Thornton Tomasetti

Project

National Mall Carousel
Final Supporting Documents
12/18/2024

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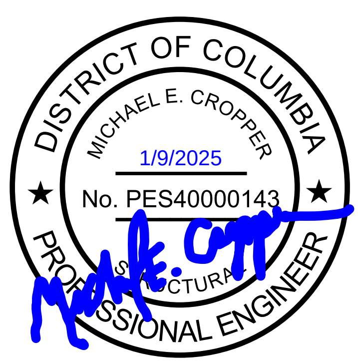

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PROJECT: National Mall Carousel	PROJECT #: M23152.00	DATE: 10/29/2024	
SUBJECT: Wind Load Calculation	BY: HWC	SHEET:	of
	CHECKED BY:	DRAWING #:	

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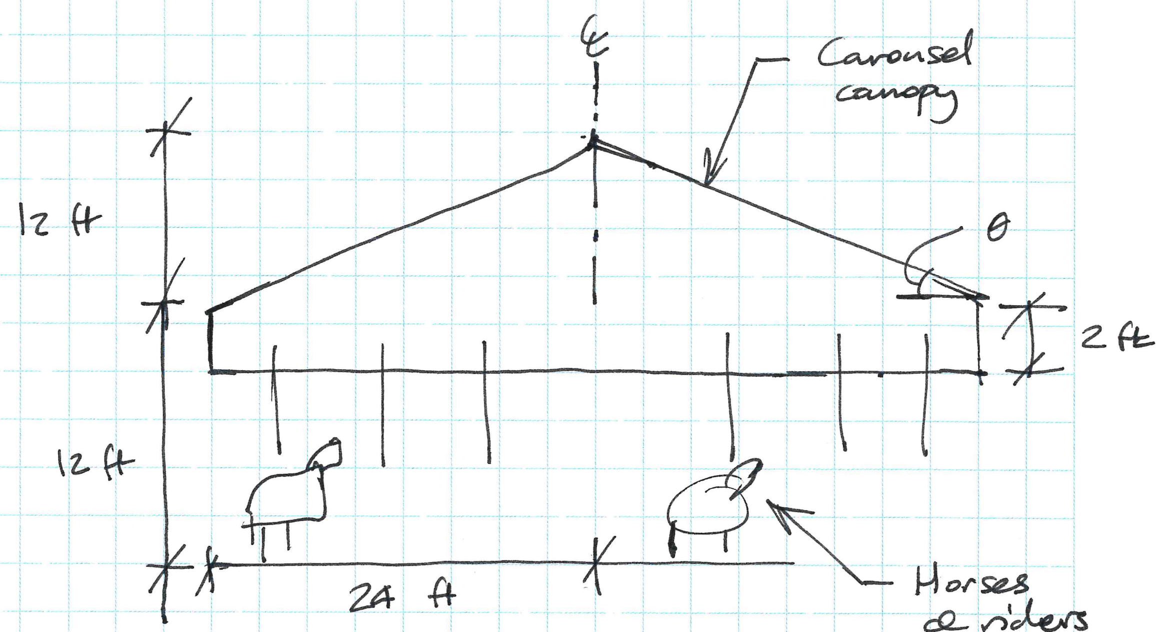
1. Grade Beam Design

Calculate Loads Imposed on Carousel Foundations

Assumed self-weight of carousel
= 30,000 lbs — treat as dead load

Assumed self-weight of riders
= 60 x 200 — treat as live load
= 12,000 lbs



Assume 30 psf snow

$\tan\theta = \frac{12}{24}$

$\theta = 26.6°$

Apply chapter 29 of ASCE 7-16

Follow procedure for wind loads on MWFRS circular bldgs, silos & tanks: (see table 29.1-2)

Risk category of structure: II

Basic wind speed
$V = 115$ mph

$K_d = 1.0$ — circular domes — table 26.6-1

Exposure category B

Topographic factor
$K_{zt} = 1.0$
Elevation factor
$K_e = 1.0$

Enclosure classification:

Treat the sides as 50 % porous

∴ partially ~~enclosed~~ open

$GC_{pi} = \pm 0.18$

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Gust effect factor

$G = 0.85$ (assume rigid due to diagonal braces & relatively low height)

$k_h = 0.66$ — table 26.10-1 for $h = 25$ ft (conservative)

$q_h = 0.00256 k_z k_{zt} k_d k_e V^2$

$= 0.00256 \times 0.66 \times 1.0 \times 1.0 \times 1.0 \times 115^2 = 22.3$ psf

Force coefficients :

Ref. fig. 29.4-6

$H/D = 12/48$

$= 0.25$

$\therefore C_f = 1.3$ — for projected walls

$F = q_z G C_f A_f$

$= 22.3 \times 0.85 \times 1.3 \times (48 \times (12-2) \times 0.5 + 48 \times 2 + 12 \times 24)$

$= 15,376$ lbs

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Calculate moment due to wind:

$$M = 22.3 \times 0.85 \times 1.3 \times (48 \times (12-2) \times 0.5 \times 5 + 48 \times 2 \times 11 + 12 \times 24 \times 18)$$
$$= 183{,}333 \text{ lb-ft}$$

Calculate worst case downward vertical load from axle, ignoring wind:

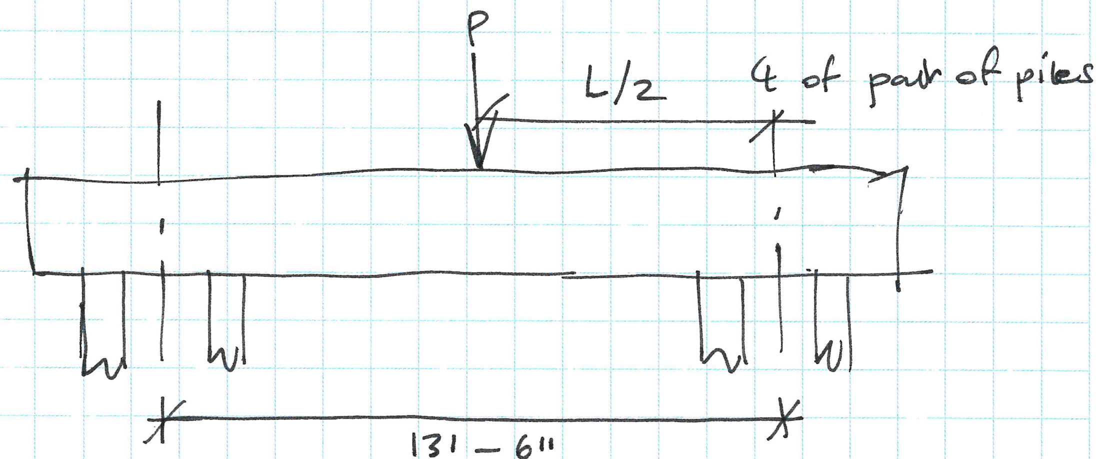
$$P = 1.2 \times \text{Dead} + 1.6 \times \text{Live} + 0.5 \text{ Snow}$$
$$= 1.2 \times 30{,}000 + 1.6 \times 12{,}000 + 0.5 \times 30 \times \pi \times 24^2$$
$$= \cancel{55{,}200 \text{ lbs}} \quad 82{,}343 \text{ lbs}$$

Compare vs. 1.4D:

$$P = 1.4 \times 30{,}000$$
$$= 42{,}000 \text{ lbs}$$

$\therefore$ $1.2D + 1.6L$ ~~+S~~ $+ 0.5S$ is worst case.

Design one grade beam for point load due to $1.2D + 1.6L + 0.5S$:



Try 30" x 30" grade beam :

Self - wt. of grade beam :

$$w_D = 2.5 \times 2.5 \times 150$$
$$= 938 \text{ lb/ft}$$

$$w_u = 1.2 \times 0.938$$
$$= 1.13 \text{ klf}$$

$$M_u = \frac{1.13 \times 13.5^2}{8} + \frac{82.3 \times 13.5}{4}$$
$$= 25.7 + 278 \text{ kip-ft}$$
$$= 303 \text{ kip-ft}$$

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Check 30" x 30" grade beam with 4 – #8 T+B & #4 @ 12" o.c. stirrups.

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4 \times 0.79 \times 60}{0.85 \times 4 \times 30} = 1.86 \text{ in.}$$

$$d = 30" - 3" - 0.5" - 1/2" = 26 \text{ in.}$$

$$\phi M_n = 0.90 \times 4 \times 0.79 \times 60 \left(26 - \frac{1.86}{2}\right)$$

$$= 4278 \text{ kip-in.}$$

$$= 357 \text{ kip-ft}$$

$$> M_u = 303 \text{ kip-ft} \therefore \text{ok}$$

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Check shear –

$$V_u = \frac{82.3}{2} + \frac{1.13 \times 13.5}{2}$$
$$= 48.8 \text{ kips}$$

$$\phi V_c = \phi 2 \lambda \sqrt{f_c'}\, b_w d$$
$$= 0.75 \times 2 \times 1 \times \sqrt{4000} \times 30 \times 26$$
$$= 73,997 \text{ lbs}$$
$$= 73.997 \text{ kips}$$

$V_u > 0.5 \phi V_c$ $\therefore$ provide minimum shear reinforcement.

$A_{v,min.}$ is greater of :

$$0.75\sqrt{f_c'}\frac{b_w}{f_{yt}} = 0.75 \times \sqrt{4000} \times \frac{30}{60000}$$
$$= 0.0237 \text{ in}^2$$

$$50\frac{b_w}{f_{yt}} = \frac{50 \times 30}{60000} = 0.025 \text{ in}^2$$

$\therefore$ #4 @ 12" o.c. $\Rightarrow$ $2 \times 0.2 = 0.4 \text{ in}^2$ is ok

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Check forces applied to piles –

Assume that vertical load is evenly distributed between all piles

∴ For worst case vertical load –

Applied force per pile $= \frac{82.3}{8}$

$= 10.3$ kips (factored)

Pile capacity

$= 10$ tons / pile

$= 20$ kips / pile ∴ ok

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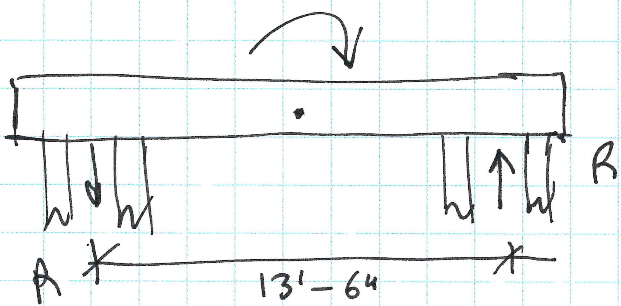
Check pile reactions – including wind.

M = 183 kip-ft



$R \times 13.5 = 183$

$R = \dfrac{183}{13.5}$

$= 13.5$ kips $\Rightarrow$ 6.75 kips per pile

Check for net uplift –

$0.9D + 1.0W$

Pile reaction under 0.9D $= 0.9 \times 30 / 8$

$= 3.4$ kips per pile

∴ Net uplift $= 3.4 - 6.75$

$= \underline{\underline{-3.35 \text{ kips}}}$

~~∴ Net uplift is present ...~~

~~[illegible]~~

Account for self-wt. of grade beams – 3.16 kips per pile (see next page)

∴ Net uplift $= 3.4 + 3.16 - 6.75 = -0.19$ kips

By inspection – accounting for grade beam self-wt. beyond ℄ of pile group will eliminate

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Calculate reactions under $1.2D + 1.0W + L$

Pile reactions due to $1.2D + L$

$= (1.2 \times 30 + 12) / 8$ — equally distributed between piles

$= 6$ kips / pile

Include self-wt. of grade beams:

pile reaction $= 6 + \dfrac{2 \times (13.5 \times 2.5 \times 2.5 \times 150)/1000}{8}$

$= 6 + 3.16$

$= 9.16$ kips

> 6.75 kips wind reaction per pile.

$\therefore$ no net uplift

Max. pile reaction

$= 9.16 + 6.75$

$= 15.91$ kips < 20 kips

$\therefore$ ok

Calculate lateral reactions –

Lateral force

$= 22.3 \times 0.85 \times 1.3 \times (48 \times (12-2) \times 0.5 + 48 \times 2 + 12 \times 24)$

$= 15{,}376$ lbs

$= 15.4$ kips total

$\Rightarrow \frac{15.4}{8} = 1.9$ kips per pile

$\therefore$ Piles designed for 5 kip per pile lateral capacity will be adequate.

2. Pilecap Design

PILE CAP DESIGN
LOCATION:
DATE: 10/29/24

MAX ASD DOWN REACTION	MAX ASD DOWNWARD LOAD ON PILE GROUP	MAX ASD DOWNWARD LOAD ON SINGLE PILE
10 KIPS	24 KIPS	7 KIPS

MAX ASD UPLIFT REACTION	MAX ASD UPLIFT LOAD ON PILE GROUP	MAX ASD UPLIFT LOAD ON SINGLE
KIPS	0 KIPS	0 KIPS

Pile Allowable Uplift	0	kips	
Pile Allowable Downward	20	kips	
Pile Strength Uplift	0	kips	
Pile Strength Downward	29	kips	
Pile Embeddment	6	in	
Rebar Clear Cover	3	in	
Design Factor	90%		
No. Pile Rows / Length	2		
No. Pile Rows / Width	2		
Row / Length Spacing	5.5	ft	66
Row / Width Spacing	5.5	ft	66
Effective Edge Distance	1.25	ft	15
Effective "Cap" Thickness	1.50	ft	18
Effective "Cap" Width	8	ft	96
Effective "Cap" Length	8	ft	96
Wall/Column Pier Length	2.50	ft	30
Wall/Column Pier Width	2.50	ft	30
Pile Tolerance	3	in	

Concrete Capacity	4000	psi
Short Direction As,min	2.52	in2
Short Direction Bar Size	#6	
Req'd # of Bars	6	
Short Direction Bar Qty.	23	
Long Direction As,min	2.52	in2
Long Direction Bar Size	#6	
Req'd # of Bars	6	
Long Direction Bar Qty.	17	

REMARKS

WORST-CASE AS-BUILT PILE FACTOR
1.1

PILE & CAP/SLAB UTILIZATION RATIOS

	PILE (DOWN)	PILE (UP)	CAP SHEAR	CAP FLEXURE
PILE CAPACITY	-	-	269%	38%
ACTUAL REACTION	31%	#DIV/0!	#DIV/0!	#DIV/0!

Note: this figure corresponds with the full design load capacity of the piles (20 kips per pile). Actual utilization is lower. If ticket booth weighs 10 kips, applied load per pile is 2.5 kips, therefore, utilization = (2.5/20)*269 = 33.6 % therefore okay

National Mall Carousel
AUGER-CAST PILE CAP DESIGN

LOCATION:	Washington, DC
DATE:	10/29/24

Total Piles	4
Pile Service Capacity (Dwn)	80
Pile Service Capacity (Up)	0
Short Direction Bar Area	10.16
Short Direction Rho,w	0.0134
Long Direction Bar Area	7.51
Long Direction Rho,w	0.0099
Flexural Depth	7.88

1-Way Shear at D Away, Short Direction		
No. Piles > D Away from Face	4	
Factored Shear Force	114	kips
Factored Shear Capacity	72	kips
Utilization	159%	NG

1-Way Shear at D Away, Long Direction		
No. Piles > D Away from Face	4	
Factored Shear Force	114	
Factored Shear Capacity	72	kips
Utilization	159%	NG

1-Way Shear & Flexure at Face, Short Direction		
No. Piles Outside Face	2	
Factored Shear Force	57	kips
Factored Moment	100	k-ft
w	21.00	in
Design Shear Stress	-158	psi
Roots f'c	-2.49	
Factored Shear Capacity	-89	kips
Utilization	-64%	OK
Factored Moment Capacity	360	k-ft
Utilization	28%	OK

1-Way Shear & Flexure at Face, Long Direction		
No. Piles Outside Face	2	
Factored Shear Force	57	kips
Factored Moment	100	k-ft
w	21	in
Design Shear Stress	-154	psi
Roots f'c	-2.43	
Factored Shear Capacity	-87	kips
Utilization	-66%	OK
Factored Moment Capacity	266	k-ft
Utilization	38%	OK

2-Way Shear at D/2 Away		
No. Piles > D/2 Away	20	
Factored Shear Force	571	kips
Critical Perimeter	152	in
Roots f'c	4.00	
Factored Shear Capacity	226	kips
Utilization	252%	NG

2-Way Shear at Face		
No. Piles Outside Face	4	
Factored Shear Force	114	kips
Critical Perimeter	120	in
w	21	
Special Upper Limit w/Beta	20.00	
Design Shear Stress	60	
Roots f'c	0.95	
Factored Shear Capacity	42	kips
Utilization	269%	NG