TENNENG
TENNIS ENGINEERING COMPANY

STRUCTURAL CALCULATIONS
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CALCULATIONS

Wood Duck Ct Bridge Replacement Project
New Bridge Substructures

December 20, 2024

REGISTERED PROFESSIONAL
ENGINEER
83622PE
Digitally signed by Timothy S Tennis
Date: 2024.12.20 14:47:21-08'00'
OREGON
JULY 8, 2014
TIMOTHY SHAYNE TENNIS

EXPIRES: 6/30/2026

Index of Calculation Sheets

Prepared for:
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Calculate Wind Loads for Bridge Design:

Calculate Wind Load On Structure:

"Wind pressure calculations are in accordance with the 8th edition of the AASHTO LRFD Bridge Design Specifications at a wind speed of 115mph, ground surface roughness C, and wind exposure category C"

$V_w := 110 \cdot mph$ "Design 3-second gust wind speed for project"

$Z := 10 \cdot ft$ "Max height of structure above ground"

$G := 1.0$ "Gust factor per AASHTO Table 3.8.1.2.1-1"

$C_D := 1.3$ "Drag coefficient per AASTO Table 3.8.1.2.1-1"

$K_d := 0.95$ "Wind directionality factor per AASHTO Temp 2.3.5.2.3b"

$$K_z := \frac{\left(2.5 \cdot \ln\left(\frac{Z \div ft}{0.0984}\right) + 7.35\right)^2}{478.4} = 0.75$$

"Exposure and elevation coefficient for Wind Exposure Category C per AASHTO EQ 3.8.1.2.1-3"

$$P_{wind} := 2.56 \cdot 10^{-6} \cdot \left(\frac{V_w}{mph}\right)^2 \cdot K_z \cdot G \cdot C_D \cdot ksf = 30\ psf$$

"Design wind pressure per AASHTO EQ 3.8.1.2.1-1"

$$R_{WS} := P_{wind} \cdot 57 \cdot in \cdot \frac{55 \cdot ft}{2} = 3.93\ kip$$

"Wind on structure reaction at each end of bridge"

Calculate Wind Load On Live Load:

$$R_{WLtran} := 0.10 \cdot klf \cdot \frac{55 \cdot ft}{2} = 2.75\ kip$$

"AASHTO 3.8.1 for transverse wind on live load reaction at each end of bridge"

$$R_{WLlong} := 0.04 \cdot klf \cdot \frac{55 \cdot ft}{2} = 1.1\ kip$$

"AASHTO 3.8.1 for longitudinal wind on live load reaction at each end of bridge"

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Concrete Sill Calculations

"HL-93 loading controls for the Strength I load case by inspection of all loads"

Sill Material Properties:

$F_y := 60 \cdot ksi$ $E_s := 29000 \cdot ksi$ "Material properties of reinforcing steel"

$\gamma_c := 150 \cdot pcf$ $f'_c := 4000 \cdot psi$ "Using 4000psi concrete"

$$E_c := 33000 \cdot ksi \cdot \left(\gamma_c \div \frac{kip}{ft^3}\right)^{1.5} \cdot \sqrt{f'_c \div ksi} = 3834.25\ ksi$$

Sill Section Properties:

$d_{cap} := 30 \cdot in$ "Depth of sill"

$b_{cap} := 48 \cdot in$ "Width of sill"

Calculate Vertical Spring Stiffness of Soil:

"Subgrade reaction modulus is calculated per Foundation Analysis By Ronald F. Scott (1981). It is conservative to use the uncorrected blow counts for shallow depths"

$b_{foot} := 4 \cdot ft$ "Width of footing"

$L_{foot} := 19.67 \cdot ft$ "Length of footing"

$N_{blow} := 30$ "Average ground improved soil values"

$K_s := 1.8 \cdot N_{blow}^{0.489} \cdot \frac{kgf}{cm^3} = 343.1\ pci$ "Subgrade reaction modulus for 30cm plate"

$$K_{s\_mod} := K_s \cdot \frac{\left(\frac{L_{foot}}{b_{foot}} + 0.5\right)}{1.5 \cdot \left(\frac{L_{foot}}{b_{foot}}\right)} = 251.99\ pci$$ "Modified for footing size per Terzaghi"

$Spring := K_{s\_mod} \cdot b_{foot} \cdot 12 \cdot in = 1742\ \frac{kip}{ft}$ "Calculate spring stiffness for beam model with springs at 12in centers"

Calculate Unfactored Loads on Sill:

$R_{DL} := 6340 \cdot plf$ "Dead load reaction from slabs & rail (weight of slabs and rail only)

$R_{DW} := 1404 \cdot plf$ "Present + future wearing surface reaction from slabs (average 4.375in thickness)"

$R_{LL} := 9650 \cdot plf$ "Live load reaction from single HL-93 lane over 10ft width with impact"

$DL_{wing} := 3250 \cdot lbf$ "Weight of wing wall"

$DL_{cap} := (2.5 \cdot ft \cdot 4 \cdot ft) \cdot \gamma_c = 1500\ plf$ "Uniform load of sill"

Load Combinations Referenced Below:

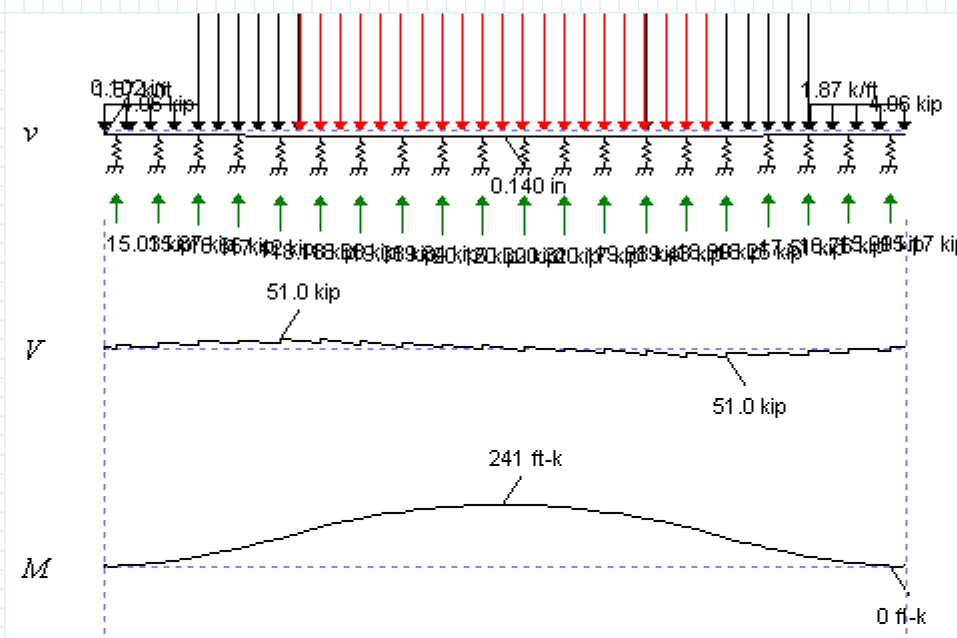
The above loads were factored and applied to a beam model using the following load combinations

Strength I = 1.25*DL + 1.5*DW + 1.75*LL

Strength II = 1.25*DL + 1.5*DW + 1.35*LL

Calculate Factored Internal Forces:

"All load combinations have been checked for the loads listed above. The following are the controlling internal forces by inspection"



Strength I Loading of Sill for Maximum Positive & Negative Moment

$M_{pos\_s1} := 241 \cdot kip \cdot ft$ $M_{neg\_s1} := 0 \cdot kip \cdot ft$

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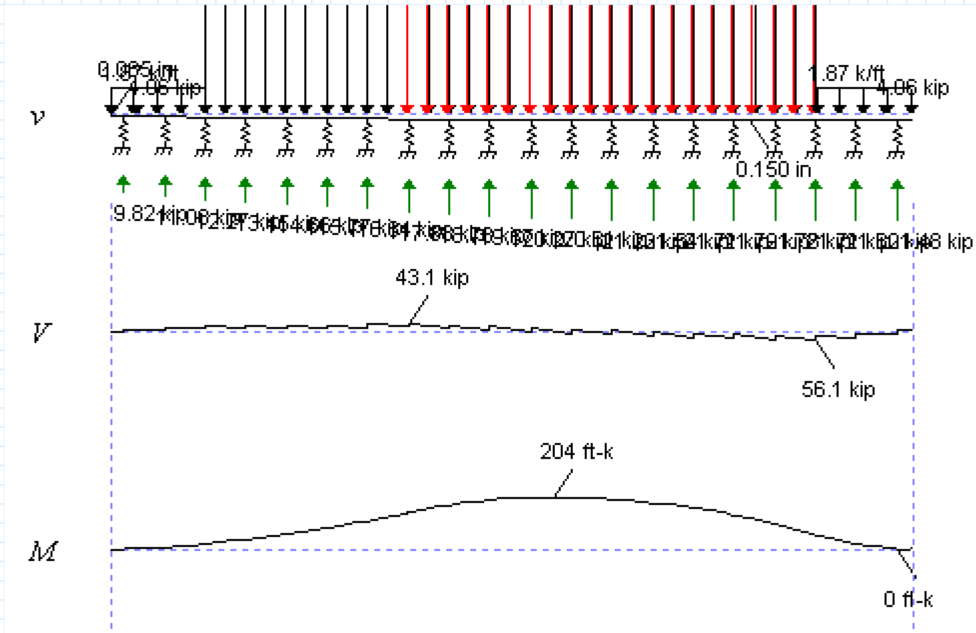

Strength I Loading of Sill for Max Shear

$$V_{s1} := 56.1 \cdot kip$$

Strain Compatibility For Positive Moment Capacity:

Concrete Beam Properties:

$h := d_{cap} = 30\ in$ "Total height of beam"

$h_{flange} := d_{cap} = 30\ in$ "Thickness of flange"

$b_{eff} := b_{cap} = 48\ in$ "Effective width of flange"

$b_{web} := b_{cap} = 48\ in$ "Width of web"

$f_r := 7.5 \cdot \sqrt{f'_c \cdot psi} = 474\ psi$ "Modulus of rupture"

Reinforcing Properties:

"Assume top and bottom mats of reinforcement only participate in vertical loading. This is conservative"

$n_{layers} := 2$ "Number of layers"

$A_{s_1} := 2.64 \cdot in^2$ $d_{s_1} := 3 \cdot in$ $f_{y_1} := 60 \cdot ksi$ $E_{s_1} := 29000 \cdot ksi$ "Properties Layer 1"

$A_{s_2} := 2.64 \cdot in^2$ $d_{s_2} := 26 \cdot in$ $f_{y_2} := 60 \cdot ksi$ $E_{s_2} := 29000 \cdot ksi$ "Properties Layer 2"

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Analysis Options:

$n_{slices} := 1000$ "Number of slices"

$\varepsilon_o := 0.003$ "Strain @ maximum compression fiber"

$c := 1.995 \cdot in$ "Depth of compression block"

$\sum F_s + \sum F = -0.04\ kip$ "Change c until this equation equals zero"

$M_n := \left| \sum M_s + \sum M \right| = 353\ kip \cdot ft$

$\phi := 0.9$ "AASHTO 5.5.4.2.1"

$\phi \cdot M_n = 318\ kip \cdot ft$ > $M_{pos\_s1} = 241\ kip \cdot ft$ OK

Strain Compatibility For Negative Moment Capacity:

Concrete Beam Properties:

$h := d_{cap} = 30\ in$ "Total height of beam"

$h_{flange} := d_{cap} = 30\ in$ "Thickness of flange"

$b_{eff} := b_{cap} = 48\ in$ "Effective width of flange"

$b_{web} := b_{cap} = 48\ in$ "Width of web"

$f_r := 7.5 \cdot \sqrt{f'_c \cdot psi} = 474\ psi$ "Modulus of rupture"

Reinforcing Properties:

"Assume top and bottom mats of reinforcement only participate in vertical loading. This is conservative"

$n_{layers} := 2$ "Number of layers"

$A_{s_1} := 2.64 \cdot in^2$ $d_{s_1} := 4 \cdot in$ $f_{y_1} := 60 \cdot ksi$ $E_{s_1} := 29000 \cdot ksi$ "Properties Layer 1"

$A_{s_2} := 2.64 \cdot in^2$ $d_{s_2} := 27 \cdot in$ $f_{y_2} := 60 \cdot ksi$ $E_{s_2} := 29000 \cdot ksi$ "Properties Layer 2"

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Analysis Options:

$n_{slices} := 1000$ "Number of slices"

$\varepsilon_o := 0.003$ "Strain @ maximum compression fiber"

$c := 2.307 \cdot in$ "Depth of compression block"

$\sum F_s + \sum F = 0.02\ kip$ "Change c until this equation equals zero"

$M_n := \left| \sum M_s + \sum M \right| = 383\ kip \cdot ft$

$\phi := 0.9$ "AASHTO 5.5.4.2.1"

$\phi \cdot M_n = 345\ kip \cdot ft$ > $M_{neg\_s1} = 0\ kip \cdot ft$ OK

Calculate Shear Capacity:

$\phi_v := 0.9$ "AASHTO 5.5.4.2.1"

$s_v := 10.5 \cdot in$ "Stirrup spacing"

$\alpha := 90 \cdot deg$ "Inclination of stirrups"

$A_s := 2.64 \cdot in^2$ "Area of main tension steel"

$A_v := 2 \cdot 0.31 \cdot in^2$ "(2) #5 legs per stirrup"

$d_v := d_{s_2} - \left(\frac{0.85 \cdot c}{2} \right) = 26.02\ in$ "Depth from compression to main tension steel"

$A_{vmin} := 0.0316 \cdot \sqrt{f'_c \cdot ksi} \cdot \frac{b_{web} \cdot s_v}{F_y} = 0.53\ in^2$ < $A_v = 0.62\ in^2$ OK

$\upsilon_u := \frac{|V_{s1}|}{\phi_v \cdot b_{cap} \cdot d_v} = 0.05\ ksi$ < $0.125 \cdot f'_c = 0.5\ ksi$ OK

$s_{vmax} := min(0.8 \cdot d_v, 24 \cdot in) = 20.82\ in$ > $s_v = 10.5\ in$ OK "AASHTO 5.8.2.9-1"

"Use simplified procedure since member is non-prestressed and has the minimum transverse reinforcement (AASHTO 5.8.3.4.1)"

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$\beta := 2.0$ $\theta := 45 \cdot deg$

$V_c := 0.0316 \cdot \beta \cdot \sqrt{f'_c \cdot ksi} \cdot (b_{web} \cdot d_v)$

$V_c = 157.87\ kip$ "AASHTO 5.8.3.3-3"

$V_s := \frac{A_v \cdot F_y \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s_v} = 92.18\ kip$ "AASHTO 5.8.3.3-4"

$V_n := min(V_c + V_s, 0.25 \cdot f'_c \cdot b_{cap} \cdot d_v) = 250.05\ kip$ "AASHTO 5.8.3.3-1"

$\phi_v \cdot V_n = 225.04\ kip$ > $V_{s1} = 56.1\ kip$ OK

Calculate Bearing Pressure Under Sill:

$A_{sill} := L_{foot} \cdot b_{foot} = 78.68\ ft^2$ "Area under sill"

$S_{sill} := \frac{b_{foot} \cdot L_{foot}^2}{6} = 257.94\ ft^3$ "Section modulus of sill footprint"

$P_{loads} := 1.0\ (R_{DL} \cdot 15\ ft + 2 \cdot DL_{wing} + DL_{cap} \cdot L_{foot}) + 1.0\ R_{DW} \cdot 15\ ft + 1.0 \cdot R_{LL} \cdot 10\ ft = 248.67\ kip$ "Total factored axial loads on sill"

$M_{loads} := 1.0 \cdot R_{LL} \cdot 10\ ft \cdot 2.5\ ft = 241.25\ kip \cdot ft$ "Total factored eccentric loading of live loads on sill"

$Pressure_{max} := \frac{P_{loads}}{A_{sill}} + \frac{M_{loads}}{S_{sill}} = 4096\ psf$ "Max factored bearing pressure under sill"

$Pressure_{min} := \frac{P_{loads}}{A_{sill}} - \frac{M_{loads}}{S_{sill}} = 2225\ psf$ "Min factored bearing pressure under sill"

Check Allowable Bearing Pressure:

$Q_{allow} := 4100\ psf$ "Allowable bearing pressure per geotech report"

$Q_{allow} = 4100\ psf$ > $Pressure_{max} = 4095.76\ psf$

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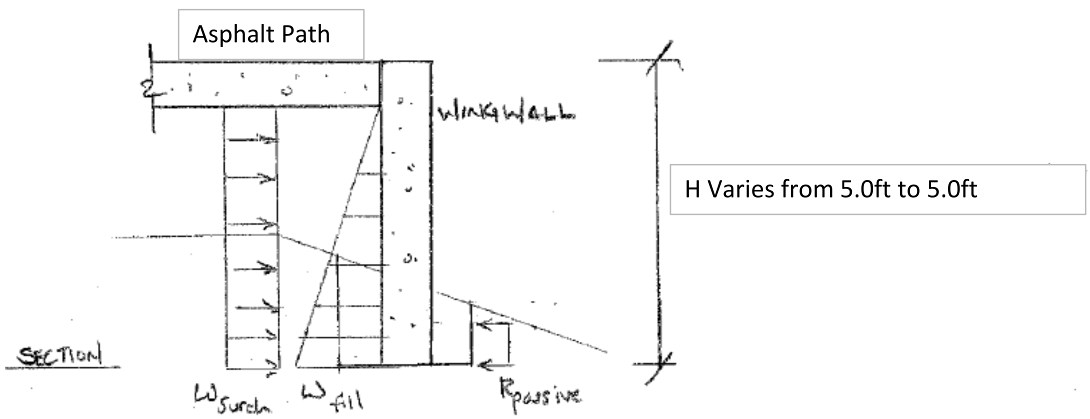
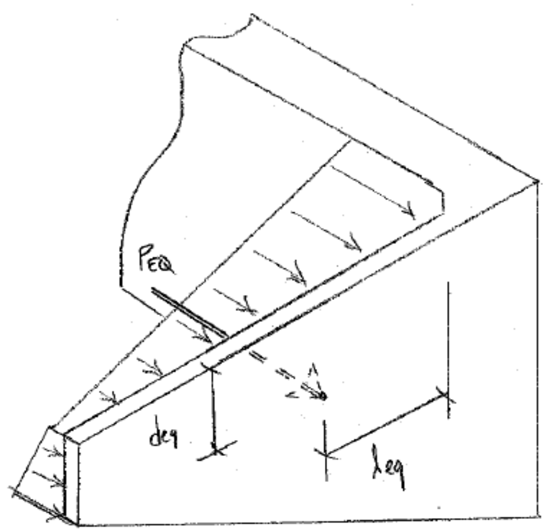
Wing Wall Calculations





Lwall = 3.0ft

$\phi'_f = 32°$

$\delta = 20°$

$\beta = 0°$

$\rho = 2.78$

$K_o = 0.449$

$\gamma = 115$ pcf

LOAD FACTORS:

STRENGTH I

EARTH PRESSURE ⇒ 1.5

SURCHARGE ⇒ 1.75

Tie wingwall to abutment / pile cap / end wall.

Length of wall from inside face of abutment to end of wall

L wall 3 ft

Height of wall at abutment

Hwall_max 5 ft

Height of wall at end of wall

Hwall_min 5 ft

Slope of top wall is even

Slope of bot wall = #DIV/0! :1

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Effective friction angle of soil
phi'f 32

Friction angle between fill and wall (Table 3.11.5.3-1)
delta 20

Angle of fill to horizontal (Fig 3.11.5.3-1)
beta 0

Angle of back of wall to horizontal (Fig 3.11.5.3-1)
theta 90

Pressure coefficient

	k_a	0.276	w/ GAMMA =	2.78
	k_o	0.449		
	k_between	0.362056		
Use?	k_o	0.449		

Soil unit weight
gamma 115 pcf

Load factor for Earth Loads at Strength I
Load factor 1.5 Strength I

Factored Earth Loads pressure
P_EH 77.4 psf/ft

Load factor for Surcharge Loads at Strength I
Load factor 1.75 Strength I
Delta_P_surch 575.0 psf

	End		Abut	
Location	0	ft	3	
Hwall	5	ft	5	ft
p_surch	2875	lb/ft	2875	lb/ft
at depth	2.5	ft	2.50	ft
p_typ	967	lb/ft	967	lb/ft
at depth	3.33333333	ft	3.33	ft
p_total	3842	lb/ft	3842	lb/ft
at depth	2.71	ft	2.71	ft

p_average	3842	lb/ft
P_resultant	11527	lb
Moment arm	1.50	ft from inside face of abutment
Wingwall moment	17290	ft*lb
	17	ft*k

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Wingwall Moment:

Dist from Abutment	Mu kip-ft	Mr kip-ft
0	17.29	42.1
1	7.68	42.1
2	1.92	42.1
3	0.00	42.1

Flexural strength of Rectangular Reinforced Concrete Section

f'c	ksi	**4**	**4**	
Beta_1		0.85	0.85	
Bar size	#4	**4**	**4**	
Spacing	in	**8**	**8**	
As	in^2 / ft	0.295	0.295	
Dist from Abut, ft		0	**3**	
Hwall	ft	**5**	**5**	
b	in	48	48	Assumes 1' of height not participating
d	in	**9.75**	**9.75**	Dist to tension steel
fy	ksi	**60**	**60**	
a	in	0.43	0.43	Depth of compression block
Mn	in*k	674	674	
	ft*k	56.2	56.2	
phi		**0.75**	**0.75**	
phi*Mn	ft*k	42.1	42.1	
E	ksi	3834	3834	
I	in^4 / ft	927	927	

Check Strength of Bearing Pads:

Calculate Slab Reaction Force:

$R_{DL} := 6340 \cdot plf$ "Dead load reaction from slabs & rail (weight of slabs and rail only)

$R_{DW} := 1404 \cdot plf$ "Present + future wearing surface reaction from slabs (average 5.75in thickness)"

$R_{LL\_s1} := 9650 \cdot plf$ "Live load reaction from single HL-93 lane over 10ft width with impact"

$W_{slab} := 30\ in$ "Width of single slab"

$R_{slab} := \left(R_{DL} + R_{DW} + R_{LL\_s1}\right) \cdot W_{slab} = 43.49\ kip$ "Service Level Reaction of Slab"

Check Compressive Stress on Bearing Pad:

$G := 0.08\ ksi$ "Design Shear Modulus per AASHTO 14.7.5.2"

$w_{bearing} := 6\ in$ "Width of bearing pads"

$L_{bearing} := 13\ in$ "Length of single bearing pads beneath slab"

$h_{bearing} := 0.5\ in$ "Thickness of bearing"

$$S := \frac{L_{bearing} \cdot w_{bearing}}{2 \cdot h_{bearing} \cdot \left(L_{bearing} + w_{bearing}\right)} = 4.11$$ "AASHTO 14.7.5.1-1"

$$\sigma_s := \frac{R_{slab}}{w_{bearing} \cdot 2\ L_{bearing}} = 278.75\ psi$$ "Compressive stress on bearing"

$\sigma_s = 278.75\ psi$ < $1.0 \cdot G \cdot S = 328.42\ psi$ *OK* "AASHTO 14.7.6.3.2-1"

< $0.80\ ksi = 800\ psi$ *OK* "AASHTO 14.7.6.3.2-2"

Check Compressive Deflection Bearing Pad:

$E_c := 30\ ksi$ "AASHTO 14.7.6.3.3"

$$\Delta := \frac{R_{slab} \cdot h_{bearing}}{2 \cdot L_{bearing} \cdot w_{bearing} \cdot E_c} = 0.005\ in$$

$\Delta = 0.005\ in$ < $0.09 \cdot h_{bearing} = 0.05\ in$ *OK* "AASHTO 14.7.6.3.3"

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