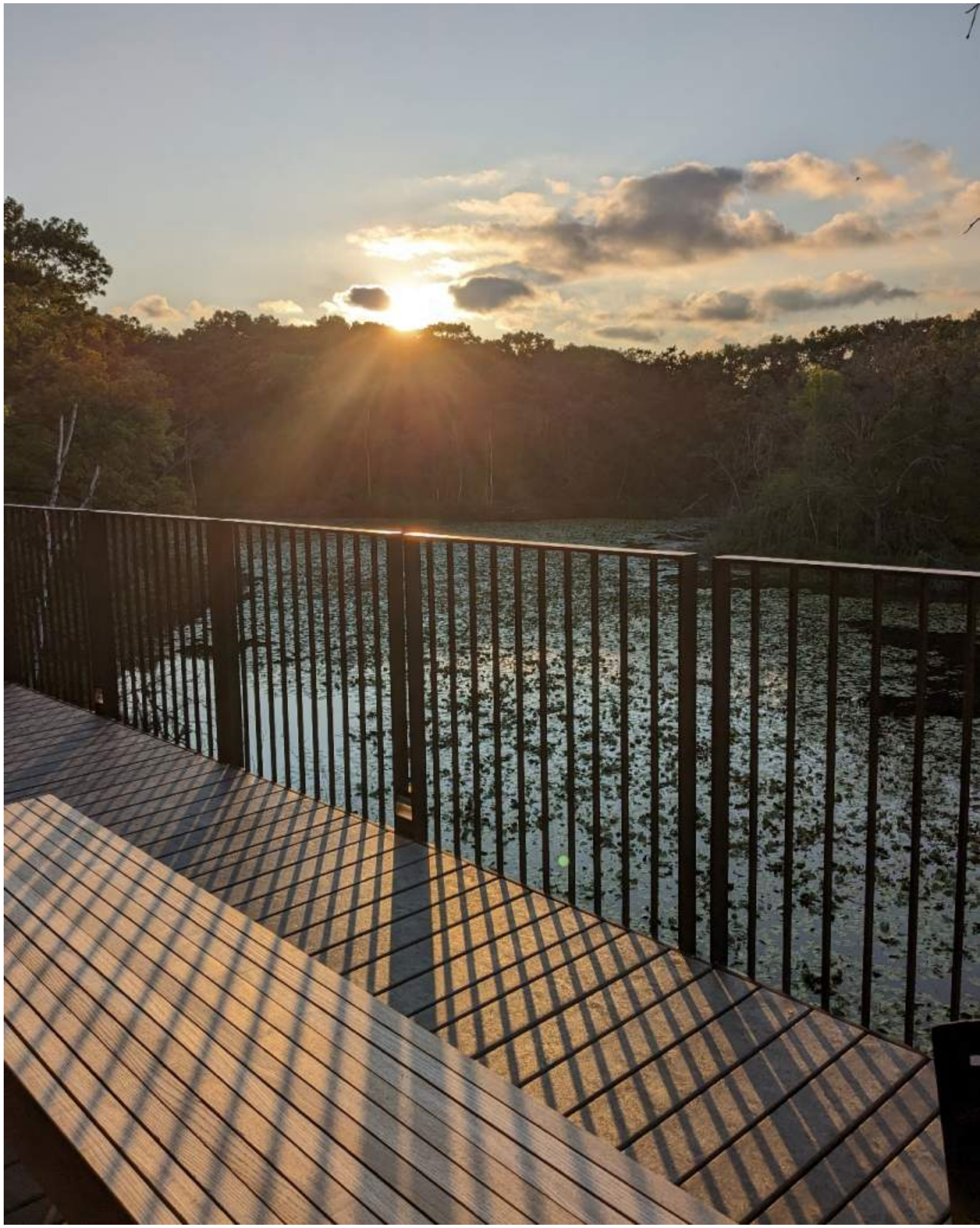
Treetop Trail at the Minnesota Zoo

A Case Study in Adaptive Reuse



UMN Structural Seminar Series

January 30, 2024

Thomas Root, PE
Minnesota Zoo

Jon Wacker, PE
HGA

Michael Osowski
PCL Construction

Fraser Reid, PE CEng MICE
Buro Happold

Craig Huhtala, PE
MBJ

Project Partners

MINNESOTA ZOO

SNOW KREILICH ARCHITECTS

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PCL

Project Partners Beyond Design

- Donors – private/public
- Community Stakeholders
- Government Partners
  - City of Apple Valley, MN
    - Fire Department
    - Building Officials
  - Fish and Wildlife Service
- Branding/PR/Marketing
- Interpretive and Wayfinding Elements

MINNESOTA ZOO BURO HAPPOLD MBJ MEYER BORGMAN JOHNSON PCL

Presentation Outline

- The Minnesota Zoo and the Monorail
- Project Introduction
- Existing Structure and Assessment
- System Selection
- Live Load Requirements
- Structural Analysis
- Thermal
- Wind / Lateral
- BREAK
- Vibrations
- Capacity Design
- Construction
- Questions

THE MINNESOTA ZOO AND THE MONORAIL

MINNESOTA ZOO

Vision: Our vision is a future where wildlife thrives in Minnesota and beyond.

Mission : To connect people, animals and the natural world to save wildlife.

Values: Stewardship, Excellence with Integrity, Smart Fun, Engage to Inspire, Diversity and Inclusion

About the Minesota Zoo

- Opened to the public in 1978
- Located on 485 acres in Apple Valley, Minnesota it is the 5th largest zoo in the US.
- Home to more than 4,400 animals and 485 species including 68 threatened and endangered species
- Annual attendance of over 1.2 million

About the monorail

- Originally referred to as the Skytrail.
- 1.25 miles long with a maximum elevation of 32 ft.
- With a single station, all rides were round trip.
- A monorail trip took about 25 minutes translating to an average speed of about 3 miles per hour.

A Brief History of the Monorail

- May 1978 - The Minnesota Zoo opens.
- September 1979 - The monorail begins operation. Exhibits and pedestrian routing were deliberately designed to incorporate views from the monorail.
- September 2013 - The monorail closes due to aging infrastructure, maintenance challenges due to mechanical obsolescence, and declining ridership.
- 2018 - Planning to convert the monorail into Treetop Trail commences.
- January 2021 - Design for Treetop Trail kicks off in earnest.
- April 2022 - Construction for Treetop Trail begins.
- July 2023 - Treetop Trail is completed with a total project cost of $39M.

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Existing Documentation

- Limited documentation was available from the original construction including a partial plan set showing:
  - Geometric alignment of the monorail
  - Typical steel section
  - Splices between track sections
  - Rigid connection to columns
  - Tabulated drilled pier foundation depths.
  - Loading diagram for a monorail train

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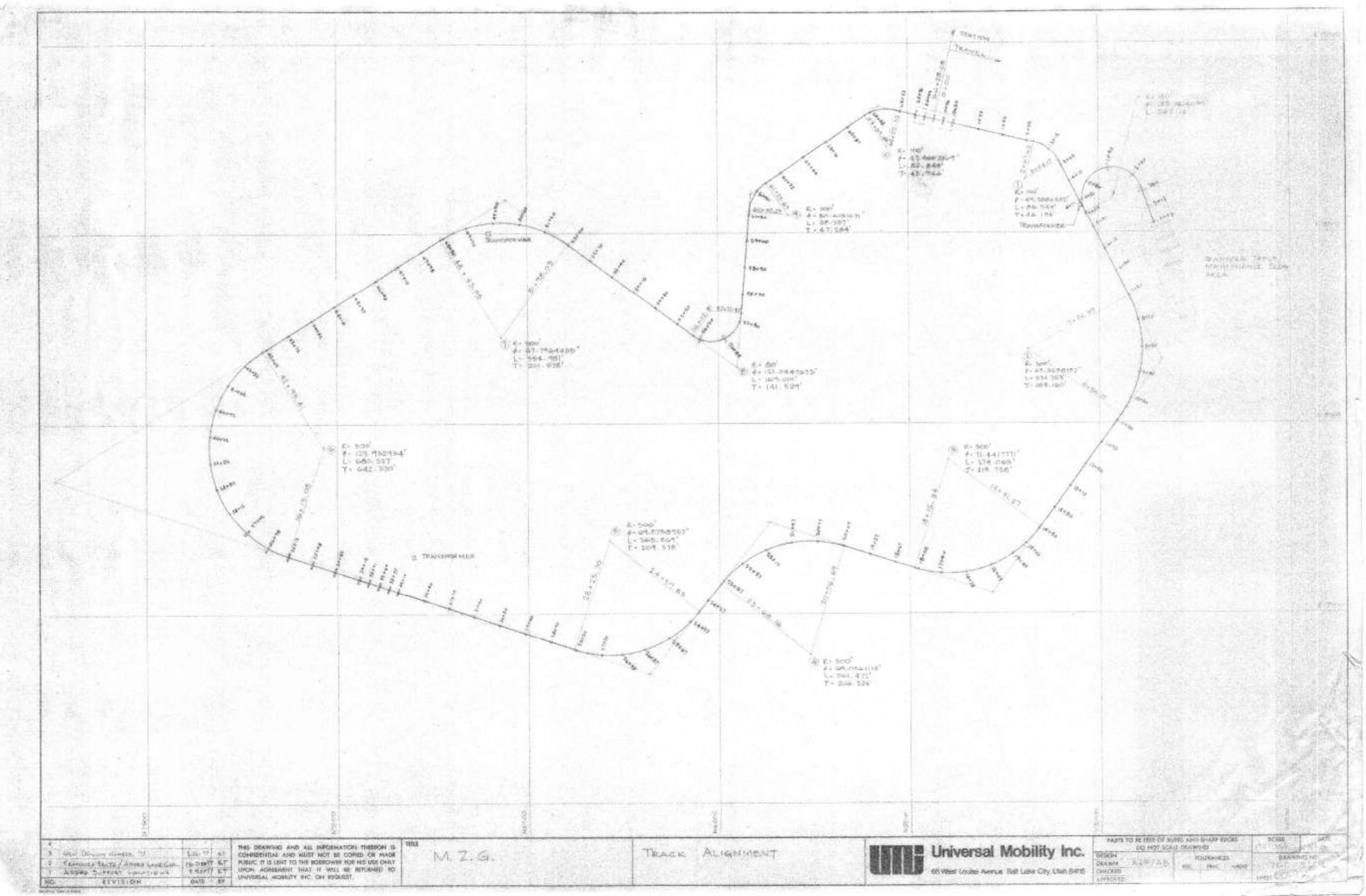
POLAR BEAR (5)
BEARDED SEAL (5)
ARCTIC FOX (5)
MOOSE (9)
WOOD BISON (10)
MUSK OX (6)



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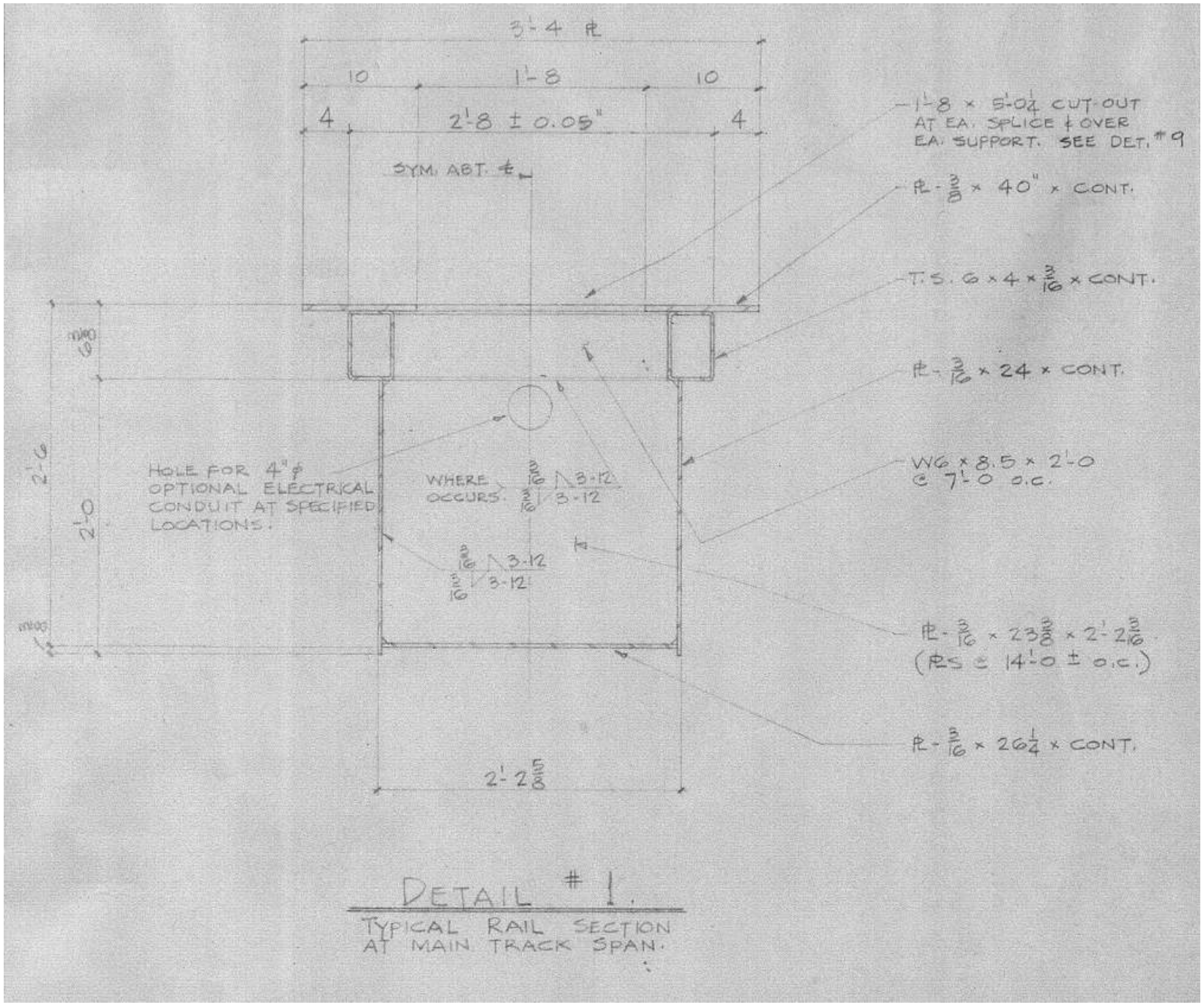
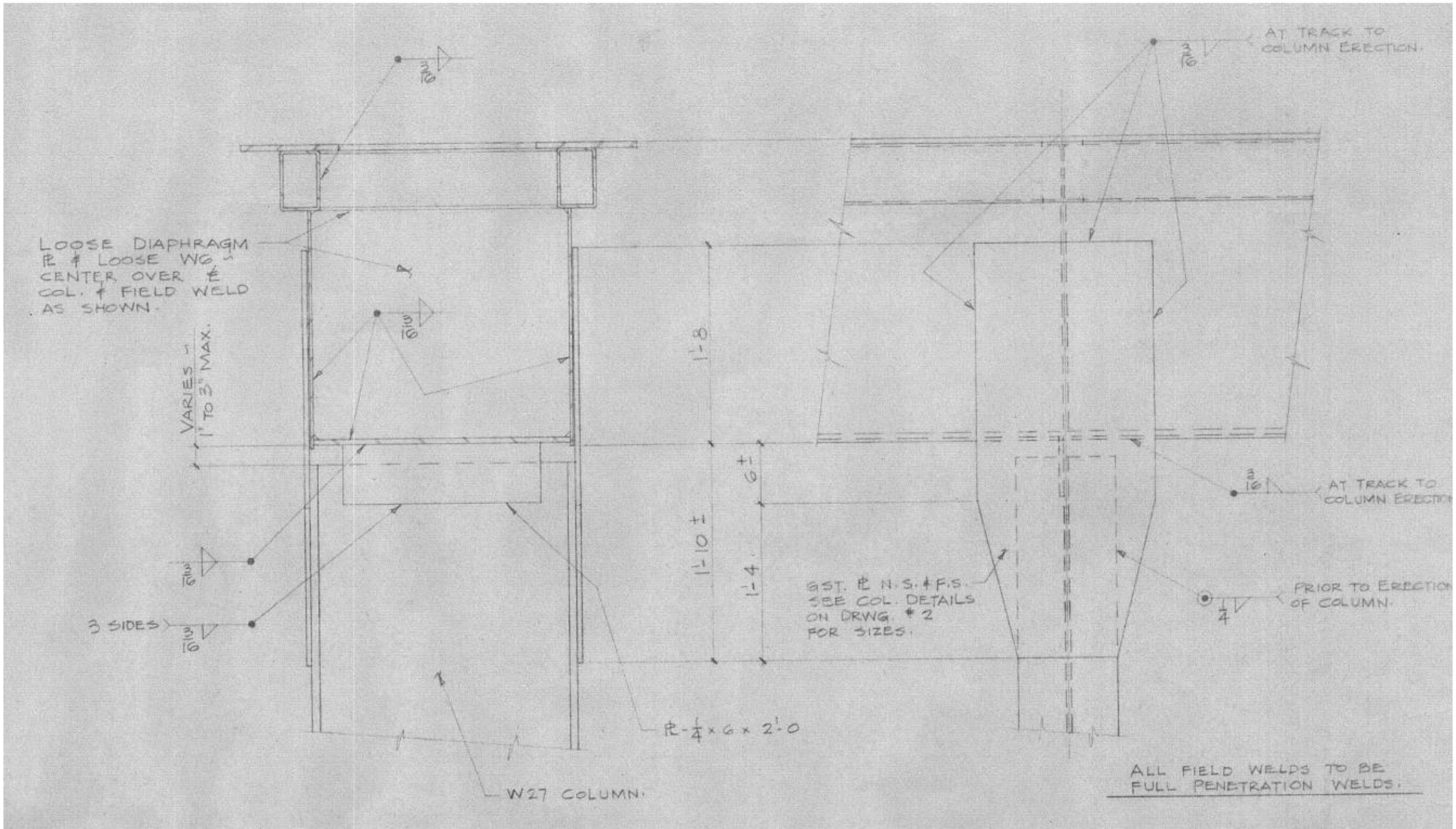


3'-4 PL
10
1'-8
10
4
2'-8 ± 0.05"
4
SYM. ABT. ℄
1'-8 × 5'-0¼ CUT-OUT AT EA. SPLICE & OVER EA. SUPPORT. SEE DET. #9
PL - 3/8 × 40" × CONT.
T.S. 6 × 4 × 3/16 × CONT.
PL - 3/16 × 24 × CONT.
W6 × 8.5 × 2'-0 @ 7'-0 O.C.
HOLE FOR 4"φ OPTIONAL ELECTRICAL CONDUIT AT SPECIFIED LOCATIONS.
WHERE OCCURS.
2'-6
2'-0
PL - 3/16 × 23⅜ × 2'-2⅜ (PLS @ 14'-0 ± O.C.)
PL - 3/16 × 26¼ × CONT.
2'-2⅝
DETAIL # 1.
TYPICAL RAIL SECTION AT MAIN TRACK SPAN.

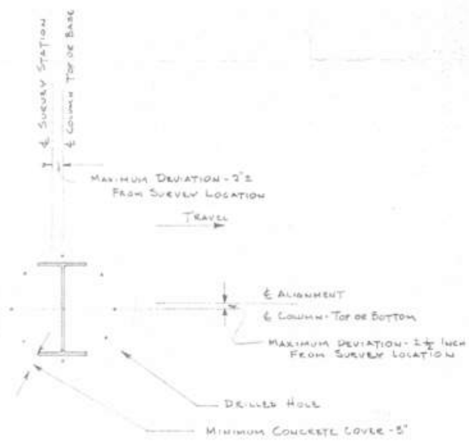


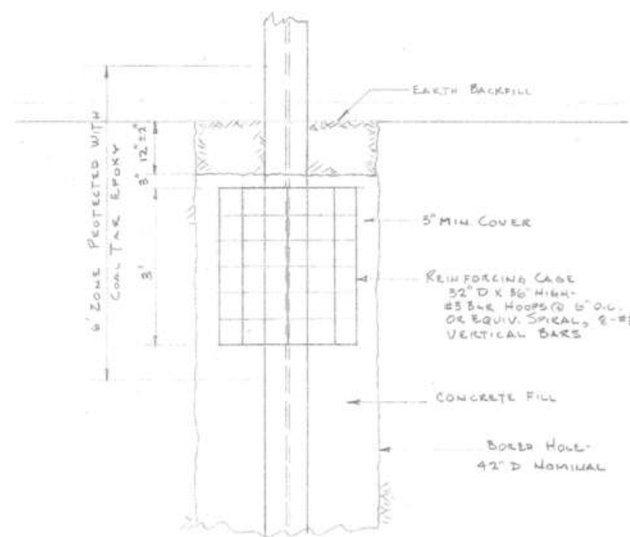
COLUMN TO TRACK CONNECTION

Zoo

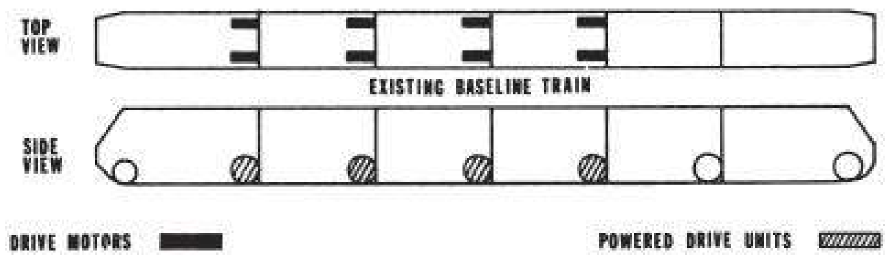
UNIVERSAL MOBILITY, INC. - MINNESOTA ZOOLOGICAL GARDENS - ZOORIDE FOUNDATIONS

Support Location Station	Type of Support	Top of Foundation Elevation	Total length of Column	Hole Information Bottom Elevation	Approx. Depth
0+00	Grade Beam	96.83	None	-----	----
0+26	Grade Beam	96.83	None		
0+52	Grade Beam	96.83	None		
1+22	84# column	98.29	34 feet	64.0	20 feet
1+92	" "	99.95	34 feet	65.5	20 feet
2+55	" "	01.43	33 feet	68.0	20 feet
3+15	" "	02.85	30 feet	72.5	19 feet
3+65	" "	04.03	31 feet	72.5	19 feet
4+10	" "	05.09	33 feet	72.0	20 feet
4+71.39	" "	05.33	34 feet	71.0	19 feet
5+04.43	" "	05.33	30 feet	75.0	17 feet
5+50	" "	05.33	29 feet	76.0	18 feet
6+15	" "	05.79	31 feet	74.5	18 feet
6+80	" "	06.25	30 feet	76.0	18 feet
7+50	" "	06.74	23 feet	83.5	16 feet
8+20	" "	07.24	29 feet	78.0	18 feet
8+90	94# column	07.73	39 feet	68.5	20 feet
9+60	94# column	08.22	36 feet	72.0	19 feet
10+30	94# column	08.72	36 feet	72.5	19 feet
11+00	94# column	09.21	39 feet	70.0	20 feet
11+70	145# column	09.71	41 feet	68.5	20 feet
12+40	145# column	10.20	43 feet	67.0	21 feet
13+10	145# column	10.69	43 feet	67.5	21 feet
13+80	145# column	11.19	41 feet	70.0	20 feet
14+50	94# column	11.97	40 feet	71.5	20 feet
15+00	84# column	12.61	35 feet	77.5	19 feet
15+44	84# column	13.17	31 feet	82.0	18 feet
16+07	84# column	13.98	34 feet	79.5	1[illegible] feet
16+75	145# column	14.85	43 feet	71.5	21 feet
17+40	145# column	15.68	43 feet	72.5	21 feet
18+05	145# column	16.51	44 feet	72.0	21 feet
18+67	94# column	17.31	38 feet	79.0	20 feet
19+37	145# column	18.20	46 feet	72.0	21 feet
20+07	145# column	19.10	48 feet	71.0	22 feet
20+77	145# column	20.00	50 feet	69.5	22 feet
21+47	145# column	20.89	49 feet	71.5	22 feet
22+17	145# column	21.79	50 feet	71.5	22 feet
22+87	145# column	22.68	51 feet	71.5	22 feet
23+57	145# column	23.58	46 feet	77.5	21 feet
24+27	94# column	24.48	39 feet	85.0	20 feet
24+97	145# column	25.37	43 feet	82.0	21 feet
25+67	84# column	26.27	23 feet	03.0	15 feet
26+37	84# column	27.16	27 feet	00.0	17 feet
26+95	84# column	27.92	28 feet	99.5	17 feet
27+50	145# column	28.62	48 feet	80.5	22 feet
28+20	145# column	29.15	67 feet	62.0	38 feet
28+90	145# column	28.77	67 feet	62.0	38 feet
29+60	145# column	28.39	66 feet	62.0	38 feet
30+30	145# column	28.01	66 feet	62.0	38 feet
31+00	145# column	27.63	66 feet	61.5	38 feet
31+70	145# column	27.25	65 feet	62.0	38 feet
32+40	84# column	26.88	29 feet	97.5	17 feet





Monorail Loading



TRAIN CONFIGURATIONS OF MINNESOTA ZOOLOGICAL GARDEN VEHICLES

The baseline train (empty) weight is carried by the bogies, identified as front-to-rear (left-to-right in Figure 9), as follows: No. 1 - 10,450 pounds; No. 2, 3, 4 and 5 - 8,100 pounds; No. 6 - 7,300 pounds; and No. 7 - 4,600 pounds.

- Information on the weight of the monorail trains was found in project documentation.
- Cars had a capacity of 96 seated passengers, possibly up to 120 with standing passengers.
- Based on length of car it was determined that the monorail exerted a load of about 1 kip / foot onto the track.

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Existing Documentation

- There was a lot important information missing from the existing documentation including:
  - Specifications
  - Geotechnical information
  - General structural notes
  - Material properties – Design strengths and material characteristics
  - Connection details at non-rigid connections

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PROJECT INTRODUCTION

"By transforming the monorail into a walking trail, visitors are given the opportunity to **immerse themselves in nature**, much like the monorail aimed to do, but with the **freedom to curate their own adventure**"

Supporting MN Zoo Mission

1 CONNECTION

2 COMMUNITY

3 COMPASSION

The trail builds literal and mindful **connections** between people, animals, and the natural world.

The trail bridges and embraces larger, more diverse **communities**, increasing local and global support.

The trail inspires and spurs **compassion** for wildlife, leading to immediate and long-term action.

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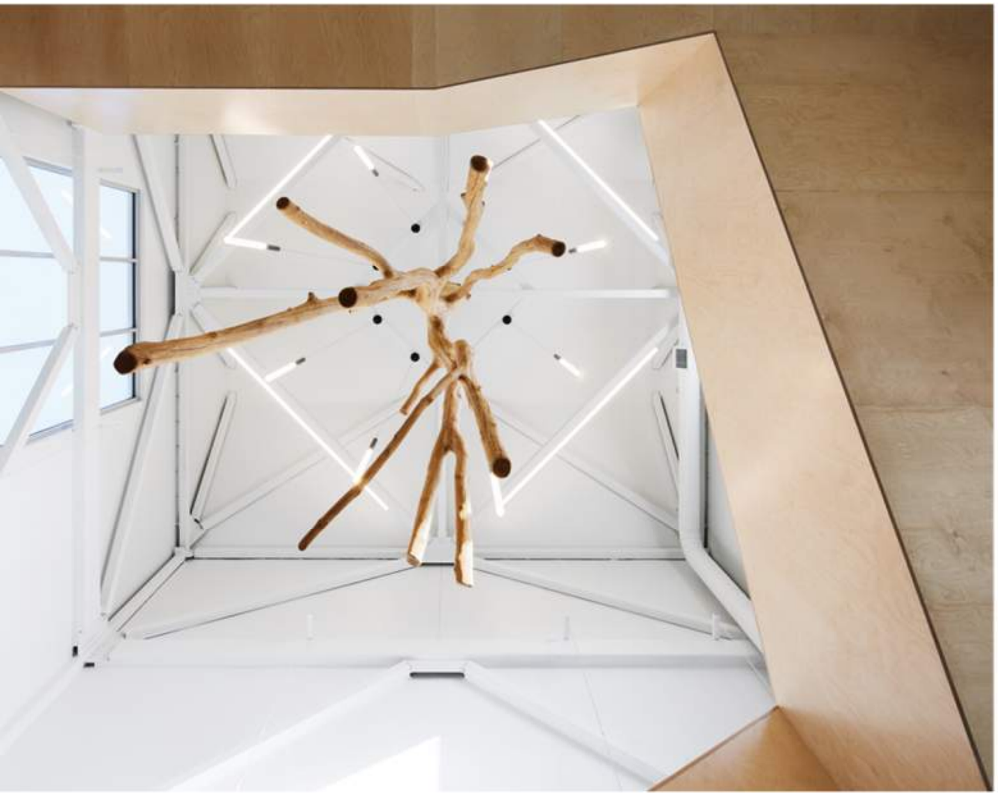
Connecting Existing Zoo Trails and Experiences



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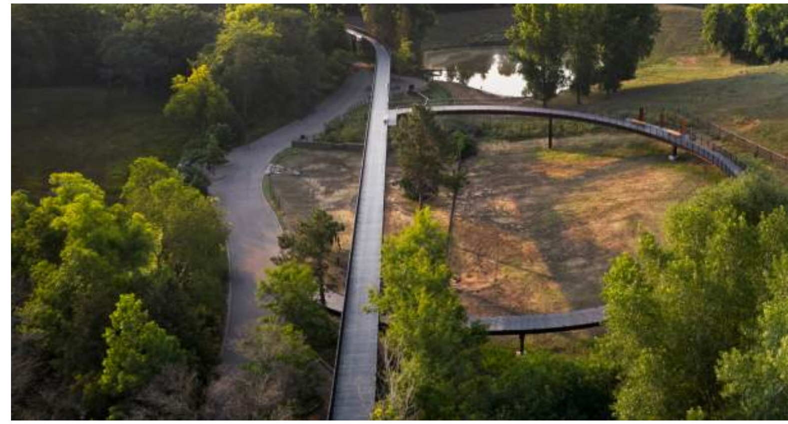
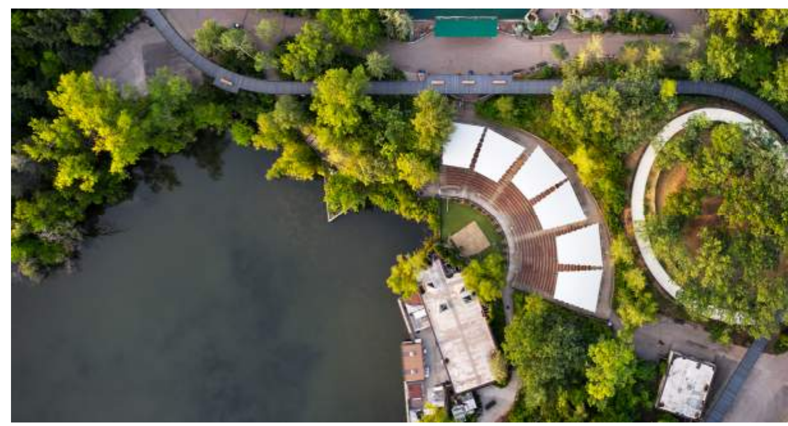
Enhancing Access





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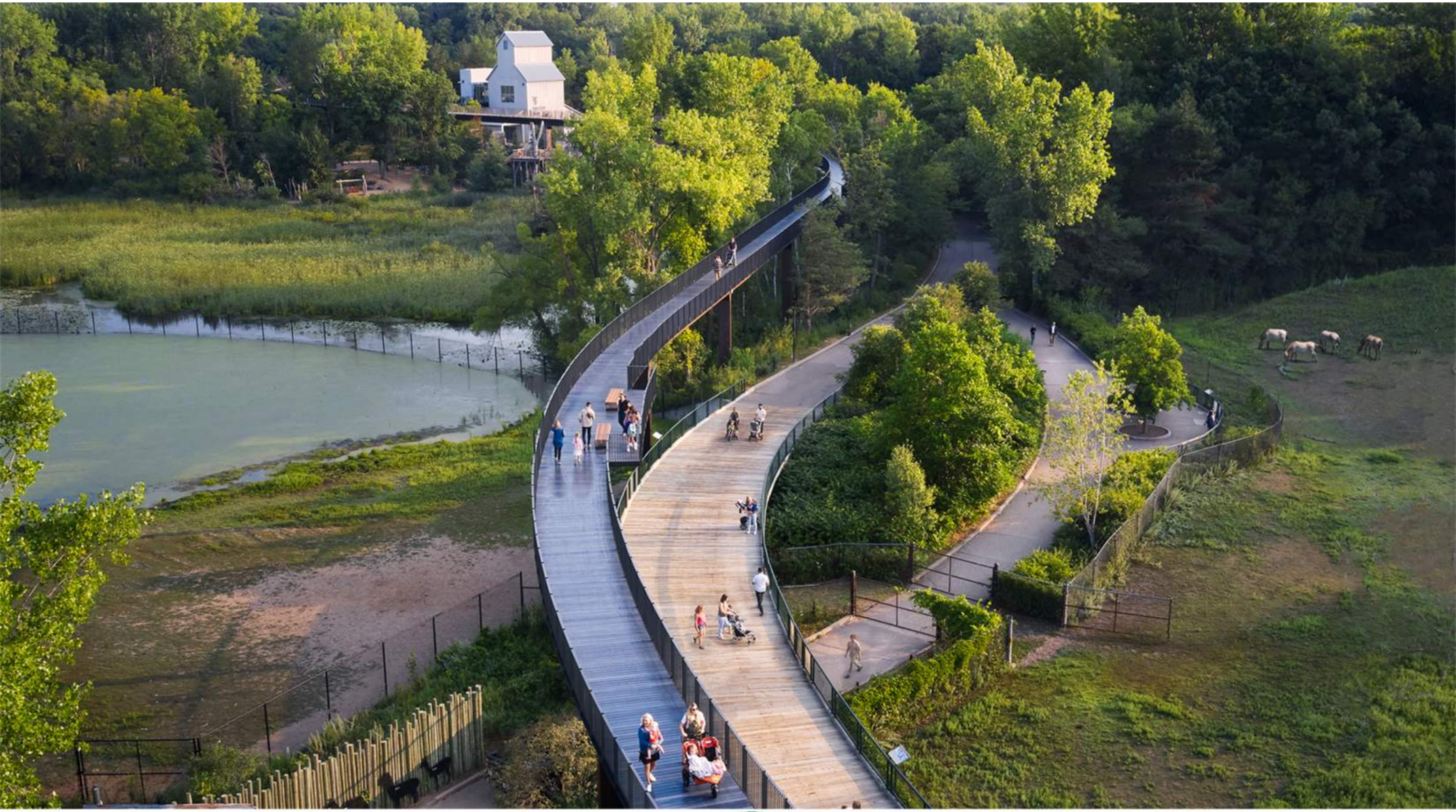

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Minnesota Zoo

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Other Project Constraints

- Disruption Avoidance – continuous operation, special events, animal needs, wildlife
- Need to be built from above
- Modular construction
- Accommodate winter construction
- Existing structural capacity
- Existing structural conditions
- Existing infrastructure - fiber optic lines
- Budget
- 1.25 miles long
- Designed and built during the global pandemic

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EXISTING CONDITIONS

Condition Assessment

- Develop confidence in the accuracy of existing drawings
- Fill in documentation gaps
- Determine extent of deterioration

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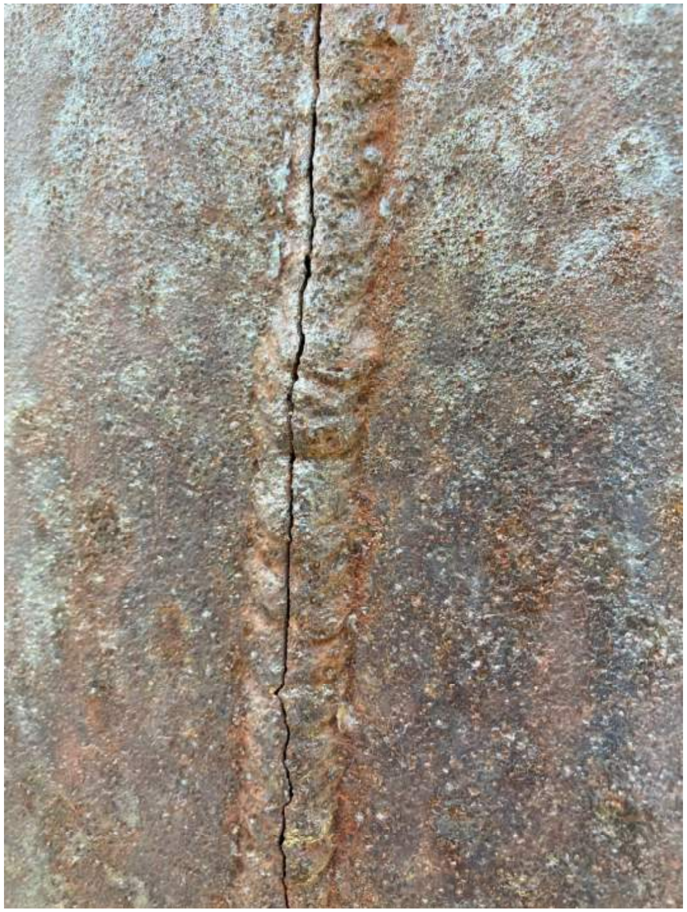
Rippling of side plates

Fractured Welds



Support Conditions

- **One-way column slider**

- **Two-way column slider**

- **One-way ground slider**

Support Conditions

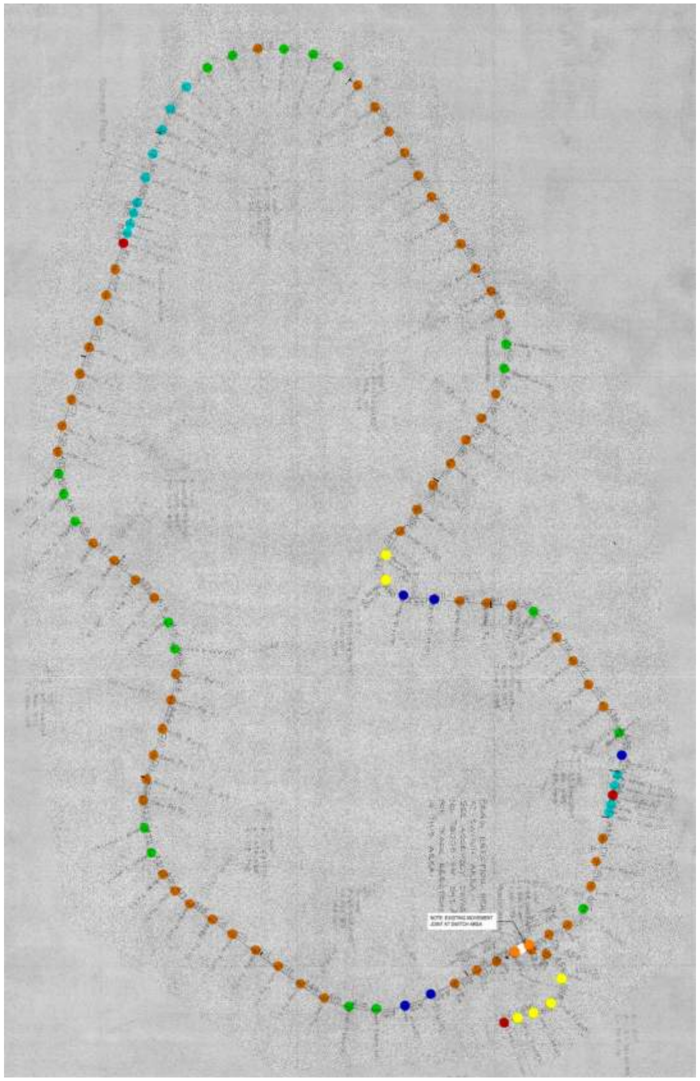
- **Fixed Column Connection**

- **Fixed Grade Connection**

Support Conditions

TRAIL SUPPORTS - KEY PLAN

SYMBOL	DESCRIPTION
COLUMN SUPPORTS	
● (orange)	FIXED
● (blue)	ONE-WAY
● (green)	TWO-WAY
GRADE SUPPORTS	
● (red)	ANCHOR
● (cyan)	ONE-WAY
● (yellow)	TWO-WAY
● (magenta)	SPECIAL



Support Conditions

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Support Conditions

Condition Assessment Scope

- Steel structure
  - Verify detail compliance with existing structure
  - Ultrasonic thickness measurements to estimate section loss
  - Visual weld inspection
  - Mag particle weld inspection
  - Ultrasonic weld inspection
  - Coupon testing for both strength and chemical composition
- Foundations
  - Top of drilled pier verification
  - Reinforcement verification using ground penetrating radar
  - Concrete sampling for compressive strength testing
  - Parallel seismic testing to estimate pier depths

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Steel Testing Methods

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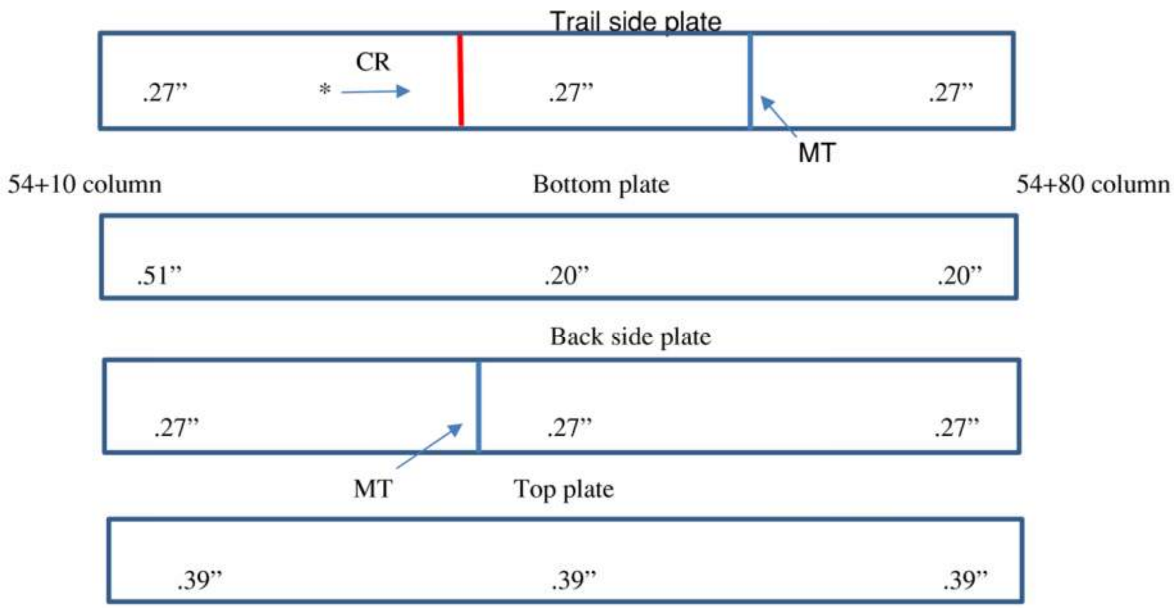
Steel Observations Results

Areas and work performed this day: Locations identified by closest column, all observations this date performed from boom lift, unless noted observations noted were on bottom plate to side plates of box beam section. see below for findings.



See above for apx. Location of thickness readings. Magnetic particle (MT) inspection performed on field weld plate splice inside plate and outside plate. See sketch for approximate location. * Note previously identified side plate splice weld crack report # 1 dated 6-28-21. 11 Angle to tube stitch welds cracked.

Steel Material Testing

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The Science You Build On.

Project B2105206

Snow Kreilich Architects, Inc.
219 North 2nd Street Suite 120
Minneapolis, MN 55401

Project Information:

Test Procedure: ASTM A370
Test Date: 9/20/2021
Gauge Length: 2 Inches
Acceptance Criteria: Results Only

Specimen Description:

Geometry Type: Rectangular for Tension

Tensile Test Results

Sample ID	Width	Thickness	Area	Yield Load	Yield Strength	Tensile Load	Tensile Strength	Elongation After Fracture
1-Side Under Track	0.505 in	0.172 in	0.087 in^2	5564 lbf	63961 lbf/in^2	6544 lbf	75342 lbf/in^2	25.0 %
2 - Side	0.504 in	0.187 in	0.094 in^2	5604 lbf	59464 lbf/in^2	7479 lbf	79357 lbf/in^2	25.3 %
3 - Top Plate	0.508 in	0.390 in	0.198 in^2	10819 lbf	54610 lbf/in^2	15993 lbf	80726 lbf/in^2	33.8 %
4 - Upright	0.510 in	0.479 in	0.244 in^2	15060 lbf	61646 lbf/in^2	20741 lbf	84905 lbf/in^2	36.6 %

Erik Knudson
Materials Technician

Sample ID	Yield Strength
1-Side Under Track	63961 lbf/in^2
2 - Side	59464 lbf/in^2
3 - Top Plate	54610 lbf/in^2
4 - Upright	61646 lbf/in^2

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Steel Material Testing

Founded 1929 St. Louis Testing Laboratories INCORPORATED

2810 Clark Avenue • St. Louis, MO 63103-2574 • (314) 531-8080 • FAX (314) 531-8085
Chemical, Metallurgical, Mechanical, Nondestructive, Environmental Testing, Analyses and Field Service.

BRAUN INTERTEC CORP
11001 Hampshire Avenue S.
Minneapolis, MN 55438

Attention: Eric O'Donnell

September 20, 2021
Lab No. 21C-1300
Invoice No. INSTL12051
P.O No. B2105206
Page 1 of 1

REPORT OF ANALYSIS

MATERIAL: Sample 1, Sample 2, Sample 3, Sample 4
SUBJECT: Compositional Analysis
TEST METHOD: ASTM E415-17
UNITS: Percent by Weight (%)
METHOD DETECTION LIMIT: 0.01% for aluminum
RESULTS:

ANALYTE	Sample 1	Sample 2	Sample 3	Sample 4
Total Carbon	0.12	0.11	0.08	0.16
Silicon	0.22	0.44	0.52	0.26
Sulfur	0.020	0.031	0.018	0.026
Manganese	0.60	0.43	0.43	1.06
Phosphorus	0.052	0.047	0.051	0.009
Nickel	0.43	0.20	0.29	0.19
Chromium	0.18	0.88	1.01	0.53
Molybdenum	0.03	0.02	0.02	0.01
Copper	0.31	0.28	0.25	0.31
Vanadium	0.01	0.01	0.01	0.05
Aluminum	0.05	0.01	<0.01	0.02
Iron	Remainder	Remainder	Remainder	Remainder

The Alloys could not be identified.

Identification of tested specimens provided by the client

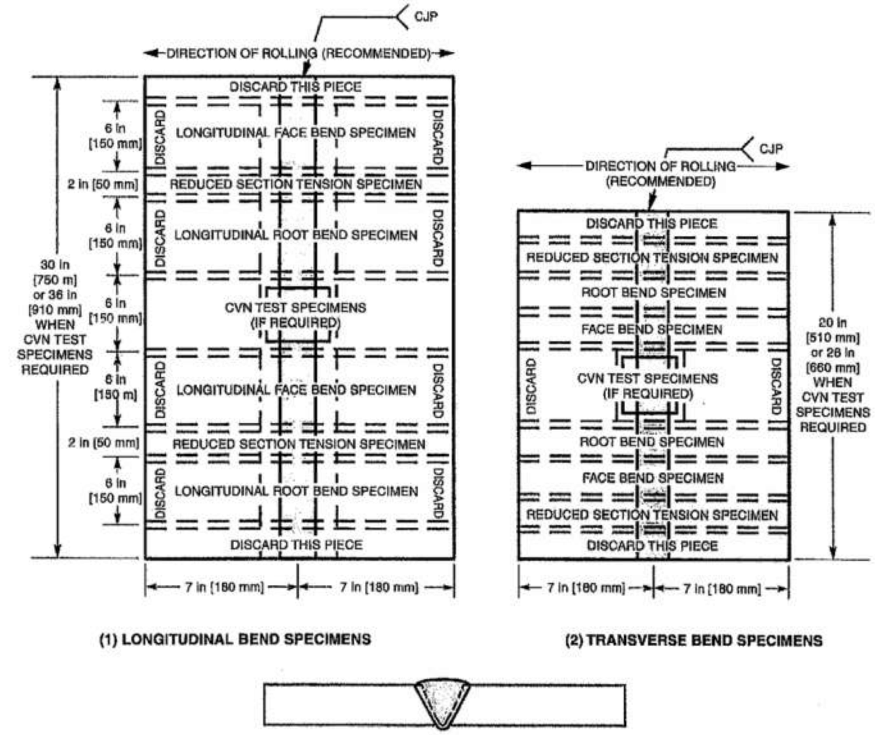
Jacob W. Long
Director Chemical and Environmental Testing

JWL/tz

ACCREDITED
CERTIFICATE # 0397.01
CERTIFICATE # 0397.02

AN OFFICIAL COPY OF TEST REPORT WILL BE PROVIDED BY THIS LABORATORY ON REQUEST. NOT OFFICIAL WITHOUT THE RAISED SEAL OF ST. LOUIS TESTING LABORATORIES, INC. SEE REVERSE FOR CONDITIONS.

MEMBER ACIL



Notes:
1. The groove configuration shown is for illustration only. The groove shape tested shall conform to the production groove shape that is being qualified.
2. When CVN tests are required, the specimens shall be removed from their locations, as shown in Figure 6.28.
3. All dimensions are minimum.
4. For 3/8 in [10 mm] plate, a side-bend test may be substituted for each of the required face- and root-bend tests. See Figure 6.8(2) for plate length and location of specimens.

Figure 6.7—Location of Test Specimens on Welded Test Plate 3/8 in [10 mm] Thick and Under—WPS Qualification (see 6.10)

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Foundation Testing

Zoo

Sep. 16, 1977

UNIVERSAL MOBILITY, INC. - MINNESOTA ZOOLOGICAL GARDENS - ZOORIDE FOUNDATIONS

Support Location Station	Type of Support	Top of Foundation Elevation	Total length of Column	Hole Information Bottom Elevation	Approx. Depth
0+00	Grade Beam	96.83	None	-----	----
0+26	Grade Beam	96.83	None		
0+52	Grade Beam	96.83	None		
1+22	84# column	98.29	34 feet	64.0	20 feet
1+92	" "	99.95	34 feet	65.5	20 feet
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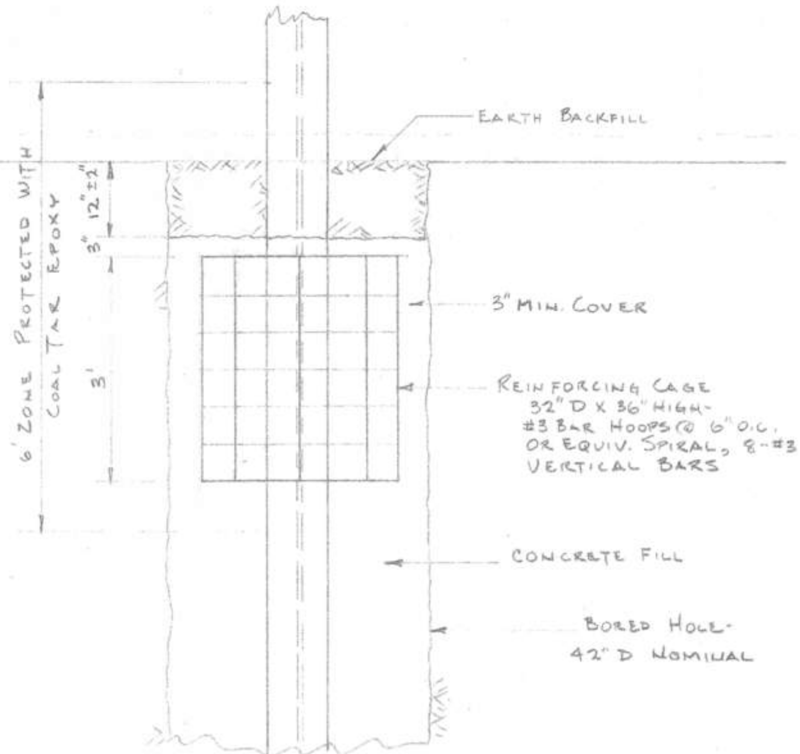
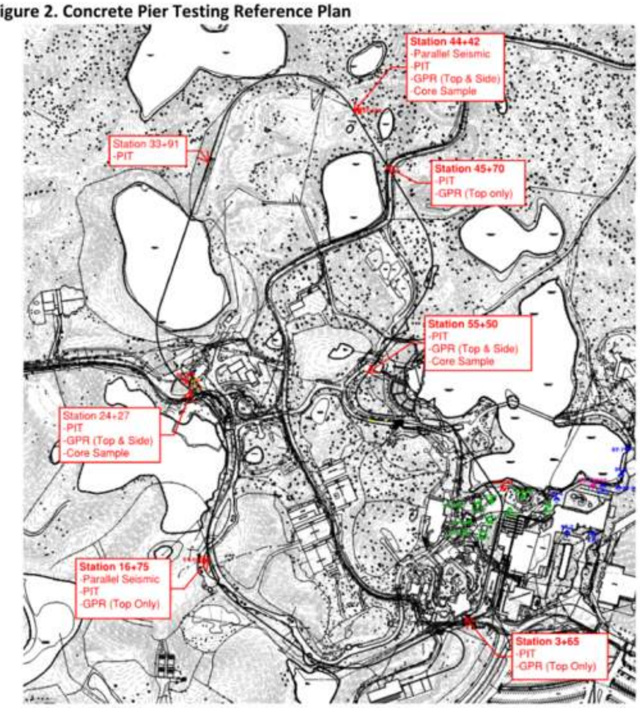
3+65	" "	04.03	31 feet	72.5	19 feet
4+10	" "	05.09	33 feet	72.0	20 feet
4+71.39	" "	05.33	34 feet	71.0	19 feet



Foundation Testing

Figure 2. Concrete Pier Testing Reference Plan



- Station 44+42
  - The concrete pier has a diameter of approximately 3 feet, 6 inches.
  - The steel column was observed to extend at least 3 feet into the pier.
  - The uneven surface of the concrete sides suggests that the pier was earth formed.
  - A concrete core was extracted for compressive strength testing. The core has a compressive strength of 4,710 psi.
  - GPR observations from the top and side indicate reinforcing consistent with a cylindrical style reinforcing cage.
    - Vertical bars are spaced at approximately 12 inches.
    - Horizontal bars are spaced at approximately 6 inches.
    - Concrete cover is approximately 6 inches.

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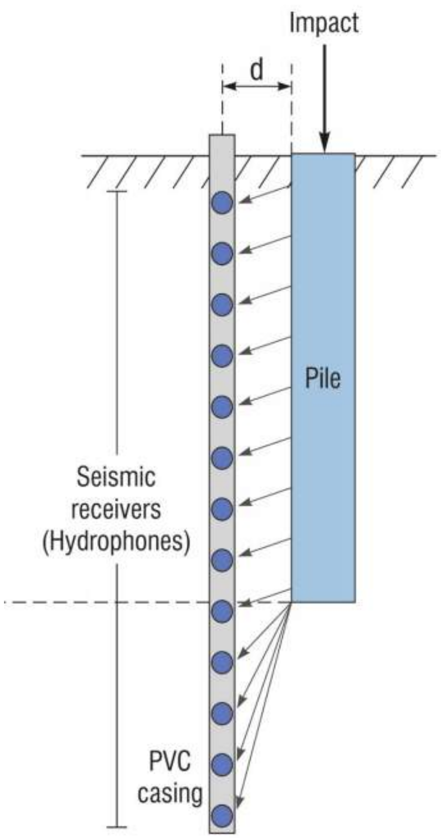
Foundation Testing



Credit:
Everest Geophysics

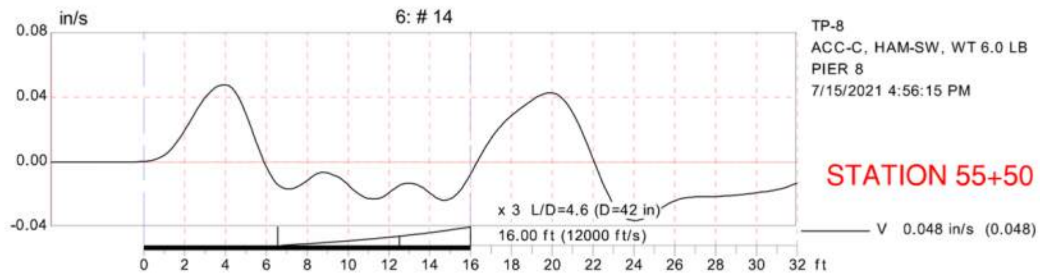


Table 1. PIT Data Results

Station Mark	As-Built Record Info	PIT Data Depth
44+42	21-feet	**20-feet**
45+70	19-feet	**17-feet**
55+50	19-feet	**16-feet**
3+65	19-feet	**19-feet**
16+75	21-feet	**21-feet**
24+27	21-feet	**20-feet**
33+91	~15 to 16-feet*	**16-feet**

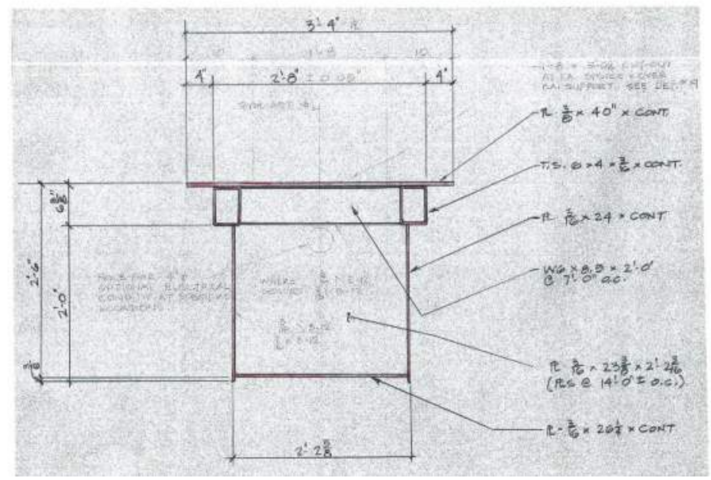
Existing Monorail Inspections

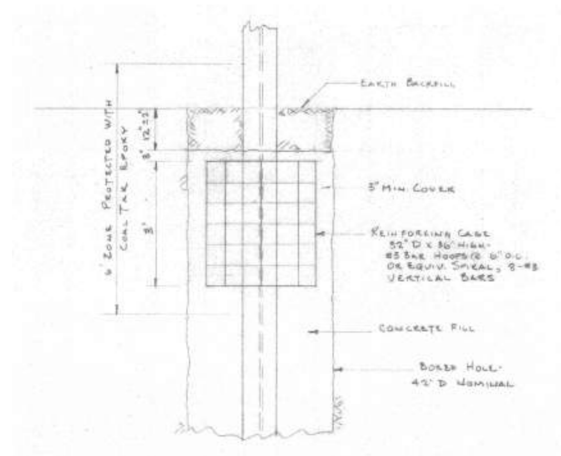
- Inspections performed at the start of construction to identify deficiencies with the existing structure requiring repair
  - 100% Visual Testing of welds
  - Ultrasonic testing of 20% top plate CJP welds, Mag particle testing of remainder.
  - Mag particle testing of 50% of side and bottom plate welds
  - Documentation of welding size and pattern
  - Measurement of column plumbness

SYSTEM SELECTION AND SHAPE FINDING

Existing Structural Condition

- 1.25-mile loop
- Typical monorail beam span: 70ft
- Typical column supports: W27
- Drilled piers with dropped columns
- 'Corten' / weathering steel
- No expansion joints: thermal expansion occurs at bends using slide bearings over columns





Existing Structural Condition

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Mindset Shift for Adaptive Reuse

Unchallenged requirements

New structure designed to suit requirements

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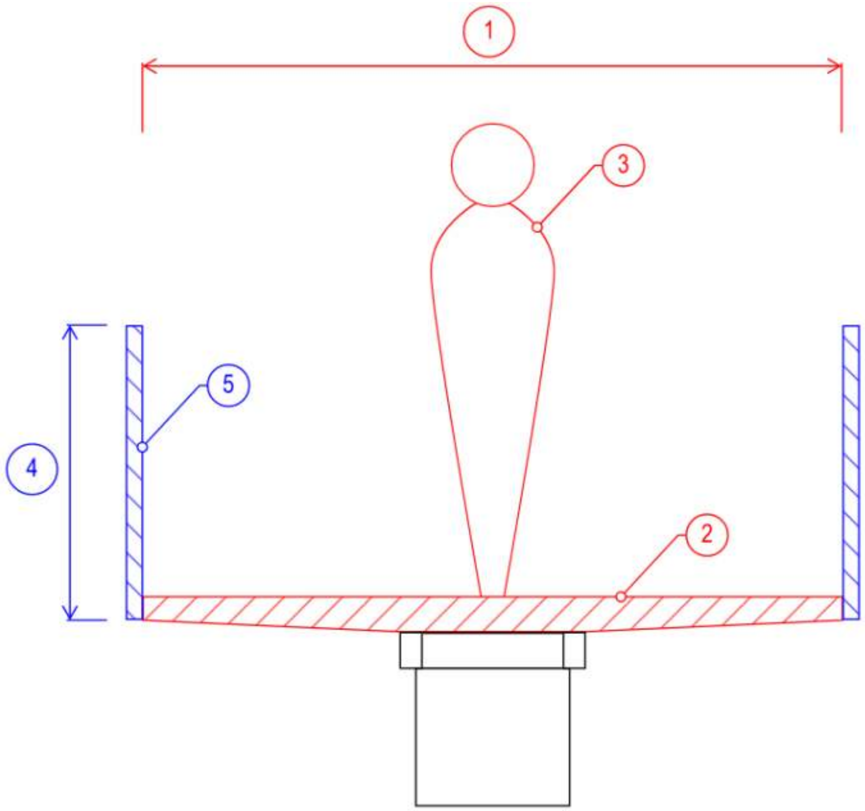
Mindset Shift for Adaptive Reuse

Unchallenged requirements → **New structure** designed to suit requirements

Existing structure → **Define requirements** based on capability of existing structure

Limited Number of Structural Variables



Gravity Design

1. Trail width
2. Weight of decking system
3. Live load requirement

Lateral Design

4. Guard rail height
5. Guard rail porosity

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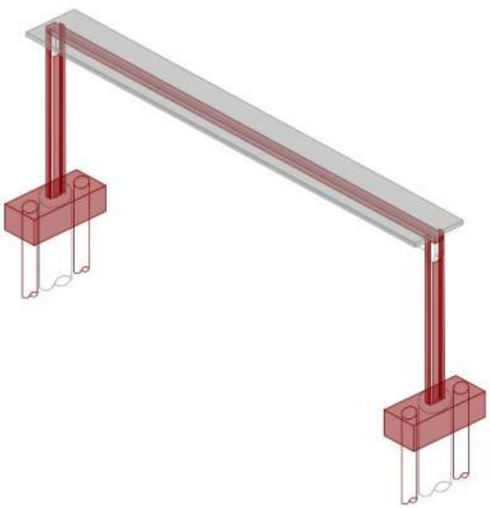
Original vs New Loading Comparison

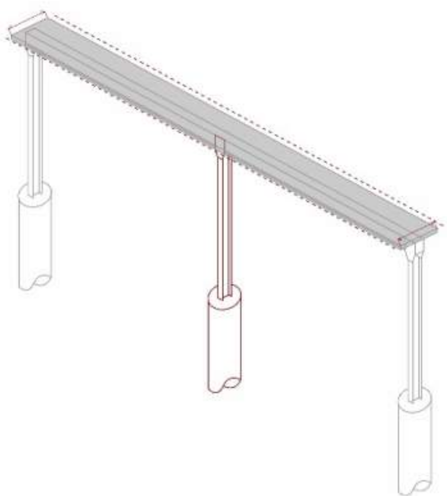
- Original structure designed for a uniform train load of **1,000 lbs/ft**
- New structure, conceptual loading:

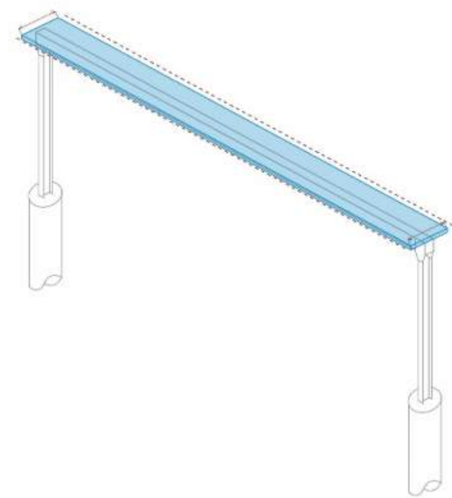
Decking system	Trail Width (ft)		
	8	10	12
'Heavy' Decking System Dead load= 65psf	**1,320**	**1,650**	**1,980**
'Light' Decking System Dead load = 25psf	**1,000**	**1,250**	**1,500**

Note: assuming 100psf live load

Three Conceptual Approaches







- 12ft wide concrete deck system
- Beam, column & foundation strengthening as required to meet demand

- Narrower concrete deck system with supplementary columns & foundations
- ‘Tune’ width dimension to reduce required strengthening of existing structures

- Narrower lightweight deck system
- ‘Tune’ width dimension to reduce required strengthening of existing structures

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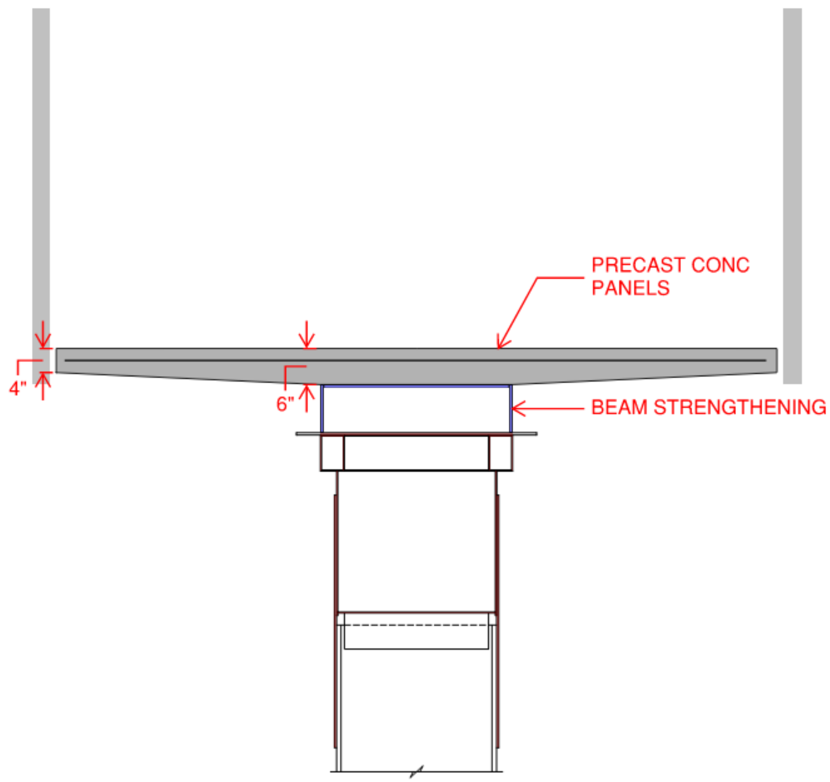
Precast Concrete



- Approx. weight: **65psf**
- Heaviest structural material meaning increased strengthening of existing structures
- Panels would be smaller vs FRP due to increased weight, meaning more regular joints
- Additional wearing surface requiring periodic maintenance / replacement
- Higher embodied carbon footprint vs structural steel framing

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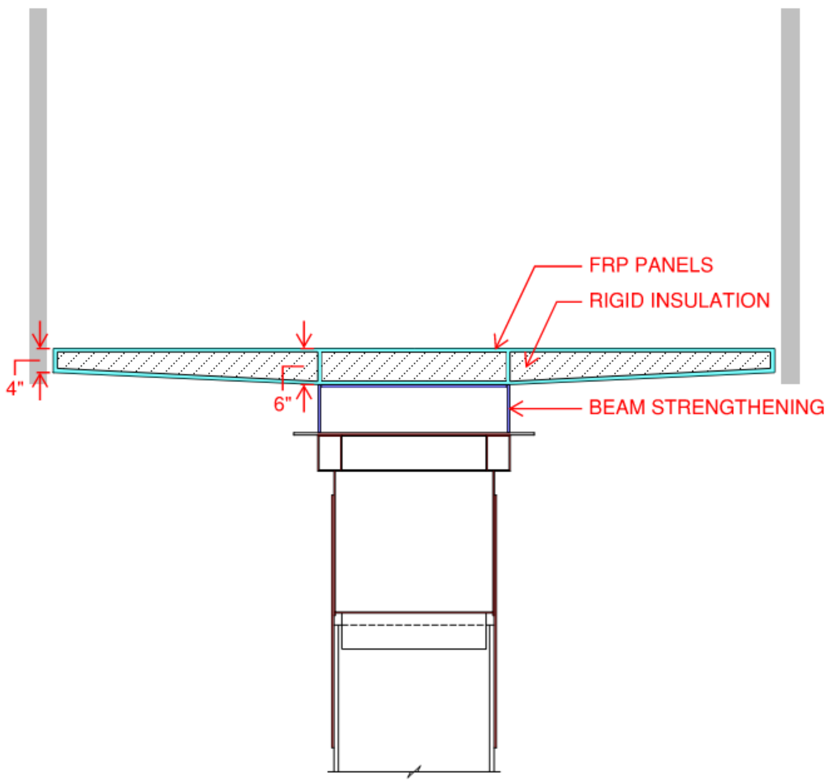
Fiber Reinforced Plastic (FRP)



- Approx. weight: **15psf**
- High strength, lightweight composite structural material
- Relatively common method of construction for bridge structures requiring lightweight decking
- Highly corrosion resistant
- Limited number of suppliers available
- Highest embodied carbon footprint
- Materials not typically recyclable at end of life

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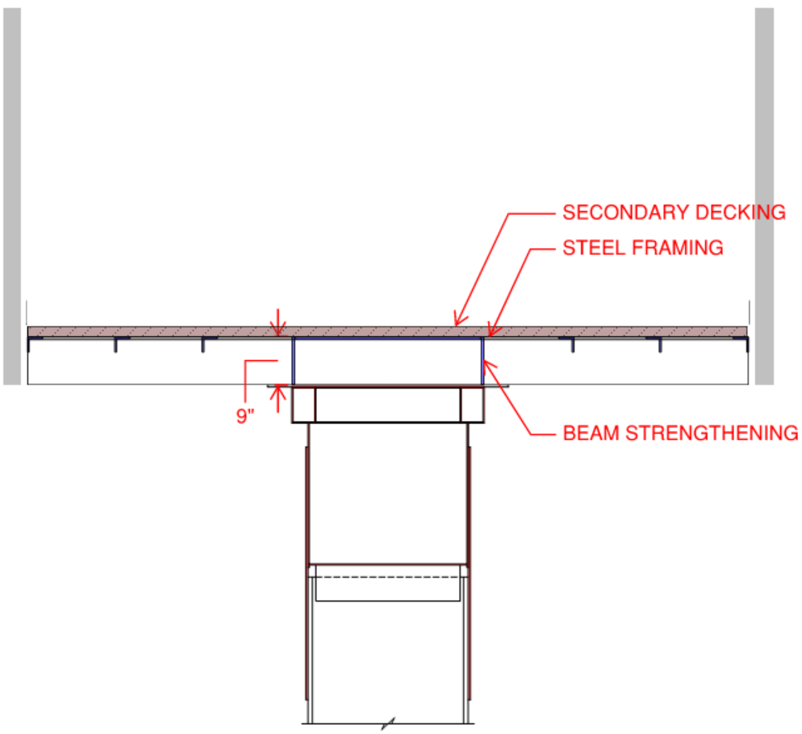
Steel Framing with Secondary Decking



- Approx. weight: **25psf**
- Lightweight structural material
- Steel framing to be weathering grade, compatible with existing corten structure
- Various options considered for secondary decking, including timber, recycled plastic and steel grating
- Decking could be easily replaced and different decking systems could be used at different locations on the TTT
- Lowest embodied carbon footprint vs FRP and precast concrete

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Tuning the Trail Width

Column capacity study

- 3 deck type options:
  - Precast concrete
  - Fiber reinforced plastic
  - Steel + wood decking
- 3 trail width options:
  - 8ft
  - 10ft
  - 12ft
- Column strength checks only (gravity + lateral)
- Results show the percentage of columns requiring reinforcement. Also indicative of expected foundation strengthening.

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Performance Considerations

- Self weight of structure: impact to existing foundations
- Speed of construction: labor cost
- Life cycle cost
- Impact on water management
- Recycled content
- Deconstruction and recyclability
- Carbon footprint
- Future maintenance
- Appearance

MO1

Structural System Options
Comparison Matrix

Description	FRP	Precast	Steel Structure w/ Decking*
Rating 1 - 3 with 3 being the best			
Maintenance	2	1	3
Life Cycle Cost	2	1	3
Water Management	1	1	3
Recycled Content	1	2	3
Recyclability / Deconstructability	1	2	3
Carbon Footprint	1	2	3
Replacement	1	1	3
Aesthetic Above	2	2	3
Aesthetic Below	3	3	2
Surface Joints	2	1	3
Wear surface Options / Flexibility	1	1	3
Impact to Existing Foundations	3	1	3
Overall Score	**20**	**18**	**35**

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Slide 58

FR0 Do we have a better quality version of this image?
Fraser Reid, 2023-10-12T19:34:15.962

MO1 New graphic installed over top of old one
Michael Osowski, 2023-10-20T12:56:10.867

LIVE LOAD REQUIREMENT

Live Load Requirement

- Live load on deck was significantly higher than self weight of structure, and original 1,000lb/ft design live load
- TTT did not fall under clear structural typology / occupancy under IBC / AASHTO; hence some engineering judgement was necessary to establish a recommended live load provision.
- Minimum code prescribed live loading for similar structures:
  - IBC (ASCE) Public assembly: **100psf** (feasibility study)
  - AASHTO Pedestrian bridges: **90psf**
  - IBC (ASCE) Walkways & elevated platforms: **60psf**

Slide 60

CH0 Didn't?

Craig Huhtala, 2024-01-23T02:33:10.396

Pedestrian Bridge Loading: IBC vs AASHTO

- IBC (ASCE) & AASHTO use different load factors for strength design
- Compare strength level live loading:
  - IBC (ASCE): 1.6x100 = **160psf**
  - AASHTO: 1.75x90 = **158psf**

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AASHTO Pedestrian Loading Illustration

50psf

100psf

150psf

Consider Reduced Occupancy for TTT

- **60psf** live load per IBC (ASCE) Walkways & Elevated Platforms
- Approx. **25-30% reduction** on strength level loading to existing beam & column structures, depending on deck system selection.
- Significant reduction in required strengthening of existing structures
- Compare 100psf vs 60psf for 70'x10' section of TTT:
  - **100psf** = 70,000lbs = approx. **400 people***
  - **60psf** = 42,000lbs = approx. **240 people***

*Using 175lbs/person

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Live Loads - Distributed

Final design live loading per IBC 2018 / ASCE 7-16:

Area	Occupancy	Live Load (psf)
Stairs and entry ramps	Stairs and exit ways	100
Assembly areas	Yards and terraces, pedestrian	100
Main walkway	Walkways and elevated platforms (other than exit ways)	60

Note: all live loads were considered unreducible

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Live Loads - Concentrated

Custom wheel loading criteria was developed for maintenance vehicle assuming John Deere Gator vehicle or similar:

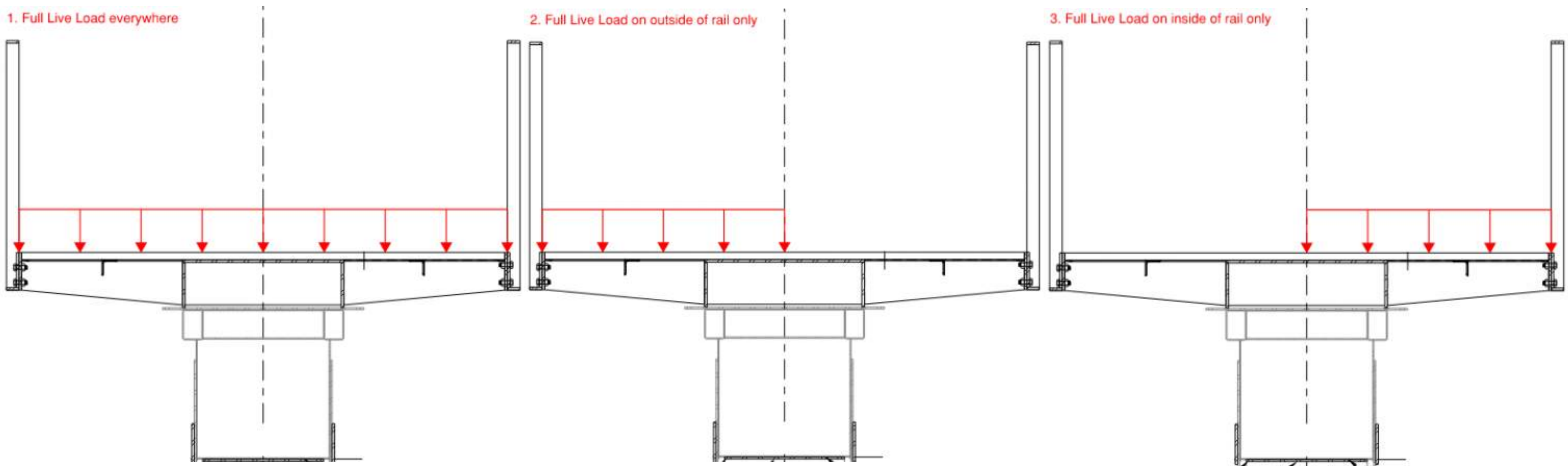
- Maximum vehicle weight = 2,000 lbs / Maximum payload weight = 1,000 lbs
- Total vehicle load = 3,000 lbs
- Wheel load = 1,000 lbs

Note: the TTT will be accessible to maintenance vehicles only, hence not subject to passenger vehicle load requirements

Live Load Patterning

- ASCE requires Live Load Patterning
- Complicated for a continuous beam
- LL effect on torsion also considered

4.3.3 Partial Loading. The full intensity of the appropriately reduced live load applied only to a portion of a structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member. Roof live loads shall be distributed as specified in Table 4.3-1.



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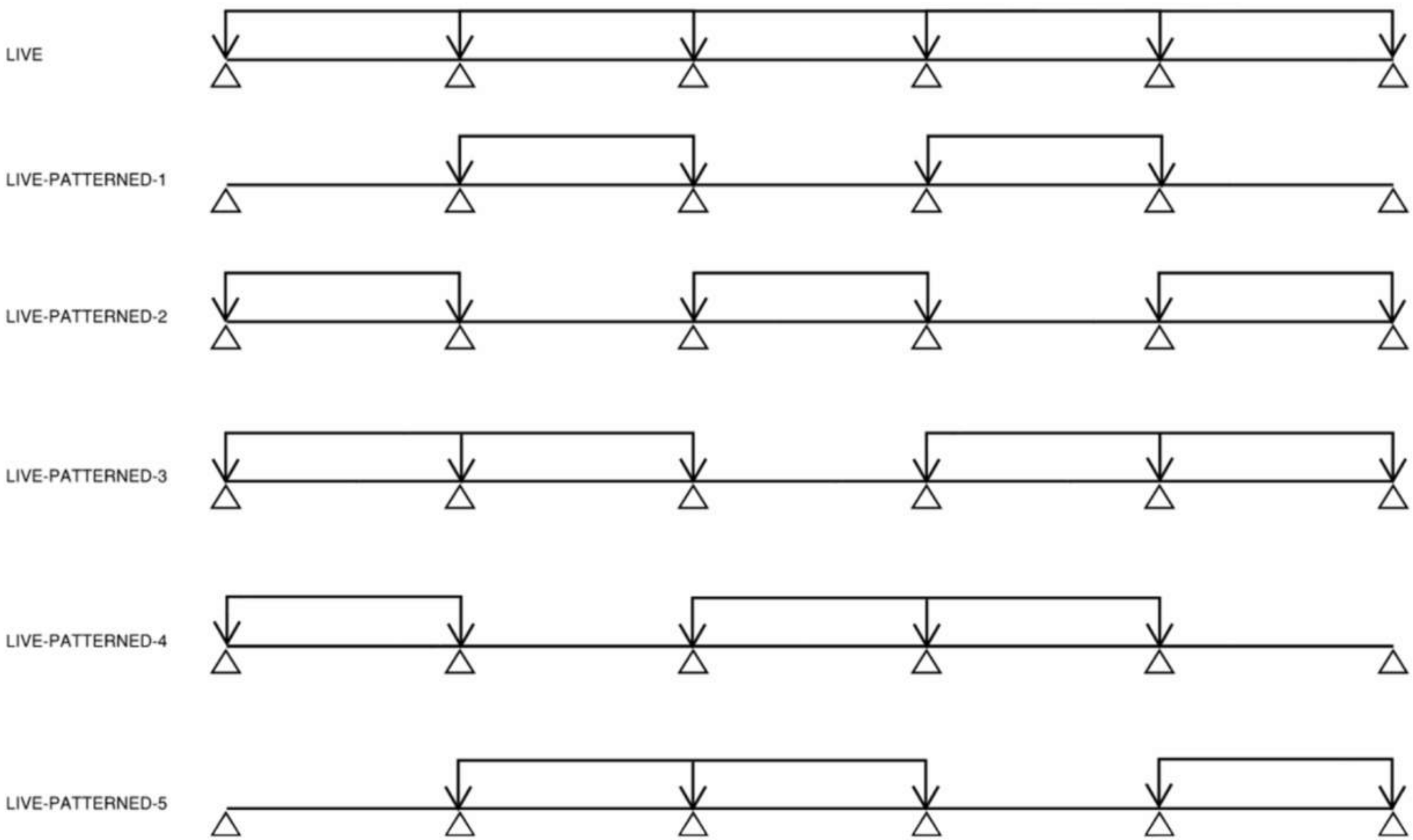
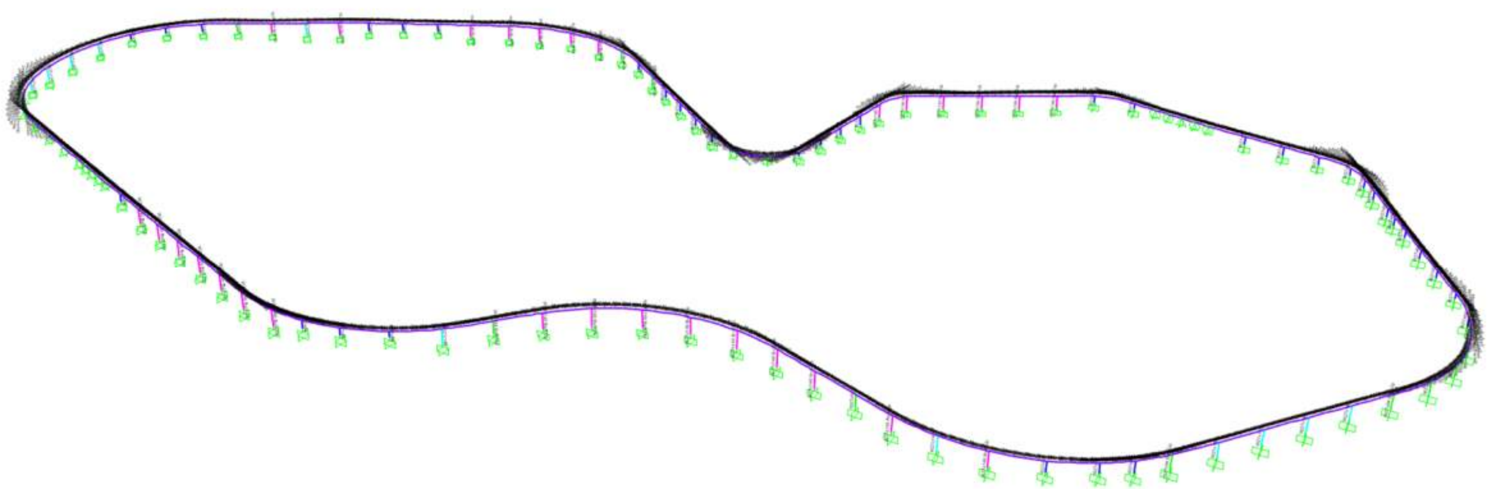
Live Load Patterning



SYSTEM BEHAVIOR

Structural Analysis – SAP2000



- Modeled initially for vibration analysis
- Ultimately used for full strength & service checks, due to complexity of 'system' behavior including influence of slide bearings on lateral loading & locked-in thermal forces.

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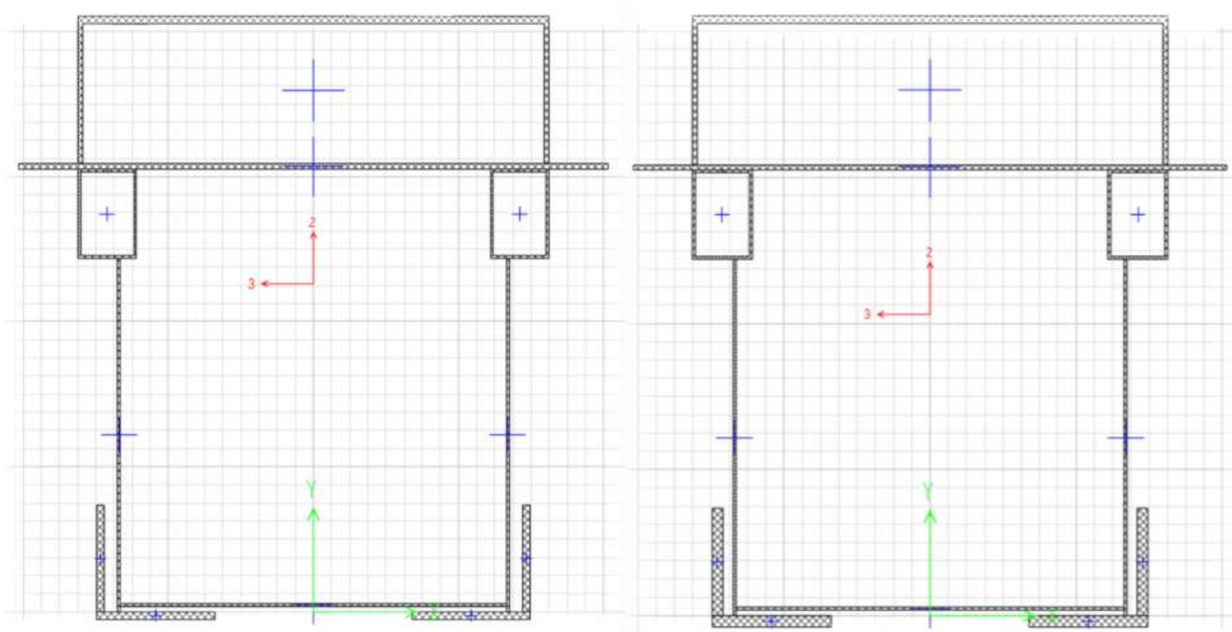
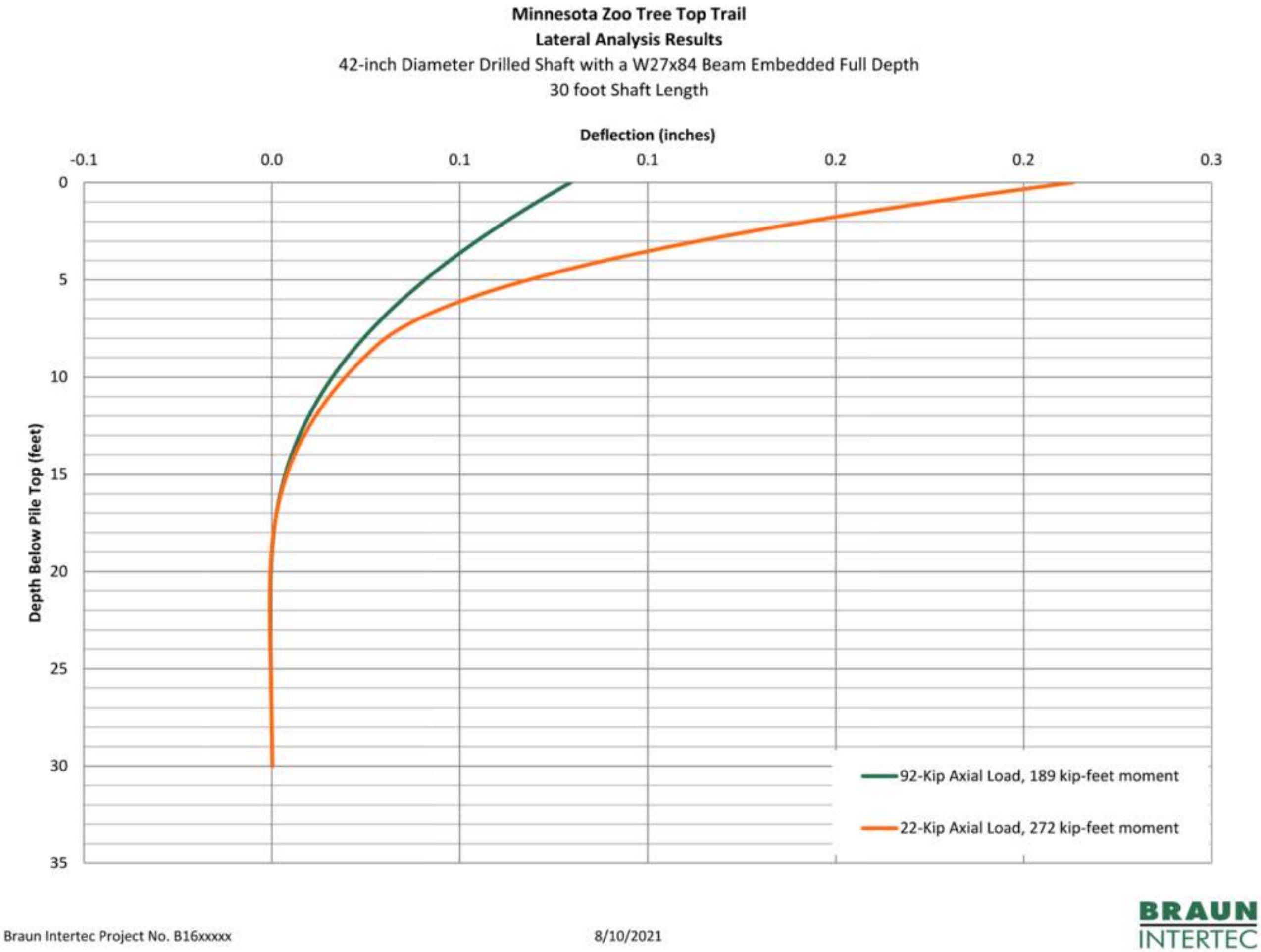
Structural Analysis Model

- Overall geometry of model
- Section types
- Boundary conditions
  - Foundations springs
  - One-way slide bearings
  - Two-way slide bearings
- Load application
- Load combinations



Structural Analysis

- Foundation springs used to soften model to better represent behavior
- LPile used for analysis
- Different pier depths and loading criteria for each column could require different spring for each column
- Sensitivity analysis justified use of a single set of average stiffness springs in all locations.



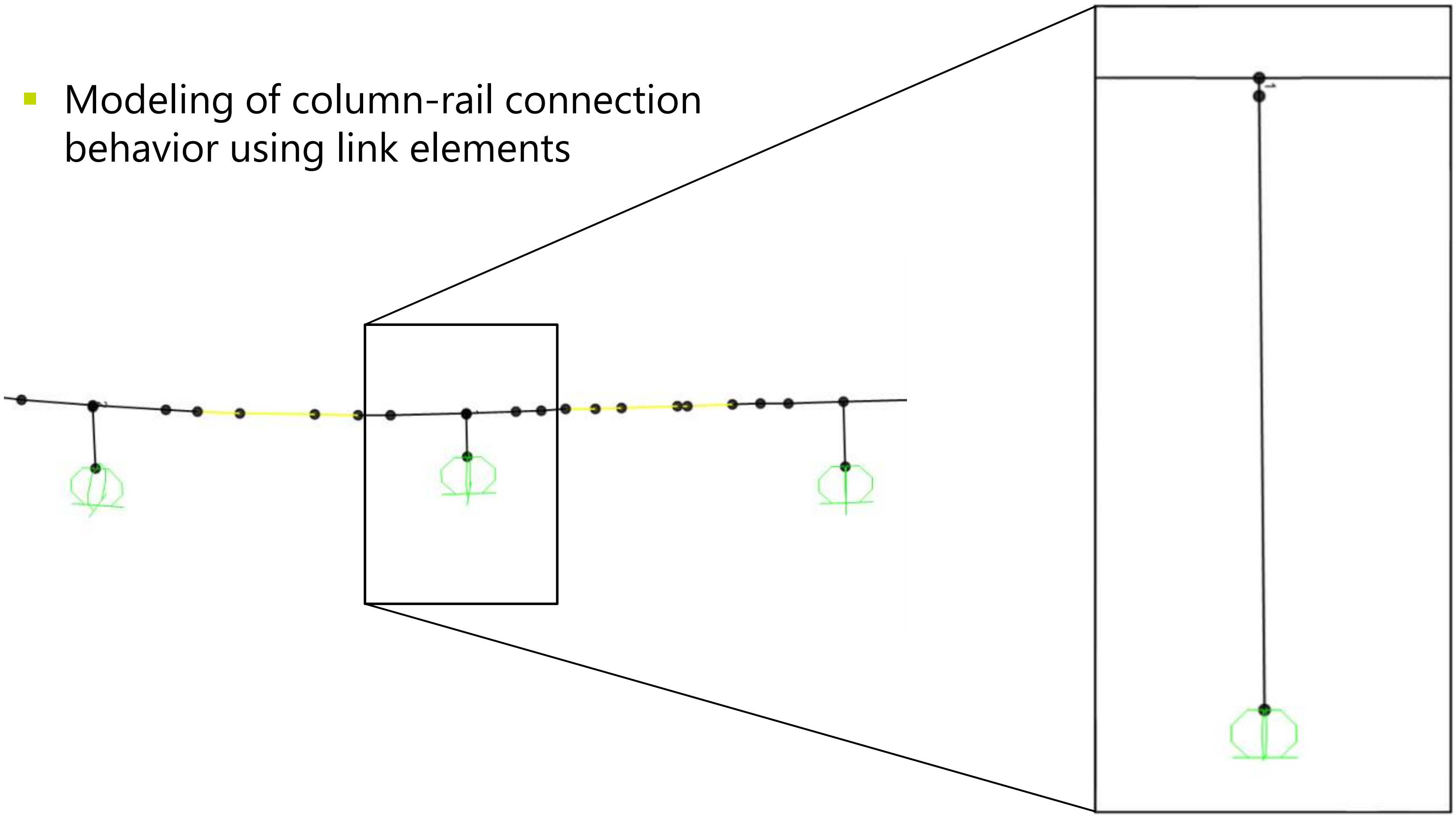
Structural Analysis

	Applied Loading		Top of Pile Movement		Stiffness	
Case	Shear (k)	Moment (k-ft)	Disp (in)	Rotation (rad)	K_Δ (k/in)	K_θ (k-ft/rad)
1	25	100	0.2329	0.00141	107	70922
2	25	425	0.4978	0.00419	50	101432
3	40	100	0.3864	0.00243	104	41152
4	40	700	0.9076	0.0076	44	92105
5	20	55	0.1749	0.00108	114	50926
6	20	130	0.2168	0.00143	92	90909
7	15	70	0.1426	0.000924	105	75758
8	15	150	0.1866	0.00129	80	116279
9	20	100	0.1999	0.00129	100	77519

- Translation Stiffness = 100 k/in
- Rotational Stiffness = 75,000 k-ft/rad

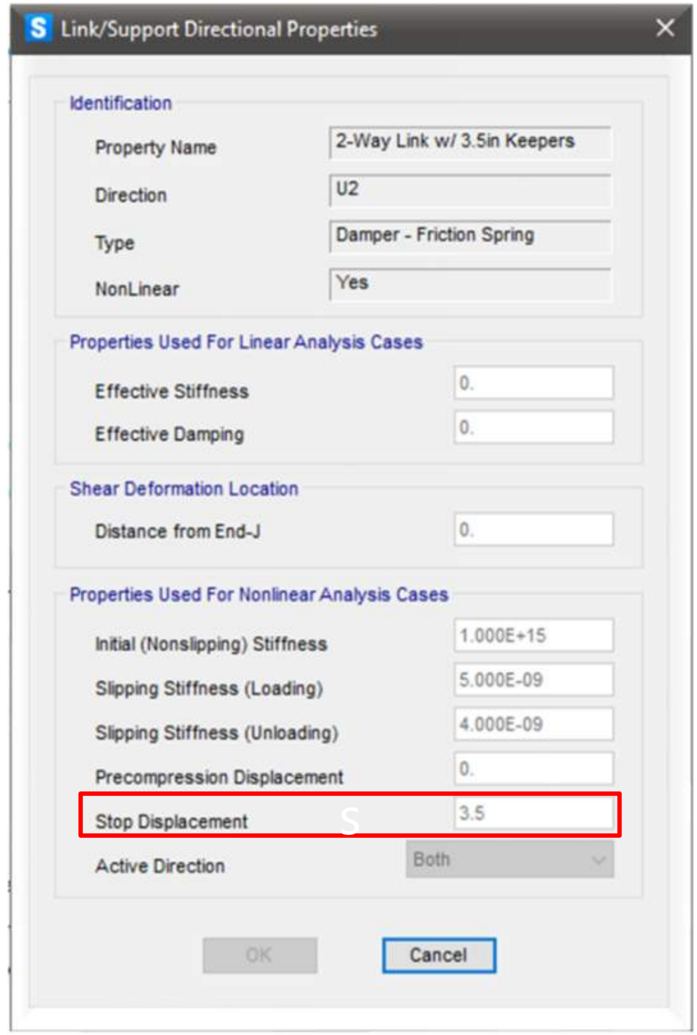
Structural Analysis

- Modeling of column-rail connection behavior using link elements



Structural Analysis

- Slide bearing connections required special consideration in the analysis model due to keeper plates
- Modeled as non-linear springs
  - Frictionless until engaging keepers, then column stiffness engaged



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Structural Analysis

- Load Combinations (more than 5!)
  - Service and Ultimate
  - Live load patterning
    - Inside/outside
    - Alternating spans
  - Temperature + and -
- 608 Load Combinations!

2.3 LOAD COMBINATIONS FOR STRENGTH DESIGN

2.3.1 Basic Combinations. Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations. Effects of one or more loads not acting shall be considered. Seismic load effects shall be combined loads in accordance with Section 2.3.6. Wind and seismic loads need not be considered to act simultaneously. Refer to Sections 1.4, 2.3.6, 12.4, and 12.14.3 for the specific definition of the earthquake load effect E. Each relevant strength limit state shall be investigated.

1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R)+(L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $0.9D + 1.0W$

Structural Analysis – Data Management

- 608 LCs, 915 Frames, 951 Points = Millions of data points
- Use of SAP2000 Table data to Excel
  - Frame forces
  - Displacements
  - Reactions
- Use of "Envelope" only is too conservative for section capacity checks
- Selection of like section types prior to export from SAP2000
  - 6 different selection sets

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Structural Analysis – Data Management

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	Model	Model 25.0													
2	Description:	Add description here...				Inputs									
3	Location saved:	0	Section	12', 0.5" Angles		Calculations									
4															
5	Capacities							Controlling LC	ULS.2.1.1-T -- 1.2D+1.6LPO+0.						
6	phi*Pnt	3650	[k]	Max DCR	0.74			P	45.446	[k]					
7	phi*Pnc	-1953	[k]	Excel RowNumber	5661			Vy	47.664	[k]					
8	phi*Vn-x	392	[k]	Controlling Frame	58			Vx	-14.979	[k]					
9	phi*Vn-y	1205	[k]					Tn	277.0273	[k-ft]					
10	phi*Tn	499	[k-ft]					My-y	352.095	[k-ft]					
11	phi*Mn-y	1420	[k-ft]					Mx-x	-195.0326	[k-ft]					
12	phi*Mn-x+	2681	[k-ft]												
13	phi*Mn-x-	-2560	[k-ft]		Axial	Vy	Vx	Tn	My-y	Mx-x					
14	TABLE: Element Forces - Frames			Max	134.74	87.15	21.01	277.03	492.91	1001.06					
15	Frame-Text	Station-ft	OutputCase-Text	CaseType-Text	P-Kip	V2-Kip	V3-Kip	T-Kip-ft	M2-Kip-ft	M3-Kip-ft	FrameElem-Text	ElemStation-ft	GroupName	RowNumber	DCR
16	3	0	ULS.1 -- 1.4D	Combination	1.115	8.03	-0.058	4.0417	0.0244	-3.5953	3-1	0		16	0.002
17	3	3.3857	ULS.1 -- 1.4D	Combination	1.149	11.34	-0.058	8.2983	0.2199	-36.3862	3-1	3.3857		17	0.015
18	3	6.7715	ULS.1 -- 1.4D	Combination	1.182	14.651	-0.058	12.5548	0.4154	-80.3863	3-1	6.7715		18	0.033
19	3	0	ULS.2.0.1 -- 1.2D+1.6L+0.5S	Combination	0.646	18.704	-0.026	3.9416	0.1116	-9.1026	3-1	0		19	0.004
20	3	3.3857	ULS.2.0.1 -- 1.2D+1.6L+0.5S	Combination	0.724	26.295	-0.026	7.5901	0.2004	-85.2806	3-1	3.3857		20	0.035
21	3	6.7715	ULS.2.0.1 -- 1.2D+1.6L+0.5S	Combination	0.801	33.886	-0.026	11.2385	0.2891	-187.1599	3-1	6.7715		21	0.076
22	3	0	ULS.2.1.1 -- 1.2D+1.6LPO+0.5S	Combination	6.512	14.611	-0.361	18.3065	-0.2947	-5.7716	3-1	0		22	0.007
23	3	3.3857	ULS.2.1.1 -- 1.2D+1.6LPO+0.5S	Combination	6.576	20.902	-0.361	32.356	0.9273	-65.8923	3-1	3.3857		23	0.035
24	3	6.7715	ULS.2.1.1 -- 1.2D+1.6LPO+0.5S	Combination	6.64	27.194	-0.361	46.4055	2.1493	-147.3126	3-1	6.7715		24	0.074
25	3	0	ULS.2.1.2 -- 1.2D+1.6LPI+0.5S	Combination	-2.487	14.53	0.149	-3.7853	0.3406	-2.3161	3-1	0		25	0.003
26	3	3.3857	ULS.2.1.2 -- 1.2D+1.6LPI+0.5S	Combination	-2.423	20.821	0.149	-2.737	-0.1655	-62.1596	3-1	3.3857		26	0.026
27	3	6.7715	ULS.2.1.2 -- 1.2D+1.6LPI+0.5S	Combination	-2.359	27.112	0.149	-1.6888	-0.6715	-143.3028	3-1	6.7715		27	0.058
28	3	0	ULS.2.2.1 -- 1.2D+1.6LP1+0.5S	Combination	0.538	17.977	-0.011	6.2946	0.159	40.3769	3-1	0		28	0.016
29	3	3.3857	ULS.2.2.1 -- 1.2D+1.6LP1+0.5S	Combination	0.616	25.569	-0.011	9.9431	0.196	-33.3408	3-1	3.3857		29	0.015
30	3	6.7715	ULS.2.2.1 -- 1.2D+1.6LP1+0.5S	Combination	0.693	33.16	-0.011	13.5915	0.2329	-132.7599	3-1	6.7715		30	0.055
31	3	0	ULS.2.2.2 -- 1.2D+1.6LP2+0.5S	Combination	1.008	9.731	-0.061	1.197	-0.0102	-53.6419	3-1	0		31	0.021
32	3	3.3857	ULS.2.2.2 -- 1.2D+1.6LP2+0.5S	Combination	1.046	13.422	-0.061	4.8455	0.195	-92.8369	3-1	3.3857		32	0.037
33	3	6.7715	ULS.2.2.2 -- 1.2D+1.6LP2+0.5S	Combination	1.084	17.113	-0.061	8.494	0.4002	-144.5282	3-1	6.7715		33	0.058
34	3	0	ULS.2.2.3 -- 1.2D+1.6LP3+0.5S	Combination	0.787	16.19	-0.037	5.7161	0.0476	35.7916	3-1	0		34	0.014
35	3	3.3857	ULS.2.2.3 -- 1.2D+1.6LP3+0.5S	Combination	0.864	23.781	-0.037	9.3646	0.1728	-31.875	3-1	3.3857		35	0.014
36	3	6.7715	ULS.2.2.3 -- 1.2D+1.6LP3+0.5S	Combination	0.942	31.372	-0.037	13.0131	0.298	-125.243	3-1	6.7715		36	0.052
37	3	0	ULS.2.2.4 -- 1.2D+1.6LP4+0.5S	Combination	0.473	20.817	-0.008166	3.7177	0.2218	-21.6886	3-1	0		37	0.009
38	3	3.3857	ULS.2.2.4 -- 1.2D+1.6LP4+0.5S	Combination	0.551	28.408	-0.008166	7.3661	0.2495	-105.019	3-1	3.3857		38	0.043

Section Capacities | 8', 0.5" Angles | 8', 0.75" Angles | 12', 0.5" Angles | 12', 0.75" Angles | 8S-R1-G | 8S-R3-G

Section capacities for each limit state

Controlling DCR

Controlling LC

Interaction check for each LC

Different data set for each section type

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THERMAL

Thermal Design

- Original 1.25-mile structure had no existing expansion joints.
- Thermal expansion of structure occurred by lateral deflection of columns and sliding of box-beam section over column and ground supports.
- In some conditions, historic movement was observed as being several inches in magnitude!
- The basic design approach was to replicate existing behavior as closely as possible.
- An alternate (traditional) approach with regularly occurring expansion joints would have required new column supports and lateral bracing which would have added significant scope and costs to the project.

Thermal Demands

EXPANSION JOINTS IN BUILDINGS

Technical Report No. 65

Prepared by the Standing Committee on Structural Engineering of the
Federal Construction Council
Building Research Advisory Board
Division of Engineering
National Research Council

NATIONAL ACADEMY OF SCIENCES
Washington, D.C. 1974

Station	Temperature (°F)		
	T_w	T_m	T_c
Minnesota			
Duluth	85	55	-19
International Falls	86	57	-29
Minneapolis/St. Paul	92	62	-14
Rochester	90	60	-17
St. Cloud	90	60	-20

Data from 1972 ASHRAE Handbook of Fundamentals

- 2021 ASHRAE Handbook of Fundamentals
  - T_C = -8 °F
  - T_W = 89 °F

Thermal Demands

Load combinations including thermal effects:

A. No temperature loading (basic load combinations)
1. 1.4D
2. 1.2D + 1.6L + 0.5S
3. 1.2D + 1.6S + 1.0L
4. 1.2D + 1.0W +1.0L + 0.5S
5. 0.9D + 1.0W

A. Full (100%) live load / Max temperature load: T = +55deg
1. 1.4D + **1.0T**
2. 1.2D + 1.6L + 0.5S + **1.0T**
3. 1.2D + 1.6S + 0.5L + **1.0T**
4. 1.2D + 1.0W + 0.5L + 0.5S + **1.0T**
5. 0.9D + 1.0W + **1.0T**

A. Full (100%) live load / Min temperature load: T = -55deg
1. 1.4D + **1.0T**
2. 1.2D + 1.6L + 0.5S + **1.0T**
3. 1.2D + 1.6S + 1.0L + **1.0T**
4. 1.2D + 1.0W +1.0L + 0.5S + **1.0T**
5. 0.9D + 1.0W + 1.0T

2.3.4 Load Combinations Including Self-Straining Forces and Effects. Where the structural effects of *T* are expected to adversely affect structural safety or performance, *T* shall be considered in combination with other loads. The load factor on *T* shall be established considering the uncertainty associated with the likely magnitude of the structural forces and effects, the probability that the maximum effect of *T* will occur simultaneously with other applied loadings, and the potential adverse consequences if the effect of *T* is greater than assumed. The load factor on *T* shall not have a value less than 1.0.

Thermal Movement

- Straight line expansion/contraction

  $L_0 \approx 6600'$
  $\alpha = 0.0000065$ in/in*°F
  $\Delta_t = 55°F$
  $dL_0 = 6600' \times 55° \times 0.0000065 = 2.36'$

- Radial expansion/contraction

  Circumference $\approx 6600'$
  $D_0 \approx 2100.8'$
  $\alpha = 0.0000065$ in/in*°F
  $\Delta_t = 55°F$
  $dD_0 = 6600' \times 55° \times 0.0000065 / \pi = 0.75'$

- Actual trail has both straight line and radial expansion behavior

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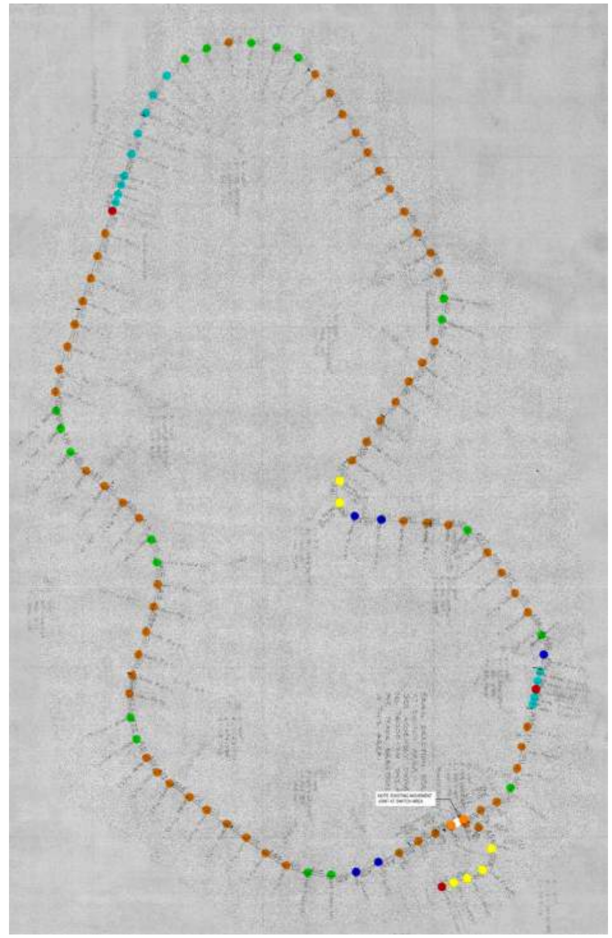
Thermal Design

Existing Structure Support Conditions

TRAIL SUPPORTS - KEY PLAN

SYMBOL	DESCRIPTION	FIXITY ASSUMPTIONS(*)					
		U1	U2	U3	R1	R2	R3
COLUMN SUPPORTS							
	FIXED	X	X	X	X	X	X
	ONE-WAY	X		X			
	TWO-WAY			X			
GRADE SUPPORTS							
	ANCHOR	X	X	X	X	X	X
	ONE-WAY	X		X		X	
	TWO-WAY			X			
	SPECIAL	X		X	X	X	X

(*) SIGN CONVENTION NOTES:
U INDICATES TRANSLATIONAL FIXITY
R INDICATES ROTATIONAL FIXITY
DIRECTION 1 = AXIS PERPENDICULAR TO BEAM
DIRECTION 2 = AXIS PARALLEL TO BEAM
DIRECTION 3 = VERTICAL AXIS
"X" INDICATES FIXITY IN THE APPLICABLE DIRECTION



Support Conditions

- **One-way column slider**

- **Two-way column slider**

- **One-way ground slider**

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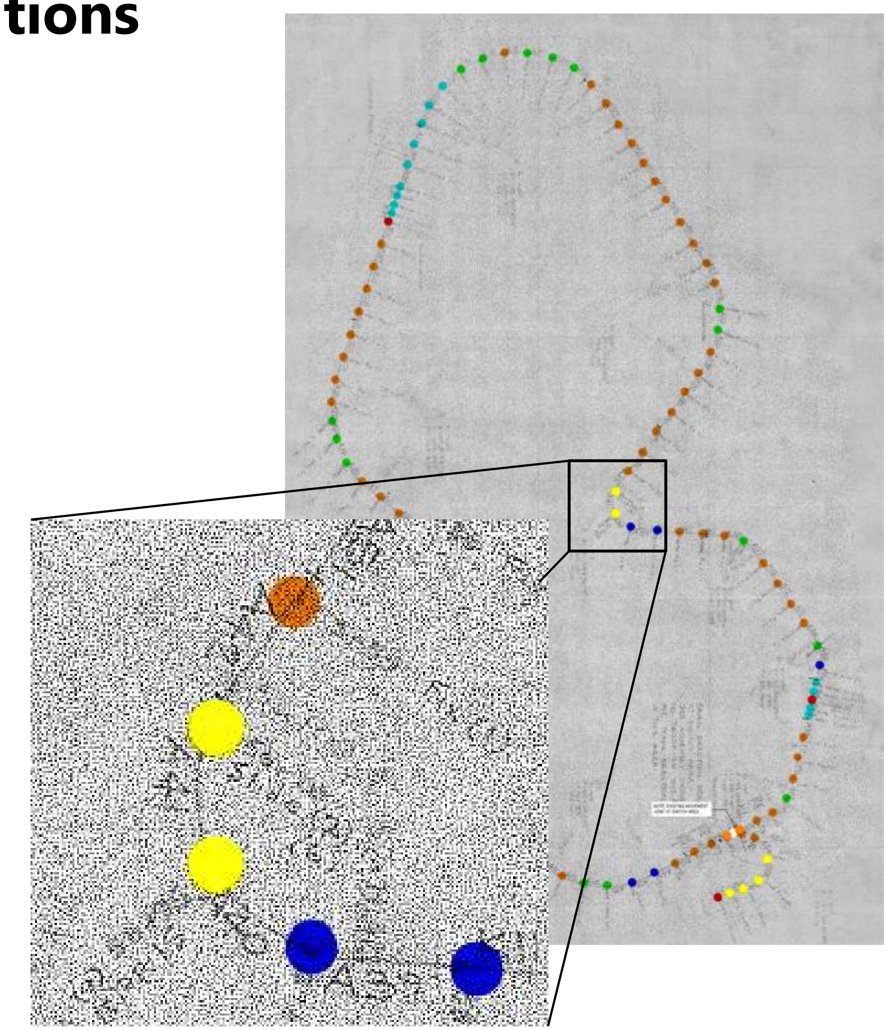
Existing Structure Support Conditions

One-way ground slider

SYMBOL	DESCRIPTION	FIXITY ASSUMPTIONS(*)					
		U1	U2	U3	R1	R2	R3
●	TWO-WAY			X			



Thermal Design & Modeling

- ASCE 7 requires temperature loading to be applied (1.0 factor) in combination with all strength design combinations
- Due to nature of existing condition, 'locked-in' thermal forces were significant
  - The addition of supports / restraints (e.g. to resist torsion) was found to exacerbate these effects
  - A 'brute force' approach to thermal design is typically a losing battle, since increased stiffness will lead to increased thermal loading!
- Thermal modeling considerations:
  - Temperature range
  - Foundation stiffness
  - Simplified section parameters to minimize design iterations

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Thermal Design Approach

- Ultimately, the design approach used a combination of:
  - ‘Fine tuning’ the existing condition
  - Replacement of bearing elements at sliding connections
  - Localized strengthening where required

WIND/LATERAL

Wind Loading

- Determination of appropriate wind loads required substantial engineering judgement.
  - Not a building
  - Not a bridge
- Continuous guardrail represented the possibility of significantly higher lateral wind loading on the structure than would have occurred during monorail operation
- Develop confidence by referencing multiple sources

Wind Loading – ASCE 7-16

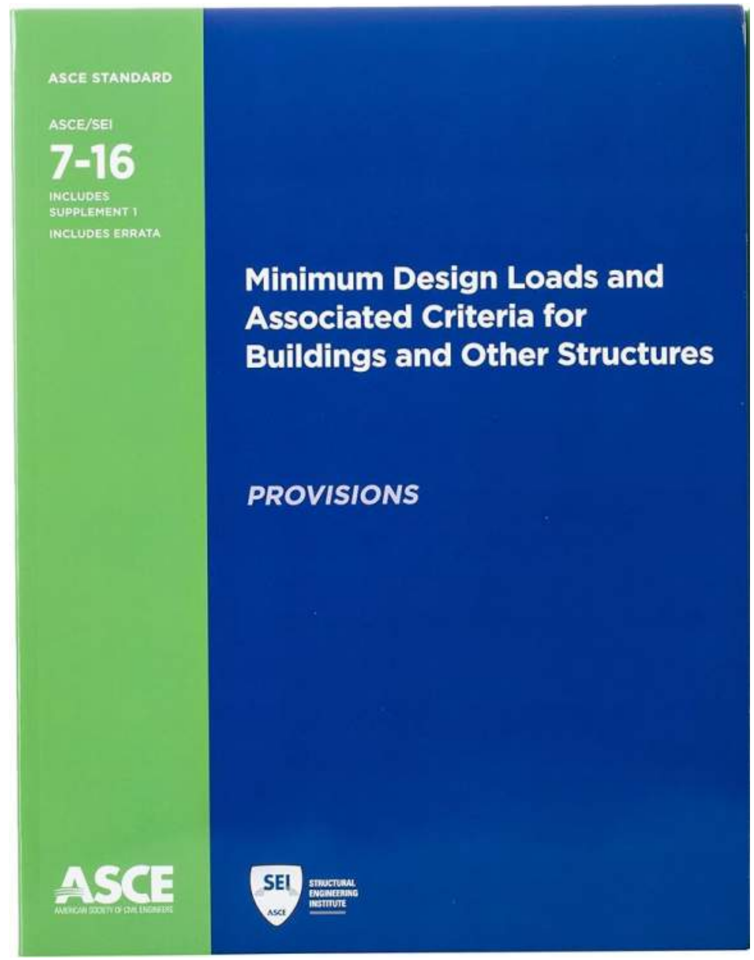
- ASCE 7-16
  - 109 mph wind speed (700yr MRI)
  - Exposure category C

26.10.2 Velocity Pressure. Velocity pressure, q_z, evaluated at height z above ground shall be calculated by the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d K_e V^2 \text{ (lb/ft}^2\text{); } V \text{ in mi/h} \qquad (26.10\text{-}1)$$

$q_z = 0.00256(1.0)(1)(.85)(1)(109\text{mph})^2 = 25.9$ psf

- Box beam and trail structure is like a solid sign.
- Guardrail is like an open sign or frame.



Wind Loading – ASCE 7-16

- Trail Structure

29.3.1 Solid Freestanding Walls and Solid Freestanding Signs. The design wind force for solid freestanding walls and solid freestanding signs shall be determined by the following formula:

$$F = q_h G C_f A_s \text{ (lb)} \qquad (29.3\text{-}1)$$

C_f = 1.95 based on applied parameters (maximum value in specification)

F = 25.9psf(0.85)*1.92*As
= 42.3 psf * As
= 42.3 psf * 3.5 ft = 148 plf

<u>Wind Load on Trail Superstructure:</u>

WL = 148 plf + 42 plf = 190 plf

- Railing

29.4 DESIGN WIND LOADS: OTHER STRUCTURES

The design wind force for other structures (chimneys, tanks, open signs, single-plane open frames, and trussed towers), whether ground or roof mounted, shall be determined by the following equation:

$$F = q_z G C_f A_f \text{ (lb)} \qquad (29.4\text{-}1)$$

Force Coefficients, C_f

		Rounded Members	
ε	Flat-Sided Members	$D\sqrt{q_z} \le 2.5$ ($D\sqrt{q_z} \le 5.3$) s.i	$D\sqrt{q_z} > 2.5$ ($D\sqrt{q_z} > 5.3$) s.i
<0.1	2.0	1.2	0.8
0.1 to 0.29	1.8	1.3	0.9
0.3 to 0.7	1.6	1.5	1.1

C_f = 1.8 for flat sided members with 15% solidity

F = 25.9psf(0.85)*1.8*Af = 25.9psf(0.85)(1.8)(0.15)As
= 5.95 psf * As
= 5.95 psf * 3.5 ft * 2 = 42 plf

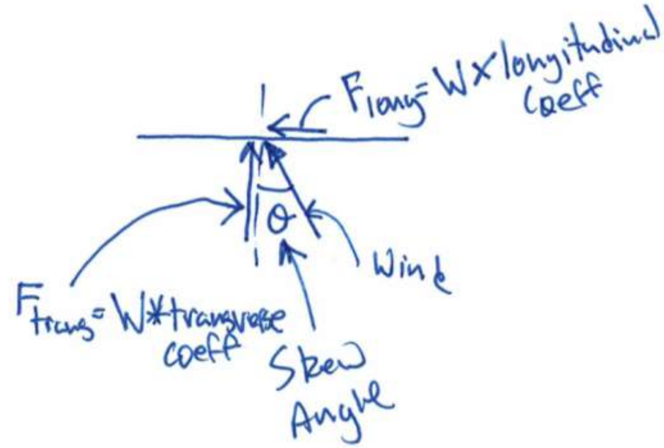
Wind Loading - AASHTO

- AASHTO LRFD Bridge Specification
  - 115 mph wind speed (700 yr MRI)
  - Exposure Category C
  - 100% Transverse, 25% longitudinal
  - 177 plf transverse, 44 plf longitudinal
- AASHTO LRFD Sign Specification
  - 115 mph wind speed (700 yr MRI)
  - Exposure Category C
  - 254 plf transverse

Use ASCE 7 wind load as it appears reasonable and is bounded by AASHTO values

Wind Loading

- Freeform shape of treetop trail makes defining primary wind directions impossible.
- Given any wind direction, it will intersect different portions of the trail at different angles.
- Utilize AASHTO Skew Angle Coefficients to account for varying angle of application.

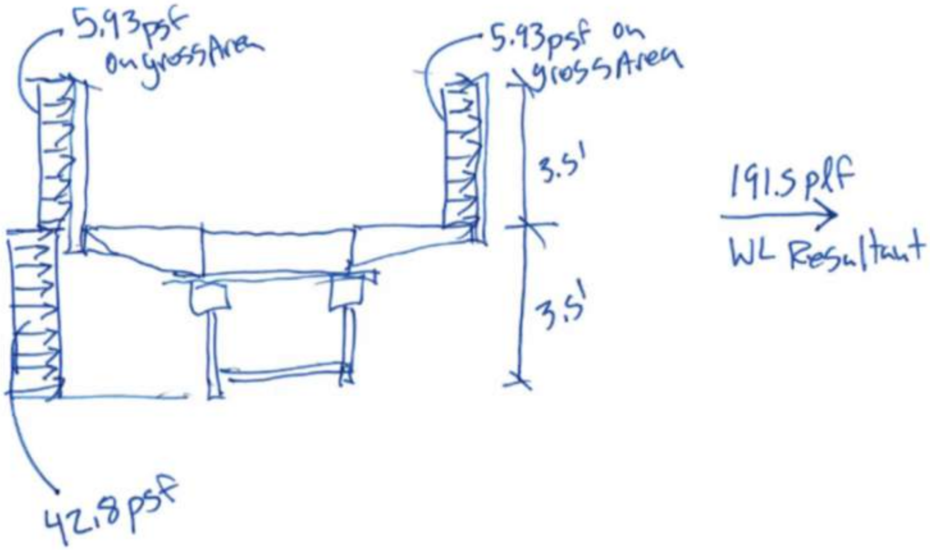


Skew Angle	Transverse Coefficient	Longitudinal Coefficient
0	1.00	0
15	0.88	0.12
30	0.82	0.24
45	0.66	0.32
60	0.34	0.38

Wind Loading

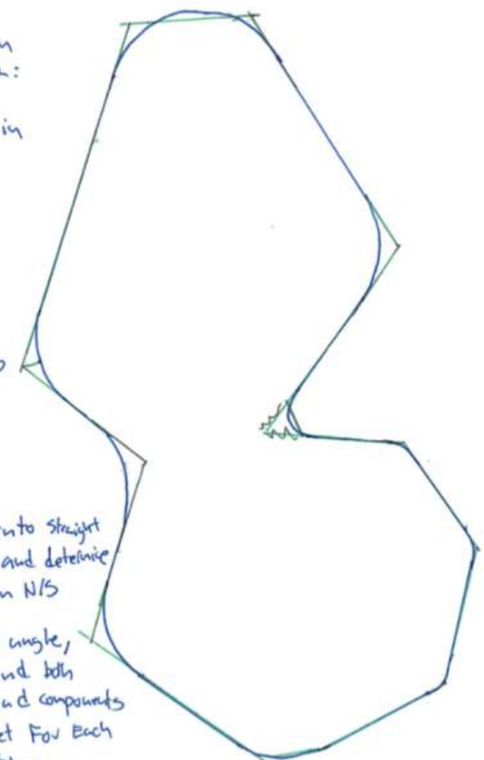
MBJ

TTT
20.658.0
WACKER
3/2/22

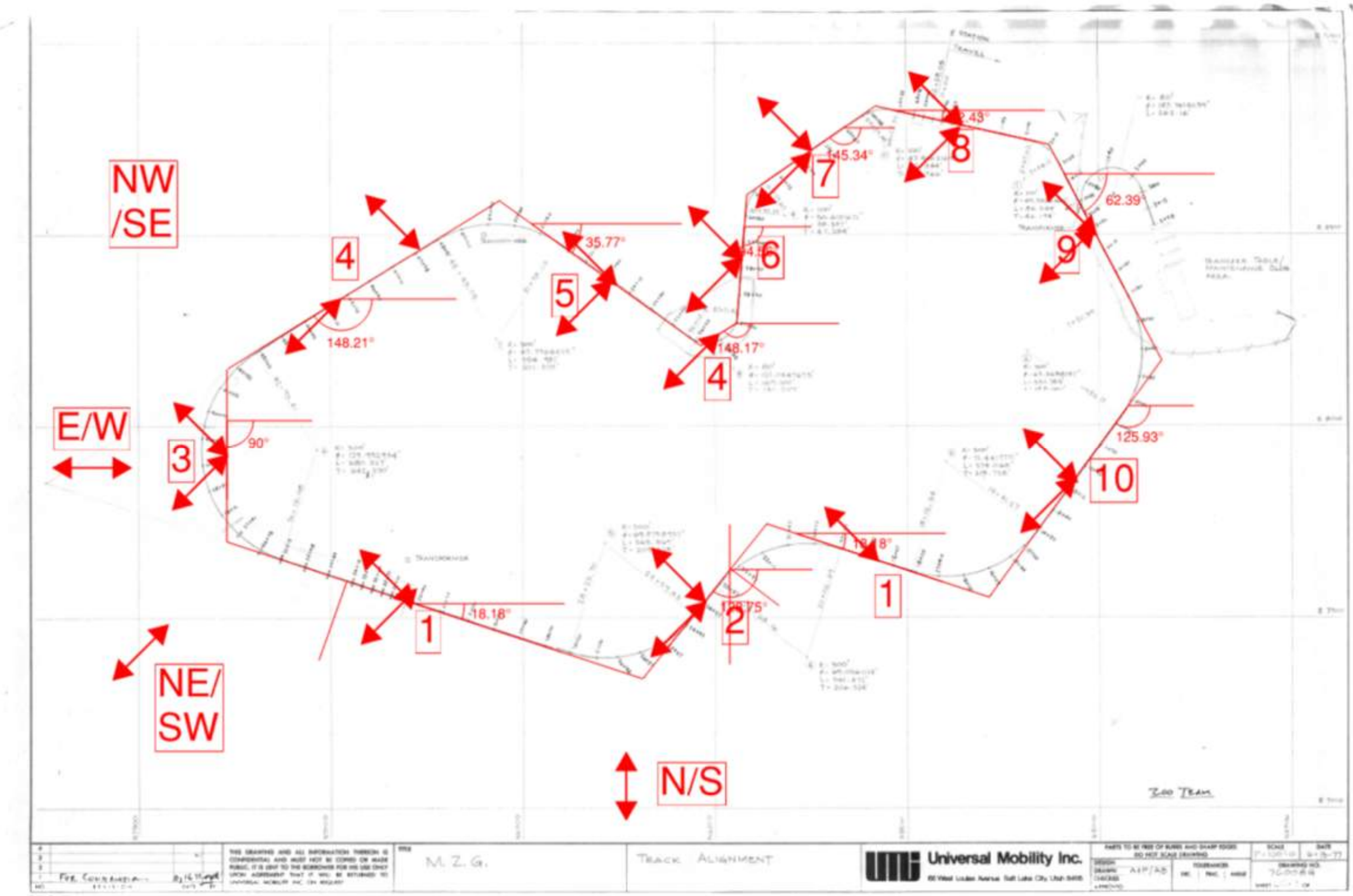


Some thoughts on implementation:

1. Consider Wind in 8 directions
2. ~~Idealize~~ this: Reduces to 4 directions with a ± in Load combos
3. Idealize trail into straight line segments and determine inclination From N/S
4. Determine skew angle, coefficients and both long & trans load components in spreadsheet For Each wind direction
5. Input line loads in SAP.

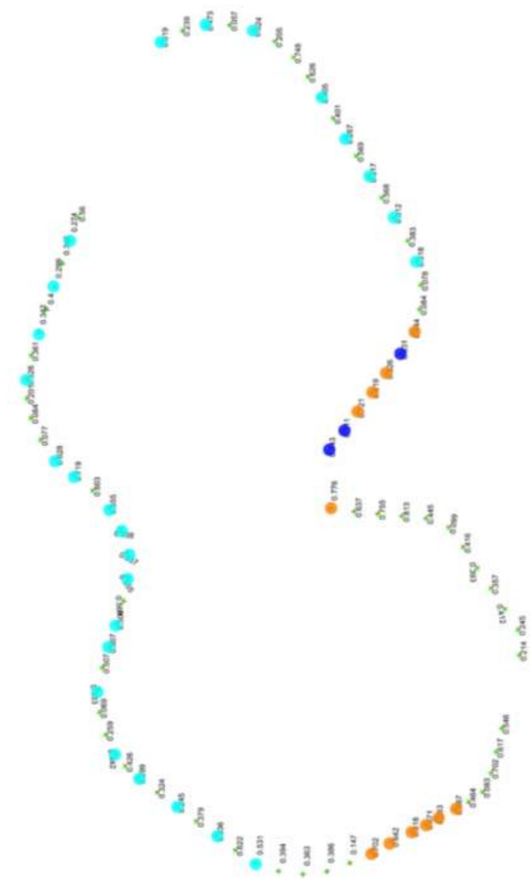


Wind Loading

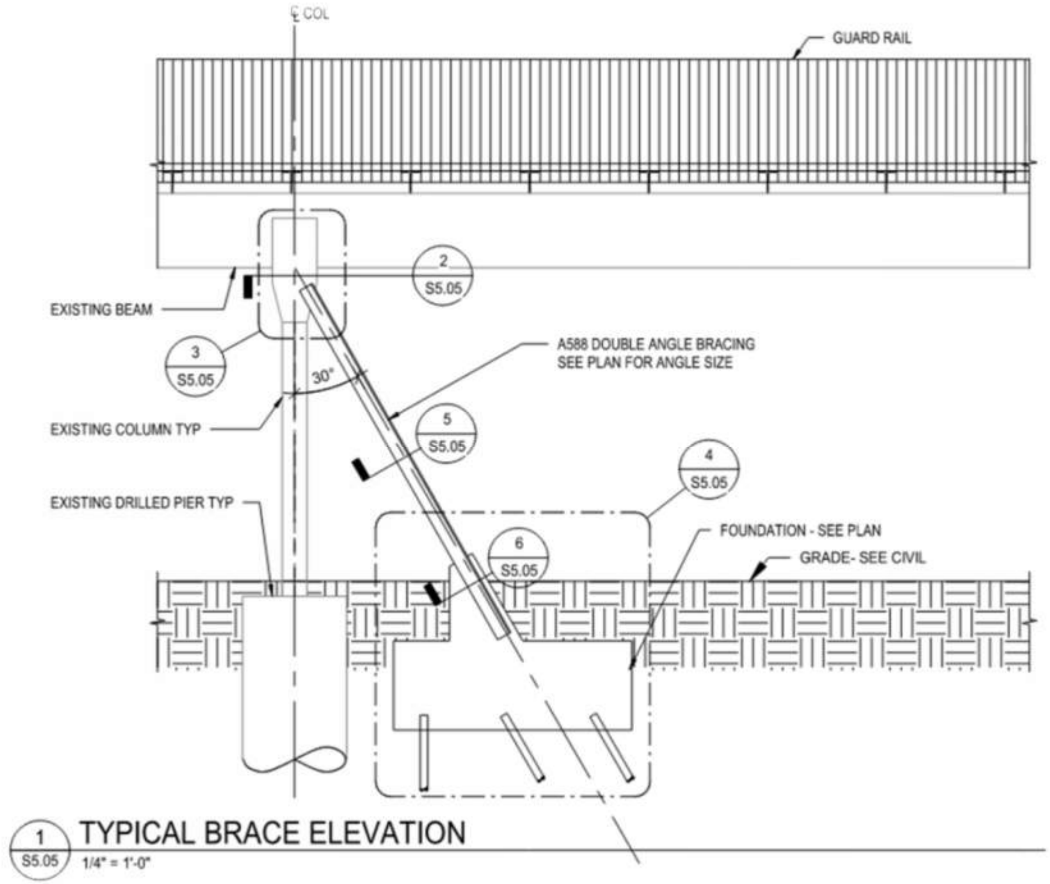


Column Reinforcement
First Pass – No Lateral Bracing

- 0.25" Strengthening Required (11)
- 0.50" Strengthening Required (3)
- 0.75" Strengthening Required (26)
- Add'l Strengthening Required (0)



Vertical Bracing



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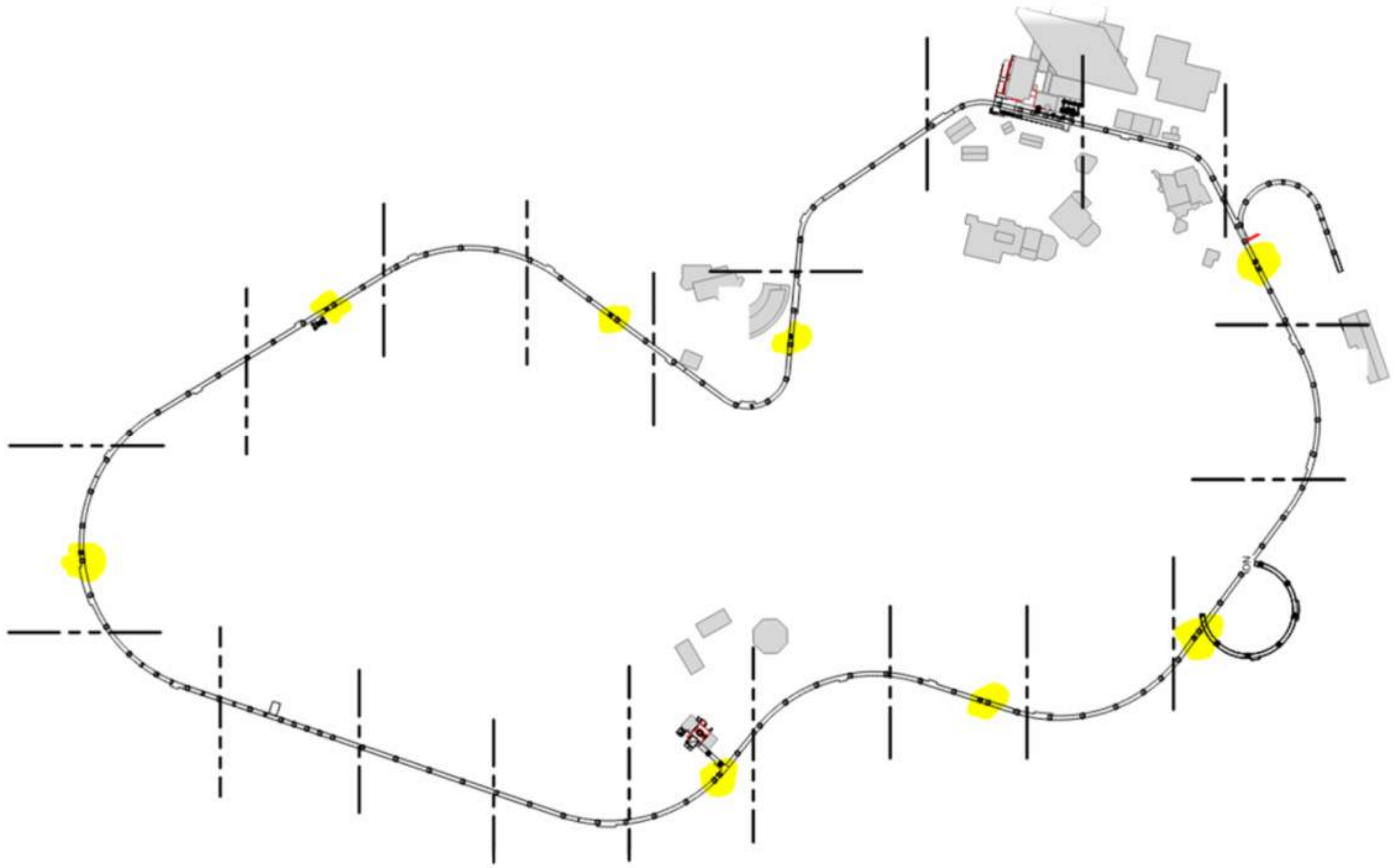
Vertical Bracing



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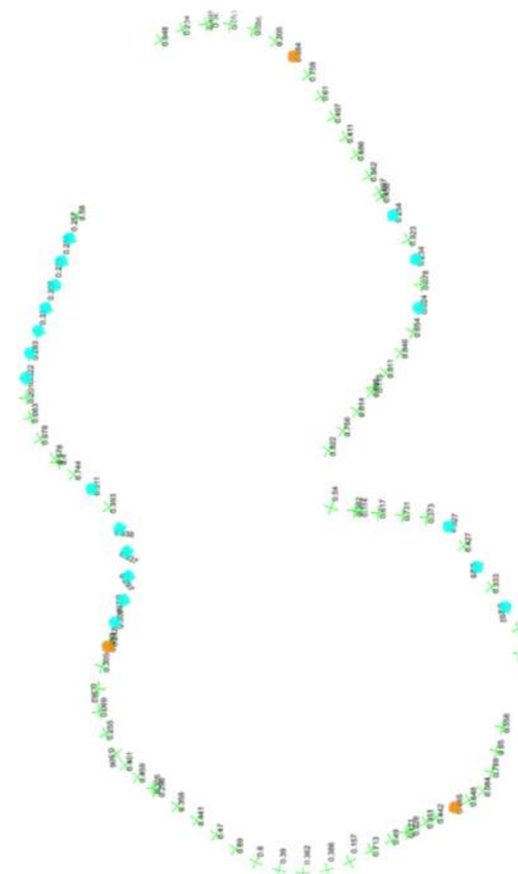
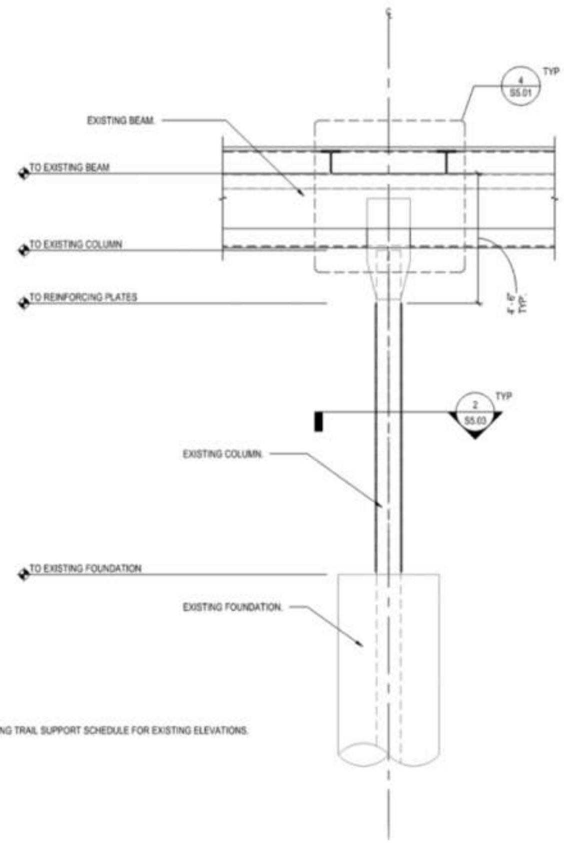
Column Reinforcement
Final Design – With Lateral Bracing

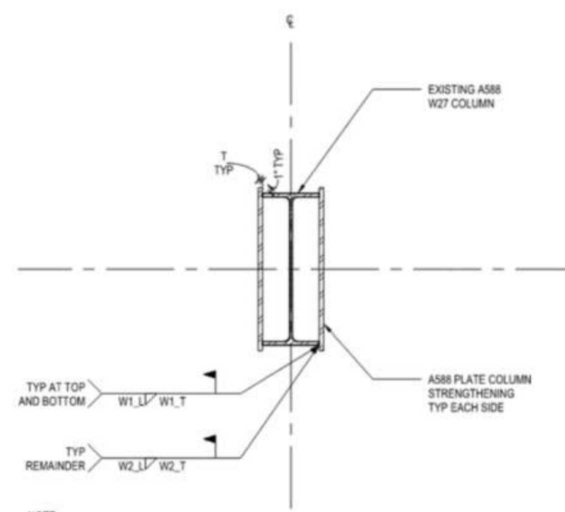
- 0.25" Strengthening Required (3)
- 0.50" Strengthening Required (0)
- 0.75" Strengthening Required (19)
- Add'l Strengthening Required (0)



Column Reinforcing



NOTES:
1. SEE EXISTING TRAIL SUPPORT SCHEDULE FOR EXISTING ELEVATIONS.



NOTE:
1. REFERENCE S4.01 FOR EXISTING COLUMN REINFORCEMENT ASSIGNMENTS.

	REINFORCEMENT TYPE
	R1
PLATE THICKNESS, T	3/4"
WELD 1 LENGTH, W1_L	96"
WELD 1 THICKNESS, W1_T	1/4"
WELD 2 LENGTH, W2_L	2"@4"
WELD 2 THICKNESS, W2_T	1/4"

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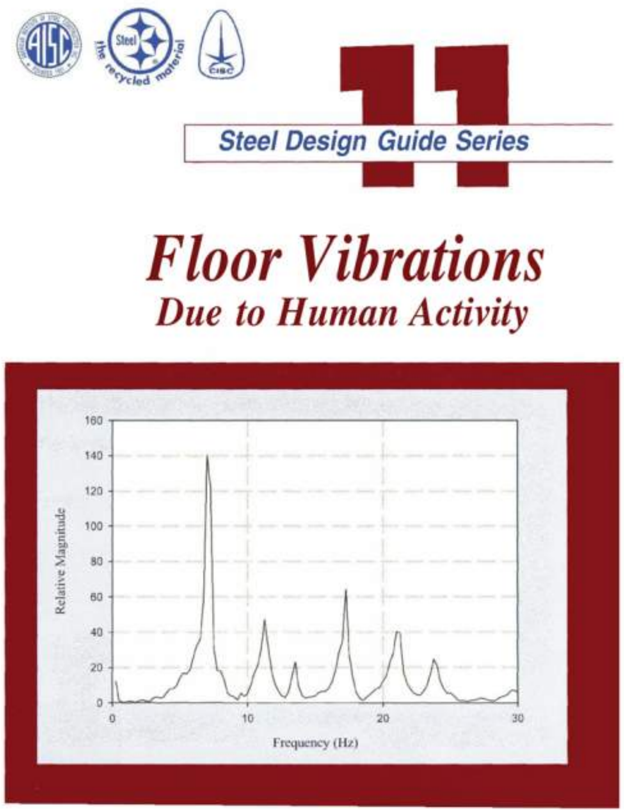
VIBRATION

Vibrations



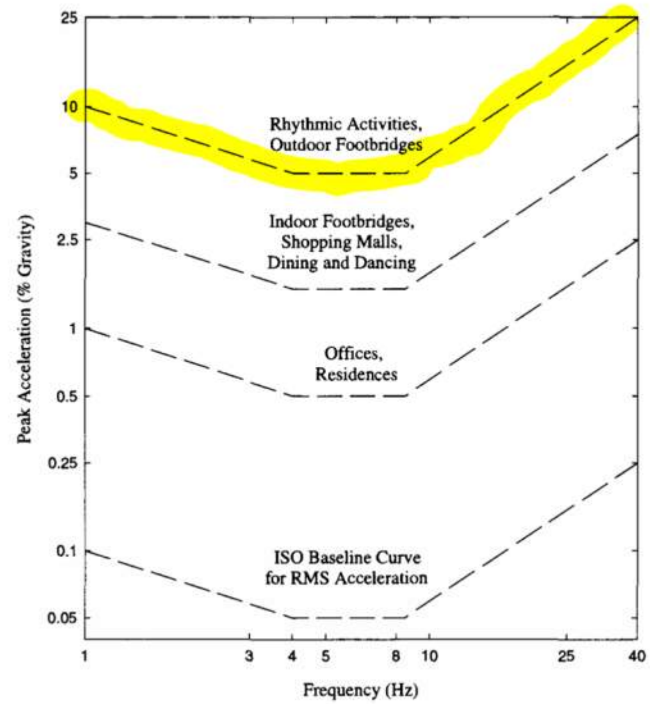


Fig. 2.1 Recommended peak acceleration for human comfort for vibrations due to human activities (Allen and Murray, 1993; ISO 2631-2: 1989).

$$\frac{a_p}{g} = \frac{P_o \exp(-0.35 f_n)}{\beta W}$$

Vibrations

$$\frac{a_p}{g} = \frac{P_o \exp(-0.35 f_n)}{\beta W}$$

- Preliminary walking vibration check
  - P_o = 92 lbs (Table 4.1)
  - f_n = 4.92 Hz (from SAP model)
  - Beta = 1.0% (Table 4.1) - possibly unconservative
  - W = 34,720 lb (Dead load of a single span)

  - a_p = 92 lbs * exp (-0.35*4.92Hz) / (0.01*34720 lb) * g = 4.74% g < 5.0% g OK!!!

    - Vibrations will be noticeable
    - This is for one person walking, what about a group?
    - What if someone starts running

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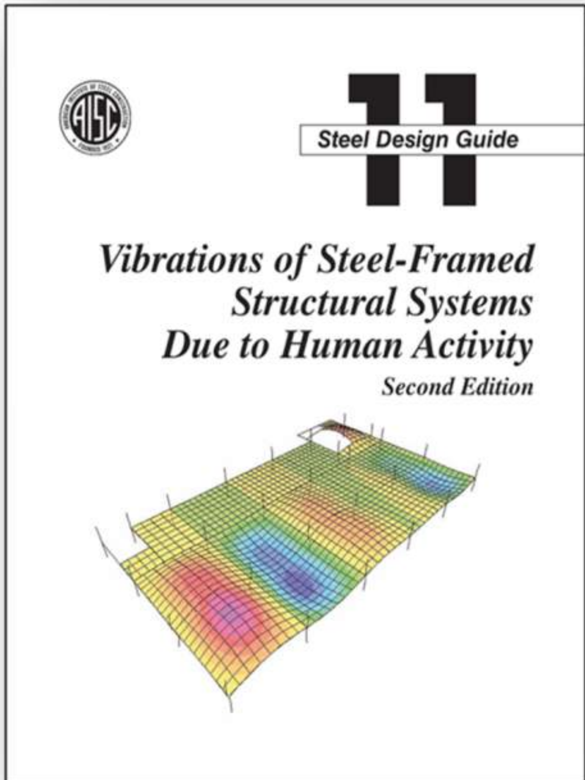
Vibrations



- Standard evaluation for walking excitation unchanged.
- Method for evaluating running excitation added!

$$\frac{a_p}{g} = \frac{0.79Q(e^{-0.173 f_n})}{\beta W}$$

- Guidance for incorporating group effects added!

P_{group} = sqrt(n) * P_o

- Finite element analysis method added!!!!!

Chapter 7! Chapter 7! Chapter 7!

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Vibrations

$$\frac{a_p}{g} = \frac{0.79Q(e^{-0.173 f_n})}{\beta W}$$

- Preliminary running vibration check
  - Q = 168 lbs (Table 4.1)
  - f_n = 4.92 Hz (from SAP model)
  - Beta = 1.0% (Table 4.1) - possibly unconservative
  - W = 34,720 lb (Dead load of a single span)

  - a_p = 0.79*168 lbs*exp(-0.173*4.92Hz)/(0.01*34720 lb)*g=16.3% g >> 5.0%g

    - Running on the trail isn't a great idea

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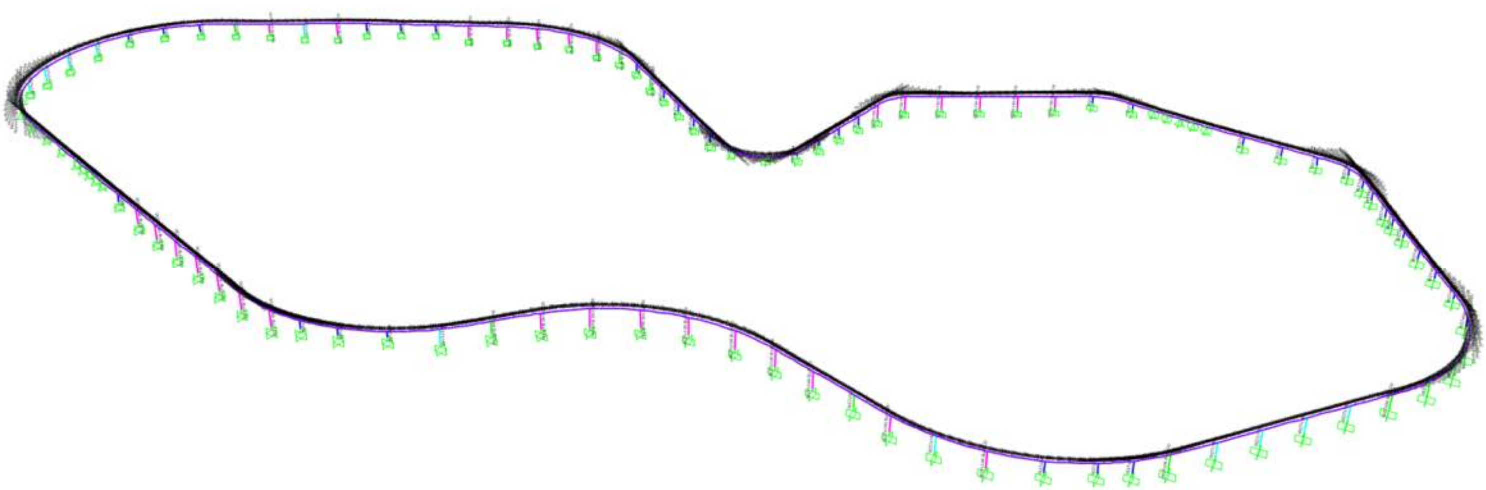
Vibrations

- Finite Element Method for Vibration Analysis



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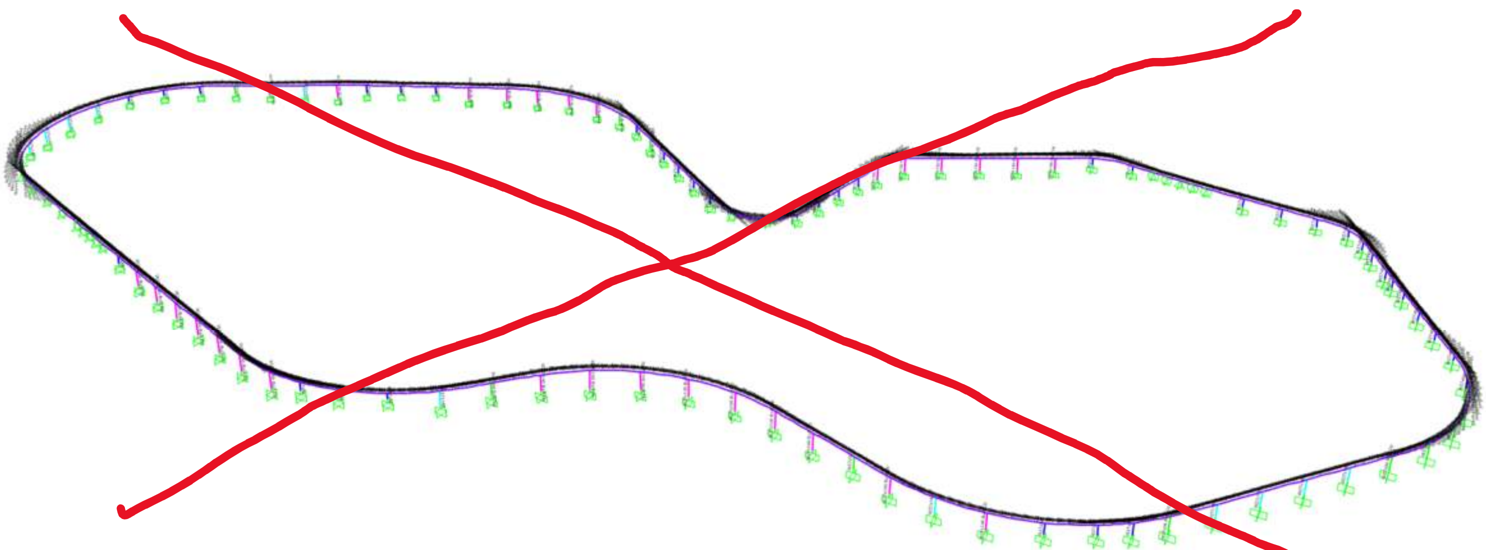
Vibrations

- Finite Element Method for Vibration Analysis



NO!

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Vibrations

- Finite Element Model for Vibrations
  - Limit model to 3 spans with mass to prevent model for predicting motion a great distances from the area of excitation overestimate modal mass and underestimating acceleration response.
  - Add a massless span on each end to capture continuity effects
  - Column bases assumed to be fully fixed
  - Beam and column connections assumed to be fixed
  - Decking and railings included only as mass
  - 1% viscous damping (probably an overestimate)
  - Assume walker and bystander are both at midspan
  - Perform eigen analysis in SAP or RISA to find all modes under 9 Hz

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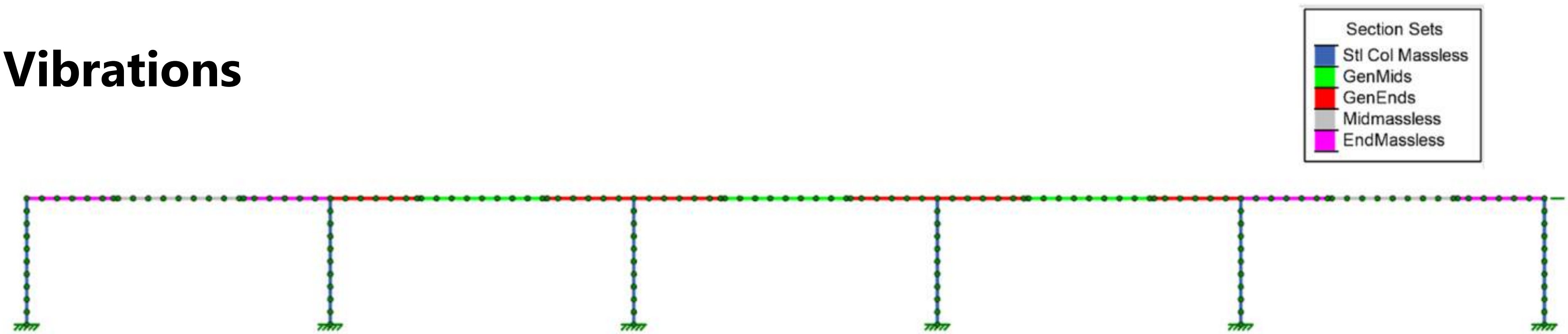
Vibrations

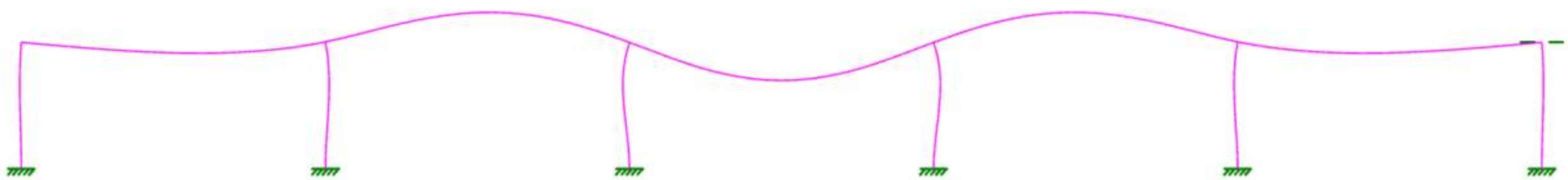


Frequencies

	Mode	Frequency (...	Period (S...	SX Participati...	SY Participati...	SZ Participati...
1	1	6.003	0.167		3.768	
2	2	7.669	0.13			
3	3	9.85	0.102		72.162	
4	4	11.353	0.088	94.321		
5	5	21.235	0.047			
6	6	23.412	0.043		0.724	
7	7	26.265	0.038			
8	8	35.976	0.028	4.65		
9	9	45.426	0.022		1.599	
10	10	45.983	0.022			
11	11	46.952	0.021		15.731	
12	12	63.154	0.016	0.689		
13	Totals:			99.665	93.985	

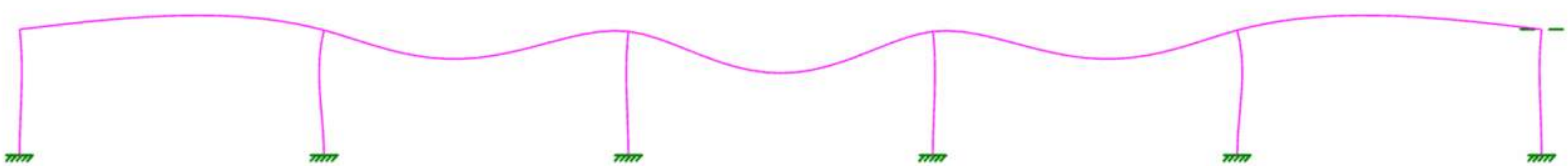
Vibrations

Mode Shape: 1 Period 0.167 Sec



Mode Shape: 2 Period 0.13 Sec

Mode Shape: 3 Period 0.102 Sec



Vibrations

- Acceleration Calculation (Chp7)

$$a_p = FRF_{Max} \alpha Q \rho \tag{7-1}$$

where

FRF_{Max} = maximum FRF magnitude at frequencies below 9 Hz, $\%g$/lb

Q = bodyweight = 168 lb

α = dynamic coefficient

ρ = resonant build-up factor

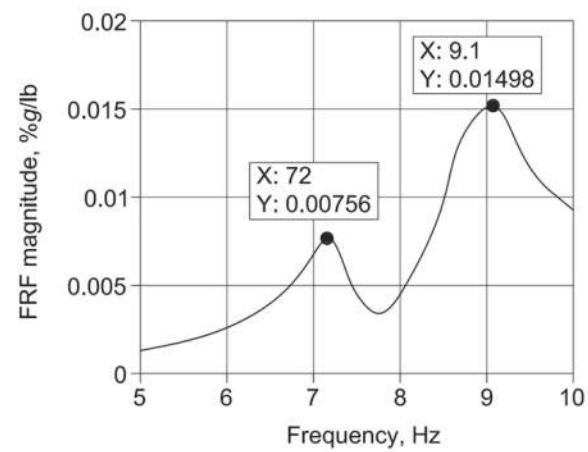
The dynamic coefficient is computed using the following equation, which approximates the Willford et al. (2007) second through fourth harmonic dynamic coefficients in Table 1-1. The equation was derived using the procedure described in Section 2.2.1.

$$\alpha = 0.09e^{-0.075 f_n} \tag{7-2}$$

where

f_n = dominant frequency, Hz

A frequency response function (FRF)—a plot of steady-state response due to sinusoidal load with unit amplitude versus frequency—is used to determine which mode(s) provides highest response, thus solving the problem.



(c) FRF magnitude plot

Fig. 7-4. Example predicted FRF for framing in Figure 7-1.

Vibrations

- Mode1:
  - f_1 = 6.00 Hz phi = 2.57
  - FRF = 0.332 $in^2/s^2/lb$
  - a = .332*.09*e^(-.075*6.00)*168lb*.75 = 2.40 in2/s = 0.6%g
- Mode 2:
  - f_2 = 7.67 Hz phi = 3.51
  - FRF = 0.614 $in^2/s^2/lb$
  - a = .614*.09*e^(-.075*7.67)*168lb*.75 = 3.92 in2/s = 1.0%g
- Mode 3:
  - f_3 = 9.85 Hz phi = 2.53 FRF = .324

- Mode 2 controls response:
  - Acceleration = 1.0%g < 5.0%g
  - 25 people walking randomly will potential reach the 5% criteria.
  - Great!

- But why did the numbers go down so much?

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Vibrations

- Chapter 4 and chapter 7 of design guide 11 use different forces to represent walking

Table 1-1. Fourier Series Parameters for Individuals

Activity	Source	Q, lb	f_{step} Range, Hz	Dynamic Coefficients, α_i	Phase Lag, ϕ_i, radians
Chp 4 Walking	Rainer et al. (1988) Allen and Murray (1993)	157	1.6–2.2	0.5, 0.2, 0.1, 0.05	—
Chp 7	Willford et al. (2007) Smith et al. (2007) Davis and Murray (2010)	168	1.6–2.2	0.4, 0.07, 0.06, 0.05	0, $-\pi/2$, π, $\pi/2$

- For second harmonic, Willford estimates about 37% of Allen and Murray excitation.
- For third harmonic, Willford estimates about 64% of Allen and Murray excitation.
    - Who's correct?

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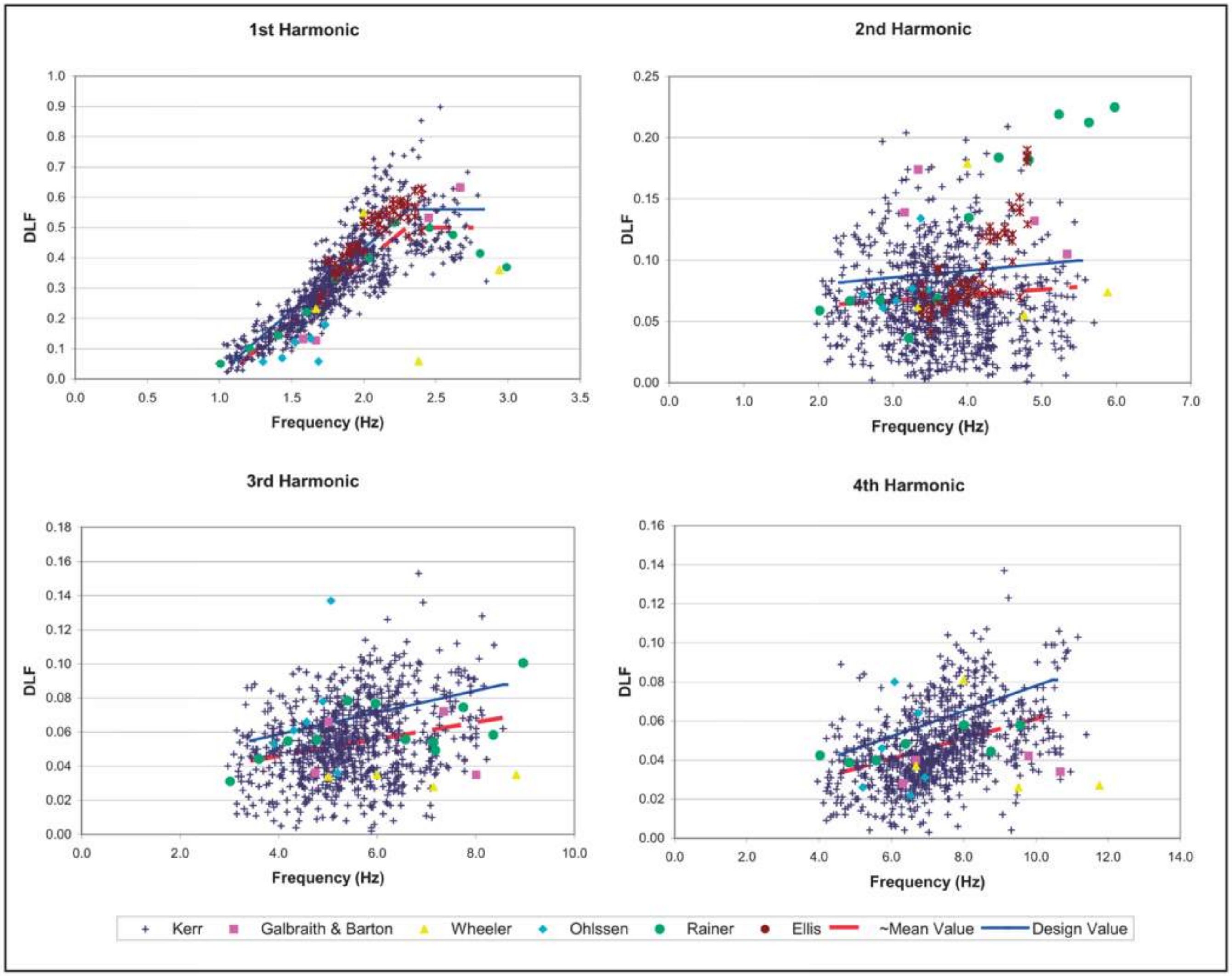
Vibrations

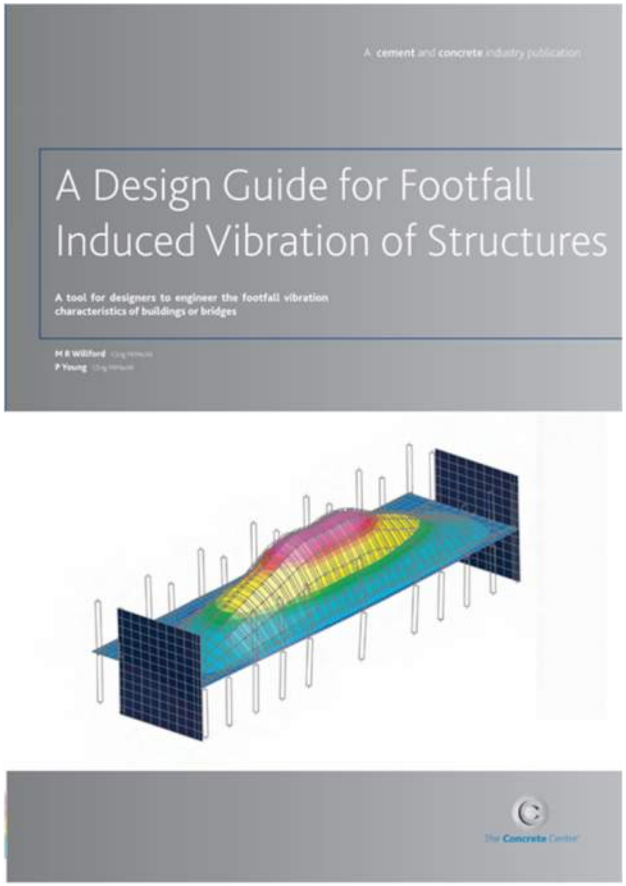
The first four harmonics of footfall forces.

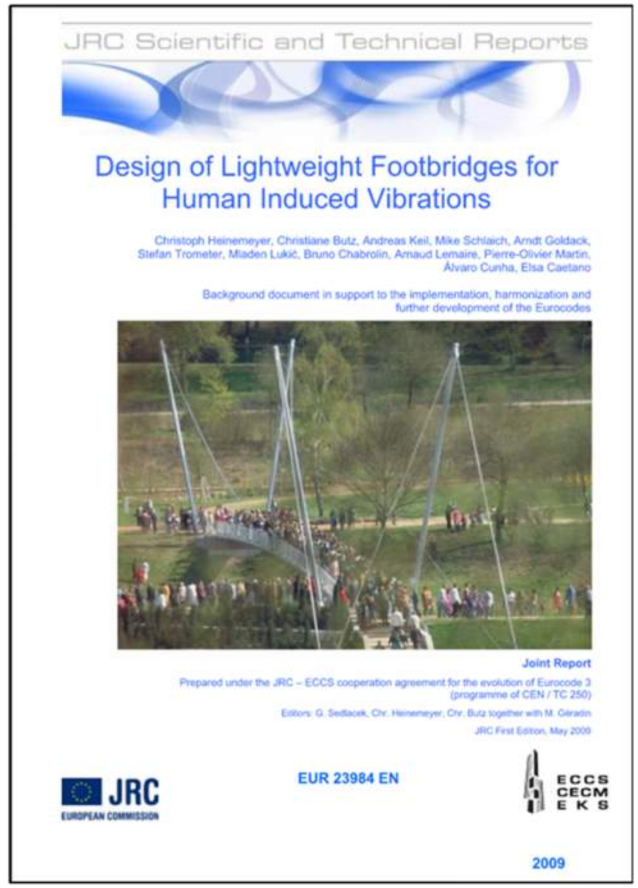


Vibrations

- Lateral and longitudinal vibrations just as concerning as vertical
- Maintain minimum frequencies to avoid first harmonic excitation
    - Longitudinal natural frequency > ~2.2 to 2.5 Hz
    - Lateral natural frequency > ~ 1.2 to 1.3 HZ

Vibration





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Vibrations

- Achieve a low probability of adverse comment

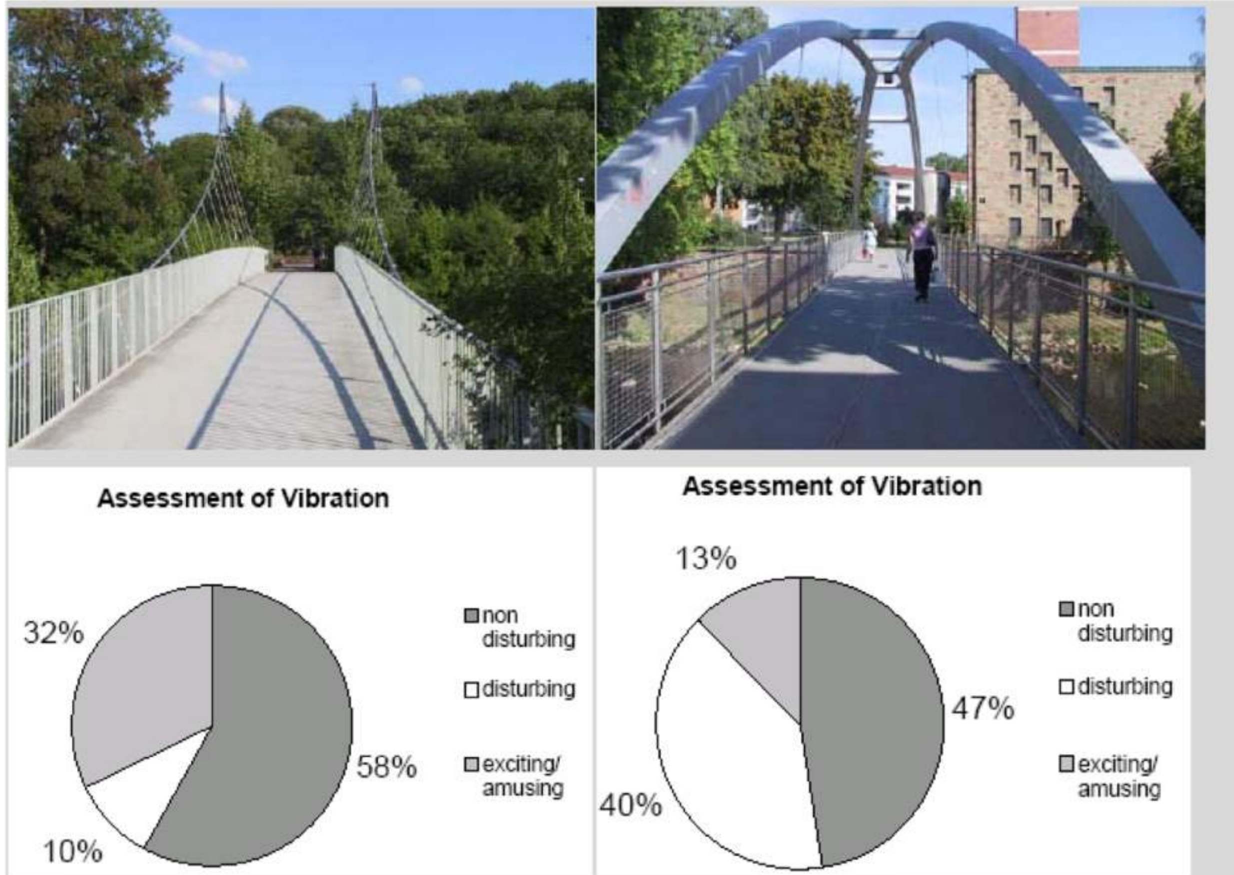
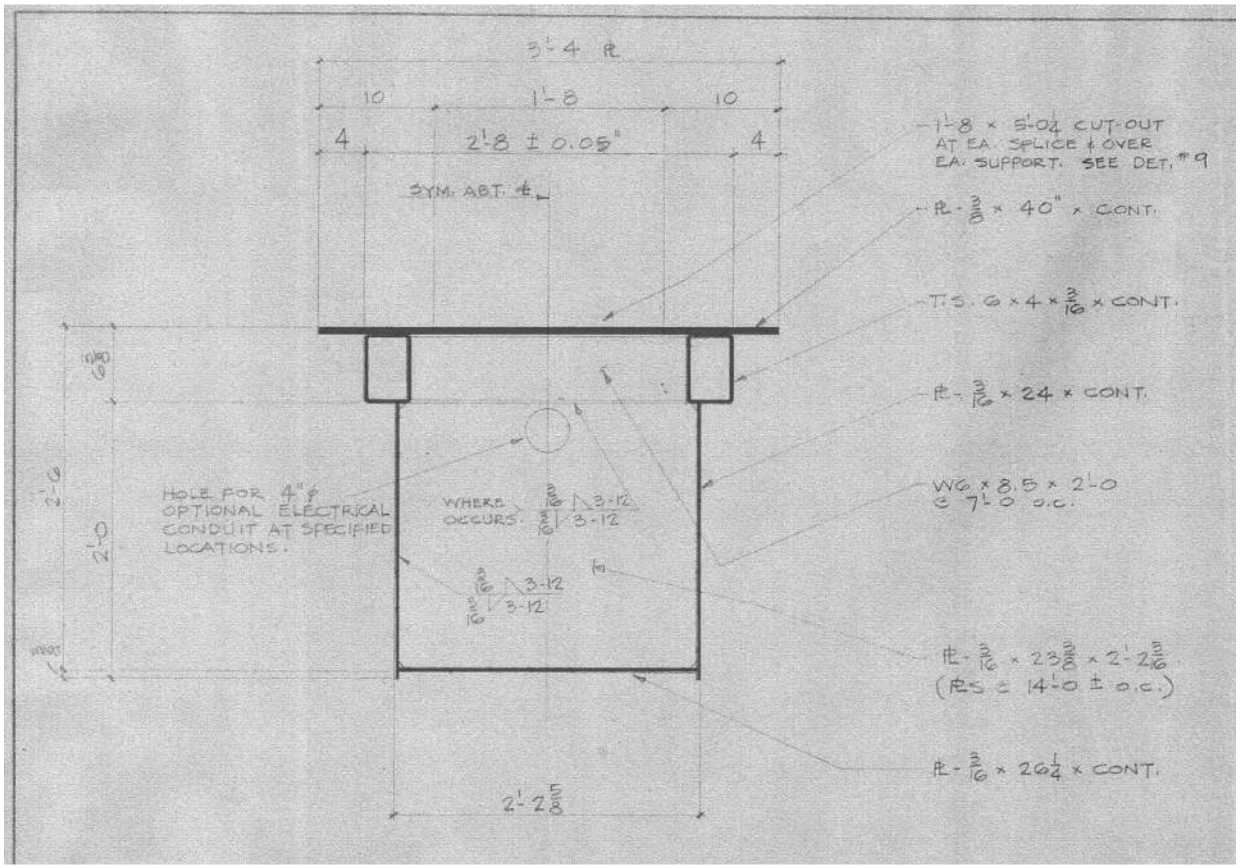


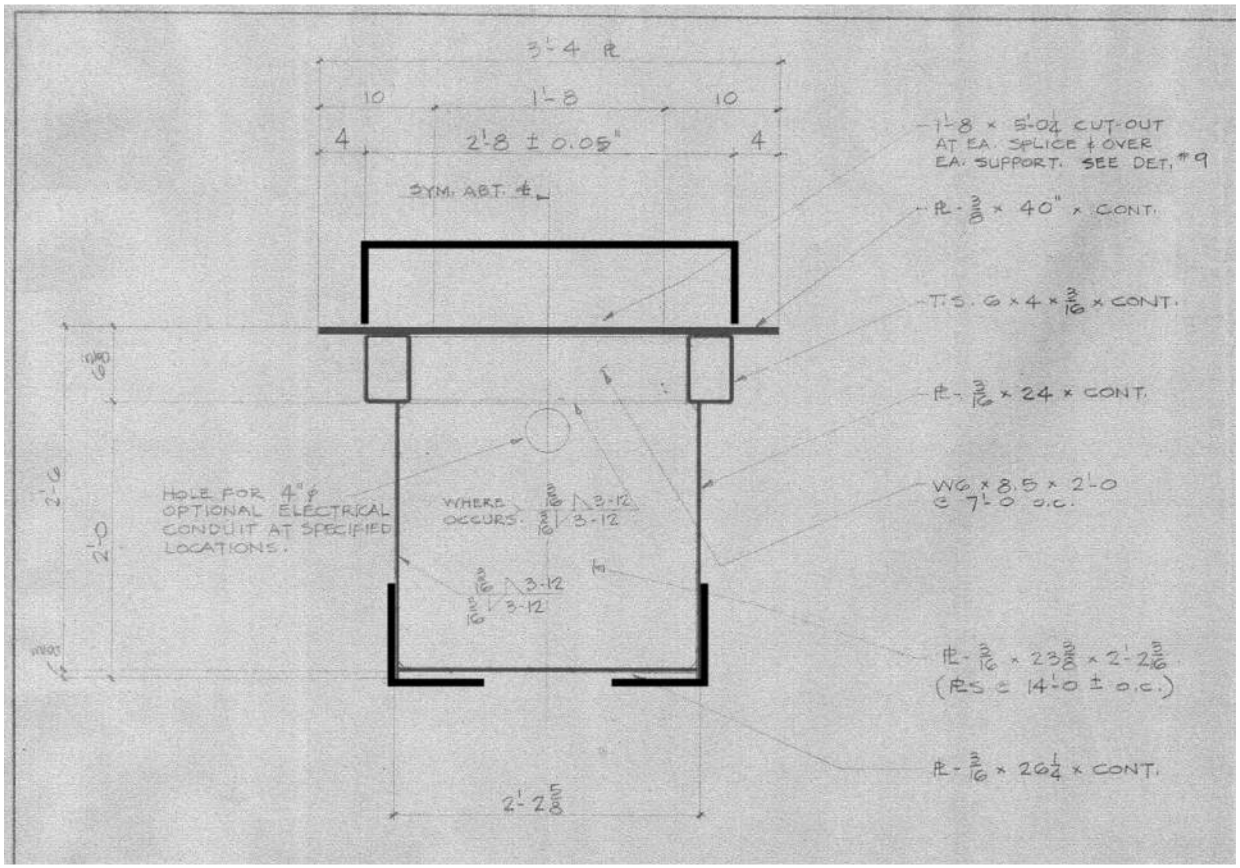
Figure 4-1: Comparison of vibration assessment of two footbridges

CAPACITY DESIGN

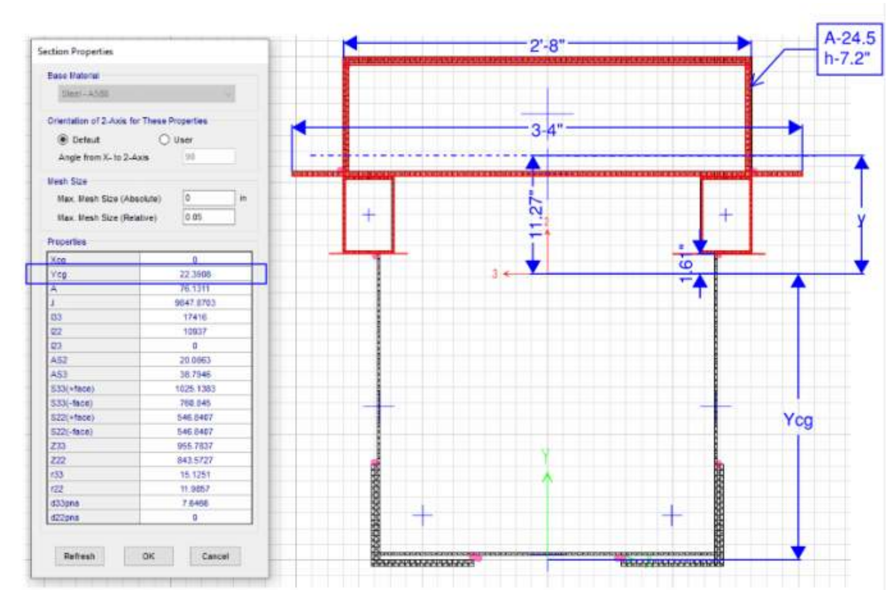
Beam Strengthening Approach



Beam Strengthening Approach

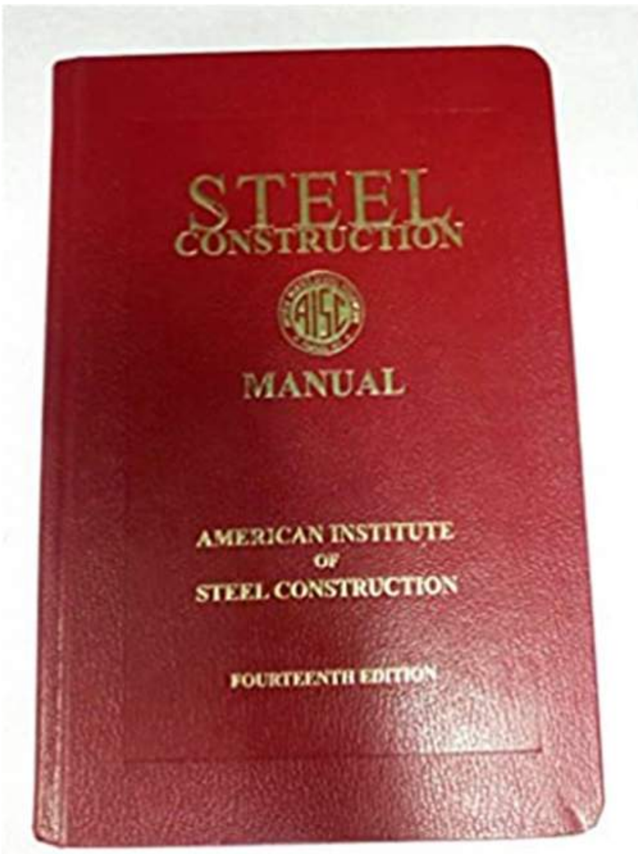


Beam Section Analysis



- Analyzed existing and new beam sections with SAP2000 and hand calculations
- Steel checks per AISC 360-16 chapter F7
  - Flexure
  - Combined shear and torsion
  - Weld capacities
- Used to determine required beam strengthening and welds

Beam Section Capacity



- How do you calculate the section capacity of an existing, built-up, singly symmetric member?
  - Tension
  - Compression
  - Major Axis Bending
  - Minor Axis Bending
  - Shear
  - Torsion
- Check existing welds and size new welds.

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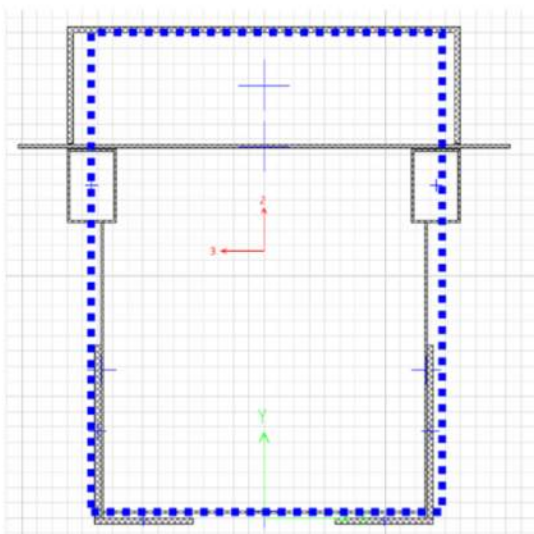
Major Axis Bending with Section F7 – Square and Rectangular HSS and Box-Shaped Members

- Section F12 directs the designer to determine F_{cr} "by analysis"
- Section F7 provides calculations for F_{cr} for geometries similar to the built up section

F1. General Provisions
~~F2.~~ Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis
~~F3.~~ Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis
~~F4.~~ Other I-Shaped Members With Compact or Noncompact Webs Bent About Their Major Axis
~~F5.~~ Doubly Symmetric and Singly Symmetric I-Shaped Members With Slender Webs Bent About Their Major Axis
~~F6.~~ I-Shaped Members and Channels Bent About Their Minor Axis
F7. Square and Rectangular HSS and Box-Shaped Members
~~F8.~~ Round HSS
~~F9.~~ Tees and Double Angles Loaded in the Plane of Symmetry
~~F10.~~ Single Angles
~~F11.~~ Rectangular Bars and Rounds
F12. Unsymmetrical Shapes
~~F13.~~ Proportions of Beams and Girders

Major Axis Bending with Section F7 – Square and Rectangular HSS and Box-Shaped Members

- Approximate the built-up section as a "box shaped member"
- Use Section F7 design calculations with section properties of the real built-up section



F7. SQUARE AND RECTANGULAR HSS AND BOX-SHAPED MEMBERS

This section applies to square and rectangular *HSS*, and doubly symmetric box-shaped members bent about either axis, having compact or noncompact webs and compact, noncompact or slender flanges as defined in Section B4.1 for flexure.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the *limit states* of *yielding (plastic moment)*, flange *local buckling* and web local buckling under pure flexure.

> **User Note:** Very long rectangular HSS bent about the major axis are subject to *lateral-torsional buckling*; however, the Specification provides no strength equation for this limit state since *beam* deflection will control for all reasonable cases.

1. Yielding

$$M_n = M_p = F_y Z \tag{F7-1}$$

where

Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)

2. Flange Local Buckling

(a) For *compact sections*, the *limit state* of flange *local buckling* does not apply.

(b) For sections with noncompact flanges

$$M_n = M_p - (M_p - F_y S)\left(3.57\frac{b}{t_f}\sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p \tag{F7-2}$$

(c) For sections with slender flanges

$$M_n = F_y S_e \tag{F7-3}$$

where

S_e = effective section modulus determined with the effective width, b_e, of the compression flange taken as:

$$b_e = 1.92 t_f \sqrt{\frac{E}{F_y}}\left[1 - \frac{0.38}{b/t_f}\sqrt{\frac{E}{F_y}}\right] \le b \tag{F7-4}$$

3. Web Local Buckling

(a) For *compact sections*, the *limit state* of web *local buckling* does not apply.

(b) For sections with noncompact webs

$$M_n = M_p - (M_p - F_y S)\left(0.305\frac{h}{t_w}\sqrt{\frac{F_y}{E}} - 0.738\right) \le M_p \tag{F7-5}$$

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Major Axis Bending with Section F7

- Start with Flange Local Buckling
- We have slender flanges
- Steps:

1. Determine b effective
2. Remove remainder of compression flange from cross section
3. Calculate new effective section properties

2. Flange Local Buckling

(a) For *compact sections*, the *limit state* of flange *local buckling* does not apply.
(b) For sections with noncompact flanges

$$M_n = M_p - (M_p - F_y S)\left(3.57\frac{b}{t_f}\sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p \quad \text{(F7-2)}$$

Use the built up section (modified with b_e) here, not an idealized box.

(c) For sections with slender flanges

$$M_n = F_y S_e \quad \text{(F7-3)}$$

where

S_e = effective section modulus determined with the effective width, b_e, of the compression flange taken as:

effective width of top flange and bottom flange. Determine which controls and use that.

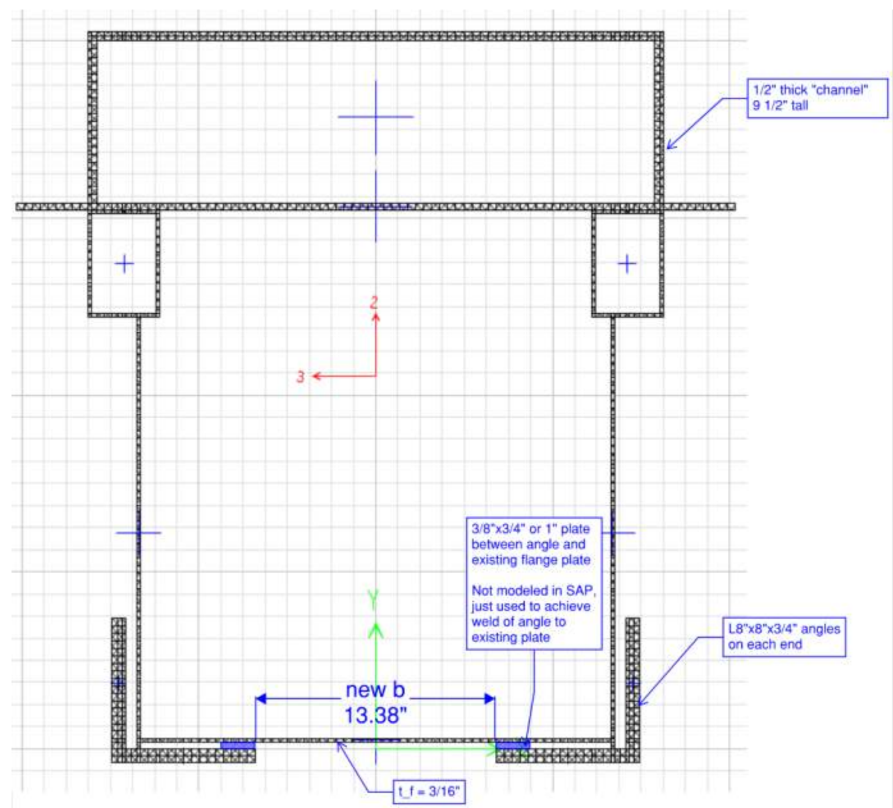
$$b_e = 1.92 t_f \sqrt{\frac{E}{F_y}}\left[1 - \frac{0.38}{b/t_f}\sqrt{\frac{E}{F_y}}\right] \le b \quad \text{(F7-4)}$$

Major Axis Bending with Section F7

- Start with Flange Local Buckling
- We have slender flanges
- Steps:

1. Determine b effective
2. Remove remainder of compression flange from cross section
3. Calculate new effective section properties (using SAP)

$$b_e = 1.92 t_f \sqrt{\frac{E}{F_y}} \left[1 - \frac{0.38}{b / t_f} \sqrt{\frac{E}{F_y}} \right] \le b$$



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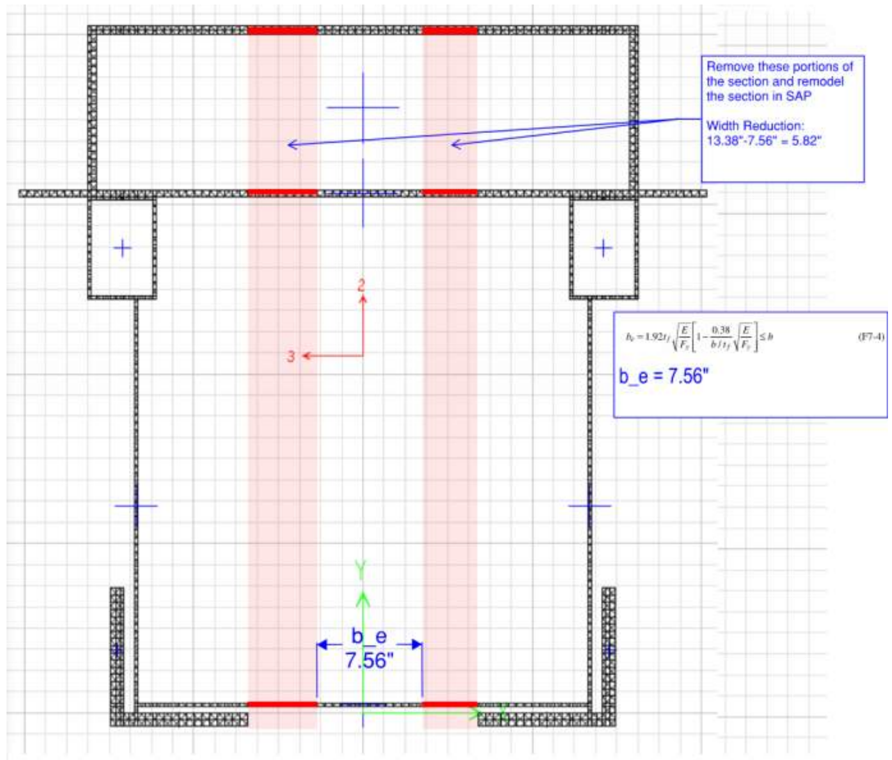
Major Axis Bending with Section F7

- Start with Flange Local Buckling
- We have slender flanges
- Steps:

1. Determine b effective
2. Remove remainder of compression flange from cross section
3. Calculate new effective section properties (using SAP)

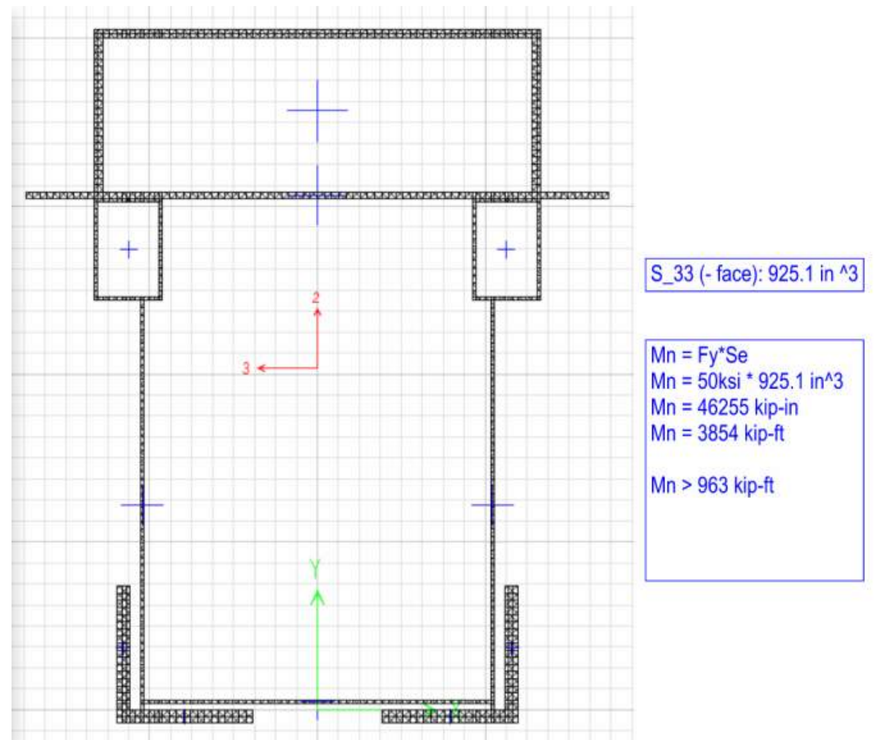
$$b_e = 1.92 t_f \sqrt{\frac{E}{F_y}}\left[1 - \frac{0.38}{b/t_f}\sqrt{\frac{E}{F_y}}\right] \le b$$



Major Axis Bending with Section F7

- Start with Flange Local Buckling
- We have slender flanges
- Steps:

1. Determine b effective
2. Remove remainder of compression flange from cross section
3. Calculate new effective section properties (using SAP)



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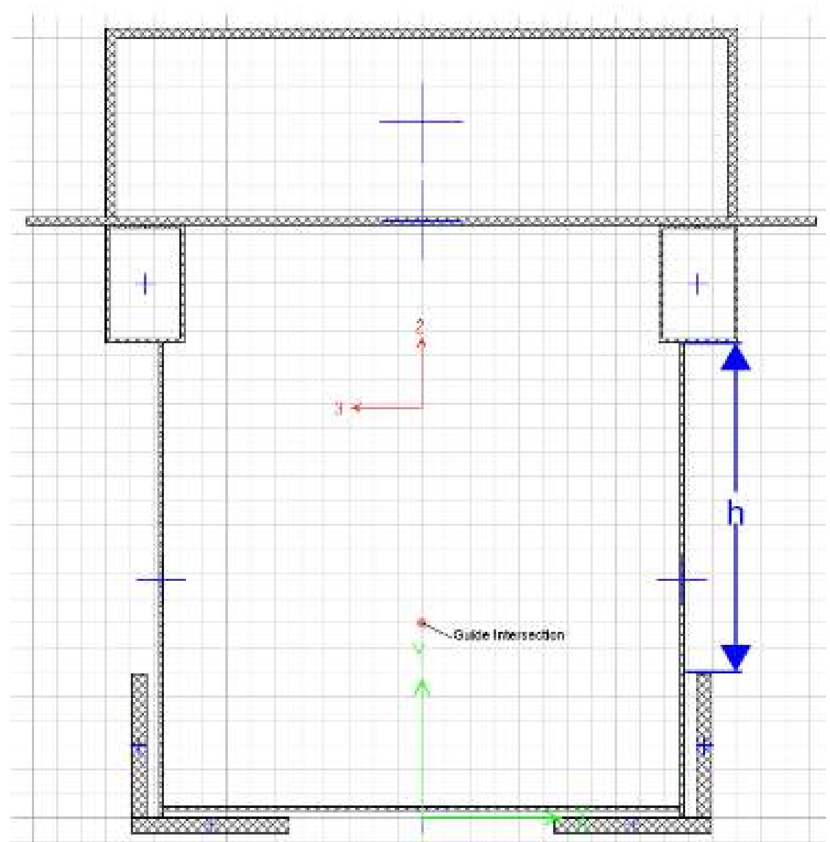
Major Axis Bending with Section F7

- Calculate Yielding and Web Local Buckling
- Yielding:
  - $Mn = M_p = F_y * Z$
- Web Local Buckling
  - We have noncompact webs
  - Portion of web that can buckle is "h"



3. Web Local Buckling

(a) For *compact sections*, the *limit state* of web *local buckling* does not apply.

(b) For sections with noncompact webs

$$M_n = M_p - (M_p - F_y S)\left(0.305\frac{h}{t_w}\sqrt{\frac{F_y}{E}} - 0.738\right) \le M_p \qquad \text{(F7-5)}$$

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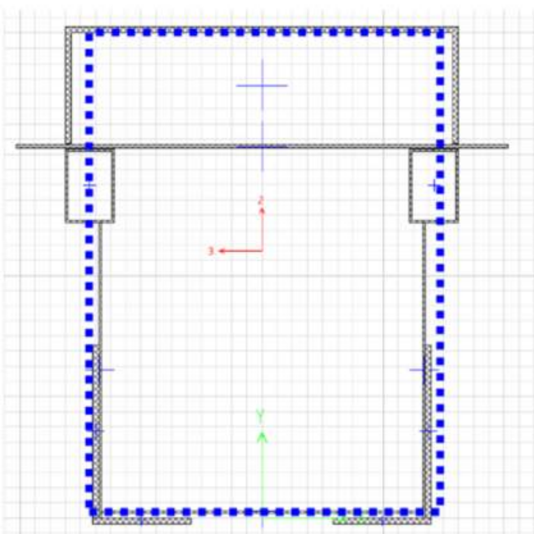
Major Axis Bending with Section F7 – Square and Rectangular HSS and Box-Shaped Members

- Lowest value of Mn from Yielding, Flange Local Buckling, and Web Local Buckling determines the nominal flexural strength.



F7. SQUARE AND RECTANGULAR HSS AND BOX-SHAPED MEMBERS

This section applies to square and rectangular *HSS*, and doubly symmetric box-shaped members bent about either axis, having compact or noncompact webs and compact, noncompact or slender flanges as defined in Section B4.1 for flexure.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the *limit states* of *yielding (plastic moment)*, flange *local buckling* and web local buckling under pure flexure.

> **User Note:** Very long rectangular HSS bent about the major axis are subject to *lateral-torsional buckling*; however, the Specification provides no strength equation for this limit state since *beam* deflection will control for all reasonable cases.

1. Yielding

$$M_n = M_p = F_y Z \qquad \text{(F7-1)}$$

where

Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)

2. Flange Local Buckling

(a) For *compact sections*, the *limit state* of flange *local buckling* does not apply.

(b) For sections with noncompact flanges

$$M_n = M_p - \left(M_p - F_y S\right)\left(3.57\frac{b}{t_f}\sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p \qquad \text{(F7-2)}$$

(c) For sections with slender flanges

$$M_n = F_y S_e \qquad \text{(F7-3)}$$

where

S_e = effective section modulus determined with the effective width, b_e, of the compression flange taken as:

$$b_e = 1.92 t_f \sqrt{\frac{E}{F_y}}\left[1 - \frac{0.38}{b/t_f}\sqrt{\frac{E}{F_y}}\right] \le b \qquad \text{(F7-4)}$$

3. Web Local Buckling

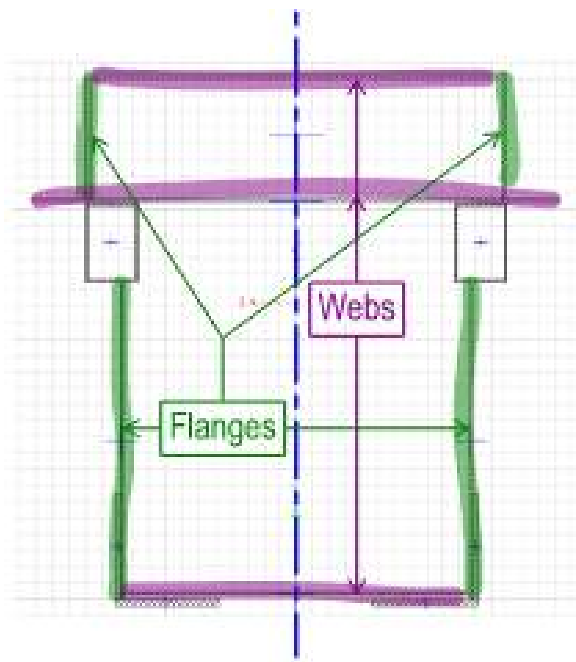
(a) For *compact sections*, the *limit state* of web *local buckling* does not apply.

(b) For sections with noncompact webs

$$M_n = M_p - \left(M_p - F_y S\right)\left(0.305\frac{h}{t_w}\sqrt{\frac{F_y}{E}} - 0.738\right) \le M_p \qquad \text{(F7-5)}$$

Minor Axis Bending with Section F7

- Use same approach as major axis with F7 checks



F7. SQUARE AND RECTANGULAR HSS AND BOX-SHAPED MEMBERS

This section applies to square and rectangular *HSS*, and doubly symmetric box-shaped members bent about either axis, having compact or noncompact webs and compact, noncompact or slender flanges as defined in Section B4.1 for flexure.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the *limit states* of *yielding (plastic moment)*, flange *local buckling* and web local buckling under pure flexure.

User Note: Very long rectangular HSS bent about the major axis are subject to *lateral-torsional buckling*; however, the Specification provides no strength equation for this limit state since *beam* deflection will control for all reasonable cases.

1. Yielding

$$M_n = M_p = F_y Z \qquad \text{(F7-1)}$$

where

Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)

2. Flange Local Buckling

(a) For *compact sections*, the *limit state* of flange *local buckling* does not apply.
(b) For sections with noncompact flanges

$$M_n = M_p - (M_p - F_y S)\left(3.57\frac{b}{t_f}\sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p \qquad \text{(F7-2)}$$

(c) For sections with slender flanges

$$M_n = F_y S_e \qquad \text{(F7-3)}$$

where

S_e = effective section modulus determined with the effective width, b_e, of the compression flange taken as:

$$b_e = 1.92 t_f \sqrt{\frac{E}{F_y}}\left[1 - \frac{0.38}{b/t_f}\sqrt{\frac{E}{F_y}}\right] \le b \qquad \text{(F7-4)}$$

3. Web Local Buckling

(a) For *compact sections*, the *limit state* of web *local buckling* does not apply.
(b) For sections with noncompact webs

$$M_n = M_p - (M_p - F_y S)\left(0.305\frac{h}{t_w}\sqrt{\frac{F_y}{E}} - 0.738\right) \le M_p \qquad \text{(F7-5)}$$

Compression Capacity with Section E7

- Calculate F_{cr} based on Q
- Q: Net reduction factor for all slender elements
  - $Q = Q_s Q_a$

E7. MEMBERS WITH SLENDER ELEMENTS

This section applies to slender-element compression members, as defined in Section B4.1 for elements in uniform compression.

The *nominal compressive strength*, P_n, shall be the lowest value based on the applicable *limit states* of *flexural buckling*, *torsional buckling*, and *flexural-torsional buckling*.

$$P_n = F_{cr} A_g \tag{E7-1}$$

The critical *stress*, F_{cr}, shall be determined as follows:

(a) When $\dfrac{KL}{r} \le 4.71\sqrt{\dfrac{E}{QF_y}}$ $\left(\text{or } \dfrac{QF_y}{F_e} \le 2.25\right)$

$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y \tag{E7-2}$$

(b) When $\dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{QF_y}}$ $\left(\text{or } \dfrac{QF_y}{F_e} > 2.25\right)$

$$F_{cr} = 0.877F_e \tag{E7-3}$$

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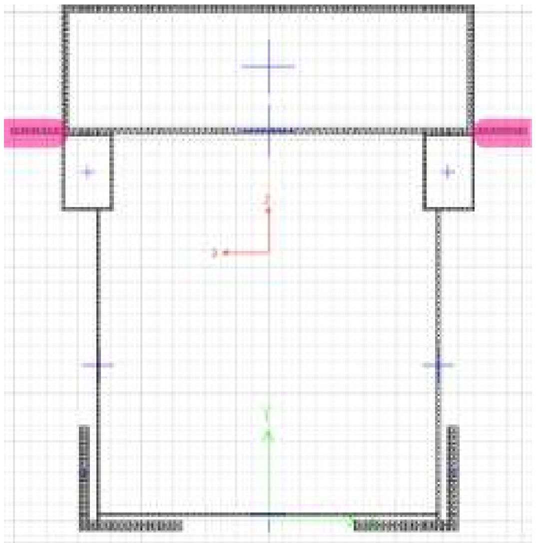
Compression Capacity with Section E7

- Calculate Q_s for slender *unstiffened* elements
- Calculate Q_a for slender *stiffened* elements
- $Q = Q_s * Q_a$
- Use Q to calculate F_{cr}

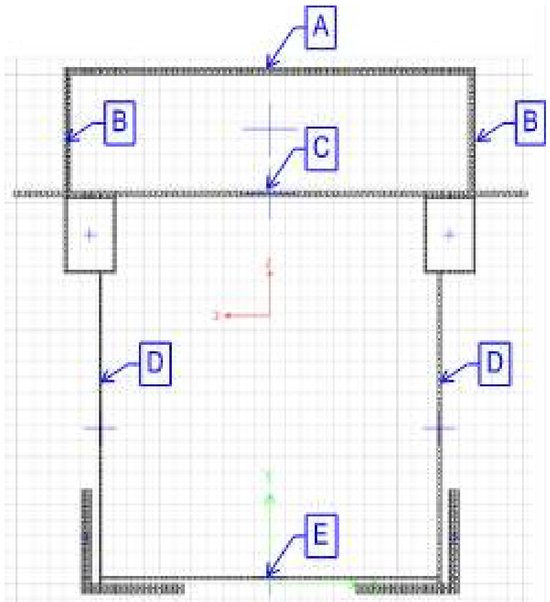
Slender *unstiffened* elements:



Compression Capacity with Section E7

- Calculate Q_s for slender stiffened elements
- Calculate Q_a for slender stiffened elements
- $Q = Q_s * Q_a$
- Use Q to calculate F_{cr}

Slender *unstiffened* elements:



Compression Capacity with Section E7

- Calculate Q_s for slender stiffened elements
- Calculate Q_a for slender stiffened elements
- $Q = Q_s * Q_a$
- Use Q to calculate F_{cr}

The critical *stress*, F_{cr}, shall be determined as follows:

(a) When $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{QF_y}}$ $\left(\text{or } \frac{QF_y}{F_e} \leq 2.25\right)$

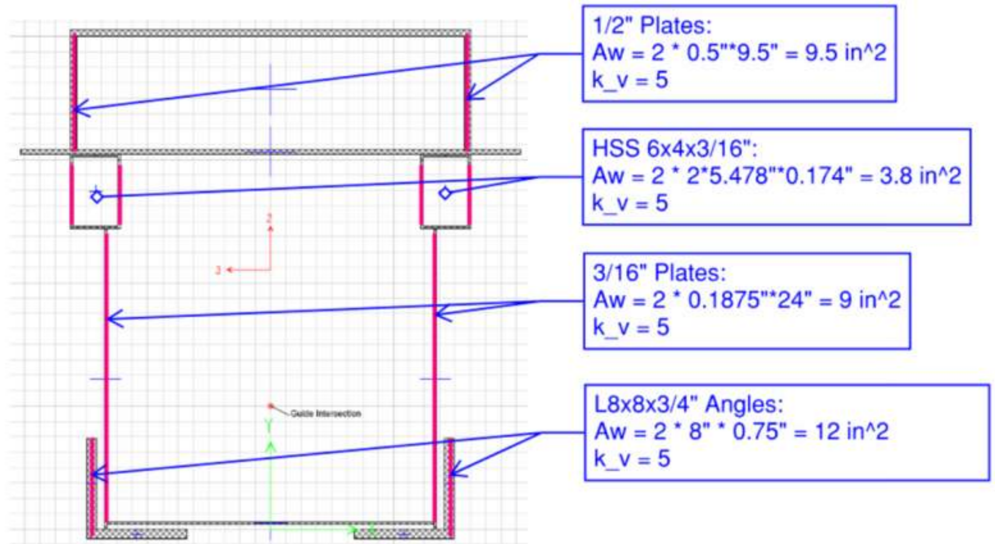
$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y$$

(b) When $\frac{KL}{r} > 4.71\sqrt{\frac{E}{QF_y}}$ $\left(\text{or } \frac{QF_y}{F_e} > 2.25\right)$

$$F_{cr} = 0.877F_e$$

Shear Capacity with Section G2 – Members with Unstiffened or Stiffened Webs

- Calculate the shear capacity of individual web elements and add them all together.



G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

1. Shear Strength

This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The *nominal shear strength*, V_n, of unstiffened or stiffened webs according to the *limit states* of *shear yielding* and *shear buckling*, is

$$V_n = 0.6 F_y A_w C_v \qquad \text{(G2-1)}$$

Calculating C_v

1/2" Plates:	HSS 6x4x3/16"	3/16" Plates:	8"x3/4" Plates:
h/tw = 9.5"/0.5" = 19 1.1*sqrt(kv*E/Fy) = 59.2 h/tw < 1.1*sqrt(kv*E/Fy) so C_v = 1	h/tw = 5.478"/0.174" = 31.5 1.1*sqrt(kv*E/Fy) = 59.2 h/tw < 1.1*sqrt(kv*E/Fy) so C_v = 1	h/tw = 24"/0.1875" = 128 1.1*sqrt(kv*E/Fy) = 59.2 1.37*sqrt(kv*E/Fy) = 73.8 h/tw > 1.37*sqrt(kv*E/Fy) so C_v = 1.51k_v*E/ ((h/tw)^2 Fy) C_v = 0.267	h/tw = 8"/0.75" = 10.7 1.1*sqrt(kv*E/Fy) = 29.0 1.37*sqrt(kv*E/Fy) = 36.14 h/tw < 1.1*sqrt(kv*E/Fy) so C_v = 1

V_n = 0.6 * (50 ksi) * [(9.5 in^2 * 1) * (3.8 in^2 * 1) * (9 in^2 * 0.267) * (12 in^2 * 1)] = 831 kips

V_c = phi_v * V_n = 0.9*831 = 747.9 kips

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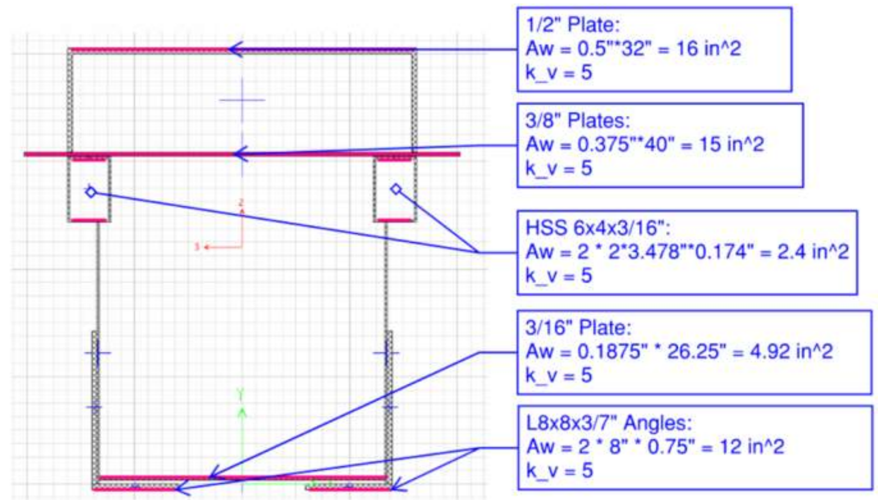
Shear Capacity with Section G2 – Members with Unstiffened or Stiffened Webs

- Calculate the shear capacity of individual web elements and add them all together.



Calculating C_v

1/2" Plates:	3/8" Plates:	HSS 6x4x3/16"	3/16" Plates:	8"x3/4" Plates:
h/tw = 32"/0.5" = 64 1.1*sqrt(kv*E/Fy) = 59.2 1.37*sqrt(kv*E/Fy) = 73.8	h/tw = 24"/0.5" = 48 1.1*sqrt(kv*E/Fy) = 59.2	h/tw = 3.478"/0.174" = 19.9 1.1*sqrt(kv*E/Fy) = 59.2	h/tw = 10.625"/0.1875" = 56.6 1.1*sqrt(kv*E/Fy) = 59.2	h/tw = 8"/0.75" = 10.67 1.1*sqrt(kv*E/Fy) = 29.0
h/tw > 1.37*sqrt(kv*E/Fy)	h/tw < 1.1*sqrt(kv*E/Fy)	h/tw < 1.1*sqrt(kv*E/Fy)	h/tw < 1.1*sqrt(kv*E/Fy)	h/tw < 1.1*sqrt(kv*E/Fy)
so C_v = 1.1sqrt(k_v*E/Fy)/(h/tw) Cv = 0.926	so C_v = 1	so C_v = 1	so C_v = 1	so C_v = 1

V_n = 0.6 * (50 ksi) * [(16 in^2 * 0.926) + (15 in^2 * 1) + (2.4 in^2 * 1) + (4.92 in^2 * 1) + (12 in^2 * 1)] = 1474 kips
V_c = phi_v * V_n = 0.9*1474 = 1327 kips (Reinforced Heavy Option)

G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

1. Shear Strength

This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The *nominal shear strength*, V_n, of unstiffened or stiffened webs according to the *limit states* of *shear yielding* and *shear buckling*, is

$$V_n = 0.6F_yA_wC_v \tag{G2-1}$$

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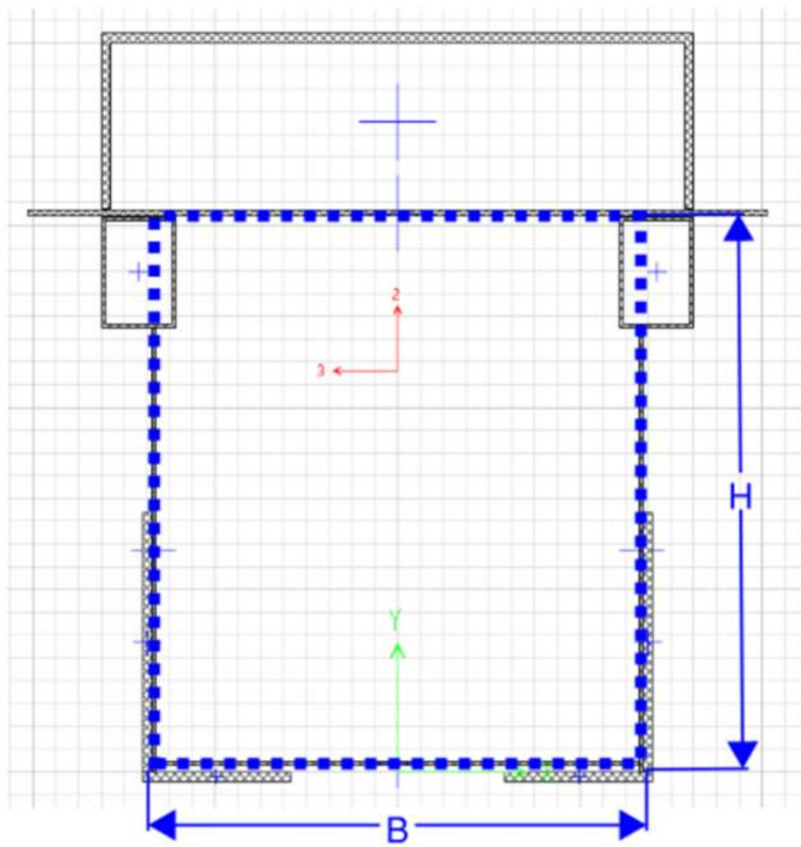
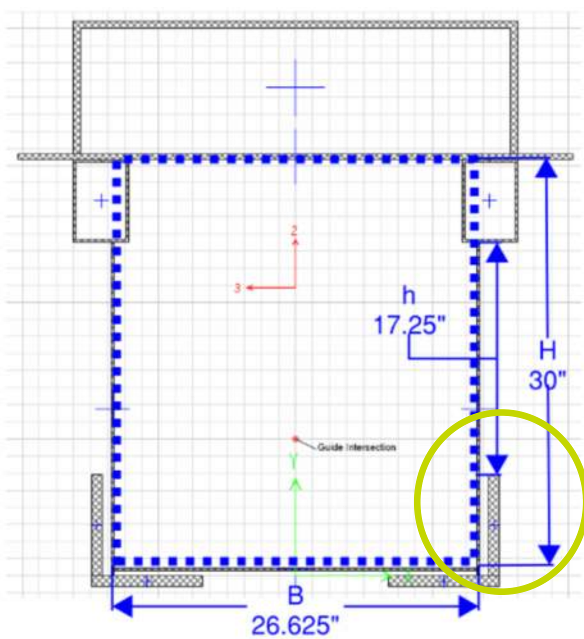
Torsion Capacity with Section H2 – HSS Subject to Combined Torsion Shear, Flexure, and Axial Force

- The beam section is required to carry torsion from dead load (due to the curvature of the beam and the eccentric deck), and torsion from wind and unbalanced live loads on the deck.
- We don't have an HSS section, but the approach is to analyze a conservative simplification "box" structure



Torsion Capacity with Section H2 – HSS Subject to Combined Torsion Shear, Flexure, and Axial Force

- Need to calculate C, the torsional constant
- Need to calculate F_{cr}



User Note: The torsional constant, C, may be conservatively taken as:

For round HSS: $C = \dfrac{\pi(D-t)^2 t}{2}$

For rectangular HSS: $C = 2(B-t)(H-t)t - 4.5(4-\pi)t^3$

The critical *stress*, F_{cr}, shall be determined as follows:

(b) For rectangular HSS

(i) When $h/t \le 2.45\sqrt{E/F_y}$

$$F_{cr} = 0.6F_y \tag{H3-3}$$

(ii) When $2.45\sqrt{\dfrac{E}{F_y}} < h/t \le 3.07\sqrt{\dfrac{E}{F_y}}$

$$F_{cr} = \frac{0.6F_y\left(2.45\sqrt{E/F_y}\right)}{\left(\dfrac{h}{t}\right)} \tag{H3-4}$$

(iii) When $3.07\sqrt{\dfrac{E}{F_y}} < h/t \le 260$

$$F_{cr} = \frac{0.458\pi^2 E}{\left(\dfrac{h}{t}\right)^2} \tag{H3-5}$$

where

h = *flat width* of longer side as defined in Section B4.1b(d), in. (mm)
t = *design wall thickness* defined in Section B4.2, in. (mm)

Capacity Diagram

Where Tr < 0.2 Tc:

(a) When $\frac{P_r}{P_c} \geq 0.2$

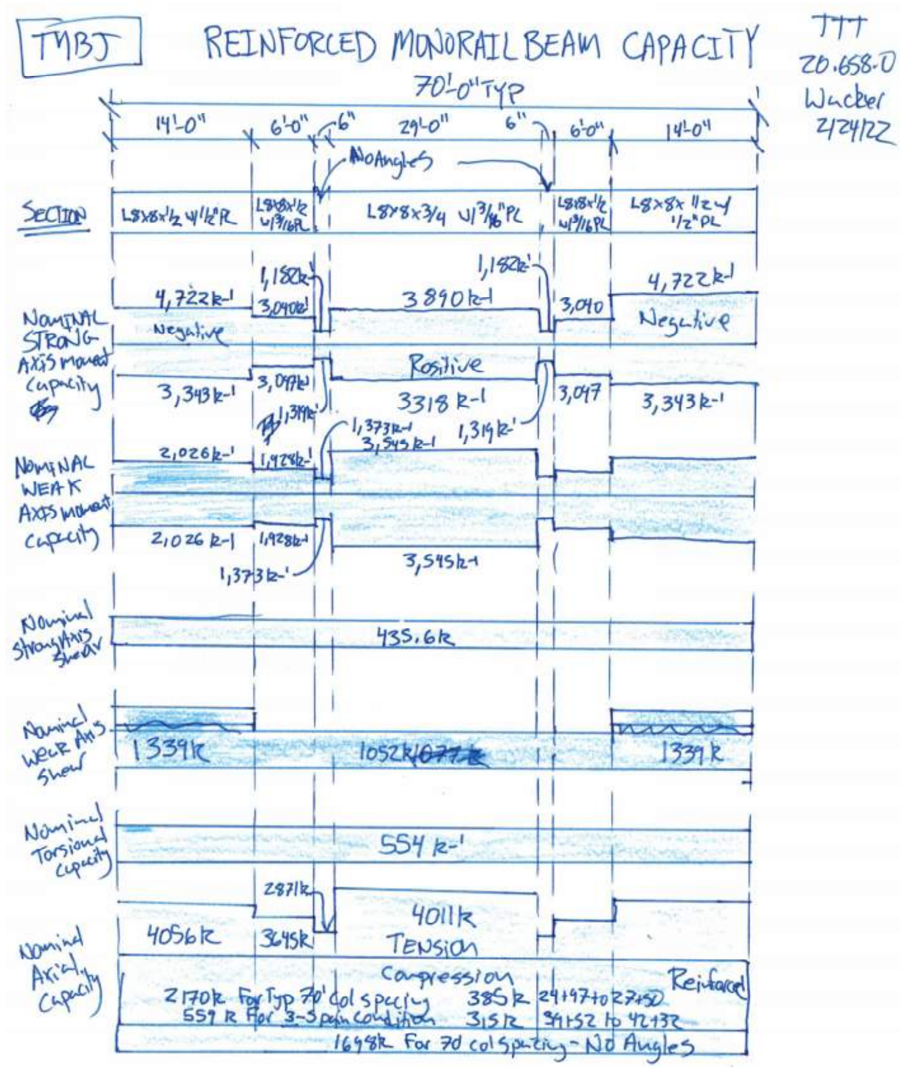
$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

(b) When $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

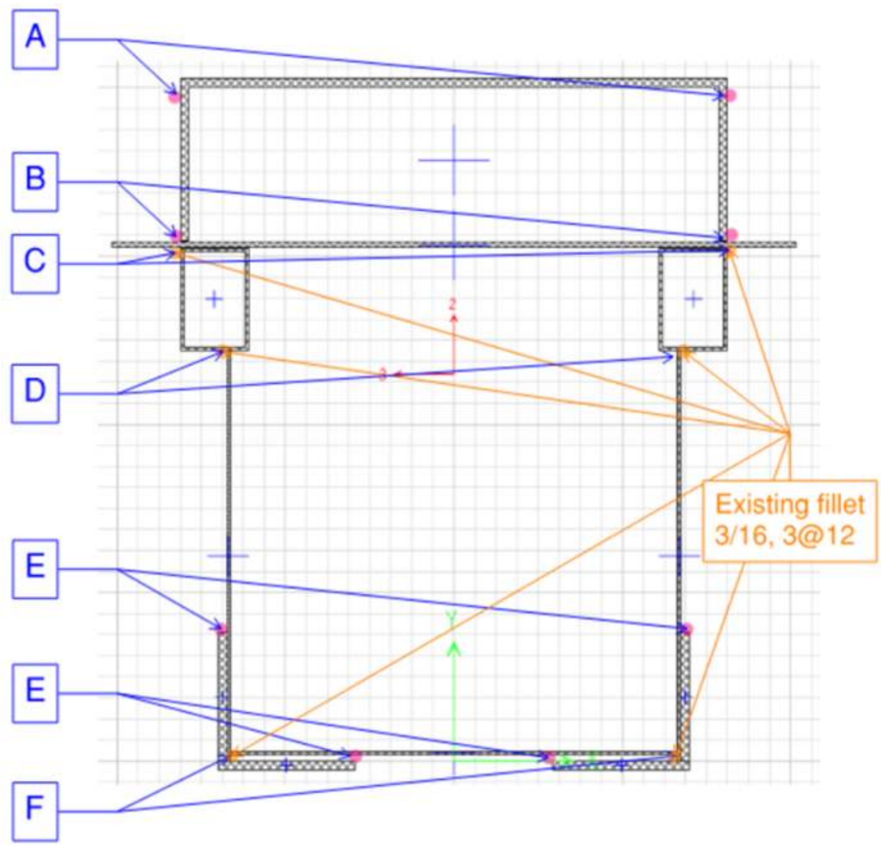
Where Tr > 0.2 Tc:

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \leq 1.0$$



Beam Weld Calculations

- New and existing welds must transfer flexural and torsional shear flows
- q denotes shear flow
  - $q = q_f + q_t$
- For flexure, $q_f = VQ/I$
  - V is shear
  - Q is A*y and changes on each weld
  - I is moment of inertia
- For torsion, $q_t = Tt/C$
  - T is torsion
  - t is member thickness
  - C is torsional shear constant, consistent for the whole section



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CONSTRUCTION CHALLENGES & INNOVATION

BUILDING IN THE WOODS

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BUILDING NEXT TO EXISTING STRUCTURES

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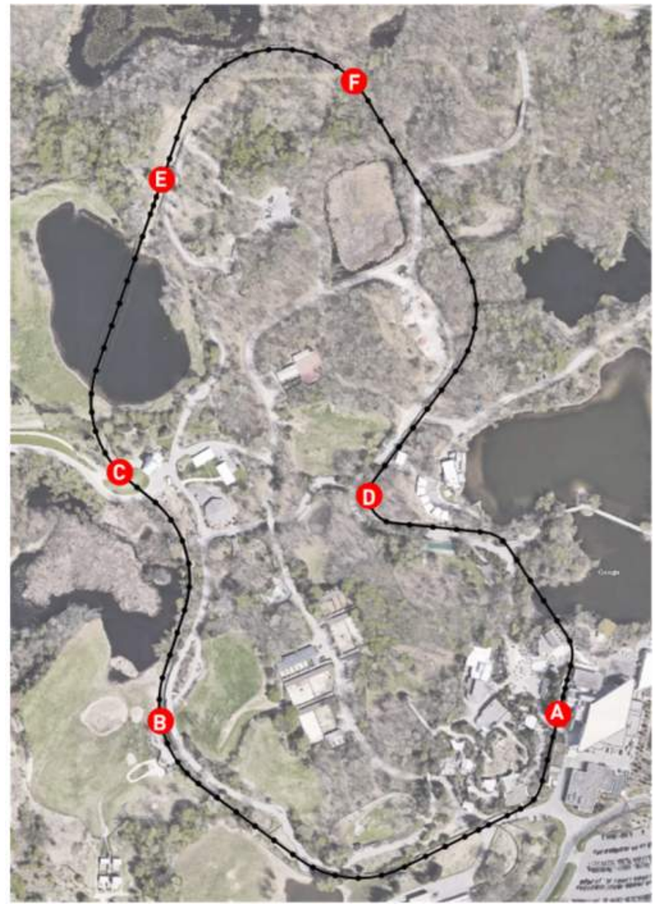
BUILDING OVER WETLANDS & MARSHES



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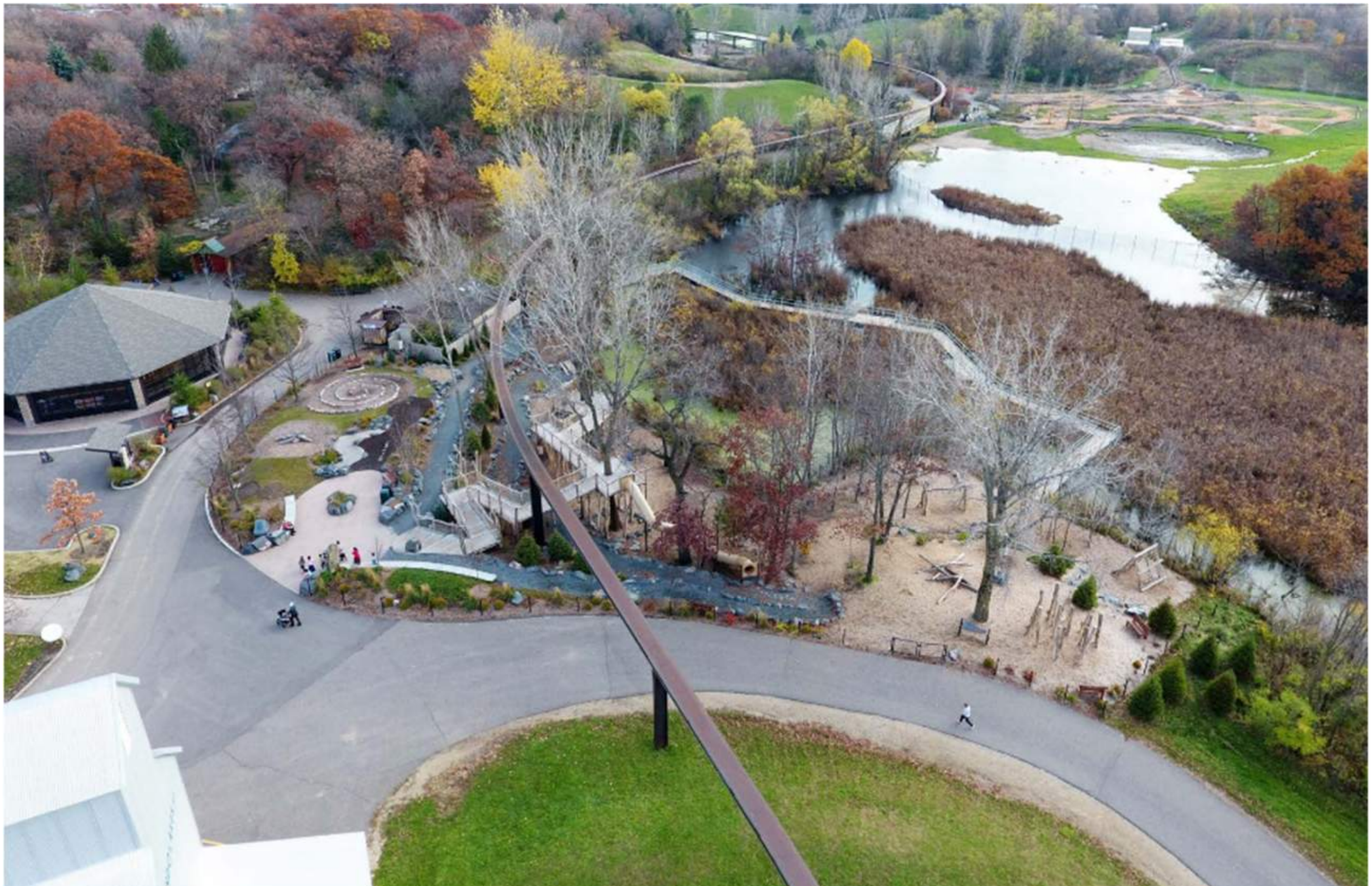
BUILDING OVER GUESTS & EXHIBITS



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BUILDING OVER WATER

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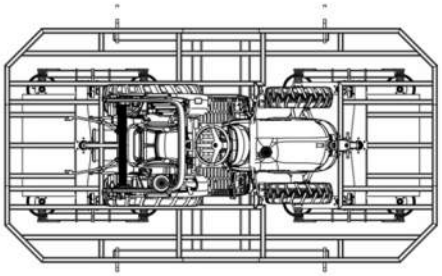
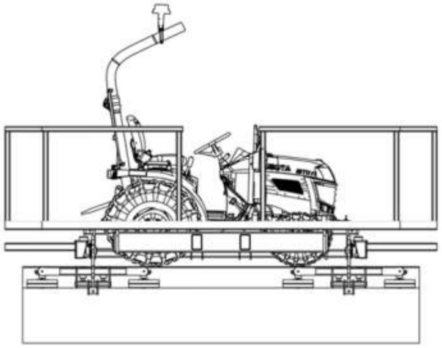
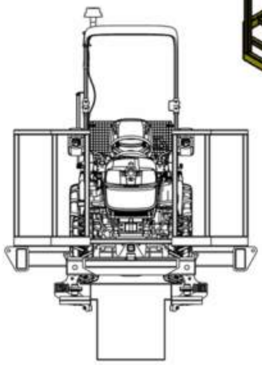
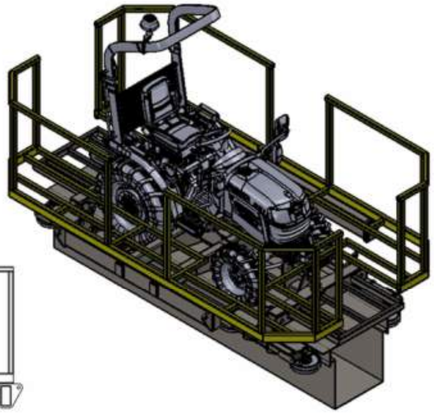
ORIGINAL TROLLEY

THE NEW TROLLEY









MINNESOTA ZOO

BURO HAPPOLD

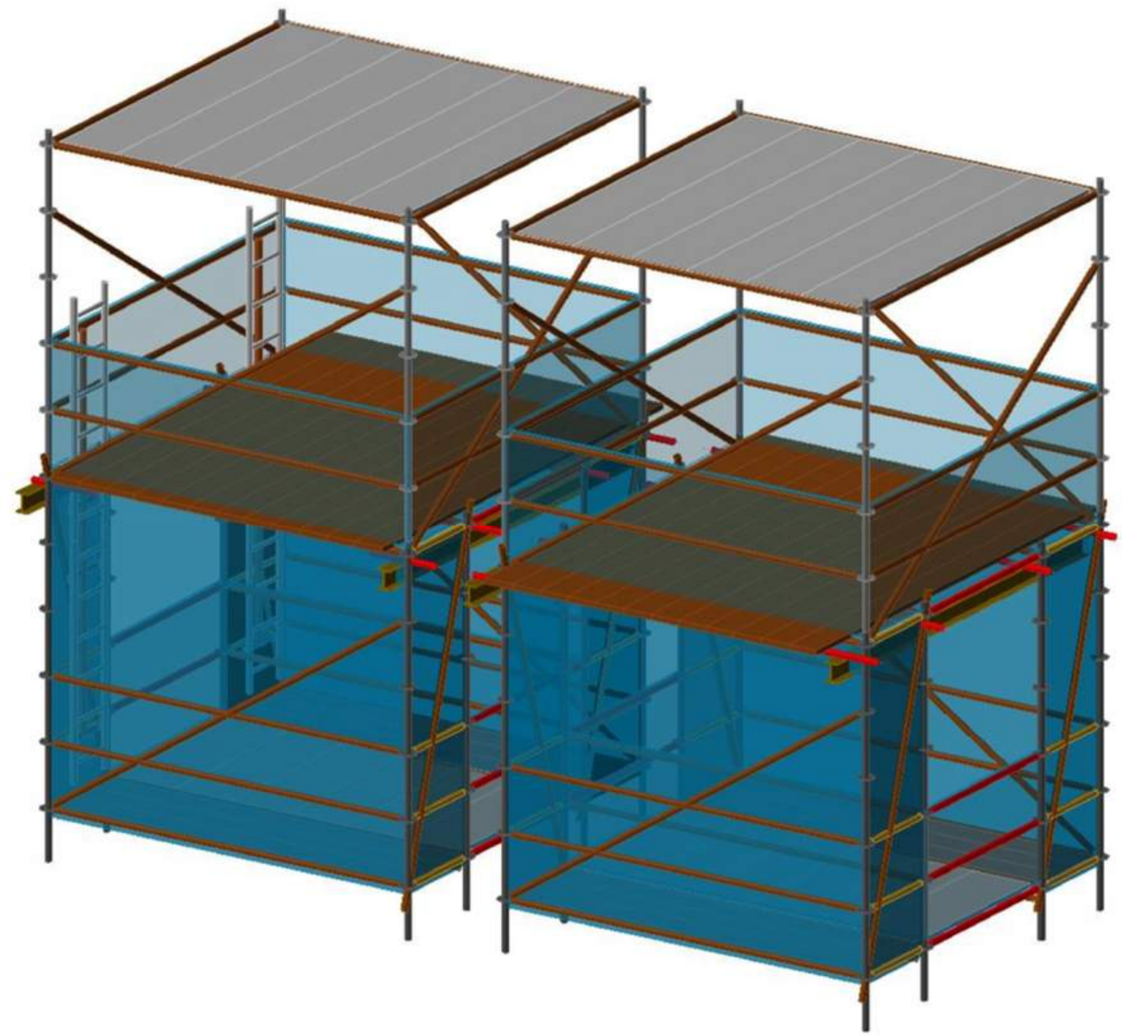
MBJ MEYER BORGMAN JOHNSON

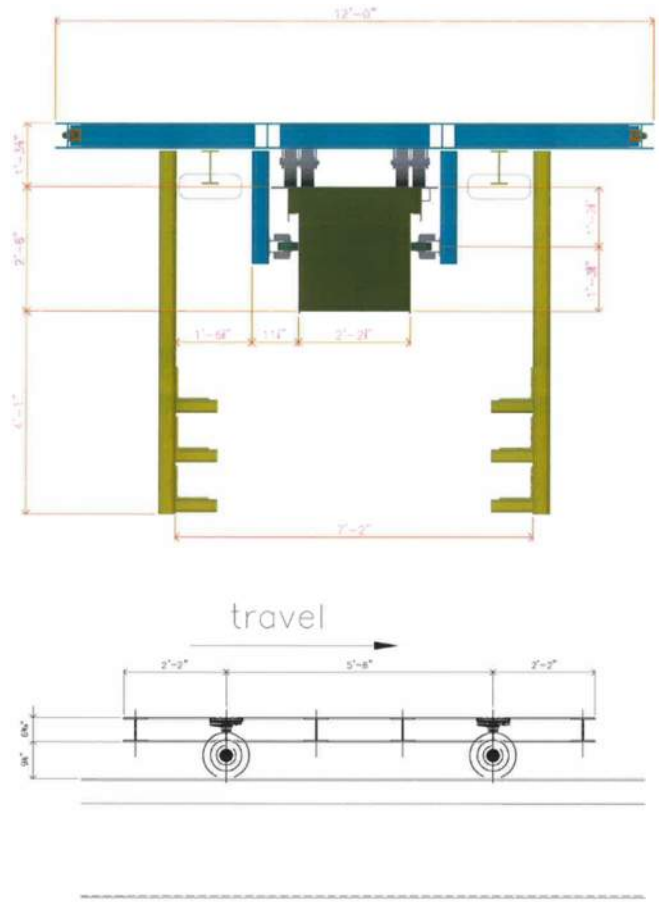
PCL

MINNESOTA ZOO BURO HAPPOLD MBJ MEYER BORGMAN JOHNSON PCL

- **Remove Bus Bars**
- **Test & Inspect the Entire Monorail**
- **Weld & Repair as Required**
- **Remove / Relocate Existing Conduit & Fiber**
- **Install 10 X 10 X 3/4" Stiffener Angle to Underside of Entire Trail**

MINNESOTA ZOO BURO HAPPOLD MBJ MEYER BORGMAN JOHNSON PCL





Section "A"

MINNESOTA ZOO

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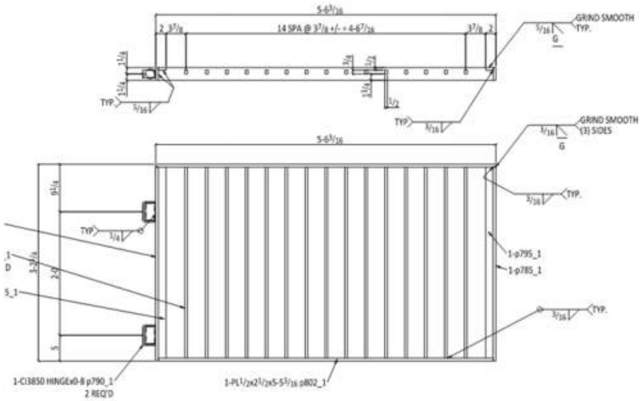
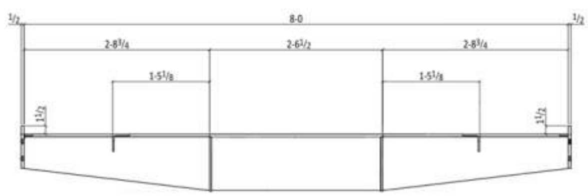
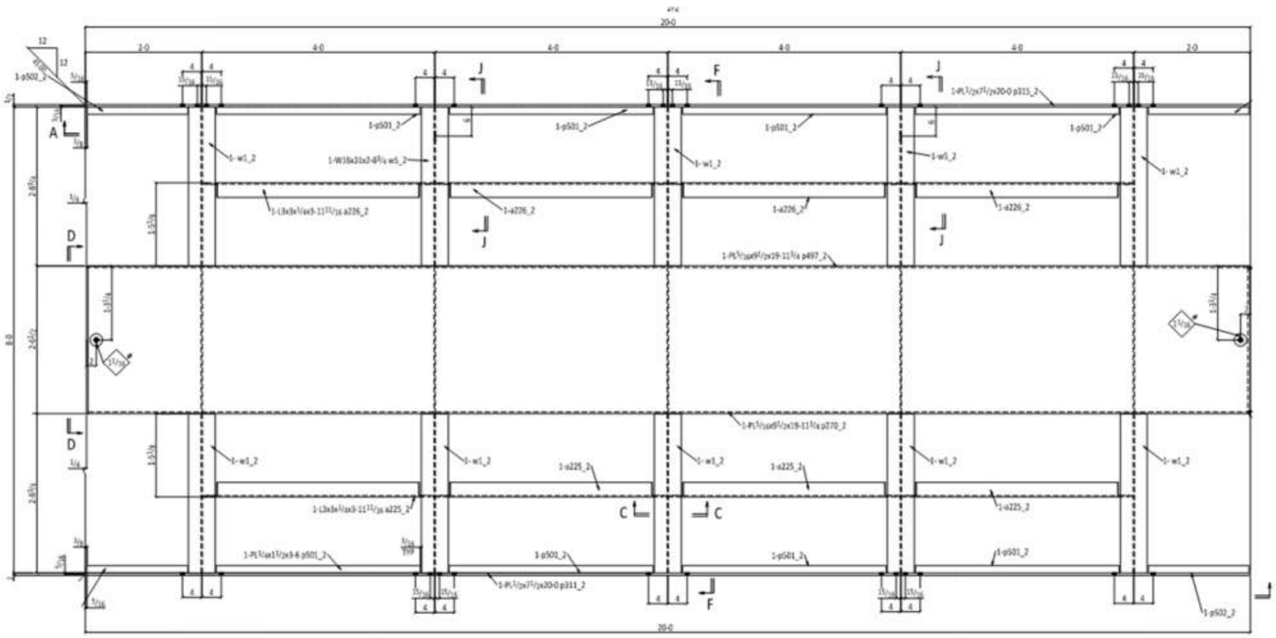
MODULAR SECTIONS



ONE GATE 1G3



Section F-F
2F2

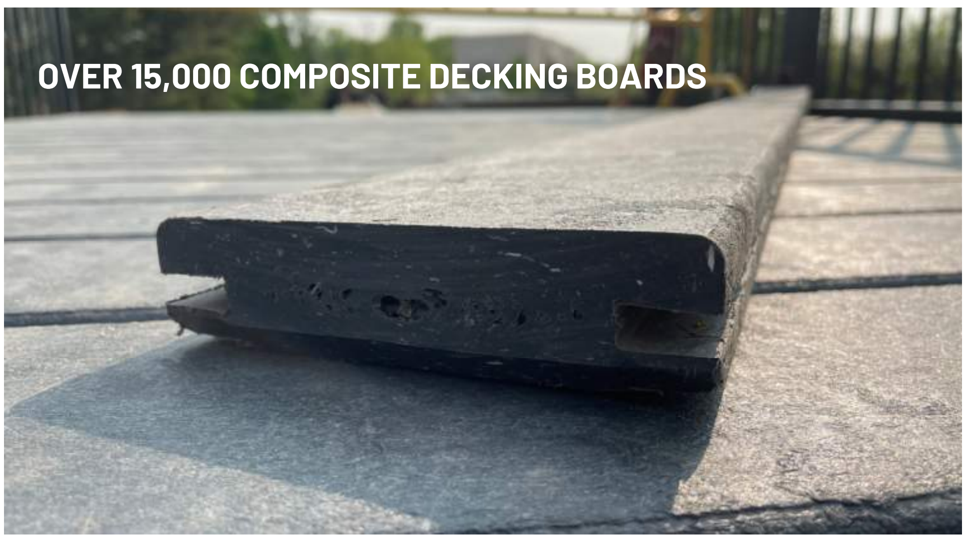


ONE FRAME 2F2

MINNESOTA ZOO BURO HAPPOLD MBJ MEYER BORGMAN JOHNSON PCL

Minnesota Zoo BURO HAPPOLD MBJ MEYER BORGMAN JOHNSON PCL


OVER 80,000 CLIPS & FASTENERS


OVER 15,000 COMPOSITE DECKING BOARDS


OVER 3,700 RAIL SECTIONS

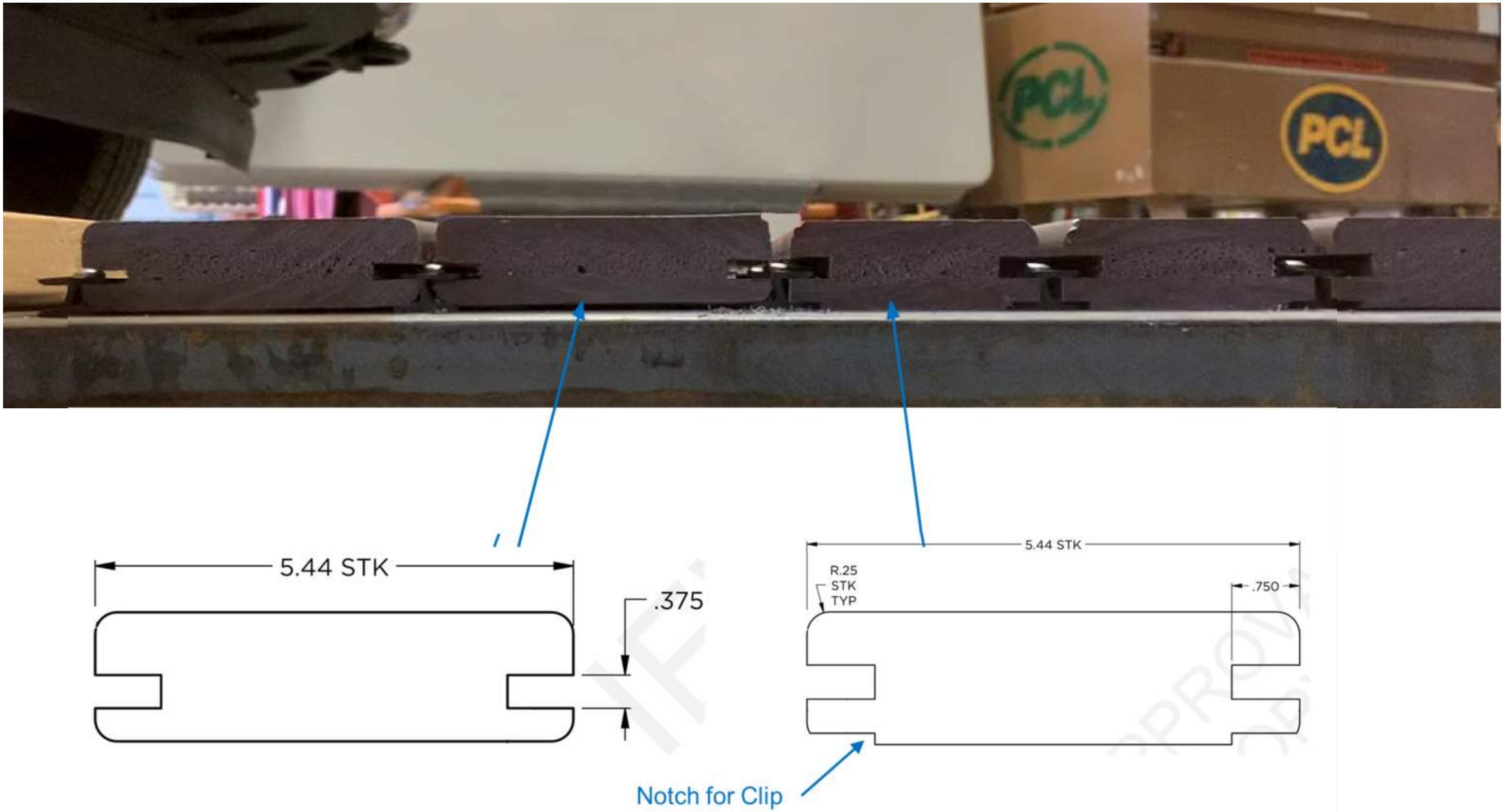
MINNESOTA ZOO · BURO HAPPOLD · MBJ MEYER BORGMAN JOHNSON · PCL

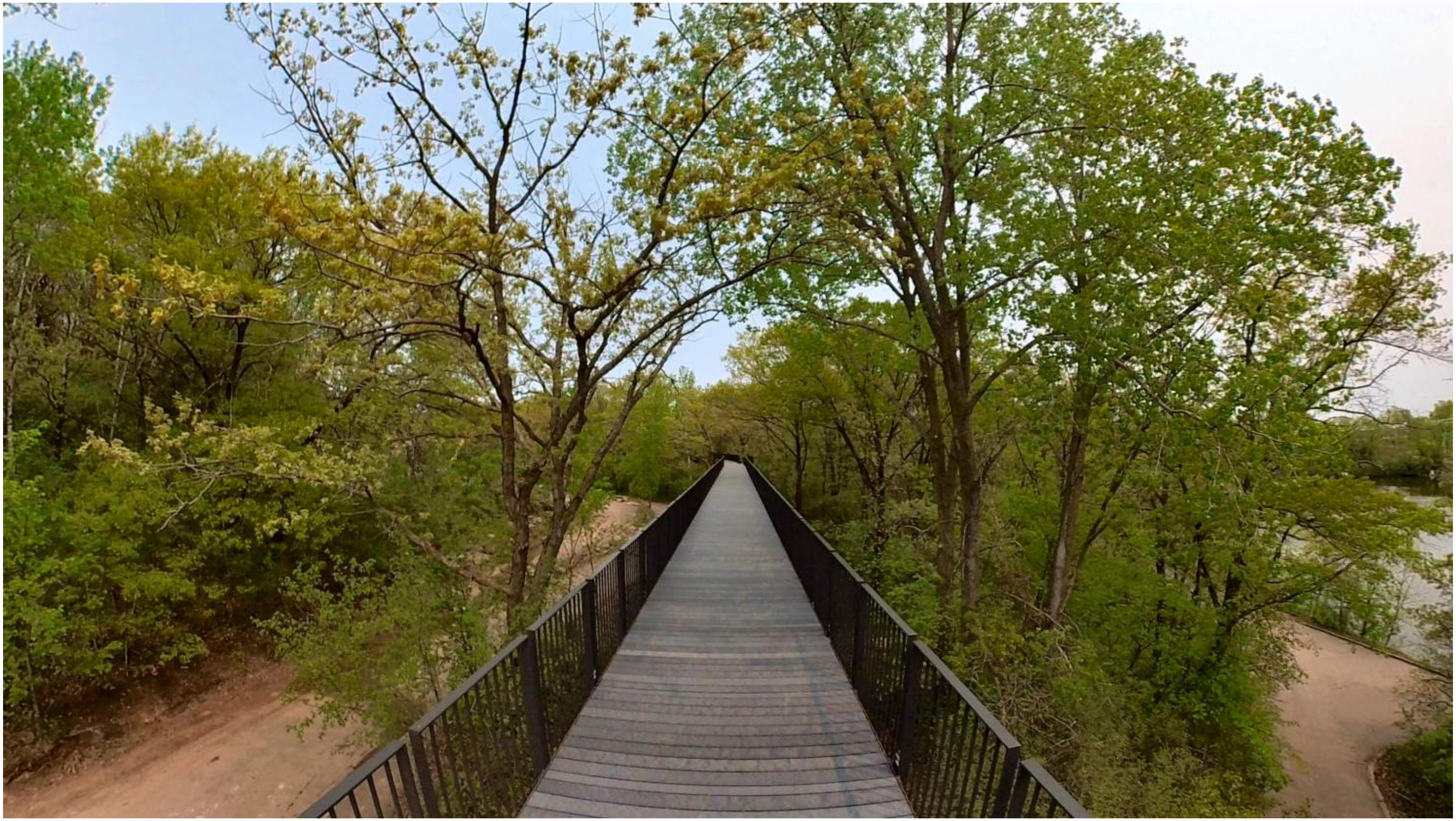
MO0

Slide 162

MO0 Added new video

Michael Osowski, 2023-11-20T14:43:05.898


PCL
5.44 STK
.375
5.44 STK
R.25
STK
TYP
.750
Notch for Clip



MINNESOTA ZOO

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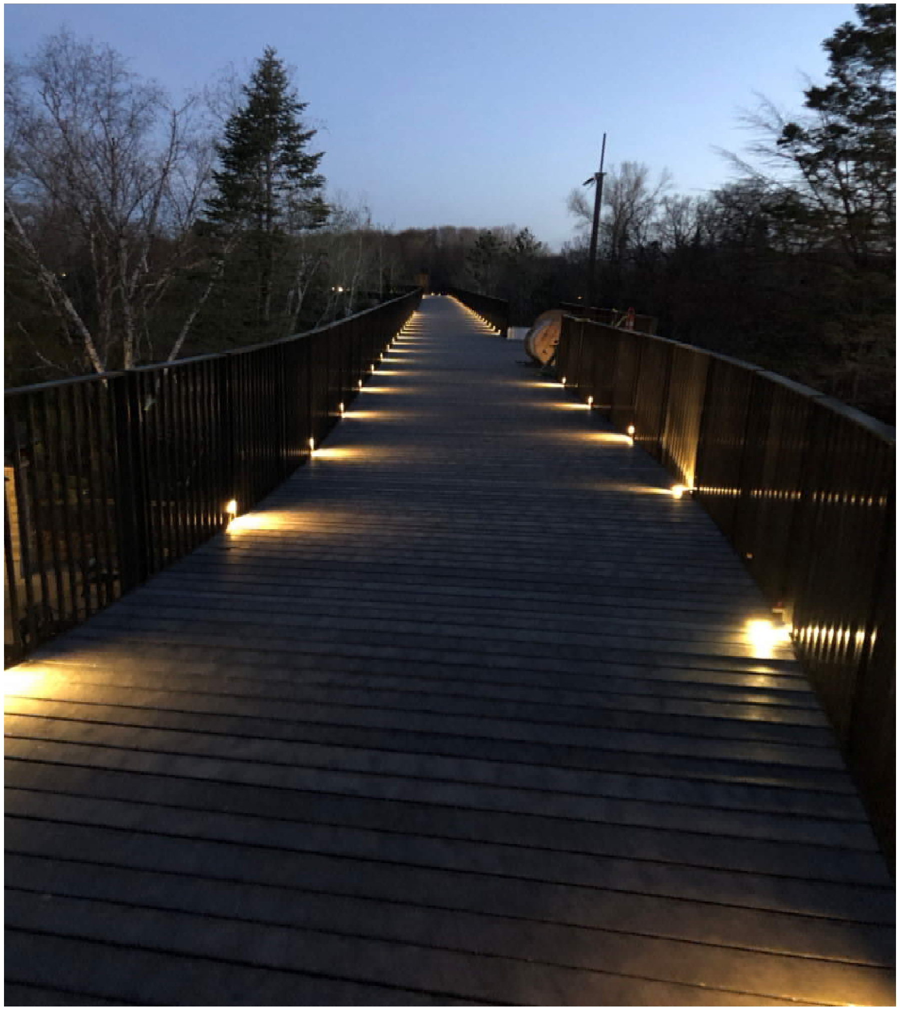

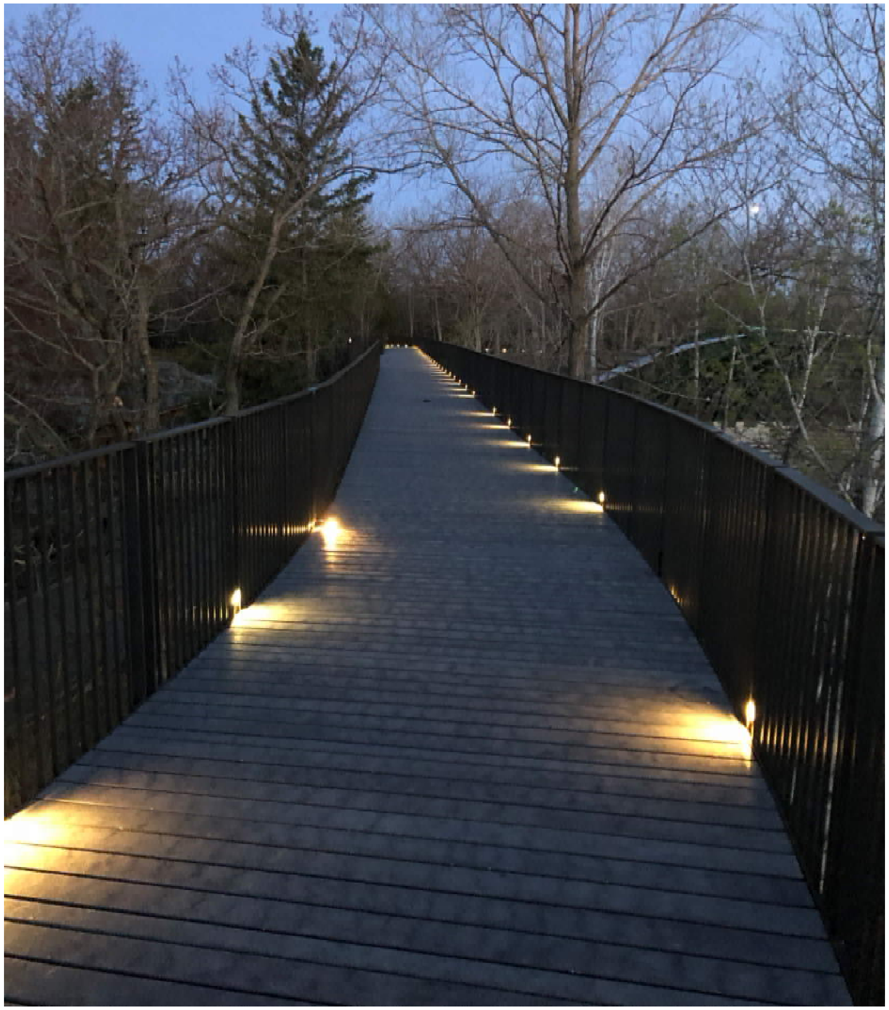

MINNESOTA ZOO

BURO HAPPOLD

MBJ MEYER BORGMAN JOHNSON

PCL

Thank you for your time!

Tom Root, Minnesota Zoo
Fraser Reid, Buro Happold
Jon Wacker,
Craig Huhtala, MBJ
Michael Osowski, PCL

MINNESOTA ZOO

BURO HAPPOLD

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