := Comment_H$$

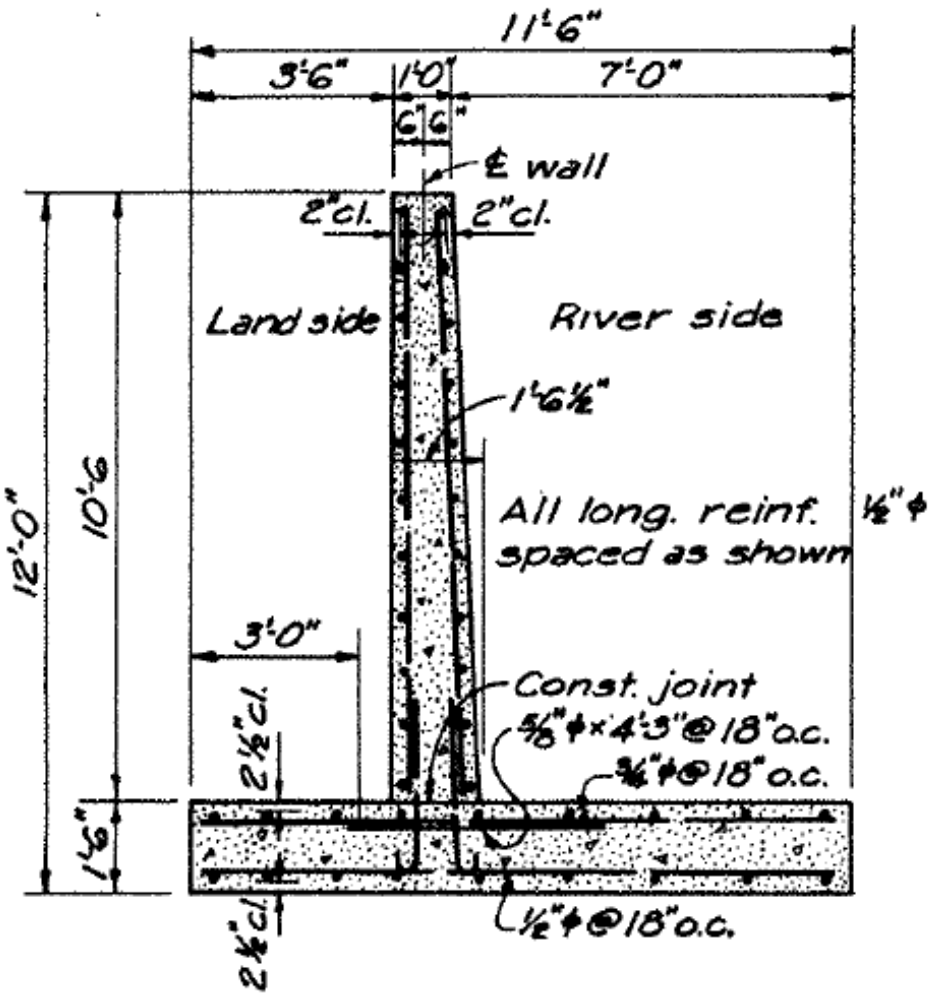
$$Mechanism := \text{if}(FoS = FS_S, Comment_S, Mechanism)$$

$$Mechanism := \text{if}(FoS = FS_T, Comment_T, Mechanism)$$

$$FoS = 0.87$$

$$Mechanism = \text{"Flexural Bottom Steel in Toe"}$$

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis. If the factor of safety is lower than 1.5 a probablility analysis is required.



Typical Section from 1946 Record Drawings

Variables

$kip := 1000lb \quad plf := \frac{lb}{ft} \quad psf \equiv \frac{lb}{ft^2} \quad ksf \equiv \frac{1000lb}{ft^2} \quad psi \equiv \frac{lb}{in^2} \quad ksi \equiv \frac{1000lb}{in^2} \quad pcf := \frac{lb}{ft^3}$

CTWALL INPUT FILE NAME: CIDMO121.OUT

Elevation of top of stem (ELTS).....	$ELTS := 12ft$
Height of stem (HTS).....	$HTS := 10.5ft$
Thickness of stem (TTS).....	$TTS := 1ft$
Thickness bottom of stem (TBS).....	$TBS := 1.541667ft$
Dist. of batter at bot.of stem (TBSR).....	$TBSR := 0.541667ft$
Depth of heel (THEEL).....	$THEEL := 1.5ft$
Distance of batter for heel (BTRH).....	$BTRH := 0ft$
Depth of toe (TTOE).....	$TTOE := 1.5ft$
Width of toe (TWIDTH).....	$TWIDTH := 3.5ft$
Distance of batter for toe (BTRT).....	$BTRT := 0ft$
Width of base (BWIDTH).....	$BWIDTH := 11.5ft$
Depth of key (HK).....	$HK := 0ft$
Width of bottom of key (TK).....	$TK := 0ft$
Dist. of batter at bot. of key (BTRK).....	$BTRK := 0ft$
Driving side soil elevation (ELSTDS).....	$ELSTDS := 5.25ft$
Resisting side soil elevation (ELSTRS).....	$ELSTRS := 6.25ft$
Driving side water elevation (WATELD).....	$WATELD := 11ft$
Resisting side water elevation (WATELR).....	$WATELR := 6.25ft$

$A := WATELD - ELSTDS$

$B := HTS - (ELTS - ELSTDS)$

$C := THEEL$

$D := TK$

$E := BTRK$

$F := BWIDTH - (TWIDTH + TBS + BTRK + TK)$

$G := TBS$

$H := TWIDTH$

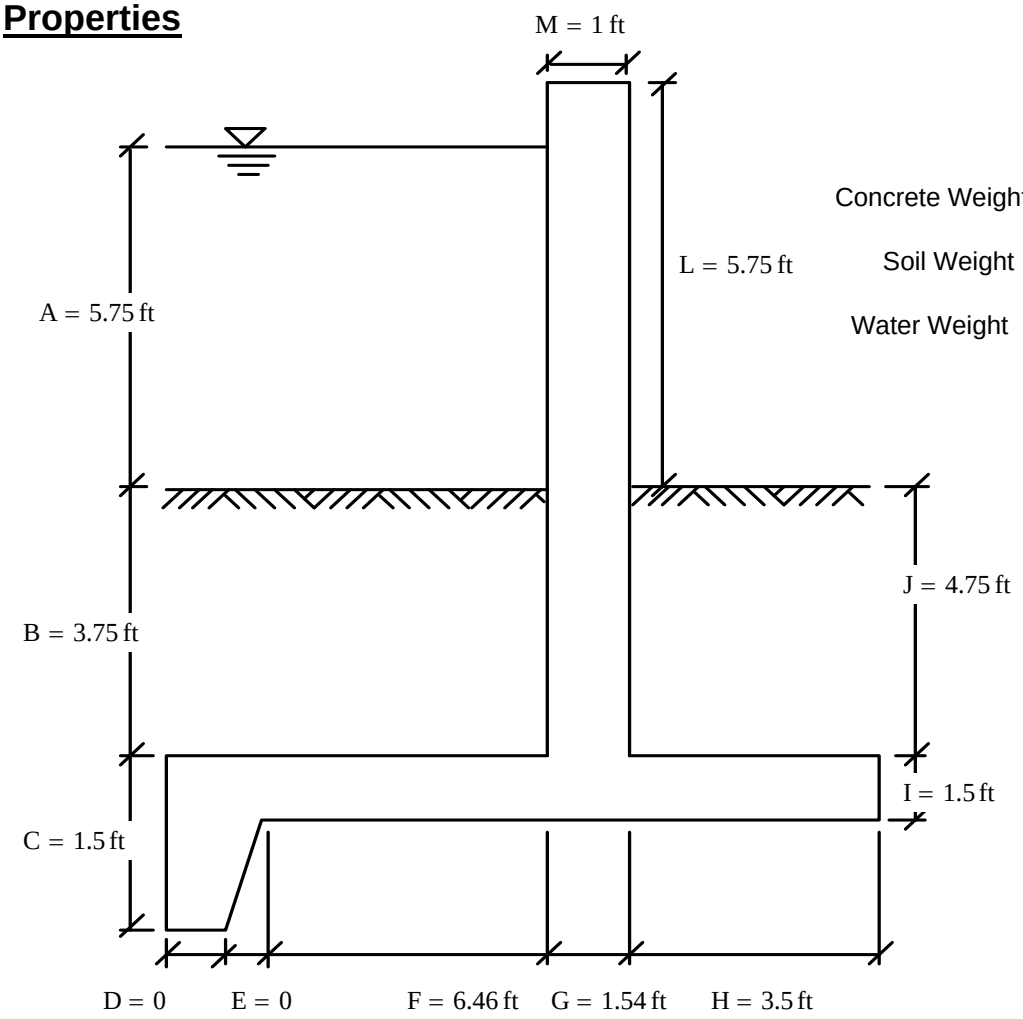
$I := TTOE + BTRT$

$J := HTS - (ELTS - ELSTRS)$

$L := ELTS - ELSTRS$

$M := TTS$

Properties



Concrete Weight $\gamma_c := 150pcf$

Soil Weight $\gamma := 115pcf$

Water Weight $\gamma_w := 62.4pcf$

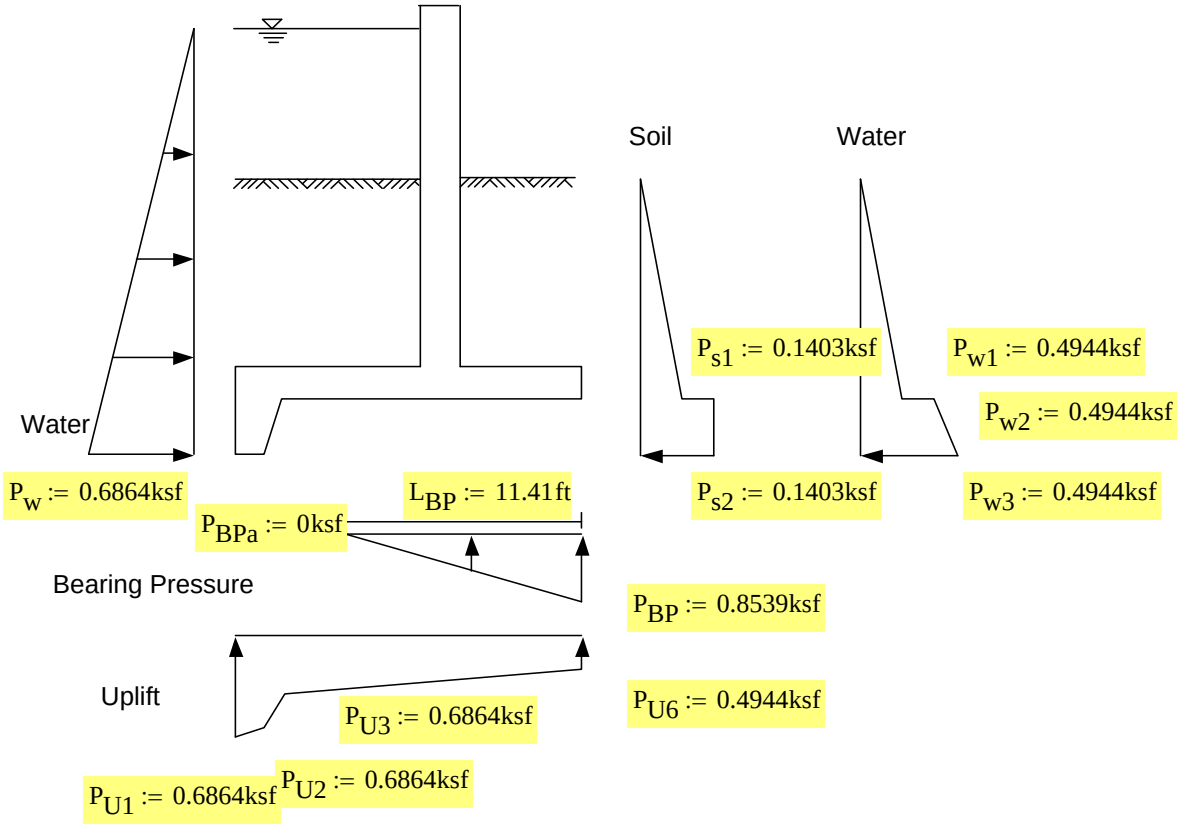
```
****************** Echoprint of Input Data ******************


Date: **/08/02                                Time: 15.43.39

JANUARY 11, 2008
CIDMO12.DAT

Company name:
  USACE
Project name:
  KANSAS CITY LEVEES FEASIBILITY STUDY PHASE 2
Project location:
  CID-MISSOURI
Wall location:
  12-FOOT WALL
Computed by: KSM

Structural geometry data:
  Elevation of top of stem (ELTS)       =    12.00 ft
  Height of stem (HTS)                  =    10.50 ft
  Thickness top of stem (TTS)           =     1.00 ft
  Thickness bottom of stem (TBS)        =     1.54 ft
  Dist. of batter at bot. of stem (TBSR)=      .54 ft
  Depth of heel (THEEL)                 =     1.50 ft
  Distance of batter for heel (BTRH)    =      .00 ft
  Depth of toe (TTOE)                   =     1.50 ft
  Width of toe (TWIDTH)                 =     3.50 ft
  Distance of batter for toe (BTRT)     =      .00 ft
  Width of base (BWIDTH)                =    11.50 ft
  Depth of key (HK)                     =      .00 ft
  Width of bottom of key (TK)           =      .00 ft
  Dist. of batter at bot. of key (BTRK) =      .00 ft

Structure coordinates:

   x (ft)    y (ft)
  ==================
       .00       .00
       .00      1.50
      6.46      1.50
      6.46     12.00
      7.46     12.00
      8.00      1.50
     11.50      1.50
     11.50       .00

NOTE: X=0 is located at the left-hand side
      of the structure.  The Y values correspond
      to the actual elevation used.

Structural property data:
  Unit weight of concrete =     .150 kcf

Driving side soil property data:

                        Moist   Saturated          Elev.
   Phi        c      Unit wt.  unit wt.  Delta   soil
  (deg)     (ksf)     (kcf)     (kcf)    (deg)   (ft)
=========================================================
   22.00     .000      .110      .115      .00    5.25

Driving side soil geometry:

  Soil      Batter    Distance
  point    (in:1ft)     (ft)
  ==============================
   1          .00    500.00
   2          .00       .00
   3          .00    500.00

Driving side soil profile:

  Soil        x          y
  point      (ft)       (ft)
  ==============================
   1     -1493.54      5.25
   2         6.46      5.25

Resisting side soil property data:

                        Moist   Saturated  Elev.
   Phi        c      Unit wt.  unit wt.  soil    Batter
  (deg)     (ksf)     (kcf)     (kcf)    (ft)   (in:1ft)
=========================================================
```

```
   22.00     .000      .110      .115     6.25     .00

Resisting side soil profile:

  Soil        x          y
  point      (ft)       (ft)
  ============================
   1         7.76      6.25
   2       507.76      6.25

Foundation property data:
  phi for soil-structure interface =    22.00 (deg)
  c for soil-structure interface   =     .000 (ksf)
  phi for soil-soil interface      =    22.00 (deg)
  c for soil-soil interface        =     .000 (ksf)

Water data:
  Driving side elevation   =    11.00 ft
  Resisting side elevation =     6.25 ft
  Unit weight of water     =    .0624 kcf
  Seepage pressures computed by Line of Creep method.

Minimum required factors of safety:
  Sliding FS   =    1.30
  Overturning  =   25.00% base in compression

Crack options:
  o  Crack *is* down to bottom of heel
  o  Computed cracks *will* be filled with water

User input failure angle data:
  Failure angle of wedge  2 =     .00 deg

Strength mobilization factor =    .6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

*Do* iterate in overturning analysis.



***** Summary of Results *****

JANUARY 11, 2008

Project name: KANSAS CITY LEVEES FEASIBILITY STUDY PHASE 2

***************    *** Satisfied ***
* Overturning *    Required base in comp. =  25.00 %
***************    Actual base in comp.   =  99.23 %
                   Overturning ratio      =   1.34

Xr (measured from toe) =   3.80 ft
Resultant ratio        =  .3308
Stem ratio             =  .3043
Base pressure at x=  11.41 ft from toe =     .0000 ksf
Base pressure at toe                   =     .8539 ksf

***********    *** Satisfied ***
* Sliding *    Min. Required =   1.30
***********    Actual FS     =   1.75



********************** Output Results **********************


Date: **/08/02                                Time: 15.43.39

JANUARY 11, 2008
CIDMO12.DAT

Company name:
  USACE
Project name:
  KANSAS CITY LEVEES FEASIBILITY STUDY PHASE 2
Project location:
  CID-MISSOURI
Wall location:
  12-FOOT WALL
Computed by: KSM

***************************
**  Overturning Results  **
```

```
***************************

Solution converged in  4 iterations.

SMF used to calculate K's =    .6667
Alpha for the SMF          =    .0000
Calculated earth pressure coefficients:
  Driving side at rest K    =    .0000
  Driving side at rest Kc   =    .0000
  Resisting side at rest K  =    .6254
  Resisting side at rest Kc =    .7908
  At-rest K's for resisting side calculated.

Depth of cracking =     5.25 ft
Crack extends to bottom of base of structure.

** Driving side pressures **

  Water pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
        11.00      .0000
          .00      .6864

** Resisting side pressures **

  Water pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
         6.25      .0000
          .00      .4944

  Earth pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
         6.25      .0000
          .00      .1403

** Uplift pressures **

  Water pressures:
     x-coord.   Pressure
       (ft)       (ksf)
    ======================
          .00      .6864
          .09      .6864
        11.50      .4944

** Forces and moments **

=========================================================================
              Part          |    Force (kips)    | Mom. Arm  |  Moment |
                            |  Vert.   |   Horiz.|    (ft)   |  (ft-k) |
=========================================================================
Structure:
  Structure weight..........    4.588                -5.16    -23.67
Structure, driving side:
  Moist soil................     .000                  .00       .00
  Saturated soil............    2.786                -8.27    -23.04
  Water above structure......     .000                  .00       .00
  Water above soil..........    2.318                -8.27    -19.17
  External vertical loads....     .000                  .00       .00
  Ext. horz. pressure loads..                .000       .00       .00
  Ext. horz. line loads......                .000       .00       .00
Structure, resisting side:
  Moist soil................     .000                  .00       .00
  Saturated soil............    1.979                -1.81     -3.58
  Water above structure......     .000                  .00       .00
  Water above soil..........     .000                  .00       .00
Driving side:
  Effective earth loads......                .000       .00       .00
  Shear (due to delta).......     .000                  .00       .00
  Horiz. surcharge effects...                .000       .00       .00
  Water loads...............                3.775      3.67     13.84
Resisting side:
  Effective earth loads......                -.439      2.08      -.91
  Water loads...............               -1.545      2.08     -3.22
Foundation:
  Vertical force on base.....   -4.872                -3.80     18.53
  Shear on base.............               -1.792       .00       .00
  Uplift....................   -6.798                -6.06     41.22
=========================================================================
** Statics Check **   SUMS =      .000       .000                   .00

Angle of base        =      .00 degrees
```

```
Normal force on base =    4.872 kips
Shear force on base  =    1.792 kips
Max. available shear force =    1.968 kips

Base pressure at x=  11.41 ft from toe =    .0000 ksf
Base pressure at toe                   =    .8539 ksf

Xr (measured from toe) =     3.80 ft
Resultant ratio        =    .3308
Stem ratio             =    .3043
Base in compression    =    99.23 %
Overturning ratio      =     1.34

Volume of concrete =     1.13 cubic yds/ft of wall

NOTE:  The engineer shall verify that the computed
bearing pressures below the wall do not exceed the
allowable foundation bearing pressure, or, perform a
bearing capacity analysis using the program CBEAR.
Also, the engineer shall verify that the base pressures
do not result in excessive differential settlement of
the wall foundation.



**********************
**  Sliding Results  **
**********************

Solution converged.  Summation of forces = 0.

           Horizontal   Vertical
  Wedge      Loads       Loads
  Number    (kips)       (kips)
  ==================================
     1        .000        .000
     2       3.775       2.318
     3        .000        .000

  Water pressures on wedges:

             Top      Bottom
    Wedge    press.   press.   x-coord.    press.
    number   (ksf)    (ksf)     (ft)       (ksf)
  ================================================
      1     .0000     .0000
      2                           .0000      .6864
      2                           .0890      .6864
      2                         11.5000      .4944
      3     .0000     .4944

Points of sliding plane:
  Point 1 (left),  x =      .00 ft,   y =      .00 ft
  Point 2 (right), x =    11.50 ft,   y =      .00 ft

Depth of cracking =     5.25 ft
Crack extends to bottom of base of structure.

          Failure   Total    Weight    Submerged   Uplift
  Wedge    angle    length  of wedge    length     force
  number   (deg)     (ft)    (kips)      (ft)      (kips)
  ========================================================
     1      .000      .000       .000     .000      .000
     2      .000    11.500      9.352   11.500     6.798
     3    38.558    10.027      2.818   10.027     2.479

  Wedge     Net force
  number     (kips)
  ===================
     1         .000
     2       -2.652
     3        2.652
  ===================
     SUM =     .000

+----------------------------+
| Factor of safety =    1.753 |
+----------------------------+
```

Loading from CTWALL Output



Assumptions

- Concrete and reinforcing strengths were not specified in the documents found. However, modifications to the CID-KS unit was under construction/design at the same time as the construction/design of the CID-MO unit. Therefore, it is a reasonable assumption to make that the same material strengths would be specified for CID-MO. The CID-KS design memorandum specifies the concrete strength and reinforcing steel properties as listed here:

Concrete Properties $f'_c := 3000 \cdot psi$

Steel Properties $F_y := 36ksi$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$

Hydraulic Load Factor $\gamma_H := 1.0$

Extreme Case Factor $\gamma_X := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.

Load Multiplication Factor EM 1110-2-2104 (3-1)

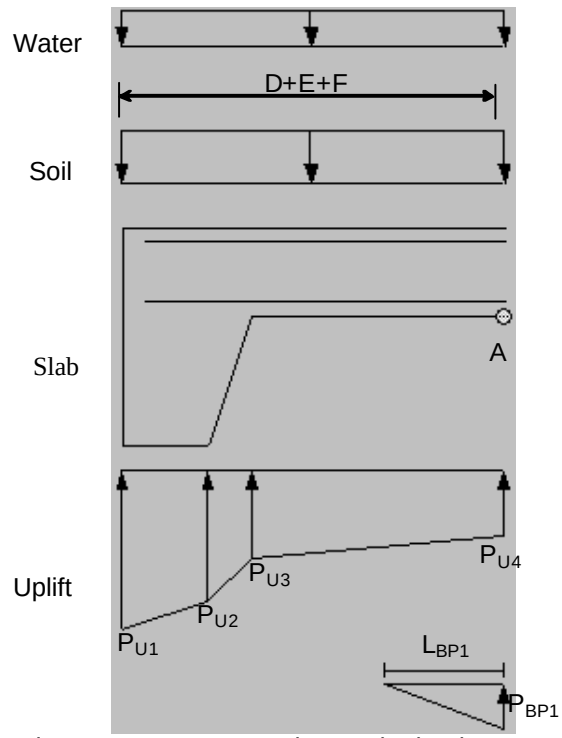
Hydraulic Factor EM 1110-2-2104 (3-2)

Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Reinforcement Checks

Location where moment is taken about.

- HEEL



$W_w := \gamma_w \cdot A \cdot (D + E + F)$

$W_w = 2317 \frac{lb}{ft}$

$W_s := \gamma \cdot B \cdot (D + E + F)$

$W_s = 2785.16 \frac{lb}{ft}$

$A_{Heel1} := 0.49 \frac{in^2}{ft}$ $C_{CH1} := 3.375in$

$d_{H1} := I - C_{CH1}$

$A_{Heel2} := 0.133 \frac{in^2}{ft}$ $C_{CH2} := 3.25in$

$d_{H2} := I - C_{CH2}$

$P_{U4} := (P_{U3} - P_{U6}) \cdot \left(\frac{G + H}{F + G + H}\right) + P_{U6}$

$P_{U4} = 578.57 \frac{lb}{ft^2}$

$L_{BP1} := if\left[\left[L_{BP} - (G + H)\right] \geq 0ft, \left[L_{BP} - (G + H)\right], 0\right]$

$L_{BP1} = 6.37\,ft$

Bearing Pressure, P_{BP1} acting on the heel at location "A"

$P_{BP1} := if\left[L_{BP} < (D + E + F + G + H), \frac{L_{BP1}}{L_{BP}} \cdot P_{BP}, P_{BPa} + \frac{L_{BP1} \cdot (P_{BP} - P_{BPa})}{L_{BP}}\right]$

$P_{BP1} = 477 \frac{lb}{ft^2}$

Slab Centroid

$$X_H := \frac{(D + E + F) \cdot I \cdot \left(\frac{D + E + F}{2}\right) + (C - I) \cdot D \cdot \left(E + F + \frac{D}{2}\right) + \left[\frac{(C - I) \cdot E}{2}\right] \cdot \left(F + \frac{2 \cdot E}{3}\right)}{(D + E + F) \cdot I + (C - I) \cdot D + \frac{(C - I) \cdot E}{2}}$$

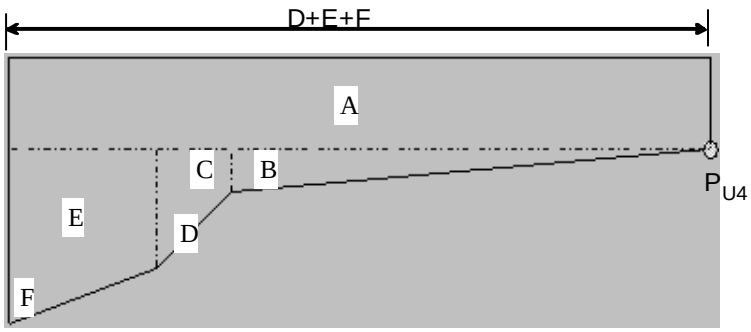
$X_H = 3.23\,ft$

- HEEL check (cont'd)

Slab Weight

$$W_H := \left[(D + E + F)\cdot I + (C - I)\cdot D + \frac{(C - I)\cdot E}{2}\right]\cdot 150pcf \qquad W_H = 1453\,\frac{lb}{ft}$$

Uplift Centroid



AREA

$$A_A := (D + E + F)\cdot P_{U4}$$

$$A_B := \frac{F\cdot(P_{U3} - P_{U4})}{2}$$

$$A_C := E\cdot(P_{U3} - P_{U4})$$

$$A_D := \frac{E\cdot(P_{U2} - P_{U3})}{2}$$

$$A_E := D\cdot(P_{U2} - P_{U4})$$

$$A_F := \frac{D\cdot(P_{U1} - P_{U2})}{2}$$

CENTROID

$$C_A := \frac{D + E + F}{2}$$

$$C_B := \frac{2\cdot F}{3}$$

$$C_C := F + \frac{E}{2}$$

$$C_D := F + \frac{2\cdot E}{3}$$

$$C_E := \left(\frac{D}{2} + E + F\right)$$

$$C_F := \left(\frac{2\cdot D}{3} + E + F\right)$$

$$X_U := \frac{A_A\cdot C_A + A_B\cdot C_B + A_C\cdot C_C + A_D\cdot C_D + A_E\cdot C_E + A_F\cdot C_F}{A_A + A_B + A_C + A_D + A_E + A_F} \qquad X_U = 3.32\,ft$$

Uplift on Heel

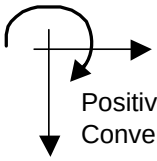
$$W_U := A_A + A_B + A_C + A_D + A_E + A_F \qquad W_U = 4085\,\frac{lb}{ft}$$

- HEEL check (cont'd)

Loading:

Bending

$$M_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[W_U \cdot X_U + \frac{P_{BP1} \cdot L_{BP1}}{2} \cdot \frac{L_{BP1}}{3} - \left[\left(W_w + W_s \right) \cdot \left(\frac{D + E + F}{2} \right) + W_H \cdot X_H \right] \right]$$



Positive Sign Convention

$$M_H = -4382.16 \cdot \frac{ft \cdot lb}{ft} \qquad M_{uH} := \left| M_H \right|$$

Note:
IF: M_H is < 0
THEN: Steel in top of heel is in tension

Shear

$$V_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left(W_w + W_s + W_H - W_U - \frac{P_{BP1} \cdot L_{BP1}}{2} \right) \qquad M_{uH} = 4.4 \cdot \frac{kip \cdot ft}{ft}$$

$$V_H = 953.17 \frac{lb}{ft} \qquad V_{uH} := \left| V_H \right| \qquad V_{uH} = 1 \cdot \frac{kip}{ft}$$

Capacity:

Flexural Capacity

$$A_{s1} := A_{Heel1} \qquad A_{s2} := A_{Heel2}$$

$$b := 12 \frac{in}{ft} \qquad b := 12 \frac{in}{ft}$$

$$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a1 = 0.58 \cdot in \qquad a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b} \qquad a2 = 0.16 \cdot in$$

$$\phi M_{H1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{H1} - \frac{a1}{2} \right) \qquad \phi M_{H2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{H2} - \frac{a2}{2} \right)$$

$$\phi M_{H1} = 21.08 \cdot \frac{kip \cdot ft}{ft} \qquad \phi M_{H2} = 5.85 \cdot \frac{kip \cdot ft}{ft}$$

$$\phi M_H := \text{if}\left(M_H > 0, \phi M_{H2}, \phi M_{H1} \right) \qquad \phi M_H = 21.08 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{H1} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{H2} \cdot \sqrt{f'_c \cdot psi}$$

$$\phi V_{c1} = 19225 \frac{lb}{ft} \qquad \phi V_{c2} = 19389 \frac{lb}{ft}$$

$$\phi V_H := \text{if}\left(M_H > 0, \phi V_{c2}, \phi V_{c1} \right) \qquad \phi V_H = 19.23 \cdot \frac{kip}{ft}$$

- HEEL check (cont'd)

Factors of Safety

Bending

$\phi M_H = 21.08 \cdot \frac{kip \cdot ft}{ft}$

$M_{uH} = 4.38 \cdot \frac{kip \cdot ft}{ft}$

$FS_1 := \frac{\phi M_H}{M_{uH}}$

$FS_1 = 4.81$

$Check1 := if\left(\phi M_H > 1.5 M_{uH}, "OKAY", "NO\ GOOD"\right)$

$Check1 = "OKAY"$

Shear

$\phi V_H = 19.23 \cdot \frac{kip}{ft}$

$V_{uH} = 0.95 \cdot \frac{kip}{ft}$

$FS_2 := \frac{\phi V_H}{V_{uH}}$

$FS_2 = 20.17$

$Check2 := if\left(\phi V_H > 1.5 V_{uH}, "OKAY", "NO\ GOOD"\right)$

$Check2 = "OKAY"$

Controlling Factor of Safety

$FS_H := \min\left(FS_1, FS_2\right)$

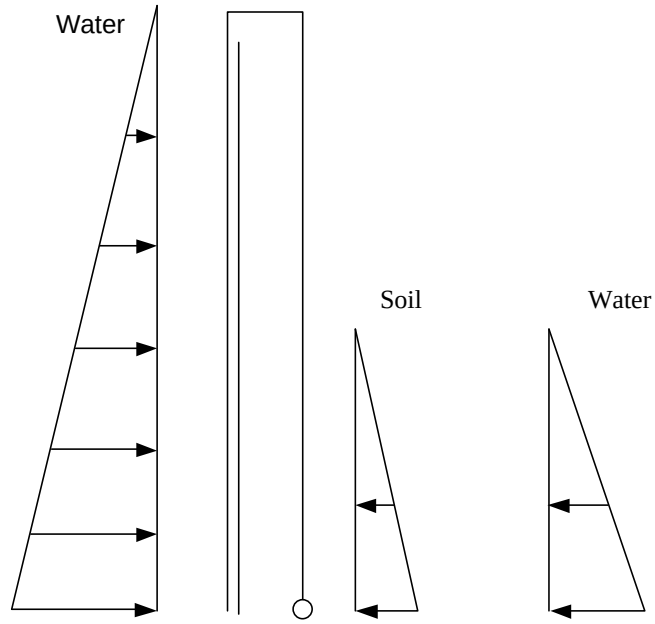
$FS_H = 4.81$

Controlling Mechanism

$Comment_H := if\left(FS_1 > FS_2, "Shear\ in\ Heel", if\left(M_H > 0, "Flexural\ Bottom\ Steel\ in\ Heel", "Flexural\ Top\ Steel\ in\ Heel"\right)\right)$

$Comment_H = "Flexural\ Top\ Steel\ in\ Heel"$

- STEM Check



Note:
P_{ss} = Soil pressure present at base of stem
P_{ws2} = Water pressure present at base of stem

$P_{ws1} := (A + B)\cdot\gamma_w$

$P_{ws1} = 593\,\frac{lb}{ft^2}$

$A_{Stem1} := 0.35\,\frac{in^2}{ft}$ $A_{Stem2} := 0.2\,\frac{in^2}{ft}$

$P_{ss} := \frac{J}{I + J}\cdot P_{s1}$ $P_{ws2} := \frac{J}{I + J}\cdot P_{w1}$

$C_{CS1} := 2.8125in$ $C_{CS2} := 2.75in$

$P_{ss} = 106.63\,\frac{lb}{ft^2}$ $P_{ws2} = 375.74\,\frac{lb}{ft^2}$

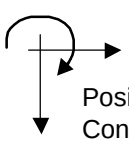
$d_{S1} := G - C_{CS1}$ $d_{S2} := G - C_{CS2}$

Loading:

Bending

$$M_S := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A + B)}{2}\cdot\frac{A + B}{3} - \frac{(P_{ss} + P_{ws2})\cdot(J)}{2}\cdot\frac{J}{3}\right]$$

Note:
IF: M_S is > 0
THEN: Riverside steel is in tension



Positive Sign Convention

$M_S = 7102.8\cdot\frac{ft\cdot lb}{ft}$ $M_{uS} := |M_S|$ $M_{uS} = 7.1\cdot\frac{kip\cdot ft}{ft}$

Shear

$$V_S := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A + B)}{2} - \frac{(P_{ss} + P_{ws2})\cdot(J)}{2}\right]$$

$V_S = 1.67\cdot\frac{kip}{ft}$ $V_{uS} := |V_S|$ $V_{uS} = 1.7\cdot\frac{kip}{ft}$

- STEM Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Stem1}$ $\qquad$ $A_{s2} := A_{Stem2}$

$b := 12\frac{in}{ft}$ $\qquad$ $b := 12\frac{in}{ft}$

$a1 := \frac{A_{s1}\cdot F_y}{0.85f'_c\cdot b}$ $\qquad a1 = 0.41\cdot in$ $\qquad$ $a2 := \frac{A_{s2}\cdot F_y}{0.85f'_c\cdot b}$ $\qquad a2 = 0.24\cdot in$

$\phi M_{S1} := \phi_B A_{s1}\cdot F_y\cdot\left(d_{S1} - \frac{a1}{2}\right)$ $\qquad$ $\phi M_{S2} := \phi_B A_{s2}\cdot F_y\cdot\left(d_{S2} - \frac{a2}{2}\right)$

$\phi M_{S1} = 16.26\cdot\frac{kip\cdot ft}{ft}$ $\qquad$ $\phi M_{S2} = 9.38\cdot\frac{kip\cdot ft}{ft}$

$\phi M_S := if\left(M_S > 0, \phi M_{S1}, \phi M_{S2}\right)$ $\qquad$ $\phi M_S = 16.26\cdot\frac{kip\cdot ft}{ft}$

Shear Capacity

$\phi V_{c1} := \phi_V\cdot 2\cdot b\cdot d_{S1}\cdot\sqrt{f'_c\cdot psi}$ $\qquad$ $\phi V_{c2} := \phi_V\cdot 2\cdot b\cdot d_{S2}\cdot\sqrt{f'_c\cdot psi}$

$\phi V_{c1} = 20622\frac{lb}{ft}$ $\qquad$ $\phi V_{c2} = 20704\frac{lb}{ft}$

$\phi V_S := if\left(M_S > 0, \phi V_{c1}, \phi V_{c2}\right)$ $\qquad$ $\phi V_S = 20.62\cdot\frac{kip}{ft}$

Factors of Safety

Bending

$\phi M_S = 16.26\cdot\frac{kip\cdot ft}{ft}$ $\qquad$ $FS_3 := \frac{\phi M_S}{M_{uS}}$

$M_{uS} = 7.1\cdot\frac{kip\cdot ft}{ft}$ $\qquad$ $FS_3 = 2.29$

$Check3 := if\left(\phi M_S > 1.5M_{uS}, "OKAY", "NO GOOD"\right)$ $\qquad$ $Check3 = "OKAY"$

Shear

$\phi V_S = 20.62\cdot\frac{kip}{ft}$ $\qquad$ $FS_4 := \frac{\phi V_S}{V_{uS}}$

$V_{uS} = 1.67\cdot\frac{kip}{ft}$ $\qquad$ $FS_4 = 12.35$

$Check4 := if\left(\phi V_S > 1.5V_{uS}, "OKAY", "NO GOOD"\right)$ $\qquad$ $Check4 = "OKAY"$

Controlling Factor of Safety

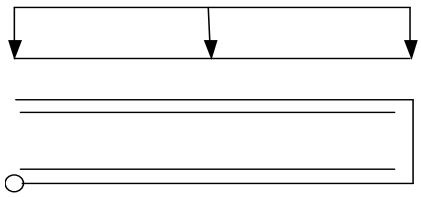
$FS_S := \min\left(FS_3, FS_4\right)$ $\qquad$ $FS_S = 2.29$

Controlling Mechanism

- TOE Check

Soil

Slab



$W_s := \gamma \cdot H \cdot (J)$ $\quad W_s = 1911.87 \frac{lb}{ft}$

$A_{Toe1} := 0.29 \frac{in^2}{ft}$ $\quad C_{CT1} := 3.375 in$

$d_{T1} := I - C_{CT1}$

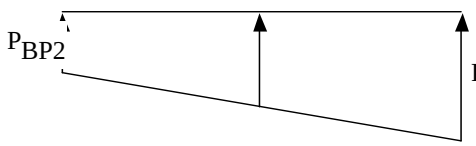
$A_{Toe2} := 0.133 \frac{in^2}{ft}$ $\quad C_{CT2} := 3.25 in$ $\quad d_{T1} = 14.63 \cdot in$

$L_{BP2} := if(H \le L_{BP}, H, L_{BP})$

$L_{BP2} = 3.5 ft$

$d_{T2} := I - C_{CT2}$

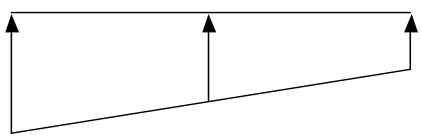
Bearing Pressure



$P_{BP} = 853.9 \frac{lb}{ft^2}$ $\quad d_{T2} = 14.75 \cdot in$

$$P_{BP2} := if\left[L_{BP} < (D + E + F + G + H), P_{BP} - P_{BP} \cdot \frac{L_{BP2}}{L_{BP}}, P_{BP} - L_{BP2} \cdot \left(\frac{P_{BP} - P_{BPa}}{L_{BP}}\right)\right] = 591.97 \frac{lb}{ft^2}$$

Uplift



$P_{U6} = 494.4 \frac{lb}{ft^2}$

$$P_{U5} := \left[\left[(P_{U3} - P_{U6}) \cdot \frac{H}{F + G + H} + P_{U6}\right]\right]$$

$$P_{U5} = 552.83 \frac{lb}{ft^2}$$

Bearing Pressure

$$L_{bp} := \frac{(L_{BP2} \cdot P_{BP2}) \cdot \left(\frac{L_{BP2}}{2}\right) + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2} \cdot \left(\frac{2L_{BP}}{3}\right)}{L_{BP2} \cdot P_{BP2} + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2}} \qquad L_{bp} = 2.81 ft$$

$$W_{bp} := L_{BP2} \cdot P_{BP2} + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2} \qquad W_{bp} = 2530 \frac{lb}{ft}$$

Uplift

$$L_u := \frac{H \cdot P_{U6} \cdot \frac{H}{2} + \frac{(P_{U5} - P_{U6}) \cdot H}{2} \cdot \frac{H}{3}}{H \cdot P_{U6} + \frac{(P_{U5} - P_{U6}) \cdot H}{2}} \qquad L_u = 1.72 ft$$

$$W_u := H \cdot P_{U6} + \frac{(P_{U5} - P_{U6}) \cdot H}{2} \qquad W_u = 1832.66 \frac{lb}{ft}$$

Toe Check (continued)

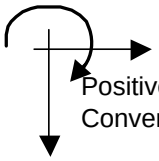
Loading:

Bending

$$M_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[\left[W_s + (H \cdot I) \cdot \gamma_c \right] \cdot \frac{H}{2} - \left(W_{bp} \cdot L_{bp} + W_u \cdot L_u \right) \right]$$

Note:
IF: $M_T < 0$
THEN: Bottom steel is in tension

$$M_T = -5.5 \cdot \frac{kip \cdot ft}{ft} \qquad M_{uT} := \left| M_T \right| \qquad M_{uT} = 5.54 \cdot \frac{kip \cdot ft}{ft}$$



Positive Sign Convention

Shear

$$V_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[W_s + (H \cdot I) \cdot \gamma_c - W_{bp} - W_u \right]$$

$$V_T = -1.66 \cdot \frac{kip}{ft} \qquad V_{uT} := \left| V_T \right| \qquad V_{uT} = 1.66 \cdot \frac{kip}{ft}$$

Capacity:

Flexural Capacity

$$A_{s1} := A_{Toe1} \qquad A_{s2} := A_{Toe2}$$

$$b := 12 \frac{in}{ft} \qquad b := 12 \frac{in}{ft}$$

$$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a1 = 0.34 \cdot in \qquad a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b} \qquad a2 = 0.16 \cdot in$$

$$\phi M_{T1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{T1} - \frac{a1}{2} \right) \qquad \phi M_{T2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{T2} - \frac{a2}{2} \right)$$

$$\phi M_{T1} = 12.58 \cdot \frac{kip \cdot ft}{ft} \qquad \phi M_{T2} = 5.85 \cdot \frac{kip \cdot ft}{ft}$$

$$\phi M_T := \text{if}\left(M_T > 0, \phi M_{T1}, \phi M_{T2} \right) \qquad \phi M_T = 5.85 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{T1} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{T2} \cdot \sqrt{f'_c \cdot psi}$$

$$\phi V_{c1} = 19225 \frac{lb}{ft} \qquad \phi V_{c2} = 19389 \frac{lb}{ft}$$

$$\phi V_T := \text{if}\left(M_{uT} > 0, \phi V_{c1}, \phi V_{c2} \right) \qquad \phi V_T = 19.23 \cdot \frac{kip}{ft}$$

Toe Check (continued)

Factors of Safety

Bending

$\phi M_T = 5.85 \cdot \frac{kip \cdot ft}{ft}$

$M_{uT} = 5.54 \cdot \frac{kip \cdot ft}{ft}$

$FS_5 := \frac{\phi M_T}{M_{uT}}$

$FS_5 = 1.06$

$Check5 := if\left(\phi M_T > 1.5 \cdot M_{uT}, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$Check5 = \text{"NO GOOD"}$

Shear

$\phi V_T = 19.23 \cdot \frac{kip}{ft}$

$V_{uT} = 1.66 \cdot \frac{kip}{ft}$

$FS_6 := \frac{\phi V_T}{V_{uT}}$

$FS_6 = 11.56$

$Check6 := if\left(\phi V_T > 1.5 V_{uT}, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$Check6 = \text{"OKAY"}$

Controlling Factor of Safety

$FS_T := \min\left(FS_5, FS_6\right)$

$FS_T = 1.06$

Controlling Mechanism

$Comment_T := if\left(FS_5 > FS_6, \text{"Shear in Heel"}, if\left(M_T > 0, \text{"Flexural Top Steel in Toe"}, \text{"Flexural Bottom Steel in Toe"}\right)\right)$

$Comment_T = \text{"Flexural Bottom Steel in Toe"}$

Overall Factor of Safety (without considering multiple layers of steel)

Factor of Safety

$$FoS := \min(FS_H, FS_S, FS_T)$$

Limiting Mechanism

$$Mechanism := Comment_H$$

$$Mechanism := \text{if}(FoS = FS_S, Comment_S, Mechanism)$$

$$Mechanism := \text{if}(FoS = FS_T, Comment_T, Mechanism)$$

$$FoS = 1.06$$

$$Mechanism = \text{"Flexural Bottom Steel in Toe"}$$

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis. If the factor of safety is lower than 1.5 a probablility analysis is required.

US Army Corps of Engineers

Probability of Failure

CIDMO 12ft floodwall
spread footing
Bending in Toe

Comp by:KSM 8-2-11
Chkd by:

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. Situation

1. This structure does not meet the strength 1.5 factor of safety for which it has been determined 99.8% reliability can be assigned. See mathcad analysis of the wall for existing condition strength check.
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength
3. Material Properties used:

   Mean Concrete Strength, $f'_{cM} := 3750 \cdot psi$

   Mean Steel Strength, $F_{yM} := 45 \cdot ksi$

4. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete Grade 40 is 14%.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength
FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength
FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength
FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength
ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength
ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. *Calculating Factors of Safety*

WATER AT TOP OF WALL

Condition under consideration from strength check: Mu for toe (from mathcad strength analysis).

$M_u := 6.75 \frac{kip \cdot ft}{ft}$

Design Concrete Strength

$\phi_B := 1.0$ Strength Reduction Factors not used in Risk and Uncertainty Analysis

$A_s := 0.133 \frac{in^2}{ft}$ Area of Steel

$dist_{CL} := 3.25in$ Distance from bottom of footing to centerline of reinforcement

$T_{toe} := 1.5ft$ Thickness of footing at toe/stem interface

$b := 12 \frac{in}{ft}$ 1 ft strip of wall analyzed

$d := T_{toe} - dist_{CL}$ $\quad d = 14.75 \cdot in$

Mean Concrete Strength and Steel Yield Strength

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cM} \cdot b}$ $\quad a = 0.156 \cdot in$

$\phi M_M := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_M = 7.32 \cdot \frac{kip \cdot ft}{ft}$

$FS_D := \frac{\phi M_M}{M_u}$ $\quad FS_D = 1.084$

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM} \cdot 0.14$ $\quad f'_{cU} = 4275 \cdot psi$

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cU} \cdot b}$ $\quad a = 0.137 \cdot in$

$\phi M_{cU} := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{cU} = 7.32 \cdot \frac{kip \cdot ft}{ft}$

$FS_{fcu} := \frac{\phi M_{cU}}{M_u}$ $\quad FS_{fcu} = 1.085$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\quad f'_{cL} = 3225 \cdot psi$

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cL} \cdot b}$ $\quad a = 0.182 \cdot in$

$\phi M_{cL} := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{cL} = 7.31 \cdot \frac{kip \cdot ft}{ft}$

$FS_{fcl} := \frac{\phi M_{cL}}{M_u}$ $\quad FS_{fcl} = 1.083$

Upper Steel Yield Strength

$F_{yU} := F_{yM} + F_{yM} \cdot 0.14$ $\quad F_{yU} = 51.3 \cdot ksi$

$a := \frac{A_s \cdot F_{yU}}{0.85 f'_{cM} \cdot b}$ $\quad a = 0.178 \cdot in$

$\phi M_{sU} := \phi_B A_s \cdot F_{yU} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{sU} = 8.34 \cdot \frac{kip \cdot ft}{ft}$

$FS_{Fyu} := \frac{\phi M_{sU}}{M_u}$ $\quad FS_{Fyu} = 1.235$

Lower Steel Yield Strength

$F_{yL} := F_{yM} - F_{yM} \cdot 0.14$ $\quad F_{yL} = 38.7 \cdot ksi$

$a := \frac{A_s \cdot F_{yL}}{0.85 f'_{cM} \cdot b}$ $\quad a = 0.135 \cdot in$

$\phi M_{sL} := \phi_B A_s \cdot F_{yL} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{sL} = 6.3 \cdot \frac{kip \cdot ft}{ft}$

$FS_{Fyl} := \frac{\phi M_{sL}}{M_u}$ $\quad FS_{Fyl} = 0.933$

$FS_D = 1.084$ $\quad FS_{fcu} = 1.085$ $\quad FS_{Fyu} = 1.235$

$FS_{fcl} = 1.083$ $\quad FS_{Fyl} = 0.933$

VI. Probability of Failure Calculation

$\Delta F_{Fy} := FS_{Fyu} - FS_{Fyl}$ $\qquad \Delta F_{Fy} = 0.302$

$\Delta F_{fc} := FS_{fcu} - FS_{fcl}$ $\qquad \Delta F_{fc} = 1.651 \times 10^{-3}$

ACI EQ (11-4)

$$\sigma_F := \sqrt{\left(\frac{\Delta F_{Fy}}{2}\right)^2 + \left(\frac{\Delta F_{fc}}{2}\right)^2} \qquad \sigma_F = 0.151$$

$$V_F := \frac{\sigma_F}{FS_D} \qquad V_F = 0.139$$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}} \qquad \beta_{LN} = 0.513$$

$R := \mathrm{cnorm}(\beta_{LN})$ $\qquad R = 69.61 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\qquad P_F = 30.39 \cdot \%$

WATER AT 1ft down from top of WALL

Condition under consideration from strength check: Mu for toe (from mathcad strength analysis).

$M_u := 5.54 \frac{kip \cdot ft}{ft}$

Design Concrete Strength

$\phi_B := 1.0$ Strength Reduction Factors not used in Risk and Uncertainty Analysis

$A_s := 0.133 \frac{in^2}{ft}$ Area of Steel

$dist_{CL} := 3.25in$ Distance from bottom of footing to centerline of reinforcement

$T_{toe} := 1.5ft$ Thickness of footing at toe/stem interface

$b := 12 \frac{in}{ft}$ 1 ft strip of wall analyzed

$d := T_{toe} - dist_{CL}$ $\quad d = 14.75 \cdot in$

Mean Concrete Strength and Steel Yield Strength

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cM} \cdot b}$ $\quad a = 0.156 \cdot in$

$\phi M_M := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_M = 7.32 \cdot \frac{kip \cdot ft}{ft}$

$FS_D := \frac{\phi M_M}{M_u}$ $\quad FS_D = 1.321$

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM} \cdot 0.14$ $\quad f'_{cU} = 4275 \cdot psi$

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cU} \cdot b}$ $\quad a = 0.137 \cdot in$

$\phi M_{cU} := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{cU} = 7.32 \cdot \frac{kip \cdot ft}{ft}$

$FS_{fcu} := \frac{\phi M_{cU}}{M_u}$ $\quad FS_{fcu} = 1.322$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\quad f'_{cL} = 3225 \cdot psi$

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cL} \cdot b}$ $\quad a = 0.182 \cdot in$

$\phi M_{cL} := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{cL} = 7.31 \cdot \frac{kip \cdot ft}{ft}$

$FS_{fcl} := \frac{\phi M_{cL}}{M_u}$ $\quad FS_{fcl} = 1.32$

Upper Steel Yield Strength

$F_{yU} := F_{yM} + F_{yM} \cdot 0.14$ $\quad F_{yU} = 51.3 \cdot ksi$

$a := \frac{A_s \cdot F_{yU}}{0.85 f'_{cM} \cdot b}$ $\quad a = 0.178 \cdot in$

$\phi M_{sU} := \phi_B A_s \cdot F_{yU} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{sU} = 8.34 \cdot \frac{kip \cdot ft}{ft}$

$FS_{Fyu} := \frac{\phi M_{sU}}{M_u}$ $\quad FS_{Fyu} = 1.505$

Lower Steel Yield Strength

$F_{yL} := F_{yM} - F_{yM} \cdot 0.14$ $\quad F_{yL} = 38.7 \cdot ksi$

$a := \frac{A_s \cdot F_{yL}}{0.85 f'_{cM} \cdot b}$ $\quad a = 0.135 \cdot in$

$\phi M_{sL} := \phi_B A_s \cdot F_{yL} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{sL} = 6.3 \cdot \frac{kip \cdot ft}{ft}$

$FS_{Fyl} := \frac{\phi M_{sL}}{M_u}$ $\quad FS_{Fyl} = 1.137$

$FS_D = 1.321$ $\quad FS_{fcu} = 1.322$ $\quad FS_{Fyu} = 1.505$

$FS_{fcl} = 1.32$ $\quad FS_{Fyl} = 1.137$

VI. Probability of Failure Calculation

$\Delta F_{Fy} := FS_{Fyu} - FS_{Fyl}$ $\qquad \Delta F_{Fy} = 0.368$

$\Delta F_{fc} := FS_{fcu} - FS_{fcl}$ $\qquad \Delta F_{fc} = 2.012 \times 10^{-3}$

ACI EQ (11-4)

$$\sigma_F := \sqrt{\left(\frac{\Delta F_{Fy}}{2}\right)^2 + \left(\frac{\Delta F_{fc}}{2}\right)^2} \qquad \sigma_F = 0.184$$

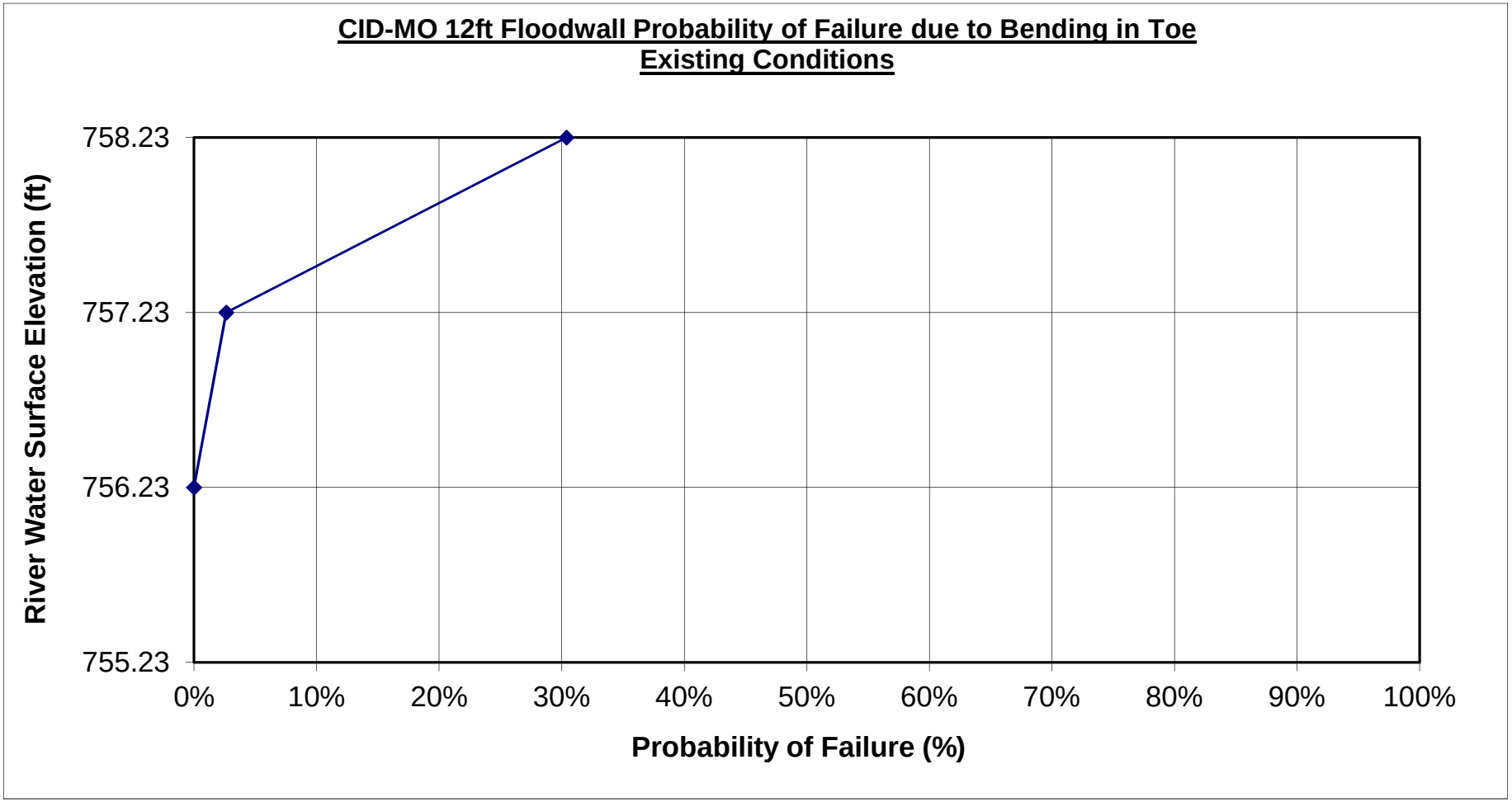
$$V_F := \frac{\sigma_F}{FS_D} \qquad V_F = 0.139$$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}} \qquad \beta_{LN} = 1.939$$

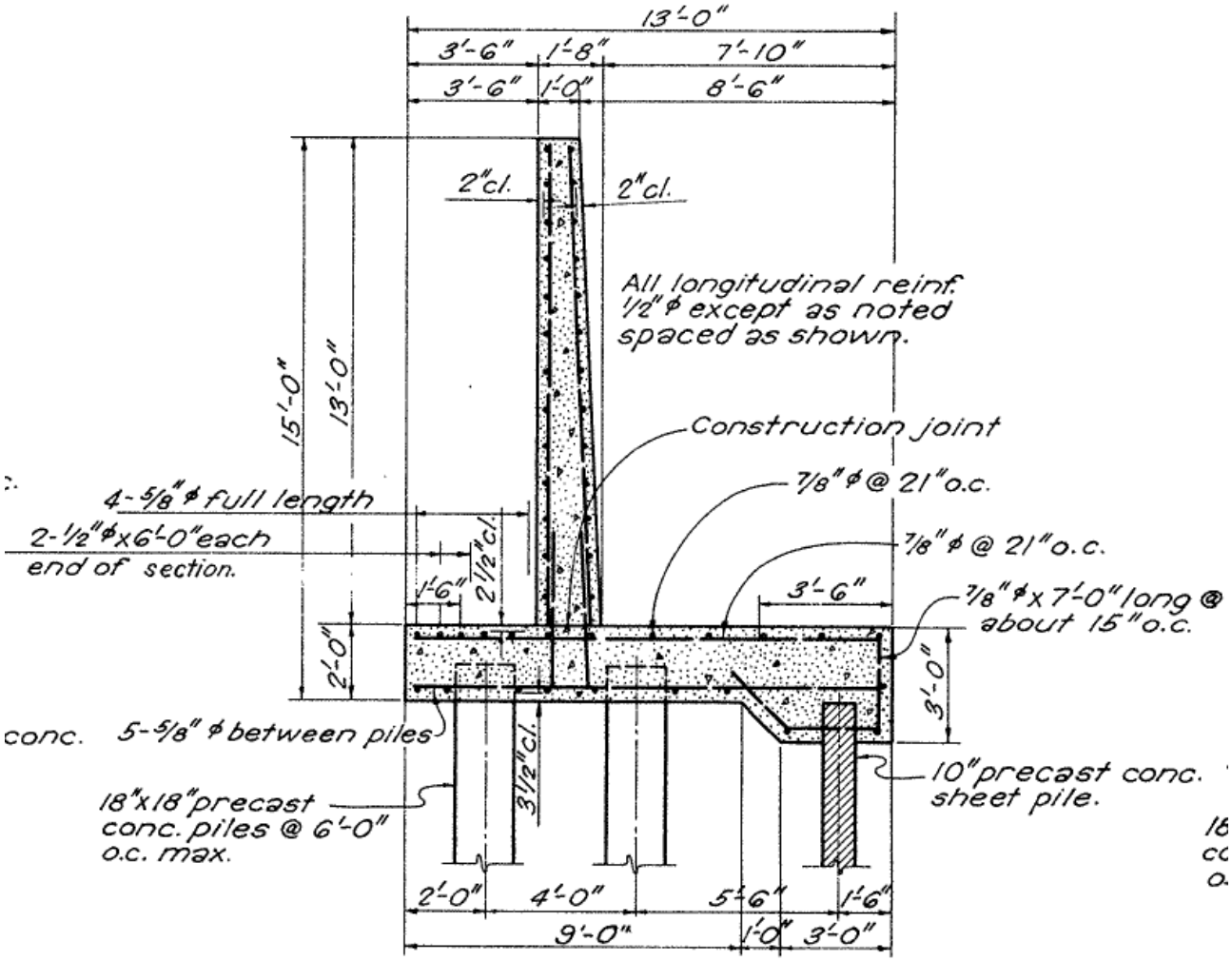
$R := \mathrm{cnorm}(\beta_{LN})$ $\qquad R = 97.37 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\qquad P_F = 2.63 \cdot \%$


CID-MO 12ft Floodwall Probability of Failure due to Bending in Toe
Existing Conditions
River Water Surface Elevation (ft)
758.23
757.23
756.23
755.23
0%
10%
20%
30%
40%
50%
60%
70%
80%
90%
100%
Probability of Failure (%)

General Structural Exhibit 2:
Pile Foundation Floodwall Sample Calculations



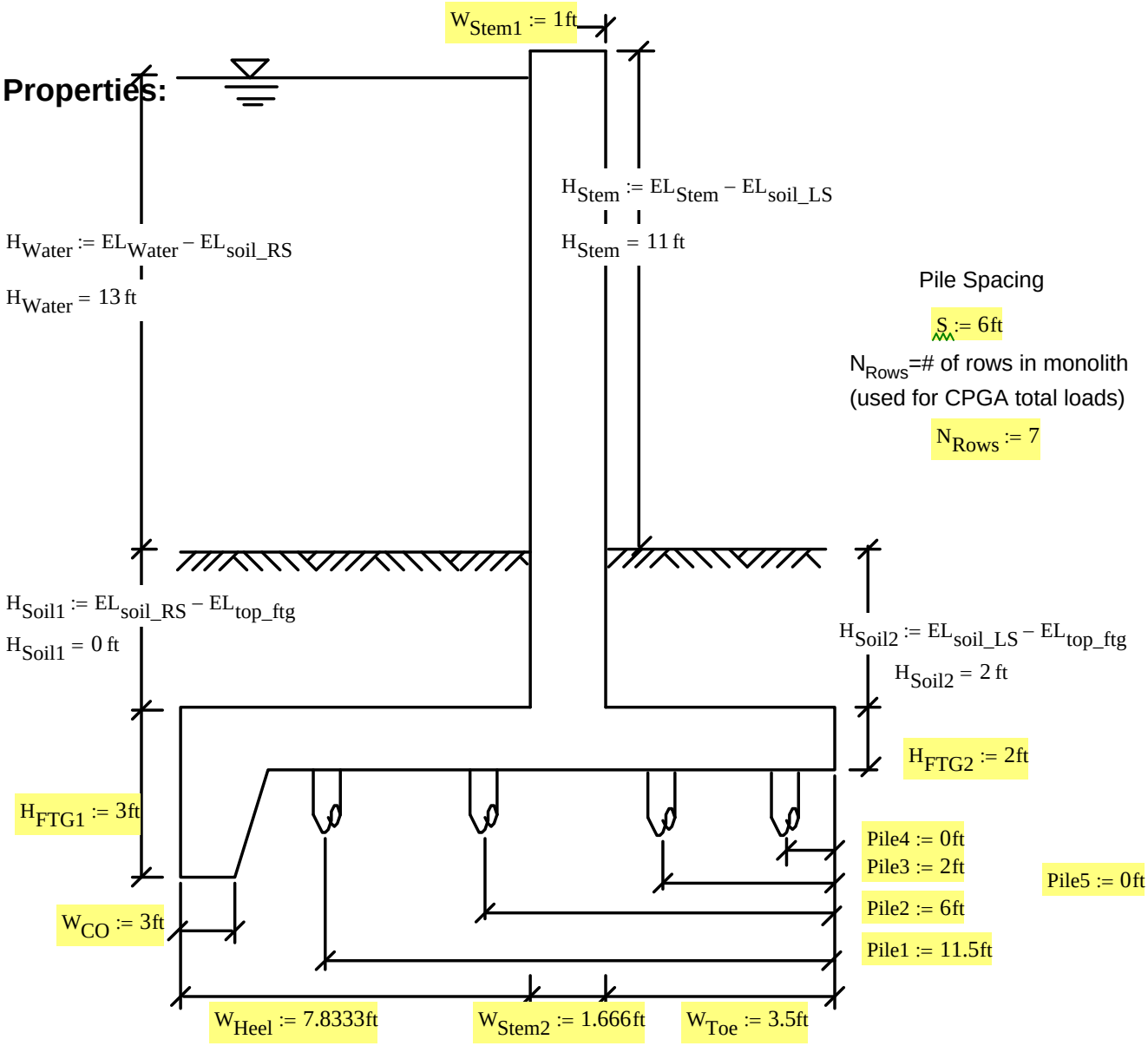
Typical Section from 1946 Record Drawings

Variables

$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $ksf \equiv \frac{1000lb}{ft^2}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$

$EL_{Stem} := 100ft$ $EL_{Water} := 100ft$ $EL_{soil_RS} := 87ft$ $EL_{soil_LS} := 89ft$ $EL_{top_ftg} := 87ft$

Properties:



Concrete Properties $\gamma_c := 150pcf$ Soil Properties $\gamma := 115pcf$ $\phi := 22deg$

Water Weight $\gamma_w := 62.4pcf$ $K_u := 1 - \sin(\phi)$ $K_u = 0.63$

Lengths:

$L_{CO} := H_{FTG1} - H_{FTG2}$ $\quad L_{W1} := H_{Water} + H_{Soil1} + H_{FTG1}$ $\quad L_{STM} := H_{Stem} + H_{Soil2}$

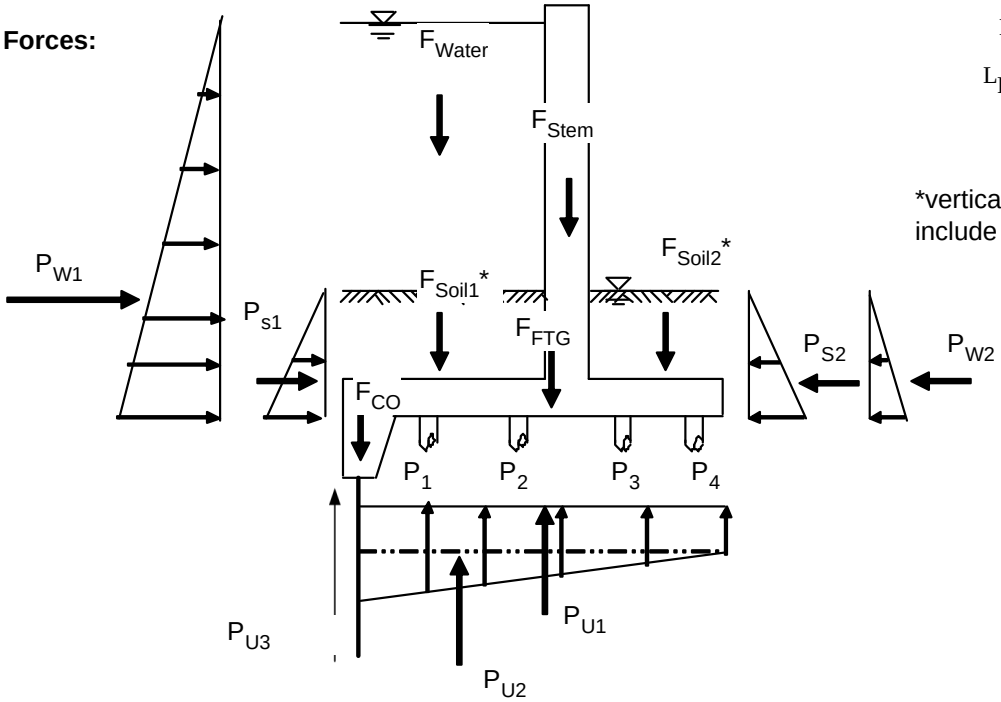
$L_{FTG} := W_{Heel} + W_{Stem2} + W_{Toe}$ $\quad L_{PU1} := H_{Soil2} + H_{FTG2}$

Forces:

$L_{W1} = 16\,ft$

$L_{PU1} = 4\,ft$

*vertical soil loads include water



$F_{Stem} := \left(\frac{W_{Stem1} + W_{Stem2}}{2}\right) \cdot L_{STM} \cdot \gamma_c$ $\quad F_{Stem} = 2599.35 \frac{lb}{ft}$

$F_{FTG} := H_{FTG2} \cdot L_{FTG} \cdot \gamma_c$ $\quad F_{FTG} = 3899.79 \frac{lb}{ft}$

$F_{CO} := L_{CO} \cdot W_{CO} \cdot \gamma_c$ $\quad F_{CO} = 450 \frac{lb}{ft}$

$F_{Soil1} := H_{Soil1} \cdot W_{Heel} \cdot \gamma$ $\quad F_{Soil1} = 0 \frac{lb}{ft}$

$F_{Soil2} := H_{Soil2} \cdot W_{Toe} \cdot \gamma$ $\quad F_{Soil2} = 805 \frac{lb}{ft}$

$F_{Water1} := H_{Water} \cdot W_{Heel} \cdot \gamma_w$ $\quad F_{Water1} = 6354.37 \frac{lb}{ft}$

$P_{W1} := \frac{1}{2} \cdot L_{W1}^2 \cdot \gamma_w$ $\quad P_{W1} = 7987.2 \frac{lb}{ft}$

$P_{W2} := -\left[\frac{1}{2} \cdot \left(H_{FTG1} + H_{Soil2}\right)^2 \cdot \gamma_w\right]$ $\quad P_{W2} = -780 \frac{lb}{ft}$

$P_{S1} := K_u \cdot \left[\frac{1}{2} \cdot \left(H_{Soil1} + H_{FTG1}\right)^2 \cdot \left(\gamma - \gamma_w\right)\right]$ $\quad P_{S1} = 148.03 \frac{lb}{ft}$

$P_{S2} := -K_u \cdot \left[\frac{1}{2} \cdot \left(H_{Soil2} + H_{FTG1}\right)^2 \cdot \left(\gamma - \gamma_w\right)\right]$ $\quad P_{S2} = -411.2 \frac{lb}{ft}$

Forces Con't

$p_{U3} := \left(L_{W1}\right)\cdot\gamma_w = 998.4\,\frac{lb}{ft^2}$ <---UA

$P_{U3} := \frac{-W_{CO}}{2}\cdot p_{U3} = -1497.6\,\frac{lb}{ft}$

$p_{U2} := -\left[L_{W1}\cdot\gamma_w - 0.5\cdot\left(L_{W1}\cdot\gamma_w - \gamma_w\cdot L_{PU1}\right)\right] = -624\,\frac{lb}{ft^2}$ <---UA'

$p_{U1} := -\left[\gamma_w\cdot L_{PU1} + \frac{L_{PU1}}{L_{FTG} - \frac{W_{CO}}{2} + L_{PU1}}\left(\left|p_{U2}\right| - \gamma_w\cdot L_{PU1}\right)\right] = -346.22\,\frac{lb}{ft^2}$ <---UB

$P_{U1} := p_{U1}\left(L_{FTG} - \frac{W_{CO}}{2}\right) = -3981.33\,\frac{lb}{ft}$

$P_{U2i} := p_{U2} - p_{U1} = -277.78\,\frac{lb}{ft^2}$

$P_{U2} := 0.5\cdot\left(P_{U2i}\right)\cdot\left(L_{FTG} - \frac{W_{CO}}{2}\right) = -1597.12\,\frac{lb}{ft}$

$P_{U2} = -1597.12\,\frac{lb}{ft}$

$P_{U1} = -3981.33\,\frac{lb}{ft}$

$P_{U3} = -1497.6\,\frac{lb}{ft}$

Inputs for CPGA

Sum moments around bottom corner of Toe:

Moment Arm
from bottom corner of Toe

$$Arm_{Stem} := -\left(W_{Toe} + \frac{W_{Stem2}}{2}\right)$$

$$Arm_{FTG} := -\frac{L_{FTG}}{2}$$

$$Arm_{CO} := -L_{FTG} + \frac{W_{CO}}{2}$$

$$Arm_{Soil1} := -\left(W_{Toe} + W_{Stem2} + \frac{W_{Heel}}{2}\right)$$

$$Arm_{Soil2} := -\frac{W_{Toe}}{2}$$

$$Arm_{Water1} := Arm_{Soil1}$$

$$Arm_{W1} := \frac{1}{3}\cdot L_{W1} - L_{CO}$$

$$Arm_{W2} := \frac{1}{3}\cdot\left(L_{PU1} + L_{CO}\right) - L_{CO}$$

$$Arm_{S1} := \frac{1}{3}\cdot\left(H_{Soil1} + H_{FTG1}\right) - L_{CO}$$

$$Arm_{S2} := Arm_{W2}$$

$$Arm_{U1} := \frac{L_{FTG} - \frac{W_{CO}}{2}}{2}$$

$$Arm_{U2} := \frac{2\left(L_{FTG} - \frac{W_{CO}}{2}\right)}{3}$$

$$Arm_{U3} := -\left(L_{FTG} - \frac{W_{CO}}{4}\right)$$

$Arm_{Stem} = -4.33\ ft$

$Arm_{FTG} = -6.5\ ft$

$Arm_{CO} = -11.5\ ft$

$Arm_{Soil1} = -9.08\ ft$

$Arm_{Soil2} = -1.75\ ft$

$Arm_{Water1} = -9.08\ ft$

$Arm_{W1} = 4.33\ ft$

$Arm_{W2} = 0.67\ ft$

$Arm_{S1} = 0\ ft$

$Arm_{S2} = 0.67\ ft$

$Arm_{U1} = -5.75\ ft$

$Arm_{U2} = -7.67\ ft$

$Arm_{U3} = -12.25\ ft$

Moment (about bottom corner of toe)

$M_{Stem} := F_{Stem}\cdot Arm_{Stem}$ $\quad M_{Stem} = -11.26\cdot\frac{kip\cdot ft}{ft}$

$M_{FTG} := F_{FTG}\cdot Arm_{FTG}$ $\quad M_{FTG} = -25.35\cdot\frac{kip\cdot ft}{ft}$

$M_{CO} := F_{CO}\cdot Arm_{CO}$ $\quad M_{CO} = -5.17\cdot\frac{kip\cdot ft}{ft}$

$M_{Soil1} := F_{Soil1}\cdot Arm_{Soil1}$ $\quad M_{Soil1} = 0\cdot\frac{kip\cdot ft}{ft}$

$M_{Soil2} := F_{Soil2}\cdot Arm_{Soil2}$ $\quad M_{Soil2} = -1.41\cdot\frac{kip\cdot ft}{ft}$

$M_{Water1} := F_{Water1}\cdot Arm_{Water1}$ $\quad M_{Water1} = -57.71\cdot\frac{kip\cdot ft}{ft}$

$M_{W1} := P_{W1}\cdot Arm_{W1}$ $\quad M_{W1} = 34.61\cdot\frac{kip\cdot ft}{ft}$

$M_{W2} := P_{W2}\cdot Arm_{W2}$ $\quad M_{W2} = -0.52\cdot\frac{kip\cdot ft}{ft}$

$M_{S1} := P_{S1}\cdot Arm_{S1}$ $\quad M_{S1} = 0\cdot\frac{kip\cdot ft}{ft}$

$M_{S2} := P_{S2}\cdot Arm_{S2}$ $\quad M_{S2} = -0.27\cdot\frac{kip\cdot ft}{ft}$

$M_{U1} := P_{U1}\cdot Arm_{U1}$ $\quad M_{U1} = 22.89\cdot\frac{kip\cdot ft}{ft}$

$M_{U2} := P_{U2}\cdot Arm_{U2}$ $\quad M_{U2} = 12.24\cdot\frac{kip\cdot ft}{ft}$

$M_{U3} := P_{U3}\cdot Arm_{U3}$ $\quad M_{U3} = 18.34\cdot\frac{kip\cdot ft}{ft}$

$$\Sigma M := \left(M_{Stem} + M_{FTG} + M_{CO} + M_{Soil1} + M_{Soil2} + M_{Water1} + M_{W1} + M_{W2} + M_{S1} + M_{S2} + M_{U1} + M_{U2} + M_{U3}\right)$$

$\Sigma M = -13.61\cdot\frac{ft\cdot kip}{ft}$

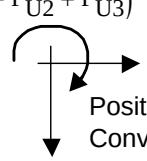
$$\Sigma Vertical := \left(F_{Stem} + F_{FTG} + F_{CO} + F_{Soil1} + F_{Soil2} + F_{Water1} + P_{U1} + P_{U2} + P_{U3}\right)$$

$\Sigma Vertical = 7.03\cdot\frac{kip}{ft}$

$$\Sigma Horizontal := \left(P_{W1} + P_{W2} + P_{S1} + P_{S2}\right)$$

$\Sigma Horizontal = 6.94\cdot\frac{kip}{ft}$

Loading about the stem.



Positive Sign Convention

Find Pile Loads Using CPGA:

$Number := if(Pile5 = 0, 4, 5)$

$Number := if(Pile4 = 0, 3, Number)$

$Number := if(Pile3 = 0, 2, Number)$

$Number := if(Pile2 = 0, 1, Number)$

$Number := if(Pile1 = 0, 0, Number)$

Total number of piles:

$Number = 3$

Concrete Pile Properties

PILES 1 THRU 14 IN CPGA

Dimension of square pile, $D := 15in$

Assume all piles to pile cap connections are fixed (details show rebar extending into base).

Note - the pile tapers from 18" to 12" over 21feet. Used a dimension at half way down the pile for analysis purposes.

Pile Properties:

$fc := 4000$ psi $\quad E_C := 57 \cdot \sqrt{fc} \quad E_C = 3605$ ksi

$I := \frac{D}{12} \quad I = 4218.75 \cdot in^4 \quad Area := D^2 \quad Area = 225 \cdot in^2$

C33=2.0

B66 = 0 torsional stiffness

Length = 20ft

Design Load Strengths:

AC = Allowable Pile Compression Load, = 37.17 k

AT = Allowable Pile Tension Load, =20.25 k

The following design strengths are from the existing pile capacity MathCAD file:

PO = Axial Compression Design Load Strength, 752 k

PT = Axial Tension Design Load Strength, Fy*As= 192 k

PB = Axial Design Load Strength at Balanced Condition, 362 k

MB = Design Moment Strength at Balanced Condition, 2057 k-in

MO = Design Moment Strength Under Pure Flexure, 1089 k-in

Soil:

NH

Soil Modulus, 0.06 k/in^3 (See Table Below)

Concrete Sheet Pile Properties

Dimension of sheet pile, $D_1 := 12in$, $D_2 := 10in$

Note - D1 is longitudinal and D2 is transverse to the wall stem

Pile Spacing, S=1ft
Assume all sheet pile to pile cap connections are fixed (details show rebar extending into base).

Pile Properties:

$fc := 4000$ psi $\quad E_C := 57\cdot\sqrt{fc} \quad E_C = 3605$ ksi

$$I_1 := \frac{D_1\cdot D_2^3}{12} \qquad I_2 := \frac{D_2\cdot D_1^3}{12}$$

$I_1 = 1000\cdot in^4 \qquad I_2 = 1440\cdot in^4$

$Area := D_1\cdot D_2 \qquad Area = 120\cdot in^2$

J00 = 0 torsional stiffness
Length = 15ft

Design Load Strengths:
AC = Allowable Pile Compression Load, = 13.43 k/lin.ft.
AT = Allowable Pile Tension Load, =10.10 k/lin.ft.
The following design strengths are from the existing pile capacity MathCAD file:
PO = Axial Compression Design Load Strength, 378 k
PT = Axial Tension Design Load Strength, Fy*As= 70 k
PB = Axial Design Load Strength at Balanced Condition, 163 k
MB = Design Moment Strength at Balanced Condition, 631 k-in
MO = Design Moment Strength Under Pure Flexure, 290 k-in

Soil:
NH
Soil Modulus, 0.06 k/in^3 (See Table Below)

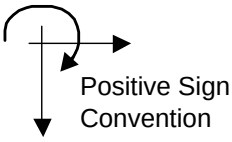
Soil Modulus Parameter k for Clays			
Average Undrained Shear Strength		Static	Cyclic
Soft Clay	c = 1.74 to 3.47 psi 250 to 500 psf 12 to 24 KPa	30 pci 8,140 KPa/m	-- --
Medium Clay	c = 3.47 to 6.94 psi 500 to 1000 psf 24 to 48 KPa	100 pci 27,150 KPa/m	-- --
Stiff Clay	c = 6.94 to 13.9 psi 1000 to 2000 psf 48 to 96 KPa	500 pci 136,000 KPa/m	200 pci 54,300 KPa/m
Very Stiff Clay	c = 13.9 to 27.8 psi 2000 to 4000 psf 96 to 192 KPa	1000 pci 271,000 KPa/m	400 pci 108,500 KPa/m
Hard Clay	c = 27.8 to 55.6 psi 4000 to 8000 psf 192 to 383 KPa	2000 pci 543,000 KPa/m	800 pci 217,000 KPa/m

Use 60 pci for k.

$\Sigma M_T := \Sigma M \cdot S \cdot N_{Rows}$

$\Sigma Vertical_T := \Sigma Vertical \cdot S \cdot N_{Rows}$

$\Sigma Horizontal_T := \Sigma Horizontal \cdot S \cdot N_{Rows}$

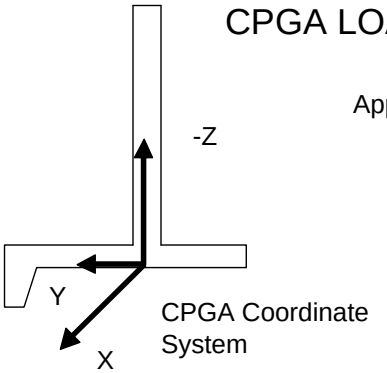


$\Sigma M_T = -571.68 \cdot ft \cdot kip$

$\Sigma Vertical_T = 295.36 \cdot kip$

$\Sigma Horizontal_T = 291.65 \cdot kip$

CPGA LOADING FOR WATER AT TOP OF WALL



Applied Loads

$PY := -\Sigma Horizontal_T$
$PZ := \Sigma Vertical_T$
$MX := -\Sigma M_T$

- Signs added to agree with CPGA sign convention.

$PY = -291.65 \cdot kip$

$PZ = 295.36 \cdot kip$

$MX = 571.68 \cdot ft \cdot kip$

```
CPGA - CASE PILE GROUP ANALYSIS PROGRAM
RUN DATE: 07-DEC-2011   RUN TIME: 14.42.38


    FOR PILES WITH UNSUPPORTED HEIGHT:
         A. CPGA CANNOT CALCULATE PMAXMOM FOR NH TYPE SOIL
         B. THE ALLOWABLE STRESS CHECKS, ASC AND AST, ARE
            NOT FULLY DEVELOPED FOR UNSUPPORTED PILES.
            WORK IS IN PROGRESS TO COMPLETE THIS ASPECT OF CPGA.

    ELASTIC CENTER LOCATION IS NOT COMPUTED FOR 3-DIMENSIONAL PROBLEMS.


CID-MO SECTION TYPE R
DATA UNKNOWN - REJECTED.



THERE ARE  56 PILES AND
            1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                    X          Y          Z
                                  -----      -----      -----
WITH DIAGONAL COORDINATES = (    -20.50 ,     2.00 ,      .00 )
                            (     20.50 ,    11.50 ,      .00 )


*****************************************************************************

         PILE PROPERTIES AS INPUT


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .36050E+04   .42180E+04   .42180E+04   .22500E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   1    2    3    4    5    6    7    8    9   10   11   12   13   14


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .36050E+04   .10000E+04   .14400E+04   .12000E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

  15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
  31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
  47   48   49   50   51   52   53   54   55   56


*****************************************************************************

         SOIL DESCRIPTIONS AS INPUT


   NH     ESOIL      LENGTH       L            LU
         K/IN**3                 FT           FT
         .60000E-01    L       .20000E+02    .00000E+00

 ESOIL(ORIGINAL)     RGROUP     RCYCLIC
   K/IN**3
   .60000E-01        .1000E+01  .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   1    2    3    4    5    6    7    8    9   10   11   12   13   14


   NH     ESOIL      LENGTH       L            LU
         K/IN**3                 FT           FT
         .60000E-01    L       .15000E+02    .00000E+00

 ESOIL(ORIGINAL)     RGROUP     RCYCLIC
   K/IN**3
   .60000E-01        .1000E+01  .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

  15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
  31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
  47   48   49   50   51   52   53   54   55   56


*****************************************************************************
```

PILE STIFFNESSES AS CALCULATED FROM PROPERTIES

.14829E+03	.00000E+00	.00000E+00	.00000E+00	.66141E+04	.00000E+00
.00000E+00	.14829E+03	.00000E+00	-.66141E+04	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.67594E+04	.00000E+00	.00000E+00	.00000E+00
.00000E+00	-.66141E+04	.00000E+00	.47570E+06	.00000E+00	.00000E+00
.66141E+04	.00000E+00	.00000E+00	.00000E+00	.47570E+06	.00000E+00
.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00

THIS MATRIX APPLIES TO THE FOLLOWING PILES -

1

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 15
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 16
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 17
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 18
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 19
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 20
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 21
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 22
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 23
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 24
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 25
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 26
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 27
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 28
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 29

```
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   30
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   31
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   32
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   33
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   34
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   35
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   36
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   37
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   38
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   39
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   40
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   41
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   42
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   43
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   44
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   45
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   46
* * * * * * * * * * * * * * * * * * *
```

```
* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   47
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   48
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   49
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   50
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   51
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   52
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   53
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   54
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   55
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   56
* * * * * * * * * * * * * * * * * * * *


*******************************************************************************
```

PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	-18.00	2.00	.00	V	.00	20.00	F
2	-12.00	2.00	.00	V	.00	20.00	F
3	-6.00	2.00	.00	V	.00	20.00	F
4	.00	2.00	.00	V	.00	20.00	F
5	6.00	2.00	.00	V	.00	20.00	F
6	12.00	2.00	.00	V	.00	20.00	F
7	18.00	2.00	.00	V	.00	20.00	F
8	-18.00	6.00	.00	V	.00	20.00	F
9	-12.00	6.00	.00	V	.00	20.00	F
10	-6.00	6.00	.00	V	.00	20.00	F
11	.00	6.00	.00	V	.00	20.00	F
12	6.00	6.00	.00	V	.00	20.00	F
13	12.00	6.00	.00	V	.00	20.00	F
14	18.00	6.00	.00	V	.00	20.00	F
15	-20.50	11.50	.00	V	.00	15.00	F
16	-19.50	11.50	.00	V	.00	15.00	F
17	-18.50	11.50	.00	V	.00	15.00	F
18	-17.50	11.50	.00	V	.00	15.00	F
19	-16.50	11.50	.00	V	.00	15.00	F
20	-15.50	11.50	.00	V	.00	15.00	F
21	-14.50	11.50	.00	V	.00	15.00	F
22	-13.50	11.50	.00	V	.00	15.00	F
23	-12.50	11.50	.00	V	.00	15.00	F
24	-11.50	11.50	.00	V	.00	15.00	F
25	-10.50	11.50	.00	V	.00	15.00	F
26	-9.50	11.50	.00	V	.00	15.00	F
27	-8.50	11.50	.00	V	.00	15.00	F
28	-7.50	11.50	.00	V	.00	15.00	F
29	-6.50	11.50	.00	V	.00	15.00	F
30	-5.50	11.50	.00	V	.00	15.00	F

31	-4.50	11.50	.00	V	.00	15.00	F
32	-3.50	11.50	.00	V	.00	15.00	F
33	-2.50	11.50	.00	V	.00	15.00	F
34	-1.50	11.50	.00	V	.00	15.00	F
35	-.50	11.50	.00	V	.00	15.00	F
36	.50	11.50	.00	V	.00	15.00	F
37	1.50	11.50	.00	V	.00	15.00	F
38	2.50	11.50	.00	V	.00	15.00	F
39	3.50	11.50	.00	V	.00	15.00	F
40	4.50	11.50	.00	V	.00	15.00	F
41	5.50	11.50	.00	V	.00	15.00	F
42	6.50	11.50	.00	V	.00	15.00	F
43	7.50	11.50	.00	V	.00	15.00	F
44	8.50	11.50	.00	V	.00	15.00	F
45	9.50	11.50	.00	V	.00	15.00	F
46	10.50	11.50	.00	V	.00	15.00	F
47	11.50	11.50	.00	V	.00	15.00	F
48	12.50	11.50	.00	V	.00	15.00	F
49	13.50	11.50	.00	V	.00	15.00	F
50	14.50	11.50	.00	V	.00	15.00	F
51	15.50	11.50	.00	V	.00	15.00	F
52	16.50	11.50	.00	V	.00	15.00	F
53	17.50	11.50	.00	V	.00	15.00	F
54	18.50	11.50	.00	V	.00	15.00	F
55	19.50	11.50	.00	V	.00	15.00	F
56	20.50	11.50	.00	V	.00	15.00	F

						910.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-291.7	295.4	571.7	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.61279E+04	.00000E+00	.00000E+00	.00000E+00	.23837E+06	-.65881E+06
.00000E+00	.55780E+04	.00000E+00	-.20972E+06	.00000E+00	-.50932E-10
.00000E+00	.00000E+00	.29651E+06	.32402E+08	.00000E+00	.00000E+00
.00000E+00	-.20972E+06	.32402E+08	.41301E+10	.00000E+00	-.34925E-09
.23837E+06	.00000E+00	.00000E+00	.00000E+00	.62484E+10	-.24561E+08
-.65881E+06	-.50932E-10	.00000E+00	-.34925E-09	-.24561E+08	.20028E+09

56 PILES 1 LOAD CASES

LOAD CASE 1. NUMBER OF FAILURES = 56. NUMBER OF PILES IN TENSION = 42.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	-.2329E-17	-.5464E-01	.7833E-02	-.6257E-04	.3701E-23	-.2167E-19

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE
# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
2	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*

3	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
4	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
5	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
6	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
7	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
8	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
9	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
10	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
11	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
12	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
13	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
14	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
15	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
16	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
17	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
18	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
19	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
20	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
21	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
22	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
23	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
24	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
25	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
26	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
27	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
28	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
29	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
30	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
31	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
32	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
33	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
34	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
35	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
36	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
37	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
38	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
39	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
40	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
41	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
42	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
43	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
44	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
45	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
46	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
47	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
48	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
49	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
50	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
51	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
52	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
53	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
54	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
55	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
56	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-7.7	42.8	331.6	.0	.0
2	.0	-7.7	42.8	331.6	.0	.0
3	.0	-7.7	42.8	331.6	.0	.0
4	.0	-7.7	42.8	331.6	.0	.0
5	.0	-7.7	42.8	331.6	.0	.0
6	.0	-7.7	42.8	331.6	.0	.0
7	.0	-7.7	42.8	331.6	.0	.0
8	.0	-7.7	22.5	331.6	.0	.0
9	.0	-7.7	22.5	331.6	.0	.0
10	.0	-7.7	22.5	331.6	.0	.0
11	.0	-7.7	22.5	331.6	.0	.0
12	.0	-7.7	22.5	331.6	.0	.0
13	.0	-7.7	22.5	331.6	.0	.0
14	.0	-7.7	22.5	331.6	.0	.0
15	.0	-4.4	-3.9	143.0	.0	.0
16	.0	-4.4	-3.9	143.0	.0	.0
17	.0	-4.4	-3.9	143.0	.0	.0
18	.0	-4.4	-3.9	143.0	.0	.0
19	.0	-4.4	-3.9	143.0	.0	.0
20	.0	-4.4	-3.9	143.0	.0	.0
21	.0	-4.4	-3.9	143.0	.0	.0

22	.0	-4.4	-3.9	143.0	.0	.0
23	.0	-4.4	-3.9	143.0	.0	.0
24	.0	-4.4	-3.9	143.0	.0	.0
25	.0	-4.4	-3.9	143.0	.0	.0
26	.0	-4.4	-3.9	143.0	.0	.0
27	.0	-4.4	-3.9	143.0	.0	.0
28	.0	-4.4	-3.9	143.0	.0	.0
29	.0	-4.4	-3.9	143.0	.0	.0
30	.0	-4.4	-3.9	143.0	.0	.0
31	.0	-4.4	-3.9	143.0	.0	.0
32	.0	-4.4	-3.9	143.0	.0	.0
33	.0	-4.4	-3.9	143.0	.0	.0
34	.0	-4.4	-3.9	143.0	.0	.0
35	.0	-4.4	-3.9	143.0	.0	.0
36	.0	-4.4	-3.9	143.0	.0	.0
37	.0	-4.4	-3.9	143.0	.0	.0
38	.0	-4.4	-3.9	143.0	.0	.0
39	.0	-4.4	-3.9	143.0	.0	.0
40	.0	-4.4	-3.9	143.0	.0	.0
41	.0	-4.4	-3.9	143.0	.0	.0
42	.0	-4.4	-3.9	143.0	.0	.0
43	.0	-4.4	-3.9	143.0	.0	.0
44	.0	-4.4	-3.9	143.0	.0	.0
45	.0	-4.4	-3.9	143.0	.0	.0
46	.0	-4.4	-3.9	143.0	.0	.0
47	.0	-4.4	-3.9	143.0	.0	.0
48	.0	-4.4	-3.9	143.0	.0	.0
49	.0	-4.4	-3.9	143.0	.0	.0
50	.0	-4.4	-3.9	143.0	.0	.0
51	.0	-4.4	-3.9	143.0	.0	.0
52	.0	-4.4	-3.9	143.0	.0	.0
53	.0	-4.4	-3.9	143.0	.0	.0
54	.0	-4.4	-3.9	143.0	.0	.0
55	.0	-4.4	-3.9	143.0	.0	.0
56	.0	-4.4	-3.9	143.0	.0	.0

NO FILES WERE GENERATED DURING THIS RUN.
Stop - Program terminated.

Pile Loading (CPGA OUTPUT)

$PileLoad1 := \frac{-3.9kip}{ft} \cdot S$ $PileLoad1 = -23.4 \cdot kip$ $PileM1 := 143 \frac{kip \cdot in}{ft} \cdot S$ $PileM1 = 858 \cdot kip \cdot in$

PileLoad 1 and Msht, These numbers are the output from CPGA for sheet piling (continuous) multiplied by the bearing pile spacing to get loading comparable to sheet piles for pile cap calcs

$PileLoad2 := 22.5kip$ $PileM2 := 331.6kip \cdot in$

$PileLoad3 := 42.8kip$ $PileM3 := 331.6kip \cdot in$

$PileLoad4 := 0kip$ $PileM4 := 0kip \cdot in$

$PileLoad5 := 0kip$ $PileM5 := 0kip \cdot in$

Check Vertical Loading

$$\Sigma Vertical - \frac{PileLoad1 + PileLoad2 + PileLoad3 + PileLoad4 + PileLoad5}{S} = 0.05 \cdot \frac{kip}{ft}$$

Checks if vertical loading calculated above and input into CPGA matches the vertical load output from CPGA. Checks if the equation above is approx. = 0.

EVALUATION OF ALLOWABLE LOAD RESULTS:

CONCRETE SHEET PILE:

Concrete Sheet Pile STRENGTH Mathcad Output:

$FS_{sheetpile} := 1.9$

$Comment_P :=$ "Pile Capacity"

CONCRETE PILING:

Concrete Pile STRENGTH Mathcad Output:

$FS_{pile} := 3.6$

$Comment_P :=$ "Pile Capacity"

See Pile Capacity Calculation for full pile check:

Check Pile loads against geotechnical allowable pile capacities:

For 18" piles:

$AC_{allow} := 37.17\,kip$ $\quad PileLoad2 = 22.5 \cdot kip$ $\quad PileLoad3 = 42.8 \cdot kip$

$AT_{allow} := 20.25\,kip$

$$FS_{PileG} := \frac{AC_{allow}}{\max(PileLoad2, PileLoad3)} = 0.87$$

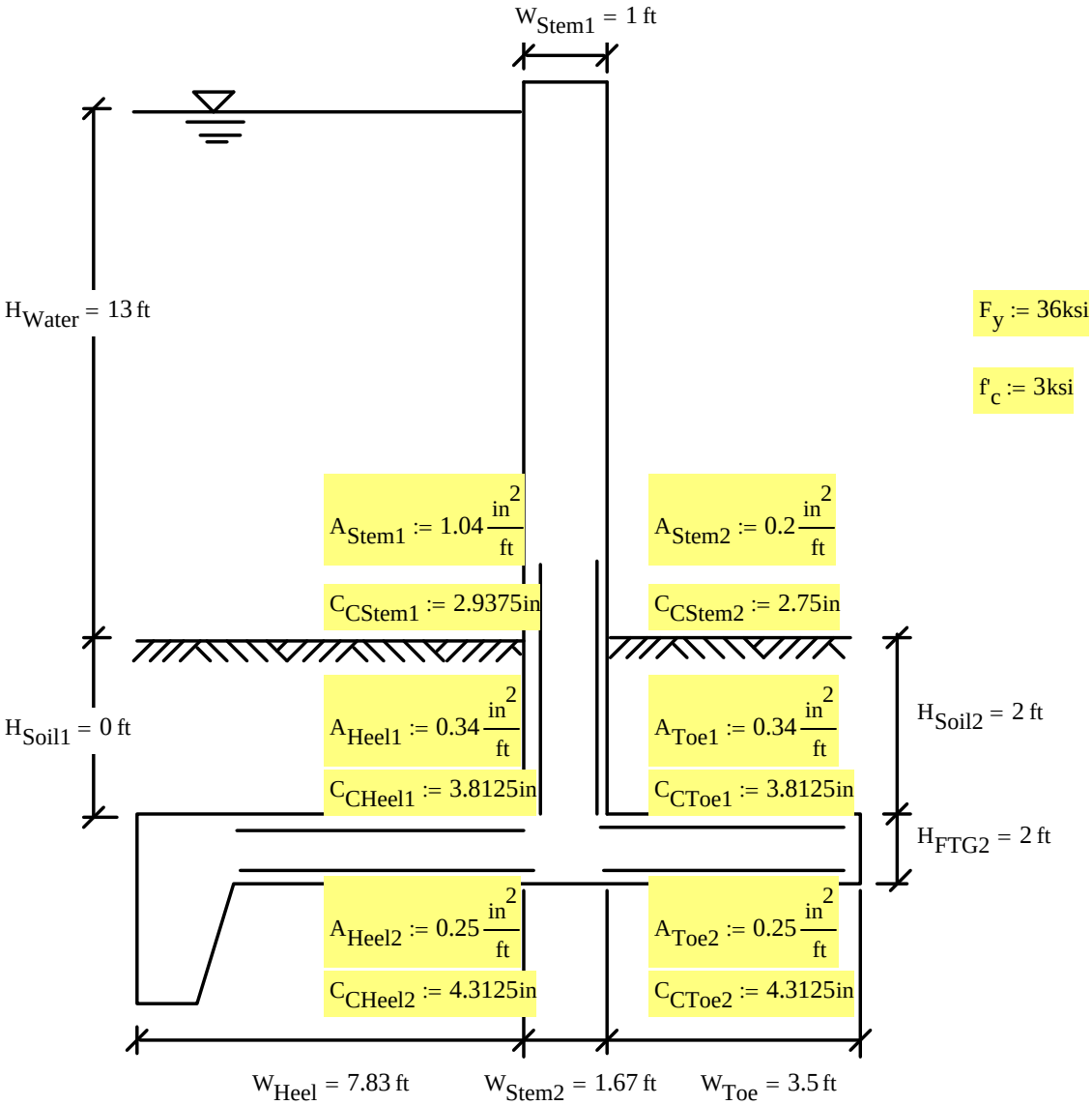
For sheet piles:

$AC_{allow} := 13.43 \frac{kip}{ft}$

$AT_{allow} := 10.10 \frac{kip}{ft}$ $\quad Pile_{sheet} := \frac{PileLoad1}{S} = -3.9 \cdot \frac{kip}{ft}$

$$FS_{SheetPileG} := \frac{\left|AT_{allow}\right|}{\left|\min\left(Pile_{sheet}\right)\right|} = 2.59$$

Typical Properties



Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength	$\phi_V := 1$
Flexural Strength	$\phi_B := 1$

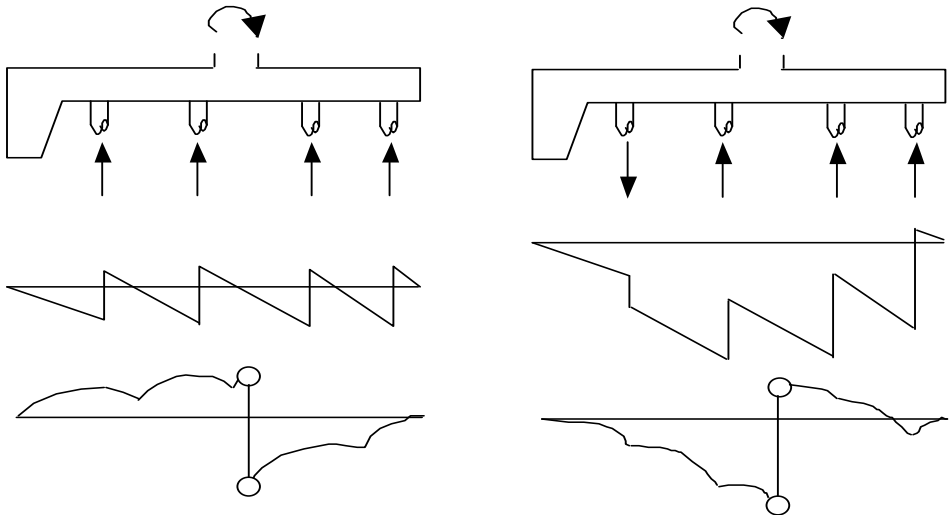
Load Factors

Dead and Live Load Factor	$\gamma_L := 1$
Hydraulic Load Factor	$\gamma_H := 1.0$
Extreme Case Factor	$\gamma_X := 1.0$

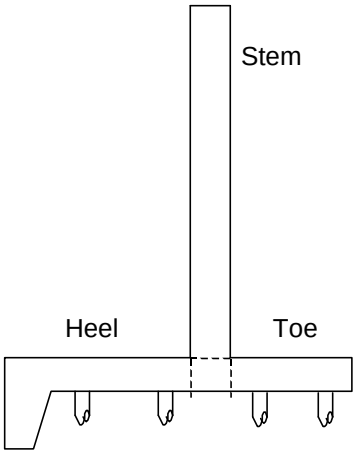
Note: Strength reduction and load factors not used in analysis of existing conditions.

Assumptions for Analysis

It has been assumed that in general the maximum bending moment from loading will occur at or near the connection of the pile cap and stem. As a result section cuts at the intersection of the pile cap and stem wall have been made to evaluate the floodwalls flexural capacity.



Typical Shear and Moment Diagrams



Section Cuts

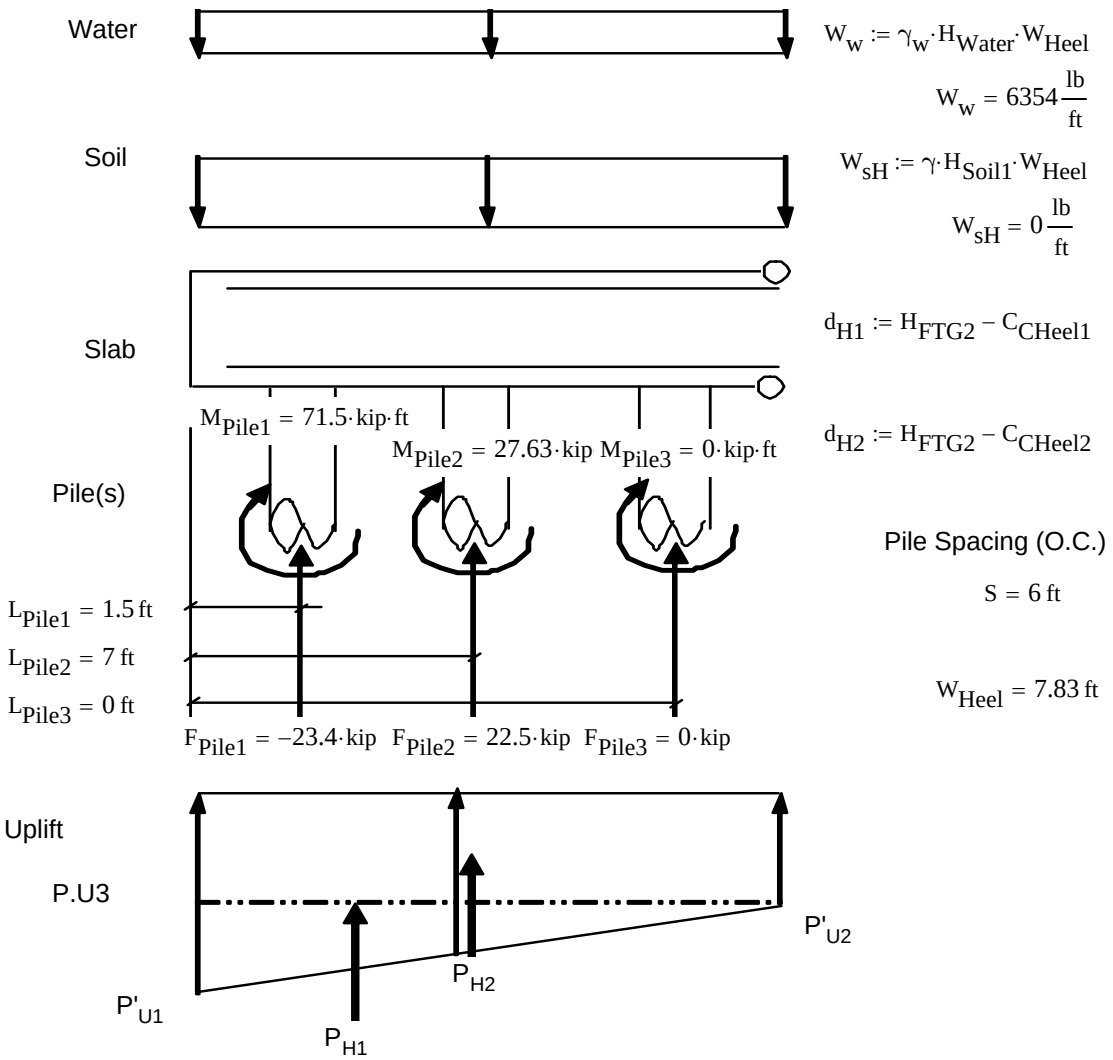
- HEEL Strength Check:

$F_{Pile1} := if\left(L_{FTG} - Pile1 < W_{Heel}, PileLoad1, 0kip\right)$ $L_{Pile1} := if\left(L_{FTG} - Pile1 < W_{Heel}, L_{FTG} - Pile1, 0ft\right)$

$F_{Pile2} := if\left(L_{FTG} - Pile2 < W_{Heel}, PileLoad2, 0kip\right)$ $L_{Pile2} := if\left(L_{FTG} - Pile2 < W_{Heel}, L_{FTG} - Pile2, 0ft\right)$

$F_{Pile3} := if\left(L_{FTG} - Pile3 < W_{Heel}, PileLoad3, 0kip\right)$ $L_{Pile3} := if\left(L_{FTG} - Pile3 < W_{Heel}, L_{FTG} - Pile3, 0ft\right)$

$M_{Pile1} := if\left(F_{Pile1} = 0, 0, PileM1\right)$ $M_{Pile2} := if\left(F_{Pile2} = 0, 0, PileM2\right)$ $M_{Pile3} := if\left(F_{Pile3} = 0, 0, PileM3\right)$



$W_w := \gamma_w \cdot H_{Water} \cdot W_{Heel}$

$W_w = 6354 \frac{lb}{ft}$

$W_{sH} := \gamma \cdot H_{Soil1} \cdot W_{Heel}$

$W_{sH} = 0 \frac{lb}{ft}$

$d_{H1} := H_{FTG2} - C_{CHeel1}$

$d_{H2} := H_{FTG2} - C_{CHeel2}$

Pile Spacing (O.C.)

$S = 6\,ft$

$W_{Heel} = 7.83\,ft$

Note: Assumes Sheet Pile reduces uplift press. by half.

$P_{U3} = -1497.6 \frac{lb}{ft}$

$P'_{U1} := p_{U2} = -624 \frac{lb}{ft^2}$

$P'_{U2} := \frac{L_{FTG} - W_{Heel}}{\left(L_{FTG} - \frac{W_{CO}}{2}\right)} \cdot \left(p_{U2} - p_{U1}\right) + p_{U1} = -471.01 \frac{lb}{ft^2}$

$P_{H1} := \frac{1}{2}\left(P'_{U1} - P'_{U2}\right) \cdot \left(W_{Heel} - \frac{W_{CO}}{2}\right) = -484.46 \frac{lb}{ft}$

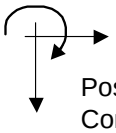
$P_{H2} := P'_{U2} \cdot \left(W_{Heel} - \frac{W_{CO}}{2}\right) = -2983.07 \frac{lb}{ft}$

- HEEL Strength Check (cont'd):

Loading:

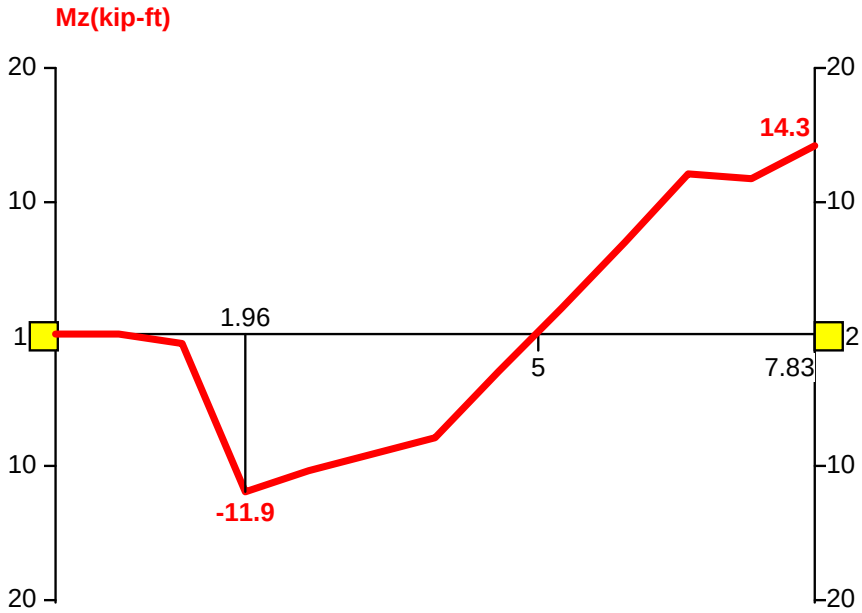
Slab Weight $W_H := \left(W_{Heel} \cdot H_{FTG2}\right) \cdot \gamma_c = 2349.99 \frac{lb}{ft}$

Bending



Positive Sign Convention

ALSO NEED TO CHECK WHERE PILE IS CAUSING LARGE MOMENTS IN THE PILE CAP AT THE PILE



$M_{Htop} := 14.3 \frac{kip \cdot ft}{ft}$

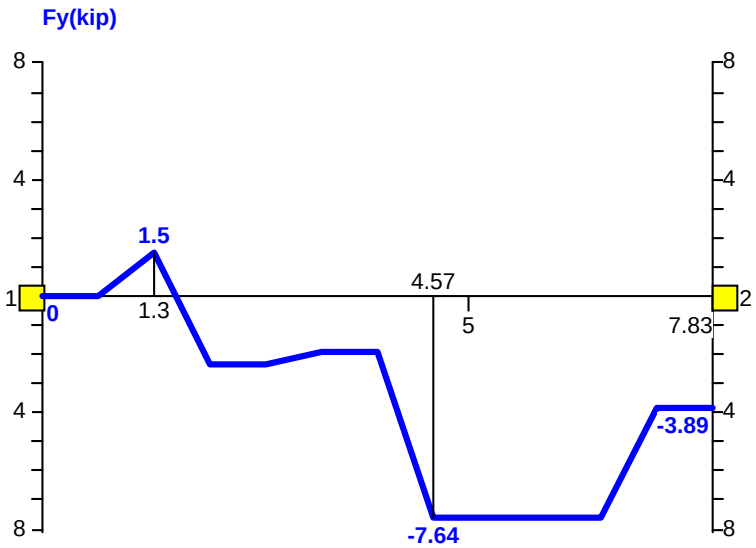
MOMENT DIAGRAM FROM STAAD

$M_{Hbottom} := 11.9 \frac{kip \cdot ft}{ft}$

Loads were input into Staad for whole moment diagram. This moment will cause tension in the steel in the BOTTOM of the heel.

- HEEL Strength Check (cont'd):

Shear



SHEAR DIAGRAM FROM STAAD

$V_H := 7.64 \frac{kip}{ft}$

$V_u := |V_H|$

$V_u = 7.64 \cdot \frac{kip}{ft}$

Capacity:

Flexural Capacity

$A_{s1} := A_{Heel1}$

$b := 12 \frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a1 = 0.4 \cdot in$

$\phi M_{H1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{H1} - \frac{a1}{2}\right)$

$\phi M_{H1} = 20.39 \cdot \frac{kip \cdot ft}{ft}$

$A_{s2} := A_{Heel2}$

$b := 12 \frac{in}{ft}$

$a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a2 = 0.29 \cdot in$

$\phi M_{H2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{H2} - \frac{a2}{2}\right)$

$\phi M_{H2} = 14.66 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{H1} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c1} = 26537 \frac{lb}{ft}$

$\phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{H2} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c2} = 25880 \frac{lb}{ft}$

$\phi V_H := \min\left[\phi V_{c1}, \left(\phi V_{c2}\right)\right]$

$\phi V_H = 25.88 \cdot \frac{kip}{ft}$

- HEEL Strength Check (cont'd):

Factors of Safety

Bending-top of heel

$\phi M_{H1} = 20.39 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_{m1} := \frac{\phi M_{H1}}{|M_{Htop}|}$

$M_{Htop} = 14.3 \cdot \frac{kip \cdot ft}{ft}$

$FS_{m1} = 1.43$

$Check1 := if(\phi M_{H1} > 1.5 M_{Htop}, "OKAY", "NO GOOD")$

$Check1 = "NO GOOD"$

Bending-bottm of heel

$\phi M_{H2} = 14.66 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_{m2} := \frac{\phi M_{H2}}{M_{Hbottom}}$

$FS_{m2} = 1.23$

$M_{Hbottom} = 11.9 \frac{1}{ft} \cdot kip \cdot ft$

$Checkm := if(\phi M_{H2} > 1.5 M_{Hbottom}, "OKAY", "NO GOOD")$

$Checkm = "NO GOOD"$

$FS_1 := min(FS_{m1}, FS_{m2})$

$FS_1 = 1.23$

Shear

$\phi V_H = 25.88 \cdot \frac{kip}{ft}$

$V_u = 7.64 \cdot \frac{kip}{ft}$ $\quad FS_2 := \frac{\phi V_H}{V_u}$

$FS_2 = 3.39$

$Check2 := if(\phi V_H > 1.5 V_u, "OKAY", "NO GOOD")$

$Check2 = "OKAY"$

Controlling Factor of Safety

$FS_H := min(FS_1, FS_2)$

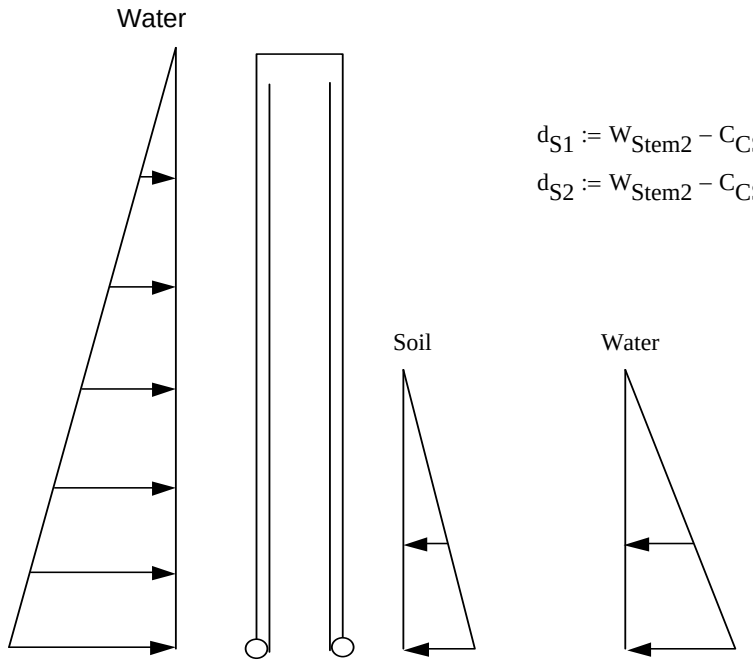
$FS_H = 1.23$

Controlling Mechanism

$Comment_H := if(FS_1 > FS_2, "Shear in Heel", if(FS_{m1} > FS_{m2}, "Flexural Bottom Steel in Heel", "Flexural Top Steel in Heel"))$

$Comment_H = "Flexural Bottom Steel in Heel"$

- STEM Strength Check



$d_{S1} := W_{Stem2} - C_{CStem1}$

$d_{S2} := W_{Stem2} - C_{CStem2}$

$P_{ws1} := \left(H_{Water} + H_{Soil1}\right) \cdot \gamma_w$

$P_{ws1} = 811 \frac{lb}{ft^2}$

$P_{ss1} := K_u \cdot \left(H_{Soil1}\right) \cdot \left(\gamma - \gamma_w\right)$

$P_{ss1} = 0 \frac{lb}{ft^2}$

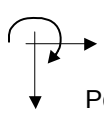
$P_{ss} := H_{Soil2} \cdot K_u \cdot \left(\gamma - \gamma_w\right)$ $\quad P_{ws2} := H_{Soil2} \cdot \gamma_w$

$P_{ss} = 65.79 \frac{lb}{ft^2}$ $\quad P_{ws2} = 124.8 \frac{lb}{ft^2}$

Loading:

Bending

$$M_S := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{P_{ws1} \cdot \left(H_{Water} + H_{Soil1}\right)}{2} \cdot \frac{H_{Water} + H_{Soil1}}{3} + \frac{P_{ss1} \cdot \left(H_{Soil1}\right)}{2} \cdot \frac{H_{Soil1}}{3} - \frac{\left(P_{ss} + P_{ws2}\right) \cdot \left(H_{Soil2}\right)}{2} \cdot \frac{H_{Soil2}}{3} \right]$$



Positive Sign Convention

$M_S = 23 \cdot \frac{kip \cdot ft}{ft}$ $\quad M_{uS} := \left|M_S\right|$

$M_{uS} = 23 \cdot \frac{kip \cdot ft}{ft}$

Shear

$$V_S := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{P_{ws1} \cdot \left(H_{Water} + H_{Soil1}\right)}{2} - \frac{\left(P_{ss} + P_{ws2}\right) \cdot \left(H_{Soil2}\right)}{2} \right]$$

$V_S = 5.08 \cdot \frac{kip}{ft}$ $\quad V_{uS} := \left|V_S\right|$

$V_{uS} = 5.08 \cdot \frac{kip}{ft}$

- STEM Strength Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Stem1}$

$b := 12\frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a1 = 1.22 \cdot in$

$\phi M_{S1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{S1} - \frac{a1}{2}\right)$

$\phi M_{S1} = 51.3 \cdot \frac{kip \cdot ft}{ft}$

$A_{s2} := A_{Stem2}$

$b := 12\frac{in}{ft}$

$a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a2 = 0.24 \cdot in$

$\phi M_{S2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{S2} - \frac{a2}{2}\right)$

$\phi M_{S2} = 10.27 \cdot \frac{kip \cdot ft}{ft}$

$A_{Stem2} = 0.2\frac{1}{ft} \cdot in^2$

$d_{S2} = 1.44\,ft$

$\phi M_S := if\left(M_S > 0, \phi M_{S1}, \phi M_{S2}\right)$ $\quad \phi M_S = 51.3 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{S1} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c1} = 22419\frac{lb}{ft}$

$\phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{S2} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c2} = 22665\frac{lb}{ft}$

$\phi V_S := if\left(M_S > 0, \phi V_{c1}, \phi V_{c2}\right)$ $\quad \phi V_S = 22.42 \cdot \frac{kip}{ft}$

Factors of Safety

Bending

$\phi M_S = 51.3 \cdot \frac{kip \cdot ft}{ft}$

$M_{uS} = 22.72 \cdot \frac{kip \cdot ft}{ft}$

$FS_3 := \frac{\phi M_S}{M_{uS}}$ $\quad FS_3 = 2.26$

$Check3 := if\left(\phi M_S > 1.5 M_{uS}, "OKAY", "NO\ GOOD"\right)$ $\quad Check3 = "OKAY"$

Shear

$\phi V_S = 22.42 \cdot \frac{kip}{ft}$

$V_{uS} = 5.08 \cdot \frac{kip}{ft}$

$FS_4 := \frac{\phi V_S}{V_{uS}}$ $\quad FS_4 = 4.41$

$Check4 := if\left(\phi V_S > 1.5 V_{uS}, "OKAY", "NO\ GOOD"\right)$ $\quad Check4 = "OKAY"$

Controlling Factor of Safety

$FS_S := \min\left(FS_3, FS_4\right)$ $\quad FS_S = 2.26$

Controlling Mechanism

$Comment_S := if\left(FS_3 > FS_4, "Shear\ in\ Stem", if\left(M_S > 0, "Stem\ Flexural\ Riverside\ Steel", "Stem\ Flexural\ Landside\ Steel"\right)\right)$

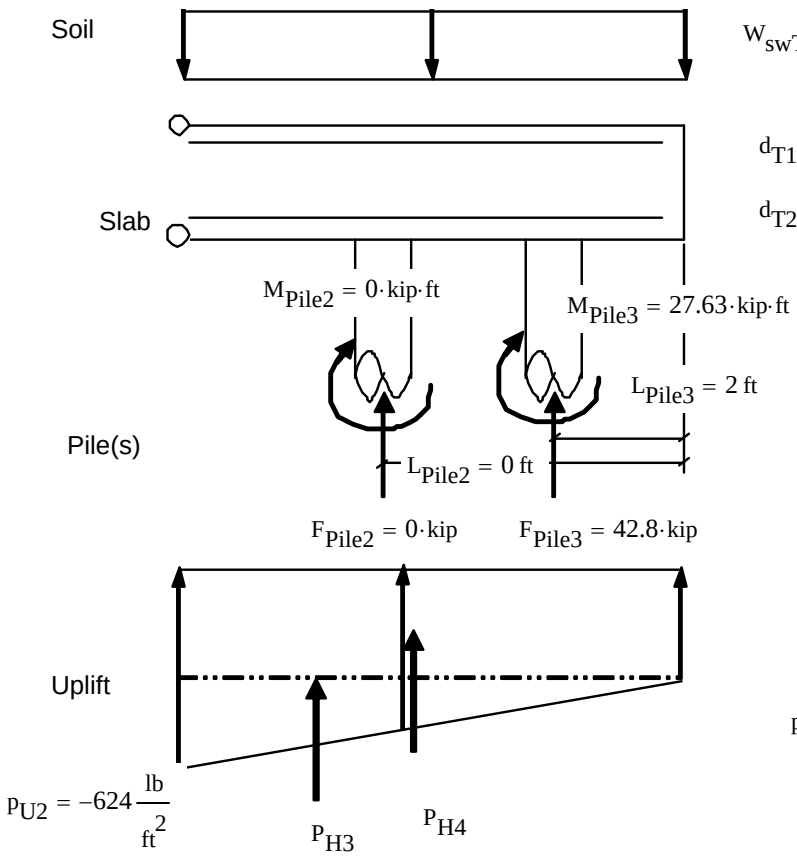
$Comment_S = "Stem\ Flexural\ Riverside\ Steel"$

- TOE Strength Check

$F_{Pile2} := \text{if}(Pile2 < W_{Toe}, PileLoad2, 0kip)$ $F_{Pile3} := \text{if}(Pile3 < W_{Toe}, PileLoad3, 0kip)$ $F_{Pile4} := \text{if}(Pile4 < W_{Toe}, PileLoad4, 0kip)$

$L_{Pile2} := \text{if}(Pile2 < W_{Toe}, Pile2, 0ft)$ $L_{Pile3} := \text{if}(Pile3 < W_{Toe}, Pile3, 0ft)$ $L_{Pile4} := \text{if}(Pile4 < W_{Toe}, Pile4, 0ft)$

$M_{Pile2} := \text{if}(Pile2 < W_{Toe} + 6in, PileM2, 0kip)$ $M_{Pile3} := \text{if}(F_{Pile3} = 0, 0, PileM3)$ $M_{Pile4} := \text{if}(F_{Pile4} = 0, 0, PileM4)$



$W_{swT} := (\gamma \cdot H_{Soil2}) \cdot W_{Toe}$ $W_{swT} = 805 \frac{lb}{ft}$

$W_{Toe} = 3.5\ ft$

$d_{T1} := H_{FTG2} - C_{CToe1}$

$d_{T2} := H_{FTG2} - C_{CToe2}$

Pile Spacing (O.C.)

$S = 6\ ft$

$M_{Pile4} = 0 \cdot kip \cdot ft$

$L_{Pile4} = 0$

Note: Assumes Sheet Piling reduces uplift pressures by half.

$p_{U1} = -346.22 \frac{lb}{ft^2}$

$$P'_{U3} := -\left[\frac{W_{Toe}}{L_{FTG} - \frac{W_{CO}}{2}} \cdot \left(|p_{U2}| - |p_{U1}|\right) + |p_{U1}|\right] \qquad P'_{U3} = -430.77 \frac{lb}{ft^2}$$

$$P_{H3} := -\left[\frac{1}{2} \cdot \left(|P'_{U3}| - |p_{U1}|\right) \cdot W_{Toe}\right] \qquad P_{H3} = -147.96 \frac{lb}{ft}$$

$$P_{H4} := p_{U1} \cdot W_{Toe} \qquad P_{H4} = -1211.78 \frac{lb}{ft}$$

Loading:

Slab Weight $W_T := (W_{Toe} \cdot H_{FTG2}) \cdot \gamma_c$ $W_T = 1050 \frac{lb}{ft}$

Pile Centroid

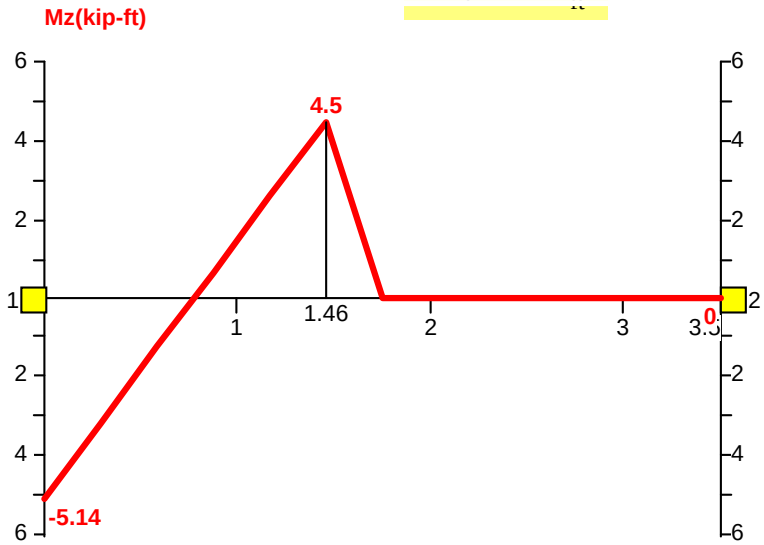
$$L_{PileT} := \frac{L_{Pile2} \cdot F_{Pile2} + L_{Pile3} \cdot F_{Pile3} + L_{Pile4} \cdot F_{Pile4}}{F_{Pile2} + F_{Pile3} + F_{Pile4}} \qquad L_{PileT} = 2\ ft$$

- TOE Strength Check (Cont'd)

Bending Positive Sign Convention

ALSO NEED TO CHECK WHERE PILE IS CAUSING LARGE MOMENTS IN THE PILE CAP AT THE PILE

$M_{Ttop} := 4.5 \frac{kip \cdot ft}{ft}$

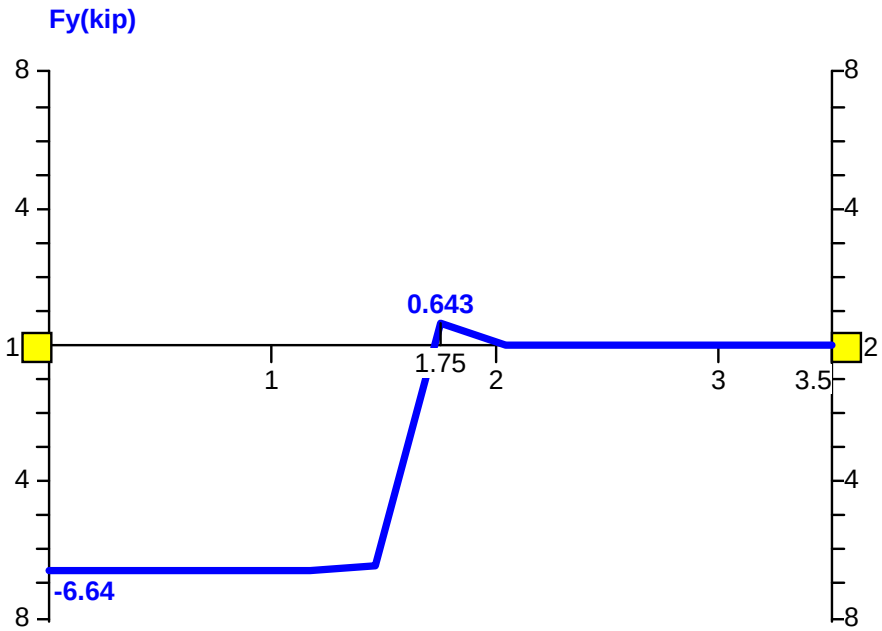


MOMENT DIAGRAM FROM STAAD

$M_{Tbottom} := 5.4 \frac{kip \cdot ft}{ft}$

Loads were input into Staad for whole moment diagram. This moment will cause tension in the steel in the BOTTOM of the heel.

Shear



$V_T := 6.64 \frac{kip}{ft}$

$V_{uT} := |V_T|$

$V_{uT} = 6.64 \cdot \frac{kip}{ft}$

- TOE Strength Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Toe1}$

$b := 12\frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85f'_c \cdot b}$ $\quad a1 = 0.4 \cdot in$

$\phi M_{T1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{T1} - \frac{a1}{2}\right)$

$\phi M_{T1} = 20.39 \cdot \frac{kip \cdot ft}{ft}$

$A_{s2} := A_{Toe2}$

$b := 12\frac{in}{ft}$

$a2 := \frac{A_{s2} \cdot F_y}{0.85f'_c \cdot b}$ $\quad a2 = 0.29 \cdot in$

$\phi M_{T2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{T2} - \frac{a2}{2}\right)$

$\phi M_{T2} = 14.66 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{T1} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c1} = 26537\frac{lb}{ft}$

$\phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{T2} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c2} = 25880\frac{lb}{ft}$

$\phi V_T := \min\left[\left(\phi V_{c1}\right), \phi V_{c2}\right]$

$\phi V_T = 25.88 \cdot \frac{kip}{ft}$

- TOE Strength Check (con'td)

Factors of Safety

Bending - top of toe

$\phi M_{T1} = 20.39 \cdot \frac{kip \cdot ft}{ft}$

$M_{Ttop} = 4.5 \cdot \frac{kip \cdot ft}{ft}$

$FS_{m1} := \frac{\phi M_{T1}}{M_{Ttop}}$

$FS_{m1} = 4.53$

$Check5 := if(\phi M_{T1} > 1.5 M_{Ttop}, "OKAY", "NO GOOD")$

$Check5 = "OKAY"$

Bending - bottom of toe

$\phi M_{T2} = 14.66 \cdot \frac{kip \cdot ft}{ft}$

$M_{Tbottom} = 5.4 \frac{1}{ft} \cdot kip \cdot ft$

$FS_{m2} := \frac{\phi M_{T2}}{M_{Tbottom}}$

$FS_{m2} = 2.71$

$Checkm := if(\phi M_{T2} > 1.5 M_{Tbottom}, "OKAY", "NO GOOD")$

$Checkm = "OKAY"$

$FS_5 := min(FS_{m1}, FS_{m2})$

$FS_5 = 2.71$

Shear

$\phi V_T = 25.88 \cdot \frac{kip}{ft}$

$V_{uT} = 6.64 \cdot \frac{kip}{ft}$

$FS_6 := \frac{\phi V_T}{V_{uT}}$

$FS_6 = 3.9$

$Check6 := if(\phi V_T > 1.5 V_{uT}, "OKAY", "NO GOOD")$

$Check6 = "OKAY"$

Controlling Factor of Safety

$FS_T := min(FS_5, FS_6)$

$FS_T = 2.71$

Controlling Mechanism

$Comment_T := if(FS_5 > FS_6, "Shear in Heel", if(FS_{m1} < FS_{m2}, "Flexural Top Steel in Toe", "Flexural Bottom Steel in Toe"))$

$Comment_T = "Flexural Bottom Steel in Toe"$

Overall Factor of Safety

Strength Factor of Safety

Sheet Pile Factor of Safety, $FS_{sheetpile} = 1.9$

Pile Factor of Safety, $FS_{pile} = 3.6$

Heel Factor of Safety, $FS_H = 1.23$

Stem Factor of Safety, $FS_S = 2.26$

Toe Factor of Safety, $FS_T = 2.71$

The strength FS for the piles should be greater than or equal to 1.5, if not a reliability analysis needs to be completed for the piles.

$$FoS := \min(FS_{sheetpile}, FS_{pile}, FS_H, FS_S, FS_T)$$

Limiting Mechanism

$$Mechanism := Comment_P$$

$$Mechanism := if(FoS = FS_H, Comment_H, Mechanism)$$

$$Mechanism := if(FoS = FS_S, Comment_S, Mechanism)$$

$$Mechanism := if(FoS = FS_T, Comment_T, Mechanism)$$

$FoS = 1.23$ $\quad$ Mechanism = "Flexural Bottom Steel in Heel"

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis.

Geotechnical Capacity Factor of Safety:

Piles $\quad FS_{PileG} = 0.87$

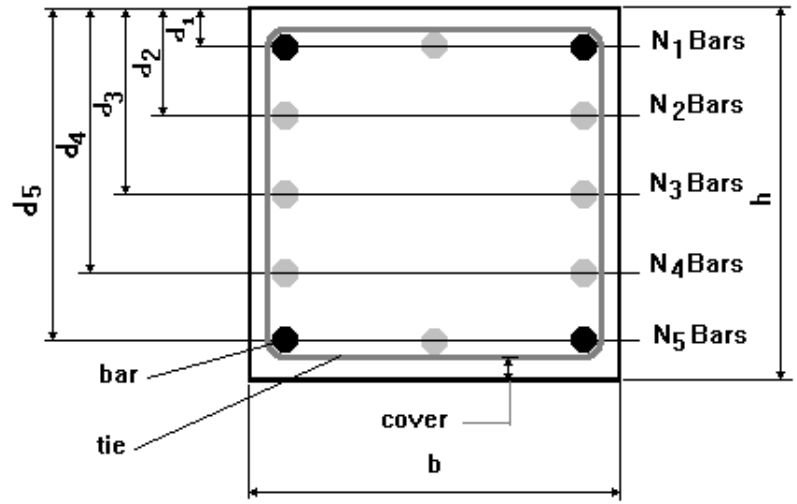
Sheet Piles $\quad FS_{SheetPileG} = 2.59$

Pile load is GREATER than the allowable geotechnical capacity.
->Reliability analysis is REQUIRED for geotechnical based axial capacity.

Units:

$kips := 1000lb$ $psf := \frac{lb}{ft^2}$ $psi := \frac{lb}{in^2}$ $ksi := \frac{kips}{in^2}$ $pcf := \frac{lb}{ft^3}$ $plf := \frac{lb}{ft}$

$ton := 2000lb$ $tsf := \frac{ton}{ft^2}$ $klf := \frac{kips}{ft}$



This program calulates the column interaction curve for a rectangular concrete column using the methods of the Concrete Reinforcing Steel Institute. Note this program also accounts for less than 1 percent steel by reducing the width. Procedure based on CRSI Manual Pg 3-5 to Pg 3-8

$h := 15in$	Column Depth	$cover := 2.25in$	Cover -in
$w := 15in$	Column Width	$N_1 := 3$	Bars in Row 1
$bar := 7$	Bar Size	$N_2 := 2$	Bars in Row 2
$tie := 2$	Tie Size	$N_3 := 3$	Bars in Row 3
$f'c := 4ksi$	Conc. Str. - ksi	$N_4 := 0$	Bars in Row 4
$fy := 40ksi$	Bar Yield - ksi	$N_5 := 0$	Bars in Row 5

$row \equiv 3$

Total Rows of Bars- NOTE: This can be changed and more rows added if required.

Axial Compression with Aci Reduction value: $\phi := 0.7$ <=== Original Code Reduction Value

$\phi := 1$ <=Reduction value used F.S. analysis

Modulus of Elasticty: $Es := 29000ksi$

Concrete Strength: $f'c := 4000psi$

FEMA 310 Allowed for concrete aging: $FEMA_Factor := 1.00$

Nominal Concrete Strength assumed orginally used: $f'c := f'c \cdot FEMA_Factor$ $f'c = 4000 \cdot psi$

NOTE - if FEMA factor is 1.00 then concrete strength is the nominal concrete strength. If trying to get the expected concrete strength the FEMA factor should be 1.25. See FEMA 310 guidance.

$areabar \equiv \left(0in^2 \quad 0in^2 \quad .05in^2 \quad 0.11in^2 \quad 0.2in^2 \quad 0.31in^2 \quad 0.44in^2 \quad 0.6in^2 \quad 0.79in^2 \quad 1in^2 \quad 1.27in^2 \quad 1.56in^2 \quad 0in^2 \quad 0in^2 \quad 2.25in^2\right)$

$diabar \equiv (0in \quad 0in \quad .25in \quad 0.38in \quad 0.5in \quad 0.63in \quad 0.75in \quad 0.88in \quad 1in \quad 1.13in \quad 1.27in \quad 1.41in \quad 0in \quad 0in \quad 1.69in)$

Calculate effective stress block per ACI 318 sect. 10.2.7.3

$$\beta 1 := if\left[f'c \le 4ksi, 0.85, if\left[f'c > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'c}{ksi} - 4\right)\right]\right]$$

$\beta 1 = 0.85$

Iterations $\quad i := 1 .. row$

Bar Diameter $\quad db := diabar_{0,bar} \quad db = 0.88\cdot in$

Bar area at i $\quad A_i := N_i\cdot areabar_{0,bar}$

Tie Diameter $\quad dt := diabar_{0,tie} \quad dt = 0.250\cdot in$

Distances from compression face $\quad d_1 := cover + \frac{db}{2} + dt \quad d_1 = 2.94\cdot in$

Spacing between rows of bars $\quad space := \frac{h - 2\cdot d_1}{row - 1} \quad space = 4.56\cdot in \quad d_i := d_1 + (i - 1)\cdot space$

$i =$	$N_i =$	$A_i =$	$d_i =$
1	3	1.8 $\cdot in^2$	2.94 $\cdot in$
2	2	1.2	7.5
3	3	1.8	12.06

Gross Area: $\quad Ag := w\cdot h \quad Ag = 225\cdot in^2$

Steel Area

$$Ast := \sum_{i=1}^{row} A_i \quad Ast = 4.8\cdot in^2$$

Reduce width if $\rho < 0.01$: $\quad \frac{Ast}{Ag} = 0.0213 \quad b := if\left(\frac{Ast}{Ag} > 0.01, w, \frac{Ast}{0.01\cdot h}\right)$

Effective Width $\quad b = 15\cdot in$

$Ag := b\cdot h \quad Ag = 225\cdot in^2$

$\frac{Ast}{Ag} = 0.0213$

Calculate Axial Load at no eccentricity, e (min) at ϕPn(max):

$Pn_1 := 0.8 \cdot [0.85 \cdot f'c \cdot (Ag - Ast) + fy \cdot Ast]$ $\quad Pn_1 = 753 \cdot kips$

Equate as the first set of points on the interaction diagram: $\quad Pn_0 := Pn_1 \quad Pn_0 = 752.544 \cdot kips \quad Mn_0 := 0kips \cdot in$

Axial Compression with Aci Reduction value: $\quad \phi := 1 \quad \phi \cdot Pn_1 = 753 \cdot kips$

Assume c=h $\quad c := h \quad c = 15 \cdot in$

Given

$$Pn_1 = \sum_{i=1}^{row} \left[\left[\left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right)\right] \cdot A_i\right] + 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b \qquad Pn_1 = 752.544 \cdot kips$$

$c := Find(c) \quad c = 13.37 \cdot in$

Calculate Width of Compression block: $\quad a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c) \quad a = 11.362 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c} \quad \varepsilon s_i =$

0.002
0.001
0

$fs_i := if\left(Es \cdot \varepsilon s_i > fy, fy, if\left(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i\right)\right) \quad fs_i =$

40	$\cdot ksi$
38.185	
8.506	

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$

$Fs_i =$

65.88
41.742
15.311

$\cdot kips$

$Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$Ms_i =$

300.413
0
-69.817

$\cdot kips \cdot in$

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 230.595 \cdot kips \cdot in$

$Fst := \sum_{i=1}^{row} Fs_i$ $\quad Fst = 122.933 \cdot kips$

$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 87.839 \cdot kips \cdot ft$

$Fc := 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$ $\quad Fc = 579.454 \cdot kips$

$Mn_1 := Mc + Mst$ $\quad Mn_1 = 1284.666 \cdot kips \cdot in$

$e_1 := \frac{Mn_1}{Pn_1}$ $\quad e_1 = 1.71 \cdot in$

$d_{row} = 12.06 \cdot in$

Find Pn and Mn at point of zero tension in bars:

$c := d_{row}$ $\quad c = 12.06 \cdot in$ $\quad a := \beta 1 \cdot c$ $\quad a = 10.251 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$i =$	$\varepsilon s_i =$	$fs_i =$	$Fs_i =$	$Ms_i =$	$A_i =$	$d_i =$
1	0.002	40 $\cdot ksi$	65.88 $\cdot kips$	300.413 $\cdot kips \cdot in$	1.8 $\cdot in^2$	2.94 $\cdot in$
2	0.001	38.185	41.742	0	1.2	7.5
3	0	8.506	15.311	-69.817	1.8	12.06

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 230.595 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 1241.391 \cdot kips \cdot in$

$Mn_2 := Mc + Mst$ $\quad Mn_2 = 1471.986 \cdot kips \cdot in$ $\quad Mn_2 = 1472 \cdot kips \cdot in$

$Mn_1 = 1284.666 \cdot kips \cdot in$ $\quad Mn_1 := if(Mn_1 < Mn_2, Mn_1, Mn_2)$ $\quad Mn_1 = 1284.666 \cdot kips \cdot in$

$Pn_2 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_2 = 646 \cdot kips$ $\quad Pn_1 = 752.544 \cdot kips$ $\quad Pn_2 := if(Pn_2 < Pn_1, Pn_2, Pn_1)$ $\quad Pn_2 = 645.734 \cdot kips$

Find Pn and Mn at point of 0.25 *fy tension in bars:

$$c := \frac{d_{row}}{1 + \frac{0.25 \cdot fy}{Es \cdot 0.003}} \qquad d_{row} = 12.06 \cdot in \qquad a := \beta 1 \cdot c$$

Find Moment:

$$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c} \qquad fs_i := if\left(Es \cdot \varepsilon s_i > fy, fy, if\left(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i\right)\right)$$

$$Fs_i := fs_i \cdot A_i - if\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$$

$d_i =$

0.245	ft
0.625	
1.005	

$\varepsilon s_i =$

0.002
0.001
-0

$fs_i =$

40	$\cdot ksi$
26.677	
-10	

$A_i =$

1.8	$\cdot in^2$
1.2	
1.8	

$Fs_i =$

65.88	$\cdot kips$
27.932	
-18	

$Ms_i =$

300.413	$\cdot kips \cdot in$
0	
82.08	

$$Mst := \sum_{i=1}^{row} Ms_i \qquad Mst = 382 \cdot kips \cdot in \qquad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right) \qquad Mc = 1361.182 \cdot kips \cdot in \qquad Mn_3 := Mc + Mst \qquad Mn_3 = 1744 \cdot kips \cdot in$$

$$Pn_3 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b \qquad Pn_3 = 545 \cdot kips$$

Find Pn and Mn at point of 0.50 *fy tension in bars:

$d_{row} = 12.06 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{0.50 \cdot fy}{Es \cdot 0.003}}$ $\quad c = 9.806 \cdot in$ $\quad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$

0.245	ft
0.625	
1.005	

$\varepsilon s_i =$

0.002
0.001
-0.001

$fs_i =$

40	$\cdot ksi$
20.458	
-20	

$A_i =$

1.8	$\cdot in^2$
1.2	
1.8	

$Fs_i =$

65.88	$\cdot kips$
20.469	
-36	

$Ms_i =$

300.413	$\cdot kips \cdot in$
0	
164.16	

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 465 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 1416.599 \cdot kips \cdot in$ $\quad Mn_4 := Mc + Mst$ $\quad Mn_4 = 1881 \cdot kips \cdot in$

$Pn_4 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_4 = 475 \cdot kips$

Find Pn and Mn at Balanced:

$d_{row} = 12.06 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{fy}{Es \cdot 0.003}}$ $\quad c = 8.262 \cdot in$ $\quad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$

0.245	ft
0.625	
1.005	

$\varepsilon s_i =$

0.002
0
-0.001

$fs_i =$

40	$\cdot ksi$
8.02	
-40	

$A_i =$

1.8	$\cdot in^2$
1.2	
1.8	

$Fs_i =$

65.88	$\cdot kips$
9.624	
-72	

$Ms_i =$

300.413	$\cdot kips \cdot in$
0	
328.32	

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 629 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 1428.557 \cdot kips \cdot in$ $\quad Mn_5 := Mc + Mst$ $\quad Mn_5 = \blacksquare \cdot kips \cdot in$

$Pn_5 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_5 = 362 \cdot kips$

Find Mn at ϕ Pn = 0.1*f'c*Ag:

$Pn_6 := \frac{0.1 \cdot f'c \cdot w \cdot h}{\phi}$ $\quad Pn_6 = 90 \cdot kips$ $\quad Pn_6 := if(Pn_6 > Pn_5, Pn_5, Pn_6)$ $\quad Pn_6 = 90 \cdot kips$

Assume c=h/2 $\quad c := h \cdot 0.5 \quad c = \blacksquare \cdot in$

Given

$$Pn_6 = \sum_{i=1}^{row} \left[\left[\begin{array}{ll} (c - d_i) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es < fy \\ + -if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \\ fy - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \geq fy \\ -fy - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \leq -fy \end{array} \right] \cdot A_i \right] \ldots$$

$$+ 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c) \quad c = 4.02 \cdot in$

$a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c) \quad a = 3.41 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$d_i =$

2.94
7.5
12.06

$\cdot in$

$\varepsilon s_i =$

0.001
-0.003
-0.006

$fs_i =$

23324.942
-40000
-40000

$\cdot psi$

$A_i =$

1.8
1.2
1.8

$\cdot in^2$

$Fs_i =$

35.865
-48
-72

$\cdot kips$

$Ms_i =$

163.544
0
328.32

$\cdot in \cdot kips$

$$Mst := \sum_{i=1}^{row} Ms_i \qquad Mst = 492 \cdot kips \cdot in$$

$$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right) \qquad Mc = 1008.729 \cdot kips \cdot in$$

$$Mn_6 := Mc + Mst \qquad Mn_6 = 1501 \cdot kips \cdot in$$

$$Pn_6 = 90 \cdot kips$$

Find Mn at ϕ Pn = 0.0:

$Pn_7 := 0lb$ $\qquad$ $Pn_7 = 0\,lb$

Assume c=0.2h $\qquad$ $c := 0.2 \cdot h$

Given

$$Pn_7 = \sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es < fy \\ + -\text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \\ fy - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \geq fy \\ -fy - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \leq -fy \end{array} \right] \cdot A_i \right] \ldots$$

$$+ 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c)$ $\qquad$ $c = 2.864 \cdot in$ $\qquad$ $a := \text{if}(\beta 1 \cdot c > h, h, \beta 1 \cdot c)$ $\qquad$ $a = 2.434 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$

$fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$

$Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$d_i =$

0.245
0.625
1.005

ft

$\varepsilon s_i =$

-0
-0.005
-0.01

$fs_i =$

-2.308
-40
-40

$\cdot ksi$

$A_i =$

1.8
1.2
1.8

$\cdot in^2$

$Fs_i =$

-4.155
-48
-72

$\cdot kips$

$Ms_i =$

-18.946
0
328.32

$\cdot kips \cdot in$

$Mst := \sum_{i=1}^{row} Ms_i$

$Mst = 309 \cdot kips \cdot in$

$Mc := 0.85 \cdot f'c \cdot a \cdot w \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$

$Mc = 780 \cdot kips \cdot in$

$Mn_7 := Mc + Mst$

$Mn_7 = 1089 \cdot kips \cdot in$

$j := 0 .. 7$ $\phi_j := 1$

Results

$\phi_j \cdot Pn_j =$ $\phi_j \cdot Mn_j =$

752.544	·kip	0
752.544		1284.666
645.734		1471.986
544.716		1743.675
475.43		1881.172
361.643		2057.29
90		1500.593
0		1089.413

·kips·in

Criteria:

Depth	$h = 15 \cdot in$	$bar = 7$
Width	$w = 15 \cdot in$	$tie = 2$
Eff. Width	$b = 15 \cdot in$	$fy = 40 \cdot ksi$

$cover = 2.25 \cdot in$ $f'c = 4000 \cdot psi$

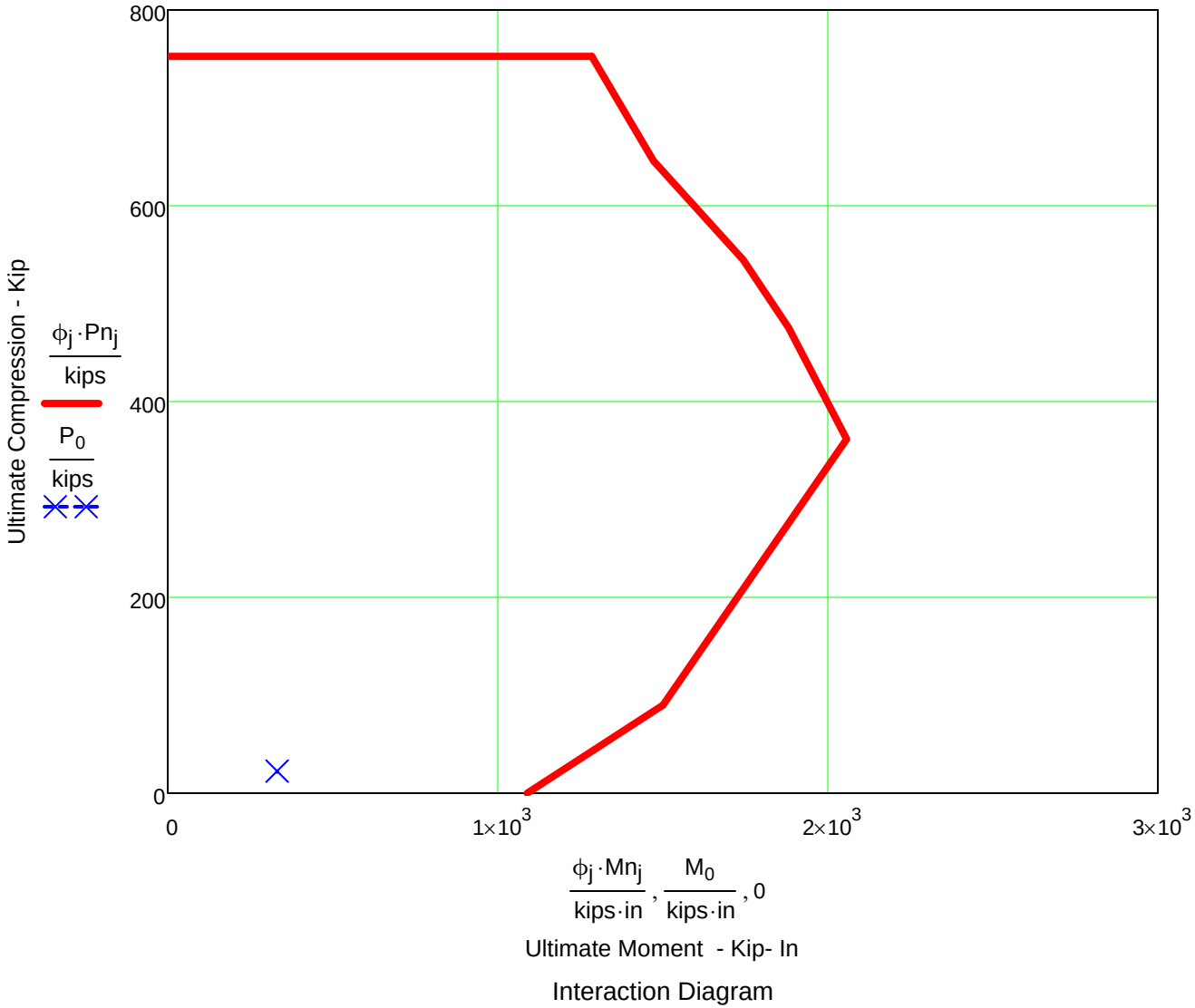
$Ag = 225 \cdot in^2$ $Ast = 4.8 \cdot in^2$

$\frac{Ast}{w \cdot h} = 0.0213$ Actual ratio ρ

$\frac{Ast}{Ag} = 0.0213$ Eff. ratio ρ

Demand Loads Water to Top of Wall:

Axial Load from CPGA output: $P_0 := 22.5kips$ Moment from CPGA output: $M_0 := 331.6kips \cdot in$



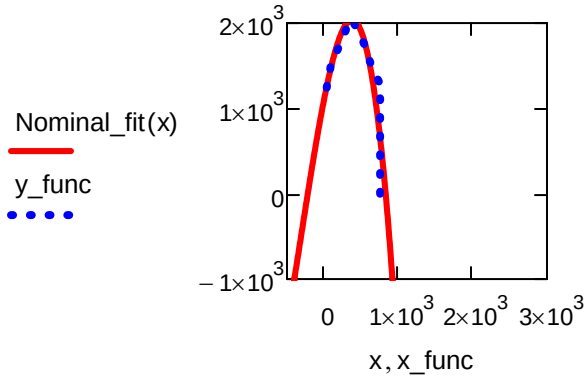
Nominal Curve Used to Find Factor of Safety:

Data Points set to function: $y_func := \frac{Mn}{kips \cdot in}$ $x_func := \frac{Pn}{kips}$

Degree of Polynominal Fit: $k := 3$

Returns a vector which interp uses to find the kth order polynomial that best fits the x and y data values

$Nominal := regress(x_func, y_func, k)$ $Nominal_fit(x) := interp(Nominal, x_func, y_func, x)$



Strength Factor of Safety:

Read the Corresponding Moment for the demand axial load ($P_0 = 22.5 \cdot kips$)

$M_{capacity} := Nominal_fit\left(\frac{P_0}{kips}\right)$ $M_{capacity} = 1205.635$

Demand Moment: $M_{demand} := \frac{M_0}{kips \cdot in}$ $M_{demand} = 331.6$

Factor of Safety: $FS_{strength} := \frac{M_{capacity}}{M_{demand}}$ $FS_{strength} = 3.6$

FEMA 310 GUIDANCE

4.2.4.4 Component Strength

Component strength for all actions shall be taken as the expected strength, Q_{CE}. Unless calculated otherwise, the expected strength shall be assumed equal to the nominal strength multiplied by 1.25. Alternatively, if allowable stresses are used, nominal strengths shall be taken as the allowable values multiplied by the following values:

Steel	1.7
Masonry	2.5
Wood	2.0

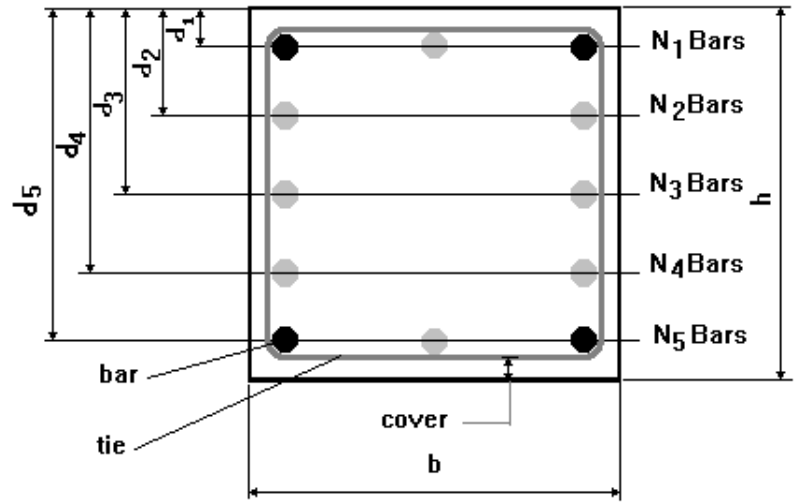
Except for wood diaphragms and wood and masonry shear walls, the allowable values shall not include a one-third increase for short term loading.

When calculating capacities of deteriorated elements, the evaluating design professional shall make

Units:

$kips := 1000lb$ $psf := \frac{lb}{ft^2}$ $psi := \frac{lb}{in^2}$ $ksi := \frac{kips}{in^2}$ $pcf := \frac{lb}{ft^3}$ $plf := \frac{lb}{ft}$

$ton := 2000lb$ $tsf := \frac{ton}{ft^2}$ $klf := \frac{kips}{ft}$



This program calulates the column interaction curve for a rectangular concrete column using the methods of the Concrete Reinforcing Steel Institute. Note this program also accounts for less than 1 percent steel by reducing the width. Procedure based on CRSI Manual Pg 3-5 to Pg 3-8

$h := 10in$	Column Depth	$cover := 2.25in$	Cover -in
$w := 12in$	Column Width	$N_1 := 2$	Bars in Row 1
$bar := 6$	Bar Size	$N_2 := 2$	Bars in Row 2
$tie := 2$	Tie Size	$N_3 := 0$	Bars in Row 3
$f'c := 4ksi$	Conc. Str. - ksi	$N_4 := 0$	Bars in Row 4
$fy := 40ksi$	Bar Yield - ksi	$N_5 := 0$	Bars in Row 5

$row \equiv 2$

Total Rows of Bars- NOTE: This can be changed and more rows added if required.

Axial Compression with Aci Reduction value: $\phi := 0.7$ <=== Original Code Reduction Value

$\phi := 1$ <=Reduction value used F.S. analysis

Modulus of Elasticty: $Es := 29000ksi$

Concrete Strength: $f'c := 4000psi$

FEMA 310 Allowed for concrete aging: $FEMA_Factor := 1.00$

Nominal Concrete Strength assumed orginally used: $f'c := f'c \cdot FEMA_Factor$ $f'c = 4000 \cdot psi$

NOTE - if FEMA factor is 1.00 then concrete strength is the nominal concrete strength. If trying to get the expected concrete strength the FEMA factor should be 1.25. See FEMA 310 guidance.

$areabar \equiv \left(0in^2 \quad 0in^2 \quad .05in^2 \quad 0.11in^2 \quad 0.2in^2 \quad 0.31in^2 \quad 0.44in^2 \quad 0.6in^2 \quad 0.79in^2 \quad 1in^2 \quad 1.27in^2 \quad 1.56in^2 \quad 0in^2 \quad 0in^2 \quad 2.25in^2\right)$

$diabar \equiv (0in \quad 0in \quad .25in \quad 0.38in \quad 0.5in \quad 0.63in \quad 0.75in \quad 0.88in \quad 1in \quad 1.13in \quad 1.27in \quad 1.41in \quad 0in \quad 0in \quad 1.69in)$

Calculate effective stress block per ACI 318 sect. 10.2.7.3 $\quad \beta 1 := if\left[f'c \le 4ksi, 0.85, if\left[f'c > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'c}{ksi} - 4\right)\right]\right]$ $\quad \beta 1 = 0.85$

Iterations $\quad i := 1 .. row$

Bar Diameter $\quad db := diabar_{0,bar} \quad db = 0.75\cdot in$

Bar area at i $\quad A_i := N_i\cdot areabar_{0,bar}$

Tie Diameter $\quad dt := diabar_{0,tie} \quad dt = 0.250\cdot in$

Distances from compression face $\quad d_1 := cover + \frac{db}{2} + dt \quad d_1 = 2.875\cdot in$

Spacing between rows of bars $\quad space := \frac{h - 2\cdot d_1}{row - 1} \quad space = 4.25\cdot in \quad d_i := d_1 + (i - 1)\cdot space$

$i =$	$N_i =$	$A_i =$	$d_i =$
1	2	0.88 $\cdot in^2$	2.875 $\cdot in$
2	2	0.88	7.125

Gross Area: $\quad Ag := w\cdot h \quad Ag = 120\cdot in^2$

Steel Area $\quad Ast := \sum_{i=1}^{row} A_i \quad Ast = 1.76\cdot in^2$

Reduce width if $\rho < 0.01$: $\quad \frac{Ast}{Ag} = 0.0147 \quad b := if\left(\frac{Ast}{Ag} > 0.01, w, \frac{Ast}{0.01\cdot h}\right)$

Effective Width $\quad b = 12\cdot in$

$Ag := b\cdot h \quad Ag = 120\cdot in^2$

$\frac{Ast}{Ag} = 0.0147$

Calculate Axial Load at no eccentricity, e (min) at ϕPn(max):

$Pn_1 := 0.8 \cdot [0.85 \cdot f'c \cdot (Ag - Ast) + fy \cdot Ast]$ $\quad Pn_1 = 378 \cdot kips$

Equate as the first set of points on the interaction diagram: $\quad Pn_0 := Pn_1 \quad Pn_0 = 377.933 \cdot kips \quad Mn_0 := 0kips \cdot in$

Axial Compression with Aci Reduction value: $\quad \phi := 1 \quad \phi \cdot Pn_1 = 378 \cdot kips$

Assume c=h $\quad c := h \quad c = 10 \cdot in$

Given

$$Pn_1 = \sum_{i=1}^{row} \left[\left[\left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right)\right] \cdot A_i\right] + 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b \qquad Pn_1 = 377.933 \cdot kips$$

$c := Find(c) \quad c = 9.08 \cdot in$

Calculate Width of Compression block: $\quad a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c) \quad a = 7.722 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c}$ $\quad \varepsilon s_i =$

0.002
0.001

$fs_i := if\left(Es \cdot \varepsilon s_i > fy, fy, if\left(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i\right)\right)$ $\quad fs_i =$

40
18.769

$\cdot ksi$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Fs_i =$

32.208
13.525

$\cdot kips$

$Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$ $\quad Ms_i =$

68.442
-28.741

$\cdot kips \cdot in$

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 39.701 \cdot kips \cdot in$ $\quad Fst := \sum_{i=1}^{row} Fs_i$ $\quad Fst = 45.733 \cdot kips$

$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 29.902 \cdot kips \cdot ft$ $\quad Fc := 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$ $\quad Fc = 315.068 \cdot kips$

$Mn_1 := Mc + Mst$ $\quad Mn_1 = 398.525 \cdot kips \cdot in$ $\quad e_1 := \frac{Mn_1}{Pn_1}$ $\quad e_1 = 1.05 \cdot in$

$d_{row} = 7.125 \cdot in$

Find Pn and Mn at point of zero tension in bars:

$c := d_{row}$ $\quad c = 7.125 \cdot in$ $\quad a := \beta 1 \cdot c$ $\quad a = 6.056 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$i =$	$\varepsilon s_i =$	$fs_i =$	$Fs_i =$	$Ms_i =$	$A_i =$	$d_i =$
1	0.002	40 $\cdot ksi$	32.208 $\cdot kips$	68.442 $\cdot kips \cdot in$	0.88 $\cdot in^2$	2.875 $\cdot in$
2	0	18.769	16.517	-35.099	0.88	7.125

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 33.343 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 487.24 \cdot kips \cdot in$

$Mn_2 := Mc + Mst$ $\quad Mn_2 = 520.584 \cdot kips \cdot in$ $\quad Mn_2 = 521 \cdot kips \cdot in$

$Mn_1 = 398.525 \cdot kips \cdot in$ $\quad Mn_1 := if(Mn_1 < Mn_2, Mn_1, Mn_2)$ $\quad Mn_1 = 398.525 \cdot kips \cdot in$

$Pn_2 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_2 = 296 \cdot kips$ $\quad Pn_1 = 377.933 \cdot kips$ $\quad Pn_2 := if(Pn_2 < Pn_1, Pn_2, Pn_1)$ $\quad Pn_2 = 295.82 \cdot kips$

Find Pn and Mn at point of 0.25 *fy tension in bars:

$$c := \frac{d_{row}}{1 + \frac{0.25 \cdot fy}{Es \cdot 0.003}}$$

$d_{row} = 7.125 \cdot in$

$a := \beta 1 \cdot c$

Find Moment:

$$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c}$$

$$fs_i := if\left(Es \cdot \varepsilon s_i > fy, fy, if\left(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i\right)\right)$$

$$Fs_i := fs_i \cdot A_i - if\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi\right) \cdot A_i$$

$$Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$$

$d_i =$

0.24
0.594

ft

$\varepsilon s_i =$

0.002
-0

$fs_i =$

40
-10

$\cdot ksi$

$A_i =$

0.88
0.88

$\cdot in^2$

$Fs_i =$

32.208
-8.8

$\cdot kips$

$Ms_i =$

68.442
18.7

$\cdot kips \cdot in$

$$Mst := \sum_{i=1}^{row} Ms_i$$

$Mst = 87 \cdot kips \cdot in$

$$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$$

$Mc = \blacksquare \cdot kips \cdot in$

$Mn_3 := Mc + Mst$

$Mn_3 = 593 \cdot kips \cdot in$

$$Pn_3 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$$

$Pn_3 = 245 \cdot kips$

Find Pn and Mn at point of 0.50 *fy tension in bars:

$d_{row} = 7.125 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{0.50 \cdot fy}{Es \cdot 0.003}}$ $\quad c = 5.793 \cdot in$ $\quad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$

0.24
0.594

ft

$\varepsilon s_i =$

0.002
-0.001

$fs_i =$

40
-20

$\cdot ksi$

$A_i =$

0.88
0.88

$\cdot in^2$

$Fs_i =$

32.208
-17.6

$\cdot kips$

$Ms_i =$

68.442
37.4

$\cdot kips \cdot in$

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 106 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 509.883 \cdot kips \cdot in$ $\quad Mn_4 := Mc + Mst$ $\quad Mn_4 = 616 \cdot kips \cdot in$

$Pn_4 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_4 = 216 \cdot kips$

Find Pn and Mn at Balanced:

$d_{row} = 7.125 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{fy}{Es \cdot 0.003}}$ $\quad c = 4.881 \cdot in$ $\quad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if\left(Es \cdot \varepsilon s_i > fy, fy, if\left(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - if\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi\right) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$

0.24	ft
0.594	

$\varepsilon s_i =$

0.001
-0.001

$fs_i =$

35.754	$\cdot ksi$
-40	

$A_i =$

0.88	$\cdot in^2$
0.88	

$Fs_i =$

28.472	$\cdot kips$
-35.2	

$Ms_i =$

60.503	$\cdot kips \cdot in$
74.8	

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 135 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 495.218 \cdot kips \cdot in$ $\quad Mn_5 := Mc + Mst$ $\quad Mn_5 = 631 \cdot kips \cdot in$

$Pn_5 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_5 = 163 \cdot kips$

Find Mn at ϕ Pn = 0.1*f'c*Ag:

$Pn_6 := \frac{0.1 \cdot f'c \cdot w \cdot h}{\phi}$ $\quad Pn_6 = 48 \cdot kips$ $\quad Pn_6 := if(Pn_6 > Pn_5, Pn_5, Pn_6)$ $\quad Pn_6 = 48 \cdot kips$

Assume c=h/2 $\quad c := h \cdot 0.5 \quad c = 5 \cdot in$

Given

$$Pn_6 = \sum_{i=1}^{row} \left[\left[\begin{array}{ll} (c - d_i) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es < fy \\ + -if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \\ fy - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \geq fy \\ -fy - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \leq -fy \end{array} \right] \cdot A_i \right] \ldots$$
$$+ 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c) \quad c = 2.62 \cdot in$

$a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c) \quad a = 2.22 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$d_i =$

2.875
7.125

$\cdot in$

$\varepsilon s_i =$

-0
-0.005

$fs_i =$

-8582.355
-40000

$\cdot psi$

$A_i =$

0.88
0.88

$\cdot in^2$

$Fs_i =$

-7.552
-35.2

$\cdot kips$

$Ms_i =$

-16.049
74.8

$\cdot in \cdot kips$

$$Mst := \sum_{i=1}^{row} Ms_i \qquad Mst = 59 \cdot kips \cdot in$$

$$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right) \qquad Mc = 352.831 \cdot kips \cdot in$$

$$Mn_6 := Mc + Mst \qquad Mn_6 = 412 \cdot kips \cdot in$$

$$Pn_6 = 48 \cdot kips$$

Find Mn at ϕ Pn = 0.0:

$Pn_7 := 0lb$ $\qquad$ $Pn_7 = 0\,lb$

Assume c=0.2h $\qquad$ $c := 0.2 \cdot h$

Given

$$Pn_7 = \sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es < fy \\ + -\text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \\ fy - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \geq fy \\ -fy - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \leq -fy \end{array} \right] \cdot A_i \right] \ldots$$

$$+ 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c)$ $\qquad$ $c = 1.993 \cdot in$ $\qquad$ $a := \text{if}(\beta 1 \cdot c > h, h, \beta 1 \cdot c)$ $\qquad$ $a = 1.694 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$

$fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$

$Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$d_i =$

0.24
0.594

ft

$\varepsilon s_i =$

-0.001
-0.008

$fs_i =$

-38.527
-40

$\cdot ksi$

$A_i =$

0.88
0.88

$\cdot in^2$

$Fs_i =$

-33.904
-35.2

$\cdot kips$

$Ms_i =$

-72.045
74.8

$\cdot kips \cdot in$

$Mst := \sum_{i=1}^{row} Ms_i$

$Mst = 3 \cdot kips \cdot in$

$Mc := 0.85 \cdot f'c \cdot a \cdot w \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$

$Mc = 287 \cdot kips \cdot in$

$Mn_7 := Mc + Mst$

$Mn_7 = 290 \cdot kips \cdot in$

$j := 0 .. 7$ $\phi_j := 1$

Results

$\phi_j \cdot Pn_j =$		$\phi_j \cdot Mn_j =$	
377.933	·kip	0	·kips·in
377.933		398.525	
295.82		520.584	
245.029		593.337	
215.517		615.725	
162.542		630.521	
48		411.582	
0		289.752	

Criteria:

Depth	$h = 10 \cdot in$	$bar = 6$
Width	$w = 12 \cdot in$	$tie = 2$
Eff. Width	$b = 12 \cdot in$	$fy = 40 \cdot ksi$

$cover = 2.25 \cdot in$ $f'c = 4000 \cdot psi$

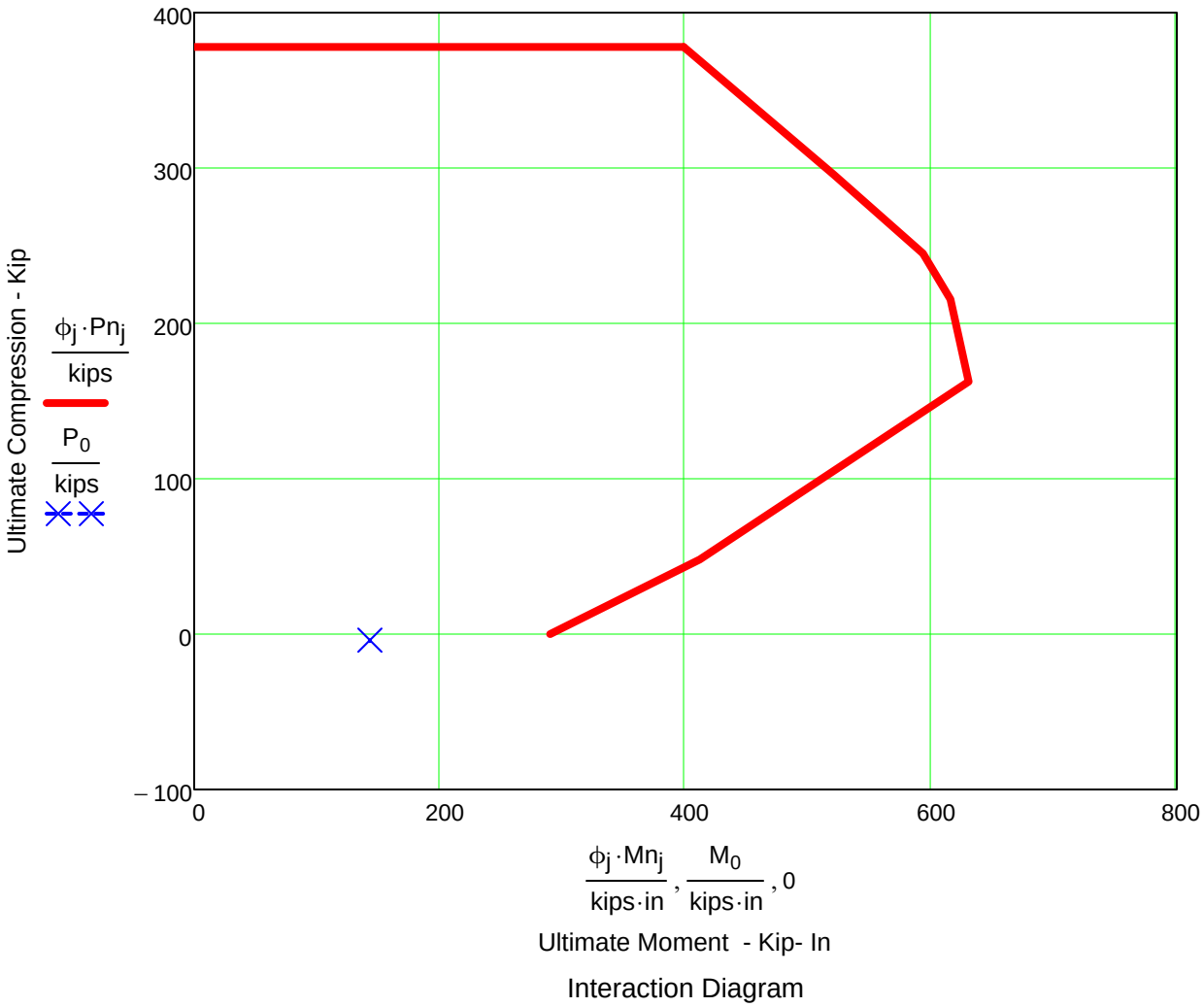
$Ag = 120 \cdot in^2$ $Ast = 1.76 \cdot in^2$

$\frac{Ast}{w \cdot h} = 0.0147$ Actual ratio ρ

$\frac{Ast}{Ag} = 0.0147$ Eff. ratio ρ

Demand Loads Water to Top of Wall:

Axial Load from CPGA output: $P_0 := -3.9kips$ Moment from CPGA output: $M_0 := 143kips \cdot in$



Interaction Diagram

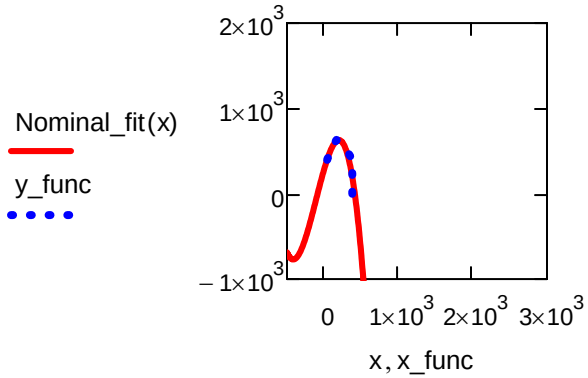
Nominal Curve Used to Find Factor of Safety:

Data Points set to function: $y_func := \frac{Mn}{kips \cdot in}$ $x_func := \frac{Pn}{kips}$

Degree of Polynominal Fit: $k := 3$

Returns a vector which interp uses to find the kth order polynomial that best fits the x and y data values

$Nominal := regress(x_func, y_func, k)$ $Nominal_fit(x) := interp(Nominal, x_func, y_func, x)$



Strength Factor of Safety:

Read the Corresponding Moment for the demand axial load ($P_0 = -3.9 \cdot kips$)

$M_{capacity} := Nominal_fit\left(\frac{P_0}{kips}\right)$ $M_{capacity} = 274.818$

Demand Moment: $M_{demand} := \frac{M_0}{kips \cdot in}$ $M_{demand} = 143$

Factor of Safety: $FS_{strength} := \frac{M_{capacity}}{M_{demand}}$ $FS_{strength} = 1.9$

FEMA 310 GUIDANCE

4.2.4.4 Component Strength

Component strength for all actions shall be taken as the expected strength, Q_{CE}. Unless calculated otherwise, the expected strength shall be assumed equal to the nominal strength multiplied by 1.25. Alternatively, if allowable stresses are used, nominal strengths shall be taken as the allowable values multiplied by the following values:

Steel	1.7
Masonry	2.5
Wood	2.0

Except for wood diaphragms and wood and masonry shear walls, the allowable values shall not include a one-third increase for short term loading.

When calculating capacities of deteriorated elements, the evaluating design professional shall make

US Army Corps of Engineers

Probability of Failure

CIDMO Type R floodwall
pile footings
18" Pile capacity

Comp by:KSM 12-8-11
Chkd by:

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. Situation

1. This structure does not meet the allowable compression loads for which it has been determined 99.8% reliability can be assigned. See mathcad sheets for calculations.
2. EM 1110-2-561 states that the coefficient of variation (COV) are the following:
   Compression Capacity 25%
   Tension Capacity 18%
3. Values used for original capacity check:
   Allowable Compression Capacity $P_{callow} := 37.17kip$
   Allowable Tension Capacity $P_{tallow} := 20.25kip$

IV. Variable Definitions

FS_M = Factor of Safety under mean material parameters

FS_u = Factor of Safety due to upper bound value of the ULTIMATE PILE COMPRESSION CAPACITY

FS_l = Factor of Safety due to lower bound value of the ULTIMATE PILE COMPRESSION CAPACITY

ΔF_C = Difference in Factors of Safety due to change in ULTIMATE PILE COMPRESSION CAPACITY

σ_F = Standard Deviation of the Factor of Safety

V_F = Coefficient of Variation of the Factor of Safety

β_{LN} = Lognormal Reliability Index

R = Reliability

P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. Calculating Factors of Safety

WATER AT TOP OF WALL

18-inch tapered piles

Actual Compression on Piles

$P_u := 42.8kip$

Ultimate Compression Capacity

$P_{cult} := 63.184kip$

$P_{cupper} := P_{cult} \cdot 1.25 = 78.98 \cdot kip$

$P_{clower} := P_{cult} \cdot 0.75 = 47.388 \cdot kip$

Mean Factor of Safety

$FS_M := \frac{P_{cult}}{P_u}$ $FS_M = 1.476$

Upper Axial Compression Capacity

$FS_u := \frac{P_{cupper}}{P_u}$ $FS_u = 1.845$

Lower Axial Compression Capacity

$FS_l := \frac{P_{clower}}{P_u}$ $FS_l = 1.107$

VI. Probability of Failure Calculation

$\Delta F_C := FS_u - FS_l$ $\qquad$ $\Delta F_C = 0.738$

$\sigma_F := \sqrt{\left(\frac{\Delta F_C}{2}\right)^2}$ $\qquad$ $\sigma_F = 0.369$

$V_F := \frac{\sigma_F}{FS_M}$ $\qquad$ $V_F = 0.25$

$\beta_{LN} := \frac{\ln\left(\frac{FS_M}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\qquad$ $\beta_{LN} = 1.459$

$R := \mathrm{cnorm}\left(\beta_{LN}\right)$ $\qquad$ $R = 92.77 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\qquad$ $P_F = 7.23 \cdot \%$

V. *Calculating Factors of Safety*

WATER AT 1-FOOT DOWN FROM TOP OF WALL

18-inch tapered piles

Actual Compression on Piles

$P_u := 38kip$

Ultimate Compression Capacity

$P_{cupper} := P_{cult} \cdot 1.25 = 78.98 \cdot kip$

$P_{clower} := P_{cult} \cdot 0.75 = 47.388 \cdot kip$

Mean Factor of Safety

$FS_M := \frac{P_{cult}}{P_u}$ $FS_M = 1.663$

Upper Axial Compression Capacity

$FS_u := \frac{P_{cupper}}{P_u}$ $FS_u = 2.078$

Lower Axial Compression Capacity

$FS_l := \frac{P_{clower}}{P_u}$ $FS_l = 1.247$

VI. Probability of Failure Calculation

$\Delta F_C := FS_u - FS_l$ $\qquad \Delta F_C = 0.831$

$\sigma_F := \sqrt{\left(\frac{\Delta F_C}{2}\right)^2}$ $\qquad \sigma_F = 0.416$

$V_F := \frac{\sigma_F}{FS_M}$ $\qquad V_F = 0.25$

$\beta_{LN} := \frac{\ln\left(\frac{FS_M}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\qquad \beta_{LN} = 1.942$

$R := \mathrm{cnorm}\left(\beta_{LN}\right)$ $\qquad R = 97.39 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\qquad P_F = 2.61 \cdot \%$

V. Calculating Factors of Safety

WATER AT 2-FOOT DOWN FROM TOP OF WALL

18-inch tapered piles

Actual Compression on Piles

$P_u := 33.6kip$

Ultimate Compression Capacity

$P_{cupper} := P_{cult} \cdot 1.25 = 78.98 \cdot kip$

$P_{clower} := P_{cult} \cdot 0.75 = 47.388 \cdot kip$

Mean Factor of Safety

$FS_M := \frac{P_{cult}}{P_u}$ $FS_M = 1.88$

Upper Axial Compression Capacity

$FS_u := \frac{P_{cupper}}{P_u}$ $FS_u = 2.351$

Lower Axial Compression Capacity

$FS_l := \frac{P_{clower}}{P_u}$ $FS_l = 1.41$

VI. Probability of Failure Calculation

$\Delta F_C := FS_u - FS_l$ $\qquad \Delta F_C = 0.94$

$\sigma_F := \sqrt{\left(\frac{\Delta F_C}{2}\right)^2}$ $\qquad \sigma_F = 0.47$

$V_F := \frac{\sigma_F}{FS_M}$ $\qquad V_F = 0.25$

$\beta_{LN} := \frac{\ln\left(\frac{FS_M}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\qquad \beta_{LN} = 2.442$

$R := \mathrm{cnorm}\left(\beta_{LN}\right)$ $\qquad R = 99.27 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\qquad P_F = 0.73 \cdot \%$

```
CPGA - CASE PILE GROUP ANALYSIS PROGRAM
RUN DATE: 07-DEC-2011   RUN TIME: 14.42.38


    FOR PILES WITH UNSUPPORTED HEIGHT:
         A. CPGA CANNOT CALCULATE PMAXMOM FOR NH TYPE SOIL
         B. THE ALLOWABLE STRESS CHECKS, ASC AND AST, ARE
            NOT FULLY DEVELOPED FOR UNSUPPORTED PILES.
            WORK IS IN PROGRESS TO COMPLETE THIS ASPECT OF CPGA.

    ELASTIC CENTER LOCATION IS NOT COMPUTED FOR 3-DIMENSIONAL PROBLEMS.


CID-MO SECTION TYPE R
DATA UNKNOWN - REJECTED.



THERE ARE  56 PILES AND
            1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                    X          Y          Z
                                  -----      -----      -----
WITH DIAGONAL COORDINATES = (    -20.50 ,     2.00 ,      .00 )
                            (     20.50 ,    11.50 ,      .00 )


*****************************************************************************

         PILE PROPERTIES AS INPUT


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .36050E+04   .42180E+04   .42180E+04   .22500E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   1    2    3    4    5    6    7    8    9   10   11   12   13   14


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .36050E+04   .10000E+04   .14400E+04   .12000E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

  15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
  31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
  47   48   49   50   51   52   53   54   55   56


*****************************************************************************

         SOIL DESCRIPTIONS AS INPUT


   NH     ESOIL      LENGTH       L            LU
         K/IN**3                 FT           FT
         .60000E-01    L       .20000E+02    .00000E+00

 ESOIL(ORIGINAL)     RGROUP     RCYCLIC
   K/IN**3
   .60000E-01        .1000E+01  .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   1    2    3    4    5    6    7    8    9   10   11   12   13   14


   NH     ESOIL      LENGTH       L            LU
         K/IN**3                 FT           FT
         .60000E-01    L       .15000E+02    .00000E+00

 ESOIL(ORIGINAL)     RGROUP     RCYCLIC
   K/IN**3
   .60000E-01        .1000E+01  .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

  15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
  31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
  47   48   49   50   51   52   53   54   55   56


*****************************************************************************
```

PILE STIFFNESSES AS CALCULATED FROM PROPERTIES

.14829E+03	.00000E+00	.00000E+00	.00000E+00	.66141E+04	.00000E+00
.00000E+00	.14829E+03	.00000E+00	-.66141E+04	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.67594E+04	.00000E+00	.00000E+00	.00000E+00
.00000E+00	-.66141E+04	.00000E+00	.47570E+06	.00000E+00	.00000E+00
.66141E+04	.00000E+00	.00000E+00	.00000E+00	.47570E+06	.00000E+00
.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00

THIS MATRIX APPLIES TO THE FOLLOWING PILES -

1

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 15
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 16
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 17
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 18
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 19
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 20
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 21
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 22
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 23
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 24
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 25
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 26
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 27
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 28
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 29

```
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   30
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   31
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   32
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   33
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   34
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   35
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   36
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   37
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   38
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   39
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   40
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   41
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   42
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   43
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   44
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   45
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   46
* * * * * * * * * * * * * * * * * * *
```

```
* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   47
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   48
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   49
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   50
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   51
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   52
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   53
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   54
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   55
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   56
* * * * * * * * * * * * * * * * * * * *


*******************************************************************************
```

PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	-18.00	2.00	.00	V	.00	20.00	F
2	-12.00	2.00	.00	V	.00	20.00	F
3	-6.00	2.00	.00	V	.00	20.00	F
4	.00	2.00	.00	V	.00	20.00	F
5	6.00	2.00	.00	V	.00	20.00	F
6	12.00	2.00	.00	V	.00	20.00	F
7	18.00	2.00	.00	V	.00	20.00	F
8	-18.00	6.00	.00	V	.00	20.00	F
9	-12.00	6.00	.00	V	.00	20.00	F
10	-6.00	6.00	.00	V	.00	20.00	F
11	.00	6.00	.00	V	.00	20.00	F
12	6.00	6.00	.00	V	.00	20.00	F
13	12.00	6.00	.00	V	.00	20.00	F
14	18.00	6.00	.00	V	.00	20.00	F
15	-20.50	11.50	.00	V	.00	15.00	F
16	-19.50	11.50	.00	V	.00	15.00	F
17	-18.50	11.50	.00	V	.00	15.00	F
18	-17.50	11.50	.00	V	.00	15.00	F
19	-16.50	11.50	.00	V	.00	15.00	F
20	-15.50	11.50	.00	V	.00	15.00	F
21	-14.50	11.50	.00	V	.00	15.00	F
22	-13.50	11.50	.00	V	.00	15.00	F
23	-12.50	11.50	.00	V	.00	15.00	F
24	-11.50	11.50	.00	V	.00	15.00	F
25	-10.50	11.50	.00	V	.00	15.00	F
26	-9.50	11.50	.00	V	.00	15.00	F
27	-8.50	11.50	.00	V	.00	15.00	F
28	-7.50	11.50	.00	V	.00	15.00	F
29	-6.50	11.50	.00	V	.00	15.00	F
30	-5.50	11.50	.00	V	.00	15.00	F

31	-4.50	11.50	.00	V	.00	15.00	F
32	-3.50	11.50	.00	V	.00	15.00	F
33	-2.50	11.50	.00	V	.00	15.00	F
34	-1.50	11.50	.00	V	.00	15.00	F
35	-.50	11.50	.00	V	.00	15.00	F
36	.50	11.50	.00	V	.00	15.00	F
37	1.50	11.50	.00	V	.00	15.00	F
38	2.50	11.50	.00	V	.00	15.00	F
39	3.50	11.50	.00	V	.00	15.00	F
40	4.50	11.50	.00	V	.00	15.00	F
41	5.50	11.50	.00	V	.00	15.00	F
42	6.50	11.50	.00	V	.00	15.00	F
43	7.50	11.50	.00	V	.00	15.00	F
44	8.50	11.50	.00	V	.00	15.00	F
45	9.50	11.50	.00	V	.00	15.00	F
46	10.50	11.50	.00	V	.00	15.00	F
47	11.50	11.50	.00	V	.00	15.00	F
48	12.50	11.50	.00	V	.00	15.00	F
49	13.50	11.50	.00	V	.00	15.00	F
50	14.50	11.50	.00	V	.00	15.00	F
51	15.50	11.50	.00	V	.00	15.00	F
52	16.50	11.50	.00	V	.00	15.00	F
53	17.50	11.50	.00	V	.00	15.00	F
54	18.50	11.50	.00	V	.00	15.00	F
55	19.50	11.50	.00	V	.00	15.00	F
56	20.50	11.50	.00	V	.00	15.00	F

						910.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-291.7	295.4	571.7	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.61279E+04	.00000E+00	.00000E+00	.00000E+00	.23837E+06	-.65881E+06
.00000E+00	.55780E+04	.00000E+00	-.20972E+06	.00000E+00	-.50932E-10
.00000E+00	.00000E+00	.29651E+06	.32402E+08	.00000E+00	.00000E+00
.00000E+00	-.20972E+06	.32402E+08	.41301E+10	.00000E+00	-.34925E-09
.23837E+06	.00000E+00	.00000E+00	.00000E+00	.62484E+10	-.24561E+08
-.65881E+06	-.50932E-10	.00000E+00	-.34925E-09	-.24561E+08	.20028E+09

56 PILES 1 LOAD CASES

LOAD CASE 1. NUMBER OF FAILURES = 56. NUMBER OF PILES IN TENSION = 42.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	-.2329E-17	-.5464E-01	.7833E-02	-.6257E-04	.3701E-23	-.2167E-19

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE
# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
2	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*

3	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
4	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
5	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
6	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
7	.0	-7.7	42.8	331.6	.0	.0	3.90	.76	.00	.00*
8	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
9	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
10	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
11	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
12	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
13	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
14	.0	-7.7	22.5	331.6	.0	.0	2.05	.83	.00	.00*
15	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
16	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
17	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
18	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
19	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
20	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
21	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
22	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
23	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
24	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
25	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
26	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
27	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
28	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
29	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
30	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
31	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
32	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
33	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
34	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
35	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
36	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
37	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
38	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
39	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
40	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
41	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
42	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
43	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
44	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
45	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
46	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
47	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
48	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
49	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
50	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
51	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
52	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
53	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
54	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
55	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*
56	.0	-4.4	-3.9	143.0	.0	.0	7.26	1.48	.00	.00*

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-7.7	42.8	331.6	.0	.0
2	.0	-7.7	42.8	331.6	.0	.0
3	.0	-7.7	42.8	331.6	.0	.0
4	.0	-7.7	42.8	331.6	.0	.0
5	.0	-7.7	42.8	331.6	.0	.0
6	.0	-7.7	42.8	331.6	.0	.0
7	.0	-7.7	42.8	331.6	.0	.0
8	.0	-7.7	22.5	331.6	.0	.0
9	.0	-7.7	22.5	331.6	.0	.0
10	.0	-7.7	22.5	331.6	.0	.0
11	.0	-7.7	22.5	331.6	.0	.0
12	.0	-7.7	22.5	331.6	.0	.0
13	.0	-7.7	22.5	331.6	.0	.0
14	.0	-7.7	22.5	331.6	.0	.0
15	.0	-4.4	-3.9	143.0	.0	.0
16	.0	-4.4	-3.9	143.0	.0	.0
17	.0	-4.4	-3.9	143.0	.0	.0
18	.0	-4.4	-3.9	143.0	.0	.0
19	.0	-4.4	-3.9	143.0	.0	.0
20	.0	-4.4	-3.9	143.0	.0	.0
21	.0	-4.4	-3.9	143.0	.0	.0

22	.0	-4.4	-3.9	143.0	.0	.0
23	.0	-4.4	-3.9	143.0	.0	.0
24	.0	-4.4	-3.9	143.0	.0	.0
25	.0	-4.4	-3.9	143.0	.0	.0
26	.0	-4.4	-3.9	143.0	.0	.0
27	.0	-4.4	-3.9	143.0	.0	.0
28	.0	-4.4	-3.9	143.0	.0	.0
29	.0	-4.4	-3.9	143.0	.0	.0
30	.0	-4.4	-3.9	143.0	.0	.0
31	.0	-4.4	-3.9	143.0	.0	.0
32	.0	-4.4	-3.9	143.0	.0	.0
33	.0	-4.4	-3.9	143.0	.0	.0
34	.0	-4.4	-3.9	143.0	.0	.0
35	.0	-4.4	-3.9	143.0	.0	.0
36	.0	-4.4	-3.9	143.0	.0	.0
37	.0	-4.4	-3.9	143.0	.0	.0
38	.0	-4.4	-3.9	143.0	.0	.0
39	.0	-4.4	-3.9	143.0	.0	.0
40	.0	-4.4	-3.9	143.0	.0	.0
41	.0	-4.4	-3.9	143.0	.0	.0
42	.0	-4.4	-3.9	143.0	.0	.0
43	.0	-4.4	-3.9	143.0	.0	.0
44	.0	-4.4	-3.9	143.0	.0	.0
45	.0	-4.4	-3.9	143.0	.0	.0
46	.0	-4.4	-3.9	143.0	.0	.0
47	.0	-4.4	-3.9	143.0	.0	.0
48	.0	-4.4	-3.9	143.0	.0	.0
49	.0	-4.4	-3.9	143.0	.0	.0
50	.0	-4.4	-3.9	143.0	.0	.0
51	.0	-4.4	-3.9	143.0	.0	.0
52	.0	-4.4	-3.9	143.0	.0	.0
53	.0	-4.4	-3.9	143.0	.0	.0
54	.0	-4.4	-3.9	143.0	.0	.0
55	.0	-4.4	-3.9	143.0	.0	.0
56	.0	-4.4	-3.9	143.0	.0	.0

NO FILES WERE GENERATED DURING THIS RUN.
Stop - Program terminated.

```
CPGA - CASE PILE GROUP ANALYSIS PROGRAM
RUN DATE: 08-DEC-2011   RUN TIME: 21.02.37


    FOR PILES WITH UNSUPPORTED HEIGHT:
         A. CPGA CANNOT CALCULATE PMAXMOM FOR NH TYPE SOIL
         B. THE ALLOWABLE STRESS CHECKS, ASC AND AST, ARE
            NOT FULLY DEVELOPED FOR UNSUPPORTED PILES.
            WORK IS IN PROGRESS TO COMPLETE THIS ASPECT OF CPGA.

    ELASTIC CENTER LOCATION IS NOT COMPUTED FOR 3-DIMENSIONAL PROBLEMS.


CID-MO SECTION TYPE R
DATA UNKNOWN - REJECTED.



THERE ARE  56 PILES AND
            1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                    X          Y          Z
                                  -----      -----      -----
WITH DIAGONAL COORDINATES = (    -20.50 ,     2.00 ,      .00 )
                            (     20.50 ,    11.50 ,      .00 )


*****************************************************************************

         PILE PROPERTIES AS INPUT


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .36050E+04   .42180E+04   .42180E+04   .22500E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   1    2    3    4    5    6    7    8    9   10   11   12   13   14


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .36050E+04   .10000E+04   .14400E+04   .12000E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

  15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
  31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
  47   48   49   50   51   52   53   54   55   56


*****************************************************************************

         SOIL DESCRIPTIONS AS INPUT


   NH     ESOIL      LENGTH       L            LU
         K/IN**3                 FT           FT
         .60000E-01    L       .20000E+02    .00000E+00

 ESOIL(ORIGINAL)     RGROUP     RCYCLIC
   K/IN**3
   .60000E-01        .1000E+01  .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   1    2    3    4    5    6    7    8    9   10   11   12   13   14


   NH     ESOIL      LENGTH       L            LU
         K/IN**3                 FT           FT
         .60000E-01    L       .15000E+02    .00000E+00

 ESOIL(ORIGINAL)     RGROUP     RCYCLIC
   K/IN**3
   .60000E-01        .1000E+01  .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

  15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
  31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
  47   48   49   50   51   52   53   54   55   56


*****************************************************************************
```

PILE STIFFNESSES AS CALCULATED FROM PROPERTIES

.14829E+03	.00000E+00	.00000E+00	.00000E+00	.66141E+04	.00000E+00
.00000E+00	.14829E+03	.00000E+00	-.66141E+04	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.67594E+04	.00000E+00	.00000E+00	.00000E+00
.00000E+00	-.66141E+04	.00000E+00	.47570E+06	.00000E+00	.00000E+00
.66141E+04	.00000E+00	.00000E+00	.00000E+00	.47570E+06	.00000E+00
.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00

THIS MATRIX APPLIES TO THE FOLLOWING PILES -

1

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 15
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 16
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 17
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 18
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 19
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 20
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 21
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 22
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 23
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 24
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 25
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 26
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 27
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 28
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 29

```
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   30
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   31
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   32
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   33
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   34
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   35
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   36
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   37
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   38
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   39
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   40
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   41
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   42
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   43
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   44
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   45
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   46
* * * * * * * * * * * * * * * * * * *
```

```
* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   47
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   48
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   49
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   50
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   51
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   52
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   53
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   54
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   55
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   56
* * * * * * * * * * * * * * * * * * * *


*******************************************************************************
```

PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	-18.00	2.00	.00	V	.00	20.00	F
2	-12.00	2.00	.00	V	.00	20.00	F
3	-6.00	2.00	.00	V	.00	20.00	F
4	.00	2.00	.00	V	.00	20.00	F
5	6.00	2.00	.00	V	.00	20.00	F
6	12.00	2.00	.00	V	.00	20.00	F
7	18.00	2.00	.00	V	.00	20.00	F
8	-18.00	6.00	.00	V	.00	20.00	F
9	-12.00	6.00	.00	V	.00	20.00	F
10	-6.00	6.00	.00	V	.00	20.00	F
11	.00	6.00	.00	V	.00	20.00	F
12	6.00	6.00	.00	V	.00	20.00	F
13	12.00	6.00	.00	V	.00	20.00	F
14	18.00	6.00	.00	V	.00	20.00	F
15	-20.50	11.50	.00	V	.00	15.00	F
16	-19.50	11.50	.00	V	.00	15.00	F
17	-18.50	11.50	.00	V	.00	15.00	F
18	-17.50	11.50	.00	V	.00	15.00	F
19	-16.50	11.50	.00	V	.00	15.00	F
20	-15.50	11.50	.00	V	.00	15.00	F
21	-14.50	11.50	.00	V	.00	15.00	F
22	-13.50	11.50	.00	V	.00	15.00	F
23	-12.50	11.50	.00	V	.00	15.00	F
24	-11.50	11.50	.00	V	.00	15.00	F
25	-10.50	11.50	.00	V	.00	15.00	F
26	-9.50	11.50	.00	V	.00	15.00	F
27	-8.50	11.50	.00	V	.00	15.00	F
28	-7.50	11.50	.00	V	.00	15.00	F
29	-6.50	11.50	.00	V	.00	15.00	F
30	-5.50	11.50	.00	V	.00	15.00	F

31	-4.50	11.50	.00	V	.00	15.00	F
32	-3.50	11.50	.00	V	.00	15.00	F
33	-2.50	11.50	.00	V	.00	15.00	F
34	-1.50	11.50	.00	V	.00	15.00	F
35	-.50	11.50	.00	V	.00	15.00	F
36	.50	11.50	.00	V	.00	15.00	F
37	1.50	11.50	.00	V	.00	15.00	F
38	2.50	11.50	.00	V	.00	15.00	F
39	3.50	11.50	.00	V	.00	15.00	F
40	4.50	11.50	.00	V	.00	15.00	F
41	5.50	11.50	.00	V	.00	15.00	F
42	6.50	11.50	.00	V	.00	15.00	F
43	7.50	11.50	.00	V	.00	15.00	F
44	8.50	11.50	.00	V	.00	15.00	F
45	9.50	11.50	.00	V	.00	15.00	F
46	10.50	11.50	.00	V	.00	15.00	F
47	11.50	11.50	.00	V	.00	15.00	F
48	12.50	11.50	.00	V	.00	15.00	F
49	13.50	11.50	.00	V	.00	15.00	F
50	14.50	11.50	.00	V	.00	15.00	F
51	15.50	11.50	.00	V	.00	15.00	F
52	16.50	11.50	.00	V	.00	15.00	F
53	17.50	11.50	.00	V	.00	15.00	F
54	18.50	11.50	.00	V	.00	15.00	F
55	19.50	11.50	.00	V	.00	15.00	F
56	20.50	11.50	.00	V	.00	15.00	F

						910.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-251.0	288.2	772.9	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.61279E+04	.00000E+00	.00000E+00	.00000E+00	.23837E+06	-.65881E+06
.00000E+00	.55780E+04	.00000E+00	-.20972E+06	.00000E+00	-.50932E-10
.00000E+00	.00000E+00	.29651E+06	.32402E+08	.00000E+00	.00000E+00
.00000E+00	-.20972E+06	.32402E+08	.41301E+10	.00000E+00	-.34925E-09
.23837E+06	.00000E+00	.00000E+00	.00000E+00	.62484E+10	-.24561E+08
-.65881E+06	-.50932E-10	.00000E+00	-.34925E-09	-.24561E+08	.20028E+09

56 PILES 1 LOAD CASES

LOAD CASE 1. NUMBER OF FAILURES = 56. NUMBER OF PILES IN TENSION = 42.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	-.2006E-17	-.4705E-01	.6922E-02	-.5445E-04	.3187E-23	-.1866E-19

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE
# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*
2	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*

3	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*
4	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*
5	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*
6	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*
7	.0	-6.6	38.0	285.3	.0	.0	3.46	.71	.00	.00*
8	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
9	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
10	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
11	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
12	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
13	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
14	.0	-6.6	20.3	285.3	.0	.0	1.85	.77	.00	.00*
15	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
16	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
17	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
18	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
19	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
20	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
21	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
22	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
23	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
24	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
25	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
26	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
27	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
28	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
29	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
30	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
31	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
32	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
33	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
34	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
35	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
36	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
37	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
38	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
39	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
40	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
41	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
42	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
43	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
44	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
45	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
46	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
47	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
48	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
49	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
50	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
51	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
52	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
53	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
54	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
55	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*
56	.0	-3.8	-2.8	123.0	.0	.0	5.37	1.26	.00	.00*

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-6.6	38.0	285.3	.0	.0
2	.0	-6.6	38.0	285.3	.0	.0
3	.0	-6.6	38.0	285.3	.0	.0
4	.0	-6.6	38.0	285.3	.0	.0
5	.0	-6.6	38.0	285.3	.0	.0
6	.0	-6.6	38.0	285.3	.0	.0
7	.0	-6.6	38.0	285.3	.0	.0
8	.0	-6.6	20.3	285.3	.0	.0
9	.0	-6.6	20.3	285.3	.0	.0
10	.0	-6.6	20.3	285.3	.0	.0
11	.0	-6.6	20.3	285.3	.0	.0
12	.0	-6.6	20.3	285.3	.0	.0
13	.0	-6.6	20.3	285.3	.0	.0
14	.0	-6.6	20.3	285.3	.0	.0
15	.0	-3.8	-2.8	123.0	.0	.0
16	.0	-3.8	-2.8	123.0	.0	.0
17	.0	-3.8	-2.8	123.0	.0	.0
18	.0	-3.8	-2.8	123.0	.0	.0
19	.0	-3.8	-2.8	123.0	.0	.0
20	.0	-3.8	-2.8	123.0	.0	.0
21	.0	-3.8	-2.8	123.0	.0	.0

22	.0	-3.8	-2.8	123.0	.0	.0
23	.0	-3.8	-2.8	123.0	.0	.0
24	.0	-3.8	-2.8	123.0	.0	.0
25	.0	-3.8	-2.8	123.0	.0	.0
26	.0	-3.8	-2.8	123.0	.0	.0
27	.0	-3.8	-2.8	123.0	.0	.0
28	.0	-3.8	-2.8	123.0	.0	.0
29	.0	-3.8	-2.8	123.0	.0	.0
30	.0	-3.8	-2.8	123.0	.0	.0
31	.0	-3.8	-2.8	123.0	.0	.0
32	.0	-3.8	-2.8	123.0	.0	.0
33	.0	-3.8	-2.8	123.0	.0	.0
34	.0	-3.8	-2.8	123.0	.0	.0
35	.0	-3.8	-2.8	123.0	.0	.0
36	.0	-3.8	-2.8	123.0	.0	.0
37	.0	-3.8	-2.8	123.0	.0	.0
38	.0	-3.8	-2.8	123.0	.0	.0
39	.0	-3.8	-2.8	123.0	.0	.0
40	.0	-3.8	-2.8	123.0	.0	.0
41	.0	-3.8	-2.8	123.0	.0	.0
42	.0	-3.8	-2.8	123.0	.0	.0
43	.0	-3.8	-2.8	123.0	.0	.0
44	.0	-3.8	-2.8	123.0	.0	.0
45	.0	-3.8	-2.8	123.0	.0	.0
46	.0	-3.8	-2.8	123.0	.0	.0
47	.0	-3.8	-2.8	123.0	.0	.0
48	.0	-3.8	-2.8	123.0	.0	.0
49	.0	-3.8	-2.8	123.0	.0	.0
50	.0	-3.8	-2.8	123.0	.0	.0
51	.0	-3.8	-2.8	123.0	.0	.0
52	.0	-3.8	-2.8	123.0	.0	.0
53	.0	-3.8	-2.8	123.0	.0	.0
54	.0	-3.8	-2.8	123.0	.0	.0
55	.0	-3.8	-2.8	123.0	.0	.0
56	.0	-3.8	-2.8	123.0	.0	.0

NO FILES WERE GENERATED DURING THIS RUN.
Stop - Program terminated.

```
CPGA - CASE PILE GROUP ANALYSIS PROGRAM
RUN DATE: 08-DEC-2011   RUN TIME: 21.03.50


    FOR PILES WITH UNSUPPORTED HEIGHT:
         A. CPGA CANNOT CALCULATE PMAXMOM FOR NH TYPE SOIL
         B. THE ALLOWABLE STRESS CHECKS, ASC AND AST, ARE
            NOT FULLY DEVELOPED FOR UNSUPPORTED PILES.
            WORK IS IN PROGRESS TO COMPLETE THIS ASPECT OF CPGA.

    ELASTIC CENTER LOCATION IS NOT COMPUTED FOR 3-DIMENSIONAL PROBLEMS.


CID-MO SECTION TYPE R
DATA UNKNOWN - REJECTED.



THERE ARE  56 PILES AND
            1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                    X          Y          Z
                                  -----      -----      -----
WITH DIAGONAL COORDINATES = (    -20.50 ,     2.00 ,      .00 )
                            (     20.50 ,    11.50 ,      .00 )


******************************************************************************

         PILE PROPERTIES AS INPUT


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .36050E+04   .42180E+04   .42180E+04   .22500E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   1    2    3    4    5    6    7    8    9   10   11   12   13   14


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .36050E+04   .10000E+04   .14400E+04   .12000E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

  15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
  31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
  47   48   49   50   51   52   53   54   55   56


******************************************************************************

         SOIL DESCRIPTIONS AS INPUT


   NH     ESOIL      LENGTH       L            LU
         K/IN**3                 FT           FT
         .60000E-01    L       .20000E+02    .00000E+00

 ESOIL(ORIGINAL)     RGROUP     RCYCLIC
   K/IN**3
   .60000E-01        .1000E+01  .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   1    2    3    4    5    6    7    8    9   10   11   12   13   14


   NH     ESOIL      LENGTH       L            LU
         K/IN**3                 FT           FT
         .60000E-01    L       .15000E+02    .00000E+00

 ESOIL(ORIGINAL)     RGROUP     RCYCLIC
   K/IN**3
   .60000E-01        .1000E+01  .1000E+01

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

  15   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
  31   32   33   34   35   36   37   38   39   40   41   42   43   44   45   46
  47   48   49   50   51   52   53   54   55   56


******************************************************************************
```

PILE STIFFNESSES AS CALCULATED FROM PROPERTIES

.14829E+03	.00000E+00	.00000E+00	.00000E+00	.66141E+04	.00000E+00
.00000E+00	.14829E+03	.00000E+00	-.66141E+04	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.67594E+04	.00000E+00	.00000E+00	.00000E+00
.00000E+00	-.66141E+04	.00000E+00	.47570E+06	.00000E+00	.00000E+00
.66141E+04	.00000E+00	.00000E+00	.00000E+00	.47570E+06	.00000E+00
.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00

THIS MATRIX APPLIES TO THE FOLLOWING PILES -

1

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 15
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 16
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 17
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 18
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 19
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 20
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 21
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 22
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 23
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 24
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 25
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 26
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 27
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 28
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE 29

```
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   30
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   31
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   32
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   33
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   34
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   35
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   36
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   37
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   38
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   39
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   40
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   41
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   42
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   43
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   44
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   45
* * * * * * * * * * * * * * * * * * *

* * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   46
* * * * * * * * * * * * * * * * * * *
```

```
* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   47
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   48
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   49
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   50
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   51
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   52
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   53
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   54
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   55
* * * * * * * * * * * * * * * * * * * *


* * * * * * * * * * * * * * * * * * * *
LENGTH LESS THAN 5T2 FOR PILE   56
* * * * * * * * * * * * * * * * * * * *


*******************************************************************************
```

PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	-18.00	2.00	.00	V	.00	20.00	F
2	-12.00	2.00	.00	V	.00	20.00	F
3	-6.00	2.00	.00	V	.00	20.00	F
4	.00	2.00	.00	V	.00	20.00	F
5	6.00	2.00	.00	V	.00	20.00	F
6	12.00	2.00	.00	V	.00	20.00	F
7	18.00	2.00	.00	V	.00	20.00	F
8	-18.00	6.00	.00	V	.00	20.00	F
9	-12.00	6.00	.00	V	.00	20.00	F
10	-6.00	6.00	.00	V	.00	20.00	F
11	.00	6.00	.00	V	.00	20.00	F
12	6.00	6.00	.00	V	.00	20.00	F
13	12.00	6.00	.00	V	.00	20.00	F
14	18.00	6.00	.00	V	.00	20.00	F
15	-20.50	11.50	.00	V	.00	15.00	F
16	-19.50	11.50	.00	V	.00	15.00	F
17	-18.50	11.50	.00	V	.00	15.00	F
18	-17.50	11.50	.00	V	.00	15.00	F
19	-16.50	11.50	.00	V	.00	15.00	F
20	-15.50	11.50	.00	V	.00	15.00	F
21	-14.50	11.50	.00	V	.00	15.00	F
22	-13.50	11.50	.00	V	.00	15.00	F
23	-12.50	11.50	.00	V	.00	15.00	F
24	-11.50	11.50	.00	V	.00	15.00	F
25	-10.50	11.50	.00	V	.00	15.00	F
26	-9.50	11.50	.00	V	.00	15.00	F
27	-8.50	11.50	.00	V	.00	15.00	F
28	-7.50	11.50	.00	V	.00	15.00	F
29	-6.50	11.50	.00	V	.00	15.00	F
30	-5.50	11.50	.00	V	.00	15.00	F

31	-4.50	11.50	.00	V	.00	15.00	F
32	-3.50	11.50	.00	V	.00	15.00	F
33	-2.50	11.50	.00	V	.00	15.00	F
34	-1.50	11.50	.00	V	.00	15.00	F
35	-.50	11.50	.00	V	.00	15.00	F
36	.50	11.50	.00	V	.00	15.00	F
37	1.50	11.50	.00	V	.00	15.00	F
38	2.50	11.50	.00	V	.00	15.00	F
39	3.50	11.50	.00	V	.00	15.00	F
40	4.50	11.50	.00	V	.00	15.00	F
41	5.50	11.50	.00	V	.00	15.00	F
42	6.50	11.50	.00	V	.00	15.00	F
43	7.50	11.50	.00	V	.00	15.00	F
44	8.50	11.50	.00	V	.00	15.00	F
45	9.50	11.50	.00	V	.00	15.00	F
46	10.50	11.50	.00	V	.00	15.00	F
47	11.50	11.50	.00	V	.00	15.00	F
48	12.50	11.50	.00	V	.00	15.00	F
49	13.50	11.50	.00	V	.00	15.00	F
50	14.50	11.50	.00	V	.00	15.00	F
51	15.50	11.50	.00	V	.00	15.00	F
52	16.50	11.50	.00	V	.00	15.00	F
53	17.50	11.50	.00	V	.00	15.00	F
54	18.50	11.50	.00	V	.00	15.00	F
55	19.50	11.50	.00	V	.00	15.00	F
56	20.50	11.50	.00	V	.00	15.00	F

						910.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-213.0	281.1	937.4	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.61279E+04	.00000E+00	.00000E+00	.00000E+00	.23837E+06	-.65881E+06
.00000E+00	.55780E+04	.00000E+00	-.20972E+06	.00000E+00	-.50932E-10
.00000E+00	.00000E+00	.29651E+06	.32402E+08	.00000E+00	.00000E+00
.00000E+00	-.20972E+06	.32402E+08	.41301E+10	.00000E+00	-.34925E-09
.23837E+06	.00000E+00	.00000E+00	.00000E+00	.62484E+10	-.24561E+08
-.65881E+06	-.50932E-10	.00000E+00	-.34925E-09	-.24561E+08	.20028E+09

56 PILES 1 LOAD CASES

LOAD CASE 1. NUMBER OF FAILURES = 56. NUMBER OF PILES IN TENSION = 42.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	-.1704E-17	-.3997E-01	.6112E-02	-.4726E-04	.2708E-23	-.1585E-19

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE
# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*
2	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*

3	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*
4	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*
5	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*
6	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*
7	.0	-5.6	33.6	241.9	.0	.0	3.06	.67	.00	.00*
8	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
9	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
10	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
11	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
12	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
13	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
14	.0	-5.6	18.3	241.9	.0	.0	1.67	.72	.00	.00*
15	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
16	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
17	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
18	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
19	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
20	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
21	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
22	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
23	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
24	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
25	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
26	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
27	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
28	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
29	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
30	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
31	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
32	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
33	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
34	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
35	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
36	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
37	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
38	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
39	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
40	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
41	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
42	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
43	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
44	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
45	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
46	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
47	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
48	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
49	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
50	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
51	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
52	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
53	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
54	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
55	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*
56	.0	-3.2	-2.0	104.4	.0	.0	3.71	1.05	.00	.00*

****

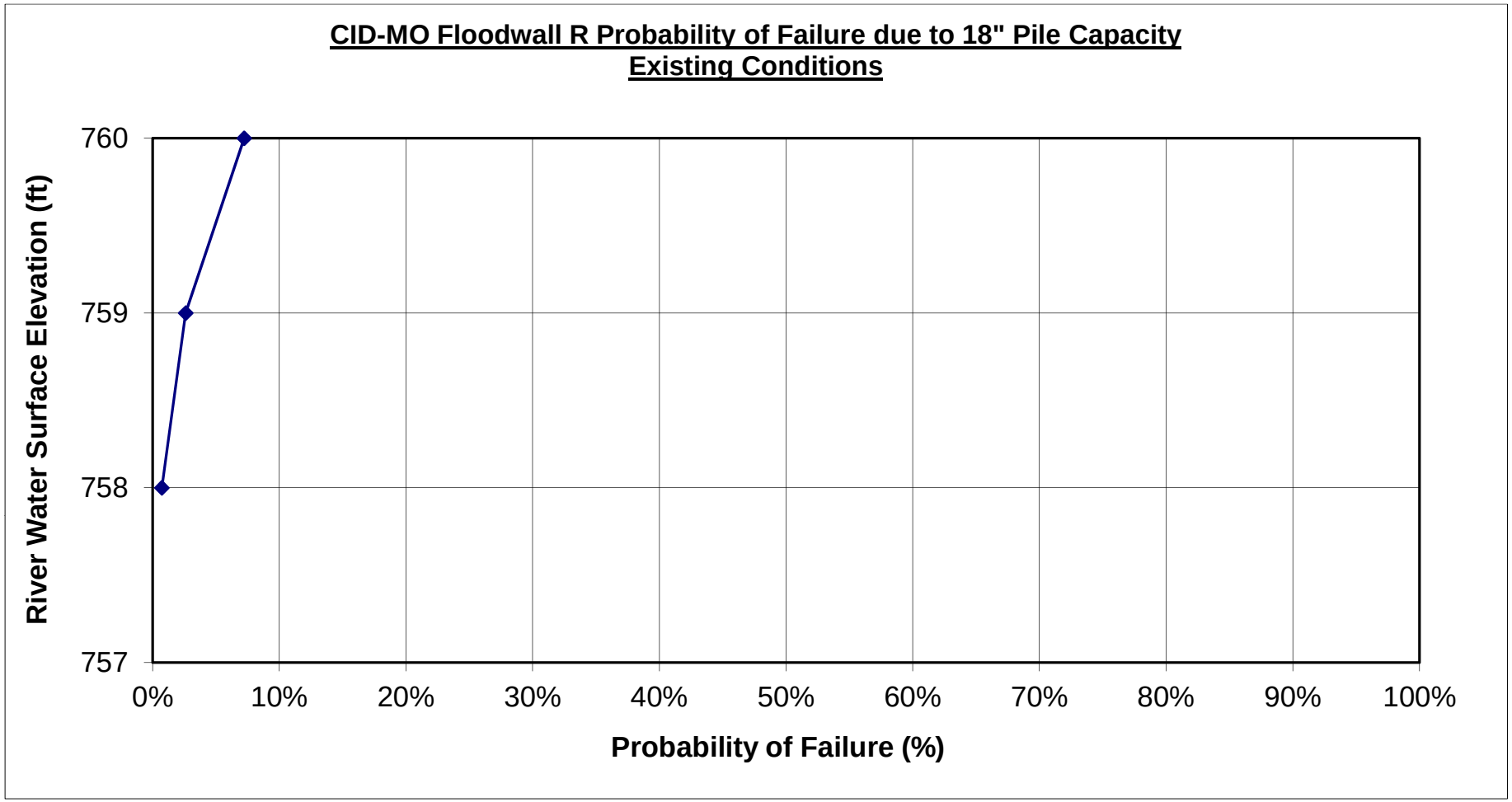
PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-5.6	33.6	241.9	.0	.0
2	.0	-5.6	33.6	241.9	.0	.0
3	.0	-5.6	33.6	241.9	.0	.0
4	.0	-5.6	33.6	241.9	.0	.0
5	.0	-5.6	33.6	241.9	.0	.0
6	.0	-5.6	33.6	241.9	.0	.0
7	.0	-5.6	33.6	241.9	.0	.0
8	.0	-5.6	18.3	241.9	.0	.0
9	.0	-5.6	18.3	241.9	.0	.0
10	.0	-5.6	18.3	241.9	.0	.0
11	.0	-5.6	18.3	241.9	.0	.0
12	.0	-5.6	18.3	241.9	.0	.0
13	.0	-5.6	18.3	241.9	.0	.0
14	.0	-5.6	18.3	241.9	.0	.0
15	.0	-3.2	-2.0	104.4	.0	.0
16	.0	-3.2	-2.0	104.4	.0	.0
17	.0	-3.2	-2.0	104.4	.0	.0
18	.0	-3.2	-2.0	104.4	.0	.0
19	.0	-3.2	-2.0	104.4	.0	.0
20	.0	-3.2	-2.0	104.4	.0	.0
21	.0	-3.2	-2.0	104.4	.0	.0

22	.0	-3.2	-2.0	104.4	.0	.0
23	.0	-3.2	-2.0	104.4	.0	.0
24	.0	-3.2	-2.0	104.4	.0	.0
25	.0	-3.2	-2.0	104.4	.0	.0
26	.0	-3.2	-2.0	104.4	.0	.0
27	.0	-3.2	-2.0	104.4	.0	.0
28	.0	-3.2	-2.0	104.4	.0	.0
29	.0	-3.2	-2.0	104.4	.0	.0
30	.0	-3.2	-2.0	104.4	.0	.0
31	.0	-3.2	-2.0	104.4	.0	.0
32	.0	-3.2	-2.0	104.4	.0	.0
33	.0	-3.2	-2.0	104.4	.0	.0
34	.0	-3.2	-2.0	104.4	.0	.0
35	.0	-3.2	-2.0	104.4	.0	.0
36	.0	-3.2	-2.0	104.4	.0	.0
37	.0	-3.2	-2.0	104.4	.0	.0
38	.0	-3.2	-2.0	104.4	.0	.0
39	.0	-3.2	-2.0	104.4	.0	.0
40	.0	-3.2	-2.0	104.4	.0	.0
41	.0	-3.2	-2.0	104.4	.0	.0
42	.0	-3.2	-2.0	104.4	.0	.0
43	.0	-3.2	-2.0	104.4	.0	.0
44	.0	-3.2	-2.0	104.4	.0	.0
45	.0	-3.2	-2.0	104.4	.0	.0
46	.0	-3.2	-2.0	104.4	.0	.0
47	.0	-3.2	-2.0	104.4	.0	.0
48	.0	-3.2	-2.0	104.4	.0	.0
49	.0	-3.2	-2.0	104.4	.0	.0
50	.0	-3.2	-2.0	104.4	.0	.0
51	.0	-3.2	-2.0	104.4	.0	.0
52	.0	-3.2	-2.0	104.4	.0	.0
53	.0	-3.2	-2.0	104.4	.0	.0
54	.0	-3.2	-2.0	104.4	.0	.0
55	.0	-3.2	-2.0	104.4	.0	.0
56	.0	-3.2	-2.0	104.4	.0	.0

NO FILES WERE GENERATED DURING THIS RUN.
Stop - Program terminated.


CID-MO Floodwall R Probability of Failure due to 18" Pile Capacity
Existing Conditions
River Water Surface Elevation (ft)
760
759
758
757
0%
10%
20%
30%
40%
50%
60%
70%
80%
90%
100%
Probability of Failure (%)

General Structural Exhibit 3:
Gatewell Sample Calculations

Gatewell 106+49, Existing Condition Strength and Uplift Stability Analysis:

Major tasks assocaited with this analysis:

1. Obtain the geometry for the gatewell and levee which are taken from the Record drawings.
2. Obtain the geotechnical parameters which are obtained from the Geotechnical Engineer.
3. Perform uplift analysis assuming gate is empty.

Calculation Convention Notes:

1. All input values are highlighted in *yellow*
2. Results are highlighted in green
3. Assumption notes are displayed in red

References:
Gatewell Sta. 106+49 elevations, location and reinforcing details from American Royal Arena Storm Sewer System for KC, MO Department of Public Works plans by Shafer, Kline and Warren 1973

Geotech Parameters:

Information Used to Calculate Structures HGL's - Blanket Information			
Station	Condition	Bottom of Landside Blanket Elevation	Top of Blanket Elevation
49+00 to 51+00	Existing	719	751
51+00 to 57+00	Existing	720	750
57+00 to 59+00	Existing	722	750
59+00 to 63+00	Existing	725	735
63+00 to 68+00	Existing	728	735
68+00 to 73+00	Existing	718	733
77+00 to 82+00	Existing	720	738
82+00 to 85+00	Existing	730	740
85+00 to 88+00	Existing	730	742
88+00 to 90+00	Existing	725	732
92+00 to 94+50	Existing	735	744
102+00 to 107+00	Existing	730	742
119+00 to 127+00	Existing	722	756
136+00 to 143+00	Existing	725	753
150+00 to 153+00	Existing	725	751
159+00 to 163+00	Existing	725	745

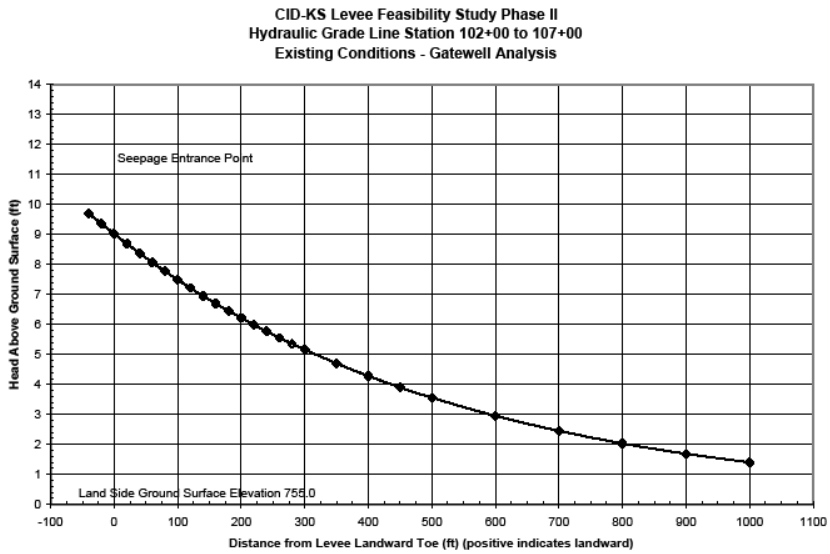
Geotech Parameters (con't):

Recommended Parameters for Phase II Feasibility Study

	Unit Weight (pcf)		Drained Shear Strength		Undrained Shear Strength*	
Material	Moist	Saturated	♦' (degrees)	c' (psf)	♦ (degrees)	c (psf)
Embankment	115	120	29	0	0	1000
Fill (cinders/rubble)	110	115	21	0	N/A	N/A
Foundation Blanket	110	115	24	0	0	500
Foundation Sand	115	120	31	0	N/A	N/A

*Estimated from average strength values for these materials

Hydraulic Gradeline:



HGL & blanket elevations provided by Glen Bellew 8/8/07

Soil Elevation Assumed by geotech: $El_{soil_geotech} \equiv 0ft$

Elevation of the HGL using geotech datum: $HGL_{geotech} \equiv 764ft$

Elevation of bottom of blanket: $El_{bot_blanket} \equiv 730ft$

Input Values

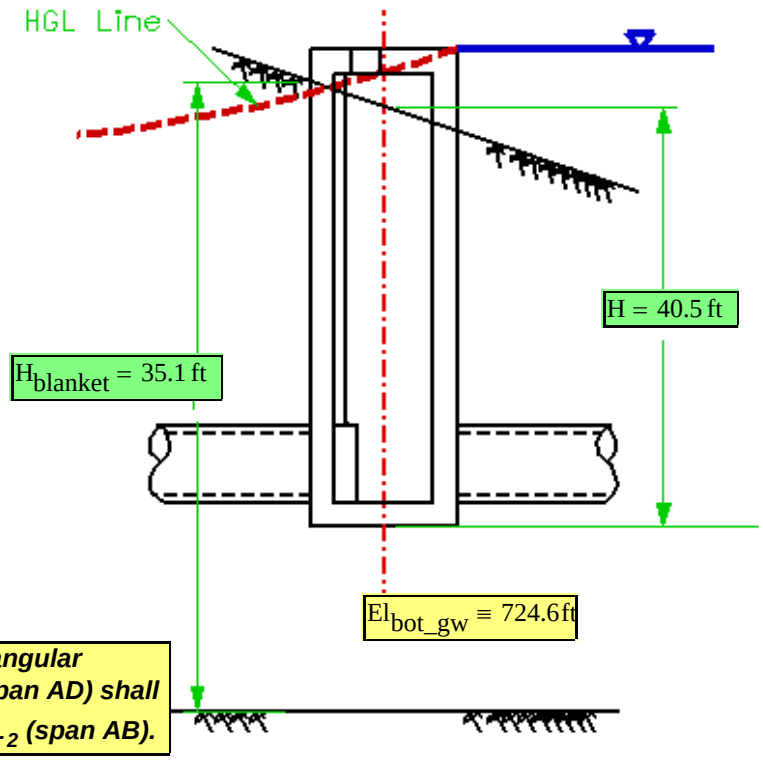
From record drwg $El_{top_gw} \equiv 765.1\,ft$

Top of floodwall EL=xxxxx'

This is determined from the Record drawings

$El_{soil_at_gw} \equiv El_{top_gw}$

$El_{soil_at_gw} = 765.1\ ft$



$H_{blanket} \equiv El_{soil_at_gw} - El_{bot_blanket}$

The HGL from the geotech needs to be adjusted using the soil elevation at the gatewell as the datum:

$HGL := HGL_{geotech} \cdots + 0ft - \left(El_{soil_at_gw} - El_{soil_geotech}\right)$

$HGL = -1.1\,ft$

Water head pressure in Wet Well under operating conditions.

$HEAD := 0ft$

Buried Height of

$H \equiv El_{soil_at_gw} - El_{bot_gw}$

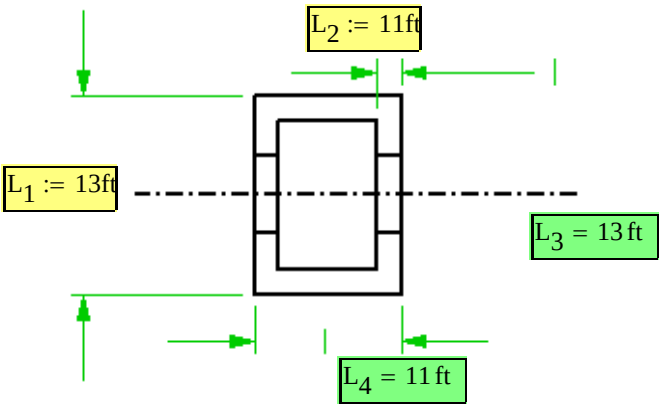
$H = 40.5\,ft$

NOTE: For rectangular gatewells, L_1 (span AD) shall be longer than L_2 (span AB).

Projected Height of Gatewell above soil

$H' := El_{top_gw} - El_{soil_at_gw}$

$H' = 0\ ft$



Width analyzed: $b := 12in$

$L_3 := L_1$

Floor Thickness $D_f := 24in$

$L_4 := L_2$

Slab clear cover $C_{cf} := 3.5in$

Slab effective depth $d_f := D_f - C_{cf}$

$d_f = 20.5 \cdot in$

Note: Upper case "L" represents the out-to-out dimension.
Lower case "l" representst the clear span dimension.

Length_Check :=	"Ok" if $L_1 \geq L_2$
	"NG - Assign larger dim to L1" otherwise

Length_Check = "Ok"

Wall Thickness

$D_1 := 18in$ Wall 1 (Span AD) wall thickness

$D_2 := 18in$ Wall 2 (Span AB) wall thickness

$D_3 := D_1$

$D_4 := D_2$

Wall 3 (Span BC) must be of the same thickness as Wall 1 (Span AD)
Wall 4 (Span CD) must be of the same thickness as Wall 2 (Span AB)

Clear Spans

$l_1 := L_1 - 2 \cdot D_2$ $l_1 = 10\,ft$ Clear span wall AD

$l_2 := L_2 - 2D_1$ $l_2 = 8\,ft$ Clear span of wall AB

$l_3 := l_1$ $l_3 = 10\,ft$ Clear span of wall BC

$l_4 := l_2$ $l_4 = 8\,ft$ Clear span of wall CD

Concrete Section Areas

$b = 1\,ft$

$A_{g1} := D_1 \cdot b$ $A_{g1} = 1.5\,ft^2$

$A_{g2} := D_2 \cdot b$ $A_{g2} = 1.5\,ft^2$

$A_{g3} := A_{g1}$ $A_{g3} = 1.5\,ft^2$

$A_{g4} := A_{g2}$ $A_{g4} = 1.5\,ft^2$

$d_f = 20.5 \cdot in$

Base Slab Reinforcment

Reinforcement information provided by the record drawings. $Bar_Sp_{slb} := 12in$

$BarNo_{slb} :=$

$BarNo_{slb} = 5$

Base Slab Reinforcement

$A_{slb} := \dfrac{A_{slb}}{\dfrac{Bar_Sp_{slb}}{12in}}$ $A_{slb} = 0.31 \cdot in^2$

This gatewell is a special case with large openings and inlet and outlet placed monolithically. Reinforcement given is above the openings.

Wall Reinforcement Areas At Elevation Bot of Gatewell:

Reinforcement information provided by the record drawings

I. WALL AD

$L_1 = 13\,ft$

END SUPPORT AND MID-SPAN REINFORCEMENT

$Bar_Spacing_{ADout} := 10in$ $Bar_Spacing_{ADin} := 6in$

$Bar_No_{ADout} :=$ $Bar_No_{ADout} = 7$

$Bar_No_{ADin} :=$ $Bar_No_{ADin} = 8$

Wall Reinforcement

The area of steel will be calculated /ft:

$A_{ADout} := \dfrac{A_{out}}{\dfrac{Bar_Spacing_{ADout}}{12in}}$ $A_{ADin} := \dfrac{A_{in}}{\dfrac{Bar_Spacing_{ADin}}{12in}}$ $A_{out} = 0.6 \cdot in^2$ $A_{in} = 0.79 \cdot in^2$

$A_{ADout} = 0.72 \cdot in^2$ $A_{ADin} = 1.58 \cdot in^2$

Reinf at wall AB is based on the Half-Section thru gate chamber detail on sheet 4 of 5.

II. WALL AB

$L_2 = 11\,ft$

END SUPPORT AND MID-SPAN REINFORCEMENT

$Bar_Spacing_{ABout} := 6in$ $Bar_Spacing_{ABin} := 6in$

$Bar_No_{ABout} :=$ $Bar_No_{ABout} = 6$

$Bar_No_{ABin} :=$ $Bar_No_{ABin} = 7$

Wall Reinforcement

The area of steel will be calculated /ft:

$$A_{ABout} := \frac{A_{out}}{\frac{Bar_Spacing_{ABout}}{12in}} \qquad A_{ABin} := \frac{A_{in}}{\frac{Bar_Spacing_{ABin}}{12in}} \qquad A_{out} = 0.44 \cdot in^2 \qquad A_{in} = 0.6 \cdot in^2$$

$$A_{ABout} = 0.88 \cdot in^2 \qquad A_{ABin} = 1.2 \cdot in^2$$

Reinforcement Bar Diameters

Reinforcement Clear Cover

Wall AD

$coverADout := 3in$

$coverADin := 3in$

Wall AB

$coverABout := 3in$

$coverABin := 3in$

Effective Depth:

These effective depths are used for the AD end analysis

$$D_1 = 18 \cdot in \qquad diaADin = 1 \cdot in \qquad diaADout = 0.875 \cdot in$$

$$d_{AD_in_end} := coverADin + \frac{diaADin}{2} \qquad d_{AD_out_end} := D_1 - coverADout - \frac{diaADout}{2}$$

$$d_{AD_in_end} = 3.5 \cdot in \qquad d_{AD_out_end} = 14.563 \cdot in$$

These effective depths are used for the AB end analysis

$$diaABin = 0.875 \cdot in \qquad diaABout = 0.75 \cdot in \qquad D_2 = 1.5\,ft$$

$$d_{AB_in_end} := coverABin + \frac{diaABin}{2} \qquad d_{AB_out_end} := D_2 - coverABout - \frac{diaABout}{2}$$

$$d_{AB_in_end} = 3.437 \cdot in \qquad d_{AB_out_end} = 14.625 \cdot in$$

These effective depths are used for the AD mid analysis

$$diaADin = 1 \cdot in \qquad diaADout = 0.875 \cdot in$$

$$d_{AD_in_mid} := D_1 - coverADin - \frac{diaADin}{2} \qquad d_{AD_out_mid} := coverADout + \frac{diaADout}{2}$$

$$d_{AD_in_mid} = 14.5 \cdot in \qquad d_{AD_out_mid} = 3.437 \cdot in$$

These effective depths are used for the AB mid analysis

$diaABin = 0.875 \cdot in$ $diaABout = 0.75 \cdot in$

$d_{AB_in_mid} := D_2 - coverABin - \frac{diaABin}{2}$

$d_{AB_in_mid} = 14.563 \cdot in$

$d_{AB_out_mid} := coverABout + \frac{diaABout}{2}$

$d_{AB_out_mid} = 3.375 \cdot in$

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete.
- The Portland Cement Association pamphlet, Engineered Concrete Structures, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Soil Properties $\phi_B := 21deg$ $\gamma_{soil} := 115pcf$

At Rest Soil Pressure $K_o := 1 - \sin(\phi_B)$ $K_o = 0.642$

Active Soil Pressure: $K_a := \tan\left(45deg - \frac{\phi_B}{2}\right)^2$ $K_a = 0.472$

Concrete Unit Weight $\gamma_c := 150pcf$ Concrete Properties $f'_c := 3.0ksi$ Water Unit Weight $\gamma_w := 62.4pcf$

Steel Properties $F_y := 40.0ksi$ Modulus of Elasticity: $Es := 29000ksi$

$\beta 1 := if\left(f'_c \le 4ksi, 0.85, if\left(f'_c > 8ksi, 0.65, 0.85 - 0.05 \cdot \frac{f'_c - 4ksi}{ksi}\right)\right)$ $\beta 1 = 0.85$ $f'_c = 3000\,psi$

Load & Resistance Factor Design Resistance and Load Factor Values

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$ Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Load Factor $\gamma_H := 1.0$ Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor $\gamma_X := 1.0$ Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) and Strength Reduction Factors (.85 for shear, 0.90 for bending) are not applied for in this initial evaluation. Instead a factor of safety is computed for the existing structure. If a strength concern is found, a design approach (applying these factors directly) will be taken for any recommended solutions.

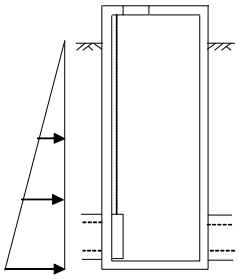
Wall Loading

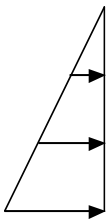
Wall Loading

WALLS

Special Case with Monolithically Placed exit and entrance:

Soil





$H = 40.5\,ft$

$H_2 := H$ OR $H_2 := H_2 - 7ft$

$H_2 = 40.5\,ft$

$W_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma_{soil} \cdot H_2$

$W_1 = 2472 \cdot psf$

Soil & Water

$H_2 = 33.5\,ft$

&

$H_3 := \text{if}\left[H < H_{blanket}, \frac{H}{H_{blanket}} \cdot \left(HGL + H_{blanket}\right) - HEAD, H + HGL - HEAD\right]$

$H_3 = 39.4\,ft$

$H_2 := H_3 - 7ft$

$W_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot \left(\gamma_{soil} - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_2 = 3152 \cdot psf$

Check to determine if water to top of wet well with reduction factor, γ_X, or soil loading is worst case

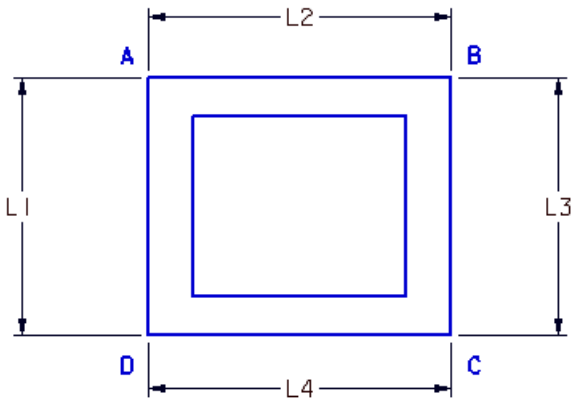
$W := \text{if}\left(W_1 > W_2, W_1, W_2\right) \cdot b$ $W = 3152 \cdot plf$

Moment Distribution of Gatewell Walls

NOTE: Check continuity of reinforcement at corners. If continuity is present, use moment distribution to determine the demand moment values. Otherwise, analyze the gatewell wall sections as simply supported beams.

The following calculations determine the distributed moment based on relative stiffness. In order for the calculations to be valid, the following must be true:
1. The structure has four walls. The walls that are parallel to each other are of the same thickness.
2. The EI value is based on the wall length and the wall thickness.
3. The walls are orthagonal.
4. The walls are exposed to uniform loading W, and $W = 3152 \cdot plf$ is the same for each wall.

Determine Moment and Shear:



Design Section Length

$L_1 = 13\,ft$ $L_2 = 11\,ft$ $L_3 = 13\,ft$ $L_4 = 11\,ft$

Moment of Inertia:

$b = 1\,ft$ $D_1 = 1.5\,ft$ $D_2 = 1.5\,ft$

$D_4 = 1.5\,ft$ $D_3 = 1.5\,ft$

$I_{beam1} := \frac{b \cdot D_1^3}{12}$ $I_{beam1} = 5832 \cdot in^4$

$I_{beam2} := \frac{b \cdot D_2^3}{12}$ $I_{beam2} = 5832 \cdot in^4$

$I_{beam3} := I_{beam1}$ $I_{beam3} = 5832 \cdot in^4$

$I_{beam4} := I_{beam2}$ $I_{beam4} = 5832 \cdot in^4$

Enter End Conditions:

All carryover factors are equal to 0.5 (idealize as a continuous beam over supports A, B, C and D).

$COF_A := 0.5$ $COF_B := COF_A$ $COF_C := COF_A$ $COF_D := COF_A$

$A := COF_A$ $B := COF_B$ $C := COF_C$ $D' := COF_D$

Fixed End Moments For A Uniform Load Between Fixed Supports using center of support to center of support distance per ACI 318-02 8.7.2 :

$L_1 = 13\,ft$ $L_2 = 132 \cdot in$ $L_3 = 13\,ft$ $L_4 = 11\,ft$ $W = 3152.38 \cdot plf$

$M_{fix_1} := \frac{W \cdot (L_1 - D_2)^2}{12}$ $M_{fix_1} = 34.742 \cdot kip \cdot ft$ $M_{fix_2} := \frac{W \cdot (L_2 - D_1)^2}{12}$ $M_{fix_2} = 23.709 \cdot kip \cdot ft$

$M_{fix_3} := \frac{W \cdot (L_3 - D_2)^2}{12}$ $M_{fix_3} = 34.742 \cdot kip \cdot ft$ $M_{fix_4} := \frac{W \cdot (L_4 - D_1)^2}{12}$ $M_{fix_4} = 23.709 \cdot kip \cdot ft$

Distribution Factors

	M_{AD}	M_{AB}	M_{BA}	M_{BC}	M_{CB}	M_{CD}	M_{DC}
COF	0.50	0.50	0.50	0.50	0.50	0.50	0.50
DF	0.46	0.54	0.54	0.46	0.46	0.54	0.54
FEM	-34.74	23.71	-23.71	34.74	-34.74	23.71	-23.71
D#1	5.057	5.976	-5.976	-5.057	5.057	5.976	-5.976
CO#1	-2.528	-2.988	2.988	2.528	-2.528	-2.988	2.988
DF#2	2.528	2.988	-2.988	-2.528	2.528	2.988	-2.988
CO#2	-1.264	-1.494	1.494	1.264	-1.264	-1.494	1.494
DF#3	1.264	1.494	-1.494	-1.264	1.264	1.494	-1.494
CO#3	-0.632	-0.747	0.747	0.632	-0.632	-0.747	0.747
DF#4	0.632	0.747	-0.747	-0.632	0.632	0.747	-0.747
CO#3	-0.316	-0.374	0.374	0.316	-0.316	-0.374	0.374
DF#4	0.316	0.374	-0.374	-0.316	0.316	0.374	-0.374
	-29.68	29.68	-29.68	29.68	-29.68	29.68	-29.68

Attach proper units to distributed values

$M_{ADend} := M'_{AD} \cdot ft \cdot kip$ $M_{ADend} := M'_{AB} \cdot ft \cdot kip$

$M_{CBend} := M'_{CB} \cdot ft \cdot kip$ $M_{DCend} := M'_{DC} \cdot ft \cdot kip$

End-of-Span Moments After Distribution

$M_{ADend} = -29.685 \cdot kip \cdot ft$ $M_{ADend} = -29.685 \cdot kip \cdot ft$

$M_{CBend} = -29.685 \cdot kip \cdot ft$ $M_{DCend} = -29.685 \cdot kip \cdot ft$

Note that the end-of-span moment value is the same at each joint because of uniform and equal loading present on all spans.

$W = 3152.38 \cdot plf$

Note: L_1 represents span AD, L_2 represents span AB, L_3 represents span BC and L_4 represents span CD

Mid-Span Moment Values

Moment Envelope Values

$$M_{UADin} := \left[\frac{W\cdot(L_1 - D_2)^2}{8} - |M_{ADend}|\right] \qquad M_{UADin} = 22.428\cdot kip\cdot ft \qquad \frac{W\cdot(L_1 - D_2)^2}{8} = 52.113\cdot kip\cdot ft$$

$$M_{UABin} := \left[\frac{W\cdot(L_2 - D_1)^2}{8} - |M_{ADend}|\right] \qquad M_{UABin} = 5.878\cdot kip\cdot ft \qquad \frac{W\cdot(L_2 - D_1)^2}{8} = 35.563\cdot kip\cdot ft$$

$$M_{UCBin} := \left[\frac{W\cdot(L_3 - D_2)^2}{8} - |M_{CBend}|\right] \qquad M_{UCBin} = 22.428\cdot kip\cdot ft \qquad \frac{W\cdot(L_3 - D_2)^2}{8} = 52.113\cdot kip\cdot ft$$

$$M_{UDCin} := \left[\frac{W\cdot(L_4 - D_1)^2}{8} - |M_{DCend}|\right] \qquad M_{UDCin} = 5.878\cdot kip\cdot ft \qquad \frac{W\cdot(L_4 - D_1)^2}{8} = 35.563\cdot kip\cdot ft$$

The demand moment will be taken at the support face IAW ACI 318-02, 8.7.2 and 8.7.3.

$$M_{UADface} := \left[M_{ADend} + \frac{W\cdot D_2}{8}\cdot(2L_1 - 3D_2)\right]$$

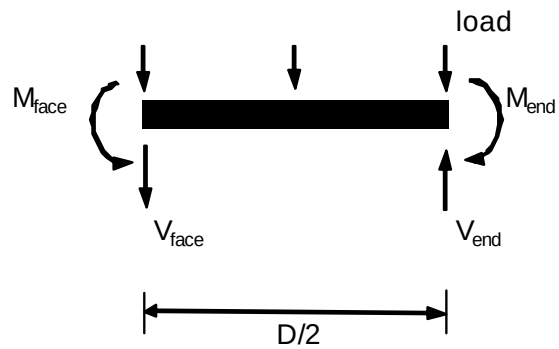
$$M_{UADface} = -16.977\cdot kip\cdot ft$$

$$M_{UABface} := \left[M_{ADend} + \frac{W\cdot D_1}{8}\cdot(2\cdot L_2 - 3D_1)\right]$$

$$M_{UABface} = -19.341\cdot kip\cdot ft$$

$$M_{UCBface} := M_{UADface} \qquad M_{UDCface} := M_{UABface}$$

Free Body Diagram

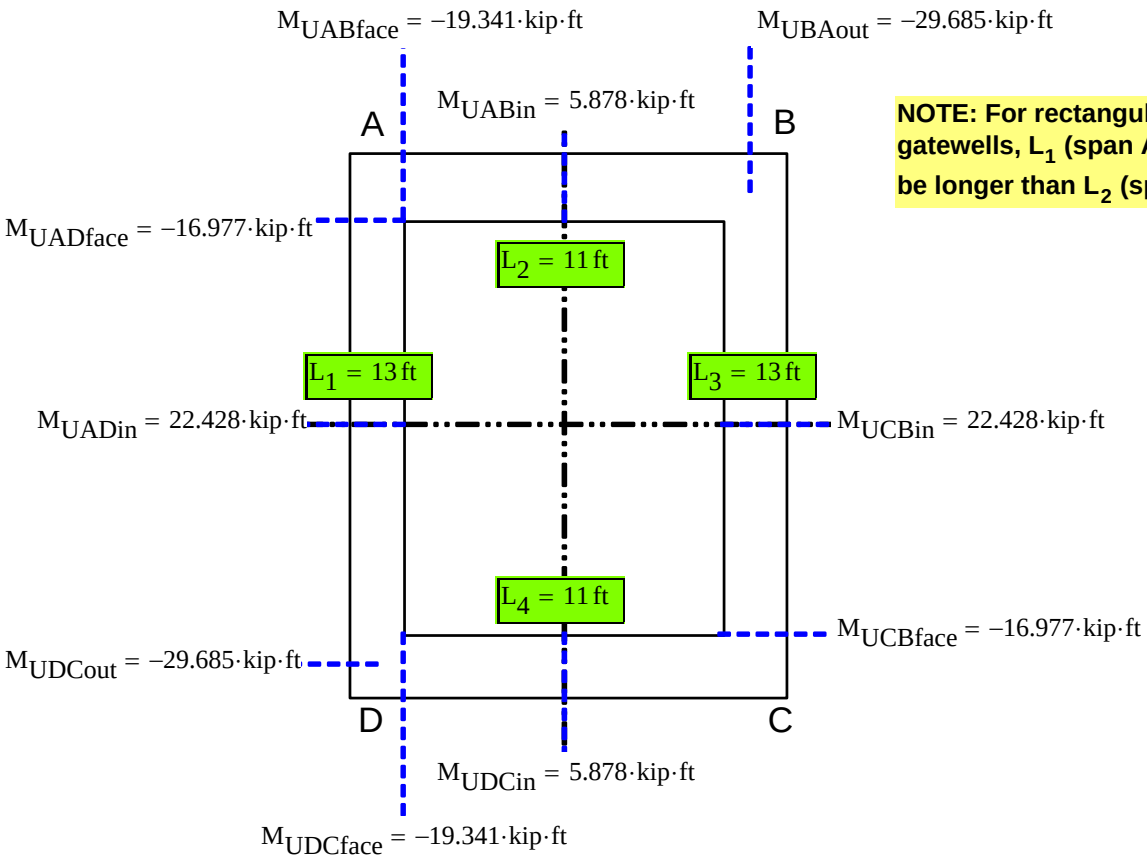


"D" represents the orthagonal wall thickness

$M_{UDCout} := M_{DCend}$ $\boxed{M_{UDCout} = -29.685 \cdot kip \cdot ft}$

$M_{UABout} := M_{ADend}$ $\boxed{M_{UABout} = -29.685 \cdot kip \cdot ft}$ $M_{UBAout} := M_{ADend}$

FACTORED MOMENT DISTRIBUTION VALUES



NOTE: For rectangular gatewells, L_1 (span AD) shall be longer than L_2 (span AB).

The "L" values are the "outside-to-outside" dimensions.

Ensure the reinforcement size entered earlier reflects the steel that will be in tension at the mid-span.
For example:
IF: The mid-span moment demand value obtained from moment distribution is negative.
THEN: The steel located near the wall's exterior face is in tension. Select the reinforcement size that is present near the wall exterior face.

Wall Thrust

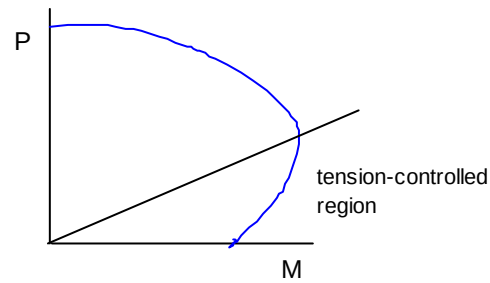
Thrust was considered when evaluating the wall section moment and shear capacity. A higher value of thrust within the tension-controlled region of the interaction diagram results in a higher nominal strength value. The calculated thrust is based upon the active (lower) value of lateral loading. This is deemed conservative relative to the at-rest (higher) value of lateral loading. Note that for lateral loading on the walls, the at-rest loading condition was used.

$K_a = 0.472$ $\gamma_{soil} = 115 \cdot pcf$ $H_2 = 33.5\ ft$

$W_{1active} := \gamma_L \cdot \gamma_H \cdot K_a \cdot \gamma_{soil} \cdot H_2$

$W_{2active} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_a \cdot \left(\gamma_{soil} - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_{1active} = 1819.748 \cdot psf$ $W_{2active} = 2854.097 \cdot psf$



$Wactive := if\left(W_{1active} > W_{2active}, W_{1active}, W_{2active}\right) \cdot b$

$Wactive = 2854.097 \cdot plf$ $W = 3152.38 \cdot plf$

Total Trust Acting On Wall AD:

$P_{ADthrust} := \frac{Wactive \cdot L_2}{2}$ $P_{ADthrust} = 15.698 \cdot kip$

Total Trust Acting On Wall AB:

$P_{ABthrust} := \frac{Wactive \cdot L_1}{2}$ $P_{ABthrust} = 18.552 \cdot kip$

Wall AD (Wall 1) Analysis

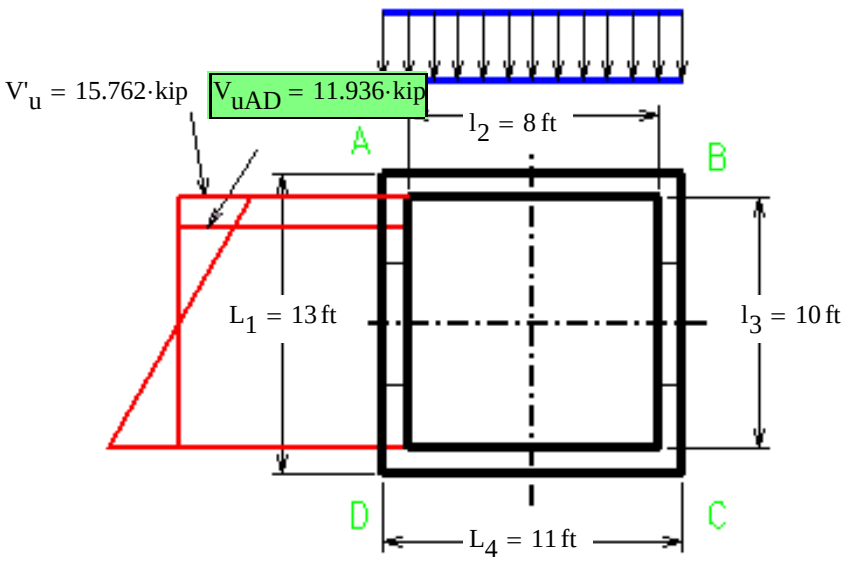
Shear Demand:

$W = 3152.38 \cdot plf \qquad l_1 = 10\,ft$

$V'_u := W \cdot \frac{l_1}{2} \qquad V'_u = 15.762 \cdot kip$

$d_{AD_out_end} = 14.563 \cdot in$

$V_{uAD} := V'_u \cdot \frac{\frac{l_1}{2} - d_{AD_out_end}}{\frac{l_1}{2}} \qquad V_{uAD} = 11.936 \cdot kip \qquad W = 3152.38 \cdot plf$



Shear Capacity and Factor of Safety:

$d_{AD_out_end} = 14.563 \cdot in \qquad \frac{P_{ADthrust}}{2000 \cdot A_{g1}} = 0.036\,psi \qquad f'_c = 3 \cdot ksi \quad b = 1\,ft$

Shear Concrete Resistance value at Distance d from Support ACI 318R-11.1.3.1

$\phi V_{n_AD} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{P_{ADthrust}}{2000 \cdot A_{g1}} \cdot \frac{in^2}{lbf} \right) \cdot \left(\sqrt{f'_c \cdot psi} \right) \cdot \left(b \cdot d_{AD_out_end} \right) \right]$ ACI EQ (11-4)

$\phi V_{n_AD} = 19.838 \cdot kip$

$V_{uAD} = 11.936 \cdot kip$

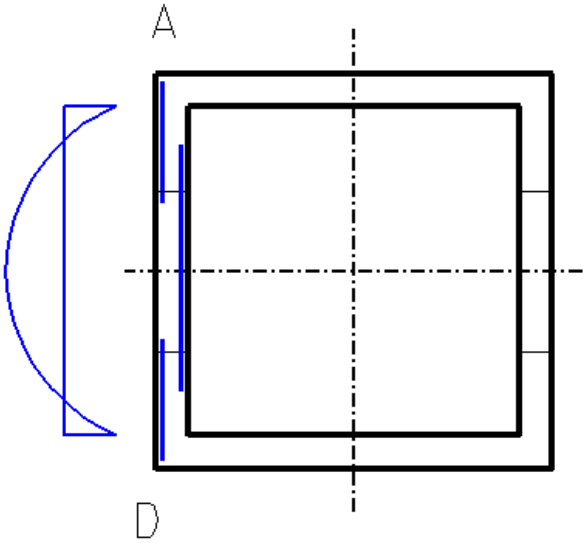
Factor of Safety $FS_{ADshear} := \frac{\phi V_{n_AD}}{V_{uAD}}$

$FS_{ADshear} = 1.662$

DEPTH OF EQUIVALENT RECTANGULAR STRESS BLOCK

Wall AD Con't

$F_y = 40 \cdot ksi$ $f'_c = 3 \cdot ksi$ $b = 1\,ft$ $W = 3.152 \cdot klf$ $L_1 = 13\,ft$



$A_{ADout} = 0.72 \cdot in^2$ $a_{AD_out} := \frac{A_{ADout} \cdot F_y}{0.85 f'_c \cdot b}$

$A_{ADin} = 1.58 \cdot in^2$ $a_{AD_in} := \frac{A_{ADin} \cdot F_y}{0.85 f'_c \cdot b}$

$a_{AD_out} = 0.941 \cdot in$ $a_{AD_in} = 2.065 \cdot in$

Negative Moment Redistribution

Wall AD Con't

ACI 318-02, 8.4 allows moment redistribution (plastic hinge formation). Refer to 8.4 for moment redistribution limitations.

Determine if redistribution is possible. To occur, the steel must yield prior to concrete crushing.

$\beta 1 = 0.85 \qquad f'_c = 3000\,psi$

Beta one value is equal to 0.85 for f'c equal to or less than 4000 psi

$c_{AD_out} := \frac{a_{AD_out}}{\beta 1} \qquad c_{AD_out} = 1.107 \cdot in \qquad c_{AD_in} := \frac{a_{AD_in}}{\beta 1} \qquad c_{AD_in} = 2.43 \cdot in$

The net tensile strain values

$\varepsilon_{tout} := 0.003 \cdot \left(\frac{d_{AD_out_end}}{c_{AD_out}} - 1 \right) \qquad \varepsilon_{tout} = 0.036$

Negative moment distribution is permissible when the tensile strain is greater than 0.0075, per ACI 318-02, 8.4

Reduce negative moment at face of wall AD end support and increase the positive mid-span moment (if applicable)

$R_{nface} := \frac{|M_{UADface}|}{\phi_B \cdot b \cdot d_{AD_out_end}^2} \qquad R_{nface} = 80.055 \cdot psi \qquad M_{UADface} = -16.977 \cdot kip \cdot ft$

$M_{UADin} = 22.428 \cdot kip \cdot ft$

$$\varepsilon'_{tface} := \begin{cases} 0.003 \cdot \left(\frac{\beta 1}{1 - \sqrt{1 - \frac{40}{17} \cdot \frac{R_{nface}}{f'_c}}} - 1 \right) & \text{if } (\varepsilon_{tout} \ge 0.0075) \wedge (M_{UADin} > 0) \\ 0 & \text{otherwise} \end{cases}$$

PCA 318-05 Notes, page 8-3

$\varepsilon'_{tface} = 0.077$

$$\varepsilon'_ADfacedelta := \begin{cases} \varepsilon'_{tface} \cdot 10 & \text{if } 0 < \varepsilon'_{tface} \le 0.020 \\ 0 & \text{if } \varepsilon'_{tface} = 0 \\ 0.20 & \text{otherwise} \end{cases} \qquad \varepsilon'_ADfacedelta = 0.200 \qquad \varepsilon'_{tface} \cdot 10 = 0.769$$

$$RedistAD := \begin{cases} \text{"Permitted"} & \text{if } \varepsilon'_{tface} \ne 0 \\ \text{"Not Permitted"} & \text{otherwise} \end{cases} \qquad RedistAD = \text{"Permitted"}$$

IF: The strain is greater than or equal to 0.0075 and the mid span moment is positive
THEN: Negative moment redistribution is permitted. OTHERWSE: Negative moment redistribution is not permitted and "N/A" is printed.

Redistributed (reduced) negative moment at the support face

$$M'_{UADface} := \begin{cases} M_{UADface} \cdot (1 - \varepsilon'_ADfacedelta) & \text{if } \varepsilon'_ADfacedelta \ne 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADface} = -13.582 \cdot kip \cdot ft \qquad M_{UADface} = -16.977 \cdot kip \cdot ft$

Wall AD Con't

$$M'_{UADout} := \begin{cases} M_{UDCout} \cdot (1 - \varepsilon'_ADfacedelta) & \text{if } \varepsilon'_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADout} = -23.748 \cdot kip \cdot ft$ $M_{UADface} = -16.977 \cdot kip \cdot ft$ $M_{UDCout} = -29.685 \cdot kip \cdot ft$

Corresponding redistributed (increased) mid-span moment

$$M'_{UADin} := \begin{cases} \frac{W \cdot (L_1 - D_2)^2}{8} \cdot (\gamma_L \cdot \gamma_H \cdot \gamma_X) + M'_{UADout} & \text{if } M'_{UADout} \neq \text{"N/A"} \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADout} = -23.748 \cdot kip \cdot ft$

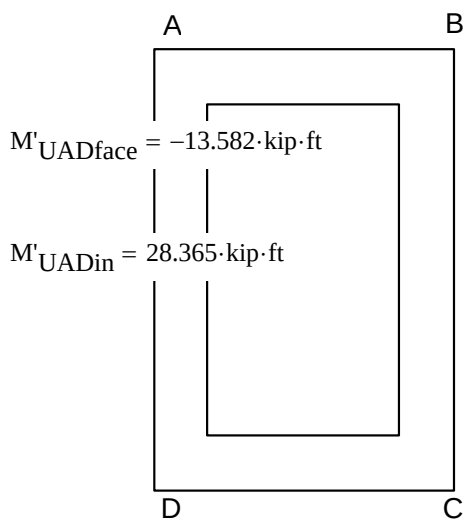
$M'_{UADin} = 28.365 \cdot kip \cdot ft$

$$\frac{W \cdot (L_1 - D_2)^2}{8} = 52.113 \cdot kip \cdot ft$$

$M_{UADin} = 22.428 \cdot kip \cdot ft$

Moments on Span AD After Redistribution

RedistAD = "Permitted"



Note: If redistribution is not allowed, "N/A" is shown.

Wall AD Magnified Moment Check

Wall AD Con't

The effects of wall slenderness shall be checked.

10.12.2 — In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy:

$$\frac{k l_u}{r} \le 34 - 12(M_1/M_2) \qquad (10\text{-}8)$$

where the term $[34 - 12M_1/M_2]$ shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

Table 8-3 Deflection Coefficient K

		K
1.	Cantilevers (deflection due to rotation at supports not included)	2.40
2.	Simple beams	1.0
3.	Continuous beams	1.2-0.2 M_o/M_a
4.	Fixed-hinged beams (midspan deflection)	0.80
5.	Fixed-hinged beams (maximum deflection using maximum moment)	0.74
6.	Fixed-fixed beams	0.60

For other types of loading, K values are given in Ref. 8.2.

M_o = Simple span moment at midspan $\left(\frac{w\ell^2}{8}\right)$

M_a = Net midspan moment.

$M_1 := M_{UADface}$ $\qquad$ $M_2 := M_{UADface}$

$M_1 = -16.977 \cdot kip \cdot ft$ $\qquad$ $M_2 = -16.977 \cdot kip \cdot ft$

From the table from the right, the k value for fixed fixed condition: $k := 0.6$

Ubraced length: $l_u := L_1$ $\qquad$ $l_u = 13\,ft$

Radius of Gyration, r:

Thickness of Wall: $D_1 = 1.5\,ft$

10.11.2 — It shall be permitted to take the radius of gyration r equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, it shall be permitted to compute the radius of gyration for the gross concrete section.

$r := 0.3 \cdot D_1$ $\qquad$ $r = 0.45\,ft$

$Slender_Ratio := k \cdot \frac{l_u}{r}$ $\qquad$ $Slender_Ratio = 17.333$

$$ADSlenderness_Check := if\left[Slender_Ratio \le \left[34 - 12 \cdot \left(\frac{M_1}{M_2}\right)\right], \text{"Ok"}, \text{"Consider Slenderness using ACI 10.10.1"}\right]$$

$ADSlenderness_Check = \text{"Ok"}$

IF: ADSlenderness_Check is OK
THEN: Moment magnification is not required

IF: ADSlenderness_Check is NOT OK
THEN: Wall is slender. Moment magnification per ACI 318-02, 10.12 shall be followed

Wall AD Mid-Span Nominal Moment Capacity

$L_1 = 13\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_1 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar_No_{ADin} = 8$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverADin = 3 \cdot in$ Cover -in

Axial Compression with ACI Reduction value: $\phi_B = 1$ $Bar_No_{ADin} = 8$ $diaADin = 1 \cdot in$

$row := 2$

Iterations $i := 1 .. 2$ $i =$

1
2

Bar area at i $A_1 := A_{ADout}$ $A_2 := A_{ADin}$ $A_i =$

0.72
1.58

$\cdot in^2$

$coverADout = 3 \cdot in$ $diaADout = 0.875 \cdot in$ $coverADin = 3 \cdot in$ $diaADin = 1 \cdot in$

$$dd_1 := \begin{cases} coverADout + \dfrac{diaADout}{2} & \text{if } M_{UADin} > 0ft \cdot kip \\ coverADin + \dfrac{diaADin}{2} & \text{otherwise} \end{cases}$$

$M_{UADin} = 22.428 \cdot kip \cdot ft$

$dd_1 = 3.437 \cdot in$

$$dd_2 := \begin{cases} D_1 - \left(coverADin + \dfrac{diaADin}{2}\right) & \text{if } M_{UADin} > 0ft \cdot kip \\ D_1 - \left(coverADout + \dfrac{diaADout}{2}\right) & \text{otherwise} \end{cases}$$

$dd_2 = 14.5 \cdot in$

Wall AD, Midspan Column Interaction Results

Wall AD Midspan

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_1 = 1.5\,ft \quad b = 1\,ft \quad \beta 1 = 0.85$

Given $\quad c := .2 \cdot D_1 \quad c = 3.6 \cdot in$

$$P_{ADthrust} = \left[\sum_{i=1}^{row} \left[\left[\begin{array}{l} \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots \quad \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) \\ F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \le -F_y \end{array} \right] \cdot A_i \right] \ldots \right] \cdot \phi_B$$

$$+ 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \quad c = 3.207 \cdot in \quad a := \text{if}\left(\beta 1 \cdot c > D_1, D_1, \beta 1 \cdot c\right) \quad a = 2.726 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(Es \cdot \varepsilon s_i > F_y, F_y, \text{if}\left(Es \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, Es \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > dd_i, 0.85 \cdot f'_c, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D_1}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.286 ft	-0	-6.262 ·ks	0.72 $\cdot in^2$	-4.509 ·kip	-25.079 ·kip·in
1.208	-0.011	-40	1.58	-63.2	347.6

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 323 \cdot kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D_1}{2} - \frac{a}{2}\right) \qquad Mc = 637 \cdot kip \cdot in$

$\phi M_{nADin1} := (Mc + Mst) \qquad \phi M_{nADin1} = 79.96 \cdot kip \cdot ft$

$$\phi M_{nADin} := \begin{cases} \phi M_{nADin1} & \text{if } M_{UADin} > 0ft \cdot kip \\ \phi M_{nADin1} \cdot (-1) & \text{otherwise} \end{cases}$$

Wall AD Mid-Span Nomimal Moment Strength $\quad \phi M_{nADin} = 79.959 \cdot kip \cdot ft$

Wall AD Mid-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ADmid}

Wall AD Mid-Span Con't

$\phi M_{nADin} = 79.959 \cdot kip \cdot ft$

$M_{UADin} = 22.428 \cdot kip \cdot ft$

$$FS_{ADin} := \left(\left| \frac{\phi M_{nADin}}{M_{UADin}} \right| \right)$$

$FS_{ADin} = 3.565$

Wall AD Mid-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ADmid}$

$\phi M_{nADin} = 79.959 \cdot kip \cdot ft$

Note: An apostrophe signifies a negative moment redistribution-related value.

$M'_{UADin} = 28.365 \cdot kip \cdot ft$

$$FS'_{ADin} := \begin{cases} \left| \frac{\phi M_{nADin}}{M'_{UADin}} \right| & \text{if } \varepsilon'_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ADin} = 2.819$

The mid-span factor of safety always decreases when negative moment redistribution is considered.

ADSlenderness_Check = "Ok"

RedistAD = "Permitted"

ADinspanCheck := if (ADSlenderness_Check ≠ "Ok" , "ANALYSIS INVALID" , "OKAY")

ADinspanCheck = "OKAY"

Wall AD End-of-Span Nominal Moment Capacity

$L_1 = 13\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_1 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar_No_{ADout} = 7$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverADout = 3 \cdot in$ Cover -in

$Bar_No_{ADout} = 7$ $diaADin = 1 \cdot in$

$A_1 := A_{ADin}$ $A_2 := A_{ADout}$ $A_i =$

1.58
0.72

$\cdot in^2$

$coverADout = 3 \cdot in$ $diaADout = 0.875 \cdot in$

$dd_1 := d_{AD_in_end}$ $dd_1 = 3.5 \cdot in$

$dd_2 := d_{AD_out_end}$ $dd_2 = 14.563 \cdot in$

Wall AD, End-of-Span Column Interaction Results

Wall AD End-of-Span

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_1 = 1.5\,ft \quad b = 1\,ft \quad \beta 1 = 0.85$

Given $\quad c := .2 \cdot D_1 \quad c = 3.6 \cdot in$

$$P_{ADthrust} = \left[\sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \\ F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \le -F_y \end{array} \right] \cdot A_i \right] \ldots \right] \cdot \phi_B + 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \quad c = 2.87 \cdot in \quad a := \text{if}\left(\beta 1 \cdot c > D_1, D_1, \beta 1 \cdot c\right) \quad a = 2.44 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(Es \cdot \varepsilon s_i > F_y, F_y, \text{if}\left(Es \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, Es \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > dd_i, 0.85 \cdot f'_c, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D_1}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.292 ft	-0.001	-19.087	1.58 $\cdot in^2$	-30.158 $\cdot kip$	-165.87 $\cdot kip \cdot in$
1.214	-0.012	-40	0.72	-28.8	160.2

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = -6 \cdot kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D_1}{2} - \frac{a}{2}\right) \qquad Mc = 581 \cdot kip \cdot in$

$\phi M_{nADface1} := (Mc + Mst) \qquad \phi M_{nADface1} = 47.93 \cdot kip \cdot ft$

$\phi M_{nADface} := \begin{cases} \phi M_{nADface1} & \text{if } M_{UADface} > 0ft \cdot kip \\ \phi M_{nADface1} \cdot (-1) & \text{otherwise} \end{cases} \qquad M_{UADface} = -16.977 \cdot kip \cdot ft$

Wall AD End-of-Span Nominal Moment Strength $\quad \phi M_{nADface} = -47.93 \cdot kip \cdot ft$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ADface}

$\phi M_{nADface} = -47.93 \cdot kip \cdot ft$ $M_{UADface} = -16.977 \cdot kip \cdot ft$

$$FS_{ADface} := \left(\left| \frac{\phi M_{nADface}}{M_{UADface}} \right| \right)$$

$FS_{ADface} = 2.823$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ADface}$

$$FS'_{ADface} := \begin{cases} \left| \frac{\phi M_{nADface}}{M'_{UADface}} \right| & \text{if } \varepsilon'_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ADface} = 3.529$

The end-of-span factor of safety always increases when negative moment redistribution is considered.

ADSlenderness_Check = "Ok"

RedistAD = "Permitted"

ADoutspanCheck := if (ADSlenderness_Check ≠ "Ok" , "ANALYSIS INVALID" , "OKAY")

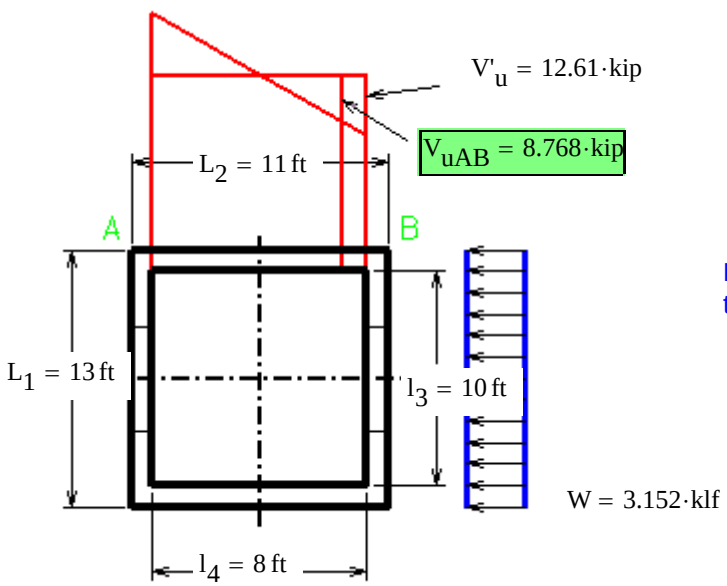
ADoutspanCheck = "OKAY"

Wall AB (Wall 2) Analysis

Shear Demand:

$W = 3152.38\ ft \cdot psf$ $\quad l_2 = 8\ ft$ $\quad d_{AB_out_end} = 14.625 \cdot in$

$V'_u := W \cdot \frac{l_2}{2}$ $\quad V'_u = 12.61 \cdot kip$ $\quad V_{uAB} := V'_u \cdot \frac{\frac{l_2}{2} - d_{AB_out_end}}{\frac{l_2}{2}}$ $\quad V_{uAB} = 8.768 \cdot kip$



Total Thrust into Wall2:

$P_{ABthrust} = 18.552 \cdot kip$

Note: Refer to wall AD calculations for derivation of the thrust values acting on wall AB.

Shear Capacity and Factor of Safety

$d_{AB_out_end} = 14.625 \cdot in$ $\quad \frac{P_{ABthrust}}{2000 \cdot A_{g2}} = 0.043\ psi$ $\quad f'_c = 3000\ psi$ $\quad b = 1\ ft$

Shear Concrete Resistance value at Distance d from Support ACI 318R-11.1.3.1

$$\phi V_{n_AB} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{P_{ABthrust}}{2000 \cdot A_{g2}} \cdot \frac{in^2}{lbf} \right) \cdot \left(\sqrt{f'_c \cdot psi} \right) \cdot \left(b \cdot d_{AB_out_end} \right) \right]$$

ACI EQ (11-4) $\quad \phi V_{n_AB} = 20.051 \cdot kip$

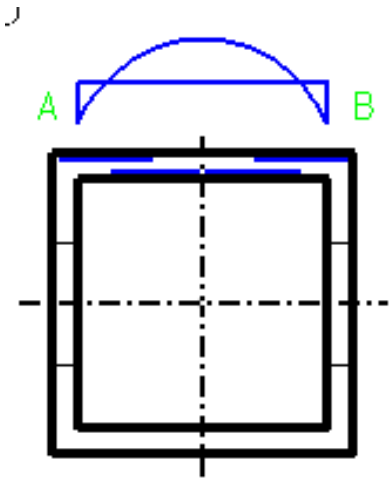
$V_{uAB} = 8.768 \cdot kip$

Factor of Safety $\quad FS_{ABshear} := \frac{\phi V_{n_AB}}{V_{uAB}}$ $\quad FS_{ABshear} = 2.287$

Wall AB con't

Depth of Equivalent Rectangular Stress Block

$F_y = 40 \cdot ksi$ $f'_c = 3 \cdot ksi$ $b = 1\,ft$ $W = 3.152 \cdot klf$ $L_1 = 13\,ft$



$A_{ABout} = 0.88 \cdot in^2$ $aABout := \frac{A_{ABout} \cdot F_y}{0.85 f'_c \cdot b}$

$A_{ABin} = 1.2 \cdot in^2$ $aABin := \frac{A_{ABin} \cdot F_y}{0.85 f'_c \cdot b}$

$aABout = 1.15 \cdot in$ $aABin = 1.569 \cdot in$

Negative Moment Redistribution

Wall AB con't

ACI 318-02, 8.4 allows moment redistribution (plastic hinge formation). Refer to 8.4 for moment redistribution limitations.

Determine if redistribution is possible. To occur, the steel must yield prior to concrete crushing.

$\beta 1 = 0.85$ $\quad f'_c = 3000\,psi$

$cABout := \frac{aABout}{\beta 1}$ $\quad cABout = 1.353 \cdot in$ $\quad cABin := \frac{aABin}{\beta 1}$ $\quad c_{AD_in} = 2.43 \cdot in$

The net tensile strain values

$\varepsilon_{tout} := 0.003 \cdot \left(\frac{d_{AB_out_end}}{cABout} - 1 \right)$ $\quad \varepsilon_{tout} = 0.029$

Negative moment distribution is permissible when the tensile strain is greater than 0.0075, per ACI 318-02, 8.4

Reduce negative moment at face of wall AB end support and increase the positive mid-span moment (if applicable)

$R_{nface} := \frac{|M_{UABface}|}{\phi_B \cdot b \cdot d_{AB_out_end}^2}$ $\quad R_{nface} = 90.425 \cdot psi$ $\quad M_{UABface} = -19.341 \cdot kip \cdot ft$

$$\varepsilon'_{tface} := \begin{cases} 0.003 \cdot \left(\frac{\beta 1}{1 - \sqrt{1 - \frac{40}{17} \cdot \frac{R_{nface}}{f'_c}}} - 1 \right) & \text{if } (\varepsilon_{tout} \ge 0.0075) \wedge (M_{UABin} > 0) \\ 0 & \text{otherwise} \end{cases}$$

PCA 318-05 Notes, page 8-3

$\varepsilon'_{tface} = 0.068$

$$\varepsilon'_ABfacedelta := \begin{cases} \varepsilon'_{tface} \cdot 10 & \text{if } 0 < \varepsilon'_{tface} \le 0.020 \\ 0 & \text{if } \varepsilon'_{tface} = 0 \\ 0.20 & \text{otherwise} \end{cases}$$

$\varepsilon'_ABfacedelta = 0.200$ $\quad \varepsilon'_{tface} \cdot 10 = 0.676$

$$RedistAB := \begin{cases} \text{"Permitted"} & \text{if } \varepsilon'_{tface} \ne 0 \\ \text{"Not Permitted"} & \text{otherwise} \end{cases}$$

RedistAB = "Permitted"

IF: The strain is greater than or equal to 0.0075 and the mid span moment is positive
THEN: Negative moment redistribution is permitted. OTHERWSE: Negative moment redistribution is not permitted and "N/A" is printed.

Redistributed (reduced) negative moment at the support face

$$M'_{UABface} := \begin{cases} M_{UABface} \cdot (1 - \varepsilon'_ABfacedelta) & \text{if } \varepsilon'_ABfacedelta \ne 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UABface} = -15.473 \cdot kip \cdot ft$ $\quad M_{UABface} = -19.341 \cdot kip \cdot ft$

Wall AB con't

$$M'_{UABout} := \begin{cases} M_{UDCout} \cdot (1 - \varepsilon'_ABfacedelta) & \text{if } \varepsilon'_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UABout} = -23.748 \cdot kip \cdot ft$ $M_{UABface} = -19.341 \cdot kip \cdot ft$ $M_{UDCout} = -29.685 \cdot kip \cdot ft$

Corresponding redistributed (increased) mid-span moment

$$M'_{UABin} := \begin{cases} \frac{W \cdot (L_2 - D_1)^2}{8} \cdot (\gamma_L \cdot \gamma_H \cdot \gamma_X) + M'_{UABout} & \text{if } M'_{UABout} \neq \text{"N/A"} \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UABin} = 11.815 \cdot kip \cdot ft$

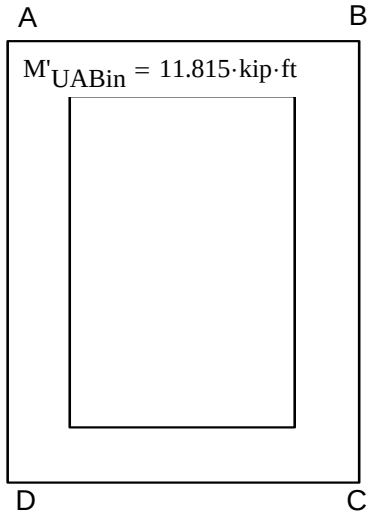
$\frac{W \cdot (L_2 - D_1)^2}{8} = 35.563 \cdot kip \cdot f$ $M_{UABin} = 5.878 \cdot kip \cdot ft$

$M'_{UABin} = 11.815 \cdot kip \cdot ft$

Moments on Span AD After Redistribution

RedistAB = "Permitted"

$M'_{UABface} = -15.473 \cdot kip \cdot ft$



Note: If redistribution is not allowed, "N/A" is shown.

Wall AB Magnified Moment Check

Wall AB Con't

The effects of wall slenderness shall be checked.

10.12.2 — In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy:

$$\frac{k l_u}{r} \le 34 - 12(M_1/M_2) \qquad (10\text{-}8)$$

where the term $[34 - 12M_1/M_2]$ shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

Table 8-3 Deflection Coefficient K

		K
1.	Cantilevers (deflection due to rotation at supports not included)	2.40
2.	Simple beams	1.0
3.	Continuous beams	1.2-0.2 M_o/M_a
4.	Fixed-hinged beams (midspan deflection)	0.80
5.	Fixed-hinged beams (maximum deflection using maximum moment)	0.74
6.	Fixed-fixed beams	0.60

For other types of loading, K values are given in Ref. 8.2.

M_o = Simple span moment at midspan $\left(\frac{w\ell^2}{8}\right)$

M_a = Net midspan moment.

$M_1 := M_{UABface}$ $\qquad$ $M_2 := M_{UABface}$

$M_1 = -19.341 \cdot kip \cdot ft$ $\qquad$ $M_2 = -19.341 \cdot kip \cdot ft$

From the table from the right, the k value for fixed fixed condition: $k := 0.6$

Ubraced length: $l_u := L_2$ $\qquad$ $l_u = 11\,ft$

Radius of Gyration, r:

Thickness of Wall: $D_2 = 1.5\,ft$

10.11.2 — It shall be permitted to take the radius of gyration r equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, it shall be permitted to compute the radius of gyration for the gross concrete section.

$r := 0.3 \cdot D_2$ $\qquad$ $r = 0.45\,ft$

$$Slender_Ratio := k \cdot \frac{l_u}{r} \qquad Slender_Ratio = 14.667$$

$$ABSlenderness_Check := if\left[Slender_Ratio \le \left[34 - 12 \cdot \left(\frac{M_1}{M_2}\right)\right], \text{"Ok"}, \text{"Consider Slenderness using ACI 10.10.1"}\right]$$

$ABSlenderness_Check = \text{"Ok"}$

IF: ADSlenderness_Check is OK
THEN: Moment magnification is not required

IF: ADSlenderness_Check is NOT OK
THEN: Wall is slender. Moment magnification per ACI 318-02, 10.12 shall be followed

Wall AB Mid-Span Nominal Moment Capacity

$L_2 = 11\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_2 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar_No_{ABin} = 7$ **Tension** bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverABin = 3 \cdot in$ Cover -in

Axial Compression with ACI Reduction value: $\phi_B = 1$ $Bar_No_{ABin} = 7$ $diaABin = \blacksquare \cdot in$

$diaABout = 0.75 \cdot in$

Bar area at i $A_1 := A_{ABout}$ $A_2 := A_{ABin}$ $A_i =$

$$\begin{array}{|r|} \hline 0.88 \\ \hline 1.2 \\ \hline \end{array} \cdot in^2$$

$coverABout = 3 \cdot in$ $diaABout = 0.75 \cdot in$ $coverABin = 3 \cdot in$ $diaABin = 0.875 \cdot in$

$$dd_1 := \begin{cases} coverABout + \dfrac{diaABout}{2} & \text{if } M_{UABin} > 0ft \cdot kip \\ coverABin + \dfrac{diaABin}{2} & \text{otherwise} \end{cases}$$

$M_{UABin} = 5.878 \cdot kip \cdot ft$

$\boxed{dd_1 = 3.375 \cdot in}$

$$dd_2 := \begin{cases} D_2 - \left(coverABin + \dfrac{diaABin}{2}\right) & \text{if } M_{UABin} > 0ft \cdot kip \\ D_2 - \left(coverABout + \dfrac{diaABout}{2}\right) & \text{otherwise} \end{cases}$$

$\boxed{dd_2 = 14.563 \cdot in}$

$coverABout = 3 \cdot in$ $diaABout = 0.75 \cdot in$

$dd_1 := d_{AB_out_mid}$ $dd_1 = 3.375 \cdot in$

$dd_2 := d_{AB_in_mid}$ $dd_2 = 14.563 \cdot in$

Wall AB, Mid-Span Column Interaction Results

Wall AB Mid-Span con't

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_2 = 18\cdot in \quad b = 1\,ft$ $\quad \beta 1 = 0.85$

Given $\quad c := .2\cdot D_2 \quad c = 3.6\cdot in$

$$P_{ABthrust} = \left[\sum_{i=1}^{row}\left[\left[\begin{array}{l} \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ldots \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \\ F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \le -F_y \end{array}\right]\cdot A_i\right] \ldots \\ + 0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b \end{array}\right]\cdot\phi_B$$

$c := \text{Find}(c) \quad c = 2.965\cdot in \quad a := \text{if}\left(\beta 1\cdot c > D_2, D_2, \beta 1\cdot c\right) \quad a = 2.521\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right)\cdot\frac{0.003}{c} \qquad fs_i := \text{if}\left(Es\cdot\varepsilon s_i > F_y, F_y, \text{if}\left(Es\cdot\varepsilon s_i < 0 - F_y, 0 - F_y, Es\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > dd_i, 0.85\cdot f'_c, 0ksi\right)\cdot A_i \qquad Ms_i := Fs_i\cdot\left(\frac{D_2}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.281 ft	-0	-12.019 ·k	0.88 $\cdot in^2$	-10.577 ·kip	-59.494 ·kip·in
1.214	-0.012	-40	1.2	-48	267

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 208\cdot kip\cdot in$

$Mc := 0.85\cdot f'_c\cdot a\cdot b\cdot\left(\frac{D_2}{2} - \frac{a}{2}\right) \qquad Mc = 597\cdot kip\cdot in$

$\phi M_{nABin1} := (Mc + Mst) \qquad \phi M_{nABin1} = 67.04\cdot kip\cdot ft$

$$\phi M_{nABin} := \begin{cases} \phi M_{nABin1} & \text{if } M_{UABin} > 0ft\cdot kip \\ \phi M_{nABin1}\cdot(-1) & \text{otherwise} \end{cases}$$

Wall AB Mid-Span Nominal Moment Strength $\qquad \phi M_{nABin} = 67.038\cdot kip\cdot ft$

Wall AB Mid-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ABmid}

Wall AB Mid-Span Con't

$\phi M_{nABin} = 67.038 \cdot kip \cdot ft$

$M_{UABin} = 5.878 \cdot kip \cdot ft$

$$FS_{ABin} := \left(\left| \frac{\phi M_{nABin}}{M_{UABin}} \right| \right)$$

$FS_{ABin} = 11.405$

Wall AB Mid-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ABmid}$

$\phi M_{nABin} = 67.038 \cdot kip \cdot ft$

Note: An apostrophe signifies a negative moment redistribution-related value.

$M'_{UABin} = 11.815 \cdot kip \cdot ft$

$$FS'_{ABin} := \begin{cases} \left| \frac{\phi M_{nABin}}{M'_{UABin}} \right| & \text{if } \varepsilon'_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ABin} = 5.674$

The mid-span factor of safety always decreases when negative moment redistribution is considered.

ABSlenderness_Check = "Ok"

RedistAB = "Permitted"

ABinspanCheck := if (ABSlenderness_Check ≠ "Ok" , "ANALYSIS INVALID" , "OKAY")

ABinspanCheck = "OKAY"

Wall AB End-of-Span Nominal Moment Capacity

$L_2 = 11\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_2 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar_No_{ABout} = 6$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverABout = 3 \cdot in$ Cover -in

$Bar_No_{ABout} = 6$ $diaABin = 0.875 \cdot in$

$A_1 := A_{ABin}$ $A_2 := A_{ABout}$

$A_i =$

1.2
0.88

$\cdot in^2$

<==The first value of A1 is the compression steel, A2 is the tension steel

$coverABout = 3 \cdot in$ $diaABout = 0.75 \cdot in$

$dd_1 := d_{AB_in_end}$ $dd_1 = 3.437 \cdot in$ Effective depth of the compression steel

$dd_2 := d_{AB_out_end}$ Effective depth of the tension steel

Wall AB, End-of-Span Column Interaction Results

Wall AB End-of-Span con't

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_2 = 18\cdot in \quad b = 1\,ft$ $\quad \beta 1 = 0.85$

Given $\quad c := .2\cdot D_2 \quad c = 3.6\cdot in$

$$P_{ABthrust} = \left[\sum_{i=1}^{row} \left[\left[\begin{array}{l} \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ldots \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \\ F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \le -F_y \end{array}\right]\cdot A_i\right] \ldots\right]\cdot\phi_B$$

$$+ 0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b$$

$c := \text{Find}(c) \quad c = 2.866\cdot in \quad a := \text{if}\left(\beta 1\cdot c > D_2, D_2, \beta 1\cdot c\right) \quad a = 2.436\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right)\cdot\frac{0.003}{c} \quad fs_i := \text{if}\left(Es\cdot\varepsilon s_i > F_y, F_y, \text{if}\left(Es\cdot\varepsilon s_i < 0 - F_y, 0 - F_y, Es\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > dd_i, 0.85\cdot f'_c, 0ksi\right)\cdot A_i \quad Ms_i := Fs_i\cdot\left(\frac{D_2}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.286 ft	-0.001	-17.335 $\cdot$ksi	1.2 $\cdot in^2$	-20.802 $\cdot$kip	-115.713 $\cdot$kip$\cdot$in
1.219	-0.012	-40	0.88	-35.2	198

$Mst := \sum_{i=1}^{2} Ms_i \quad Mst = 82\cdot kip\cdot in$

$Mc := 0.85\cdot f'_c\cdot a\cdot b\cdot\left(\frac{D_2}{2} - \frac{a}{2}\right) \quad Mc = 580\cdot kip\cdot in$

$\phi M_{nABface1} := (Mc + Mst) \quad \phi M_{nABface1} = 55.2\cdot kip\cdot ft$

$$\phi M_{nABface} := \begin{cases} \phi M_{nABface1} & \text{if } M_{UABface} > 0ft\cdot kip \\ \phi M_{nABface1}\cdot(-1) & \text{otherwise} \end{cases}$$

Wall AB End-of-Span Nominal Moment Strength $\quad \phi M_{nABface} = -55.204\cdot kip\cdot ft$

Wall AB End-of-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ABface}

$\phi M_{nABface} = -55.204 \cdot kip \cdot ft$ $M_{UABface} = -19.341 \cdot kip \cdot ft$

$$FS_{ABface} := \left(\left| \frac{\phi M_{nABface}}{M_{UABface}} \right| \right)$$

$FS_{ABface} = 2.854$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ABface}$

$\varepsilon'_ABfacedelta = 0.2$

$$FS'_{ABface} := \begin{cases} \left| \frac{\phi M_{nABface}}{M'_{UABface}} \right| & \text{if } \varepsilon'_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ABface} = 3.568$

The end-of-span factor of safety always increases when negative moment redistribution is considered.

ABSlenderness_Check = "Ok"

RedistAB = "Permitted"

ABoutspanCheck := if (ABSlenderness_Check ≠ "Ok" , "ANALYSIS INVALID" , "OKAY")

ABoutspanCheck = "OKAY"

Strength Analysis Results Summary

Span AD Moment Factor of Safety Analysis Summary and Recommendations

$FS_{ADface} = 2.823$ $FS_{ADin} = 3.565$ $FS'_{ADface} = 3.529$ $FS'_{ADin} = 2.819$

$FS_{ADno_redist} := \min(FS_{ADface}, FS_{ADin})$ $FS'_{ADredist} := \min(FS'_{ADface}, FS'_{ADin})$

$FS_{ADno_redist} = 2.823$ $FS'_{ADredist} = 2.819$

$$FS_{ADmoment} := \begin{cases} \max(FS_{ADno_redist}, FS'_{ADredist}) & \text{if } FS'_{ADredist} \neq \text{"N/A"} \\ FS_{ADno_redist} & \text{otherwise} \end{cases}$$

$FS_{ADin} = 3.565$

$FS_{ADmoment} = 2.823$

$$AD_Recommendation1 := \begin{cases} \text{"Perform Simple Beam Analysis"} & \text{if } FS_{ADin} \geq 2.0 \wedge FS_{ABin} \geq 2.0 \\ \text{"Simple Beam Analysis Not Recommended"} & \text{otherwise} \end{cases}$$

AD_Recommendation1 = "Perform Simple Beam Analysis"

$$AD_Recommendation2 := \begin{cases} \text{"CAUTION - Reliability Questionable"} & \text{if } FS_{ADmoment} < 1.5 \\ \text{"Span AD OKAY for Moment Strength"} & \text{if } FS_{ADmoment} \geq 1.5 \end{cases}$$

AD_Recommendation2 = "Span AD OKAY for Moment Strength"

A listing of the span AD moment demand-capacity values, factors of safety and recommendations are provided in the following table

δ

Scalar Conversion Equations To Allow Text Entry Into Tables

Span AD Moment Strength Analysis Summary						
	End-Support			Mid-Span		
Analysis	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS
Moment Distribution	-16.98	-47.93	2.82	22.43	79.96	3.57
Redistribution of Negative Moments	-13.58		3.53	28.36		2.82
Slenderness Check	Ok					
Redistribution of Negative Moments	Permitted					
Recommendations: Perform Simple Beam Analysis Span AD OKAY for Moment Strength						

N/A: Not Applicable

Span AB Moment Factor of Safety Analysis Summary and Recommendations

$FS_{ABno_redist} := \min(FS_{ABface}, FS_{ABin})$ $FS'_{ABredist} := \min(FS'_{ABface}, FS'_{ABin})$

$FS_{ABno_redist} = 2.854$ $FS'_{ABredist} = 3.568$

$$FS_{ABmoment} := \begin{cases} \max(FS_{ABno_redist}, FS'_{ABredist}) & \text{if } FS'_{ABredist} \neq \text{"N/A"} \\ FS_{ABno_redist} & \text{otherwise} \end{cases}$$

$FS_{ABin} = 11.405$

$FS_{ABmoment} = 3.568$

$$AB_Recommendation1 := \begin{cases} \text{"Perform Simple Beam Analysis"} & \text{if } FS_{ADin} \geq 2.0 \wedge FS_{ABin} \geq 2.0 \\ \text{"Simple Beam Analysis Not Required"} & \text{otherwise} \end{cases}$$

AB_Recommendation1 = "Perform Simple Beam Analysis"

$$AB_Recommendation2 := \begin{cases} \text{"CAUTION - Reliability Questionable"} & \text{if } FS_{ABmoment} < 1.5 \\ \text{"Span AB OKAY for Moment Strength"} & \text{if } FS_{ABmoment} \geq 1.5 \end{cases}$$

AB_Recommendation2 = "Span AB OKAY for Moment Strength"

A listing of the span AB moment demand-capacity values, factors of safety and recommendations are provided in the following table

AB Scalar Conversion for Table

Span AB Moment Strength Analysis Summary						
	End-Support			Mid-Span		
Analysis	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS
Moment Distribution	-19.34	-55.20	2.85	5.88	67.04	11.41
Redistribution of Negative Moments	-15.47		3.57	11.81		5.67
Slenderness Check	Ok					
Redistribution of Negative Moments	Permitted					
Recommendations: Perform Simple Beam Analysis Span AB OKAY for Moment Strength						

N/A: Not Applicable

Shear Factor of Safety Analysis Summary and Recommendations

Span AD

$$FS_{ADshear} = 1.662$$

Span AB

$$FS_{ABshear} = 2.287$$

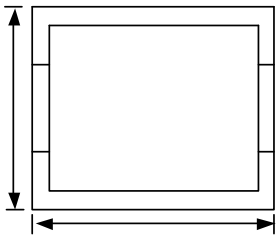
NOTES TO STRENGTH SUMMARY RESULTS:

1. If simple beam analysis is performed on one wall, it must be performed on all four gatewell walls. This is required because fixity is lost when an adjacent wall is analyzed as a simple span. Stability should be considered when the gatewell is analyzed as having pinned supports.
2. Moment redistribution is not permitted when the mid-span and end-support moment values are negative.
3. Moment redistribution is not permitted when the reinforcement strain is less than 0.0075
4. Simple span analysis is recommended when the mid-span moment factor of safety is greater than 2.0. Final decision to use simple span analysis rests with the engineer.
5. Reliability analysis is recommended when the 1.0 < FS < 1.5
6. Caution is provided in the recommendations when the FS < 1.0

This calculation is a supplement to the typical analysis above. In the previous full section analysis, the load and section analyzed were taken above the inlet and outlet openings because for this case, the opening is large and the wall with the opening will govern failure when the load is taken at the very base. The following calculations check the strength of the wall without an opening at top of the base slab.

Properties of the Wall without an opening (Wall AB & CD)

Plan



$L_2 = 11\,ft$

$D_f = 2\,ft$

$H'' := H - D_f - y$

$H'' = 38.5\,ft$

Analyze wall at, $y := 0ft$ above base slab

Wall Thickness $D_2 = 18 \cdot in$ $l_2 = 8\,ft$

Clear Cover $C_c := coverABin$

MID-SPAN REINFORCEMENT

$Bar_Spacing := 6in$

$Bar_No :=$

$Bar_No = 7$

Wall Reinforcement

The area of steel will be calculated /ft:

$$A_s := \frac{A_{out}}{\frac{Bar_Spacing}{12in}} \qquad A_s = 1.2 \cdot in^2$$

Reinforcement Bar Diameters

Effective Depth:

$$D_2 = 18 \cdot in \qquad dia = 0.875 \cdot in$$

$$d := D_2 - C_c - \frac{dia}{2}$$

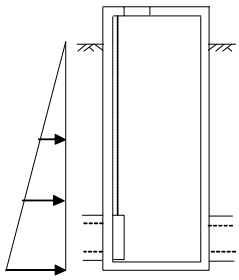
$$d = 14.563 \cdot in$$

Wall Loading

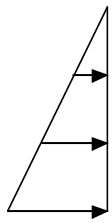
(Wall without opening near base, cont'd)

Wall Loading

WALLS



Soil



$H'' = 38.5\,ft$

$H'_2 := H''$

$H'_2 = 38.5\,ft$

Analysis of continuous wall perpendicular to opening. The walls above the opening were analyzed above.

$W'_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma_{soil} \cdot H'_2$

$W'_1 = 2841 \cdot psf$

Soil & Water

$H'_2 = 38.5\,ft$

&

$H'_3 := \text{if}\left[H'' < H_{blanket}, \frac{H''}{H_{blanket}} \cdot \left(HGL + H_{blanket}\right) - HEAD, H'' + HGL - HEAD\right]$

$H'_3 = 37.4\,ft$

$W'_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot \left(\gamma_{soil} - \gamma_w\right) \cdot H'_2 + \gamma_w \cdot H'_3\right]$

$W'_2 = 3633 \cdot psf$

Check to determine if water to top of wet well with reduction factor, γ_X, or soil loading is worst case

$W' := \text{if}\left(W'_1 > W'_2, W'_1, W'_2\right) \cdot b$ $\boxed{W' = 3633 \cdot plf}$

(Wall without opening near base, cont'd)

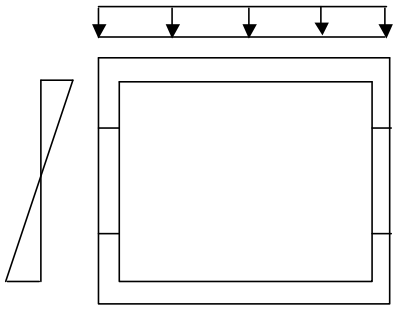
WALL without opening:

SHEAR

$f'_c = 3000\,psi$

$V'_u := W' \cdot \frac{l_2}{2}$

$V'_u = 14533\,lbf$



$V_u := V'_u \cdot \frac{\frac{l_2}{2} - d}{\frac{l_2}{2}}$

$b := 12in$

$V_u = 10 \cdot kip$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$\phi V_n := \phi_V \cdot \left[2 \cdot \left(\sqrt{f'_c \cdot psi}\right) \cdot (d) \cdot b\right]$ ACI EQ (11-4) $\phi V_n = 19 \cdot kip$

Factor of Safety $FS_{opening_shear} := \frac{\phi V_n}{V_u}$ $FS_{opening_shear} = 1.9$

Bending

NOTE: Assumed to be simply supported with no thrust because of opening in adjacent walls.

Demand Moment:

$M_u := \frac{W' \cdot (L_2 - D_1)^2}{8}$ $M_u = 40.986 \cdot kip \cdot ft$

Moment Capacity:

$a := \frac{A_s \cdot F_y}{0.85 f'_c \cdot b}$ $a = 1.569 \cdot in$

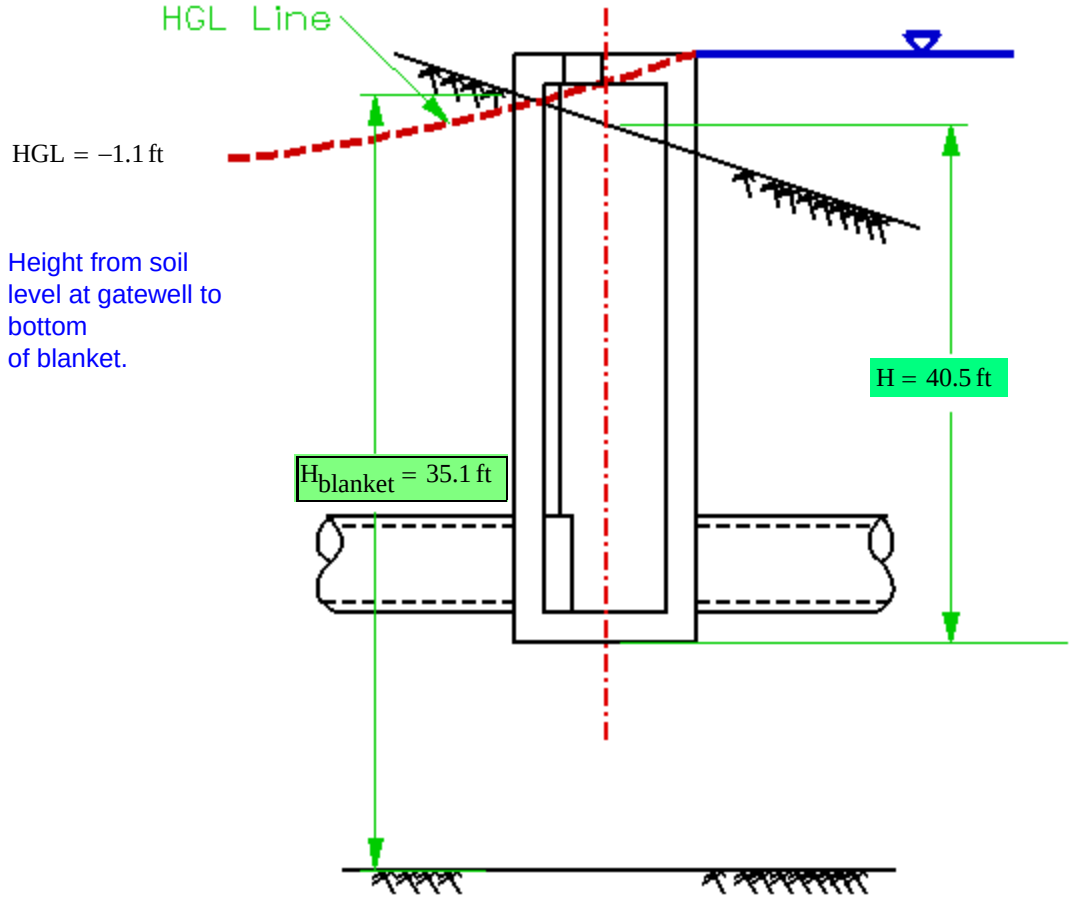
$\phi M_n := \phi_B \cdot A_s \cdot F_y \cdot \left(d - \frac{a}{2}\right)$ $\phi M_n = 55.1\,ft \cdot \frac{kip \cdot ft}{ft}$

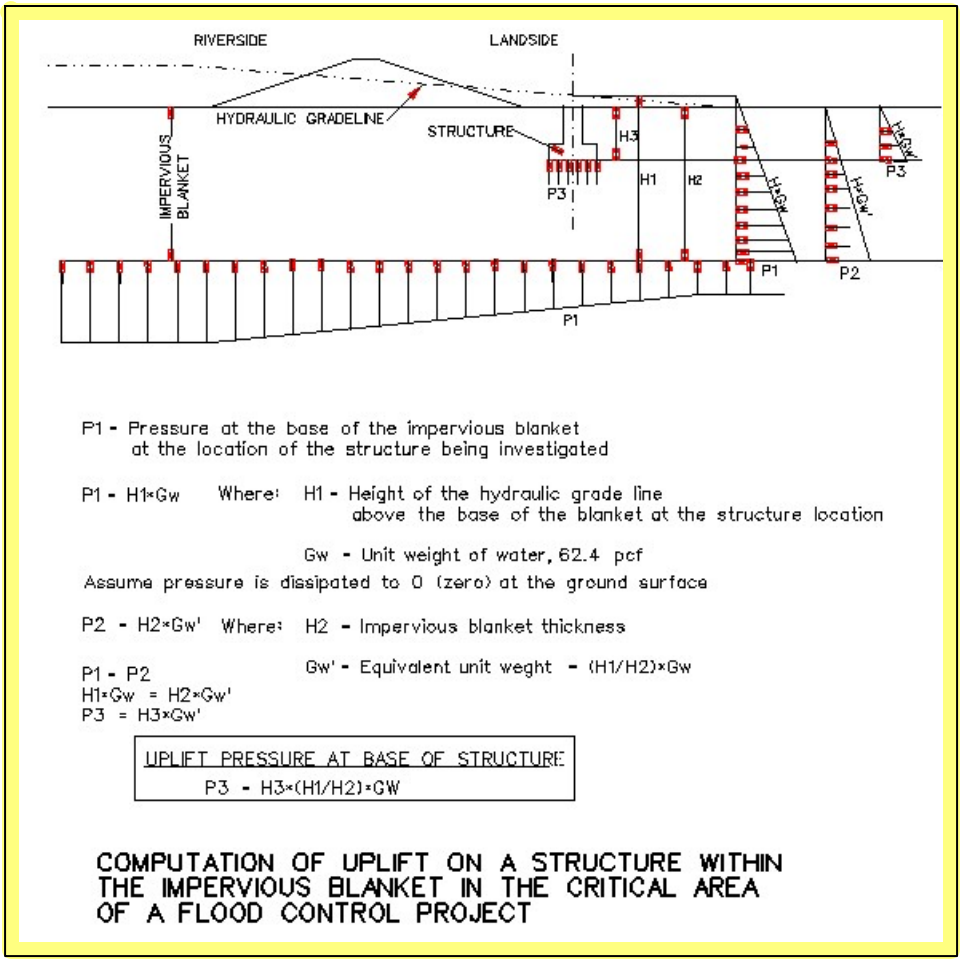
Factor of Safety $FS_{opening_mom} := \frac{\phi M_n}{M_u}$ $FS_{opening_mom} = 1.3$

Uplift Analysis:

Uplift Using Local Protection Guidance

UPLIFT





<< =from KC levees uplift guidance.

$A_{base} := L_1 \cdot L_2$ $\qquad A_{base} = 143\,ft^2$

Dissipated Head, if structural foundation in blanket.

$H_{blanket} = 35.1\,ft \quad HGL = -1.1\,ft \quad H = 40.5\,ft \quad L_1 = 13\,ft \quad L_2 = 11\,ft$

$$U_1 := \left(\frac{H_{blanket} + HGL}{H_{blanket}} \cdot H\right) \cdot L_1 \cdot L_2 \cdot \gamma_w \qquad U_1 = 350.1 \cdot kip$$

Full Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL) \cdot \left(L_1 \cdot L_2\right) \cdot \gamma_w\right] \qquad U_2 = 351.6 \cdot kip$$

Uplift

$H = 40.5\,ft$ $H_{blanket} = 35.1\,ft$ $H' = 0\,ft$

$U := if(H > H_{blanket}, U_2, U_1)$ $U = 351.6 \cdot kip$

$U_{uplift} := \frac{U}{L_1 \cdot L_2}$ $U_{uplift} = 2459 \cdot psf$

Weight of Structure

$Walls := (L_1 \cdot L_2 - l_1 \cdot l_2) \cdot (H + H') \cdot \gamma_c$ $Slabs := 2 \cdot (l_1 \cdot l_2) \cdot D_f \cdot \gamma_c$ $W_S := (Walls + Slabs)$ $W_S = 430.7 \cdot kip$

Surcharge Loads

$Soil := 0lb$

WEIGHT OF SOIL LOCATED ON TOP OF THE BASE SLAB HEEL OR WALL STEP. EQUAL TO ZERO IF NO HEEL IS PRESENT.

Weight of Surcharge Water

$W_{water} := HEAD \cdot l_1 \cdot l_2 \cdot \gamma_w$ $W_{water} = 0 \cdot lb$ $W_g := 0lb$

Skin Friction

SOIL (Soil parameters entered previously)

$\phi = \blacksquare \cdot deg$ INTERNAL FRICTION ANGLE

$\gamma_{soil} = 115 \cdot pcf$ DRY SOIL UNIT WEIGHT

$\gamma_w = 62.4 \cdot pcf$ WATER UNIT WEIGHT

$\gamma_{sub} := (\gamma_{soil} - \gamma_w)$ SUBMERGED SOIL UNIT WEIGHT $\gamma_{sub} = 52.6 \cdot pcf$

$K_o = 0.642$ AT-REST SOIL COEFFICIENT. CALCULATED PREVIOUSLY

$\mu := 0.25$ FRICTION COEFICIENT BETWEEN STRUCTURE AND SOIL.

$U_{uplift} = 2458.56 \cdot psf$ UPLIFT FORCE ACTING ON BASE OF STRUCTURE

$c := 5ft$ ASSUMED CRACK DEPTH

STRUCTURE

$H = 40.5\,ft$ DEPTH OF STRUCTURE BELOW GRADE

$per := 2 \cdot L_1 + 2 \cdot L_2$ STRUCTURE PERIMETER $per = 48\,ft$

$W_S = 430.725 \cdot kip$ STRUCTURE DEAD WEIGHT

Skin Friction (Continued)

1. Effective Lateral Load that Contributes to Side Friction

$R_1 := (H - c)\cdot\left(\frac{c + H}{2}\right)\cdot\gamma_{soil}$ $\quad R_1 = 92876.875\cdot\frac{lbf}{ft}$

$R_2 := (H - c)\cdot U_{uplift}\cdot\left(\frac{c}{2\cdot H} + \frac{1}{2}\right)$ $\quad R_2 = 49027.025\cdot\frac{lbf}{ft}$

$R_1 - R_2 = 43849.85\cdot\frac{lbf}{ft}$

2. Force Created By Side Friction, Pv

$K_o = 0.642$ $\quad \mu = 0.25$ $\quad per = 48\,ft$

$P_{v1} := (R_1 - R_2)\cdot K_o\cdot\mu\cdot per$ $\quad P_{v1} = 337.626\cdot kip$

Ignore_Skin_Friction :=

$P_v := \begin{cases} 0kip & \text{if Ignore_Skin_Friction} = \text{"Yes"} \\ P_{v1} & \text{otherwise} \end{cases}$ $\quad P_v = 338\cdot kip$

Final Uplift Factor of Safety:

$$FS_{Uplift} := \frac{W_S + W_{water} + Soil + P_v}{U}$$

$FS_{Uplift} = 2.185$

NOTE: EM 1110-2-2100 defines 1.1 Factor of Safety acceptable for extreme loading case. The gatewells have minimal overhangs to produce weight of water above structure, W_G, therefore W_G ignored (conservative).

Base Slab Analysis

NOTE:
1. The following calculation assumes two opposite edges fixed and the other opposite edges simply supported. This occurs when a large pipe takes up most of the wall so that wall should be simply supported. Coef came from "Theory of Plates and Shells" second edition, by Timoshenko. This is conservative for gatewells without large pipe openings. So if FS is critical than may want to reanalize using the table of coeff for all sides fixed from "EM 27 - Moments and Reactions for Rect. Plates".
2. Only the end-of-span moment is checked, but in cases where the base slab top and bottom reinf is not the same, it may be necessary to check the mid-span.

Slab Loading:

Weight of structure: $W_S = 430.725 \cdot kip$ Uplift: $U = 351.574 \cdot kip$ Skin Friction: $P_{v1} = 337.626 \cdot kip$

Load acting up on baseslab:

$$W := \text{if}\left(U > W_S, U + P_{v1}, W_S\right) = 430.725 \cdot kip \qquad W_{UNIFORM} := \frac{\gamma_L \cdot \gamma_H \cdot \gamma_X \cdot W}{L_1 \cdot L_2} = 3.012 \cdot ksf$$

END-OF-SPAN MOMENT IS CHECKED HERE:

Plate Coefficient and Demand Moment:

$k := 1 .. 5$

a/b=L_2/L_1 $\frac{l_2}{l_1} = 0.8$

From plates tables:

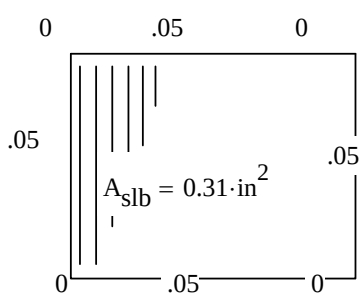
a/b	Coefficient
1	.0697
.9091	.0787
.769	.0938
.625	.109
.5	.1191

$ab_k :=$

0.5
.625
.769
0.9091
1

$Coeff_k :=$

.1191
.109
.0938
.0787
.0697



$$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right) = 0.09$$

$l_2 = 8\,ft \qquad b = 1\,ft$

$$M_{uFe} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b \qquad M_{uFe} = 17.438 \cdot kip \cdot ft$$

Moment Capacity:

$$a := \frac{A_{slb} \cdot F_y}{0.85 f'_c \cdot b} = 4.9 \cdot \frac{in^2}{ft} \qquad d_f = 1.708\,ft$$

$$\phi M_{nFe} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right) \qquad \phi M_{nFe} = 21\,ft \cdot \frac{kip \cdot ft}{ft}$$

Factor of Safety $FS_{slabmomente} := \frac{\phi M_{nFe}}{M_{uFe}}$ $FS_{slabmomente} = 1.203$

Base Slab Analysis, con't

MID-SPAN MOMENT (My) IS CHECKED HERE:

Plate Coefficient and Demand Moment:

a/b=L_2/L_1 $\frac{l_2}{l_1} = 0.8$

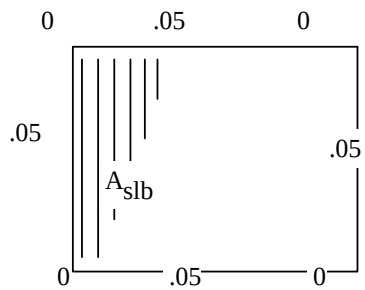
From plates tables:

a/b	Coefficient
1	0.0332
.9091	0.0371
.769	0.0426
.625	0.0469
.5	0.0474

$k := 1 .. 5$

$ab_k :=$ $Coeff_k :=$

0.5	0.0474
.625	0.0469
.769	0.0426
0.9091	0.0371
1	0.0332



$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right) = 0.041$

$l_2 = 8\,ft$ $b = 1\,ft$

$M_{uFm1} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b$ $M_{uFm1} = 7.977 \cdot kip \cdot ft$

$A_{slb} := 0.44in^2$ $C_{cf} := 3.0625in$

$d_f := D_f - C_{cf}$

Moment Capacity:

$a := \frac{A_{slb} \cdot F_y}{0.85f'_c \cdot b} = 6.9 \cdot \frac{in^2}{ft}$ $d_f = 1.745\,ft$

$\phi M_{nFm1} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$ $\phi M_{nFm1} = 30.3\,ft \cdot \frac{kip \cdot ft}{ft}$

Factor of Safety $FS_{slabmomentm1} := \frac{\phi M_{nFm1}}{M_{uFm1}}$ $FS_{slabmomentm1} = 3.797$

Base Slab Analysis, con't

MID-SPAN MOMENT (Mx) IS CHECKED HERE:

Plate Coefficient and Demand Moment:

$k := 1 .. 5$

a/b=L_2/L_1 $\frac{l_2}{l_1} = 0.8$

From plates tables:

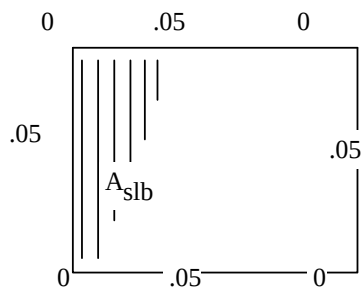
a/b	Coefficient
1	0.0244
.9091	0.0307
.769	0.0446
.625	0.065
.5	0.0869

$ab_k :=$

0.5
.625
.769
0.9091
1

$Coeff_k :=$

0.0869
0.065
0.0446
0.0307
0.0244



$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right) = 0.042$

$l_2 = 8\,ft$ $b = 1\,ft$

$M_{uFm2} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b$ $M_{uFm2} = 8.005 \cdot kip \cdot ft$

$A_{slb} := 1.06in^2$ $C_{cf} := 2.375in$

$d_f := D_f - C_{cf}$

Moment Capacity:

$a := \frac{A_{slb} \cdot F_y}{0.85f'_c \cdot b} = 16.6 \cdot \frac{in^2}{ft}$ $d_f = 1.802\,ft$

$\phi M_{nFm2} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$ $\phi M_{nFm2} = 74\,ft \cdot \frac{kip \cdot ft}{ft}$

Factor of Safety $FS_{slabmomentm2} := \frac{\phi M_{nFm2}}{M_{uFm2}}$ $FS_{slabmomentm2} = 9.24$

Base Slab Analysis, con't

Note that there is a concentration of reinforcing in one direction in the base slab. Assume that the base slab acts as a plate until reinforcing in the weak direction begins to yield and then the moments will redistribute to reinforcing in the other direction (a plastic hinge will form in the base slab at the wall). When this happens the base slab should be analyzed as if it were bending as if it were a simple span:

$L := 10ft$ $\quad b = 1\,ft$

$$M_{ss} := \frac{W_{UNIFORM} \cdot L^2 \cdot b}{8} = 37.651\,ft \cdot \frac{kip \cdot ft}{ft}$$

$A_{slb} := 1.06in^2$ $\quad C_{cf} := 2.375in$

$d_f := D_f - C_{cf}$

Moment Capacity:

$$a := \frac{A_{slb} \cdot F_y}{0.85f'_c \cdot b} = 16.6 \cdot \frac{in^2}{ft} \qquad d_f = 1.802\,ft$$

$$\phi M_{nFm3} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right) \qquad \phi M_{nFm3} = 74\,ft \cdot \frac{kip \cdot ft}{ft}$$

Factor of Safety $\quad FS_{slabmomentm3} := \dfrac{\phi M_{nFm3}}{M_{ss}}$ $\quad FS_{slabmomentm3} = 1.964$

**********Overall Factors of Safety**********

Uplift Stability

$FS_{Uplift} = 2.2$

Strength

Wall AD $L_1 = 13\,ft$

Wall AB $L_2 = 11\,ft$

WALL MOMENT

The overall wall moment factor of safety, $FS_{wallmoment}$

$FS_{ADmoment} = 2.823$ $FS_{ABmoment} = 3.568$ $FS_{opening_mom} = 1.345$

$FS_{wallmoment} := \min(FS_{ADmoment}, FS_{ABmoment}, FS_{opening_mom})$ $FS_{wallmoment} = 1.34$

WALL SHEAR

$FS_{ADshear} = 1.662$ $FS_{ABshear} = 2.287$ $FS_{opening_shear} = 1.891$

$FS_{wallshear} := \min(FS_{ADshear}, FS_{ABshear}, FS_{opening_shear})$ $FS_{wallshear} = 1.662$

BASE SLAB MOMENT FACTOR OF SAFETY

$FS_{slabmoment} := \max(\min(FS_{slabmomente}, FS_{slabmomentm1}, FS_{slabmomentm2}), FS_{slabmomentm3})$

$FS_{slabmoment} = 1.964$

Summary :

$FS_{strength} := \min(FS_{wallmoment}, FS_{wallshear}, FS_{slabmoment})$ $FS_{strength} = 1.3$

$$Mechanism := \begin{cases} \text{"Wall Bending"} & \text{if } FS_{strength} = FS_{wallmoment} \\ \text{"Wall Shear"} & \text{if } FS_{strength} = FS_{wallshear} \\ \text{"Base Slab Moment"} & \text{otherwise} \end{cases}$$

Mechanism = "Wall Bending"

$FS_{Uplift} = 2.2$

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis. The reasoning being the load factor (live load neglecting hydraulic structure) divided by the strength reduction factor is approximately 1.75. Because this is an existing structure in the field that is routinely inspected with no visible history of problems, a factor of safety of 1.5 is deemed acceptable. If modifications are required for strength, the modified structure must be evaluated from a standard design approach (include factors - no FS calculation)

END OF ANALYSIS

US Army Corps of Engineers

Probability of Failure

CIDKS GATEWELL 106+49

Bending in wall AB/CD

Comp by:KSM 7-3-08
Chkd by:

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. Situation

1. This structure does not meet the strength 1.5 factor of safety for which it has been determined 99.8% reliability can be assigned. See mathcad analysis of the wall for existing condition strength check.
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength
3. Material Properties used:

   Mean Concrete Strength, $f'_{cM} := 3750 \cdot psi$

   Mean Steel Strength, $F_{yM} := 45 \cdot ksi$

4. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete Grade 40 is 14%.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters

FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength

FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength

FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength

FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength

ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength

ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength

σ_F = Standard Deviation of the Factor of Safety

V_F = Coefficient of Variation of the Factor of Safety

β_{LN} = Lognormal Reliability Index

R = Reliability

P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. *Calculating Factors of Safety*

WATER AT TOP OF WALL

Condition under consideration from strength check: Mu for wall AB/CD (from mathcad strength analysis).

$M_u := 40.986 \frac{kip \cdot ft}{ft}$

Design Concrete Strength

$\phi_B := 1.0$ Strength Reduction Factors not used in Risk and Uncertainty Analysis

$A_s := 1.2 \frac{in^2}{ft}$ Area of Steel

$dist_{CL} := 3.4375in$ Distance from inside wall to centerline of reinforcement

$T_{wall} := 1.5ft$ Thickness of wall

$b := 12 \frac{in}{ft}$ 1 ft strip of wall analyzed

$d := T_{wall} - dist_{CL}$ $d = 14.563 \cdot in$

Mean Concrete Strength and Steel Yield Strength

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cM} \cdot b}$ $a = 1.412 \cdot in$

$\phi M_M := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\phi M_M = 62.35 \cdot \frac{kip \cdot ft}{ft}$

$FS_D := \frac{\phi M_M}{M_u}$ $FS_D = 1.521$

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM} \cdot 0.14$ $f'_{cU} = 4275 \cdot psi$

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cU} \cdot b}$ $a = 1.238 \cdot in$

$\phi M_{cU} := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\phi M_{cU} = 62.74 \cdot \frac{kip \cdot ft}{ft}$

$FS_{fcu} := \frac{\phi M_{cU}}{M_u}$ $FS_{fcu} = 1.531$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\quad f'_{cL} = 3225 \cdot psi$

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cL} \cdot b}$ $\quad a = 1.642 \cdot in$

$\phi M_{cL} := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{cL} = 61.84 \cdot \frac{kip \cdot ft}{ft}$

$FS_{fcl} := \frac{\phi M_{cL}}{M_u}$ $\quad FS_{fcl} = 1.509$

Upper Steel Yield Strength

$F_{yU} := F_{yM} + F_{yM} \cdot 0.14$ $\quad F_{yU} = 51.3 \cdot ksi$

$a := \frac{A_s \cdot F_{yU}}{0.85 f'_{cM} \cdot b}$ $\quad a = 1.609 \cdot in$

$\phi M_{sU} := \phi_B A_s \cdot F_{yU} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{sU} = 70.58 \cdot \frac{kip \cdot ft}{ft}$

$FS_{Fyu} := \frac{\phi M_{sU}}{M_u}$ $\quad FS_{Fyu} = 1.722$

Lower Steel Yield Strength

$F_{yL} := F_{yM} - F_{yM} \cdot 0.14$ $\quad F_{yL} = 38.7 \cdot ksi$

$a := \frac{A_s \cdot F_{yL}}{0.85 f'_{cM} \cdot b}$ $\quad a = 1.214 \cdot in$

$\phi M_{sL} := \phi_B A_s \cdot F_{yL} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{sL} = 54.01 \cdot \frac{kip \cdot ft}{ft}$

$FS_{Fyl} := \frac{\phi M_{sL}}{M_u}$ $\quad FS_{Fyl} = 1.318$

$FS_D = 1.521$ $\quad FS_{fcu} = 1.531$ $\quad FS_{Fyu} = 1.722$

$FS_{fcl} = 1.509$ $\quad FS_{Fyl} = 1.318$

VI. Probability of Failure Calculation

$\Delta F_{Fy} := FS_{Fyu} - FS_{Fyl}$ $\Delta F_{Fy} = 0.404$

$\Delta F_{fc} := FS_{fcu} - FS_{fcl}$ $\Delta F_{fc} = 0.022$

ACI EQ (11-4)

$$\sigma_F := \sqrt{\left(\frac{\Delta F_{Fy}}{2}\right)^2 + \left(\frac{\Delta F_{fc}}{2}\right)^2}$$ $\sigma_F = 0.202$

$$V_F := \frac{\sigma_F}{FS_D}$$ $V_F = 0.133$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$$ $\beta_{LN} = 3.101$

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 99.9 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 0.1 \cdot \%$

Gatewell 106+49, Future Condition Strength and Uplift Stability Analysis:

Major tasks assocaited with this analysis:

1. Obtain the geometry for the gatewell and levee which are taken from the Record drawings.
2. Obtain the geotechnical parameters which are obtained from the Geotechnical Engineer.
3. Perform uplift analysis assuming gate is empty.

Calculation Convention Notes:

1. All input values are highlighted in *yellow*
2. Results are highlighted in green
3. Assumption notes are displayed in red

References:
Gatewell Sta. 106+49 elevations, location and reinforcing details from American Royal Arena Storm Sewer System for KC, MO Department of Public Works plans by Shafer, Kline and Warren 1973

Geotech Parameters:

Information Used to Calculate Structures HGL's - Blanket Information			
Station	Condition	Bottom of Landside Blanket Elevation	Top of Blanket Elevation
49+00 to 51+00	Existing	719	751
51+00 to 57+00	Existing	720	750
57+00 to 59+00	Existing	722	750
59+00 to 63+00	Existing	725	735
63+00 to 68+00	Existing	728	735
68+00 to 73+00	Existing	718	733
77+00 to 82+00	Existing	720	738
82+00 to 85+00	Existing	730	740
85+00 to 88+00	Existing	730	742
88+00 to 90+00	Existing	725	732
92+00 to 94+50	Existing	735	744
102+00 to 107+00	Existing	730	742
119+00 to 127+00	Existing	722	756
136+00 to 143+00	Existing	725	753
150+00 to 153+00	Existing	725	751
159+00 to 163+00	Existing	725	745

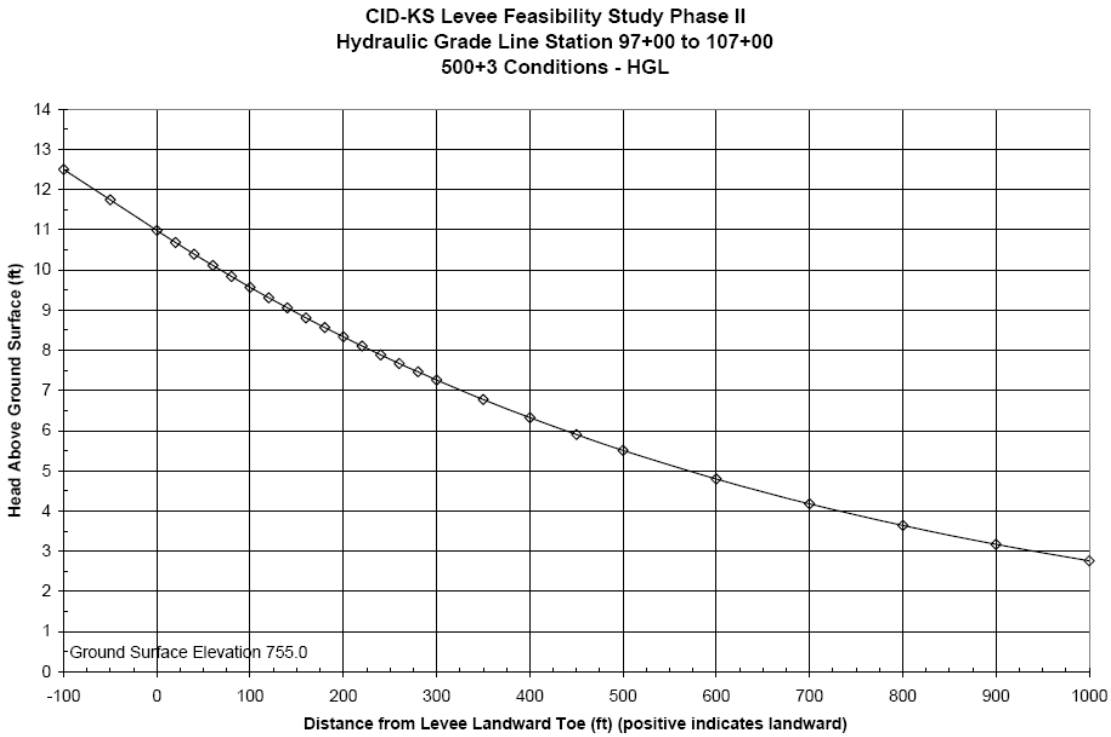
Geotech Parameters (con't):

Recommended Parameters for Phase II Feasibility Study

	Unit Weight (pcf)		Drained Shear Strength		Undrained Shear Strength*	
Material	Moist	Saturated	♦' (degrees)	c' (psf)	♦ (degrees)	c (psf)
Embankment	115	120	29	0	0	1000
Fill (cinders/rubble)	110	115	21	0	N/A	N/A
Foundation Blanket	110	115	24	0	0	500
Foundation Sand	115	120	31	0	N/A	N/A

*Estimated from average strength values for these materials

Hydraulic Gradeline:



HGL & blanket elevations provided by Glen Bellew 6/2/08

Soil Elevation Assumed by geotech: $El_{soil_geotech} \equiv 0ft$

Elevation of the HGL using geotech datum: $HGL_{geotech} \equiv 767ft$

Elevation of bottom of blanket: $El_{bot_blanket} \equiv 730ft$

Input Values

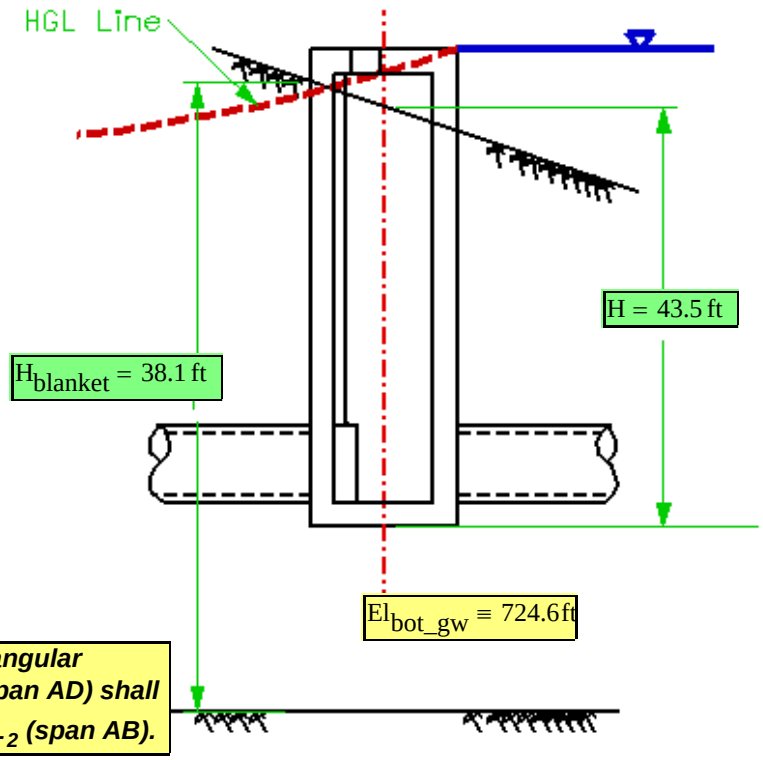
From record drwg $El_{top_gw} \equiv 768.1\,ft$

RAISED GATEWELL 3 FEET FOR FC

This is determined from the Record drawings

$El_{soil_at_gw} \equiv El_{top_gw}$

$El_{soil_at_gw} = 768.1\ ft$



$H_{blanket} \equiv El_{soil_at_gw} - El_{bot_blanket}$

The HGL from the geotech needs to be adjusted using the soil elevation at the gatewell as the datum:

$HGL := HGL_{geotech} \cdots + 0ft - \left(El_{soil_at_gw} - El_{soil_geotech}\right)$

$HGL = -1.1\,ft$

Water head pressure in Wet Well under operating conditions.

$HEAD := 0ft$

NOTE: For rectangular gatewells, L_1 (span AD) shall be longer than L_2 (span AB).

Buried Height of

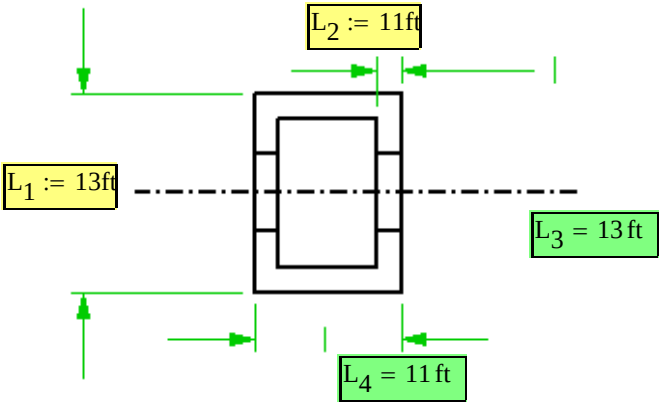
$H \equiv El_{soil_at_gw} - El_{bot_gw}$

$H = 43.5\,ft$

Projected Height of Gatewell above soil

$H' := El_{top_gw} - El_{soil_at_gw}$

$H' = 0\,ft$



Width analyzed: $b := 12in$

$L_3 := L_1$

Floor Thickness $D_f := 24in$

$L_4 := L_2$

Slab clear cover $C_{cf} := 3.5in$

Slab effective depth $d_f := D_f - C_{cf}$

$d_f = 20.5 \cdot in$

Note: Upper case "L" represents the out-to-out dimension.
Lower case "l" representst the clear span dimension.

Length_Check :=	"Ok" if $L_1 \geq L_2$
	"NG - Assign larger dim to L1" otherwise

Length_Check = "Ok"

Wall Thickness

$D_1 := 18in$ Wall 1 (Span AD) wall thickness

$D_2 := 18in$ Wall 2 (Span AB) wall thickness

$D_3 := D_1$

$D_4 := D_2$

Wall 3 (Span BC) must be of the same thickness as Wall 1 (Span AD)
Wall 4 (Span CD) must be of the same thickness as Wall 2 (Span AB)

Clear Spans

$l_1 := L_1 - 2 \cdot D_2$ $l_1 = 10\,ft$ Clear span wall AD

$l_2 := L_2 - 2D_1$ $l_2 = 8\,ft$ Clear span of wall AB

$l_3 := l_1$ $l_3 = 10\,ft$ Clear span of wall BC

$l_4 := l_2$ $l_4 = 8\,ft$ Clear span of wall CD

Concrete Section Areas

$b = 1\,ft$

$A_{g1} := D_1 \cdot b$ $A_{g1} = 1.5\,ft^2$

$A_{g2} := D_2 \cdot b$ $A_{g2} = 1.5\,ft^2$

$A_{g3} := A_{g1}$ $A_{g3} = 1.5\,ft^2$

$A_{g4} := A_{g2}$ $A_{g4} = 1.5\,ft^2$

$d_f = 20.5 \cdot in$

Base Slab Reinforcment

Reinforcement information provided by the record drawings. $Bar_Sp_{slb} := 12in$

$BarNo_{slb} :=$

$BarNo_{slb} = 5$

Base Slab Reinforcement

$$A_{slb} := \frac{A_{slb}}{\frac{Bar_Sp_{slb}}{12in}}$$

$A_{slb} = 0.31 \cdot in^2$

This gatewell is a special case with large openings and inlet and outlet placed monolithically. Reinforcement given is above the openings.

Wall Reinforcement Areas At Elevation Bot of Gatewell:

Reinforcement information provided by the record drawings

I. WALL AD

$L_1 = 13\,ft$

END SUPPORT AND MID-SPAN REINFORCEMENT

$Bar_Spacing_{ADout} := 10in$

$Bar_Spacing_{ADin} := 6in$

$Bar_No_{ADout} :=$

$Bar_No_{ADout} = 7$

$Bar_No_{ADin} :=$

$Bar_No_{ADin} = 8$

Wall Reinforcement

The area of steel will be calculated /ft:

$$A_{ADout} := \frac{A_{out}}{\frac{Bar_Spacing_{ADout}}{12in}}$$

$$A_{ADin} := \frac{A_{in}}{\frac{Bar_Spacing_{ADin}}{12in}}$$

$A_{out} = 0.6 \cdot in^2$

$A_{in} = 0.79 \cdot in^2$

$A_{ADout} = 0.72 \cdot in^2$

$A_{ADin} = 1.58 \cdot in^2$

Reinf at wall AB is based on the Half-Section thru gate chamber detail on sheet 4 of 5.

II. WALL AB

$L_2 = 11\,ft$

END SUPPORT AND MID-SPAN REINFORCEMENT

$Bar_Spacing_{ABout} := 6in$

$Bar_Spacing_{ABin} := 6in$

$Bar_No_{ABout} :=$

$Bar_No_{ABout} = 6$

$Bar_No_{ABin} :=$

$Bar_No_{ABin} = 7$

Wall Reinforcement

The area of steel will be calculated /ft:

$$A_{ABout} := \frac{A_{out}}{\frac{Bar_Spacing_{ABout}}{12in}} \qquad A_{ABin} := \frac{A_{in}}{\frac{Bar_Spacing_{ABin}}{12in}} \qquad A_{out} = 0.44 \cdot in^2 \qquad A_{in} = 0.6 \cdot in^2$$

$$A_{ABout} = 0.88 \cdot in^2 \qquad A_{ABin} = 1.2 \cdot in^2$$

Reinforcement Bar Diameters

Reinforcement Clear Cover

Wall AD

$coverADout := 3in$

$coverADin := 3in$

Wall AB

$coverABout := 3in$

$coverABin := 3in$

Effective Depth:

These effective depths are used for the AD end analysis

$$D_1 = 18 \cdot in \qquad diaADin = 1 \cdot in \qquad diaADout = 0.875 \cdot in$$

$$d_{AD_in_end} := coverADin + \frac{diaADin}{2} \qquad d_{AD_out_end} := D_1 - coverADout - \frac{diaADout}{2}$$

$$d_{AD_in_end} = 3.5 \cdot in \qquad d_{AD_out_end} = 14.563 \cdot in$$

These effective depths are used for the AB end analysis

$$diaABin = 0.875 \cdot in \qquad diaABout = 0.75 \cdot in \qquad D_2 = 1.5\,ft$$

$$d_{AB_in_end} := coverABin + \frac{diaABin}{2} \qquad d_{AB_out_end} := D_2 - coverABout - \frac{diaABout}{2}$$

$$d_{AB_in_end} = 3.437 \cdot in \qquad d_{AB_out_end} = 14.625 \cdot in$$

These effective depths are used for the AD mid analysis

$$diaADin = 1 \cdot in \qquad diaADout = 0.875 \cdot in$$

$$d_{AD_in_mid} := D_1 - coverADin - \frac{diaADin}{2} \qquad d_{AD_out_mid} := coverADout + \frac{diaADout}{2}$$

$$d_{AD_in_mid} = 14.5 \cdot in \qquad d_{AD_out_mid} = 3.437 \cdot in$$

These effective depths are used for the AB mid analysis

$diaABin = 0.875 \cdot in$ $diaABout = 0.75 \cdot in$

$d_{AB_in_mid} := D_2 - coverABin - \frac{diaABin}{2}$

$d_{AB_in_mid} = 14.563 \cdot in$

$d_{AB_out_mid} := coverABout + \frac{diaABout}{2}$

$d_{AB_out_mid} = 3.375 \cdot in$

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete.
- The Portland Cement Association pamphlet, Engineered Concrete Structures, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Soil Properties $\phi_B := 21deg$ $\gamma_{soil} := 115pcf$

At Rest Soil Pressure $K_o := 1 - \sin(\phi_B)$ $K_o = 0.642$

Active Soil Pressure: $K_a := \tan\left(45deg - \frac{\phi_B}{2}\right)^2$ $K_a = 0.472$

Concrete Unit Weight $\gamma_c := 150pcf$ Concrete Properties $f'_c := 3ksi$ Water Unit Weight $\gamma_w := 62.4pcf$

Steel Properties $F_y := 40ksi$ Modulus of Elasticity: $Es := 29000ksi$

$\beta 1 := if\left(f'_c \le 4ksi, 0.85, if\left(f'_c > 8ksi, 0.65, 0.85 - 0.05 \cdot \frac{f'_c - 4ksi}{ksi}\right)\right)$ $\beta 1 = 0.85$ $f'_c = 3000\,psi$

Load & Resistance Factor Design Resistance and Load Factor Values

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$ Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Load Factor $\gamma_H := 1.0$ Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor $\gamma_X := 1.0$ Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

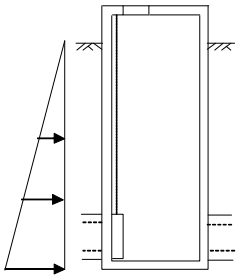
Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) and Strength Reduction Factors (.85 for shear, 0.90 for bending) are not applied for in this initial evaluation. Instead a factor of safety is computed for the existing structure. If a strength concern is found, a design approach (applying these factors directly) will be taken for any recommended solutions.

Wall Loading

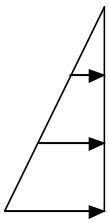
Wall Loading

WALLS

Special Case with Monolithically Placed exit and entrance:



Soil



$H = 43.5\,ft$

$H_2 := H$ OR

$H_2 = 43.5\,ft$ $H_2 := H_2 - 7ft$

$W_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma_{soil} \cdot H_2$

$W_1 = 2693 \cdot psf$

Soil & Water

$H_2 = 36.5\,ft$ &

$H_3 := \text{if}\left[H < H_{blanket}, \frac{H}{H_{blanket}} \cdot \left(HGL + H_{blanket}\right) - HEAD, H + HGL - HEAD\right]$

$H_3 = 42.4\,ft$

$H_3 := H_3 - 7ft$

$W_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot \left(\gamma_{soil} - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_2 = 3441 \cdot psf$

Check to determine if water to top of wet well with reduction factor, γ_X, or soil loading is worst case

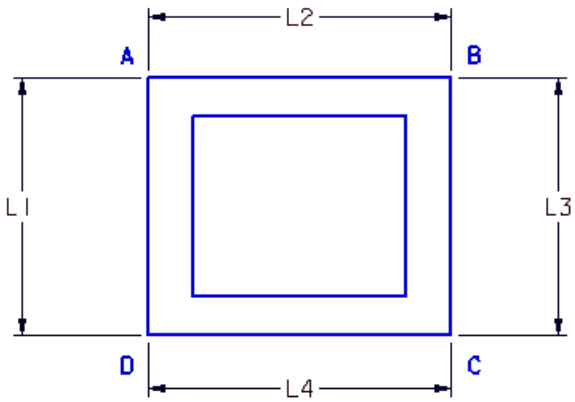
$W := \text{if}\left(W_1 > W_2, W_1, W_2\right) \cdot b$ $W = 3441 \cdot plf$

Moment Distribution of Gatewell Walls

NOTE: Check continuity of reinforcement at corners. If continuity is present, use moment distribution to determine the demand moment values. Otherwise, analyze the gatewell wall sections as simply supported beams.

The following calculations determine the distributed moment based on relative stiffness. In order for the calculations to be valid, the following must be true:
1. The structure has four walls. The walls that are parallel to each other are of the same thickness.
2. The EI value is based on the wall length and the wall thickness.
3. The walls are orthagonal.
4. The walls are exposed to uniform loading W, and $W = 3441 \cdot plf$ is the same for each wall.

Determine Moment and Shear:



Design Section Length

$L_1 = 13\,ft$ $L_2 = 11\,ft$ $L_3 = 13\,ft$ $L_4 = 11\,ft$

Moment of Inertia:

$b = 1\,ft$ $D_1 = 1.5\,ft$ $D_2 = 1.5\,ft$

$D_4 = 1.5\,ft$ $D_3 = 1.5\,ft$

$I_{beam1} := \frac{b \cdot D_1^3}{12}$ $I_{beam1} = 5832 \cdot in^4$

$I_{beam2} := \frac{b \cdot D_2^3}{12}$ $I_{beam2} = 5832 \cdot in^4$

$I_{beam3} := I_{beam1}$ $I_{beam3} = 5832 \cdot in^4$

$I_{beam4} := I_{beam2}$ $I_{beam4} = 5832 \cdot in^4$

Enter End Conditions:

All carryover factors are equal to 0.5 (idealize as a continuous beam over supports A, B, C and D).

$COF_A := 0.5$ $COF_B := COF_A$ $COF_C := COF_A$ $COF_D := COF_A$

$A := COF_A$ $B := COF_B$ $C := COF_C$ $D' := COF_D$

Fixed End Moments For A Uniform Load Between Fixed Supports using center of support to center of support distance per ACI 318-02 8.7.2 :

$L_1 = 13\,ft$ $L_2 = 132 \cdot in$ $L_3 = 13\,ft$ $L_4 = 11\,ft$ $W = 3440.829 \cdot plf$

$M_{fix_1} := \frac{W \cdot (L_1 - D_2)^2}{12}$ $M_{fix_1} = 37.921 \cdot kip \cdot ft$ $M_{fix_2} := \frac{W \cdot (L_2 - D_1)^2}{12}$ $M_{fix_2} = 25.878 \cdot kip \cdot ft$

$M_{fix_3} := \frac{W \cdot (L_3 - D_2)^2}{12}$ $M_{fix_3} = 37.921 \cdot kip \cdot ft$ $M_{fix_4} := \frac{W \cdot (L_4 - D_1)^2}{12}$ $M_{fix_4} = 25.878 \cdot kip \cdot ft$

Distribution Factors

	M_{AD}	M_{AB}	M_{BA}	M_{BC}	M_{CB}	M_{CD}	M_{DC}
COF	0.50	0.50	0.50	0.50	0.50	0.50	0.50
DF	0.46	0.54	0.54	0.46	0.46	0.54	0.54
FEM	-37.92	25.88	-25.88	37.92	-37.92	25.88	-25.88
D#1	5.520	6.523	-6.523	-5.520	5.520	6.523	-6.523
CO#1	-2.760	-3.262	3.262	2.760	-2.760	-3.262	3.262
DF#2	2.760	3.262	-3.262	-2.760	2.760	3.262	-3.262
CO#2	-1.380	-1.631	1.631	1.380	-1.380	-1.631	1.631
DF#3	1.380	1.631	-1.631	-1.380	1.380	1.631	-1.631
CO#3	-0.690	-0.815	0.815	0.690	-0.690	-0.815	0.815
DF#4	0.690	0.815	-0.815	-0.690	0.690	0.815	-0.815
CO#3	-0.345	-0.408	0.408	0.345	-0.345	-0.408	0.408
DF#4	0.345	0.408	-0.408	-0.345	0.345	0.408	-0.408
	-32.40	32.40	-32.40	32.40	-32.40	32.40	-32.40

Attach proper units to distributed values

$M_{ADend} := M'_{AD} \cdot ft \cdot kip$ $M_{ADend} := M'_{AB} \cdot ft \cdot kip$

$M_{CBend} := M'_{CB} \cdot ft \cdot kip$ $M_{DCend} := M'_{DC} \cdot ft \cdot kip$

End-of-Span Moments After Distribution

$M_{ADend} = -32.401 \cdot kip \cdot ft$ $M_{ADend} = -32.401 \cdot kip \cdot ft$

$M_{CBend} = -32.401 \cdot kip \cdot ft$ $M_{DCend} = -32.401 \cdot kip \cdot ft$

Note that the end-of-span moment value is the same at each joint because of uniform and equal loading present on all spans.

$W = 3440.829 \cdot plf$

Note: L_1 represents span AD, L_2 represents span AB, L_3 represents span BC and L_4 represents span CD

Mid-Span Moment Values

Moment Envelope Values

$$M_{UADin} := \left[\frac{W\cdot(L_1 - D_2)^2}{8} - |M_{ADend}|\right] \qquad M_{UADin} = 24.48\cdot kip\cdot ft \qquad \frac{W\cdot(L_1 - D_2)^2}{8} = 56.881\cdot kip\cdot ft$$

$$M_{UABin} := \left[\frac{W\cdot(L_2 - D_1)^2}{8} - |M_{ADend}|\right] \qquad M_{UABin} = 6.416\cdot kip\cdot ft \qquad \frac{W\cdot(L_2 - D_1)^2}{8} = 38.817\cdot kip\cdot ft$$

$$M_{UCBin} := \left[\frac{W\cdot(L_3 - D_2)^2}{8} - |M_{CBend}|\right] \qquad M_{UCBin} = 24.48\cdot kip\cdot ft \qquad \frac{W\cdot(L_3 - D_2)^2}{8} = 56.881\cdot kip\cdot ft$$

$$M_{UDCin} := \left[\frac{W\cdot(L_4 - D_1)^2}{8} - |M_{DCend}|\right] \qquad M_{UDCin} = 6.416\cdot kip\cdot ft \qquad \frac{W\cdot(L_4 - D_1)^2}{8} = 38.817\cdot kip\cdot ft$$

The demand moment will be taken at the support face IAW ACI 318-02, 8.7.2 and 8.7.3.

$$M_{UADface} := \left[M_{ADend} + \frac{W\cdot D_2}{8}\cdot(2L_1 - 3D_2)\right]$$

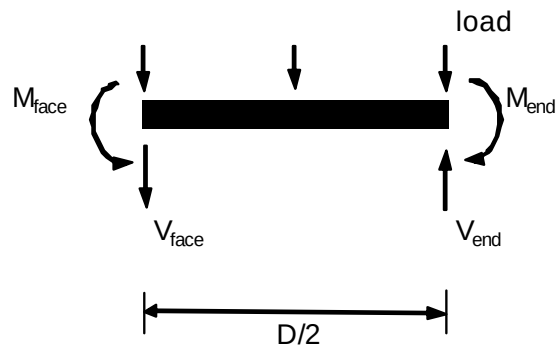
$$M_{UADface} = -18.53\cdot kip\cdot ft$$

$$M_{UABface} := \left[M_{ADend} + \frac{W\cdot D_1}{8}\cdot(2\cdot L_2 - 3D_1)\right]$$

$$M_{UABface} = -21.111\cdot kip\cdot ft$$

$$M_{UCBface} := M_{UADface} \qquad M_{UDCface} := M_{UABface}$$

Free Body Diagram

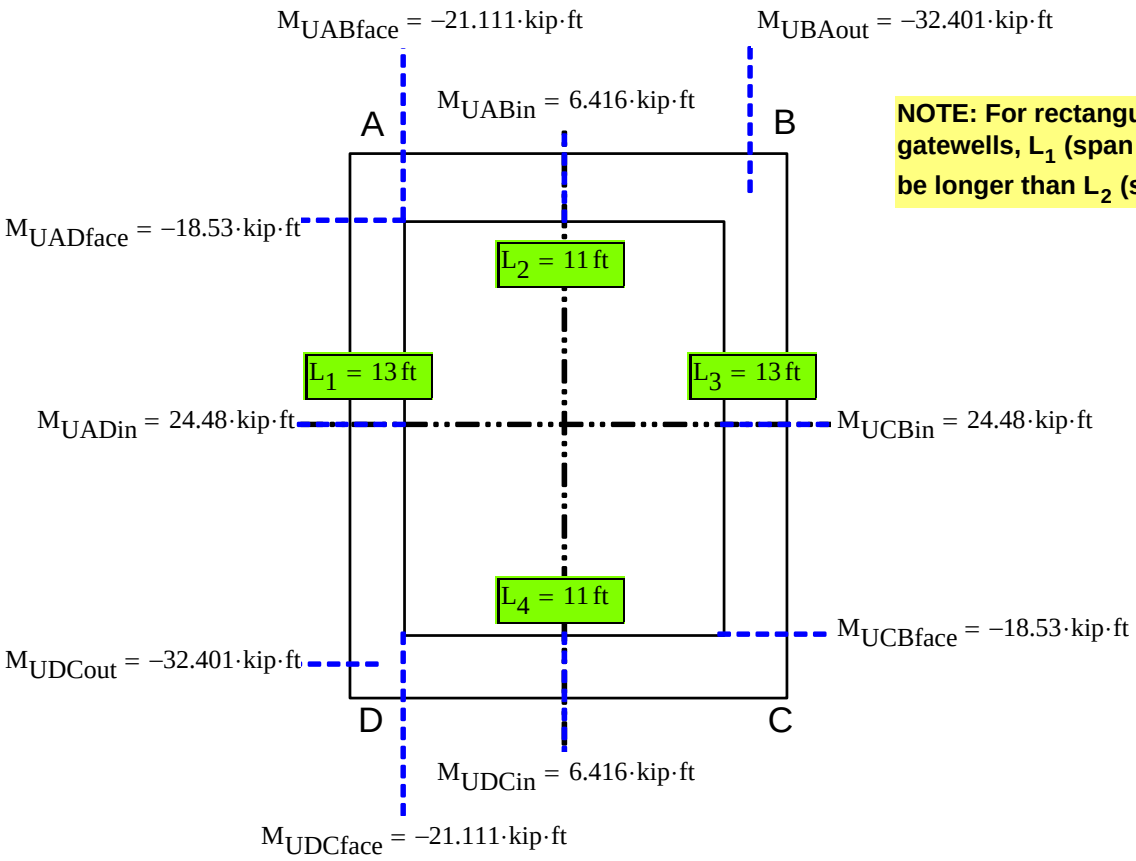


"D" represents the orthagonal wall thickness

$M_{UDCout} := M_{DCend}$ $M_{UDCout} = -32.401 \cdot kip \cdot ft$

$M_{UABout} := M_{ADend}$ $M_{UABout} = -32.401 \cdot kip \cdot ft$ $M_{UBAout} := M_{ADend}$

FACTORED MOMENT DISTRIBUTION VALUES



NOTE: For rectangular gatewells, L_1 (span AD) shall be longer than L_2 (span AB).

The "L" values are the "outside-to-outside" dimensions.

Ensure the reinforcement size entered earlier reflects the steel that will be in tension at the mid-span.
For example:
IF: The mid-span moment demand value obtained from moment distribution is negative.
THEN: The steel located near the wall's exterior face is in tension. Select the reinforcement size that is present near the wall exterior face.

Wall Thrust

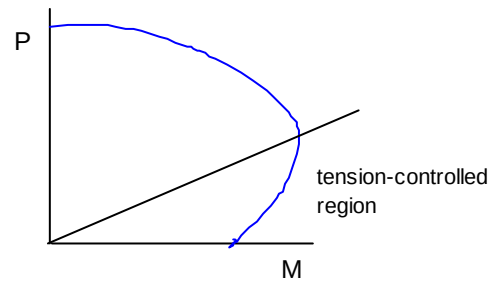
Thrust was considered when evaluating the wall section moment and shear capacity. A higher value of thrust within the tension-controlled region of the interaction diagram results in a higher nominal strength value. The calculated thrust is based upon the active (lower) value of lateral loading. This is deemed conservative relative to the at-rest (higher) value of lateral loading. Note that for lateral loading on the walls, the at-rest loading condition was used.

$K_a = 0.472$ $\gamma_{soil} = 115 \cdot pcf$ $H_2 = 36.5\ ft$

$W_{1active} := \gamma_L \cdot \gamma_H \cdot K_a \cdot \gamma_{soil} \cdot H_2$

$W_{2active} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_a \cdot \left(\gamma_{soil} - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_{1active} = 1982.711 \cdot psf$ $W_{2active} = 3115.835 \cdot psf$



$Wactive := if\left(W_{1active} > W_{2active}, W_{1active}, W_{2active}\right) \cdot b$

$Wactive = 3115.835 \cdot plf$ $W = 3440.829 \cdot plf$

Total Trust Acting On Wall AD:

$P_{ADthrust} := \frac{Wactive \cdot L_2}{2}$ $P_{ADthrust} = 17.137 \cdot kip$

Total Trust Acting On Wall AB:

$P_{ABthrust} := \frac{Wactive \cdot L_1}{2}$ $P_{ABthrust} = 20.253 \cdot kip$

Wall AD (Wall 1) Analysis

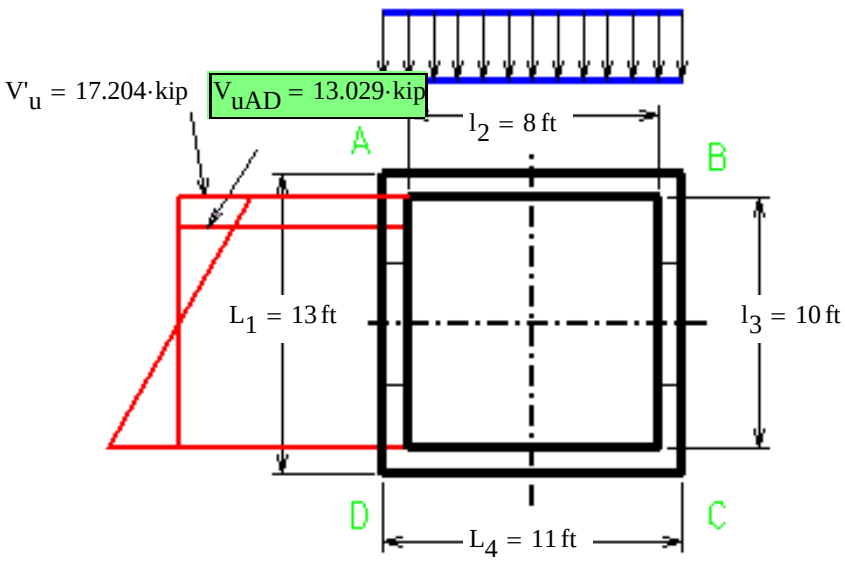
Shear Demand:

$W = 3440.829 \cdot plf \qquad l_1 = 10\,ft$

$V'_u := W \cdot \frac{l_1}{2} \qquad V'_u = 17.204 \cdot kip$

$d_{AD_out_end} = 14.563 \cdot in$

$V_{uAD} := V'_u \cdot \frac{\frac{l_1}{2} - d_{AD_out_end}}{\frac{l_1}{2}} \qquad V_{uAD} = 13.029 \cdot kip \qquad W = 3440.829 \cdot plf$



Shear Capacity and Factor of Safety:

$d_{AD_out_end} = 14.563 \cdot in \qquad \frac{P_{ADthrust}}{2000 \cdot A_{g1}} = 0.04\,psi \qquad f'_c = 3 \cdot ksi \quad b = 1\,ft$

Shear Concrete Resistance value at Distance d from Support ACI 318R-11.1.3.1

$\phi V_{n_AD} := \phi_V \cdot \left[2 \cdot \left(1 + \frac{P_{ADthrust}}{2000 \cdot A_{g1}} \cdot \frac{in^2}{lbf} \right) \cdot \left(\sqrt{f'_c \cdot psi} \right) \cdot \left(b \cdot d_{AD_out_end} \right) \right]$ ACI EQ (11-4)

$\phi V_{n_AD} = 19.902 \cdot kip$

$V_{uAD} = 13.029 \cdot kip$

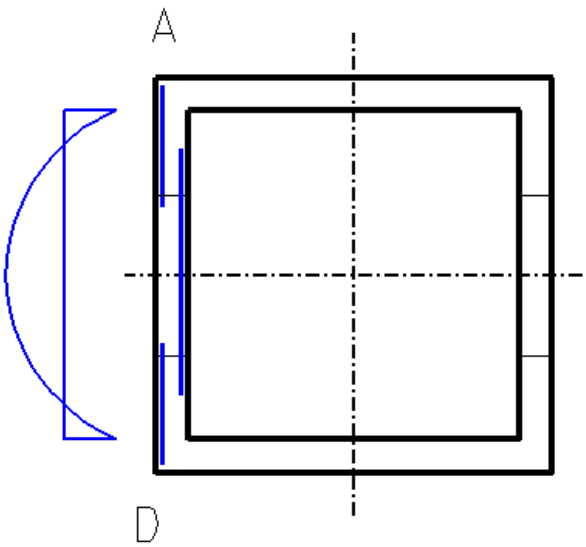
Factor of Safety $FS_{ADshear} := \frac{\phi V_{n_AD}}{V_{uAD}}$

$FS_{ADshear} = 1.528$

DEPTH OF EQUIVALENT RECTANGULAR STRESS BLOCK

Wall AD Con't

$F_y = 40 \cdot ksi$ $f'_c = 3 \cdot ksi$ $b = 1\,ft$ $W = 3.441 \cdot klf$ $L_1 = 13\,ft$



$A_{ADout} = 0.72 \cdot in^2$ $a_{AD_out} := \frac{A_{ADout} \cdot F_y}{0.85 f'_c \cdot b}$

$A_{ADin} = 1.58 \cdot in^2$ $a_{AD_in} := \frac{A_{ADin} \cdot F_y}{0.85 f'_c \cdot b}$

$a_{AD_out} = 0.941 \cdot in$ $a_{AD_in} = 2.065 \cdot in$

Negative Moment Redistribution

Wall AD Con't

ACI 318-02, 8.4 allows moment redistribution (plastic hinge formation). Refer to 8.4 for moment redistribution limitations.

Determine if redistribution is possible. To occur, the steel must yield prior to concrete crushing.

$\beta 1 = 0.85 \qquad f'_c = 3000\,psi$

Beta one value is equal to 0.85 for f'c equal to or less than 4000 psi

$c_{AD_out} := \frac{a_{AD_out}}{\beta 1}$ $\quad c_{AD_out} = 1.107 \cdot in$ $\quad c_{AD_in} := \frac{a_{AD_in}}{\beta 1}$ $\quad c_{AD_in} = 2.43 \cdot in$

The net tensile strain values

$$\varepsilon_{tout} := 0.003 \cdot \left(\frac{d_{AD_out_end}}{c_{AD_out}} - 1 \right) \qquad \varepsilon_{tout} = 0.036$$

Negative moment distribution is permissible when the tensile strain is greater than 0.0075, per ACI 318-02, 8.4

Reduce negative moment at face of wall AD end support and increase the positive mid-span moment (if applicable)

$$R_{nface} := \frac{|M_{UADface}|}{\phi_B \cdot b \cdot d_{AD_out_end}^2} \qquad R_{nface} = 87.38 \cdot psi \qquad M_{UADface} = -18.53 \cdot kip \cdot ft$$

$M_{UADin} = 24.48 \cdot kip \cdot ft$

$$\varepsilon'_{tface} := \begin{cases} 0.003 \cdot \left(\dfrac{\beta 1}{1 - \sqrt{1 - \frac{40}{17} \cdot \frac{R_{nface}}{f'_c}}} - 1 \right) & \text{if } (\varepsilon_{tout} \ge 0.0075) \wedge (M_{UADin} > 0) \\ 0 & \text{otherwise} \end{cases} \qquad \varepsilon'_{tface} = 0.07$$

PCA 318-05 Notes, page 8-3

$$\varepsilon'_ADfacedelta := \begin{cases} \varepsilon'_{tface} \cdot 10 & \text{if } 0 < \varepsilon'_{tface} \le 0.020 \\ 0 & \text{if } \varepsilon'_{tface} = 0 \\ 0.20 & \text{otherwise} \end{cases} \qquad \varepsilon'_ADfacedelta = 0.200 \qquad \varepsilon'_{tface} \cdot 10 = 0.701$$

$$RedistAD := \begin{cases} \text{"Permitted"} & \text{if } \varepsilon'_{tface} \ne 0 \\ \text{"Not Permitted"} & \text{otherwise} \end{cases} \qquad RedistAD = \text{"Permitted"}$$

IF: The strain is greater than or equal to 0.0075 <u>and</u> the mid span moment is positive
THEN: Negative moment redistribution is permitted. OTHERWSE: Negative moment redistribution is not permitted and "N/A" is printed.

Redistributed (reduced) negative moment at the support face

$$M'_{UADface} := \begin{cases} M_{UADface} \cdot (1 - \varepsilon'_ADfacedelta) & \text{if } \varepsilon'_ADfacedelta \ne 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADface} = -14.824 \cdot kip \cdot ft \qquad M_{UADface} = -18.53 \cdot kip \cdot ft$

Wall AD Con't

$$M'_{UADout} := \begin{cases} M_{UDCout} \cdot (1 - \varepsilon'_ADfacedelta) & \text{if } \varepsilon'_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADout} = -25.921 \cdot kip \cdot ft$ $M_{UADface} = -18.53 \cdot kip \cdot ft$ $M_{UDCout} = -32.401 \cdot kip \cdot ft$

Corresponding redistributed (increased) mid-span moment

$$M'_{UADin} := \begin{cases} \frac{W \cdot (L_1 - D_2)^2}{8} \cdot (\gamma_L \cdot \gamma_H \cdot \gamma_X) + M'_{UADout} & \text{if } M'_{UADout} \neq \text{"N/A"} \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADout} = -25.921 \cdot kip \cdot ft$

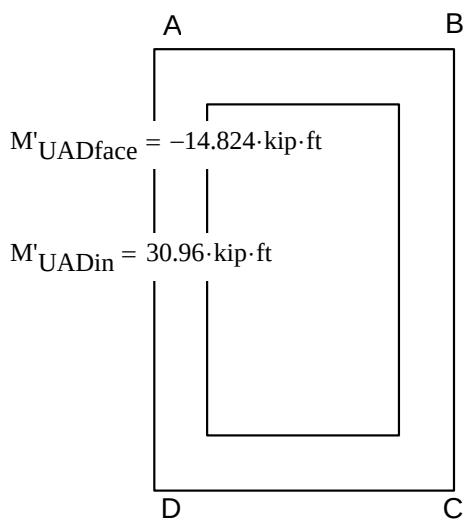
$M'_{UADin} = 30.96 \cdot kip \cdot ft$

$$\frac{W \cdot (L_1 - D_2)^2}{8} = 56.881 \cdot kip \cdot ft$$

$M_{UADin} = 24.48 \cdot kip \cdot ft$

Moments on Span AD After Redistribution

RedistAD = "Permitted"



Note: If redistribution is not allowed, "N/A" is shown.

Wall AD Magnified Moment Check

Wall AD Con't

The effects of wall slenderness shall be checked.

10.12.2 — In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy:

$$\frac{k\ell_u}{r} \le 34 - 12(M_1/M_2) \qquad (10\text{-}8)$$

where the term $[34 - 12M_1/M_2]$ shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

Table 8-3 Deflection Coefficient K

		K
1.	Cantilevers (deflection due to rotation at supports not included)	2.40
2.	Simple beams	1.0
3.	Continuous beams	1.2-0.2 M_o/M_a
4.	Fixed-hinged beams (midspan deflection)	0.80
5.	Fixed-hinged beams (maximum deflection using maximum moment)	0.74
6.	Fixed-fixed beams	0.60

For other types of loading, K values are given in Ref. 8.2.

M_o = Simple span moment at midspan $\left(\frac{w\ell^2}{8}\right)$

M_a = Net midspan moment.

$M_1 := M_{UADface}$ $\qquad$ $M_2 := M_{UADface}$

$M_1 = -18.53 \cdot kip \cdot ft$ $\qquad$ $M_2 = -18.53 \cdot kip \cdot ft$

From the table from the right, the k value for fixed fixed condition: $k := 0.6$

Ubraced length: $l_u := L_1$ $\qquad$ $l_u = 13\,ft$

Radius of Gyration, r:

Thickness of Wall: $D_1 = 1.5\,ft$

10.11.2 — It shall be permitted to take the radius of gyration r equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, it shall be permitted to compute the radius of gyration for the gross concrete section.

$r := 0.3 \cdot D_1$ $\qquad$ $r = 0.45\,ft$

$Slender_Ratio := k \cdot \frac{l_u}{r}$ $\qquad$ $Slender_Ratio = 17.333$

$$ADSlenderness_Check := if\left[Slender_Ratio \le \left[34 - 12 \cdot \left(\frac{M_1}{M_2}\right)\right], \text{"Ok"}, \text{"Consider Slenderness using ACI 10.10.1"}\right]$$

$ADSlenderness_Check = \text{"Ok"}$

IF: ADSlenderness_Check is OK
THEN: Moment magnification is not required

IF: ADSlenderness_Check is NOT OK
THEN: Wall is slender. Moment magnification per ACI 318-02, 10.12 shall be followed

Wall AD Mid-Span Nominal Moment Capacity

$L_1 = 13\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_1 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar_No_{ADin} = 8$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverADin = 3 \cdot in$ Cover -in

Axial Compression with ACI Reduction value: $\phi_B = 1$ $Bar_No_{ADin} = 8$ $diaADin = 1 \cdot in$

$row := 2$

Iterations $i := 1 .. 2$ $i =$

1
2

Bar area at i $A_1 := A_{ADout}$ $A_2 := A_{ADin}$ $A_i =$

0.72
1.58

$\cdot in^2$

$coverADout = 3 \cdot in$ $diaADout = 0.875 \cdot in$ $coverADin = 3 \cdot in$ $diaADin = 1 \cdot in$

$$dd_1 := \begin{cases} coverADout + \dfrac{diaADout}{2} & \text{if } M_{UADin} > 0ft \cdot kip \\ coverADin + \dfrac{diaADin}{2} & \text{otherwise} \end{cases}$$

$M_{UADin} = 24.48 \cdot kip \cdot ft$

$dd_1 = 3.437 \cdot in$

$$dd_2 := \begin{cases} D_1 - \left(coverADin + \dfrac{diaADin}{2}\right) & \text{if } M_{UADin} > 0ft \cdot kip \\ D_1 - \left(coverADout + \dfrac{diaADout}{2}\right) & \text{otherwise} \end{cases}$$

$dd_2 = 14.5 \cdot in$

Wall AD, Midspan Column Interaction Results

Wall AD Midspan

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_1 = 1.5\,ft \quad b = 1\,ft \quad \beta 1 = 0.85$

Given $\quad c := .2 \cdot D_1 \quad c = 3.6 \cdot in$

$$P_{ADthrust} = \left[\sum_{i=1}^{row} \left[\left[\begin{array}{l} \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots \quad \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) \\ F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \le -F_y \end{array} \right] \cdot A_i \right] \ldots \right] \cdot \phi_B$$

$$+ 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \quad c = 3.237 \cdot in \quad a := \text{if}\left(\beta 1 \cdot c > D_1, D_1, \beta 1 \cdot c\right) \quad a = 2.752 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(Es \cdot \varepsilon s_i > F_y, F_y, \text{if}\left(Es \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, Es \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > dd_i, 0.85 \cdot f'_c, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D_1}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.286 ft	-0	-5.375 ·ks	0.72 $\cdot in^2$	-3.87 ·kip	-21.526 ·kip·in
1.208	-0.01	-40	1.58	-63.2	347.6

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 326 \cdot kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D_1}{2} - \frac{a}{2}\right) \qquad Mc = 642 \cdot kip \cdot in$

$\phi M_{nADin1} := (Mc + Mst) \qquad \phi M_{nADin1} = 80.67 \cdot kip \cdot ft$

$$\phi M_{nADin} := \begin{cases} \phi M_{nADin1} & \text{if } M_{UADin} > 0ft \cdot kip \\ \phi M_{nADin1} \cdot (-1) & \text{otherwise} \end{cases}$$

Wall AD Mid-Span Nomimal Moment Strength $\quad \phi M_{nADin} = 80.673 \cdot kip \cdot ft$

Wall AD Mid-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ADmid}

Wall AD Mid-Span Con't

$\phi M_{nADin} = 80.673 \cdot kip \cdot ft$

$M_{UADin} = 24.48 \cdot kip \cdot ft$

$$FS_{ADin} := \left(\left| \frac{\phi M_{nADin}}{M_{UADin}} \right| \right)$$

$FS_{ADin} = 3.295$

Wall AD Mid-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ADmid}$

$\phi M_{nADin} = 80.673 \cdot kip \cdot ft$

Note: An apostrophe signifies a negative moment redistribution-related value.

$M'_{UADin} = 30.96 \cdot kip \cdot ft$

$$FS'_{ADin} := \begin{cases} \left| \frac{\phi M_{nADin}}{M'_{UADin}} \right| & \text{if } \varepsilon'_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ADin} = 2.606$

The mid-span factor of safety always decreases when negative moment redistribution is considered.

ADSlenderness_Check = "Ok"

RedistAD = "Permitted"

ADinspanCheck := if (ADSlenderness_Check ≠ "Ok" , "ANALYSIS INVALID" , "OKAY")

ADinspanCheck = "OKAY"

Wall AD End-of-Span Nominal Moment Capacity

$L_1 = 13\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_1 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar_No_{ADout} = 7$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverADout = 3 \cdot in$ Cover -in

$Bar_No_{ADout} = 7$ $diaADin = 1 \cdot in$

$A_1 := A_{ADin}$ $A_2 := A_{ADout}$ $A_i =$

1.58
0.72

$\cdot in^2$

$coverADout = 3 \cdot in$ $diaADout = 0.875 \cdot in$

$dd_1 := d_{AD_in_end}$ $dd_1 = 3.5 \cdot in$

$dd_2 := d_{AD_out_end}$ $dd_2 = 14.563 \cdot in$

Wall AD, End-of-Span Column Interaction Results

Wall AD End-of-Span

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_1 = 1.5\,ft \quad b = 1\,ft \quad \beta 1 = 0.85$

Given $\quad c := .2 \cdot D_1 \quad c = 3.6 \cdot in$

$$P_{ADthrust} = \left[\sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \\ F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \le -F_y \end{array} \right] \cdot A_i \right] \ldots \right] \cdot \phi_B + 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \quad c = 2.887 \cdot in \quad a := \text{if}\left(\beta 1 \cdot c > D_1, D_1, \beta 1 \cdot c\right) \quad a = 2.454 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(Es \cdot \varepsilon s_i > F_y, F_y, \text{if}\left(Es \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, Es \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > dd_i, 0.85 \cdot f'_c, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D_1}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.292 ft	-0.001	-18.458 ·k	1.58 ·in^2	-29.164 ·kip	-160.403 ·kip·in
1.214	-0.012	-40	0.72	-28.8	160.2

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = -0 \cdot kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D_1}{2} - \frac{a}{2}\right) \qquad Mc = 584 \cdot kip \cdot in$

$\phi M_{nADface1} := (Mc + Mst) \qquad \phi M_{nADface1} = 48.63 \cdot kip \cdot ft$

$\phi M_{nADface} := \begin{cases} \phi M_{nADface1} & \text{if } M_{UADface} > 0ft \cdot kip \\ \phi M_{nADface1} \cdot (-1) & \text{otherwise} \end{cases} \qquad M_{UADface} = -18.53 \cdot kip \cdot ft$

Wall AD End-of-Span Nominal Moment Strength $\quad \phi M_{nADface} = -48.629 \cdot kip \cdot ft$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ADface}

$\phi M_{nADface} = -48.629 \cdot kip \cdot ft$ $M_{UADface} = -18.53 \cdot kip \cdot ft$

$$FS_{ADface} := \left(\left| \frac{\phi M_{nADface}}{M_{UADface}} \right| \right)$$

$FS_{ADface} = 2.624$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ADface}$

$$FS'_{ADface} := \begin{cases} \left| \frac{\phi M_{nADface}}{M'_{UADface}} \right| & \text{if } \varepsilon'_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ADface} = 3.28$

The end-of-span factor of safety always increases when negative moment redistribution is considered.

ADSlenderness_Check = "Ok"

RedistAD = "Permitted"

ADoutspanCheck := if (ADSlenderness_Check ≠ "Ok" , "ANALYSIS INVALID" , "OKAY")

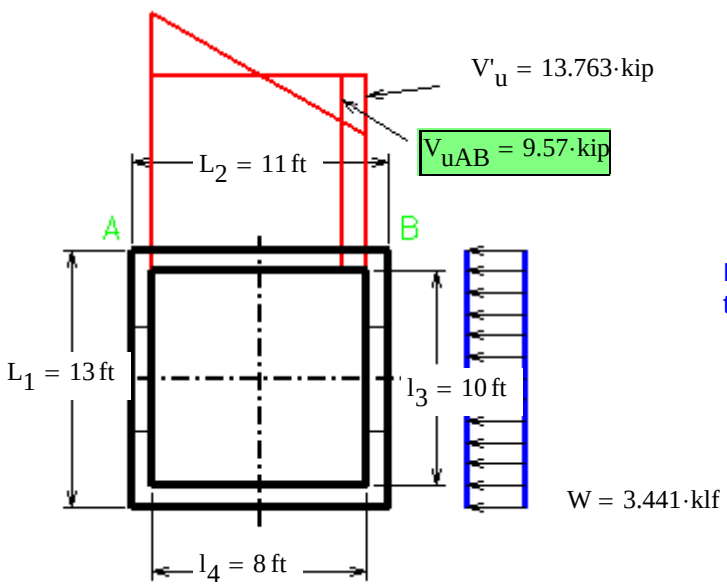
ADoutspanCheck = "OKAY"

Wall AB (Wall 2) Analysis

Shear Demand:

$W = 3440.829\,ft\cdot psf$ $\quad l_2 = 8\,ft$ $\quad d_{AB_out_end} = 14.625\cdot in$

$V'_u := W\cdot\frac{l_2}{2}$ $\quad V'_u = 13.763\cdot kip$ $\quad V_{uAB} := V'_u\cdot\frac{\frac{l_2}{2} - d_{AB_out_end}}{\frac{l_2}{2}}$ $\quad V_{uAB} = 9.57\cdot kip$



Total Thrust into Wall2:

$P_{ABthrust} = 20.253\cdot kip$

Note: Refer to wall AD calculations for derivation of the thrust values acting on wall AB.

Shear Capacity and Factor of Safety

$d_{AB_out_end} = 14.625\cdot in$ $\quad \frac{P_{ABthrust}}{2000\cdot A_{g2}} = 0.047\,psi$ $\quad f'_c = 3000\,psi$ $\quad b = 1\,ft$

Shear Concrete Resistance value at Distance d from Support ACI 318R-11.1.3.1

$$\phi V_{n_AB} := \phi_V\cdot\left[2\cdot\left(1 + \frac{P_{ABthrust}}{2000\cdot A_{g2}}\cdot\frac{in^2}{lbf}\right)\cdot\left(\sqrt{f'_c\cdot psi}\right)\cdot\left(b\cdot d_{AB_out_end}\right)\right]$$

ACI EQ (11-4) $\quad \phi V_{n_AB} = 20.126\cdot kip$

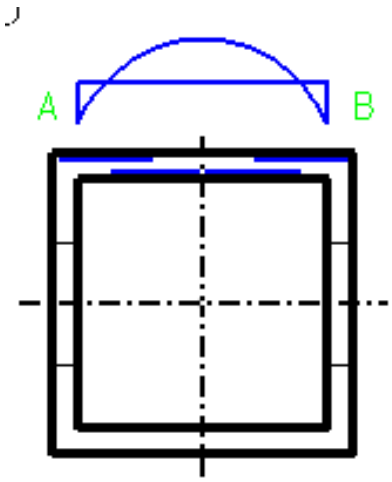
$V_{uAB} = 9.57\cdot kip$

Factor of Safety $\quad FS_{ABshear} := \frac{\phi V_{n_AB}}{V_{uAB}}$ $\quad FS_{ABshear} = 2.103$

Wall AB con't

Depth of Equivalent Rectangular Stress Block

$F_y = 40 \cdot ksi$ $f'_c = 3 \cdot ksi$ $b = 1\,ft$ $W = 3.441 \cdot klf$ $L_1 = 13\,ft$



$A_{ABout} = 0.88 \cdot in^2$ $aABout := \frac{A_{ABout} \cdot F_y}{0.85 f'_c \cdot b}$

$A_{ABin} = 1.2 \cdot in^2$ $aABin := \frac{A_{ABin} \cdot F_y}{0.85 f'_c \cdot b}$

$aABout = 1.15 \cdot in$ $aABin = 1.569 \cdot in$

Negative Moment Redistribution

Wall AB con't

ACI 318-02, 8.4 allows moment redistribution (plastic hinge formation). Refer to 8.4 for moment redistribution limitations.

Determine if redistribution is possible. To occur, the steel must yield prior to concrete crushing.

$\beta 1 = 0.85$ $\quad f'_c = 3000\,psi$

$cABout := \frac{aABout}{\beta 1}$ $\quad cABout = 1.353 \cdot in$ $\quad cABin := \frac{aABin}{\beta 1}$ $\quad c_{AD_in} = 2.43 \cdot in$

The net tensile strain values

$\varepsilon_{tout} := 0.003 \cdot \left(\frac{d_{AB_out_end}}{cABout} - 1 \right)$ $\quad \varepsilon_{tout} = 0.029$

Negative moment distribution is permissible when the tensile strain is greater than 0.0075, per ACI 318-02, 8.4

Reduce negative moment at face of wall AB end support and increase the positive mid-span moment (if applicable)

$R_{nface} := \frac{|M_{UABface}|}{\phi_B \cdot b \cdot d_{AB_out_end}^2}$ $\quad R_{nface} = 98.7 \cdot psi$ $\quad M_{UABface} = -21.111 \cdot kip \cdot ft$

$$\varepsilon'_{tface} := \begin{cases} 0.003 \cdot \left(\frac{\beta 1}{1 - \sqrt{1 - \frac{40}{17} \cdot \frac{R_{nface}}{f'_c}}} - 1 \right) & \text{if } (\varepsilon_{tout} \ge 0.0075) \wedge (M_{UABin} > 0) \\ 0 & \text{otherwise} \end{cases}$$

PCA 318-05 Notes, page 8-3

$\varepsilon'_{tface} = 0.062$

$$\varepsilon'_ABfacedelta := \begin{cases} \varepsilon'_{tface} \cdot 10 & \text{if } 0 < \varepsilon'_{tface} \le 0.020 \\ 0 & \text{if } \varepsilon'_{tface} = 0 \\ 0.20 & \text{otherwise} \end{cases}$$

$\varepsilon'_ABfacedelta = 0.200$ $\quad \varepsilon'_{tface} \cdot 10 = 0.616$

$$RedistAB := \begin{cases} \text{"Permitted"} & \text{if } \varepsilon'_{tface} \ne 0 \\ \text{"Not Permitted"} & \text{otherwise} \end{cases}$$

$RedistAB = \text{"Permitted"}$

IF: The strain is greater than or equal to 0.0075 <u>and</u> the mid span moment is positive
THEN: Negative moment redistribution is permitted. OTHERWSE: Negative moment redistribution is not permitted and "N/A" is printed.

Redistributed (reduced) negative moment at the support face

$$M'_{UABface} := \begin{cases} M_{UABface} \cdot (1 - \varepsilon'_ABfacedelta) & \text{if } \varepsilon'_ABfacedelta \ne 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UABface} = -16.889 \cdot kip \cdot ft$ $\quad M_{UABface} = -21.111 \cdot kip \cdot ft$

Wall AB con't

$$M'_{UABout} := \begin{cases} M_{UDCout} \cdot (1 - \varepsilon'_ABfacedelta) & \text{if } \varepsilon'_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UABout} = -25.921 \cdot kip \cdot ft$ $M_{UABface} = -21.111 \cdot kip \cdot ft$ $M_{UDCout} = -32.401 \cdot kip \cdot ft$

Corresponding redistributed (increased) mid-span moment

$$M'_{UABin} := \begin{cases} \frac{W \cdot (L_2 - D_1)^2}{8} \cdot (\gamma_L \cdot \gamma_H \cdot \gamma_X) + M'_{UABout} & \text{if } M'_{UABout} \neq \text{"N/A"} \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

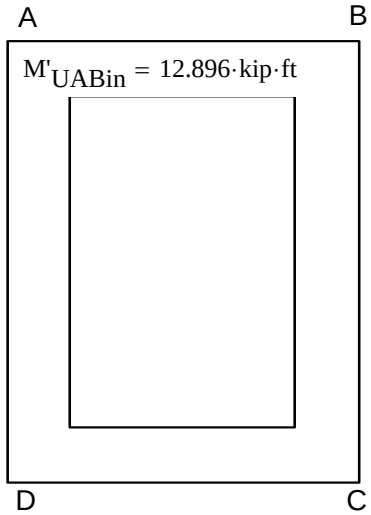
$M'_{UABin} = 12.896 \cdot kip \cdot ft$

$\frac{W \cdot (L_2 - D_1)^2}{8} = 38.817 \cdot kip \cdot f$ $M_{UABin} = 6.416 \cdot kip \cdot ft$

$M'_{UABin} = 12.896 \cdot kip \cdot ft$

Moments on Span AD After Redistribution RedistAB = "Permitted"

$M'_{UABface} = -16.889 \cdot kip \cdot ft$



Note: If redistribution is not allowed, "N/A" is shown.

Wall AB Magnified Moment Check

Wall AB Con't

The effects of wall slenderness shall be checked.

10.12.2 — In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy:

$$\frac{k\ell_u}{r} \le 34 - 12(M_1/M_2) \qquad (10\text{-}8)$$

where the term $[34 - 12M_1/M_2]$ shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

Table 8-3 Deflection Coefficient K

		K
1.	Cantilevers (deflection due to rotation at supports not included)	2.40
2.	Simple beams	1.0
3.	Continuous beams	1.2-0.2 M_o/M_a
4.	Fixed-hinged beams (midspan deflection)	0.80
5.	Fixed-hinged beams (maximum deflection using maximum moment)	0.74
6.	Fixed-fixed beams	0.60

For other types of loading, K values are given in Ref. 8.2.

M_o = Simple span moment at midspan $\left(\frac{w\ell^2}{8}\right)$

M_a = Net midspan moment.

$M_1 := M_{UABface}$ $\qquad$ $M_2 := M_{UABface}$

$M_1 = -21.111 \cdot kip \cdot ft$ $\qquad$ $M_2 = -21.111 \cdot kip \cdot ft$

From the table from the right, the k value for fixed fixed condition: $k := 0.6$

Ubraced length: $l_u := L_2$ $\qquad$ $l_u = 11\,ft$

Radius of Gyration, r:

Thickness of Wall: $D_2 = 1.5\,ft$

10.11.2 — It shall be permitted to take the radius of gyration r equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, it shall be permitted to compute the radius of gyration for the gross concrete section.

$r := 0.3 \cdot D_2$ $\qquad$ $r = 0.45\,ft$

$$Slender_Ratio := k \cdot \frac{l_u}{r} \qquad Slender_Ratio = 14.667$$

$$ABSlenderness_Check := if\left[Slender_Ratio \le \left[34 - 12 \cdot \left(\frac{M_1}{M_2}\right)\right], \text{"Ok"}, \text{"Consider Slenderness using ACI 10.10.1"}\right]$$

$ABSlenderness_Check = \text{"Ok"}$

IF: ADSlenderness_Check is OK
THEN: Moment magnification is not required

IF: ADSlenderness_Check is NOT OK
THEN: Wall is slender. Moment magnification per ACI 318-02, 10.12 shall be followed

Wall AB Mid-Span Nominal Moment Capacity

$L_2 = 11\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_2 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar_No_{ABin} = 7$ **Tension** bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverABin = 3 \cdot in$ Cover -in

Axial Compression with ACI Reduction value: $\phi_B = 1$ $Bar_No_{ABin} = 7$ $diaABin = 0.875 \cdot in$

$diaABout = 0.75 \cdot in$

Bar area at i $A_1 := A_{ABout}$ $A_2 := A_{ABin}$ $A_i = \blacksquare \cdot in^2$

$coverABout = 3 \cdot in$ $diaABout = 0.75 \cdot in$ $coverABin = 3 \cdot in$ $diaABin = 0.875 \cdot in$

$$dd_1 := \begin{cases} coverABout + \dfrac{diaABout}{2} & \text{if } M_{UABin} > 0ft \cdot kip \\ coverABin + \dfrac{diaABin}{2} & \text{otherwise} \end{cases}$$

$M_{UABin} = 6.416 \cdot kip \cdot ft$

$dd_1 = 3.375 \cdot in$

$$dd_2 := \begin{cases} D_2 - \left(coverABin + \dfrac{diaABin}{2}\right) & \text{if } M_{UABin} > 0ft \cdot kip \\ D_2 - \left(coverABout + \dfrac{diaABout}{2}\right) & \text{otherwise} \end{cases}$$

$dd_2 = 14.563 \cdot in$

$coverABout = 3 \cdot in$ $diaABout = \blacksquare \cdot in$

$dd_1 := d_{AB_out_mid}$ $dd_1 = 3.375 \cdot in$

$dd_2 := d_{AB_in_mid}$ $dd_2 = 14.563 \cdot in$

Wall AB, Mid-Span Column Interaction Results

Wall AB Mid-Span con't

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_2 = 18\cdot in \quad b = 1\,ft$ $\quad \beta 1 = 0.85$

Given $\quad c := .2\cdot D_2 \quad c = 3.6\cdot in$

$$P_{ABthrust} = \left[\sum_{i=1}^{row}\left[\left[\begin{array}{l} \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ldots \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \\ F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \le -F_y \end{array}\right]\cdot A_i\right] \ldots \right]\cdot\phi_B$$
$$+ 0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b$$

$c := \text{Find}(c) \quad c = 2.996\cdot in \quad a := \text{if}\left(\beta 1\cdot c > D_2, D_2, \beta 1\cdot c\right) \quad a = 2.547\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right)\cdot\frac{0.003}{c} \qquad fs_i := \text{if}\left(Es\cdot\varepsilon s_i > F_y, F_y, \text{if}\left(Es\cdot\varepsilon s_i < 0 - F_y, 0 - F_y, Es\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > dd_i, 0.85\cdot f'_c, 0ksi\right)\cdot A_i \qquad Ms_i := Fs_i\cdot\left(\frac{D_2}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.281 ft	-0	-10.999 $\cdot$	0.88 $\cdot in^2$	-9.679 $\cdot kip$	-54.443 $\cdot kip\cdot in$
1.214	-0.012	-40	1.2	-48	267

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 213\cdot kip\cdot in$

$Mc := 0.85\cdot f'_c\cdot a\cdot b\cdot\left(\frac{D_2}{2} - \frac{a}{2}\right) \qquad Mc = 602\cdot kip\cdot in$

$\phi M_{nABin1} := (Mc + Mst) \qquad \phi M_{nABin1} = 67.89\cdot kip\cdot ft$

$$\phi M_{nABin} := \begin{cases} \phi M_{nABin1} & \text{if } M_{UABin} > 0ft\cdot kip \\ \phi M_{nABin1}\cdot(-1) & \text{otherwise} \end{cases}$$

Wall AB Mid-Span Nominal Moment Strength $\qquad \phi M_{nABin} = 67.892\cdot kip\cdot ft$

Wall AB Mid-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ABmid}

Wall AB Mid-Span Con't

$\phi M_{nABin} = 67.892 \cdot kip \cdot ft$

$M_{UABin} = 6.416 \cdot kip \cdot ft$

$$FS_{ABin} := \left(\left| \frac{\phi M_{nABin}}{M_{UABin}} \right| \right)$$

$FS_{ABin} = 10.582$

Wall AB Mid-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ABmid}$

$\phi M_{nABin} = 67.892 \cdot kip \cdot ft$

Note: An apostrophe signifies a negative moment redistribution-related value.

$M'_{UABin} = 12.896 \cdot kip \cdot ft$

$$FS'_{ABin} := \begin{cases} \left| \frac{\phi M_{nABin}}{M'_{UABin}} \right| & \text{if } \varepsilon'_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ABin} = 5.265$

The mid-span factor of safety always decreases when negative moment redistribution is considered.

ABSlenderness_Check = "Ok"

RedistAB = "Permitted"

ABinspanCheck := if (ABSlenderness_Check ≠ "Ok" , "ANALYSIS INVALID" , "OKAY")

ABinspanCheck = "OKAY"

Wall AB End-of-Span Nominal Moment Capacity

$L_2 = 11\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_2 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar_No_{ABout} = 6$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverABout = 3 \cdot in$ Cover -in

$Bar_No_{ABout} = 6$ $diaABin = 0.875 \cdot in$

$A_1 := A_{ABin}$ $A_2 := A_{ABout}$

$A_i =$

1.2
0.88

$\cdot in^2$

<==The first value of A1 is the compression steel, A2 is the tension steel

$coverABout = 3 \cdot in$ $diaABout = 0.75 \cdot in$

$dd_1 := d_{AB_in_end}$ $dd_1 = 3.437 \cdot in$ Effective depth of the compression steel

$dd_2 := d_{AB_out_end}$ Effective depth of the tension steel

Wall AB, End-of-Span Column Interaction Results

Wall AB End-of-Span con't

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_2 = 18\cdot in \quad b = 1\,ft$ $\quad \beta 1 = 0.85$

Given $\quad c := .2\cdot D_2 \quad c = 3.6\cdot in$

$$P_{ABthrust} = \left[\sum_{i=1}^{row} \left[\left[\begin{array}{l} \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ldots \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \\ F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \le -F_y \end{array}\right]\cdot A_i\right] \ldots\right]\cdot\phi_B$$

$$+ 0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b$$

$c := \text{Find}(c) \quad c = 2.891\cdot in \quad a := \text{if}\left(\beta 1\cdot c > D_2, D_2, \beta 1\cdot c\right) \quad a = 2.457\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right)\cdot\frac{0.003}{c} \quad fs_i := \text{if}\left(Es\cdot\varepsilon s_i > F_y, F_y, \text{if}\left(Es\cdot\varepsilon s_i < 0 - F_y, 0 - F_y, Es\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > dd_i, 0.85\cdot f'_c, 0ksi\right)\cdot A_i \quad Ms_i := Fs_i\cdot\left(\frac{D_2}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.286 ft	-0.001	-16.45 ·ks	1.2 ·in^2	-19.739 ·kip	-109.801 ·kip·in
1.219	-0.012	-40	0.88	-35.2	198

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 88\cdot kip\cdot in$

$Mc := 0.85\cdot f'_c\cdot a\cdot b\cdot\left(\frac{D_2}{2} - \frac{a}{2}\right) \qquad Mc = 584\cdot kip\cdot in$

$\phi M_{nABface1} := (Mc + Mst) \qquad \phi M_{nABface1} = 56.05\cdot kip\cdot ft$

$$\phi M_{nABface} := \begin{cases} \phi M_{nABface1} & \text{if } M_{UABface} > 0ft\cdot kip \\ \phi M_{nABface1}\cdot(-1) & \text{otherwise} \end{cases}$$

Wall AB End-of-Span Nominal Moment Strength $\quad \phi M_{nABface} = -56.046\cdot kip\cdot ft$

Wall AB End-of-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ABface}

$\phi M_{nABface} = -56.046 \cdot kip \cdot ft$ $M_{UABface} = -21.111 \cdot kip \cdot ft$

$$FS_{ABface} := \left(\left| \frac{\phi M_{nABface}}{M_{UABface}} \right| \right)$$

$FS_{ABface} = 2.655$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ABface}$

$\varepsilon'_ABfacedelta = 0.2$

$$FS'_{ABface} := \begin{cases} \left| \frac{\phi M_{nABface}}{M'_{UABface}} \right| & \text{if } \varepsilon'_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ABface} = 3.319$

The end-of-span factor of safety always increases when negative moment redistribution is considered.

ABSlenderness_Check = "Ok"

RedistAB = "Permitted"

ABoutspanCheck := if (ABSlenderness_Check ≠ "Ok" , "ANALYSIS INVALID" , "OKAY")

ABoutspanCheck = "OKAY"

Strength Analysis Results Summary

Span AD Moment Factor of Safety Analysis Summary and Recommendations

$FS_{ADface} = 2.624$ $FS_{ADin} = 3.295$ $FS'_{ADface} = 3.28$ $FS'_{ADin} = 2.606$

$FS_{ADno_redist} := \min(FS_{ADface}, FS_{ADin})$ $FS'_{ADredist} := \min(FS'_{ADface}, FS'_{ADin})$

$FS_{ADno_redist} = 2.624$ $FS'_{ADredist} = 2.606$

$$FS_{ADmoment} := \begin{cases} \max(FS_{ADno_redist}, FS'_{ADredist}) & \text{if } FS'_{ADredist} \neq \text{"N/A"} \\ FS_{ADno_redist} & \text{otherwise} \end{cases}$$

$FS_{ADin} = 3.295$

$FS_{ADmoment} = 2.624$

$$AD_Recommendation1 := \begin{cases} \text{"Perform Simple Beam Analysis"} & \text{if } FS_{ADin} \geq 2.0 \wedge FS_{ABin} \geq 2.0 \\ \text{"Simple Beam Analysis Not Recommended"} & \text{otherwise} \end{cases}$$

AD_Recommendation1 = "Perform Simple Beam Analysis"

$$AD_Recommendation2 := \begin{cases} \text{"CAUTION - Reliability Questionable"} & \text{if } FS_{ADmoment} < 1.5 \\ \text{"Span AD OKAY for Moment Strength"} & \text{if } FS_{ADmoment} \geq 1.5 \end{cases}$$

AD_Recommendation2 = "Span AD OKAY for Moment Strength"

A listing of the span AD moment demand-capacity values, factors of safety and recommendations are provided in the following table

δ

Scalar Conversion Equations To Allow Text Entry Into Tables

Span AD Moment Strength Analysis Summary						
	End-Support			Mid-Span		
Analysis	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS
Moment Distribution	-18.53	-48.63	2.62	24.48	80.67	3.30
Redistribution of Negative Moments	-14.82		3.28	30.96		2.61
Slenderness Check	Ok					
Redistribution of Negative Moments	Permitted					
Recommendations: Perform Simple Beam Analysis Span AD OKAY for Moment Strength						

N/A: Not Applicable

Span AB Moment Factor of Safety Analysis Summary and Recommendations

$FS_{ABno_redist} := \min(FS_{ABface}, FS_{ABin})$ $FS'_{ABredist} := \min(FS'_{ABface}, FS'_{ABin})$

$FS_{ABno_redist} = 2.655$ $FS'_{ABredist} = 3.319$

$$FS_{ABmoment} := \begin{cases} \max(FS_{ABno_redist}, FS'_{ABredist}) & \text{if } FS'_{ABredist} \neq \text{"N/A"} \\ FS_{ABno_redist} & \text{otherwise} \end{cases}$$

$FS_{ABin} = 10.582$

$FS_{ABmoment} = 3.319$

$$AB_Recommendation1 := \begin{cases} \text{"Perform Simple Beam Analysis"} & \text{if } FS_{ADin} \geq 2.0 \wedge FS_{ABin} \geq 2.0 \\ \text{"Simple Beam Analysis Not Required"} & \text{otherwise} \end{cases}$$

AB_Recommendation1 = "Perform Simple Beam Analysis"

$$AB_Recommendation2 := \begin{cases} \text{"CAUTION - Reliability Questionable"} & \text{if } FS_{ABmoment} < 1.5 \\ \text{"Span AB OKAY for Moment Strength"} & \text{if } FS_{ABmoment} \geq 1.5 \end{cases}$$

AB_Recommendation2 = "Span AB OKAY for Moment Strength"

A listing of the span AB moment demand-capacity values, factors of safety and recommendations are provided in the following table

AB Scalar Conversion for Table

Span AB Moment Strength Analysis Summary						
	End-Support			Mid-Span		
Analysis	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS
Moment Distribution	-21.11	-56.05	2.65	6.42	67.89	10.58
Redistribution of Negative Moments	-16.89		3.32	12.90		5.26
Slenderness Check	Ok					
Redistribution of Negative Moments	Permitted					
Recommendations: Perform Simple Beam Analysis Span AB OKAY for Moment Strength						

N/A: Not Applicable

Shear Factor of Safety Analysis Summary and Recommendations

Span AD

$$FS_{ADshear} = 1.528$$

Span AB

$$FS_{ABshear} = 2.103$$

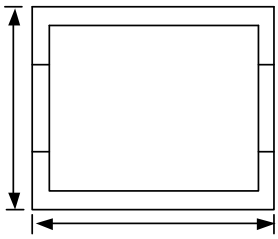
NOTES TO STRENGTH SUMMARY RESULTS:

1. If simple beam analysis is performed on one wall, it must be performed on all four gatewell walls. This is required because fixity is lost when an adjacent wall is analyzed as a simple span. Stability should be considered when the gatewell is analyzed as having pinned supports.
2. Moment redistribution is not permitted when the mid-span and end-support moment values are negative.
3. Moment redistribution is not permitted when the reinforcement strain is less than 0.0075
4. Simple span analysis is recommended when the mid-span moment factor of safety is greater than 2.0. Final decision to use simple span analysis rests with the engineer.
5. Reliability analysis is recommended when the 1.0 < FS < 1.5
6. Caution is provided in the recommendations when the FS < 1.0

This calculation is a supplement to the typical analysis above. In the previous full section analysis, the load and section analyzed were taken above the inlet and outlet openings because for this case, the opening is large and the wall with the opening will govern failure when the load is taken at the very base. The following calculations check the strength of the wall without an opening at top of the base slab.

Properties of the Wall without an opening (Wall AB & CD)

Plan



$L_2 = 11\,ft$

$D_f = 2\,ft$

$H'' := H - D_f - y$

$H'' = 41.5\,ft$

Analyze wall at, $y := 0ft$ above base slab

Wall Thickness $D_2 = 18 \cdot in$ $l_2 = 8\,ft$

Clear Cover $C_c := coverABin$

MID-SPAN REINFORCEMENT

$Bar_Spacing := 6in$

$Bar_No :=$

$Bar_No = 7$

Wall Reinforcement

The area of steel will be calculated /ft:

$$A_s := \frac{A_{out}}{\frac{Bar_Spacing}{12in}} \qquad A_s = 1.2 \cdot in^2$$

Reinforcement Bar Diameters

Effective Depth:

$$D_2 = 18 \cdot in \qquad dia = 0.875 \cdot in$$

$$d := D_2 - C_c - \frac{dia}{2}$$

$$d = 14.563 \cdot in$$

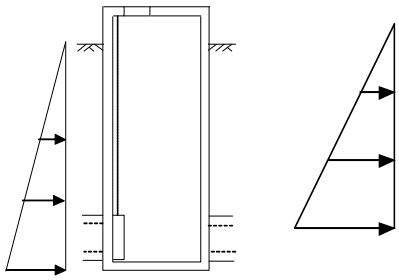
Wall Loading

(Wall without opening near base, cont'd)

Wall Loading

WALLS

Soil



$H'' = 41.5\,ft$

$H'_2 := H''$

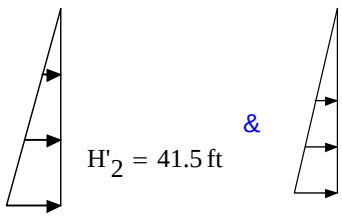
$H'_2 = 41.5\,ft$

Analysis of continuous wall perpendicular to opening. The walls above the opening were analyzed above.

$W'_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma_{soil} \cdot H'_2$

$W'_1 = 3062 \cdot psf$

Soil & Water



$H'_2 = 41.5\,ft$ &

$H'_3 := \text{if}\left[H'' < H_{blanket}, \frac{H''}{H_{blanket}} \cdot (HGL + H_{blanket}) - HEAD, H'' + HGL - HEAD\right]$

$H'_3 = 40.4\,ft$

$W'_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot (\gamma_{soil} - \gamma_w) \cdot H'_2 + \gamma_w \cdot H'_3\right]$

$W'_2 = 3922 \cdot psf$

Check to determine if water to top of wet well with reduction factor, γ_X, or soil loading is worst case

$W' := \text{if}(W'_1 > W'_2, W'_1, W'_2) \cdot b$ $\boxed{W' = 3922 \cdot plf}$

(Wall without opening near base, cont'd)

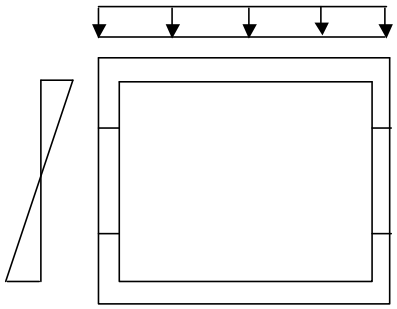
WALL without opening:

SHEAR

$f'_c = 3000\,psi$

$V'_u := W' \cdot \frac{l_2}{2}$

$V'_u = 15686\,lbf$



$V_u := V'_u \cdot \frac{\frac{l_2}{2} - d}{\frac{l_2}{2}}$

$b := 12in$

$V_u = 11 \cdot kip$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$\phi V_n := \phi_V \cdot \left[2 \cdot \left(\sqrt{f'_c \cdot psi}\right) \cdot (d) \cdot b\right]$ ACI EQ (11-4) $\phi V_n = 19 \cdot kip$

Factor of Safety $FS_{opening_shear} := \frac{\phi V_n}{V_u}$ $FS_{opening_shear} = 1.8$

Bending

NOTE: Assumed to be simply supported with no thrust because of opening in adjacent walls.

Demand Moment:

$M_u := \frac{W' \cdot (L_2 - D_1)^2}{8}$ $M_u = 44.24 \cdot kip \cdot ft$

Moment Capacity:

$a := \frac{A_s \cdot F_y}{0.85 f'_c \cdot b}$ $a = 1.569 \cdot in$

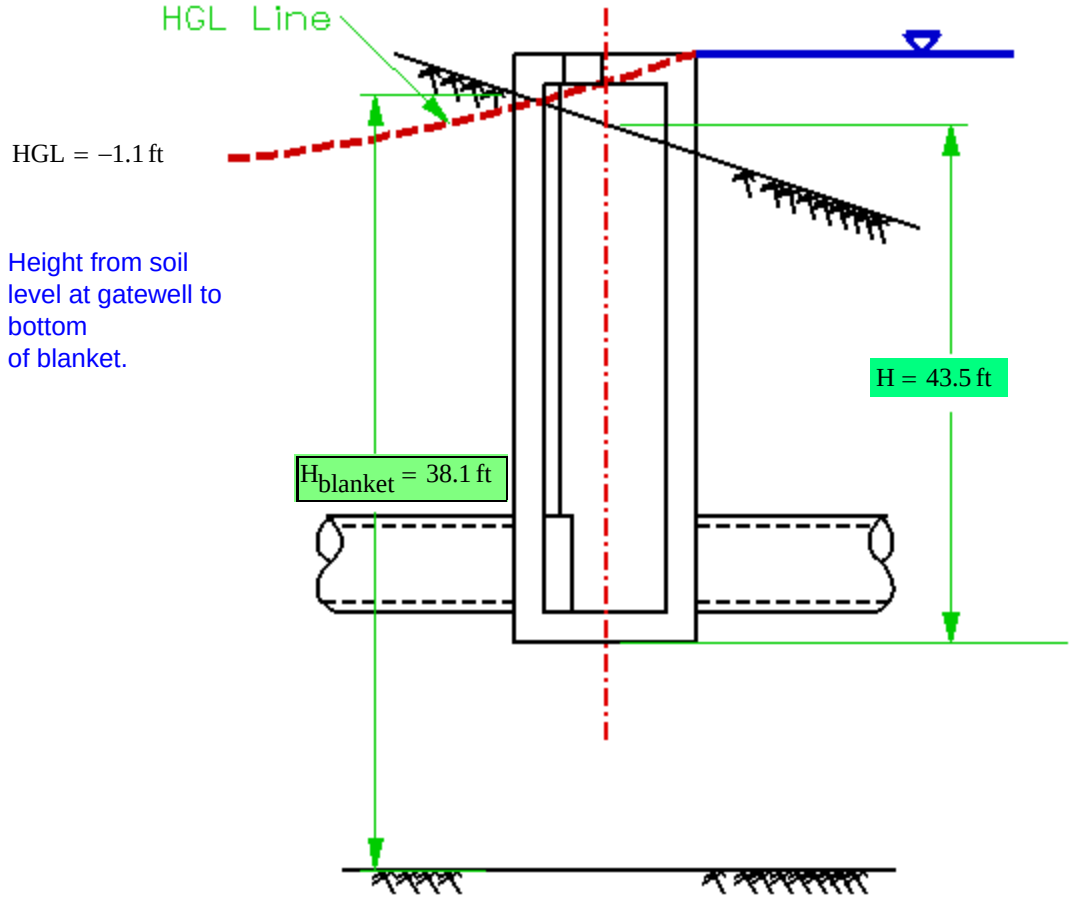
$\phi M_n := \phi_B \cdot A_s \cdot F_y \cdot \left(d - \frac{a}{2}\right)$ $\phi M_n = 55.1\,ft \cdot \frac{kip \cdot ft}{ft}$

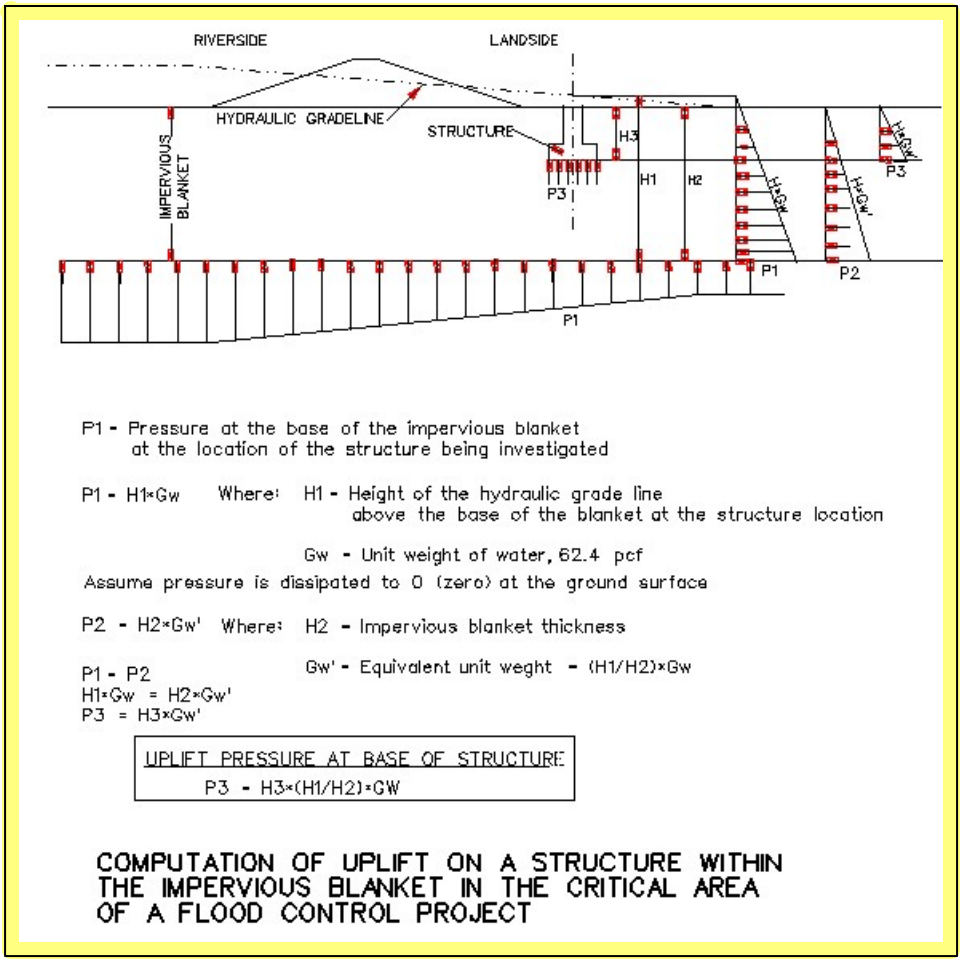
Factor of Safety $FS_{opening_mom} := \frac{\phi M_n}{M_u}$ $FS_{opening_mom} = 1.2$

Uplift Analysis:

Uplift Using Local Protection Guidance

UPLIFT





<< =from KC levees uplift guidance.

$A_{base} := L_1 \cdot L_2$ $\qquad A_{base} = 143\,ft^2$

Dissipated Head, if structural foundation in blanket.

$H_{blanket} = 38.1\,ft \quad HGL = -1.1\,ft \quad H = 43.5\,ft \quad L_1 = 13\,ft \quad L_2 = 11\,ft$

$$U_1 := \left(\frac{H_{blanket} + HGL}{H_{blanket}} \cdot H\right) \cdot L_1 \cdot L_2 \cdot \gamma_w \qquad U_1 = 377.0 \cdot kip$$

Full Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL) \cdot (L_1 \cdot L_2) \cdot \gamma_w\right] \qquad U_2 = 378.3 \cdot kip$$

Uplift

$H = 43.5\,ft$ $H_{blanket} = 38.1\,ft$ $H' = 0\,ft$

$U := if(H > H_{blanket}, U_2, U_1)$ $U = 378.3 \cdot kip$

$U_{uplift} := \frac{U}{L_1 \cdot L_2}$ $U_{uplift} = 2646 \cdot psf$

Weight of Structure

$Walls := (L_1 \cdot L_2 - l_1 \cdot l_2) \cdot (H + H') \cdot \gamma_c$ $Slabs := 2 \cdot (l_1 \cdot l_2) \cdot D_f \cdot \gamma_c$ $W_S := (Walls + Slabs)$ $W_S = 459.1 \cdot kip$

Surcharge Loads

$Soil := 0lb$

WEIGHT OF SOIL LOCATED ON TOP OF THE BASE SLAB HEEL OR WALL STEP. EQUAL TO ZERO IF NO HEEL IS PRESENT.

Weight of Surcharge Water

$W_{water} := HEAD \cdot l_1 \cdot l_2 \cdot \gamma_w$ $W_{water} = 0 \cdot lb$ $W_g := 0lb$

Skin Friction

SOIL (Soil parameters entered previously)

$\phi = \blacksquare \cdot deg$ INTERNAL FRICTION ANGLE

$\gamma_{soil} = 115 \cdot pcf$ DRY SOIL UNIT WEIGHT

$\gamma_w = 62.4 \cdot pcf$ WATER UNIT WEIGHT

$\gamma_{sub} := (\gamma_{soil} - \gamma_w)$ SUBMERGED SOIL UNIT WEIGHT $\gamma_{sub} = 52.6 \cdot pcf$

$K_o = 0.642$ AT-REST SOIL COEFFICIENT. CALCULATED PREVIOUSLY

$\mu := 0.25$ FRICTION COEFICIENT BETWEEN STRUCTURE AND SOIL.

$U_{uplift} = 2645.76 \cdot psf$ UPLIFT FORCE ACTING ON BASE OF STRUCTURE

$c := 5ft$ ASSUMED CRACK DEPTH

STRUCTURE

$H = 43.5\,ft$ DEPTH OF STRUCTURE BELOW GRADE

$per := 2 \cdot L_1 + 2 \cdot L_2$ STRUCTURE PERIMETER $per = 48\,ft$

$W_S = 459.075 \cdot kip$ STRUCTURE DEAD WEIGHT

Skin Friction (Continued)

1. Effective Lateral Load that Contributes to Side Friction

$R_1 := (H - c)\cdot\left(\frac{c + H}{2}\right)\cdot\gamma_{soil}$ $R_1 = 107366.875\cdot\frac{lbf}{ft}$

$R_2 := (H - c)\cdot U_{uplift}\cdot\left(\frac{c}{2\cdot H} + \frac{1}{2}\right)$ $R_2 = 56785.004\cdot\frac{lbf}{ft}$

$R_1 - R_2 = 50581.871\cdot\frac{lbf}{ft}$

2. Force Created By Side Friction, Pv

$K_o = 0.642$ $\mu = 0.25$ $per = 48\,ft$

$P_{v1} := (R_1 - R_2)\cdot K_o\cdot\mu\cdot per$ $P_{v1} = 389.459\cdot kip$

Ignore_Skin_Friction :=

$P_v := \begin{cases} 0kip & \text{if Ignore_Skin_Friction} = \text{"Yes"} \\ P_{v1} & \text{otherwise} \end{cases}$ $P_v = 389\cdot kip$

Final Uplift Factor of Safety:

$FS_{Uplift} := \frac{W_S + W_{water} + Soil + P_v}{U}$ $FS_{Uplift} = 2.243$

NOTE: EM 1110-2-2100 defines 1.1 Factor of Safety acceptable for extreme loading case. The gatewells have minimal overhangs to produce weight of water above structure, W_G, therefore W_G ignored (conservative).

Base Slab Analysis

NOTE:
1. The following calculation assumes two opposite edges fixed and the other opposite edges simply supported. This occurs when a large pipe takes up most of the wall so that wall should be simply supported. Coef came from "Theory of Plates and Shells" second edition, by Timoshenko. This is conservative for gatewells without large pipe openings. So if FS is critical than may want to reanalize using the table of coeff for all sides fixed from "EM 27 - Moments and Reactions for Rect. Plates".
2. Only the end-of-span moment is checked, but in cases where the base slab top and bottom reinf is not the same, it may be necessary to check the mid-span.

Slab Loading:

Weight of structure: $W_S = 459.075 \cdot kip$ Uplift: $U = 378.344 \cdot kip$ Skin Friction: $P_{v1} = 389.459 \cdot kip$

Load acting up on baseslab:

$$W := \text{if}\left(U > W_S, U + P_{v1}, W_S\right) = 459.075 \cdot kip \qquad W_{UNIFORM} := \frac{\gamma_L \cdot \gamma_H \cdot \gamma_X \cdot W}{L_1 \cdot L_2} = 3.21 \cdot ksf$$

END-OF-SPAN MOMENT IS CHECKED HERE:

Plate Coefficient and Demand Moment:

$k := 1 .. 5$

a/b=L_2/L_1 $\frac{l_2}{l_1} = 0.8$

From plates tables:

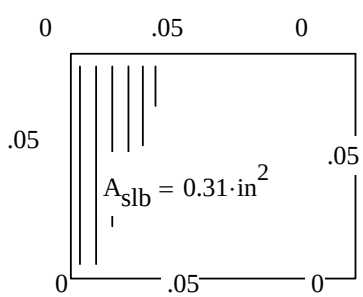
a/b	Coefficient
1	.0697
.9091	.0787
.769	.0938
.625	.109
.5	.1191

$ab_k :=$

0.5
.625
.769
0.9091
1

$Coeff_k :=$

.1191
.109
.0938
.0787
.0697



$$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right) = 0.09$$

$l_2 = 8\,ft \qquad b = 1\,ft$

$$M_{uFe} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b \qquad M_{uFe} = 18.586 \cdot kip \cdot ft$$

Moment Capacity:

$$a := \frac{A_{slb} \cdot F_y}{0.85 f'_c \cdot b} = 4.9 \cdot \frac{in^2}{ft} \qquad d_f = 1.708\,ft$$

$$\phi M_{nFe} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right) \qquad \phi M_{nFe} = 21\,ft \cdot \frac{kip \cdot ft}{ft}$$

Factor of Safety $FS_{slabmomente} := \frac{\phi M_{nFe}}{M_{uFe}}$ $FS_{slabmomente} = 1.129$

Base Slab Analysis, con't

MID-SPAN MOMENT (My) IS CHECKED HERE:

Plate Coefficient and Demand Moment:

a/b=L_2/L_1 $\frac{l_2}{l_1} = 0.8$

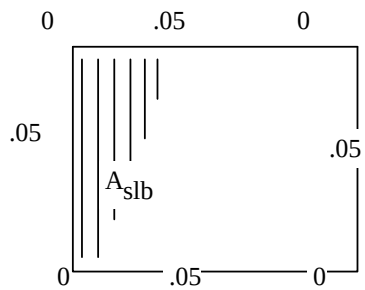
From plates tables:

a/b	Coefficient
1	0.0332
.9091	0.0371
.769	0.0426
.625	0.0469
.5	0.0474

$k := 1 .. 5$

$ab_k :=$ $Coeff_k :=$

0.5	0.0474
.625	0.0469
.769	0.0426
0.9091	0.0371
1	0.0332



$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right) = 0.041$

$l_2 = 8\,ft$ $b = 1\,ft$

$M_{uFm1} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b$ $M_{uFm1} = 8.503 \cdot kip \cdot ft$

$A_{slb} := 0.44in^2$ $C_{cf} := 3.0625in$

$d_f := D_f - C_{cf}$

Moment Capacity:

$a := \frac{A_{slb} \cdot F_y}{0.85f'_c \cdot b} = 6.9 \cdot \frac{in^2}{ft}$ $d_f = 1.745\,ft$

$\phi M_{nFm1} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$ $\phi M_{nFm1} = 30.3\,ft \cdot \frac{kip \cdot ft}{ft}$

Factor of Safety $FS_{slabmomentm1} := \frac{\phi M_{nFm1}}{M_{uFm1}}$ $FS_{slabmomentm1} = 3.562$

Base Slab Analysis, con't

MID-SPAN MOMENT (Mx) IS CHECKED HERE:

Plate Coefficient and Demand Moment:

a/b=L_2/L_1 $\frac{l_2}{l_1} = 0.8$

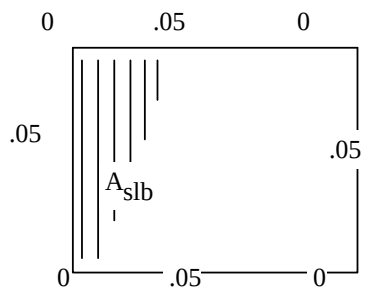
From plates tables:

a/b	Coefficient
1	0.0244
.9091	0.0307
.769	0.0446
.625	0.065
.5	0.0869

$k := 1 .. 5$

$ab_k :=$ $Coeff_k :=$

0.5	0.0869
.625	0.065
.769	0.0446
0.9091	0.0307
1	0.0244



$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right) = 0.042$

$l_2 = 8\,ft$ $b = 1\,ft$

$M_{uFm2} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b$ $M_{uFm2} = 8.532 \cdot kip \cdot ft$

$A_{slb} := 1.06in^2$ $C_{cf} := 2.375in$

$d_f := D_f - C_{cf}$

Moment Capacity:

$a := \frac{A_{slb} \cdot F_y}{0.85f'_c \cdot b} = 16.6 \cdot \frac{in^2}{ft}$ $d_f = 1.802\,ft$

$\phi M_{nFm2} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$ $\phi M_{nFm2} = 74\,ft \cdot \frac{kip \cdot ft}{ft}$

Factor of Safety $FS_{slabmomentm2} := \frac{\phi M_{nFm2}}{M_{uFm2}}$ $FS_{slabmomentm2} = 8.669$

Base Slab Analysis, con't

Note that there is a concentration of reinforcing in one direction in the base slab. Assume that the base slab acts as a plate until reinforcing in the weak direction begins to yield and then the moments will redistribute to reinforcing in the other direction (a plastic hinge will form in the base slab at the wall). When this happens the base slab should be analyzed as if it were bending as if it were a simple span:

$L := 10ft$ $\quad b = 1\,ft$

$$M_{ss} := \frac{W_{UNIFORM} \cdot L^2 \cdot b}{8} = 40.129\,ft \cdot \frac{kip \cdot ft}{ft}$$

$A_{slb} := 1.06in^2$ $\quad C_{cf} := 2.375in$

$d_f := D_f - C_{cf}$

Moment Capacity:

$$a := \frac{A_{slb} \cdot F_y}{0.85f'_c \cdot b} = 16.6 \cdot \frac{in^2}{ft} \qquad d_f = 1.802\,ft$$

$$\phi M_{nFm3} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right) \qquad \phi M_{nFm3} = 74\,ft \cdot \frac{kip \cdot ft}{ft}$$

Factor of Safety $\quad FS_{slabmomentm3} := \dfrac{\phi M_{nFm3}}{M_{ss}}$ $\quad FS_{slabmomentm3} = 1.843$

**********Overall Factors of Safety**********

Uplift Stability

$FS_{Uplift} = 2.2$

Strength

Wall AD $L_1 = 13\,ft$

Wall AB $L_2 = 11\,ft$

WALL MOMENT

The overall wall moment factor of safety, $FS_{wallmoment}$

$FS_{ADmoment} = 2.624$ $FS_{ABmoment} = 3.319$ $FS_{opening_mom} = 1.246$

$FS_{wallmoment} := \min(FS_{ADmoment}, FS_{ABmoment}, FS_{opening_mom})$ $FS_{wallmoment} = 1.25$

WALL SHEAR

$FS_{ADshear} = 1.528$ $FS_{ABshear} = 2.103$ $FS_{opening_shear} = 1.752$

$FS_{wallshear} := \min(FS_{ADshear}, FS_{ABshear}, FS_{opening_shear})$ $FS_{wallshear} = 1.528$

BASE SLAB MOMENT FACTOR OF SAFETY

$FS_{slabmoment} := \max(\min(FS_{slabmomente}, FS_{slabmomentm1}, FS_{slabmomentm2}), FS_{slabmomentm3})$

$FS_{slabmoment} = 1.843$

Summary :

$FS_{strength} := \min(FS_{wallmoment}, FS_{wallshear}, FS_{slabmoment})$ $FS_{strength} = 1.2$

$$Mechanism := \begin{cases} \text{"Wall Bending"} & \text{if } FS_{strength} = FS_{wallmoment} \\ \text{"Wall Shear"} & \text{if } FS_{strength} = FS_{wallshear} \\ \text{"Base Slab Moment"} & \text{otherwise} \end{cases}$$

Mechanism = "Wall Bending"

$FS_{Uplift} = 2.2$

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis. The reasoning being the load factor (live load neglecting hydraulic structure) divided by the strength reduction factor is approximately 1.75. Because this is an existing structure in the field that is routinely inspected with no visible history of problems, a factor of safety of 1.5 is deemed acceptable. If modifications are required for strength, the modified structure must be evaluated from a standard design approach (include factors - no FS calculation)

END OF ANALYSIS

General Structural Exhibit 4:
Stoplog Gap Sample Calculations

This exhibit includes calculations specific to stoplog gaps. The check of the general floodwall section that is part of the gap is similar to that found in the exhibits for floodwalls calculations. See those exhibits for sample calculations.

FOR A FEASIBILITY STUDY BASED UPON RISK ANALYSIS, A DETAILED & PRECISE ANALYSIS IS BEYOND BUDGET AND SCOPE. AN ESTIMATE FOR TORSION WILL BE BASED UPON THE MOMENT TRANSFERRED INTO THE PILE CAP FROM THE ABUTMENT. SOIL RESISTANCE TO PILE CAP IS IGNORED.

THIS ESTIMATE FOR TORSION WAS TESTED ON L385's R1A CLOSURE. MAXIMUM TORSION, OCCURING NEAR ABUTMENT FACE, RANGED FROM 13000 TO 15000 K-ft, DEPENDING UPON PILE CONFIGURATION. THE MORE PILES, THE STIFFER THE PILE CAP AND THUS HIGHER TORSION. R1A WAS FINALLY DESIGNED WITH THE FEWER PILE CONFIGURATION; HOWEVER, AN EARLIER TRIAL RUN WITH 3 ROWS OF PILES AT 5'-6" SPACING YIELDED THE 15000 K-ft MAX TORSION. CID MO'S PILE CAPS ARE NOT VERY UNIFORM. PILES VARY FROM 6' TO 7' IN THE CLOSURE TO APPROXIMATELY 4' AT THE ABUTMENTS.

R1A'S MOMENT AT ABUTMENT:

68' CLOSURE WIDTH; ABUTMENT WIDTH IS 2'

$w = 68'/2 = 34'$ $\qquad h = 16' - 2\frac{3}{4}''$

$M_{R1A} = \frac{1}{2} h^2 \gamma w (h/3)$ (IGNORING ABUTMENT WIDTH)

$M_{R1A} = 1512$ K-ft. > LPGF OUTPUTS (13000 - 15000 K-ft)

THIS IS A CRUDE ESTIMATE FOR TORSION, BUT A CONSERVATIVE ONE. THIS METHOD WILL BE USED TO DETERMINE THE TORSION OF CID MO'S PILE CAPS AND THEN COMPARE TO THE CRACKING TORSION. A TORSION WITH FS ≥ 1.5 WHEN COMPARED TO $\frac{1}{4} T_{CR}$ (ACI) WILL BE CONSIDERED RELIABLE.

NOTE: NONE OF THE PILE CAPS MEET A FS OF 1.5, THUS A PROBABILITY OF FAILURE WAS ESTABLISHED. REALIZE THAT MARGINALLY EXCEEDING THE $\frac{1}{4} T_R$ LIMIT BY ACI DOES NOT CONSTITUTE FAILURE. IT MAY MEAN SOME CRACKING WILL OCCUR OR THAT MOMENT AND SHEAR FS WILL BE REDUCED. IF THE FS FOR BOTH MOMENT AND SHEAR EXCEED, SAY 3.0, MEANING THERE'S ADEQUATE REINFORCING FOR SHEAR, FLEXURE, AND TORSION, THEN THE PILE CAP WOULD NOT BE INADEQUATELY DESIGNED. IT IS RECOMMENDED THAT A THOROUGH ANALYSIS BE COMPLETED BEFORE RECOMMENDING MODIFICATIONS TO THE WALL.

MONOLITH 145: ABUTMENT WIDTH = 2'-6"

OPENING = 17'-6" $H = 11'-0"$ $h_{sill} = 2'$

$w = 17.5'/2 = 8.75'$

$T_{145} = M_{145} = 1/2\, \gamma_{H_2O} \left[(H^2 w)(H/3 + h_{sill}) \right] = 187$ K-ft.

(241 K-ft w/ ABUTMENT WIDTH)

LOAD FACTORS = 1 FOR RELIABILITY.

$T_{145} = 187$ K-ft (241 w/ ABUTMENT WIDTH)

TORSIONAL THRESHOLD (τ_ℓ):

SEE ACI 11.6.1, WITH $\phi = 1$ FOR RELIABILITY.

$w = 13'$ $h = 2'$ (PILE CAP DIMENSIONS)

$A_{cp} = w * h = 3744\ in^2$

$p_{cp} = 2(w+h) = 360\ in$

$$\tau_\ell = \phi \sqrt{f'_c} \left(A_{cp}^2 / p_{cp} \right) \qquad f'_c = 3{,}000\ psi$$

$\tau_\ell = 178$ K-ft $< T_{145}$ N.G.

RELIABILITY ANALYSIS REQUIRED

AGAIN, THE $1/4 T_{cr}$ CRITERION WAS JUDGED BY ACI TO REPRESENT THE THRESHOLD FOR TORSION THAT DOES NOT CAUSE SIGNIFICANT REDUCTION TO MOMENT AND SHEAR. A LARGE FS IN MOMENT AND SHEAR WOULD PERMIT SOME OF THE EXCESS TORSIONAL STRESS TO BE ABSORBED BY THE REINFORCED SECTION WITHOUT CRACKING.

MONOLITH 158:

OPENING = 17'-6" $H = 9.79'$ $h_{sill} = 3.21'$

$w = 17.5'/2 = 8.75'$

$$T_{158} = M_{158} = \tfrac{1}{2}\gamma_{H_2O}(H^2 w)(H/3 + h_{sill}) = 169 \text{ k-ft}$$

LOAD FACTOR = 1 FOR RELIABILITY

$T_{158} = 169$ k-ft

$\tau_e = 178$ k-ft (SAME AS M145)

$$FS = \frac{\tau_e}{T_{158}} = 1.05 < 1.5 \quad \text{N.G.}$$

MONOLITH 162: THE CENTER POST IS ASSUMED TO CONTROL:

TRIBUTARY WIDTH = $2(15'/2) - 4.5' = 10.5'$ POSTWELL LENGTH = 4.5'

$H = 9'-1\tfrac{1}{2}''$ $h_{sill} = 3'-10\tfrac{1}{2}''$

$$T_{162} = \tfrac{1}{2}\gamma_{H_2O}(H^2 w)(H/3 + h_{sill}) = 189 \text{ k-ft}$$

LOAD FACTOR = 1 FOR RELIABILITY

$T_{162} = 189$ k-ft

TORSIONAL THRESHOLD (τ_e):

$w = 13'$ $h = 2'$ (PILE CAP DIMENSIONS)

$A_{cp} = w * h = 3744 \text{ in}^2$

$p_{cp} = 2(w+h) = 360 \text{ in}$

$$\tau_e = \phi\sqrt{f'_c}\left(A_{cp}^2/p_{cp}\right) \qquad f'_c = 3000 \text{ psi}$$

$\tau_e = 178$ k-ft $< T_{162}$ N.G.

RELIABILITY ANALYSIS REQUIRED.

LEVEE MONOLITH: OPENING = 24'

$W = 24'/2 = 12'$ $H = 6'-10\frac{1}{2}''$ $h_{sill} = 3'-1\frac{1}{2}''$

$T_{Lev} = \frac{1}{2}\gamma_{H_2O}(H^2 w)(H/3 + h_{sill}) = 95.9$ K-ft

LOAD FACTOR = 1 FOR RELIABILITY

$T_{Lev} = 95.9$ K-ft.

TORSIONAL THRESHOLD (τ_R):

$w = 10'$ $h = 2'$ (PILE CAP DIMENSIONS)

$A_{cp} = w * h = 2880$ in^2

$P_{cp} = 2(w+h) = 288$ in

$\tau_R = \phi\sqrt{f'_c}\,(A_{cp}^2/P_{cp})$ $f'_c = 3$ ksi

$FS = \tau_R / T_{Lev} = 1.37 < 1.5$ N.G.

22/437

US Army Corps of Engineers

Probability of Failure
CID MO Monolith 145
Torsion in Pile Cap

Comp by: FRS
Chkd by: PDM
NOV 11

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. Situation

1. CID MO ~~M32~~ M145 sill does not meet the strength 1.5 factor of safety for which it has been determined 99.8% reliability can be assigned. See mathcad analysis of the wall for existing condition strength check.
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength.
3. Material Properties used:

   Mean Concrete Strength, $f'_{cM} := 3750 \cdot psi$

   Mean Steel Strength, $F_{yM} := 50 \cdot ksi$

4. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete Grade 40 is 14%.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength
FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength
FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength
FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength
ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength
ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. *Caclulating Factors of Safety*

Condition under consideration from strength check: Tu for Pile Cap (from hand-computation analysis - Water to TOW). $T_u := 187kip \cdot ft$

Design Concrete Strength

$\phi_B := 1.0$ Strength Reduction Factors not used in Risk and Uncertainty Analysis

$w_{cap} := 13ft$ Width of Pile Cap

$T_{toe} := 2.0ft$ Thickness of concrete section

$b := 12\frac{in}{ft}$ 1 ft strip of wall analyzed

$A_{cp} := T_{toe} \cdot w_{cap} = 3.744 \times 10^3 \cdot in^2$

$P_{cp} := 2 \cdot (T_{toe} + w_{cap}) = 360 \cdot in$

Mean Concrete Strength

$\phi\tau_{lM} := \phi_B \cdot \sqrt{f'_{cM} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}}\right)$ $\phi\tau_{lM} = 198.703 \cdot kip \cdot ft$

$FS_D := \frac{\phi\tau_{lM}}{T_u}$ $FS_D = 1.063$

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM} \cdot 0.14$ $f'_{cU} = 4275 \cdot psi$

$\phi\tau_{lU} := \phi_B \cdot \sqrt{f'_{cU} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}}\right)$ $\phi\tau_{lU} = 212.16 \cdot kip \cdot ft$

$FS_{fcu} := \frac{\phi\tau_{lU}}{T_u}$ $FS_{fcu} = 1.135$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\quad f'_{cL} = 3225 \cdot psi$

$\phi\tau_{lL} := \phi_B \cdot \sqrt{f'_{cL} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}}\right)$ $\quad \phi\tau_{lL} = 184.27 \cdot kip \cdot ft$

$FS_{fcl} := \frac{\phi\tau_{lL}}{T_u}$ $\quad FS_{fcl} = 0.985$

VI. Probability of Failure Calculation

$\Delta F_{fc} := FS_{fcu} - FS_{fcl}$ $\quad \Delta F_{fc} = 0.149$

$\sigma_F := \sqrt{\left(\frac{\Delta F_{fc}}{2}\right)^2}$ $\quad \sigma_F = 0.075$

$V_F := \frac{\sigma_F}{FS_D}$ $\quad V_F = 0.07$

$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\quad \beta_{LN} = 0.831$ Taylor Series method. See ETL 1110-2-547, pg B-8.

$R := cnorm(\beta_{LN})$ $\quad R = 79.7 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\quad P_F = 20.3 \cdot \%$

Condition under consideration from strength check: Tu for Pile Cap (from hand-computation analysis - Water 1ft below TOW). $T_{u1} := 145.6 kip \cdot ft$

Design Concrete Strength

$\phi_B := 1.0$ Strength Reduction Factors not used in Risk and Uncertainty Analysis

$w_{cap} := 13ft$ Width of Pile Cap

$T_{toe} := 2.0ft$ Thickness of concrete section

$b := 12 \frac{in}{ft}$ 1 ft strip of wall analyzed

$A_{cp} := T_{toe} \cdot w_{cap} = 3.744 \times 10^3 \cdot in^2$

$P_{cp} := 2 \cdot (T_{toe} + w_{cap}) = 360 \cdot in$

Mean Concrete Strength

$\phi\tau_{lM} := \phi_B \cdot \sqrt{f'_{cM} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}} \right)$ $\phi\tau_{lM} = 198.703 \cdot kip \cdot ft$

$FS_D := \frac{\phi\tau_{lM}}{T_{u1}}$ $FS_D = 1.365$

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM} \cdot 0.14$ $f'_{cU} = 4275 \cdot psi$

$\phi\tau_{lU} := \phi_B \cdot \sqrt{f'_{cU} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}} \right)$ $\phi\tau_{lU} = 212.16 \cdot kip \cdot ft$

$FS_{fcu} := \frac{\phi\tau_{lU}}{T_{u1}}$ $FS_{fcu} = 1.457$

26/437

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\qquad f'_{cL} = 3225 \cdot psi$

$\phi\tau_{IL} := \phi_B \cdot \sqrt{f'_{cL} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}}\right)$ $\qquad \phi\tau_{IL} = 184.27 \cdot kip \cdot ft$

$FS_{fcl} := \frac{\phi\tau_{IL}}{T_{u1}}$ $\qquad FS_{fcl} = 1.266$

VI. Probability of Failure Calculation

$\Delta F_{fc} := FS_{fcu} - FS_{fcl}$ $\qquad \Delta F_{fc} = 0.192$

$\sigma_F := \sqrt{\left(\frac{\Delta F_{fc}}{2}\right)^2}$ $\qquad \sigma_F = 0.096$

$V_F := \frac{\sigma_F}{FS_D}$ $\qquad V_F = 0.07$

$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\qquad \beta_{LN} = 4.402$ Taylor Series method. See ETL 1110-2-547, pg B-8.

$R := cnorm(\beta_{LN})$ $\qquad R = 100 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\qquad P_F = 5.37 \times 10^{-4} \cdot \%$

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Comp. By [illegible] Date Aug 08 Project CID-MO Sheet 42 of 437

Chkd. By KSM Date 9/28/11 Subject POF FOR PILE-FOUNDED CLOSURES Sheet ____ of ____

TO ASSIST WITH UNDERSTANDING THE POF & CPGA ANALYSES FOR TENSION PULLOUT OF SHEETPILE.

1. IN CPGA INPUT, THE ALLOWABLES WERE USED SO THAT THE ALF AND CBF OUTPUT OF 1.0 WAS THE THRESHOLD. A FS=1.5 WAS USED FOR STRUCTURAL STRENGTH AND A FS = 1.7 WAS USED FOR GEOTECHNICAL STRENGTHS. THESE FS ARE USED AS THRESHOLDS FOR THE FEASIBILITY ANALYSIS, THEY DO NOT CONSTITUTE DESIGN PARAMETERS.
2. THE POF FOR PULLOUT USED ULTIMATE GEOTECHNICAL STRENGTHS FOR THE MEAN.

2011 REVISION

Chkd: KSM 9/28/11

43/

M145R.dat

```
10 CID-MO M145 GEOTECH REV STRENGTH 4ft abutment-
20 SEP 2011 Existing conditions h20 TOW-
30 GT FS is 1.7 and Structural FS is 1.5
40 LOAD 1 0 -60.56 31.44 -170.63 0 0
51 PILE 1 0 2 0
52 PILE 2 0 6 0
53 PILE 3 -1.5 11.5 0
54 PILE 4 -.5 11.5 0
55 PILE 5 .5 11.5 0
56 PILE 6 1.5 11.5 0
110 SOIL  NH 0.05 LEN 33  0  1 to 2
111 PROP 3122 8748 8748 324 2 0 1 to 2
112 DLS S 81.1 43.1 536 128 274 1802 905 H 18 1 TO 2
120 SOIL  NH 0.05 LEN 15  0  3 TO 6
121 PROP 3122 1000 1440 120 2 0 3 to 6
121 DLS S 15.2 8.26 205 56.3 72 333 195 H 10 3 TO 6
140 FIX 1 TO 6
150 PFO 1 TO 6
500 FOUT 1 2 3 4 5 6 7 M145R.txt
```

```
*********************************
* CASE PROGRAM    #  X0080      *  CPGA - CASE PILE GROUP ANALYSIS PROGRAM
* VERSION NUMBER # 1993/03/29   *  RUN DATE 06-SEP-2011   RUN TIME 13.57.54
*********************************


CID-MO M145 GEOTECH REV STRENGTH 4FT ABUTMENT
SEP 2011 EXISTING CONDITIONS H20 TOW
GT FS IS 1.7 AND STRUCTURAL FS IS 1.5


THERE ARE     6 PILES AND
              1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                      X          Y          Z
                                    -----      -----      -----
WITH DIAGONAL COORDINATES = (      -1.50 ,     2.00 ,      .00 )
                            (       1.50 ,    11.50 ,      .00 )

*******************************************************************************

         PILE PROPERTIES AS INPUT

      E           I1           I2           A           C33          B66
    KSI         IN**4        IN**4        IN**2
  .31220E+04   .87480E+04   .87480E+04   .32400E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   1    2


      E           I1           I2           A           C33          B66
    KSI         IN**4        IN**4        IN**2
  .31220E+04   .10000E+04   .14400E+04   .12000E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   3    4    5    6

*******************************************************************************

         SOIL DESCRIPTIONS AS INPUT

   NH      ESOIL       LENGTH        L            LU
         K/IN**3                    FT           FT
         .50000E-01     L        .33000E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   1    2


   NH      ESOIL       LENGTH        L            LU
         K/IN**3                    FT           FT
         .50000E-01     L        .15000E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -
```

45/

3 4 5 6

****

PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	.00	2.00	.00	V	.00	33.00	F
2	.00	6.00	.00	V	.00	33.00	F
3	-1.50	11.50	.00	V	.00	15.00	F
4	-.50	11.50	.00	V	.00	15.00	F
5	.50	11.50	.00	V	.00	15.00	F
6	1.50	11.50	.00	V	.00	15.00	F

						126.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-60.6	31.4	-170.6	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.66258E+03	.00000E+00	.00000E+00	.00000E+00	.29315E+05	-.61195E+05
.00000E+00	.61826E+03	.00000E+00	-.26988E+05	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.26868E+05	.27882E+07	.00000E+00	.00000E+00
.00000E+00	-.26988E+05	.27882E+07	.34850E+09	.00000E+00	.00000E+00
.29315E+05	.00000E+00	.00000E+00	.00000E+00	.51547E+07	-.24726E+07
-.61195E+05	.00000E+00	.00000E+00	.00000E+00	-.24726E+07	.72376E+07

LOAD CASE 1. NUMBER OF FAILURES = 6. NUMBER OF PILES IN TENSION = 4.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	.0000E+00	-.1039E+00	.1541E-01	-.1372E-03	.0000E+00	.0000E+00

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE

46/437

# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-16.3	61.9	807.7	.0	.0	.76	1.40	.00	.00*
2	.0	-16.3	28.2	807.7	.0	.0	.35	1.57	.00	.00*
3	.0	-7.0	-14.7	229.5	.0	.0	1.78	3.88	.00	.00*
4	.0	-7.0	-14.7	229.5	.0	.0	1.78	3.88	.00	.00*
5	.0	-7.0	-14.7	229.5	.0	.0	1.78	3.88	.00	.00*
6	.0	-7.0	-14.7	229.5	.0	.0	1.78	3.88	.00	.00*

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-16.3	61.9	807.7	.0	.0
2	.0	-16.3	28.2	807.7	.0	.0
3	.0	-7.0	-14.7	229.5	.0	.0
4	.0	-7.0	-14.7	229.5	.0	.0
5	.0	-7.0	-14.7	229.5	.0	.0
6	.0	-7.0	-14.7	229.5	.0	.0

2011 REVISION

47/437

US Army Corps of Engineers

CID-MO Floodwall
Floodwall Analysis - Existing, M145
4 foot Stop Log Abutment Monolith
Kansas City Levees

Comp by: Sheffield
Chkd by: Muller 11/9/11

WATER EL @ T.O.W.

Assumptions: For Feasibility Analysis, it is assumed that the sheetpile cutoff is adequately strength designed for a 50% head differential, the pile cap under the closure is adequately designed, and that the floodwall sections of the monolith are accounted for by analysis of adjacent floodwall monoliths. Soil Surfaces assumed horizontal.

Variables

$kip := 1000lb$

$psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$ $plf := \frac{lb}{ft}$

$ksf \equiv \frac{1000lb}{ft^2}$ $psf \equiv \frac{lb}{ft^2}$

$EL_{top_ftg} := 746.9ft$

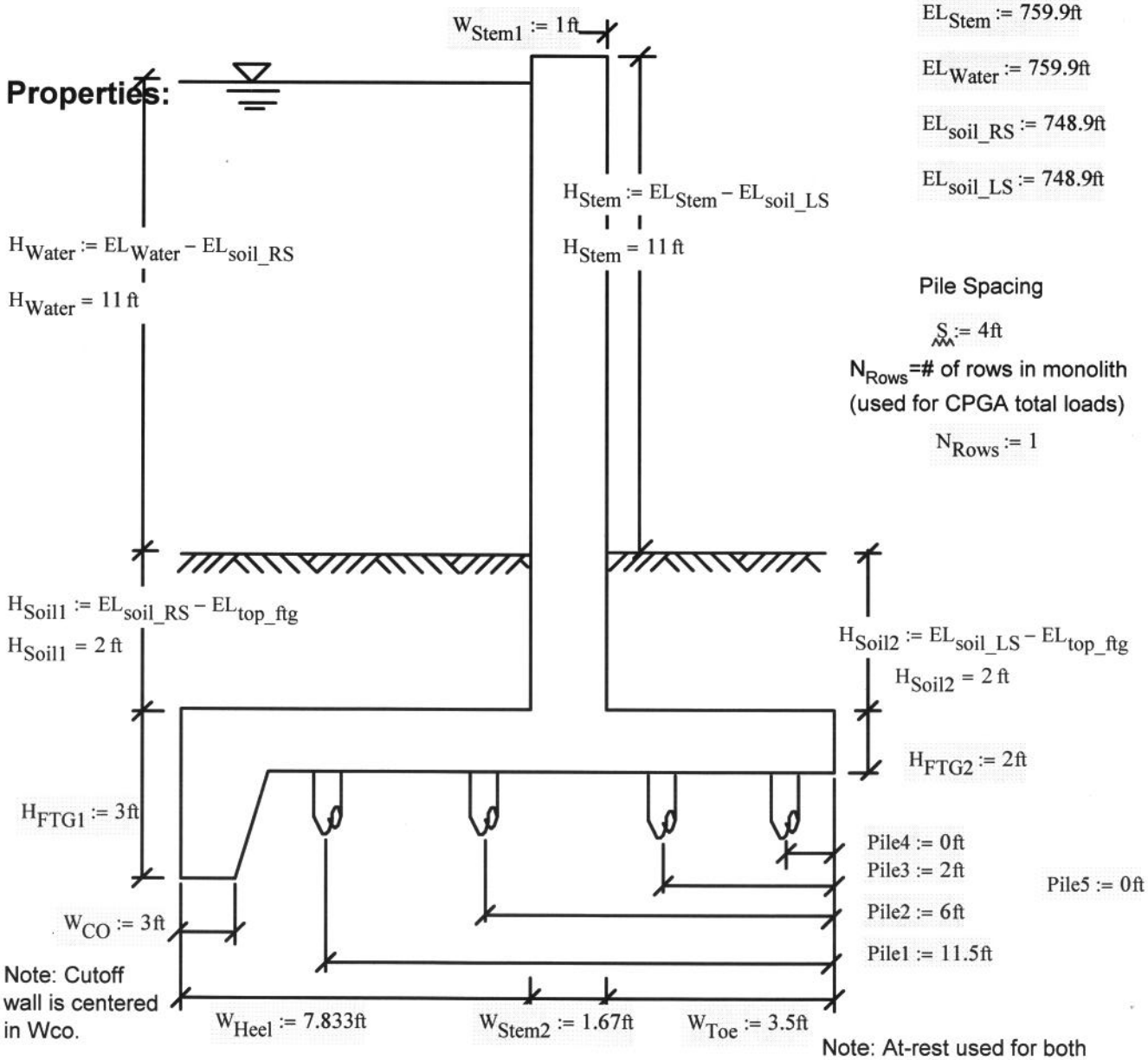
$EL_{Stem} := 759.9ft$

$EL_{Water} := 759.9ft$

$EL_{soil_RS} := 748.9ft$

$EL_{soil_LS} := 748.9ft$

Properties:



Pile Spacing

$S := 4ft$

N_{Rows}=# of rows in monolith (used for CPGA total loads)

$N_{Rows} := 1$

Note: Cutoff wall is centered in Wco.

Concrete Properties $\gamma_c := 150pcf$

Water Weight $\gamma_w := 62.4pcf$

Soil Properties $\gamma_{sat} := 115pcf$ $\phi := 22deg$

$K_o := 1 - \sin(\phi)$ $K_o = 0.63$

Note: At-rest used for both driving and resisting for Feasibility and to require piles to fully resist stability.

48/437

Lengths:

$L_{CO} := H_{FTG1} - H_{FTG2}$ $\quad L_{W1} := H_{Water} + H_{Soil1} + H_{FTG1}$ $\quad L_{STM} := H_{Stem} + H_{Soil2}$

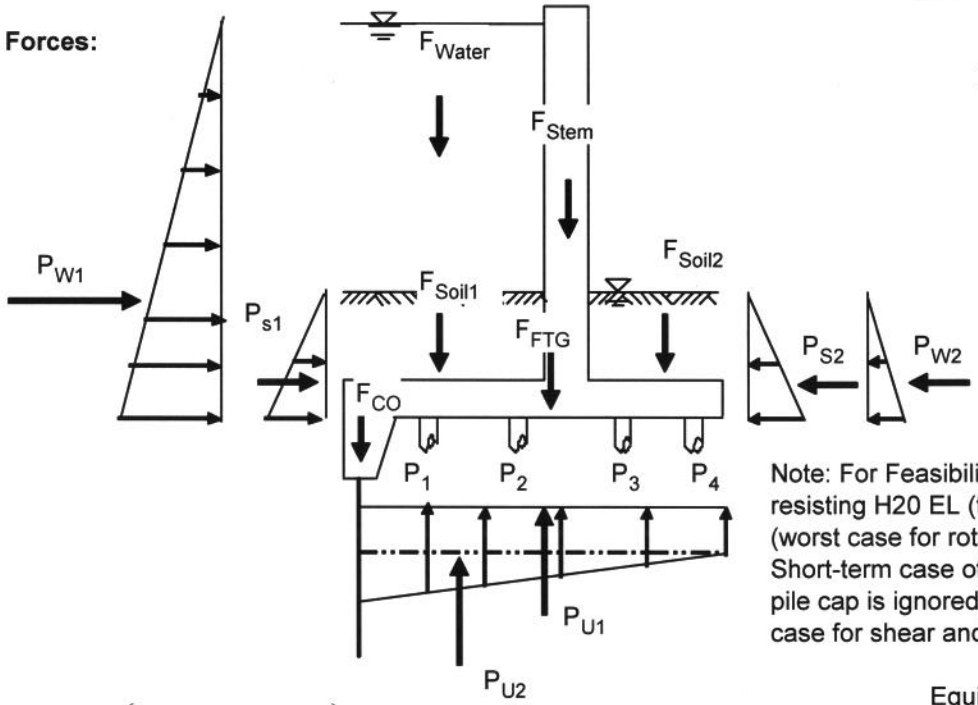
$L_{FTG} := W_{Heel} + W_{Stem2} + W_{Toe}$ $\quad L_{PU1} := H_{Soil2} + H_{FTG2}$

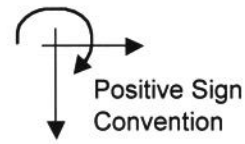
$L_{FTG} = 13\,ft$

$L_{W1} = 16\,ft$

$L_{PU1} = 4\,ft$

Forces:





Note: For Feasibility, only long-term case of resisting H20 EL (top of soil) is considered (worst case for rotation and pile pull-out). Short-term case of little or no resisting H20 on pile cap is ignored, though this is the worst case for shear and sliding.

$F_{Stem} := \left(\frac{W_{Stem1} + W_{Stem2}}{2}\right) \cdot L_{STM} \cdot \gamma_c = 2603.25 \cdot plf$

$F_{closure} := 8.26 \frac{kip}{ft}$ Equivalent Point Load acting on abutment from Stop Logs per tributary width of pile cap.

$F_{closure}$ = 1/2*.0624*11²*17.5/2*(1/4ft)

$F_{FTG} := H_{FTG2} \cdot L_{FTG} \cdot \gamma_c = 3900.9 \cdot plf$

$F_{CO} := L_{CO} \cdot W_{CO} \cdot \gamma_c = 450 \cdot plf$ (Treated Rectangular)

$F_{Soil1} := H_{Soil1} \cdot W_{Heel} \cdot (\gamma_{sat} - \gamma_w) = 824.03 \cdot plf$

$F_{Soil2} := H_{Soil2} \cdot W_{Toe} \cdot (\gamma_{sat} - \gamma_w) = 368.2 \cdot plf$

$F_{Water1} := (H_{Water} + H_{Soil1}) \cdot W_{Heel} \cdot \gamma_w = 6354.13 \cdot plf$

$F_{Water2} := H_{Soil2} \cdot W_{Toe} \cdot \gamma_w = 436.8 \cdot plf$

$P_{W1} := \frac{1}{2} \cdot L_{W1}^2 \cdot \gamma_w = 7987.2 \cdot plf$

$P_{W2a} := -\left[\frac{1}{2} \cdot (L_{PU1})^2 \cdot \gamma_w\right] = -499.2 \cdot plf$

Cutoff wall assumed to be 50% effective. See Figure C-3 of EM 1110-2-2100

$q_{uCO} := (1 - 0.5) \cdot \gamma_w \left[(H_{Water} + H_{Soil1} + H_{FTG2}) - L_{PU1}\right] + L_{PU1} \cdot \gamma_w = 592.8 \cdot psf$

$q_{umax} := \left[(1 - 0.5)(L_{W1} - L_{PU1})\gamma_w\right] + L_{PU1} \cdot \gamma_w = 624 \cdot psf$

$q_{utoe} := L_{PU1} \cdot \gamma_w + \frac{L_{PU1}}{L_{FTG} - .5W_{CO} + L_{PU1}} (q_{umax} - L_{PU1} \cdot \gamma_w) = 346.2 \cdot psf$

Note: Soil below base of pile cap acting on key is essentially equal and opposite since all resistance is by piles.

$P_{W2b} := -\left[\frac{1}{2} \cdot (q_{uCO} + q_{umax}) \cdot L_{CO}\right] = -608.4 \cdot plf$

$P_{S1} := K_o \cdot \left[\frac{1}{2} \cdot (H_{Soil1} + H_{FTG2})^2 \cdot (\gamma_{sat} - \gamma_w)\right] = 263.17 \cdot plf$ $\quad P_{S2} := -K_o \cdot \left[\frac{1}{2} \cdot (H_{Soil2} + H_{FTG2})^2 \cdot (\gamma_{sat} - \gamma_w)\right] = -263.17 \cdot plf$

$P_{U1} := -(L_{FTG} - .5W_{CO}) \cdot (q_{utoe}) = -3982.35 \cdot plf$ $\quad P_{U2} := -0.5(L_{FTG} - .5 \cdot W_{CO}) \cdot (q_{umax} - q_{utoe}) = -1597.76 \cdot plf$

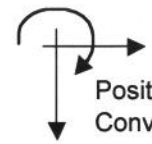
$P_{U3} := -.5W_{CO} \cdot (L_{W1} \cdot \gamma_w) = -1497.6 \cdot plf$

49

Inputs for CPGA

Assumes perfectly rigid pile cap.

Sum moments around bottom corner of Toe:



Positive Sign Convention

Moment Arm (from bottom corner of Toe)

$$Arm_{Stem} := \left(W_{Toe} + \frac{W_{Stem2}}{2}\right) = 4.34\,ft$$

$$Arm_{FTG} := \frac{L_{FTG}}{2} = 6.5\,ft$$

$$Arm_{CO} := L_{FTG} - \frac{W_{CO}}{2} = 11.5\,ft$$

$$Arm_{closure} := 7.67 \cdot ft$$

$$Arm_{Soil1} := \left(W_{Toe} + W_{Stem2} + 0.5W_{Heel}\right) = 9.09\,ft$$

$$Arm_{Soil2} := 0.5W_{Toe} = 1.75\,ft$$

$$Arm_{Water1} := Arm_{Soil1} = 9.09\,ft$$

$$Arm_{Water2} := Arm_{Soil2} = 1.75\,ft$$

$$Arm_{W1} := \frac{1}{3} \cdot L_{W1} - L_{CO} = 4.33\,ft$$

$$Arm_{W2a} := 0.333 \cdot \left(L_{PU1}\right) = 1.33\,ft$$

$$Arm_{W2b} := L_{CO} - \frac{L_{CO} \cdot \left(2q_{uCO} + q_{umax}\right)}{3\left(q_{uCO} + q_{umax}\right)} = 0.5\,ft$$

$$Arm_{S1} := \frac{1}{3} \cdot \left(H_{Soil1} + H_{FTG2}\right) = 1.33\,ft$$

$$Arm_{S2} := Arm_{W2a} = 1.33\,ft$$

$$Arm_{U1} := \frac{L_{FTG} - 0.5W_{CO}}{2} = 5.75\,ft$$

$$Arm_{U2} := \frac{2\left(L_{FTG} - 0.5W_{CO}\right)}{3} = 7.67\,ft$$

$$Arm_{U3} := L_{FTG} - 0.25 \cdot W_{CO} = 12.25\,ft$$

Moment (about bottom corner of toe)

$$M_{Stem} := -F_{Stem} \cdot Arm_{Stem} = -11.29 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{FTG} := -F_{FTG} \cdot Arm_{FTG} = -25.36 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{CO} := -F_{CO} \cdot Arm_{CO} = -5.18 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{closure} := F_{closure} \cdot Arm_{closure} = 63.35 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{Soil1} := -F_{Soil1} \cdot Arm_{Soil1} = -7.49 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{Soil2} := -F_{Soil2} \cdot Arm_{Soil2} = -0.64 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{Water1} := -F_{Water1} \cdot Arm_{Water1} = -57.74 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{Water2} := -F_{Water2} \cdot Arm_{Water2} = -0.76 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{W1} := P_{W1} \cdot Arm_{W1} = 34.61 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{W2a} := P_{W2a} \cdot Arm_{W2a} = -0.66 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{W2b} := -P_{W2b} \cdot Arm_{W2b} = 0.31 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{S1} := P_{S1} \cdot Arm_{S1} = 0.35 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{S2} := P_{S2} \cdot Arm_{S2} = -0.35 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{U1} := -P_{U1} \cdot Arm_{U1} = 22.9 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{U2} := -P_{U2} \cdot Arm_{U2} = 12.25 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{U3} := -P_{U3} \cdot Arm_{U3} = 18.35 \cdot \frac{kip \cdot ft}{ft}$$

$$\Sigma M := \begin{pmatrix} M_{Stem} + M_{FTG} + M_{CO} + M_{closure} + M_{Soil1} + M_{Soil2} + M_{Water1} + M_{Water2} + M_{W1} + M_{W2a} \cdots \\ + M_{W2b} + M_{S1} + M_{S2} + M_{U1} + M_{U2} + M_{U3} \end{pmatrix}$$

$$\Sigma M = 42.66 \cdot \frac{ft \cdot kip}{ft}$$

$$\Sigma Vertical := \begin{pmatrix} F_{Stem} + F_{FTG} + F_{CO} + F_{Soil1} + F_{Soil2} + F_{Water1} + F_{Water2} \cdots \\ + P_{U1} + P_{U2} + P_{U3} \end{pmatrix}$$

$$\Sigma Vertical = 7.86 \cdot \frac{kip}{ft}$$

$$\Sigma Horizontal := \left(F_{closure} + P_{W1} + P_{W2a} + P_{W2b} + P_{S1} + P_{S2}\right)$$

$$\Sigma Horizontal = 15.14 \cdot \frac{kip}{ft}$$

5Φ/437

Find Pile Loads Using CPGA:

Number := if(Pile5 = 0, 4, 5)
Number := if(Pile4 = 0, 3, Number)
Number := if(Pile3 = 0, 2, Number)
Number := if(Pile2 = 0, 1, Number)
Number := if(Pile1 = 0, 0, Number)

Total number of piles:

Number = 3

Concrete Pile Properties

Dimension of square pile $D := 18in$

Assume all piles to pile cap connections are fixed (details show rebar and pile extending into base).

Pile Properties:

$fc := 3000 \cdot psi$

$E_C := 57000 \cdot \sqrt{fc \cdot psi}$ $E_C = 3122.02 \cdot ksi$

$I := \frac{D^4}{12}$ $I = 8748 \cdot in^4$

C33=2.0 for full skin friction
B66 = 0 torsional stiffness
Length = 34ft

$Area := D^2$ $Area = 324 \cdot in^2$

Allowable Design Load Strengths from Geotechs (FS = 1.7 for extreme):
AC = Allowable Pile Compression Load, = 137,963/1.7 = 81.1 k
AT = Allowable Pile Tension Load, = 73,264/1.7 '= 43.1 k

The following allowable design strengths are from the existing pile capacity MathCAD file (FS = 1.5):
PO = Axial Compression Design Load Strength, 536 k (P_{n0} in Mathcad)
PT = Axial Tension Design Load Strength, Fy*As= 128 k
PB = Axial Design Load Strength at Balanced Condition, 274 k (P_{n5} in Mathcad)
MB = Design Moment Strength at Balanced Condition, 1802 k-in (M_{n5} in Mathcad)
MO = Design Moment Strength Under Pure Flexure, 905 k-in (M_{n7} in Mathcad)

Soil:
NH
Soil Modulus, 0.05 k/in^3 (per Geotech)

Concrete Sheet Pile Properties

Dimension of rectangular pile $D_{sp} := 10in$ $W_{sp} := 12in$

Pile Spacing, S=1ft

Assume all sheet pile to pile cap connections are fixed (details show rebar extending into base).

Pile Properties:

$fc := 3000 \cdot psi$

$I_{1sp} := \frac{\left(W_{sp} \cdot D_{sp}^{3}\right)}{12}$ $I_{1sp} = 1000 \cdot in^4$ $I_{2sp} := \frac{D_{sp} \cdot W_{sp}^{3}}{12}$ $I_{2sp} = 1440 \cdot in^4$

C33=2.0 for full skin friction

B66 = 0 torsional stiffness

Length = 15ft

$E_C := 57000 \cdot \sqrt{fc \cdot psi}$ $Area := D_{sp} \cdot W_{sp}$

$E_C = 3122.02 \cdot ksi$ $Area = 120 \cdot in^2$

Allowable Design Load Strengths from Geotechs (FS = 1.7 for extreme):

AC = Allowable Pile Compression Load, = 5'/4'*20,730klf/1.7 = 15.2 k

AT = Allowable Pile Tension Load, = 5'/4'*11.231klf/1.7 = 8.26 k

Assumes two 2.5ft wide sheetpiles engaged; strength is equally divided among four 1' wide piles.

The following allowable design strengths are from the existing pile capacity MathCAD file (FS = 1.5):

PO = Axial Compression Design Load Strength, 205 k (P_{n0} in Mathcad)

PT = Axial Tension Design Load Strength, Fy*As= 56.3 k

PB = Axial Design Load Strength at Balanced Condition, 72 k (P_{n5} in Mathcad)

MB = Design Moment Strength at Balanced Condition, 333 k-in (M_{n5} in Mathcad)

MO = Design Moment Strength Under Pure Flexure, 195 k-in (M_{n7} in Mathcad)

Soil:

NH

Soil Modulus, 0.05 k/in^3 (per Geotech)

52/

$\Sigma M_T := \Sigma M \cdot S \cdot N_{Rows}$

$\Sigma Vertical_T := \Sigma Vertical \cdot S \cdot N_{Rows}$

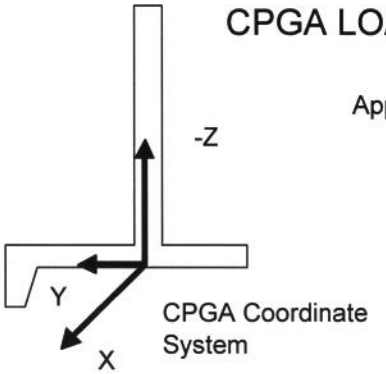
$\Sigma Horizontal_T := \Sigma Horizontal \cdot S \cdot N_{Rows}$

$\Sigma M_T = 170.63 \cdot ft \cdot kip$

$\Sigma Vertical_T = 31.44 \cdot kip$

$\Sigma Horizontal_T = 60.56 \cdot kip$

CPGA LOADING FOR WATER AT TOP OF WALL



Applied Loads

$PY := -\Sigma Horizontal_T$
$PZ := \Sigma Vertical_T$
$MX := -\Sigma M_T$

- Signs added to agree with CPGA sign convention.

$PY = -60.56 \cdot kip$

$PZ = 31.44 \cdot kip$

$MX = -170.63 \cdot ft \cdot kip$

Loads input into CPGA filename: M145R.dat

Pile Loading (CPGA OUTPUT)

$PileLoad1 := \frac{-14.7kip}{ft} \cdot S$ $PileLoad1 = -58.8 \cdot kip$ $PileM1 := 229.5 \frac{kip \cdot in}{ft} \cdot S$ $PileM1 = 918 \cdot kip \cdot in$

PileLoad 1 and Msht, These numbers are the output from CPGA for sheet piling (continuous) multiplied by the bearing pile spacing to get loading comparable to sheet piles for pile cap calcs

$PileLoad2 := 28.2kip$ $PileM2 := 807.7kip \cdot in$

$PileLoad3 := 61.9kip$ $PileM3 := 807.7kip \cdot in$

$PileLoad4 := 0kip$ $PileM4 := 0kip \cdot in$

$PileLoad5 := 0kip$ $PileM5 := 0kip \cdot in$

Check Vertical Loading

$$\Sigma Vertical - \frac{PileLoad1 + PileLoad2 + PileLoad3 + PileLoad4 + PileLoad5}{S} = 0.03 \cdot \frac{kip}{ft}$$

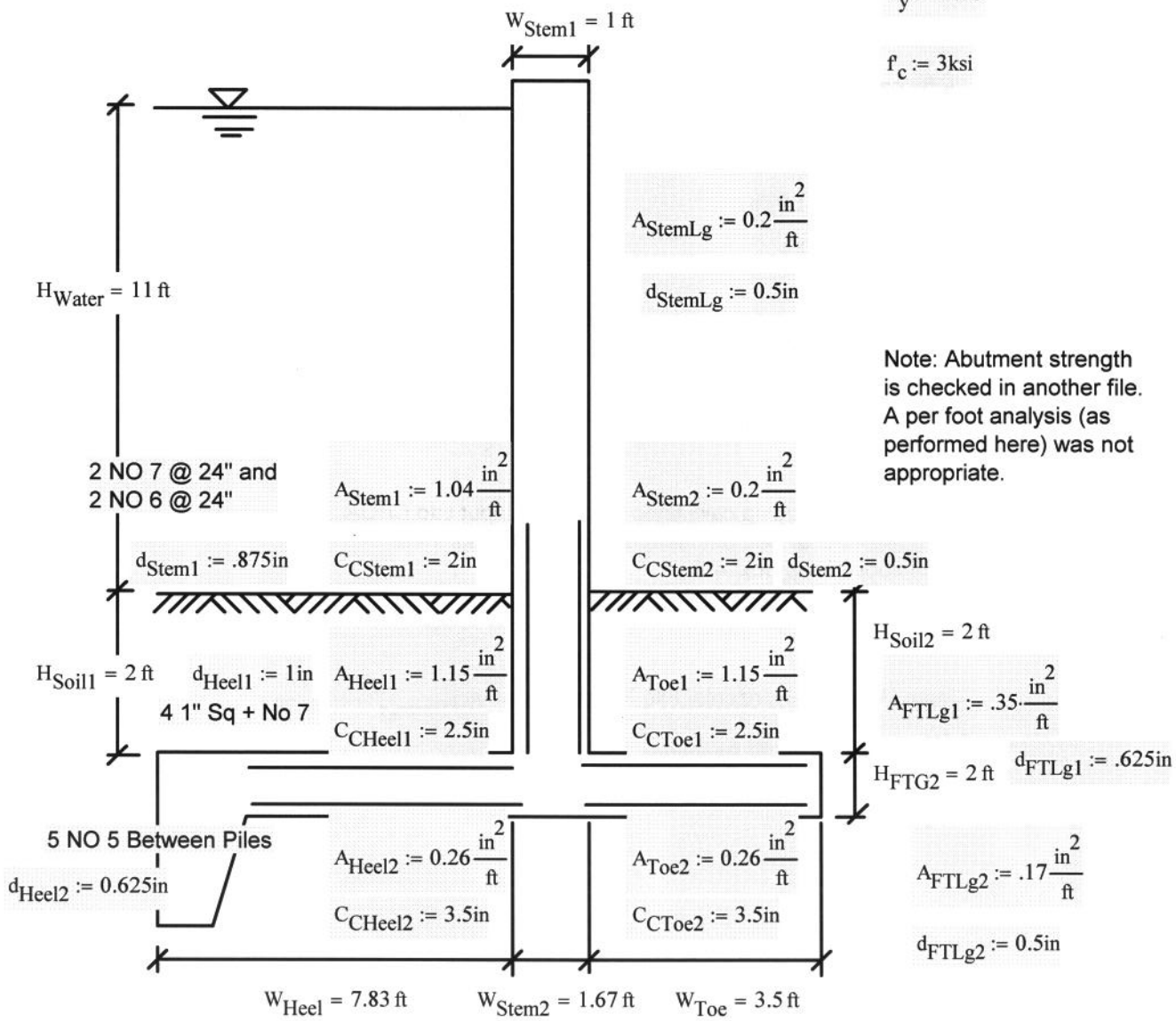
Checks if vertical loading calculated above and input into CPGA matches the vertical load output from CPGA. Checks if the equation above is approx. = 0.

Input Maximum Horizontal Pile Reaction: $Pile_{Hormax} := 16.3 \cdot kip$

Sheetpile Horizontal Reaction: $Sht_{Hormax} := 7 \cdot kip$

5A/437

Typical Properties



Assumptions:

1. Pile cap is assumed rigid; load is distributed uniformly. Since abutment is checked in seperate file, the stem check in this file is for wall portion of stoplog gap closure only.

2. Equations in this file are established for a horizontal footer with uniform thickness.

3. Resisting Side water EL is the EL of the top of resisting grade.

4. Rebar Cutoff lengths assumed sufficient for feasibility study.

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength	$\phi_V := 1$	$\phi_T := 1.0$	Tensile Strength
Flexural Strength	$\phi_B := 1$		

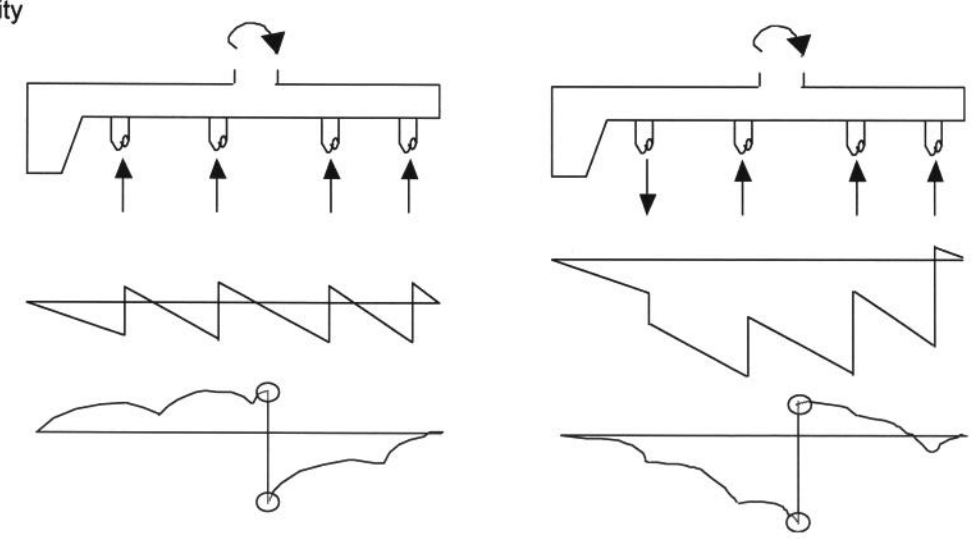
Load Factors

Dead and Live Load Factor	$\gamma_L := 1$
Hydraulic Load Factor	$\gamma_H := 1.0$
Extreme Case Factor	$\gamma_X := 1.0$

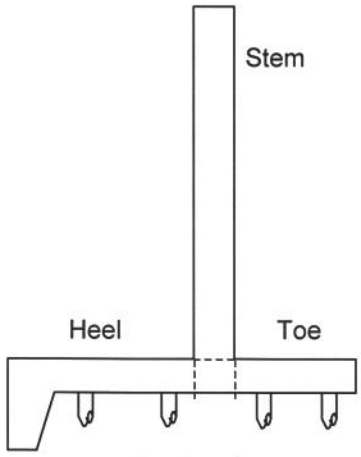
Note: Strength reduction and load factors not used in analysis of existing conditions.

Assumptions for Analysis

It has been assumed that in general the maximum transverse bending moment from loading will occur at or near the connection of the pile cap and stem. As a result section cuts at the intersection of the pile cap and stem wall have been made to evaluate the floodwalls flexural capacity



Typical Shear and Moment Diagrams



Section Cuts

56/437

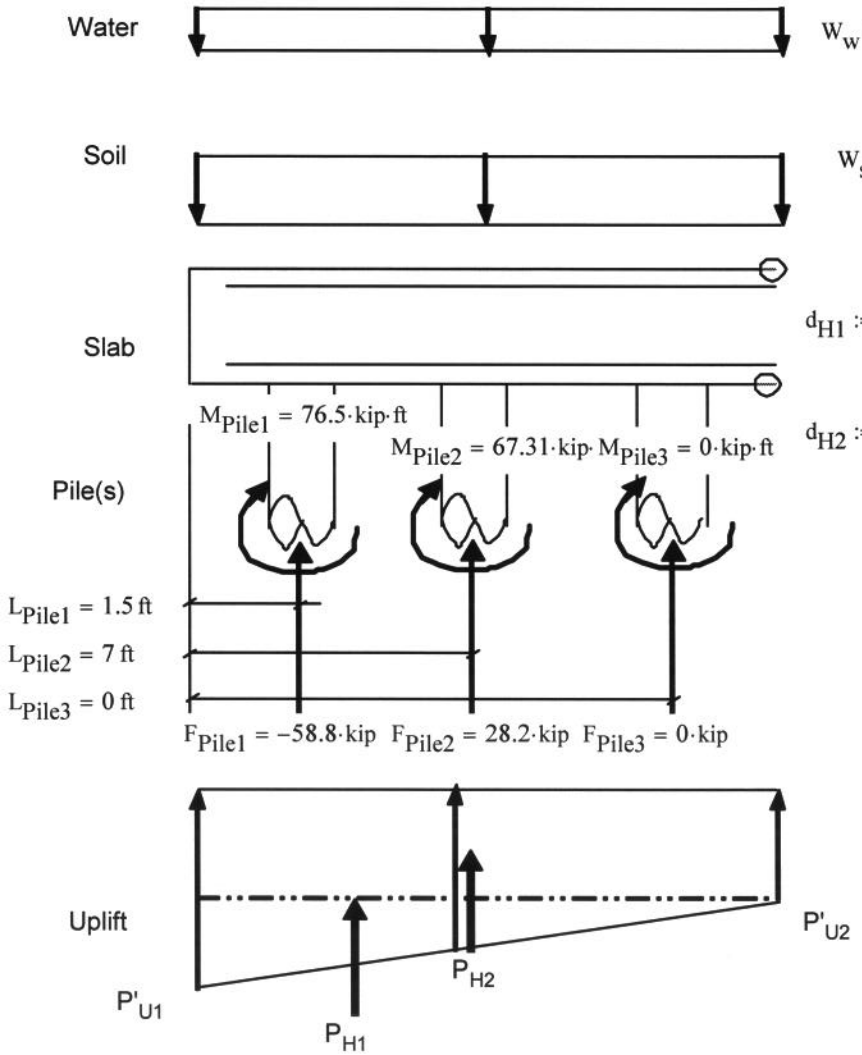
- HEEL Strength Check:

$F_{Pile1} := if(L_{FTG} - Pile1 < W_{Heel}, PileLoad1, 0kip)$ $L_{Pile1} := if(L_{FTG} - Pile1 < W_{Heel}, L_{FTG} - Pile1, 0ft)$

$F_{Pile2} := if(L_{FTG} - Pile2 < W_{Heel}, PileLoad2, 0kip)$ $L_{Pile2} := if(L_{FTG} - Pile2 < W_{Heel}, L_{FTG} - Pile2, 0ft)$

$F_{Pile3} := if(L_{FTG} - Pile3 < W_{Heel}, PileLoad3, 0kip)$ $L_{Pile3} := if(L_{FTG} - Pile3 < W_{Heel}, L_{FTG} - Pile3, 0ft)$

$M_{Pile1} := if(F_{Pile1} = 0, 0, PileM1)$ $M_{Pile2} := if(F_{Pile2} = 0, 0, PileM2)$ $M_{Pile3} := if(F_{Pile3} = 0, 0, PileM3)$



$W_w := \gamma_w \cdot (H_{Water} + H_{Soil1}) \cdot W_{Heel}$

$W_w = 6354 \frac{lb}{ft}$

$W_{sH} := (\gamma_{sat} - \gamma_w) \cdot H_{Soil1} \cdot W_{Heel}$

$W_{sH} = 824.03 \frac{lb}{ft}$

$d_{H1} := H_{FTG2} - C_{CHeel1} - d_{FTLg1} - \frac{1}{2} d_{Heel1}$

$d_{H2} := H_{FTG2} - C_{CHeel2} - d_{FTLg2} - \frac{1}{2} d_{Heel2}$

Pile Spacing (O.C.)

$S = 4\,ft$

Note: Assumes Sheet Pile reduces uplift press. by half.

$P'_{U1} := q_{umax} = 624 \cdot psf$

$$P'_{U2} := \frac{(q_{umax} - q_{utoe})(W_{Toe} + W_{Stem2})}{(L_{FTG} - .5W_{CO})} + q_{utoe} = 471.06 \cdot psf$$

$$P'_{U2p} := \frac{(q_{umax} - q_{utoe})(L_{FTG} - L_{Pile2})}{(L_{FTG} - .5W_{CO})} + q_{utoe} = 491.1 \cdot psf$$

$P_{H1} := \frac{1}{2}(P'_{U1} - P'_{U2}) \cdot (W_{Heel} - 0.5W_{CO}) = 484.29 \cdot plf$ $P_{H2} := P'_{U2} \cdot (W_{Heel} - .5W_{CO}) = 2983.2 \cdot plf$

$P_{H1p} := \frac{1}{2}(P'_{U1} - P'_{U2p}) \cdot (L_{Pile2} - 0.5W_{CO}) = 365.67 \cdot plf$ $P_{H2p} := P'_{U2} \cdot (L_{Pile2} - .5W_{CO}) = 2592.23 \cdot plf$

$P_{H3} := -P_{U3} = 1497.6 \cdot plf$

57/

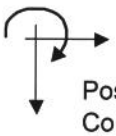
- HEEL Strength Check (cont'd):

Loading: $F_{CO} = 450 \cdot plf$

Slab Weight $W_H := \left(W_{Heel} \cdot H_{FTG2}\right) \cdot \gamma_c$

Bending

$$M_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{F_{Pile1} \cdot \left(W_{Heel} - L_{Pile1}\right) + F_{Pile2} \cdot \left(W_{Heel} - L_{Pile2}\right) + F_{Pile3} \cdot \left(W_{Heel} - L_{Pile3}\right)}{S} + P_{H1} \cdot \frac{2\left(W_{Heel} - .5W_{CO}\right)}{3} \ldots \right.$$
$$+ P_{H2} \cdot \frac{\left(W_{Heel} - .5W_{CO}\right)}{2} + P_{H3} \cdot \left(W_{Heel} - .25W_{CO}\right) - \left(W_H + W_w + W_{sH}\right) \cdot \frac{W_{Heel}}{2} \ldots$$
$$\left. + -F_{CO} \cdot \left(W_{Heel} - .5W_{CO}\right) + \frac{M_{Pile1} + M_{Pile2} + M_{Pile3}}{S} \right]$$



Positive Sign Convention

$M_H = -69.32 \cdot \frac{kip \cdot ft}{ft}$ $\quad M_{uH} := \left|M_H\right|$ $\quad M_{uH} = 69.32 \cdot \frac{kip \cdot ft}{ft}$

Shear

Shear at Stem

$$V_{Ha} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\left(F_{CO} + W_H + W_w + W_{sH}\right) - \frac{F_{Pile1} + F_{Pile2}}{S} - P_{H2} - P_{H3} - P_{H1} \right] = 12662.96 \cdot plf$$

Shear at Pile 2

$$V_{Hb} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\left[F_{CO} + \left(W_H + W_w + W_{sH}\right) \cdot \frac{L_{Pile2}}{W_{Heel}} \right] - \frac{F_{Pile1}}{S} - P_{H1p} - P_{H2p} - P_{H3} \right] = 19212.95 \frac{lb}{ft}$$

$V_H := \max\left(V_{Ha}, V_{Hb}\right)$ $\quad V_{uH} := \left|V_H\right|$ $\quad V_{uH} = 19.21 \cdot \frac{kip}{ft}$

Capacity:

Flexural Capacity

Top Steel: $A_{s1} := A_{Heel1}$

$b := 12 \frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a1 = 1.5 \cdot in$

$\phi M_{H1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{H1} - \frac{a1}{2}\right)$

$\phi M_{H1} = 75.22 \cdot \frac{kip \cdot ft}{ft}$

Bottom Steel: $A_{s2} := A_{Heel2}$

$b := 12 \frac{in}{ft}$

$a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a2 = 0.34 \cdot in$

$\phi M_{H2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{H2} - \frac{a2}{2}\right)$

$\phi M_{H2} = 16.92 \cdot \frac{kip \cdot ft}{ft}$

$\phi M_H := if\left(M_H > 0, \phi M_{H2}, \phi M_{H1}\right)$ $\quad \phi M_H = 75.22 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{H1} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c1} = 26784 \frac{lb}{ft}$

$\phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{H2} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c2} = 25880 \frac{lb}{ft}$

$\phi V_H := if\left(M_H > 0, \phi V_{c2}, \phi V_{c1}\right)$ $\quad \phi V_H = 26.78 \cdot \frac{kip}{ft}$

58/437

- HEEL Strength Check (cont'd):

Factors of Safety

Bending $\phi M_H = 75.22 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_1 := \frac{\phi M_H}{M_{uH}}$

$M_{uH} = 69.32 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_1 = 1.09$

$Check1 := if(\phi M_H > 1.5 M_{uH}, "OKAY", "NO GOOD")$ $\quad Check1 = "NO GOOD"$

Shear $\phi V_H = 26.78 \cdot \frac{kip}{ft}$ $\quad FS_2 := \frac{\phi V_H}{V_{uH}}$

$V_{uH} = 19.21 \cdot \frac{kip}{ft}$ $\quad FS_2 = 1.39$

$Check2a := if(\phi V_H > 1.5 V_{uH}, "OKAY", "NO GOOD")$ $\quad Check2a = "NO GOOD"$

Controlling Factor of Safety

$FS_H := \min(FS_1, FS_2)$ $\quad FS_H = 1.09$

Controlling Mechanism

$Comment_H := if(FS_1 > FS_2, "Shear in Heel", if(M_H > 0, "Flexural Bottom Steel in Heel", "Flexural Top Steel in Heel"))$

$Comment_H = "Flexural Top Steel in Heel"$

Check Longitudinal Flexure Across Piles:

Note: For Feasibility and due to rigid body assumptions, an average of the total load is used to span a middle pile, minus P_{u3} and F_{CO}. Calculations at present assume no My Reaction at pile.

Length of span (pile spacing): $S = 4\,ft$

Number of piles: $Number = 3$

Tributary Pile Width: $W_{trib} := \frac{L_{FTG}}{Number} = 4.33\,ft$

Uniform Load:

$$\omega := \frac{\Sigma Vertical - (P_{U3} + F_{CO})}{L_{FTG}} \cdot W_{trib} = 2969.07 \cdot plf$$

$M_{\omega neg} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot 0.106 \cdot \omega \cdot S^2 = 5.04 \cdot kip \cdot ft$ $\quad M_{\omega pos} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot 0.078 \omega \cdot S^2 = 3.71 \cdot kip \cdot ft$

$roe_b := 0.0371$ $\quad roe_{max} := \frac{1}{2} \cdot roe_b = 0.02$ $\quad roe_{min} := 0.0050$

$roe_{neg} := \frac{A_{FTLg1}}{b \cdot d_{H1}} = 0.0014$ $\quad roe_{pos} := \frac{A_{FTLg2}}{b \cdot d_{H2}} = 0.0007$

Check Steel Ratios (0 is a false Statement, 1 is a true Statement):

$roe_{neg} > roe_{min} = 0$ Not Good; a 3/4 Flexure Reduction is incurred.

$roe_{neg} \le roe_{max} = 1$ OK

$roe_{pos} > roe_{min} = 0$ Not Good; a 3/4 Flexure Reduction is incurred.

$roe_{pos} \le roe_{max} = 1$ OK

$$a_{neg} := \frac{A_{FTLg1} \cdot F_y}{0.85 \cdot f'_c \cdot b} = 0.46 \cdot in \qquad a_{pos} := \frac{A_{FTLg2} \cdot F_y}{0.85 \cdot f'_c \cdot b} = 0.22 \cdot in$$

$$\phi M_{yneg} := \phi_B \cdot \frac{3}{4} \cdot A_{FTLg1} \cdot ft \cdot F_y \cdot \left(d_{H1} - \frac{a_{neg}}{2}\right) = 17.63 \cdot kip \cdot ft \qquad \phi M_{ypos} := \phi_B \cdot \frac{3}{4} \cdot A_{FTLg2} \cdot ft \cdot F_y \cdot \left(d_{H2} - \frac{a_{pos}}{2}\right) = 8.32\, ft \cdot kip \cdot \frac{ft}{ft}$$

Factor of Safety:

$$FS_{2b} := \frac{\phi M_{yneg}}{M_{\omega neg}} \qquad FS_{2b} = 3.5$$

$$Check2b := if\left(\phi M_{yneg} > 1.5 M_{\omega neg}, \text{"OKAY"}, \text{"NO GOOD"}\right) \qquad Check2b = \text{"OKAY"}$$

$$FS_{2c} := \frac{\phi M_{ypos}}{M_{\omega pos}} \qquad FS_{2c} = 2.25$$

$$Check2c := if\left(\phi M_{ypos} > 1.5 M_{\omega pos}, \text{"OKAY"}, \text{"NO GOOD"}\right) \qquad Check2c = \text{"OKAY"}$$

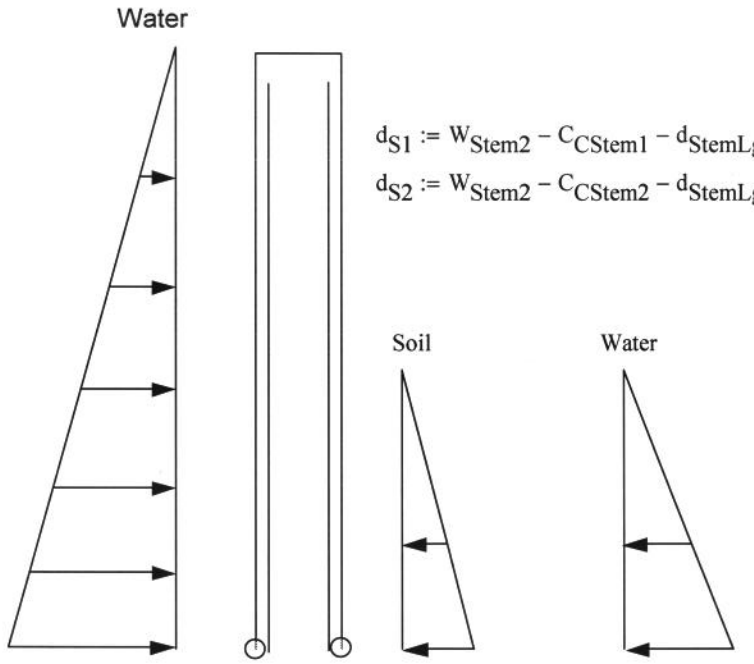
$$FS_{LF} := \min\left(FS_{2b}, FS_{2c}\right) = 2.25$$

Shear is sufficient by inspection since it is adequate, from concrete capacity only, in the primary loading direction.

60/437

STEM Strength Check

For stoplog gap sections other than abutment. Abutment was checked in another file.



$d_{S1} := W_{Stem2} - C_{CStem1} - d_{StemLg} - 0.5d_{Stem1}$

$d_{S2} := W_{Stem2} - C_{CStem2} - d_{StemLg} - 0.5d_{Stem2}$

$P_{ws1} := (H_{Water} + H_{Soil1}) \cdot \gamma_w$

$P_{ws1} = 811 \frac{lb}{ft^2}$

$P_{ss1} := K_o \cdot (H_{Soil1}) \cdot (\gamma_{sat} - \gamma_w)$

$P_{ss1} = 65.79 \frac{lb}{ft^2}$

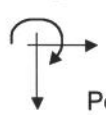
$P_{ss} := H_{Soil2} \cdot K_o \cdot (\gamma_{sat} - \gamma_w)$ $\quad P_{ws2} := H_{Soil2} \cdot \gamma_w$

$P_{ss} = 65.79 \frac{lb}{ft^2}$ $\quad P_{ws2} = 124.8 \frac{lb}{ft^2}$

Loading:

Bending

$$M_S := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{P_{ws1} \cdot (H_{Water} + H_{Soil1})}{2} \cdot \frac{H_{Water} + H_{Soil1}}{3} + \frac{P_{ss1} \cdot (H_{Soil1})}{2} \cdot \frac{H_{Soil1}}{3} - \frac{(P_{ss} + P_{ws2}) \cdot (H_{Soil2})}{2} \cdot \frac{H_{Soil2}}{3} \right]$$



Positive Sign Convention

$M_S = 23 \cdot \frac{kip \cdot ft}{ft}$ $\quad M_{uS} := |M_S|$

$M_{uS} = 23 \cdot \frac{kip \cdot ft}{ft}$

Shear

$$V_S := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{P_{ws1} \cdot (H_{Water} + H_{Soil1})}{2} + \frac{P_{ss1} \cdot H_{Soil1}}{2} - \frac{(P_{ss} + P_{ws2}) \cdot (H_{Soil2})}{2} \right]$$

$V_S = 5.15 \cdot \frac{kip}{ft}$ $\quad V_{uS} := |V_S|$

$V_{uS} = 5.15 \cdot \frac{kip}{ft}$

$A_{ts} := \frac{1}{2} \cdot \left(\frac{W_{Stem1} + W_{Stem2}}{2} \right) \cdot 0.0028 = 0.27 \frac{1}{ft} \cdot in^2$ Per face.

61/

- STEM Strength Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Stem1}$

$b := 12\frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a1 = 1.36 \cdot in$

$\phi M_S := \phi_B A_{s1} \cdot F_y \cdot \left(d_{S1} - \frac{a1}{2}\right)$ $\quad \phi M_S = 56.93 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\mu := 1.0$ For Concrete placed against roughened concrete.

$\phi V_{S1} := \phi_V \cdot A_{Stem2} \cdot F_y \cdot \mu = 8000\frac{lb}{ft}$ Does not rely on any flexural reinforcing.

$\phi V_{S2} := \phi_V \cdot \left(2 \cdot b \cdot d_{S1} \cdot \sqrt{f'_c \cdot psi}\right) = 22481.82\frac{lb}{ft}$

$\phi V_S := \min\left(\phi V_{S1}, \phi V_{S2}\right) = 8000\frac{lb}{ft}$ $\quad \phi V_S = 8000\frac{lb}{ft}$

Temperature and Shrinkage

$A_{StemLg} = 0.2\frac{1}{ft} \cdot in^2$ $\quad A_{ts} = 0.27\frac{1}{ft} \cdot in^2$ $\quad FS_{3a} := \frac{A_{StemLg}}{A_{ts}}$ $\quad FS_{3a} = 0.74$

Temperature and Shrinkage is not a failure mechanism for Risk and Reliability; however, it is an indication of structural condition.

Factors of Safety

Bending

$\phi M_S = 56.93 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_{3b} := \frac{\phi M_S}{M_{uS}}$

$M_{uS} = 22.77 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_{3b} = 2.5$

$Check3b := if\left(\phi M_S > 1.5 M_{uS}, "OKAY", "NO\ GOOD"\right)$ $\quad Check3b = "OKAY"$

Shear

$\phi V_S = 8 \cdot \frac{kip}{ft}$ $\quad FS_4 := \frac{\phi V_S}{V_{uS}}$

$V_{uS} = 5.15 \cdot \frac{kip}{ft}$ $\quad FS_4 = 1.55$

$Check4 := if\left(\phi V_S > 1.5 V_{uS}, "OKAY", "NO\ GOOD"\right)$ $\quad Check4 = "OKAY"$

Controlling Factor of Safety

$FS_S := \min\left(FS_{3b}, FS_4\right)$ $\quad FS_S = 1.55$

Controlling Mechanism

$Comment_S := if\left(FS_{3b} > FS_4, "Shear\ in\ Stem", if\left(M_S > 0, "Stem\ Flexural\ Riverside\ Steel", "Stem\ Flexural\ Landside\ Steel"\right)\right)$

$Comment_S = "Shear\ in\ Stem"$

Note: No Cut-off bar checks were made for Feasibility Study.

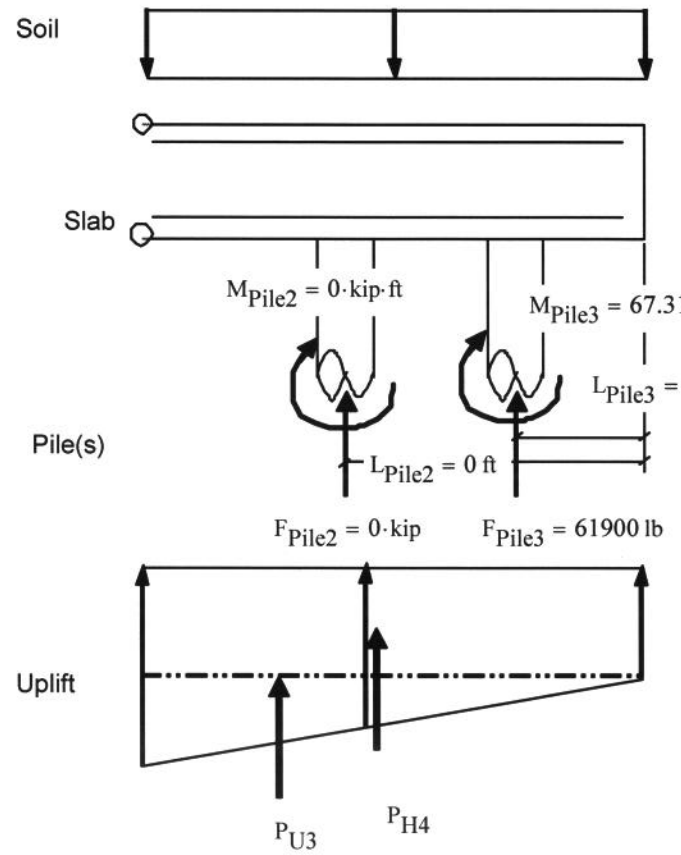
- TOE Strength Check

62/437

$F_{Pile2} := if(Pile2 < W_{Toe}, PileLoad2, 0kip)$ $F_{Pile3} := if(Pile3 < W_{Toe}, PileLoad3, 0kip)$ $F_{Pile4} := if(Pile4 < W_{Toe}, PileLoad4, 0kip)$

$L_{Pile2} := if(Pile2 < W_{Toe}, Pile2, 0ft)$ $L_{Pile3} := if(Pile3 < W_{Toe}, Pile3, 0ft)$ $L_{Pile4} := if(Pile4 < W_{Toe}, Pile4, 0ft)$

$M_{Pile2} := if(Pile2 < W_{Toe} + 6in, PileM2, 0kip)$ $M_{Pile3} := if(F_{Pile3} = 0, 0, PileM3)$ $M_{Pile4} := if(F_{Pile4} = 0, 0, PileM4)$



$W_{swT} := (\gamma_{sat} \cdot H_{Soil2}) \cdot W_{Toe}$ $W_{swT} = 805 \frac{lb}{ft}$

$d_{T1} := H_{FTG2} - C_{CToe1} - d_{FTLg1} - 0.5 d_{Heel1}$

$d_{T2} := H_{FTG2} - C_{CToe2} - d_{FTLg2} - 0.5 d_{Heel2}$

Pile Spacing (O.C.)

$S = 4\,ft$

Note: Assumes Sheet Piling reduces uplift pressures by half.

$q_{utoe} = 346.2 \cdot psf$

$P_{H4} := q_{utoe} \cdot W_{Toe} = 1211.7 \frac{lb}{ft}$

$$P'_{U3} := \frac{(q_{umax} - q_{utoe})(W_{Toe})}{(L_{FTG} - 0.5 W_{CO})} + q_{utoe} = 430.73 \cdot psf \qquad P_{H3} := \frac{1}{2} \cdot (P'_{U3} - q_{utoe}) \cdot W_{Toe} = 147.92 \frac{lb}{ft}$$

Loading:

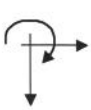
Slab Weight $W_T := (W_{Toe} \cdot H_{FTG2}) \cdot \gamma_c$ $W_T = 1050 \frac{lb}{ft}$

Pile Centroid

$$L_{PileT} := \frac{L_{Pile2} \cdot F_{Pile2} + L_{Pile3} \cdot F_{Pile3} + L_{Pile4} \cdot F_{Pile4}}{F_{Pile2} + F_{Pile3} + F_{Pile4}} \qquad L_{PileT} = 2\,ft$$

Bending

$$M_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[(W_T + W_{swT}) \cdot \frac{W_{Toe}}{2} - \left(\frac{F_{Pile2} + F_{Pile3} + F_{Pile4}}{S} \right) \cdot (W_{Toe} - L_{PileT}) - P_{H3} \cdot \frac{W_{Toe}}{3} \cdots + -P_{H4} \cdot \frac{W_{Toe}}{2} + \frac{M_{Pile2} + M_{Pile3} + M_{Pile4}}{S} \right]$$



Positive Sign Convention

$M_T = -5.43 \cdot \frac{kip \cdot ft}{ft}$ $M_{uT} := |M_T|$ $M_{uT} = 5.43 \cdot \frac{kip \cdot ft}{ft}$

- TOE Strength Check (Cont'd)

63/437

Shear

Shear at Stem.

$$V_{Ta} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\left(W_T + W_{swT}\right) - \left(\frac{F_{Pile2} + F_{Pile3} + F_{Pile4}}{S}\right) - P_{H3} - P_{H4} \right] = -14979.62 \frac{lb}{ft}$$

$$V_{Ta} := \left|V_{Ta}\right|$$

Shear at Pile

$$V_{Tb} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\left(W_T + W_{swT}\right) \cdot \frac{L_{Pile3}}{W_{Toe}} - \left[q_{utoe} \cdot L_{Pile3} + .5 \cdot \frac{\left(q_{umax} - q_{utoe}\right) \cdot L_{Pile3}}{\left(L_{FTG} - .5 W_{CO}\right)} \cdot L_{Pile3} \right] - \frac{F_{Pile3}}{S} \right] = -15155.7 \frac{lb}{ft}$$

$$V_{Tb} := \left|V_{Tb}\right| \qquad V_{uT} := \max\left(V_{Ta}, V_{Tb}\right) \qquad V_{uT} = 15.16 \cdot \frac{kip}{ft}$$

Capacity:

Flexural Capacity

Top Steel: $A_{s1} := A_{Toe1}$

$$b := 12 \frac{in}{ft}$$

$$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a1 = 1.5 \cdot in$$

$$\phi M_{T1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{T1} - \frac{a1}{2}\right)$$

$$\phi M_{T1} = 75.22 \cdot \frac{kip \cdot ft}{ft}$$

Bottom Steel: $A_{s2} := A_{Toe2}$

$$b := 12 \frac{in}{ft}$$

$$a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b} \qquad a2 = 0.34 \cdot in$$

$$\phi M_{T2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{T2} - \frac{a2}{2}\right)$$

$$\phi M_{T2} = 16.92 \cdot \frac{kip \cdot ft}{ft}$$

$$\phi M_T := if\left(M_T > 0, \phi M_{T1}, \phi M_{T2}\right) \qquad \phi M_T = 16.92 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{T1} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c1} = 26784 \frac{lb}{ft}$$

$$\phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{T2} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c2} = 25880 \frac{lb}{ft}$$

$$\phi V_T := if\left(M_{uT} > 0, \phi V_{c1}, \phi V_{c2}\right) \qquad \phi V_T = 26.78 \cdot \frac{kip}{ft}$$

Factors of Safety

Bending

$$\phi M_T = 16.92 \cdot \frac{kip \cdot ft}{ft} \qquad M_{uT} = 5.43 \cdot \frac{kip \cdot ft}{ft} \qquad FS_5 := \frac{\phi M_T}{M_{uT}} \qquad FS_5 = 3.11$$

$$Check5 := if\left(\phi M_T > 1.5 M_{uT}, \text{"OKAY"}, \text{"NO GOOD"}\right) \qquad Check5 = \text{"OKAY"}$$

Shear

$$\phi V_T = 26.78 \cdot \frac{kip}{ft} \qquad V_{uT} = 15.16 \cdot \frac{kip}{ft} \qquad FS_6 := \frac{\phi V_T}{V_{uT}} \qquad FS_6 = 1.77$$

$$Check6 := if\left(\phi V_T > 1.5 V_{uT}, \text{"OKAY"}, \text{"NO GOOD"}\right) \qquad Check6 = \text{"OKAY"}$$

Controlling Factor of Safety

$$FS_T := \min\left(FS_5, FS_6\right) \qquad FS_T = 1.77$$

Controlling Mechanism

$$Comment_T := if\left(FS_5 > FS_6, \text{"Shear in Heel"}, if\left(M_T > 0, \text{"Flexural Top Steel in Toe"}, \text{"Flexural Bottom Steel in Toe"}\right)\right)$$

$$Comment_T = \text{"Shear in Heel"}$$

64/437

Pile Checks:

Tension Pile Embedment: Pullout Force per foot: $T_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \frac{-PileLoad1 \cdot 1ft}{S} = 14.7 \cdot kip$

Equations are established for straight bars.

$\psi_t := 1.0$ $\psi_e := 1.0$ $\psi_s := 0.8$ $\lambda := 1.0$

Embedment Length $l_{em} := 18in$

Bar diameter: $d_b := .75in$ $c_b := 2in$ K_{tr} is assumed zero.

Number of bars per foot: $N_b := 4.8$ Area of 1 bar: $A_{bt} := 0.44in^2$

$$c_{mod} := if\left(\frac{c_b}{d_b} > 2.5, 2.5, \frac{c_b}{d_b}\right) = 2.5 \qquad l_d := \frac{3}{40} \cdot \frac{F_y}{\sqrt{f'_c \cdot psi}} \cdot \frac{\psi_t \cdot \psi_s \cdot \psi_e \cdot \lambda}{c_{mod}} \cdot d_b = 13.15 \cdot in$$

$Check7 := if(l_{em} > l_d, "OKAY", "NO GOOD")$ $Check7 = "OKAY"$

$T_n := \phi_T \cdot N_b \cdot A_{bt} \cdot F_y = 84.48 \cdot kip$

$Check8 := if(T_n > 1.5 \cdot T_u, "OKAY", "NO GOOD")$ $FS_8 := \frac{T_n}{T_u} = 5.75$ $Check8 = "OKAY"$

Punching Shear Check (assuming 3rd pile in max compression and distance to edge of pile cap is > d/2):

Side of a square pile: $D = 18 \cdot in$ Compression Force: $F_c := PileLoad3 \cdot (\gamma_L \cdot \gamma_H \cdot \gamma_X) = 61.9 \cdot kip$

Pile Embedment into pile cap: $Pile_{em} := 12in$ Height of concrete cone: $h_{cone} := H_{FTG2} - Pile_{em} = 1\ ft$

$\alpha_s := 20$ $d := d_{H1} - h_{cone} = 8.37 \cdot in$ $b_o := 4 \cdot (D + d) = 105.5 \cdot in$

$$\alpha_{mod} := if\left(\frac{\alpha_s \cdot d}{b_o} + 2 > 4, 4, \frac{\alpha_s \cdot d}{b_o} + 2\right) = 3.59$$

$V_c := \phi_V \cdot \alpha_{mod} \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot d = 173.62 \cdot kip$

$Check9 := if(V_c > 1.5 \cdot F_c, "OKAY", "NO GOOD")$ $FS_9 := \frac{V_c}{F_c} = 2.8$ $Check9 = "OKAY"$

Vertical Bearing Check (assuming 3rd pile in max compression):

$B_{pile} := \phi_V \cdot D^2 \cdot 0.85 \cdot f'_c = 826.2 \cdot kip$

$Check10 := if(B_{pile} > 1.5 \cdot F_c, "OKAY", "NO GOOD")$ $FS_{10} := \frac{B_{pile}}{F_c} = 13.35$ $Check10 = "OKAY"$

Horzontal Shear and Bearing Checks.

$B_{pile2} := \phi_V \cdot Pile_{em} \cdot D \cdot 0.85 \cdot f'_c = 550.8 \cdot kip$ $FS_{11} := \frac{B_{pile2}}{Pile_{Hormax}} = 33.79$

$Check11 := if(B_{pile2} > 1.5 \cdot Pile_{Hormax}, "OKAY", "NO GOOD")$ $Check11 = "OKAY"$

$B_{sht} := \phi_V \cdot Pile_{em} \cdot W_{sp} \cdot 0.85 \cdot f'_c = 367.2 \cdot kip$ $FS_{12} := \frac{B_{sht}}{Sht_{Hormax}} = 52.46$

$Check12 := if(B_{pile2} > 1.5 \cdot Pile_{Hormax}, "OKAY", "NO GOOD")$ $Check12 = "OKAY"$

$FS_B := \min(FS_{10}, FS_{11}, FS_{12}) = 13.35$

Shear. Due to the continuous construction of the sheet pile, b = 12".

$Sht_{Hormax} = 7 \cdot kip$

For 1 ft wide pile.

$d_f := 18in - \frac{D_{sp}}{2} - 2.5in - .5in - .5 \cdot \frac{7}{8}in = 9.56 \cdot in$ $\quad b_o := 12in$

$V_c := \phi_V \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot d = 11.01 \cdot kip$

$Check13 := if(V_c > 1.5 \cdot Sht_{Hormax}, "OKAY", "NO GOOD")$ $\quad FS_{13} := \frac{V_c}{Sht_{Hormax}} = 1.57$ $\quad$ Check13 = "OKAY"

Overall Factor of Safety

Factor of Safety

Heel Factor of Saftey:	$FS_H = 1.09$
Longitudinal Flexure of Pile Cap:	$FS_{LF} = 2.25$
Stem Factor of Safety:	$FS_S = 1.55$
Toe Factor of Safety:	$FS_T = 1.77$
Tension Pile Embedment:	Check7 = "OKAY"
Tension Pile Factor of Safety:	$FS_8 = 5.75$
Punching Shear Factor of Safety:	$FS_9 = 2.8$
Bearing Factor of Safety:	$FS_B = 13.35$
Pile Cap Shear Factor of Safety:	$FS_{13} = 1.57$

$FoS := \min(FS_H, FS_{LF}, FS_S, FS_T, FS_8, FS_9, FS_B, FS_{13})$

$FoS = 1.09$

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis.

2011 REVISION
66/437

Comp by: Sheffield
Chkd by: Marx 9/28/11

Units:

CID MO 10"x12" (16') Sheetpile

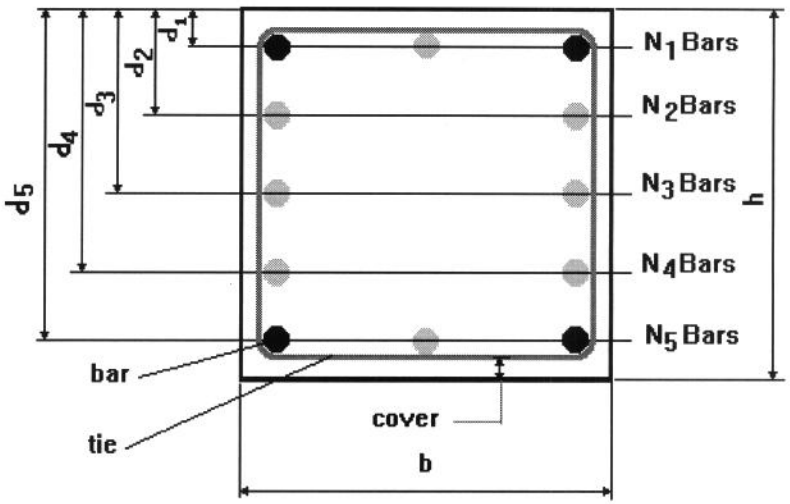
$kips := 1000lb$

$psi := \frac{lb}{in^2}$ $\quad psf := \frac{lb}{ft^2}$

$ton := 2000lb$ $\quad tsf := \frac{ton}{ft^2}$ $\quad klf := \frac{kips}{ft}$ $\quad ksi := \frac{kips}{in^2}$ $\quad pcf := \frac{lb}{ft^3}$ $\quad plf := \frac{lb}{ft}$

Note: The file has been used by many Engineers on previous projects who have also verified the results with independent programs. It was not closely re-examined for a Reliability Analysis.

This program calulates the column interaction curve for a rectangular concrete column using the methods of the Concrete Reinforcing Steel Institute. Note this program also accounts for less than 1 percent steel by reducing the width. Procedure based on CRSI Manual Pg 3-5 to Pg 3-8



$h := 10in$	Column Depth	$cover := 2.5in$	Cover -in	
$w := 12in$	Column Width	$N_1 := 2.4$	Bars in Row 1	
$bar := 6$	Bar Size	$N_2 := 2.4$	Bars in Row 2	
$tie := 2$	Tie Size	$N_3 := 0$	Bars in Row 3	
$f'c := 3ksi$	Conc. Str. - ksi	$N_4 := 0$	Bars in Row 4	
$fy := 40ksi$	Bar Yield - ksi	$N_5 := 0$	Bars in Row 5	$row \equiv 2$

Total Rows of Bars- NOTE: This can be changed and more rows added if required.

$$A_v := \left[\pi \cdot \left(\frac{tie \cdot in}{8 \cdot 2}\right)^2\right] = 0.049 \cdot in^2 \qquad s_v := 8 \cdot in$$

Modulus of Elasticty: $Es := 29000ksi$

Concrete Strength: $f'c := 3000psi$

FEMA 310 Allowed for concrete aging: $FEMA_Factor := 1.00$

Nominal Concrete Strength assumed orginally used: $f'c := f'c \cdot FEMA_Factor$ $\quad f'c = 3000 \cdot psi$

NOTE - if FEMA factor is 1.00 then concrete strength is the nominal concrete strength. If trying to get the expected concrete strength the FEMA factor should be 1.25. See FEMA 310 guidance.

67/437

$areabar \equiv (0in^2\ \ 0in^2\ \ .05in^2\ \ 0.11in^2\ \ 0.2in^2\ \ 0.31in^2\ \ 0.44in^2\ \ 0.6in^2\ \ 0.79in^2\ \ 1in^2\ \ 1.27in^2\ \ 1.56in^2\ \ 0in^2\ \ 0in^2\ \ 2.25in^2)$

$diabar \equiv (0in\ \ 0in\ \ .25in\ \ 0.38in\ \ 0.5in\ \ 0.63in\ \ 0.75in\ \ 0.88in\ \ 1in\ \ 1.13in\ \ 1.27in\ \ 1.41in\ \ 0in\ \ 0in\ \ 1.69in)$

Calculate effective stress block per ACI 318 sect. 10.2.7.3 $\beta 1 := if\left[f'c \le 4ksi, 0.85, if\left[f'c > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'c}{ksi} - 4\right)\right]\right]$ $\beta 1 = 0.85$

Iterations $i := 1 .. row$

Bar Diameter $db := diabar_{0,bar}$ $db = 0.75\cdot in$

Bar area at i $A_i := N_i\cdot areabar_{0,bar}$

Tie Diameter $dt := diabar_{0,tie}$ $dt = 0.250\cdot in$

Distances from compression face $d_1 := cover + \frac{db}{2} + dt$ $d_1 = 3.125\cdot in$

Spacing between rows of bars $space := \frac{h - 2\cdot d_1}{row - 1}$ $space = 3.75\cdot in$ $d_i := d_1 + (i - 1)\cdot space$

$i =$	$N_i =$	$A_i =$	$d_i =$
1	2.4	1.056 $\cdot in^2$	3.125 $\cdot in$
2	2.4	1.056	6.875

Gross Area: $Ag := w\cdot h$ $Ag = 120\cdot in^2$

Steel Area $Ast := \sum_{i=1}^{row} A_i$ $Ast = 2.11\cdot in^2$

Reduce width if $\rho < 0.01$: $\frac{Ast}{Ag} = 0.0176$ $b := if\left(\frac{Ast}{Ag} > 0.01, w, \frac{Ast}{0.01\cdot h}\right)$

Effective Width $b = 12\cdot in$

$Ag := b\cdot h$ $Ag = 120\cdot in^2$

$\frac{Ast}{Ag} = 0.0176$

68/

Calculate Axial Load at no eccentricity, e (min) at ϕPn(max):

$Pn_1 := 0.8\cdot[0.85\cdot f'c\cdot(Ag - Ast) + fy\cdot Ast]$ $\quad Pn_1 = 308\cdot kips$

Equate as the first set of points on the interaction diagram: $\quad Pn_0 := Pn_1 \quad Pn_0 = 308.076\cdot kips \quad Mn_0 := 0kips\cdot in$

Axial Compression with Aci Reduction value: $\quad \phi := 1 \quad \phi\cdot Pn_1 = 308\cdot kips$

Assume c=h $\quad c := h \quad c = 10\cdot in$

Given

$$Pn_1 = \sum_{i=1}^{row}\left[\left[\left(c - d_i\right)\cdot\frac{0.003}{c}\cdot Es - if\left(d_i < \beta1\cdot c, 0.85\cdot f'c, 0\right)\right]\cdot A_i\right] + 0.85\cdot f'c\cdot\beta1\cdot c\cdot b \qquad Pn_1 = 308.076\cdot kips$$

$c := Find(c) \quad c = 8.94\cdot in$

Calculate Width of Compression block: $\quad a := if(\beta1\cdot c > h, h, \beta1\cdot c) \quad a = 7.598\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right)\cdot\frac{0.003}{c}$ $\quad \varepsilon s_i =$

0.002
0.001

$fs_i := if\left(Es\cdot\varepsilon s_i > fy, fy, if\left(Es\cdot\varepsilon s_i < 0 - fy, 0 - fy, Es\cdot\varepsilon s_i\right)\right)$ $\quad fs_i =$

40
20.086

$\cdot ksi$

69/

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Fs_i =$

39.547
18.518

$\cdot kips$

$Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$ $\quad Ms_i =$

74.151
-34.722

$\cdot kips \cdot in$

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 39.429 \cdot kips \cdot in$ $\quad Fst := \sum_{i=1}^{row} Fs_i$ $\quad Fst = 58.065 \cdot kips$

$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 23.27 \cdot kips \cdot ft$ $\quad Fc := 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$ $\quad Fc = 232.497 \cdot kips$

$Mn_1 := Mc + Mst$ $\quad Mn_1 = 318.666 \cdot kips \cdot in$ $\quad e_1 := \frac{Mn_1}{Pn_1}$ $\quad e_1 = 1.03 \cdot in$

$d_{row} = 6.875 \cdot in$

7Φ/437

Find Pn and Mn at point of zero tension in bars:

$c := d_{row}$ $\quad c = 6.875 \cdot in$ $\quad a := \beta 1 \cdot c$ $\quad a = 5.844 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$i =$	$\varepsilon s_i =$	$fs_i =$	$Fs_i =$	$Ms_i =$	$A_i =$	$d_i =$
1	0.002	40 $\cdot ksi$	39.547 $\cdot kips$	74.151 $\cdot kips \cdot in$	1.056 $\cdot in^2$	3.125 $\cdot in$
2	0	20.086	21.211	-39.771	1.056	6.875

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 34.38 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 371.608 \cdot kips \cdot in$

$Mn_2 := Mc + Mst$ $\quad Mn_2 = 405.988 \cdot kips \cdot in$ $\quad Mn_2 = 406 \cdot kips \cdot in$

$Mn_1 = 318.666 \cdot kips \cdot in$ $\quad Mn_1 := if(Mn_1 < Mn_2, Mn_1, Mn_2)$ $\quad Mn_1 = 318.666 \cdot kips \cdot in$

$Pn_2 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_2 = 240 \cdot kips$ $\quad Pn_1 = 308.076 \cdot kips$ $\quad Pn_2 := if(Pn_2 < Pn_1, Pn_2, Pn_1)$ $\quad Pn_2 = 239.577 \cdot kips$

71/

Find Pn and Mn at point of 0.25 *fy tension in bars:

$$c := \frac{d_{row}}{1 + \frac{0.25 \cdot fy}{Es \cdot 0.003}}$$

$d_{row} = 6.875 \cdot in$

$a := \beta 1 \cdot c$

Find Moment:

$$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$$

$$fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$$

$$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$$

$$Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.26 ft	0.001	40 ·ksi	1.056 $\cdot in^2$	39.547 ·kips	74.151 ·kips·in
0.573	-0	-10	1.056	-10.56	19.8

$$Mst := \sum_{i=1}^{row} Ms_i$$

$Mst = 94 \cdot kips \cdot in$

$$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$$

$Mc = 381.609 \cdot kips \cdot in$

$Mn_3 := Mc + Mst$

$Mn_3 = 476 \cdot kips \cdot in$

$$Pn_3 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$$

$Pn_3 = 189 \cdot kips$

72/437

Find Pn and Mn at point of 0.50 *fy tension in bars:

$d_{row} = 6.875 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{0.50 \cdot fy}{Es \cdot 0.003}}$ $\quad c = 5.59 \cdot in$ $\quad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.26 ft	0.001	38.364 ·ksi	1.056 $\cdot in^2$	37.819 ·kips	70.911 ·kips·in
0.573	-0.001	-20	1.056	-21.12	39.6

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 111 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 381.555 \cdot kips \cdot in$ $\quad Mn_4 := Mc + Mst$ $\quad Mn_4 = 492 \cdot kips \cdot in$

$Pn_4 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_4 = 162 \cdot kips$

Find Pn and Mn at Balanced:

$d_{row} = 6.875 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{fy}{Es \cdot 0.003}}$ $\quad c = 4.71 \cdot in$ $\quad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$

0.26	ft
0.573	

$\varepsilon s_i =$

0.001
-0.001

$fs_i =$

29.273	$\cdot ksi$
-40	

$A_i =$

1.056	$\cdot in^2$
1.056	

$Fs_i =$

28.219	$\cdot kips$
-42.24	

$Ms_i =$

52.911	$\cdot kips \cdot in$
79.2	

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 132 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 367.298 \cdot kips \cdot in$ $\quad Mn_5 := Mc + Mst$ $\quad Mn_5 = 499 \cdot kips \cdot in$

$Pn_5 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_5 = 108 \cdot kips$

74/437

Find Mn at ϕ Pn = 0.1*f'c*Ag:

$Pn_6 := \frac{0.1 \cdot f'c \cdot w \cdot h}{\phi}$ $\quad Pn_6 = 36 \cdot kips$ $\quad Pn_6 := if(Pn_6 > Pn_5, Pn_5, Pn_6)$ $\quad Pn_6 = 36 \cdot kips$

Assume c=h/2 $\quad c := h \cdot 0.5$ $\quad c = 5 \cdot in$

Given

$$Pn_6 = \sum_{i=1}^{row} \left[\left[\begin{cases} (c - d_i) \cdot \frac{0.003}{c} \cdot Es \ldots + -if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es < fy \\ fy - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \geq fy \\ -fy - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \leq -fy \end{cases} \right] \cdot A_i \right] \ldots + 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c)$ $\quad c = 3.07 \cdot in$

$a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c)$ $\quad a = 2.61 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
3.125 $\cdot in$	-0	-1540.645 $\cdot psi$	1.056 $\cdot in^2$	-1.627 $\cdot kips$	-3.05 $\cdot in \cdot kips$
6.875	-0.004	-40000	1.056	-42.24	79.2

75/

$$Mst := \sum_{i=1}^{row} Ms_i \qquad Mst = 76 \cdot kips \cdot in$$

$$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right) \qquad Mc = 295.107 \cdot kips \cdot in$$

$$Mn_6 := Mc + Mst \qquad Mn_6 = 371 \cdot kips \cdot in$$

$$Pn_6 = 36 \cdot kips$$

76/437

Find Mn at ϕ Pn = 0.0:

$Pn_7 := 0lb$ $\qquad$ $Pn_7 = 0\,lb$

Assume c=0.2h $\qquad$ $c := 0.2 \cdot h$

Given

$$Pn_7 = \sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es < fy \\ + -\text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \\ fy - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \geq fy \\ -fy - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \leq -fy \end{array} \right] \cdot A_i \right] \ldots$$

$$+ 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c)$ $\qquad$ $c = 2.503 \cdot in$ $\qquad$ $a := \text{if}(\beta 1 \cdot c > h, h, \beta 1 \cdot c)$ $\qquad$ $a = 2.127 \cdot in$

Find Moment:

77/

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$

$fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$

$Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$d_i =$

0.26
0.573

ft

$\varepsilon s_i =$

-0.001
-0.005

$fs_i =$

-21.639
-40

$\cdot ksi$

$A_i =$

1.056
1.056

$\cdot in$

$Fs_i =$

-22.851
-42.24

$\cdot kips$

$Ms_i =$

-42.846
79.2

$\cdot kips \cdot in$

$Mst := \sum_{i=1}^{row} Ms_i$

$Mst = 36 \cdot kips \cdot in$

$Mc := 0.85 \cdot f'c \cdot a \cdot w \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$

$Mc = 256 \cdot kips \cdot in$

$Mn_7 := Mc + Mst$

$Mn_7 = 293 \cdot kips \cdot in$

$Pn_8 := -fy \cdot Ast = -84.48 \cdot kips$

$Mn_8 := 0$

78/437

$j := 0..8$ $\quad \phi_j := 1$

Results

$\phi_j \cdot Pn_j =$		$\phi_j \cdot Mn_j =$	
308.076	·kip	0	·kips·in
308.076		318.666	
239.577		405.988	
189.371		475.56	
162.094		492.066	
108.477		499.409	
36		371.257	
0		292.58	
-84.48		0	

Criteria:

Depth	$h = 10 \cdot in$	$bar = 6$
Width	$w = 12 \cdot in$	$tie = 2$
Eff. Width	$b = 12 \cdot in$	$fy = 40 \cdot ksi$

$cover = 2.5 \cdot in$ $\quad f'c = 3000 \cdot psi$

$Ag = 120 \cdot in^2$ $\quad Ast = 2.112 \cdot in^2$

$\frac{Ast}{w \cdot h} = 0.0176$ Actual ratio ρ

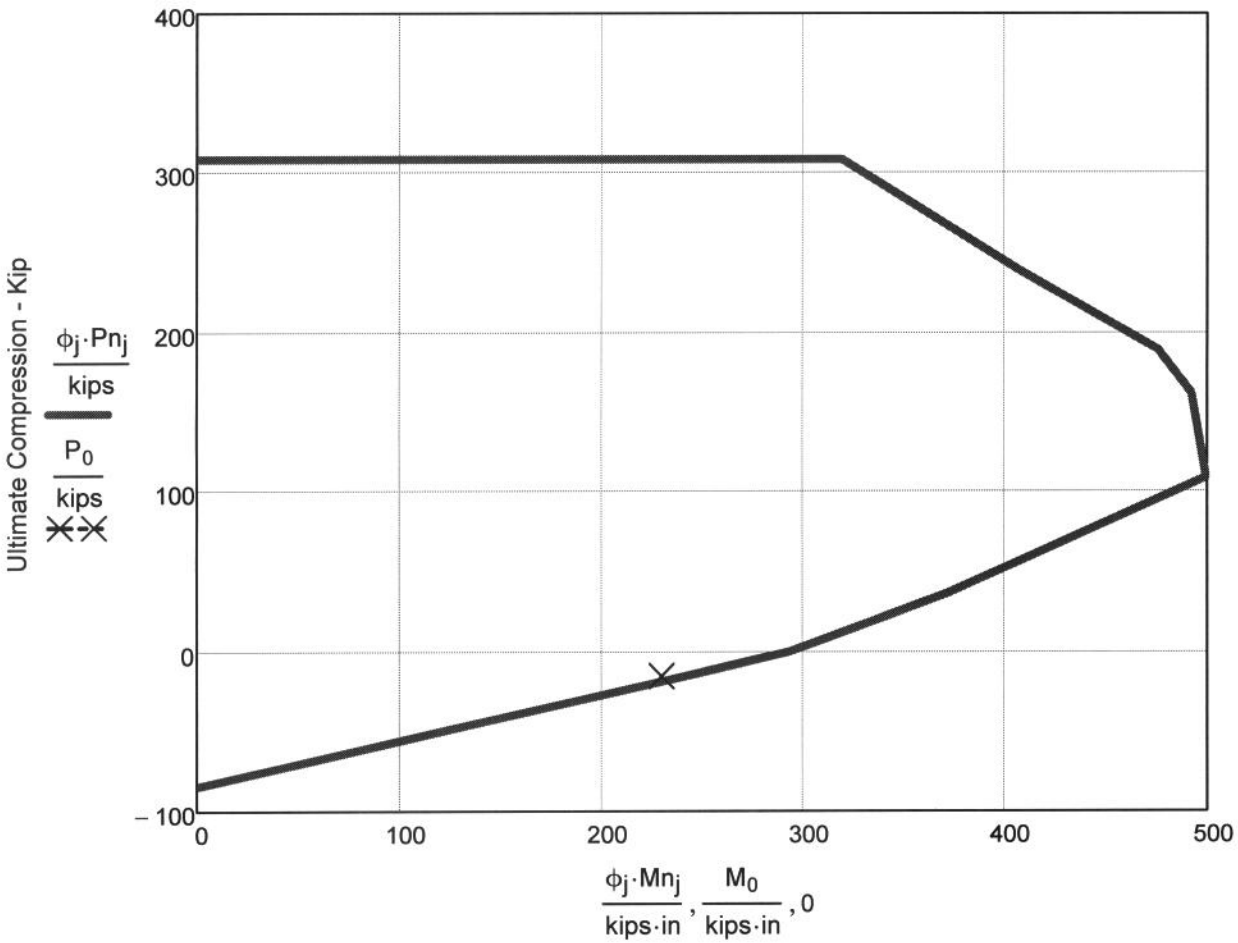
$\frac{Ast}{Ag} = 0.0176$ Eff. ratio ρ

79/437

Demand Loads Water to Top of Wall:

Axial Load from CPGA output: $P_0 := -14.7kips$ Moment from CPGA output: $M_0 := 229.5kips \cdot in$

$V_{max} := 7kips$



Interaction Diagram

Nominal Curve Used to Find Factor of Safety:

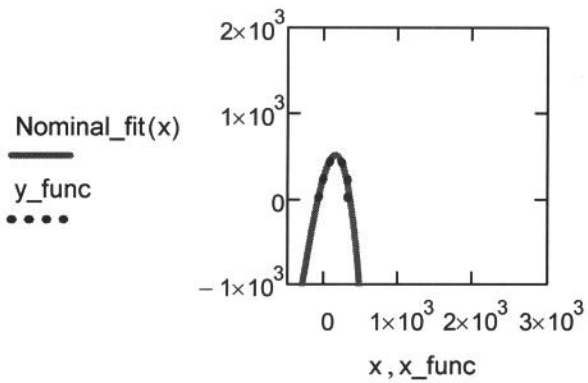
80/437

Data Points set to function: $y_func := \frac{Mn}{kips \cdot in}$ $x_func := \frac{Pn}{kips}$

Degree of Polynominal Fit: $k := 3$

Returns a vector which interp uses to find the kth order polynomial that best fits the x and y data values

Nominal := regress(x_func, y_func, k) Nominal_fit(x) := interp(Nominal, x_func, y_func, x)



Allowable Axial Capacity (geotech):
(assuming minimum of two 2.5' long sheet piles engaged for pullout).

Allowable Axial Compression Load for 15ft sheet pile (To be calculated by Geotechs)

$$P_c := \frac{20.73kips}{1ft}\left(\frac{5 \cdot ft}{4}\right) = 25.912 \cdot kips$$

Allowable Axial Tension Load for 15ft sheet pile (To be calculated by Geotechs)

$$P_t := \frac{-11.231kips}{1ft}\left(\frac{5ft}{4}\right) = -14.039 \cdot kips$$

Tension Pile Geotech FS:

$$FS_{GT} := \frac{P_t}{P_0} = 0.955$$

FS_{GT} Must be > 1.7 to meet Feasibility Criteria without necessitating R&R analysis.

8//

Strength Factor of Safety:

Read the Corresponding Moment for the demand axial load ($P_0 = -14.7 \cdot kips$)

$M_{capacity} := Nominal_fit\left(\frac{P_0}{kips}\right)$ $\qquad M_{capacity} = 244.675$

Demand Moment: $\qquad M_{demand} := \frac{M_0}{kips \cdot in}$ $\qquad M_{demand} = 229.5$

Factor of Safety: $\qquad FS_{strength} := \frac{M_{capacity}}{M_{demand}}$ $\qquad FS_{strength} = 1.07$

Check Shear Through Pile:

d for pile: $\qquad d_{sht} := 6.875 \cdot in$

$$V_c := 2\left(1 + \frac{\frac{P_0}{lb}}{500 \cdot \frac{h \cdot w}{in^2}}\right) \cdot \sqrt{f'c \cdot psi} \cdot w \cdot d_{sht} = 6.823 \cdot kips \qquad V_s := \frac{A_v \cdot fy \cdot d_{sht}}{s_v} = 1.687 \cdot kips$$

$V_n := V_c + V_s = 8.511 \cdot kips$ $\qquad FS_{shear} := \frac{V_n}{V_{max}} = 1.216$

82/437

FEMA 310 GUIDANCE

4.2.4.4 Component Strength

Component strength for all actions shall be taken as the expected strength, Q_{CE}. Unless calculated otherwise, the expected strength shall be assumed equal to the nominal strength multiplied by 1.25. Alternatively, if allowable stresses are used, nominal strengths shall be taken as the allowable values multiplied by the following values:

Steel	1.7
Masonry	2.5
Wood	2.0

Except for wood diaphragms and wood and masonry shear walls, the allowable values shall not include a one-third increase for short term loading.

When calculating capacities of deteriorated elements, the evaluating design professional shall make

2011 REVISION

83/437

CID MO 18"x18" (34') Piles

Comp by: Sheffield
Chkd by: Marx 9/28/11

Units:

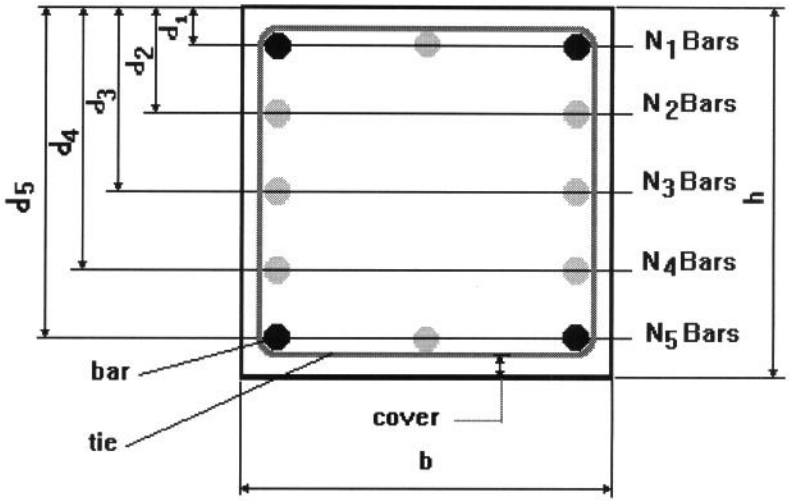
$kips := 1000lb$ $psf := \frac{lb}{ft^2}$

$psi := \frac{lb}{in^2}$

$ton := 2000lb$ $tsf := \frac{ton}{ft^2}$ $klf := \frac{kips}{ft}$ $plf := \frac{lb}{ft}$ $pcf := \frac{lb}{ft^3}$ $ksi := \frac{kips}{in^2}$

Note: The file has been used by many Engineers on previous projects who have also verified the results with independent programs. It was not closely re-examined for a Reliability Analysis.

This program calulates the column interaction curve for a rectangular concrete column using the methods of the Concrete Reinforcing Steel Institute. Note this program also accounts for less than 1 percent steel by reducing the width. Procedure based on CRSI Manual Pg 3-5 to Pg 3-8



$h := 18in$	Column Depth	$cover := 2.25in$	Cover -in	
$w := 18in$	Column Width	$N_1 := 3$	Bars in Row 1	
$bar := 7$	Bar Size	$N_2 := 2$	Bars in Row 2	
$tie := 2$	Tie Size	$N_3 := 3$	Bars in Row 3	
$f'c := 3ksi$	Conc. Str. - ksi	$N_4 := 0$	Bars in Row 4	
$fy := 40ksi$	Bar Yield - ksi	$N_5 := 0$	Bars in Row 5	$row \equiv 3$

Total Rows of Bars- NOTE: This can be changed and more rows added if required.

Modulus of Elasticty: $Es := 29000ksi$

Concrete Strength: $f'c := 3000psi$

FEMA 310 Allowed for concrete aging: $FEMA_Factor := 1.00$

Nominal Concrete Strength assumed orginally used: $f'c := f'c \cdot FEMA_Factor$ $f'c = 3000 \cdot psi$

NOTE - if FEMA factor is 1.00 then concrete strength is the nominal concrete strength. If trying to get the expected concrete strength the FEMA factor should be 1.25. See FEMA 310 guidance.

$areabar \equiv (0in^2 \;\; 0in^2 \;\; .05in^2 \;\; 0.11in^2 \;\; 0.2in^2 \;\; 0.31in^2 \;\; 0.44in^2 \;\; 0.6in^2 \;\; 0.79in^2 \;\; 1in^2 \;\; 1.27in^2 \;\; 1.56in^2 \;\; 0in^2 \;\; 0in^2 \;\; 2.25in^2)$

84/

$diabar \equiv (0in \;\; 0in \;\; .25in \;\; 0.38in \;\; 0.5in \;\; 0.63in \;\; 0.75in \;\; 0.88in \;\; 1in \;\; 1.13in \;\; 1.27in \;\; 1.41in \;\; 0in \;\; 0in \;\; 1.69in)$

Calculate effective stress block per ACI 318 sect. 10.2.7.3 $\beta 1 := if\left[f'c \le 4ksi, 0.85, if\left[f'c > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'c}{ksi} - 4\right)\right]\right]$ $\beta 1 = 0.85$

Iterations $i := 1 .. row$

Bar Diameter $db := diabar_{0,bar}$ $db = 0.88\cdot in$

Bar area at i $A_i := N_i\cdot areabar_{0,bar}$

Tie Diameter $dt := diabar_{0,tie}$ $dt = 0.250\cdot in$

Distances from compression face $d_1 := cover + \frac{db}{2} + dt$ $d_1 = 2.94\cdot in$

Spacing between rows of bars $space := \frac{h - 2\cdot d_1}{row - 1}$ $space = 6.06\cdot in$ $d_i := d_1 + (i - 1)\cdot space$

$i =$	$N_i =$	$A_i =$	$d_i =$
1	3	1.8 $\cdot in^2$	2.94 $\cdot in$
2	2	1.2	9
3	3	1.8	15.06

Gross Area: $Ag := w\cdot h$ $Ag = 324\cdot in^2$

Steel Area $Ast := \sum_{i=1}^{row} A_i$ $Ast = 4.8\cdot in^2$

Reduce width if $\rho < 0.01$: $\frac{Ast}{Ag} = 0.0148$ $b := if\left(\frac{Ast}{Ag} > 0.01, w, \frac{Ast}{0.01\cdot h}\right)$

Effective Width $b = 18\cdot in$

$Ag := b\cdot h$ $Ag = 324\cdot in^2$

$\frac{Ast}{Ag} = 0.0148$

Calculate Axial Load at no eccentricity, e (min) at ϕPn(max):

85/437

$Pn_1 := 0.8\cdot[0.85\cdot f'c\cdot(Ag - Ast) + fy\cdot Ast]$ $\quad Pn_1 = 805\cdot kips$

Equate as the first set of points on the interaction diagram: $\quad Pn_0 := Pn_1 \quad Pn_0 = 804.768\cdot kips \quad Mn_0 := 0kips\cdot in$

Axial Compression with Aci Reduction value: $\quad \phi := 1 \quad \phi\cdot Pn_1 = 805\cdot kips$

Assume c=h $\quad c := h \quad c = 18\cdot in$

Given

$$Pn_1 = \sum_{i=1}^{row}\left[\left[\left(c - d_i\right)\cdot\frac{0.003}{c}\cdot Es - if\left(d_i < \beta1\cdot c, 0.85\cdot f'c, 0\right)\right]\cdot A_i\right] + 0.85\cdot f'c\cdot\beta1\cdot c\cdot b \qquad Pn_1 = 804.768\cdot kips$$

$c := Find(c) \quad c = 16.1\cdot in$

Calculate Width of Compression block: $\quad a := if(\beta1\cdot c > h, h, \beta1\cdot c) \quad a = 13.687\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right)\cdot\frac{0.003}{c}$ $\quad \varepsilon s_i =$

0.002
0.001
0

$fs_i := if\left(Es\cdot\varepsilon s_i > fy, fy, if\left(Es\cdot\varepsilon s_i < 0 - fy, 0 - fy, Es\cdot\varepsilon s_i\right)\right)$ $\quad fs_i =$

40	$\cdot ksi$
38.373	
5.631	

86/

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $Fs_i =$

67.41
42.988
10.136

$\cdot kips$

$Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$ $Ms_i =$

408.505
0
-61.423

$\cdot kips \cdot in$

$Mst := \sum_{i=1}^{row} Ms_i$ $Mst = 347.082 \cdot kips \cdot in$ $Fst := \sum_{i=1}^{row} Fs_i$ $Fst = 120.533 \cdot kips$

$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $Mc = 112.901 \cdot kips \cdot ft$ $Fc := 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$ $Fc = 628.227 \cdot kips$

$Mn_1 := Mc + Mst$ $Mn_1 = 1701.896 \cdot kips \cdot in$ $e_1 := \frac{Mn_1}{Pn_1}$ $e_1 = 2.11 \cdot in$

$d_{row} = 15.06 \cdot in$

87/437

Find Pn and Mn at point of zero tension in bars:

$c := d_{row}$ $\quad c = 15.06 \cdot in$ $\quad a := \beta 1 \cdot c$ $\quad a = 12.801 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$i =$	$\varepsilon s_i =$	$fs_i =$	$Fs_i =$	$Ms_i =$	$A_i =$	$d_i =$
1	0.002	40 $\cdot ksi$	67.41 $\cdot kips$	408.505 $\cdot kips \cdot in$	1.8 $\cdot in^2$	2.94 $\cdot in$
2	0.001	38.373	42.988	0	1.2	9
3	0	5.631	10.136	-61.423	1.8	15.06

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 347.082 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 1527.378 \cdot kips \cdot in$

$Mn_2 := Mc + Mst$ $\quad Mn_2 = 1874.46 \cdot kips \cdot in$ $\quad Mn_2 = 1874 \cdot kips \cdot in$

$Mn_1 = 1701.896 \cdot kips \cdot in$ $\quad Mn_1 := if(Mn_1 < Mn_2, Mn_1, Mn_2)$ $\quad Mn_1 = 1701.896 \cdot kips \cdot in$

$Pn_2 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_2 = 708 \cdot kips$ $\quad Pn_1 = 804.768 \cdot kips$ $\quad Pn_2 := if(Pn_2 < Pn_1, Pn_2, Pn_1)$ $\quad Pn_2 = 708.099 \cdot kips$

88/

Find Pn and Mn at point of 0.25 *fy tension in bars:

$$c := \frac{d_{row}}{1 + \frac{0.25 \cdot fy}{Es \cdot 0.003}} \qquad d_{row} = 15.06 \cdot in \qquad a := \beta 1 \cdot c$$

Find Moment:

$$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c} \qquad fs_i := if\left(Es \cdot \varepsilon s_i > fy, fy, if\left(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i\right)\right)$$

$$Fs_i := fs_i \cdot A_i - if\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$$

$d_i =$

0.245	ft
0.75	
1.255	

$\varepsilon s_i =$

0.002
0.001
-0

$fs_i =$

40	$\cdot ksi$
29.032	
-10	

$A_i =$

1.8	$\cdot in^2$
1.2	
1.8	

$Fs_i =$

67.41	$\cdot kips$
31.778	
-18	

$Ms_i =$

408.505	$\cdot kips \cdot in$
0	
109.08	

$$Mst := \sum_{i=1}^{row} Ms_i \qquad Mst = 518 \cdot kips \cdot in \qquad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right) \qquad Mc = 1717.649 \cdot kips \cdot in \qquad Mn_3 := Mc + Mst \qquad Mn_3 = 2235 \cdot kips \cdot in$$

$$Pn_3 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b \qquad Pn_3 = 608 \cdot kips$$

Find Pn and Mn at point of 0.50 *fy tension in bars:

89/437

$d_{row} = 15.06 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{0.50 \cdot fy}{Es \cdot 0.003}}$ $\quad c = 12.245 \cdot in$ $\quad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$

0.245	ft
0.75	
1.255	

$\varepsilon s_i =$

0.002
0.001
-0.001

$fs_i =$

40	$\cdot ksi$
23.056	
-20	

$A_i =$

1.8	$\cdot in^2$
1.2	
1.8	

$Fs_i =$

67.41	$\cdot kips$
24.607	
-36	

$Ms_i =$

408.505	$\cdot kips \cdot in$
0	
218.16	

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 627 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 1813.434 \cdot kips \cdot in$ $\quad Mn_4 := Mc + Mst$ $\quad Mn_4 = 2440 \cdot kips \cdot in$

$Pn_4 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_4 = 534 \cdot kips$

90/437

Find Pn and Mn at Balanced:

$d_{row} = 15.06 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{fy}{Es \cdot 0.003}}$ $\quad c = 10.317 \cdot in$ $\quad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$

0.245	ft
0.75	
1.255	

$\varepsilon s_i =$

0.002
0
-0.001

$fs_i =$

40	$\cdot ksi$
11.104	
-40	

$A_i =$

1.8	$\cdot in^2$
1.2	
1.8	

$Fs_i =$

67.41	$\cdot kips$
13.324	
-72	

$Ms_i =$

408.505	$\cdot kips \cdot in$
0	
436.32	

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 845 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 1857.727 \cdot kips \cdot in$ $\quad Mn_5 := Mc + Mst$ $\quad Mn_5 = 2703 \cdot kips \cdot in$

$Pn_5 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_5 = 411 \cdot kips$

91/

Find Mn at ϕ Pn = 0.1*f'c*Ag:

$Pn_6 := \frac{0.1 \cdot f'c \cdot w \cdot h}{\phi}$ $\quad Pn_6 = 97.2 \cdot kips$ $\quad Pn_6 := if(Pn_6 > Pn_5, Pn_5, Pn_6)$ $\quad Pn_6 = 97.2 \cdot kips$

Assume c=h/2 $\quad c := h \cdot 0.5$ $\quad c = 9 \cdot in$

Given

$$Pn_6 = \sum_{i=1}^{row} \left[\left[\begin{cases} (c - d_i) \cdot \frac{0.003}{c} \cdot Es \ldots + -if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es < fy \\ fy - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \geq fy \\ -fy - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \leq -fy \end{cases} \right] \cdot A_i \right] \ldots + 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c)$ $\quad c = 4.37 \cdot in$

$a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c)$ $\quad a = 3.72 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
2.94 $\cdot in$	0.001	28479.727 $\cdot psi$	1.8 $\cdot in^2$	46.674 $\cdot kips$	282.841 $\cdot in \cdot kips$
9	-0.003	-40000	1.2	-48	0
15.06	-0.007	-40000	1.8	-72	436.32

$$Mst := \sum_{i=1}^{row} Ms_i$$

$Mst = 719 \cdot kips \cdot in$

92/437

$$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$$

$Mc = 1217.971 \cdot kips \cdot in$

$Mn_6 := Mc + Mst$

$Mn_6 = 1937 \cdot kips \cdot in$

$Pn_6 = 97 \cdot kips$

93/437

Find Mn at ϕ Pn = 0.0:

$Pn_7 := 0lb$ $\qquad$ $Pn_7 = 0lb$

Assume c=0.2h $\qquad$ $c := 0.2 \cdot h$

Given

$$Pn_7 = \sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es < fy \\ + -\text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \\ fy - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \geq fy \\ -fy - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \leq -fy \end{array} \right] \cdot A_i \right] \ldots$$

$$+ 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c)$ $\qquad$ $c = 2.998 \cdot in$ $\qquad$ $a := \text{if}(\beta 1 \cdot c > h, h, \beta 1 \cdot c)$ $\qquad$ $a = 2.548 \cdot in$

Find Moment:

94/

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$

$fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$

$Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$d_i =$

0.245	ft
0.75	
1.255	

$\varepsilon s_i =$

0
-0.006
-0.012

$fs_i =$

1.684	$\cdot ksi$
-40	
-40	

$A_i =$

1.8	$\cdot in^2$
1.2	
1.8	

$Fs_i =$

3.032	$\cdot kips$
-48	
-72	

$Ms_i =$

18.371	$\cdot kips \cdot in$
0	
436.32	

$Mst := \sum_{i=1}^{row} Ms_i$

$Mst = 455 \cdot kips \cdot in$

$Mc := 0.85 \cdot f'c \cdot a \cdot w \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$

$Mc = 904 \cdot kips \cdot in$

$Mn_7 := Mc + Mst$

$Mn_7 = 1358 \cdot kips \cdot in$

95/437

$j := 0..7$ $\phi_j := 1$

Results

$\phi_j \cdot Pn_j =$		$\phi_j \cdot Mn_j =$	
804.768	·kip	0	·kips·in
804.768		1701.896	
708.099		1874.46	
608.18		2235.234	
533.757		2440.098	
411.24		2702.552	
97.2		1937.132	
0		1358.37	

Criteria:

Depth	$h = 18 \cdot in$	$bar = 7$
Width	$w = 18 \cdot in$	$tie = 2$
Eff. Width	$b = 18 \cdot in$	$fy = 40 \cdot ksi$

$cover = 2.25 \cdot in$ $f'c = 3000 \cdot psi$

$Ag = 324 \cdot in^2$ $Ast = 4.8 \cdot in^2$

$\frac{Ast}{w \cdot h} = 0.0148$ Actual ratio ρ

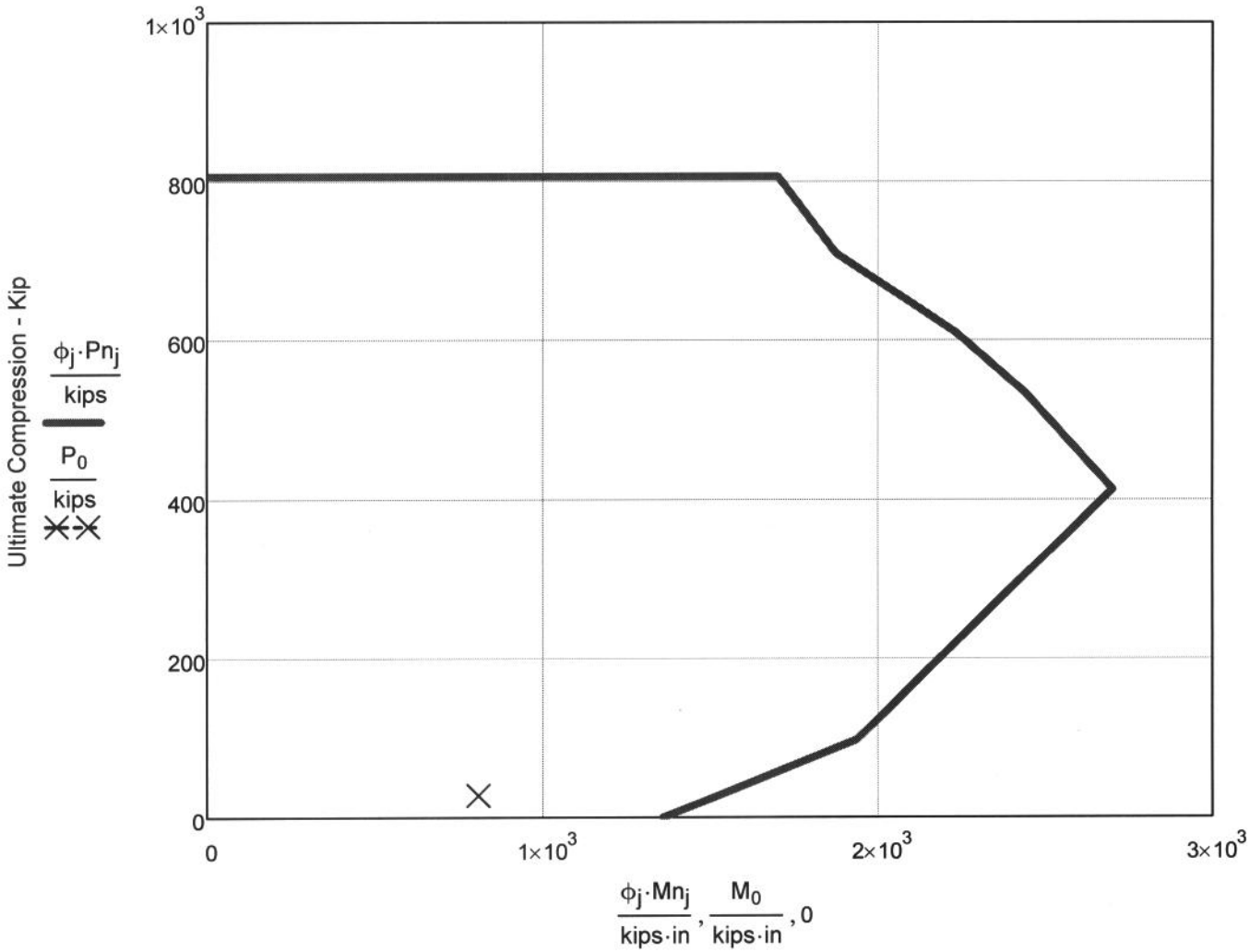
$\frac{Ast}{Ag} = 0.0148$ Eff. ratio ρ

96/

Demand Loads Water to Top of Wall:

Axial Load from CPGA output: $P_0 := 28.2kips$ Moment from CPGA output: $M_0 := 807.7kips \cdot in$

$P_{0GT} := 61.9kips$ $V_{max} := 16.3kips$



Interaction Diagram

97/437

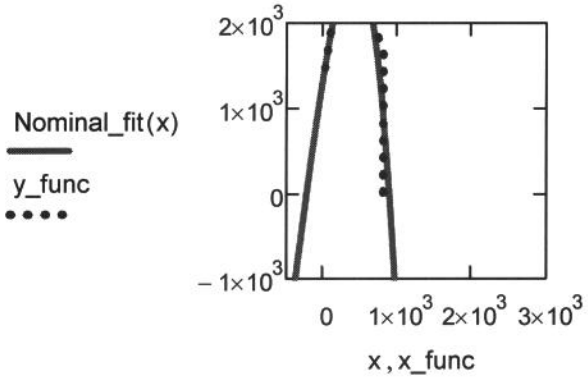
Nominal Curve Used to Find Factor of Safety:

Data Points set to function: $y_func := \frac{Mn}{kips \cdot in}$ $x_func := \frac{Pn}{kips}$

Degree of Polynominal Fit: $k := 3$

Returns a vector which interp uses to find the kth order polynomial that best fits the x and y data values

$Nominal := regress(x_func, y_func, k)$ $Nominal_fit(x) := interp(Nominal, x_func, y_func, x)$



Allowable Axial Capacity (geotech):

Allowable Axial Compression Load for 33ft pile (To be calculated by Geotechs)

$P_c := 137.963kips$

Allowable Axial Tension Load for 339ft pile (To be calculated by Geotechs)

$P_t := 73.264kips$

Compression Pile Geotech FS:

$FS_{GT} := \frac{P_c}{P_{0GT}} = 2.229$

FS_{GT} Must be > 1.7 to meet Feasibility Criteria without necessitating R&R analysis.

98/

Strength Factor of Safety:

Read the Corresponding Moment for the demand axial load ($P_0 = 28.2 \cdot kips$)

$M_{capacity} := Nominal_fit\left(\frac{P_0}{kips}\right)$ $\quad M_{capacity} = 1542.921$

Demand Moment: $M_{demand} := \frac{M_0}{kips \cdot in}$ $\quad M_{demand} = 807.7$

Factor of Safety: $FS_{strength} := \frac{M_{capacity}}{M_{demand}}$ $\quad FS_{strength} = 1.9$

Check Shear through pile (conservatively ignoring shear reinforcing):

d for pile: $d_{pile} := 14.8in$

$V_{Pile} := 2 \cdot \sqrt{f'c \cdot psi} \cdot w \cdot d_{pile} = 29.183 \cdot kips$

$FS_{shear} := \frac{V_{Pile}}{V_{max}} = 1.79$

FEMA 310 GUIDANCE

4.2.4.4 Component Strength

Component strength for all actions shall be taken as the expected strength, Q_{CE}. Unless calculated otherwise, the expected strength shall be assumed equal to the nominal strength multiplied by 1.25. Alternatively, if allowable stresses are used, nominal strengths shall be taken as the allowable values multiplied by the following values:

Steel	1.7
Masonry	2.5
Wood	2.0

Except for wood diaphragms and wood and masonry shear walls, the allowable values shall not include a one-third increase for short term loading.

When calculating capacities of deteriorated elements, the evaluating design professional shall make

2011 REVISION

KPD/437

US Army Corps of Engineers

CID-MO Floodwall
Floodwall Analysis - Existing, M145
4 foot Stop Log Abutment Monolith
Kansas City Levees

Comp by: Sheffield
Chkd by: Marx 9/28/11

Water EL: TOW - 1'

Assumptions: For Feasibility Analysis, it is assumed that the sheetpile cutoff is adequately strength designed for a 50% head differential, the pile cap under the closure is adequately designed, and that the floodwall sections of the monolith are accounted for by analysis of adjacent floodwall monoliths. Soil Surfaces assumed horizontal.

Variables

$kip := 1000lb$

$psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$ $plf := \frac{lb}{ft}$

$ksf \equiv \frac{1000lb}{ft^2}$ $psf \equiv \frac{lb}{ft^2}$

$EL_{top_ftg} := 746.9ft$

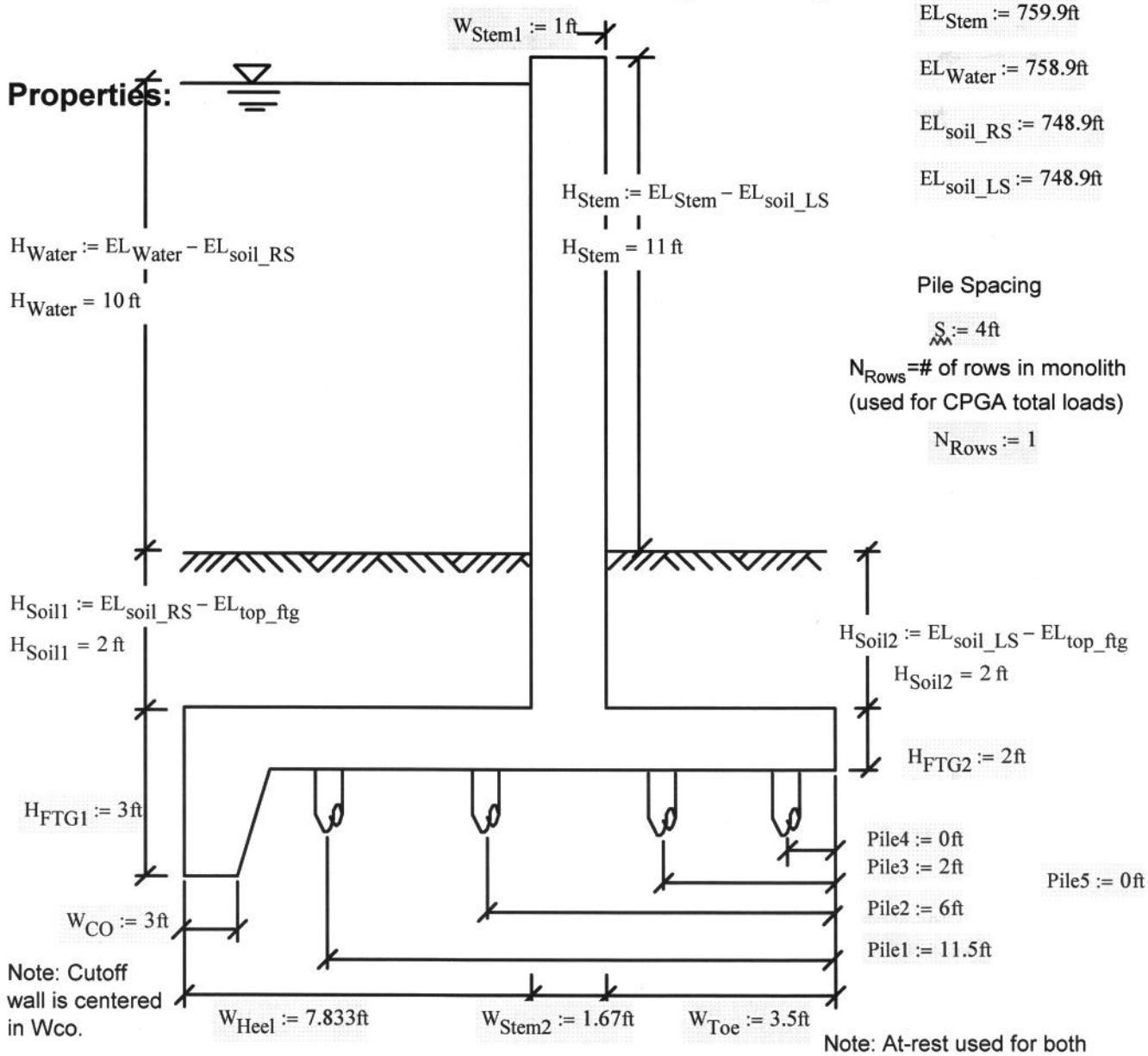
$EL_{Stem} := 759.9ft$

$EL_{Water} := 758.9ft$

$EL_{soil_RS} := 748.9ft$

$EL_{soil_LS} := 748.9ft$

Properties:



Pile Spacing

$S := 4ft$

N_{Rows}=# of rows in monolith (used for CPGA total loads)

$N_{Rows} := 1$

Note: Cutoff wall is centered in Wco.

Concrete Properties $\gamma_c := 150pcf$

Water Weight $\gamma_w := 62.4pcf$

Soil Properties $\gamma_{sat} := 115pcf$ $\phi := 22deg$

$K_o := 1 - \sin(\phi)$ $K_o = 0.63$

Note: At-rest used for both driving and resisting for Feasibility and to require piles to fully resist stability.

181/437

Lengths:

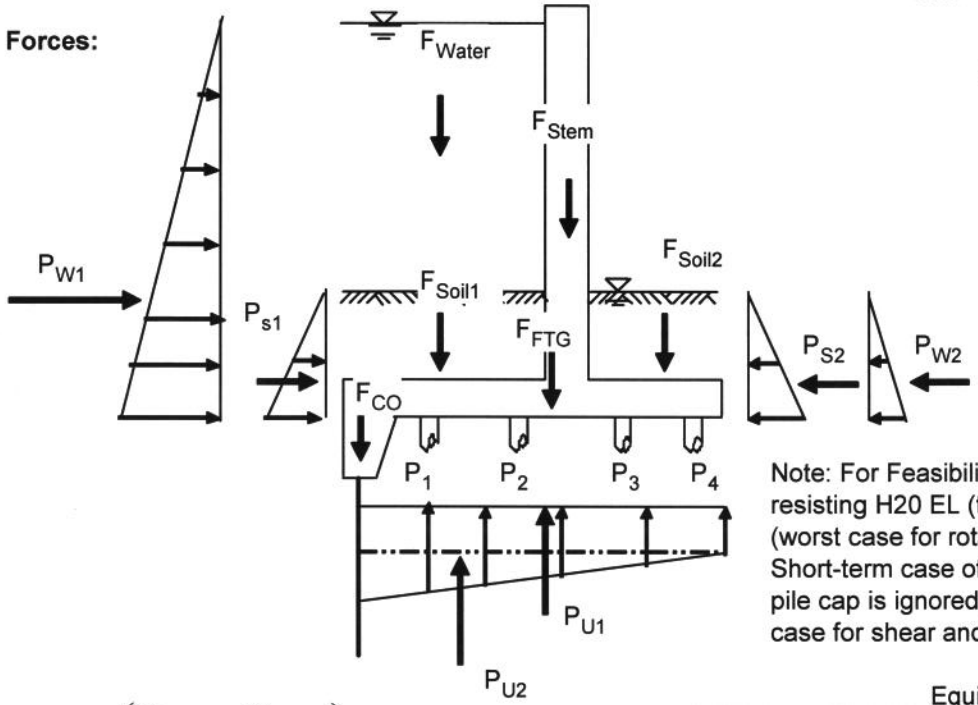
$L_{CO} := H_{FTG1} - H_{FTG2}$ $\quad L_{W1} := H_{Water} + H_{Soil1} + H_{FTG1}$ $\quad L_{STM} := H_{Stem} + H_{Soil2}$

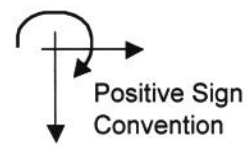
$L_{FTG} := W_{Heel} + W_{Stem2} + W_{Toe}$ $\quad L_{PU1} := H_{Soil2} + H_{FTG2}$ $\quad L_{FTG} = 13\,ft$

$L_{W1} = 15\,ft$

$L_{PU1} = 4\,ft$

Forces:





Note: For Feasibility, only long-term case of resisting H20 EL (top of soil) is considered (worst case for rotation and pile pull-out). Short-term case of little or no resisting H20 on pile cap is ignored, though this is the worst case for shear and sliding.

$F_{Stem} := \left(\frac{W_{Stem1} + W_{Stem2}}{2}\right) \cdot L_{STM} \cdot \gamma_c = 2603.25 \cdot plf$

$F_{closure} := 6.825 \frac{kip}{ft}$ Equivalent Point Load acting on abutment from Stop Logs per tributary width of pile cap.

$F_{closure}$ = 1/2*.0624*10²*17.5/2*(1/4ft)

$F_{FTG} := H_{FTG2} \cdot L_{FTG} \cdot \gamma_c = 3900.9 \cdot plf$

$F_{CO} := L_{CO} \cdot W_{CO} \cdot \gamma_c = 450 \cdot plf$ (Treated Rectangular)

$F_{Soil1} := H_{Soil1} \cdot W_{Heel} \cdot (\gamma_{sat} - \gamma_w) = 824.03 \cdot plf$

$F_{Soil2} := H_{Soil2} \cdot W_{Toe} \cdot (\gamma_{sat} - \gamma_w) = 368.2 \cdot plf$

$F_{Water1} := (H_{Water} + H_{Soil1}) \cdot W_{Heel} \cdot \gamma_w = 5865.35 \cdot plf$

$F_{Water2} := H_{Soil2} \cdot W_{Toe} \cdot \gamma_w = 436.8 \cdot plf$

$P_{W1} := \frac{1}{2} \cdot L_{W1}^2 \cdot \gamma_w = 7020 \cdot plf$

$P_{W2a} := -\left[\frac{1}{2} \cdot (L_{PU1})^2 \cdot \gamma_w\right] = -499.2 \cdot plf$

Cutoff wall assumed to be 50% effective. See Figure C-3 of EM 1110-2-2100

$q_{uCO} := (1 - 0.5) \cdot \gamma_w \left[(H_{Water} + H_{Soil1} + H_{FTG2}) - L_{PU1}\right] + L_{PU1} \cdot \gamma_w = 561.6 \cdot psf$

$q_{umax} := \left[(1 - 0.5)(L_{W1} - L_{PU1})\gamma_w\right] + L_{PU1} \cdot \gamma_w = 592.8 \cdot psf$

$q_{utoe} := L_{PU1} \cdot \gamma_w + \frac{L_{PU1}}{L_{FTG} - .5W_{CO} + L_{PU1}} (q_{umax} - L_{PU1} \cdot \gamma_w) = 338.15 \cdot psf$

Note: Soil below base of pile cap acting on key is essentially equal and opposite since all resistance is by piles.

$P_{W2b} := -\left[\frac{1}{2} \cdot (q_{uCO} + q_{umax}) \cdot L_{CO}\right] = -577.2 \cdot plf$

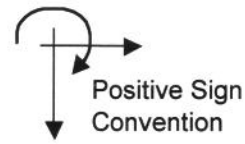
$P_{S1} := K_o \cdot \left[\frac{1}{2} \cdot (H_{Soil1} + H_{FTG2})^2 \cdot (\gamma_{sat} - \gamma_w)\right] = 263.17 \cdot plf$ $\quad P_{S2} := -K_o \cdot \left[\frac{1}{2} \cdot (H_{Soil2} + H_{FTG2})^2 \cdot (\gamma_{sat} - \gamma_w)\right] = -263.17 \cdot plf$

$P_{U1} := -(L_{FTG} - .5W_{CO}) \cdot (q_{utoe}) = -3889.75 \cdot plf$ $\quad P_{U2} := -0.5(L_{FTG} - .5 \cdot W_{CO}) \cdot (q_{umax} - q_{utoe}) = -1464.62 \cdot plf$

$P_{U3} := -.5W_{CO} \cdot (L_{W1} \cdot \gamma_w) = -1404 \cdot plf$

Inputs for CPGA

Assumes perfectly rigid pile cap.



Sum moments around bottom corner of Toe:

Moment Arm (from bottom corner of Toe)

$$Arm_{Stem} := \left(W_{Toe} + \frac{W_{Stem2}}{2}\right) = 4.34\,ft$$

$$Arm_{FTG} := \frac{L_{FTG}}{2} = 6.5\,ft$$

$$Arm_{CO} := L_{FTG} - \frac{W_{CO}}{2} = 11.5\,ft$$

$$Arm_{closure} := 4\cdot ft + \frac{H_{Water}}{3} = 7.33\,ft$$

$$Arm_{Soil1} := \left(W_{Toe} + W_{Stem2} + 0.5W_{Heel}\right) = 9.09\,ft$$

$$Arm_{Soil2} := 0.5W_{Toe} = 1.75\,ft$$

$$Arm_{Water1} := Arm_{Soil1} = 9.09\,ft$$

$$Arm_{Water2} := Arm_{Soil2} = 1.75\,ft$$

$$Arm_{W1} := \frac{1}{3}\cdot L_{W1} - L_{CO} = 4\,ft$$

$$Arm_{W2a} := 0.333\cdot\left(L_{PU1}\right) = 1.33\,ft$$

$$Arm_{W2b} := L_{CO} - \frac{L_{CO}\cdot\left(2q_{uCO} + q_{umax}\right)}{3\left(q_{uCO} + q_{umax}\right)} = 0.5\,ft$$

$$Arm_{S1} := \frac{1}{3}\cdot\left(H_{Soil1} + H_{FTG2}\right) = 1.33\,ft$$

$$Arm_{S2} := Arm_{W2a} = 1.33\,ft$$

$$Arm_{U1} := \frac{L_{FTG} - 0.5W_{CO}}{2} = 5.75\,ft$$

$$Arm_{U2} := \frac{2\left(L_{FTG} - 0.5W_{CO}\right)}{3} = 7.67\,ft$$

$$Arm_{U3} := L_{FTG} - 0.25\cdot W_{CO} = 12.25\,ft$$

Moment (about bottom corner of toe)

$$M_{Stem} := -F_{Stem}\cdot Arm_{Stem} = -11.29\cdot\frac{kip\cdot ft}{ft}$$

$$M_{FTG} := -F_{FTG}\cdot Arm_{FTG} = -25.36\cdot\frac{kip\cdot ft}{ft}$$

$$M_{CO} := -F_{CO}\cdot Arm_{CO} = -5.18\cdot\frac{kip\cdot ft}{ft}$$

$$M_{closure} := F_{closure}\cdot Arm_{closure} = 50.05\cdot\frac{kip\cdot ft}{ft}$$

$$M_{Soil1} := -F_{Soil1}\cdot Arm_{Soil1} = -7.49\cdot\frac{kip\cdot ft}{ft}$$

$$M_{Soil2} := -F_{Soil2}\cdot Arm_{Soil2} = -0.64\cdot\frac{kip\cdot ft}{ft}$$

$$M_{Water1} := -F_{Water1}\cdot Arm_{Water1} = -53.3\cdot\frac{kip\cdot ft}{ft}$$

$$M_{Water2} := -F_{Water2}\cdot Arm_{Water2} = -0.76\cdot\frac{kip\cdot ft}{ft}$$

$$M_{W1} := P_{W1}\cdot Arm_{W1} = 28.08\cdot\frac{kip\cdot ft}{ft}$$

$$M_{W2a} := P_{W2a}\cdot Arm_{W2a} = -0.66\cdot\frac{kip\cdot ft}{ft}$$

$$M_{W2b} := -P_{W2b}\cdot Arm_{W2b} = 0.29\cdot\frac{kip\cdot ft}{ft}$$

$$M_{S1} := P_{S1}\cdot Arm_{S1} = 0.35\cdot\frac{kip\cdot ft}{ft}$$

$$M_{S2} := P_{S2}\cdot Arm_{S2} = -0.35\cdot\frac{kip\cdot ft}{ft}$$

$$M_{U1} := -P_{U1}\cdot Arm_{U1} = 22.37\cdot\frac{kip\cdot ft}{ft}$$

$$M_{U2} := -P_{U2}\cdot Arm_{U2} = 11.23\cdot\frac{kip\cdot ft}{ft}$$

$$M_{U3} := -P_{U3}\cdot Arm_{U3} = 17.2\cdot\frac{kip\cdot ft}{ft}$$

$$\Sigma M := \begin{pmatrix} M_{Stem} + M_{FTG} + M_{CO} + M_{closure} + M_{Soil1} + M_{Soil2} + M_{Water1} + M_{Water2} + M_{W1} + M_{W2a} \cdots \\ + M_{W2b} + M_{S1} + M_{S2} + M_{U1} + M_{U2} + M_{U3} \end{pmatrix}$$

$$\Sigma Vertical := \begin{pmatrix} F_{Stem} + F_{FTG} + F_{CO} + F_{Soil1} + F_{Soil2} + F_{Water1} + F_{Water2} \cdots \\ + P_{U1} + P_{U2} + P_{U3} \end{pmatrix}$$

$$\Sigma Horizontal := \left(F_{closure} + P_{W1} + P_{W2a} + P_{W2b} + P_{S1} + P_{S2}\right)$$

$$\Sigma M = 24.55\cdot\frac{ft\cdot kip}{ft}$$

$$\Sigma Vertical = 7.69\cdot\frac{kip}{ft}$$

$$\Sigma Horizontal = 12.77\cdot\frac{kip}{ft}$$

Find Pile Loads Using CPGA:

103/437

Number := if(Pile5 = 0, 4, 5)
Number := if(Pile4 = 0, 3, Number)
Number := if(Pile3 = 0, 2, Number)
Number := if(Pile2 = 0, 1, Number)
Number := if(Pile1 = 0, 0, Number)

Total number of piles:

Number = 3

Concrete Pile Properties

Dimension of square pile $D := 18in$

Assume all piles to pile cap connections are fixed (details show rebar and pile extending into base).

Pile Properties:

$fc := 3000 \cdot psi$

$E_C := 57000 \cdot \sqrt{fc \cdot psi}$ $E_C = 3122.02 \cdot ksi$

$I := \frac{D^4}{12}$ $I = 8748 \cdot in^4$

C33=2.0 for full skin friction
B66 = 0 torsional stiffness
Length = 34ft

$Area := D^2$ $Area = 324 \cdot in^2$

Allowable Design Load Strengths from Geotechs (FS = 1.7 for extreme):
AC = Allowable Pile Compression Load, = 137.963/1.7 = 81.1 k
AT = Allowable Pile Tension Load, = 73.264/1.7 '= 43.1 k

The following allowable design strengths are from the existing pile capacity MathCAD file (FS = 1.5):
PO = Axial Compression Design Load Strength, 536 k (P_{n0} in Mathcad)
PT = Axial Tension Design Load Strength, Fy*As= 128 k
PB = Axial Design Load Strength at Balanced Condition, 274 k (P_{n5} in Mathcad)
MB = Design Moment Strength at Balanced Condition, 1802 k-in (M_{n5} in Mathcad)
MO = Design Moment Strength Under Pure Flexure, 905 k-in (M_{n7} in Mathcad)

Soil:
NH
Soil Modulus, 0.05 k/in^3 (per Geotech)

Concrete Sheet Pile Properties 104/437

Dimension of rectangular pile $D_{sp} := 10in$ $W_{sp} := 12in$

Pile Spacing, S=1ft

Assume all sheet pile to pile cap connections are fixed (details show rebar extending into base).

Pile Properties:

$fc := 3000 \cdot psi$

$I_{1sp} := \frac{\left(W_{sp} \cdot D_{sp}^{3}\right)}{12}$ $I_{1sp} = 1000 \cdot in^4$ $I_{2sp} := \frac{D_{sp} \cdot W_{sp}^{3}}{12}$ $I_{2sp} = 1440 \cdot in^4$

C33=2.0 for full skin friction

B66 = 0 torsional stiffness

Length = 15ft

$E_C := 57000 \cdot \sqrt{fc \cdot psi}$ $Area := D_{sp} \cdot W_{sp}$

$E_C = 3122.02 \cdot ksi$ $Area = 120 \cdot in^2$

Allowable Design Load Strengths from Geotechs (FS = 1.7 for extreme):

AC = Allowable Pile Compression Load, = 5'/4'*20.73klf/1.7 = 15.2 k

AT = Allowable Pile Tension Load, = 5'/4'*11.231klf/1.7 = 8.26 k

Assumes two 2.5ft wide sheetpiles engaged; strength is equally divided among four 1' wide piles.

The following allowable design strengths are from the existing pile capacity MathCAD file (FS = 1.5):

PO = Axial Compression Design Load Strength, 205 k (P_{n0} in Mathcad)

PT = Axial Tension Design Load Strength, Fy*As= 56.3 k

PB = Axial Design Load Strength at Balanced Condition, 72 k (P_{n5} in Mathcad)

MB = Design Moment Strength at Balanced Condition, 333 k-in (M_{n5} in Mathcad)

MO = Design Moment Strength Under Pure Flexure, 195 k-in (M_{n7} in Mathcad)

Soil:

NH

Soil Modulus, 0.05 k/in^3 (per Geotech)

105/437

$\Sigma M_T := \Sigma M \cdot S \cdot N_{Rows}$

$\Sigma Vertical_T := \Sigma Vertical \cdot S \cdot N_{Rows}$

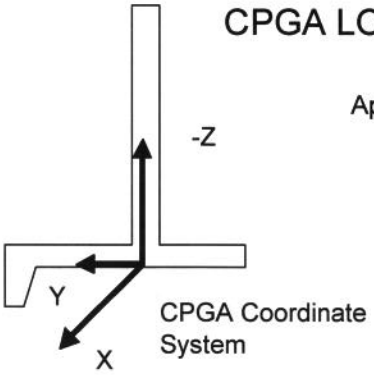
$\Sigma Horizontal_T := \Sigma Horizontal \cdot S \cdot N_{Rows}$

$\Sigma M_T = 98.19 \cdot ft \cdot kip$

$\Sigma Vertical_T = 30.76 \cdot kip$

$\Sigma Horizontal_T = 51.07 \cdot kip$

CPGA LOADING FOR WATER AT TOP OF WALL



Applied Loads

$PY := -\Sigma Horizontal_T$
$PZ := \Sigma Vertical_T$
$MX := -\Sigma M_T$

- Signs added to agree with CPGA sign convention.

$PY = -51.07 \cdot kip$

$PZ = 30.76 \cdot kip$

$MX = -98.19 \cdot ft \cdot kip$

Loads input into CPGA filename: M145RL1.dat

Shear. Due to the continuous construction of the sheet pile, b = 12".

$Sht_{Hormax} = 5.9 \cdot kip$

For 1 ft wide pile.

$d_f := 18in - \frac{D_{sp}}{2} - 2.5in - .5in - .5 \cdot \frac{7}{8}in = 9.56 \cdot in$ $\quad b_o := 12in$

$V_c := \phi_V \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot d = 11.01 \cdot kip$

$Check13 := if\left(V_c > 1.5 \cdot Sht_{Hormax}, "OKAY", "NO GOOD"\right)$ $\quad FS_{13} := \frac{V_c}{Sht_{Hormax}} = 1.87$ $\quad$ Check13 = "OKAY"

Overall Factor of Safety

Factor of Safety

Heel Factor of Saftey:	$FS_H = 1.37$
Longitudinal Flexure of Pile Cap:	$FS_{LF} = 2.31$
Stem Factor of Safety:	$FS_S = 1.83$
Toe Factor of Safety:	$FS_T = 2.11$
Tension Pile Embedment:	Check7 = "OKAY"
Tension Pile Factor of Safety:	$FS_8 = 7.35$
Punching Shear Factor of Safety:	$FS_9 = 3.33$
Bearing Factor of Safety:	$FS_B = 15.83$
Pile Cap Shear Factor of Safety:	$FS_{13} = 1.87$

$FoS := \min\left(FS_H, FS_{LF}, FS_S, FS_T, FS_8, FS_9, FS_B, FS_{13}\right)$

$FoS = 1.37$

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis.

107/437

M145RL1.dat

2011 REVISION

Chkd: Marx 9/20/11

```
10 CID-MO M145 GEOTECH REV STRENGTH 4ft abutment-
20 SEP 2011 Existing conditions h20 TOW-1ft-
30 GT FS is 1.7 and Structural FS is 1.5
40 LOAD 1 0 -51.07 30.76 -98.19 0 0
51 PILE 1 0 2 0
52 PILE 2 0 6 0
53 PILE 3 -1.5 11.5 0
54 PILE 4 -.5 11.5 0
55 PILE 5 .5 11.5 0
56 PILE 6 1.5 11.5 0
110 SOIL  NH 0.05 LEN 33  0  1 to 2
111 PROP 3122 8748 8748 324 2 0 1 to 2
112 DLS S 81.1 43.1 536 128 274 1802 905 H 18 1 TO 2
120 SOIL  NH 0.05 LEN 15  0  3 TO 6
121 PROP 3122 1000 1440 120 2 0 3 to 6
121 DLS S 15.2 8.26 205 56.3 72 333 195 H 10 3 TO 6
140 FIX 1 TO 6
150 PFO 1 TO 6
500 FOUT 1 2 3 4 5 6 7 M145RL1.txt
```

108/437


```
*********************************
* CASE PROGRAM    #  X0080       *  CPGA - CASE PILE GROUP ANALYSIS PROGRAM
* VERSION NUMBER # 1993/03/29   *  RUN DATE 06-SEP-2011   RUN TIME 14.38.26
*********************************


CID-MO M145 GEOTECH REV STRENGTH 4FT ABUTMENT
SEP 2011 EXISTING CONDITIONS H20 TOW-1FT
GT FS IS 1.7 AND STRUCTURAL FS IS 1.5


THERE ARE     6 PILES AND
              1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                      X          Y          Z
                                    -----      -----      -----
WITH DIAGONAL COORDINATES = (      -1.50 ,     2.00 ,      .00 )
                            (       1.50 ,    11.50 ,      .00 )


*******************************************************************************

         PILE PROPERTIES AS INPUT

      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .31220E+04   .87480E+04   .87480E+04   .32400E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   1    2


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .31220E+04   .10000E+04   .14400E+04   .12000E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   3    4    5    6


*******************************************************************************

         SOIL DESCRIPTIONS AS INPUT

   NH      ESOIL       LENGTH        L            LU
         K/IN**3                    FT           FT
         .50000E-01     L        .33000E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   1    2


   NH      ESOIL       LENGTH        L            LU
         K/IN**3                    FT           FT
         .50000E-01     L        .15000E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -
```

3 4 5 6

****

PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	.00	2.00	.00	V	.00	33.00	F
2	.00	6.00	.00	V	.00	33.00	F
3	-1.50	11.50	.00	V	.00	15.00	F
4	-.50	11.50	.00	V	.00	15.00	F
5	.50	11.50	.00	V	.00	15.00	F
6	1.50	11.50	.00	V	.00	15.00	F

						126.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-51.1	30.8	-98.2	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.66258E+03	.00000E+00	.00000E+00	.00000E+00	.29315E+05	-.61195E+05
.00000E+00	.61826E+03	.00000E+00	-.26988E+05	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.26868E+05	.27882E+07	.00000E+00	.00000E+00
.00000E+00	-.26988E+05	.27882E+07	.34850E+09	.00000E+00	.00000E+00
.29315E+05	.00000E+00	.00000E+00	.00000E+00	.51547E+07	-.24726E+07
-.61195E+05	.00000E+00	.00000E+00	.00000E+00	-.24726E+07	.72376E+07

LOAD CASE 1. NUMBER OF FAILURES = 6. NUMBER OF PILES IN TENSION = 4.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	.0000E+00	-.8757E-01	.1296E-01	-.1138E-03	.0000E+00	.0000E+00

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE

# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-13.7	52.2	681.8	.0	.0	.64	1.26	.00	.00*
2	.0	-13.7	24.3	681.8	.0	.0	.30	1.40	.00	.00*
3	.0	-5.9	-11.5	193.6	.0	.0	1.39	3.23	.00	.00*
4	.0	-5.9	-11.5	193.6	.0	.0	1.39	3.23	.00	.00*
5	.0	-5.9	-11.5	193.6	.0	.0	1.39	3.23	.00	.00*
6	.0	-5.9	-11.5	193.6	.0	.0	1.39	3.23	.00	.00*

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-13.7	52.2	681.8	.0	.0
2	.0	-13.7	24.3	681.8	.0	.0
3	.0	-5.9	-11.5	193.6	.0	.0
4	.0	-5.9	-11.5	193.6	.0	.0
5	.0	-5.9	-11.5	193.6	.0	.0
6	.0	-5.9	-11.5	193.6	.0	.0

2011 REVISION

111/437

US Army Corps of Engineers

CID-MO Floodwall
Floodwall Analysis - Existing, M145
4 foot Stop Log Abutment Monolith
Kansas City Levees

Comp by: Sheffield
Chkd by: Marx 9/28/11

Water EL: TOW - 2'

Assumptions: For Feasibility Analysis, it is assumed that the sheetpile cutoff is adequately strength designed for a 50% head differential, the pile cap under the closure is adequately designed, and that the floodwall sections of the monolith are accounted for by analysis of adjacent floodwall monoliths. Soil Surfaces assumed horizontal.

Variables

$kip := 1000lb$

$psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$ $plf := \frac{lb}{ft}$

$ksf \equiv \frac{1000lb}{ft^2}$ $psf \equiv \frac{lb}{ft^2}$

$EL_{top_ftg} := 746.9ft$

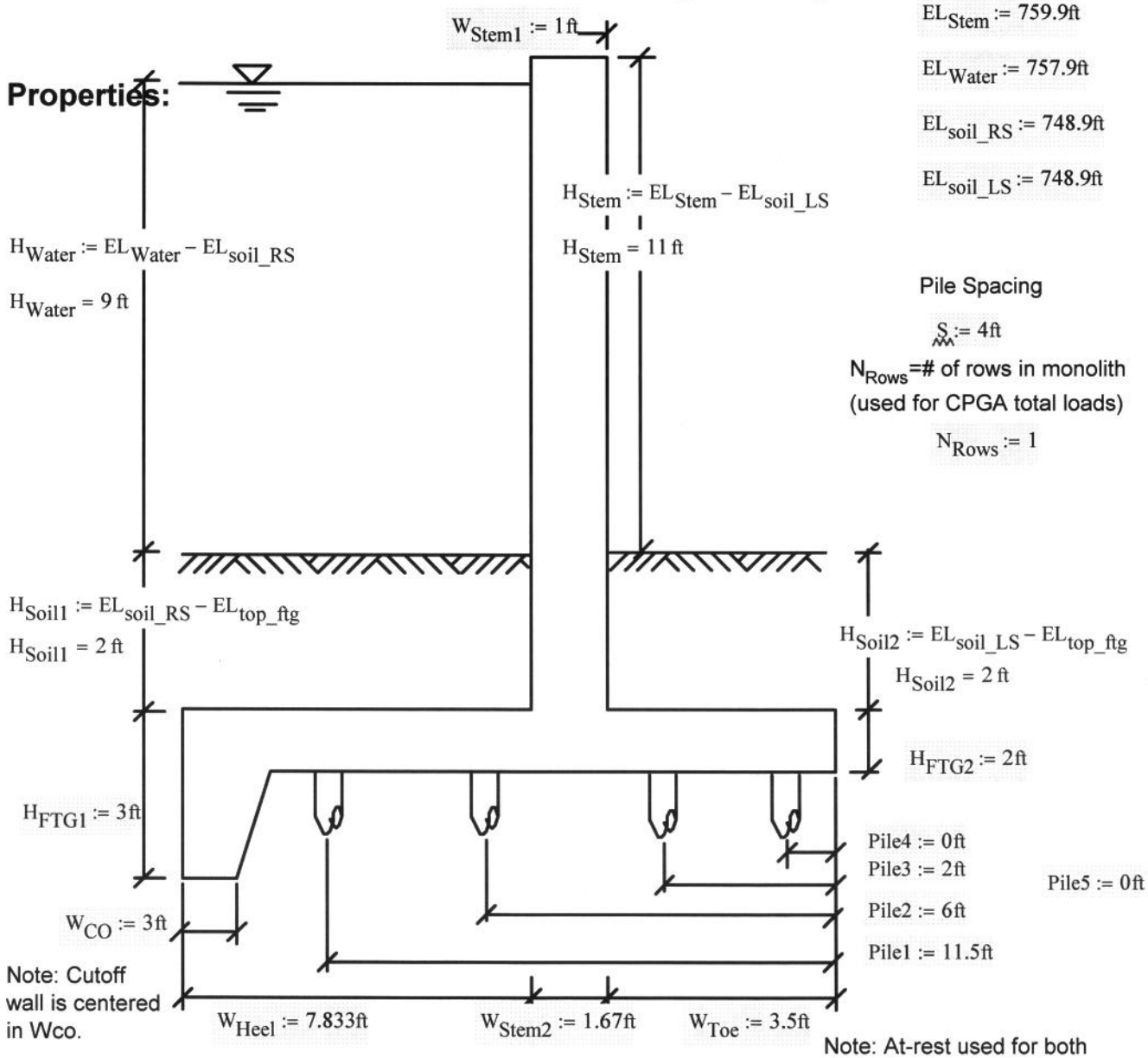
$EL_{Stem} := 759.9ft$

$EL_{Water} := 757.9ft$

$EL_{soil_RS} := 748.9ft$

$EL_{soil_LS} := 748.9ft$

Properties:



Pile Spacing

$S := 4ft$

N_{Rows}=# of rows in monolith (used for CPGA total loads)

$N_{Rows} := 1$

Note: Cutoff wall is centered in Wco.

Concrete Properties $\gamma_c := 150pcf$

Water Weight $\gamma_w := 62.4pcf$

Soil Properties $\gamma_{sat} := 115pcf$ $\phi := 22deg$

$K_o := 1 - \sin(\phi)$ $K_o = 0.63$

Note: At-rest used for both driving and resisting for Feasibility and to require piles to fully resist stability.

112/437

Lengths:

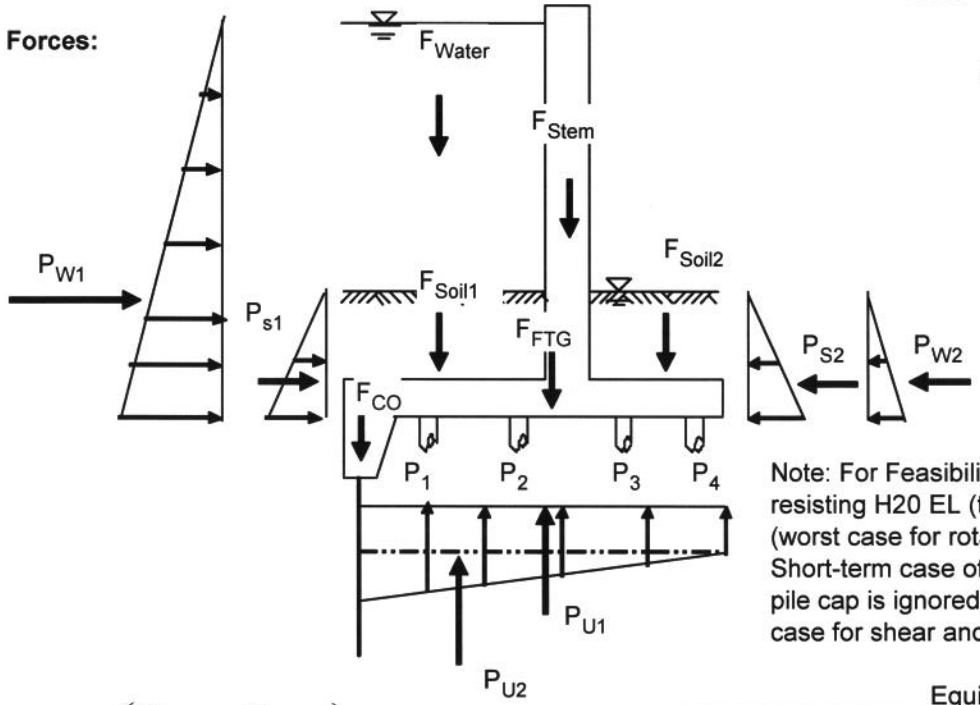
$L_{CO} := H_{FTG1} - H_{FTG2}$ $\quad L_{W1} := H_{Water} + H_{Soil1} + H_{FTG1}$ $\quad L_{STM} := H_{Stem} + H_{Soil2}$

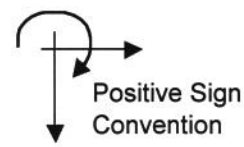
$L_{FTG} := W_{Heel} + W_{Stem2} + W_{Toe}$ $\quad L_{PU1} := H_{Soil2} + H_{FTG2}$ $\quad L_{FTG} = 13\,ft$

$L_{W1} = 14\,ft$

$L_{PU1} = 4\,ft$

Forces:





Note: For Feasibility, only long-term case of resisting H20 EL (top of soil) is considered (worst case for rotation and pile pull-out). Short-term case of little or no resisting H20 on pile cap is ignored, though this is the worst case for shear and sliding.

$F_{Stem} := \left(\frac{W_{Stem1} + W_{Stem2}}{2}\right) \cdot L_{STM} \cdot \gamma_c = 2603.25 \cdot plf$

$F_{closure} := 5.53 \frac{kip}{ft}$ Equivalent Point Load acting on abutment from Stop Logs per tributary width of pile cap.

$F_{closure}$ = 1/2*.0624*10²*17.5/2*(1/4ft)

$F_{FTG} := H_{FTG2} \cdot L_{FTG} \cdot \gamma_c = 3900.9 \cdot plf$

$F_{CO} := L_{CO} \cdot W_{CO} \cdot \gamma_c = 450 \cdot plf$ (Treated Rectangular)

$F_{Soil1} := H_{Soil1} \cdot W_{Heel} \cdot (\gamma_{sat} - \gamma_w) = 824.03 \cdot plf$

$F_{Soil2} := H_{Soil2} \cdot W_{Toe} \cdot (\gamma_{sat} - \gamma_w) = 368.2 \cdot plf$

$F_{Water1} := (H_{Water} + H_{Soil1}) \cdot W_{Heel} \cdot \gamma_w = 5376.57 \cdot plf$

$F_{Water2} := H_{Soil2} \cdot W_{Toe} \cdot \gamma_w = 436.8 \cdot plf$

$P_{W1} := \frac{1}{2} \cdot L_{W1}^2 \cdot \gamma_w = 6115.2 \cdot plf$

$P_{W2a} := -\left[\frac{1}{2} \cdot (L_{PU1})^2 \cdot \gamma_w\right] = -499.2 \cdot plf$

Cutoff wall assumed to be 50% effective. See Figure C-3 of EM 1110-2-2100

$q_{uCO} := (1 - 0.5) \cdot \gamma_w \left[(H_{Water} + H_{Soil1} + H_{FTG2}) - L_{PU1}\right] + L_{PU1} \cdot \gamma_w = 530.4 \cdot psf$

$q_{umax} := \left[(1 - 0.5)(L_{W1} - L_{PU1})\gamma_w\right] + L_{PU1} \cdot \gamma_w = 561.6 \cdot psf$

$q_{utoe} := L_{PU1} \cdot \gamma_w + \frac{L_{PU1}}{L_{FTG} - .5W_{CO} + L_{PU1}} (q_{umax} - L_{PU1} \cdot \gamma_w) = 330.1 \cdot psf$

Note: Soil below base of pile cap acting on key is essentially equal and opposite since all resistance is by piles.

$P_{W2b} := -\left[\frac{1}{2} \cdot (q_{uCO} + q_{umax}) \cdot L_{CO}\right] = -546 \cdot plf$

$P_{S1} := K_o \cdot \left[\frac{1}{2} \cdot (H_{Soil1} + H_{FTG2})^2 \cdot (\gamma_{sat} - \gamma_w)\right] = 263.17 \cdot plf$ $\quad P_{S2} := -K_o \cdot \left[\frac{1}{2} \cdot (H_{Soil2} + H_{FTG2})^2 \cdot (\gamma_{sat} - \gamma_w)\right] = -263.17 \cdot plf$

$P_{U1} := -(L_{FTG} - .5W_{CO}) \cdot (q_{utoe}) = -3797.15 \cdot plf$ $\quad P_{U2} := -0.5(L_{FTG} - .5 \cdot W_{CO}) \cdot (q_{umax} - q_{utoe}) = -1331.47 \cdot plf$

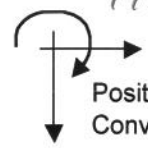
$P_{U3} := -.5W_{CO} \cdot (L_{W1} \cdot \gamma_w) = -1310.4 \cdot plf$

113/437

Inputs for CPGA

Assumes perfectly rigid pile cap.

Sum moments around bottom corner of Toe:



Moment Arm (from bottom corner of Toe)

$$Arm_{Stem} := \left(W_{Toe} + \frac{W_{Stem2}}{2}\right) = 4.34\,ft$$

$$Arm_{FTG} := \frac{L_{FTG}}{2} = 6.5\,ft$$

$$Arm_{CO} := L_{FTG} - \frac{W_{CO}}{2} = 11.5\,ft$$

$$Arm_{closure} := 4\cdot ft + \frac{H_{Water}}{3} = 7\,ft$$

$$Arm_{Soil1} := \left(W_{Toe} + W_{Stem2} + 0.5W_{Heel}\right) = 9.09\,ft$$

$$Arm_{Soil2} := 0.5W_{Toe} = 1.75\,ft$$

$$Arm_{Water1} := Arm_{Soil1} = 9.09\,ft$$

$$Arm_{Water2} := Arm_{Soil2} = 1.75\,ft$$

$$Arm_{W1} := \frac{1}{3}\cdot L_{W1} - L_{CO} = 3.67\,ft$$

$$Arm_{W2a} := 0.333\cdot\left(L_{PU1}\right) = 1.33\,ft$$

$$Arm_{W2b} := L_{CO} - \frac{L_{CO}\cdot\left(2q_{uCO} + q_{umax}\right)}{3\left(q_{uCO} + q_{umax}\right)} = 0.5\,ft$$

$$Arm_{S1} := \frac{1}{3}\cdot\left(H_{Soil1} + H_{FTG2}\right) = 1.33\,ft$$

$$Arm_{S2} := Arm_{W2a} = 1.33\,ft$$

$$Arm_{U1} := \frac{L_{FTG} - 0.5W_{CO}}{2} = 5.75\,ft$$

$$Arm_{U2} := \frac{2\left(L_{FTG} - 0.5W_{CO}\right)}{3} = 7.67\,ft$$

$$Arm_{U3} := L_{FTG} - 0.25\cdot W_{CO} = 12.25\,ft$$

Moment (about bottom corner of toe)

$$M_{Stem} := -F_{Stem}\cdot Arm_{Stem} = -11.29\cdot\frac{kip\cdot ft}{ft}$$

$$M_{FTG} := -F_{FTG}\cdot Arm_{FTG} = -25.36\cdot\frac{kip\cdot ft}{ft}$$

$$M_{CO} := -F_{CO}\cdot Arm_{CO} = -5.18\cdot\frac{kip\cdot ft}{ft}$$

$$M_{closure} := F_{closure}\cdot Arm_{closure} = 38.71\cdot\frac{kip\cdot ft}{ft}$$

$$M_{Soil1} := -F_{Soil1}\cdot Arm_{Soil1} = -7.49\cdot\frac{kip\cdot ft}{ft}$$

$$M_{Soil2} := -F_{Soil2}\cdot Arm_{Soil2} = -0.64\cdot\frac{kip\cdot ft}{ft}$$

$$M_{Water1} := -F_{Water1}\cdot Arm_{Water1} = -48.85\cdot\frac{kip\cdot ft}{ft}$$

$$M_{Water2} := -F_{Water2}\cdot Arm_{Water2} = -0.76\cdot\frac{kip\cdot ft}{ft}$$

$$M_{W1} := P_{W1}\cdot Arm_{W1} = 22.42\cdot\frac{kip\cdot ft}{ft}$$

$$M_{W2a} := P_{W2a}\cdot Arm_{W2a} = -0.66\cdot\frac{kip\cdot ft}{ft}$$

$$M_{W2b} := -P_{W2b}\cdot Arm_{W2b} = 0.28\cdot\frac{kip\cdot ft}{ft}$$

$$M_{S1} := P_{S1}\cdot Arm_{S1} = 0.35\cdot\frac{kip\cdot ft}{ft}$$

$$M_{S2} := P_{S2}\cdot Arm_{S2} = -0.35\cdot\frac{kip\cdot ft}{ft}$$

$$M_{U1} := -P_{U1}\cdot Arm_{U1} = 21.84\cdot\frac{kip\cdot ft}{ft}$$

$$M_{U2} := -P_{U2}\cdot Arm_{U2} = 10.21\cdot\frac{kip\cdot ft}{ft}$$

$$M_{U3} := -P_{U3}\cdot Arm_{U3} = 16.06\cdot\frac{kip\cdot ft}{ft}$$

$$\Sigma M := \begin{pmatrix} M_{Stem} + M_{FTG} + M_{CO} + M_{closure} + M_{Soil1} + M_{Soil2} + M_{Water1} + M_{Water2} + M_{W1} + M_{W2a} \cdots \\ + M_{W2b} + M_{S1} + M_{S2} + M_{U1} + M_{U2} + M_{U3} \end{pmatrix}$$

$$\Sigma M = 9.28\cdot\frac{ft\cdot kip}{ft}$$

$$\Sigma Vertical := \begin{pmatrix} F_{Stem} + F_{FTG} + F_{CO} + F_{Soil1} + F_{Soil2} + F_{Water1} + F_{Water2} \cdots \\ + P_{U1} + P_{U2} + P_{U3} \end{pmatrix}$$

$$\Sigma Vertical = 7.52\cdot\frac{kip}{ft}$$

$$\Sigma Horizontal := \left(F_{closure} + P_{W1} + P_{W2a} + P_{W2b} + P_{S1} + P_{S2}\right)$$

$$\Sigma Horizontal = 10.6\cdot\frac{kip}{ft}$$

114/437

Find Pile Loads Using CPGA:

$Number := if(Pile5 = 0, 4, 5)$
$Number := if(Pile4 = 0, 3, Number)$
$Number := if(Pile3 = 0, 2, Number)$
$Number := if(Pile2 = 0, 1, Number)$
$Number := if(Pile1 = 0, 0, Number)$

Total number of piles:

$Number = 3$

Concrete Pile Properties

Dimension of square pile $D := 18in$

Assume all piles to pile cap connections are fixed (details show rebar and pile extending into base).

Pile Properties:

$fc := 3000 \cdot psi$

$E_C := 57000 \cdot \sqrt{fc \cdot psi}$ $E_C = 3122.02 \cdot ksi$

$I := \frac{D^4}{12}$ $I = 8748 \cdot in^4$

C33=2.0 for full skin friction
B66 = 0 torsional stiffness
Length = 34ft

$Area := D^2$ $Area = 324 \cdot in^2$

Allowable Design Load Strengths from Geotechs (FS = 1.7 for extreme):
AC = Allowable Pile Compression Load, = 137.963/1.7 = 81.1 k
AT = Allowable Pile Tension Load, = 73.264/1.7 '= 43.1 k

The following allowable design strengths are from the existing pile capacity MathCAD file (FS = 1.5):
PO = Axial Compression Design Load Strength, 536 k (P_{n0} in Mathcad)
PT = Axial Tension Design Load Strength, Fy*As= 128 k
PB = Axial Design Load Strength at Balanced Condition, 274 k (P_{n5} in Mathcad)
MB = Design Moment Strength at Balanced Condition, 1802 k-in (M_{n5} in Mathcad)
MO = Design Moment Strength Under Pure Flexure, 905 k-in (M_{n7} in Mathcad)

Soil:
NH
Soil Modulus, 0.05 k/in^3 (per Geotech)

Concrete Sheet Pile Properties

Dimension of rectangular pile $D_{sp} := 10in \quad W_{sp} := 12in$

Pile Spacing, S=1ft

Assume all sheet pile to pile cap connections are fixed (details show rebar extending into base).

Pile Properties:

$fc := 3000 \cdot psi$

$I_{1sp} := \frac{\left(W_{sp} \cdot D_{sp}^{3}\right)}{12}$ $\quad I_{1sp} = 1000 \cdot in^4$ $\quad I_{2sp} := \frac{D_{sp} \cdot W_{sp}^{3}}{12}$ $\quad I_{2sp} = 1440 \cdot in^4$

C33=2.0 for full skin friction

B66 = 0 torsional stiffness

Length = 15ft

$E_C := 57000 \cdot \sqrt{fc \cdot psi}$ $\quad Area := D_{sp} \cdot W_{sp}$

$E_C = 3122.02 \cdot ksi$ $\quad Area = 120 \cdot in^2$

Allowable Design Load Strengths from Geotechs (FS = 1.7 for extreme):

AC = Allowable Pile Compression Load, = 5'/4'*20.73klf/1.7 = 15.2 k

AT = Allowable Pile Tension Load, = 5'/4'*11.231klf/1.7 = 8.26 k

Assumes two 2.5ft wide sheetpiles engaged; strength is equally divided among four 1' wide piles.

The following allowable design strengths are from the existing pile capacity MathCAD file (FS = 1.5):

PO = Axial Compression Design Load Strength, 205 k (P_{n0} in Mathcad)

PT = Axial Tension Design Load Strength, Fy*As= 56.3 k

PB = Axial Design Load Strength at Balanced Condition, 72 k (P_{n5} in Mathcad)

MB = Design Moment Strength at Balanced Condition, 333 k-in (M_{n5} in Mathcad)

MO = Design Moment Strength Under Pure Flexure, 195 k-in (M_{n7} in Mathcad)

Soil:

NH

Soil Modulus, 0.05 k/in^3 (per Geotech)

116/437

$\Sigma M_T := \Sigma M \cdot S \cdot N_{Rows}$

$\Sigma Vertical_T := \Sigma Vertical \cdot S \cdot N_{Rows}$

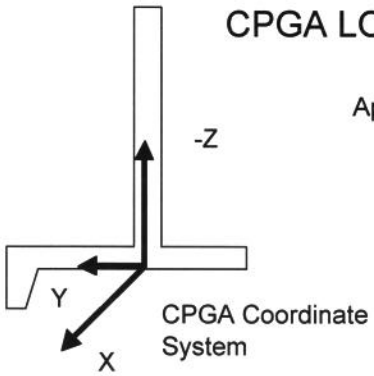
$\Sigma Horizontal_T := \Sigma Horizontal \cdot S \cdot N_{Rows}$

$\Sigma M_T = 37.1 \cdot ft \cdot kip$

$\Sigma Vertical_T = 30.08 \cdot kip$

$\Sigma Horizontal_T = 42.4 \cdot kip$

CPGA LOADING FOR WATER AT TOP OF WALL



Applied Loads

$PY := -\Sigma Horizontal_T$
$PZ := \Sigma Vertical_T$
$MX := -\Sigma M_T$

- Signs added to agree with CPGA sign convention.

$PY = -42.4 \cdot kip$

$PZ = 30.08 \cdot kip$

$MX = -37.1 \cdot ft \cdot kip$

Loads input into CPGA filename: M145RL2.dat

Shear. Due to the continuous construction of the sheet pile, b = 12".

$Sht_{Hormax} = 4.9 \cdot kip$

For 1 ft wide pile.

$d_f := 18in - \frac{D_{sp}}{2} - 2.5in - .5in - .5 \cdot \frac{7}{8}in = 9.56 \cdot in$ $\quad b_o := 12in$

$V_c := \phi_V \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot d = 11.01 \cdot kip$

$Check13 := if(V_c > 1.5 \cdot Sht_{Hormax}, "OKAY", "NO GOOD")$ $\quad FS_{13} := \frac{V_c}{Sht_{Hormax}} = 2.25$ $\quad$ Check13 = "OKAY"

Overall Factor of Safety

Factor of Safety

Heel Factor of Saftey:	$FS_H = 1.78$
Longitudinal Flexure of Pile Cap:	$FS_{LF} = 2.39$
Stem Factor of Safety:	$FS_S = 2.19$
Toe Factor of Safety:	$FS_T = 2.53$
Tension Pile Embedment:	Check7 = "OKAY"
Tension Pile Factor of Safety:	$FS_8 = 9.71$
Punching Shear Factor of Safety:	$FS_9 = 3.96$
Bearing Factor of Safety:	$FS_B = 18.86$
Pile Cap Shear Factor of Safety:	$FS_{13} = 2.25$

$FoS := \min(FS_H, FS_{LF}, FS_S, FS_T, FS_8, FS_9, FS_B, FS_{13})$

$FoS = 1.78$

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis.

118/437

2011 REVISION

Chkd: Marx 9/20/11

M145RL2.dat

```
10 CID-MO M145 GEOTECH REV STRENGTH 4ft abutment-
20 SEP 2011 Existing conditions h20 TOW-2ft-
30 GT FS is 1.7 and Structural FS is 1.5
40 LOAD 1 0 -42.4 30.08 -37.1 0 0
51 PILE 1 0 2 0
52 PILE 2 0 6 0
53 PILE 3 -1.5 11.5 0
54 PILE 4 -.5 11.5 0
55 PILE 5 .5 11.5 0
56 PILE 6 1.5 11.5 0
110 SOIL  NH 0.05 LEN 33  0  1 to 2
111 PROP 3122 8748 8748 324 2 0 1 to 2
112 DLS S 81.1 43.1 536 128 274 1802 905 H 18 1 TO 2
120 SOIL  NH 0.05 LEN 15  0  3 TO 6
121 PROP 3122 1000 1440 120 2 0 3 to 6
121 DLS S 15.2 8.26 205 56.3 72 333 195 H 10 3 TO 6
140 FIX 1 TO 6
150 PFO 1 TO 6
500 FOUT 1 2 3 4 5 6 7 M145RL2.txt
```

119/437

```
*********************************
* CASE PROGRAM    #  X0080      *  CPGA - CASE PILE GROUP ANALYSIS PROGRAM
* VERSION NUMBER # 1993/03/29   *  RUN DATE 06-SEP-2011   RUN TIME 15.08.53
*********************************


CID-MO M145 GEOTECH REV STRENGTH 4FT ABUTMENT
SEP 2011 EXISTING CONDITIONS H20 TOW-2FT
GT FS IS 1.7 AND STRUCTURAL FS IS 1.5


THERE ARE     6 PILES AND
              1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                      X          Y          Z
                                    -----      -----      -----
WITH DIAGONAL COORDINATES = (      -1.50 ,     2.00 ,      .00 )
                            (       1.50 ,    11.50 ,      .00 )


*******************************************************************************

         PILE PROPERTIES AS INPUT


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .31220E+04   .87480E+04   .87480E+04   .32400E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   1    2


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .31220E+04   .10000E+04   .14400E+04   .12000E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   3    4    5    6


*******************************************************************************

         SOIL DESCRIPTIONS AS INPUT


   NH      ESOIL       LENGTH        L            LU
          K/IN**3                   FT           FT
          .50000E-01    L        .33000E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   1    2


   NH      ESOIL       LENGTH        L            LU
          K/IN**3                   FT           FT
          .50000E-01    L        .15000E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -
```

3 4 5 6

****

PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	.00	2.00	.00	V	.00	33.00	F
2	.00	6.00	.00	V	.00	33.00	F
3	-1.50	11.50	.00	V	.00	15.00	F
4	-.50	11.50	.00	V	.00	15.00	F
5	.50	11.50	.00	V	.00	15.00	F
6	1.50	11.50	.00	V	.00	15.00	F

						126.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-42.4	30.1	-37.1	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.66258E+03	.00000E+00	.00000E+00	.00000E+00	.29315E+05	-.61195E+05
.00000E+00	.61826E+03	.00000E+00	-.26988E+05	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.26868E+05	.27882E+07	.00000E+00	.00000E+00
.00000E+00	-.26988E+05	.27882E+07	.34850E+09	.00000E+00	.00000E+00
.29315E+05	.00000E+00	.00000E+00	.00000E+00	.51547E+07	-.24726E+07
-.61195E+05	.00000E+00	.00000E+00	.00000E+00	-.24726E+07	.72376E+07

LOAD CASE 1. NUMBER OF FAILURES = 6. NUMBER OF PILES IN TENSION = 4.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	.0000E+00	-.7266E-01	.1082E-01	-.9344E-04	.0000E+00	.0000E+00

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE

# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-11.4	43.8	566.4	.0	.0	.54	1.13	.00	.00*
2	.0	-11.4	20.9	566.4	.0	.0	.26	1.24	.00	.00*
3	.0	-4.9	-8.7	160.7	.0	.0	1.05	2.64	.00	.00*
4	.0	-4.9	-8.7	160.7	.0	.0	1.05	2.64	.00	.00*
5	.0	-4.9	-8.7	160.7	.0	.0	1.05	2.64	.00	.00*
6	.0	-4.9	-8.7	160.7	.0	.0	1.05	2.64	.00	.00*

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-11.4	43.8	566.4	.0	.0
2	.0	-11.4	20.9	566.4	.0	.0
3	.0	-4.9	-8.7	160.7	.0	.0
4	.0	-4.9	-8.7	160.7	.0	.0
5	.0	-4.9	-8.7	160.7	.0	.0
6	.0	-4.9	-8.7	160.7	.0	.0

US Army Corps of Engineers

Probability of Failure

CID MO Monolith 145

Pullout of Sheetpile: H2O to TOW

Comp by: FRS
Chkd by: Marx 9/28/11

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996

III. Situation

1. CID MO M145 did not meet the 1.7 geotech factor of safety for which it has been determined 99.8% reliability can be assigned. See mathcad and CPGA analyses of a 4' wide pile cap for existing condition.
2. Material Properties used:

Mean (Ultimate) Pullout, $A_T := 11.231 \cdot kip \cdot \frac{5}{4} = 14.039 \cdot kip$ for 12" of pile length

(assuming minimum of two 2.5' long sheet piles engaged for pullout - strength is equally divided among four 1' wide piles).

4. From *Geotechnical Data Supplied by Glenn Bellew,* the amount of variation anticipated in one standard deviation for pullout is 18%.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength
FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength
FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength
FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength
ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength
ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

123/437

V. Caclulating Factors of Safety

Tension Load on Sheetpile (from M145R.dat CPGA file): $P_u := 14.7 \cdot kip$

Mean Sheetpile Pullout:

$\phi := 1.0$ For Risk and Reliability

$$FS_D := \frac{\phi \cdot A_T}{P_u} \qquad FS_D = 0.955$$

Upper Sheetpile Pullout

$$A_{TU} := A_T + A_T \cdot 0.18 \qquad A_{TU} = 16.566 \cdot kip$$

$$FS_{ATu} := \frac{\phi \cdot A_{TU}}{P_u} \qquad FS_{ATu} = 1.127$$

Lower Sheetpile Pullout

$$A_{TL} := A_T - A_T \cdot 0.18 \qquad A_{TL} = 11.512 \cdot kip$$

$$FS_{ATl} := \frac{\phi \cdot A_{TL}}{P_u} \qquad FS_{ATl} = 0.783$$

VI. Probability of Failure Calculation

$\Delta F_{fc} := FS_{ATu} - FS_{ATl}$ $\Delta F_{fc} = 0.344$

$\sigma_F := \sqrt{\left(\frac{\Delta F_{fc}}{2}\right)^2}$ $\sigma_F = 0.172$

$V_F := \frac{\sigma_F}{FS_D}$ $V_F = 0.18$

$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\beta_{LN} = -0.347$ Taylor Series method. See ETL 1110-2-547, pg B-8.

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 36.43 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 63.57 \cdot \%$

125/437

POF For Water 1 ft Below TOW:

Tension Load on Sheetpile (see M145RL1.dat CPGA file). $P_{u1} := 11.5kip$

Ultimate Pullout at Water EL 759ft: $A_{T1} := A_T = 14.039 \cdot kip$

Mean Concrete Strength

$$FS_{D1} := \frac{\phi \cdot A_{T1}}{P_{u1}} \qquad FS_{D1} = 1.221$$

Upper Sheetpile Pullout

$$A_{TU1} := A_{T1} + A_{T1} \cdot 0.18 \qquad A_{TU1} = 16.566 \cdot kip$$

$$FS_{ATu1} := \frac{\phi \cdot A_{TU1}}{P_{u1}} \qquad FS_{ATu1} = 1.44$$

Lower Sheetpile Pullout

$$A_{TL1} := A_{T1} - A_{T1} \cdot 0.18 \qquad A_{TL1} = 11.512 \cdot kip$$

$$FS_{ATl1} := \frac{\phi \cdot A_{TL1}}{P_{u1}} \qquad FS_{ATl1} = 1.001$$

126/437

VI. Probability of Failure Calculation

$\Delta F_{fc} := FS_{ATu1} - FS_{ATl1}$ $\Delta F_{fc} = 0.439$

$\sigma_F := \sqrt{\left(\frac{\Delta F_{fc}}{2}\right)^2}$ $\sigma_F = 0.22$

$V_F := \frac{\sigma_F}{FS_{D1}}$ $V_F = 0.18$

$\beta_{LN} := \frac{\ln\left(\frac{FS_{D1}}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\beta_{LN} = 1.028$ Taylor Series method. See ETL 1110-2-547, pg B-8.

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 84.8 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 15.2 \cdot \%$

127/437

POF For Water 2 ft Below TOW:

Tension Load on Sheetpile (see M145L2.dat CPGA file). $P_{u2} := 8.7kip$

Ultimate Pullout at Water EL 758ft: $A_{T2} := A_T = 14.039\,kip$

Mean Concrete Strength

$$FS_{D2} := \frac{\phi \cdot A_{T2}}{P_{u2}} \qquad FS_{D2} = 1.614$$

Upper Sheetpile Pullout

$$A_{TU2} := A_{T2} + A_{T2} \cdot 0.18 \qquad A_{TU2} = 16.566 \cdot kip$$

$$FS_{ATu2} := \frac{\phi \cdot A_{TU2}}{P_{u2}} \qquad FS_{ATu2} = 1.904$$

Lower Sheetpile Pullout

$$A_{TL2} := A_{T2} - A_{T2} \cdot 0.18 \qquad A_{TL2} = 11.512 \cdot kip$$

$$FS_{ATl2} := \frac{\phi \cdot A_{TL2}}{P_{u2}} \qquad FS_{ATl2} = 1.323$$

VI. Probability of Failure Calculation

$\Delta F_{fc} := FS_{ATu2} - FS_{ATl2}$ $\Delta F_{fc} = 0.581$

$\sigma_F := \sqrt{\left(\frac{\Delta F_{fc}}{2}\right)^2}$ $\sigma_F = 0.29$

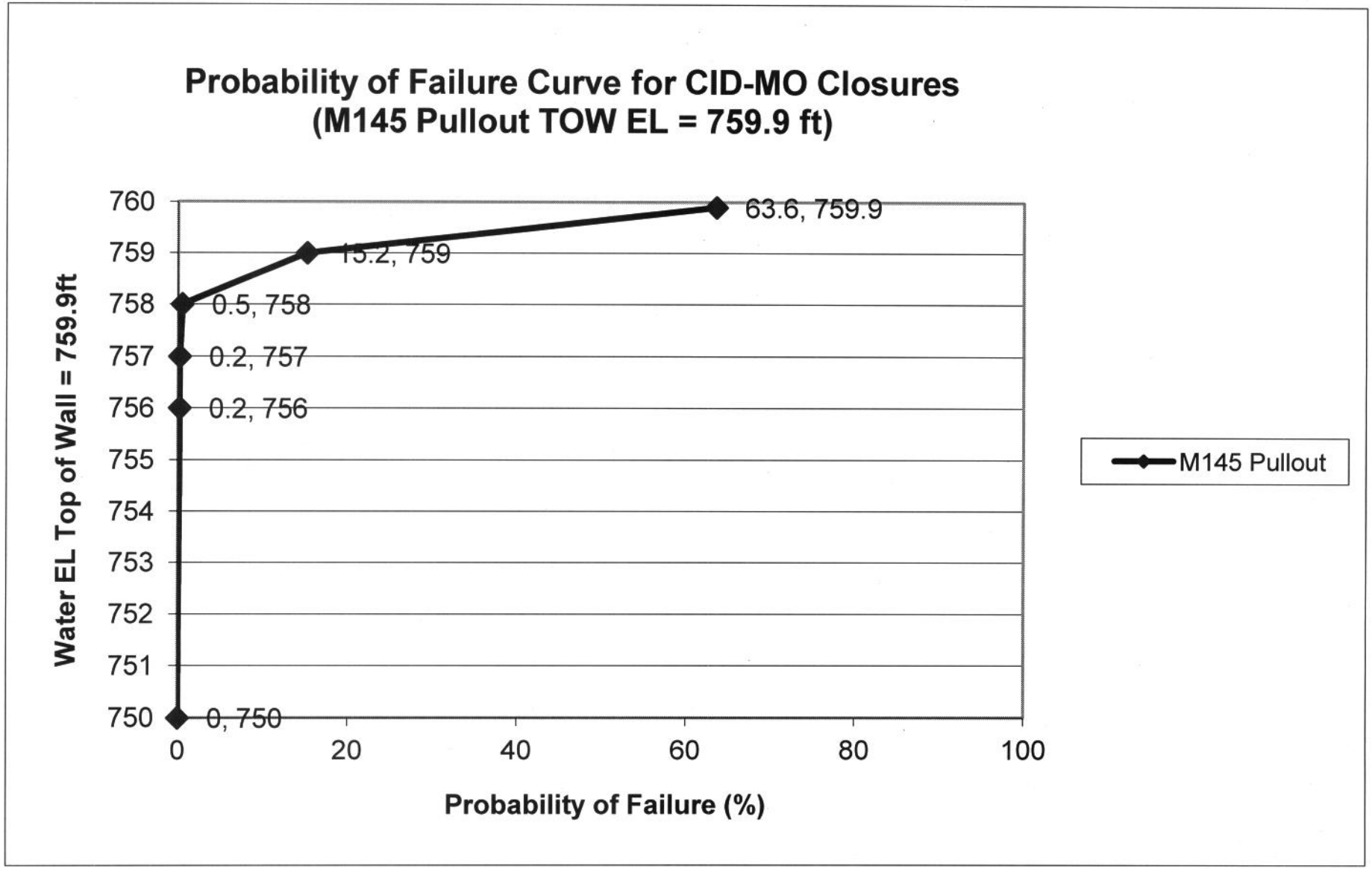
$V_F := \frac{\sigma_F}{FS_{D2}}$ $V_F = 0.18$

$\beta_{LN} := \frac{\ln\left(\frac{FS_{D2}}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\beta_{LN} = 2.59$ Taylor Series method. See ETL 1110-2-547, pg B-8.

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 99.52 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 0.48 \cdot \%$


Probability of Failure Curve for CID-MO Closures
(M145 Pullout TOW EL = 759.9 ft)
Water EL Top of Wall = 759.9ft
760
759
758
757
756
755
754
753
752
751
750
0
20
40
60
80
100
Probability of Failure (%)
63.6, 759.9
15.2, 759
0.5, 758
0.2, 757
0.2, 756
0, 750
M145 Pullout

VKSM 11/16/11

PER THE 2005 STRUCTURAL SUMMIT, A LOAD TO TOP OF WALL MAY BE CHECKED, AND IF $FS_{SLIDE} > 1.3$, % BASE IN COMPRESSION > 25%, AND $q_a = q_u/2$ > BEARING THEN A RELIABILITY STUDY MAY BE FOREGONE WITH RELIABILITY ASSUMED 99.8%.

FOR FEASIBILITY, FOOTERS ARE ANALYZED RIGID. IF THE FOOTERS ARE SIZED SUCH THAT THE TORSIONAL SHEAR STRESS IS LESS THAN 1/4 OF THE CRACKING SHEAR STRESS, AND HORIZONTAL FOOTER STEEL (PARALLEL TO WALL ALIGNMENT) IS PRESENT, IT SEEMS REASONABLE TO ANALYZE THE CLOSURES AS RIGID BODIES; HOWEVER THIS NEEDS ESTABLISHED IN DESIGN.

ALL SOIL PROFILES, LIKE STRUCTURE GEOMETRIES, ARE TAKEN FROM ORIGINAL DRAWINGS. FOR A (FEASIBILITY) RELIABILITY ANALYSIS THIS 50-YEAR OLD DATA SHOULD BE SUFFICIENT.

CREEP WILL BE ESTABLISHED BY LINE OF CREEP METHOD & DETERMINED BY CTWALL.

FILE NOMENCLATURE:

MXXX DRR OR MXXX UNR

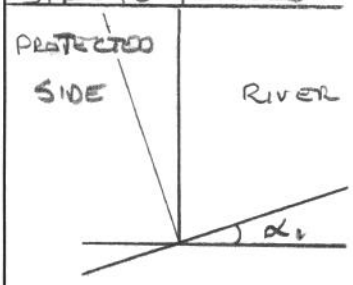
MXXX ≡ MONOLITH NUMBER

DR ≡ DRAINED SOIL STRENGTHS

UN ≡ UNDRAINED SOIL STRENGTHS

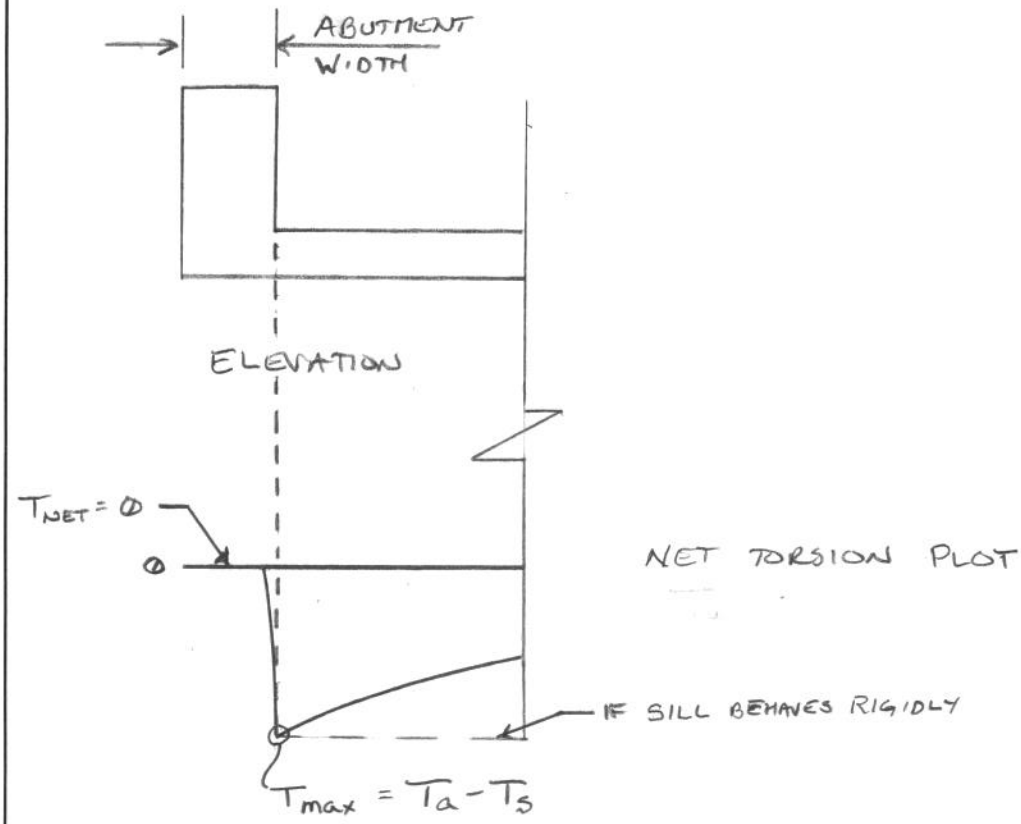
LAST "R" ≡ RIGID BODY ASSUMPTION (i.e. ALL LOADS ARE UNIFORM).

SPREAD FOOTERS:



ROTATION AT ABUTMENT DUE TO T_a

ROTATION AT SILL DUE TO T_s $(< T_a)$



→ NOTE: THE BENEFIT OF RESISTANCE FROM SOIL UNDER TOE IS IGNORED.

→ NOTE: THE STABILITY OF THE ABUTMENTS IS DEPENDENT UPON THEIR CONNECTION TO THE REST OF THE MONOLITH. AFTER PEAK LOADING, IT IS RECOMMENDED THAT THE FOOTERS BE INSPECTED FOR CRACKS, AND ANY DELAMINATION BE REMEDIED.

SPREAD FOOTERS:

THOUGH THE FOOTER WILL BE ANALYZED RIGIDLY IN STABILITY CALCS, THE LOCAL LOAD AT THE ABUTMENTS CREATES A TWISTING OF THE FOOTER'S CROSS-SECTION. THIS TWISTING IS NOT PRESENT IN THE FOOTER'S CROSS-SECTION UNDER THE STOP LOGS. FOR A SPREAD FOOTER, THE TWISTING IS THE SOLE RESULT OF THE DRIVING WATER LOAD AT THE ABUTMENT, AS ALL UPLIFT AND RESISTING FORCES ARE EQUIVALENT BETWEEN THE TWO CROSS-SECTION LOCATIONS.

MONOLITH 10: ABUTMENT LENGTH = 2'

ABUTMENT TORSION: $\frac{1}{2}\gamma_{H_2O}\left[(9'-9'')^3/3 + (6'-7'')^2\left(\frac{10'}{2'}\right)(5.36')\right] = \underline{45.9\ K\text{-}ft/ft.}$

$(9'-9'') - (6'-7'') + (6'-7'')/3 = 5.36'$

DEDUCT LOAD ON SILL:

$-\left(\frac{1}{2}\gamma\left[(9'-9'')^3/3 - (6'-7'')^2(5.36')\right]\right) = \underline{-2.4\ K\text{-}ft/ft}$

LOAD FACTORS = 1 FOR RELIABILITY

$\underline{\underline{T_u = T_{max} = 43.5\ K\text{-}ft}}$

TORSIONAL THRESHOLD (τ_ℓ):

USING ACI 11.6.1: $\phi = 1$ FOR RELIABILITY

$\tau_\ell = \phi\sqrt{f'_c}\left(A_{cp}^2/p_{cp}\right)$ $f'_c = 3{,}000$ psi

$w = 11'-0''$ $h = 1'-3''$

$A_{cp} = (11')(1.25') = 13.75\ ft^2 = 1980\ in^2$

$p_{cp} = 2(11' + 1.25') = 294\ in$

$\underline{\underline{\tau_\ell = 60.9\ K\text{-}ft/ft}}$

NEED H_2O 2' BELOW TOW FOR FS = 3, 2.5' WITH 18% REDUCTION IN STRENGTH DUE TO DETERIORATION

$\underline{\underline{\tau_\ell > T_u \quad OK.}}$ $\underline{\underline{FS = 1.40}}$

FS < 1.5 ; FS > 1.0. OK FOR RISK ANALYSIS, BUT NOT GOOD ENOUGH TO PASS FOR FUTURE CONDITIONS. ACI CRITERIA NOT MET. ADEQUATE LOAD FACTORS NOT USED.

ANY TORSIONAL SHEAR STRESS BEYOND $\frac{1}{4}T_{CR}$ MIGHT BE ADEQUATELY HANDLED BY REBAR AT CRITICAL MOMENT IF FS FOR FLEXURE ≥ 1.5. LONGITUDINAL REBAR IS ASSUMED ADEQUATE (ACI 11.6.3.7).

PM 4/30/08
181/437

US Army Corps of Engineers

Probability of Failure

CID MO Monolith 10
Torsion in Pile Cap

Comp by: FRS
Chkd by: ~~K Mark~~ PM

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. Situation

1. CID MO ~~M32 sill~~ M10 FOOTER does not meet the strength 1.5 factor of safety for which it has been determined 99.8% reliability can be assigned. See mathcad analysis of the wall for existing condition strength check.
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength.
3. Material Properties used:

   Mean Concrete Strength, $f'_{cM} := 3750 \cdot psi$

   Mean Steel Strength, $F_{yM} := 50 \cdot ksi$

4. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete Grade 40 is 14%.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength
FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength
FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength
FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength
ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength
ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. Caclulating Factors of Safety

Condition under consideration from strength check: Tu for Pile Cap (from hand-computation analysis). $T_u := 43.5 kip \cdot ft$

Design Concrete Strength

$\phi_B := 1.0$ Strength Reduction Factors not used in Risk and Uncertainty Analysis

$w_{cap} := 11ft$ Width of Pile Cap

$T_{toe} := 1.25ft$ Thickness of concrete section

$b := 12 \frac{in}{ft}$ 1 ft strip of wall analyzed

$A_{cp} := T_{toe} \cdot w_{cap} = 1.98 \times 10^3 \cdot in^2$

$P_{cp} := 2 \cdot (T_{toe} + w_{cap}) = 294 \cdot in$

Mean Concrete Strength

$\phi\tau_{lM} := \phi_B \cdot \sqrt{f'_{cM} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}} \right)$ $\phi\tau_{lM} = 68.048 \cdot kip \cdot ft$

$FS_D := \frac{\phi\tau_{lM}}{T_u}$ $FS_D = 1.564$

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM} \cdot 0.14$ $f'_{cU} = 4275 \cdot psi$

$\phi\tau_{lU} := \phi_B \cdot \sqrt{f'_{cU} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}} \right)$ $\phi\tau_{lU} = 72.66 \cdot kip \cdot ft$

$FS_{fcu} := \frac{\phi\tau_{lU}}{T_u}$ $FS_{fcu} = 1.67$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\qquad f'_{cL} = 3225 \cdot psi$

$\phi\tau_{lL} := \phi_B \cdot \sqrt{f'_{cL} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}} \right)$ $\qquad \phi\tau_{lL} = 63.11 \cdot kip \cdot ft$

$FS_{fcl} := \frac{\phi\tau_{lL}}{T_u}$ $\qquad FS_{fcl} = 1.451$

VI. Probability of Failure Calculation

$\Delta F_{fc} := FS_{fcu} - FS_{fcl}$ $\qquad \Delta F_{fc} = 0.22$

$\sigma_F := \sqrt{\left(\frac{\Delta F_{fc}}{2} \right)^2}$ $\qquad \sigma_F = 0.11$

$V_F := \frac{\sigma_F}{FS_D}$ $\qquad V_F = 0.07$

$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}} \right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\qquad \beta_{LN} = 6.349$ Taylor Series method. See ETL 1110-2-547, pg B-8.

$R := cnorm(\beta_{LN})$ $\qquad R = 100 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\qquad P_F = 1.08 \times 10^{-8} \cdot \%$

MONOLITH 10: STA 5+23.95

PER GEOTECH DATA:

	DRAINED	UNDRAINED
FOUNDATION = BLANKET	$\phi = 22°$ $C = 0$	$C = 600$ psf

$\gamma_{SAT} = 115$ pcf $\gamma_m = 110$ pcf

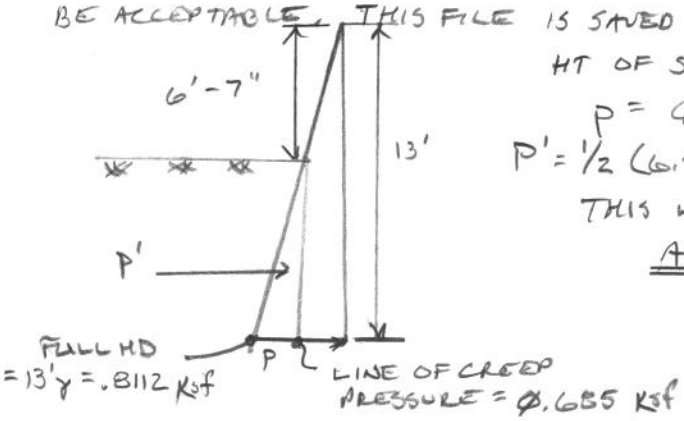
ULTIMATE BEARING, $q_u = 3400$ psf

RESULTS:

$q_{max} = 1.06$ ksf FOR FS = 2 $q_a = 1.7$ ksf

$q_a > q_{max}$ OK

$FS_{SLIDE} = 1.35$ WITH DRAINED STRENGTHS. THIS CASE ASSUMES SOME AMOUNT OF HORIZONTAL AND UPLIFT RELIEF WITH THE LINE OF CREEP METHOD. IT IS THE ENGINEER'S JUDGMENT TO TEST THE SLIDING WITH FULL HORIZONTAL PRESSURE (NO CHANGE MADE TO UPLIFT). SINCE THIS IS A VERY CONSERVATIVE ANALYSIS, A SLIDING FS > 1.1 WILL BE ACCEPTABLE. THIS FILE IS SAVED AS M10 FLHD.



HT OF SOIL = 13' − (6'-7") = 6.417'

p = 0.8112 − 0.685 = 0.1262 ksf

P' = ½ (6.417')(p) = 0.40 ksf

THIS WILL BE A SURCHARGE APPLIED AT EL 747.37'

CTWALL $FS_{SLIDE} = 1.18$ OK

MONOLITH 18: STA 8+67.90

PER GEOTECH DATA:

	DRAINED	UNDRAINED
USE FOUNDATION = BLANKET	$\phi = 22°$	$C = 0.6$ ksf

$\gamma_{SAT} = 115$ pcf $\gamma_m = 110$ pcf

$q_u = 3663$ psf.

MONOLITH 32: STA 14+80.69

PER GEOTECH DATA:

	DRAINED	UNDRAINED
USE FOUNDATION = BLANKET	$\phi = 22°$	$C = 0.6$ ksf

$\gamma_{SAT} = 115$ pcf $\gamma_m = 110$ pcf

$q_u = 40{,}000$ psf

NOTE: ANALYSIS IN CTWALL DOES NOT ACCOUNT FOR 'Z' SHAPE OF MONOLITH (IN PLAN). THUS ACTUAL SLIDING FS IS > CTWALL'S COMPUTATION.

SPREAD FOUNDATION

MONOLITH 10 CONSISTS OF A FLOOD WALL AND CLOSURE ON ONE FOOTING. IT IS ASSUMED THAT THE FOOTER BEHAVES RIGIDLY.

THEREFORE THE CONCENTRATION OF H_2O LOADS AT ABUTMENTS WILL NOT CAUSE A CONCERN; THEY WILL DISTRIBUTE ACROSS THE MONOLITH LENGTH. FURTHERMORE, THE DIFFERENCE BETWEEN A RESOLVED TOTAL FORCE AND ECCENTRICITY FROM C.G. WILL NOT BE A CONCERN, IT ONLY MEANS THAT IF THE MONOLITH BEGINS TO SLIDE, IT WILL ALSO ROTATE (IN PLAN). THIS WILL ACTUALLY HELP STABILITY BY INCREASING SIDE FRICTION (IGNORED IN ANALYSIS) FROM BEARING ON ADJACENT MONOLITHS.

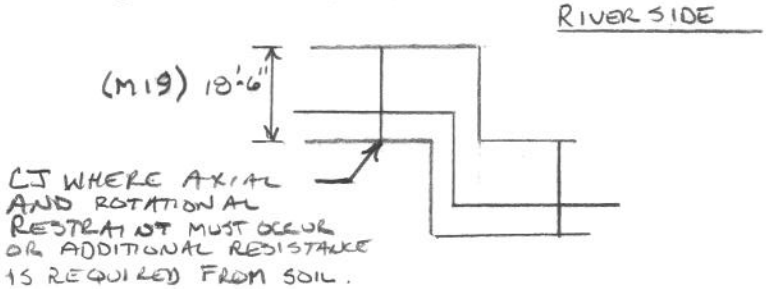
MONOLITH 10 IS THE ONLY MONOLITH WITH STOP LOG GAP ON A SPREAD FOOTING WITHOUT "Z" GEOMETRY.

→ A GENERAL 2-D STABILITY ANALYSIS WILL BE SUFFICIENT FOR CALCULATING SLIDING FS, BEARING, AND OVERTURNING.

MONOLITHS 18 & 32:

THESE ARE THE ONLY TWO MONOLITHS ON SPREAD FOOTINGS WITH STOP LOG GAPS. THEY ARE 'Z' SHAPED IN PLAN AND HAVE SIMILAR FOOTPRINT SIZES. THE GAP WIDTH IS THE SAME (17'-6"), BUT THE HEIGHT OF M32 (11'-11") IS MUCH TALLER THAN M18 (8'-7¾").

IT WILL BE INVESTIGATED HERE IF THIS 'Z' SHAPE WILL REQUIRE MORE SHEAR STRENGTH FOR STABILITY.



PDM 4/30/08

SEE COMMENTS ON DRAINED W/ ADDED WATER PRESS RUN

191/437

********** Echoprint of Input Data **********

Date: **/01/17 Time: 10.13.57

Company name:

Project name:
CID MO Closures
Project location:

Wall location:
M10 Drained Case, Rigid Body Analysis
Computed by: FRS

Structural geometry data:

Elevation of top of stem (ELTS)	=	758.25 ft
Height of stem (HTS)	=	9.75 ft
Thickness top of stem (TTS)	=	1.00 ft
Thickness bottom of stem (TBS)	=	1.50 ft
Dist. of batter at bot. of stem (TBSR)	=	.00 ft
Depth of heel (THEEL)	=	3.25 ft
Distance of batter for heel (BTRH)	=	.00 ft
Depth of toe (TTOE)	=	1.25 ft
Width of toe (TWIDTH)	=	3.50 ft
Distance of batter for toe (BTRT)	=	.00 ft
Width of base (BWIDTH)	=	11.00 ft
Depth of key (HK)	=	2.00 ft
Width of bottom of key (TK)	=	1.00 ft
Dist. of batter at bot. of key (BTRK)	=	.50 ft

Structure coordinates:

x (ft)	y (ft)
.00	745.25
.00	748.50
6.00	748.50
6.50	758.25
7.50	758.25
7.50	748.50
11.00	748.50
11.00	747.25
1.50	747.25
1.00	745.25

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = .150 kcf

Driving side soil property data:

Phi	c	Moist Unit wt.	Saturated unit wt.	Delta	Elev. soil

192/237

(deg)	(ksf)	(kcf)	(kcf)	(deg)	(ft)
22.00	.000	.110	.115	11.00	752.17

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	.00	500.00
2	.00	.00
3	.00	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1493.81	752.17
2	6.19	752.17

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
22.00	.000	.110	.115	751.67	.00

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	7.50	751.67
2	507.50	751.67

Foundation property data:
phi for soil-structure interface = 22.00 (deg)
c for soil-structure interface = .000 (ksf)
phi for soil-soil interface = 22.00 (deg)
c for soil-soil interface = .000 (ksf)

Water data:
Driving side elevation = 758.25 ft
Resisting side elevation = 751.67 ft
Unit weight of water = .0624 kcf
Seepage pressures computed by Line of Creep method.

Minimum required factors of safety:
Sliding FS = 1.33
Overturning = 75.00% base in compression

Crack options:
- o Crack depth is to be calculated
- o Computed cracks *will* be filled with water

Strength mobilization factor = .6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

Project name: CID MO Closures

```
***************    *** Satisfied ***
* Overturning *    Required base in comp. =  75.00 %
***************    Actual base in comp.   =  96.14 %
                   Overturning ratio      =   1.54

Xr (measured from toe) =   3.53 ft
Resultant ratio        =  .3205
Stem ratio             =  .3182
Base pressure at x=  10.58 ft from toe =     .0000 ksf
Base pressure at toe                   =    1.0569 ksf

***********    *** Satisfied ***
* Sliding *    Min. Required =   1.33
***********    Actual FS     =   1.35
```

********** Output Results **********

Date: **/01/17 Time: 10.13.57

Company name:

Project name:
CID MO Closures
Project location:

Wall location:
M10 Drained Case, Rigid Body Analysis
Computed by: FRS

```
**************************
**  Overturning Results  **
**************************

Solution converged in  1 iterations.

SMF used to calculate K's =     .6667
Alpha for the SMF         = -34.3032
Calculated earth pressure coefficients:
  Driving side at rest K    =    .4788
  Driving side at rest Kc   =    .8497
  Resisting side at rest K  =    .6254
  Resisting side at rest Kc =    .7908
  At-rest K's for resisting side calculated.

Depth of cracking =      .00 ft
```

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
758.25	.0000
752.17	.3794
745.25	.6850

Earth pressures:

Elevation (ft)	Pressure (ksf)
752.17	.0000
745.25	.2347

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
751.67	.0000
747.25	.3564
747.25	.5106
745.25	.6688

Earth pressures:

Elevation (ft)	Pressure (ksf)
751.67	.0000
747.25	.0950

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
747.25	1.7357
745.25	1.7357

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
.00	.6850
1.00	.6688
1.50	.5106
11.00	.3564

** Forces and moments **

Part	Force (kips)		Mom. Arm	Moment
	Vert.	Horiz.	(ft)	(ft-k)

Structure:				
Structure weight.....	4.266		-5.34	-22.79
Structure, driving side:				
Moist soil.....	.000		.00	.00
Saturated soil.....	2.572		-7.95	-20.45
Water above structure.....	.000		.00	.00
Water above soil.....	2.407		-7.83	-18.84
External vertical loads....	.000		.00	.00
Ext. horz. pressure loads..		.000	.00	.00
Ext. horz. line loads.....		.000	.00	.00
Structure, resisting side:				
Moist soil.....	.000		.00	.00
Saturated soil.....	1.276		-1.75	-2.23
Water above structure.....	.000		.00	.00
Water above soil.....	.000		.00	.00
Driving side:				
Effective earth loads.....		.812	.31	.25
Shear (due to delta).....	.158		-11.00	-1.74
Horiz. surcharge effects...		.000	.00	.00
Water loads.....		4.836	2.51	12.16
Resisting side:				
Effective earth loads.....		-.210	1.48	-.31
Balancing earth load.....		-3.471	-.99	3.45
Water loads.....		-1.967	-.04	.09
Foundation:				
Vertical force on base.....	-5.589		-3.53	19.70
Uplift.....	-5.090		-6.03	30.71
** Statics Check ** SUMS =	.000	.000		.00

Angle of base = 10.30 degrees
Normal force on base = 6.119 kips
Shear force on base = 2.416 kips
Max. available shear force = 2.861 kips

Base pressure at x= 10.58 ft from toe = .0000 ksf
Base pressure at toe = 1.0569 ksf

Xr (measured from toe) = 3.53 ft
Resultant ratio = .3205
Stem ratio = .3182
Base in compression = 96.14 %
Overturning ratio = 1.54

Volume of concrete = 1.05 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

```
***********************
**  Sliding Results  **
***********************
```

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	.000	3.632
2	1.153	2.407
3	.000	.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	.3794	.6850		
2			.0000	.6850
2			11.0000	.3564
3	.0000	.3564		

Points of sliding plane:
Point 1 (left), x = .00 ft, y = 745.25 ft
Point 2 (right), x = 11.00 ft, y = 747.25 ft

Depth of cracking = .00 ft

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	-35.859	11.813	3.810	11.813	6.287
2	10.305	11.180	9.091	11.180	5.822
3	36.877	7.365	1.497	7.365	1.312

Wedge number	Net force (kips)
1	-4.215
2	2.822
3	1.393
SUM =	.000

```
+-----------------------------+
| Factor of safety =    1.351 |
+-----------------------------+
```

PDM 4/30/08

197/437

********** Echoprint of Input Data **********

Date: **/01/17 Time: 10.11.24

Company name:

Project name:
CID Missouri Closures
Project location:

Wall location:
Monolith 10 Drained Case, Rigid Body Analysis
Computed by: FRS + FULL HEAD ON U/S FACE.

Structural geometry data:

Elevation of top of stem (ELTS)	=	758.25	ft
Height of stem (HTS)	=	9.75	ft
Thickness top of stem (TTS)	=	1.00	ft
Thickness bottom of stem (TBS)	=	1.50	ft
Dist. of batter at bot. of stem (TBSR)	=	.00	ft
Depth of heel (THEEL)	=	3.25	ft
Distance of batter for heel (BTRH)	=	.00	ft
Depth of toe (TTOE)	=	1.25	ft
Width of toe (TWIDTH)	=	3.50	ft
Distance of batter for toe (BTRT)	=	.00	ft
Width of base (BWIDTH)	=	11.00	ft
Depth of key (HK)	=	2.00	ft
Width of bottom of key (TK)	=	1.00	ft
Dist. of batter at bot. of key (BTRK)	=	.50	ft

Structure coordinates:

x (ft)	y (ft)
.00	745.25
.00	748.50
6.00	748.50
6.50	758.25
7.50	758.25
7.50	748.50
11.00	748.50
11.00	747.25
1.50	747.25
1.00	745.25

NOTE: X=0 is located at the left-hand side
of the structure. The Y values correspond
to the actual elevation used.

Structural property data:
Unit weight of concrete = .150 kcf

Driving side soil property data:

Phi	c	Moist Unit wt.	Saturated unit wt.	Delta	Elev. soil

(deg)	(ksf)	(kcf)	(kcf)	(deg)	(ft)
22.00	.000	.110	.115	11.00	752.17

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	.00	500.00
2	.00	.00
3	.00	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1493.81	752.17
2	6.19	752.17

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
22.00	.000	.110	.115	751.67	.00

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	7.50	751.67
2	507.50	751.67

Foundation property data:
phi for soil-structure interface = 22.00 (deg)
c for soil-structure interface = .000 (ksf)
phi for soil-soil interface = 22.00 (deg)
c for soil-soil interface = .000 (ksf)

Water data:
Driving side elevation = 758.25 ft
Resisting side elevation = 751.67 ft
Unit weight of water = .0624 kcf
Seepage pressures computed by Line of Creep method.

Horizontal line load data:

TO ESTABLISH FULL HD ON DRIVING SIDE

Elevation (ft)	Force (kips)
747.37	.40

CONCUR
CONSERVATIVE TO ASSUME NO CRACK (DRAINED) + FULL PRESSURE ON R.S + THIS WILL MAKE UPLIFT INCONSISTENT. SUGGEST CHANGING CRACK OPTION TO FORCE A CRACK TO HEEL OR DON'T ADD LOAD

Minimum required factors of safety:
Sliding FS = 1.33
Overturning = 75.00% base in compression

Crack options:

USED ON PREVIOUS RUN

199/437

CAN BE CHANGED TO FORCE CRACK

- o Crack depth is to be calculated
- o Computed cracks *will* be filled with water

Strength mobilization factor = .6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

Project name: CID Missouri Closures

```
***************    *** Satisfied ***
* Overturning *    Required base in comp. =  75.00 %
***************    Actual base in comp.   =  93.85 %
                   Overturning ratio      =   1.53

Xr (measured from toe) =   3.44 ft
Resultant ratio        =  .3128
Stem ratio             =  .3182
Base pressure at x=  10.32 ft from toe =     .0000 ksf
Base pressure at toe                   =    1.0827 ksf

***********    *** Not Satisfied ***
* Sliding *    Min. Required =   1.33
***********    Actual FS     =   1.18
```

CONCUR.

THIS MAY BE CONSERVATIVE
→ EXTRA H_2O PRESS WITH DRAINED CONDITION
– COMPUTED CRACK CASE (NO CRACK) IS MORE REALISTIC

To increase stability try one or a combination of the following:

1. Increase the base width
2. Slope the base of the structure
3. Lower the wall base
4. Add a key

********** Output Results **********

Date: **/01/17 Time: 10.11.24

Company name:

Project name:
CID Missouri Closures
Project location:

Wall location:
Monolith 10 Drained Case, Rigid Body Analysis
Computed by: FRS

****
** Overturning Results **

```
***************************

Solution converged in  1 iterations.

SMF used to calculate K's =     .6667
Alpha for the SMF          = -34.3032
Calculated earth pressure coefficients:
  Driving side at rest K    =     .4788
  Driving side at rest Kc   =     .8497
  Resisting side at rest K  =     .6254
  Resisting side at rest Kc =     .7908
  At-rest K's for resisting side calculated.

Depth of cracking =      .00 ft

** Driving side pressures **

  Water pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
       758.25      .0000
       752.17      .3794
       745.25      .6850

  Earth pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
       752.17      .0000
       745.25      .2347

** Resisting side pressures **

  Water pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
       751.67      .0000
       747.25      .3564
       747.25      .5106
       745.25      .6688

  Earth pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
       751.67      .0000
       747.25      .0950

  Balancing earth pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
       747.25     1.9357
       745.25     1.9357

** Uplift pressures **

  Water pressures:
     x-coord.   Pressure
```

(ft)	(ksf)
.00	.6850
1.00	.6688
1.50	.5106
11.00	.3564

** Forces and moments **

Part	Force (kips) Vert.	Force (kips) Horiz.	Mom. Arm (ft)	Moment (ft-k)
Structure:				
Structure weight.....	4.266		-5.34	-22.79
Structure, driving side:				
Moist soil.....	.000		.00	.00
Saturated soil.....	2.572		-7.95	-20.45
Water above structure.....	.000		.00	.00
Water above soil.....	2.407		-7.83	-18.84
External vertical loads....	.000		.00	.00
Ext. horz. pressure loads..		.000	.00	.00
Ext. horz. line loads.....		.400	.12	.05
Structure, resisting side:				
Moist soil.....	.000		.00	.00
Saturated soil.....	1.276		-1.75	-2.23
Water above structure.....	.000		.00	.00
Water above soil.....	.000		.00	.00
Driving side:				
Effective earth loads.....		.812	.31	.25
Shear (due to delta).....	.158		-11.00	-1.74
Horiz. surcharge effects...		.000	.00	.00
Water loads.....		4.836	2.51	12.16
Resisting side:				
Effective earth loads.....		-.210	1.48	-.31
Balancing earth load.....		-3.871	-1.00	3.87
Water loads.....		-1.967	-.04	.09
Foundation:				
Vertical force on base.....	-5.589		-3.44	19.23
Uplift.....	-5.090		-6.03	30.71
** Statics Check ** SUMS =	.000	.000		.00

Angle of base = 10.30 degrees
Normal force on base = 6.191 kips
Shear force on base = 2.809 kips
Max. available shear force = 2.890 kips

Base pressure at x= 10.32 ft from toe = .0000 ksf
Base pressure at toe = 1.0827 ksf

Xr (measured from toe) = 3.44 ft
Resultant ratio = .3128
Stem ratio = .3182
Base in compression = 93.85 %
Overturning ratio = 1.53

Volume of concrete = 1.05 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

```
***********************
**  Sliding Results  **
***********************
```

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	.000	3.379
2	1.553	2.407
3	.000	.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	.3794	.6850		
2			.0000	.6850
2			11.0000	.3564
3	.0000	.3564		

Points of sliding plane:
Point 1 (left), x = .00 ft, y = 745.25 ft
Point 2 (right), x = 11.00 ft, y = 747.25 ft

Depth of cracking = .00 ft

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	-37.846	11.279	3.544	11.279	6.003
2	10.305	11.180	9.091	11.180	5.822
3	35.753	7.565	1.560	7.565	1.348

Wedge number	Net force (kips)
1	-4.155
2	2.711
3	1.445
SUM =	.000

```
+-----------------------------+
| Factor of safety =    1.181 |
+-----------------------------+
```

204/437 TDM 4/30/08

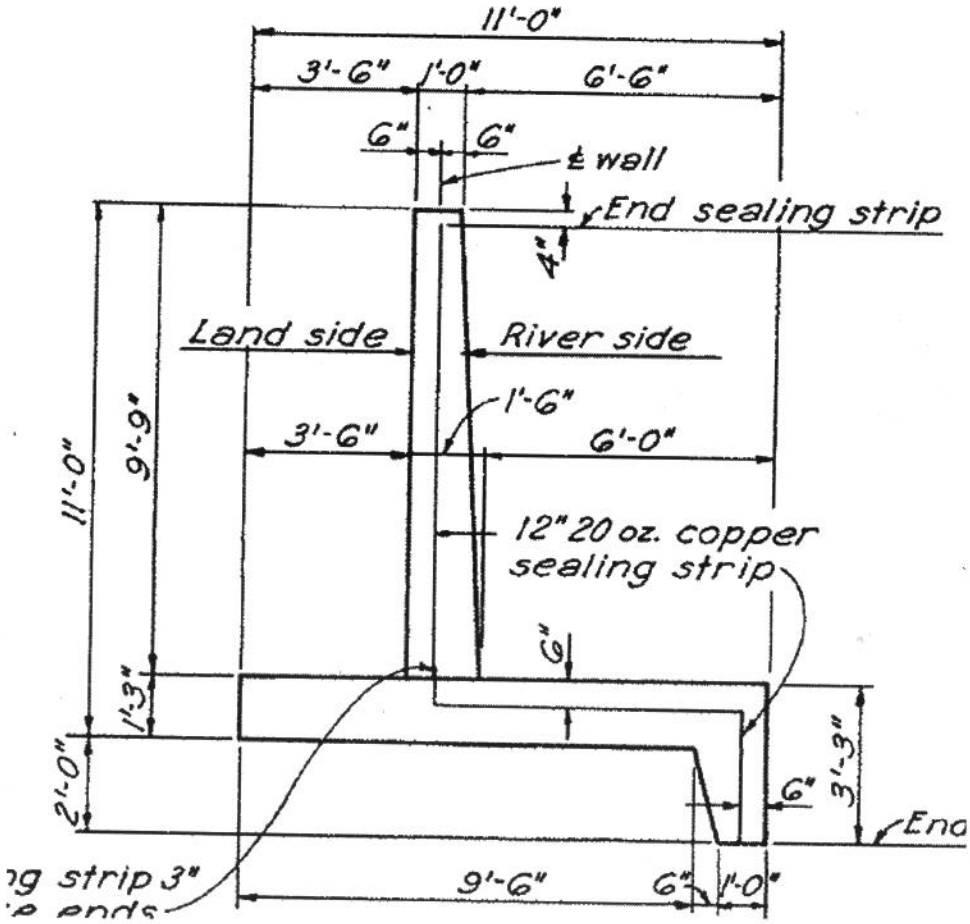
US Army Corps of Engineers

CID Missouri
Monolith 10 Stoplog Gap
Footer and Wall Strength Analysis

Kansas Citys Levees

Comp by: FRS
Chkd by: ~~[illegible]~~ P. MULLER

DRAINED



Typical Floodwall Section

Assumptions — Taken from Record Drawings File No. A-10-1176

1. The length of the base in bearing compression must be less than (D+E+F+G+H). Otherwise, the bending moment, M_H will be incorrect.

2. Equations in this file are established for a horizontal footer with uniform thickness.

3. Resisting Side Water EL is the EL of the top of resisting grade.

4. Temperature and Shrinkage steel assumed sufficient.

Variables

$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $ksf \equiv \frac{1000lb}{ft^2}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$

CTWALL INPUT FILE NAME: M10DRR.OUT

Elevation of top of stem (ELTS).....	$ELTS := 758.25ft$
Height of stem (HTS).....	$HTS := 9ft + 9in$
Thickness top of stem (TTS).....	$TTS := 1.0ft$
Thickness bottom of stem (TBS).....	$TBS := 1.5ft$
Dist. of batter at bot.of stem LS (TBSR).....	$TBSR := 0ft$
Depth of heel (THEEL).....	$THEEL := 3ft + 3in$
Distance of batter for heel (BTRH).....	$BTRH := 0ft$ *
Depth of toe (TTOE).....	$TTOE := 1ft + 3in$ *
Width of toe (TWIDTH).....	$TWIDTH := 3ft + 6in$
Distance of batter for toe (BTRT).....	$BTRT := 0ft$ *
Width of base (BWIDTH).....	$BWIDTH := 11ft$
Depth of key (HK).....	$HK := 2ft + 0in$ *
Width of bottom of key (TK).....	$TK := 1ft$
Dist. of batter at bot. of key (BTRK).....	$BTRK := 0ft + 6in$
Driving side soil elevation (ELSTDS).....	$ELSTDS := 752.17ft$
Resisting side soil and water elevation (ELSTRS).....	$ELSTRS := 751.67ft$
Driving side water elevation (WATELD).....	$WATELD := 758.25ft$

$A := WATELD - ELSTDS$

$B := HTS - (ELTS - ELSTDS)$

$C := THEEL$

$D := TK$

$E := BTRK$

$F := BWIDTH - (TWIDTH + TBS + BTRK + TK)$

$G := TBS$

$H := TWIDTH$

$I := TTOE + BTRT$

$J := HTS - (ELTS - ELSTRS)$

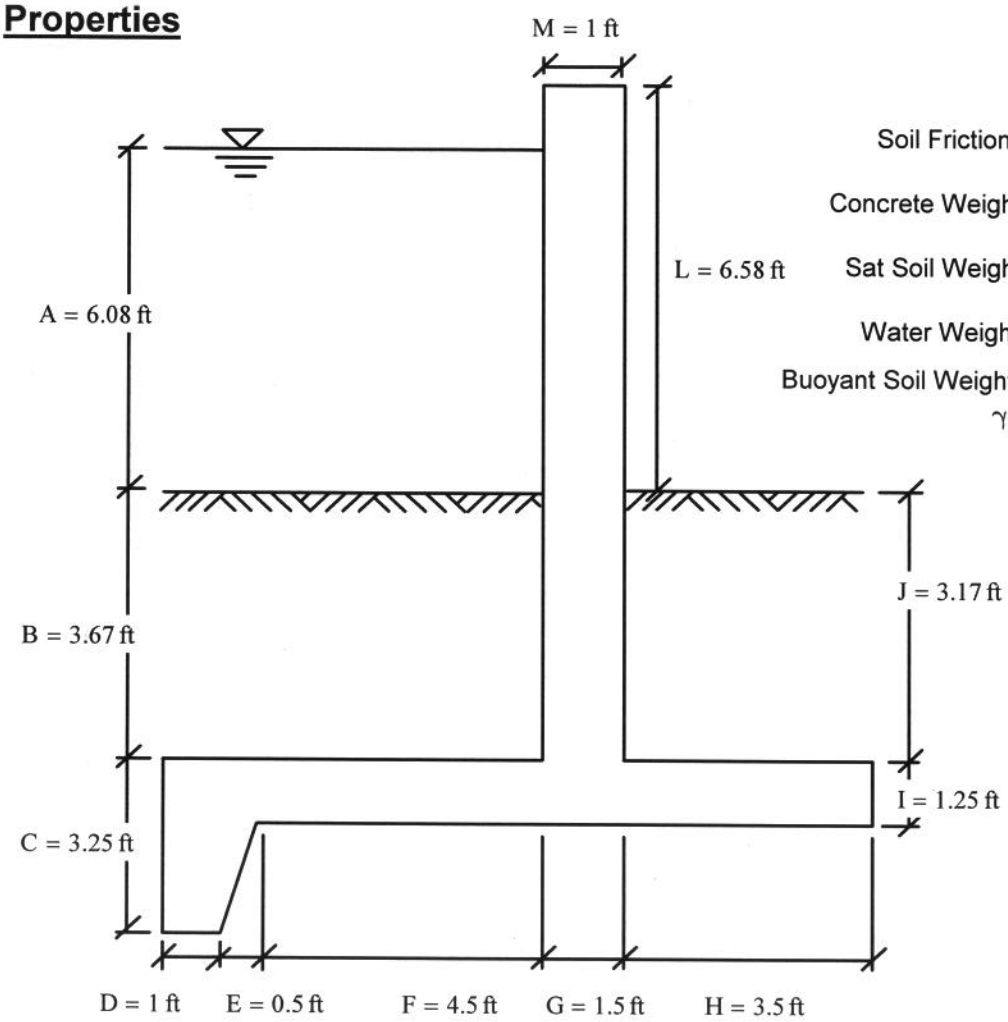
$L := ELTS - ELSTRS$

$M := TTS$

* Equations in this file are established for a horizontal footer with uniform thickness.

286/437

Properties



Soil Friction $\phi := 22 \cdot deg$

Concrete Weight $\gamma_c := 150pcf$

Sat Soil Weight $\gamma_{sat} := 115pcf$

Water Weight $\gamma_w := 62.4pcf$

Buoyant Soil Weight

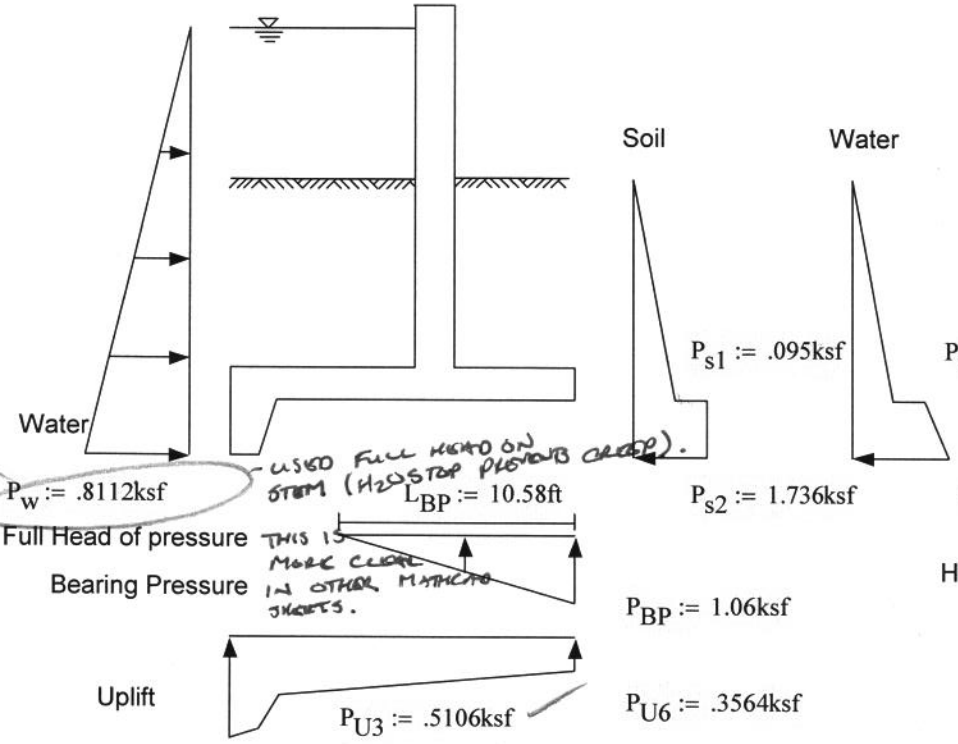
$\gamma_b := \gamma_{sat} - \gamma_w = 52.6 \cdot pcf$

$Base := D + E + F + G + H = 11\,ft$

$Verify := if(BWIDTH = Base, "OKAY", \overline{"Check\ Dimensions"})$

Verify = "OKAY"

Pressures below were taken from corresponding CTWALL output.



$P_{s1} := .095ksf$ $P_{w1} := .3564ksf$

$P_{w2} := .5106ksf$

$P_w := .8112ksf$ USED FULL HEAD ON STEM (H2O STOP PROVIDES CREEP).

$L_{BP} := 10.58ft$ $P_{s2} := 1.736ksf$ $P_{w3} := .6688ksf$

THIS IS MORE CLOSE IN OTHER MATHCAD SHEETS.

Hand Check of Soil Pressure:

$P_{BP} := 1.06ksf$

$P_{U3} := .5106ksf$ $P_{U6} := .3564ksf$

INCONSISTENT $P_{U1} := 0.685ksf$ $P_{U2} := .6688ksf$

Driving Soil Pressure at bottom of Key: $P_{s3} := .2347ksf$

Driving Soil Pressure at top of footer:

$$P_{s4} := \frac{B}{B + C} \cdot P_{s3} = 0.12 \cdot ksf$$

Driving Soil Pressure at base of footer:

$$P_{s5} := \frac{B}{(B + I)} \cdot P_{s3} = 0.18 \cdot ksf$$

Note, the presence of any cutoff wall could impact the uplift pressures. Reducing the uplift would cause an uncoservative design for the heel.

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete. As concrete ages it continues to hydrate and gain compressive strength. FEMA 310, Section 4.2.4.4, states, "Unless calculated otherwise the expected strength shall be assumed equal to the nominal strength multiplied by 1.25." Using FEMA's guidance the concrete strength becomes 3750 psi. However, 3000 psi is used conservatively until a reliability calculation is performed
- The Portland Cement Association pamphlet, *Engineered Concrete Structures*, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Concrete Properties $f'_c := 3000 \cdot psi$ Steel Properties $F_y := 40.0ksi$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength	$\phi_V := 1.0$
Flexural Strength	$\phi_B := 1.0$

Load Factors

Dead and Live Load Factor	$\gamma_L := 1.0$
Hydraulic Load Factor	$\gamma_H := 1.0$
Extreme Case Factor	$\gamma_X := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.

Load Multiplication Factor EM 1110-2-2104 (3-1)

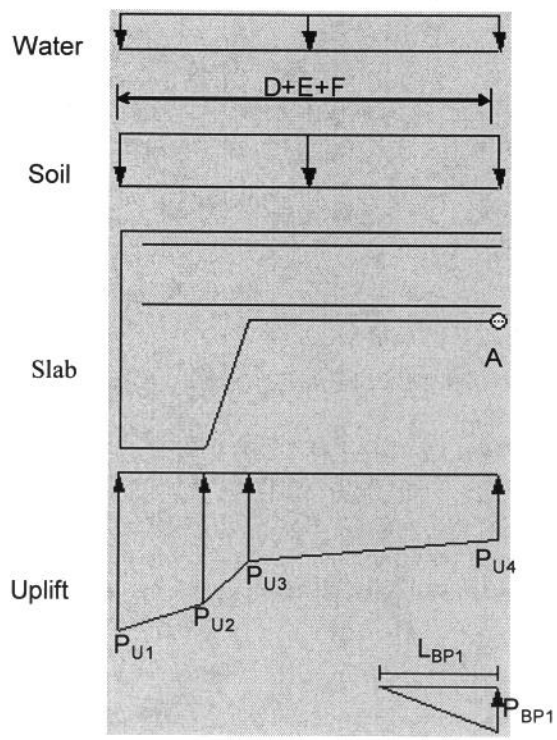
Hydraulic Factor EM 1110-2-2104 (3-2)

Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Reinforcement Checks

- HEEL

O Location where moment is taken about.



$$W_w := \gamma_w \cdot (A + B) \cdot (D + E + F)$$

$$W_w = 3650 \frac{lb}{ft}$$

$$W_s := \gamma_b \cdot B \cdot (D + E + F)$$

$$W_s = 1158.25 \frac{lb}{ft}$$

Drag on edge of Key due to tendency to rotate:

$$\delta_p := \left[\frac{1}{2} \cdot \left(P_{s4} + P_{s3}\right) \cdot C \cdot \tan(\phi)\right] = 235.81 \frac{lb}{ft}$$

$$\delta_d := \text{if}\left(L_{BP} < BWIDTH, \delta_p, 0\right) = 235.81 \frac{lb}{ft}$$

$$P_{U4} := \left(P_{U3} - P_{U6}\right) \cdot \left(\frac{G + H}{F + G + H}\right) + P_{U6}$$

$$P_{U4} = 437.56 \frac{lb}{ft^2}$$

$$L_{BP1} := \text{if}\left[\left[L_{BP} - (G + H)\right] \geq 0ft, \left[L_{BP} - (G + H)\right], 0\right]$$

$$L_{BP1} = 5.58\,ft$$

Bearing Pressure, P_{BP1} acting on the heel at location "A"

$$P_{BP1} := \frac{L_{BP1}}{L_{BP}} \cdot P_{BP} \qquad P_{BP1} = 559 \frac{lb}{ft^2}$$

Heel and Key Centroid

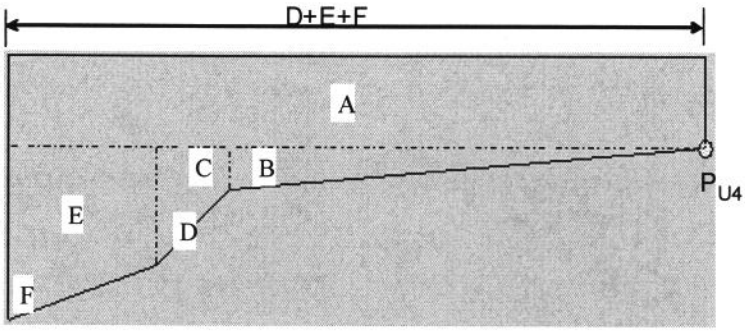
$$X_H := \frac{(D + E + F) \cdot I \cdot \left(\frac{D + E + F}{2}\right) + (C - I) \cdot D \cdot \left(E + F + \frac{D}{2}\right) + \left[\frac{(C - I) \cdot E}{2}\right] \cdot \left(F + \frac{2 \cdot E}{3}\right)}{(D + E + F) \cdot I + (C - I) \cdot D + \frac{(C - I) \cdot E}{2}} \qquad X_H = 3.59\,ft$$

- HEEL check (cont'd)

Heel and Key Weight

$$W_H := \left[(D + E + F)\cdot I + (C - I)\cdot D + \frac{(C - I)\cdot E}{2}\right]\cdot\gamma_c \qquad W_H = 1500\,\frac{lb}{ft}$$

Uplift Centroid



AREA	CENTROID
$A_A := (D + E + F)\cdot P_{U4}$	$C_A := \frac{D + E + F}{2}$
$A_B := \frac{F\cdot(P_{U3} - P_{U4})}{2}$	$C_B := \frac{2\cdot F}{3}$
$A_C := E\cdot(P_{U3} - P_{U4})$	$C_C := F + \frac{E}{2}$
$A_D := \frac{E\cdot(P_{U2} - P_{U3})}{2}$	$C_D := F + \frac{2\cdot E}{3}$
$A_E := D\cdot(P_{U2} - P_{U4})$	$C_E := \left(\frac{D}{2} + E + F\right)$
$A_F := \frac{D\cdot(P_{U1} - P_{U2})}{2}$	$C_F := \left(\frac{2\cdot D}{3} + E + F\right)$

$$X_U := \frac{A_A\cdot C_A + A_B\cdot C_B + A_C\cdot C_C + A_D\cdot C_D + A_E\cdot C_E + A_F\cdot C_F}{A_A + A_B + A_C + A_D + A_E + A_F} \qquad X_U = 3.24\,ft$$

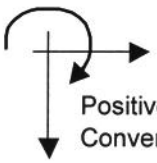
Uplift on Heel $\quad W_U := A_A + A_B + A_C + A_D + A_E + A_F \qquad W_U = 3105\,\frac{lb}{ft}$

- HEEL check (cont'd)

Loading: Note, since the amount of base in compression is > 75%, bearing of water-side of key against soil due to rotation should be marginally different from counter-moment produced from sliding force on key. Water loads on each side of key are also similar and so their effects on the heel are ignored.

Bending

$$M_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[W_U \cdot X_U + \frac{P_{BP1} \cdot L_{BP1}}{2} \cdot \frac{L_{BP1}}{3} - \left[(W_w + W_s) \cdot \left(\frac{D + E + F}{2} \right) + W_H \cdot X_H + \delta_d \cdot (D + E + F) \right] \right]$$



Positive Sign Convention

$$M_H = -8275.73 \cdot \frac{ft \cdot lb}{ft}$$

Note:
IF: M_H is < 0
THEN: Steel in top of heel is in tension

$$M_{uH} := |M_H| \qquad M_{uH} = 8.3 \cdot \frac{kip \cdot ft}{ft}$$

Shear

$$V_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left(W_w + W_s + \delta_d + W_H - W_U - \frac{P_{BP1} \cdot L_{BP1}}{2} \right)$$

$$V_H = 1879.6 \frac{lb}{ft} \qquad V_{uH} := |V_H| \qquad V_{uH} = 1.9 \cdot \frac{kip}{ft}$$

Capacity:
Flexural Capacity

Top Steel: $A_{Heel1} := .44 \frac{in^2}{ft}$ $C_{CH1} := 3.375in$ Bottom Steel: $A_{Heel2} := .35 \frac{in^2}{ft}$ $C_{CH2} := 3.375in$

$$A_{s1} := A_{Heel1} \qquad d_{H1} := I - C_{CH1} \qquad A_{s2} := A_{Heel2} \qquad d_{H2} := I - C_{CH2}$$

$$b := 12 \frac{in}{ft} \quad a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \quad a1 = 0.58 \cdot in \qquad b_c := 12 \frac{in}{ft} \quad a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b_c} \quad a2 = 0.46 \cdot in$$

$$\phi M_{H1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{H1} - \frac{a1}{2} \right) \qquad \phi M_{H2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{H2} - \frac{a2}{2} \right)$$

$$\phi M_{H1} = 16.63 \cdot \frac{kip \cdot ft}{ft} \qquad \phi M_{H2} = 13.3 \cdot \frac{kip \cdot ft}{ft}$$

$$\phi M_H := if(M_H > 0, \phi M_{H2}, \phi M_{H1}) \qquad \phi M_H = 16.63 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{H1} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c2} := \phi_V \cdot 2 \cdot b_c \cdot d_{H2} \cdot \sqrt{f'_c \cdot psi}$$

$$\phi V_{c1} = 15281 \frac{lb}{ft} \qquad \phi V_{c2} = 15281 \frac{lb}{ft}$$

$$\phi V_H := if(M_H > 0, \phi V_{c2}, \phi V_{c1}) \qquad \phi V_H = 15.28 \cdot \frac{kip}{ft}$$

211/437

- HEEL check (cont'd)

Factors of Safety

Bending $\phi M_H = 16.63 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_1 := \frac{\phi M_H}{M_{uH}}$

$M_{uH} = 8.28 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_1 = 2.01$

$Check1 := if(\phi M_H > 1.5 M_{uH}, "OKAY", "NO GOOD")$ $\quad Check1 = "OKAY"$

Shear $\phi V_H = 15.28 \cdot \frac{kip}{ft}$ $\quad FS_2 := \frac{\phi V_H}{V_{uH}}$

$V_{uH} = 1.88 \cdot \frac{kip}{ft}$ $\quad FS_2 = 8.13$

$Check2 := if(\phi V_H > 1.5 V_{uH}, "OKAY", "NO GOOD")$ $\quad Check2 = "OKAY"$

Controlling Factor of Safety

$FS_H := \min(FS_1, FS_2)$ $\quad FS_H = 2.01$

Controlling Mechanism

$Comment_H := if(FS_1 > FS_2, "Shear in Heel", if(M_H > 0, "Flexural Bottom Steel in Heel", "Flexural Top Steel in Heel"))$

$Comment_H = "Flexural Top Steel in Heel"$

Key Check

Hydrostatic Load is similar to each side of key, and thus neglected in analysis.

$M_K := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[(P_{s5} - P_{s2}) \cdot \frac{(C - I)^2}{2} + \frac{1}{2} \cdot (P_{s3} - P_{s5}) \cdot \frac{2(C - I)^2}{3} \right] = -3.04 \cdot kip \cdot \frac{ft}{ft}$ $\quad M_K > 0$ means water-side steel is in tension.

$V_K := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[(P_{s5} - P_{s2}) \cdot (C - I) + \frac{1}{2}(P_{s3} - P_{s5}) \cdot (C - I) \right] = -3062.23 \frac{lb}{ft}$ $\quad M_{uK} := |M_K| = 3042.35\ lb$

$V_{uK} := |V_K| = 3062.23 \frac{lb}{ft}$

Steel in Tension: $A_{Key} := 0.16 \frac{in^2}{ft}$ $\quad C_K := 3.375in$ $\quad d_K := (D + E) - C_K$

$b := 12 \frac{in}{ft}$ $\quad a_k := \frac{A_{Key} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_k = 0.21 \cdot in$ $\quad \phi M_K := \phi_B A_{Key} \cdot F_y \cdot \left(d_K - \frac{a_k}{2} \right)$ $\quad \phi M_K = 7.74 \cdot \frac{kip \cdot ft}{ft}$

Shear:

$\phi V_K := \phi_V \cdot 2 \cdot b \cdot d_K \cdot \sqrt{f'_c \cdot psi}$ $\quad \phi V_K = 19225 \frac{lb}{ft}$

$FS_{KM} := \frac{\phi M_K}{M_K} = -2.55$ $\quad Check_M := if(\phi M_K > 1.5 M_{uK}, "OKAY", "NO GOOD")$

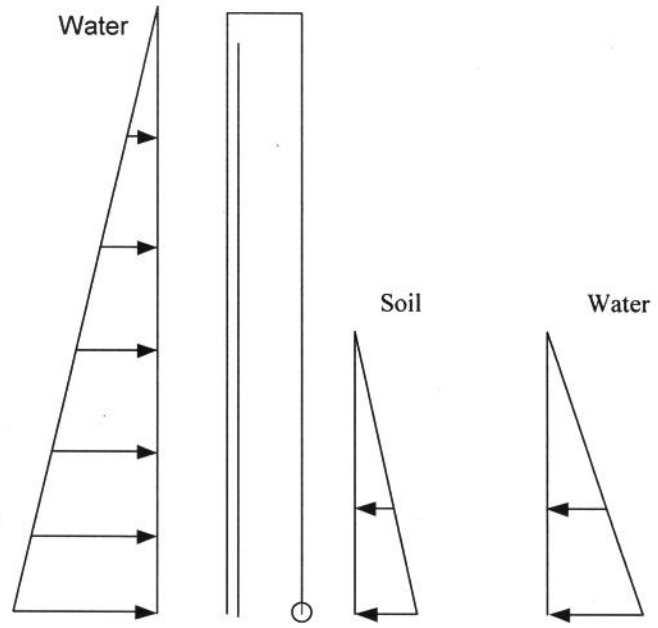
$Check_M = "OKAY"$

$FS_{KV} := \frac{\phi V_K}{V_K} = -6.28$ $\quad Check_V := if(\phi V_K > 1.5 V_{uK}, "OKAY", "NO GOOD")$

$Check_V = "OKAY"$

L +

- STEM Check



Note:
P_{ss2} = Resistive Soil pressure present at base of stem
P_{ws2} = Resistive Water pressure present at base of stem

$P_{ws1} := (A + B)\cdot\gamma_w$

$P_{ws1} = 608\,\frac{lb}{ft^2}$

$P_{ss2} := \frac{J}{I + J}\cdot P_{s1}$ $\qquad P_{ws2} := J\cdot\gamma_w$

Driving Soil Pressure at base of Stem:

$P_{ss1} := P_{s4} = 0.12\cdot ksf$

$P_{ss2} = 0.07\cdot ksf$ $\qquad P_{ws2} = 0.2\cdot ksf$

Waterside Steel: $A_{Stem1} := .51\,\frac{in^2}{ft}$ $\quad C_{CS1} := 2.81in$ $\quad d_{S1} := G - C_{CS1}$

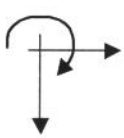
Resistive Side Steel: $A_{Stem2} := 0.2\cdot\frac{in^2}{ft}$

Loading:

Bending

$$M_{uS} := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A + B)}{2}\cdot\frac{A + B}{3} + \frac{P_{ss1}\cdot B^2}{2\cdot 3} - \frac{\left(P_{ss2} + P_{ws2}\right)\cdot\left(J^2\right)}{2\cdot 3}\right]$$

Note:
IF: M_{uS} is > 0
THEN: Riverside steel is in tension



$M_{uS} = 9.5\cdot\frac{kip\cdot ft}{ft}$

Positive Sign Convention

Shear

$$V_{uS} := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A + B)}{2} + \frac{P_{ss1}\cdot B}{2} - \frac{\left(P_{ss2} + P_{ws2}\right)\cdot(J)}{2}\right]$$

$V_{uS} = 2.8\cdot\frac{kip}{ft}$

- STEM Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Stem1}$

$b := 12\frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a1 = 0.67 \cdot in$

$\phi M_S := \phi_B A_{s1} \cdot F_y \cdot \left(d_{S1} - \frac{a1}{2}\right)$ $\quad \phi M_S = 25.26 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

NICE!

$\mu := 1.0$

$\phi V_S := \phi_V \cdot \left(A_{Stem2} \cdot F_y \cdot \mu\right) = 8000\frac{lb}{ft}$ $\quad \phi V_S = 8 \cdot \frac{kip}{ft}$

Factors of Safety

Bending

$\phi M_S = 25.26 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_3 := \frac{\phi M_S}{M_{uS}}$

$M_{uS} = 9.47 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_3 = 2.67$

$Check3 := if\left(\phi M_S > 1.5 M_{uS}, "OKAY", "NO GOOD"\right)$ $\quad Check3 = "OKAY"$

Shear

$\phi V_S = 8 \cdot \frac{kip}{ft}$ $\quad FS_4 := \frac{\phi V_S}{V_{uS}}$

$V_{uS} = 2.77 \cdot \frac{kip}{ft}$ $\quad FS_4 = 2.89$

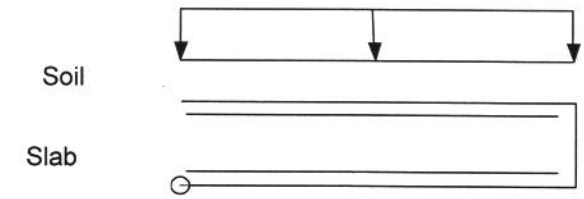
$Check4 := if\left(\phi V_S > 1.5 V_{uS}, "OKAY", "NO GOOD"\right)$ $\quad Check4 = "OKAY"$

Controlling Factor of Safety

$FS_S := \min\left(FS_3, FS_4\right)$ $\quad FS_S = 2.67$

Note: No bar cut-offs checks are made for the reliability Study.

- TOE Check

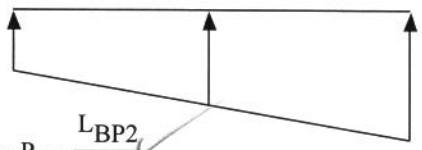


$W_s := \gamma_{sat} \cdot H \cdot (J)$ $\quad W_s = 1275.92 \frac{lb}{ft}$

$L_{BP2} := if(H \le L_{BP}, H, L_{BP})$

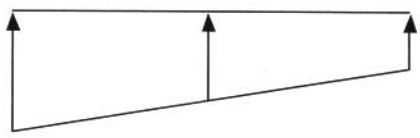
$L_{BP2} = 3.5\,ft$

Bearing Pressure



$P_{BP2} := P_{BP} - P_{BP} \cdot \frac{L_{BP2}}{L_{BP}}$ $\quad P_{BP} = 1060 \frac{lb}{ft^2}$

$P_{BP2} = 709.34 \frac{lb}{ft^2}$



Uplift

$P_{U6} = 356.4 \frac{lb}{ft^2}$

$$P_{U5} := \left[\left[(P_{U3} - P_{U6}) \cdot \frac{H}{F + G + H} + P_{U6}\right]\right]$$

$P_{U5} = 413.21 \frac{lb}{ft^2}$

Bearing Pressure

$$L_{bp} := \frac{(L_{BP2} \cdot P_{BP2}) \cdot \left(\frac{L_{BP2}}{2}\right) + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2} \cdot \left(\frac{2L_{BP}}{3}\right)}{L_{BP2} \cdot P_{BP2} + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2}}$$

$L_{bp} = 2.8\,ft$

$$W_{bp} := L_{BP2} \cdot P_{BP2} + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2}$$

$W_{bp} = 3096 \frac{lb}{ft}$

Uplift

$$L_u := \frac{H \cdot P_{U6} \cdot \frac{H}{2} + \frac{(P_{U5} - P_{U6}) \cdot H}{2} \cdot \frac{H}{3}}{H \cdot P_{U6} + \frac{(P_{U5} - P_{U6}) \cdot H}{2}}$$

$L_u = 1.71\,ft$

$$W_u := H \cdot P_{U6} + \frac{(P_{U5} - P_{U6}) \cdot H}{2}$$

$W_u = 1346.82 \frac{lb}{ft}$

Toe Check (continued)

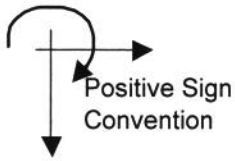
Loading:

Bending

$$M_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[\left[W_s + (H \cdot I) \cdot \gamma_c \right] \cdot \frac{H}{2} - \left(W_{bp} \cdot L_{bp} + W_u \cdot L_u \right) \right]$$

Note:
IF: $M_T < 0$
THEN: Bottom steel is in tension

$$M_T = -7.6 \cdot \frac{kip \cdot ft}{ft} \qquad M_{uT} := \left| M_T \right| \qquad M_{uT} = 7.59 \cdot \frac{kip \cdot ft}{ft}$$



Shear

$$V_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[W_s + (H \cdot I) \cdot \gamma_c - W_{bp} - W_u \right]$$

$$V_T = -2.51 \cdot \frac{kip}{ft} \qquad V_{uT} := \left| V_T \right| \qquad V_{uT} = 2.51 \cdot \frac{kip}{ft}$$

Capacity:

Flexural Capacity

Area of bottom (tension) steel: $A_{Toe} := .352 \frac{in^2}{ft}$ $\quad C_{CT} := 3.375in$ $\quad d_T := I - C_{CT} = 11.63 \cdot in$

$$b := 12 \frac{in}{ft} \qquad A_{s1} := A_{Toe}$$

$$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a1 = 0.46 \cdot in$$

$$\phi M_T := \phi_B A_{s1} \cdot F_y \cdot \left(d_T - \frac{a1}{2} \right) \qquad \phi M_T = 13.37 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_T := \phi_V \cdot 2 \cdot b \cdot d_T \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_T = 15.28 \cdot \frac{kip}{ft}$$

Toe Check (continued)

Factors of Safety

Bending

$\phi M_T = 13.37 \cdot \frac{kip \cdot ft}{ft}$

$M_{uT} = 7.59 \cdot \frac{kip \cdot ft}{ft}$

$FS_5 := \frac{\phi M_T}{M_{uT}}$

$FS_5 = 1.76$

$Check5 := if(\phi M_T > 1.5 \cdot M_{uT}, "OKAY", "NO GOOD")$

Check5 = "OKAY"

Shear

$\phi V_T = 15.28 \cdot \frac{kip}{ft}$

$V_{uT} = 2.51 \cdot \frac{kip}{ft}$

$FS_6 := \frac{\phi V_T}{V_{uT}}$

$FS_6 = 6.09$

$Check6 := if(\phi V_T > 1.5 V_{uT}, "OKAY", "NO GOOD")$

Check6 = "OKAY"

Controlling Factor of Safety

$FS_T := \min(FS_5, FS_6)$

$FS_T = 1.76$

End of Analysis

PDM 4/30/08
217/437

********** Echoprint of Input Data **********

Date: **/01/17 Time: 10.16.16

Company name:

Project name:
CID MO Closures
Project location:

Wall location:
M10 Undrained Case, Rigid Body Analysis
Computed by: FRS

Structural geometry data:

Elevation of top of stem (ELTS)	=	758.25 ft
Height of stem (HTS)	=	9.75 ft
Thickness top of stem (TTS)	=	1.00 ft
Thickness bottom of stem (TBS)	=	1.50 ft
Dist. of batter at bot. of stem (TBSR)	=	.00 ft
Depth of heel (THEEL)	=	3.25 ft
Distance of batter for heel (BTRH)	=	.00 ft
Depth of toe (TTOE)	=	1.25 ft
Width of toe (TWIDTH)	=	3.50 ft
Distance of batter for toe (BTRT)	=	.00 ft
Width of base (BWIDTH)	=	11.00 ft
Depth of key (HK)	=	2.00 ft
Width of bottom of key (TK)	=	1.00 ft
Dist. of batter at bot. of key (BTRK)	=	.50 ft

Structure coordinates:

x (ft)	y (ft)
.00	745.25
.00	748.50
6.00	748.50
6.50	758.25
7.50	758.25
7.50	748.50
11.00	748.50
11.00	747.25
1.50	747.25
1.00	745.25

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = .150 kcf

Driving side soil property data:

Phi	c	Moist Unit wt.	Saturated unit wt.	Delta	Elev. soil

(deg)	(ksf)	(kcf)	(kcf)	(deg)	(ft)
.00	.600	.110	.115	.00	752.17

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	.00	500.00
2	.00	.00
3	.00	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1493.81	752.17
2	6.19	752.17

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
.00	.600	.110	.115	751.67	.00

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	7.50	751.67
2	507.50	751.67

Foundation property data:
phi for soil-structure interface = .00 (deg)
c for soil-structure interface = .300 (ksf)
phi for soil-soil interface = .00 (deg)
c for soil-soil interface = .600 (ksf)

Water data:
Driving side elevation = 758.25 ft
Resisting side elevation = 751.67 ft
Unit weight of water = .0624 kcf
Seepage pressures computed by Line of Creep method.

Minimum required factors of safety:
Sliding FS = 1.33
Overturning = 75.00% base in compression

Crack options:
o Crack *is* down to bottom of heel
o Computed cracks *will* be filled with water

Strength mobilization factor = .6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

Project name: CID MO Closures

```
***************    *** Satisfied ***
* Overturning *    Required base in comp. =  75.00 %
***************    Actual base in comp.   =  78.63 %
                   Overturning ratio      =   1.33

Xr (measured from toe) =   2.88 ft
Resultant ratio        =  .2621
Stem ratio             =  .3182
Base pressure at x=   8.65 ft from toe =     .0000 ksf
Base pressure at toe                   =    1.0609 ksf

***********    *** Satisfied ***
* Sliding *    Min. Required =   1.33
***********    Actual FS     =   5.89
```

********** Output Results **********

Date: **/01/17 Time: 10.16.16

Company name:

Project name:
CID MO Closures
Project location:

Wall location:
M10 Undrained Case, Rigid Body Analysis
Computed by: FRS

```
***************************
**  Overturning Results  **
***************************
```

Solution converged in 1 iterations.

```
SMF used to calculate K's =    .6667
Alpha for the SMF         =    .0000
Calculated earth pressure coefficients:
  Driving side at rest K    =    .0000
  Driving side at rest Kc   =    .0000
  Resisting side at rest K  =   1.0000
  Resisting side at rest Kc =   1.0000
  At-rest K's for resisting side calculated.

Depth of cracking =     6.92 ft
```

Crack extends to bottom of base of structure.

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
758.25	.0000
745.25	.8112

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
751.67	.0000
747.25	.3921
747.25	.6147
745.25	.7878

Earth pressures:

Elevation (ft)	Pressure (ksf)
751.67	.0000
747.25	.1162

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
747.25	1.3735
745.25	1.3735

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
.00	.8112
1.00	.7878
1.50	.6147
11.00	.3921

** Forces and moments **

Part	Force (kips) Vert.	Force (kips) Horiz.	Mom. Arm (ft)	Moment (ft-k)
Structure:				
Structure weight.....	4.266		-5.34	-22.79
Structure, driving side:				
Moist soil.....	.000		.00	.00

Saturated soil.....	2.572		-7.95	-20.45
Water above structure.....	.000		.00	.00
Water above soil.....	2.407		-7.83	-18.84
External vertical loads....	.000		.00	.00
Ext. horz. pressure loads..		.000	.00	.00
Ext. horz. line loads.....		.000	.00	.00
Structure, resisting side:				
Moist soil.....	.000		.00	.00
Saturated soil.....	1.276		-1.75	-2.23
Water above structure.....	.000		.00	.00
Water above soil.....	.000		.00	.00
Driving side:				
Effective earth loads.....		.000	.00	.00
Shear (due to delta).....	.000		.00	.00
Horiz. surcharge effects...		.000	.00	.00
Water loads.....		5.273	2.33	12.29
Resisting side:				
Effective earth loads.....		-.257	1.47	-.38
Balancing earth load.....		-2.747	-1.01	2.77
Water loads.....		-2.269	-.08	.19
Foundation:				
Vertical force on base.....	-4.588		-2.88	13.23
Uplift.....	-5.932		-6.10	36.21

==========

** Statics Check ** SUMS =	.000	.000	.00

Angle of base = 10.30 degrees
Normal force on base = 5.005 kips
Shear force on base = 1.882 kips
Max. available shear force = 6.708 kips

Base pressure at x= 8.65 ft from toe = .0000 ksf
Base pressure at toe = 1.0609 ksf

Xr (measured from toe) = 2.88 ft
Resultant ratio = .2621
Stem ratio = .3182
Base in compression = 78.63 %
Overturning ratio = 1.33

Volume of concrete = 1.05 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Horizontal Vertical

Wedge Number	Loads (kips)	Loads (kips)
1	.000	.000
2	5.273	2.407
3	.000	.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	.0000	.0000		
2			.0000	.8112
2			11.0000	.3921
3	.0000	.3921		

Points of sliding plane:
Point 1 (left), x = .00 ft, y = 745.25 ft
Point 2 (right), x = 11.00 ft, y = 747.25 ft

Depth of cracking = 6.92 ft
Crack extends to bottom of base of structure.

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	.000	.000	.000	.000	.000
2	10.305	11.180	9.091	11.180	6.727
3	45.128	6.237	1.118	6.237	1.223

Wedge number	Net force (kips)
1	.000
2	-2.024
3	2.024
SUM =	.000

Factor of safety = 5.887

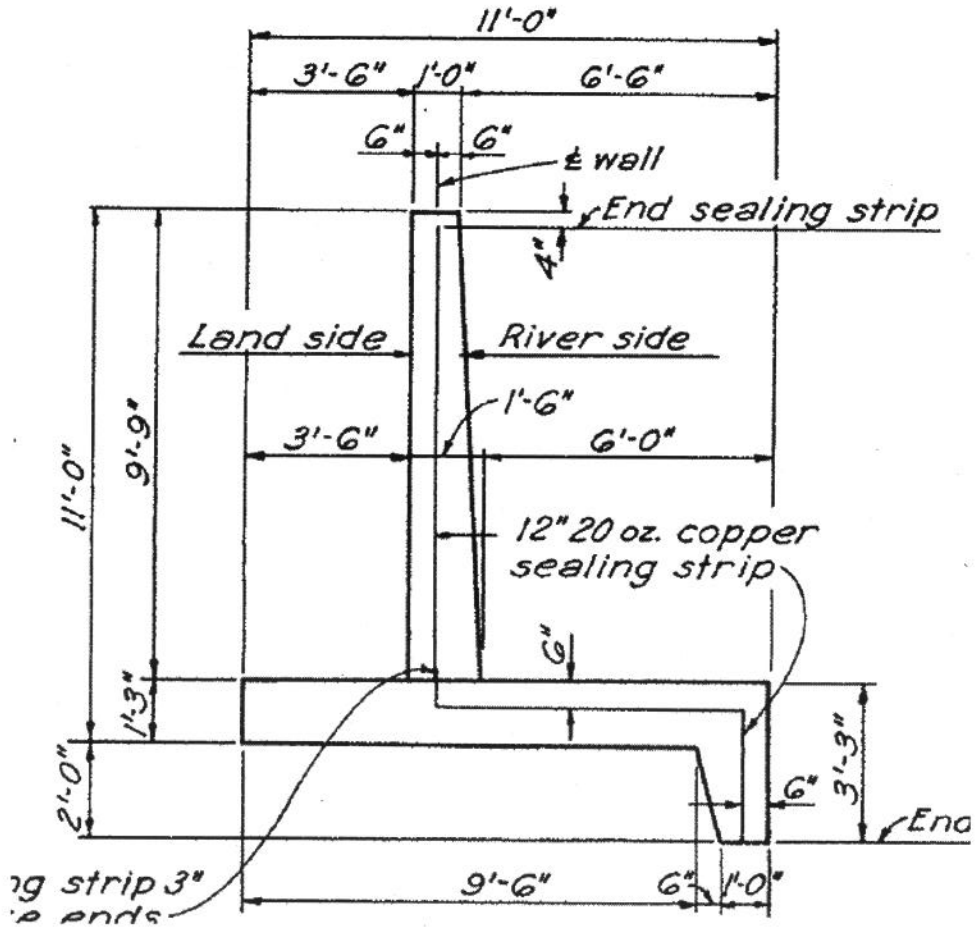
PDM 4/30/08
223/437

US Army Corps of Engineers

CID Missouri
Monolith 10 Stoplog Gap
Footer and Wall Strength Analysis
Kansas Citys Levees

Comp by: FRS
Chkd by: [redacted] PMILLER

UNDRAINED



Typical Floodwall Section

Assumptions — Taken from Record Drawings File No. A-10-1176

1. The length of the base in bearing compression must be less than (D+E+F+G+H). Otherwise, the bending moment, M_H will be incorrect.

2. Equations in this file are established for a horizontal footer with uniform thickness.

3. Resisting Side Water EL is the EL of the top of resisting grade.

4. Temperature and Shrinkage steel assumed sufficient.

Variables

$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $ksf \equiv \frac{1000lb}{ft^2}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$

CTWALL INPUT FILE NAME: M10UNR.OUT

Elevation of top of stem (ELTS).....	$ELTS := 758.25ft$
Height of stem (HTS).....	$HTS := 9ft + 9in$
Thickness top of stem (TTS).....	$TTS := 1.0ft$
Thickness bottom of stem (TBS).....	$TBS := 1.5ft$
Dist. of batter at bot.of stem LS (TBSR).....	$TBSR := 0ft$
Depth of heel (THEEL).....	$THEEL := 3ft + 3in$
Distance of batter for heel (BTRH).....	$BTRH := 0ft$ *
Depth of toe (TTOE).....	$TTOE := 1ft + 3in$ *
Width of toe (TWIDTH).....	$TWIDTH := 3ft + 6in$
Distance of batter for toe (BTRT).....	$BTRT := 0ft$ *
Width of base (BWIDTH).....	$BWIDTH := 11ft$
Depth of key (HK).....	$HK := 2ft + 0in$ *
Width of bottom of key (TK).....	$TK := 1ft$
Dist. of batter at bot. of key (BTRK).....	$BTRK := 0ft + 6in$
Driving side soil elevation (ELSTDS).....	$ELSTDS := 752.17ft$
Resisting side soil and water elevation (ELSTRS).....	$ELSTRS := 751.67ft$
Driving side water elevation (WATELD).....	$WATELD := 758.25ft$

$A := WATELD - ELSTDS$

$B := HTS - (ELTS - ELSTDS)$

$C := THEEL$

$D := TK$

$E := BTRK$

$F := BWIDTH - (TWIDTH + TBS + BTRK + TK)$

$G := TBS$

$H := TWIDTH$

$I := TTOE + BTRT$

$J := HTS - (ELTS - ELSTRS)$

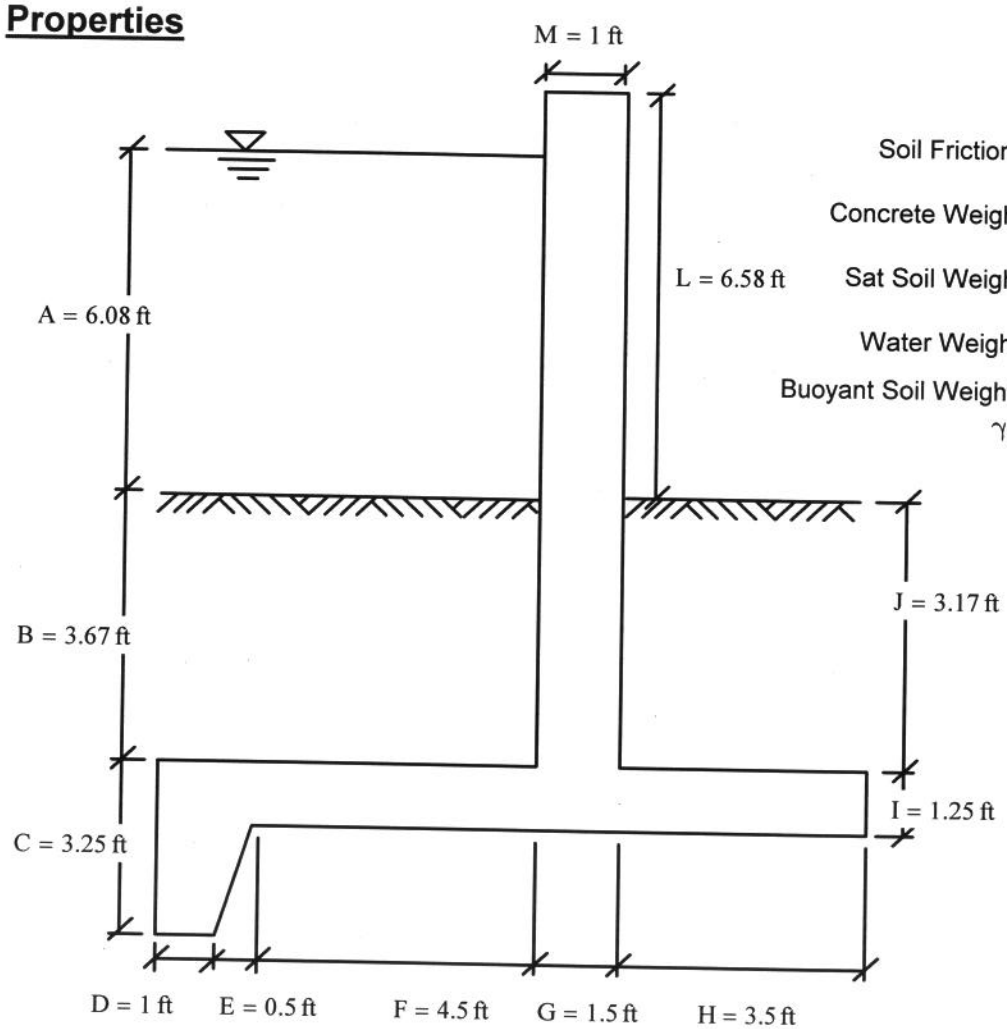
$L := ELTS - ELSTRS$

$M := TTS$

* Equations in this file are established for a horizontal footer with uniform thickness.

225/437

Properties



Soil Friction $\phi := 0 \cdot deg$ ✓

Concrete Weight $\gamma_c := 150pcf$

Sat Soil Weight $\gamma_{sat} := 115pcf$

Water Weight $\gamma_w := 62.4pcf$

Buoyant Soil Weight

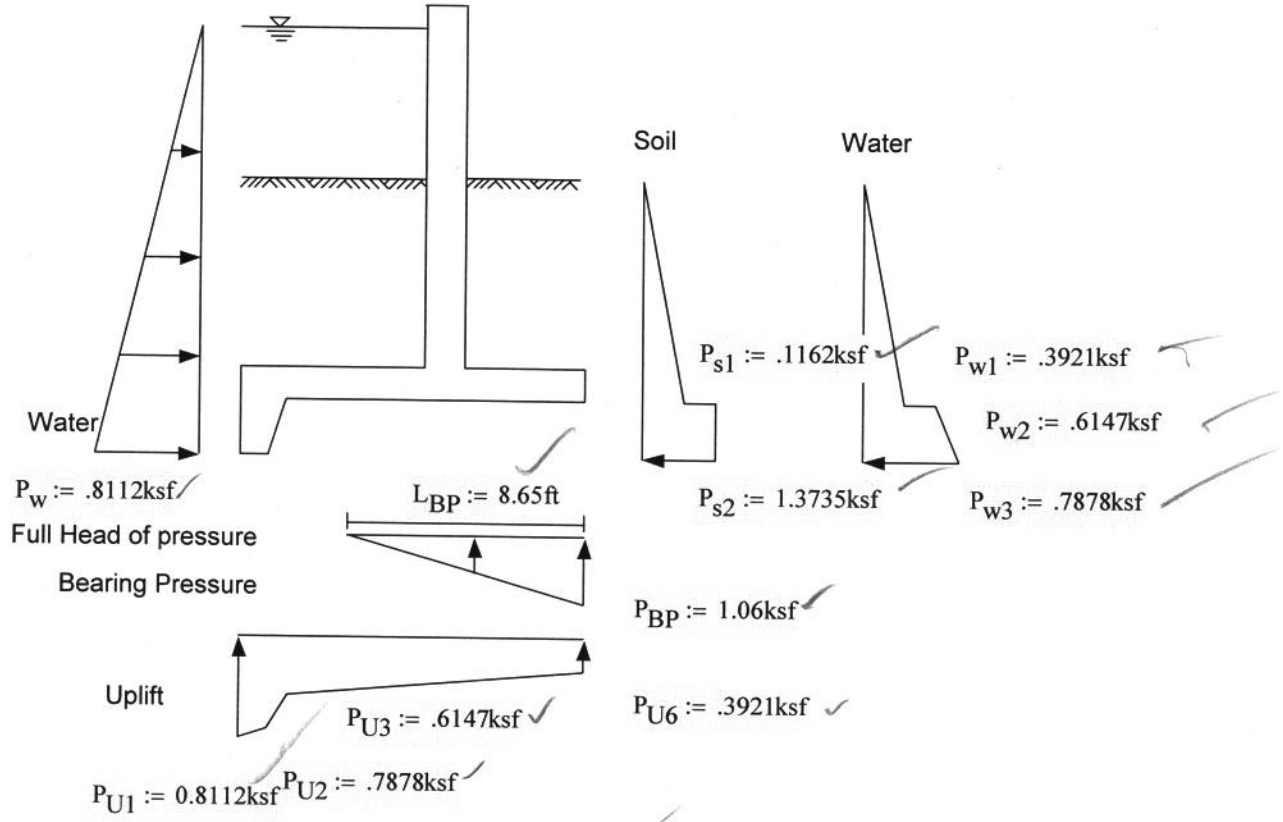
$\gamma_b := \gamma_{sat} - \gamma_w = 52.6 \cdot pcf$

$Base := D + E + F + G + H = 11\,ft$

$Verify := if(BWIDTH = Base, "OKAY", \overline{"Check\ Dimensions"})$

Verify = "OKAY"

Pressures below were taken from corresponding CTWALL output.



Driving Soil Pressure at bottom of Key: $P_{s3} := 0ksf$

Driving Soil Pressure at top of footer:

$$P_{s4} := \frac{B}{B + C} \cdot P_{s3} = 0 \cdot ksf$$

Driving Soil Pressure at base of footer:

$$P_{s5} := \frac{B}{(B + I)} \cdot P_{s3} = 0 \cdot ksf$$

Note, the presence of any cutoff wall could impact the uplift pressures. Reducing the uplift would cause an uncoservative design for the heel.

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete. As concrete ages it continues to hydrate and gain compressive strength. FEMA 310, Section 4.2.4.4, states, "Unless calculated otherwise the expected strength shall be assumed equal to the nominal strength multiplied by 1.25." Using FEMA's guidance the concrete strength becomes 3750 psi. However, 3000 psi is used conservatively until a reliability calculation is performed
- The Portland Cement Association pamphlet, *Engineered Concrete Structures*, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Concrete Properties $f'_c := 3000 \cdot psi$ Steel Properties $F_y := 40.0ksi$

Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength	$\phi_V := 1.0$
Flexural Strength	$\phi_B := 1.0$

Load Factors

Dead and Live Load Factor	$\gamma_L := 1.0$
Hydraulic Load Factor	$\gamma_H := 1.0$
Extreme Case Factor	$\gamma_X := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.

Load Multiplication Factor EM 1110-2-2104 (3-1)

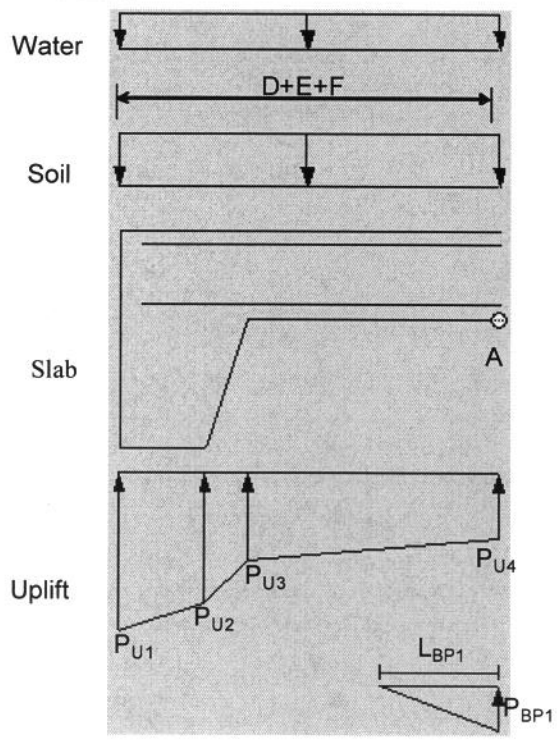
Hydraulic Factor EM 1110-2-2104 (3-2)

Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Reinforcement Checks

- HEEL

○ Location where moment is taken about.



$$W_w := \gamma_w \cdot (A + B) \cdot (D + E + F)$$

$$W_w = 3650 \frac{lb}{ft}$$

$$W_s := \gamma_b \cdot B \cdot (D + E + F)$$

$$W_s = 1158.25 \frac{lb}{ft}$$

Drag on edge of Key due to tendency to rotate:

$$\delta_p := \left[\frac{1}{2} \cdot \left(P_{s4} + P_{s3}\right) \cdot C \cdot \tan(\phi)\right] = 0$$

$$\delta_d := \text{if}\left(L_{BP} < BWIDTH, \delta_p, 0\right) = 0$$

$$P_{U4} := \left(P_{U3} - P_{U6}\right) \cdot \left(\frac{G + H}{F + G + H}\right) + P_{U6}$$

$$P_{U4} = 509.26 \frac{lb}{ft^2}$$

$$L_{BP1} := \text{if}\left[\left[L_{BP} - (G + H)\right] \geq 0ft, \left[L_{BP} - (G + H)\right], 0\right]$$

$$L_{BP1} = 3.65\,ft$$

Bearing Pressure, P_{BP1} acting on the heel at location "A"

$$P_{BP1} := \frac{L_{BP1}}{L_{BP}} \cdot P_{BF} \qquad P_{BP1} = 447 \frac{lb}{ft^2}$$

Heel and Key Centroid

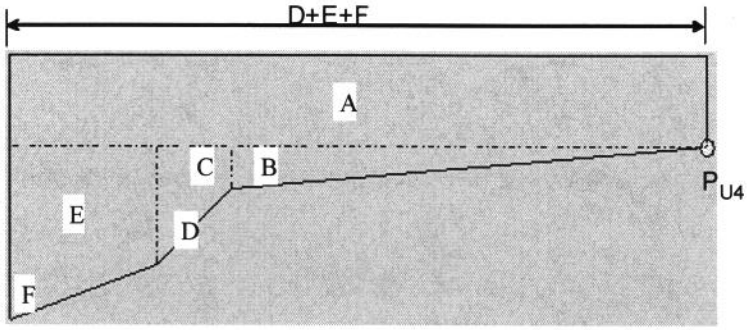
$$X_H := \frac{(D + E + F) \cdot I \cdot \left(\frac{D + E + F}{2}\right) + (C - I) \cdot D \cdot \left(E + F + \frac{D}{2}\right) + \left[\frac{(C - I) \cdot E}{2}\right] \cdot \left(F + \frac{2 \cdot E}{3}\right)}{(D + E + F) \cdot I + (C - I) \cdot D + \frac{(C - I) \cdot E}{2}} \qquad X_H = 3.59\,ft$$

- HEEL check (cont'd)

Heel and Key Weight

$$W_H := \left[(D + E + F)\cdot I + (C - I)\cdot D + \frac{(C - I)\cdot E}{2}\right]\cdot\gamma_c \qquad W_H = 1500\,\frac{lb}{ft}$$

Uplift Centroid



AREA

$$A_A := (D + E + F)\cdot P_{U4}$$

$$A_B := \frac{F\cdot(P_{U3} - P_{U4})}{2}$$

$$A_C := E\cdot(P_{U3} - P_{U4})$$

$$A_D := \frac{E\cdot(P_{U2} - P_{U3})}{2}$$

$$A_E := D\cdot(P_{U2} - P_{U4})$$

$$A_F := \frac{D\cdot(P_{U1} - P_{U2})}{2}$$

CENTROID

$$C_A := \frac{D + E + F}{2}$$

$$C_B := \frac{2\cdot F}{3}$$

$$C_C := F + \frac{E}{2}$$

$$C_D := F + \frac{2\cdot E}{3}$$

$$C_E := \left(\frac{D}{2} + E + F\right)$$

$$C_F := \left(\frac{2\cdot D}{3} + E + F\right)$$

$$X_U := \frac{A_A\cdot C_A + A_B\cdot C_B + A_C\cdot C_C + A_D\cdot C_D + A_E\cdot C_E + A_F\cdot C_F}{A_A + A_B + A_C + A_D + A_E + A_F} \qquad X_U = 3.24\,ft$$

Uplift on Heel

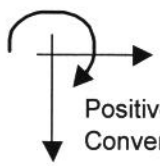
$$W_U := A_A + A_B + A_C + A_D + A_E + A_F \qquad W_U = 3679\,\frac{lb}{ft}$$

- HEEL check (cont'd)

Loading: Note, since the amount of base in compression is > 75%, bearing of water-side of key against soil dut to rotation should be marginally different from counter-moment produced from sliding force on key. Water loads on each side of key are also similar and so their effects on the heel are ignored.

Bending

$$M_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[W_U \cdot X_U + \frac{P_{BP1} \cdot L_{BP1}}{2} \cdot \frac{L_{BP1}}{3} - \left[(W_w + W_s) \cdot \left(\frac{D + E + F}{2} \right) + W_H \cdot X_H + \delta_d \cdot (D + E + F) \right] \right]$$



Positive Sign Convention

$$M_H = -6884.06 \cdot \frac{ft \cdot lb}{ft}$$

Note:
IF: M_H is < 0
THEN: Steel in top of heel is in tension

$$M_{uH} := |M_H| \qquad M_{uH} = 6.9 \cdot \frac{kip \cdot ft}{ft}$$

Shear

$$V_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left(W_w + W_s + \delta_d + W_H - W_U - \frac{P_{BP1} \cdot L_{BP1}}{2} \right)$$

$$V_H = 1813.33 \frac{lb}{ft} \qquad V_{uH} := |V_H| \qquad V_{uH} = 1.8 \cdot \frac{kip}{ft}$$

Capacity:
Flexural Capacity

Top Steel: $A_{Heel1} := .44 \frac{in^2}{ft}$ $C_{CH1} := 3.375in$ Bottom Steel: $A_{Heel2} := .35 \frac{in^2}{ft}$ $C_{CH2} := 3.375in$

$A_{s1} := A_{Heel1}$ $d_{H1} := I - C_{CH1}$ $A_{s2} := A_{Heel2}$ $d_{H2} := I - C_{CH2}$

$b := 12 \frac{in}{ft}$ $a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $a1 = 0.58 \cdot in$ $b_c := 12 \frac{in}{ft}$ $a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b_c}$ $a2 = 0.46 \cdot in$

$$\phi M_{H1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{H1} - \frac{a1}{2} \right) \qquad \phi M_{H2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{H2} - \frac{a2}{2} \right)$$

$$\phi M_{H1} = 16.63 \cdot \frac{kip \cdot ft}{ft} \qquad \phi M_{H2} = 13.3 \cdot \frac{kip \cdot ft}{ft}$$

$$\phi M_H := if(M_H > 0, \phi M_{H2}, \phi M_{H1}) \qquad \phi M_H = 16.63 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{H1} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c2} := \phi_V \cdot 2 \cdot b_c \cdot d_{H2} \cdot \sqrt{f'_c \cdot psi}$$

$$\phi V_{c1} = 15281 \frac{lb}{ft} \qquad \phi V_{c2} = 15281 \frac{lb}{ft}$$

$$\phi V_H := if(M_H > 0, \phi V_{c2}, \phi V_{c1}) \qquad \phi V_H = 15.28 \cdot \frac{kip}{ft}$$

- HEEL check (cont'd)

Factors of Safety

Bending $\phi M_H = 16.63 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_1 := \frac{\phi M_H}{M_{uH}}$

$M_{uH} = 6.88 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_1 = 2.42$

$Check1 := if(\phi M_H > 1.5 M_{uH}, "OKAY", "NO GOOD")$ $\quad Check1 = "OKAY"$

Shear $\phi V_H = 15.28 \cdot \frac{kip}{ft}$ $\quad FS_2 := \frac{\phi V_H}{V_{uH}}$

$V_{uH} = 1.81 \cdot \frac{kip}{ft}$ $\quad FS_2 = 8.43$

$Check2 := if(\phi V_H > 1.5 V_{uH}, "OKAY", "NO GOOD")$ $\quad Check2 = "OKAY"$

Controlling Factor of Safety

$FS_H := \min(FS_1, FS_2)$ $\quad FS_H = 2.42$

Controlling Mechanism

$Comment_H := if(FS_1 > FS_2, "Shear in Heel", if(M_H > 0, "Flexural Bottom Steel in Heel", "Flexural Top Steel in Heel"))$

$Comment_H = "Flexural Top Steel in Heel"$

Key Check

Hydrostatic Load is similar to each side of key, and thus neglected in analysis.

$$M_K := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[(P_{s5} - P_{s2}) \cdot \frac{(C - I)^2}{2} + \frac{1}{2} \cdot (P_{s3} - P_{s5}) \cdot \frac{2(C - I)^2}{3} \right] = -2.75 \cdot kip \cdot \frac{ft}{ft}$$

$M_K > 0$ means water-side steel is in tension.

$$V_K := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[(P_{s5} - P_{s2}) \cdot (C - I) + \frac{1}{2} (P_{s3} - P_{s5}) \cdot (C - I) \right] = -2747 \frac{lb}{ft}$$

$M_{uK} := |M_K| = 2747\,lb$

$V_{uK} := |V_K| = 2747 \frac{lb}{ft}$

Steel in Tension: $A_{Key} := 0.16 \frac{in^2}{ft}$ $\quad C_K := 3.375in$ $\quad d_K := (D + E) - C_K$

$b := 12 \frac{in}{ft}$ $\quad a_k := \frac{A_{Key} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_k = 0.21 \cdot in$ $\quad \phi M_K := \phi_B A_{Key} \cdot F_y \cdot \left(d_K - \frac{a_k}{2} \right)$ $\quad \phi M_K = 7.74 \cdot \frac{kip \cdot ft}{ft}$

Shear:

$\phi V_K := \phi_V \cdot 2 \cdot b \cdot d_K \cdot \sqrt{f'_c \cdot psi}$ $\quad \phi V_K = 19225 \frac{lb}{ft}$

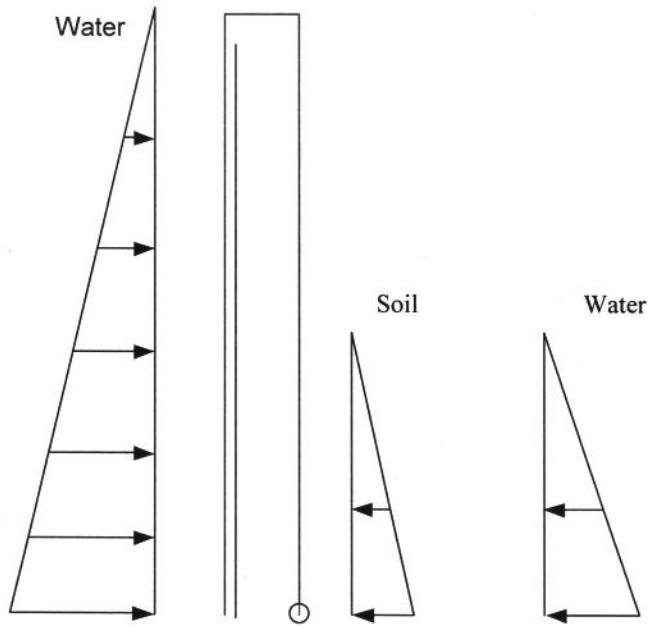
$FS_{KM} := \frac{\phi M_K}{M_K} = -2.82$ $\quad Check_M := if(\phi M_K > 1.5 M_{uK}, "OKAY", "NO GOOD")$

$Check_M = "OKAY"$

$FS_{KV} := \frac{\phi V_K}{V_K} = -7$ $\quad Check_V := if(\phi V_K > 1.5 V_{uK}, "OKAY", "NO GOOD")$

$Check_V = "OKAY"$

- STEM Check



Note:
P_{ss2} = Resistive Soil pressure present at base of stem
P_{ws2} = Resistive Water pressure present at base of stem

$P_{ws1} := (A + B)\cdot\gamma_w$

$P_{ws1} = 608\,\frac{lb}{ft^2}$

$P_{ss2} := \frac{J}{I + J}\cdot P_{s1}$ $\qquad P_{ws2} := J\cdot\gamma_w$

Driving Soil Pressure at base of Stem:
$P_{ss1} := P_{s4} = 0\cdot ksf$

$P_{ss2} = 0.08\cdot ksf$ $\qquad P_{ws2} = 0.2\cdot ksf$

Waterside Steel: $A_{Stem1} := .51\,\frac{in^2}{ft}$ $\qquad C_{CS1} := 2.81in$ $\qquad d_{S1} := G - C_{CS1}$

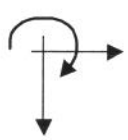
Resistive Side Steel: $A_{Stem2} := .20\cdot\frac{in^2}{ft}$

Loading:

Bending

$$M_{uS} := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A + B)}{2}\cdot\frac{A + B}{3} + \frac{P_{ss1}\cdot B^2}{2\cdot 3} - \frac{\left(P_{ss2} + P_{ws2}\right)\cdot\left(J^2\right)}{2\cdot 3}\right]$$

Note:
IF: M_{uS} is > 0
THEN: Riverside steel is in tension



$M_{uS} = 9.2\cdot\frac{kip\cdot ft}{ft}$

Positive Sign Convention

Shear

$$V_{uS} := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A + B)}{2} + \frac{P_{ss1}\cdot B}{2} - \frac{\left(P_{ss2} + P_{ws2}\right)\cdot(J)}{2}\right]$$

$V_{uS} = 2.5\cdot\frac{kip}{ft}$

- STEM Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Stem1}$

$b := 12\frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a1 = 0.67 \cdot in$

$\phi M_S := \phi_B A_{s1} \cdot F_y \cdot \left(d_{S1} - \frac{a1}{2}\right)$ $\quad \phi M_S = 25.26 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\mu := 1.0$

$\phi V_S := \phi_V \cdot A_{Stem2} \cdot F_y \cdot \mu = 8000\frac{lb}{ft}$ $\quad \phi V_S = 8 \cdot \frac{kip}{ft}$

Factors of Safety

Bending

$\phi M_S = 25.26 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_3 := \frac{\phi M_S}{M_{uS}}$

$M_{uS} = 9.17 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_3 = 2.75$

$Check3 := if\left(\phi M_S > 1.5 M_{uS}, "OKAY", "NO GOOD"\right)$ $\quad Check3 = "OKAY"$

Shear

$\phi V_S = 8 \cdot \frac{kip}{ft}$ $\quad FS_4 := \frac{\phi V_S}{V_{uS}}$

$V_{uS} = 2.52 \cdot \frac{kip}{ft}$ $\quad FS_4 = 3.17$

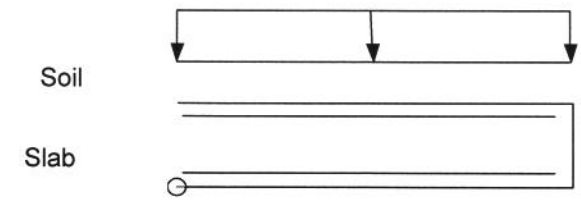
$Check4 := if\left(\phi V_S > 1.5 V_{uS}, "OKAY", "NO GOOD"\right)$ $\quad Check4 = "OKAY"$

Controlling Factor of Safety

$FS_S := \min\left(FS_3, FS_4\right)$ $\quad FS_S = 2.75$

Note: No bar cut-offs checks are made for the reliability Study.

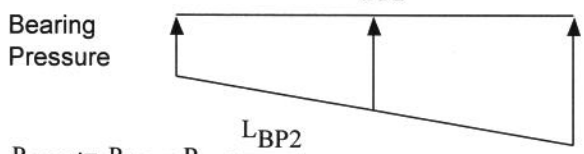
- TOE Check



$W_s := \gamma_{sat} \cdot H \cdot (J)$ $\quad W_s = 1275.92 \frac{lb}{ft}$

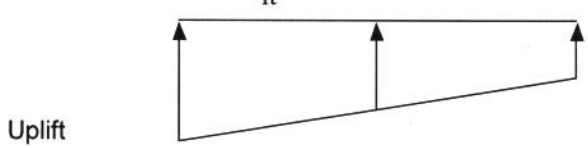
$L_{BP2} := if(H \le L_{BP}, H, L_{BP})$

$L_{BP2} = 3.5\,ft$



$P_{BP2} := P_{BP} - P_{BP} \cdot \frac{L_{BP2}}{L_{BP}}$ $\quad P_{BP} = 1060 \frac{lb}{ft^2}$

$P_{BP2} = 631.1 \frac{lb}{ft^2}$



$P_{U6} = 392.1 \frac{lb}{ft^2}$

$$P_{U5} := \left[\left[\left(P_{U3} - P_{U6}\right) \cdot \frac{H}{F + G + H} + P_{U6}\right]\right]$$

$P_{U5} = 474.11 \frac{lb}{ft^2}$

Bearing Pressure

$$L_{bp} := \frac{\left(L_{BP2} \cdot P_{BP2}\right) \cdot \left(\frac{L_{BP2}}{2}\right) + \frac{\left(P_{BP} - P_{BP2}\right) \cdot \left(L_{BP2}\right)}{2} \cdot \left(\frac{2L_{BP}}{3}\right)}{L_{BP2} \cdot P_{BP2} + \frac{\left(P_{BP} - P_{BP2}\right) \cdot \left(L_{BP2}\right)}{2}}$$

$L_{bp} = 2.77\,ft$

$$W_{bp} := L_{BP2} \cdot P_{BP2} + \frac{\left(P_{BP} - P_{BP2}\right) \cdot \left(L_{BP2}\right)}{2}$$

$W_{bp} = 2959 \frac{lb}{ft}$

Uplift

$$L_u := \frac{H \cdot P_{U6} \cdot \frac{H}{2} + \frac{\left(P_{U5} - P_{U6}\right) \cdot H}{2} \cdot \frac{H}{3}}{H \cdot P_{U6} + \frac{\left(P_{U5} - P_{U6}\right) \cdot H}{2}}$$

$L_u = 1.69\,ft$

$$W_u := H \cdot P_{U6} + \frac{\left(P_{U5} - P_{U6}\right) \cdot H}{2}$$

$W_u = 1515.87 \frac{lb}{ft}$

Toe Check (continued)

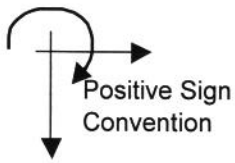
Loading:

Bending

$$M_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[\left[W_s + (H \cdot I) \cdot \gamma_c \right] \cdot \frac{H}{2} - \left(W_{bp} \cdot L_{bp} + W_u \cdot L_u \right) \right]$$

Note:
IF: $M_T < 0$
THEN: Bottom steel is in tension

$$M_T = -7.4 \cdot \frac{kip \cdot ft}{ft} \qquad M_{uT} := \left| M_T \right| \qquad M_{uT} = 7.38 \cdot \frac{kip \cdot ft}{ft}$$



Shear

$$V_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[W_s + (H \cdot I) \cdot \gamma_c - W_{bp} - W_u \right]$$

$$V_T = -2.54 \cdot \frac{kip}{ft} \qquad V_{uT} := \left| V_T \right| \qquad V_{uT} = 2.54 \cdot \frac{kip}{ft}$$

Capacity:

Flexural Capacity

Area of bottom (tension) steel: $A_{Toe} := .352 \frac{in^2}{ft}$ $\quad C_{CT} := 3.375in$ $\quad d_T := I - C_{CT} = 11.63 \cdot in$

$$b := 12 \frac{in}{ft} \qquad A_{s1} := A_{Toe}$$

$$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a1 = 0.46 \cdot in$$

$$\phi M_T := \phi_B A_{s1} \cdot F_y \cdot \left(d_T - \frac{a1}{2} \right) \qquad \phi M_T = 13.37 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_T := \phi_V \cdot 2 \cdot b \cdot d_T \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_T = 15.28 \cdot \frac{kip}{ft}$$

Toe Check (continued)

Factors of Safety

Bending

$\phi M_T = 13.37 \cdot \frac{kip \cdot ft}{ft}$

$M_{uT} = 7.38 \cdot \frac{kip \cdot ft}{ft}$

$FS_5 := \frac{\phi M_T}{M_{uT}}$

$FS_5 = 1.81$

$Check5 := if(\phi M_T > 1.5 \cdot M_{uT}, "OKAY", "NO GOOD")$

$Check5 = "OKAY"$

Shear

$\phi V_T = 15.28 \cdot \frac{kip}{ft}$

$V_{uT} = 2.54 \cdot \frac{kip}{ft}$

$FS_6 := \frac{\phi V_T}{V_{uT}}$

$FS_6 = 6.01$

$Check6 := if(\phi V_T > 1.5 V_{uT}, "OKAY", "NO GOOD")$

$Check6 = "OKAY"$

Controlling Factor of Safety

$FS_T := \min(FS_5, FS_6)$

$FS_T = 1.81$

End of Analysis

US Army Corps of Engineers

CID Missouri
Monolith 10 Stoplog Gap
Analyzed at Top of Footer

Kansas Citys Levees

Comp by: Sheffield
Chkd by: K. Marx 9/28/11

- Guide Post Strength Analysis

Tributary length of load,

$B := \frac{24.02ft}{2}$

Depth of Section, $D := 30in$

Width of section, $b := 24in$
Width for shear, $b_s := 17in$

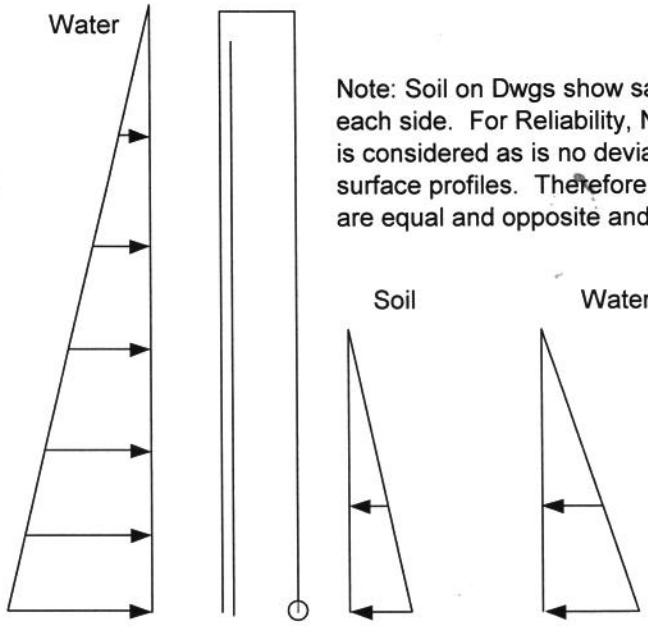
Height of water driving:

$H_{w1} := 6ft + 7in$

Note: Vertical Cracks are assumed on each side of Guide Post, reducing sill's ability to resist load.

Full Height of Guide:

$H_w := 9ft + 9in$



$\gamma_w := 62.4pcf$

Note: Soil on Dwgs show same EL each side. For Reliability, No Passive is considered as is no deviation from surface profiles. Therefore soil loads are equal and opposite and ignored.

Height of water resisting:

$H_{w2} := 3ft + 2in$

Resistance at Footer:

$w_{w1} := (H_{w1} \cdot \gamma_w \cdot B)$

$w_{w1} = 4934 \cdot plf$

4 No 7 $A_1 := 2.41in^2$

$C_1 := 2in + \frac{1}{2}in + \frac{1}{2} \cdot \left(\frac{7}{8}\right)in$ $d_1 := D - C_1$

$d_1 = 27.0625 \cdot in$

$w_{w2} := H_{w2} \cdot \gamma_w \cdot b$

$w_{w2} = 395 \cdot plf$

6 No 4 Area of dowels, $A_v := 1.18in^2$

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete. As concrete ages it continues to hydrate and gain compressive strength. FEMA 310, Section 4.2.4.4, states, "Unless calculated otherwise the expected strength shall be assumed equal to the nominal strength multiplied by 1.25." Using FEMA's guidance the concrete strength becomes 3750 psi. However, 3750 psi is only used for the median strength in reliability calcs; 3000 psi is used for the following initial calculations.
- The Portland Cement Association pamphlet, *Engineered Concrete Structures*, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Concrete Properties $f'_c := 3000 \cdot psi$

Steel Properties $F_y := 40.0ksi$ $E_s := 29000ksi$

Load & Resistance Factor Design

Strength Reduction Factors:

Shear Strength	$\phi_V := 1$	Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.
Flexural Strength	$\phi_B := 1$	

Load Factors:

Dead and Live Load Factor	$\gamma_L := 1.0$	Load Multiplication Factor EM 1110-2-2104 (3-1)
Hydraulic Load Factor	$\gamma_H := 1$	Hydraulic Factor EM 1110-2-2104 (3-2)
Extreme Case Factor	$\gamma_X := 1.0$	Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

All set to equal one for existing conditions reliability analysis

- Flexural Check of Stop Log Guide

DEMAND

Bending:

$$M_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{1}{2} \cdot w_{w1} \cdot H_{w1} \cdot \left[\frac{H_{w1}}{3} + \left(H_w - H_{w1}\right) \right] + \frac{1}{2} H_{w1} \cdot \gamma_w \cdot b \cdot \left(H_w - H_{w1}\right)^2 + \frac{1}{6} \cdot \left(H_w - H_{w1}\right)^3 \cdot \gamma_w \cdot b - \frac{1}{6} \cdot w_{w2} \cdot H_{w2}^{\,2} \right]$$

$$M_u = 91.2 \cdot kip \cdot ft$$

Shear:

$$V_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{1}{2} \cdot w_{w1} \cdot H_{w1} - \frac{1}{2} \cdot w_{w2} \cdot H_{w2} + H_{w1} \cdot \gamma_w \cdot \left(H_w - H_{w1}\right) \cdot b + \frac{1}{2} \cdot \left(H_w - H_{w1}\right)^2 \cdot \gamma_w \cdot b \right] \qquad V_u = 18.8419 \cdot kip$$

CAPACITY

Flexural Capacity (of tension steel only):

For fy = 40,000 psi and f'c = 3,000 psi:

$roe_b := 0.0371$ $\qquad roe_{max} := \frac{1}{2} \cdot roe_b = 0.0186$ $\qquad roe_{min} := 0.0050$

Note: Compression steel is ignored since $\rho < 0.75\ \rho_b$

$$roe := \frac{A_1}{b \cdot d_1} = 0.0037$$

Check Steel Ratios (0 is a false Statement, 1 is a true Statement):

$roe > roe_{min} = 0$ Not Good; however, requirement need not be met if As provided is at least one-third greater than required. So for flexural capacity, 3/4 factor will be applied.

$roe \le roe_{max} = 1$ OK

Calculate Capacity:

$$a := \frac{A_1 F_y}{0.85 \cdot f'_c \cdot b} = 1.5752 \cdot in$$

$$\phi M_n := \phi_B \cdot \left[\frac{3}{4} A_1 F_y \cdot \left(d_1 - \frac{a}{2} \right) \right] = 158.3064 \cdot kip \cdot ft$$

3/4 As is used since ρ_{min} is not met.

- Flexural Check of Stop Log Guide (cont'd)

Shear Capacity: $\mu := 1.0$ For Concrete placed against roughened concrete.

$\phi V_n := \phi_V \cdot (A_v \cdot F_y \cdot \mu) = 47200\ \text{lbf}$ $\phi V_n = 47 \cdot \text{kip}$

FACTORS OF SAFETY

Bending:

$\phi M_n = 158.3064 \cdot \text{kip} \cdot \text{ft}$

$M_u = 91.1845 \cdot \text{kip} \cdot \text{ft}$

$FS_B := \frac{\phi M_n}{M_u} = 1.7361$ Check := if($FS_B > 1.5$, "OKAY", "NO GOOD") $FS_B = 1.7$ Check = "OKAY"

Shear:

$\phi V_n = 47.2 \cdot \text{kip}$

$V_u = 18.8419 \cdot \text{kip}$

$FS_V := \frac{\phi V_n}{V_u}$ Check1 := if($FS_V > 1.5$, "OKAY", "NO GOOD") $FS_V = 2.5051$ Check1 = "OKAY"

Controlling Factor of Safety

$FS_S := \min(FS_B, FS_V)$ $FS_S = 1.7$

- Guide Post Strength Analysis

Tributary length of load,

$$B := \frac{24.02ft}{2}$$

Depth of Section,

$D := 20in$

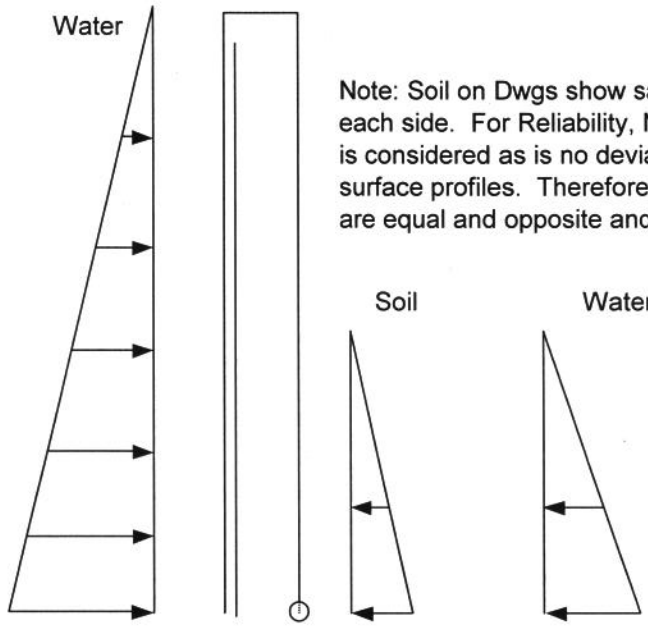
Width of section,

Width for shear, $b_s := 17in$

Height of water driving:

$$H_{w1} := 6ft + 7in$$

Note: Width of reinforced sill is assumed to strengthen abutment such that its small height and additional load will not control.



$\gamma_w := 62.4pcf$

Note: Soil on Dwgs show same EL each side. For Reliability, No Passive is considered as is no deviation from surface profiles. Therefore soil loads are equal and opposite and ignored.

Height of water resisting:

$H_{w2} := 0ft$

$w_{w2} := H_{w2} \cdot \gamma_w \cdot b$

$w_{w2} = 0 \cdot plf$

$$w_{w1} := H_{w1} \cdot \gamma_w \cdot B$$

$$w_{w1} = 4934 \cdot plf$$

4 No 7 $\quad A_1 := 2.41in^2$

Note: Compression steel is ignored since $\rho < 0.75\ \rho_b$

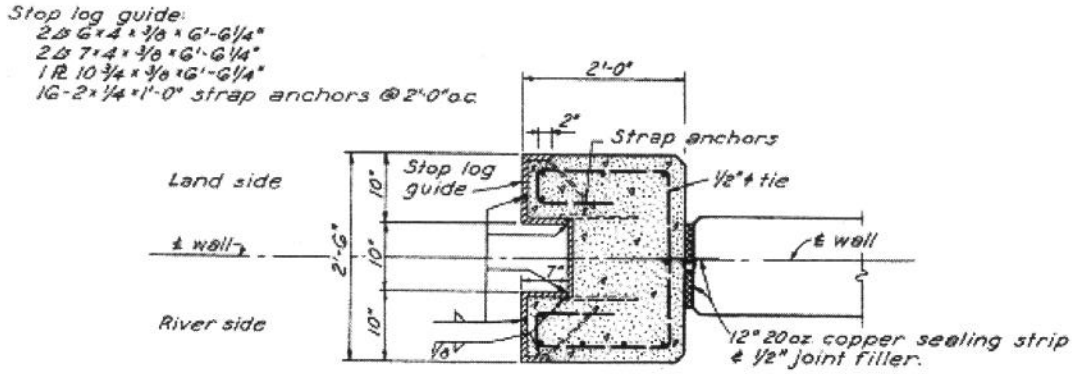
$$C_1 := 2in + \frac{1}{2}in + \frac{1}{2} \cdot \left(\frac{7}{8}\right)in$$

$$d_1 := D - C_1$$

$$d_1 = 27.0625 \cdot in$$

Area of stirrup, $A_v := .20in^2 \cdot 2$

Stirrup spacing, $s_s := 12in$



From Record Drawings
File No. A-10-1176

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete. As concrete ages it continues to hydrate and gain compressive strength. FEMA 310, Section 4.2.4.4, states, "Unless calculated otherwise the expected strength shall be assumed equal to the nominal strength multiplied by 1.25." Using FEMA's guidance the concrete strength becomes 3750 psi. However, 3750 psi is only used for the median strength in reliability calcs; 3000 psi is used for the following initial calculations.
- The Portland Cement Association pamphlet, *Engineered Concrete Structures*, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Concrete Properties $f'_c := 3000 \cdot psi$

Steel Properties $F_y := 40.0ksi$ $E_s := 29000ksi$

Load & Resistance Factor Design

Strength Reduction Factors:

Shear Strength	$\phi_V := 1$	Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.
Flexural Strength	$\phi_B := 1$	

Load Factors:

Dead and Live Load Factor	$\gamma_L := 1.0$	Load Multiplication Factor EM 1110-2-2104 (3-1)
Hydraulic Load Factor	$\gamma_H := 1$	Hydraulic Factor EM 1110-2-2104 (3-2)
Extreme Case Factor	$\gamma_X := 1.0$	Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

All set to equal one for existing conditions reliability analysis

- Flexural Check of Stop Log Guide

DEMAND

Bending:

$$M_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left(\frac{1}{2} \cdot w_{w1} \cdot H_{w1} \cdot \frac{H_{w1}}{3} - \frac{1}{2} \cdot w_{w2} \cdot H_{w2} \cdot \frac{H_{w2}}{3} \right) \qquad M_u = 35.6 \cdot kip \cdot ft$$

Shear:

$$V_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left(\frac{1}{2} \cdot w_{w1} \cdot H_{w1} - \frac{1}{2} \cdot w_{w2} \cdot H_{w2} \right) \qquad V_u = 16.2401 \cdot kip$$

CAPACITY

Flexural Capacity (of tension steel only):

For fy = 40,000 psi and f'c = 3,000 psi:

$roe_b := 0.0371$ $\qquad roe_{max} := \frac{1}{2} \cdot roe_b = 0.0186$ $\qquad roe_{min} := 0.0050$

Note: Compression steel is ignored since $\rho < 0.75\ \rho_b$

$$roe := \frac{A_1}{b \cdot d_1} = 0.0037$$

Check Steel Ratios (0 is a false Statement, 1 is a true Statement):

$roe > roe_{min} = 0$ Guide Post does not meet Minimum Steel Ratio, ρ_{min}, thus should the section crack, immediate failure may occur!

$roe \le roe_{max} = 1$ OK

Calculate Capacity:

$$a := \frac{A_1 F_y}{0.85 \cdot f'_c \cdot b} = 1.5752 \cdot in$$

$$\phi M_n := \phi_B \cdot \left[A_1 F_y \cdot \left(d_1 - \frac{a}{2} \right) \right] = 211.0752 \cdot kip \cdot ft$$

Since ρ_{min} is not met, compute ϕM_n based upon Unreinforced Section (ACI 22.5):

$$S_m := \frac{b \cdot D^2}{6} = 3600 \cdot in^3$$

$$\phi M_n := 5 \cdot \sqrt{f'_c \cdot psi} \cdot S_m = 82.1584 \cdot kip \cdot ft$$

- Flexural Check of Stop Log Guide (cont'd)

Shear Capacity:

$$\phi V_n := \phi_V \cdot \left(2 \cdot b_s \cdot d_1 \cdot \sqrt{f'_c \cdot psi} + \frac{A_v \cdot F_y \cdot d_1}{s_s}\right)$$

$\phi V_n = 86 \cdot kip$

FACTORS OF SAFETY

Bending:

$\phi M_n = 82.1584 \cdot kip \cdot ft$

$M_u = 35.638 \cdot kip \cdot ft$

$FS_B := \frac{\phi M_n}{M_u}$ Check := if$\left(FS_B > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$FS_B = 2.3$ Check = "OKAY"

Shear:

$\phi V_n = 86.4807 \cdot kip$

$V_u = 16.2401 \cdot kip$

$FS_V := \frac{\phi V_n}{V_u}$ Check1 := if$\left(FS_V > 1.5, \text{"OKAY"}, \text{"NO GOOD"}\right)$

$FS_V = 5.3251$ Check1 = "OKAY"

Controlling Factor of Safety

$FS_S := \min\left(FS_B, FS_V\right)$

$FS_S = 2.3$

US Army Corps of Engineers

CID Missouri
Monolith 10 Stoplog Gap
Sill Analysis

Kansas Citys Levees

Comp by: Sheffield
Chkd by: K. Marx 9/28/11

Analysis will be performed on a per foot basis. Soil on Dwgs show same EL each side. For Reliability, No Passive is considered as is no deviation from surface profiles. Therefore soil loads are equal and opposite and ignored.

Depth of Section, $D := 30in$

$\gamma_w := 62.4pcf$

Height to top of sill:

6.583

$H_{w1} := 6ft + 7in$ $\quad w_{w1} := H_{w1} \cdot \gamma_w = 410.8 \cdot psf$

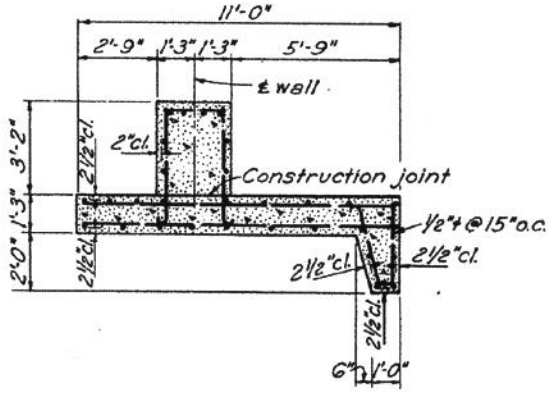
Height of sill:

9.75

$h_{sill} := 3ft + 2in$

$H_T := H_{w1} + h_{sill} = 9.75\,ft$

$w_{wT} := H_T \cdot \gamma_w = 608.4 \cdot psf$



No 4 @ 12" OC $\quad A_1 := 0.20in^2$

Note: Compression steel is ignored since $\rho < 0.75\,\rho_b$

$$C_1 := 2in + \frac{1}{2}in + \frac{1}{2} \cdot \left(\frac{4}{8}\right)in$$

$d_1 := D - C_1 \qquad d_1 = 27.25 \cdot in$

From Record Drawings
File No. A-10-1176

Area of dowels,
$A_v := .20in^2$

No 4 @ 12" OC

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete.
- The Portland Cement Association pamphlet, *Engineered Concrete Structures*, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Concrete Properties $\quad f'_c := 3000 \cdot psi$

Steel Properties $\quad F_y := 40.0ksi \qquad E_s := 29000ksi$

Load & Resistance Factor Design

Strength Reduction Factors:

Shear Strength	$\phi_V := 1$	Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.
Flexural Strength	$\phi_B := 1$	
Load Factors:		
Dead and Live Load Factor	$\gamma_L := 1.0$	Load Multiplication Factor EM 1110-2-2104 (3-1)
Hydraulic Load Factor	$\gamma_H := 1$	Hydraulic Factor EM 1110-2-2104 (3-2)
Extreme Case Factor	$\gamma_X := 1.0$	Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

All set to equal one for existing conditions reliability analysis

- Flexural Check of Sill:

DEMAND

Bending:

$$M_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[w_{w1} \cdot \frac{(H_T - H_{w1})^2}{2} + \frac{1}{2} \cdot (w_{wT} - w_{w1}) \cdot \frac{(H_T - H_{w1})^2}{3} \right] \cdot 12in \quad M_u = 2.4 \cdot kip \cdot ft$$

Shear:

$$V_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left(\frac{1}{2} \cdot w_{wT} \cdot H_T - \frac{1}{2} \cdot w_{w1} \cdot H_{w1} \right) \cdot 12in \qquad V_u = 1.6137 \cdot kip$$

CAPACITY

Flexural Capacity (of tension steel only):

For fy = 40,000 psi and f'c = 3,000 psi:

$roe_b := 0.0371$ $\quad roe_{max} := \frac{1}{2} \cdot roe_b = 0.0186$ $\quad roe_{min} := 0.0050$

Note: Compression steel is ignored since $\rho < 0.75\ \rho_b$

$$roe := \frac{A_1}{12in \cdot d_1} = 0.0006$$

Check Steel Ratios (0 is a false Statement, 1 is a true Statement):

$roe > roe_{min} = 0$ Not Good; however, requirement need not be met if As provided is at least one-third greater than required. So for flexural capacity, a 3/4 factor will be applied.

$roe \le roe_{max} = 1$ OK

Calculate Capacity:

$$a := \frac{A_1 F_y}{0.85 \cdot f'_c \cdot 12in} = 0.2614 \cdot in$$

$$\phi M_n := \phi_B \cdot \left[\frac{3}{4} A_1 F_y \cdot \left(d_1 - \frac{a}{2} \right) \right] = 13.5596 \cdot kip \cdot ft$$

- Flexural Check of Stop Log Guide (cont'd)

Shear Capacity: $\mu := 1.0$ For Concrete placed against roughened concrete.

$\phi V_n := \phi_V \cdot (A_v \cdot F_y \cdot \mu) = 8000\,lbf$ $\phi V_n = 8 \cdot kip$

FACTORS OF SAFETY

Bending:

$\phi M_n = 13.5596 \cdot kip \cdot ft$

$M_u = 2.39 \cdot kip \cdot ft$

$FS_B := \frac{\phi M_n}{M_u}$ $Check := if(FS_B > 1.5, "OKAY", "NO GOOD")$ $FS_B = 5.7$ $Check = "OKAY"$

Shear:

$\phi V_n = 8 \cdot kip$

$V_u = 1.6137 \cdot kip$

$FS_V := \frac{\phi V_n}{V_u}$ $Check1 := if(FS_V > 1.5, "OKAY", "NO GOOD")$ $FS_V = 4.9574$ $Check1 = "OKAY"$

Controlling Factor of Safety

$FS_S := \min(FS_B, FS_V)$ $FS_S = 5$