A Structural Redesign of Terminal E at Logan Airport

A Major Qualifying Project
submitted to the faculty of

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degree of Bachelor of Science

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Abstract

Terminal E at Logan Airport now sees around 6 million international passengers a year after being built in 1970. With this high amount of traffic, renovating Terminal E to accommodate more passenger traffic would reduce crowding and improve efficiency. The objective of this MQP is to conduct a structural redesign of Terminal E with a focus on improving efficiency and sustainability.

Capstone Design Statement

This MQP focused on the structural redesign of Terminal E at Logan Airport. Terminal E is currently undergoing a massive renovation to modernize and accommodate more international traffic. Our team took inspiration from the current Terminal E design to redesign the terminal to make it more efficient and sustainable. This included designing the new layout, structural members, foundations, and cladding of the new terminal. During the planning and design of the terminal, our team considered the capstone design criteria to produce a better design by considering different important factors like cost, sustainability, and constructability.

The purpose of capstone design criteria serves to produce a well-rounded and cohesive project. Our team goal was to design a more efficient and sustainable alternative to the current terminal design. To achieve this, our team focused on cost, sustainability, and constructability to meet this capstone requirement.

Economic

Logan Airport's Terminal E is the international terminal and serves as the gateway for all international passengers to the city of Boston. With such a major transportation infrastructure project, much of the funding needed for an airport redesign comes from the federal government. As a result, the terminal must be designed in a way that is safe but not overtly costly. This includes making design decisions that could save costs on the overall project. In the case of our project, cambering longer spans allowed us to reduce section size and therefore save cost. In addition, we choose a roofing and cladding material for our terminal that would last, allowing us to save project costs in the long run.

Sustainability

With Logan airport's susceptibility to climate change, the focus should be producing a sustainable design. Terminal E at Logan is currently rated LEED Gold, just below the highest level of Platinum. While we did not have full details on how Terminal E achieved LEED Gold status, our team considered sustainability when designing our terminal. This included adding more windows to allow natural light in, utilizing photovoltaic glass, and designing a more space-efficient building to help reduce the carbon footprint. Other actions that could improve sustainability that were outside the scope of our project include: reusing water and runoff, implementing solar panels on the roof, and working to lower airport vehicle emissions.

Constructability

Constructability must be considered in the design. Designing a terminal that is difficult to construct can increase costs and put a project off schedule. For our Terminal E redesign, we laid out the terminal and determined structural framing sizes in such a way to ease the construction process. This included using more uniform beam and girder sizes to reduce the amount of framework needed and ensuring there was access for construction.

Professional Licensure Statement

In the current civil engineering industry, an engineer needs to have their Professional Engineer (PE) license to stamp or sign off on any engineering work for a project. To sign off on structural engineering work in some states, such as Illinois and Washington, structural engineers must have their Structural Engineering (SE) License. But in the state of Massachusetts, only a PE is required to sign off on any structural work. PEs and SEs are responsible for ensuring that any work they sign off on is both ethical and safe for the community.

Obtaining PE licensure varies from state to state. In general, engineers must first graduate from an ABET-accredited four-year program. After graduation, engineers must pass the 6-hour long Fundamental Engineering (FE) Exam to obtain their Engineer-in-Training (EIT) certification. Any engineers that work in industry directly after an undergraduate degree must have three years of working experience under a PE. For those who obtain their graduate degree, only one year of working experience is required. The cumulation is usually an 8-hour long exam that engineers must pass to obtain their license.

In some states, SEs are designated differently than PEs, and an SE is required to sign off on structural engineering work. Illinois and many states on the West Coast require an SE to sign off on structural work for certain types of buildings. The SE exam is a total of 16-hours long, with two 8-hour sections: vertical and lateral.

It is important to clarify that no one on this MQP team has their PE. Any of the engineering work that is presented in this report is the result of an undergraduate project, and an academic exercise. This project should not be used as a professionally designed project by a licensed professional. It would not necessarily reflect real-world, safe results.

Table of Contents

Table of Figures

List of Tables

Executive Summary

The Massachusetts Port Authority (Massport) is the sponsor of an ongoing project at Boston Logan International Airport, Terminal E Modernization. Since its inception in 1970, Logan Airport Terminal E has experienced a steady increase in passengers. It was built with the intention of building twelve gates that would carry more than 1.5 million international passengers per year but has shown an increase of up to 10 million in 2019. Terminal E is currently undergoing a full modernization and expansion (renovation crescent) within the footprint of the airport to efficiently accommodate current and projected international operations and passengers and to meet regional economic goals while minimizing environmental and community impacts.

The goal of this project was to structurally redesign Terminal E by taking inspiration and modeling it after the current design but highlighting our own distinct differences. We chose to modify the design in a way that we thought would improve the overall functionality of Terminal E. Then, we created a layout schematic of structural members, connections, gates, and the foundation using Revit software. We also looked for the best cladding system that gives the structure a modern look but is also friendly to the environment.

To start with this design process, it was necessary to visit the project site to learn more about the design of a project of this magnitude. Our team, together with our advisor, had the opportunity to tour and photograph the new expansion of Terminal E, which allowed us to learn about the process and materials used in the construction of this project. We were also given access to the Revit files of the existing Terminal E and the new design. Once these files were reviewed, we proceeded to start with our own design. The modernization project, like any other construction project, had to follow codes and be approved by entities that ensure construction complies with the corresponding safety. To carry out the designs, we abided by the Federal Aviation Administration (FAA), the International Building Code (IBC), the Massachusetts Building Code (MBC) standards, and American Institute of Steel Construction (AISC) standards. At the same time, we abided by the LEED guidelines since the intention is for this to be a sustainable project.

The first step was to design the terminal layout. We referenced the height of each floor, the full size of the entire terminal, the distance between the existing gates, and the new gates that are being added in the renovation crescent of the terminal. After the site tour and seeing the available land that could be worked with, we added an extra section compared to the original design. This extra section had three gates designed by our team, and one more gate in the renovation crescent, making a total of eight additional gates to the current structure of Terminal E. After reviewing FAA standards, we concluded that the best option for the extra section would be a pier shape.

Once the terminal layout was determined, we proceeded to design the structural framing. Steel with composite slab decking was the best material for design because it allows for smaller section depth and makes erection easier. An Excel sheet we created helped us choose the sizes of our beams, columns, and connections. In the three sections of the terminal—the main building, the renovation crescent, and the pier—the size of the girders and beams varied. In large sections, we decided to camber the members to reduce the size of the sections and save cost. As for the columns, there was also a variation in sizes, but most remained constant from one floor to another. For the connections, we selected a single-angle connection design. Larger beam and girder sizes would require a greater number of bolts. Another factor was the type of bolt and the size of the bolt.

To determine the foundation of the structure, we relied on the geotechnical report sent to us by Massport. From this report we could obtain the soil properties needed for completing a foundation design. Based on these numbers, we chose to use the Vesic equation because it allowed the building to support more weight and gave us a better representation of how strong and deep our piles need to be driven into the ground. Once the members and their sizes were determined, we used RISA, which determined if the beams passed the unit verification by comparing the capacity of the beams with the applied loads. Based on this result, we determined whether the beams performed as predicted by hand calculations. Finally, we proceeded to look for the best options for cladding. In our investigation, we were able to find that the current design will have aluminum composite panel for most of the structure and photovoltaic glass as well. Both materials improve the characteristics of the building but also make it more sustainable.

Despite the different challenges presented in the process, the group was able to achieve its goal of completing an alternative design for Terminal E (based on the current one) and in turn adding some possible changes that could help operations run more efficiently while offering quality services, providing comfort for passengers, and creating structures that are cost effective and environmentally friendly.

1 Introduction

Boston Logan International Airport is a world-class facility that serves as the primary airport for New England. Logan Airport's Terminal E is the airport's international terminal. In recent years, there has been an increase in passenger traffic, reaching 10 million in 2019. To improve the experience of travelers, the terminal is undergoing expansion and modernization. This modernization will allow the terminal to accommodate more international demand, which is good for the economy while at the same time reducing environmental impacts. Among the various updates added were additional gates, comfortable waiting areas, and dynamic glass to provide shade from sun glares. Our group was presented with the opportunity to structurally redesign Terminal E based on the current design while adding new ideas and possible improvements. As shown in Figure 1.1, our team and advisor were able to go on a site visit to learn more about the actual design.

Figure 1.1: Photo of team's site visit to Terminal E taken on the new roof

Our Massport contact, Swikriti Khanal (Project Manager), gave us a tour of the terminal, along with access to the files of the existing terminal and the terminal's renovation. From this, we were able to obtain the necessary measurements and the available space there was to start our design from scratch. Our team did research since every project must meet regular safety and efficiency standards. The team faced real-world design challenges due to building codes, transportation department standards, and available space, but we were able to meet the goal of presenting an efficient and environmentally friendly terminal.

2 Background

2.1 Current Design

Since its conception in 1970, Logan Airport's Terminal E has seen a steady increase in passengers. It was built with the intention to construct twelve gates that would transport over 1.5 million international passengers per year. As Figure 2.1 shows, international traffic increased to 6 million passengers in 2016, and almost reached 10 million in 2019.

Figure 2.1: Logan Airport Traffic Data From 2011 to 2021

The original terminal was designed in the shape of a rectangle with a small crescent jutting off the west side of the rectangle housing gates E1-E3. In order to accommodate the increasing passenger traffic, Terminal E was expanded in 2017 to twelve gates, three of which can handle Group VI aircraft. (Airport Technology, 2022)

Despite the upgrade, Terminal E still faced issues with unconsolidated ticketing areas and congestion at security checkpoints and gates. For this reason, another larger renovation called the

Terminal E Modernization Project is undergoing with a planned completion date of 2023 to add four new gates, making a total of sixteen gates.

These four new gates will be included as an extension in the shape of a crescent on the east side of the main rectangular part of the terminal. Seen in Figure 2.2, this modernization will add 320,000 square feet of space, expand baggage claim and ticketing areas, and increase sustainability. The new modernization project is owned by Massport, with Suffolk chosen as the main construction contractor, and AECOM and their partner company Luis Vidal and Architects as the architectural designers for the project. The most striking part of the new modernization will be the new roof of the building holding the four new gates, shown in Figure 2.3. The roof is comprised of three levels separating the club and the gate level with another smaller roof section in between. Each roof section is curved reaching the highest points at the middle of the terminal. As Luis Vidal explains, the roof of the extension was designed to follow the path of the sun, with two skylights facing the south. (Luis Vidal and Architects, 2017).

Figure 2.2: A conceptual image of the proposed extension of Terminal E

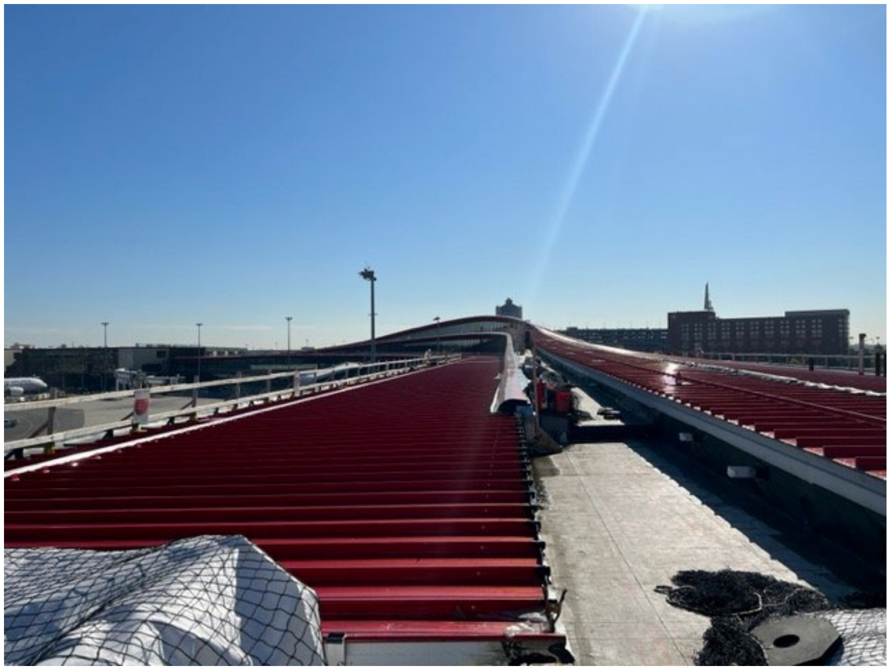

Figure 2.3: Photo of the Roof Taken from the Team's Site Visit

As each section of Terminal E has different purposes and different layouts, for the purpose of this report, we will name each section as follows: renovation crescent, main building, and E-C connector (Yu, 2022). Figure 2.4 represents a visual guide of Terminal E. The green section represents the main building, the red section represents E-C connector, and the blue section represents renovation crescent which is the new modernization that will be completed in 2023.

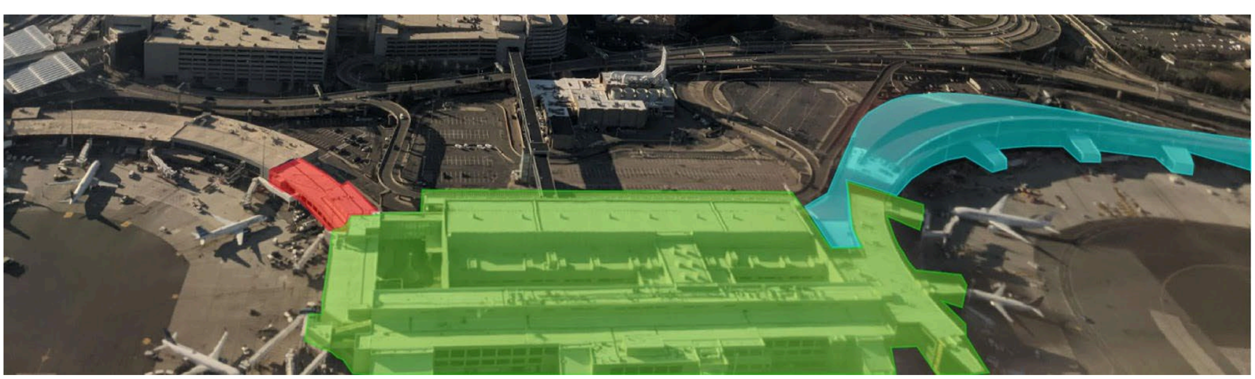

Figure 2.4: Bird's Eye View of Terminal E

2.2 Terminal E Levels

2.2.1 Renovation Crescent

Currently, Terminal E's renovation crescent consists of four levels. The main purposes of each level and what they are composed of are listed in Table 2.1. (Hussain, 2022). While this portion of the terminal has four floors, its primary purpose is to house the four new gates, along with the concessions and amenities needed for departing passengers. Figure 2.5 provides a detailed view of the third floor, to better break down the concessions and the departure locations.

Table 2.1 Purpose of Renovation Crescent Levels

Level	Main purpose	Comprised of
1	Shadow level	Storage space
2	Arrivals and Mechanical	Mechanical systems
3	Departures	Food and concessions Gates E13-E16
4	Clubs	Airline clubs and lounges

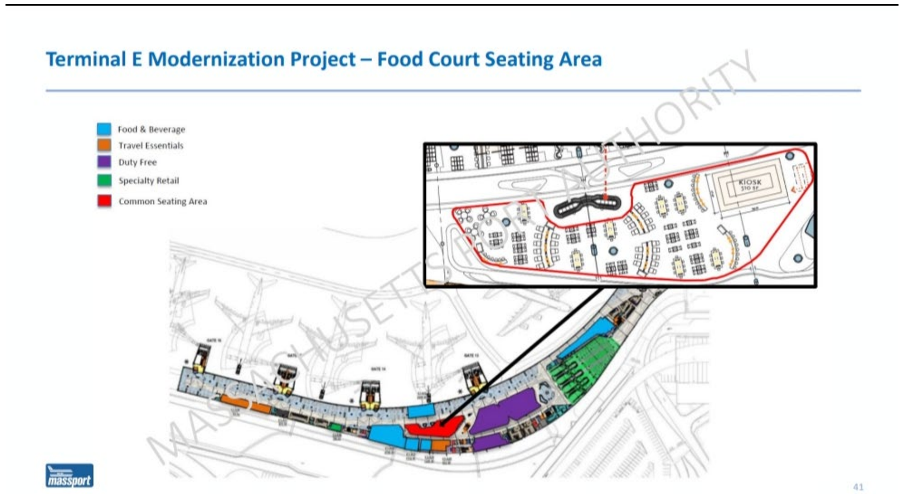

Figure 2.5: Visualization of Interior Plans for Renovation Crescent

2.2.2 Rectangle

The main building of the terminal consists of three levels. The purpose and amenities of each level is shown below in Table 2.2.

Table 2.2 Purpose of Main Building Levels

Level	Main purpose	Comprised of
1	Arrivals	• Storage space
2	Arrivals	• Mechanical systems • Customs • Transportation ○ Shuttles, taxis, trains, etc. • Gates E1a, E1b, E2
3	Departures	• Ticketing • Food and concessions • Gates E3-E12 • US Customs and Border Protection • Security checkpoint

On the first floor, there are locations for baggage claim and concessions. Figure 2.6 shows the location for baggage drop-off and check-in at the back of the level and the areas for car pickup and drop-off at the front of the terminal. On the second and third floors, we can find restaurants and shopping areas. Figure 2.7 shows the combined layout of levels two and three on the E-C connector and the original main building, including the security checkpoint, access to gates E1A to E12, shopping areas represented by the green dots, restaurants by the orange dots, and amenities by the green dots.

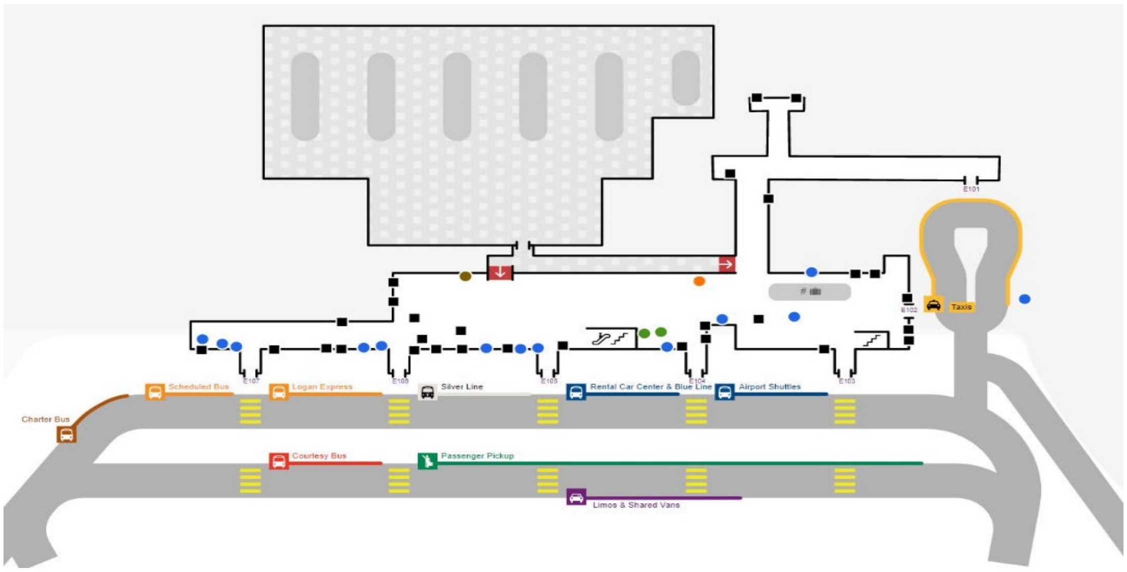

Figure 2.6: First Floor Layout of Main Building

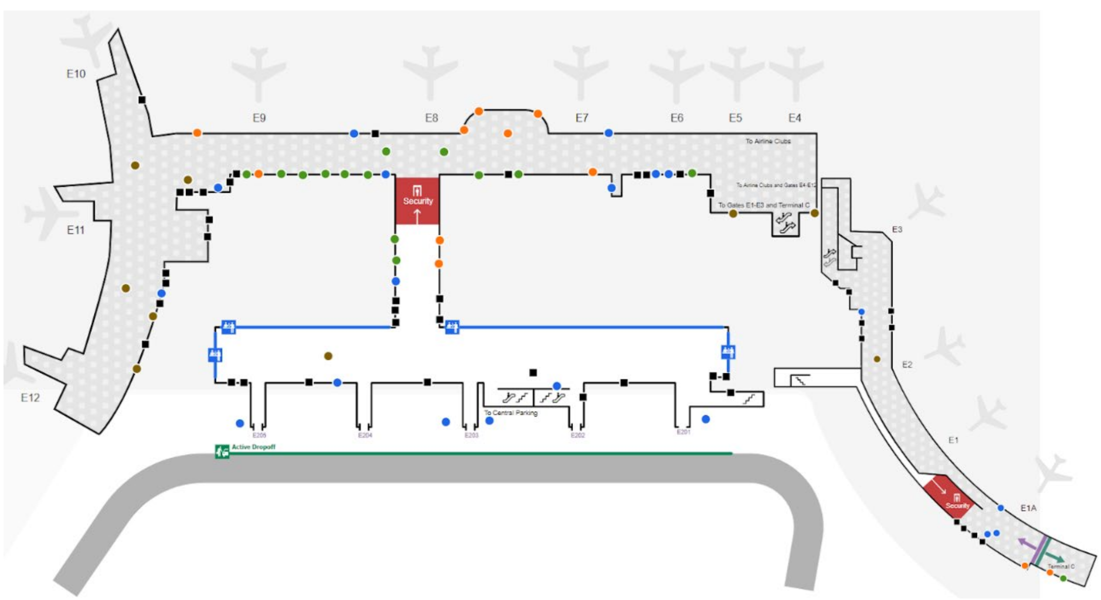

Figure 2.7: Second and Third Floor Layout of Main Building

2.3 Terminal Shape Design

Terminal shape design and layout is one important factor that is considered when designing an airport terminal. Depending on the available tarmac space and the layout of surrounding roadways, different terminal shapes can be the most efficient to implement with the least wasted space. When redesigning the terminal, it should be taken into consideration that the work would only be conducted within the existing airport footprint on land that is already impervious and paved. Understanding the available land, we considered several shapes that would make the best use of land and would allow us to design the best possible terminal. There are several different terminal shapes as shown in Figure 2.8, that have been studied and implemented in the past that are determined to be the best possible shapes for an airport terminal.

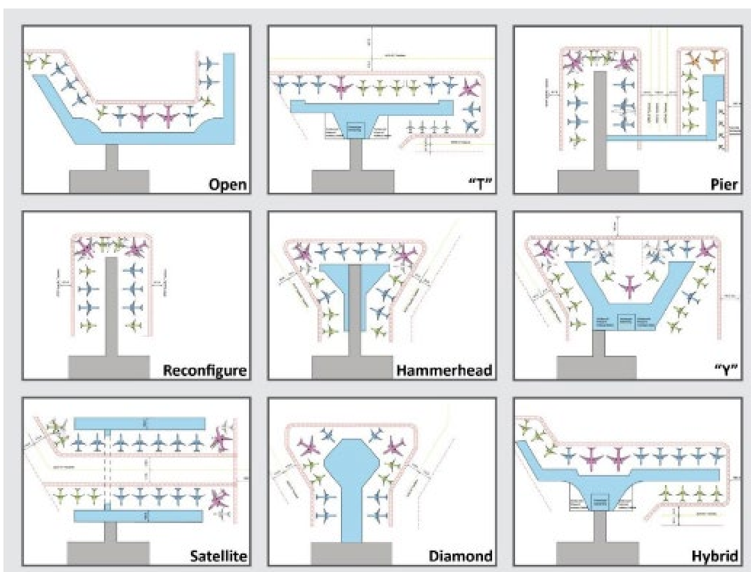

Figure 2.8: Common Terminal Shapes

Two common designs include piers and T-shapes (Black, 2018, p.15). A pier shape is a straight, narrow extension off the main part of a terminal, with aircraft parked on both sides. Pier shapes are commonly used at aircraft terminals because they are simple to design and allow enough space for aircraft. Implementing moving walkways in pier-shapes is also easier because there are no curves, resulting in shorter walking distances for passengers (Ashford, N. J, 2023). Pier-shapes can be found in most major airports around the world, including Terminal E at the San Francisco International Airport and Terminal 3 at Chicago O'Hare's Terminal 3 (Figure 2.9).

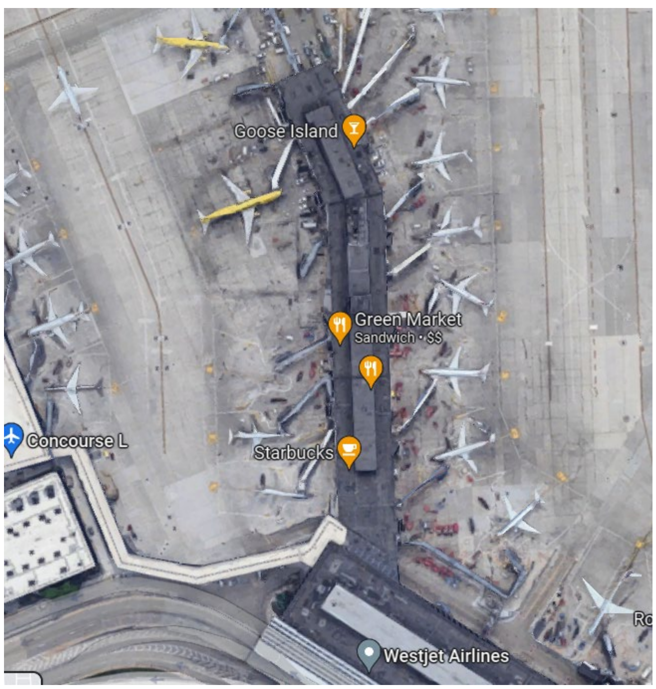

Figure 2.9: Bird's Eye View of Example Pier Shape

A T-shape terminal features a pier-shape with another straight section perpendicular to the end of the pier. Like pier-shapes, aircraft can be parked on either side of a T-shape, and implementing moving walkways to reduce a passenger's linear walking distance is easier because of the straight design. T-shapes can be found implemented in airports across the world, including Terminal C at Logan Airport (Figure 2.10)

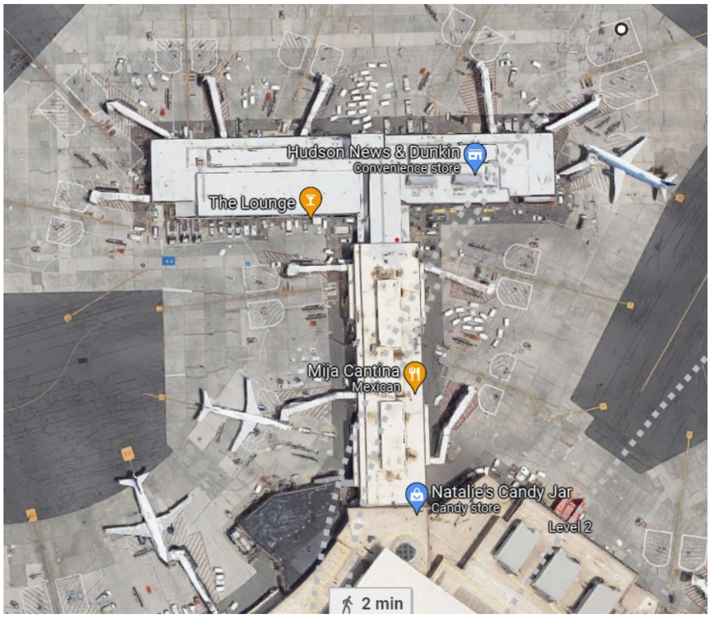

Figure 2.10: Bird's Eye View of Example T-Shape

Currently, Terminal E at Logan Airport has a hybrid shape. The main building of Terminal E is the main part of the terminal because it houses customs and ticketing and would be considered a linear shape. The two crescents coming off this rectangle are not quite like a linear shape but could be considered hybrid.

2.4 Design Considerations and Codes

2.4.1 FAA Design Codes

To verify if the modernization project complies with safety requirements, the layout plans must go through the approval of different entities. In the case of airport systems and everything related to them, the revisions to the airport layout plan require the approval of the FAA. As part of the Department of Transportation, the FAA (Federal Aviation Administration) is responsible for the creation and oversight of standards for maintaining and running aircraft. The FAA's Airport Division provides leadership and assistance to the aviation community to achieve the goal of a secure and efficient airport system. (FAA Mission and Responsibilities, n.d.)

The modernization of Terminal E will include new gates, so the design must be governed by FAA standards, including consideration of the size of the aircraft allowed and the appropriate spacing between the aircraft at the boarding gates. The FAA developed the Airplane Design Group (ADG) to categorize the aircraft based on their size. Table 2.3 shows how the aircraft are divided into six groups based on wingspan or tail height, with Group I being the smallest aircraft and Group VI the largest aircraft (AC 150/5300-13B, 2022). A typical commercial group III jet could be a Boeing 737, while a typical Group V jet could be a Boeing 787.

Table 2.3 FAA Aircraft Groups

Group #	Tail Height (ft)	Wingspan (ft)
I	<20	<49
II	20 - <30	49 - <79
III	30 - <45	79 - <118
IV	45 - <60	118 - <171
V	60 - <66	171 - <214
VI	66 - <80	214 - <262

One of the reasons for the renovation at Terminal E is to provide access for Group VI (e.g., the Airbus A380) planes. These are the largest commercial planes, with wingspans between 214 ft and 262 ft; they are often used in international travel and certain models can hold around 853 passengers (Airbus A380). At this moment, the gates in Terminal E are only large enough to accommodate up to Group V (e.g., the Boeing 747) planes. In comparison to Group VI, these planes have an average wingspan between 171 ft and 214 ft, with the capacity for around 524 passengers (Boeing 747-400). When redesigning the terminal, it will be important to accommodate Group VI aircraft at several gates.

The FAA also provides regulations on the distance between the airplanes at their parking gates with guidelines that are dependent on the planes themselves. As shown in Figure 2.11, airplanes are required to have at least 25 ft of clearance from wing to wing when parked next to one another, and 45 ft of clearance when parked next to a pier. Therefore, if the gates are to be modified for a larger class of plane, they need to be the length of the wingspan of the largest desired plane plus 25 ft. If the plane is going to be docked next to a pier, half of the wingspan and 57.5 ft will be required for the gate.

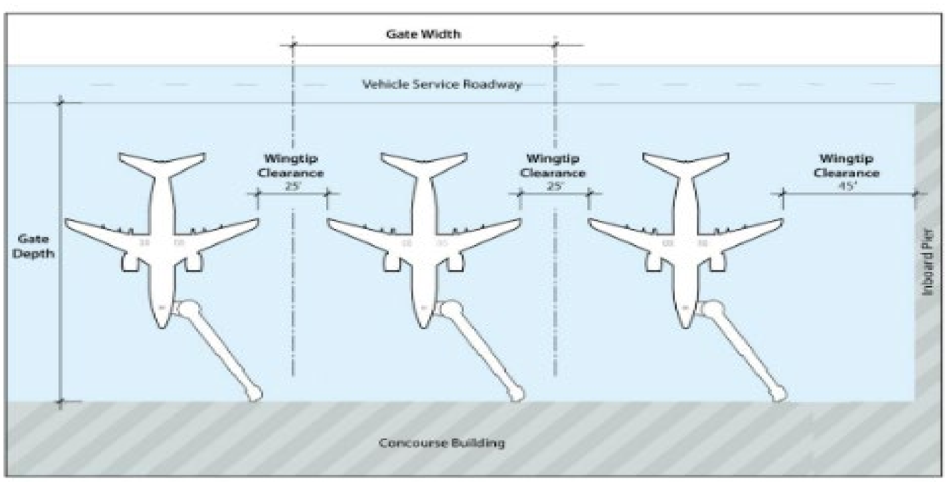

Figure 2.11: FAA Spacing Requirements for Aircraft

In addition to complying with FAA codes, the project must follow other codes, such as IBC and MBC for design development.

2.4.2 International Building Code and Massachusetts Building Code

To determine the loading and design considerations of the terminal, we followed the guidelines of the International Building Code (IBC). Information related to the height, structural design, building material, and foundation were determined from the IBC. This will be used in conjunction with the Massachusetts Building Code (MBC). The MBC follows the same format as the IBC; however, town-specific data is provided to allow buildings to be up to both state and federal code. Several of the chapters that were used for this project are:

- Ch 3. Occupancy
- Ch 5. General Building Heights
- Ch. 16 Structural Design
- Ch. 18 Soils and Foundations

When designing the terminal, geographical location should be considered. Since the terminal is on the waterfront, consideration for high winds and water needed to be incorporated in the design process. For Massport projects, floodproofing is a requirement, but more research is required into the flood proofing strategies for buildings on the coastline. According to Massport standards, the exteriors must contain design features that protect from airborne storm debris, extreme winds, and water. The windows, doors, and openings are required to be water intrusion resistant. (Massport, Sustainability and Resiliency Design Standards and Guidelines, 2018, p.10)

2.4.3 Construction Noise

One aspect to this project is that the additional noise created at the airport during construction might become a problem for residents in the nearby area. Currently, the noise from the airport reaches a maximum of 75 dB, with most of the surrounding area experiencing around 60 to 65 dB (Massport, 2015). The city of Boston allows a maximum of 86 dB from 50 ft away for construction projects.

The FAA, in collaboration with Massport and an engaged public advisory council, took part in a comprehensive noise analysis of the nearby locations surrounding Terminal E from Massport, as seen in Figure 2.12 (Massport, n.d.). This analysis examined the projected sound levels at each spot, based on the equipment needed for the project. Based on these results, the maximum construction sound levels, L_{max}, are only at 70 dB, as it is shown in Figure 2.12, which is well under the city ordinance, and around the general noise level created by the airport. Construction of the terminal should therefore not have any effect on the surrounding area.

	Project Sound Levels		City of Boston Criteria[1]	
Receptor Locations	L_{10}[2]	Lmax[3]	L_{10}	Lmax
Receptor 1 - East Boston Memorial Park (Tennis Court) - Boston	60 - 69	54 - 70	80	N/A
Receptor 2 - East Boston Memorial Park (Football Field) - Boston	58 - 67	52 - 67	80	N/A
Receptor 3 - Intersection of Bremen Street and Putnam Street - Boston	55 - 65	49 - 65	75	86
Receptor 4 - Swift Terrace - Boston	55 - 66	48 - 64	75	86
Receptor 5 - Intersection of Short Street and Coleridge Street – Boston	55 - 66	48 - 63	75	86
Receptor 6 - Intersection of Thurston Street and Bayswater Street – Boston	50 - 60	43 - 59	75	86
Receptor 7 - New Court Road near Albert Ave – Winthrop	50 - 60	43 - 58	75	86
Receptor 8 - Intersection of Foam Street and Grand View Avenue – Winthrop	44 - 57	37 - 53	75	86
Receptor 9 - Intersection of East 1st St. and Farragut Road – South Boston	45 - 56	37 - 53	80	N/A

1 City of Boston's noise criteria for residential or recreational use.
2 L_{10} represents total sound level of all equipment.
3 L_{max} represents sound level of noisiest piece of equipment.

Figure 2.12: Projected Sound Levels as Compared with the City of Boston's Noise Criteria

2.5 Sustainability Considerations

2.5.1 LEED Guidelines

Massport is dedicated to reaching LEED® criteria for new construction projects. For our terminal design, our team followed the sustainability considerations that Massport does. Leadership in Energy and Environmental Design, usually LEED, is an internationally recognized green building certification system, providing independent third-party verification that a building was designed and built using strategies aimed at improving "performance in energy efficiency, emissions reduction, water and natural resource conservation, and more". (Massport, n.d.) On average, Massport's LEED certified buildings are 28% more energy efficient than ordinary buildings of the same kind and perform 9% better than design. On-site solar generates up to 7% of the power utilized in these buildings. Green Bus Depot, Terminal A, Rental Car Center, and Terminal E New Large Aircraft Wing are all LEED-certified structures at Boston Logan Airport. The John A Volpe Terminal E New Large Wing Aircraft also received LEED-Gold certification in 2017. (Massport, 2018 Annual Sustainability and Resiliency Report, 2018, p.16).

Just as other current airport projects comply with LEED certification and are therefore more sustainable, the modernization of Terminal E aims to be another sustainable project. In the 2018 report, Massport developed policies to reach energy and greenhouse gas (GHG) emissions reduction goals. They implemented initiatives including an energy-efficient heating, ventilation, and air conditioning (HVAC) system in the Terminal E New Large Aircraft Wing. Following these initiatives would make a project more sustainable because this system increases engineering efficiency while also providing airports with improved control and management of their systems. In addition to this idea, the materials used within the terminal should be taken into consideration too. Using renewable and recyclable resources is more beneficial for the environment, reduces the carbon footprint, and increases the level of LEED certification of the building.

2.5.2 Sustainability and Resiliency Design Guidelines (SRDG)

In addition to pursuing LEED, we also followed the guidelines of the Sustainability and Resiliency Design Standards (SRDG). Some material options for this project were sustainable concrete or recycled materials according to Massport's Sustainability and Resiliency Design Standards and Guidelines (SRDSG) (Massport, Sustainability and Resiliency Design Standards and Guidelines, 2018, p.19). Locally sourced materials would also reduce the carbon footprint of the building process; however, preference was given to durable materials that would increase the lifespan of the building.

Logan Airport is built around protected wetlands with endangered species. Any construction or additional building would need to take the safety of the wetlands into account, according to the Massachusetts Wetlands Protection Act. However, it was noted in a prior report that construction shouldn't have any more effect on the wetlands than the current airport does. To avoid any problem, during construction the crew must dispose of materials properly and act with respect to the wetlands. (Massport, Boston-Loan International Airport Runway Safety Area, 2011, p.51).

2.5.3 Massport Floodproofing Design Guide

As a result of climate change, Logan International Airport is becoming increasingly vulnerable to flooding dangers induced by strong storms and rising sea levels.

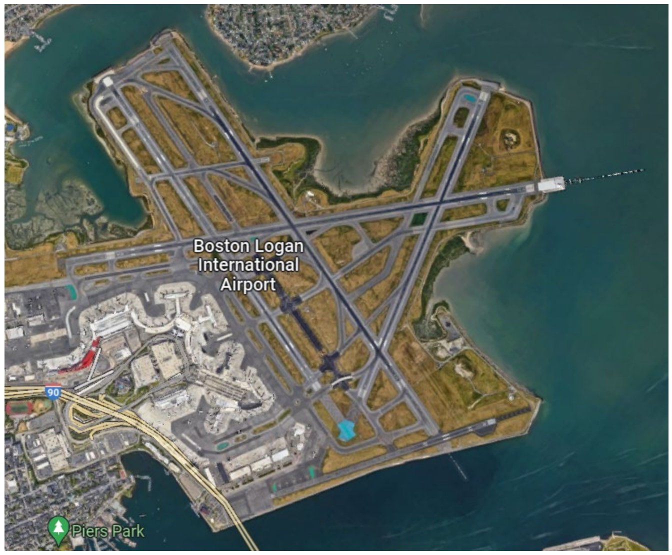

Figure 2.13 Birds-eye-view of Logan Airport showing the Surrounding Ocean

Logan Airport is surrounded by land to the north, south, and west and by Boston Harbor to the east, as shown in Figure 2.13. This harbor lies on Massachusetts Bay, an arm of the Atlantic Ocean. Considering the possible threat of rising sea levels to the airport, our team followed the standards of the Massport Floodproofing Design Guide. Our design must meet these standards since these guidelines assist in making infrastructure and operations more robust to expected flooding hazards. There are some projects that have been developed under these guidelines, from which the team took reference to make the project more sustainable. In 2017 Massport modified the Civil Air Terminal at Hanscom Field due to a heavy precipitation event that caused significant damage to the building and major impacts. Putting floodproof doors at side entrances and enhancing overall site drainage were some of the changes of this project (Massport, Sustainability Report Final, 2019). In 2018, the resiliency measures taken at the airport by Massport in the face of possible flooding was to locate air intakes and HVAC equipment above ground level, that is, above potential flood levels. This measure helped and continues to help protect valuable equipment from potential flooding and keep the airport running smoothly (Massport, 2018 Annual Sustainability and Resiliency Report, 2018, p.19). Implementing several of these measures would make the project more sustainable.

At the same time, we will be guided by the Design Flood Elevations (DFE) standards found in this guide. For existing installations, the DFE indicates that the lowest floor shall be elevated to or above the design flood elevation, which is 13.7 ft (NAVD88) for installations at Logan Airport. The DFE will also be useful to determine the minimum effective levels of protection provided by wet and dry waterproofing designs (Massport, Massport Floodproofing Design Guide, 2018). By following these guidelines and using several ideas from projects already built successfully, our project will be more sustainable.

2.6 Possible Changes

2.6.1 Increasing Number of Gates

One thing that can be improved upon with the current terminal E design is the addition of more gates. With the increasing traffic demand placed at Logan Airport, more gates are needed to serve the growing number of passengers. To determine the number of gates required to service an airport, we need an awareness of present capacity as well as future requirements based on anticipated activity (Transportation Research Board, 2010, p. 17). Massport's evaluation planning team performed this analysis. They examined terminal traffic projections, arrival and departure times at each gate, the number of passengers who use each gate, and the types of flights that depart from there, among other things. Collecting and properly analyzing this data is beyond the scope of our project; therefore, our team will design a terminal with seventeen gates based on Terminal E's 2023 design. Our team plans to redesign Terminal E to match this number of gates and, if the length is permissible by FAA standards, add an additional gate.

2.6.2 Additional INS Corridor

An INS Corridor, standing for Immigration and Naturalization Service Corridor, is the security checkpoint (TSA) that international passengers go through before boarding their flights. The current design of terminal E at Logan Airport features one INS corridor in the center of the main building on the third floor. This could be a drawback because on heavy travel days when there are more passengers traveling through the terminal, this would become a congestion point. Increased congestion at this one point in the airport will result in crowding and delays for passengers to get to their gates. Our team plans to shift the existing INS corridor more towards the side of the main building and add an additional INS corridor for a TSA security checkpoint on the other side. Since the INS corridor has only one opening, long waiting lines are created to carry out the check-in process and for passengers to access the boarding gates. Even the distance between the security point at the INS and some gates causes long walks for the passengers. Two INS corridors will split the number of passengers going through each, thereby decreasing congestion and resulting in less stressful airport travel. Another advantage of adding an additional INS corridor is that it can decrease the linear walking distance to a gate as passengers can go through the security checkpoint closest to their gate. Although adding another corridor will need an increase in staffing and will be more costly during the construction process, the benefit of decreased crowding will allow the terminal to operate more efficiently for years to come.

2.6.3 Increasing Gate Seating

The gates at the renovation crescent are designed slightly differently than the rest of the terminal. To board their flights, passengers must descend a level from the main part of the terminal to access the ramp to their aircraft (either by stairs or elevator). The gate seating is located on the third floor in the building before descending a level into the passenger boarding bay, as shown in Figure 2.14.

Figure 2.14: Photo of Passenger Boarding Bay from Renovation Crescent

While these boarding bays are an efficient use of space, no resting space in the passenger boarding bay can be inconvenient for physically impaired passengers. Although there is an elevator to provide assistance, adding additional seating in the boarding bay will allow space for passengers to rest if before boarding their plane if there are long weights, and improve the experience of physically impaired passengers. The Kuala Lumpur International Airport features a similar design where passengers walk down a flight of stairs to access the boarding ramp but include seating at the bottom of the stairs. Taking inspiration from this airport, our team plans to widen the passenger boarding bay slightly to allow space for 10-20 seats at the bottom of the stairs. This will increase overall gate seating and benefit impaired passengers by providing them with a spot to rest before boarding their flight. Including this in our design will provide a better experience for passengers at Terminal E.

2.6.4 Roof Design

Massport's current design for the new terminal roof is shown previously in Figure 2.2 and 2.3. It features a curved end roof that is angled downwards. There are three segments in the terminal roof dividing the fourth level the third level and on in-between the third and fourth level. The material used to construct this was aluminum. The three segments, while eye-catching, is harder to construct and therefore is more costly. Our team plans to take inspiration from the nearby Atlantic Ocean to create a roof with two segments that replicated an ocean wave.

3 Methodology

This section covers the methods used in the design of our terminal. During the design process, we determined loading, section sizes for structural members, foundations, connections, and the use of software. The procedure used for each of these steps, any relevant equations, and how we utilized design software is described in more detail below.

3.1 Loading

3.1.1 Load Combinations

There are two main design methods: Load and Resistance Factor Design (LRFD), and Allowable Stress Design (ASD). The main differences between the two is that both use a different procedure for calculating design loads, and LRFD uses resistance factors while ASD uses safety factors. For this project we used LRFD design as our team was most familiar with this design methodology.

Figure 3.1 shows the LRFD load combinations used from ASCE Standard Section 2.2 (American Society of Civil Engineers, 2010):

Basic Combinations	**Symbols**
1. 1.4D	D = dead load
2. 1.2D + 1.6L + 0.5(Lr or S or R)	L = live load
3. 1.2D + 1.6(Lr or S or R) + (L or 0.5W)	Lr = roof live load
4. 1.2D + 1.0W + L + 0.5(Lr or S or R)	S = snow load
5. 1.2D + 1.0E + L + 0.2S	R = rain load
6. 0.9D + 1.0W	E = earthquake load
7. 0.9D + 1.0E	W = wind load

Figure 3.1: LRFD Load Combinations

The methods for calculating the described loads above are explained in detail in the following sections. To produce a more conservative design, for the design of each section we chose the load combination that was the largest.

3.1.2 Dead Loads

The dead load is a combination of all objects that loads the structure permanently. This includes the self-weight of the structure. As an airport terminal has long spans, the structure was designed with steel framing because it is easier to erect and allowed us to keep smaller section sizes compared to what would be needed for reinforced or prestressed concrete. Composite slabs were used for the flooring because of the higher strength to weight ratio, and cheaper cost.

To calculate dead load of the structure we used the following procedures:

1) Roof: Our terminal was designed with an aluminum roof. We assumed a conservative estimate of 10 psf.
2) Floor Slabs: Steel: A Vulcan 3VLI-36 composite flooring slab with a depth of 5-1/2 in and deck gage of 19 was used, which gave a dead load of 39.1 psf. The selected composite slab is highlighted in blue in Table 3.1.

Table 3.1 Vulcan Composite Slab Properties

3VLI-36/3VLJ-36/3PLVLI-36 COMPOSITE DECK-SLABS
LIGHT WEIGHT CONCRETE (110 pcf)
LRFD

Slab Depth			Maximum Unshored Spans			Composite Deck-Slab Properties			
		Deck	Maximum Unshored Construction Clear Span			Concrete + Deck	Deflection $I_d = (I_{cr}+I_u)/2$	Moment $ØM_{no}$	Shear $ØV_{no}$
Total	Topping	Gage	1	2	3	(psf)	(in^4/ft)	(kip-ft/ft)	(kip/ft)
5"	2"	22	11'-1"	11'-10"	12'-2"	33.8	5.75	4.95	4.02
		20	12'-7"	13'-8"	14'-1"	34.2	6.19	5.86	5.48
		19	13'-1"	15'-3"	15'-5"	34.5	6.59	6.70	5.61
		18	13'-6"	16'-7"	15'-10"	34.8	6.94	7.47	5.61
		16	14'-3"	17'-9"	16'-9"	35.6	7.66	9.09	5.61
5½"	2½"	22	10'-7"	11'-4"	11'-8"	38.4	7.51	5.45	4.30
		20	12'-2"	13'-1"	13'-6"	[illegible]	8.07	6.46	5.77
		19	12'-9"	14'-7"	14'-11"	39.1	8.57	7.39	6.36
		18	13'-2"	15'-11"	15'-5"	39.4	9.02	8.23	6.36
		16	13'-10"	17'-3"	16'-3"	40.2	9.93	10.02	6.36
6¼"	3¼"	22	9'-11"	10'-6"	11'-0"	45.2	10.78	6.26	4.75
		20	11'-9"	12'-4"	12'-9"	45.6	11.57	7.43	6.21
		19	12'-3"	13'-10"	14'-3"	45.9	12.27	8.51	7.44
		18	12'-8"	15'-1"	14'-10"	46.2	12.89	9.49	7.55
		16	13'-4"	16'-7"	15'-8"	47.0	14.16	11.56	7.55

Note:
1. Maximum unshored spans do not consider web-crippling. Required bearing should be determined based on specific span conditions.

3.1.3 Live Loads

The 9th edition of the Massachusetts Building Code does not include any amendments for dead and live loads, so the IBC manual was used to find the following loads. According to the IBC, there are no specific live loads used for airports, so we determined the most appropriate live load by the section of the terminal (e.g., gate seating, concession areas, check in, security checkpoints).

1) Level 1: Level 1 is considered a ghost floor, meaning that it is not for use by passengers. Here, HVAC and other mechanical systems for the building will be stored. The following live load was used: 100 psf for corridors, and 10 psf for Mechanical, Electrical, and Plumbing (MEP).

2) Level 2: Level 2 is the arrivals floor of the terminal, including the baggage claim area and a large open space leading to bus and passenger pick up. The following live loads were used: 100 psf for baggage claims and lobby area, and 10 psf for MEP.
3) Level 3: Level 3 is the departure floor of the terminal, including three gates and two concession areas. For simplicity, our team assumed the same live load for the concession area and gates. The following live loads were used: 100 psf for gate and concession areas, and 10 psf for MEP.
4) Level 4: Level 4 is the departure floor of the terminal, including nine gates, shopping, and concessions. As shopping, concessions, and restaurants are in the same section of this floor, we assumed the live loads. 100 psf was used for gates and concession area, and 10 psf for MEP.
5) Roof: Our roof is non occupiable and designed with an ordinary pitch of 7 degrees, therefore 20 psf was used.

3.1.4 Snow Loads

To determine the snow load, equation 7.4-1 for sloped roofs of the ASCE standard manual was used.

$$p_s=p_f*Cs$$

To find p_s, p_f was first calculated using the following procedure:

1) Found the exposure factor, C_e. From ASCE section 26.7, the terrain category at Logan Airport is C and fully exposed, so C_e is 0.9.
2) We found the thermal factor, C_t= 1.0, and ground snow load, p_g, which from MSBC section 16, ground snow load for Boston is 40 psf.

3) Using equation 7.3-1 $p_f = 0.7C_eC_tp_f$, $p_f = 0.7*0.9*1.0*40$ psf = 25.2 psf. To find p_s for a sloped roof, Cs was determined with graph 7.4-1, yielding a value of 1.0. Therefore, p_s was the same as p_f at 25.2 psf.

3.1.5 Wind Load

There are three procedures that can be used to determine the design wind load: simplified, analytical, and wind tunnel procedure. The different procedures are described in more detail in sections 6.4, 6.5, and 6.6 in ASCE 7.

1) Simplified procedure: the basic wind speed, importance factor, exposure category, and height and exposure category are determined to solve for design wind load. For this procedure to be used, the mean roof height must be equal to or less than 60 ft, which would disqualify this procedure being used for Terminal E.
2) Analytical procedure: This process is used for regular shaped buildings that do not respond to crosswind loading, vortex shedding, or wind channeling effects. As Terminal E is not excluded by these provisions, this procedure can be used.
3) Wind-tunnel Procedure: This process uses a wind tunnel to analyze the forces and pressures acting on a structure, making it an infeasible option for our team.

The simplified and wind-tunnel procedure would not apply to Terminal E, so our team determined wind loads using the analytical procedure. The design procedure for this process is in accordance with ASCE 7 section 6.5.3.

1) Basic wind speed determined from MBC; Wind directionality Factor determined from table 26.5 1D for risk category III building. V=125 mph. Kd=0.85

2) Surface Roughness and Exposure Categories: Logan airport could be considered either class B or class C surface roughness. In the east-west direction it is flat ground with some residential houses, therefore exposure category C would be appropriate. In the north-south direction lies the highly urban city of Boston, which would result in a surface roughness category B. For the most conservative estimate, the surface roughness category C was used, and therefore the exposure category was C.
3) Topographic factor K_{zt}: Logan airport does not lie near a ridge or a hill, therefore K_{zt}=1.0
4) Gust effect factor G or G_f: assuming a rigid structure, G=0.836
5) Internal pressure coefficient C_p or GC_{pf} or force coefficients Cf: From table 26.13-1 GC_{pi}= 0.18 or -0.18
6) Find the following values from table 26.11-1 for exposure category C, α=9.8 and z_g= 2460 ft
7) Wall Pressure Coefficients: windward wall coefficient C_p= 0.80, and leeward wall coefficient C_p= -0.50
8) Height-evaluated Velocity Press Q_z: from equation 26.10-1

$$Q_z=0.00256K_zK_{zt}K_eV^2$$

In order to find Q_z, K_z must be found, which differs with height. Table 3.3 shows the correct K_z values for different heights. Using the values in this table, table 3.2 and equation 26.10-1, Q_z was calculated for heights up to 160 ft.

Table 3.2 ASCE Table 26.10-1

Height above Ground Level, z or h		Exposure		
ft	m	B	C	D
0–15	0–4.6	0.57 (0.70)*	0.85	1.03
20	6.1	0.62 (0.70)*	0.90	1.08
25	7.6	0.66 (0.70)*	0.94	1.12
30	9.1	0.70	0.98	1.16
40	12.2	0.74	1.04	1.22
50	15.2	0.79	1.09	1.27
60	18.3	0.83	1.13	1.31
70	21.3	0.86	1.17	1.34
80	24.4	0.90	1.21	1.38
90	27.4	0.92	1.24	1.40
100	30.5	0.95	1.26	1.43
120	36.6	1.00	1.31	1.48
140	42.7	1.04	1.34	1.52
160	48.8	1.08	1.39	1.55
180	54.9	1.11	1.41	1.58
200	61.0	1.14	1.44	1.61
250	76.2	1.21	1.51	1.68
300	91.4	1.27	1.57	1.73
350	106.7	1.33	1.62	1.78
400	121.9	1.38	1.66	1.82
450	137.2	1.42	1.70	1.86
500	152.4	1.46	1.74	1.89

*Use 0.70 in Chapter 28, Exposure B, when $z < 30$ ft (9.1 m).

Table 3.3 Qz Values for Different Heights

Height, z	K_z	q_z
0 ft	0.85	31.2 psf
16 ft	0.86	31.6 psf
32 ft	1.00	36.6 psf
48 ft	1.08	39.9 psf
64 ft	1.15	42.4 psf
80 ft	1.21	44.4 psf
96 ft	1.25	46.1 psf
112 ft	1.30	47.7 psf
128 ft	1.33	49.0 psf
144 ft	1.37	50.3 psf
160 ft	1.40	51.4 psf

9. Using the Q_z values calculated in the previous step, we found the internal wind pressure on the leeward and windward walls (Table 3.4) at each height using the following equation:

$$F = q_z GCpiC$$

Table 3.4 Windward and Leeward Pressures

Height, z	K_z	q_z	Walls			
			WW	LW	WW + LW	Side
0 ft	0.85	28.9 psf	19.3 psf	-18.0 psf	37.3 psf	-25.2 psf
10 ft	0.85	28.9 psf	19.3 psf		37.3 psf	
20 ft	0.90	30.7 psf	20.5 psf		38.5 psf	
30 ft	0.98	33.4 psf	22.3 psf		40.3 psf	
40 ft	1.04	35.5 psf	23.7 psf		41.7 psf	
50 ft	1.09	37.2 psf	24.9 psf		42.9 psf	
60 ft	1.14	38.6 psf	25.8 psf		43.8 psf	
70 ft	1.17	39.9 psf	26.7 psf		44.7 psf	
80 ft	1.21	41.1 psf	27.5 psf		45.4 psf	
90 ft	1.24	42.1 psf	28.1 psf		46.1 psf	
100 ft	1.27	43.0 psf	28.8 psf		46.8 psf	

3.1.6 Seismic Loads

Seismic activity causes lateral motion that buildings must have the ability to withstand. To determine the loads earthquake activity applies on buildings, the following ASCE-7 procedure was followed.

1) Determined risk-targeted maximum earthquake spectral-response accelerations at short periods. Logan Airport would be considered risk category III because it is a high occupancy gathering space, so important category I=1.25. The values shown in Figure 3.2 were utilized from the ASCE-7 Hazard Tool:

Seismic B - Rock

Site Soil Class:
Results:

PGA_M :	0.15	T_L :	6
S_{MS} :	0.2	S_S :	0.26
S_{M1} :	0.054	S_1 :	0.055
S_{DS} :	0.14	V_{S30} :	1080
S_{D1} :	0.036		

Figure 3.2: Seismic Design Factors

Where:

S_{MS}= Maximum spectral response acceleration parameter at short periods
S_{M1}= Spectral response acceleration parameter at a period of 1 second
S_{DS}= Design spectral response acceleration parameter at short periods
S_{D1}= Design spectral response acceleration parameter at a period of 1 second
T_L= Long-period transition period
S_S= 0.2 second mapped spectral response acceleration value
S_1= 1 second mapped spectral response acceleration value

2) Determined the site coefficients F_a and F_v from ASCE-7 tables 11.4-1 and 11.4-2. F_a= 1.0, and F_v= 1.0

3) Found the following seismic design coefficient and factors from table 12.2-1:

   a. Response Modification Coefficient R= 3.5 for steel and composite concrete ordinary braced frames

   b. Overstrength Factor Ω_0= 2.5

   c. Deflection Amplification Factor C_d= 3

4) Using the seismic design coefficients, calculated remaining seismic response coefficients:

   a. $C_{S(max)}= S_{D1}/(T*(R/I))= 0.022$

   b. $C_{S(min)}= 0.044S_{DS}*I= 0.01$

c. Seismic Response Coefficient $C_{S}= S_{DS}/(R/I)= 0.062$. Because $C_{S(max)} < 0.062$, use $C_s=0.022$

5) Found the following fundamental period coefficients:

a. Period Coefficient $C_T= 0.02$ from table 12.8-2

b. Period Exponent x= 0.75 from table 12.8-2

c. Approximate Period $T_a= C_T*height^x= 0.589$

d. Upper Limit Coefficient $C_u= 1.7$ from table 12.8-1

e. Period max $T_{max}= C_u*T_a= 1.002$

f. Fundamental Period $T= T_a= 0.589$ because $T_{a} < T_{ma}$

6) Calculated the seismic base shear using equations 12.8-1

a. $V= C_S*W= 1745.95$

7) Determined the structure weight distribution using the effective seismic weight W. This includes the dead load and other loads as specified in section 12.7-2. After, we found the total weight of the building, which is the sum of all the floors, as shown in Table 3.5.

Table 3.5 Calculated Weights

Seismic Level x	Height, hx (ft.)	Weight, Wx (kips)
4	83.000	1345.00
3	60.000	25740.00
2	43.000	25740.00
1	28.000	25740.00

Total Weight, $W = \Sigma W_x =$ 78565.00 kips

8) Finally, we found the seismic shear vertical distribution, as seen in Table 3.6. Since 0.5 < T < 2.5 sec, the distribution exponent k will be an interpolation between k=1 and k=2 (per section 12.8.3). Therefore, k= 1.04.

   a. $C_{vx}= W_x$*height ^{k} (equation 12.8-11)

   b. Lateral Force $F_x= C_{vx}$*V (equation 12.8-11)

Table 3.6 Seismic Lateral Forces

Seismic Level x	Weight, Wx (kips)	hx^k (ft.)	Wx*h^k (ft-kips)	Cvx (%)	Shear, Fx (kips)	Σ Story Shears
4	1345.00	101.096	135973.5	0.033	57.41	57.41
3	25740.00	72.030	1854059.2	0.448	782.77	840.17
2	25740.00	50.860	1309131.3	0.317	552.70	1392.87
1	25740.00	32.490	836290.6	0.202	353.07	1745.95
Σ =	78565.00		4135454.7	1.000	1745.95	

3.2 Structural Member Design

3.2.1 Structural Design

The structural design provides required sizes and information for floors, roof, beams, girders, columns, and material quality to ensure that the building will be structurally adequate to carry the design loads and withstand environmental conditions.

As was mentioned before, since the airport terminal has long spans, the structure will need to be designed with steel framing and composite slabs. The use of this material will be beneficial to the structure due to its technical properties. Steel has high strength and is lightweight, which is good for our long spans. The moments of inertia of a steel structure can be accurately calculated since it follows Hooke's law up to high stresses. Steel also has a great speed of erection. Steel frames that are correctly maintained can last indefinitely. A steel member loaded until it has large deformations will still be able to withstand large forces due to its ductility and strength. A steel member can also easily accommodate modifications and have connections attached to it. Due to all these characteristics, steel is one of the most cost-efficient ways to raise a structure and reduces life-cycle costs. (McCormac, 2008, pp. 1-3).

Having decided on steel as the best material for the structure, it is critical to select the best steel member sections and sizes to provide crucial structural support.

3.2.2 Beams

Beams are the members that support transverse loads. Joist beams are the spaced beams that support the roofs and floors of a building. Spandrel beams support the exterior walls. For our design, we used wide-flange beams (W beams), which are shaped like an I as shown in Figure 3.3 (*S*tructural Steel Dimensioning Tool). W-beams were used because they are the most economical for long spans and can facilitate connections. Also, the flanges of these beams are designed to resist bend stress, while the web resists shear. Because of their wider profile, they are efficient at dispersing weight loads over a larger area, which was necessary in our terminal design (McCormac, 2008, p. 236).

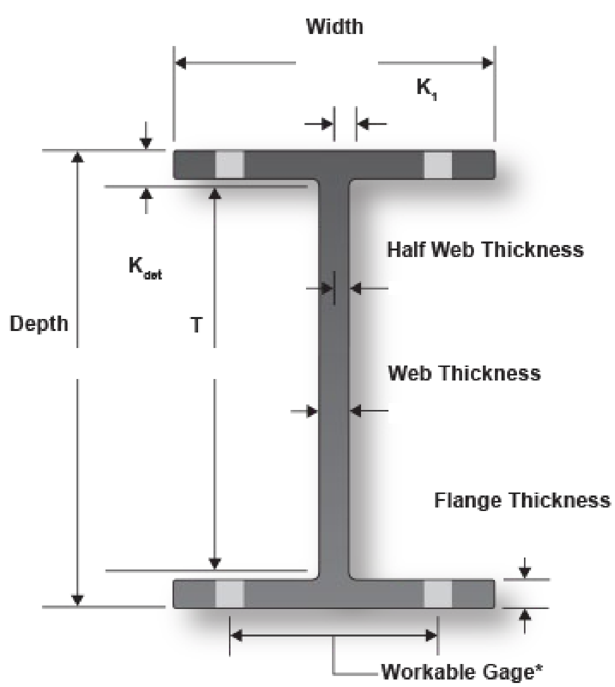

Figure 3.3: Diagram of a Wide-flange Beam

This type of beam has a wide range of sizes, so to choose the correct beam we used the 15th edition of the Steel Construction Manual, published by the American Institute of Steel Construction (AISC). This manual provides detailed information for steel shapes. To get the correct size of the beam, we needed to know the loading that the beams will withstand when placed on the roof and on other levels. Since the dead load and live load for floor and roof are different, the procedure to get the right size of the beam were a little bit different.

For the roof, we made use of the data in Table 3.7:

Table 3.7: Data to calculate the Beam Sizes for the Roof

Length of the beam (L)	Varies
Tributary width	Varies
Wind load	13 psf * Tributary width
Snow load	25.2 osf * Tributary width
Dead Load (D) of roofing material	10 psf * Tributary width
Roof live load (Lr)	20 psf * Tributary width
Fy	50 ksi (AISC)

After obtaining all these values we proceeded to determine a suitable beam size using the following procedure:

1) We checked the different load combinations to find the governing load combination per unit length (W_u)
2) Once Wu was determined, we proceeded to calculate the max moment of the beam:
    a) $M_u = (W_u * L^2)/8$

3) The next step was to get the plastic section modulus: $Z_x = M_u/(\Phi * F_y)$, where $\Phi=0.9$ per AISC

4) We compared our calculated plastic section modulus with Z_x from AISC Table 3-2, then proceeded to follow the guidelines from Chapter 16 in the IBC codes for the deflection limits. To ensure that the size chosen is code compliant, the actual deflection must be less than the deflection limit.

    a) Actual deflection = $(5W_uL^4)/(384EI)$

        i. Where E is Elastic modulus (29000 ksi) and I is moment of inertia

        ii. The limits of deflection are shown in Table 3.8 from chapter 16 in IBC.

Table 3.8: IBC Deflection Limits

CONSTRUCTION	L or L_r	S or W^f	$D + L^{d,g}$
Roof members:[e]			
Supporting plaster or stucco ceiling	l/360	l/360	l/240
Supporting nonplaster ceiling	l/240	l/240	l/180
Not supporting ceiling	l/180	l/180	l/120
Floor members	l/360	—	l/240

For the floor slab, we used the same data previously mentioned for the roof but with some exceptions. Wind load and snow load was not a factor, and a composite slab dead load of 39.1 psf (Table 2.1) and live load of 110 psf was used. The same procedure was followed. Shear is only a concern for girders and not beams, so shear was not checked (McCormac, 2008, p. 236). We created a comprehensive spreadsheet in Excel to be able to record all the data and make the calculations easier. An example of this spreadsheet can be found in Appendix E.

3.2.3 Girders

Another fundamental structural member in a building is a girder. A girder is a supporting large beam, and a structure's primary horizontal support for smaller beams. The process to get the correct size of a girder is similar to that of beams, but the dead load and live load are different. To record all the data and size we also created a comprehensive spreadsheet in Excel which can be found in Appendix G.

For the roof, we made use of the data in Table 3.9:

Table 3.9: Data to calculate Girder Sizes for the Roof

Length of the beam (L)	Varies
Tributary width	Varies
Wind load	13 psf * Tributary width
Snow load	25.2 psf * Tributary width
Dead Load (D) of roofing material	10 psf * Tributary width
Roof live load (Lr)	20 psf * Tributary width
Fy	50 ksi (AISC)

After obtaining all these values we proceeded to determine a suitable girder size using the following procedure:

1) We checked the different load combinations to find the governing load combination per unit length (W_u).
2) Found the max moment of the beam: $M_u = (W_u * L^2)/8$
3) The next step was to get the plastic section modulus: $Z_x = M_u/(\Phi * F_y)$, where $\Phi = 0.9$

4) We compared our calculated plastic section modulus with Z_x from AISC Table 3-2. Once girder size was chosen, we proceeded to follow the guidelines from Chapter 16 in the IBC codes for deflection limits. To ensure that the size chosen is safe to use, the actual deflection must be less than the deflection limit.

For the floor slab, we made use of the same information previously mentioned for the roof but with some exceptions. Wind load and snow load was not a factor, and we used a composite slab dead load of 39.1 psf (Table 2.1) and live load of 110 psf. The same procedure was followed.

Although we knew that there would only be long spans for the girders, it was still important to check the shear, to make sure that the sizes chosen were correct for the applied forces. To calculate shear (for roof and floor slabs) we used the data from Table 1-1 W- Shapes (dimensions) according to the selected girder size. We used:

A= Area
D= depth
t_w= web thickness
Web Area A_w=d*t_w
k_{des}= distance

To calculate height of the W-flange (or the depth), the following equation was used:

$$h = d - 2k_{des}$$

If h/t_w, the width-to-thickness ratio of the web. is less than $2.24\,(E/F_y)^{1/2}$ where F_y=50 ksi and E= 29000ksi, we use:

- C_v=1.0
- ϕ_v=1.0

Almost all current W-shapes fall into this class. The exceptions are listed in AISC Specification manual with their respected equations for example:

- If the shape falls in the exception, we use:
  - $h/t_w < 1.10\ ((k_v\ E)/F_y)^{1/2}$ where $k_v = 5.34$ (web plate buckling coefficient) for webs without transverse stiffeners where C_v is also 1.0
  - ϕ_v=0.9

Once we got all the values, we proceeded to check shear:

1) We first checked the nominal shear strength.
   a. $V_n = 0.6*F_y*A_w*C_v$
2) Then factored shear force V_u
   a. $V_u = W_u * L/2$
3) Then we checked if our factored shear force V_u is less than the design shear stress $\phi_v V_n$
   a. If the design shear stress is greater than the factored shear force, we use the chosen girder size.

3.2.4 Columns

Columns are vertical structural components found where an axial force operates parallel to the longitudinal axis and convey forces operating vertically to the foundations and the ground below. They support compressing stress from the roof and floors, and as a result can suffer from buckling. For this we used the K factor procedure which is a method of making simple solutions for complicated frame buckling situations. K, or the effective length factor, must be multiplied by the length of the column to find its effective length which is the distance between points of zero moment in the column, that is, the distance between its inflection points. The AISC Specification (C1-3a) states that K=1.0 should be used for columns in frames with sidesway inhibited, unless an analysis shows that a smaller value can be used. This is often quite conservative, and an analysis made as described herein may result in some savings (McCormac, 2008). In the case of our terminal, sidesway was inhibited and we wanted to go with a conservative number, so K=1.0.

To determine column section, we used the following procedure:

1) Calculated the tributary area the column supports.
2) Calculated the normal strength: P_u= 1.2 (Composite slab dead load+ Steel Dead load) + 1.6 (Live Load)* Tributary width
3) Assumed the effective slenderness ratio KL/r is 50 where K is the effective length coefficient, L is unbraced length and r is radius of gyration.
4) Checked for the design stress $\varphi_c F_{cr}$ in Table 4-14 from AISC Manual.
5) Calculated the area required= P_u/ ($\varphi_c F_{cr}$)
    a. We used Table 4-1a to check area given (A_g). We selected a higher number of our minimum Area required and obtained the radius in the y direction (r_y).
6) Recalculated to the effective slenderness ratio KL/r where K= 1.0, L is the length of the column, and r is now the radius in y direction.
7) Got the new design stress: Based on the new design stress, we proceeded to get allowable strength $\varphi_c P_n$

$$\varphi_c P_n = \varphi_c F_{cr} * Ag$$

8) If this value was greater than normal strength, we proceeded to use the chosen size. If not, we proceeded to go back to Table 4-1a and pick a larger column based on a higher A_g and r_y. We proceeded to do the procedure again to verify the allowable strength was greater than normal strength.

3.3 Foundations

3.3.1 Foundation Design

When considering the foundation requirements for the terminal design, the soil conditions that the terminal will be built on must first be understood. The soil holds the loads from the superstructure so that the terminal will be supported by the ground below. The calculations used by foundations will determine the type of support necessary for the building to be structurally sound, under the condition that the building's loading does not exceed the bearing capacity. This then helped determine the type of support needed for the building to stand and not collapse or sink into the soil if it exceeds the bearing capacity of the soil or the allowable settlement. The foundation's geotechnical report was sent to us by Massport. This report provided us with soil characteristics that were used to dictate what our foundation layout would look like. Based on the recommendations from the geotechnical report as shown in Appendix O and example of the report data, this determined whether we would use a deep or shallow foundation for the building.

After we had determined the soil conditions, we created the bearing capacity design and parameters for the load that will be applied to the soil. The type of foundations required for additional support would depend on the load applied to the soil. *Foundations Design 3nd Ed.* by Coduto was referenced for all design steps of the footings, which was the selected foundation for the building. For comparison, Massport used a deep foundations configuration for terminal E, as the building load is too much for the topsoil to handle and would need the additional support underneath. Figure 3.4 shows an example sketch of pile caps and columns, which will be added to the terminal.

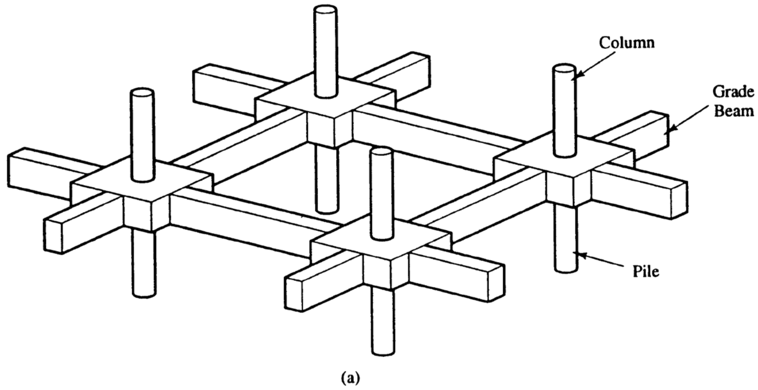

Figure 3.4: Design of Piles and Columns

The first step will be determining whether we will be using a shallow or deep foundation. To determine the type of foundation that was needed, we took variables from the geotechnical report of the soil to calculate the 'bearing capacity and column load' that will be applied to the soil. The method we chose for our foundations bearing capacity will be using Vesic's equation:

$$q_{ult} = c' N_c s_c d_c i_c g_c + \sigma'_{zD} N_q s_q d_q i_q b_q g_q + 0.5\gamma' B N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$$

Where:

c'= Soil Cohesion

N_c= Bearing capacity factor - cohesion

q=D_f*$\boldsymbol{\gamma}$ - Surcharge

Nq= Bearing capacity factor - Surcharge

$\boldsymbol{\gamma}$= Soil Unit Weight

N$\boldsymbol{\gamma}$= Bearing Capacity Factor - Soil

B= Footing Width

We chose this formula because it uses specific parameters based on the footing such as shape, depth and inclination factors, all of which are considered for bearing capacity.

The shape factors depend on the dimensions for the foundation. The width, length, and height are used in the three equations below:

$$S_c = 1 + (\frac{B}{L})(\frac{N_q}{N_c})$$

$$S_q = 1 + (\frac{B}{L})tan\text{Ø}'$$

$$S_{\gamma} = 1 - 0.4(\frac{B}{L})$$

To find depth factor:

$$d_c = 1 + 0.4k$$

$$d_q = 1 + 2 * k * \tan(\phi') * (1 - \sin(\phi'))^2$$

$$d_{\gamma} = 1$$

K varied depending on if D/B ≤ 1: K= D/B, D/B > 1 K = tan-1(D/B)

To find Inclination factor

$$i_c = 1 - \frac{mV}{Ac'N_c} \geq 0$$

$$i_q = \left[1 - \frac{V}{P + \frac{Ac'}{tan\phi'}}\right]^m \geq 0$$

$$i_{\gamma} = \left[1 - \frac{V}{P + \frac{Ac'}{tan\phi'}}\right]^{m+1} \geq 0$$

$$m = \frac{2 + \frac{B}{L}}{1 + \frac{B}{L}}$$

The next equation determined the allowable bearing capacity:

$$q_a = \frac{q_{ult}}{F}$$

Where:

q_{ult}= bearing capacity

F = Factor of safety based on the category. (Using category B (F=2.5))

3.3.2 IBC Standards

The factors and variables collected for the equations were found in the IBC section 1806.2 for presumptive load-bearing values. We specified the minimum compressive strength f'c as 4000 psi. Table 1809.7 describes the footing that supports walls of light frame construction, to determine the thickness of the footing based on the number of floors. In this case the footing base had a width of 18 in and thickness of 8 in. Table 1810.3.2.6 specifies the allowable stresses for materials used in deep foundation. Because we used concrete for the base of the square footing, we had a maximum allowable stress of 0.4*f'_c or 1600 psi.

3.3.3 Deep Foundations

For deep foundations, additional equations were used. For a deep foundation, we used pile caps to distribute the load of the building, as described in IBC section 1810.3.11.
These pile caps needed to factor in upward and downward load capacities as shown in the following equations:

$$\left(P_{upward}\right)_a = \frac{W_f + \sum f_s A_s}{F}$$

Where:

W_f = effective weight of foundation

F_s = friction factor of soil

A_s = surface area of contacted soil

F= factor of safety

Rankine's formula was then used to determine the length of the pile:

$$h = \frac{p}{\gamma}\left(\frac{1-sin\varphi}{1+sin\varphi}\right)^2 \varphi$$

For clay (undrained conditions assumed):

$$q_t' = N_c * s_u$$

$$N_c = \ 6.5@s_u = 500\ psf; 8\ @\ s_u = 1000\ psf; 9@s_u \geq 2000\ psf$$

Side friction determined the stress from the soil compactness on the pile driven into the ground to determine how strong it will be:

$$f_n = \sigma_x' tan\varphi_f$$

Where:

$\sigma_x{}'$= horizontal effective stress

ϕ_f = soil- pile interface friction angle (.9)

β Method for silts and clay will assume that the shaft resistance of the pile is a function of the effective stress of the soil along the pile shaft.

$$\beta = 0.25\ to\ 0.35$$

The last equations determined the settlement to see how much the building will sink into the ground, to give the support needed.

$$W_s = (Q_P + \alpha_s Q_s) * \left(\frac{L}{AE_p}\right)$$

Where:

Qp = point load of the pile tip

Qs = Shaft friction load

α= .67

L = length of pile

A = area of cross section

Ep modulus of elasticity

$$W_{pp} = \frac{C_p * Q_p}{Bq_o}$$

Cp = empirical coefficient – 0.03

B = pile diameter

qo = bearing capacity

$$W_{ps} = \frac{C_s Q_s}{Dq_o}$$

D = embedded length

$$W_o = W_s + W_{pp} + W_{ps}$$

Wo = total settlement depth

3.4 Connections

Connections are used to join different members of the beams, girders, and columns of the structure. For steel, there are several methods of connecting members that can be used: riveted, welded, or bolted. Riveted connections were used extensively decades ago, but because of their cost and need for high-skill workers, are not common anymore. Nowadays, bolted and welded connections are most common. For this project, our team used bolted connections because they are faster to erect, require less skilled-labor, and are cheaper.

Common bolted connections include single/double angle, single-plate, and end-plate shear connections. Figure 3.5 shows an example of a single web and double web angle connection. Our team used single angle connections because they are cost-effective and strong.

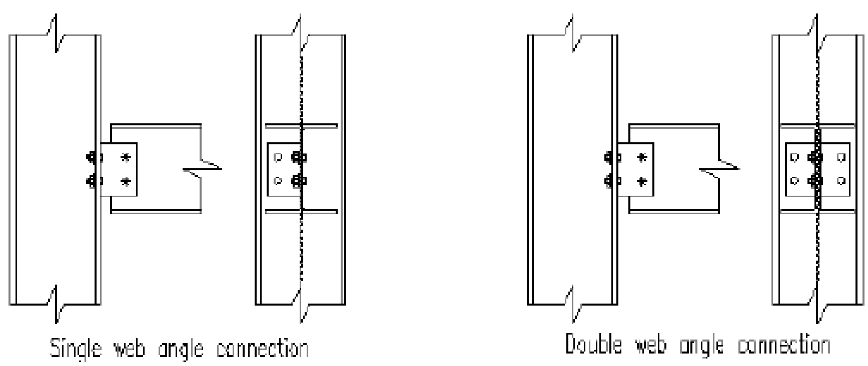

Figure 3.5: Single Web Angle and Double Web Angle Connections Sketch

The following procedure was used for designing single-angle bolted connections:

1) Found the design loads R_{DL} and R_{LL} using the loading and area on that specific bay.
2) Calculated the number of A325-N bolts required for the connection. Number of bolts $n = \frac{V_u}{\emptyset R_n}$, where V_u is the reaction found in step 1.
   a) We found the factored reaction using $\emptyset R_n = \emptyset F_{nv} A_b$, where $\emptyset = 0.75$
      i) F_{nv}=54 ksi for A325-N bolts
      ii) A_b= area of the bolt

3) Determined the bearing and tear out strength at each bolt hole, with the smallest of the two being the governing value.

   a) Tearout = $1.2L_c tF_u$, where L_c= distance in loading direction from bolt hole to bolt hole

   b) Bearing = $2.4d_b tF_u$, where d_b= diameter of the bolt

4) Calculated the bearing or tear out capacity at each hole, depending on what governed in last step.

   a) $ØR_n = n\, Ø1.2L_c tF_u$ if tear out is governing

   b) $ØR_n = n\, Ø2.4d_b tF_u$ if bearing is governing

5) Using ASCE table J3.4, found the minimum edge distance from center of standard hole to edge of connected part.

6) Used ASCE table 1-7A to find angle legs.

   a) For 2 in gage distance, <3½ x 3½ x t

To find the angle leg thickness, three different limit states must be checked: bolt bearing/tear out, shear rupture, and shear yield. The largest angle leg thickness determined from these three limit state checks will be the angle thickness.

1) Bolt bearing/tear out on angle leg

   a. Found the capacity for load transfer in the vicinity of each bolt $ØR_n = Ø(1.2L_c tF_u) < Ø(2.4d_b tF_u$

   b. Determined the clear distances L_{c1} and L_{c2}, and used these to find out which sets of bolts are governed by tear out or bearing.

   c. Calculated the total capacity of all bolt holes, which can be compared to the design load to find minimum angle thickness.

2) Angle Shear Rupture

a. Found the factored reaction $\emptyset R_n = \emptyset(0.6F_u)(L - nd_e)$

i. $L - nd_e$ is the net distance on shear plane thru angle leg

b. Used the factored reaction to compare to the design load to determine thickness.

i. $\emptyset R_n t \geq R_{total}$

3) Angle Shear Yield

a. Found shear yield $\emptyset R_n = \emptyset(0.6F_y)Lt$, where Lt= gross area through shear plane

b. Compared shear yield to design load to determine thickness.

i. $\emptyset R_n t \geq R_{total}$

3.5 Software

Considering the scope of this project, different types of software were critical in determining the supporting loads and conditions used in the construction of the terminal. While calculations can be done entirely by hand, this is quite tedious. Software and technology allowed us to alter materials and loading conditions to determine the effects on the structure.

3.5.1 Revit

One of the more popular 3D modeling software, Revit, was developed in 2000 and is commonly used by structural and architectural engineers to model and test a building's structure and materials. Massport used Revit to model the annex for Terminal E, and we were fortunate enough to obtain that model for analysis with our project. Our own model was created in Revit, using our own materials and structural choices while referencing the model provided by Massport. The model was constructed from scratch over the course of two months and provided an overview of the structural plans for Levels 1 through 4, along with floor plans for each level. It provided a visual representation of our project, along with the sizes and lengths of the structural elements we chose for this project, that could be referenced throughout the course of our project.

3.5.2 RISA

Like Revit, RISA is also a modeling software that is used for modeling and testing a building's materials and structural components. The RISA was constructed using nodes, beams, and plates. By modelling sections of the main rectangle, pier, and renovation crescent, we could have a good grasp on how seismic loads would affect our design. The model reached a maximum height of 79 ft on the fourth floor, and 62 ft on the third floor. Basic loading conditions were then tested upon the model. Line loads were placed upon the beams, and the analysis determined whether the beams passed the Unity Check, comparing the capacity of the beams to the demand from the loads. The results are displayed on RISA in a color-coded system, that codes the beams, girders, and columns to the various degrees of passing. The color-coded system is shown below in Figure 3.6. Based on this result, we determined if the beams performed as the hand-calculations predicted. If we found that members are failing the code check, we can either increase the member size or place lateral bracing in the model to support the member. Lateral bracing is preferred, because increasing the member size excessively can be costly. If lateral bracing suffices, then the member size will not be increased; however, there may be scenarios where the member sizes will need to be increased to improve stability.

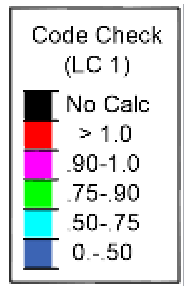

Figure 3.6: RISA's code check parameters

3.5.3 Excel

While the models provided a good resource for determining loading conditions and materials, calculations were still confirmed by our team. With the scale of the terminal, automating calculations made the procedure of choosing structural member sizes much quicker. This is where Microsoft Excel was utilized. Using Excel allowed us to compute calculations with less human error and helped us keep track of results on such a large project. An example of one of our spreadsheets, on Seismic, is shown below in Figure 3.7.

Wind Load	13	psf	585	plf			
Snow load	25.2	psf	1134	plf			
Rain loads							
Dead load (composite deck)			391	plf			
Dead load (beam)			84	plf			
Dead Load (roofing material)			391	plf			
Live load	110	psf	4950	plf			
Live load roof	20	psf	900	plf	0.02	k/ft	
Unfactored LL+DL			5425	plf	5.425	k/ft	
Unfactored LL			4950	plf	4.95	k/ft	
Wu1	665	plf	8.49	k/ft			
Wu2	8490	plf					
Wu3	8490	plf					
Wu4	1832	plf					
Mu	2149031.25	ft-lb	25788375	in-lb			
Z>=	573.075		Fy	50,000	psi	50	ksi
			phi	0.9			
Choose Steel Beam	W40x149		unit weight	84	LB/FT	(from AISC table)	
Zx	598	in^3					
moment of Inertia	9800	in^4					
Elastic modulus	29000	ksi					
Use inertia tables for new Section	W40x167						
New Ix	11600	in^4	Actual deflection LL+DL (with new Ix)	1.487908856	<	2.25	
new Zx (found with same section on Zx ables)			Actual Deflection LL (with new Ix)	1.357631122	<	1.5	

Figure 3.7: Sample Excel Spreadsheet

4 Results and Discussion

4.1 Design Choices

While our project was similar in many aspects to the current Massport project, we wanted to make sure that our design stood out with its own distinct differences. Along with our own structural member layout and section sizes, we chose to alter the design in ways that we thought would improve the overall layout and functionality of Terminal E.

4.1.1 Addition of the Pier

When we first considered the project, it was important to make sure that the overall shape of the building would be modified to better suit the needs of Logan Airport. The different terminal shapes shown in Figure 2.8 were referenced to decide what shape would work best. Since Logan already has an Open Structure, that was immediately eliminated. Hammerhead, Y and diamond shapes would not fit well with the existing structure of the building, and these were eliminated. Satellite was a possible option, but Logan has runway space across from the terminal that cannot be compromised. This left Hybrid, I, and Pier, which are quite similar. Because of Terminal C, as well as the roads and other infrastructure directly behind the terminal, we could not include any gates on the “backside” of the terminal, due to the parking and infrastructure already in place (Figure 4.1). Thus, we decided to place a Pier formation at a 90° angle to the current renovation. With the open space on both sides of the Pier for gates, we felt this was the best option.

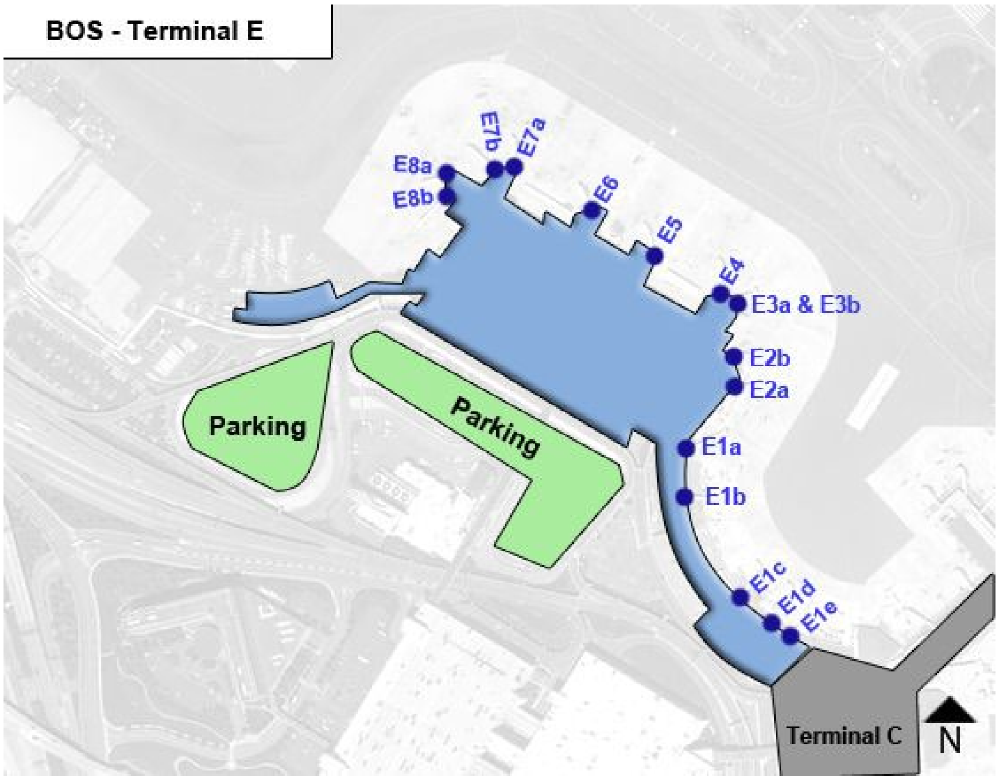

Figure 4.1: Existing Gate Overview of Terminal E

4.1.2 Number of Gates

The original plans from Massport for their project were to include 7 additional gates in the renovation. However, due to some constraints, the project was reduced to 4 additional gates, along the crescent. Our team was determined to match or exceed this number, and thus we designed our terminal to include the 4 additional gates, as well as 4 more. 3 gates were located on the pier, which we designed ourselves, and 1 gate was added to the crescent for a total of 8 gates. A visual of this is shown in Figure 4.2.

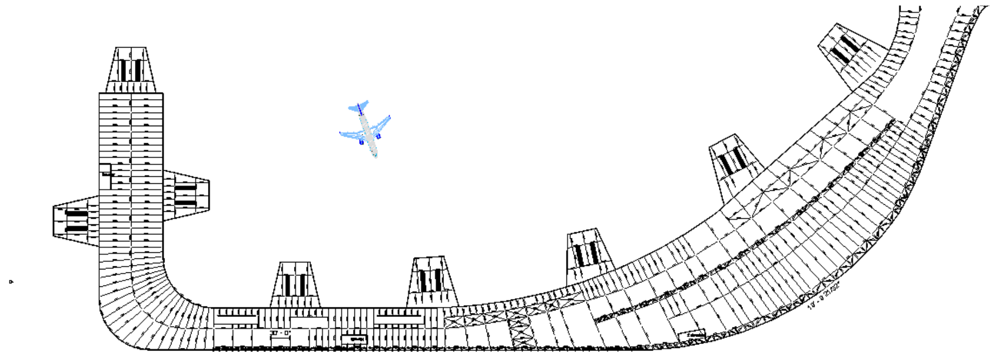

Figure 4.2: Gate Orientation of Crescent and Pier

4.1.3 Gate Spacing

With the Pier shape selected, we then proceeded to determine how many gates could be added. From Table 2.3 in FAA Design Codes, a space of 45 ft is needed from the end of the renovation section, as well as a minimum space of 25 ft between wingtips from gate to gate. Since planes have different wingspans, the spacing for the gates will determine the maximum class of aircraft that can use said gate. For the gates along the pier, the appropriate restricting dimensions were calculated by hand, and used to determine the placement of the gates. Along the outer edge of the pier the length is 400 ft, and along the inner edge the length is 262 ft. We started by placing the gate on the inner pier. From Table 2.3, we determined that in order to accommodate Group V aircraft, we would need spacing of 107 ft from the middle of the gate on either side, with a 45 ft clearance from the end of the terminal. Since these restrictions added up to 259 ft, and the length of the inner edge of the pier was 262 ft, Group V aircraft would have enough space to use this gate within the appropriate FAA regulations. After completing the rest of the calculations for the other two gates on the pier, we determined that the topmost gate would be able to include Group IV aircraft, and the innermost gate would be able to include Group IV aircraft. Since a gate was added along the crescent as well, there were now a total of 5 gates spanning this distance. After some calculations, we determined that we could space them appropriately, so that each would be able to accommodate Group V aircraft.

4.1.4 Gate Design

When structurally designing the placements of beams and girders, we had to account for the other interior elements of the gates. To board their planes, passengers must descend a level to the gate. This means that each gate must fit one or two flights of stairs and one elevator. To work around these elements, our team decided to lay the girders east and west. These girders support the beams running north and south that are placed at relatively equal intervals around the placement of stairs and elevators. A relatively equal beam spacing allowed us to keep more uniform beam and girder sizes, preventing any one member from being a considerably larger section size. Similar to Massport's design, we made two different gate sizes: large and small. Figures 4.3 and 4.4 give a visual example of our designed gates, with beam sizes included.

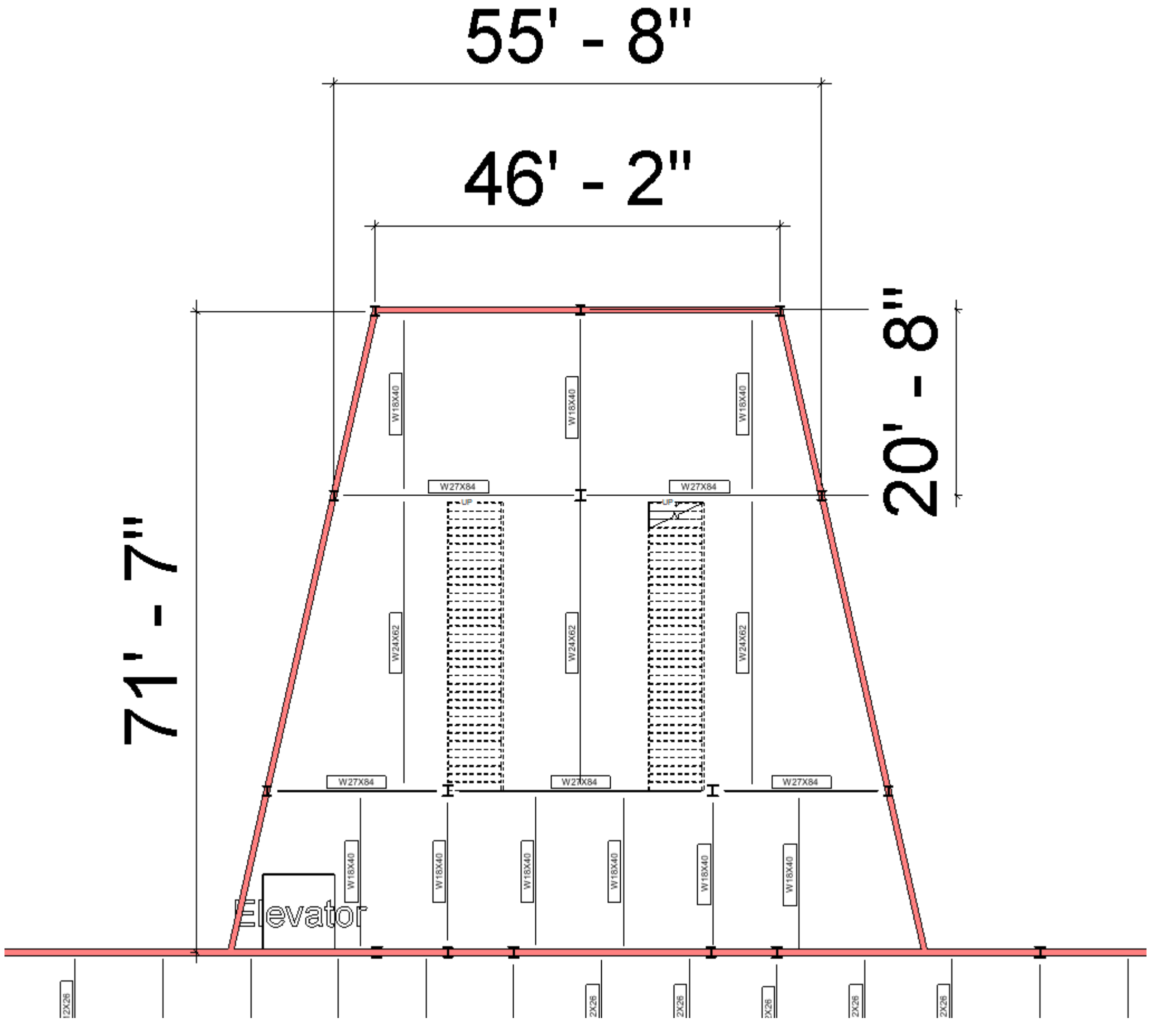

Figure 4.3: Structural Design of Larger Gate

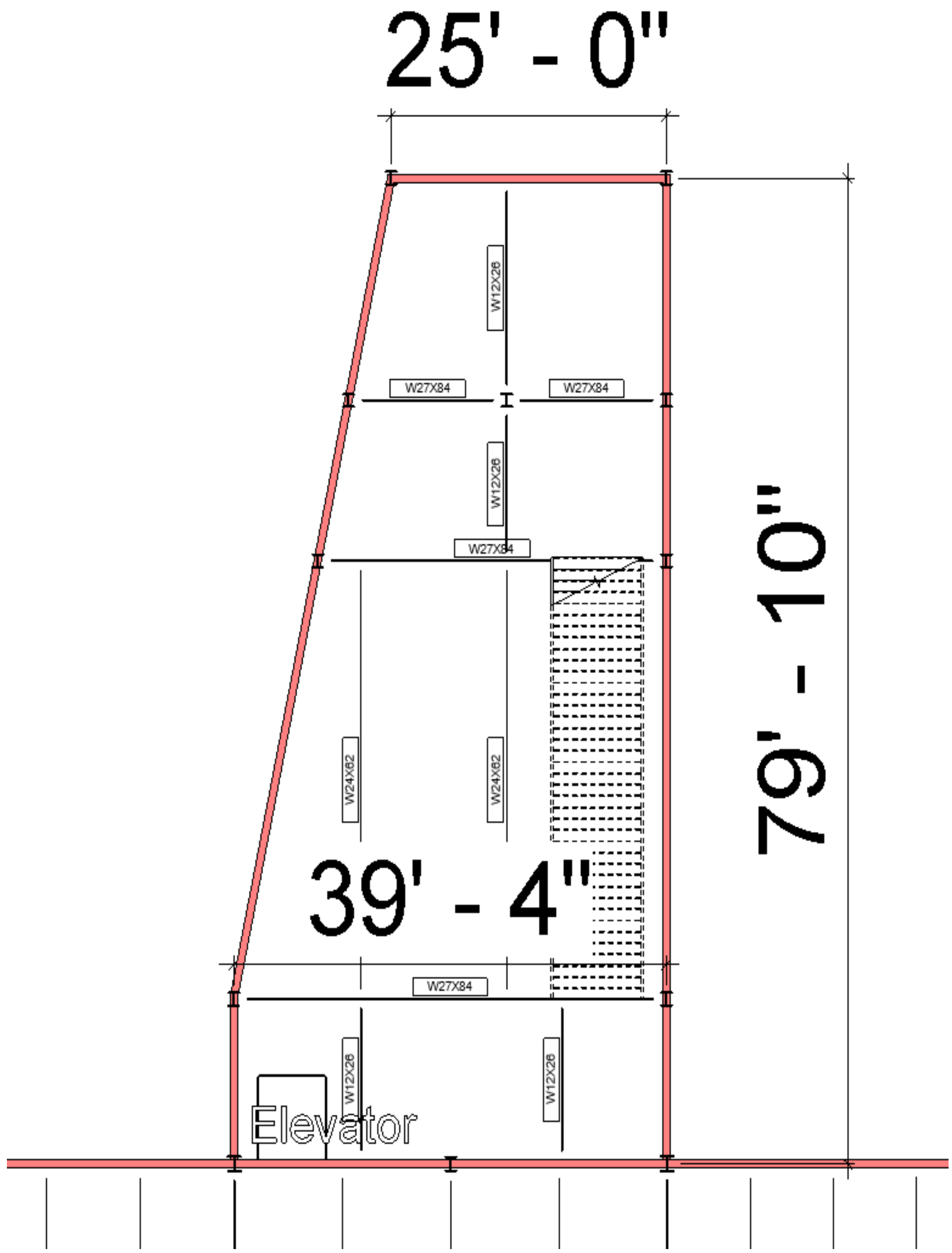

Figure 4.4: Structural Design of Smaller Gate

4.1.5 Roof

The inspiration of our roof design came from the nearby Atlantic Ocean. Our team wanted to emulate a wave-shaped design, so we designed the roof to have two different, downward sloping levels. The design shown in Appendix C has a simplified design of a non-curved roof with only 2 sections dividing the fourth level and third level. As compared to the Terminal E's current design with three levels, the two levels on our terminal makes construction easier and less costly, while keeping an eye-catching design. This also gave a structural advantage as the fourth floor more support, along with a clear view of the city of Boston.

4.1.6 Second INS Corridor

In addition to all the external elements that were added to the project, we also chose to make an internal change that would improve passenger flow within the main rectangle of the terminal. In the current design of Terminal E, the INS corridor consists of only one opening, creating a potential bottleneck for passengers on the third floor. Having two security corridors, as shown in Figure 4.5 would allow for more volume to flow through and would be especially useful for higher passenger traffic during the holidays. This would also provide the outgoing passengers with less of a walking distance than one opening; the two entrances would divert passengers towards their gates and provide a more direct route for the passengers.

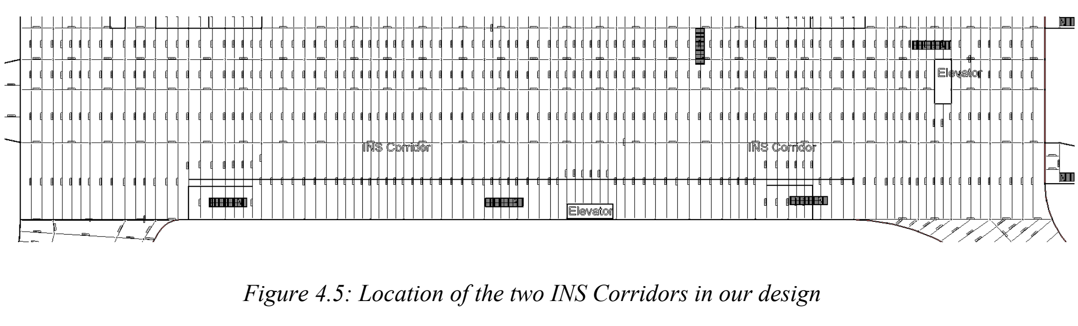

Figure 4.5: Location of the two INS Corridors in our design

4.2 Structural Member Design

4.2.1 Beams

To determine beam section sizes, our team created a spreadsheet to automate the process. With three different sections in the terminal, beam lengths varied considerably and therefore section sizes did too. For example, some of our beams in the renovation crescent were significantly larger than what lengths would be needed in the E-C connector. As we kept consistent beam spacing of 10 ft, this meant the longer beams needed a larger section size. When determining section sizes, the beams would typically meet checks for strength but not deflection. This required us to increase the beam section by several sizes to meet service load deflection checks. In some cases, the beams were 70-80 ft, which resulted in significantly higher deflection. In these situations, we decided to camber the beam 1.75-4.75” (depending on beam length) to decrease the section size and therefore save cost. In the pier where the spans were smaller, a typical beam size was a W30x90 uncambered. In the curved section of the renovation crescent spans were much longer and therefore required camber, the largest of which was W33x118 c=4.25”. The main building had similarly long beam spans at the ticketing hall and gate area to preserve open space. In this section, the smallest section sizes used were W16x26 and the largest was a W33x130 c=4.75”. The smallest part of Terminal E was the E-C Connector which had smaller spans, and a typical size was a W30x90. In Tables 4.1 and 4.2 we can see the results for the beams. The beam calculations can be found in Appendixes D and E.

Table 4.1 Beam Sizes for Second and Third Floors

Beams				
Levels	**Location**	**Length (ft)**	**Smallest Size**	**Largest Size**
Second Floor	**Pier**	20-40	W30X90	
		40-60	W30X90	W33x130(c=1.75”)
		60-80	NA	
	Renovation Crescent	20-40	W12X26	W33X118
		40-60	W30X108 (c=4.25”)	W33X118(c=4.25”)
		60-80	W30X108 (c=4.25”)	
	Main Building	20-40	W16X26	W21X44
		40-60	W24X76 (c=2.25”)	W30X108
		60-80	W33X130 (c=4.75”)	
	E-C Connector	20-40	W21X44	W30X90
		40-60	W30X90	
		60-80	NA	
Third Floor	**Pier**	20-40	W30X90	
		40-60	W30X90	W33x130(c=1.75”)
		60-80	NA	
	Renovation Crescent	20-40	W12X26	W30X90
		40-60	W30X90	W30X108 (c=4.25”)
		60-80	W30X108 (c=4.25”)	W36X135(C=2.75”)
	Main Building	20-40	W16X26	W21X44
		40-60	W24X76 (c=2.25”)	W30X108
		60-80	W33X130 (c=4.75”)	
	E-C Connector	20-40	W21X44	W30X90
		40-60	W30X90	
		60-80	NA	

Table 4.2 Beam Sizes for Fourth Floor and Roof

Beams				
Levels	**Location**	**Length (ft)**	**Smallest Size**	**Largest Size**
Fourth Floor	**Pier**	20-40	NA	
		40-60	NA	
		60-80	NA	
	Renovation Crescent	20-40	W12X26	
		40-60	W30X108	
		60-80	W30x108	
	Main Building	20-40	NA	
		40-60	NA	
		60-80	NA	
	E-C Connector	20-40	NA	
		40-60	NA	
		60-80	NA	
Roof	**Pier**	20-40	W16X31	
		40-60	W21X62	W24X62(c=3.25”)
		60-80	NA	
	Renovation Crescent	20-40	W12X26	W16X31
		40-60	W27X84 (c=2.75”)	
		60-80	NA	
	Main Building	20-40	W14X26	
		40-60	W24X55	
		60-80	W30X108 (c=4.5”)	
	E-C Connector	20-40	W16X26	W24X76
		40-60	W27X84	
		60-80	NA	

4.2.2 Girders

Since the girders are responsible for receiving the load of the beams, they also varied considerably in their lengths. For example, there were sections in both the main building and the renovation crescent that required large girders to respect the open spaces that gave the structure its functionality. To support these 70- to 80-ft-long beams that created large tributary widths, we decided to also camber the girders. One of the largest sizes we had for the girder on the main building was W44x262 c=3.25" on the second and third floor. This was to decrease section size while also making the structure strong enough to support the weight of the beams and respect the open spaces. On the renovation crescent section, it went as high as W36x182 c=2". There were sections where uncambered girders were enough for the structure, as in the E-C connector for both floors and roof, which was W33x118, the biggest size. It should be noted that some members were reviewed again as it was necessary to choose larger members, some of which did not meet the h/t_w limit for shear. The necessary calculations were made to ensure that the deflection and shear limits were met. Once the beams and girders were designed, we proceeded to design the columns. Tables 4.3 and 4.4 show the results for the girders, and Appendixes F and G show the calculations for the girders.

Table 4.3 Girder Sizes for Second and Third Floors

Girders				
Levels	**Location**	**Length (ft)**	**Smallest Size**	**Largest Size**
Second Floor	**Pier**	20-40	W24x84	
		40-60	NA	
		60-80	NA	
	Renovation Crescent	20-40	W24X68	W30X108
		40-60	W30X116 (c=2”)	W40X149(c=1.75”)
		60-80	NA	
	Main Building	20-40	W24X76 (c=1.75”)	W33X118
		40-60	W44X262 (c=3.25”)	
		60-80	NA	
	E-C Connector	20-40	W21X44	W33X118
		40-60	NA	
		60-80	NA	
Third Floor	**Pier**	20-40	W24x84	
		40-60	NA	
		60-80	NA	
	Renovation Crescent	20-40	W24X68	W40X149(c=1.75”)
		40-60	W33X130 (c=2”)	W40X199(c=2.75”)
		60-80	NA	
	Main Building	20-40	W24X76 (c=1.75”)	W33X118
		40-60	W44X262 (c=3.25”)	
		60-80	NA	
	E-C Connector	20-40	W21X44	W33X118
		40-60	NA	
		60-80	NA	

Table 4.4 Girder Sizes for Fourth Floor and Roof

Girders				
Levels	**Location**	**Length (ft)**	**Smallest Size**	**Largest Size**
Fourth Floor	**Pier**	20-40	NA	
		40-60	NA	
		60-80	NA	
	Renovation Crescent	20-40	NA	
		40-60	W36X182(c=2”)	
		60-80	W36X182(c=2”)	
	Main Building	20-40	NA	
		40-60	NA	
		60-80	NA	
	E-C Connector	20-40	NA	
		40-60	NA	
		60-80	NA	
Roof	**Pier**	20-40	W18X35(c=1”)	W24X62
		40-60	NA	
		60-80	NA	
	Renovation Crescent	20-40	W21X55	W27X84
		40-60	W27X84(c=2.75”)	
		60-80	NA	
	Main Building	20-40	W24X62 (c=2”)	W30X90 (c=2.25”)
		40-60	W36X160 (c=3.5”)	
		60-80	NA	
	E-C Connector	20-40	W12X26	W21X44
		40-60	W30X99	
		60-80	NA	

4.2.3 Columns

With the columns supporting the loads from both the beams and the girders, they were designed to be much larger in thickness than the beams and girders. While there was some variety within the column sizes, most of the sizes remained consistent from floor to floor. The largest column size overall was the W14x370, with the smallest size being a W10x49. The ranges of the columns, as well as the location that they are in, can be seen in Table 4.5 below, and the calculations for the columns are shown in Appendixes H and I.

Table 4.5 Determined Column Sizes

Floor	Location	Column Size
1	E/C Connector	W14x176
		W12x136
		W14x109
		W14x90
		W12x72
		W10x49
	Rectangle	W14x176
		W12x136
		W14x109
		W14x90
		W12x72
		W10x49
	Renovation	W14x370
		W12x136
		W14x145
		W14x109
		W14x90
		W10x49
	Pier	W12x136
		W14x145
		W14x109
		W10x49

Floor	Location	Column Size
2	E/C Connector	W14x176
		W12x136
		W14x109
		W14x90
		W12x72
		W10x49
	Rectangle	W14x176
		W12x136
		W14x109
		W14x90
		W12x72
		W10x49
	Renovation	W14x370
		W12x136
		W14x145
		W14x109
		W14x90
		W10x49
	Pier	W12x136
		W14x145
		W14x109
		W10x49

Floor	Location	Column Size
3	E/C Connector	W14x176
		W12x136
		W14x109
		W14x90
		W12x72
		W10x49
	Rectangle	W14x176
		W12x136
		W14x109
		W14x90
		W12x72
		W10x49
	Renovation	W14x370
		W12x136
		W14x145
		W14x109
		W14x90
		W10x49
	Pier	W12x136
		W14x145
		W14x109
		W10x49

Floor	Location	Column Size
4	Renovation	W14x370
		W14x109
		W10x49

4.2.4 Connections

When designing single angle connections, important factors to consider were the length and section sizes of the structural members making up the connection. Larger beam and girder sizes in the renovation crescent and main building (where our spans were longest) would require a higher number of bolts. Another factor was bolt type and bolt size. Our team decided to design our connections with A325N bolts because they are cheaper and can be galvanized against corrosion unlike A490 bolts. The standard bolt size we used had a diameter of 3/4”, but in the sections where spans were much larger and would therefore require a much higher number of bolts, we increased bolt size to 7/8” or 1”. The larger bolt diameter allowed us to use less bolts at these larger connections, which worked around the issue of too many bolts not fitting in a connection or adding to overall cost. With these factors in mind, a typical connection designed in the pier was 3 ½” x 3 ½” x 1/4”, with 8 3/4" bolts. In the renovation crescent the largest connection had 14-7/8” bolts, while the largest connection in the main building was 16-1” bolts. Table 4.6 shows all the connections results, and Appendixes P and Q show the calculations for the connections.

Table 4.6 Determined Connection Sizes

Connections		
Location	**Smallest Size**	**Largest Size**
Pier	3 ½” x 3 ½” x 1/4” 8 3/4" bolts	3 ½” x 3 ½” x 7/16” 10 3/4" bolts
Renovation Crescent	3 ½” x 3 ½” x 1/4” 10 3/4" bolts	3 ½” x 3 ½” x 7/16” 14 7/8" bolts
Main Building	3 ½” x 3 ½” x 1/4” 8 3/4" bolts	3 ½” x 3 ½” x 1/2” 16 1" bolts
B-C Connector	3 ½” x 3 ½” x 1/4” 8 3/4" bolts	3 ½” x 3 ½” x 7/16” 10 3/4" bolts

4.3 Revit and RISA Model

We constructed a scale model of our design for Terminal E for Revit. The beams, girders, and columns were appropriately sized and labelled along with the four floors of the terminal. Foundations, shear walls, stairs, and elevators were also included in the Revit design. Interior design choices, such as the stairs and elevators, were included as well. The model was constructed to scale, so that the dimensions could be used to properly place the gates and columns. The structural framing plans and elevation views created in Revit, along with an overview of the model, are shown in Appendixes A, B, and C.

Using RISA, we determined if lateral bracing would be required for parts of the terminal. By creating a small cross-sectional bay in each component of the terminal, we got an idea how our design behaved under seismic conditions. Using the bracing options, as shown in Figure 4.6, we worked with various types of bracing to improve the model's results where possible. Diagonal K and X bracing were the main bracing options that were utilized. Along the pier, for example, the model showed that the bracing should be applied to the first and second floors. A W21x55 beam was determined to be the lightest member possible to support these calculations. This process was repeated for the crescent and main rectangle of Terminal E, and the results are displayed in Table 4.7.

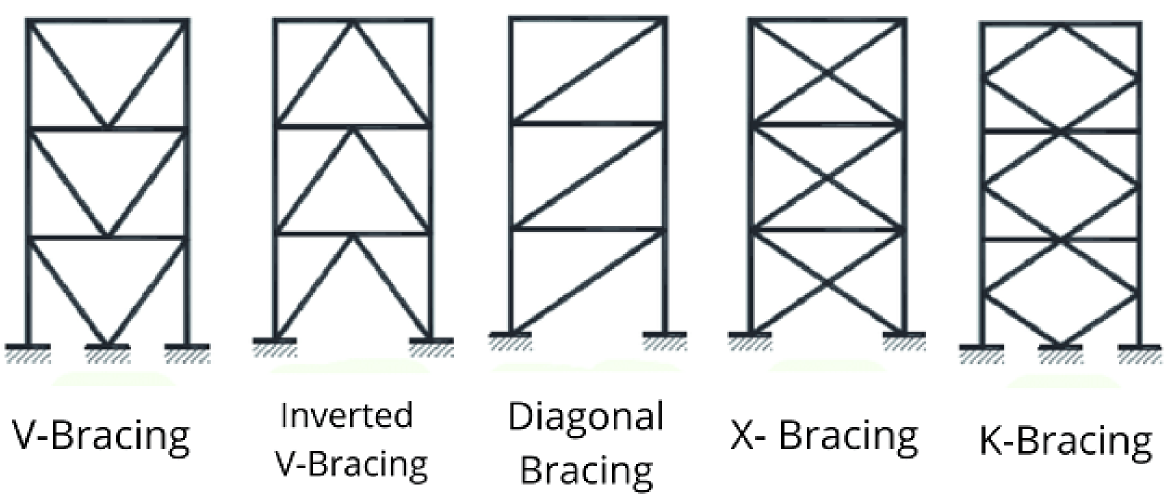

Figure 4.6: A Diagram of the Most Common Bracing Types

In the main rectangle, lateral bracing was not needed, however, the models provided some insight on the current beam sizing for this section. From this, the beam sizes along the main entrance area were increased, as seen in Appendix R. Other alterations that were made to the beam sizing are included in Table 4.7.

Table 4.7: RISA results

Location	**Bracing Type**	**Bracing Size**	**Bracing Location**	**Beam Increase**	**Beam Location**
Pier & E/C Connector	Diagonal	W21x55	Along First two floors	None	None
Renovation Crescent	K Bracing	W12x26	Along Fourth floor, facing the tarmac	W27x84 to W30x90	Roof
Main Building	None	None	None	W12x26 to W24x55	All affected floors

4.4 Cladding

Building cladding is the exterior element of a facility that protects the structure from external factors. From commercial to residential, all types of structures require an efficient and useful cladding material. There are various types of cladding, so we looked for the best material according to weather conditions and design. We decided on aluminum composite panels to be our cladding material. Aluminum composite cladding is common in buildings of this nature, and through research, found that this cladding is being used for the terminal's actual design. In addition to the aluminum material, it was decided to cover some sections of the building with photovoltaic glass. This was done with the intention of improving not only the appearance but also the sustainability of the building.

4.4.1 Aluminum Composite Panel

Aluminum was chosen as the cladding material since it provides insulation, protects the building from inclement weather, resists oxidation and corrosion in humid climates, and is not damaged by sunlight while improving the aesthetic characteristics of the building facility. Aluminum composite material is fully recyclable, lightweight, durable, flexible, and excellent at reducing noise. Manufacturers like LYMO Construction Co. Inc, who is the current manufacturer and installer, use aluminum composite material (consisting of two strong 0.020-inch sheets of aluminum with 85% recycled content) that is bonded to a mix of low-density polyethylene core and core fire retardant (Aluminum Composite Panel (ACM panel)). Figure 4.7 shows the aluminum paneling currently being used in the Massport project.

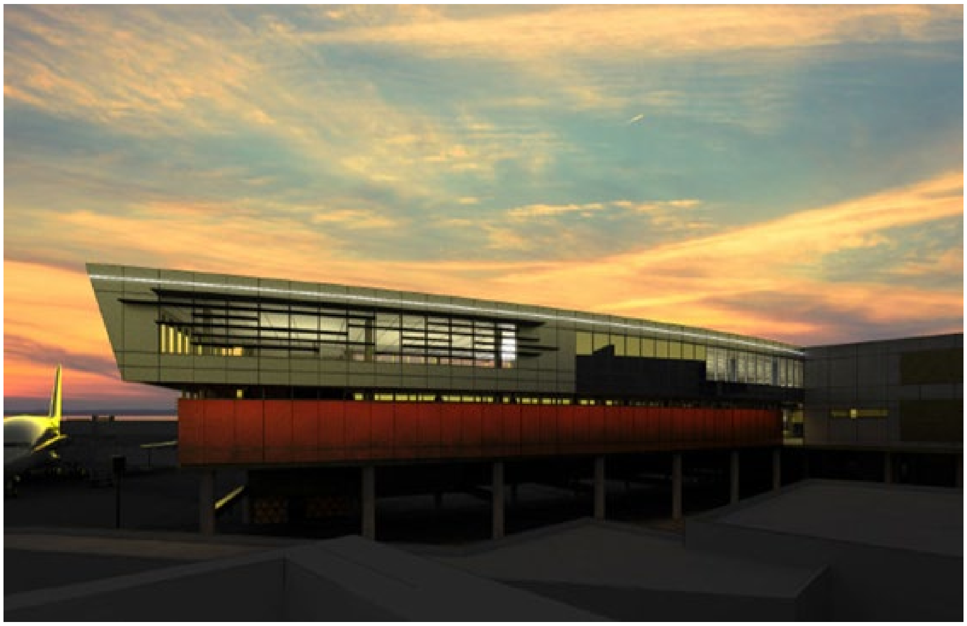

Figure 4.7: Current Design of Terminal E Extension

This thickness is based on the guidelines in Chapter 14 of the International Building Code, which states that the minimum thickness for aluminum siding should be 0.020 inches (ICC) *2018 International Building Code (IBC),* chapter 14). In the same chapter, we found that exterior wall envelope test assemblies should not be less than 4 ft by 8 ft in size and should be subject to a minimum test exposure duration of 2 hours.

LYMO uses the 3000 Rainscreen system for the Aluminum Composite Panel (ACP), as seen in Figure 4.8 (LYMO 3000 Panel System). This system offers a high-tech industrial look in a non-sequential, easy-to-mount system. The open joint systems allow easy installation, and the spline joint covers the panel joints. The extruded frame and non-welded corners of the 3000 series provide a more crisp and clean edge for an improved aesthetic. The design also has the added advantage of being singularly removable.

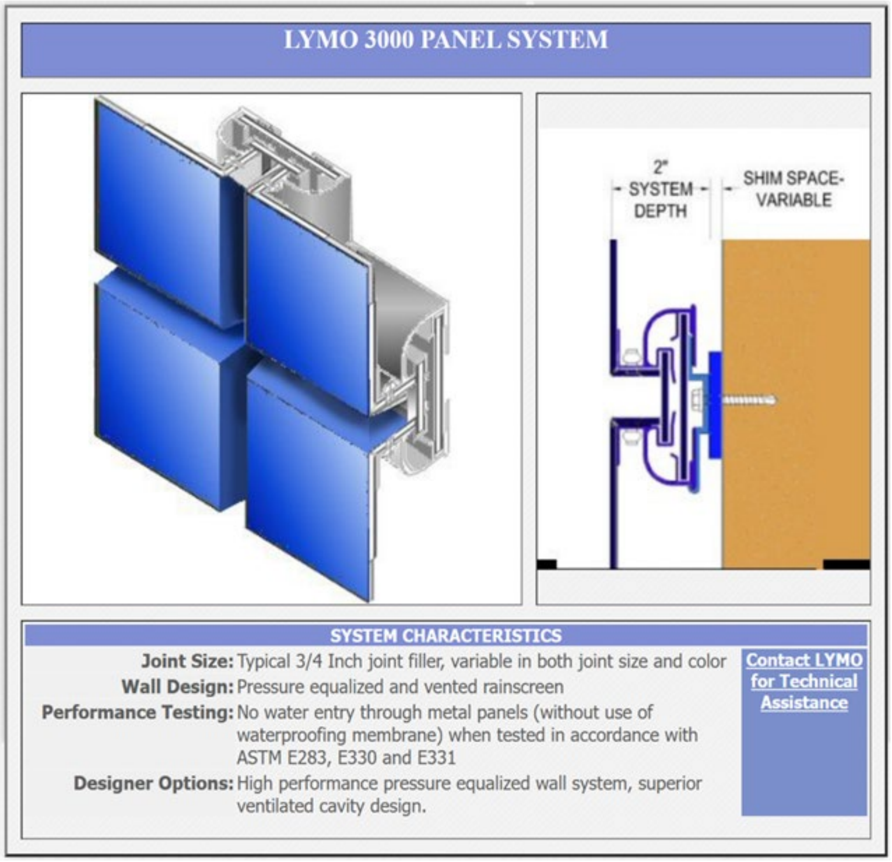

Figure 4.8: Design of LYMO 3000 Cladding System

To install these panels, the following steps are used:

1) Weather Resistance Barrier: The base wall must be waterproofed before cladding may be installed to provide further wind protection.
2) Flashing: The install crew runs flashing along the base of the substrate to complete the waterproofing operation. The flashing creates a gutter that allows water to drain away from the building.
3) Grid: The installers draw a grid onto the barrier that has been applied to the foundation wall. A full grid will contain panel measurements, lines for underlying extrusions, and fastener backplate positions.

4) Extrusions: Extrusions are fastened to the substrate by installers. Extrusions are attached to the vertical perimeter first, then mitered at the top to connect to horizontal extrusions. This part of the foundation wall is framed by vertical and horizontal perimeter extrusions. Installers secure segmented backplates horizontally along the substrate's bottom. A top cap is snap-secured onto these backplates using a sled tool and a mallet to complete the afore mentioned perimeter frame. Next, installers operate in accordance with the grid's vertical lines. For continuous parts of the wall, full backplates are utilized. Half backplates run alongside windows and other complicated sections and are joined together to form full backplates. Three-inch spaces are left at the top and bottom of all backplates to accommodate the upcoming top caps. Installers utilize segmented backplates to follow the grid's horizontal lines, and affix clips to the backplates to ensure that each extrusion receives adequate structural support from the foundation wall.
5) Cladding Panels: Each panel has E-brackets. These fit into the vertical and horizontal extrusions attached to the base wall. The installation crew begins with a lower perimeter corner and works its way up. Top caps are inserted between two parallel panels to fill the space between them. The top caps, which form a grid, cross one another. Horizontal caps go in first, followed by verticals.
6) Post-Installation: The installer removes the protective film, revealing the ACP's hue and texture.

4.4.2 Photovoltaic Glass

Photovoltaic glass (PV glass) is a technique that converts light into electricity. This facade is a good choice for this project since it generates energy, allows for glazing of facades and balconies, has extra thermal features, and may provide significant noise reduction. PV glass may be fitted into existing building facades, updating and making them more energy efficient.

Our intention is to replace the standard glass in windows and skylights with photovoltaic glass. The glass is solid in parts without vision but semi-transparent in places with vision, meant to improve aesthetics by creating a uniform picture of the facade while enabling natural light to enter the building through its windows and visual contact with the outside. PV glass may be simply installed as rainscreen cladding over an existing structure, saving energy and boosting the building's appearance. A breakdown of the layers of PV Glass is shown in Figure 4.9 (Building Integrated Photovoltaics).

The type of glass to be installed depends on the place and the type of application that will be made. In this case, amorphous silicon glass is one of the best options, as it has visible light transmission levels of up to 30%, works well in low temperatures, and is good for rainscreen cladding.

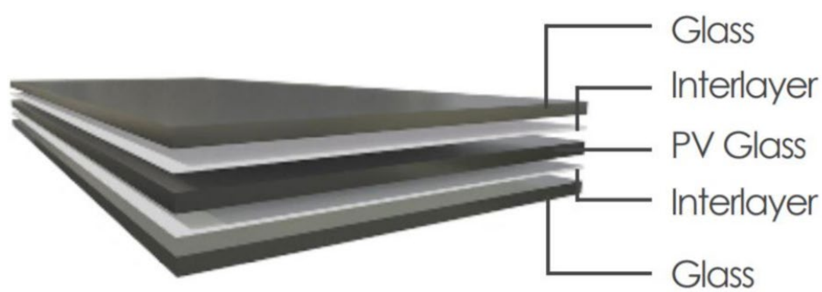

Figure 4.9: Layers in Photovoltaic Glass

To install PV glass, the following steps were followed:

1) Attach brackets to an existing solid wall.
2) Install low weight vertical aluminum profiles on the brackets.

3) Install a robust cable tray on the wall.
4) Insulate the space between the glass and the existing wall.
5) Install the PV glass using clamps from the ground up. Continue to daisy-chain the glass units in accordance with the electrical design.

4.5 Foundations

The result from the foundations reflects the building and the soil that it will be built upon. This included considering the layers in the ground and how strong the soil is based on the load being applied. The soil classification gave us a better understanding of how deep our piles must go to reach the strongest layer and give the footings the support necessary for the building to not sink into the ground.

The soil classification is based on the geotechnical report, with the soil layers shown in Table 4.7. These layers explain the type, thickness, or location it is found underground. Based on the layers, we can get the density per layer and determine the maximum strength that can be applied to the soil itself.

Table 4.8 Geotechnical Report of Soil Layers

Soil Type	Depth	Description
Pavement	12 -16 in	Airside asphalt runways
Granular/Cohesive fill	13 – 25 ft	The granular fill was usually found in the top 5 to 10 feet and consisted of sand and gravel with some silt and cobbles. Cohesive fill was usually encountered beneath the granular fill and consisted of silt or clay with some sand and gravel
Organic Soil	5 ft	Samples collected in this layer consisted of fibrous organic matter with a sulfuric odor
Marine deposit	25 ft	Stratified deposits of fine sand, silt, and clay were encountered below the organic soil and fill
Clay	20 - 30 ft	Silty, low plasticity clay, referred to as Boston Blue Clay, was encountered in every boring that extended deep enough.
Glacial Till	Various depths	The till consisted of clayey sand and gravel
Bedrock	50 – 179 ft	The bedrock consisted of medium-hard, slightly weathered argillite.
Groundwater	6 – 9.5 ft	Groundwater levels may be influenced by tidal fluctuations in Boston Harbor.

Table 4.9 Soil Type Classifications

Soil Type	Classification	Dry unit weight, γ_d (pcf)	Saturated unit weight, γ_s (pcf)
GP, Poorly graded gravel	Sand	110-130	125-140
GW, Well graded gravel	Sand	110-140	125-150
GM, Silty gravel	Sand	100-130	125-140
GC, Clayey gravel	Sand	100-130	125-140
SP, Poorly graded sand	Sand	95-125	120-135
SW, Well graded sand	Sand	95-135	120-145
SM, Silty sand	Sand	80-135	110-140
SC, Clayey sand	Clay	85-130	110-135
ML, Low plasticity silt	Clay	75-110	80-130
MH, High plasticity silt	Clay	75-110	75-130
CL, Low plasticity clay	Clay	80-110	75-130
CH, High plasticity clay	Clay	80-110	70-125
PT, Peat	Clay	30	70

Table 5-2 Relative density of cohesionless soils versus N

N Value (Blows/ft)	Classification	Relative Density Dr (%)
0-4	Very loose	0-15
4-10	Loose	15-35
10-30	Medium Dense	35-65
30-50	Dense	65-85
>50	Very Dense	85-100

Based on the unit weights from Table 4.9 and the type of soil and depth from Table 4.8, we can determine that clayey gravel is 115 pcf and silty sand is 108 pcf. Using the appropriate depth of 20 feet from the clay gravel and 5 feet for the silty sand:

$$\text{Clayey gravel } \sigma z = \gamma * H = 115 \text{ pcf} * 20 \text{ ft} = 2300 \text{ psf}$$

$$\text{Silty sand } \sigma z = \gamma * H = 108 \text{ pcf} * 5 \text{ ft} = 540 \text{ psf}$$

After calculating the soil strength using tables 4.8 and 4.9, we used the information to determine the proper settlement and bearing capacity required for the building. Based on the weakness of the soil, a deep foundation was needed to support the building. The bearing capacity for the pile caps was then determined by calculating the result of Vesic's equations. The work for this is shown in Appendix L.

Small footing sizes that were calculated with the bearing capacity had a length of 18 ft and width of 18 ft, along with the depth of 6ft. For the larger footings, we used a length and width of 24 ft. Based on the completed calculations that can be found in Appendixes M and N, we got values of 154,110 psf for Vesic's, with a bearing capacity of 152.481 lb/ft^2 for the 24 ft^2 footing. Based on these numbers, we chose to use Vesic's equation because it is a more conservative number, allowed the building to hold more weight, and gave us a better representation of how strong and deep our piles must go down into the ground.

The next step was determining the settlement based on the bearing capacity. This represents the strength that the soil once the terminal is built with support from the piles and foundation included in the design. The settlement we found was 0.23 in, as further detailed in Appendixes J and K. This settlement number will determine the depth of our foundation, and the number of piles needed to disperse the pressure of the building. Based on the calculations we determined that the settlements will be 0.85 in in depth once the footings get applied to the soil for larger sections, and settlement load of .23 in in depth for smaller sections of the building.

Next, we determined the length of the pile caps by using the Rankine's formula from methodology for the bearing capacity and the new settlement for the foundation. Each pile should be around 20.44 ft into the ground, not including the 6 ft from the square footing of the foundation, to give a total of 26 ft into the soil. These requirements gave the terminal the best support foundations necessary for the building to satisfy the requirements for the soil typing of the area.

5 Conclusion

Our changes to the terminal will reduce crowding, improve overall efficiency, as well as increase the revenue for the airport. With the addition of the pier, three more gates can be utilized without compromising the taxiways or surrounding infrastructure. The pier design also provides two gates that can support Group VI aircraft. The addition of the extra gate to the renovation crescent can increase the influx of passengers through the terminal, which will in turn increase airport profits. With the flow of passengers expected to increase, another modification was included to improve the overall experience at Terminal E. By expanding the gate sizes, it was possible to include some additional seating directly before the loading bridge for the elderly, or individuals with disabilities. This would be in addition to the seating that is currently designed in the main areas of the terminal, and we believe that its inclusion would improve the passenger experience at the terminal. To reduce crowding before security, the addition of the second INS corridor was included. This would limit the potential bottleneck created by having a singular corridor and allow passengers to flow through more quickly. This will require additional staffing and security; however, the improvement of the passenger experience will improve the airport experience and can generate more jobs for the community. Adding a second INS corridor can also decrease the linear walking distance for some passengers to their gate.

Cost was a consideration for the structural elements of the terminal. When examining the steel members used to design the building, the objective was to ensure that the section sizes chosen would support the required loading safely without costing more than necessary. For the structural steel, A992 steel was used because its properties, such as its good strength-to-weight ratio, would make it well suited to build a large structure like Terminal E. When calculating the sizes of the beams, girders, and columns, the least-weighted member (those bolded on AISC table 3-2) was chosen to reduce the overall cost of the steel. Cambering was also utilized in the process to reduce the section sizes beam and girders with long spans or larger tributary widths. By cambering a beam instead of choosing a larger size that supported loads, cost would be saved from the difference of the weight between the two members. Using lateral bracing instead of increasing the member size would likewise help to reduce the cost. For the columns, using the smallest size that would support the loading ensured the safest and cheapest option. Cost was also a factor in the smallest details, such as the bolts used for this project. We utilized A325N bolts because they were a cheaper choice that met the standards required for the project. By ensuring that the cheapest structural elements available were chosen without sacrificing structural integrity, the project could be built within a reasonable budget.

When choosing the building cladding, the goal was to get a material that would be durable from the strong winds and waters from the harbor nearby. Aluminum cladding satisfies these conditions, while providing a pleasing aesthetic to the outer terminal. We also chose to include photovoltaic glass as part of the design for the glazing in the terminal. While this is costly to install, the benefits of the glass will be shown through its use in the years to come. The glass provides several benefits along with the electricity that it generates, including glare reduction and heat retention. Since the terminal sits along the harbor, the reflection of the light from the water would be a potential issue for those looking out. By capturing direct and indirect energy from the sun, as well as reducing any impact from glare, the efficiency of the building will be improved. The heat retention properties will also decrease the overall cost of heating the building. With windows being a primary source of heat loss in the winter, PV glass will limit this amount, and reduce the heating bill for the building. Likewise in the summer, the glass will prevent heat from entering, making the building slightly cooler and requiring less air conditioning within. Along with this, since glass generates electricity, it will help to provide a daily source of clean energy. While the amount may vary from day to day, it will have an overall impact on the amount of electricity used. Given these factors, we believe PV glass would be well worth the cost, paying off over the long run.

Our team achieved our objective of creating an alternative design for Terminal E, incorporating several modifications aimed at enhancing operational efficiency. This includes improving passenger comfort, and promoting cost-effectiveness and environmental sustainability, despite encountering various challenges during the process. This project has taught us a great deal about the process of designing a building, and we intend to build upon this knowledge in the future.

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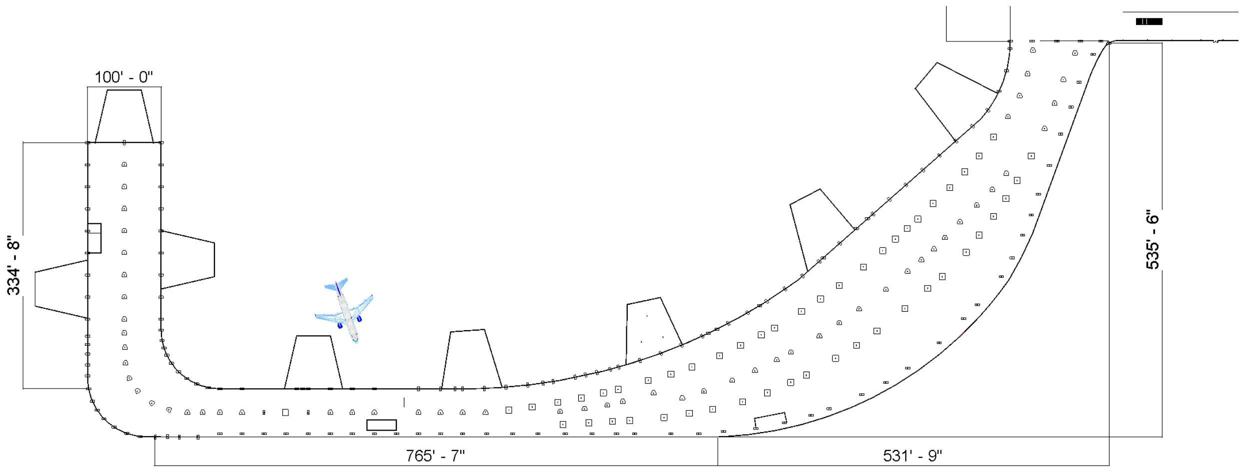
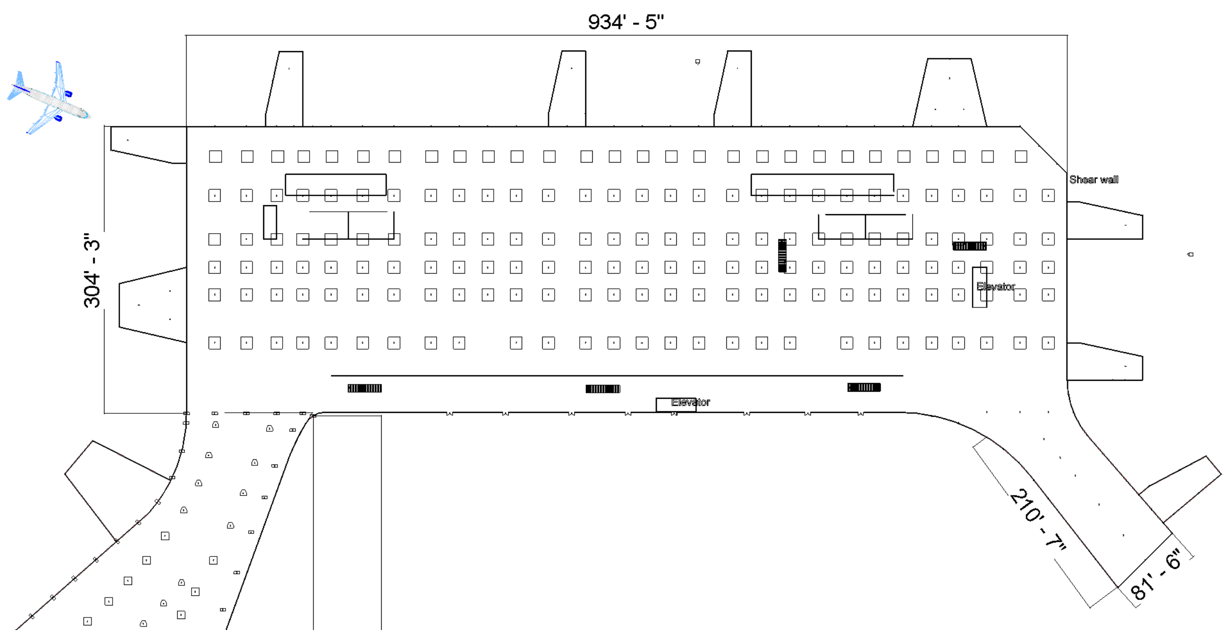
Appendix

Appendix A: Structural Framing Plans

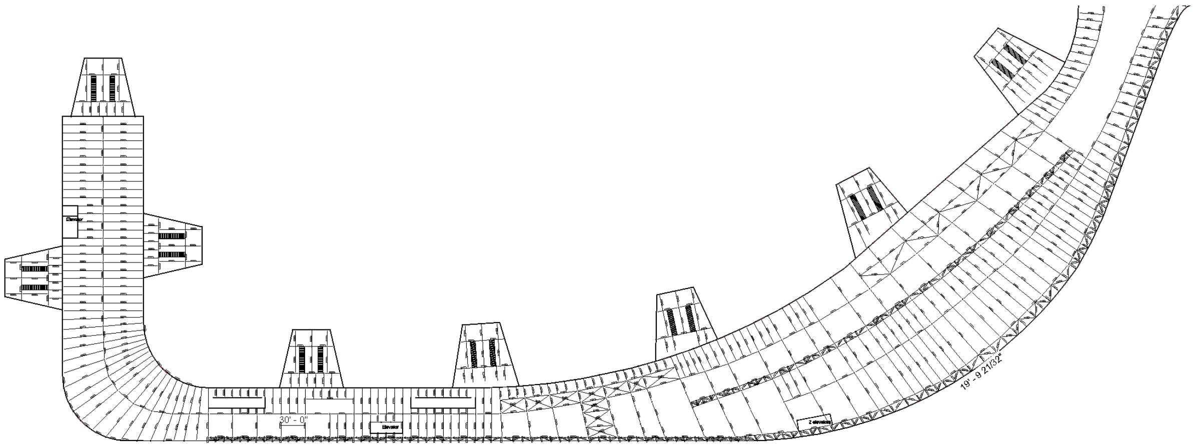
Structural Plan Level 1, Pier and Crescent with Overall Dimensions



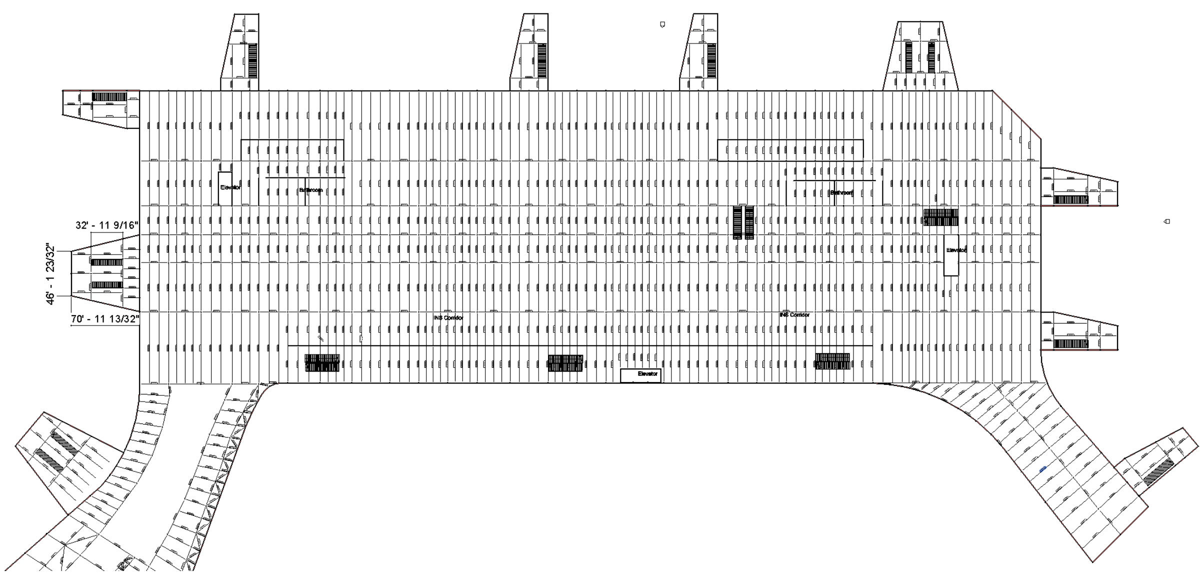
Structural Plan Level 1, Main Building and E/C Connector with Overall Dimensions



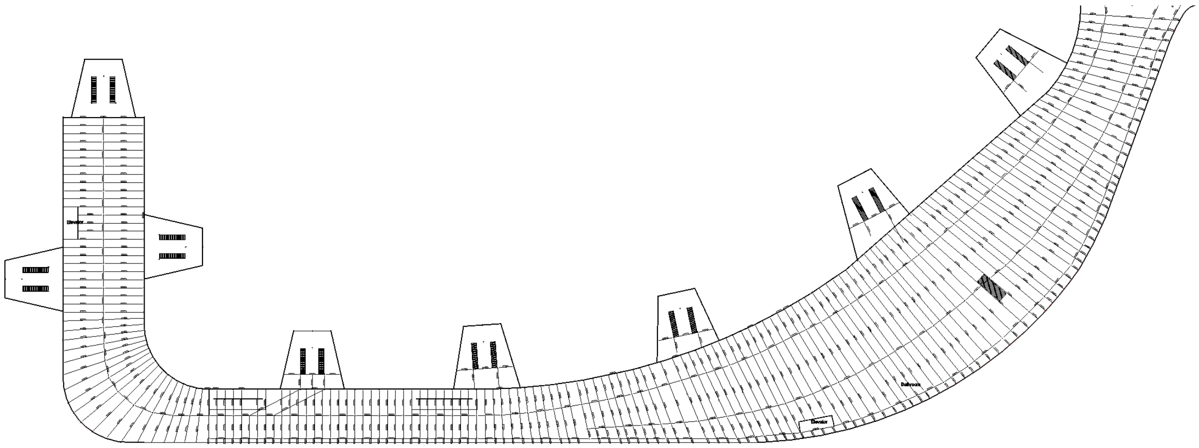
Structural Plan Level 2, Pier and Crescent



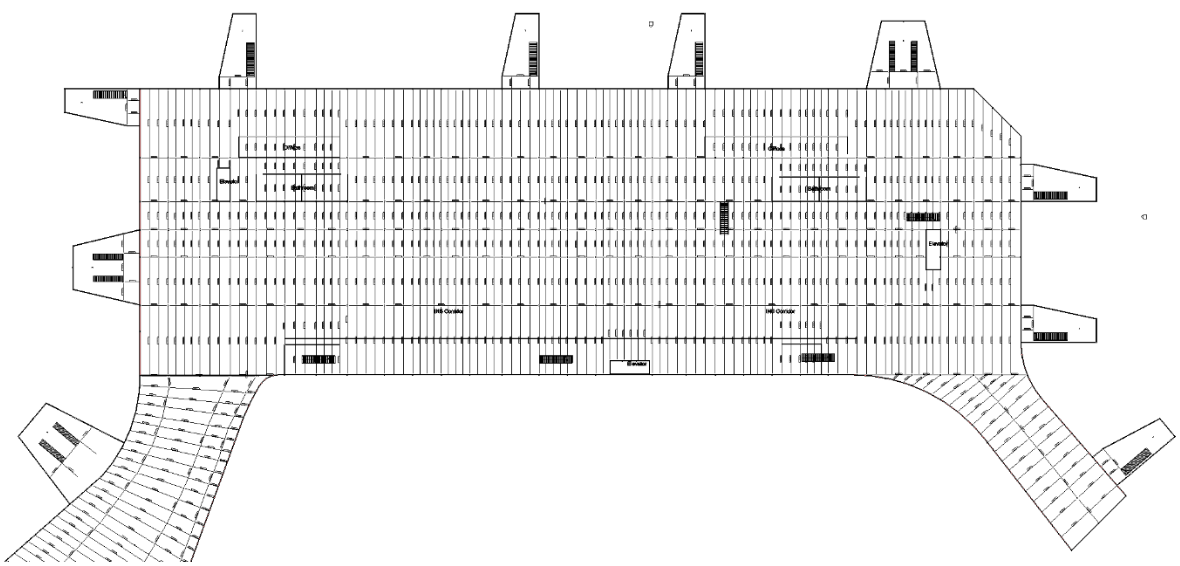
Structural Plan Level 2, Main Building and E/C Connector



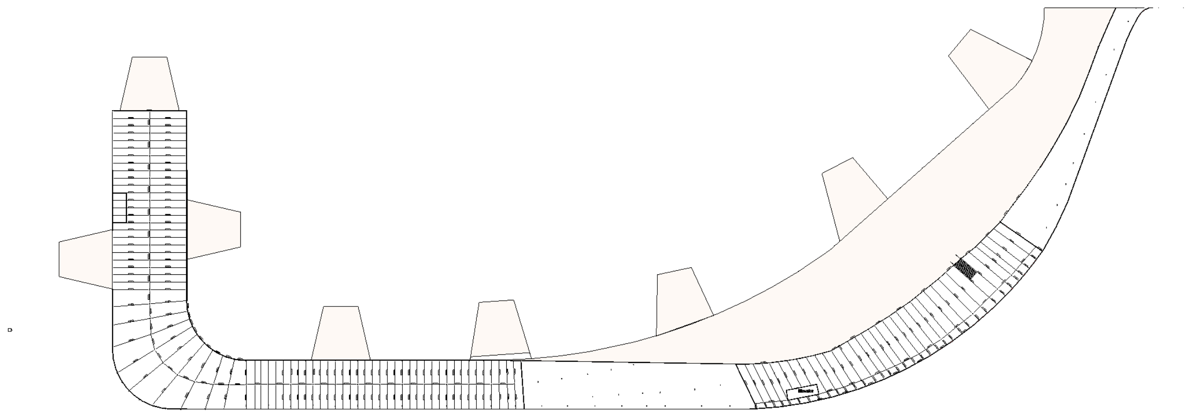
Structural Plan Level 3, Pier and Crescent



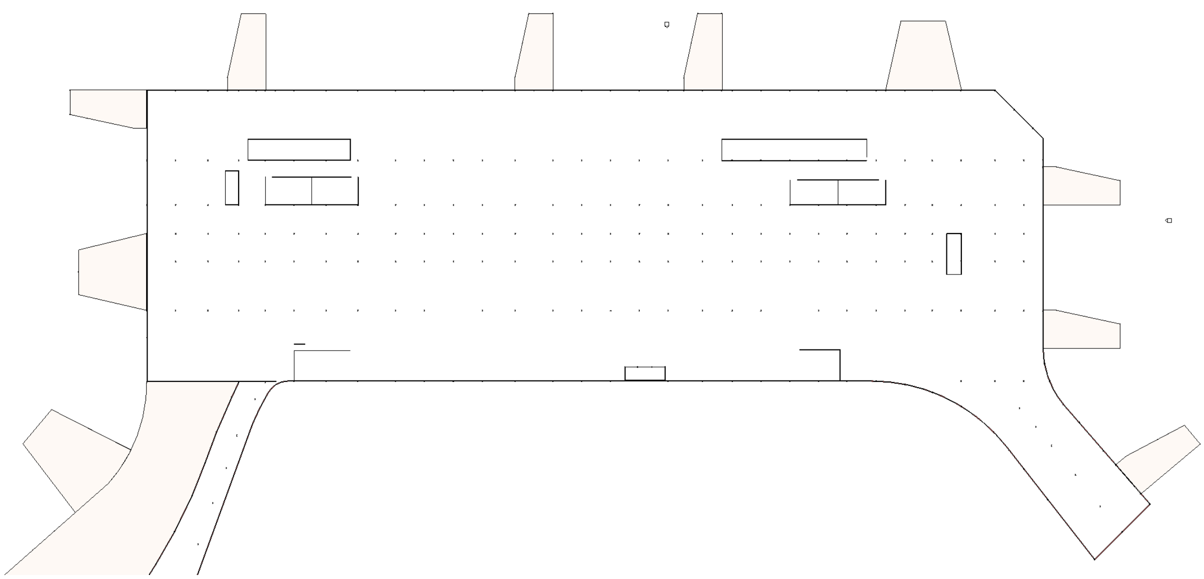
Structural Plan Level 3, Main Building and E/C Connector



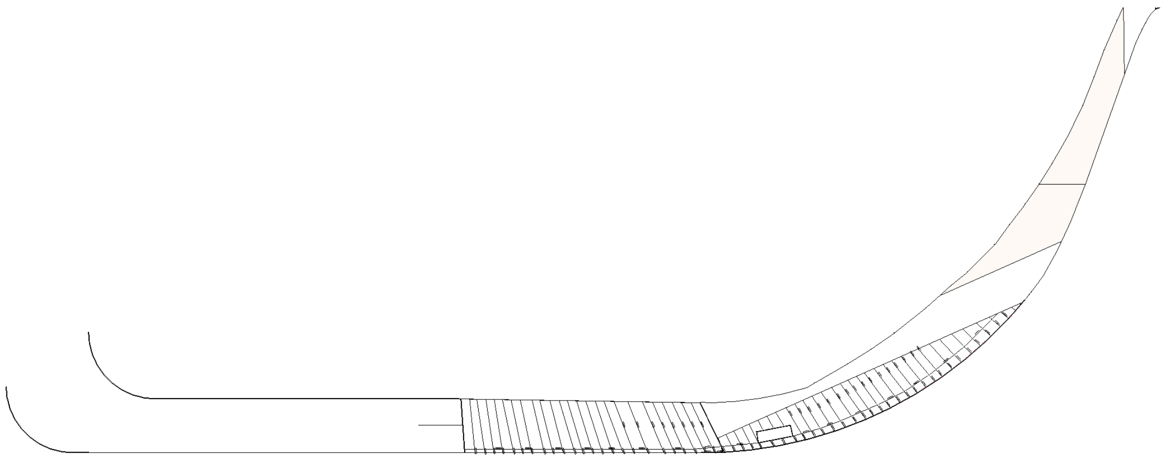
Structural Plan Level 4, Pier and Crescent



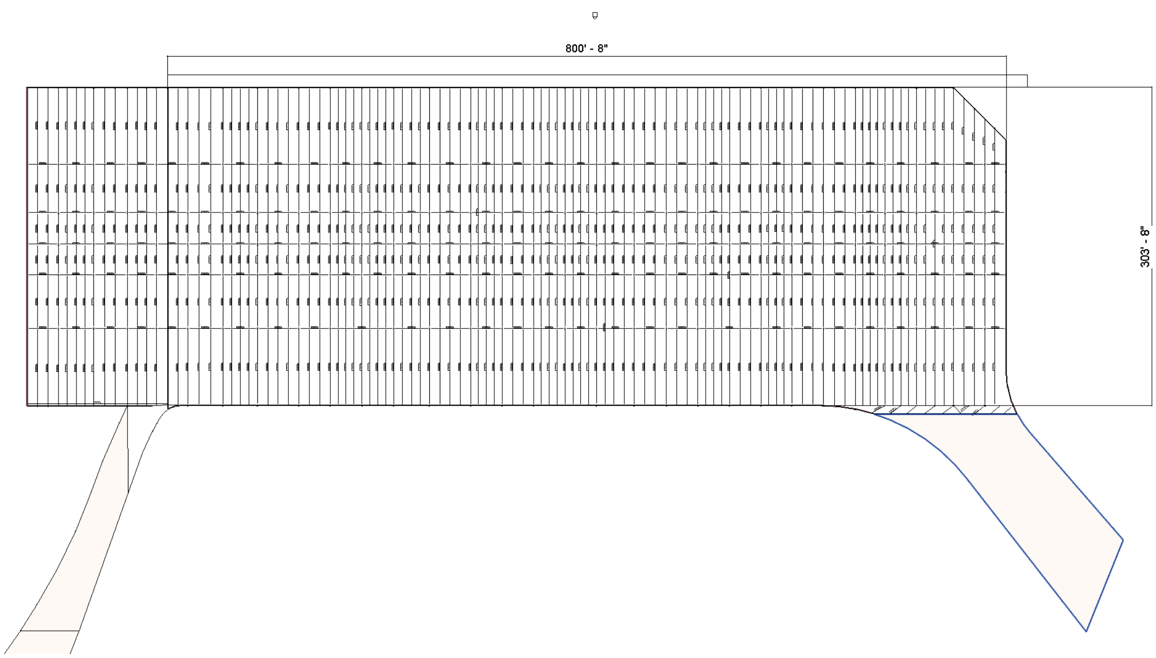
Structural Plan Level 4, Main Building and E/C Connector



Structural Plan Roof, Pier and Crescent

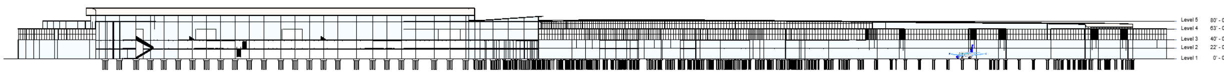


Structural Plan Roof, Main Building and E/C Connector



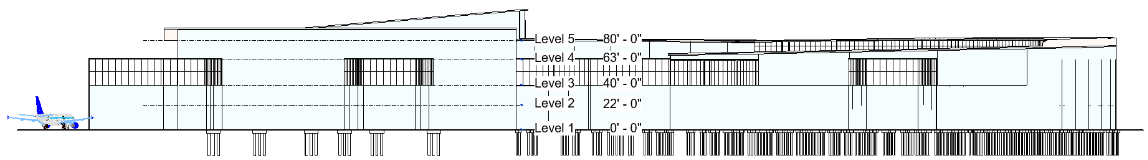
Appendix B: Elevation Views

Elevation View, North

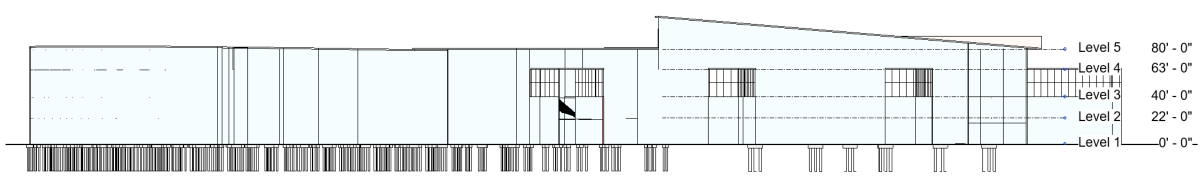


Elevation View, South

Elevation View, West

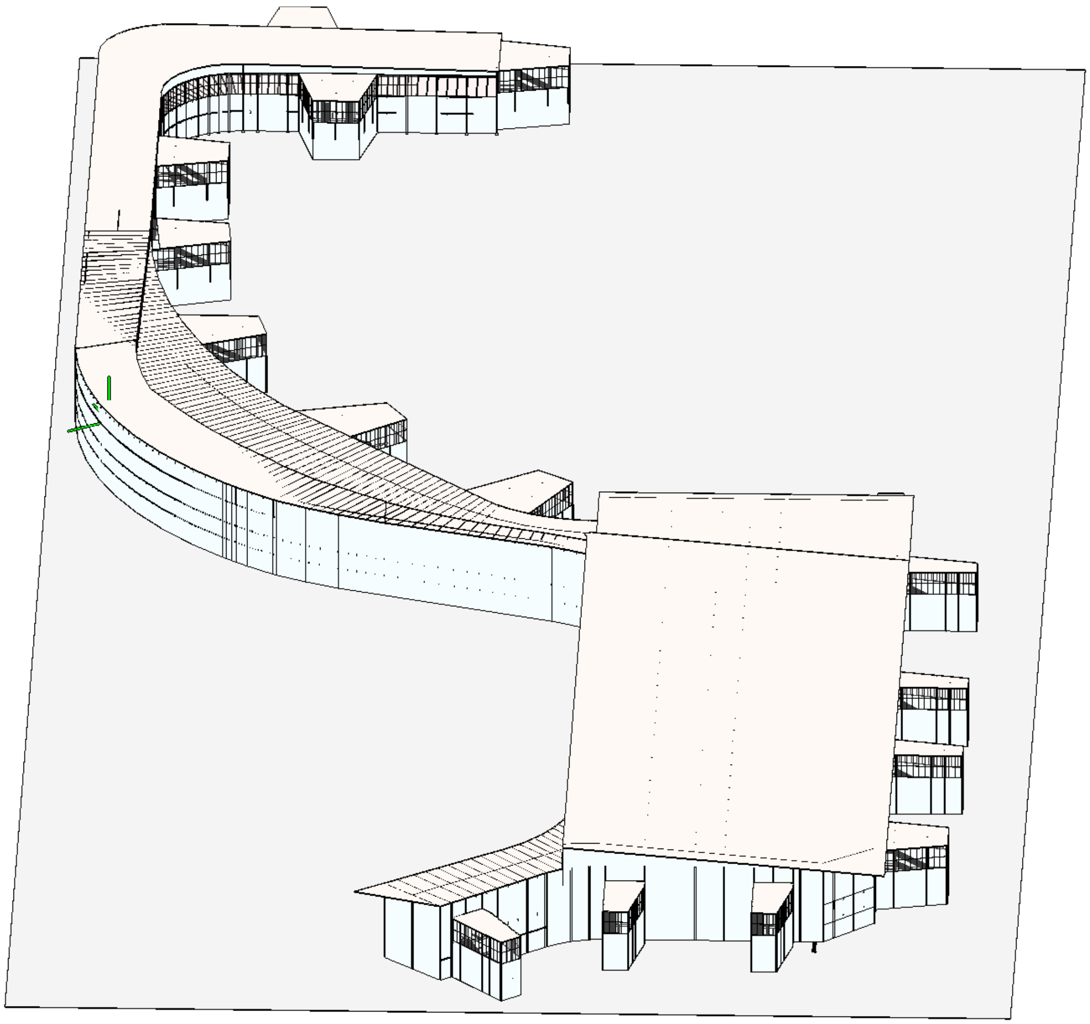


Elevation View, East

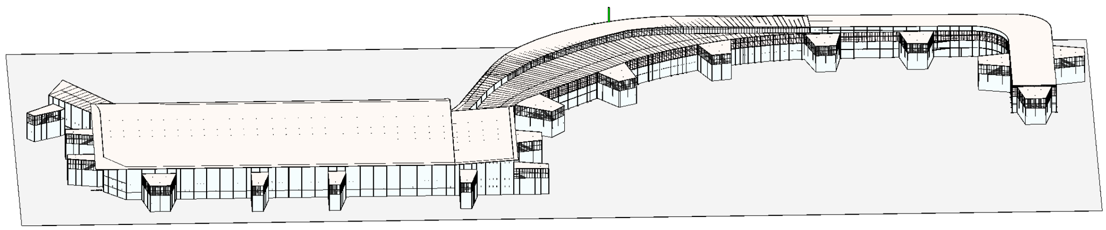


Appendix C: 3D view of Terminal

3D View of Terminal E



3D View of Terminal E, Alternative View



Appendix D: Beam Hand Calculations

Beam Hand Calculations, Page 1

Beam hand calculation

Length = 51 ft = 612 in

Tributary width = 10 ft

Dead load of composite deck = 391 plf = 0.391K/ft

Unfactored Live load = 110 psf = 1100 plf = 1.1 k/ft

Unfactored LL + DL = 0.391 + 1.1 = 1.491 k/ft

Largest Load Combination

$W_u = 1.2(DL) + 1.6(LL)$

$= 1.2(391\ plf) + 1.6(1100\ plf) = 2229.2\ plf$

$M_u = \frac{W_u(L)^2}{8} = \frac{2229.2(51)^2}{8} = 724768.65\ lb \cdot ft$

$= 8697223.8\ lb \cdot in$

$Z = \frac{M_u}{0.9(F_y)}$ where $F_y = 50ksi = 50{,}000psi$

$= \frac{8697223.8}{0.9(50{,}000)} = 193.27$

Choosing W-Shape from AISC Table

W 24X76 => $Z = 200\ in^3$, $I = 2100\ in^4$ (moment of inertia)

Elastic Modulus (E) = 29,000 ksi

Allowable Deflec. LL+DL = L/240 = 612/240 = 2.55 in

Allowable Deflection LL = L/360 = 612/360 = 1.7 in

Allowable Deflection NO Camber = 1.7 in

Actual Deflection $= \frac{5 W_u L^4}{384 E I}$ where W_u is the unfactored LL+DL and unfactored LL

Actual Deflection LL+DL = $\frac{5 \cdot 1.491 \cdot 612^4}{384 \cdot 2100 \cdot 29000 \cdot 12} = 3.73$ in

Actual Deflection LL = $\frac{5 \cdot 1.1 \cdot 612^4}{384 \cdot 2100 \cdot 29000 \cdot 12} = 2.75$ in

Actual		Allowable
3.73	>	2.55
2.75	>	1.7

Since actual deflection is greater than the allowable, we need to pick a new size.

W-Shape from AISC Table

W 30 × 116 => Z = 378 in^3, I = 4930 in^4

New Actual Deflection LL+DL = $\frac{5 \cdot 1.491 \cdot 612^4}{384 \cdot 4930 \cdot 29000 \cdot 12} = 1.59$ in

New Actual Deflection LL = $\frac{5 \cdot 1.1 \cdot 612^4}{384 \cdot 4930 \cdot 29000 \cdot 12} = 1.17$ in

New Actual		Allowable
1.59	<	2.55
1.17	<	1.7

Now both actual deflections are smaller than the allowable deflections but the member is too big. We try to camber the section to check if we can use our first option.

Actual Deflection = $\frac{5 \cdot 0.391 \cdot 612^4}{384 \cdot 2100 \cdot 29000 \cdot 12} = 3.26$ in

Camber = $3.26 \cdot 0.8 = 2.61$ in

We check maximum camber = $L/180 = 612/180 = 3.4$ in

We round down to quarter of inch (no greater than maximum camber)

With a camber of 2.25 in

The Net deflection $3.26 - 2.25 = 1.01$ in

$1.01 < 1.7$ $\quad 1.01 < 2.55$

* If we camber the section, we can use our first size which was W24X76

Appendix E: Beam Calculations Excel Spreadsheet

Beam Calculations, Excel Spreadsheet

Denisty concrete (pcf)	110		thickness (in)	4	tributary beam width (ft)	10		thickness of 6" is from concrete slab
Slab weight	366.6666667	plf	(ft)	0.3333333				
Length of beam (ft)	51	ft						
	612	in						
Wind load	13	psf	130	plf				
Snow load	25.2	psf	252	plf				
Dead load (composite deck)			391	plf	0.391	k/ft		
Dead Load (roofing material)	10	psf	100	plf				
Unfactored LL+DL	1491	plf			1.491	k/ft		
Unfactored LL	110	psf	1100	plf	1.1	K/ft		
Wu1	547.4	plf						
Wu2	2229.2	plf						
Wu3	2229.2							
Wu4	1825.2	plf						
Mu	724768.65	ft-lb	8697223.8	in-lb				
Z>=	193.27164		Fy	50,000	psi			
			phi	0.9				
Choose Steel Beam from Z (A10)	W24x76		unit weight	76	LB/FT	0.076	k/ft	
Zx	200	in^3						
moment of Inertia	2100	in^4						
Elastic modulus	29000	ksi						
Use inertia tables for new Section	W30x116		Actual deflection LL+DL (with new Ix)	1.587435	<	2.55		
New Ix	4930	in^4	Actual deflection LL (with new Ix)	1.1711459	<	1.7		
new Zx (found with same section on Zx	378	in^3	Actual deflection (w/o camber)	1.587435	<	1.7		
Update Wu	2320.4	plf						
Update Mu	754420.05	ft-lb	9053040.6	in-lb				
Z>=	201.17868	<	378					
Actual deflection LL+DL	3.72669262	in	Allowable LL+DL	2.55	in	L/240		
Actual deflection LL	2.749404347	in	Allowable Deflection LL	1.7	in	L/360		
			Allowable Non camber	1.7	in	L/360		
Actual Deflection DL	3.256794422	in						
Camber	2.605435538	in						
round down to quarter of inch	2.25	in						
Max camber	3.4	in	Net DL deflection	1.0067944	in			
Notes:								
If actual deflection is too high, so use I tables to choose larger section								
Updated Wu uses unit weight from chosen section								
Live load of 110 psf is from 100 psf building live load and 10 psf MEP								

Appendix F: Girder Hand Calculations

Girder Hand Calculations, Page 1

Girder hand calculation

Length = 36 ft = 432 in

Tributary width = 40.5 ft

Dead load of composite deck = 391 plf = 0.391 k/ft

Dead load of the beam = 84 plf

Live load = 110 · 40.5 = (100 psf building live load and 10 psf MEP · tributary width)

= 4455 plf

Unfactored Live load = 4455 plf = 4.455 K/ft

Unfactored LL + DL = (Composite deck DL + Beam DL) + LL

= (391 + 84) + 4455 = 4930 plf = 4.93 k/ft

Largest Load Combination

$W_u = 1.2(\text{Composite deck DL} + \text{Beam DL}) + 1.6(LL)$

$= 1.2(391 + 84) + 1.6(4455)$

$= 7698$ plf

$M_u = \frac{W_u (L)^2}{8} = \frac{7698 \cdot (36)^2}{8} = 1247076$ lb·ft

$= 14964912$ lb·in

$Z = \frac{M_u}{0.9(F_y)} = \frac{14964912}{0.9(50{,}000)} = 332.55$

Choosing W-Shape from AISC Table

W 27×84 => $Z = 244\ in^3$, $I = 2850\ in^4$, $E = 29000$ ksi

Allowable Deflec. LL + DL = L/240 = 432/240 = 1.8 in

Allowable Deflec. LL = L/360 = 432/360 = 1.2 in

Allowable NO camber = 1.2 in

Actual deflection = $\frac{5 W_u L^4}{384 EI}$ where W_u is the unfactored LL + DL and unfactored LL

Girder Hand Calculations, Page 2

Actual deflection LL+DL $= \dfrac{5 \cdot 4.93 \cdot 432^4}{384 \cdot 12 \cdot 2850 \cdot 29000} = 2.25$ in

Actual deflection LL $= \dfrac{5 \cdot 4.455 \cdot 432^4}{384 \cdot 12 \cdot 2850 \cdot 29000} = 2.04$ in

Actual		Allowable
2.25	>	1.8
2.04	>	1.2

Since actual deflection is greater th the allowable, we need to pick a new size.

W-Shape from AISC Table

W33×118 => $Z = 415\ in^3$, $I = 5900\ in^4$

New Actual Deflection LL+DL $= \dfrac{5 \cdot 4.93 \cdot 432^4}{384 \cdot 12 \cdot 5900 \cdot 29000} = 1.09$ in

New Actual Deflection LL $= \dfrac{5 \cdot 4.455 \cdot 432^4}{384 \cdot 12 \cdot 5900 \cdot 29000} = 0.98$ in

Actual deflection NO camber = 1.09 in

New Actual		Allowable
1.09	<	1.8
0.98	<	1.2

Both deflections are smaller than the allowable and the size is a good size for the space where it is and strong enough to support the weight of the beams.

Now we need to verify the shear strength.

Choosing Steel Beam W 33X118

Values from table 1-1 in AISC manual

$A = 34.7\ in^2$ $K_{des} = 1.44\ in$

$d = 32.9\ in^2$ $h = d - 2 \cdot K_{des} = 32.9 - 2 \cdot 1.44 = 30.02\ in$

$t_w = 0.55\ in$ $h/t_w = 30.02/0.55 = 54.58$

We check the equation $2.24\sqrt{E/F_y}$. If our h/t_w is smaller than this equation $C_v = 1$ and $\phi_v = 1$. If not we check the equation $1.1\sqrt{\frac{K_v E}{F_y}}$ where $K_v = 5.34$ and $C_v = 1$ and $\phi_v = 0.9$

$$2.24\sqrt{\frac{29000}{50}} = 53.95 < 54.58 \text{ NO Good}$$

$$1.1\sqrt{\frac{5.34(29000)}{50}} = 61.22 > 54.58 \text{ OK!!!}$$

$$V_u = W_u \cdot \frac{L}{2} = 7698 \cdot \frac{36}{2} = 138564\ lb$$

$V_n = 0.6\ F_y\ A_w\ C_v$ where $A_w = d \cdot t_w$

$$0.6 \cdot 50 \cdot (32.9 \cdot 0.55) \cdot 1 = 542.85\ k = 542850\ lb$$

$$\phi_v V_n = 0.9 \cdot 542.85 = 488.565K = 488565\ lb$$

$$V_u = 138564 < 488565\ lb \text{ GOOD!!!}$$

Appendix G: Girder Spreadsheet

Girder Calculations, Excel Spreadsheet

1	Denisty concrete (pcf)	110		thickness (in)	4	tributary beam width (ft)	40.5			
2	Slab weight	1485	plf	(ft)	0.333333333					
3	Length of beam (ft)	36	ft							
4		432	in	For Vulcraft composite deck slab with lightweight concrete 6-1/4", 19 deck gage						
5										
6	Wind Load	13	psf	526.5	plf					
7	Snow load	25.2	psf	1020.6	plf					
8	Rain loads									
9	Dead load (composite deck)			391	plf	0.391				
10	Dead load (beam)			84	plf					
11	Dead Load (roofing material)			391	plf					
12	Live load	110	psf	4455	plf					
13	Live load roof	20	psf	810	plf	0.02	k/ft			
14	Unfactored LL+DL			4930	plf	4.93	k/ft			
15	Unfactored LL			4455	plf	4.455	k/ft			
16										
17	Wu1	665	plf	7.698	k/ft					
18	Wu2	7698	plf							
19	Wu3	7698	plf							
20	Wu4	6061.8	plf							
21										
22	Mu	1247076.00	ft-lb	14964912	in-lb					
23	Z>=	332.5536		Fy	50,000	psi	50	ksi		
24				phi	0.9					
25	Choose Steel Beam	W27x84		unit weight	84	LB/FT				
26	Zx	244	in^3			0.084				
27	moment of Inertia	2850	in^4							
28	Elastic modulus	29000	ksi							
29										
30	Use inertia tables for new Section	W33x118								
31	New Ix	5900	in^4	Actual deflection LL+DL (with new Ix)	1.088903593	<	1.8			
32	new Zx (found with same section on Zx tables)	415	in^3	Actual Deflection LL (with new Ix)	0.983988947	<	1.2			
33				Actual deflection (w/o camber)	1.088903593	<	1.2			
34	Update Wu	7798.8	plf				original+1.2*unit weight			
35	Update Mu	1263405.6	ft-lb	15160867.2	in-lb					
36	Z>=	336.90816	<	415						
37										
38										
39	Actual Deflection LL+DL	2.254221474	in	Allowable LL+DL deflection Other mem	1.8	in	L/240			
40	Actual Deflection LL	2.03702975	in	Allowable LL deflection Other member	1.2	in	L/360			
41				Allowable Non camber	1.2	in	L/360			
42										
43	Choose Steel Beam	W33x118								
44	A=	34.7	in^2		Table 1-1					
45	d=	32.9	in		Table 1-1					
46	tw=	0.55	in		Table 1-1					
47	kdes=	1.44	in		Table 1-1					
48	h=	30.02	in							
49	h/tw	54.58	if <	53.95		Cv=	1	ϕv=	1	
50				61.22	OK!!!	kv=	5.34	Buildings		
51						Cv=	1	ϕv=	0.9	
52										
53	Vu=	138564.00	lb	<	488565	lb	OK!	489	kips	Table3-2
54	Vn=	542.85	k	542850	lb					
55	ϕvVn=	488.565	k	488565	lb					
56										

Appendix H: Column Hand Calculations

Column Hand Calculations

Column hand calculations

Friday, February 17, 2023 4:52 PM

Variables

Density concrete (pcf): 150

Slab weight (psf): 39.1 (SW)

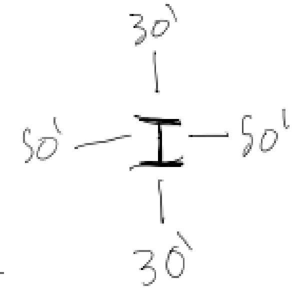
thickness (in): 6 = .5 ft

height of Column (ft): 23 = 276 in

Dead Load concrete: $C = P \cdot (t) = 110 \cdot .5 = 55$ psf

Dead Load steel: 84 psf (DLS)

Live Load = 110 psf (LL)



$$\text{Tributary area} = \left(\frac{N}{2} + \frac{S}{2}\right) \cdot \left(\frac{W}{2} + \frac{E}{2}\right)$$

$$\left(\frac{30}{2} + \frac{30}{2}\right) \cdot \left(\frac{50}{2} + \frac{50}{2}\right) = 1500\,ft^2$$

$$P_u = (1.2)(SW + DLS) + (1.6 \cdot LL) \cdot \text{tributary area}$$

$$= (1.2)(39.1\,psf + 84\,psf) + (1.6 \cdot 110\,psf) \cdot 1500\,ft^2 = 485{,}580\,lb = 485.58\,kips$$

assume $\frac{KL}{r} = 50$ $[P_n > P_u)$

AISC table 4-14 $\phi_c F_{cr} = 37.7\,ksi$

$$A_{req} = \frac{P_u}{\phi_c F_{cr}} = \frac{485.58k}{37.7\,ksi} = 12.88\,in^2$$

W10x88 $(A_g = 26\,in,\ r_y = 2.63)$

$$\left(\frac{KL}{r}\right)_y = \frac{276}{2.63} = 105$$

$$\phi_c f_{cr} = 20.1$$

$$\phi_c P_n = (\phi_c f_{cr})(A_g) = (20.1) \cdot (26) = 522.6 > 485.58 \checkmark$$

W10x88

Appendix I: Column Spreadsheet

Column Calculations, Excel Spreadsheet

Denisty concrete (pcf)	110		thickness (in)	6	Column height	30	
Slab weight	39.1	psf	(ft)	0.5	K (eccentricity)	1	
Height of Column (ft)	40	ft					Add MEP and dead load of steel
	480	in					
Bay size							
North	21	Ft					Keep all dead load and live load in psf
South	28	Ft					
East	33	Ft					
West	0	Ft					
Dead load concrete	55	psf					
Dead load steel	62	psf					
Live load	110	psf		level 1-2	22		
Tributary area	404.25	ft^2		level 2-3	18		
				level 3-4	23		
				level 4-5	17		
Pu	120,191.61	lb			63		
	120.19	kips					
Choose Column Size							
Size:	W14x()						
Grade							
Steel	Fy	50000	psi				
Notes:							
Live load of 110 psf is from 100 psf building live load and 10 psf MEP							
Would dead load of steel be taken from Girder design spreadsheet?							

Appendix J: Settlement Hand Calculations

Settelment

Monday, February 27, 2023 4:49 PM

$Q_s = 24{,}900$ kips

$q_0 = 154.11$ ksf

$L = 45$ ft

$A_{req} = 11\,ft^2$ $\quad 2000 \cdot 32.67 + ($

$B = 3.75$ ft

$D = 30$

$E_s = 719410$ ksf

$C_p = .03$

$C_s = (.93 + (.16 \cdot \frac{D}{B}) \cdot C_p = 0.07$

$$w_s = (Q_p + \alpha_s \cdot Q_s)\frac{L}{AE_p} = (0 + .667 \cdot 1000)\frac{45}{11 \cdot 719410} = 0.04$$

$$w_{pp} = \frac{C_p Q_p}{B q_0} = \frac{0.03 \cdot 0}{3.75 \cdot 154.11} = 0$$

$$w_{ps} = \frac{C_p \cdot Q_s}{D q_0} = \frac{.03 \cdot 1000}{30 \cdot 154.11} = 0.14$$

$w_0(ft) = .04 + 0 + .14 = .18$ ft or .218 in

Appendix K: Settlement Spreadsheet

Settlement Spreadsheet, Page 1

Given Variable	Value	Units						
Q's	24,900	kips						
q0	154.11	ksf						
L	45	ft						
A	11	Ft^2	11.0446617					
B	3.75	ft						
D	30							
Es	719910	ksf						
alpha	0.667							
Cp	0.03							
Computed Variable								
Cs	0.07							
Vesic Method Load-Settlement								
Load at Pile Head, kips	Mobilized Shaft Resistance Qs, kips	Mobilized Toe Resistance Qp, kips	Ws, ft.	Wpp, ft.	Wps, ft.	Wo, ft.	Vesic Method Wo, in.	
0	0	0	0.000	0.000	0.000	0.000	0.000	
50	50	0	0.000	0.000	0.001	0.001	-0.011	
100	100	0	0.000	0.000	0.001	0.002	-0.022	
150	150	0	0.001	0.000	0.002	0.003	-0.033	
200	200	0	0.001	0.000	0.003	0.004	-0.044	
250	250	0	0.001	0.000	0.004	0.005	-0.054	
300	300	0	0.001	0.000	0.004	0.005	-0.065	
350	350	0	0.001	0.000	0.005	0.006	-0.076	
400	400	0	0.002	0.000	0.006	0.007	-0.087	
450	450	0	0.002	0.000	0.006	0.008	-0.098	
500	500	0	0.002	0.000	0.007	0.009	-0.109	
550	550	0	0.002	0.000	0.008	0.010	-0.120	
600	600	0	0.002	0.000	0.009	0.011	-0.131	
650	650	0	0.002	0.000	0.009	0.012	-0.141	
700	700	0	0.003	0.000	0.010	0.013	-0.152	
750	750	0	0.003	0.000	0.011	0.014	-0.163	
800	800	0	0.003	0.000	0.011	0.015	-0.174	
850	850	0	0.003	0.000	0.012	0.015	-0.185	
900	900	0	0.003	0.000	0.013	0.016	-0.196	
950	950	0	0.004	0.000	0.014	0.017	-0.207	
1000	1,000	0	0.004	0.000	0.014	0.018	-0.218	
TAMWAVE Load-Settlement Curve								

4. SETTLEMENTS OF PILE FOUNDATIONS

a. Single Pile. The settlement at the top of pile can be broken down into three components (after Reference 6).

(1) Settlement due to axial deformation of pile shaft; Ws

$$Ws = (Q_p + \alpha_s Q_s)\ \frac{L}{AE_p}$$

where: Q_p = point load transmitted to the pile tip in the working stress range.

Q_s = shaft friction load transmitted by the pile in the working stress range(in force units)

α_s = 0.5 for parabolic or uniform distribution of shaft friction

0.67 for triangular distribution of shaft friction starting from zero friction at pile head to a maximum value at pile point

0.33 for triangular distribution of shaft friction starting from a maximum at pile head to zero at the pile point.

L = pile length

A = pile cross sectional area

E_p = modulus of elasticity of the pile

(2) Settlement of pile point caused by load transmitted at the point W_{pp}:

$$W_{pp} = \frac{C_p\ Q_p}{Bq_o}$$

where: C_p = empirical coefficient depending on soil type and method of construction, see Table 5

B = pile diameter

q_o = ultimate end bearing capacity

(3) Settlement of pile points caused by load transmitted along the

where : $C_s = (0.93 + 0.16\ D/B)\ C_p$

D = embedded length

(4) Total settlement of a single pile, W_o:

$$W_o = W_s + W_{pp} + W_{ps}$$

TABLE 5
Typical* Values of Coefficient C_p for Estimating Settlement of a Single Pile

Soil Type	Driven Piles	Bored Piles
Sand (dense to loose)	0.02 to 0.04	0.09 to 0.18
Clay (stiff to soft)	0.02 to 0.03	0.03 to 0.06
Silt (dense to loose)	0.03 to 0.05	0.09 to 0.12

* Bearing stratum under pile tip assumed to extend at least 10 pile diameters below tip and soil below tip is of comparable or higher stiffness.

Appendix L: Pile Caps Spreadsheet

Pile Caps, Excel Spreadsheet

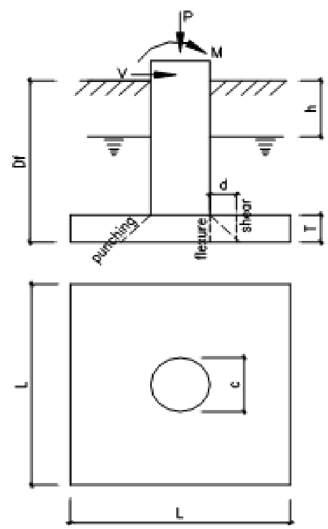
Daniel T. Li
Engineering International

PROJECT :
CLIENT :
JOB NO. : DATE :
PAGE :
DESIGN BY :
REVIEW BY :

Deep Footing Design Based on ACI 318-02

INPUT DATA

PEDESTAL DIAMETER	c	=	45	in
SQUARE FOOTING LENGTH	L	=	18	ft
FOOTING EMBEDMENT DEPTH	D_f	=	6	ft
FOOTING THICKNESS	T	=	18	in
WATER TABLE	h	=	13	ft
CONCRETE STRENGTH	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
AXIAL DEAD LOAD	P_{DL}	=	115	k
AXIAL LIVE LOAD	P_{LL}	=	365.4	k
LATERAL LOAD (0=WIND, 1=SEISMIC)		=	0	Wind,ASD
WIND AXIAL LOAD	P_{LAT}	=	-79.18	k, ASD, uplift
WIND MOMENT LOAD	M_{LAT}	=	867.4	ft-k, ASD
WIND SHEAR LOAD	V_{LAT}	=	11.95	k, ASD
SURCHARGE	q_s	=	0.1	ksf
BBACKFILL SOIL WEIGHT	w_s	=	0.1	kcf
ALLOW SOIL PRESSURE	Q_a	=	4	ksf
FOOTING REINFORCING SIZE		#	10	
PEDESTAL VERT. REINF. SIZE	22	#	10	vertical
PEDESTAL SHEAR. REINF.	#	4	spiral @	3 in o.c.



DESIGN SUMMARY

TOP FOOTING REINF., E. WAY => 5 # 10
BOT. FOOTING REINF., E. WAY => 14 # 10 @ 16 in o.c.

THE FOOTING DESIGN IS ADEQUATE.

Appendix M: Bearing Capacity Hand Calculations

Bearing Capacity, Hand Calculations

foundation
Tuesday, December 13, 2022 11:15 AM

- Boring for Shallow Foundation Table 1 — Vesic equation

Average = 3000 ft² Structural footprint area for each boring

Minimum depth of boring

average $15S^{.7}+D = 15\cdot(9)^{.7} + 20ft = 60ft$

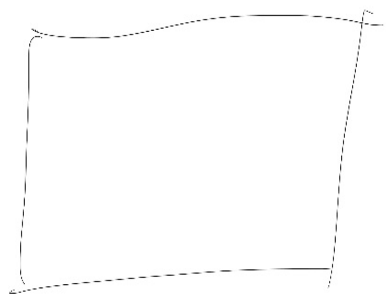


table 4.1

unit weight γ — $\gamma_w = 62.43$ — $D_w = 3ft$

Lean Clay CL = A: 95 lb/ft³ B: 103

silty sand SM = A: 108 B: 125

Sand with gravel SW = A: 115 B: 133

$B = 24.67ft$, $L = 1011ft$, $D = 6ft$, $C = 1795\ lb/ft^2$

Table 6.4 Vesics

Friction angle = 17° ϕ — $N_c = 12.3$, $N_q = 4.77$, $N_\gamma = 3.53$ — $\gamma' = 125 - 62.4 = 62.57$

$$q_{ult} = c' N_c s_c d_c + \sigma'_{zD} N_q s_q d_q + .5\gamma' B N_\gamma s_\gamma d_\gamma$$

$$S_c = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right) = 1 + \left(\frac{24.67}{1011}\right)\left(\frac{4.77}{12.34}\right) = 1.94$$

$$S_q = 1 + \left(\frac{B}{L}\right)\tan\phi = 1 + \left(\frac{24.67}{1011}\right)\tan(17) = 1.75$$

$$S_\gamma = 1 - .4\left(\frac{B}{L}\right) = 1 - .4\left(\frac{24.67}{1011}\right) = .02$$

$$k = D/B = \frac{6}{24.67} = .0024 \leq 1$$

$$d_c = 1 + .4(k) = 1$$

$$d_q = 1 + 2(k)\tan\phi(1 - \sin\phi)^2 = 1$$

$$d_\gamma = 1$$

$$\sigma'_{zD} = \gamma \cdot D - \gamma_w \cdot (D - D_w) = 125\cdot 6 - 62.43(6-3) = 562.8$$

$$q_{ult} = (1796\ lb/ft^2 \cdot 12.34 \cdot 1.94 \cdot 1) + 562.8 \cdot 4.77 \cdot 1.75 \cdot 1 + .5 \cdot 62.6 \cdot 24.67 \cdot 3.53 \cdot .02 \cdot 1$$

$$q_{ult} = 54{,}332\ lb/ft^2$$

$$q_a = \frac{q_{ult}}{FS} = \frac{54332}{2} = 27{,}166\ lb/ft^2$$

Used for Column sheer weight

Appendix N: Bearing Capacity Spreadsheet

Bearing Capacity, Capacity Spreadsheet

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date	February 17, 2023				
Identification	Example 6.4				
Input			**Results**		
Units of Measurement				Terzaghi	Vesic
	E	SI or E	Bearing Capacity		
			q ult =	132,680 lb/ft^2	154,110 lb/ft^2
Foundation Information			q a =	66,340 lb/ft^2	77,055 lb/ft^2
Shape	SQ	SQ, CI, CO, or RE			
B =	18	ft	Allowable Column Load		
L =		ft	P =	21,494 k	24,966 k
D =	6	ft			
Soil Information					
c =	2000	lb/ft^2			
phi =	31	deg			
gamma =	115	lb/ft^3			
Dw =	8	ft			
Factor of Safety					
F =	2				

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Unit conve	1000
Gamma w	62.4
phi (radiar	0.54105
Terzaghi Computations	
a theta =	3.50152
Nc =	40.41
Nq =	25.28
N gamma =	23.72
gamma' =	59.5333
coefficient	1.3
coefficient	0.4
sigma zD'	690
Vesic Computation	
Nc =	32.67
sc =	1.63
dc =	1.13
Nq =	20.63
sq =	1.60
dq =	1.09
N gamma =	25.99
s gamma =	0.60
d gamma =	1.00
B/L =	1
k =	0.33333
W sub f	0

Appendix O: Geotechnical Report Boring Data

NORTHING (ft): 2,960,073	EASTING (ft): 785,192
GROUND SURFACE EL. (ft): 12.12	DATE START/END: 5/31/2018 - 6/4/2018
VERT./HORIZ. DATUMS: NAVD 88/	DRILLING COMPANY: Geosearch Inc.

BORING GEI-102

PAGE 2 of 5

Elev. (ft)	Depth (ft)	Sample No.	Sample Depth (ft)	Pen./ Rec. (in)	Blows per 6 in. or RQD	Drilling Remarks/ Field Test Data	Layer Name	Soil and Rock Description
	25	S4	24 to 26	24/24	4-4-6-9		COHESIVE FILL	S4: SANDY LEAN CLAY (CL); ~70% high plasticity fines, ~30% fine sand, light brown.
	30	S5	29 to 31	24/24	1-0-3-3		COHESIVE FILL	S5: SANDY LEAN CLAY (CL); Similar to S4.
-20								
	35	S6	34 to 36	24/24	1-0-3-4		COHESIVE FILL	S6: SANDY LEAN CLAY (CL); Similar to S4.
	40	S7	39 to 41	24/2	1-2-4-4	4" casing to 39 ft.	CLAY	S7: LEAN CLAY (CL); ~100% high plasticity fines, gray.
-30								
	45	S8	44 to 46	24/24	1-2-5-3		CLAY	S8: LEAN CLAY (CL); Similar to S7, sandy lean clay seam.
	50	S9	49 to 51	24/24	WOH/12"-2-3		CLAY	S9: LEAN CLAY (CL); Similar to S7, organic odor.
-40								
	55							

NOTES:

PROJECT NAME: Terminal E Modernization

CITY/STATE: Boston, Massachusetts

GEI PROJECT NUMBER: 1702980

GEI Consultants

Appendix P: Connections Hand Calculations

Connections Hand Calculations, Page 1

W30x90 girder, $l = 30'$ 30x90 beam, $l = 50'$ beam spacing $= 10'$

trib area $= 750 ft^2$

$R_{DL} = 750 ft^2 \times \left(37.1 psf + \frac{90 plf}{10 ft} + 10 psf\right) + (90 plf \cdot 30 ft)/2 = 44925$ lb

$R_{LL} = 100 psf \times 750 ft^2 = 75{,}000$ lb

$R_u = 1.2(44925 \text{ lb}) + 1.6(75{,}000 \text{ lb}) = 173{,}910 \text{ lb} = 173.9^K$

Number A325N bolts:

$\phi R_n = \phi F_{nv} A_b$; $\phi = 0.75$ $F_{nv} = 54$ ksi

$A_b = \pi/4 (3/4")^2 = 0.4418 in^2$

$\phi R_n = 0.75(54 ksi)(0.4418 in^2) = 17.9^K/bolt$

$n = \frac{V_u}{\phi R_n} = \frac{173.9^K}{17.9^K/bolt} = 9.72$ Use 10 bolts

Bearing / Tearout at Each Bolt Hole:

$R_n = 1.2 L_c t F_u \leq 2.4 d_b t F_u$

$L_c = 3" - 1/2 (3/4" + 1/8") = 2.56"$

$1.2 L_c t F_u = 1.2(2.56")t F_u = 3.075 t F_u$ tearout

$2.4 d_b t F_u = 2.4(3/4")t F_u = 1.8 t F_u$ bearing

* bearing governs at each bolt hole

$\phi R_n = 10 \text{ bolts } \phi 2.4 d_b t_w F_u \geq 173.9^K$

$\phi R_n = 10(0.75)(2.4)(3/4")(0.470)(F_u = 65 ksi) = 412^K \geq 173.9^K$ OK

From table J3.4, for diameter $= 3/4"$, minimum edge distance $= 1"$

$\angle 3\frac{1}{2} \times 3\frac{1}{2}$

Determine angle thickness w/ 3 limit checks: bolt bearing / tearout, shear rupture, & shear yield

1) Bolt bering / tearout on angle leg

$\phi R_n = \phi(1.2 L_c t F_u) \leq \phi(2.4 d_b t F_u)$; $\phi = 0.75$

top 5 bolts: $L_{c1} = 3" - (3/4" + 1/16" + 1/16") = 2.13"$

bottom 5 bolts: $L_{c2} = 1" - 1/2(3/4" + 1/8") = 0.56"$

Tearout Capacities:

top 5: $\phi R_{n1} = (\phi = 0.75)(1.2)(2.13")tF_u = 1.92 t F_u$

bottom 5: $\phi R_{n2} = (\phi = 0.75)(1.2)(0.56")tF_u = 0.504 t F_u$

* Top 5 governed by bearing, bottom 5 by tearout

Bearout Capacity:

$\phi R_n = (\phi = 0.75)(2.4)(d_b = 3/4")tF_u = 1.35 t F_u$

Total Capacity $= [(5 \times 1.3 t F_u) + (5 \times 0.504 t F_u)] = 9.28 t F_u$

Required thickness; t: $9.28 t F_u \geq 173.9^k \rightarrow t \geq \frac{173.9^k}{(9.28)(58ksi)} = 0.323"$

2) Angle Shear Rupture

$\phi R_n = \phi(0.6 F_u)(L - nd_e)t = (0.75)(0.6 \times 58ksi)(20.25")t$

$L - nd_e = 29" - 10(3/4" + 1/8") = 20.25"$

$\downarrow$

$\phi R_n = 528.25\, t \geq 173.9^k$

$t \geq 0.329"$

3) Angle Shear Yield

$\phi R_n = \phi(0.6 F_y)Lt$; $\phi = 1.0$

$\phi R_n = (0.75)(0.6 \times 58ksi)(29")(t) = 626.4t \geq 173.9^k$

$t \geq 0.278"$

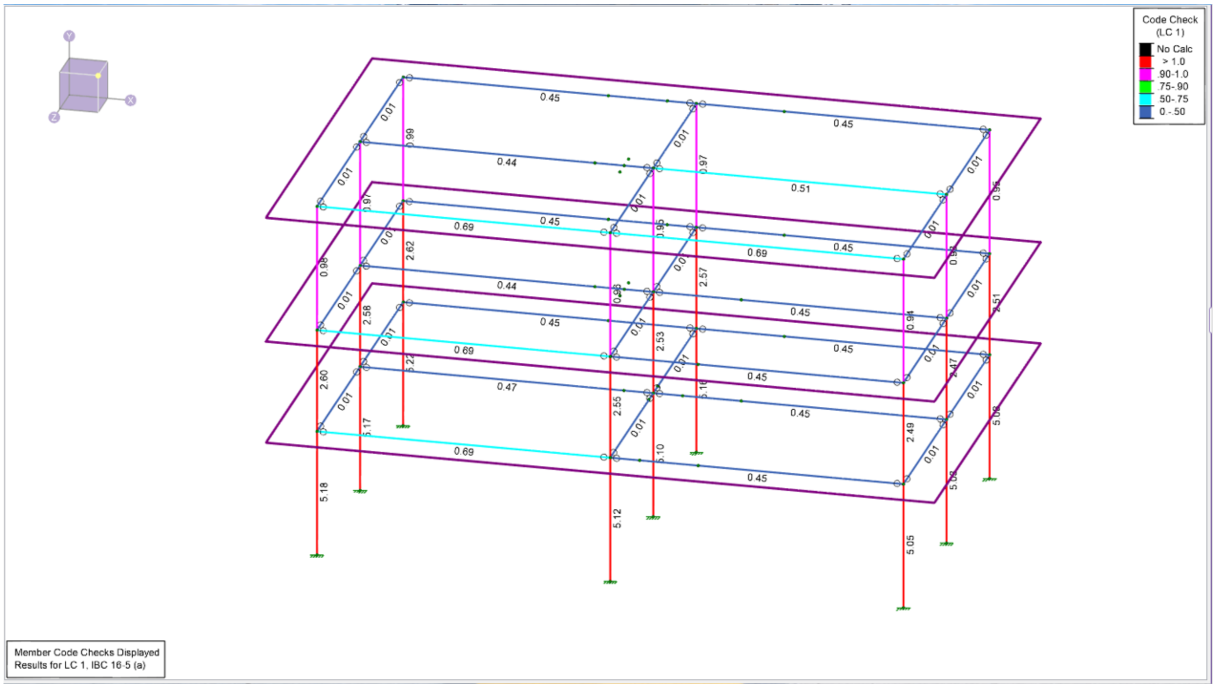
Appendix Q: Connections Spreadsheet

Connections, Excel Spreadsheet

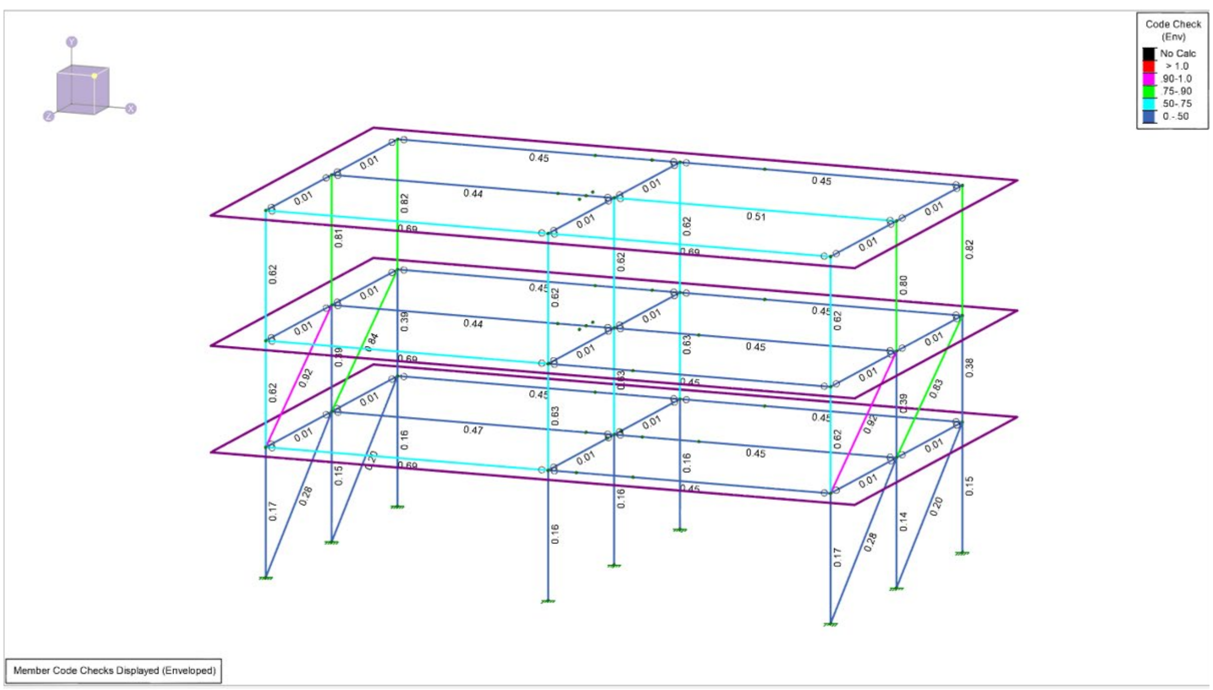
composite slab	39.1	psf		Tributary area	750	sq ft						
MEP	10	psf		Length girder	30	ft						
Live load	100	psf		girder weight	90	plf						
				beam weight	90	plf	beam spacing	10	ft			
Rdl	44925	lb					**Determine Angle Thickness w/ 4 limit states**					
Rll	75000	lb					1) Bolt Bearing on Angle Leg					
Ru	173910	lb	173.91	kips			Lc1	2.125	in	Fu (strut ai	58	ksi
							Lc2	0.5625	in			
Minimum # Bolts							Tearout Capac	1.9125	1.91tFu			
phi	0.75						Tearout Capac	0.50625	0.5tFu			
Fnv	54	ksi	(for A325-N)				Bearing Capac	1.35	1.35tFu		Governed by bearing	
diameter bolt	0.75	in	Area of bolt	0.441786467			Total Capacity	9.28125	tFu >	173.91	kips	
φRn	17.89235	kips/bolt					t	0.323065	in			
n	9.719795	10	Use 9 bolts									
							2) Angle Shear Rupture					
Bearing/Tearout Strength at each bolt hole							L	29				
Lc	2.5625	in		tw (girder)	0.47	in	L-nde	20.25	in			
Tearout	3.075	3.075tFu		Fu	65	ksi	ϕRn	528.525	t			
Bearing	1.8	1.8tFu	Bearing governs				t	0.329048	in			
ϕRn	412.425	kips	>	173.91		OK						
							3) Angle Shear Yield					
							ϕ	1		Fy	36	ksi
							ϕRn	626.4				
							t	0.277634	in			

Appendix R: RISA Results

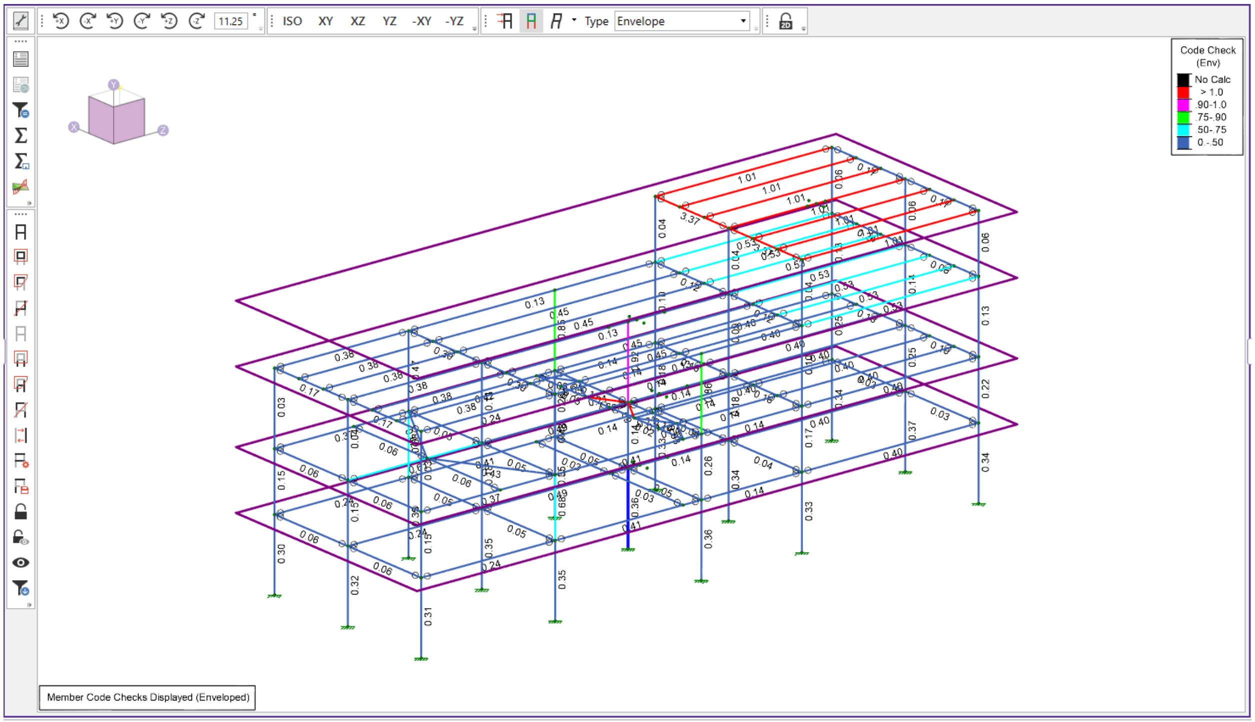
RISA Model Pier Initial Condition



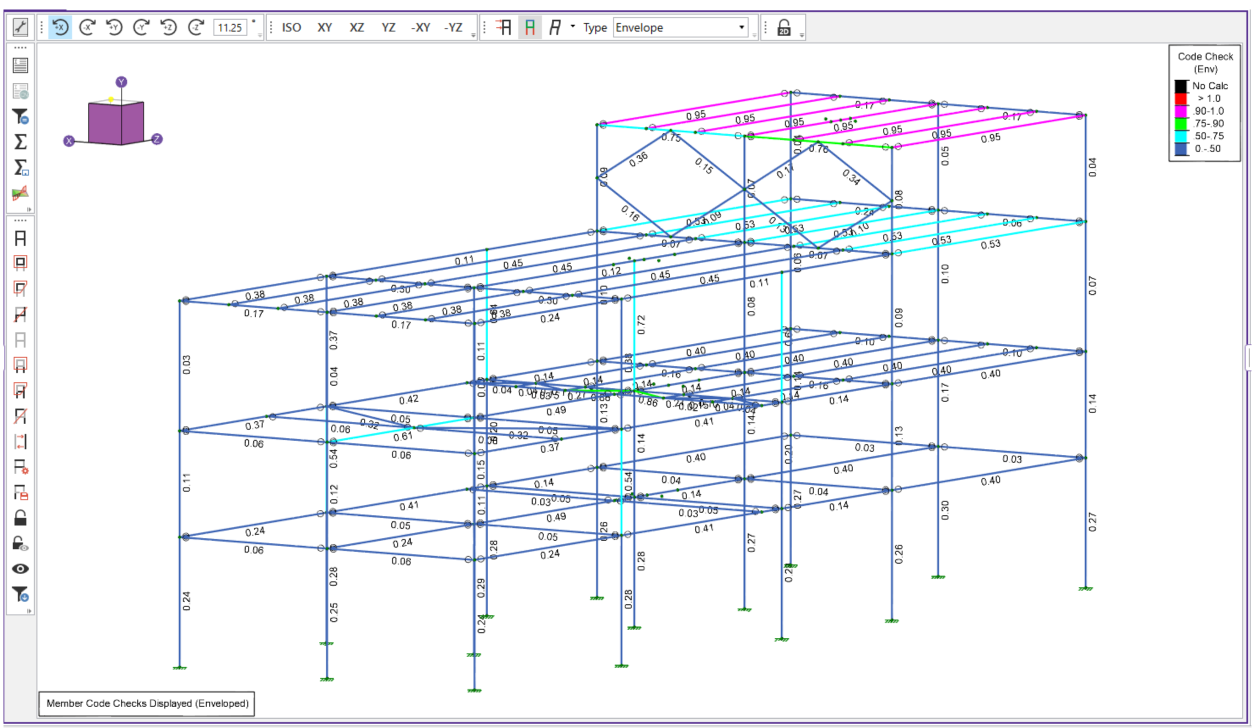
RISA Model Pier Final Bracing



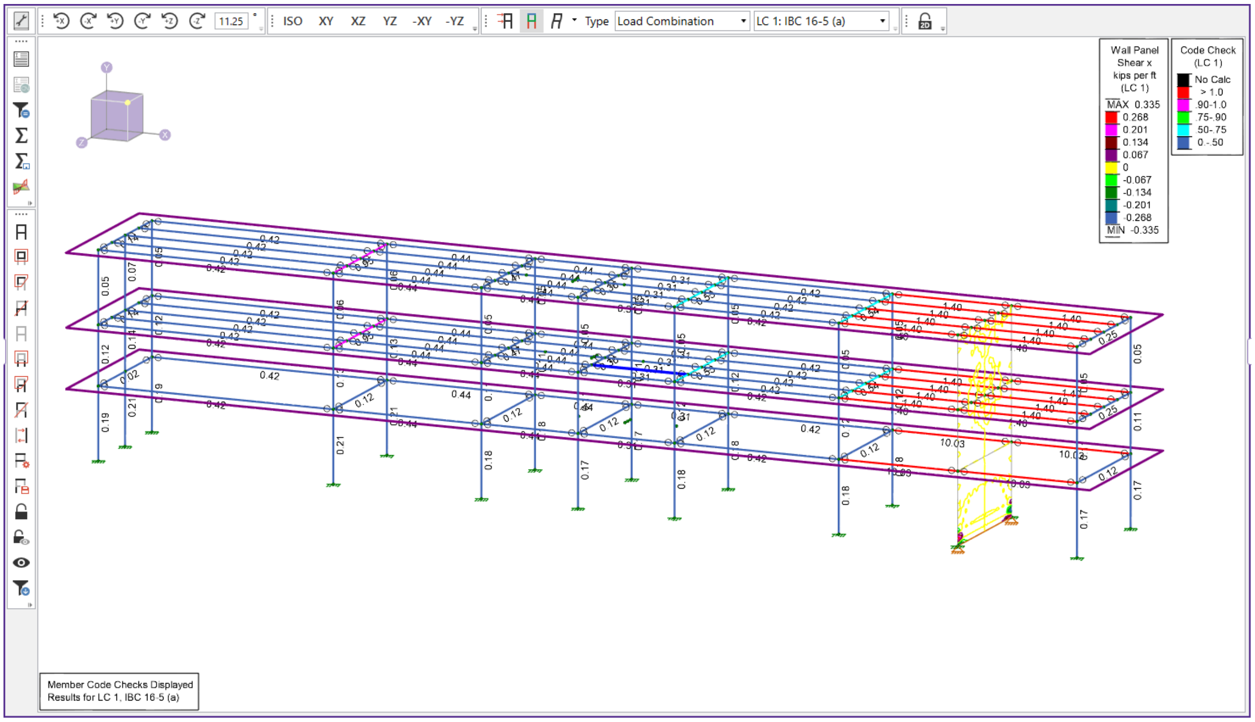
RISA Model Crescent Initial Condition



RISA Model Crescent Final Bracing



RISA Model Main Rectangle Initial Condition



RISA Model Main Rectangle Final Bracing