RADBUG 2023

Madison, Wisconsin

AI Engineers
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Substructure Analysis Overview

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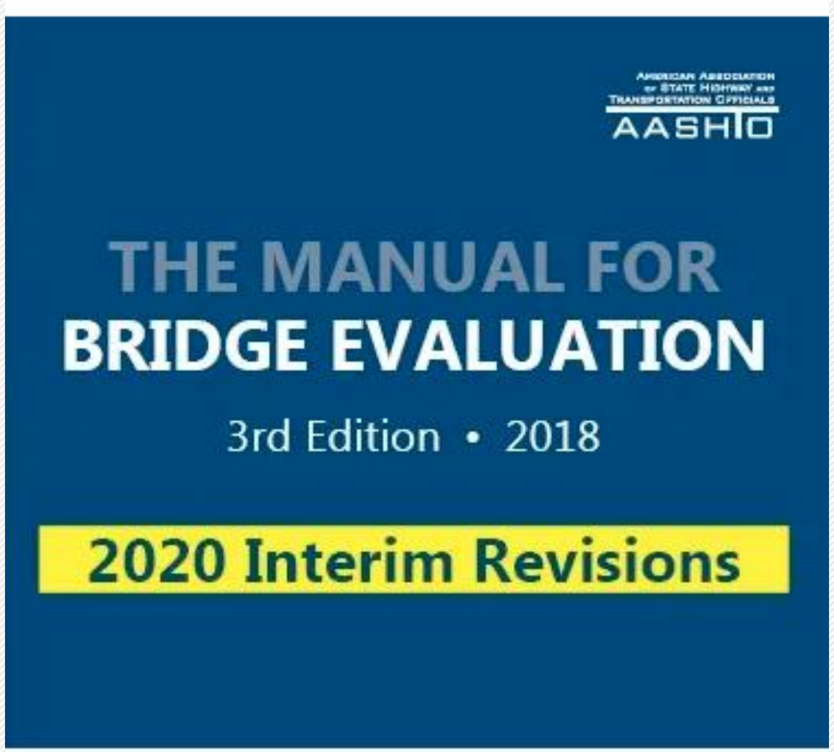
Discussion Points

1. Purpose of the presentation
2. Criteria to determine Substructure rating
3. Method of Substructure analysis
4. Force effects
5. Capacity calculation
6. Load rating
7. Points for panel discussion

Purpose of The Presentation

Wishlist:

- Substructure analysis guidance in MBE
- Implementation in BrR



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Criteria For Substructure Rating

Deterioration:

- Reduction of capacity
- Increase in the span length of pier cap due to loss of support

Criteria for Substructure Rating

Erosion and Undermining:

- Increase in the buckling length and danger of overturning in Abutments

Criteria for Substructure Rating

Unusual Geometry and Configuration:

- Straddle bents
- Steel Integral pier cap (Fracture critical)

Method of Substructure Analysis

ASD

- Masonry and Timber -ASD method

LFR & LRFR

- RC and Steel Structure - LFR and LRFR

Force Effects

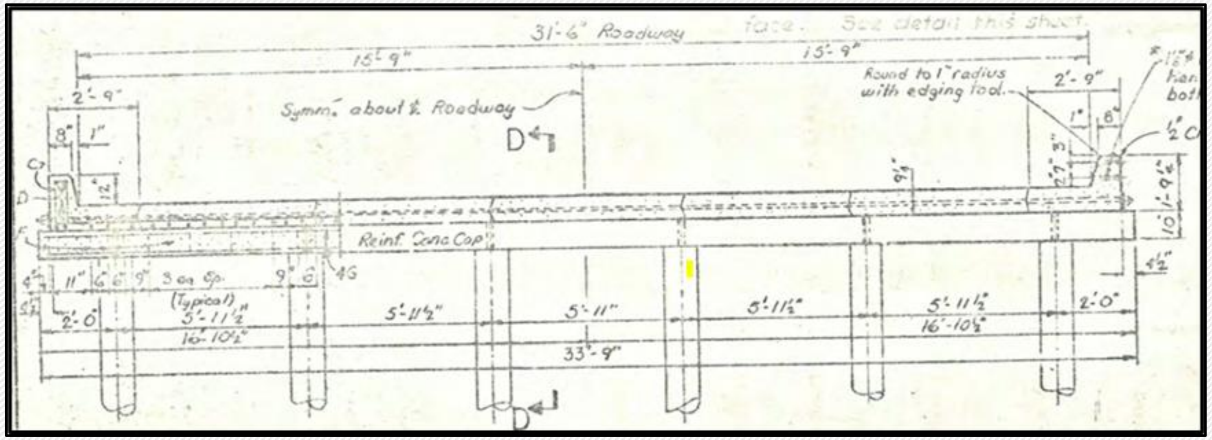
Steel and Concrete Substructures

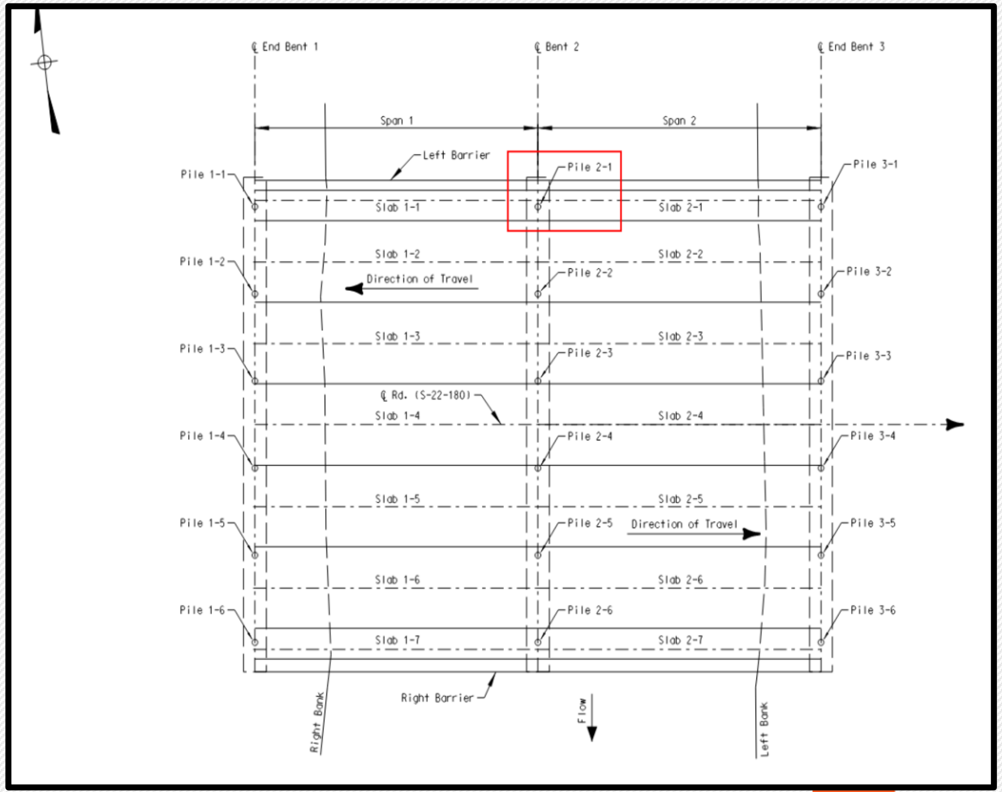
- Concrete pier cap with steel and timber columns: BrR:
- Concrete pier cap with concrete columns. BrD or Combination of BrR and other independent programs:

Force Effects

Sample Bridge

- Two simple spans: Concrete pier caps and timber columns





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Force Effects

Reinforced Concrete Pier Caps

- Two BrR models were created: Floorline (Force effect) and Girder line (capacity).
- Pier cap was modeled as Floorline using Rolled beam properties.
- DL from superstructure was input in to the Floorline model
- Rating analysis was performed
- Moment, shear and reaction were obtained from BrR output

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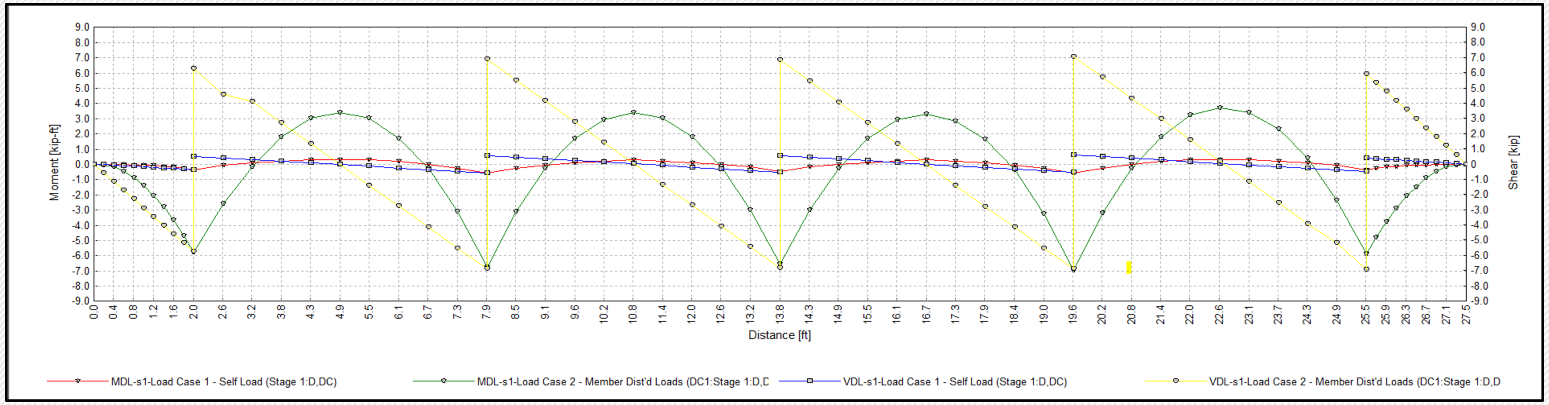
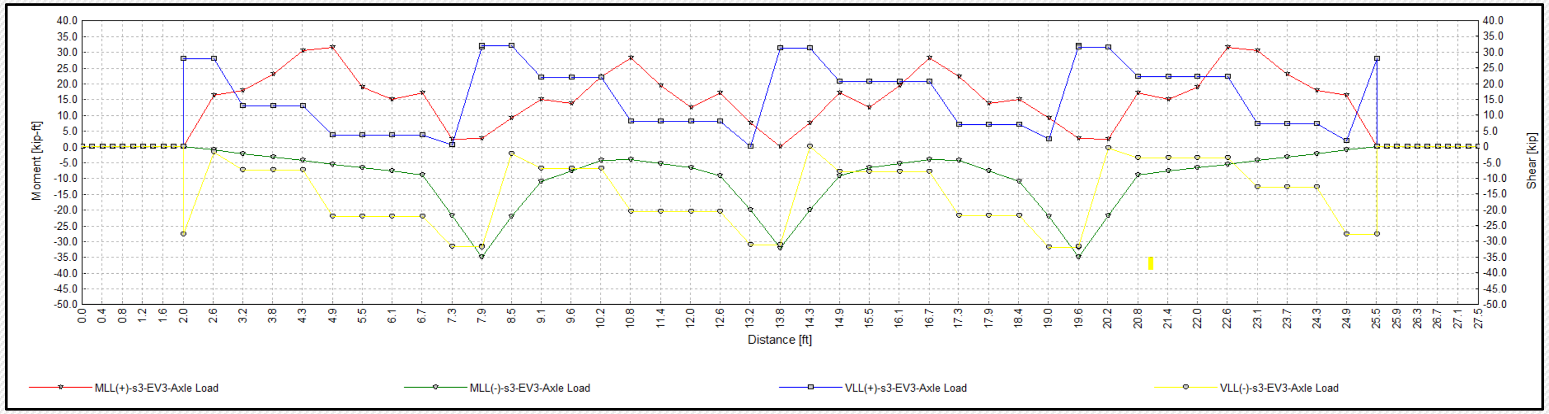
Force Effects

Moment and Shear Diagram Floorline: BrR output





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Capacity and Load Rating

Reinforced Concrete Pier Caps

- The pier cap was modeled in BrR as girder line for capacity calculation.
- RF=$(\phi Mn - M_{DL} * \gamma_{DL}) / \gamma_{LL} * V_{LL+I}$

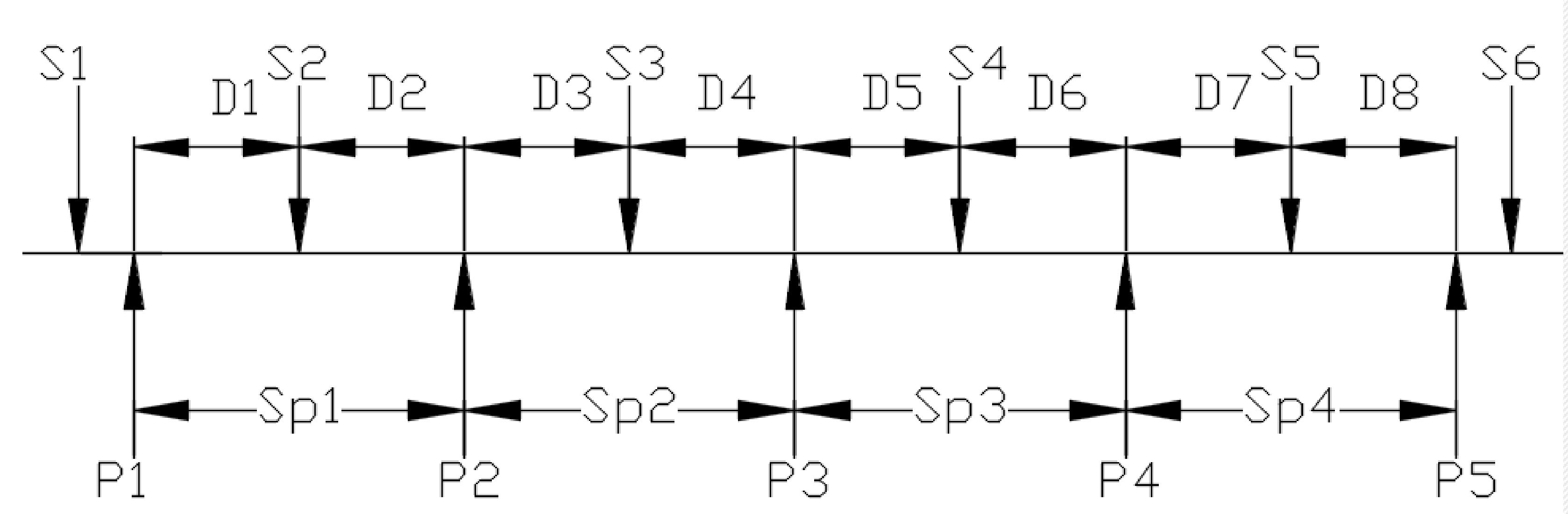
Span 2

Location (ft)	Percent	Limit State	Units	Capacity
0.59	10.0	Flexure	kip-ft	-33.38
1.17	20.0	Flexure	kip-ft	33.38
1.76	30.0	Flexure	kip-ft	33.38
2.35	40.0	Flexure	kip-ft	33.38
2.94	50.0	Flexure	kip-ft	33.38
3.52	60.0	Flexure	kip-ft	33.38
4.11	70.0	Flexure	kip-ft	33.38
4.70	80.0	Flexure	kip-ft	33.38
5.29	90.0	Flexure	kip-ft	-33.38
5.88	100.0	Flexure	kip-ft	-33.38

Force Effects

Steel and Timber Columns

- Live load distribution on columns, “LLDF” using lever rules.
- Example LL on pile “P3”
- LLDF=(D3/Sp2+D6/Sp3)
- Where “S” centerline of the beams and “P” centerline of the piles.



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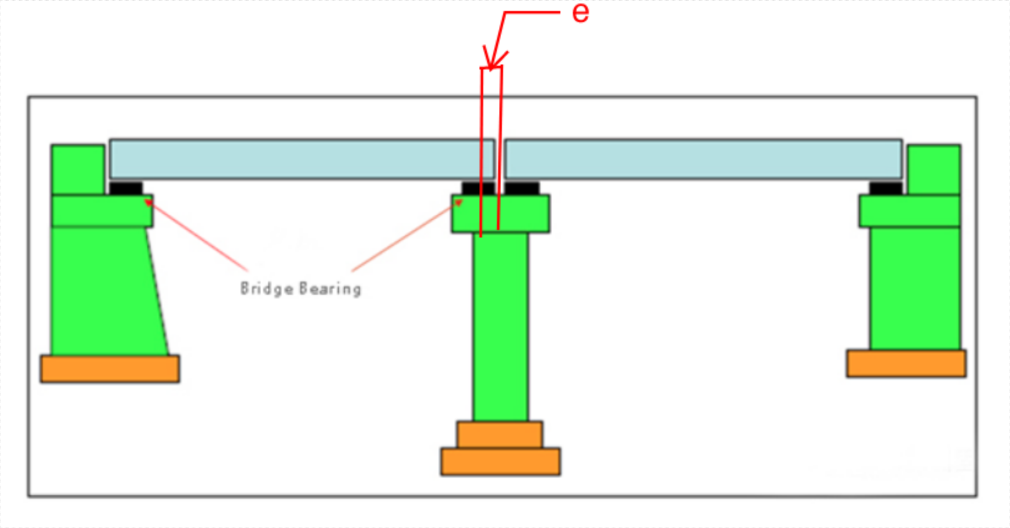
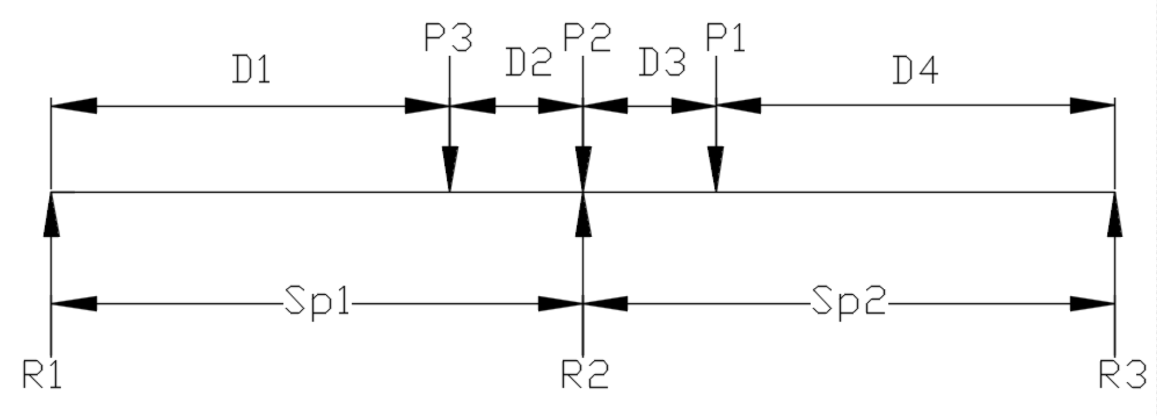
Force Effect

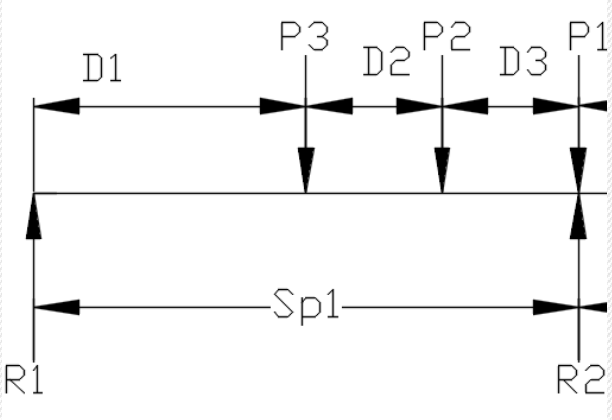
Steel and Timber Columns

- DL reaction from the Floorline model.
- LL reaction=LLDF*R2
- LL Reaction from the superstructure model with eccentricity (Combined Axial and bending)
- LL Reaction from continuous model maximum reaction (Axial only)





Continuous span model



Superstructure (simple span) model

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Capacity and Load Rating

Timber Columns: Combined Axial and Bending

- Capacity based on NDS Supplemental Table 6A (see panel discussion for detail)
- Rating based on research report FHWA-ICT-12-014
- Iterate by changing fcLL until the sum of the 1st and 2nd formula becomes "1"
- Perform rating using the 'Proposed Modification'
- The controlling rating will be the minimum between Axial and combined Axial and Bending rating.

Fc'DL	
F_c'	Inv.
F_c'	Opr.
Fc'LL	Inv.
	Opr.

F'B	
	Inv.
	Opr.
F_{CE}	Inv.
	Opr.

1st — 2nd

$$\left(\frac{f_{cDL}}{F_{CDL}'}+\frac{f_{cLL}}{F_{CLL}'}\right)^2+\frac{f_{cLL}\left(\frac{6e_{LL}}{d}\right)\times\left[1+.234\frac{f_{cLL}}{F_{cE}}\right]}{F_b'\left(1-\left(f_{cLL}/F_{cE}\right)\right)}\le 1.0$$

e	2
f_{cLL}	f_{cLL}
f_{cLL}/F'_{CLL}	f_{cLL}/F'_{CLL}
$f_{cLL}*(6e_{LL}/d)$	$f_{cLL}*(6e_{LL}/d)$
$1+0.234*(f_{cLL}/F_{cE})$	$1+0.234*(f_{cLL}/F_{cE})$
$F_b'(1-(f_{cLL}/F_{cE})$	$F_b'(1-(f_{cLL}/F_{cE})$
1st	1st
2nd	2nd
Formula	Formula

$$e_{LL} = 0.6 \times physical\ eccentricity\ of\ the\ deck \quad (2$$

2.2 PROPOSED MODIFICATION

Further analysis of Equation 2.1 reveals that the difference between the pile capacity and dead load in the numerator can be interpreted as the concentric structural live load capacity. This is compared to the denominator, which represents the live load in the pile due to the HS20-44 loading. The proposed modified load rating equation is shown below.

$$\text{Structural Pile Rating} = \left(\frac{\text{LL Stress Capacity, } f_{cLL}\big|_{\max}}{\text{Stress due to HS20-44 Live Load}}\right)\times 20 \qquad (2.5)$$

Capacity and Load Rating

Steel Columns

- Capacity based on AASHTO LRFD 6.9
- Load rating based on MBE H6A
- Change the value of “RF” until the criteria are satisfied

Column Rating: Is based on MBE Appendix H6A

If $\frac{P_u}{P_r} < 0.2$ then:

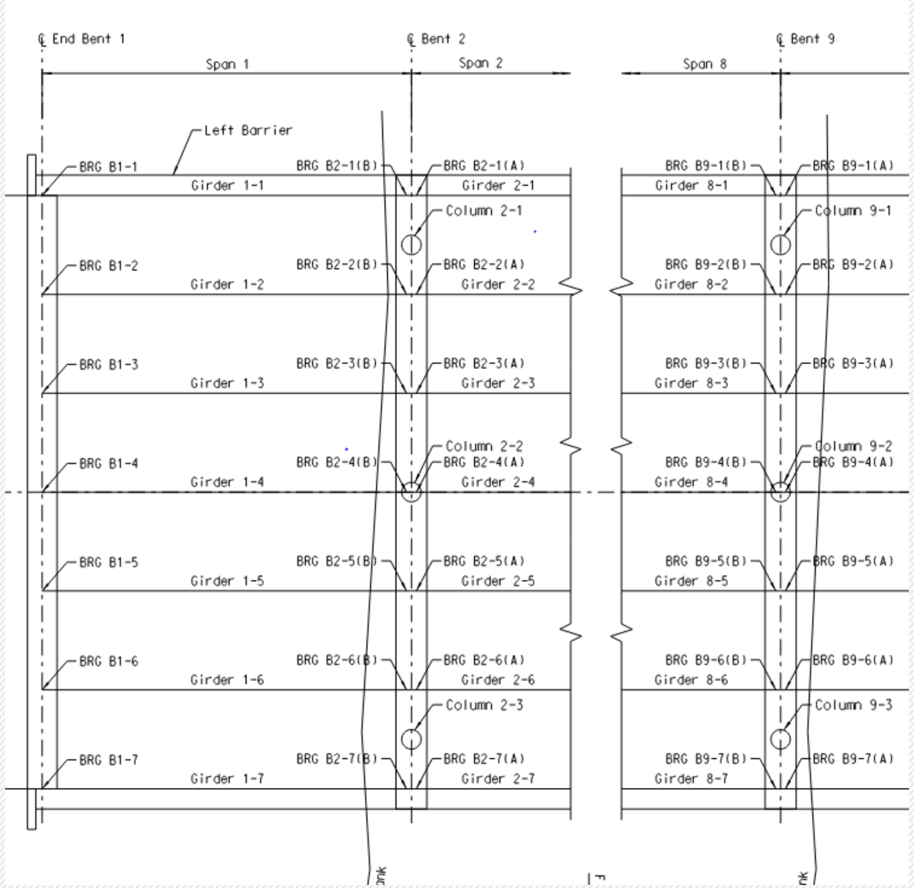
$$\gamma_D\left[\frac{1}{2}\frac{P_{DL}}{P_r}+\delta_b\left(\frac{M_{DL}}{M_r}\right)\right]+RF\times\gamma_L\left[\frac{P_{LL+IM}}{2P_r}+\delta_b\left(\frac{M_{LL+IM}}{M_r}\right)\right]=1.0$$

If $\frac{P_u}{P_r} > 0.2$ and $M_{uy} = 0$ then:

$$\gamma_D\left[\frac{P_{DL}}{P_r}+\frac{8}{9}\delta_b\left(\frac{M_{DL}}{M_r}\right)\right]+RF\times\gamma_L\left[\frac{P_{LL+IM}}{P_r}+\frac{8}{9}\delta_b\left(\frac{M_{LL+IM}}{M_r}\right)\right]=1.0$$

Force Effect

Concrete Pier caps with Concrete Columns



Force Effect

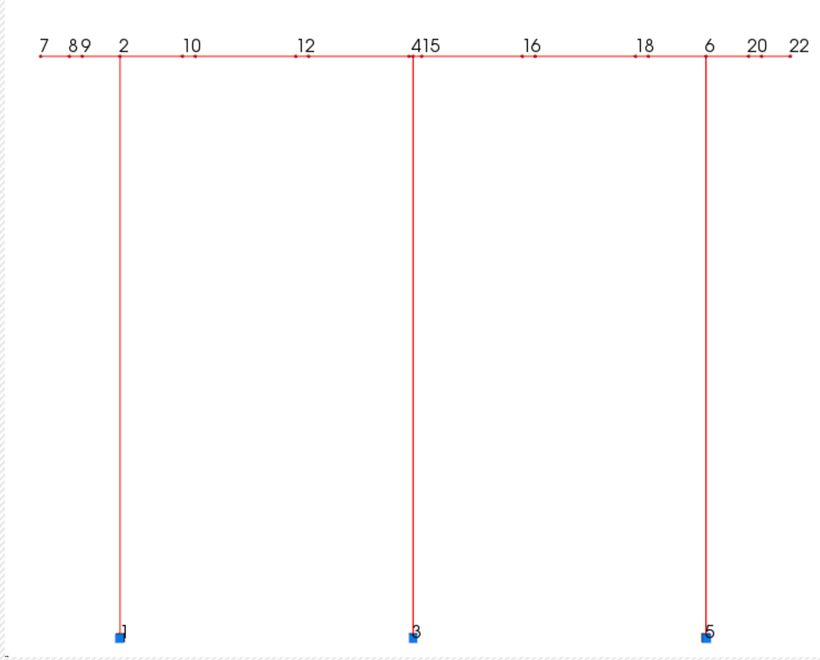
Concrete Pier caps with Concrete Columns

- Use BrD if available

Alternatively:

- DL and LL reaction from BrR
- Use any independent programs (STAAD Pro, RC-Pier…) and input the reactions at the stringer locations.
- Perform analysis
- Moment and shear from result output

Model



Force Effect

Concrete Pier caps with Concrete Columns:

- BrR reactions:

EV3 -G1-G7

Span	Location (	% Span	Positive M	Negative	Positive S	Negative	Positive A	Negative	Positive R	Negative	Positive X	Negative	Positive Y	Negative	% Impact	% Impact	Neg Reaction
1	0	0	0	0	57.59	0	0	0	57.59	0	0	0	0	0	33	0	
1	2.08	3.2	115.67	0	55.54	-0.74	0	0			0	0	0	0			
1	6.51	10	333.15	0	50.41	-3.16	0	0			0	0	0	0			
1	11.66	17.9	537.14	0	44.74	-6.67	0	0			0	0	0	0			
1	13.03	20	582.64	0	43.42	-7.61	0	0			0	0	0	0			
1	19.54	30	748.46	0	36.63	-12.06	0	0			0	0	0	0			
1	22.67	34.8	798.39	0	33.68	-15.01	0	0			0	0	0	0			
1	26.05	40	840.79	0	30.02	-17.92	0	0			0	0	0	0			
1	32.56	50	865	0	23.69	-23.69	0	0			0	0	0	0			
1	33.99	52.2	866.85	0	22.29	-24.9	0	0			0	0	0	0			
1	39.08	60	840.79	0	17.64	-29.55	0	0			0	0	0	0			
1	42.67	65.5	795.66	0	14.36	-32.83	0	0			0	0	0	0			
1	45.59	70	748.46	0	11.69	-35.5	0	0			0	0	0	0			
1	52.1	80	582.64	0	7.26	-41.45	0	0			0	0	0	0			
1	58.61	90	333.15	0	2.97	-47.4	0	0			0	0	0	0			
1	63.25	97.1	104.49	0	0.62	-51.64	0	0			0	0	0	0			
1	65.13	100	0	0	0	-53.35	0	0	53.35	0	0	0	0	0	33	0	

EV3 -G2/G6

Span	Location (	% Span	Positive M	Negative	Positive S	Negative	Positive A	Negative	Positive R	Negative	Positive X	Negative	Positive Y	Negative	% Impact	% Impact	Neg Reaction
1	0	0	0	0	82.27	0	0	0	82.27	0	0	0	0	0	33	0	
1	2.08	3.2	131.47	0	79.34	-1.06	0	0			0	0	0	0			
1	6.51	10	378.63	0	72.02	-4.52	0	0			0	0	0	0			
1	13.03	20	662.18	0	62.04	-10.87	0	0			0	0	0	0			
1	14.94	22.9	727.38	0	59.42	-12.76	0	0			0	0	0	0			
1	19.54	30	850.64	0	52.32	-17.23	0	0			0	0	0	0			
1	22.67	34.8	907.38	0	48.11	-21.45	0	0			0	0	0	0			
1	26.05	40	955.58	0	42.88	-25.6	0	0			0	0	0	0			

Force Effect

Concrete Pier caps with Concrete Columns

- Sample Result Output

Loadcase ID: SELF Name: Self Load of Structure

Memb	Node	Fx kip	Fy kip	Fz kip	Mx kft	My kft
1	1	0.35	86.88	0.00	-0.00	0.00
1	2	-0.35	-32.04	0.00	-0.00	0.00
2	3	-0.01	102.41	0.00	-0.00	0.00
2	4	0.01	-47.57	0.00	-0.00	0.00
3	5	-0.33	87.80	0.00	-0.00	0.00
3	6	0.33	-32.96	0.00	-0.00	0.00
4	7	0.00	0.00	0.00	0.00	0.00
4	8	0.00	4.31	0.00	0.00	0.00
5	8	0.00	-4.31	0.00	0.00	0.00
5	9	0.00	6.23	0.00	0.00	0.00
6	9	0.00	-6.23	0.00	0.00	0.00
6	2	0.00	11.92	0.00	0.00	0.00
7	2	0.35	20.12	0.00	0.00	0.00
7	10	-0.35	-10.73	0.00	0.00	0.00
8	10	0.35	10.73	0.00	0.00	0.00
8	11	-0.35	-8.81	0.00	0.00	0.00
9	11	0.35	8.81	0.00	0.00	0.00
9	12	-0.35	6.27	0.00	0.00	0.00
10	12	0.35	-6.27	0.00	0.00	0.00
10	13	-0.35	8.20	0.00	0.00	0.00

Loadcase ID: DC1 Name:

Memb	Node	Fx kip	Fy kip	Fz kip	Mx kft	My kft	Mz kft
1	1	1.58	214.81	-0.04	0.17	0.51	-20.60
1	2	-1.58	-214.81	0.04	-1.61	-0.51	-42.75
2	3	-0.10	283.64	-0.01	-0.08	0.54	1.86
2	4	0.10	-283.64	0.01	-0.18	-0.54	2.24
3	5	-1.48	220.63	0.04	-0.08	0.52	20.23
3	6	1.48	-220.63	-0.04	1.78	-0.52	39.03
4	7	0.00	0.00	0.00	0.00	0.00	0.00
4	8	0.00	0.00	0.00	0.00	0.00	0.00
5	8	0.00	-48.46	0.00	-45.55	0.00	0.00
5	9	0.00	48.46	0.00	45.55	0.00	-42.40
6	9	0.00	-96.92	0.00	0.00	0.00	42.40
6	2	0.00	96.92	0.00	0.00	0.00	-293.11
7	2	1.58	117.89	-0.04	1.61	0.51	335.86
7	10	-1.58	-117.89	0.04	-1.61	-0.36	167.23
8	10	1.58	64.13	-0.04	-48.93	0.36	-167.23
8	11	-1.58	-64.13	0.04	48.93	-0.33	223.35
9	11	1.58	10.37	-0.04	1.61	0.33	-223.35
9	12	-1.58	-10.37	0.04	-1.61	-0.08	294.42
10	12	1.58	-43.19	-0.04	-48.74	0.08	-294.42
10	13	-1.58	43.19	0.04	48.74	-0.05	256.63
11	13	1.58	-96.75	-0.04	1.61	0.05	-256.63

Loadcase ID: DC2 Name: EV3

Memb	Node	Fx kip	Fy kip	Fz kip	Mx kft	My kft	Mz kft
1	1	3.18	288.47	-0.05	0.24	0.77	-41.77
1	2	-3.18	-288.47	0.05	-2.39	-0.77	-85.51
2	3	-0.12	431.60	-0.01	-0.14	0.81	2.18
2	4	0.12	-431.60	0.01	-0.27	-0.81	2.51
3	5	-3.06	296.65	0.06	-0.13	0.78	41.44
3	6	3.06	-296.65	-0.06	2.69	-0.78	81.15
4	7	0.00	0.00	0.00	0.00	0.00	0.00
4	8	0.00	0.00	0.00	0.00	0.00	0.00
5	8	0.00	-57.59	0.00	-54.13	0.00	0.00
5	9	0.00	57.59	0.00	54.13	0.00	-50.39
6	9	0.00	-115.18	0.00	0.00	0.00	50.39
6	2	0.00	115.18	0.00	0.00	0.00	-348.33
7	2	3.18	173.29	-0.05	2.39	0.77	433.84
7	10	-3.18	-173.29	0.05	-2.39	-0.54	305.69
8	10	3.18	91.02	-0.05	-74.94	0.54	-305.69
8	11	-3.18	-91.02	0.05	74.94	-0.49	385.34
9	11	3.18	8.75	-0.05	2.39	0.49	-385.34
9	12	-3.18	-8.75	0.05	-2.39	-0.12	445.35
10	12	3.18	-67.47	-0.05	-69.25	0.12	-445.35
10	13	-3.18	67.47	0.05	69.25	-0.07	386.32

Capacity and Load Rating

Capacity and Flexural Rating

- Capacity based on Axial-Bending Interaction

Formulas for capacity calculation (Interpolation)

$$M_x = \frac{P_o - P_x}{P_0 - P_b} * M_b$$

If compression controls

$$M_x = \frac{(P_x)*(M_b - M_n)}{P_b} + M_n$$

If tension controls

P_b= Balance load strength	M_b= Balanced moment strength
P_o= Pure design axial load strength	M_n= Pure design moment strength
P_x= Actual factored axial force	M_x= Flexural moment capacity

- $RF=(\phi Mn - M_{DL}*\gamma_{DL}) / \gamma_{LL}*V_{LL+I}$

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Points for Panel Discussion

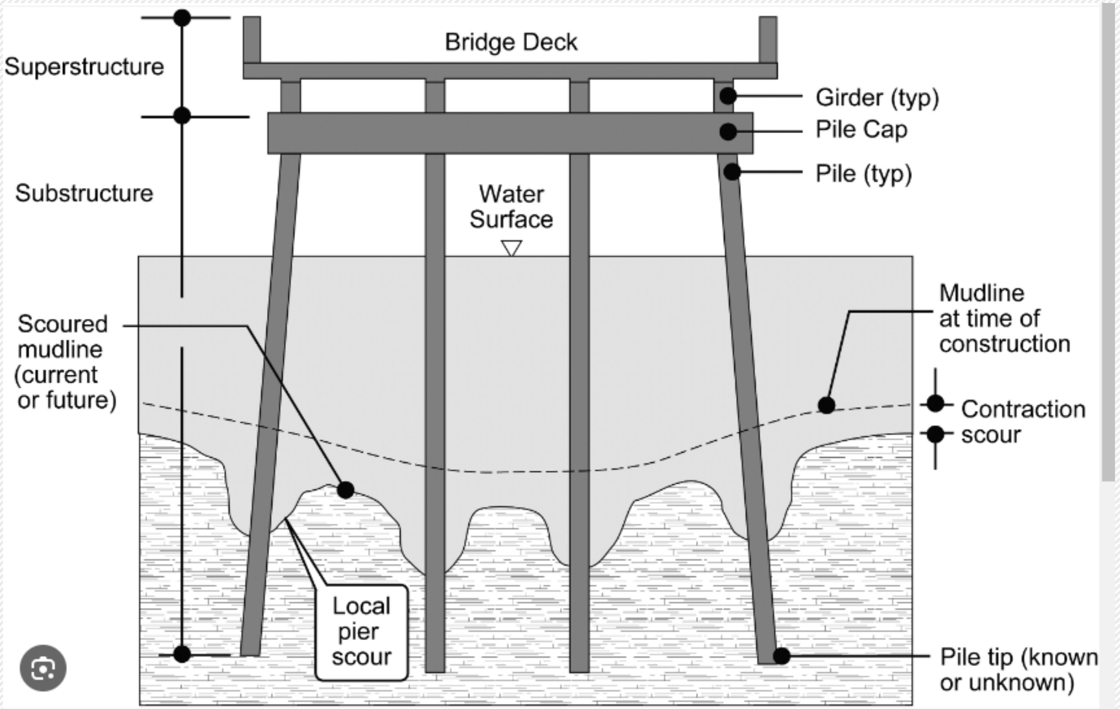
Parameters require Engineering judgment:

- Embedment length of columns
- Timber species and capacity adjustment factors
- Timber inventory capacity MBE 6B.5.2.7
- Wind load

Panel Discussion

Embedment Length

- Design plans not available
- Foundation information not available
- Available: Stream profile
- Is it good enough to assume the unsupported length to be above the scour mudline with some factor of safety?



Panel Discussion

Timber

- Species (Southern pine, Red pine…)
- Condition treatment factor (0.74 to 1.0)
- Load sharing factor (load path)

ROUND TIMBER POLES AND PILES

Table 6.3.1 Applicability of Adjustment Factors for Round Timber Poles and Piles

		ASD only	ASD and LRFD							LRFD only		
		Load Duration Factor	Temperature Factor	Condition Treatment Factor	Size Factor	Column Stability Factor	Critical Section Factor	Bearing Area Factor	Load Sharing Factor	Format Conversion Factor K_F	Resistance Factor ϕ	Time Effect Factor
$F_c' = F_c$	x	C_D	C_t	C_{ct}	-	C_P	C_{cs}	-	C_{ls}	2.40	0.90	λ

Table 6A Reference Design Values for Treated Round Timber Piles Graded per ASTM D25

(Tabulated design values are for normal load duration and wet service conditions. See NDS 6.3 for a comprehensive description of design value adjustment factors.)

Species	Bending F_b	Shear parallel to grain F_v	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain F_c	Modulus of elasticity E	Modulus of elasticity E_{min}	Specific Gravity[4] G
	Design values in pounds per square inch (psi)						
Pacific Coast Douglas Fir[1]	2,050	160	490	1,300	1,700,000	690,000	0.50
Red Pine[2]	1,350	125	270	850	1,300,000	520,000	0.42
Southern Pine (Grouped)[3]	1,950	160	440	1,250	1,500,000	600,000	0.55

1. Pacific Coast Douglas Fir reference design values apply to this species as defined in ASTM Standard D 1760.
2. Red Pine reference design values apply to Red Pine grown in the United States.
3. Southern Pine reference design values apply to Loblolly, Longleaf, Shortleaf, and Slash Pines.
4. Specific gravity, G, based on weight and volume when oven-dry.

4

REFERENCE DESIG

Panel Discussion

Timber Capacity

- Capacity based on NDS Supplemental Table 6A

Table 2.3.3 Temperature Factor, C_t

Reference Design Values	In-Service Moisture Conditions[1]	C_t $T \le 100°F$	C_t $100°F < T \le 125°F$	C_t $125°F < T \le 150°F$
F_t, E, E_{min}	Wet or Dry	1.0	0.9	0.9
F_b, F_v, F_c, and $F_{c\perp}$	Dry	1.0	0.8	0.7
	Wet	1.0	0.7	0.5

1. Wet and dry service conditions for sawn lumber, structural glued laminated timber, prefabricated wood I-joists, structural composite lumber, and wood structural panels are specified in 4.1.4, 5.1.5, 7.1.4, 8.1.4, and 9.3.3, respectively

6.3.5 Condition Treatment Factor, C_{ct}

Reference design values are based on air dried conditioning. If kiln-drying, steam-conditioning, or boultonizing is used prior to treatment (see reference 20) then the reference design values shall be multiplied by the condition treatment factors, C_{ct}, in Table 6.3.5.

Table 6.3.5 Condition Treatment Factor, C_{ct}

Air Dried	Kiln Dried	Boulton Drying	Steaming (Normal)	Steaming (Marine)
1.0	0.90	0.95	0.80	0.74

6.3.8 Column Stability Factor, C_P

Reference compression design values parallel to grain, F_c, shall be multiplied by the column stability factor, C_P, specified in 3.7 for the portion of a timber pole or pile standing unbraced in air, water, or material not capable of providing lateral support.

6.3.9 Critical Section Factor, C_{cs}

Reference compression design values parallel to grain, F_c, for round timber piles and poles are based on the strength at the tip of the pile. Reference compression design values parallel to grain, F_c, in Table 6A and Table 6B shall be permitted to be multiplied by the critical section factor. The critical section factor, C_{cs}, shall be determined as follows:

$$C_{cs} = 1.0 + 0.004L_c \quad (6.3\text{-}1)$$

3.7.1 Creosote

Creosote has been widely used to protect wood from biological attack since 1865. It is a distillate of tar produced by the carbonization of bituminous coal consisting of various polyaromatic hydrocarbons over a wide range of boiling temperatures. Common applications for creosote pressure treated timber products include timber piling for foundations on land, in fresh water, and in salt water, bridge timber and railroad ties.

3.7 Solid Columns

3.7.1 Column Stability Factor, C_P

3.7.1.1 When a compression member is supported throughout its length to prevent lateral displacement in all directions, $C_P = 1.0$.

3.7.1.2 The effective column length, ℓ_e, for a solid column shall be determined in accordance with principles of engineering mechanics. One method for determining effective column length, when end-fixity conditions are known, is to multiply actual column length by the appropriate effective length factor specified in Appendix G, $\ell_e = (K_e)(\ell)$.

3.7.1.3 For solid columns with rectangular cross section, the slenderness ratio, ℓ_e/d, shall be taken as the larger of the ratios ℓ_{e1}/d_1 or ℓ_{e2}/d_2 (see Figure 3F) where each ratio has been adjusted by the appropriate buckling length coefficient, K_e, from Appendix G.

3.7.1.4 The slenderness ratio for solid columns, ℓ_e/d, shall not exceed 50, except that during construction ℓ_e/d shall not exceed 75.

3.7.1.5 The column stability factor shall be calculated as follows:

$$C_P = \frac{1+(F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1+(F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} \quad (3.7\text{-}1)$$

where:

F_c^* = reference compression design value parallel to grain multiplied by all applicable adjustment factors except C_P (see 2.3), psi

$$F_{cE} = \frac{0.822\, E_{min}'}{(\ell_e/d)^2}$$

c = 0.8 for sawn lumber

c = 0.85 for round timber poles and piles

c = 0.9 for structural glued laminated timber or structural composite lumber

AI Engineers

Panel Discussion

Timber Operating Capacity

- MBE 6B.5.2.7 only limits the operating capacity
- Limit to inventory capacity should also be included

Operating:

$$P/A=(4.8E)/(l/r)^2$$

Inventory: Operating/1.33

$$P/A=(3.6E)/(l/r)^2$$

6B.5.2.7—Timber

Determining allowable stresses for timber in existing bridges will require sound judgment on the part of the Engineer making the field investigation.

(1) Inventory Stress

The inventory unit stresses should be equal to the allowable stresses for stress-grade lumber given in the AASHTO Standard Specifications.

Allowable inventory unit stresses for timber columns should be in accordance with the applicable provisions of the AASHTO Standard Specifications.

(2) Operating Stress

The maximum allowable Operating unit stresses should not exceed 1.33 times the allowable stresses for stress-grade lumber given in the current AASHTO Standard Specifications. Reduction from the maximum allowable stress will depend upon the grade and condition of the timber and should be determined at the time of the inspection.

Allowable operating stress in lb/in.^{2} of cross-sectional area of simple solid columns should be determined by the following formulas but the allowable operating stress should not exceed 1.33 times the values for compression parallel to grain given in the design stress table of the AASHTO Standard Specifications.

$$\frac{P}{A}=\frac{4.8E}{(l/r)^2} \quad (6B.5.2.7\text{-}1)$$

where:

P = Total load, lb

A = Cross-sectional area, in.^{2}

E = Modulus of elasticity

ℓ = Unsupported overall length between points of lateral support of simple columns, in.

r = Least radius of gyration of the section, in.

For columns of square or rectangular cross-section, this formula becomes:

C6B.5.2.7

The material and member properties based on as-built information may need to be adjusted for field conditions such as weathering or decay. The Engineer's judgment and experience are required in assessing actual member resistance.

Eq. 6B.5.2.7-1 is based on the Euler long-column formula with two adjustments as follows. First, E is reduced by dividing by 2.74. This corresponds to a safety factor of 1.66 for solid timber members according to the National Design Specifications for Wood Construction (2005). Then the Euler allowable stress is multiplied by 1.33 to provide an operating level allowable stress as shown in Eq. 6B.5.2.7-1.

For square and rectangular columns, substituting $d/\sqrt{12}$ for the radius of gyration, r, in Eq. 6B.5.2.7-1 results in Eq. 6B.5.2.7-2.

Panel Discussion

Wind Load

- Superstructure: No wind load
- Substructure should we consider wind load if so, why?

AI Engineers

QUESTIONS?