TRON 2.0: THE EXPERIENCE

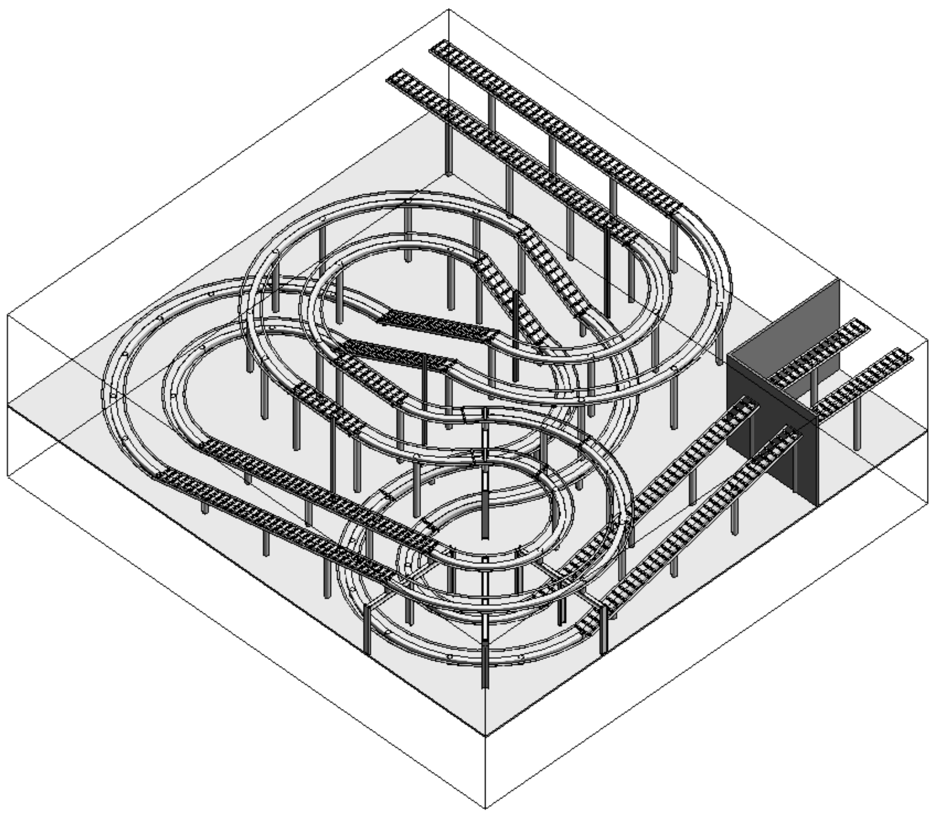

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Table of Contents

Executive Summary

The goal of this senior project was to come up with a structural-engineering-related schematic design for a roller coaster to be installed in Disneyland called TRON 2.0: The Experience. This involved looking at both design processes that we had already learned through our curriculum as well as researching concepts that were previously foreign to us, namely dynamic application of loads and the effects of fatigue stresses.

A story was established based on the source material: the TRON movies produced by Walt Disney Studios. Thematic elements from the story were used as the basis for design implications that would become necessary as the process continued. These design implication influenced the design of the Ride System, Track Layout, and Ride Vehicle Design. These factors would end up influencing applicable loads, which would be utilized in the design of the Track Layout

The site was chosen to be constructed on Disneyland proper so that realistic site parameters and dimensions would give a sense of scale to the project and eliminate ambiguity. However, existing elements on the site would cause challenges for us to work around.

Once the site was chosen, Queue and Show buildings were designated simple designs to tie back to story-driven implications and reinforce the standards set by the Walt Disney Imagineering process model. In addition, we gained more information such as crowd capacity to dictate further mechanical design and dynamic applications.

The mechanical side of the process saw the development of a ride system used to propel the Ride Vehicles, all designed by the Mechanical Engineers on the project. The Ride Vehicle weights became relevant for the design of the track. A Track Layout was created and inputted into simulation software so that dynamic accelerations could be developed as new loading conditions for Track Design.

Finally, all of the preceding elements of design were culminated into a schematic design of a roller coaster track making sure to account for subjects such as dynamic applications, torsion, and fatigue.

All designs were replicated in modeling software to confirm constructability and viability of the resulting schematic design.

Background & Project Overview

For this senior project we aimed to pursue an aspect of Architectural Engineering (ARCE) that we hadn't experienced before. We decided that our knowledge of dynamic load applications and repeated impact loading in particular were viable subjects to expand on. In addition, we knew that we needed to approach the project with some form of interdisciplinary interaction. Lastly, we understood that our interest in the projects of Walt Disney Imagineering (WDI) and the possibility of obtaining positions within the company were important factors to consider; thus, we hoped to establish a project that could serve as material for supplementing our respective project portfolios. The answer, it seemed, to accomplish all of this was to conceptually design a roller coaster attraction to be installed at a Disney theme park.

Firstly, we felt that it was important to emulate the WDI process in a way that would still meet project requirements, considering that WDI was a primary influence on the project. To do this, we began by formulating our own outline of how the project would be carried out. The way we established this process was as follows:

The Idea – It is crucial to the WDI process that an idea be established at the forefront of the project in order to place restrictions on design. These restrictions, in a sense, grounded us to a specific focus and gave us real-world constraints. Recognizing these constraints allowed us to imagine the attraction in an existing space and gave us explicit requirements to meet, further honing our conceptual design to a product for theoretical installation.

The idea we chose to theme our project around was based on the franchise of TRON. The TRON films are a pair of video-game-themed action-adventure movies that were produced by the Walt Disney Company in the years 1982 (TRON) and 2010 (TRON: Legacy). Elements and terminology from the films were applied to the project and are referenced at various times in the process of the design. These terms were not created for this project and are property of the Walt Disney Company.

The idea was chosen on the mentality that the theme of TRON could work as a functioning attraction in a Disney theme park. The attraction ElecTRONica which premiered in 2010, lasted until 2012 and was the inspiration for the project, essentially confirming that TRON would work as a viable and visually interesting subject for an attraction design. In addition, the plans for a TRON-themed roller coaster were announced during the summer of 2015 in preparation of the new Disneyland Shanghai theme park. This further confirmed the notion that TRON could work as a marketable subject for WDI to install seamlessly into one of its theme parks.

The Story – The second half of the idea portion of the project was to develop an actual story for the guests to experience. An element like the story further established the aforementioned design constraints and helped us in later decisions to be made on crowd capacity, ride systems, and necessary building space.

The Site – The site was one of the first aspects to be considered for the building process, and choosing the site was directly correlated to the idea and story in terms of theming. For instance, an attraction that simulates space travel would feel out of place in an area themed to a jungle environment. Once the site was chosen, several requirements were made aware of in the form of soil conditions, available space, and existing elements.

The Queue – The queue was imperative in determining crowd capacity. From crowd capacity we established elements like Ride Vehicle Design, ride cycles, and Track Design. The queue is most effective when placed within a building as it allows for more opportunities to keep guests entertained and protects them from environmental exposure. For this reason, it was decided that a Queue Building would be necessary. However, the Queue Building was given a simplistic design for this project so that we could continue to remain focused on the areas of design that were related to aspects beyond our standard curriculum.

The Show Building – The Show Building, in essence, houses the track. This was decided to be necessary as a part of the theming of the ride as a part of the story that was established early on. We knew that the ride would not work if built outdoors. The Show Building must fit within the site constraints and should be designed to encompass the Track Layout within. Like the Queue Building, the Show Building was given a relatively simple conceptual design.

The remainder of the process was the crucial focus of the project. Our next steps took us past our comfort level and exposed us to resources utilized by our Mechanical Engineering (ME) teammates in tandem with exploring areas of design that weren't covered in our ARCE courses.

The Ride System – The Ride System was essential for determining the "feel" of the ride and more importantly, the points of additional accelerations to be applied to the Track Design. With the ride system chosen, a simulation was ran to figure out the maximum loading points to consider for conservative design decisions. Knowledge of ME-related materials was required to research the ideal models for ride system installation.

The Ride Vehicle – Choosing the proper Ride Vehicle can have a tremendous effect on the service loads to be applied to the Track Design. It has a secondary relationship to crowd capacity in terms of the amount of guests that can board the vehicle at one time. These two aspects are interlocked in their significance when inputted into the ride simulation for the sake of retrieved accelerations. Again, ME participation was required to accurately represent the Ride Vehicle as a digital model.

<u>The Track Layout</u> – Track Layout was established on the parameters set by available space and safety concerns. A certain ride envelope was evoked as to prevent guest interaction with the surrounding structural elements. In addition, required geometries were applied to turn radii and elevation changes to ensure that a healthy level of g-forces will be experienced by any guest at one time. The track layout was modeled in the same way as the Ride Vehicle and served as the project's final point of sole application of ME-related concepts.

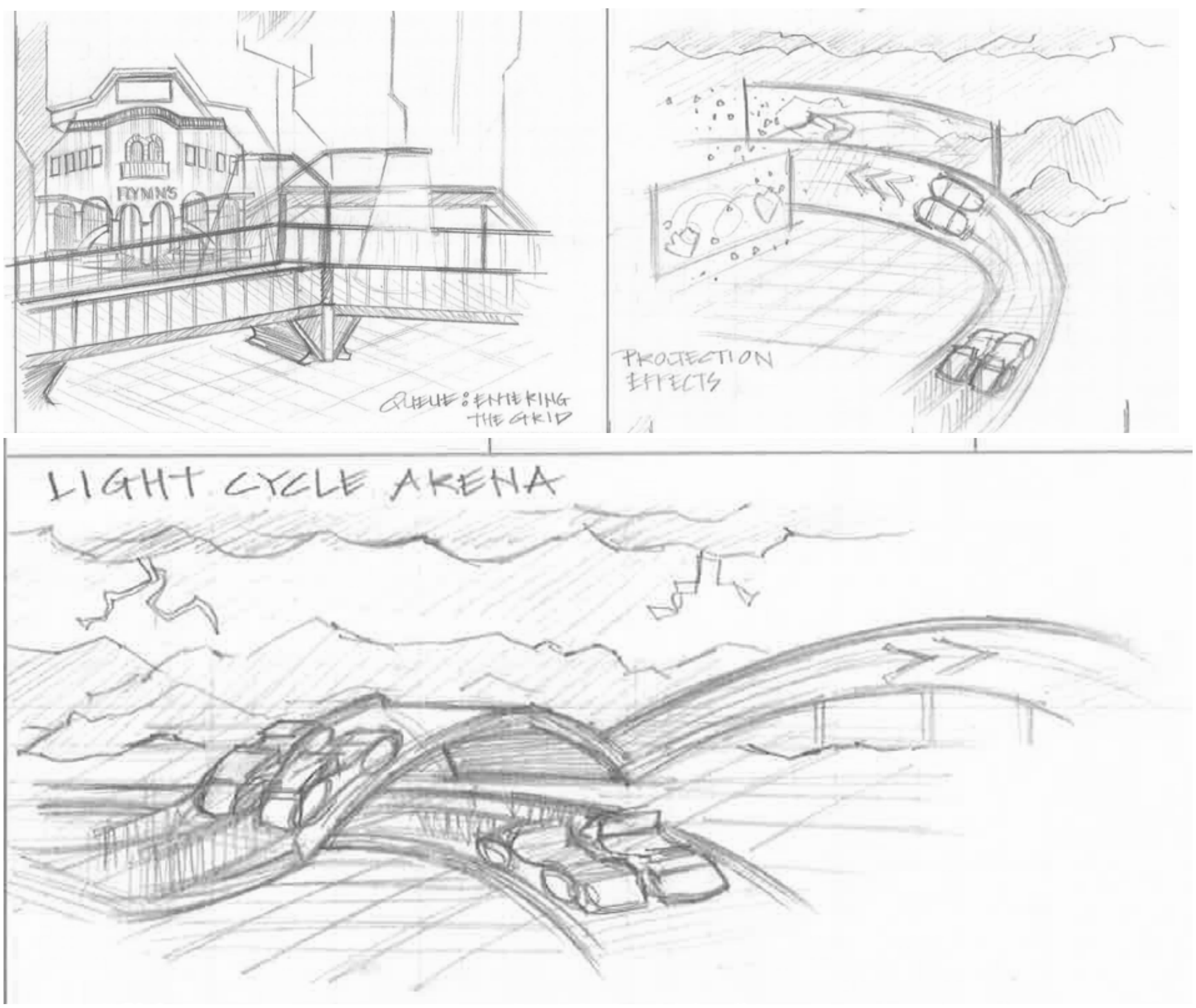
<u>The Track Design</u> – The bulk of this project focused on the design of the track. This is mainly because Track Design was the ultimate merging of the two sides of the process: ME and ARCE. The design considerations as applied in the Ride System, Ride Vehicle, and Track Layout culminated into outputted accelerations. These accelerations were then applied to a designated Track Configuration that carried load through each element and ended with the foundation supports at the base. The track was designed in a way that worked conceptually and is a conservative simplicity of what an actual roller coaster track may be. Connections and maintenance access were not taken into account.

Each part of our WDI-inspired process has been outlined here but can be found explained in more detail in its respective section within this report. We would like to recognize and emphasize that the design of this attraction is purely conceptual and was done so to grasp at a general understanding of what goes into the design of such structures.

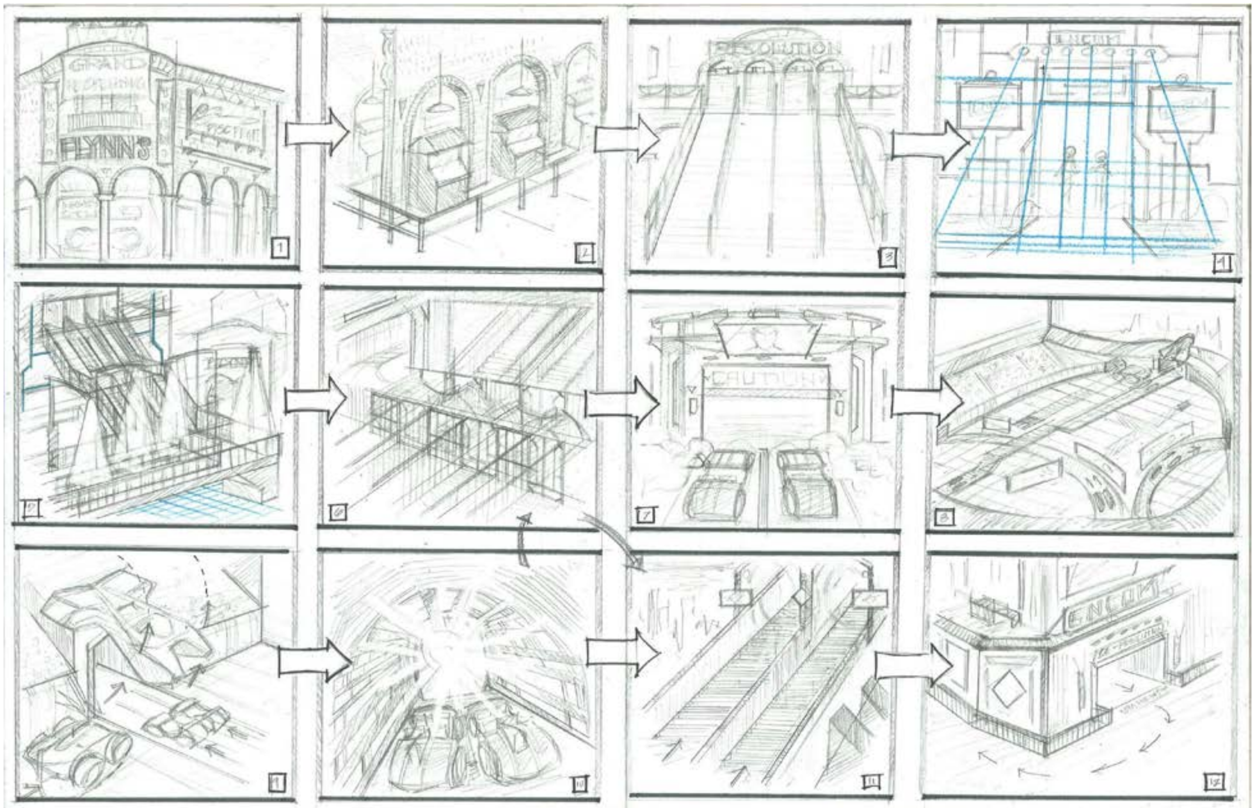
Story Development

One of the crucial elements of the WDI process, story development was applied in the context of this project as a way of retrieving constraints on design. Our period of Story Development was kept necessarily brief as to maintain the true focus of the project. In our approach, we set up a sequence of two pitch meetings where we iteratively evolved the story to its best form. When a story element met the favor of the group, it was advanced to the next level. Conversely, any story elements that were deemed too unrealistic or costly were cut from the project.

To enhance this storytelling process, sketches and storyboarding were helpful in attaching visuals from which design developments could be made.



Left to Right - Top to Bottom - An example of interior space within the Queue Building, an example of a bank turn and incorporation of projection effects, an example of overlapping track levels and side-by-side Ride Vehicle orientation



Storyboard for concept of ride - show elements listed in order of guest experience

This step of the process, although not immediately evident, had large impacts on the design process later on. This development gave us the bases for design, such as ride duration and Track Layout. This sense of realism simulated many of the architectural and aesthetic requirements and served as a guide for the remainder of the design process.

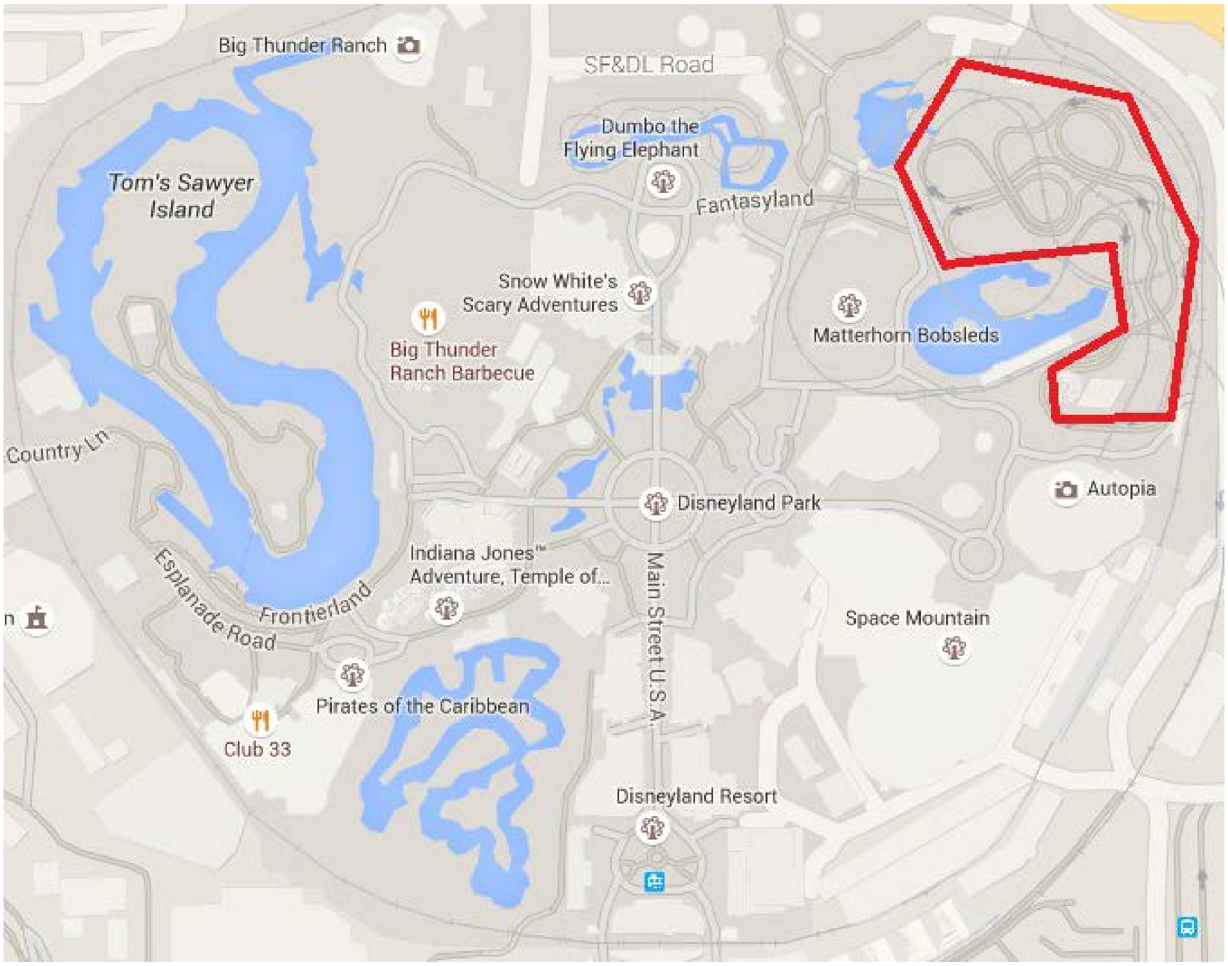
Geotechnical Information

The decision to build on a real-world site allowed for existing geotechnical information to be accessed. However, site information within Disneyland itself was not available for use on this project due to confidentiality restrictions. Instead, information for typical soil in Anaheim was researched through the Anaheim City Website, and a specific site approximately 5 miles southeast of Disneyland was researched through the USDA-NRCS Official Soil Series. All the data retrieved from this research is referenced in the Geotechnical Information section of the report Appendix.

The most useful piece of information to be taken away from this research, as it was essential for the Foundation Design, was the type of soil found on the site: Poorly Graded Sand. Information concerning this type of soil was researched through the Geotechnical Info Website. This information included Soil Unit Weight, the Lateral Earth Pressure Coefficient, the Angle of Internal Friction, and the Factor of Safety. A further description of these factors is referenced on the Soil Parameter portion of the report Appendix.

Existing Conditions

The site chosen for analysis was the current site for the Autopia attraction at Disneyland Park. In terms of size, the site makes up for a large portion of the park as can be seen in the picture below:



Map of Disneyland Park - Red represents the area occupied by Autopia - Google Maps

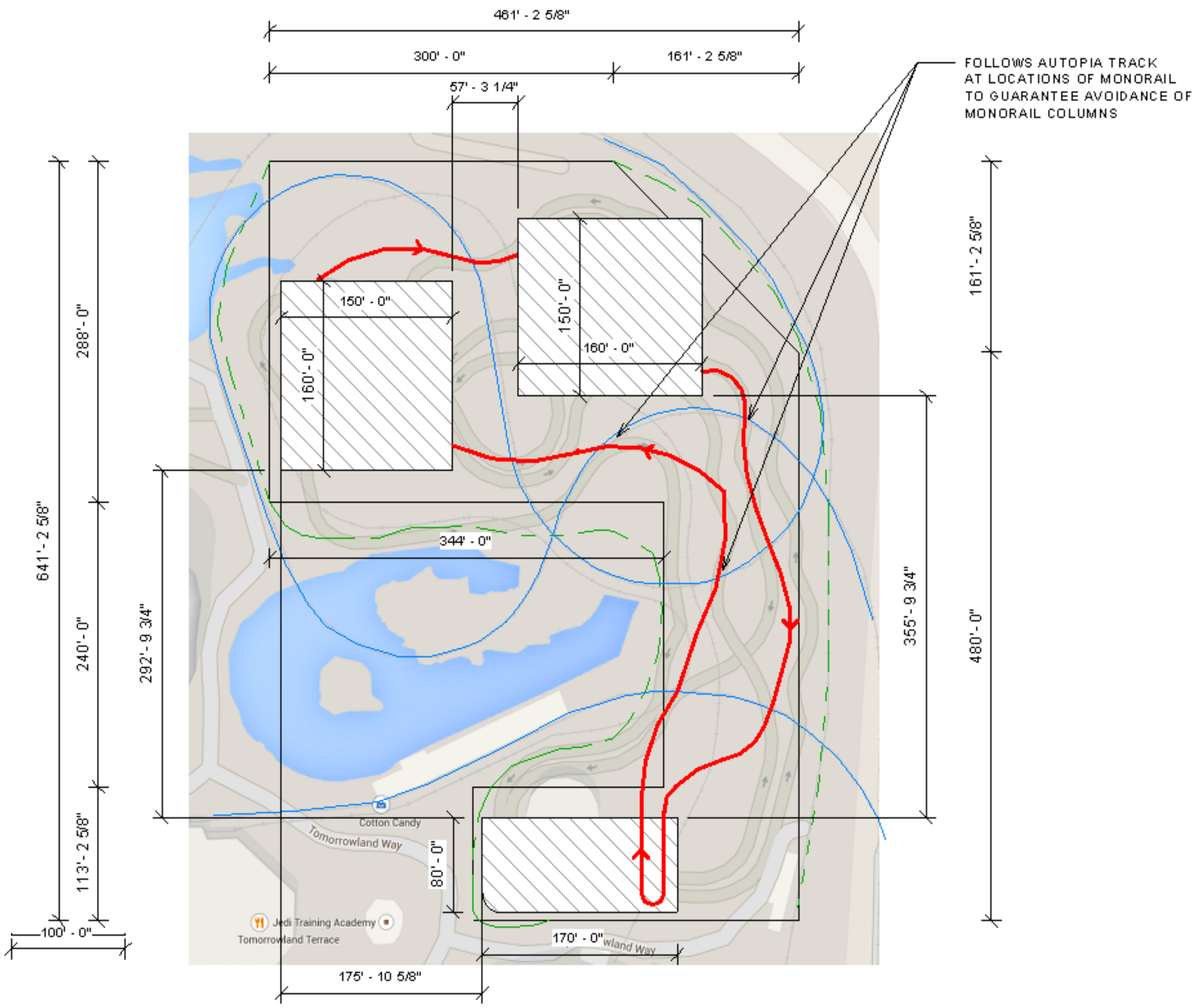
The reason that a real-world site was selected was so that it would present us with realistic parameters and eliminate any unnecessary ambiguity in design. However, this also brought about some new issues to work around. One of the challenges that arose from selecting this site was the characteristic of existing conditions.

Sharing the same space as the Autopia attraction are two structures that cause limitations for building and track placement. These structures are the Peoplemover track and the Monorail track. The Peoplemover track has been defunct for over 20 years and contributes no real purpose to the park. Therefore, we proposed to simulate its demolition to allow for more space for design.

The Monorail, on the other hand, is a key transportation element of the park and would not be able to be removed. Therefore, the Monorail track became an element to be worked around.

The first step taken into consideration was the placement of all three buildings within the parameters of the Monorail. It was decided that the two Show Buildings would need to be dimensioned as 160’ x 150’ and that one building would be a 90° rotation of the other. These buildings would also need to be built into subterranean space to account for easy-to-accommodate architectural aesthetics (particularly keeping in mind the point of view of passengers on the nearby Monorail).

For further information on the design of the Show Buildings see Show Building Design.



Placement of Queue and Show Buildings within chosen site to include established dimensions.
Blue Line - Monorail track
Red Line - Proposed layout of TRON 2.0: The Experience track
Modeled with information from Google Maps

The next aspect of design considered was the location of the columns that supported the Monorail. Information concerning the placement of these columns was not made available for this project and so alternative measures were taken. In order to mark the theoretical placement of the Monorail track, certain areas of the Autopia track were considered. It was deemed that in places where the Autopia track intersected with the Monorail track, placement of new track would be acceptable.

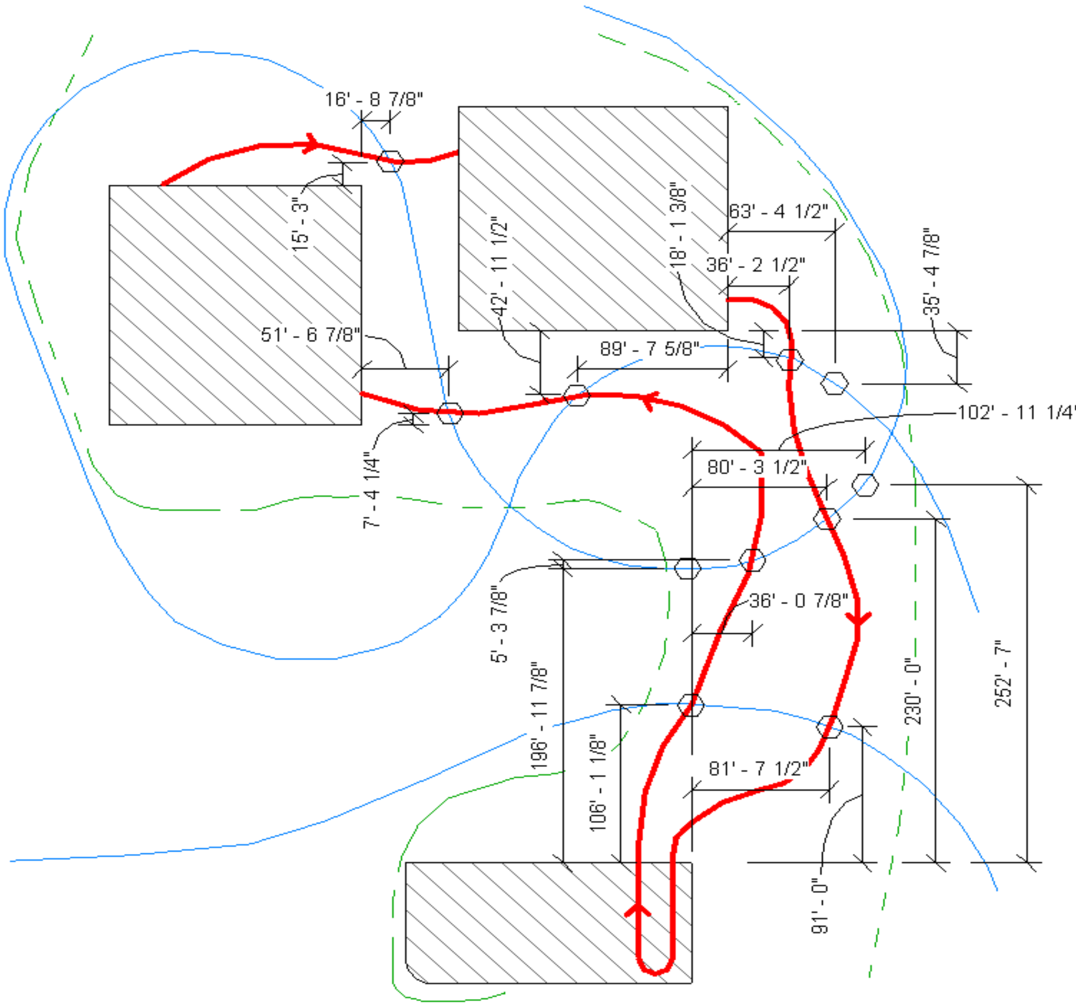


Diagram corresponding to areas of Monorail and Autopia track intersections. The hexagon shapes mark areas where these intersections occur. Modeled with info from *<u>Google Maps</u>.*

Ride Vehicle Design

Concept Generation

The first step in the Ride Vehicle design was to generate multiple concepts based on the Story Development. Potential Ride Vehicle designs were pitched and eventually narrowed down to three basic concepts as listed below:

Light Plane - One of the major vehicles depicted in TRON: Legacy was a Light Plane. This concept would allow for a large Ride Vehicle with a very high passenger capacity.

Light Car - This Ride Vehicle accounted for a more immersive experience through depicted locations within the attraction. The idea was based on the "Light Cycle" model as it appears in the film, but was altered to increase passenger capacity.

Suspended Rail - The Suspended Rail concept was designed to give a unique experience to Disneyland guests. In this concept, passengers would be suspended from a rail with rotational capabilities to allow easy-to-view access to show scenes.

To enforce an unbiased decision-making process, a decision matrix was created to weigh the three Ride Vehicle designs. The first step of the decision matrix was to come up with critical criteria for the Ride Vehicle. The most important factors in this design were: cost, time/complexity, safety, space required, audience/draw, story applicability, high capacity, comfort, and weight. Each factor was rated with a level of importance based on a scale from one (least important) to ten (most important). The ratings for each criteria were averaged to give each criteria a scale factor (see Table 1).

Each Ride Vehicle was evaluated using a second decision matrix and rated on a scale of one (easiest to achieve) to ten (hardest to achieve). The rating was then scaled by the scale factor and the scaled totals were summed for each of the concepts to give a final score (see Table 2). This method proved to be too biased but was rectified by reevaluating each Ride Vehicle on a one to three scale (one was easily achievable, two was moderately achievable, and three was difficult to achieve). The process was repeated to produce more concrete results (see Table 3). Based on the results of the decision matrices, the Light Car concept was chosen for design development.

Factor	Jordan	Alec	Alex	Will	Kelly	Total	Scale Factor
Cost	7	4	2	7	2	22	4.4
Time/Complexity	9	8	9	9	9	44	8.8
Safety	10	6	6	10	10	42	8.4
Space	6	8	7	6	5	32	6.4
Audience/Draw	10	7	6	9	10	42	8.4
Story	2	10	8	6	7	33	6.6
Capacity	6	6	8	10	8	38	7.6
Comfort	7	3	5	9	5	29	5.8
Weight	1	7	6	5	7	26	5.2

Table 1 - Individual Criteria Ratings

Factor	Light Jet	Scaled Total	Light Car	Scaled Total	Suspended Rail	Scaled Total
Cost	3	13.2	1	4.4	5	22
Time/Complexity	6	52.8	3	26.4	6	52.8
Safety	1	8.4	1	8.4	1	8.4
Space	5	32	1	6.4	3	19.2
Audience/Draw	4	33.6	8	67.2	6	50.4
Story	7	46.2	1	6.6	7	46.2
Capacity	3	22.8	3	22.8	5	38
Comfort	3	17.4	3	17.4	2	11.6
Weight	4	20.8	2	10.4	2	10.4
Total Scored		247.2		170		259

Table 2 - Decision Matrix Trial 1

Factor	Light Jet	Scaled Total	Light Car	Scaled Total	Suspended Rail	Scaled Total
Cost	2	8.8	1	4.4	3	13.2
Time/Complexity	2	17.6	1	8.8	2	17.6
Safety	1	8.4	1	8.4	1	8.4
Space	3	19.2	1	6.4	2	12.8
Audience/Draw	1	8.4	3	25.2	2	16.8
Story	2	13.2	1	6.6	2	13.2
Capacity	1	7.6	1	7.6	2	15.2
Comfort	2	11.6	2	11.6	1	5.8
Weight	2	10.4	1	5.2	1	5.2
Total Scored		105.2		84.2		108.2

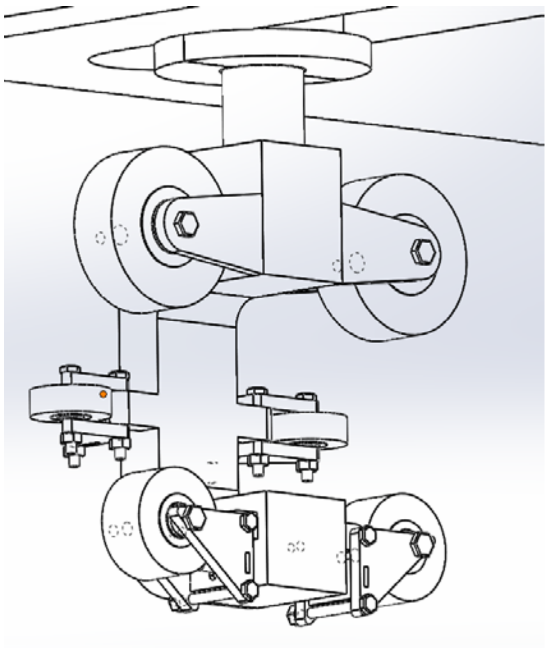
Table 3 - Decision Matrix Trial 2

Design Development

Nomenclature

Vehicle Shell - the structural framing of the Ride Vehicle. The vehicle shell functions as a barrier to keep passengers within the Ride Vehicle during the course of the ride.

Wheel Carriage - element of the Ride Vehicle that is composed of three different types of wheels and the components that hold them together and connect them to the Vehicle Shell.



An example of the Wheel Carriage as modeled in SOLIDWORKS

Up-stop Wheels – wheels that hug the bottom of the Guide Rail* – resist uplift

Tractor / Running Wheels – wheels that hug top of the Guide Rail* – resist gravity

Side Friction Wheels – wheels that hug the side of the Guide Rail* – resist lateral movement

Guide Bar - structural bar that holds the two adjacent Wheel Carriages together one either side of the Ride Vehicle.

LIM - acronym for linear induction motor; a magnetic system that provides propulsion to and causes acceleration of the ride vehicle.

Safety Envelope - theoretical space surrounding the Ride Vehicle that shall not coincide with any structural members (or any other obstructions). A safety envelope is crucial to prevent passengers from making contact with said obstructions.

*for Guide Rail definition see Track Design Overview - Nomenclature

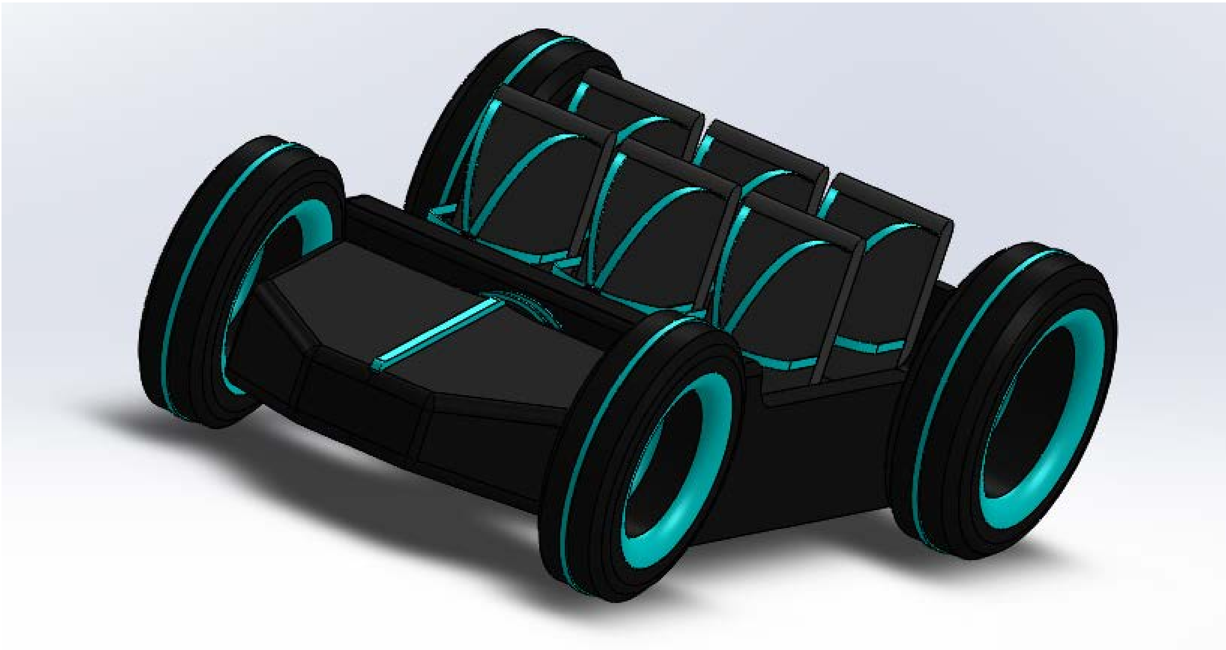
The Ride Vehicle design was broken up into two parts: Vehicle Shell Design and Wheel Carriage Design. A factor of safety of five was used during the initial design because of the high chance of death or severe injury in the event of failure. In a second phase of design, the factor of safety could be reduced to lower the overall weight and construction costs of the Ride Vehicle. The Ride Vehicle was designed in a program called SOLIDWORKS, a computer-aided engineering software program and solid modeler that utilizes a parametric feature-based approach to create models and assemblies.

Vehicle Shell Design

The primary goal of the Vehicle Shell (See Nomenclature) Design was to develop a comfortable Ride Vehicle that was simultaneously lightweight and strong. The Ride Vehicle was modeled after the "Light Cycle" in TRON: Legacy and thus consisted of two parts: a nose and a body. The nose was purely aesthetic and thus was designed to be hollow. The body was lengthened to provide enough space for two rows of three people in order to maximize passenger capacity. The total dimensions of the car (including exterior aesthetic elements) were around 10' x 5' x 16', and the size of the seats was designed based on Anthropometric Data (see Figure 10 - Appendix). The seats themselves were composed of black leather and filled with high-strength foam to provide comfort for the passengers while maintaining durability. A racecar style seat belt was used to keep passengers firmly locked into their seats while maintaining comfort. The seats were mounted to a metal frame that attached directly to the Wheel Carriage to ensure that if the plastic frame of the Ride Vehicle failed, the seats would remain attached to the Wheel Carriage.

Wheel Carriage Design

The Wheel Carriage (See Nomenclature) followed a traditional design: it consisted of two pairs of Running Wheels (to carry the downward forces), two pairs of Side Friction Wheels (to carry the lateral forces), and two pairs of Up-Stop wheels (to carry the upward forces). Each Wheel Carriage pair was connected with a Guide Bar (see Nomenclature). The Wheel Carriage was created in SOLIDWORKS, and each component was assigned a material so the weight of the Wheel Carriage could be calculated. Once the Wheel Carriage and Vehicle Shell were completed, they were combined in an assembly so that total weight of the Ride Vehicle could be calculated. The thickness and dimensions of the Wheel Carriage were sized based on the simulated weight, and it was assumed that the top 95th percentile of the population would be able to board the Ride Vehicle (see Figure 10 - Appendix). Critical points, such as the connection points in the wheels and Guide Bar, were checked for shear and bending failure. The locations where the Wheel Carriage was attached to the Vehicle Shell were spaced such that the weight of the passengers was evenly distributed amongst the two wheel carriages.



Completed Ride Vehicle Design as modeled in SOLIDWORKS

Track Layout Design

Column Placement

The largest limiting criteria for the track design was the column placement. Coordination between the ME and ARCE teams allowed the column placement to be large enough to fit two cars side-by-side while maintaining the 3' Safety Envelope (see Nomenclature) around each Ride Vehicle. To minimize column placement, certain design requirements were enforced for the Show Building (see Show Building Design).

Multi-Story Elevations

To simulate the story element of a multi-level arena experience as seen in TRON: Legacy, the Track Layout was designed to fluidly move between two stories that were located 20' apart in elevation.

Turn Radius

To ensure that the ride would be safe for small children (ages 10-12 depending on their weight), the turn radii of the Track Layout were kept as large as Show Building geometries would allow. When it was necessary for the turn radius to be small, the speed of the Ride Vehicle was limited to minimize the gravitational forces experienced by the passengers.

LIM Placement

It was decided that the Ride System would utilize a combination of linear induction motors (LIMs) and gravity for the propulsion of the ride vehicle. Due to the placement of the Monorail supports and track, the designed Track Layout had to remain low to the ground (see Site Analysis - Existing Conditions). The height and space limitations imposed by the Monorail made it problematic to use solely gravity because there was no substantial source of potential energy. LIMs were chosen as the alternative because they allowed the Ride Vehicles to remain low to the ground and had the added bonus of simulating the smooth acceleration of a "Light Cycle."

Safety Envelope

A Safety Envelope (see Nomenclature) of 3' around each of the Ride Vehicles was maintained to ensure that passengers could extend their arms upward or outward and not hit anything. This Safety Envelope was critical in column placement and was the determining factor for bay spacing in the Show Building.

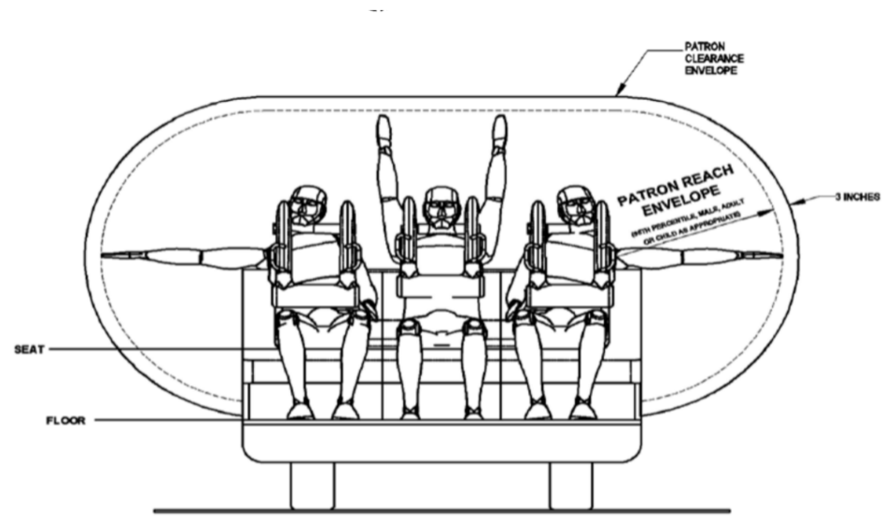


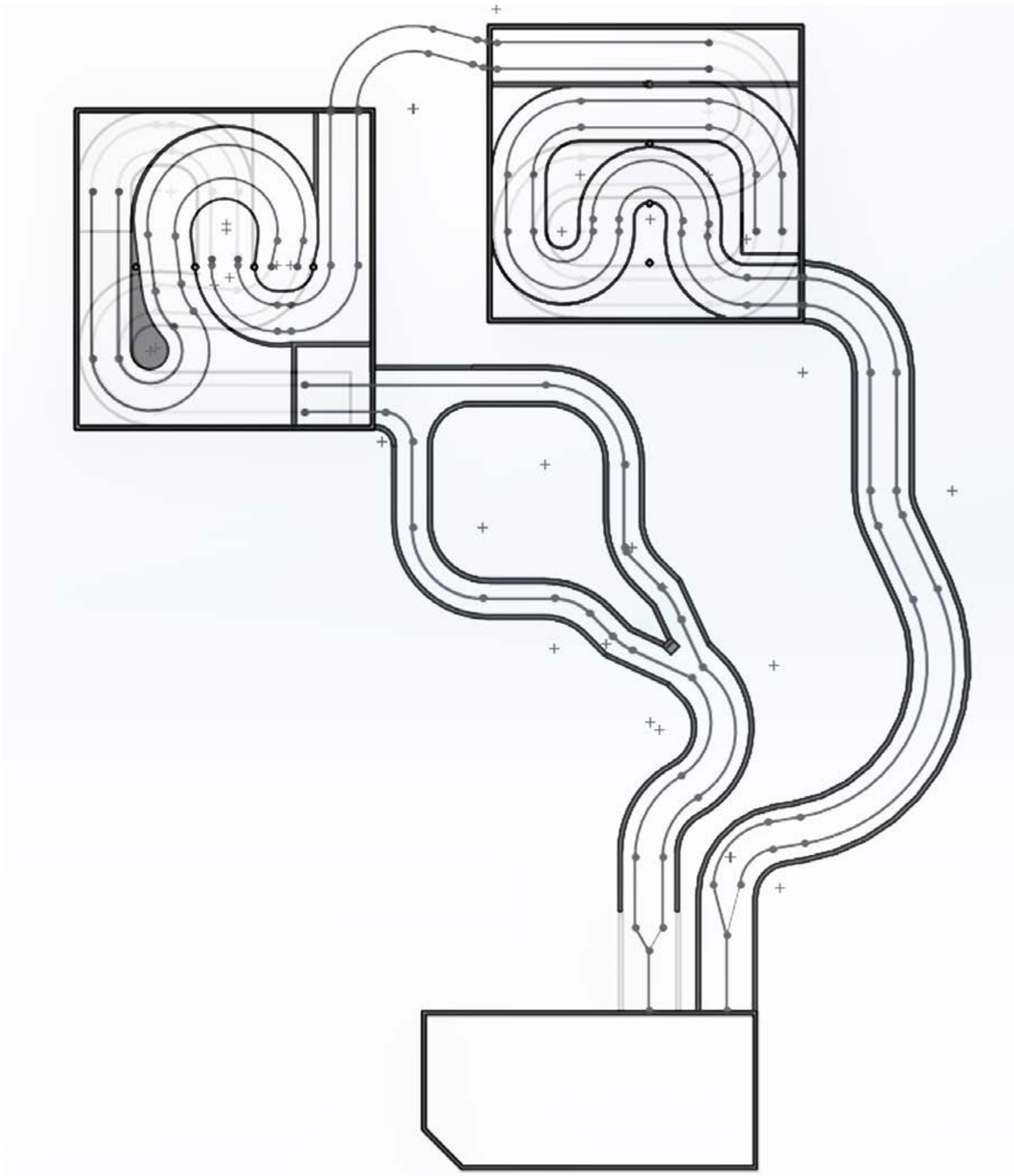
**FIG. 4 Sample Patron Clearance Envelope Illustration
Front View Configuration**

Safety Envelope as defined by *<u>ASTM F2291 – 13 Standard for Amusement Ride Design</u>*

<u>Simulation</u>

Once the SOLIDWORKS track design was completed, a Track Layout was modeled in the roller coaster simulation software, NoLimits 2. NoLimits Rollercoaster Simulation 2 (NoLimits 2) is a rollercoaster simulation software package for Microsoft Windows. NoLimits 2 is a cost-effective alternative that outputted all necessary information for dynamic analysis.

From here, the maximum gravitational force of the ride was determined to be 2.5 Gs (2.5 times the force of gravity). However, the NoLimits 2 could not model banked turns, which would have significantly reduced the gravitational forces. The maximum speed was found to be approximately 28 mph.



Completed Track Layout as modeled in SOLIDWORKS - includes Queue and Show Buildings

Queue Building Design Overview

The term queue refers mainly to the area encompassing the formation of passengers as they prepare to board the Ride Vehicles. Whereas the term *line* indicates the passengers themselves, the *queue* is in actuality what envelops the *line*. A typical queue serves multiple purposes. First and foremost, it is meant to establish a sense of crowd control. A well-designed queue guides guests in an orderly manner whilst covering maximum space in order to maximize the number of people that can be loaded onto the attraction as quickly as possible. In this respect, a queue should also be diverse and have many changing directions to remain interesting, rather than remain in one continuous orientation as this makes the line seem longer than it is.

A factor that is important to WDI but not especially relevant to this project is that the queue also portrays some aspect of the story. For this project, we chose a building shape and aesthetic that most accurately represents one of the main buildings seen in both versions of the source material; this way, the building appears relevant to guests but also gives us some design parameters to consider. For instance, the building as it appears in both films has a brick finish on the exterior; so, when developing gravity loads for the exterior members, we took into account a brick veneer as additional weight to the structure. In addition, linoleum floors, as they appear in both films, were considered for floor weights.

Concept art of Queue Building- Used to dictate design decisions

The final design element worth mentioning is the choice to build in subterranean space. With the dimension constraints defined in the site design (See Existing Conditions), we knew that there was a limited amount of space to work with. In order to maximize ride occupancy and to allow for separate loading / unloading locations, it seemed necessary to develop an underground space. By establishing a basement level portion of the queue, we could space the line out further and help regulate the amount of passengers boarding the ride at one period of time. For further information on the layout of the Queue Building, see Revit Modeling.

General Assumptions

1. All design was in accordance with Load and Resistance Factor Design (LRFD) Loading Configuration.
2. Live loading conditions were based on the ASCE 7-10.
3. The roof materials (vapor barrier, insulation, and fireproofing) selected for the roof design were based on standard roof materials (see Fig. 1 & 2 - Appendix).
4. All decking was chosen to be Verco brand due to our easy accessibility to said catalog.
5. The weight of Mechanical, Electrical, & Plumbing (MEP) Systems was assumed to be 5 psf.
6. A suspended T-Bar ceiling was selected to allow space for HVAC and MEP components.
7. A brick veneer was included to simulate the texture as reflected in the architectural concept.
8. Miscellaneous loads were taken as 2-5% of the total load.
9. Linoleum was chosen as a floor finishing material to simulate texture as defined in architectural concept.
10. All columns were designed to be pinned-pinned connections for simplicity of design.
11. Walls at the basement level are considered to be concrete retaining walls and were not designed due to not being within scope of project.
12. Foundations and connections were not designed due to not being within scope of project.
13. Seismic Design Considerations were based on the ASCE 7-10.
14. The lateral force resisting system chosen for the ground floor of the structure is a braced frame configuration due it being a cost effective solutions and coincided with architectural considerations.
15. Due to the slope of the roof being slightly sloped for rainwater considerations, the total roof height was conservatively considered 24’ in all locations. Braced frames were modeled as 20’ high to account for the 4’ parapet on the south / west orientation.

Queue Building Design

Gravity Loading Components - Roof

Firestone V-Force Waterproofing Membrane

1" Rigid Insulation

Vapor Liner Sheet

Spray Fireproofing

For selection of these materials, see Assumption 3.

18" Verco Decking - PLB-36 / HSB-36

The live and dead loads up to the deck were combined to retrieve an applied gravity load. Typical deck span was also observed. With this information, a 22 Gauge PLB-36 Verco Deck was selected from the Verco Catalog (See Assumption 4).

MEP (See Assumption 5)

Suspended T-bar ceiling (See Assumption 6)

Brick Veneer Finish – Exterior Walls

Brick veneer walls have an interior composed of a frame of steel channels attached to a layer of drywall (See Assumption 7). Weights of the channels, drywall, and brick veneer were totaled.

Gravity Loading Components - Floor

Linoleum Floor Finish (See Assumption 9)

Normal Weight Concrete Fill

Verco 20 Gage PLW2 or W2 Formlock Deck

The live and dead loads up to the deck were combined to retrieve an applied gravity load. Typical deck span was also observed. With this information, a Verco 20 Gage PLW2 or W2 Formlock Deck was selected from the Verco Catalog (See Assumption 4).

Design Process – Gravity Systems

Materiality was the first aspect of design established. Steel was chosen for its ease of use in creating open environments and creating thematic parallels to the source material.

The design for the gravity-resisting systems was split into several categories.

First, a system was designed for the roof level and the floor level (over the basement space). Then, in each system, structural elements were divided into interior and exterior members. Of these members there were three types: beams, girders, and columns. Beams spanned longer distances than the girders. Columns varied in height between floors and extended to the basement level. Foundations and connections were not designed (See Assumption 12).

To begin the process, all loads (dead and live) over the tributary area were applied as a distributed load to the first structural member (roof beam). The loads were factored for LRFD conditions (See Assumption 1). Live load was analyzed for roof reduction according to the ASCE 7-10 (See Assumption 2).

The applied loads were analyzed for bending and shear demands according to the specified equations (see Gravity System Design - Queue Building - Appendix). Deflection demand was also calculated for maximum controlling live load conditions.

An initial steel member was selected from the AISC Steel Construction Manual to be evaluated for capacity. Using the given equation for plastic moment (Mp) based on yield stress (Fy) and section modulus (Z) and multiplied by a reduction factor (φ), bending capacity was determined. Using the given equation for shear (Vn) based on Fy and the area of the section and multiplied by φ, shear capacity was determined. Lastly, a controlling live load deflection was calculated using the equation specified for a simple span beam (see Gravity System Design - Queue Building - Appendix).

All capacities were compared with demands to confirm that the selected member would work for design. An additional check was performed to establish whether or not a camber of the member was required.

The maximum shear force was taken from the beam analysis and added to the self-weight of the beam to be used as the demand load for roof girders. These girders were designed using the same process used for beam design, with differences occurring in the loads inputted and the length of the members.

All floor members were designed with the same methodology used for roof members with the exceptions of weights applied (See Gravity Loading Components - Floor).

Columns spanning from the roof to the ground floor were then sized. Dead and live loads were based on the tributary area of the columns and factored based on LRFD; live load reductions were also considered (see Assumptions 1 & 2). An effective length factor (K) of 1.0 was chosen (Assumption 10) to determine the effective length of the column. Based on these criteria, columns were sized using Table 4-1 in the AISC Steel Construction Manual.

Design Process – Lateral Systems

In order to design the lateral system, base shear was first determined. Seismic lateral loads were considered the governing case over potential wind conditions based on the seismic conditions of the area. The latitude and longitude for our chosen site was inputted into the USGS Website. From this database, we retrieved seismic design response spectra accelerations (see Fig. 5 - Appendix). Factors of Response Modification (R), Importance (I), and Risk Category were retrieved from the ASCE 7-10 based on the parameters outlined in our site location and materiality of design. Next, period and the seismic response coefficient were calculated from the aforementioned factors retrieved from ASCE 7-10. Seismic weight was taken as the weights of the roof and wall cladding multiplied over the respective area of coverage. Lastly, base shear (V) was calculated as the multiplication of our seismic weights with the selected governing seismic response coefficient.

Even though only two braced frames were required in each direction, a total of four braced frames was used in each direction to increase redundancy, and were used to divide up the base shear into corresponding lateral loads. The frames were modeled in RISA with applied distributed loads and point loads taken from the gravity calculation load take-offs. The braced frames were modeled as 20’ in height (see Assumption 15). Using the yield stress (Fy) associated with HSS members, a limiting width-thickness ratio was determined and compared with the actual width to thickness ratio of the selected HSS section. Allowable deflection was likewise checked, but with consideration of the height of the frame. Story drift was checked by using the retrieved story drift from RISA and, amplified based on the criteria set by ASCE 7-10, was compared to the allowable story drift, also set by ASCE 7-10.

Selected Size: HSS 6x6x1/2 - Weight = 35.2 plf & φPn = 58.6 k & b/t = 9.9

Show Building Design Overview

While the space dedicated to pre-ride boarding and unloading is referred to as the Queue Building, the area that encompasses the actual attraction is called the Show Building. Since we decided early in our story process that the only way to simulate the world of TRON for our guests was to build indoors, we found it necessary to design several Show Buildings to cover our Track Layout. However, the elements of design involved with building these structures were not considered the focus of the project and so it was decided that a relatively simple design was adequate for acquiring the information needed for the development of other aspects of the project.

Due to the constraints established from Existing Conditions, we found that in order to cover the total amount of track that we expected, in addition to avoiding the Monorail track, two separate buildings would be required to be installed. This also limited the building height, and therefore it became necessary to have the Show Buildings utilize subterranean space. However, because we desired to keep design as simple as possible, both buildings were identical in structure and dimension, with one oriented at a 90° rotation of the other (so that it could fit within our site parameters - see Existing Conditions). In addition, the roof design of these structures used many of the same systems and elements of the Queue Building that was already designed.

Since our Track Layout would need to fit within the Show Buildings it was deemed crucial that they essentially be designed as “hangar-like” structures, meaning that a minimal amount of columns would be necessary. Our solution to this problem was to create a steel truss system that would serve as girders for our roof beams to frame into. For ease of design, they were modeled in RISA in order to retrieve axial and moment demands.

General Assumptions

1. All design was in accordance with Load and Resistance Factor Design (LRFD) Loading Configuration.

2. Live loading conditions were based on the ASCE 7-10.

3. All assumptions about roof materials and weights made for the Queue Building Design also applied to the Show Building Design (Assumptions 3-8 for Queue Building Design).

4. All beam sizes that were used in the Queue Building roof design were applied to the Show Building roof design.

5. All column sizes that were used in the Queue Building design were also applied to the Show Building design.

6. Foundations, connections, and lateral systems were not designed due to not being within scope of project.

7. Walls for the underground portion are considered to be concrete retaining walls and were not designed due to not being within scope of project.

8. All connections between truss members are assumed to be pinned-pinned connections for simplicity of design.

9. The trusses were designed to be 8' deep to allow for reasonable member sizes as well as provide adequate space for the track and Anthropometric considerations.

Show Building Design

For simplicity of design and to stay within the scope of the project, the beams and columns designed for the Queue Building were applied to the Show Buildings (See Assumptions 3-5). In addition, foundations, connections, and lateral systems were not designed (See Assumption 6).

In place of girders, trusses were designed for the beams to frame into. This was done to minimize the amount of columns used so that the Track Layout would fit within the structure.

A single truss design was duplicated for all trusses used in the structure. The truss was modeled in RISA to obtain a maximum demand axial force. Bay width was modeled as 10'. Height was modeled as 8' to account for two levels of track configuration and Anthropometric requirements (see Assumption 9). Point loads from beams were placed at the location of vertical web members with their values corresponding to the maximum shear values of the roof beams designed for the Queue Building.

The truss system was first modeled in RISA in order to obtain axial values from which to size members. Then a simple span beam equivalent was used to confirm the axial analysis of the truss. A simple span beam with the same loads and application points was modeled in RISA. A maximum moment demand was retrieved and divided by the height of the truss. The resulting axial value was compared to the axial demand retrieved from the initial RISA analysis of the truss to confirm the accuracy of the design.

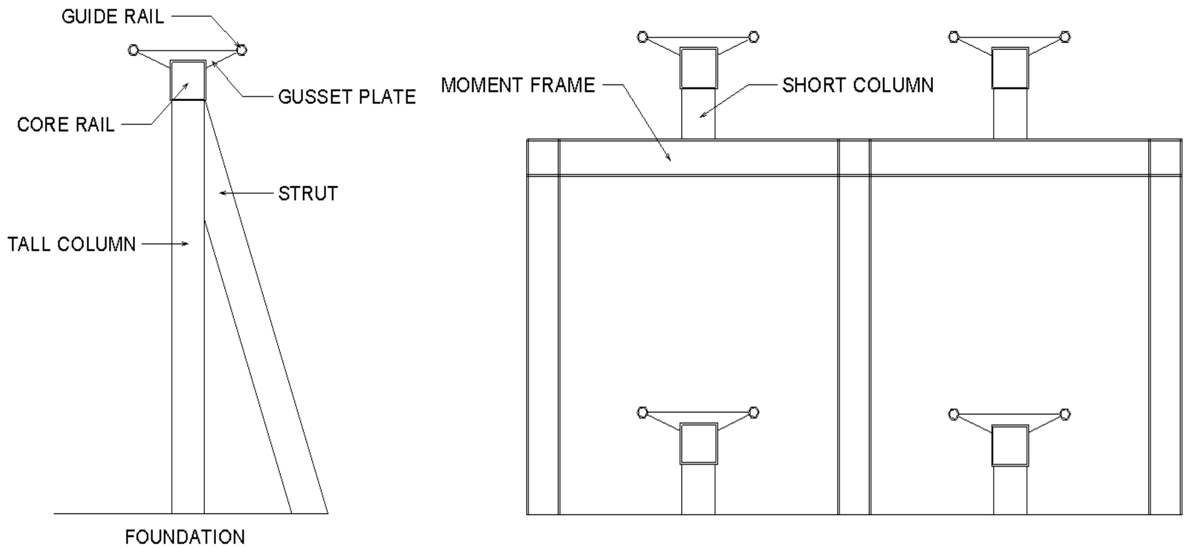
Based on the axial demand, an HSS section was chosen with a sufficient capacity (φPn) based on effective length (KL) from the AISC Steel Construction Manual.

Truss Members Selected Size: HSS 4.4x4.5x3/16 - Weight = 10.7 plf & φPn = 88.4 k

All other structural member sizes replicated from Queue Building Design

Track Design Overview

The term Track Design refers to the conceptual design of the structural members composing the Track Layout to which the Ride Vehicle is attached. For organizational purposes, Track Design has been divided into two categories with their components defined as follows:
Rail System – Guide Rails, Gusset Plates, and Core Rail
Support System – Short Columns, Tall Columns, Struts, Moment Frame, and Foundations



Schematic of Track Configuration to define track elements

Nomenclature

Guide Rails – small continuous round HSS sections that directly support the Ride Vehicle. The wheel assembly fits around the diameter of the Guide Rails. The Guide Rails are supported at 2’ on center by Gusset Plates connected to the Core Rail.
Gusset Plates – steel plates that connect the Guide Rails to the Core Rail. They are trapezoidal in shape and are doubly cantilevered to support each Guide Rail on either side of the Core Rail.
Core Rail - jumbo square HSS section that supports the Gusset Plates and Guide Rails. The Core Rail sits directly on the columns (either tall or short) and spans a maximum of 20’ for transportation and maintenance purposes.
Short Columns – squat round HSS sections that support the portions of the Rail System that are on the lowermost levels of the Track Configuration and that are connected to the Moment Frame. Short Columns are 2’ in height.
Tall Column - round HSS sections that support the portions of the Rail System that are on the uppermost levels of the Track Configuration. They are supported laterally by a Strut. Tall Columns are 17’ in height.
Strut – round HSS sections that provide lateral support for the Tall Columns in the direction perpendicular to the Rail System. Struts are approximately 17.72’ in length.

Moment Frame - wide flange sections that form a moment frame with two bays. Each bay is 14' in length, and the height of the frame is 15'. The Moment Frame is used as a vertical and lateral support system for the Rail System when Tall Column placement was made otherwise impossible.

Foundation - HP sections that serve as steel driven piles. Foundations support the Short Column, Tall Column and Moment Frame where they occur along the Ride System.

General Assumptions

1. All design was in accordance with Allowable Strength Design (ASD) Loading Configuration.
2. The factor of safety (Ω) was used in accordance with ASD load combinations as laid out in AISC Steel Construction Manual.
3. The forces due to earthquake loading did not govern for the design and therefore were not analyzed as part of this conceptual design. Preliminary calculations determined that the dynamic forces generated by the motions of the Ride Vehicle will govern over seismic loading patterns. These dynamic forces were accounted for by applying maximum accelerations to the design weights. Maximum accelerations were observed from a ride simulation ran by the program NoLimits 2. More information on this program can be found in the section on Track Layout within Mechanical Design.
4. Live loading conditions were based on Anthropometric Reference Data for Children and Adults: United States, 2007–2010 (See Fig 10 - Appendix).
5. Due to the dynamic and repetitive loading, the yield stress of the steel (Fy) is based on Fatigue Stress (Fsr) – Eq. A-3-1 AISC Steel Construction Manual. Fatigue will control for repeated loading conditions.

For the calculation of Fsr, the number of stress range fluctuations in design life (Nsr) is based on the number of ride vehicles that will pass a given portion of track per operational day. The assumptions and observations that were made in accordance with this design consideration are as follows:

6. A typical operational day for Disneyland is 16 hours long according to the listed park hours on the official Disneyland Website.
7. In order to cycle passengers through the ride efficiently, six sets of two ride vehicles need be simultaneously on the track during one ride cycle.
8. The duration of a typical ride cycle is approximately 3 mins as reported by the NoLimits 2 ride simulation.
9. According to the ASTM F2291 – 13 Standard for Amusement Ride Design, the design life of a standard amusement park ride is 35000 operational hours (approximately 6 years). At this point, inspection and/or refurbishment of structural components is required.

With these assumptions and observations made, it was determined that approximately 1,920 ride vehicles will pass a given portion of track per operational day. The number of stress fluctuations was determined to be 4,200,000 in design life.

Rail System Design

Guide Rail Design

Design Assumptions

1. Lateral forces were applied to one set of the Wheel Carriage on the interior side of the Guide Rail since the centrifugal force from ride motion places the full weight of the car on the side friction wheels on the outermost Guide Rail on a given turn (see Fig. 14 - Appendix).

2. The unrestrained length of the Guide Rail is determined by the segments in between the points where the Gusset Plates connect with the Guide Rail. These segments are assumed straight due to the small unrestrained length (2’ on center) and to allow for application of a simple span beam analysis. Thus, the restraints are considered under pin condition.

Design Process

This section highlights the process involved when calculating the conceptual design of the Guide Rails (see Nomenclature - Track Design Overview). The forces that are applied to the Guide Rail fall into two broad categories: self-weight of the structure and dynamic loads generated by the movements of the fully-loaded Ride Vehicles with passengers.

The design weight of the Ride Vehicle with passengers was approximately 19,000 lbs. The weights were generated from the SOLIDWORKS outputted mass properties (see Fig. 9 - Appendix), and the weight of the passengers is as stated in the Anthropometric Reference Data for Children and Adults: United States, 2007–2010 (See General Assumption 4). These weights were converted into vertical and lateral components. The vertical components are equally distributed over all four Running Wheels. The horizontal components are assumed to be distributed over two Side Friction Wheels (See Design Assumption 1). The weights were adjusted for dynamic movement with the accelerations retrieved from the NoLimits 2 simulation at points of extreme behavior (i.e. maximum accelerations).

The moment demand was developed based on a conservative assumption of a point load applied to the center of a simple span beam (See Design Assumption 2). The moment demand was related to moment capacity with a factor of safety (Ω). Since round HSS sections are compact in bending, the moment capacity is a function of the plastic moment. This allowed for the determination of a required section modulus (Z) using yield stress (Fy). However, stress due to fatigue (Fsr) was considered instead of Fy (See General Assumption 5). Using the determined section modulus, a round HSS size was chosen from the AISC Steel Construction Manual.

Selected Size: HSS6x0.500 – Weight = 29 plf & Z = 14.3 in^3

Gusset Plate Design

Design Assumptions

1. Each Gusset Plate is spaced 2'on center along the length of the Core Rail. This estimation is based on the observation of real-world representations of roller coaster track.

2. Only vertical acceleration is taken into consideration. The application of lateral acceleration causes an eccentricity in the Gusset Plate that reduces the moment demand due to the vertical acceleration. It is more conservative therefore to consider only vertical acceleration, as it increases moment demand.

3. The distance between the two Guide Rails is 5' based on the distance of the wheel assemblies as developed in the design of the Ride Vehicle in SOLIDWORKS.

4. A minimum value of 1' is considered for the width of the Core Rail in order to estimate a reasonable value for the eccentricity of the Gusset Plate. This was chosen based on the observation that most Core Rails used in roller coaster design are at least 1' in width or diameter. Using a minimum width maximizes the eccentricity which in turn conservatively maximizes moment demand (see Fig. 16 - Appendix).

5. The height of the Gussets is estimated to be four times the thickness. This estimated ratio is based on the observation of real-world representations of roller coaster track in addition to the material properties of steel.

Design Process

This section highlights the process involved when calculating the conceptual design of the Gusset Plates (see Nomenclature - Track Design Overview). The forces that are applied to the Gusset Plates fall into two broad categories: self-weight of the structure and dynamic loads generated by the movements of the fully-loaded Ride Vehicles with passengers.

The design weight of the Ride Vehicle with passengers was approximately 19,000 lbs. The weights were generated from the SOLIDWORKS outputted mass properties (see Fig. 9 - Appendix), and the weight of the passengers is as stated in the Anthropometric Reference Data for Children and Adults: United States, 2007–2010 (See General Assumption 4). Self-weight of the Guide Rail was included. These weights were converted into a vertical component. The lateral component was not taken into consideration (See Design Assumption 2). The weights were adjusted for dynamic movement with the accelerations retrieved from the NoLimits 2 simulation at points of extreme behavior (i.e. maximum accelerations).

In addition, the design weight of the Guide Rails above the Gusset Plates was accounted for by multiplying the weight of a single Guide Rail over its span between each Gusset Plate. When added to the design weight of the Ride Vehicle with passengers, total design weight in the vertical component became approximately 12,000 lbs.

In order to find moment demand, an analysis of the eccentricity in the gusset was conducted. Only the vertical eccentricity in the gusset is considered (see Design Assumption 2). The vertical eccentricity is the estimated distance from the edge of the Core Rail to the center of the Guide Rail. The distance between the two Guide Rails is 5' and the assumed minimum width of the Core Rail is 1' (See Design Assumptions 3 & 4).

The moment demand was related to moment capacity with a factor of safety (Ω). Moment capacity is directly related to bending stress, which is a factor of the moment of inertia for the steel cross-section (I) and the distance of the neutral axis to the outermost dimension of the section (c). Both moment of inertia and neutral axis distance are calculated from the height of the section (h). By substituting the value of the factored moment demand (Mu) for moment capacity (Mn) and the value of stress based on fatigue (Fsr) for bending stress (σ_b) (See General Assumption 4), both moment of inertia and neutral axis length can be equated. Once equated, the height of the cross section can be determined based on the assumption that the thickness of the section is four times the height (see Design Assumption 5). The derived height of the section can be related back to width using the same assumption. The retrieved data was adjusted to whole numbers for ease of design.

Buckling of the Gusset Plate was checked using the parameters outlined in section E7-4 of the <u>AISC Steel Construction Manual.</u> Required thickness was determined and compared to the thickness sized based on moment capacity.

Selected Size: 2.5 in x 9.5 in Gusset Plate– Weight = 266 lbs.

Core Rail Design

Design Assumptions

1. The maximum span of the Core Rail is 20’. This was done so that the Core Rail can be transported in a reasonable fashion and uninstalled in a way that allows for ease of maintenance access.

2. Axial Stresses were ignored to account for placement of LIMs. A characteristic effect of LIMs is a kickback force that applies an acceleration in the orientation of the ride vehicles’ course of movement.

3. Due to the square section properties and the fact that horizontal accelerations were less than vertical accelerations, analysis for the horizontal orientation of the Core Rail was assumed to be valid for the selected design.

4. When designing for combined bending stresses, it was important to look at the area of track where the resultant accelerations were highest, resulting from both vertical and lateral accelerations (2.6 g - vertical and 1.3 g - lateral).

5. When designing for torsion, it was more conservative to maximize the accelerations individually, so a location with the maximum lateral acceleration was chosen (1.8 g).

Design Process

This section highlights the process involved when calculating the conceptual design of the Core Rail (see Nomenclature - Track Design Overview). The forces that are applied to the Core Rail fall into two broad categories: self-weight of the structure and dynamic loads generated by the movements of the fully-loaded Ride Vehicles with passengers.

The design weight of the Ride Vehicle with passengers was approximately 19,000 lbs. The weights were generated from the SOLIDWORKS outputted mass properties (see Fig. 9 - Appendix), and the weight of the passengers is as stated in the Anthropometric Reference Data for Children and Adults: United States, 2007–2010 (See General Assumption 4). These weights were converted into a vertical and lateral component. The weights were adjusted for dynamic movement with the accelerations retrieved from the NoLimits 2 simulation at points of extreme behavior (i.e. maximum accelerations).

In addition, the design weights of the Guide Rails and Gusset Plates above the Core Rail were accounted for. For the Guide Rails, the self-weight of two Guide Rails was multiplied by the maximum span of the Core Rail. For the Gusset Plates, the self-weight of each was multiplied by the maximum span of the Core Rail and divided by their spacing. When added to the design weight of the Ride Vehicle with passengers, total design weight in the vertical component became approximately 53,000 lbs.

The shear and moment demand was developed by applying the total design weight to the maximum span of the Core Rail (See Design Assumption 1). The moment demand was related to moment capacity with a factor of safety (Ω). Since square HSS sections are compact in bending, the moment capacity is a function of the plastic moment. This allowed for the determination of a required section modulus (Z). Using the determined section modulus, a jumbo square HSS size was chosen from the AISC Website.

To account for self-weight of the Core Rail total moment demand was increased by the moment demand from the self-weight of the Core Rail. Section modulus was re-evaluated and it was determined that the selected design remained valid.

Due to the square section properties and the fact that horizontal accelerations were less than vertical accelerations, analysis for the horizontal orientation of the Core Rail was assumed to be valid for the selected design (See Design Assumption 3).

Combined bending stress was analyzed by relating the moment demands in both orientations to a stress value (fx and fy). Each demand stress was related with a ratio to fatigue stress (Fsr) to confirm that combination of the bending stresses remained lower than the governing value of Fsr (See General Assumption 5).

Effects from torsion were also analyzed. To account for torsion in the horizontal direction the factored weight was multiplied by the distance from the Guide Rail to the center of the Core Rail. To account for torsion in the vertical direction the applied weight was multiplied by the moment arm (b). Moment arm was defined by factors outlined in the torsion section of Design of Welded Structures. These factors included: radius of the turn (r) and the angle between supports (α). The addition of these torsional values were applied to an equation to develop shear stress from Design of Welded Structures. This equation also took into account the area enclosed and thickness of the selected HSS section. The determined shear stress was compared to fatigue stress to confirm that the section was valid.

Finally, combined shear stress was analyzed, accounting for self-weight of the Core Rail. The shear demands in both orientations were related to a stress value (fvx and fvy). Each demand stress was related with a ratio to fatigue stress Fsr to confirm that combination of the shear stresses remained lower than the governing value of Fsr.

Selected Size: HSS20x20x7/8 - Weight = 221 plf & Z = 14.3 in^3

Support System Design

Short Column Design

Design Assumptions

1. Buckling was not analyzed due to the short length of the column.

2. There is full fixity at the base of the column due to the use of a moment-carrying foundation design.

3. All instances along the track where there are negative vertical accelerations are less than 1.0g and therefore tension in the column does not have to be considered.

Design Process

This section highlights the process involved when calculating the conceptual design of the Short Columns (see Nomenclature - Track Design Overview). The forces that are applied to the Short Columns fall into two broad categories: self-weight of the structure and dynamic loads generated by the movements of the fully-loaded Ride Vehicles with passengers. These forces were analyzed as axial loads applied vertically on the Short Column and as bending loads determined from lateral movement.

The design weight of the Ride Vehicle with passengers was approximately 19,000 lbs. The weights were generated from the SOLIDWORKS outputted mass properties (See Fig. 9 - Appendix), and the weight of the passengers is as stated in the Anthropometric Reference Data for Children and Adults: United States, 2007–2010 (See General Assumption 4). These weights were converted into a vertical and lateral component. The weights were adjusted for dynamic movement with the accelerations retrieved from the NoLimits 2 simulation at points of extreme behavior (i.e. maximum accelerations).

In addition, the design weights of the entire Rail System above the Short Columns needed to be considered. For the Guide Rails, self-weight was multiplied by the maximum span of the Core Rail and twice multiplied per rail. For the Gusset Plates, the self-weight of each was multiplied by the maximum span of the Core Rail and divided by their spacing. For the Core Rail, self-weight was multiplied by the maximum span of the Core Rail. When added to the design weight of the Ride Vehicle with passengers, total design weight in the vertical component became approximately 41,000 lbs.

To design for axial forces, a demand axial force was determined from the vertical component design weight. The demand axial force was related to axial capacity with a factor of safety (Ω). Capacity was related to fatigue stress (Fsr) with section area. The required area was determined, and a minimum section size was selected from the AISC Steel Construction Manual.

To design for bending forces, a demand moment was determined by multiplying the lateral component design weight by the height of the Short Column. The moment demand was related to moment capacity with a factor of safety (Ω). Since round HSS sections are compact in bending, the moment capacity is a function of the plastic moment. This allowed for the determination of a required section modulus (Z).

Using the determined section modulus, a round HSS size was chosen from the AISC Steel Construction Manual.

Combined bending and axial stress was checked by adding the stresses determined from bending and axial and comparing the total with fatigue stress (Fsr).

Selected Size: HSS 16x0.625 - Weight = 103.0 plf - Z = 138.0 in^2

NOTE: Size Re-evaluated to account for Steel Driven Piles (See Foundation Design)

Selected Size: HSS 18x0.500 - Weight = 93.54 plf - Z = 143.0 in^2

Tall Column and Strut Design

Design Assumptions

1. The column and strut were assumed to not have moment resisting fixity at the base (pinned condition).

Design Process

This section highlights the process involved when calculating the conceptual design of the Tall Columns and Struts (see Nomenclature - Track Design Overview). The forces that are applied to the Tall Columns and Struts fall into two broad categories: self-weight of the structure and dynamic loads generated by the movements of the fully-loaded Ride Vehicles with passengers. Tall Columns and Struts were analyzed for axial forces caused by lateral and vertical accelerations.

The design weight of the Ride Vehicle with passengers was approximately 19,000 lbs. The weights were generated from the SOLIDWORKS outputted mass properties (see Fig. 9 - Appendix), and the weight of the passengers is as stated in the Anthropometric Reference Data for Children and Adults: United States, 2007–2010 (See General Assumption 4). These weights were converted into a vertical and lateral component. The weights were adjusted for dynamic movement with the accelerations retrieved from the No Limits 2 simulation at points of extreme behavior (i.e. maximum accelerations).

In addition, the design weights of the entire Rail System above the Tall Columns and Struts needed to be considered. For the Guide Rails, self-weight was multiplied by the maximum span of the Core Rail and twice multiplied per rail. For the Gusset Plates, the self-weight of each was multiplied by the maximum span of the Core Rail and divided by their spacing. For the Core Rail, self-weight was multiplied by its maximum span. When added to the design weight of the Ride Vehicle with passengers, total design weight in the vertical component became approximately 57,000 lbs.

Axial demand for the Strut was determined by finding the maximum lateral force on the tall column from the adjusted design weight (developed with maximum lateral accelerations). The maximum lateral force was converted into an axial resultant by multiplying the maximum lateral force by the ratio of the diagonal length of the Strut with its lateral distance from the Tall Column.

An initial round HSS section was selected to begin design. The parameters outlined for determining Fcr (critical stress) were observed which included an analysis of kL/r (unrestrained length and radius of gyration) and Fe, the elastic buckling stress. An axial capacity was determined by multiplying Fcr by the gross area (A) of the section selected. Axial capacity was related to axial demand with a factor of safety (Ω) to verify that the capacity determined with the selected section was greater than the demand. The addition of self-weight of the Strut on demand was verified as well.

Axial demand on the Tall Column took into consideration both lateral and vertical accelerations. The total design weight on the Tall Column (developed with maximum vertical accelerations) was taken as compressive force. To determine the tensile force on the Tall Column, the maximum lateral force (developed with maximum lateral accelerations) was converted into an axial resultant by multiplying the maximum lateral force by the ratio of the height of the Tall Column with its lateral distance from the base of the Strut. Compressive and tensile forces were combined to develop a singular axial compression force on the Tall Column.

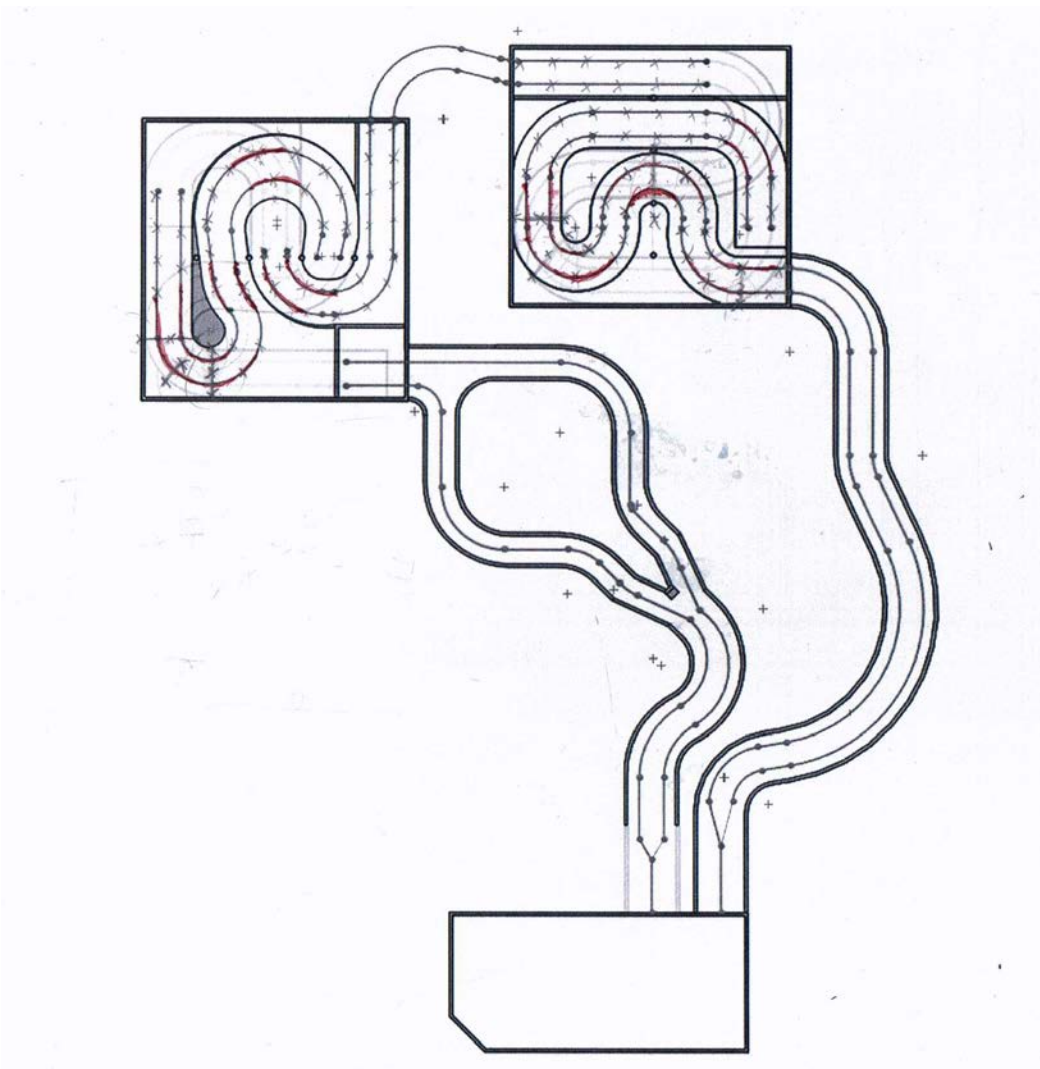
An axial capacity was determined by multiplying Fcr by the gross area of the section selected. Axial capacity was related to axial demand with a factor of safety (Ω) to verify that the capacity determined with the selected section was greater than the demand.

Moment Frame Design

Design Assumptions

1. The height of the Moment Frames as analyzed in RISA was modeled as 20' tall to account for the 5' height of track sitting on top of the 15' tall Moment Frames. This simplified analysis of the frame while still being conservative at the critical locations of design (see Fig. 20 & 21 - Appendix)

2. The Moment Frames were only necessary in areas of the Track Layout that had maximum accelerations of 1.0 g in both the vertical and horizontal directions. Therefore, the design weights applied to the Moment Frame remained the same after accounting for dynamic movement.



Track overlay - Moment Frame placement is drawn in red

Design Process

This section highlights the process involved when calculating the conceptual design of the Moment Frames (see Nomenclature). The forces that are applied to the Moment Frames include self-weight of the structure and the fully-loaded Ride Vehicles with passengers. The Moment Frame was designed to consider forces due to buckling and combined bending and axial applications.

The design weight of the Ride Vehicle with passengers was approximately 19,000 lbs. The weights were generated from the SOLIDWORKS outputted mass properties (See Fig. 9 - Appendix), and the weight of the passengers is as stated in the Anthropometric Reference Data for Children and Adults: United States, 2007–2010 (See General Assumption 4). These weights were converted into a vertical and lateral component.

In addition, the design weights of the entire Rail System and Short Columns on the Moment Frame needed to be considered. For the Guide Rails, self-weight was multiplied by the maximum span of the Core Rail and twice multiplied per rail. For the Gusset Plates, the self-weight of each was multiplied by the maximum span of the Core Rail and divided by their spacing. For the Core Rail, the self-weight was multiplied by the maximum span of the Core Rail. For the Short Column, the self-weight was multiplied by the height of the Short Column. When added to the design weight of the Ride Vehicle with passengers, total design weight in the vertical component became approximately 26,000 lbs.

Moment and axial demand were observed from an analysis of the Moment Frame as modeled in RISA (See Design Assumptions 1 & 2). These demands were multiplied by a factor of safety (Ω) to determine required flexural and axial strengths (Mr & Pr).

An initial wide flange section was selected to begin design. The parameters outlined for determining Fcr (critical stress) were observed which included an analysis of kL/r (unrestrained length and radius of gyration) and Fe, the elastic buckling stress. To determine moment capacity (Mc), Fcr was multiplied by the section modulus of the selected section (Z). To determine axial capacity (Pc), Fcr was multiplied by the gross area of the selected section (Ag).

Combined axial and bending forces were checked within the parameters outlined in section H1-1b of the AISC Steel Construction Manual using the determined capacities and required strengths.

Selected Size: W18x175 - Weight = 175 plf & Z = 398 in^3 & A = 51.4 in^2

Foundation Design

Design Assumptions

1. For simplicity of design, calculations were made to only consider the required depth of the selected steel driven piles.

2. Soil pressure normally increases as depth increases, however, in the special case of piles driven in sand (see Geotechnical Information) it was determined that Effective Vertical Overburden Pressure (Pv) increased as depth increased until a certain depth of penetration was reached, known as critical depth (dc), after which it remained constant.

3. Tests as outlined in Soils and Foundations indicate that critical depth (dc) is 10 pile diameters for loose sand (see Geotechnical Information).

4. Since the axial demand from the Tall Column design was less than that of the Short Column design, it was established that the selected size would be adequate for the Tall Column Foundation.

5. Soil assumed to be poorly graded sand for the continuous depth analyzed. Soil unit weight (γ), lateral earth pressure coefficient (K), angle of internal friction (φ), and factor of safety (FS), were all based on assumptions for typical poorly graded sand from the Geotechnical Info Website.

Design Process

This section highlights the process involved when calculating the conceptual design of the Foundations (see Nomenclature - Track Design Overview). The design of the Foundations was split up into 3 different categories based on the element that the corresponding Foundations were supporting. These categories were Short Column Foundations, Tall Column Foundations, and Moment Frame Foundations. An analysis of the required Foundation depth was performed for each category (See Design Assumption 1).

For each element of Foundation design, an initial HP size and depth was selected.

The height of the cross section (d) of the selected HP size was used to calculate the critical depth (dc) (See Design Assumption 3). To determine the Effective Vertical Overburden Pressure (Pv), dc was multiplied by the Unit Weight of Soil (γ) (See Geotechnical Information - Soil Parameters - Appendix). The area of soil pressure adjacent to the selected pile (PvL) was determined based on the geometries defined by the selected depth and dc (See Design Assumption 2).

The value for bearing from the friction between the soil and the pile (qfric) was determined by multiplying PvL by the coefficient of lateral earth pressure (K), the coefficient of friction between sand and pile surface ($\tan\delta$), and the surface area of the pile (SA). K and $\tan\delta$ were retrieved from the Geotechnical Info Website and SA was determined from the selected diameter (d).

The value for bearing within the area beneath pile's tip (qtip) was determined by multiplying Pv by the bearing capacity factor (Nq*) by the area of the pile's tip (Atip). Nq* was retrieved from a graph in Soils Foundations based on the angle of internal friction (φ) (See Geotechnical Information) and Atip was retrieved from the AISC Steel Construction Manual based on the selected HP size.

The axial demand from the Short Column design was compared to the ultimate bearing capacity of a single pile (qult) which is the combination of qfric and qtip divided by a Factor of Safety. This Factor of Safety was retrieved from the Geotechnical Info Website.

Since the axial demand from the Tall Column design was less than that of the Short Column design, it was established that the selected size would be adequate for the Tall Column Foundation (See Design Assumption 4).

The Foundations underneath the Moment Frame were sized based on the tension demand caused from uplift, as it was the more conservative case as it applies to driven pile design. Tensile forces were retrieved from an analysis of the Moment Frame as modeled in RISA (See Moment Frame Design). The self-weight of the Moment Frame was also determined and applied as a compressive force. Tensile and compressive forces were combined to produce an axial tension demand to be applied to the Moment Frame Foundation. The process performed for the Moment Frame Foundations is very similar to the process used in designing the Short Column Foundations, with the only difference being that qtip is not taken into account since the foundation is considered to be in tension and end bearing pressure provides no capacity.

The axial demand determined was compared to qult which is equal to qfric divided by a Factor of Safety. This Factor of Safety was retrieved from the Geotechnical Info Website.

Lastly, lateral accelerations due to the placement of LIMs was taken into account, since the propulsion force of the LIM caused a force to be applied at the top of the columns. A demand moment (Ms) was determined by applying maximum lateral acceleration to the design weight of the Ride Vehicle with passengers and multiplying over the design height of the Moment Frame. A process as outlined in section 18-12 of the IBC 2012 was used to calculate a design depth to be compared with the selected depth. This process included the demand moment (Ms), the lateral bearing pressure of the soil (S), and the width of the member (b).

Based on the separate criteria established, it was decided to use an HP 18x181 and a depth of 33 feet be applied to all columns supporting the track, as different depths could cause errors during the construction process.

Selected Size: HP18x181 - 33 ft deep Steel Driven Pile

Conclusion

As stated in the Background of this report, our goal was to come up with a conceptual / schematic design that could be further developed into an attraction at a Disneyland park. Even though it wasn't designed so it could be signed off immediately by WDI and slated for installation, we still hoped to deliver a safe and viable design. As a result of the design assumptions inferred within this report, we feel that we have accomplished this task.

Our process took us through a wide range of design processes involved on a typical WDI project and gave us exposure to the dependency that each element has on one another. Story drafting, storyboarding, aesthetic considerations, geotechnical research, mechanical design, and schematic design were all skills that needed to be accessed while completing this project. We also came into contact with such programs as Revit, SOLIDWORKS, and NoLimits 2. As a result we have broadened our skill set and made ourselves more comfortable with software and engineering processes that would have otherwise not have been included in our curriculum.

This project also furthered our understanding to us on concepts relating to the ARCE curriculum that we had not previously been exposed to before. Dynamic forces, other than earthquake and wind forces, had to be researched and applied. We also had to look further into to the implications of these loads, including additional torsional and fatigue stresses on the structure, with which we had previously had been unfamiliar.

Furthermore, we were able to reach out to other disciplines and resources. Our project encouraged us to make contact with departmental professionals (i.e. Melinda Keller - an ME professor with professional experience in the amusement park industry) and extracurricular on-campus associations (i.e. Cal Poly Amusement Park Engineers and Designers or CAPED - a club who has interviewed and worked with professionals in the amusement park industry). Specifically we'd like to give credit to Robert Cory Molloy (the current president of CAPED) who helped give us access to SOLIDWORKS and the ASTM F2291. Likewise, ME team member Kelly Rorden (ex-president of CAPED) used her own access to materials such as SOLIDWORKS and NoLimits 2 alongside her knowledge of industry standards to aid in the development of Ride Vehicle Design and Track Layout.

Next Steps

With the completion of this senior project, we would appreciate the opportunity to show our project to representatives of WDI in order to gain a professional opinion on the work performed. The easiest way for this to be accomplished would be to possibly enter the Imaginations Competition that WDI puts on every year. The rules regarding the theme of the competition are generally strict, but as long as the competition requirements are met by our project, we feel that it would be beneficial to enter and a fun experience as well.

It would be likewise beneficial, given the time, to further our design development, advancing from a simple schematic design to a more detailed analysis. Since the goal of our project dictated a complete overview of the design process, assumptions were made in place of a detailed design where we felt it was appropriate.

One of these ignored components was the design of connection details - specifically welds. If continued further, the project would become more viable if such elements were considered within the design process. Other efforts made in the pursuit of realism could include the designing of maintenance access for the Track Design in addition to elements such as safety protocol.

Overall, TRON 2.0: The Experience was an excellent albeit tiny glance into the immense capacity of projects by WDI.

Geotechnical Information

Properties of Typical Soil in Anaheim:

- Well drained
- Moderately deep soils over weathered fine grained sandstone and shale
- Fine, loamy, mixed
- 0-9": Clay loam, moderate, medium subangular blocky structure; hard, friable, sticky, plastic
- 9"-17": Moderate,medium subangular blocky structure; hard, friable, sticky, plastic
- 17-26": Clay loam, weak, medium subangular blocky structure; hard, friable, sticky, plastic
- 26"-54": Weathered, fractured, fine grained sandstone and shale

Site Analysis Report for Different Project, Approximately 5 Miles South East of Disneyland:

- Soil Type:
  - Poorly graded sand, silty sand, lean clay, sandy silt
  -
- Groundwater:
  - Average per testing of area: 60' depth
  - Historic high: 20' depth
  - Another recorded at South Douglass and Katella: 34' depth
  - Site is within a state of california hazard zone for liquefaction, potential for liquifaction is high
    - 2007 CBC: Sites subject to liquefaction shall be classified as site class F, and requires site response analysis
    - ASCE 7-05: For short period (<1/2 second) structure on liquefiable soils, site class D or E may be used to estimate seismic design loading
- Foundation:
  - Shallow foundations NOT recommended
  - Deep foundation systems such as driven piles with a structurally supported slab
  - Consider ground improvement, deep soil mixing
  - At walls and floor slabs that extend below design ground water elevation should be waterproofed and designed for hydrostatic pressures
- Settlement:
  - Predicted seismically induced settlement: 3"-6"
  - Predicted differential settlement: 2"-4"

Conclusion:

- Potentially shallow groundwater, consider liquefaction, site response analysis
- Site Class D
- Likely Deep Foundations, ground improvement may be necessary

Anaheim Soil Information retrieved from

Anaheim City Website and the USDA-NRCS Official Soil Series

Soil Parameters - Poorly Graded Sand

NOTE: These are the factors used in Foundation Design that were not retrieved from *Soils and Foundations* text - for values retrieved from the text See Foundation Design

γ = 90 pcf - Soil Unit Weight - Loose Sand

K = 0.6 - Lateral Earth Pressure Coefficient - Silty Sand

φ = 30 - Angle of Internal Friction - Loose Sand

FS = 1.7 - Factor of Safety Considered for Uplift

Information retrieved from Geotechnical Info Website

Standard Roof Materials - Queue Building

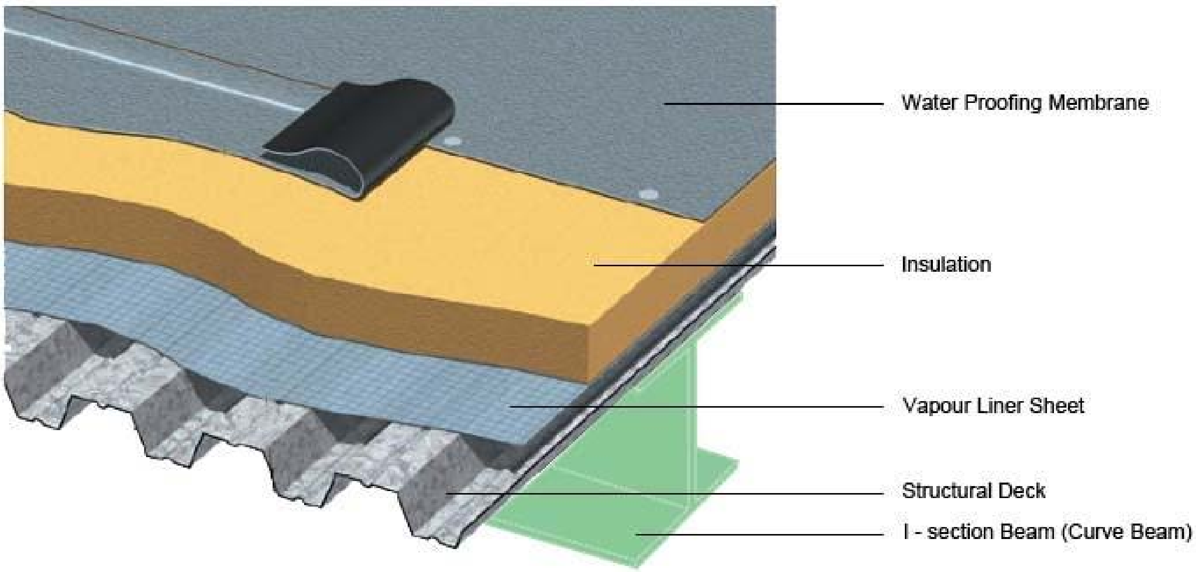


Fig. 1 - Typical Roof Materials - Cross-Section - Tegral Building Products

Boise Cascade
Engineered Wood Products

Technical Note

Weights of Building Materials – Pounds Per Square Foot [PSF]

ROOF	
Fiberglass shingles	3
Asphalt shingles [1]	2
Wood shingles [1]	3
Spanish clay tile [1]	19
Concrete roof tile	12
Composition Roofing:	
Three-ply ready roofing [1]	1
Four-ply felt and gravel [1]	5.5
Five-ply felt and gravel [1]	6
20 gage metal deck [1]	2.5
18 gage metal deck [1]	3
0.05" thick polyvinyl chloride polymer membrane[4]	0.35

1" fiberglass batt insulation	0.04
1" loose fiberglass insulation	0.04
1" loose cellulose insulation	0.14
1" rigid insulation [1]	1.5
Blowing wool insulation R-38 (16"deep)	0.62
3/16" slate [1]	7
1/4" slate [1]	10
Single-ply (no ballast) [1]	0.7
Single-ply (ballasted)	11
Dry gravel [1]	8.7
2x8 rafters @ 16" o.c., fiberglass shingles, 15# felt, 3/8" sheathing	8
Skylight: metal frame w/ 3/8" wire glass [1]	8

Fig. 2 - Typical Roof Materials - Standard Weights - Boise Cascade

TECHNICAL INFORMATION SHEET

Firestone BUILDING PRODUCTS

V-Force™ Vapor Barrier Membrane

Storage:

- All material should be stored out of the weather in a clean, dry area in its original unopened packaging at a minimum of 50 °F (10 °C) and a maximum of 140 °F (60 °C) so that it will be 50 °F (10 °C) or above at the time of application.
- If material must be stored temporarily on the roof before application, it must be elevated from the roof surface on a pallet, stored on end, and covered from the weather with a light colored opaque tarp in a neat safe manner that does not exceed the allowable load limit of the storage area.
- Do not stack V-Force material more than two (2) pallets high.

Shelf Life:

Shelf life of eighteen (18) months can be expected when stored in the original, unopened container between 50 °F and 140 °F (10 °C and 60 °C)

Precautions:

1. Refer to Material Safety Data Sheet (MSDS) for safety information.
2. Hot asphalt cannot be used to adhere roofing materials to V-Force membrane.
3. Take care when transporting and handling Firestone V-Force Vapor Barrier Membrane rolls to avoid punctures and other types of physical damage.
4. Isolate waste products, petroleum products, grease, oil (mineral and vegetable) and animal fats from all Firestone membranes.

LEED® Information:

Post Consumer Recycled Content: 0%
Post Industrial Recycled Content: 0%
Manufacturing Location: Drummondville, QC

FM APPROVED

CLASSIFIED C UL US

Product Packaging

Roll Width:	45" (1.14 m)	Boxed Rolls per Pallet:	16
Roll Length:	133.8' (42.3 m)	Weight Per Pallet:	1,372 lb (622 kg)
Net Coverage:	468 ft^2 (43.5 m^2)	Pallet Size:	48" x 39" (1.2 m x 1 m)
Roll Weight:	82 lb (37 kg)	Coverage per Pallet:	7,488 ft^2 (696 m^2)

Fig. 3 - Firestone Vapor Barrier - Technical Info

Sarnafil® G476 Waterproofing Membrane

Warranty: Upon successful completion of the waterproofing system application, a Sika Sarnafil warranty may be available. Consult with your Sika Sarnafil Regional Office for further information.

Maintenance: Sarnafil G476 requires no maintenance. Although the membrane is typically not easily accessible, regular standard maintenance of plaza decks and green roofs should include regular inspection of drains and termination sealants at least twice per year and after each storm.

Technical: Sika Sarnafil provides technical support. Technical staff is available to advise applicators as to the proper installation method.

Technical Data (as manufactured):

Parameters	ASTM Test Method	Typical Physical Properties
Reinforcing Material	--	Fiberglass
Overall Thickness(1), min.	D638	(see note 1)
Weight, lb/ft^2 (kg/m^2)	--	0.49 (2.38)
Tensile Strength, min., psi (MPa)	D638	1600 (11.1)
Elongation at Break, min.	D638	240% M.D. 240% C.M.D.
Seam Strength(2), min., (% of tensile strength)	D638	90
Retention of Properties After Heat Aging	D3045	--
Tensile Strength, min., (% of original)	D638	95
Elongation, min., (% of original)	D638	95
Tearing Resistance, min., lbf (N)	D1004	21.3 (94.7)
Low Temperature Bend, -40°F (-40°C)	D2136	Pass
Linear Dimensional Change	D1204	0.002%
Weight Change After Immersion in Water	D570	2.0%
Static Puncture Resistance, 56 lbf (250 N)	D5602	Pass
Dynamic Puncture Resistance, 117.7 ft-pdl (5 J)	D5635	Pass
VOC Content		0 g/L

(1)Typical Physical Properties data is applicable for 0.048 in. (1.2 mm) membrane thickness and greater. (2)Failure occurs through membrane rupture not seam failure.

Fig. 4 - Sika Sarnafil Waterproofing Membrane - Technical Info

Gravity System Design - Queue Building

Gravity calculations for the Queue Building were performed by hand at the beginning of the project. For this reason, their page numbers do not coincide with the page numbers assigned to this Appendix. The regular page number references will continue after the insertion of the following gravity calculations.

Project: TRON: THE EXPERIENCE – QUEUE BUILDING

Date: 3/8/15

Subject: ROOF DEAD LOAD TAKE-OFF

By: JD

Reviewed:

Page: RGD-1

Reference:

Check:

ROOF DEAD LOAD TAKE-OFF (INTERIOR MEMBERS)

MATERIAL	CALCULATIONS	DECKING	BEAMS	GIRDERS	COLUMNS	SEISMIC
FIRESTONE V-FORCE VAPOR BARRIER	(SEE MATERIAL RESEARCH)	.49PSF				
1" RIGID INSULATION	1.5PSF/1"	1.5PSF				
VAPOR LINER SHEET		.18PSF				
18" VERCO DECKING PLB-36/HSB-36	(SEE PAGE GD-2)	1.9PSF				
SPRAY FIREPROOFING		3.0PSF				
MEP			5.0PSF			
SUSP. T-BAR CEILING +ACOUST. FIBER TILES	2PSF + 1PSF		3.0PSF			
W12 x 45 BEAMS			4.5PSF			
W12 x 45 GIRDERS				1.5PSF		
W10 x 33 COLUMNS					1.3 PSF	
MISCELLANEOUS	2-5% (TOTAL)	.43PSF	1.0PSF	1.0PSF	1.2PSF	
PREVIOUS MEMBER TOTAL			7.5PSF	21 PSF	23.5PSF	
TOTAL		7.5PSF	21 PSF	23.5PSF	26 PSF	

ROOF LIVE LOAD (UNREDUCED) = 20PSF

Units Listed: ☐

Numbers Checked: ☐

Sources Referenced: ☐

Project: TRON: THE EXPERIENCE - QUEUE BUILDING			Date: 3/8/15
Subject: ROOF DECK DESIGN	By: JD	Reviewed:	Page: RGD-2

Reference:

GD-1

VERCO DECKING STEEL ROOF DECKS P. 28-29 WWW.VERCODECK.COM

LOADING

LIVE = 20PSF

DEAD = 7.5PSF

DEAD + LIVE = 27.5PSF

SPAN = 10'

USE:

⟹ 22 GAGE PLB-36, 1½" DEEP, DOUBLE SPAN, W_{GALV} = 1.9PSF

(SEE ATTATCHED)

Check:

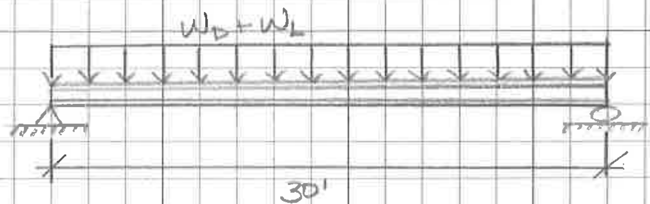
Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

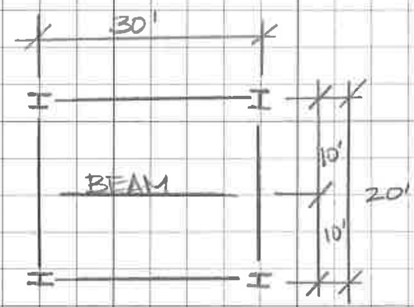
Project: TRON: THE EXPERIENCE – QUEUE BUILDING			Date: 3/8/15
Subject: ROOF INTERIOR BEAM DESIGN	By: JD	Reviewed:	Page: RGD-3

Reference:

Check:

SCHEMATIC





LOADS

LIVE LOAD REDUCTION:

ASCE 7-10 SEC. 4.8 P. 15

$L_r = L_o R_1 R_2$

R_1: $A_T = (30')(10') = 300FT^2$

$200FT^2 < A_T = 300FT^2 < 600FT^2$

$R_1 = 1.2 - .001(A_T)$

$= 1.2 - .001(300FT^2)$

$= .9$

$R_1 = .9$

IBC 2012 1507.4.2 P. 320

R_2: $F = 1/2''$ RISE PER 1' RUN

$F = 1/2 \leq 4$

$R_2 = 1$

$R_2 = 1$

$L_r = L_o R_1 R_2$

$= (20PSF)(.9)(1.0)$

$= 18PSF$

DEAD LOAD:

RGD-1

D: ASSUME WEIGHT OF BEAM = 4.5PSF + 17PSF MISCELLANEOUS

D = 21.0PSF

FACTURED LOADS

DEAD = 1.2(21.0PSF)

= 25.2 PSF

LIVE = 1.6(18PSF)

= 28.8PSF

w_D = (25.2PSF)(10')

= 252PLF

= .252KLF

w_L = (28.8PSF)(10')

= 288PLF

= .288KLF

TOTAL FACTURED LOADS

$w_{TOTAL} = w_D + w_L$

= 252 PLF + 288PLF

= 540 PLF

= .540 KLF

Units Listed: ☐	Numbers Checked: ☐	Sources Referenced: ☐

Project: TRON: THE EXPERIENCE — QUEUE BUILDING			Date: 3/8/15
Subject: ROOF INTERIOR BEAM DESIGN	By: JD	Reviewed:	Page: RGD-4

Reference:

Check:

SHEAR AND MOMENT DEMAND

$M_{u_D} = w_D l^2/8$
$= (252 PLF)(30')^2/8$
$= 28350 \# FT$
$= 28.35 KFT$

$V_{u_D} = w_D l/2$
$= (252 PLF)(30')/2$
$= 3780\#$
$= 3.78K$

$M_{u_L} = w_L l^2/8$
$= (288 PLF)(30')^2/8$
$= 32400 \# FT$
$= 32.4 KFT$

$V_{u_L} = w_L l/2$
$= (288 PLF)(30')/2$
$= 4320\#$
$= 4.32K$

$M_{u_{TOT}} = 28.35 KFT + 32.4 KFT$
$= 60.75K$

$V_{u_{TOT}} = 3.78K + 4.32K$
$= 8.1 K$

TRY A W12 x 45

PROPERTIES:

AISC 2011 P.1-26

$A = 13.1 IN^2$ $I_x = 348 IN^4$ $E = 29000 KSI$
$d = 12.1 IN.$ $S_x = 57.7 IN^3$ $F_y = 50 KSI$
$t_w = .335 IN.$ $Z_x = 64.2 IN^3$

CHECK MOMENT CAPACITY:

$\phi M_{px} = \phi F_y Z_x$
$= (.9)(50 KSI)(64.2 IN^3)$
$= 2889 KIN (1'/12'')$
$= 240.8 KFT > M_u = 60.75 KFT$ ✓ W12X45 ADEQUATE FOR M

CHECK SHEAR CAPACITY:

$\phi V_n = .6 F_y A_w$
$= .6 F_y t_w d$
$= (.6)(50 KSI)(.335'')(12.1 IN)$
$= 121.6K > V_u = 8.10 K$ ✓ W12X45 ADEQUATE FOR V

CHECK DEFLECTION LIMIT:

$\Delta_{MAX_L} = 5 w_L l^4 / 384 EI$
$= 5(.288 KLF)(30')^4(12''/1')^3 / 384(29000 KSI)(348 IN^4)$
$= .520 IN.$

$\Delta_{CAP_L} = L/360$
$= (30')(12''/1')/360$
$= 1''$

$\Delta_{CAP_L} = 1'' > \Delta_{MAX_L} = .520''$ ✓ W12X45 ADEQUATE FOR Δ

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

	Project: TRON: THE EXPERIENCE – QUEUE BUILDING			Date: 3/8/15
	Subject: ROOF INTERIOR BEAM DESIGN	By: JD	Reviewed:	Page: RGD-5

Reference: | Check:

RGD-3

CHECK SELF WEIGHT ASSUMPTION

RECALL: ASSUMED BEAM WEIGHT = 4.5 PSF

FOR W12X45, SELF WEIGHT = 45 PLF

W/ $W_{TRIB} = 10'$,

SELF WEIGHT = 45 PLF / 10' = 4.5 PSF

⟶ DEAD LOAD ASSUMPTION IS OKAY

CAMBER FOR DEAD LOAD DEFLECTION

$\Delta_{MAX_{D+L}} = 5 w_{D+L} \ell^4 / 384EI$

$= (.520\text{ IN})(.540\text{ KLF} / .288\text{ KLF})$

$= .975''$

$\Delta_{CAP_{D+L}} = L/240$

$= (30')(12''/1')/240$

$= 1.5''$

$\Delta_{CAP_{D+L}} = 1.5'' > \Delta_{MAX_{D+L}} = .975''$ ✓ W12X45 ADEQUATE FOR Δ

$\Delta_{MAX_{D+L}} < \Delta_{CAP_{D+L}}$ ⇒ NO CAMBER NECESSARY

CONCLUSION

FOR TYPICAL INTERIOR BEAM @ ROOF USE W12X45 W/ 0" CAMBER

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE – QUEUE BUILDING			Date: 3/8/15
Subject: ROOF INTERIOR GIRER DESIGN	By: JD	Reviewed:	Page: RGD-6

SCHEMATIC

LOADS

LIVE LOAD REDUCTION

$L_r = L_0 R_1 R_2$

R_1: $A_T = (30')(20') = 600 FT^2$

$A_T = 600 FT^2 \geq 600 FT^2$

$R_1 = .6$

$R_1 = .6$

RGD-3 $R_2 = 1$

ASCE 7-10 SEC. 4.8 P. 15

$L_r = L_0 R_1 R_2$

$= (20PSF)(.6)(1)$

$= 12 PSF$

DEAD LOAD:

D: ASSUME WEIGHT OF GIRDER = 1.5PSF + 1.0PSF MISCELLANEOUS

D = 23.5PSF

FACTORED LOADS

DEAD = 1.2(23.5PSF)
= 28.2PSF

LIVE = 1.6(12PSF)
= 19.2 PSF

$w_D = (28.2PSF)(30')$
= 846PLF
= .846KLF

$w_L = (19.2PSF)(30')$
= 576PLF
= .576KLF

RGD-4 $P_D = (3.78k)(2) = 7.56k$

$P_L = (4.32k)(2) = 8.64k$

TOTAL FACTORED LOADS

$w_{TOTAL} = w_D + w_L$
= 846PLF + 576PLF
= 1422PLF
= 1.422 KLF

$P_{TOTAL} = P_D + P_L$
= 7.56k + 8.64k
= 16.2k

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE - QUEUE BUILDING		Date: 3/8/15
Subject: ROOF INTERIOR GIRDER DESIGN	By: JD	Reviewed:	Page: RGD-7

Reference: Check:

SHEAR AND MOMENT DEMAND

$M_{uD} = w_D l^2/8 + P_D l/4$
$= (.846\text{KLF})(20')^2/8 + (7.56\text{K})(20')/4$
$= 80.1\ \text{KFT}$

$V_{uD} = w_D l/2 + P_D/2$
$= (.846\text{KLF})(20')/2 + (7.56\text{K})/2$
$= 12.24\text{K}$

$M_{uL} = w_L l^2/8 + P_L l/4$
$= (.576\text{KLF})(20')^2/8 + (8.64\text{K})(20')/4$
$= 72.0\text{KFT}$

$V_{uL} = w_L l/2 + P_L/2$
$= (.576\text{KLF})(20')/2 + (8.64\text{K})/2$
$= 10.08\text{K}$

$M_{uTOT} = M_{uD} + M_{uL}$
$= 80.1\text{KFT} + 72.0\text{KFT}$
$= 152.1\ \text{KFT}$

$V_{uTOT} = V_{uD} + V_{uL}$
$= 12.24\text{K} + 10.08\text{K}$
$= 22.32\text{K}$

TRY A W12X45

CHECK MOMENT CAPACITY:

RGD-4 $\phi M_{px} = 240.8\text{KFT} > M_u = 152.1\ \text{KFT}$ ✓ W12X45 ADEQUATE FOR M

CHECK SHEAR CAPACITY

RGD-4 $\phi V_{nx} = 121.6\text{K} > 22.32\text{K}$ ✓ W12x45 ADEQUATE FOR V

CHECK DEFLECTION LIMIT

RGD-4 $\Delta_{MAX\,L} = 5w_L l^4/384EI + P_L l^3/48EI$
$= \dfrac{5(.576\text{KLF})(20')^4(12''/1')^3}{384(29000\text{KSI})(348\text{IN}^4)} + \dfrac{(8.64\text{K})(20')^3(12''/1')^3}{48(29000\text{KSI})(348\text{IN}^4)}$
$= .205'' + .2465''$
$= .4515''$

$\Delta_{CAP\,L} = L/360$
$= (20')(12''/1')/360$
$= .667''$

$\Delta_{CAP\,L} = .667'' > \Delta_{MAX\,L} = .4515''$ ✓ W12x45 ADEQUATE FOR Δ

CHECK SELF-WEIGHT ASSUMPTION

RECALL: ASSUMED WEIGHT = 1.5PSF
FOR W12x45, SELF WEIGHT = 45PLF
W/ $W_{TRIB} = 30'$
SELF WEIGHT = 45PLF/30' = 1.5PSF = 1.5PSF
$\Longrightarrow$ DEAD LOAD ASSUMPTION IS OKAY

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

	Project: TRON: THE EXPERIENCE - QUEUE BUILDING			Date: 3/8/15
	Subject: ROOF INTERIOR GIRDER DESIGN	By: JD	Reviewed:	Page: RGD-8

Reference:

Check:

CAMBER FOR DEAD LOAD DEFLECTION

$$\Delta_{MAX_{D+L}} = (.205")\left(\frac{1.422KLF}{.576KLF}\right) + (.2465")\left(\frac{16.2K}{8.64K}\right)$$

$$= .506" + .462"$$

$$= .968"$$

$$\Delta_{CAP_{D+L}} = L/240$$

$$= (20')(12")/240$$

$$= 1"$$

$\Delta_{CAP_{D+L}} = 1" > \Delta_{MAX_{D+L}} = .968"$ ✓ W12X45 ADEQUATE FOR Δ

$\Delta_{MAX_{D+L}} < \Delta_{CAP_{D+L}} \Rightarrow$ NO CAMBER NECESSARY

CONCLUSION

FOR TYPICAL INTERIOR GIRDER @ ROOF USE W12X45 W/ 0" CAMBER

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON : THE EXPERIENCE - QUEUE BUILDING

Date: 3/8/15

Subject: ROOF DEAD LOAD TAKE-OFF

By: JD

Reviewed:

Page: RGD-9

Reference:

Check:

ROOF DEAD LOAD TAKE-OFF (PERIMETER MEMBERS)

MATERIAL	CALCULATIONS	DECKING	BEAMS	GIRDERS	COLUMNS	SEISMIC
FIRESTONE V-FORCE VAPOR BARRIER		.49 PSF	.49PSF	.49PSF		
1" RIGID INSULATION	1.5PSF/1"	1.5PSF	1.5PSF	1.5PSF		
VAPOR LINER SHEET		.18PSF	.18PSF	.18PSF		
1.5" VERCO DECK PLB-36/HSB-36	(SEE PAGE GD-2)	1.9PSF	1.9PSF	1.9PSF		
SPRAY FIREPROOFING		3.0PSF	3.0PSF	3.0PSF		
MEP			5.0PSF	5.0PSF		
SUSP. TBAR CEIL. +ACOUST. FIBER TILES	2PSF + 1PSF		3.0PSF	3.0PSF		
BRICK VENEER FINISH	(SEE PAGE RGD-10)		8.5PSF	8.0PSF	8.5 PSF (conserv.)	
W12 x 45 BEAMS			9.0PSF	9.0PSF	9.0 PSF	
W12 x 45 GIRDERS				3.0PSF	3.0PSF	
W10 x 33 COLUMNS					5.28PSF	
MISCELLANEOUS	2-5% (TOTAL)	.43PSF	1.0PSF	1.0PSF	1.22PSF	
TOTAL W/O BRICK VENEER		7.5PSF	25.5PSF	29.5PSF	30 PSF	

Units Listed: ☐

Numbers Checked: ☐

Sources Referenced: ☐

Project: TRON: THE EXPERIENCE - QUEUE BUILDING

Date: 3/8/15

Subject: BRICK VENEER FINISH WEIGHT

By: JD

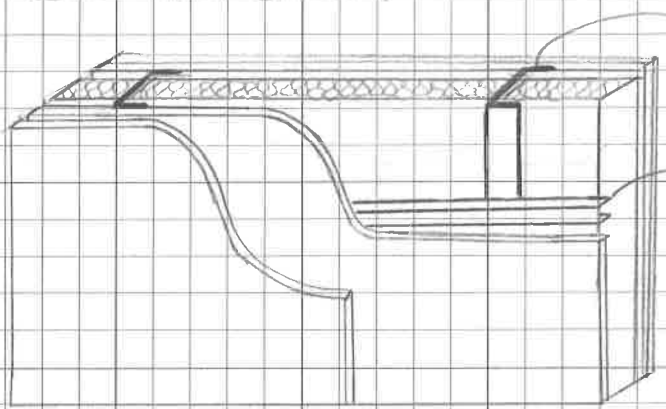
Reviewed:

Page: RGD-10

Reference:

Check:

ASSUME 25PLF SELF-WEIGHT OF CHANNELS SUPPORTING VENEER
(SEE WALL SECTION)



<u>WEIGHT OF SOUTH WALL PER BAY:</u>

2 STUDS, 1 BEAM $\Rightarrow$ 2(20') + (30') = 70'

CHANNEL WEIGHT = 70' (25PLF) = 1750#
= 1750# / A_{WALL}
= 1750# / (30')(20')
= 2.92 PSF

BRICK WEIGHT = 1 PSF

DRYWALL WEIGHT = (2.3PSF)(2 SIDES) = 4.6PSF

TOTAL: ≈ 8.5PSF

<u>WEIGHT OF EAST WALL PER BAY</u>

1 STUD, 1 BEAM $\Rightarrow$ (20') + (20') = 40

CHANNEL WEIGHT = (40')(25PLF) = 1000#
= 1000# / A_{WALL}
= 1000# / (20')(20')
= 2.5PSF

BRICK WEIGHT = 1.0PSF

DRYWALL WEIGHT = (2.3PSF)(2 SIDES) = 4.6PSF

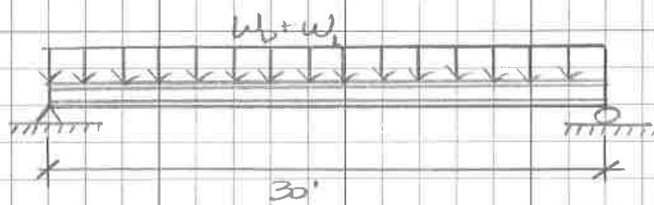
TOTAL ≈ 8.0PSF

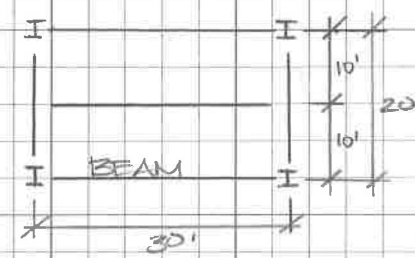
Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

	Project: TRON: THE EXPERIENCE - QUEUE BUILDING			Date: 3/8/15
	Subject: ROOF PERIMETER BEAM DESIGN	By: JD	Reviewed:	Page: RGD-11

Reference: | Check:

SCHEMATIC





LOADS

LIVE LOAD REDUCTION

ASCE 7-10 SEC. 4.8 P.15

$L_r = L_0 R_1 R_2$

R_1: $A_T = (30')(5') = 150FT^2$

$A_T = 150FT^2 < 200FT^2$

$R_1 = 1.0$

$R_1 = 1.0$

R_2: F = 1/2" RISE PER 1' RUN

F = 1/2 < 4

$R_2 = 1$

$R_2 = 1.0$

$L_r = (20PSF)(1.0)(1.0)$
$= 20PSF$

DEAD LOAD:

D: ASSUME WEIGHT OF BEAM = 9.0PSF

D = 25.5PSF (W/O BRICK VENEER)

FACTORED LOADS

DEAD LOAD = 1.2(25.5PSF)
= 30.6 PSF

LIVE LOAD = 1.6(20PSF)
= 32PSF

$w_D = (30.6PSF)(5')$
= 153PLF
= .153KLF

$w_L = (32PSF)(5')$
= 160PLF
= .160KLF

DEAD LOAD OF BRICK VENEER

RGD-9

$w_D = 1.2(8.5PSF)(10')$ — TRIB HEIGHT OF WALL
= 102 PLF = .102KLF

TOTAL FACTORED LOAD

w_{TOTAL} = .153KLF + .160KLF + .102KLF
= .415KLF

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE - QUEUE BUILDING			Date: 3/8/15
Subject: ROOF PERIMETER BEAM DESIGN	By: JD	Reviewed:	Page: RGD-12

Reference: | Check:

SHEAR AND MOMENT DEMAND

$M_{u_D} = w_D l^2/8$
$= (255PLF)(30')^2/8$
$= 28687.5\# FT$
$= 28.7\ KFT$

$V_{u_D} = w_D l/2$
$= (255PLF)(30')/2$
$= 3825\#$
$= 3.83K$

$M_{u_L} = w_L l^2/8$
$= (160PLF)(30')^2/8$
$= 18000\# FT$
$= 18\ KFT$

$V_{u_L} = w_L l/2$
$= (160PLF)(30')/2$
$= 2400\# FT$
$= 2.4K$

$M_{u_{TOT}} = 28.7KFT + 18KFT$
$= 46.7KFT$

$V_{u_{TOT}} = 3.83K + 2.4K$
$= 6.23K$

TRY A W12x45

PROPERTIES:

AISC 2011 P.1-26

$A = 13.1\ IN^2$ $\quad I_x = 348\ IN^4$ $\quad F_y = 50KSI$
$d = 12.1''$ $\quad S_x = 57.7\ IN^3$ $\quad E = 29000KSI$
$t_w = .335''$ $\quad Z_x = 64.2\ IN^3$

CHECK MOMENT CAPACITY:

$\phi M_{px} = \phi F_y Z_x$
$= (.9)(50KSI)(57.7IN^3)$
$= 2596.5\ KIN$
$= 216.4KFT > M_u = 46.7\ KFT$ ✓ W12x45 ADEQUATE FOR M

CHECK SHEAR CAPACITY:

$\phi V_{nx} = .6 F_y A_w$
$= .6 F_y t_w d$
$= .6(50KSI)(.335'')(12.1'')$
$= 121.6\ K > V_u = 6.23K$ ✓ W12x45 ADEQUATE FOR V

CHECK DEFLECTION LIMIT:

$\Delta_{MAX_L} = 5w_L l^4/384EI$
$= [5(.160KLF)(30')^4/384(29000KSI)(348IN^4)](12''/1')^3$
$= .319''$

$\Delta_{CAP_L} = L/360$
$= (30')(12''/1')/360$
$= 1''$

$\Delta_{CAP_L} = 1'' > \Delta_{MAX_L} = .319''$ ✓ W12x45 ADEQUATE FOR Δ

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE – QUEUE BUILDING			Date: 3/8/15
Subject: ROOF PERIMETER BEAM DESIGN	By: JD	Reviewed:	Page: RGD-13

Reference: GD-9

CHECK SELF WEIGHT ASSUMPTION

RECALL: ASSUMED BEAM WEIGHT = 9.0 PSF

FOR W12X45, SELF-WEIGHT = 45 PLF

W/ W_{TRIB} = 5'

SELF WEIGHT = 45 PLF / 5' = 9.0 PSF

⟶ DEAD LOAD ASSUMPTION IS OKAY

CAMBER FOR DEAD LOAD DEFLECTION

$\Delta_{MAX_{D+L}} = 5w_{D+L}\ell^4/384EI$

$= (.319'')(.415\,KLF/.160\,KLF)$

$= .827''$

$\Delta_{CAP_{D+L}} = L/240$

$= (30')(12''/1')/240$

$= 1.5''$

$\Delta_{CAP_{D+L}} = 1.5'' > \Delta_{MAX_{D+L}} = .827''$ ✓ W12x45 ADEQUATE FOR Δ

$\Delta_{CAP_{D+L}} > \Delta_{MAX_{D+L}} \Rightarrow$ NO CAMBER NECESSARY

CONCLUSION:

FOR TYPICAL PERIMETER BEAM @ ROOF USE W12x45 W/ 0" CAMBER

Check:

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE – QUEUE BUILDING

Date: 3/8/15

Subject: ROOF PERIMETER GIRDER DESIGN

By: JD

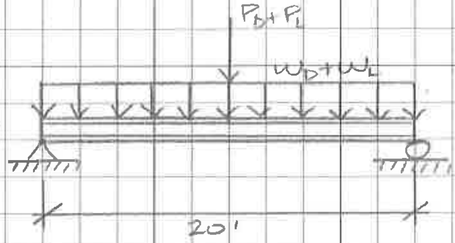
Reviewed:

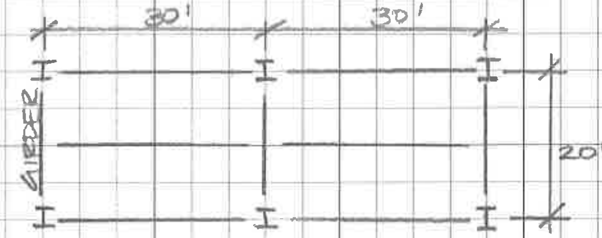
Page: RGD-14

Reference:

Check:

SCHEMATIC





LOADS

LIVE LOAD REDUCTION:

$L_r = L_o R_1 R_2$

R_1: $A_T = (20')(15') = 300FT^2$

$200FT^2 < 300FT^2 < 600FT^2$

$R_1 = 1.2 - .001 A_T$

$= 1.2 - .001(300FT^2)$

$= .9$

$R_1 = .9$

$R_2 = 1$

$L_r = L_o R_1 R_2$

$= (20PSF)(.9)(1.0)$

$= 18PSF$

DEAD LOAD:

D: ASSUME WEIGHT OF GIRDER = 3.0PSF

D = 29.5PSF

FACTORED LOADS

DEAD = 1.2(29.5PSF)

= 35.4 PSF

$w_D = (35.4PSF)(15')$

= 531 PLF

= .531 KLF

RGD-4 P_D = 3.78K

LIVE = 1.6(18PSF)

= 28.8 PSF

$w_L = (28.8PSF)(15')$

= 432 PLF

= .432 KLF

P_L = 4.32K

TOTAL FACTORED LOADS

$w_{TOTAL} = w_D + w_L + w_D$

= 531PLF + 432PLF + 96PLF

= 1059 PLF

= 1.059 KLF

BRICK VENEER WEIGHT

$w_D = 1.2(8.0PSF)(10')$

= 96PLF

= .096 KLF

P_{TOTAL} = 3.78K + 4.32K

= 8.1K

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE - QUEUE BUILDING			Date: 3/8/15
Subject: ROOF PERIMETER GIRDER DESIGN	By: JD	Reviewed:	Page: RGD-15

Reference: | Check:

SHEAR AND MOMENT

$M_{u_D} = w_D l^2/8 + P_D l/4$
$= \frac{(.627 KLF)(20')^2}{8} + \frac{(3.78K)(20')}{4}$
$= 50.3 KFT$

$V_{u_D} = w_D l/2 + P_D/2$
$= \frac{(.627 KLF)(20')}{2} + \frac{(3.78K)}{2}$
$= 8.16K$

$M_{u_L} = w_L l^2/8 + P_L l/4$
$= \frac{(.432 KLF)(20')^2}{8} + \frac{(4.32K)(20')}{4}$
$= 43.2 KFT$

$V_{u_L} = w_L l/2 + P_L/2$
$= \frac{(.432 KLF)(20')}{2} + \frac{(4.32K)}{2}$
$= 6.48K$

$M_{u_{TOT}} = 50.3 KFT + 43.2 KFT$
$= 93.5 KFT$

$V_{u_{TOT}} = 8.16K + 6.48K$
$= 14.64K$

TRY A W12 X 45

PROPERTIES:

AISC 2011 P.1-26

$A = 13.1 \ IN^2$ $I_x = 348 \ IN^4$ $F_y = 50 KSI$
$d = 12.1 \ IN$ $S_x = 57.7 \ IN^3$ $E = 29000 KSI$
$t_w = .335''$ $Z_x = 64.2 \ IN^3$

CHECK MOMENT CAPACITY:

$\phi M_{px} = \phi F_y Z_x$
$= (.9)(50 KSI)(64.2 \ IN^3)$
$= 2889 \ KIN$
$= 240.8 \ KFT > M_u = 93.5 KFT$ ✓ W12X45 ADEQUATE FOR M

CHECK SHEAR CAPACITY:

$\phi V_{nx} = .6 F_y A_w$
$= .6 F_y t_w d$
$= .6 (50 KSI)(.335'')(12.1'')$
$= 121.6K > V_u = 14.64K$ ✓ W12X45 ADEQUATE FOR V

CHECK DEFLECTION LIMITS:

$\Delta_{MAX_L} = 5 w_L l^4/384EI + P_L l^3/48EI$
$= \frac{5(.432 KLF)(20')^4(12''/1')^3}{384(29000 KSI)(307 IN^4)} + \frac{(4.32K)(20')^3(12''/1')^3}{48(29000 KSI)(307 IN^4)}$
$= .1540'' + .1232''$
$= .2772''$

$\Delta_{CAP_L} = L/360$
$= (20')(12''/1')/360$
$= .667''$

$\Delta_{CAP_L} = .667'' > \Delta_{MAX_L} = .2772''$ ✓ W12X45 ADEQUATE FOR Δ

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE - QUEUE BUILDING

Date: 3/8/15

Subject: ROOF PERIMETER GIRDER DESIGN

By: JD

Reviewed:

Page: RGD-16

Reference:

Check:

CHECK SELF-WEIGHT ASSUMPTION

RGD-12 RECALL: ASSUMED WEIGHT = 3.0

FOR W10X40, SELF-WEIGHT = 45 PLF

W/ W_{TRIB} = 15'

SELF WEIGHT = 45PLF/15' = 3.0 PSF = 3.0PSF

$\Longrightarrow$ DEAD LOAD ASSUMPTION IS OKAY

CAMBER FOR DEAD LOAD DEFLECTION

$\Delta_{MAX_{D/L}} = (.1540'')\dfrac{(1.059 KLF)}{(.432 KLF)} + (.1232'')\dfrac{(8.1K)}{(4.32K)}$

$= .3775'' + .231''$

$= .6085''$

$\Delta_{CAP_{D/L}} = L/240$

$= (20')(12''/1')/240$

$= 1''$

$\Delta_{CAP_{D/L}} = 1'' > \Delta_{MAX_{D/L}} = .6085''$

$\Delta_{CAP_{D/L}} > \Delta_{MAX_{D/L}} \Longrightarrow$ NO CAMBER NECESSARY

CONCLUSION

FOR TYPICAL PERIMETER GIRDER @ ROOF USE W12x45 W/ 0'' CAMBER

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE - QUEUE BUILDING			Date: 3/8/15
Subject: BRICK VENEER FINISH WEIGHT	By: JD	Reviewed:	Page: RGD-17

Reference:

Check:

@ CORNER GIRDER

APPROXIMATELY 28.3'

2 STUDS, 1 BEAM $\Rightarrow$ 2(20') + (30.0') = 70'

CHANNEL WEIGHT = 70'(25PLF) = 1750#

= 1750# / A_{WALL}

= 1750# / (30.0')(20')

= 2.92 PSF

BRICK WEIGHT = 1 PSF

DRYWALL WEIGHT = (2.3PSF)(2 SIDES) = 4.6PSF

TOTAL: ≈ 8.5PSF

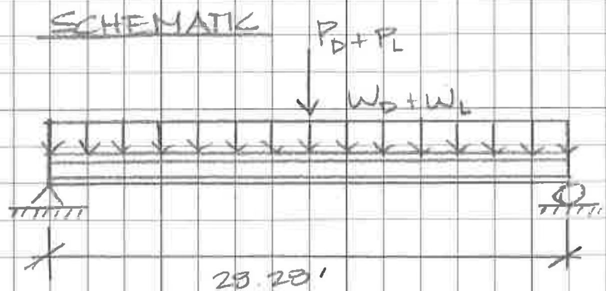
Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

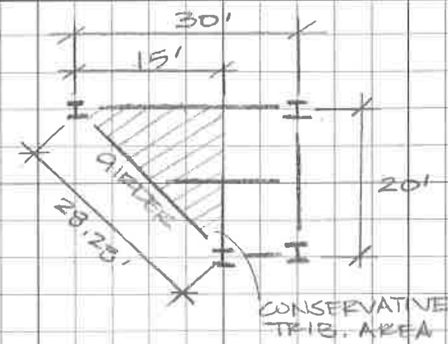
Project: TRON: THE EXPERIENCE – QUEUE BUILDING	Date: 3/8/15
Subject: ROOF CORNER GIRDER DESIGN	By: JD · Reviewed: · Page: RGD-18

Reference:

Check:

SCHEMATIC





LOADS

LIVE LOAD REDUCTION:

$L_r = L_0 R_1 R_2$

$R_1: A_T = \frac{1}{2}(15')(20') = 150\,FT^2$ (CONSERVATIVELY)

$A_T = 150\,FT^2 \leq 200\,FT^2$

$R_1 = 1$

$R_2 = 1$

$L_r = 20\,PSF$

DEAD LOAD:

D: ASSUME WEIGHT OF GIRDER = 3.2 PSF

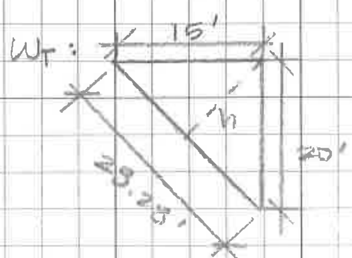
D = 29.7 PSF

FACTORED LOADS

DEAD = 1.2 (29.7 PSF)
= 35.6 PSF

LIVE = 1.6 (20 PSF)
= 32 PSF

W_T:



$20^2 = \sqrt{(14.14')^2 + (h)^2}^{\,2}$
$= 14.14'$

$W_T = 15'$ (CONSERVATIVELY)

$W_D = (35.6\,PSF)(15')$
= 534 PLF
= .534 KLF

$W_L = (32\,PSF)(15')$
= 480 PLF
= .480 KLF

RGD-4 $P_D = 3.78K$

$P_L = 4.32K$

TOTAL FACTORED LOADS

W_{TOTAL} = .534KLF + .480KLF + .102KLF
= 1.116 KLF
= 1116 PLF

WEIGHT OF VENEER

$W_D = 1.2(8.5PSF)(10')$
= 102 PLF
= .102 KLF

P_{TOTAL} = 3.78K + 4.32K = 8.1K

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE – QUEUE BUILDING			Date: 3/8/15
Subject: ROOF CORNER GIRDER DESIGN	By: JD	Reviewed:	Page: RGD-19

Reference: | Check:

SHEAR AND MOMENT DEMAND

$M_{uD} = w_D l^2/8 + P_D l/4$
$= (.636KLF)(28.28')^2/8 + (3.73K)(28.28')/4$
$= 90.3KFT$

$V_{uD} = w_D l/2 + P_D/2$
$= (.636KLF)(28.28')/2 + (3.73K)/2$
$= 10.9K$

$M_{uL} = w_L l^2/8 + P_L l/4$
$= (.48KLF)(28.28')^2/8 + (4.32K)(28.28')/4$
$= 78.5KFT$

$V_{uL} = w_L l/2 + P_L/2$
$= (.48KLF)(28.28')/2 + (4.32K)/2$
$= 8.9K$

$M_{uTOTAL} = 90.3KFT + 78.5KFT$
$= 168.8KFT$

$V_{uTOTAL} = 10.9K + 8.9K$
$= 19.8K$

TRY A W14 X 48

PROPERTIES:

AISC 2011 P.1-26

$A = 14.1 IN^2$ $I_x = 484 IN^4$ $E = 29000KSI$
$d = 13.8 IN$ $S_x = 70.2 IN^3$ $F_y = 50KSI$
$t_w = .34 IN$ $Z_x = 78.4 IN^3$

CHECK MOMENT CAPACITY:

$\phi M_{px} = \phi F_y Z_x$
$= (.9)(50KSI)(70.2IN^3)$
$= 3159 \ KIN$
$= 263.3 \ KFT > 168.8 \ KFT$ ✓ W14 X 48 ADEQUATE FOR M

CHECK SHEAR CAPACITY:

$\phi V_{nx} = .6 F_y A_w$
$= .6 F_y d t_w$
$= .6(50KSI)(14.1")(.34")$
$= 143.8 \ K > 19.7K$ ✓ W14 X 48 ADEQUATE FOR V

CHECK DEFLECTION LIMIT:

$\Delta_{MAX_L} = 5 w_L l^4/384EI + P_L l^3/48EI$
$= \dfrac{5(.48KLF)(28.28')^4(12''/1')^3}{384(29000KSI)(484IN^4)} + \dfrac{(4.32K)(28.28')^3(12''/1')^3}{48(29000KSI)(484IN^4)}$
$= .492'' + .251''$
$= .743$

$\Delta_{CAP_L} = L/360$
$= (28.28')(12''/1')/360$
$= .943''$

$\Delta_{CAP_L} = .943'' > \Delta_{MAX_L} = .743''$ ✓ W14X48 ADEQUATE FOR Δ

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE - QUEUE BUILDING			Date: 3/8/15
Subject: ROOF CORNER GIRDER DESIGN	By: JD	Reviewed:	Page: RGD-20

Reference: RGD-15

CHECK SELF-WEIGHT ASSUMPTION

RECALL: ASSUMED WEIGHT = 3.2 PSF

FOR W14X48, SELF-WEIGHT = 48 PLF

W/ W_{TRIB} = 15'

SELF WEIGHT = 48 PLF / 15' = 3.2 PSF = 3.2 PSF

$\Longrightarrow$ DEAD LOAD ASSUMPTION IS OKAY

CAMBER FOR DEAD LOAD

$\Delta_{MAX_{D+L}} = (.492'')\dfrac{(1.116\,KLF)}{(.48\,KLF)} + (.251'')\dfrac{(8.1K)}{(4.32K)}$

$= 1.144'' + .471''$

$= 1.615''$

$\Delta_{CAP_{D+L}} = L/240$

$= (28.28')(12''/1')/240$

$= 1.414''$

$\Delta_{CAP_{D+L}} = 1.414'' \ngtr \Delta_{MAX_L} = 1.615''$

$\Delta_{MAX_D} = \Delta_{MAX_{D+L}} - \Delta_{MAX_L}$

$= 1.615'' - .743''$

$= .872''$

$\Longrightarrow$ CAMBER .75''

CONCLUSION

FOR GIRDER AT CORNER @ ROOF, USE W14X48 W/ .75" CAMBER

Check:

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: The Experience - Queue Building	Date: 3/8/15

Subject: Column Design (Interior)	By: AV	Reviewed:	Page: RGD-21

Reference: | Check:

Roof to Ground Floor

$P_D = A_T(D) = (20' \times 30')(25.0 \text{ psf})$
$= 15000\#$

$P_L = A_T(L_r)$

$\Rightarrow L_r = R_1(L)$, $R_1 = 0.6$ w/ $A_T = 600$ sf
$= 0.6(20 \text{ psf})$ (ASCE 7-10 - 4.8.2)
$= 12 \text{ psf}$

$P_L = (20' \times 30')(12 \text{ psf}) = 7200\#$

$P_{R,tot} = 1.2D + 1.6L$
$= 1.2(15000\#) + 1.6(7200\#)$
$= 30{,}240\# = 30.2K$

Pinned-Pinned Condition $\Rightarrow K = 1.0$
$\Rightarrow L = 24'$

W/ $KL = 24'$ & $P_{R,tot} = 30.2K$

$\boxed{\text{Use W10X33}}$ w/ $\phi P_n = 143K$

Units Listed: ☐	Numbers Checked: ☐	Sources Referenced: ☐

Project: TRON: The Experience – Queue Building			Date: 4/12/15
Subject: Column Design (Exterior)	By: AB	Reviewed:	Page: RGD-22

Reference: | Check:

Roof to Ground Floor

$P_D = (A_T)(D) = (10' \times 15')(36\,psf) = 5400\#$

$P_{brick} = (10' \times 15')(8.5\,psf) = 1275\#$

$P_L = A_T(L_r)$

ASCE 7-10 4.8.2 — $\Rightarrow L_r = R_1(L)$ $\quad R_1 = 1$ w/ $A_T = 150\,ft^2$

$= 20\,psf$

$P_L = (10' \times 15')(20\,psf) = 3000\#$

ASCE 7-10 2.3.2 — $P_{r,tot} = 1.2D + 1.6L = 1.2(5400\# + 1275\#) + 1.6(3000\#)$

$= 12{,}810\# = 12.8^k$

Pinned-Pinned Cond. $\Rightarrow$ $K = 1.0$
$L = 24'$

w/ $KL = 24'$ & $P_{r,tot} = 12.8^k$ $\Rightarrow$ $\boxed{\text{use W}10\times33}$ w/ $\phi P_n = 143^k$

Units Listed: ☐	Numbers Checked: ☐	Sources Referenced: ☐

Project: TRON: THE EXPERIENCE - QUEUE BUILDING

Date: 3/8/15

Subject: FLOOR DEAD LOAD TAKE OFF

By: JD

Reviewed:

Page: F GD-1

Reference:

Check:

FLOOR DEAD LOAD TAKE-OFF (INTERIOR MEMBERS)

MATERIAL	CALCULATIONS	DECKING	BEAMS	GIRDERS	COLUMNS	SEISMIC
LINOLEUM FLOOR		1.0PSF				
NW CONCRETE FILL	(SEE PAGE)	36.3PSF				
20 GAGE PLW2 OR W2 FORMLOK DECK	(SEE PAGE GD-)	2.1PSF				
SPRAY FIREPROOFING		3.0PSF				
MEP			5.0PSF			
SUSP. T-BAR CEILING +ACOUST. FIBER TILES	2PSF+1PSF		3.0PSF			
W 18 x 71 BEAMS			7.1PSF			
W 18 x 97 GIRDERS				3.23PSF		
W 10 x 39 COLUMNS					0.65PSF	
MISCELLANEOUS	2-5% (TOTAL)	2.6PSF	2.9PSF	3.77PSF	1.35PSF	
PREVIOUS MEMBER TOTAL			45PSF	63PSF	70PSF	
TOTAL		45PSF	63PSF	70PSF	75PSF	

Units Listed: ☐

Numbers Checked: ☐

Sources Referenced: ☐

	Project: TRON: THE EXPERIENCE - QUEUE BUILDING			Date: 3/8/15
	Subject: FLOOR DECK DESIGN	By: JD	Reviewed:	Page: FGD-2

Reference:

GD-

LOADING

LIVE: 100PSF

DEAD: 45 PSF

TOTAL = (100 + 45 PSF) = 145 PSF

SPAN = 10'-0"

USE

VERCO STEEL FLOOR DECKS CATALOG P. 45

20 GAGE PLW2 OR W2 FORMLOK, 4" DEPTH

4" DEPTH, TRIPLE SPAN

GALV WEIGHT = 2.1 PSF

CONCRETE = 36.3 PSF

NO SHORING REQUIRED

Check:

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE – QUEUE BUILDING

Date: 3/8/15

Subject: FLOOR INTERIOR BEAM DESIGN

By: JD

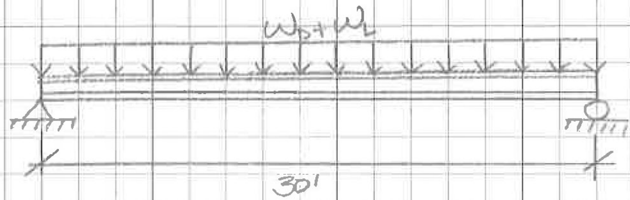
Reviewed:

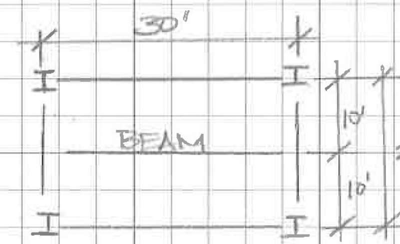
Page: FGD-3

Reference:

Check:

SCHEMATIC





LOADS

ASCE 7-10 SEC. 4.7 P.15

LIVE LOAD REDUCTION:

$L = L_o[.25 + (15/\sqrt{K_{LL}A_T})]$

$L_o = 100PSF$

$A_T = (10')(30') = 300FT^2 < 400FT^2$

$\Longrightarrow$ LIVE LOAD NEED NOT BE REDUCED

$L = 100PSF$

FGD-1

DEAD LOAD = 64PSF

* ASSUME 8PSF SELF WEIGHT OF BEAM

FACTORED LOADS

DEAD = 1.2(64PSF)
= 76.8PSF

LIVE = 1.6(100PSF)
= 160PSF

w_D = (76.8PSF)(10')
= 768PLF
= .768KLF

w_L = (160PSF)(10')
= 1600PLF
= 1.60KLF

TOTAL FACTORED LOADS

$w_{TOTAL} = w_D + w_L$
= 768PLF + 1600PLF
= 2368PLF
= 2.37KLF

MOMENT DEMAND

$M_{uD} = w_D l^2/8$
$= (768PLF)(30')^2/8$
= 86400#FT
= 86.4KFT

$M_{uL} = w_L l^2/8$
$= (1600PLF)(30')^2/8$
= 180000#FT
= 180KFT

$M_{uTOT} = M_{uD} + M_{uL}$
= 86400#FT + 180000#FT
= 266400#FT
= 266.4KFT

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE - QUEUE BUILDING			Date: 3/8/15
Subject: FLOOR INTERIOR BEAM DESIGN	By: JD	Reviewed:	Page: FGD-4

Reference: | Check:

SHEAR DEMAND

$V_{u_D} = w_D l/2$
$= (768PLF)(30')/2$
$= 11520\#$
$= 11.52K$

$V_{u_L} = w_L l/2$
$= (1600PLF)(30')/2$
$= 24000\#$
$= 24.0K$

$V_{u_{TOT}} = V_{u_D} + V_{u_L}$
$= 11520\# + 24000\#$
$= 35520\#$
$= 35.52K$

TRY A W18x71

PROPERTIES:

AISC 2011 P. 1-20

$A = 20.9 IN^2$ $I_x = 1170 IN^4$ $E = 29000KSI$
$d = 18.5 IN$ $S_x = 127 IN^3$ $F_y = 50KSI$
$t_w = .495 IN$ $Z_x = 146 IN^3$

CHECK MOMENT CAPACITY:

$\phi M_{px} = \phi F_y Z_x$
$= (.9)(50KSI)(146IN^3)$
$= 6570 KIN$
$= 547.5 KFT > M_u = 296.4 KFT$ ✓ W18x71 ADEQUATE FOR M

CHECK SHEAR CAPACITY

$\phi V_{nx} = .6 F_y A_w$
$= .6 F_y t_w d$
$= .6(50KSI)(.495'')(18.5'')$
$= 274.7K > V_u = 35.52K$ ✓ W18x71 ADEQUATE FOR V

CHECK DEFLECTION LIMIT:

$\Delta MAX_L = 5 w_L l^4 / 384EI$
$= 5(1.60KLF)(30')^4(12''/1')^3 / 384(29000KSI)(1170 IN^4)$
$= .859''$

$\Delta CAP_L = L/360$
$= (30')(12''/1')/360$
$= 1''$

$\Delta CAP_L = 1'' > \Delta MAX_L = .859''$ ✓ W18x71 ADEQUATE FOR Δ

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE – QUEUE BUILDING			Date: 3/8/15
Subject: FLOOR INTERIOR BEAM DESIGN	By: JD	Reviewed:	Page: FGD-5

Reference: FGD-3

Check:

CHECK SELF WEIGHT ASSUMPTION

RECALL: ASSUME SELF WEIGHT OF BEAM = 8.0PSF

FOR W18X71, SELF WEIGHT = 71PLF

W/ W_{TRIB} = 10'

SELF WEIGHT = 71PLF/10' = 7.1PSF < 8.0PSF

$\Longrightarrow$ DEAD LOAD ASSUMPTON IS OKAY

CAMBER FOR DEAD LOAD DEFLECTION

$\Delta MAX_{D+L} = 5w_{D+L}\ell^4/384EI = (\Delta MAX_L)(w_{TOTAL})/(w_L)$

$= (.859'')(2.37KLF/1.40KLF)$

$= 1.27''$

$\Delta CAP_{D+L} = L/240$

$= (30')(12'')/240$

$= 1.5''$

$\Delta CAP_{D+L} = 1.5'' > \Delta MAX_{D+L} = 1.27''$ ✓ W18X71 ADEQUATE FOR Δ

$\Delta MAX_D = \Delta MAX_{D+L} - \Delta MAX_L$

$= 1.27'' - .859''$

$= .413'' \Longrightarrow$ CAMBER BEAM .25"

CONCLUSION

FOR TYPICAL INTERIOR BEAM @ FLOOR USE W18X71 W/ .25" CAMBER

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE – QUEUE BUILDING

Date: 3/8/15

Subject: FLOOR INTERIOR GIRDER DESIGN

By: JD

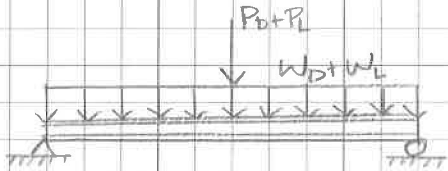
Reviewed:

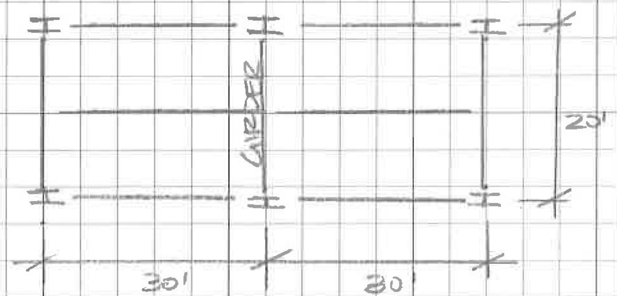
Page: FGD-6

Reference:

Check:

SCHEMATIC:





LOADS

LIVE LOAD REDUCTION:

$L = L_o[.25 + (15/\sqrt{K_{LL}A_T})]$

$L_o = 100PSF$

$A_T = 20' \times 30' = 600FT^2 \geq 400FT^2$

$\Longrightarrow$ LIVE LOAD SHALL BE REDUCED

$L = (100PSF)[.25 + (15/\sqrt{(2)(600FT^2)})]$

$= 68.3PSF$

DEAD LOAD = 70PSF

* ASSUME GIRDER SELF-WEIGHT = 35PSF

FACTORED LOADS

DEAD = 1.2(70PSF)
= 84.0 PSF

LIVE = 1.6(68.3PSF)
= 109.3PSF

$W_D = (84.0PSF)(30')$
$= 2520PLF$
$= 2.520KLF$

$W_L = (109.3PSF)(30')$
$= 3278PLF$
$= 3.28KLF$

FGD-4 $P_D = 2(11.52K)$
$= 23.04K$

$P_L = 2(24.0K)$
$= 48K$

TOTAL FACTORED LOADS

$W_{TOTAL} = W_D + W_L$
$= 2520PLF + 3278PLF$
$= 5798PLF$
$= 6.798KLF$

$P_{TOTAL} = P_D + P_L$
$= 23.04K + 48K$
$= 71.24K$

MOMENT DEMAND

$M_{uD} = W_D \ell^2/8 + P_D \ell/4$
$= (2.52KLF)(20')^2/8 + (23.04K)(20')/4$
$= 241.2KFT$

$M_{uL} = W_L \ell^2/8 + P_L \ell/4$
$= (3.278KLF)(20')^2/8 + (48K)(20')/4$
$= 403.9KFT$

$M_{uTOT} = 241.2KF + 403.9KFT = 645.1KFT$

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: THE EXPERIENCE - QUEUE BUILDING			Date: 3/3/15
Subject: FLOOR INTERIOR GIRDER DESIGN	By: JD	Reviewed:	Page: FGD-7

Reference:

Check:

SHEAR DEMAND

$V_{u_D} = w_D l/2 + P_D/2$
$= (2.32\ KLF)(20')/2 + (23.04K)/2$
$= 36.72K$

$V_{u_L} = w_L l/2 + P_L/2$
$= (3.278\ KLF)(20')/2 + (48K)/2$
$= 56.78K$

$V_{u_{TOTAL}} = 36.72K + 56.78K = 93.5K$

TRY A W18x97

PROPERTIES:

AISC 2011 P.1-20

$A = 28.5 IN^2$ $\quad I_x = 1750 IN^4$ $\quad E = 29000 KSI$
$d = 18.6 IN.$ $\quad S_x = 188 IN^3$ $\quad F_y = 50 KSI$
$t_w = .535 IN.$ $\quad Z_x = 211 IN^3$

CHECK MOMENT CAPACITY:

$\phi M_{px} = \phi F_y Z_x$
$= .9(50KSI)(211IN^3)$
$= 9495 KIN$
$= 791.25 KFT \geq M_u = 645.1 KFT$ ✓ W18x97 ADEQUATE FOR M

CHECK SHEAR CAPACITY:

$\phi V_{nx} = .6 F_y A_w$
$= .6 F_y t_w d$
$= .6(50KSI)(.535)(18.6")$
$= 298.53K > 93.5K$ ✓ W18x97 ADEQUATE FOR V

CHECK DEFLECTION LIMIT:

$\Delta_{MAX_L} = 5w_L l^4/384EI + P_L l^3/48EI$
$= \dfrac{5(3.278 KLF)(20')^4(12"/1')^3}{384(29000KSI)(1750IN^4)} + \dfrac{(48K)(20')^3(12"/1')^3}{48(29000KSI)(1750IN^4)}$
$= .2325" + .2724"$
$= .505"$

$\Delta_{CAP_L} = L/360$
$= (20')(12"/1')/360$
$= .667"$

$\Delta_{CAP_L} = .667" > \Delta_{MAX_L} = .505"$ ✓ W18x97 ADEQUATE FOR Δ

CHECK SELF WEIGHT ASSUMPTION

RECALL: ASSUMED WEIGHT = 3.5
FOR W18x97, SELF WEIGHT = 97PLF
W/ $W_{TRIB} = 30'$
SELF-WEIGHT = 97PLF/30' = 3.23PSF < 3.5PSF
⟹ DEAD LOAD ASSUMPTION IS OKAY

Units Listed: ☐	Numbers Checked: ☐	Sources Referenced: ☐

Project: TRON: THE EXPERIENCE - QUEUE BUILDING

Date: 3/3/15

Subject: FLOOR INTERIOR GIRDER DESIGN

By: JD

Reviewed:

Page: FGD-8

Reference:

Check:

CAMBER FOR DEAD LOAD DEFLECTION

$\Delta MAX_{D+L} = 5w_{D+L}\ell^4/384EI + P_{D+L}\ell^3/48EI$

$= (.2325")\frac{(6.798 KLF)}{(3.278 KLF)} + (.2724")\frac{(71.24K)}{(48K)}$

$= .4822" + .4043"$

$= .886"$

$\Delta CAP_{D+L} = L/240$

$= (20')(12"/1')/240$

$= 1"$

$\Delta CAP_{D+L} = 1" > \Delta MAX_{D+L} = .886"$ ✓ W18x97 ADEQUATE FOR Δ

$\Delta MAX_D = \Delta MAX_{D+L} - \Delta MAX_L$

$= .886" - .505"$

$= .381" \Longrightarrow$ CAMBER .25"

CONCLUSION:

FOR TYPICAL INTERIOR GIRDER @ FLOOR USE W18x97 W/ .25" CAMBER

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: The Experience - Queue Building	Date: 3/18/15
Subject: Floor Dead Load Take-OFF	By: AV
Reviewed:	Page: FGD-9

Reference:

Floor Dead Load Take-OFF (Exterior Members)

Material	Calculations	Decking	Beams	Girders	Columns	Seismic
Linoleum Floor		1.0 psf				
NW Concrete Fill		36.3 psf				
20 Gage PLW2 or W2 Formlock Deck		2.1 psf				
Spray Fireproofing		3.0 psf				
MEP			5.0 psf			
Susp. T-Bar Ceiling + Acoust. Fiber Tiles	2 psf + 1 psf		3.0 psf			
Brick Veneer Finish			6.9 psf	8 psf		
W 18 x 35 Beams			7 psf			
W 24 x 62 Girders				1.13 psf		
W 10 x 33 Columns					1.1 psf	
Miscellaneous	2-5% (Total)	2.0 psf	3 psf	2.87 psf	2.9 psf	
Previous Member Total			45 psf	63 psf	70 psf	
Total w/o Brick Veneer		45 psf	63 psf	70 psf	74 psf	

Check:

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: The Experience - Queue Building

Date: 3/8/15

Subject: Floor Perimeter Beam Design

By: AV

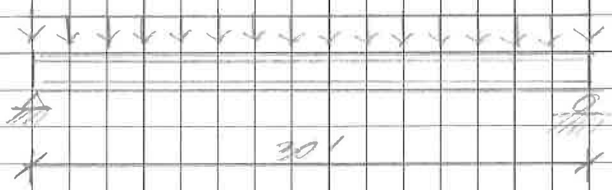
Reviewed:

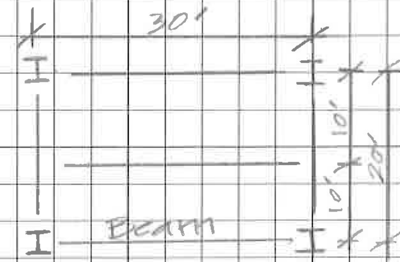
Page: FGD-10

Reference:

Check:

Schematic





Loads

ASCE 7-10 Sec. 4.7

Live Load Reduction:

$L = L_o[0.25 + (15/\sqrt{K_{LL}A_T})]$

$L_o = 100psf$

$A_T = 30'(5') = 150 ft^2 < 400 ft^2$ ✓

⇒ Reduction not needed

$L = \underline{100psf}$

Dead Load

FGD-9

$D = \underline{64psf}$ * Assume 8psf Self-Weight

see → FGD-10

Factored Loads

$\omega_D = 1.2(D)(\omega_T)$
$= 1.2(64psf)(5')$
$= 384 plf$
$= 0.384 klf$

(Brick Veneer)

$\omega_D = 1.2(8.5psf)(10')$
$= 102 plf = 0.102 klf$

$\omega_L = 1.6(L)(\omega_T)$
$= 1.6(100psf)(5')$
$= 800 plf$
$= 0.80 klf$

$\omega_{tot} = \omega_D + \omega_L$
$= 0.102 klf + 0.384 klf + 0.80 klf$
$= 1.286 klf$

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: The Experience - Queue Building

Date: 3/8/15

Subject: Floor Perimeter Beam Design

By: AV

Reviewed:

Page: FGD-11

Reference:

Check:

Moment Demand

$M_{u,tot} = \omega_D l^2/8 + \omega_L l^2/8$
$= (0.486KLF)(30')^2/8 + (0.80KLF)(30')^2/8$
$= 144.68$ K-ft

Shear Demand

$V_{u,tot} = \omega_D(l/2) + \omega_L(l/2)$
$= 0.486KLF(30'/2) + 0.80KLF(30'/2)$
$= 19.29$ K

Try a W18X35

AISC-11 T.1-1

Properties:

$A = 10.3\ in^2$ $I_x = 510\ in^4$ $E = 29000$ ksi
$d = 17.7$ in $S_x = 57.6\ in^3$ $F_y = 50$ ksi
$t_w = 0.30$ in $Z_x = 66.5\ in^3$

Check Moment Capacity

$\phi M_{px} = \phi F_y Z_x$
$= (0.9)(50ksi)(66.5 in^3)$
$= 2993$ k-in $(1/12)$
$= 249$ k-ft $> M_u = 145$ K-ft ✓ Adequate for M

Check Shear Capacity

$\phi V_{nx} = 0.6 F_y A_w$
$= 0.6(50ksi)(10.3 in^2)$
$= 309$ K $> V_u = 19$ K ✓ Adequate for V

Check Deflection Limit

$\Delta_{Max_L} = 5\omega_L l^4/384EI$
$= 5(0.80KLF)(30')^4(12)^3/384(29000ksi)(510 in^4)$
$= 0.986$ in

$\Delta_{cap_L} = L/360$
$= (30')(12''/1')/360$
$= 1''$

$\Delta_{cap_L} = 1'' > \Delta_{Max_L} = 0.986''$ ✓

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: The Experience – Queue Building			Date: 3/8/15
Subject: Floor Perimeter Beam Design	By: AY	Reviewed:	Page: FGD-12

Reference:

Check Self Weight Assumption

Assume Self Weight of Beam = 8 psf

(FGD-10) For W 18X35, Self-Weight = 35 plf

$w/_{\text{trib}} = 5'$

Self Weight = 35 plf / 5' = 7 psf

⇒ Dead Load Assumption is okay

Camber for Dead Load Deflection

$\Delta_{max\ D+L} = 5 w_{D+L}(l^4)/384EI = \Delta_{max\ L}(w_{tot}/w_L)$

$= 0.920''(0.8\ klf/1.200\ klf)$

$= 0.613$

$\Delta_{cap\ D+L} = L/240$

$= (30')(12''/1')/240$

$= 1.5''$

$\Delta_{cap\ D+L} = 1.5'' > \Delta_{max\ D+L} = 0.613''$ ✓ Adequate for Δ

⇒ No Camber Required

Conclusion

For typical perimeter beam at floor use W 18X35

Units Listed: ☐	Numbers Checked: ☐	Sources Referenced: ☐

Check:

Project: TRON: The Experience - Queue Building			Date: 3/8/15
Subject: Floor Perimeter Girder Design	By: AV	Reviewed:	Page: FGD-13

Reference: | Check:

Schematic

Loads

ASCE 7-10 Sec. 4.7

Live Load Reduction

$L = L_o\,[0.25 + (15/\sqrt{K_{LL} A_T})]$

$L_o = 100$ psf

$A_T = 20'(15') = 300\ ft^2 < 400\ ft^2$

$\Rightarrow$ Reduction not needed

$L = \underline{100\text{ psf}}$

Dead Load

$D = \underline{70\text{ psf}}$ (63psf + 3.5psf + 3.5psf) ← see Dead Load take-off

see → FGD-10

Factored Loads * $W_{D\,brick} = 1.2(8\text{ psf})(10') = 96$ plf

$\omega_D = 1.2(D)(w_t)$
$= 1.2(70\text{psf})(15') + 96\text{plf}$
$= 1.36$ klf

$\omega_L = 1.6(L)(w_t)$
$= 1.6(100\text{ psf})(15')$
$= 2.4$ klf

$\omega_{tot} = \omega_D + \omega_L = 1.36\text{klf} + 2.4\text{klf} = 3.76$ klf

$P_D = 2(11.52k)$
$= 23.04k$

$P_L = 2(24k)$
$= 48k$

$P_{tot} = P_D + P_L = 23.04k + 48k = 71.04k$

Moment Demand

$M_{uD} = \omega_D l^2/8 + P_D l/4$
$= 1.35\text{klf}(20')^2/8 + 23.04k(20')/4$
$= 182.7$ k-ft

$M_{uL} = \omega_L l^2/8 + P_L l/4$
$= 2.4\text{klf}(20')^2/8 + 48k(20')/4$
$= 360$ k-ft

$M_{u\,tot} = 183\text{k-ft} + 360\text{k-ft} = 543$ k-ft

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: the Experience - Queue Building | Date: 3/13/15

Subject: Floor Perimeter Girder Design | By: AV | Reviewed: | Page: FGD-14

Reference: | Check:

Shear Demand

$V_{uD} = \omega_D l/2 + P_D/2$
$= 1.35 k/ft (20')/2 + 23.04 k/2$
$= 25 k$

$V_{uL} = \omega_L l/2 + P_L/2$
$= 2.4 k/ft (20')/2 + 48 k/2$
$= 48 k$

$V_{utot} = 25k + 48k = 73k$

Try A W24X62

AISC-11 T1.1

Properties

$A = 18.2 in^2$ $\quad I_x = 1550 in^4$ $\quad E = 29000 ksi$
$d = 23.7 in$ $\quad S_x = 131 in^3$ $\quad F_y = 50 ksi$
$t_w = 0.43 in$ $\quad Z_x = 153 in^3$

Check Moment Capacity

$\phi M_{px} = \phi F_y Z_x$
$= (0.9)(50 ksi)(153 in^3)(1/12)$
$= 574 k\text{-}ft > M_u = 543 k\text{-}ft$ ✓ Adequate for M

Check Shear Capacity

$\phi V_{nx} = 0.6 F_y A_w$
$= 0.6(50 ksi)(18.2 in^2)$
$= 546 k > V_u = 73 k$ ✓ Adequate for V

Check Deflection Limit

$\Delta_{Max,L} = 5\omega_L l^4/384EI + P_L l^3/48EI$
$= 5(2.4 k/ft)(20')^4(12)^3/384(29000 ksi)(1550 in^4)$
$+ 48k(20')^3(12^3)/48(29000 ksi)(1550 in^4)$
$= 0.192'' + 0.306 = 0.50''$

$\Delta_{cap,L} = L/360$
$= (20')(12''/1')/360$
$= 0.667''$

$\Delta_{cap,L} = 0.667'' > \Delta_{Max,L} = 0.5''$

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

	Project: TRON: The Experience - Queue Building		Date: 3/8/15
	Subject: Floor Perimeter Girder Design	By: AY	Reviewed:	Page: FGD-15

Reference: FGD-13

Check:

<u>Check Self Weight Assumption</u>

Assume Self Weight of Girder = 3.5 psf
for W 24×62
w/ Wt = 15'
Self-Weight = 62 plf / 15' = 4.13 psf
⇒ Dead Load Assumption

<u>Camber for Dead Load Deflection</u>

$\Delta_{max,D+L} = 5\omega_{D+L}\,l^4/384EI + P_{D+L}\,l^3/48EI$

$= \Delta_{max\,L}(\omega_{tot}/\omega_L) + \Delta_{max\,L}(P_{tot}/P_L)$

$= 0.192''(1.286\,klf/0.8\,klf) + 0.306''(71.04k/48k)$

$= 0.762''$

$\Delta_{cap,D+L} = L/240$

$= (20')(12''/1')/240$

$= 1''$

$\Delta_{cap,D+L} = 1'' > \Delta_{max,D+L} = 0.762''$ ✓

⇒ No Camber Required

<u>Conclusion:</u>

<u>For typical perimeter Girder at Floor use W24X62</u>

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TROH: The Experience - Queue Building

Date: 3/8/15

Subject: Column Design (Interior)

By: AB

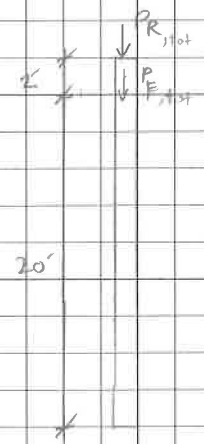
Reviewed:

Page: FGD-16

Reference:

Check:

Ground Floor to Basement



FGD-21 $P_{R_D} = 14292^{\#}$ $P_{R_L} = 7200^{\#}$ (from previous calcs)

2' splice used for constructibility

$P_{L_D} = A_T(D) = (20' \times 30')(75\ psf)$
$= 45{,}000\ \#$

$P_L = A_T(L)$

ASCE 7-10 4.7.2 $\Rightarrow L = L_0[.25 + (15/\sqrt{K_{LL} A_T})]$ $K_{LL} = 4$ for interior columns

$= 100_{psf}[.25 + (15/\sqrt{4(20' \times 30')})] =$

$= 55.62\ psf + 15\ psf(\text{partitions}) = 70.62\ psf$

$P_L = (20' \times 30')(70.62\ psf)$
$= 42372^{\#}$

ASCE 7-10 2.3.2 $P_{tot} = 1.2D + 1.6L = 1.2(14292^{\#} + 45000^{\#}) + 1.6(7200^{\#} + 42372^{\#})$
$= 150466^{\#} = 150.5^k$

Pinned - Pinned Condition $\Rightarrow K = 1.0$
$\Rightarrow L = 20'$

w/ $KL = 20$ & $P_{tot} = 150.5^k$ $\Rightarrow$ $\boxed{\text{Use W10} \times 39}$ w/ $\phi P_n = 177^k$

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Project: TRON: the Experience – Queue Building		Date: 4/12/15
Subject: Column Design (Exterior)	By: AB	Reviewed:	Page: FGD-17

Reference: | Check:

Ground Floor to Basement

RGD-22

$P_{RD} = 6675^{\#}$ $\quad P_{RL} = 3000^{\#}$

2' splice used for constructibility

$P_{LD} = A_T(D) = (15' \times 10')(74\,psf)$
$= 11{,}100^{\#}$

$P_L = A_T(L)$

ASCE 7-10 4.7.2

$\Rightarrow L = L_o[.25 + (15/\sqrt{K_{LL}A_T})]$ $\quad K_{LL} = 4$ for exterior columns w/o cantilever

$= 100\,psf[.25 + (15/\sqrt{4(10\times15)})]$

$= 86.24\,psf + 15\,psf\,(\text{partitions}) = 101.24\,psf$

$P_L = (15' \times 10')(101.24\,psf)$
$= 15{,}185^{\#}$

ASCE 7-10 2.3.2

$P_{tot} = 1.2(D) + 1.6(L) = 1.2(6675^{\#} + 11{,}100^{\#}) + 1.6(3000^{\#} + 15{,}185^{\#})$
$= 50426^{\#} = 50.4^k$

Pinned-Pinned Conditions $\Rightarrow$ $K = 1.0$
$L = 20'$

w/ $KL = 20'$ & $P_{tot} = 50.4^k$ $\Rightarrow$ $\boxed{\text{use } W10\times33}$ w/ $\phi P_n = 143^k$

Units Listed: ☐ Numbers Checked: ☐ Sources Referenced: ☐

Base Shear Calculation

Description: Northeast Disneyland Park - Anaheim, CA

Latitude: 33.813° N

Longitude: 117.917° W

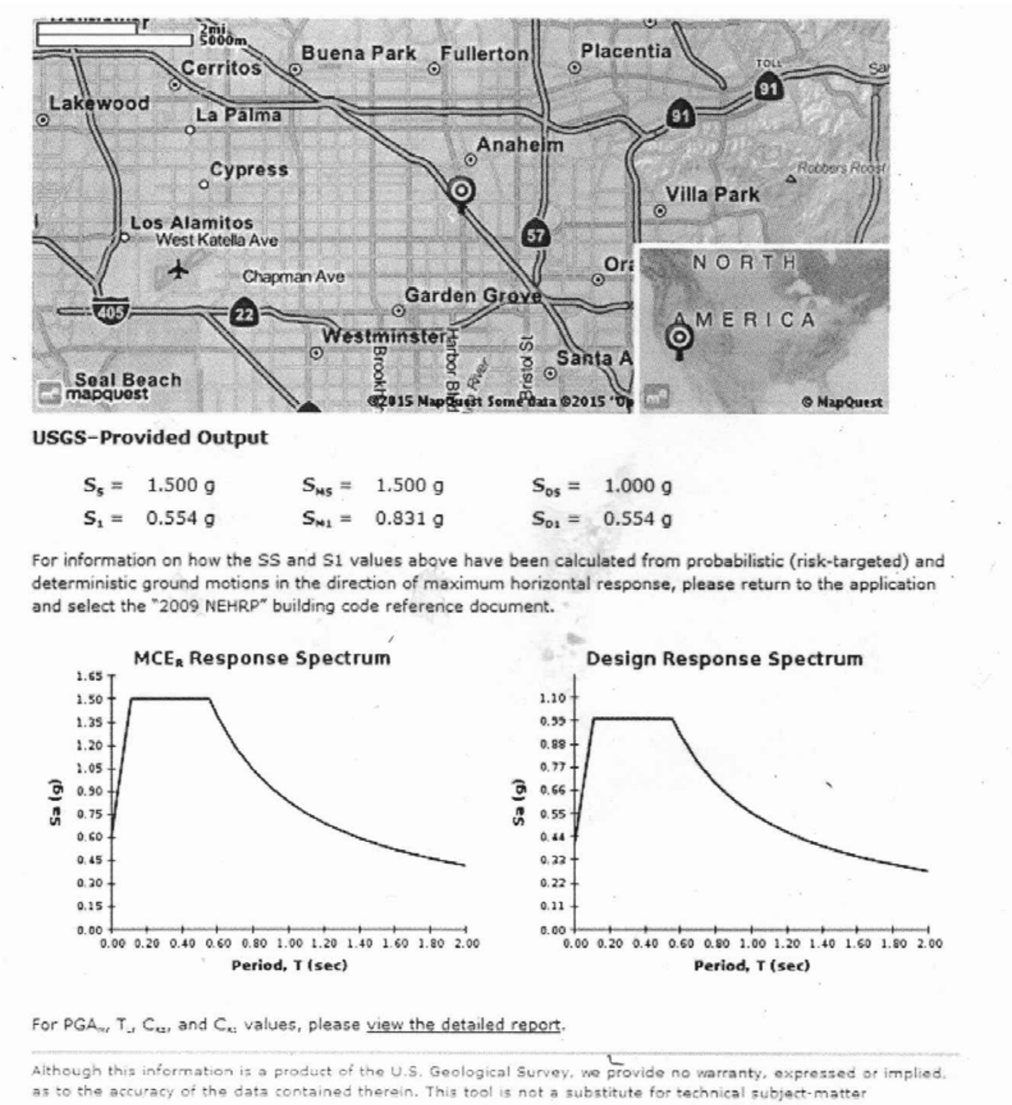


Fig. 5 - USGS Seismic Design Maps and Response Spectra

R = 3.25 - *T.12.2-1 - ASCE 7-10*

I = 1.25 - *T.1.5-2 - ASCE 7-10*

Risk Cateogory - III - *T.1.5-1 - ASCE 7-10*

<u>Period</u>

hn = 24’

Ct = 0.02

x = 0.75

T = Ct*hn^x - *T.12.8-2 - ASCE 7-10*

= 0.02*24^0.75 = 0.217

<u>Seismic Response Coefficient</u>

Cs = SDS / (R/I) = 1.0 / (3.25/1.25) = <u>0.385</u> - *Eq 12.8-2 - ASCE 7-10*

Csmax = SD1 / (T*(R/I)) - *Eq 12.8-3 - ASCE 7-10*

= 0.554 / (0.217*(3.25/1.25)) = 0.983 > 0.385 → Good

Csmin = 0.044*SDS*I - Eq 12.8-5

= 0.044*1.0*1.25 = 0.055 < 0.385 → Good

<u>Seismic Weight</u>

Aroof = 30’*20’*20 + 20’*20”*4 = 13600 ft^2

Perimeter of Cladding = (30’*5 + 20’)*2 + 20’*4*2 = 500’

Droof = 26 psf - *See Roof Load Take-Off*

Wcladding = 8.5 psf - *See Brick Veneer Wall Calculation*

Wx = Droof*Aroof + Wcladding*(hn/2)*Perimeter of Cladding

= 26 psf*13600ft^2 + 8.5 psf*(24’/2)*(500ft) = 404.6 k

V = Cs*Wx - *Eq.12.8-1 - ASCE 7-10*

= 0.385*(404.6 k) = 156 k

Use Base Shear - V = 156 k

Lateral System Design – Queue Building

V = 156 k (See Base Shear Calculation)

Select Ordinary Single Braced Frame –
2 Frames per line of action (4 Frames total)

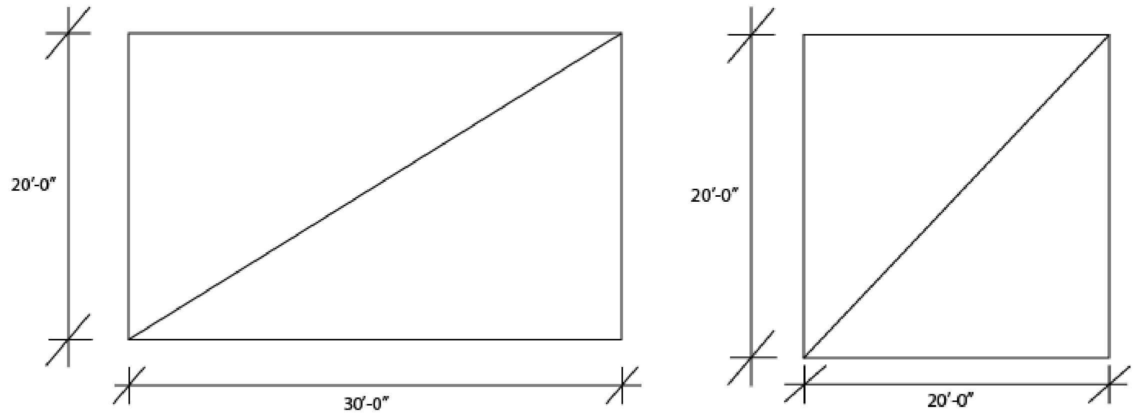


Fig. 6 - Braced Frame Geometries

RISA Analysis – Axial Forces & Deflections

Lateral loads = 156 k /4 = 39 k
Distributed loads on Beam retrieved from Load Take Off
Point loads on Girder from Load Take Off

Results:

20' x 20' → Axial Load for Brace 1 = 55.2 k

20' x 30' → Axial Load for Brace 2 = 46.9 k

f_y = 46 ksi (for HSS size members)
E = 29000 ksi
$\lambda_r = 1.40*\text{sqrt}(E/f_y) = 1.40*\text{sqrt}(29000\text{ksi} / 46\text{ksi}) = 35.15$ - *T. B4.1a – AISC Steel Manual*

Choose HSS6x6x12

T.4-4 - AISC Steel Manual

At effective length KL = 36' → ΦPn = 58.6 k
Weight = 35.2 plf
b/t = 9.9 < 35.15 → Good

0.020 h_{sx} - assumed allowable deflection - *T.12.12-1 - ASCE 7-10*
0.020*20'*12 = 4.8 in

Story Drift (Lateral System) = 0.155 in - RISA Analysis
C_d = 2.0 - Ordinary Braced Frame - *T.12.2-1 - ASCE 7-10*
$\delta_x = \delta_{xe} * C_d / I$ - *Eq 12.8-15 - ASCE 7-10*
= 0.155in*(2.0)/1.25 = 0.248 in < 4.8 in

Selected Size: HSS 6x6x1/2 - Weight = 35.2 plf & φPn = 58.6 k & b/t = 9.9

Truss System Design – Show Building

Modeled with a pinned base at each side

Applied Loads - 8.1 k (See roof interior beam gravity calcs)

6.23 k (See roof exterior beam gravity calcs)

Truss Geometries

Height of Truss = 8’

Width of Typical Truss Bay = 10’

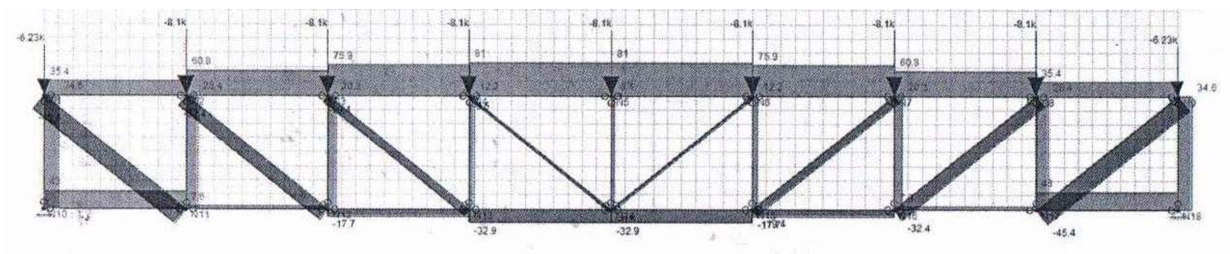


Fig. 7 - RISA Output for Axial Force Diagram of Truss

Output:

Pmax = 81 k (Axial Force Compression)

<u>Truss Modeled as Simple Span Beam</u>

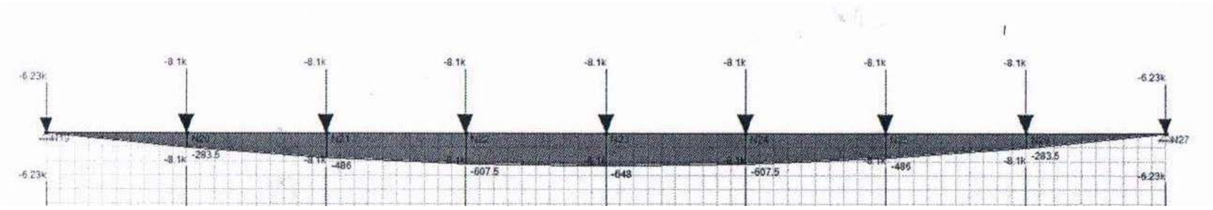


Fig. 8 - RISA Output for Moment Diagram of Simple Span Beam

Output:

Mmax = 648 k-ft

Check Truss – M / htruss = 648 k-ft / 8 ft = 81 k → Good

Choose HSS 4.4x4.5x3/16 - φPn = 88.4 k > 81 k

T.4-1 - AISC Steel Manual – Compression Capacities based on KL

Selected Size: HSS 4.4x4.5x3/16 - Weight = 10.7 plf & φPn = 88.4 k

Service Loads

Weight of ride vehicle = 7,860,760 g = 17,330 lbs = 17.33 k

Weight of passengers = 0.3 k per person * 6 passengers per ride vehicle = 1.8 k - (See Anthropometric Reference Data)

Factored Loads

Weight of ride vehicle = 17.33 k * 1.5 = 26 k – *Impact Load Factor - 4.6.3 - ASCE 7-10*

Weight of passengers = 1.8 k * 1.6 = 2.88 k - Assume as Live Load - *2.3.2 - ASCE 7-10*

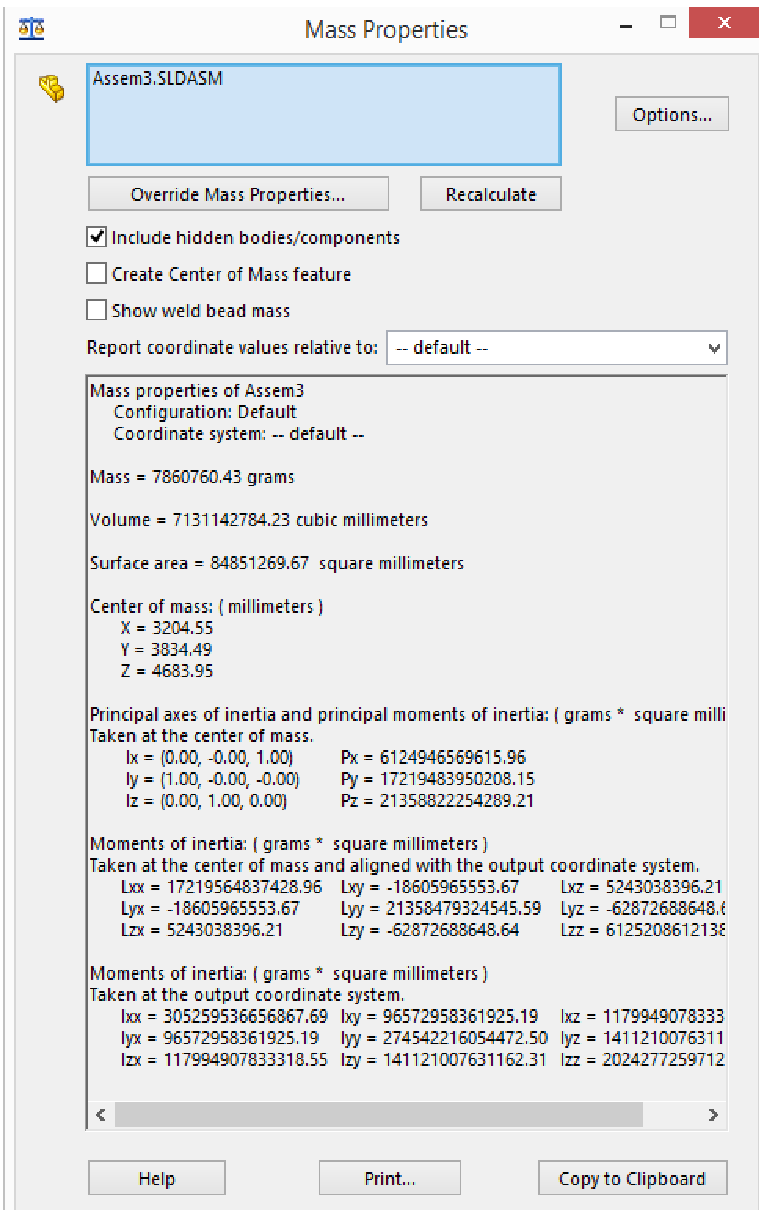


Fig. 9 - SOLIDWORKS Mass Properties

Page 10 □ Series 11, No. 252

Table 6. Weight in pounds for males aged 20 and over and number of examined persons, mean, standard error of the mean, and selected percentiles, by race and ethnicity and age: United States, 2007–2010

Race and ethnicity and age	Number of examined persons	Mean	Standard error of the mean	Percentile								
				5th	10th	15th	25th	50th	75th	85th	90th	95th
All racial and ethnic groups[1]							Pounds					
20 years and over	5,651	195.5	0.99	135.5	146.6	153.7	165.2	189.8	218.0	236.4	252.2	273.6
20–29 years	894	183.9	1.87	128.7	137.9	143.9	153.2	176.5	206.6	224.0	240.4	257.5
30–39 years	948	199.5	1.66	139.6	149.4	156.0	166.2	191.1	222.9	242.7	259.7	282.2
40–49 years	933	200.6	2.12	142.1	153.2	162.0	173.1	193.7	221.9	239.4	256.1	278.5
50–59 years	934	201.3	2.43	140.6	152.2	160.8	172.3	195.4	226.9	242.1	259.3	279.0
60–69 years	933	199.4	1.67	135.9	149.8	156.2	168.5	195.1	223.0	238.7	253.8	280.8
70–79 years	649	190.6	1.95	138.1	147.2	155.1	165.5	186.8	209.8	227.0	241.1	259.9
80 years and over	360	174.9	1.79	127.1	135.5	141.3	152.2	171.8	194.4	207.9	214.5	230.3
Non-Hispanic white												
20 years and over	2,738	199.2	0.92	140.4	151.8	159.1	170.3	194.0	221.7	239.6	254.9	273.6
20–39 years	796	194.7	1.71	135.7	145.3	153.0	162.8	188.1	217.2	238.0	250.3	270.6
40–59 years	832	204.9	1.56	146.3	159.0	165.4	177.3	198.5	227.4	246.7	259.5	283.7
60 years and over	1,110	196.3	1.21	140.1	152.0	159.1	168.8	191.8	217.1	232.7	248.1	266.7
Non-Hispanic black												
20 years and over	1,094	199.4	1.63	134.3	143.5	149.7	163.1	191.3	224.5	245.7	264.7	292.0
20–39 years	356	198.1	3.14	135.2	141.6	146.8	158.9	188.7	223.5	246.3	264.5	296.1
40–59 years	372	203.1	2.77	133.4	146.0	156.9	169.5	196.4	227.7	249.8	266.3	291.9
60 years and over	366	193.6	2.54	127.8	140.8	147.8	158.7	188.0	216.0	234.7	256.5	283.9
Hispanic[2]												
20 years and over	1,541	186.1	1.95	133.2	143.4	148.5	157.5	180.0	205.4	221.0	236.4	266.7
20–39 years	573	185.1	2.66	129.2	140.4	146.1	154.9	176.1	204.6	221.4	241.2	271.7
40–59 years	577	189.4	1.77	141.6	150.3	155.0	165.2	184.2	207.7	223.3	232.4	258.0
60 years and over	391	180.8	2.50	129.1	138.8	146.5	154.7	177.1	200.5	212.9	223.7	241.2
Mexican American												
20 years and over	991	185.4	2.30	133.2	143.3	148.4	157.1	179.6	204.6	218.8	235.6	267.6
20–39 years	386	185.2	3.27	130.9	140.0	146.0	155.0	176.6	204.8	220.6	241.4	276.2
40–59 years	371	187.4	1.98	141.1	150.0	154.6	163.0	182.8	206.6	219.8	229.3	255.3
60 years and over	234	180.6	3.17	130.7	138.9	148.0	155.8	177.2	199.3	208.9	215.2	240.6

[1]Persons of other races and ethnicities are included.

[2]Mexican-American persons are included in the Hispanic group.

SOURCE: CDC/NCHS, National Health and Nutrition Examination Survey.

Fig. 10 - Anthropometric Reference Data for Children and Adults: United States, 2007–2010

Operational Hours

A1.2.4.1 For this example, the previously calculated operational hours to be used in the design for applicable components = 19 950 operational hours. Because the general reduction for load and unload time was taken into account in the previous calculation, the number or ride cycles per hour (not including load and unload time) = (60 min/(ride cycle time – load and unload time)) = (60 min/(7 min – 3 min)) = 15 ride cycles per hour. The number of load cycles per ride cycle for this particular applicable component = 8 load cycles.

Calculation for Determining the Total Number of Load Cycles for an Applicable Component:

$$\left(\frac{\text{Operational hours for design}}{1}\right) \times \left(\frac{\text{Ride cycles (without load and unload time)}}{\text{Operational hour}}\right) \times \left(\frac{\text{Number of load cycles}}{\text{Ride cycle}}\right) = \text{Total Number of Load Cycles}$$

$$\left(\frac{19\,950 \text{ operational hours}}{1}\right) \times \left(\frac{15 \text{ ride cycles}}{1 \text{ operational hour}}\right) \times \left(\frac{8 \text{ load cycles}}{1 \text{ ride cycle}}\right) = 2\,394\,000 \text{ load cycles}$$

Fig. 11 - ASTM F2291 – 13 Standard for Amusement Ride Design

Operational Hours per day = 16 hrs (Disneyland typical park hours - *Disneyland Website*)

Ride Duration = 3 mins = 180 s (No Limits 2 Simulation)

6 pairs of ride vehicles (12 total ride vehicles) on the track at one time

180 s / 6 = 30 s difference between each set of ride vehicles

12 ride vehicles * 6 passenger per ride vehicle = 72 passenger on-ride at one time (per 3 min interval)

72 passenger per interval x 20 interval / hr = 1440 passenger per hr

1440 passenger per hr x 16 hrs / day = 23,040 passenger per day

<u>Per Individual Track</u>

Every 30 s a ride vehicle will go over a certain point - (2) 30 s intervals per min

In one hr 120 ride vehicles will pass over a point

In one day, 120*16 hr = 1920 ride vehicles will pass a given point on the track

Fatigue Stresses

Fy = Fsr = (Cf / Nsr)^0.333 - *Eq. A-3-1 – AISC Steel Manual*

- ➔ Cf = 120*10^8 per T.A-3.1
- ➔ Nsr = number of stress range fluctuations in design life
  (35,000 operational hours - *ASTM F2291*)
  Nsr = 1920 ride vehicles per day * 1 day / 16 hours * 35,000 hours = 4,200,000

Fy = Fsr = (120*10^8 / 4,200,000)^0.333 = **14.15 ksi**

Wheel Distances

Horizontal distance between wheel carriages = 60 in (SOLIDWORKS)

Vertical distance between wheels = 6 in (SOLIDWORKS)

Dynamic Applications

The accelerations retrieved from NoLimits 2 were plotted onto images of the Track Layout for the sake of easy accessibility. Elements circled in blue represent points of acceleration considered for the project.

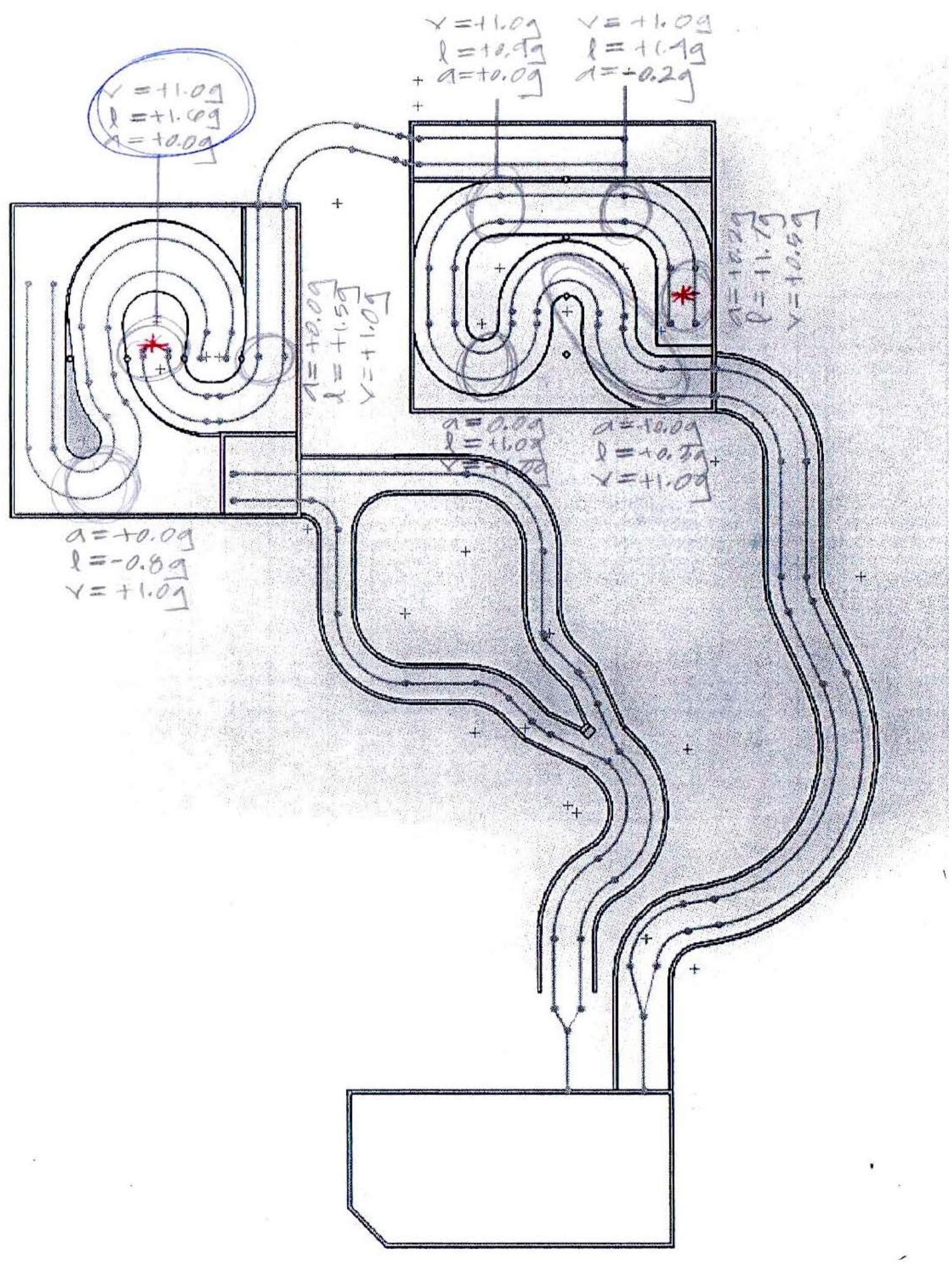


Fig. 12 - Upper portion of the Track Layout as modeled in SOLIDWORKS

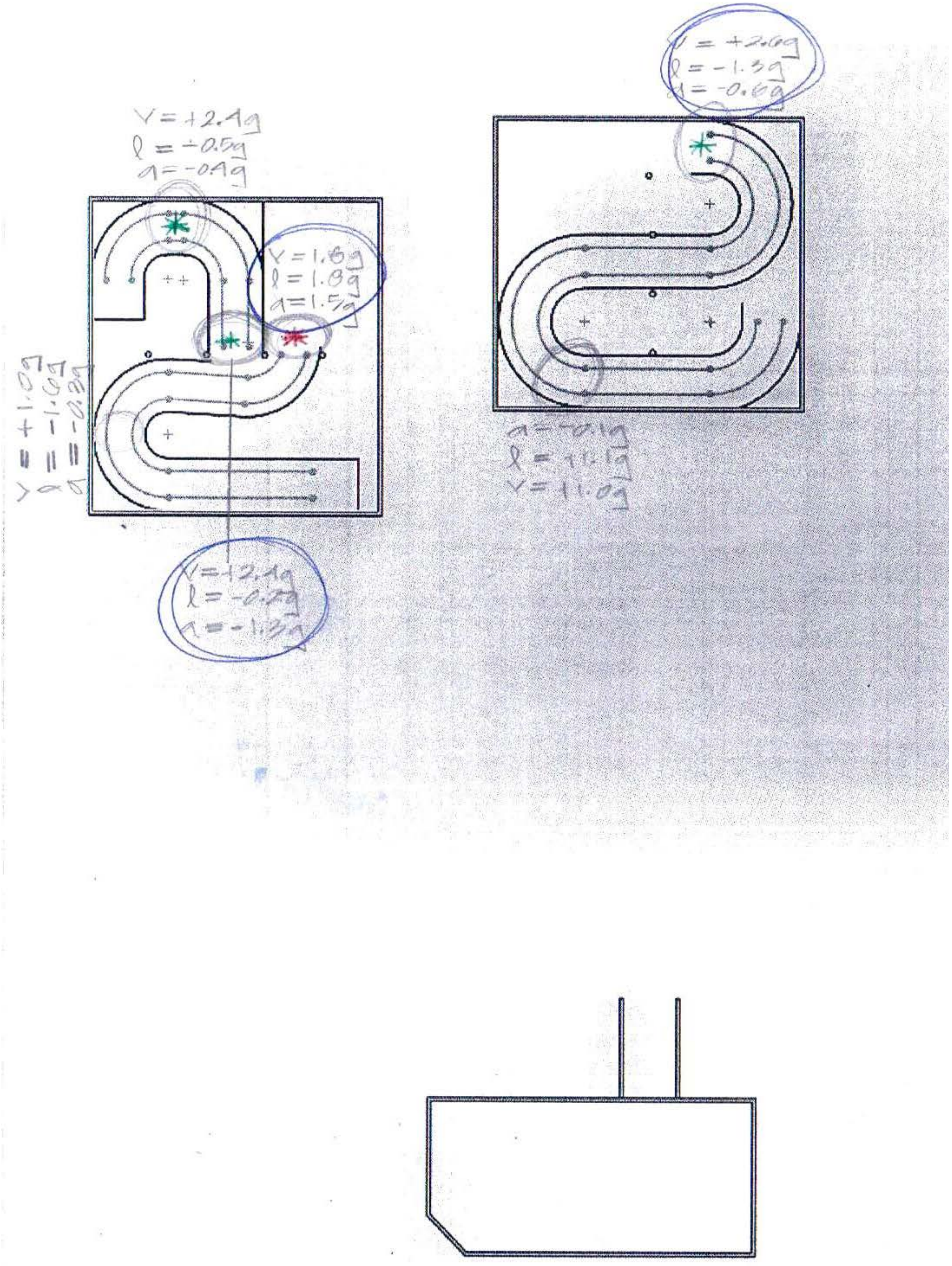


Fig. 13 - Upper portion of the Track Layout as modeled in SOLIDWORKS

Design of Guide Rails

Controlling Vertical acceleration = 1.8 g (No Limits 2 Simulation)

Controlling Lateral acceleration = 1.8 g (No Limits 2 Simulation)

Self-Weight (See Service Loads) = 17.33 k + 1.8 k = 19.13 k

Vertical Weight per Wheel = 19.13 k / 4 = 4.78 k per g (resisted by all four wheels)

Lateral Weight per Wheel = 19.13 k / 2 = 9.57 k per g (resisted by two outermost wheels)

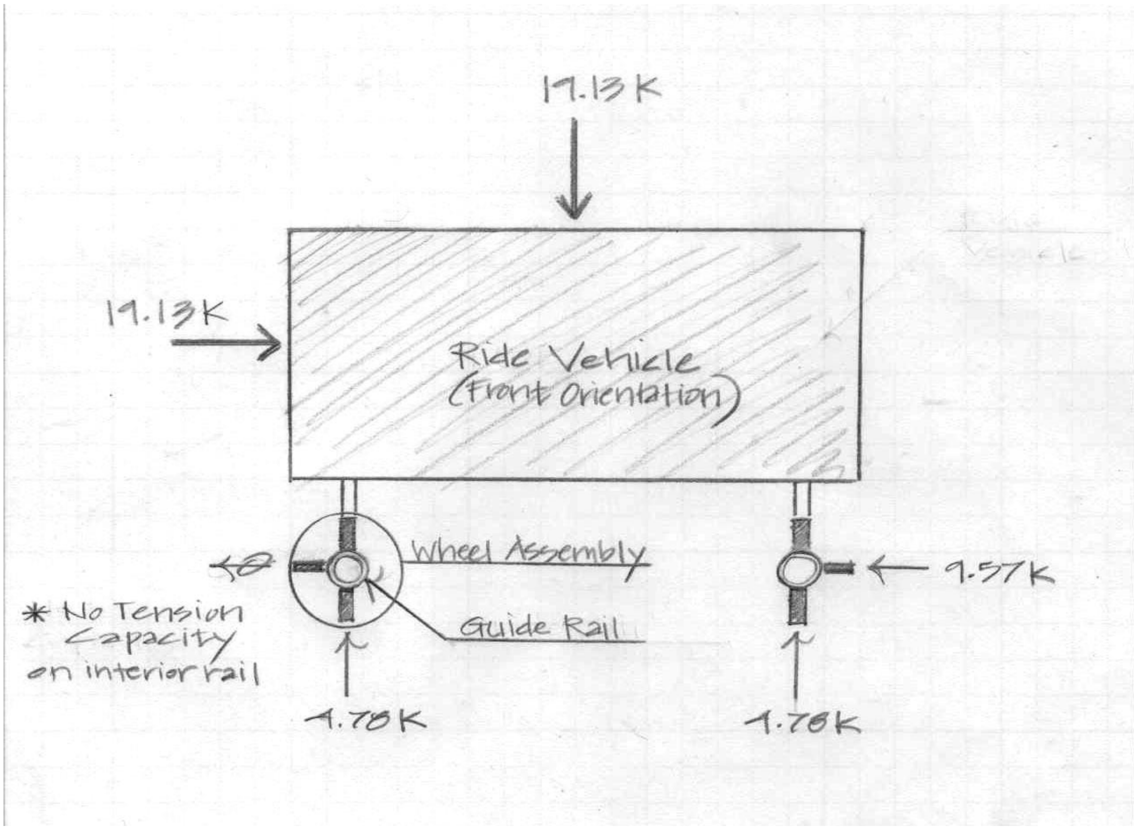


Fig. 14 - Distribution of weights on wheel assembly

Max Force per wheel (Vert. Comp.) = 1.8 g * 4.78 k = 8.61 k

Max Force per wheel (Lat. Comp.) = 1.8 g* 9.57 k = 17.22 k

Max Resultant Force per wheel = sqrt (8.61^2 + 17.22^2) = 19.25 k

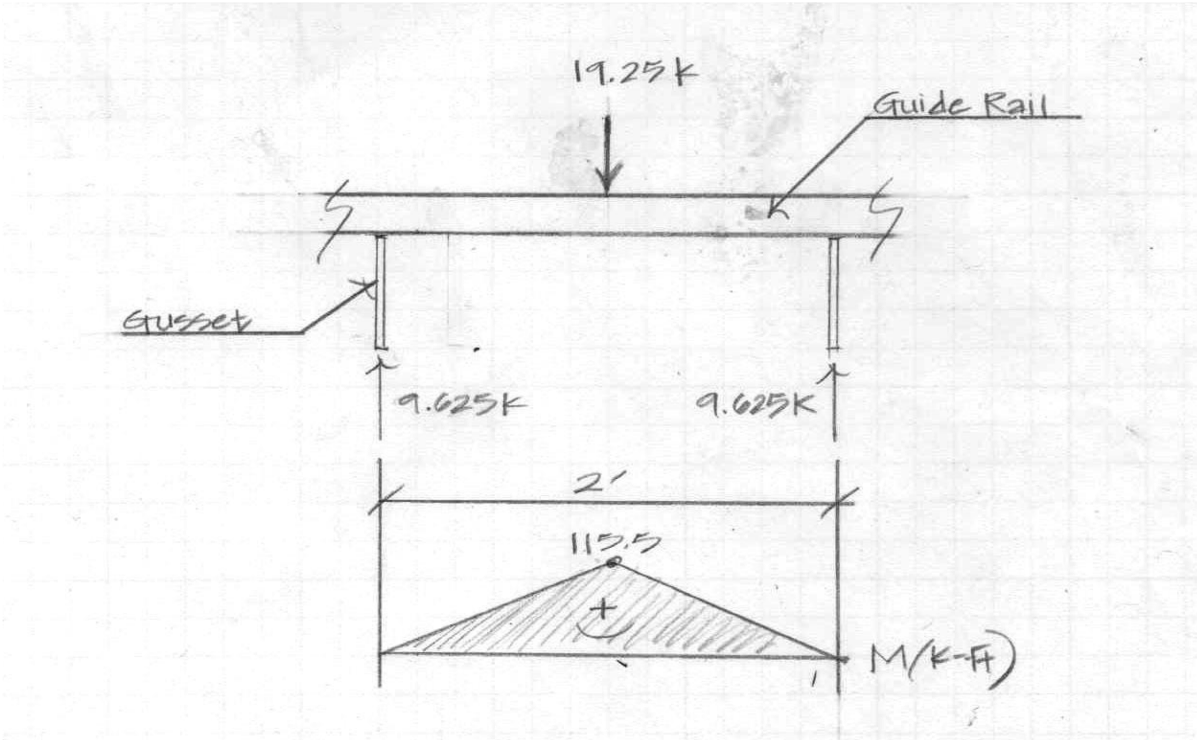


Fig. 15 - Analysis of Guide Rail as Simple Span Beam with corresponding Moment Demands

Mu = P*L/4 = 19.25 k * 2'/4 = 9.62 k-ft = 115.50 k-in

Mu < Mn / Ω - *Eq 2-2 - AISC Steel Manual*

- ➔ Mn = Mp = Fy*Z - *Eq F8-1 - AISC Steel Manual*
- ➔ Assume Round HSS Compact Section - *T.B4.1b - AISC Steel Manual*
- ➔ Ω = 1.67 – *General Requirements 16.1-46 - AISC Steel Manual*

Fy = Fsr = 14.15 ksi

Z = Mu * Ω / Fy = 115.50 k-in * 1.67 / 14.15 ksi = 13.63 in^3 - *Eq F2-1 - AISC Steel Manual*

Based on Z – Select HSS6x0.5 -> Z = 14.3 in^3 > 13.63 in^3 → Good

Selected Size: HSS6x0.500 – Weight = 29 plf Z = 14.3 in^3

Design of Gusset Plates

Weight of Guide Rail = 0.029 klf * 2' = 0.058 k

Self-Weight (ASD) = 19.13 k

Controlling Vertical acceleration = 2.6 g

Max Force per wheel (Vert. Comp.) = 2.6 g * (19.13 k) / 4 + 0.058 k = 12.49 k

Lat. Comp. application will cause an eccentricity opposite of the eccentricity caused by the Vert. Comp. which reduces it. It is more conservative therefore to only consider the Vert. Comp.

Assume min 1' wide HSS Core Rail (to maximize length of Gusset)

Creates eccentricity = 5'/2 – 1'/2 = 2'

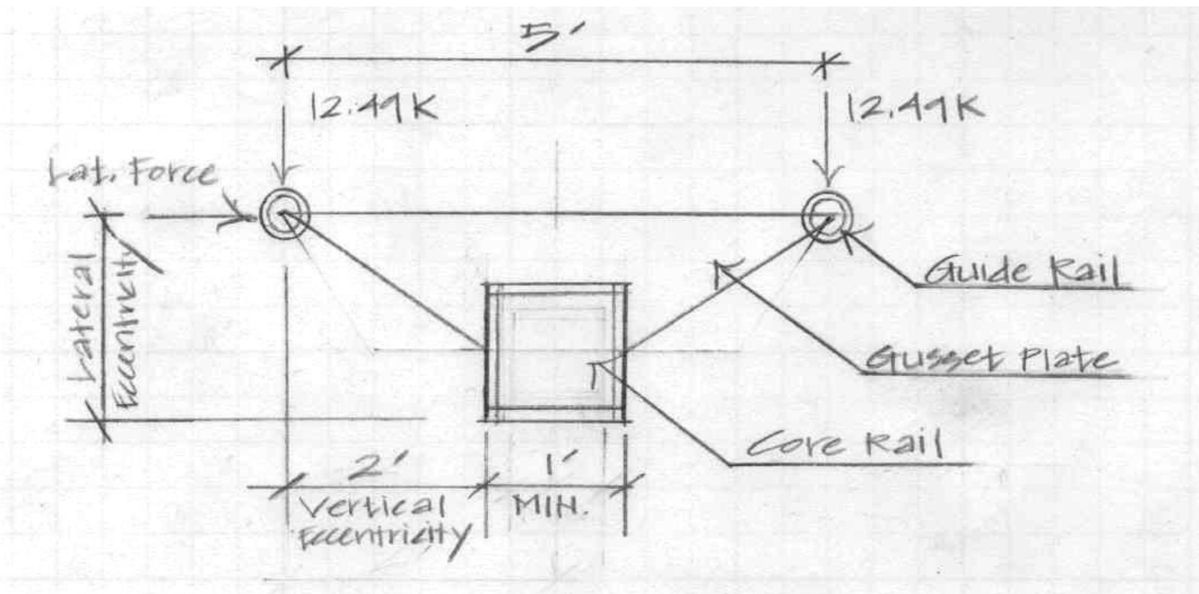


Fig. 16 - Effect of vertical and horizontal eccentricities on Gusset Plate

Mu = P*L = 12.49 k * 2' = 24.98 k-ft = 299.76 k-in

σ_b = Mn*c / I

- ➔ σ_b = Fy = Fsr = 14.15 ksi
- ➔ Mu < Mn / Ω   Mn = 299.76 k-in *1.67 = 500.6 k-in
- ➔ c = h / 2

I = Mu*c / σ_b = (500.6 k-in * h/2) / 14.15 ksi = 17.70*h

I = b*h^3 / 12 = 17.70*h → b*h^2 = 212.27

Assume h = 4*b

16*b^3 = 212.27 → b = 2.34 in

h = 9.47 in

Use a Gusset Plate with Cross Section of 2.5 in x 9.5 in @ 2' o.c.

Weight = 40 in * 9.5 in * 2.5 in * 0.28 pci = 266 lbs.

<u>Check Buckling of Flange</u>

Qs = 1.0 – Not Slender when b/t < 0.56*sqrt (E/Fy) - *Eq E7-4 - AISC Steel Manual*

b = sqrt(9.5"^2 + 24"^2) = 25.81 in

t = 25.81 in / 0.56*sqrt(29000 ksi /36 ksi) = 1.62 in < 2.5 in → Good

Selected Size: 2.5 in x 9.5 in Gusset Plate – Weight = 266 lbs

Design of Core Rail – Vertical Bending

Weight of Gussets = 266 lbs * 20' / (2' o.c.) = 2.66 k

Weight of Guide Rails = 0.029 klf * 20' * 2 rails = 1.16 k

Self-Weight (ASD) = 19.13 k

Controlling Vertical acceleration = 2.6 g

Total Point Load = 1.16 k + 2.66 k + (2.6 g * 19.13 k) = 53.56 k

Vx = P / 2 = 53.56 k / 2 = 26.78 k

Mx = PL / 4 = 53.56 k * 20'/ 4 = 267.79 k-ft = 3213.48 k-in

Mu < Mn / Ω - *Eq 2-2 - AISC Steel Manual*

- ➔ Mn = Mp = Fy*Z – *Eq F7-1 – AISC Steel Manual*
- ➔ Assume square HSS Compact Section – *T.B4.1b - AISC Steel Manual*
- ➔ Ω = 1.67 – *General Requirements 16.1-46 - AISC Steel Manual*

Fy = Fsr = 14.15 ksi

Z = M * Ω / Fy = 3213.48 k-in * 1.67 / 14.15 ksi = 379.26 in^3 - *Eq F2-1 - AISC Steel Manual*

Based on Z – Select HSS20x20x7/8 -> Z = 433 in^3 in > 379.26 in^3 → Good

Weight of Core Rail = 0.221 klf

<u>Account for Self-Weight of Core Rail</u>

Mself = wl^2 / 8 = 0.221 klf * (20')^2 / 8 = 11.05 k-ft = 132.6 k-in

Mx = 3213.48 k-in + 132.6 k-in = 3346.08 k-in

Z = Mx * Ω / Fy = 3349.08 k-in * 1.67 / 14.15 ksi = 394.91 in^3 - *Eq F2-1 - AISC Steel Manual*

Z = 433 in^3 in > 394.91 in^3 → Good

Design of Core Rail – Horizontal Bending

Core Rails sized as square section

Horizontal accelerations are less than Vertical accelerations – 1.8 g < 2.6 g

To simplify, assume capacity will check in the horizontal direction

<u>Design of Core Rail – Combined Bending Stress</u>

$M_x = 3294$ k-in

$M_y = P*L / 4 = (19.13\text{ k} * 1.3\text{ g}) * 20' / 4 = 124.35$ k-ft $= 1492.14$ k-in

$f_x = M_x / Z = 3214$ k-in / 433 in^3 = 7.609 ksi

$f_y = M_y / Z = 1492.14$ k-in / 433 in^3 = 3.446 ksi

Ignore Axial Stress to account for placement of LIMs which generate kick back force

$(M_{rx} / M_{cr}) + (M_{ry} / M_{cy}) < 1.0$ - *Eq H1-1b - AISC Steel Manual*

Equation altered to analyze for stress instead of moment

➔ $(f_x / F_y) + (f_y / F_y)$

(7.609 ksi / 14.15 ksi) + (3.446 ksi / 14.15 ksi) = 0.7183 < 1.0 → Good

<u>Design of Core Rail – Torsion</u>

$b = 2*r*\sin^2(\alpha/4)$ – *Fig. 18.2-2 - Design of Welded Structures*

➔ b- moment arm of torsional application
➔ r- radius of turn = 16’ - SOLIDWORKS observed geometries
➔ α - angle between supports = 71.62°

$b = 2*16'*\sin^2(71.62/4) = 3.025'$

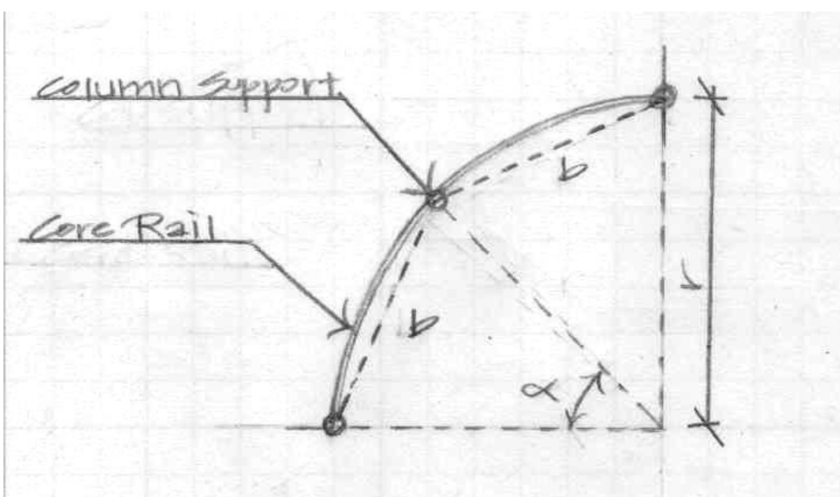


Fig. 17 - Torsional Geometries applied to Core Rail

Torsion due to Horizontal accel. = 1.8 g * 19.13 k * (10 in + 20 in /2) = 688.68 k-in

Torsion due to Vertical accel. = 53.56 k * b = 53.56 k * 3.025’ = 162.02 k-ft = 1944.23 k-in

Total Torsion = 688.68 k-in + 1944.23 k-in = 2632.91 k-in

<u>Shear stress at midpoint of the cross section</u>

τ = T / (2*A*t) – *T.2 – 2.10-4 - Design of Welded Structures*

- ➔ A - area enclosed by section = (20 - 0.75 in * 20 - 0.75 in) = 370.56 in ^2
- ➔ t - thickness of section = 0.698 in - *Jumbo HSS Dimension Table - AISC Website*

τ = 2601.33 k-in / (2*370.56 in^2*0.698in) = 5.03 ksi

Fy = Fsr = 14.15 ksi

14.15 ksi > 5.03 ksi → Good

<u>Design of Core Rail – Combined Shear Stress</u>

Account for self weight of Core Rail

Vx = 26.53 + 0.221 klf * 20’/2 = 28.56 k

Vy = 19.13 k * 1.3 g / 2= 12.43 k

τx = Vx / A = 28.56 k / 60.8 in^2 = 0.470 ksi

τy = Vy / A = 12.43 k / 60.8 in^2 = 0.204 ksi

τ = τx + τy = 0.470 ksi + 0.204 ksi = 0.674 ksi < 14.15 ksi → Good

Selected Size: HSS20x20x7/8 - Weight = 221 plf & Z = 14.3 in^3

<u>Design of Short Column - Axial</u>

Weight of Core Rail = 0.221klf * 20’ = 4.42 k

Weight of Gussets = 1.79 k

Weight of Guide Rail = 1.16 k

Self-Weight (ASD) = 19.13 k

Controlling Vertical acceleration = 1.8 g

Total Point Load = 4.42k + 1.16 k + 1.79 k + 1.8 g * 19.13 k = 41.8 k

Buckling not analyzed due to short length considered for column

Pu = Pn / Ω -> Pn = Pu* Ω = 41.8 k * 1.67 = 69.81 k

fy = Pn / A = 69.81 k / A < Fsr = 14.15 ksi

A = 4.93 in^2 → HSS 5x0.375 (initial min size based on Compression)

<u>Design of Short Column - Bending</u>

Max Lateral Force at Short Column = 19.13 k * 1.8 g = 34.43 k

Mu = 34.43 k * 2’ = 68.86 k-ft = 826.42 k-in

Mu > Mn / Ω - *Eq 2-2 - AISC Steel Manual*

Fy = Fsr = 14.15 ksi

Z = Mu * Ω / Fy = 826.42 k-in * 1.67 / 14.15 ksi = 97.54 in^3 - *Eq F2-1 - AISC Steel Manual*

Based on Z – Select HSS 16x0.625 -> Z = 138.0 in^3 > 97.54 in^3 → Good

A = 28.1 in^2

σ_b = (826.42 k-in * 1.67 / 138 in^3) + (68.14 k / 28.1 in^2) = 12.43 ksi < 14.15 ksi → Good

Selected Size: HSS 16x0.625 - Weight = 0.103 klf - Z = 138.0 in^2

<u>NOTE</u>: Size Re-evaluated to account for Steel Driven Piles (See Foundation Design)

Selected Size: HSS 18x0.500 - Weight = 0.093 klf - Z = 143 in^3 - A = 25.6 in^2

Design of Tall Column Strut – Axial

Self-Weight (ASD) = 19.13 k

Controlling Lateral Acceleration = 1.3 g

Max Lateral Force at Tall Column = 19.13 k * 1.3 g = 24.87 k

Axial Force in Strut = 24.87 k*(17.72' / 5') = 88.16 k

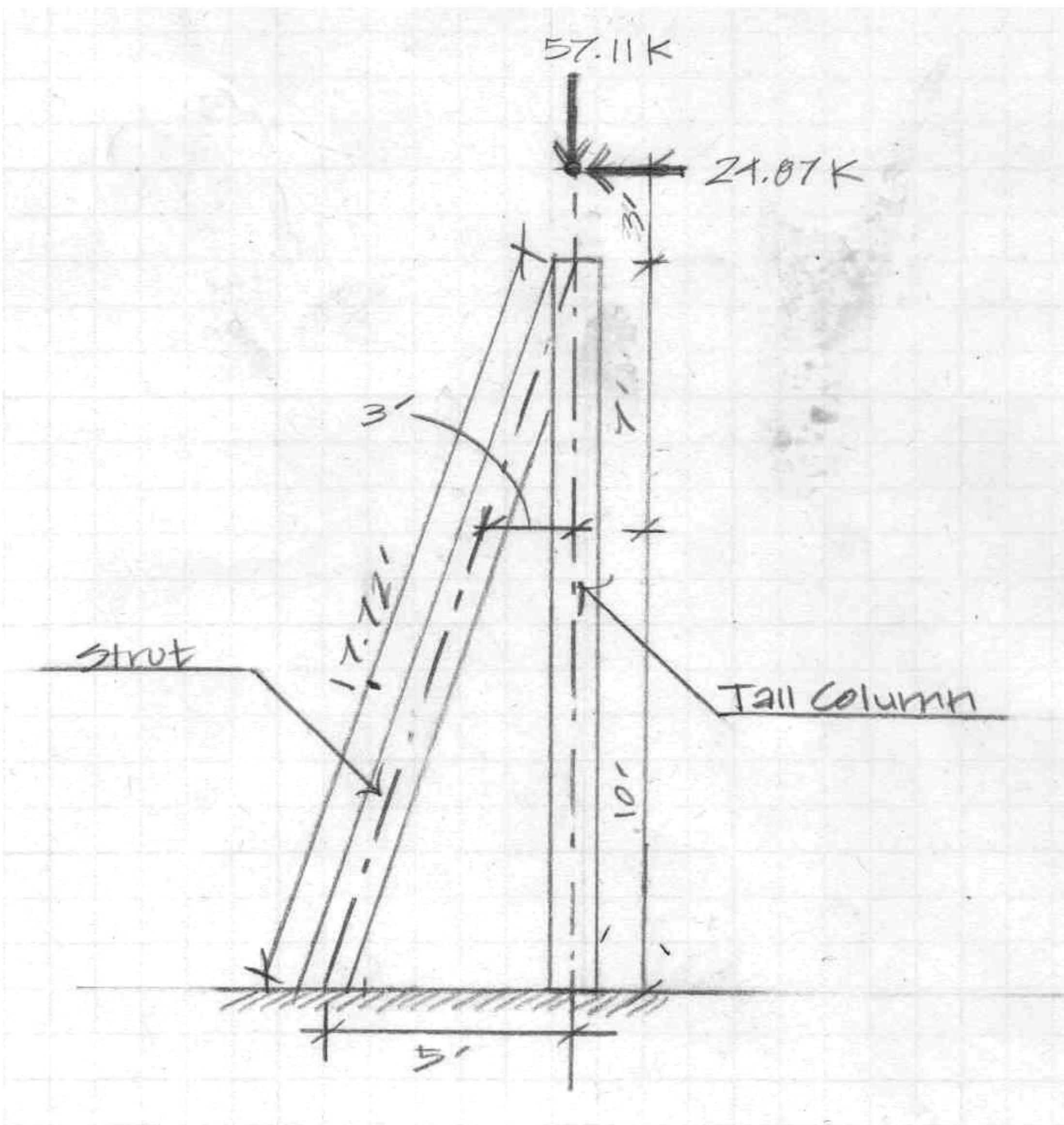


Fig. 18 - Dimensions of Tall Column and Strut to analyze component forces

Try HSS 8.625x0.5

A = 11.9 in^2

r = 2.89 in – *T.1-13 - AISC Steel Manual*

E = 29000 ksi

k = 1.0 – Pinned-Pinned Condition – *T.C-A-7.1 - AISC Steel Manual*

L = 17.72’*12 = 212.64 in

kL/r = 1.0*212.64 in /2.89 in = 73.58

Fe = (π^2 * E) / (kL/r)^2

= (π^2 * 29000ksi) / (73.58)^2 = 52.87 ksi – *Eq E 3-4 - AISC Steel Manual*

Fsr = 14.15 ksi

4.71*sqrt(E/Fsr) = 4.71*sqrt(29000ksi / 14.15 ksi) = 213.23 > kL/r = 73.58

Fcr = [0.658^(Fsr / Fe)]*Fsr - *Eq E 3-2 - AISC Steel Manual*

= [0.658^(14.15 ksi / 52.87 ksi)]*14.15 ksi = 12.65 ksi

Pn = Fcr*A / Ω

= 12.65 ksi * 11.9 in^2 / 1.67 = 90.14 k > 88.16 k → Good *Eq E 3-1 - AISC Steel Manual*

Select HSS 8.625x0.5

<u>Check addition of self-weight</u>

Weight = 0.043 klf

0.043 klf * 17.72’ = 0.76 k

88.16 k + 0.76 k = 88.92 k < 90.14 k → Good

<u>Design of Tall Column – Axial</u>

Weight of Core Rail = 4.42 k

Weight of Gussets = 1.79 k

Weight of Guide Rail = 1.16 k

Self-Weight (ASD) = 19.13 k

Controlling Vertical acceleration = 2.6 g

Axial Force - Tall Column (Compressive) = 4.42k + 1.79 k + 1.16 k + 2.6 g * 19.13 k = 57.11 k

Max Lateral Force at Tall Column = 19.13 k * 1.3 g = 24.87 k

Axial Force in Tall Column (Tensile) = 24.87 k*(17' / 5') = 87.95 k

Pu = 87.95 k – 56.11 k = 31.84 k

Pn = Fcr*A / Ω

= 12.65 ksi * 11.9 in^2 / 1.67 = 90.14 k > 31.84 k → Good *Eq E 3-1 - AISC Steel Manual*

Selected Size: HSS 8.625x0.5 - Weight = 0.043 klf - A = 11.9 in^2

<u>NOTE</u>: Size Re-evaluated to account for Steel Driven Piles (See Foundation Design)

Selected Size: HSS 18x0.500 - Weight = 0.093 klf - Z = 143 in^3 - A = 25.6 in^2

Moment Frame Design

Weight of Core Rail = 4.42 k

Weight of Guide Rail = 1.16 k

Weight of Gussets = 1.79 k

Self-Weight (ASD) = 19.13 k

Controlling Vertical and Lateral acceleration = 1.0 g

Vertical Force = 4.42k + 1.79 k + 1.16 k + 1.0 g * 19.13 k = 26.65 k

Lateral Force = 1.0 g * 19.13 k = 19.13 k

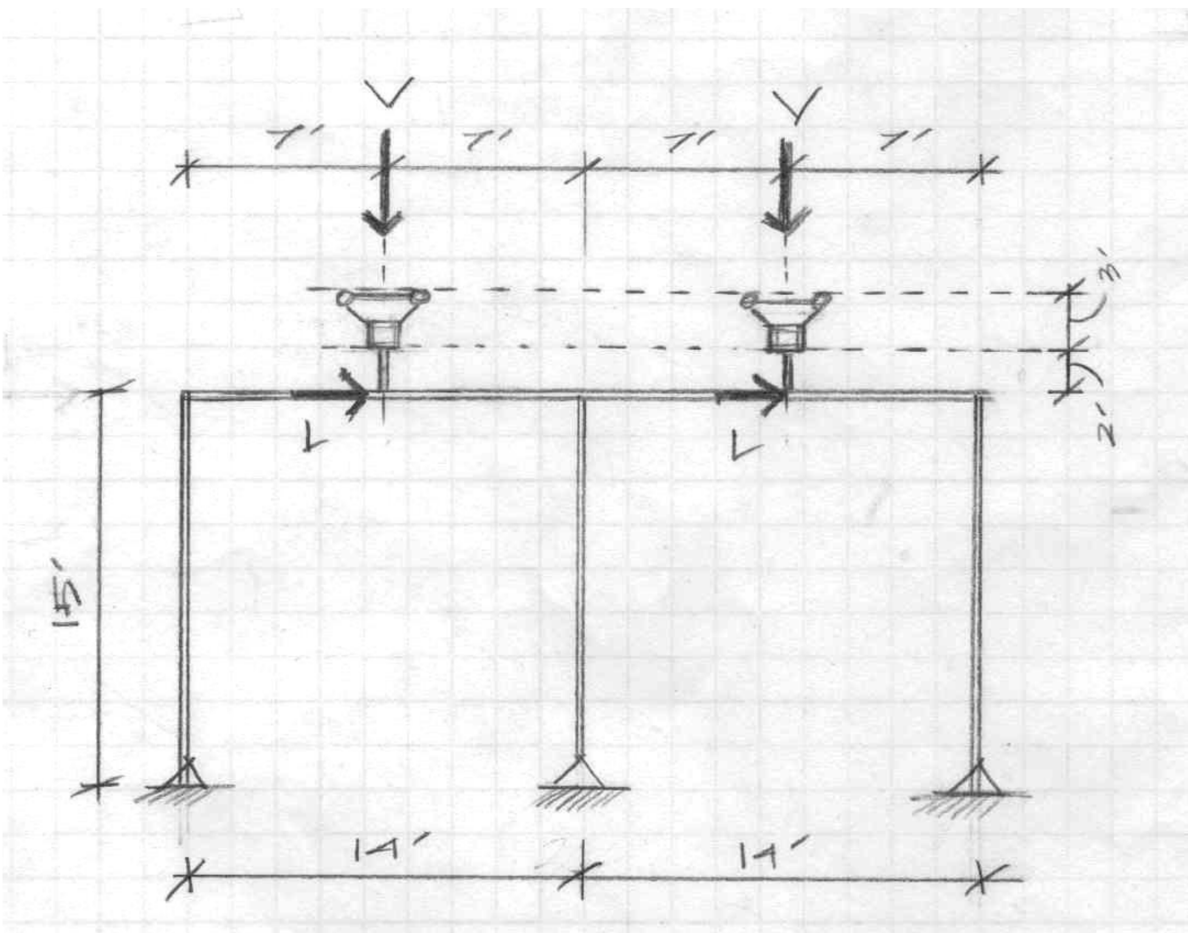


Fig. 19 - Forces applied and dimensions of Moment Frame

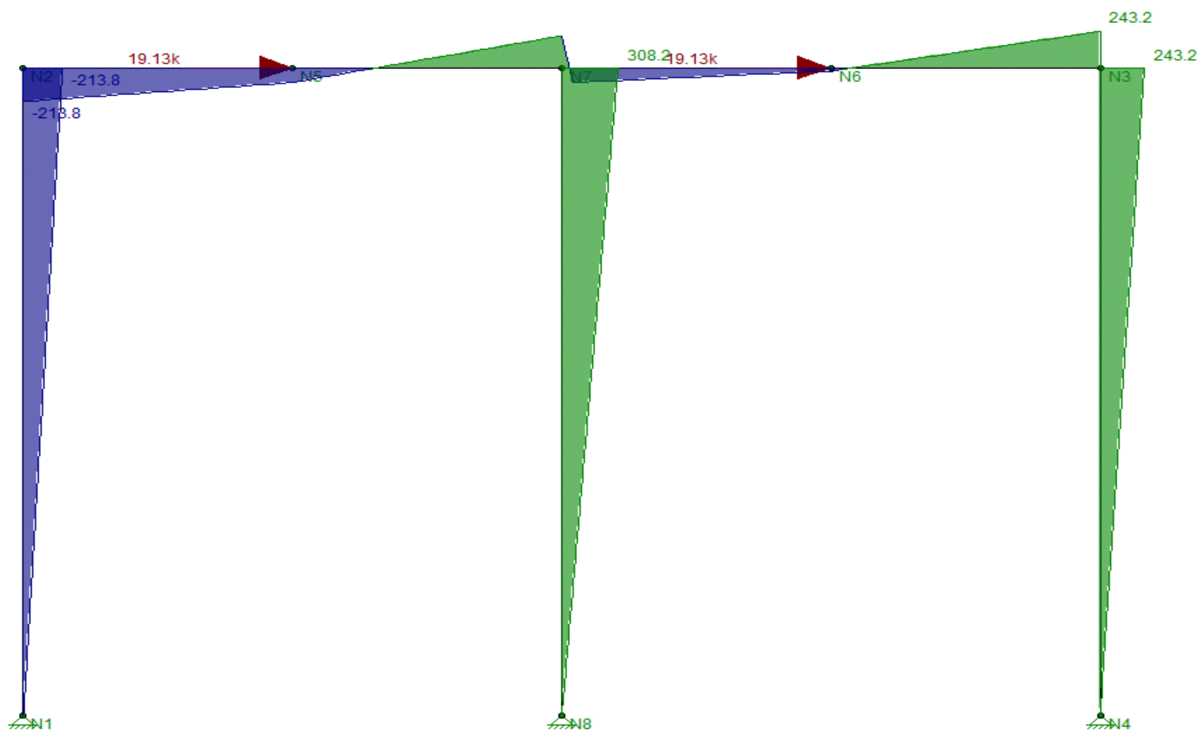


Fig. 20 - RISA Analysis Model - Bending Moment Diagram - Lateral Load Case

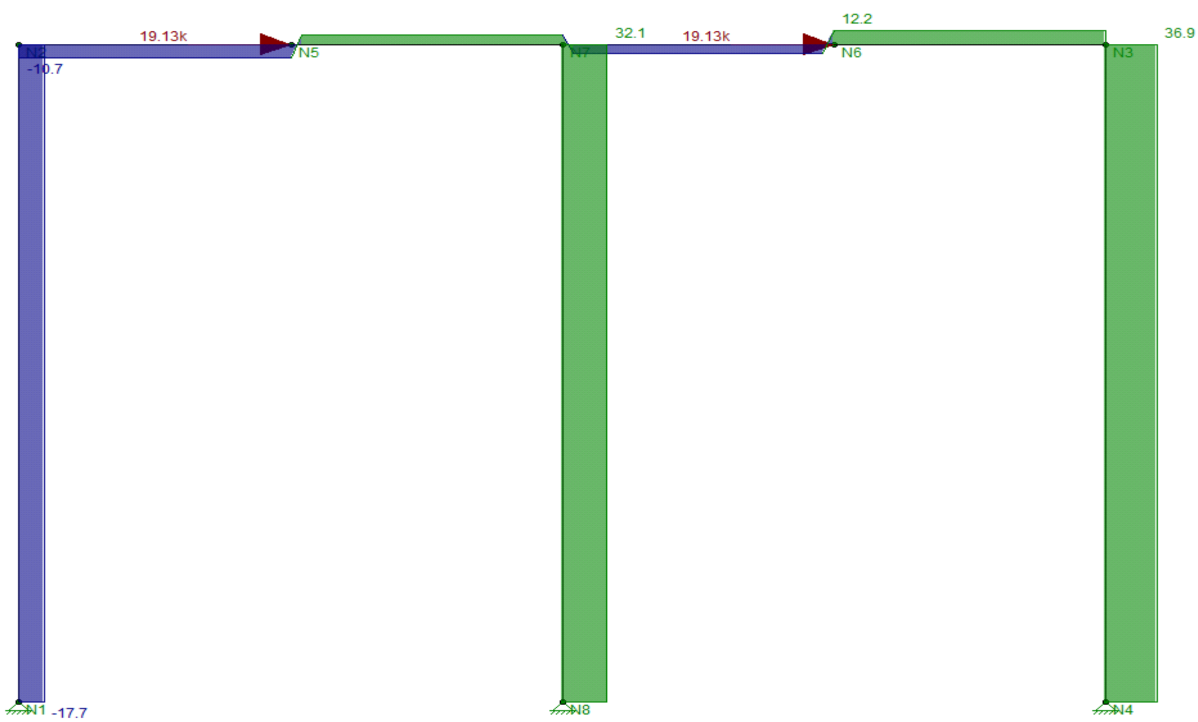


Fig. 21 - RISA Analysis Model - Axial Force Diagram - Lateral Load Case

Demand from RISA Analysis

Mu = 308.2 k-ft = 3698.4 k-in

Pu = 31.2 k

Choose W18x211

Z = 490 in^3

A = 62.3 in^2

r = 8.35 in – *T.1-13 - AISC Steel Manual*

E = 29000 ksi

k = 1.0 – Pinned-Pinned Condition – *T.C-A-7.1 AISC Steel Manual*

L = 15'*12 = 180 in

kL/r = 1.0*180 in /8.35 in = 21.56

Fe = (π^2 * E) / (kL/r)^2

= (π^2 * 29000ksi) / (21.56)^2 = 615.92 ksi – *Eq E 3-4 - AISC Steel Manual*

Fsr = 14.15 ksi

4.71*sqrt(E/Fsr) = 4.71*sqrt(29000ksi / 14.15 ksi) = 213.23 > kL/r = 21.56

Fcr = [0.658^(Fsr / Fe)]*Fsr - *Eq E 3-2 - AISC Steel Manual*

= [0.658^(14.15 ksi / 615.92 ksi)]*14.15 ksi = 14.01 ksi

Mr = Mu * Ω = 3698.4 k-in * 1.67 = 6176.3 k-in

Pr = Pu * Ω = 31.2 k * 1.67 = 52.1 k

Mc = Fy*Z = 14.15 ksi * 490 in^3 = 6933.5 k-in - *Eq F 2-1 - AISC Steel Manual*

Pc = Fcr*Ag = 14.01 ksi * 62.3 in^2 = 873.11 k - *Eq E 3-1 - AISC Steel Manual*

Combined Bending and Axial

Pr / (2*Pc) + Mr / Mc < 1.0 – *H1-1b - AISC* – Pr / Pc = 52.1 k / 873.11 k = 0.06 < 0.2

52.1 k / (2*873.11 k) + 6176.3 k-in / 6933.5 k-in = 0.92 < 1.0 → Good

Selected Size: W18x175 - Weight = 175 plf & Z = 398 in^3 & A = 51.4 in^2

Foundation Design

Short Column Foundations

Axial Demand = 41.8 k (From Short Column Design)

Use HP18 x 181

r = 18 in / 2 = 9 in = 0.75 ft

d = 18 in = 1.5 ft

Surface Area = SA = 4*d = 4*1.5 ft = 6 ft

dc = 10*d = 10*1.5 ft = 15 ft

Use Pile Depth L = 20 ft

Pv = γ* dc = 90 pcf * 15 ft = 1350 psf

PvL = 0.5*Pv*L + Pv*(L-dc) = 0.5*1350psf*20ft + 1350psf*(20ft – 15ft) = 16875 plf

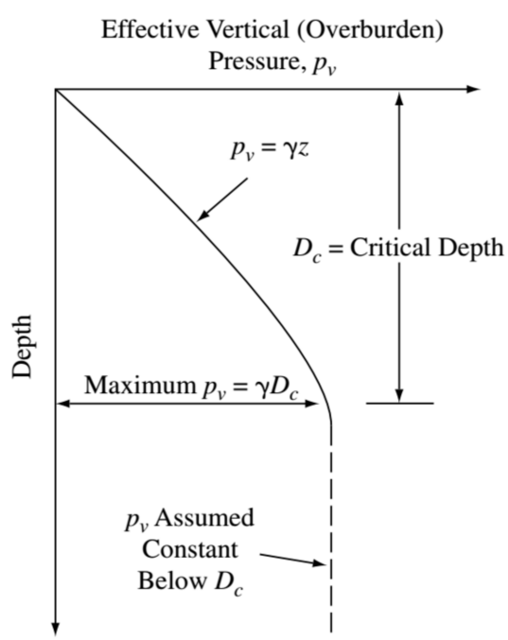


Fig. 22 - Figure from Soils and Foundations (10-1)

qult = (qfric + qtip) / FS

qfric = PvL*K*tanδ*SA = 16875 plf * 0.6 * 0.2 * 6ft = 12.15k

Material	Tan δ
Concrete	0.45
Wood	0.4
Steel (smooth)	0.2
Steel (rough, rusted)	0.4
Steel (corrugated)	Use tan ϕ of sand

Fig. 23 - Table 10-5 from <u>Soils and Foundations</u>

qtip = Pv*Nq*A tip = 1350 psf* 20 * 2.25ft^2 = 60.75k

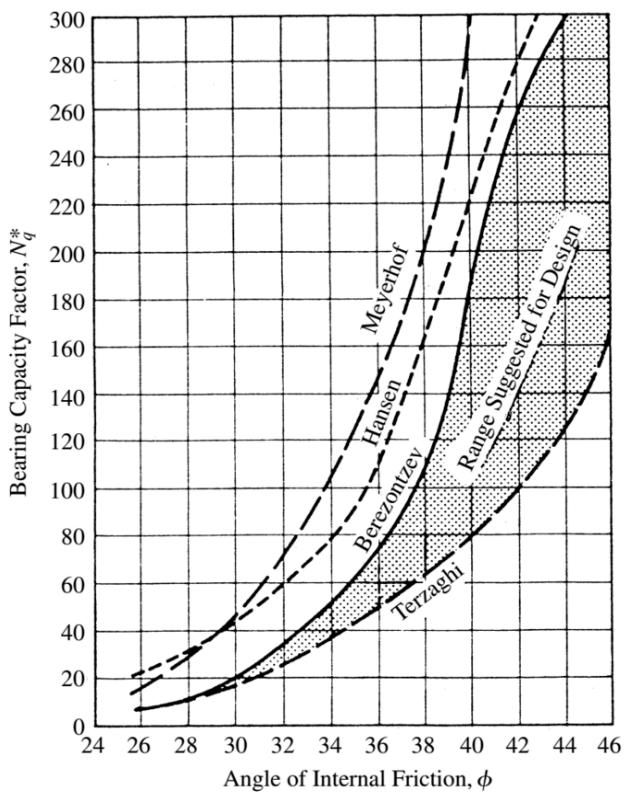


Fig. 24 - Figure from <u>Soils and Foundations</u> (10-2)

qult = (12.15 k + 60.75 k) / 1.7 = 42.88 k > 41.8 k → Good

Required depth: 22 ft

Selected Size: HP18x181 - 33 ft deep Steel Driven Pile (for constructability)

<u>Tall Column Foundations</u>

P = 31.84 k < 41.8 k - Foundation chosen for Short Column will account for Axial Force applied

Selected Size: HP18x181 - 33 ft deep Steel Driven Pile (for constructability)

<u>Moment Frame Foundations</u>

Demand

Tension = 17.7 k

Compression = 157 pcf* (15' + 7') = 3.45 k

Pu = 17.7 k – 3.45 k = 14.25 k

Use HP18 x 181

r = 18 in / 2 = 9 in = 0.75 ft

d = 18 in = 1.5 ft

Surface Area = SA = 4*d = 4*1.5 ft = 6 ft

dc = 10*d = 10*1.5 ft = 15 ft

Use Pile Length L = 33 ft

Pv = γ* dc = 90 pcf * 15 ft = 1350 psf

PvL = 0.5*Pv*L + Pv*(L-dc) = 0.5*1350psf*33ft + 1350psf*(33ft – 15ft) = 34425 plf

qult = qfric / FS

qfric = PvL*K*tanδ*SA = 34425 plf * 0.6 * 0.2 * 6ft = 24.79 k

qult = 24.79 k / 1.7 = 14.58 k > 14.25 k → Good

Selected Size: HP18x181 - 33 ft deep Steel Driven Pile

Check for Lateral Design

Based on an alteration of *Eq 18-2 - IBC 2012*

$$d = \sqrt{\frac{4.25Ph}{S_3 b}} \qquad \textbf{(Equation 18-2)}$$

or alternatively

$$d = \sqrt{\frac{4.25M_g}{S_3 b}} \qquad \textbf{(Equation 18-2)}$$

where:

M_g = Moment in the post at grade, in foot-pounds (kN-m).

S_3 = Allowable lateral soil-bearing pressure as set forth in Section 1806.2 based on a depth equal to the depth of embedment in pounds per square foot (kPa).

Fig. 25 - Eq 18-2 from IBC 2012

Demand – Max Acceleration = 1.5 g (Account for LIMs)

Ms = 19.13 k * 1.5 g * 20ft = 573900 lb-ft

S = 150 – *T.1806.2 - IBC 2012*

b = 1.5 ft - width of pile

L = (4.25*Ms) / (S*b) = (4.25*573900) / (150*1.5) = 22.13 ft < 33 ft → Good

Chosen Moment Frame Foundation adequate for Lateral Application

Revit Modeling

Autodesk Revit (or Revit) is a building information modeling software programed for the use of architects, structural engineers, and contractors. It allows users to design a building and structure and its components in 3D, annotate the model with 2D drafting elements, and access building information from the building model's database.

Revit was crucial in analyzing the designs of this project in 3D and 2D space. Revit was applied to the Queue Building, the Show Building, and the Track Layout portions of the design process.

For the Queue Building, Revit was helpful in accounting for areas where guests would line up while maintaining space for elements that guests expect to see at a Disney theme park (i.e. Gift Shop, interactive Queue elements, disability access, Fastpass service, etc.). Determining crowd capacity was an important factor to take into account for the sake of Ride Vehicle weight considerations and stress range fluctuations for fatigue design.

For the Track Layout, Revit was utilized in displaying visually how elements of the Track Configuration would fit together and ultimately confirmed constructability of the schematic design of this project.

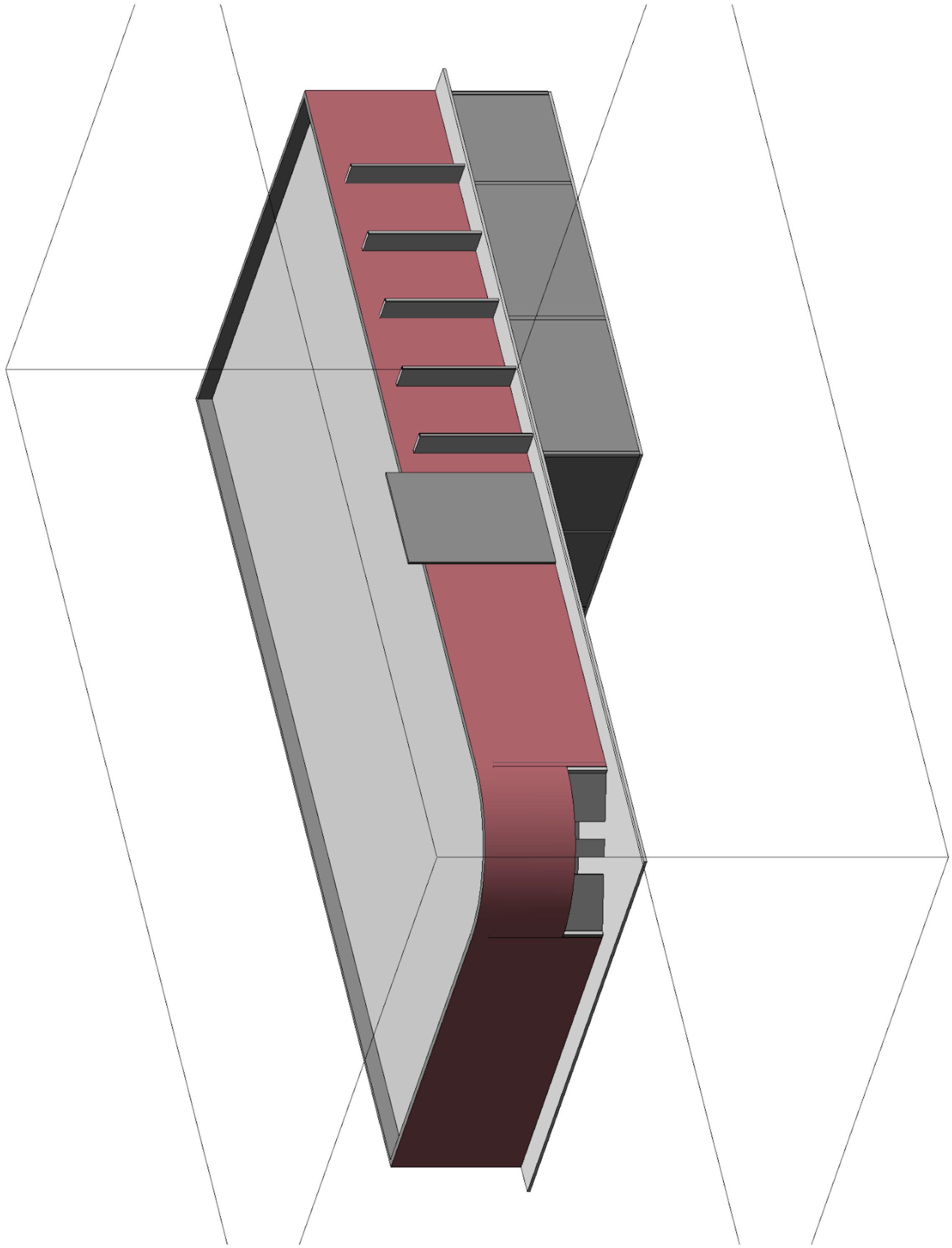


Fig. 26 - Queue Building - Isometric View
Reflective of Flynn's Arcade architectural considerations from Story Development

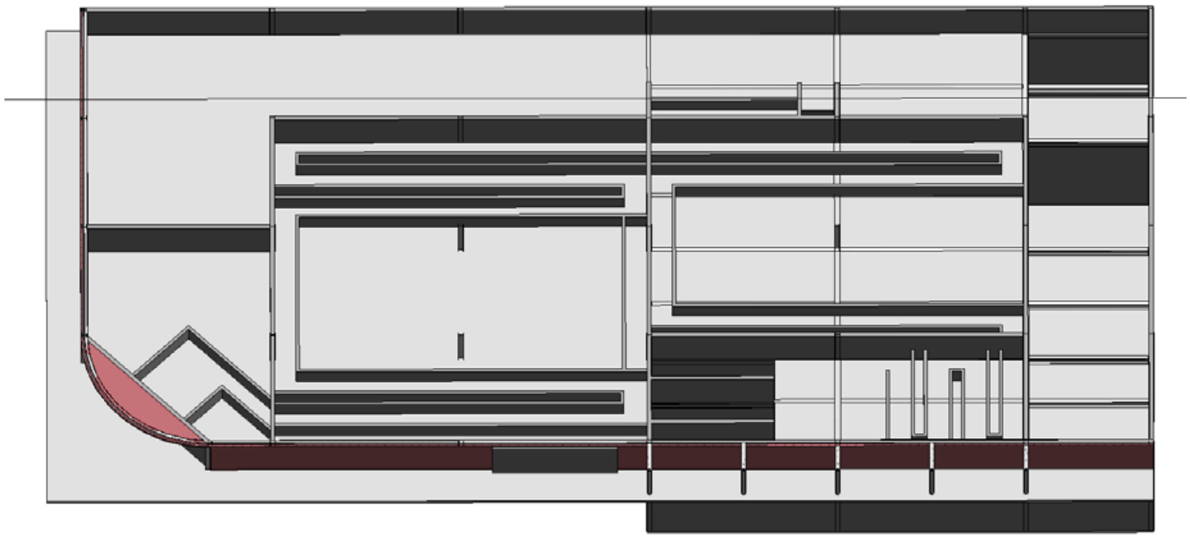


Fig. 27 - Queue Building - 3D Section of Ground Floor - Shows line space and room divisions

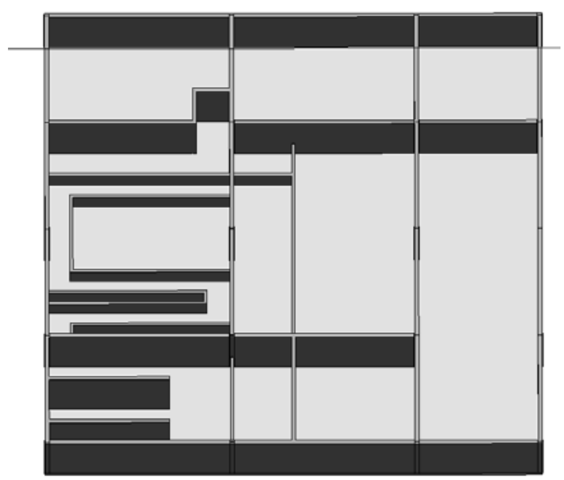


Fig. 28 - Queue Building - 3D Section of Basement Floor -Shows line space and room divisions

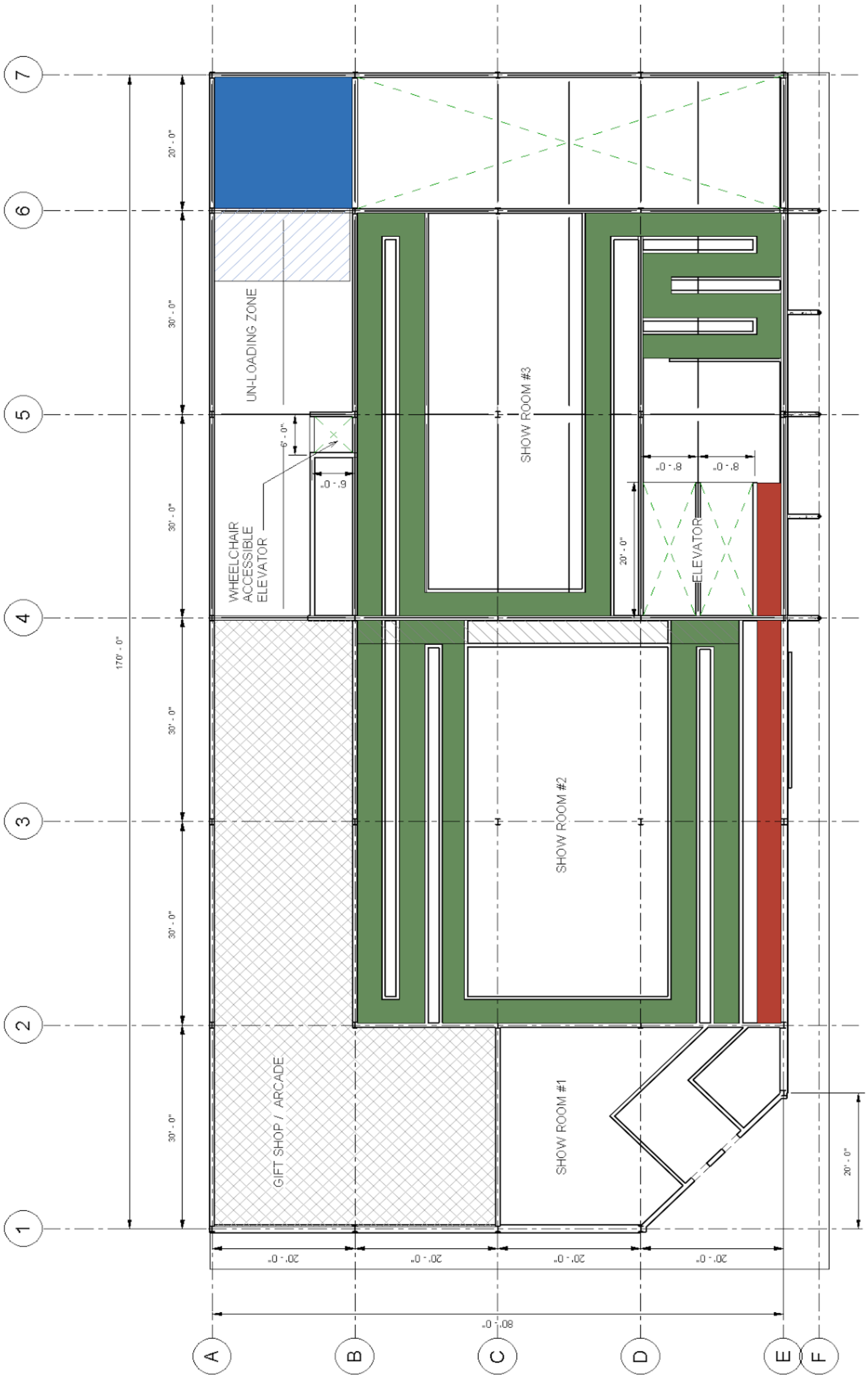


Fig. 29 - Queue Building - Annotated Ground Floor Plan

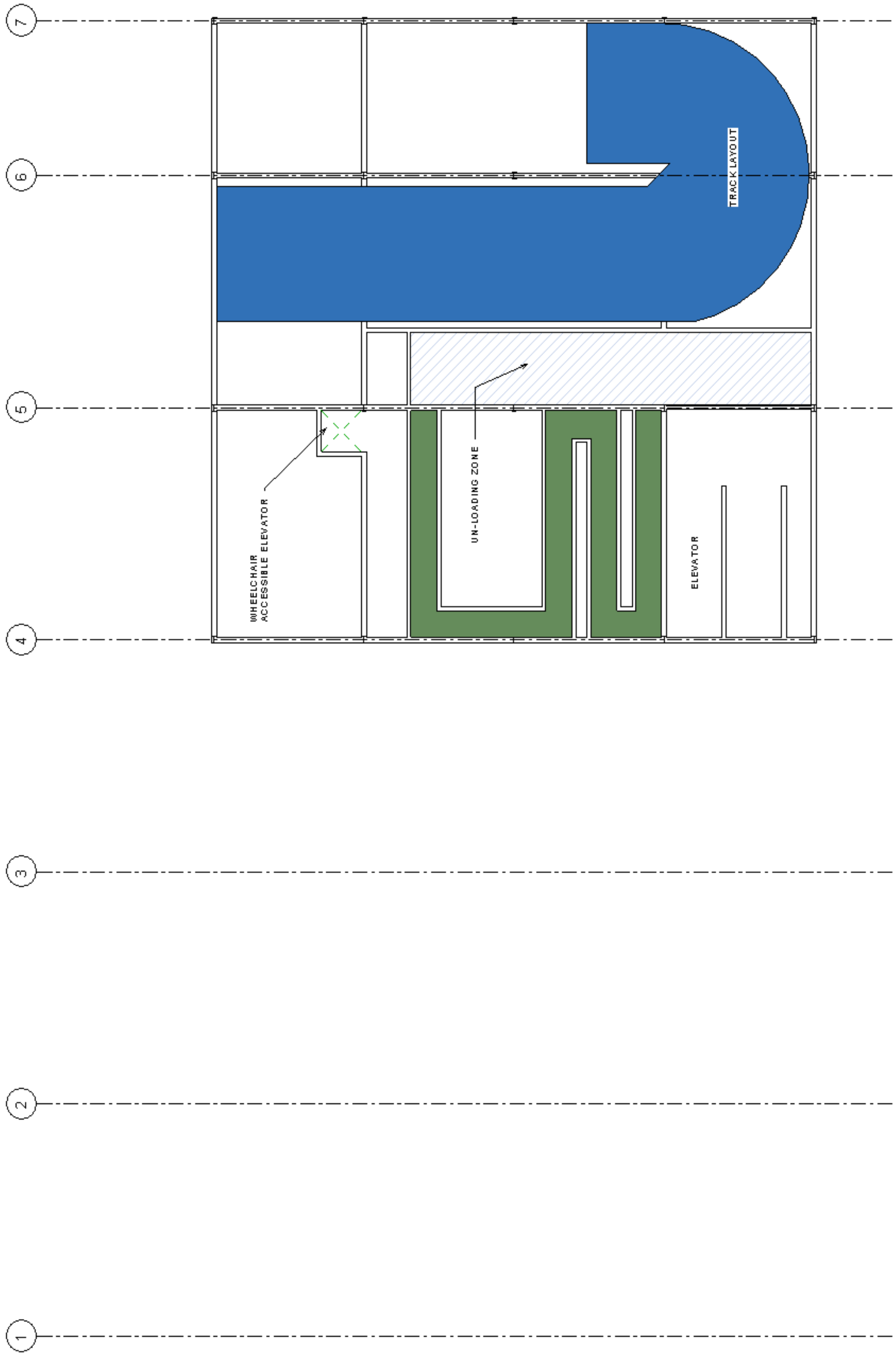


Fig. 30 - Queue Building - Annotated Basement Floor Plan

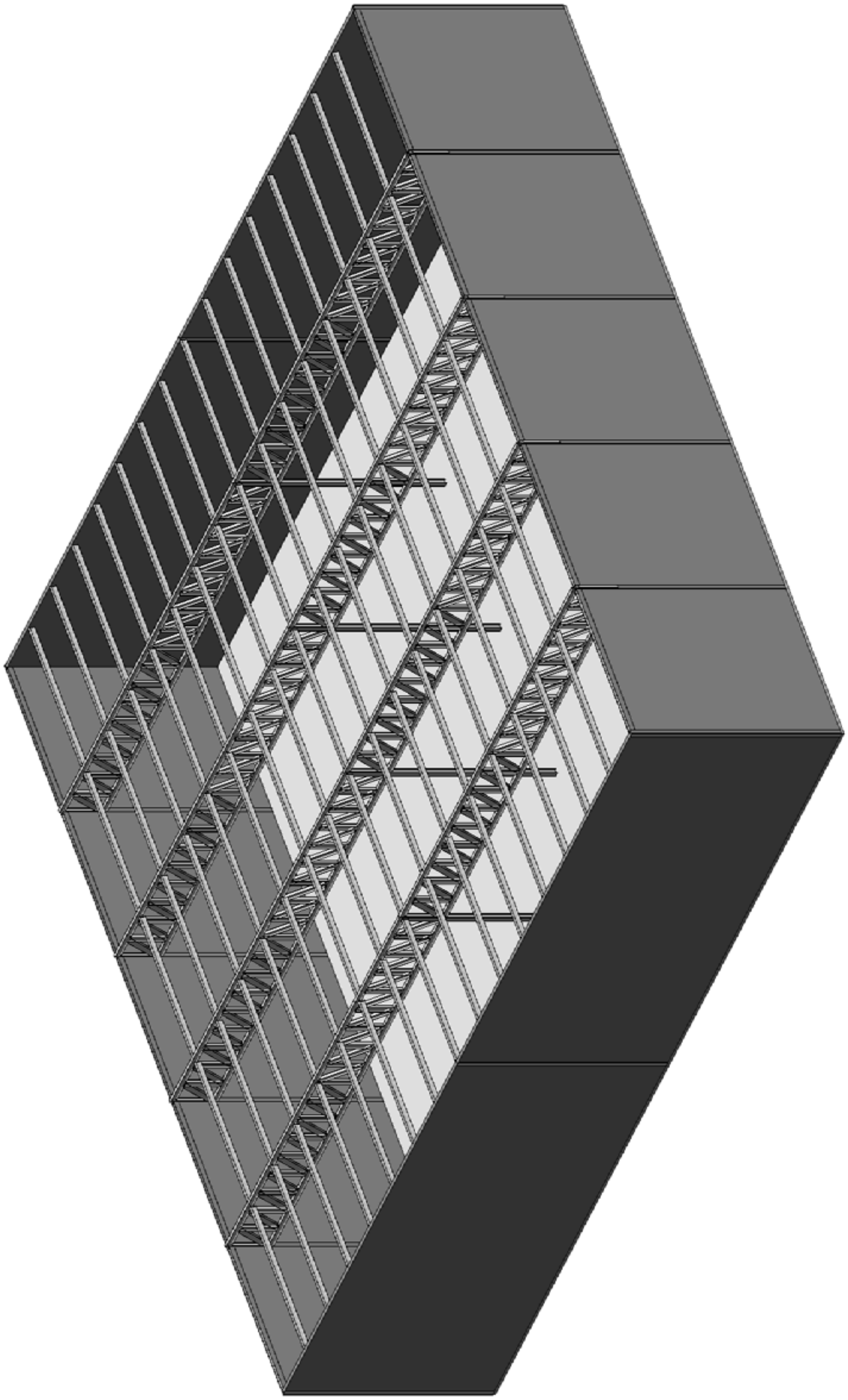


Fig. 31 - Show Building - Isometric View
Shows Truss System used to minimize column placement

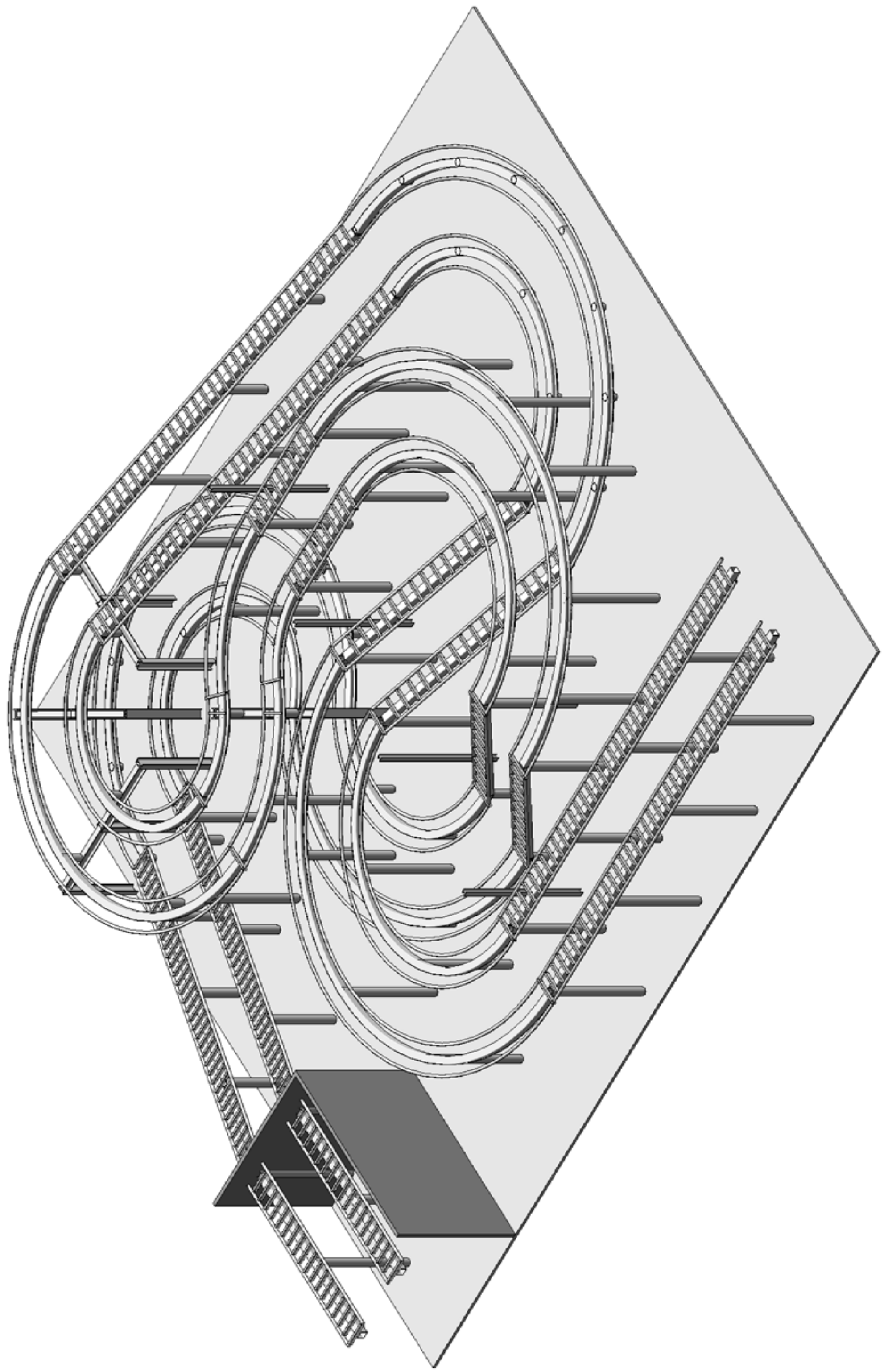


Fig. 32 - Show Building - Track Configuration - Isometric View

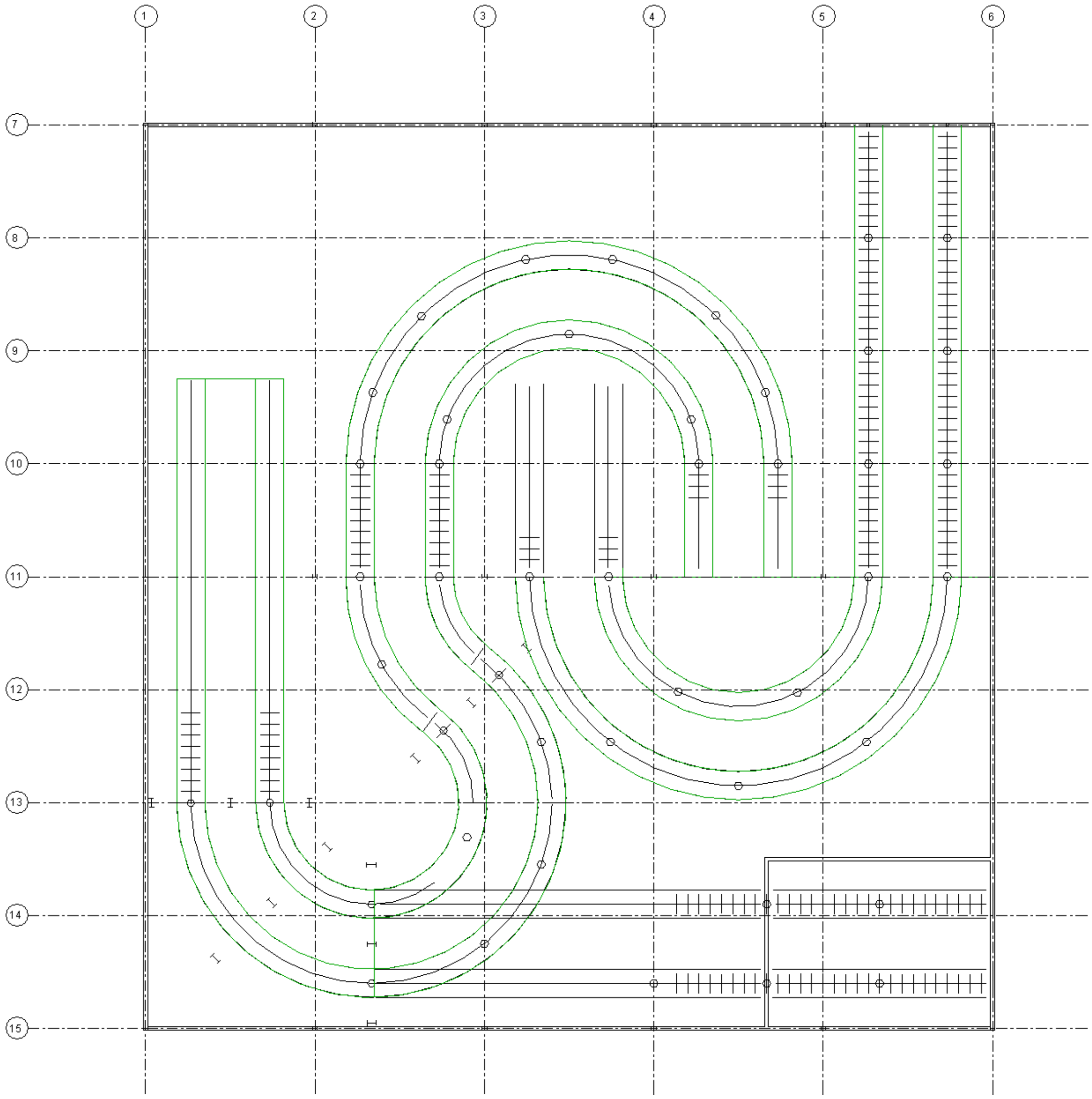


Fig. 33 - Show Building - Ground Level Track Placement

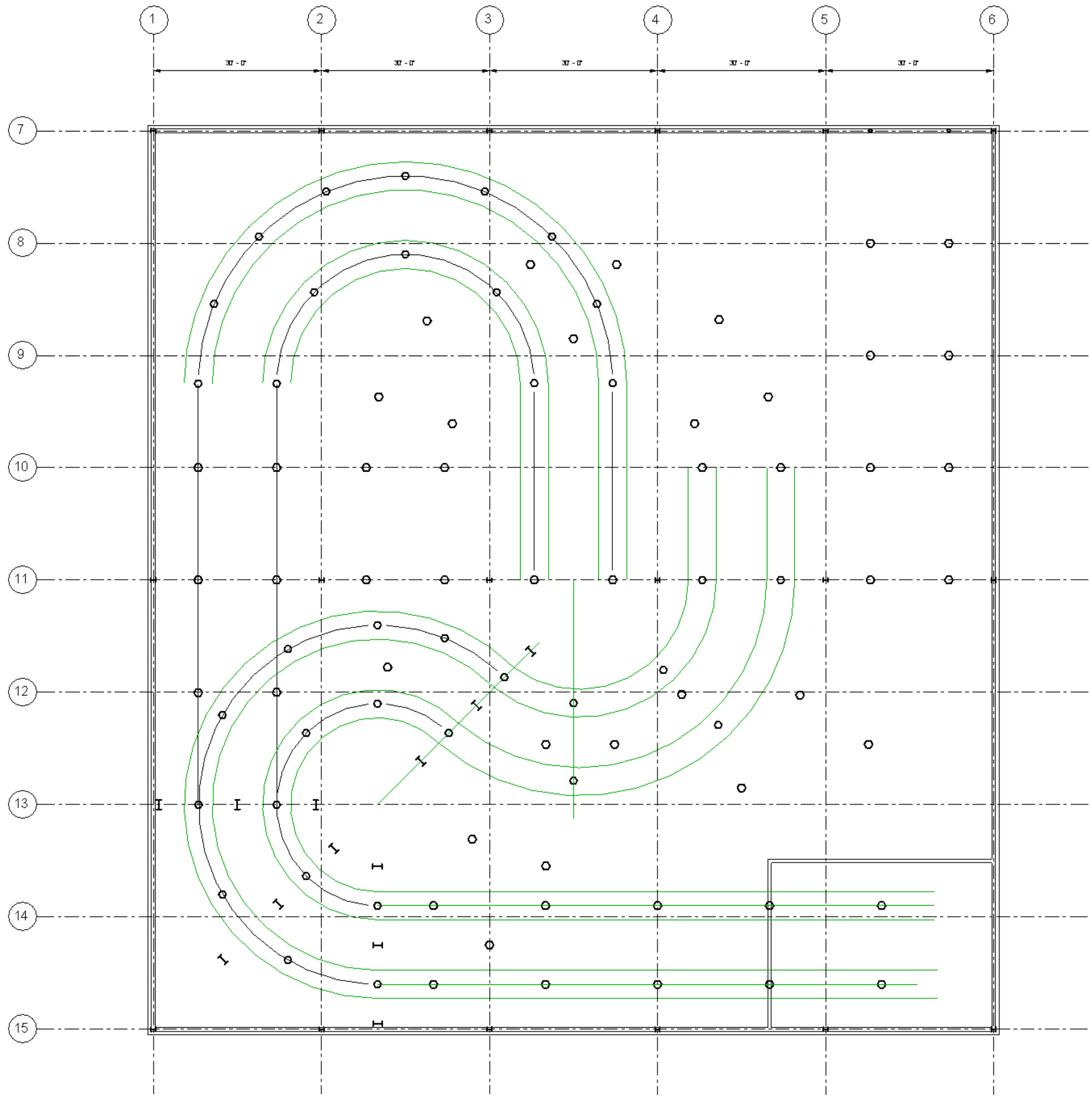


Fig. 34 - Show Building - Basement Level Track Placement

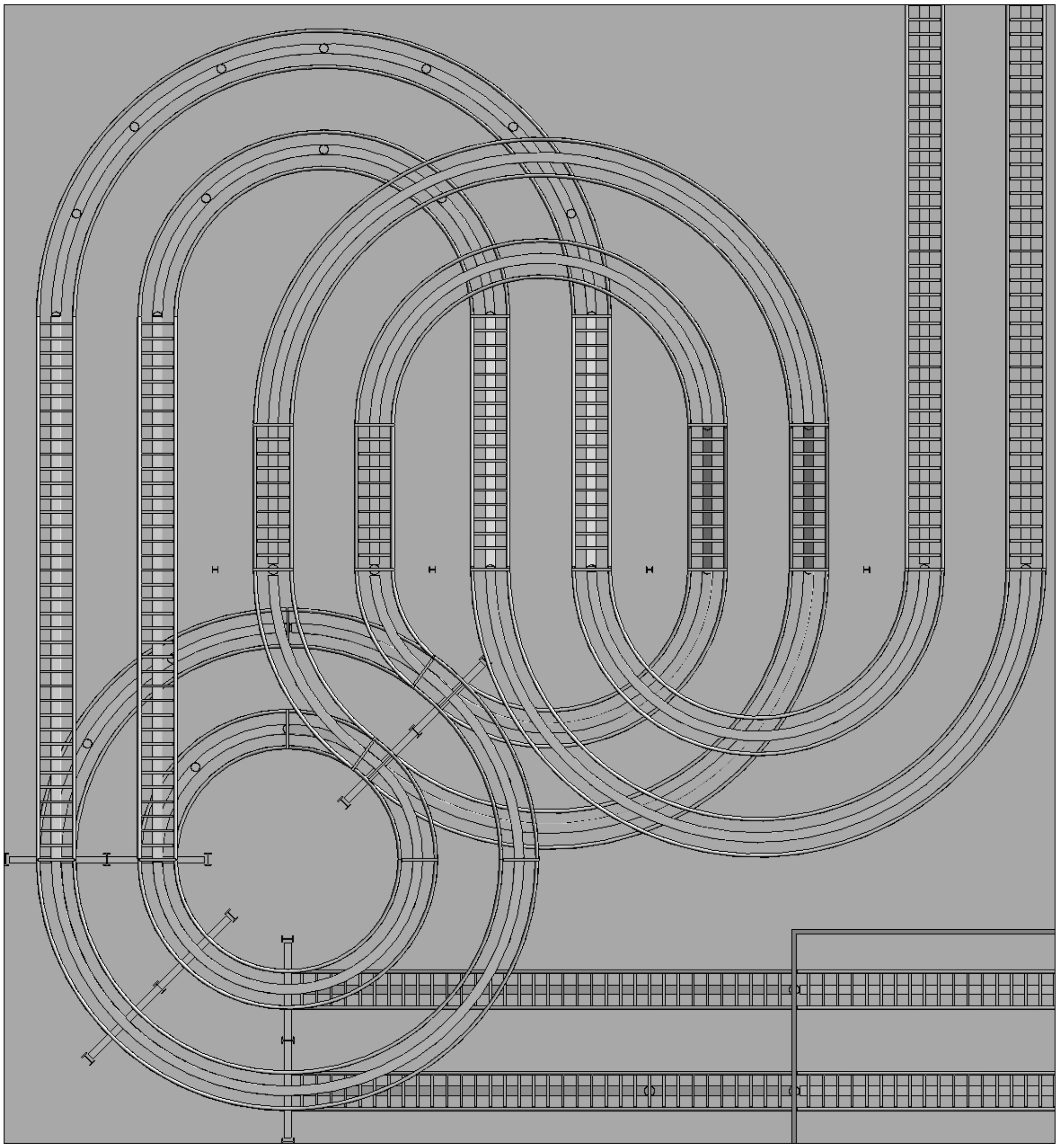


Fig. 35 - Show Building - Overview of Track Layout

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