SENIOR THESIS: FINAL REPORT

The New York City Bus Depot

New York, NY

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New York City, NY

Kaitlyn Triebl | Structural Option

Project Team

Engineers / Architect:
STV Incorporated
Construction Manager:
Silverite Construction Company
Owner / Occupant:
New York City Transit Authority

Project Information

Function:
Bus Depot and Heavy Repair Shop
Building Size:
390,000 SF; 3 stories
Delivery Method:
Design-Build
Estimated Cost:
$150 million
Dates of Construction:
Summer 2011—Spring 2012

Architectural

- Space for fleet of 150 busses
- Refueling, Servicing, Fare Collection, Bus Washing, Maintenance, and Parking Stations
- Offices for Employees on Mezzanine Level
- Corrugated Metal and Brick Veneer
- Neighborhood Artwork Displayed

Structural

- Steel Framed Buildings
- Moment Frames in N-S Plane
- Braced Frames in E-W Plane
- Concrete Slabs with Sacrificial Metal Decking
- Pile Foundation

Lighting / Electrical

- Lit with Industrial Fluorescent Lights and LED Fixtures
- Utilizes Daylighting in Design with south-facing windows and Brise Soleil
- Main Powerboards and Switchboards 480Y/277 Volt Systems; Secondary Panels 208Y/120 Volt Systems
- Emergency Generator Powered

Mechanical

- 30 Custom Roof Mounted Indirect Fire Heat Recovery HV Units with Air Inlets and Outlets for equipment
- VAV Boxes Serve Areas in Separate zones
- Door Air Curtains used at Entrances (Exterior and Hazmat Storage)
- 3 Boilers

For more information, please visit: http://www.engr.psu.edu/ae/thesis/

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EXECUTIVE SUMMARY:

The senior thesis final report represents a full academic year of study on a particular building. This study includes analyses on the building's existing systems and proposals for new systems to better the constructability, efficiency, and functionality of the building. This report presents a redesign of the New York City Bus Depot's lateral system. This redesign replaces the moment frames of the original bus depot design with buckling restrained braced frames for the purpose of making construction more efficient both in terms of cost and schedule. The redesign also serves to solve deflection problems on the third floor mezzanine and high roof as well as to alleviate the lateral loads transferred to the foundation in the weak soils of the site.

The New York City Bus Depot is a three story building divided into three separate structures. The third structure, C, contains a third floor mezzanine and a high roof structure. This mezzanine level contains office space, and, in its original design, is not currently connected to a lateral system frame in the East-West direction. Instead, East-West lateral forces are resisted only by posts continuous from the third floor to the high roof. This causes large deflections under design conditions which can be supplemented by the vibrations of the busses and other large vehicles below. For these reasons, the laterally braced frames along column lines 1 and 5 are moved east to span between column lines S.1 and U and connect both the roof and high roof sections for consistent lateral control.

This shift in frames results in a need for reevaluating the exterior architecture of the building. The move of the lateral frames allows for an additional set of windows to be placed between column lines Q and R on the north and south facades. Analysis shows, however, that daylighting levels are already satisfactory for a majority of the day; the addition of a window, small relative to the length of the wall in the bus parking area, does not add any measurable amount of light to the space.

A study of construction impacts is also conducted, as there are changes in materials and methods. This change from moment frames to buckling restrained braced frames, which leads to the increased frame stiffness, lowers the cost for the lateral systems of the building by 8%. By decreasing the complexity of the connections, the lateral frame erection time is shortened, aiding in the value engineering of the design.

The following pages present a summary of the original design and the newly proposed design of the New York City Bus Depot. The methods for properly completing this redesign are outlined within the report and include the codes, standards, references, and analysis programs utilized. The redesign relies heavily on the 2006 international Building Code, 2010 Yew York State Building Code, Ram Structural System, and SAP 2000.

The goals of this redesign are met, as is stated in the outcome of each section of the report and visible through the numerous calculations and analyses present. This design represents an innovative way to lower the forces transferred into the site class E soils of the building site while economizing the design by decreasing construction costs and schedule durations.

BUILDING INTRODUCTION & EXISTING CONDITIONS:

The New York City Bus Depot is a new design-build project that broke ground in June of 2011. This $150 million project is slated for completion in the summer of 2012. The building site can be seen below in Figure 1 highlighted in red. It is in an area that is currently zoned to be commercial specifically for heavy automotive repair shops that are used for community purposes. The region where this building is to be located was once the place of a river that ran through this part of the city. For this reason, the water table on the site is high and the soil is liquefiable. There is also a portion of the site where there is no solid rock creating a need for piles to be driven down as deep as 150 feet.

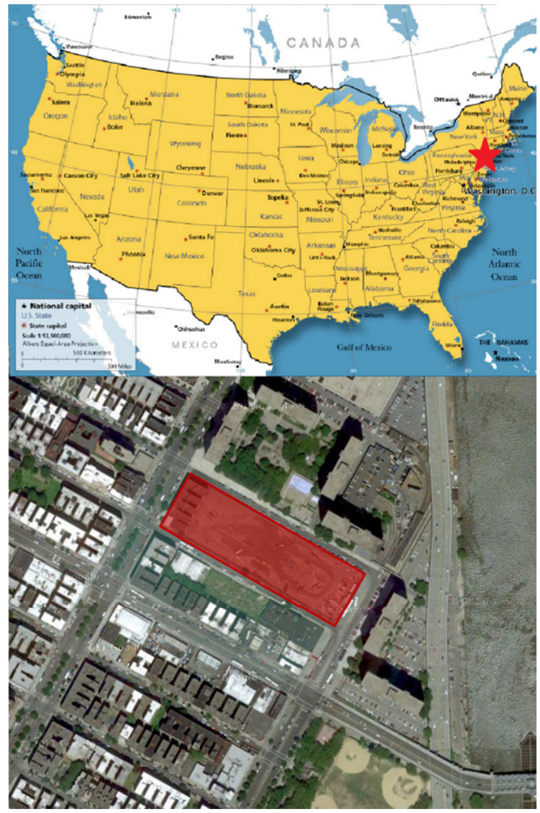


Figure 1: Aerial view of the building site highlighted in red. (*Image courtesy of Google Maps).*

The New York City Bus Depot is on a plot of land that is being reused. It was once a former trolley barn in the 1800s and, prior to the most recent demolition, an out-of-date, undersized bus depot that needed expansion for use by the New York City Transit Authority. This new and more environmentally friendly 390,000 square foot bus station will contain facilities for a fleet of 150 busses. The depot will be three stories tall, with each story at an approximate height of 25 feet. On the first floor, facilities will be available for bus refueling, servicing, fare collection, bus washing, and maintenance. The second and third floors will house parking for each of the 150 busses stationed out of the depot. Included in the space will also be offices for employees stationed at the bus depot.

Figure 2: Rendering of the New York City Bus Depot showing its south face and both the corrugated metal and brick veneer facades. *(Image courtesy of STV Inc.)*

Externally, this new facility has a modern appearance with a corrugated metal and brick veneer anchored onto CMU walls as seen in Figure 2. Large, rectangular expanses of windows with aluminum frames help to provide well lit spaces while using minimal electric lighting. The brise soleil that line the tops of the windows on the East façade to control the sunlight entering the building, helping to achieve the most energy efficient performance possible. To pay homage to the vibrant culture of the neighborhood in which the depot is located, artwork will be placed at street level for any passer-by to see. All of these features will help give life to an area of the borough looking to be renewed and revitalized.

In order to be an environmentally friendly facility, the New York City Bus Depot plans to employ green technologies. Two major highlights for this are located on top of the building: a green roof and a white roof. This green roof will help to minimize carbon dioxide emissions (particularly important for such a crowded borough of the city), and the white roof will help to regulate heat gain for the building. Other technologies to be included in the building are a rain water collection system, low emission boilers, heat recovery units, water efficient fixtures, recycled materials, and day-light centered lighting design. In addition to a rain water collection system, a water reclamation system is planned to recycle the water used in bus washing facility. All of these features aim to lead the New York City Bus Depot to a LEED certification upon completion of construction.

Structurally, this building is one which is steel framed. It has unique floor framing due to the multitudes of point loads applied from busses and their towing counterparts. Floors on levels two and three are also ramped like an over-sized parking garage for this bus fleet. Unique loading patterns are also created due to the busses as well as the mixed use occupancy of the building. At the present time, the building is at a 65% submittal stage with its contract documents and more information will be provided as updates are received.

Additional information on existing systems within the building can be found in the abstract on page one of this report and expanded upon on the senior thesis e-portfolio website in building statistics one and two. See the abstract for the exact web address.

EXISTING STRUCTURAL OVERVIEW

The New York City Bus Depot is a three story, 80' tall building that rests on piles grouped together with caps scattered throughout the site. The piles are deep due to the site class E classification that indicates the chance for liquefaction of the soil. The building itself can be treated as three separate buildings, as shown in Figure 3, due to the large expansion gaps that separate the framing systems of the building. The first floor consists of a heavily reinforced slab that is 14" to 18" thick for travel by heavy busses and towing vehicles. The framing system consists of heavy steel beams that are designed to resist the loads caused by the traveling busses. On top of each level of this steel framing sits a 6" reinforced concrete slab. This slab is supported by 2" 18 gage metal deck, however this deck is considered as sacrificial and all designs are calculated as though there is simply a concrete deck sitting upon the steel beams.

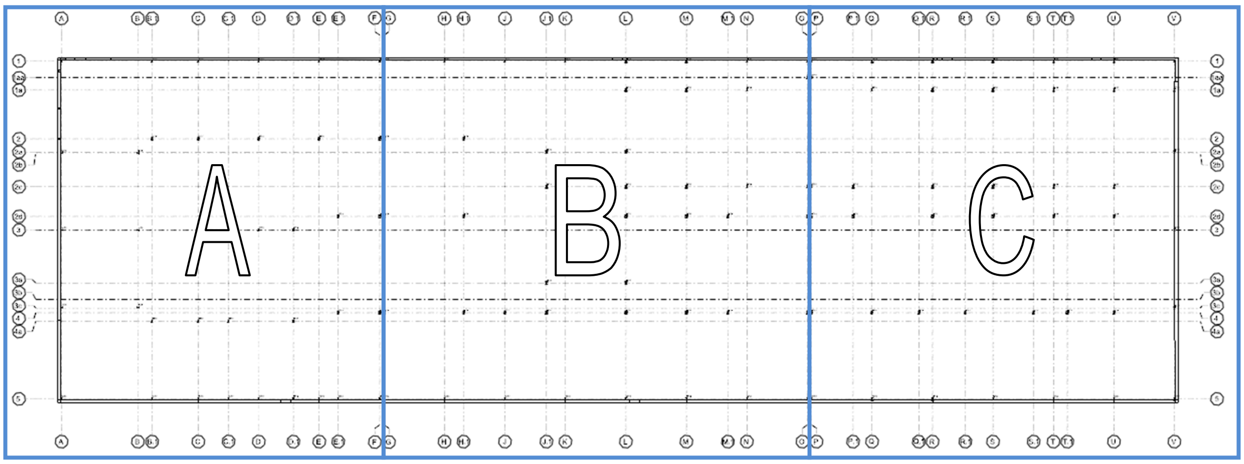


Figure 3: Depiction of the -6" Expansions joints that separate the structure into three distinct structural systems as denoted by the blue boxes. *(Image courtesy of STV Inc.)*

FOUNDATIONS:

The New York City bus depot requires the use of deep pile foundations due to the site's soil conditions. The site contains layers of organic material that compress under long-term loading, making the site unsuitable to maintain a shallow foundation. Another reason for the pile foundation lies in the liquefaction potential of the soils. Those below the water table, which is about 8' below the site surface, consist of a stratum of sand and a stratum of silt and clay all over weathered rock and bedrock. When tested, it was deemed that these would likely not liquefy during a strong earthquake, but there were some local areas that showed liquefaction potential if the 2500-year event were to occur in the city.

The piles recommended for the site are steel HP12x102 piles that possess the ability to maintain 220 tons (or a service load of 200 tons after subtracting 20 tons of downdrag). These piles are used to support the ground floor structural slabs, columns, and heavy equipment requiring extra reinforcing. They terminate at an elevation 107'-6" above sea level. These piles are required to be driven down to bedrock, which is between 35' and 100' below grade depending on the area of the site. The piles must be hammered into the ground and have a final driving resistance no less than 5 blows per quarter inch of penetration. Also, because of the low pH of the ground water, corrosion effects must be taken into consideration. Due to the effects of this, the piles are to be analyzed for strength at a size 1/8" thinner in the webs and flanges than prescribed. In addition to being able to maintain 200 tons of compression, the piles are to withstand a lateral load of 5.5kips for a single pile and 3.8kips for each pile when analyzed in groups in the pile caps.

Floor Systems:

Two flooring systems are considered in the New York City Bus Depot. On the first floor, there is a slab on grade with a thickness still to be determined. This thickness is to be between 14" and 18" due to the heavy, concentrated loads imposed by the various busses and maintenance vehicles utilizing the facility and the long spans of the slab between piles.

The typical framed flooring system on the second floor, third floor, and third floor mezzanine consists of steel beams and girders supporting a 6" one-way concrete slab on a 2" gage sacrificial composite form deck. This slab on deck is to be reinforced with a rebar layout that yet to be determined on the design drawings. Analysis presented later in this report yields a theoretical value for this reinforcing. The span of this deck is also yet to be determined since the reinforcement has also yet to be determined.

What controls the design of the thickness of the slab is not the distributed load, but instead the point loads induced by the buses. Worst case loadings of the tires of the busses are treated as 4.5"x4.5" squares with the applied point loads dictated in the dead load section of this report. This 4.5"x4.5" square is used in the evaluation of punching shear, which controls the thickness of the slab.

Various beam sizes are used in construction of this structure because of the varying spans, many of which are much longer than the conventional 30 feet bays. Smaller spans under 30'-0" are generally made up of inlay beams of W14s, W16s, and W18s. Larger spans are made of W 24s, W27s, and W30s. Examples of these spans include W27x84s that span 49'-10" and W30x99s that span 55'-6". Girders utilized on these floors include W30s, W33s, W40s, and W44s.

On the west end of the building, ramps are utilized to lead busses to the parking areas on the second and third floors. These are also steel framed with same metal decking described as typical on other areas of the floor. They utilize W24x76s that span the following: 45'-0" on the North and South ends of the ramp and 44'-2" on the West end.

Framing System

The rest of the framing system of the New York City Bus Depot consists of steel columns. They are all W14s with the exception of one W15x655 in a moment frame that supports 1001kips of service dead load and 573kips of service live load. The columns can be expected to support rather large axial loads due to the heavy imposed loads seen in appendix B and the heavy materials.

LATERAL SYSTEM

The lateral system for this building consists of two types of frames: braced and moment. Braced frames flank the interior runs of the ramps on the west side of the building and also run east to west on the exterior lines between column lines O and P as shown in blue on Figure 4. The moment frames are those which run north and south. They are located at column lines F, H.1, J.1, L, M, P.1, Q.1, S, T, U, and V respectively as shown in Figure 4 in orange.

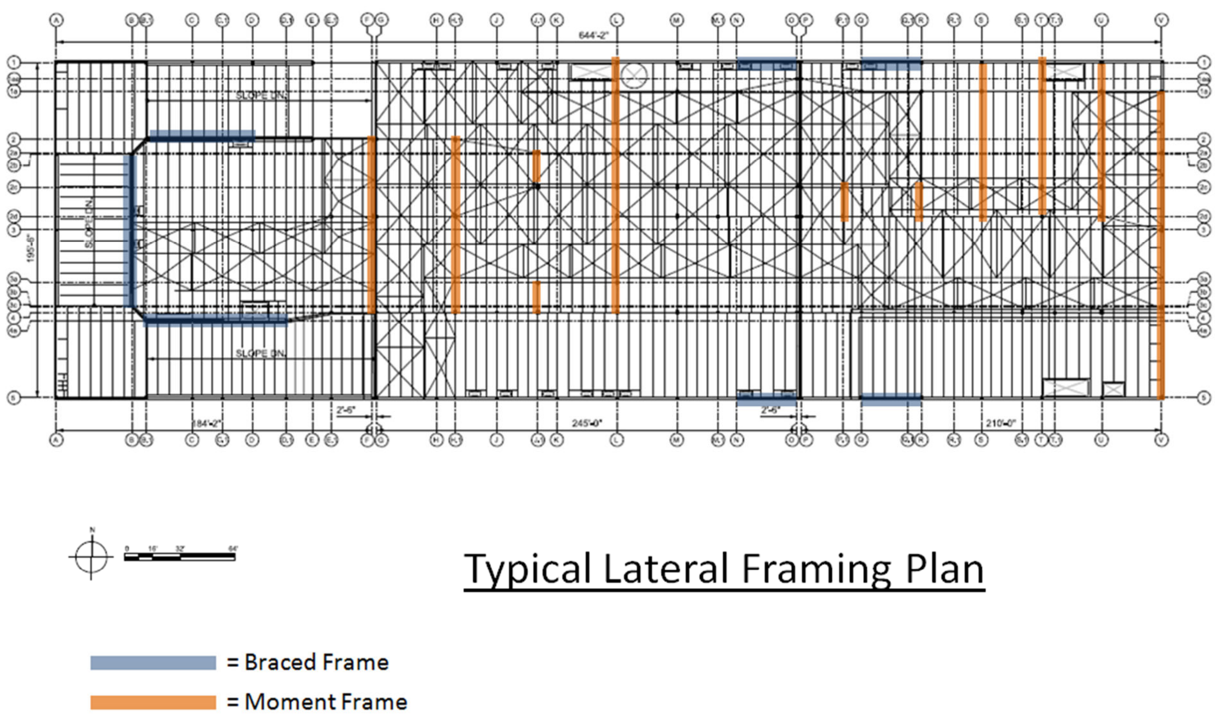


Figure 4: Locations of Moment and Braced Frames. *(Image courtesy of STV Inc.)*

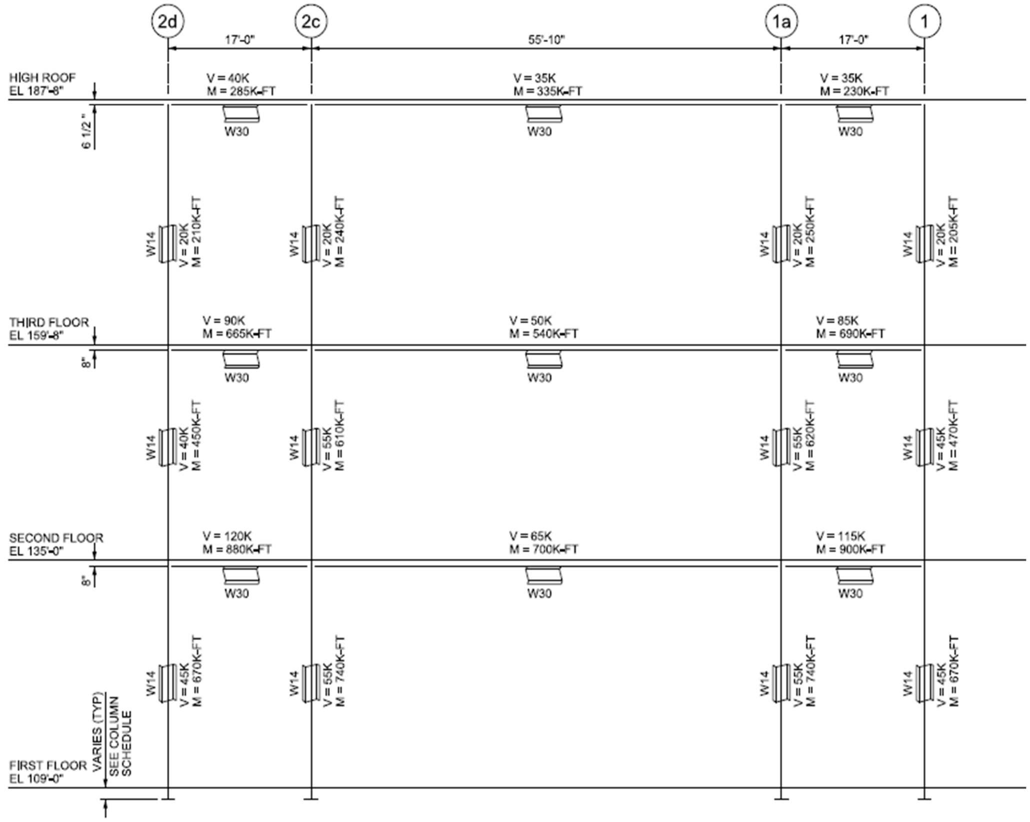


Figure 5: Typical moment frame construction *(Image courtesy of STV Inc.)*

The moment frames are constructed of W14 columns and W30 beams assembled such that the controlling seismic loads may be resisted. The moment frames are required to resist service loads ranging from shears of 5kips along the first floor columns of the frame running along F, to 455kips on the second floor beam along column line V between columns 5 and 3c. These must also resist moments of 1895kip-ft along column line V to 65kip-ft in first-floor column 2F. A typical construction of a moment frame is shown in Figure 5.

The braced frames are constructed of W14 columns of significant weight with W12 members that act as bracing. The diagonal lines that can be seen in Figure 6 show the ramp in the garage. This location, on the west end of the bus depot, is most heavily reinforced with these braced frames due to the vibrations that the walls will have to handle from the traveling busses.

With the exception of one frame, all of the braced frames run from east to west. It is easy to use the braded frames on the west end of the building because there will be no interference with architectural features on the façade there. Windows are in place in the bus parking and office areas to the east, but not in the location of the ramp. Also, on the interior, where these are located will not interfere with bus travel lanes: a key component to the functionality of the bus depot.

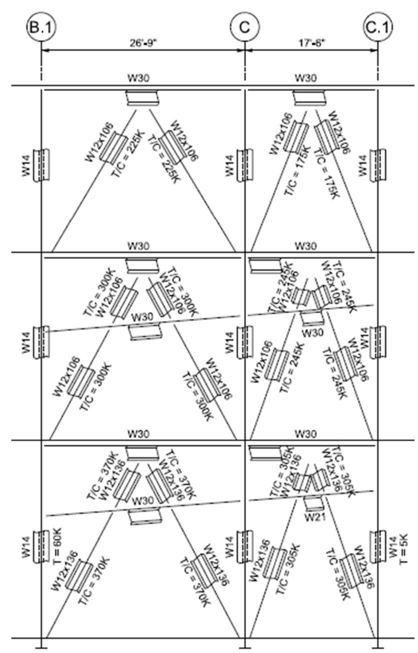


Figure 6: Typical braced frame construction. *(Image courtesy of STV Inc.)*

ROOF SYSTEMS

The roof of the building is framed similarly to the floors below with respect to size and bay spacing. Certain bays, particularly those above the ramp, utilize smaller W21s because they do not need to be concerned with carrying the weight of the busses. Overall, the roof maintains a similar beam sizing because significant weight is still expected to be carried by the system. The roof will be supporting a green roof as well as a series of air handlers stationed along the north and south edges of the roof.

The decking on the roof will consist of a 4 ½" concrete covering on a 2" 18 gage cold form metal deck. Reinforcement and span for the roof deck/slab system is yet to be determined at this stage of the project.

It should also be noted that the roof has two levels to it. The main roof consists of a diaphragm at 72' and a parapet extending up to 80". The 69' swath of the roof furthest east is actually a bulkhead above the 3rd floor mezzanine where the office space is located. This tops off at a level of 93.' This high level is used in computing wind loads so that the highest factor of safety is considered. See the Wind Load section for more details and Appendix B for calculations.

DESIGN CODES

- 2010 Building code of New York State
  - Adopts 2006 Family of Codes (IBC, IRC, IFC, IMC, IPC, IFGC, IPMC, IEBC) and 2009 IECC
- North American Specifications for the Design of Cold Formed Structural Steel Members "AISI-NASPEC" (Metal Decking)
- 2008 New York City Building Code (Foundations)
- AISC Manual of Steel Construction – Allowable Stress Design, Thirteenth Edition
- Structural Welding Code – Steel (AWS D.1 – Modified by AISC Section J2)
- Details and Detailing of Concrete Reinforcement ACI 315
- Building Code Requirements for Structural Concrete ACI 318-08
- 2008 Building Code Requirements for Masonry Structures (ACI 530-08/ASCE 5-08/ TMS 402-08)
- Specifications for Masonry Structures (ACI 530.1-08/ASCE 6-08/TMS 602-08)

MATERIALS USED

Table 1: Material properties

Material Properties		
Material		**Strength**
Steel	**Grade**	**fy = ksi**
Wide Flange Shapes	**A992**	**50**
Hollow Structural Shapes	A500, GR. B	46
Plates	A572	50
Pipe Shapes	A53, GR. B	46
Anchor Rods	F1554	36
Sag Rods	A36	36
Welding Electrodes	E70XX	70
Welding Electrodes (Gr. 65)	E80XX	80
Steel Reinforcement	A615	60
Bolts (3/4”-1” dia.)	A325	N/A
Bolts (1-1/8” dia)	A490	N/A
Deck	Gage	
2” Form Galvanized Metal	18	
Concrete	Weight (pcf)	f’c = psi
Formed Slabs	150	5,000
Structural SOG	150	5,000
Slabs on Metal Deck	150	5,000
Foundations	150	5,000
Masonry	Grade	fy = ksi
Concrete Masonry Units	C90	1.9
Mortar	C270, Type M	N/A

GRAVITY LOADS:

DEAD AND LIVE LOADS:

The dead and live load distributions on the floors and roof can be seen in the plans in Appendix B. Tables 2 and 3 respectively compare the dead and live loads utilized in the design with those outlined in the New York State Building Code (2010 Edition):

Table 2: Dead loads and floor weight

Floor	Distributed Floor Dead Load (psf)	Area (ft^2)	Col. Wt (lb)	Exterior Façade (lb)	Weight per floor (k):
Floor 1	200	125902	502.5	1047696	25682.9
Floor 2	100	125902	922.3	1934208	13512.5
Floor 3	100	125902	622.2	1450656	13212.4
Floor 3 (Mezz)	100	13489.5	30	1128288	1378.95
Roof	100	112412.5	189.9	1128288	11431.15
High Roof	100	13489.5	18.4	564144	1367.35

In the New York State Building Code, dead loads are dictated to be the actual weight of construction materials. No superimposed loads are suggested in the code, but in this project, they are included. The distributed floor dead load in the chart above does not include these superimposed values. This includes the slab weight and a 15psf beam allowance. Added to this, for total construction weight per floor, is the weight of the columns per floor, and the weight of the exterior façade, which is assumed to be 48psf. The additional superimposed dead loads are 10psf for the first floor; 35psf for the second floor, third floor, and third floor mezzanine; and 95psf for the roves for miscellaneous permanent and semi-permanent equipment such as the air handlers on the roof, maintenance equipment on the first floor, and office materials on the third floor mezzanine.

Table 3: Live loads analyzed vs. prescribed

Floor	Function	Assigned Live Load (psf)	NYS Code 2010 Perscribed LL (psf)	Notes
Floor 1	Maintenance	250	50	See Chart: Concentrated Loads
	Storage	300	250	
Floor 2	Bus Parking	175	50	See Chart: Concentrated Loads
	Future Shop	250	250	
	Office	150	50	Compact, Versitile
	Vault	600	250	Undisclosed Use
Floor 3	Bus Parking	100	50	See Chart: Concentrated Loads
	Office	150	50	Compact, Versitile
Floor 3 (Mezz)	Office	150	50	Compact, Versitile
Roof	Roof	30	100	Green Roof

The live loads prescribed in the design documents (seen in appendix B) for the New York City Bus Depot are generally close to those dictated in the 2010 New York State Building Code. The reason for some of the larger discrepancies is due to the unique occupancy of the structure. Live loads for bus and truck parking garages are generally defined in linearly distributed loads along lanes and concentrated loads.

Table 4 states the New York State Building Code's minimums for bus and truck parking facilities, and Table 5 depicts the concentrated loads expected for the facility by the design engineers. These values are show in tables 3, 4, and 5 respectively

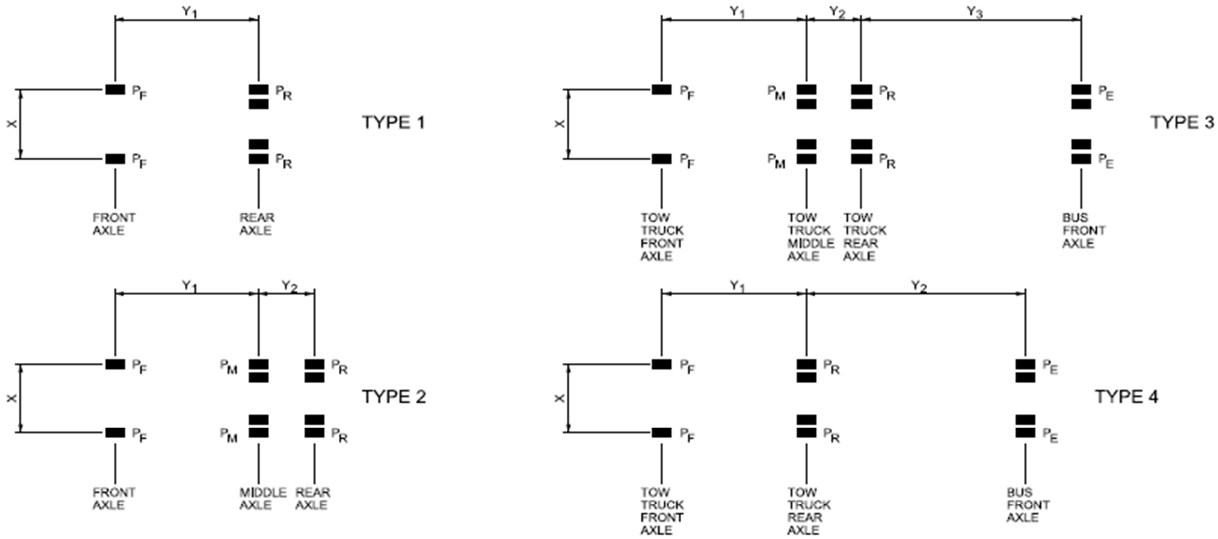
Table 4: Required uniform and concentrated loads for parking structures

(Table courtesy of 2010 New York State Building Code)

LOADING CLASS[a]	UNIFORM LOAD (pounds/linear foot of lane)	CONCENTRATED LOAD (pounds)[b]	
		For moment design	For shear design
H20-44 and HS20-44	640	18,000	26,000
H15-44 and HS15-44	480	13,500	19,500

a. An H loading class designates a two-axle truck with a semitrailer. An HS loading class designates a tractor truck with a semitrailer. The numbers following the letter classification indicate the gross weight in tons of the standard truck and the year the loadings were instituted.

b. See Section 1607.6.1 for the loading of multiple spans.



CONCENTRATED WHEEL LOAD DIAGRAMS

NOTE: THERE ARE SLIGHT VARIATIONS IN LOAD FOR P_F, P_M, P_R AND P_E. HOWEVER DESIGN IS BASED ON THE HIGHEST VALUE.

CONCENTRATED WHEEL LOAD TABLE

VEHICLE	TYPE	LOCATION	X	Y_1 (DIMENSIONS IN FEET)	Y_2	Y_3	P_F	P_M (LOADS IN KIPS)	P_R	P_E
STANDARD HS20 TRUCK	2	1st, 2nd, 3rd	6.0	14.0	14.0	-	4.0	16.0	16.0	-
MCI 2915 BUS	2	1st, 2nd, 3rd	6.67	26.5	4.0	-	5.7	8.9	4.8	-
ORION HYBRID "NEW GEN" 3877 BUS	1	1st, 2nd, 3rd	6.17	23.83	-	-	6.0	-	11.35	-
VAN HOOL DOUBLE DECKER BUS TD 925	2	1st, 2nd, 3rd	7.17	16.58	4.25	-	5.72	8.91	5.72	-
TOW TRUCK E050-08	2	1st, 2nd, 3rd	7.0	21.67	4.0	-	9.75	8.5	6.6	-
TOW TRUCK E052-03	1	1st	7.0	23.0	-	-	9.9	-	9.1	-
TOW TRUCK E050-08 LIFTING MCI 2915 BUS	3	1st, 2nd, 3rd	7.0	21.67	4.0	45.33	5.57	15.44	15.07	10.08
TOW TRUCK E052-03 LIFTING MCI 2915 BUS	4	1st	7.0	23.0	45.33	-	5.88	-	23.94	10.08
TOW TRUCK E050-08 LIFTING ORION 3877 BUS	3	1st, 2nd, 3rd	7.0	21.67	4.0	39.67	4.31	15.59	15.65	7.77
TOW TRUCK E052-03 LIFTING ORION 3877 BUS	4	1st	7.0	23.0	39.67	-	4.73	-	26.57	7.77
TOW TRUCK E050-08 LIFTING DOUBLE DECKER BUS TD 925	3	1st, 2nd, 3rd	7.0	21.67	4.0	35.67	5.57	15.87	15.45	10.08
TOW TRUCK E052-03 LIFTING DOUBLE DECKER BUS TD 925	4	1st	7.0	23.0	35.67	-	5.88	-	24.69	10.08
OPEN TOP CONTAINER TRUCK	2	1st, 2nd	7.0	9.0	4.5	-	13.0	13.5	13.5	-

NOTE: WHEN TOWED BUS LOADS ARE APPLIED SIMULTANEOUSLY WITH OTHER WHEEL LOADS ON A COMMON MEMBER, TOWED BUS WHEEL LOADS ARE REDUCED BY 25%. SIMULTANEOUS VEHICLE LOADS HAVE BEEN ANALYZED PER THE STALL LAYOUT SHOWN ON THE ARCHITECTURAL DRAWINGS. COMBINATIONS OF VEHICLE TYPES WERE PLACED IN EACH STALL GROUP IN SUCH A WAY TO PRODUCE THE WORST CASE LOADING FOR THE MEMBER BEING STUDIED.

Figure 7: Concentrated wheel loads and values, with corresponding wheel load diagrams *(Image courtesy of STV Inc.)*

Snow Loads

Snow Loads depicted in Table 6 for the New York City Bus Depot are minimal. It is assumed they are included in the distributed Live loads where applicable so no additional calculations were necessary for them. The chart on the right is a display of the design criteria for the snow loading.

Table 5: Snow design criteria *(Information courtesy of STV Inc.)*

SNOW DESIGN CRITERIA
SNOW IMPORTANCE FACTOR 1ST 1.0
OCCUPANCY CATEGORY: I
GROUND SNOW LOAD: 25 PSF
EXPOSURE FACTOR: CS=0.90
THERMAL FACTOR: C1=1.00
FLAT ROOF SNOW LOAD: 15, 75 PSF
SNOW DRIFT LOAD: INCLUDED WHERE APPLICABLE

Lateral Loads:

Wind Loads:

Table 6: Wind design criteria

Design Criteria	
Importance Factor (I):	1.0
Occupancy Category:	II
Exposure:	C
Basic Wind Speed (V):	100 mph
Directionality Factor (kd):	1
Topographic Factor (kzt):	1.0
Gust Factor (G):	0.85 (rigid)

Wind loads were calculated to be lower than those provided in the drawings. Not all values were given. Those assumed included topographic factor and GC_{pi} (assumed +/- 0.18 for an enclosed system). Table 7, to the left, is a table of the design criteria used in the analysis. Figures 7, 8, 9, and 10 in this section show the achieved values through calculations shown in Technical Report 2. The values received show that wind is not the controlling factor in the lateral system, but instead seismic forces are.

Wind Pressures N-S Direction										
Type	Floor	Elevation (ft)	k_z (interpolated)	Velocity Pressure (psf)	Cp	Wind Pressure (psf):	Internal Pressure		Net Pressure	
							$+GC_{pi}$	$-GC_{pi}$	$+GC_{pi}$	$-GC_{pi}$
Windward Walls	1st	0	0.85	21.76	0.8	14.80	5.76	-5.76	20.56	9.04
	2nd	26	0.91	23.30	0.8	15.84	5.76	-5.76	21.60	10.08
	3rd	51	1.10	28.16	0.8	19.15	5.76	-5.76	24.91	13.39
	3rd (Mezz)	65	1.15	29.44	0.8	20.02	5.76	-5.76	25.78	14.26
	Roof	79	1.21	30.98	0.8	21.06	5.76	-5.76	26.82	15.30
	Parapet	84	1.22	31.23	0.8	21.24	5.76	-5.76	27.00	15.48
	Bulkhead	93	1.25	32.00	0.8	21.76	5.76	-5.76	27.52	16.00
Leeward Walls	All	All	1.25	32.00	-0.5	-13.60	5.76	-5.76	-7.84	-19.36
Side Walls	All	All	1.25	32.00	-0.7	-19.04	5.76	-5.76	-13.28	-24.80
Roof	N/A	0 to 46.5	1.25	32.00	-0.9	-24.48	5.76	-5.76	-18.72	-30.24
	N/A	46.5 to 93	1.25	32.00	-0.9	-24.48	5.76	-5.76	-18.72	-30.24
	N/A	93 to 186	1.25	32.00	-0.5	-13.60	5.76	-5.76	-7.84	-19.36
	N/A	>186	1.25	32.00	-0.3	-8.16	5.76	-5.76	-2.40	-13.92

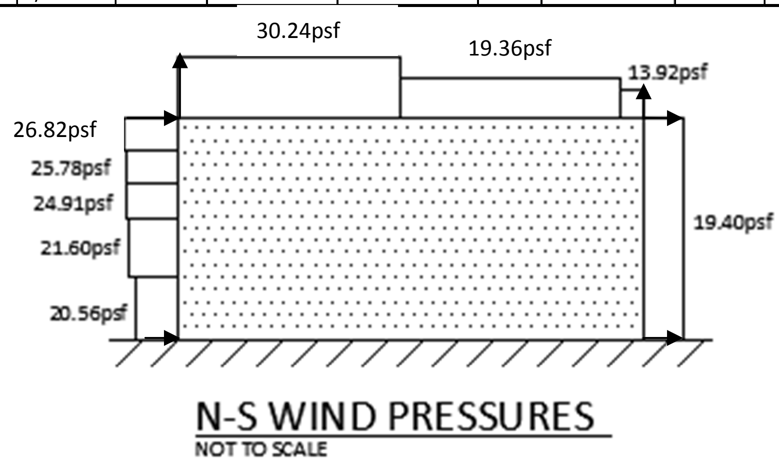


Figure 8: Table stating north-south wind pressures and diagram showing them applied

Wind Forces N-S								
Floor	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (K)	Overturning Moment (k.ft)
		Height (ft)	Area (ft2)	Height (ft)	Area (ft2)			
1st	0	0.0	0.0	13.0	8372.0	172.10	1437.63	0.00
2nd	26	13.0	8372.0	12.5	8050.0	354.74	1265.53	4611.57
3rd	51	12.5	8050.0	7.0	4508.0	312.80	910.80	3910.06
3rd (Mezz)	65	7.0	4508.0	7.0	4508.0	232.43	597.99	1626.98
Roof	79	7.0	4508.0	2.5	1610.0	164.11	365.57	1148.75
Parapet	84	2.5	1610.0	4.5	2898.0	121.71	201.46	304.26
Bulkhead	93	4.5	2898.0	0.0	0.0	79.75	79.75	358.89
							Total Base Shear:	1437.63
							Total Overturning Moment:	133699.95

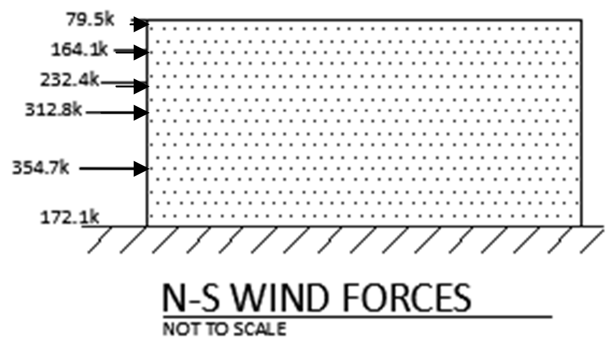


Figure 6: Table stating north-south wind forces and diagram showing them applied.

Wind Pressures E-W Direction										
Type	Floor	Elevation (ft)	k_z (interpolated)	Velocity Pressure (psf)	Cp	Wind Pressure (psf):	Internal Pressure		Net Pressure	
							$+GC_{pi}$	$-GC_{pi}$	$+GC_{pi}$	$-GC_{pi}$
Windward Walls	1st	0	0.85	21.76	0.8	14.80	5.76	-5.76	20.56	9.04
	2nd	26	0.91	23.30	0.8	15.84	5.76	-5.76	21.60	10.08
	3rd	51	1.10	28.16	0.8	19.15	5.76	-5.76	24.91	13.39
	3rd (Mezz)	65	1.15	29.44	0.8	20.02	5.76	-5.76	25.78	14.26
	Roof	79	1.21	30.98	0.8	21.06	5.76	-5.76	26.82	15.30
	Parapet	84	1.22	31.23	0.8	21.24	5.76	-5.76	27.00	15.48
	Bulkhead	93	1.25	32.00	0.8	21.76	5.76	-5.76	27.52	16.00
Leeward Walls	All	All	1.25	32.00	-0.3	-7.34	5.76	-5.76	-1.58	-13.10
Side Walls	All	All	1.25	32.00	-0.7	-19.04	5.76	-5.76	-13.28	-24.80
Roof	N/A	0 to 46.5	1.25	32.00	-0.9	-24.48	5.76	-5.76	-18.72	-30.24
	N/A	46.5 to 93	1.25	32.00	-0.9	-24.48	5.76	-5.76	-18.72	-30.24
	N/A	93 to 186	1.25	32.00	-0.5	-13.60	5.76	-5.76	-7.84	-19.36
	N/A	>186	1.25	32.00	-0.3	-8.16	5.76	-5.76	-2.40	-13.92

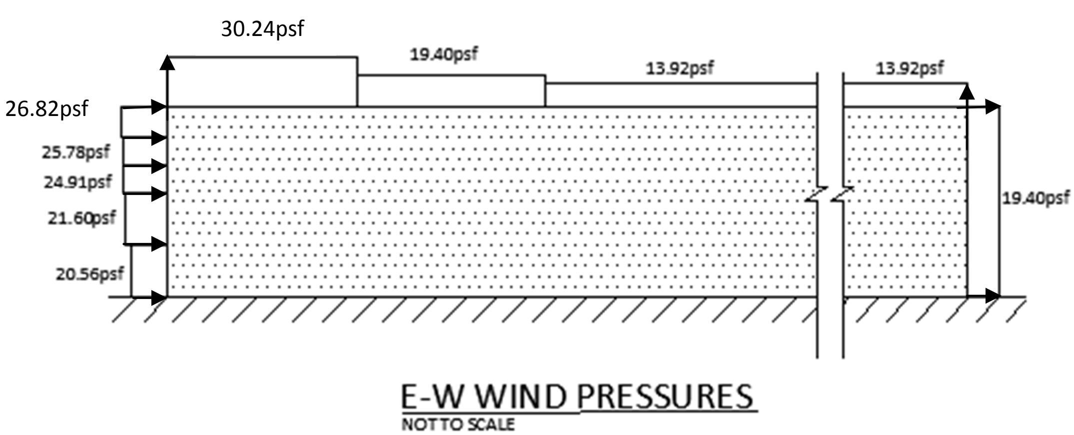


Figure 9: Table stating east-west wind pressures and diagram showing them applied.

Wind Forces E-W								
Floor	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (K)	Overturning Moment (k.ft)
		Height (ft)	Area (ft2)	Height (ft)	Area (ft2)			
1st	0	0.0	0.0	13.0	2541.5	52.25	436.42	0.00
2nd	26	13.0	2541.5	12.5	2443.8	107.69	384.18	1399.94
3rd	51	12.5	2443.8	7.0	1368.5	94.96	276.49	1186.98
3rd (Mezz)	65	7.0	1368.5	7.0	1368.5	70.56	181.53	493.90
Roof	79	7.0	1368.5	2.5	488.8	49.82	110.98	348.73
Parapet	84	2.5	488.8	4.5	879.8	36.95	61.16	92.37
Bulkhead	93	4.5	879.8	0.0	0.0	24.21	24.21	108.95
							Total Base Shear:	436.42
							Total Overturning Moment:	40587.48

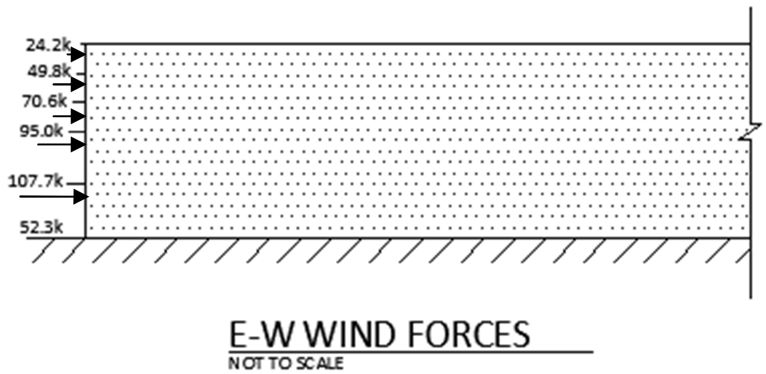


Figure 10: Table stating east-west wind forces and diagram showing them applied.

SEISMIC LOADS:

The following series of charts present in Figures 11, 12, and 13 show a summary of the results of the seismic analysis of the New York City Bus Depot. There are three sets of results for the three buildings that were analyzed separately due to the 6" expansion joint separating them. For the 65% submittal drawings that have been the guide so far, the building was analyzed as one entity, but here, the building is further divided for greater accuracy in consideration of the expansion joints. There are discrepancies between the computer model and the hand calculation shown in the Appendix of Technical Report 3. This is likely due to simplifications made for hand calculations that were not made for the RAM Structural System model.

For further detail on the calculations, see Technical Report 3.

Building A:

Building Dimensions:		
N-S	195.5	ft
E-W:	184.167	ft
Mezz/High F	68	ft
Beam Allow	15	psf

Base Shears		
Direction	C_s	V (k)
(NS)	0.05	1130.33
(EW)	0.053	1198.14

N-S Seismic Analysis									
Floor	Distributed Floor Dead Load (psf)	Area (ft^2)	Elevation (ft):	Weight (k):	$w_x h_x^k$	C_{vx}	NS Story Force Fx(k)=CvxV	NS Story Shear (k)	NS Overturning Moment (k-ft)
Floor 1	200.00	36004.65	0.00	7703.43	0.00	0.00	0.00	1130.33	0.00
Floor 2	100.00	36004.65	26.00	4522.76	117591.89	0.22	252.04	1130.33	29388.45
Floor 3	100.00	36004.65	51.00	4222.66	215355.91	0.41	461.58	878.28	44792.52
Roof	100.00	22710.65	79.00	2460.96	194416.22	0.37	416.70	416.70	32919.44
								Total Overturning Moment:	237679.90

E-W Seismic Analysis									
Floor	Distributed Floor Dead Load (psf)	Area (ft^2)	Elevation (ft):	Weight (k):	$w_x h_x^k$	C_{vx}	EW Story Force	EW Story Shear (k)	EW Overturning
Floor 1	200	36004.65	0	7703.43	0.00	0.00	0.00	1198.14	0.00
Floor 2	100	36004.65	26	4522.76	210011.28	0.20	235.87	1198.14	6132.61
Floor 3	100	36004.65	51	4222.66	433614.98	0.41	487.01	962.27	24837.28
Roof	100	22710.65	79	2460.96	423165.37	0.40	475.27	475.27	37546.28
								Total Overturning Moment:	68516.17

Figure 11: Building A Seismic Analysis

Building B:

Building Dimensions:		
N-S	195.5	ft
E-W:	210	ft
Mezz/High Roof (EW):	68	ft
Beam Allowance:	15	psf

Base Shears		
Direction	C_s	V (k)
(NS)	0.05	1404.457377
(EW)	0.053	1488.724819

N-S Seismic Analysis									
Floor	Distributed Floor Dead Load (psf)	Area (ft^2)	Elevation (ft):	Weight (k):	$w_x h_x^k$	C_{vx}	NS Story Force	NS Story Shear (k)	NS Overturning Moment (k-ft)
Floor 1	200	41055	0	8713.5	0	0.00	0.00	1404.46	0.00
Floor 2	100	41055	26	5027.8	130722.8	0.16	223.94	1404.46	36515.89
Floor 3	100	41055	51	4727.7	241112.7	0.29	413.04	1180.52	60206.63
Floor 3 (Mezz)	100	13294	65	1359.4	88361	0.11	151.37	767.48	49886.40
Roof	100	27761	79	2966	234314	0.29	401.39	616.12	48673.16
High Roof	100	13294	93	1347.8	125345.4	0.15	214.72	214.72	19969.28
								Total Overturning Moment:	237679.9002

E-W Seismic Analysis									
Floor	Distributed Floor Dead Load (psf)	Area (ft^2)	Elevation (ft):	Weight (k):	$w_x h_x^k$	C_{vx}	EW Story Force	EW Story Shear (k)	EW Overturning
Floor 1	200	41055	0	8713.5	0.00	0.00	0.00	1488.72	0.00
Floor 2	100	41055	26	5027.8	233462.21	0.14	204.98	1488.72	5329.52
Floor 3	100	41055	51	4727.7	485475.79	0.29	426.25	1283.74	21738.80
Floor 3 (Mezz)	100	13294	65	1359.4	185762.99	0.11	163.10	857.49	10601.58
Roof	100	27761	79	2966	510006.68	0.30	447.79	694.39	35375.36
High Roof	100	13294	93	1347.8	280865.48	0.17	246.60	246.60	22933.97
								Total Overturning Moment:	95979.22462

Figure 12: Building B Seismic Analysis

Building C:

Building Dimensions:		
N-S	195.5	ft
E-W:	210	ft
Mezz/High Roof (EW):	68	ft
Beam Allowance:	15	psf

Base Shears		
Direction	C_s	V (k)
(NS)	0.05	1404.46
(EW)	0.053	1488.72

N-S Seismic Analysis									
Floor	Distributed Floor Dead Load (psf)	Area (ft^2)	Elevation (ft):	Weight (k):	$w_x h_x^k$	C_{vx}	NS Story Force Fx(k)=CvxV	NS Story Shear (k)	NS Overturning Moment (k-ft)
Floor 1	200	41055	0	8713.5	0	0.00	0.00	1404.46	0.00
Floor 2	100	41055	26	5027.8	130722.8	0.16	223.94	1404.46	36515.89
Floor 3	100	41055	51	4727.7	241112.7	0.29	413.04	1180.52	60206.63
Floor 3 (Mezz)	100	13294	65	1359.4	88361	0.11	151.37	767.48	49886.40
Roof	100	27761	79	2966	234314	0.29	401.39	616.12	48673.16
High Roof	100	13294	93	1347.8	125345.4	0.15	214.72	214.72	19969.28
								Total Overturning Moment:	237679.90

E-W Seismic Analysis									
Floor	Distributed Floor Dead Load (psf)	Area (ft^2)	levation (ft	Weight (k):	$w_x h_x^k$	C_{vx}	EW Story Force	EW Story Shear (k)	EW Overturning
Floor 1	200	41055	0	8713.5	0.00	0.00	0.00	1488.72	0.00
Floor 2	100	41055	26	5027.8	233462.21	0.14	204.98	1488.72	5329.52
Floor 3	100	41055	51	4727.7	485475.79	0.29	426.25	1283.74	21738.80
Floor 3 (Mezz)	100	13294	65	1359.4	185762.99	0.11	163.10	857.49	10601.58
Roof	100	27761	79	2966	510006.68	0.30	447.79	694.39	35375.36
High Roof	100	13294	93	1347.8	280865.48	0.17	246.60	246.60	22933.97
								Total Overturning Moment:	95979.22

Figure 13: Building C Seismic Analysis

PROBLEM STATEMENT:

The New York City Bus Depot is comprised of a steel framing system that utilizes moment resisting braced frames in the north-south direction and concentrically braced frames in the east-west direction. The twelve moment frames of the system cover long spans and require costly connections. Though this system is deemed acceptable as noted in technical report three, there are more efficient and cost effective ways to design the lateral system.

The current lateral system provides little to no support to the third floor mezzanine, which has a drift significantly greater than that of the floors below. This is a problem due to the posts attached to the third floor that serve as the sole lateral force resisting elements for the third floor mezzanine and high roof levels of the structure.

PROPOSED SOLUTION:

The proposed solution for the New York City Bus Depot is to redesign the current lateral system to decrease the cost of connections and the number of frames. In this analysis, a system of bucking restrained braced frames is compared to the system of moment frames for the three separate buildings. Drift will be closely examined, particularly for the third floor mezzanine, which has a story drift above the acceptable amount for non-structural damage according to analysis in technical report three. The braced frames should be able to create narrower, more efficient frames due to their increased stiffness.

The buckling restrained braced frame connections require significantly less welding than the moment connections, which should decrease costs for both materials and labor. In addition to this impact in the construction phase of the project, the steel erection process should be expedited, potentially helping to shorten the critical path of the schedule. For these reasons, a construction management breadth will be thoroughly studied as a part of this thesis.

The east-west oriented braced frames in Building C are moved to help stabilize the third floor mezzanine and high roof. For this reason, the relocation of the braced frames will be assessed and analyzed both structurally and architecturally. The movement of the braced frames on the North and South façades will have an impact on the window placement, which will facilitate a need for a façade daylighting study. For this reason, a daylighting breadth will be studied for the bus depot.

A study on a bracing method, unusual in New York City and on the rest of the east coast, is conducted. The use of buckling-restrained braced frames (BRBFs) is utilized in place of the traditional steel bracing method. The scheme is compared to the moment frames in terms of economy, labor intensity, and effectiveness in resisting lateral forces.

The alternate design of the New York City Bus Depot is examined utilizing the following codes and standards (a complete list and bibliography can be seen at the end of this report):

- IBC 2006
- 2010 New York State Code
- ASCE 7-10

- AISC Steel Manual (13th edition)
- SEAOC Buckling Restrained Brace Design Recommendations
- Star Seismic™ Design guides

In addition to these resources, computer-aided design programs are utilized. These include, but are not limited to:

- RAM Structural System
- SAP 2000
- Microsoft Excel
- RS Means CostWorks

Goals:

The goals for each of the design studies carried out in this report are as follows:

Structural Solution: Lateral System Redesign

◊ Create a bracing scheme to replace the moment frame scheme
◊ Allow functional bus flow throughout the depot
◊ Utilize buckling restrained braced frames to lower seismic forces on the structures
◊ Control Drift of the 3rd Floor Mezzanine and High Roof
◊ Decrease lateral loads on the building to help with design on poor soils

Construction Solution: Effects of Lateral System Redesign

◊ Decrease the construction time required for making connections and assembling frames
◊ Decrease the cost of lateral system erection
◊ Decrease the skilled laborers necessary on site

Daylighting Solution: Effects of Lateral System Redesign

◊ Examine the effects of an additional window on the southern building façade
◊ Obtain maximum available daylighting values outdoors
◊ Obtain light levels indoors
◊ Determine adequacy of interior lighting in parking area

STRUCTURAL DEPTH:

PROBLEM

The moment frames used in the current New York City Bus Depot provide open bays for bus travel, but they are inefficient for distributing the controlling seismic loads through the buildings. The twelve moment frames of the system cover long spans and require costly connections. Such connections require large lengths of intensive welding over large beam and column sections installed by highly skilled laborers. Though this moment frame lateral load resisting system was deemed acceptable as noted in technical report three, there may be more efficient and cost effective ways to design the lateral system.

The current lateral system also provides little to no support to the third floor mezzanine, which has a drift significantly greater than that of the floors below. According to analysis in Technical Report 3, the drifts are greater than those permitted by code. This problem is likely attributable to the posts attached to the third floor that serve as the sole lateral force resisting elements for the third floor mezzanine and high roof levels of the structure.

PROPOSED SOLUTION/EXPECTED OUTCOME

In order to decrease the number of lateral frames necessary to resist the controlling earthquake forces on the site of the New York City Bus Depot, a buckling restrained brace system is designed. This new bracing technique is to sufficiently decrease the base shear and overturning moment experienced by the structure due to the lower ductility of the design. With a design executed that does not interfere with the bus flow throughout the depot, there is the potential to make for a more efficient and cost effective solution than previously noted..

Buckling restrained braces are a fairly new technology that has been codified within the past ten to fifteen years. The braces are composite members that have the ability to yield in compression and tension. This allows them to efficiently and predictably dissipate energy more effectively than typical steel bracing members. Figure 14 below shows sample test results of the hysteretic performance of a buckling restrained braced frame:

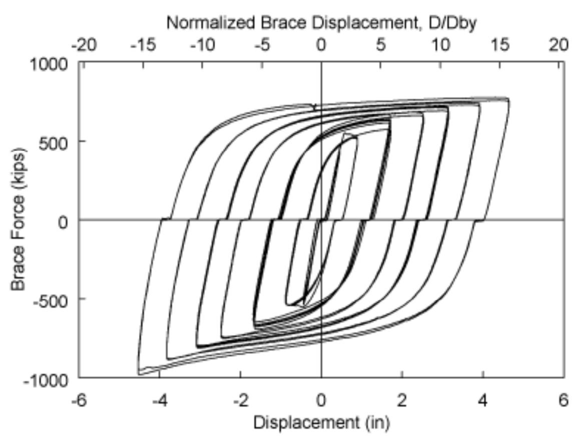


Figure 14: Hysteretic Analysis of a Buckling Restrained Braced System (image courtesy of Coffman Engineers)

Buckling restrained brace members are made of two major components: a yielding steel core and an outer shell filled with concrete. The steel core is the primary lateral load resisting component of the system. The outer shell and concrete casing serve to stabilize the inner steel core and prevent it from buckling, though to insure the two components act separately, it is not bonded to the inner steel core. In this way, the ductility benefits of the steel are insured without the concerns of unequal strains in compression and tension, such as the formation of a plastic hinge after buckling. Below, a section of the POWERCAT™ buckling restrained brace is shown in Figure 15:

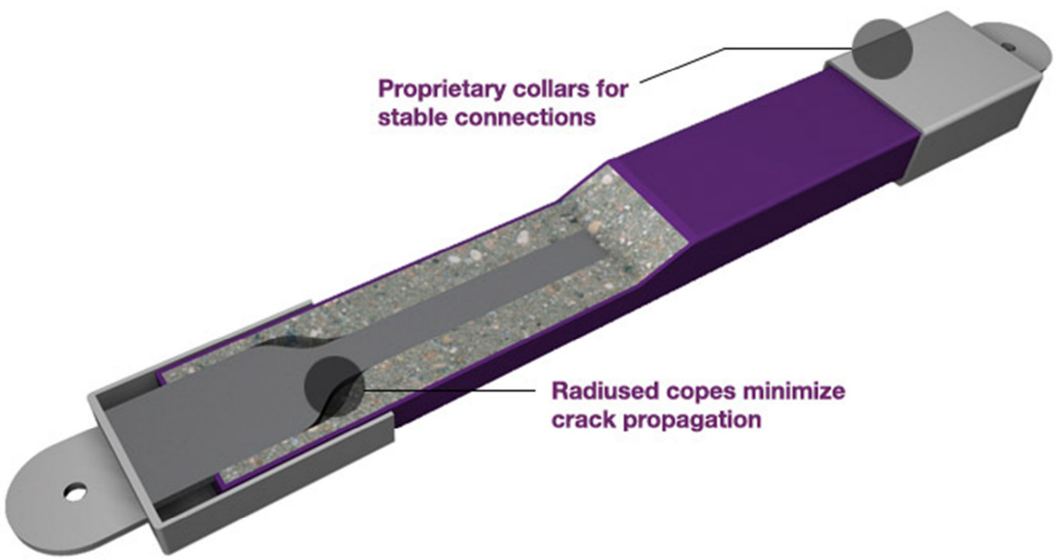


Figure 15: Star Seismic's POWERCAT™ buckling restrained brace featuring true pinned connections (Image courtesy of Star Seismic).

Buckling restrained braced frames are becoming increasingly popular for high seismic activity regions, such as those found in California and Japan where most buckling restrained braced frames are implemented. New York City does not have as much of a seismic concern as these areas, so many may be skeptical of the implementation of such a bracing scheme on the east coast. According to A. Christopher Cerino, a structural engineer at STV Incorporated, these bracing schemes do have benefits on the east coast.

For one, the buckling restrained braced frames are very useful for buildings on sites with weak soils, such as the New York City Bus Depot, which is in Site Class E. The buckling restrained braced frames greatly reduce the lateral forces imposed on the building and carried by the foundation. In addition to this, the detailing is simpler and construction methods are more efficient. These benefits are seen on another project in the vicinity of the New York City Bus Depot. A note of caution that the braces are best implemented by experienced design teams. STV, Incorporated is a large design firm with locations throughout the country, including an office in Southern California. The experience of an office that frequently designs for seismic concernts makes it easier to coordinate with the brace manufacturer, who must be included in the design process from very early on. The braces can be a difficult item to sell to contractors and designers who are not experienced with the system, but, as seen later in the report, their benefits can be valuable.

PROCESS

ELIMINATION OF INTERFERING BAYS

To begin the design of the new lateral system, the travel pattern for the buses needs to be assessed. The purpose of the open moment frames of the previous lateral system was to allow for large, open bays for the busses to easily travel through at least two buses wide. The bus travel path is highlighted in gray in figure 16 below, with the original moment frames highlighted in orange and the original braced frames highlighted in blue. The first floor is shown because it controls the bus path. The upper levels are open for parking and contain offices.

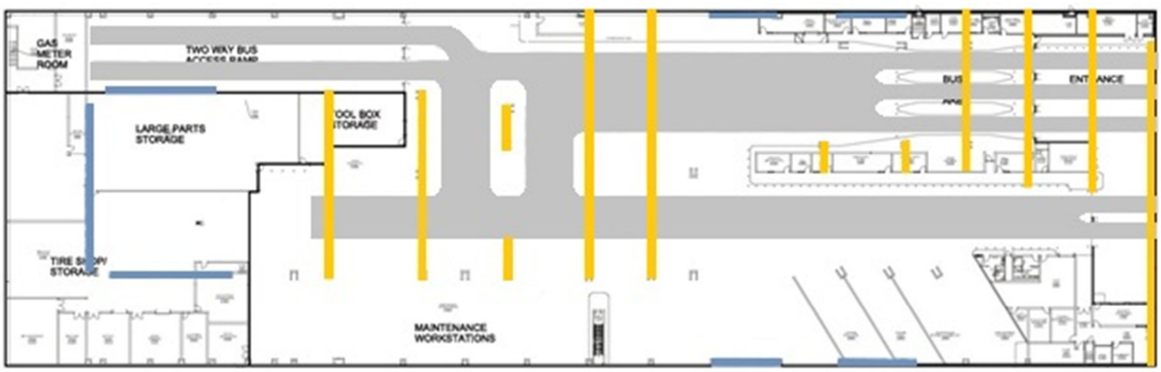


Figure 16: First floor plan (Image courtesy of STV, Incorporated) with bus path shown in grey, moment frames in orange, and braced frames in blue.

In order to create braced frames without interfering with the flow of the bus traffic, the bays depicted in figure 17 could not utilize any bracing schemes. These bays are shown in figure 17 below in red:

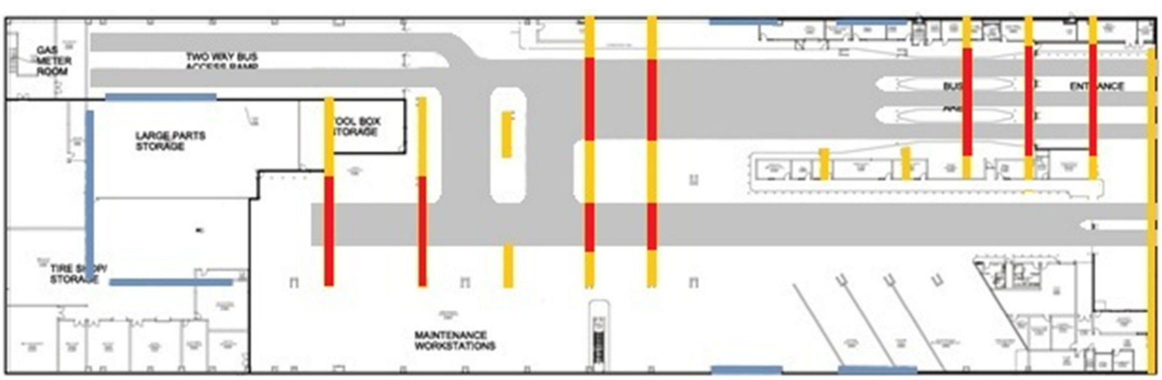


Figure 17: First floor plan (Image courtesy of STV, Incorporated) with bus path shown in grey, moment frames in orange, braced frames in blue, and unusable bays in red.

Comparison of Framing Types (SAP Models and Stiffness Analysis)

Once the bays available for cross-bracing are determined, the proper frame style is assessed. This is done by modeling various frame styles in SAP2000 for each of the available frames and open bays; bracing styles analyzed included cross braces, diagonal braces, chevron braces, and inverted braces (it was later noted that cross bracing could not be used in buckling restrained braced frames). Eccentric braces are not modeled because of the stiffness decrease with the increase in link length, making the concentric brace with a link length of zero the stiffest design. These bracing styles are analyzed for stiffness in comparison to the amount of bracing used in the connection. The results of the modeled frame styles are compared to the original moment frames for a rough understanding of how many braces are needed and what size of brace is needed. From this SAP2000 analysis, relative stiffness is determined utilizing arbitrary members; diagonal and chevron brace orientations are selected for all frames.

Once the most efficient brace configurations are selected, the overall stiffness for each orientation (north-south and east-west) is calculated. The stiffnesses are used, in relation to distance from the central point of the structure, as a comparison to the original system to determine whether or not it is logical for lateral force resisting frames to be removed. From this analysis, and the previous architectural analysis, braced frames H.1 and M in building B are removed, and frames S and V are removed from Building C. A sample of these stiffness comparisons can be seen in Appendix C of this report.

Recalculation of Lateral Forces

Once the frames, their locations, and their relative stiffnesses are determined, the lateral forces on the structures of the New York City Bus Depot are recalculated. The changes in the lateral forces and their distributions are a result of the change in ductility of the structure. The increase in the response modification factor is dramatic: from the original ordinary moment frame value of 3.5 to the buckling restrained brace frame with non moment beam-column connection of 7.0. This causes a significant reduction in the seismic response coefficient, but an increase in weight of the structure. In this case, the increased weight in the structure is negated by the decrease in column sizes due to the lower lateral loads. A sample of the decreased column size calculations can be seen in Appendix G for frames one and five of Building C. This brings the base shear and overturning moments far below the levels experienced with the ordinary moment frames as seen in table 8 below:

Table 8: Old and New Results of Seismic Loading

	Overturning Moment (kip.ft)				Base Shears (kips)			
	NS Frames		EW Frames		NS Frames		EW Frames	
	Old	New	Old	New	Old	New	Old	New
A	315391	51408	68516	31026	1130	543	1198	543
B	124674	58697	95979	35186	1404	608	1489	608
C	215251	103321	95979	43462	1404	674	1489	674

With a new base shear and overturning moment calculated, new story forces and story shears are obtained as seen in the tables in figures 18, 19, and 20 below:

Building Dimensions:	
N-S	195.5 ft
E-W:	184.167 ft
Mezz/H. Roof (EW):	68 ft
Beam Allowance:	15 psf

N-S Seismic Analysis									
Floor	Distributed Floor Dead Load (psf)	Area (ft^2)	Elevation (ft):	Weight (k):	$w_x h_x^k$	C_{vx}	NS Story Force Fx(k)=CvxV	NS Story Shear (k)	NS Overturning Moment (k-ft)
Floor 1	200.00	36004.65	0.00	7703.43	0.0	0.00	0.00	542.56	0.00
Floor 2	100.00	36004.65	26.00	4522.76	117591.9	0.22	120.98	542.56	14106.46
Floor 3	100.00	36004.65	51.00	4222.66	215355.9	0.41	221.56	421.58	21500.41
Roof	100.00	22710.65	79.00	2460.96	194416.2	0.37	200.02	200.02	15801.33
								Total Overturning Moment:	51408.20

E-W Seismic Analysis									
Floor	Distributed Floor Dead Load (psf)	Area (ft^2)	levation (ft	Weight (k):	$w_x h_x^k$	C_{vx}	EW Story Force	EW Story Shear (k)	EW Overturning
Floor 1	200	36004.65	0	7703.43	0.0	0.00	0.00	542.56	0.00
Floor 2	100	36004.65	26	4522.76	210011.3	0.20	106.81	542.56	2777.03
Floor 3	100	36004.65	51	4222.66	433615.0	0.41	220.53	435.75	11247.07
Roof	100	22710.65	79	2460.96	423165.4	0.40	215.22	215.22	17002.09
								Total Overturning Moment:	31026.19

Figure 18: Building A seismic analysis results for story forces, story shears, and overturning moments.

Building Dimensions:	
N-S	195.5 ft
E-W:	210 ft
Mezz/High Roof (EW):	68 ft
Beam Allowance:	15 psf

N-S Seismic Analysis									
Floor	Distributed Floor Dead Load (psf)	Area (ft^2)	Elevation (ft):	Weight (k):	$w_x h_x^k$	C_{vx}	NS Story Force	NS Story Shear (k)	NS Overturning Moment (k-ft)
Floor 1	200	41055	0	8713.5	0	0.00	0.00	608.01	0.00
Floor 2	100	41055	26	5027.8	130722.8	0.22	131.12	608.01	15808.13
Floor 3	100	41055	51	4727.7	241112.7	0.40	241.85	476.88	24320.99
Roof	100	27761	79	2966	234314	0.39	235.03	235.03	18567.47
								Total Overturning Moment:	58696.60

E-W Seismic Analysis									
Floor	Distributed Floor Dead Load (psf)	Area (ft^2)	Elevation (ft):	Weight (k):	$w_x h_x^k$	C_{vx}	EW Story Force	EW Story Shear (k)	EW Overturning
Floor 1	200	41055	0	8713.5	0.00	0.00	0.00	608.01	0.00
Floor 2	100	41055	26	5027.8	233462.21	0.19	115.50	608.01	3003.07
Floor 3	100	41055	51	4727.7	485475.79	0.40	240.18	492.50	12249.34
Roof	100	27761	79	2966	510006.68	0.41	252.32	252.32	19933.24
								Total Overturning Moment:	35185.6442

Figure 19: Building B seismic analysis results for story forces, story shears, and overturning moments.

Building Dimensions:	
N-S	195.5 ft
E-W:	210 ft
Mezz/High Roof (EW):	68 ft
Beam Allowance:	15 psf

N-S Seismic Analysis									
Floor	Distributed Floor Dead Load (psf)	Area (ft^2)	Elevation (ft):	Weight (k):	$w_x h_x^k$	C_{vx}	NS Story Force	NS Story Shear (k)	NS Overturning Moment (k-ft)
Floor 1	200	41055	0	8713.5	0	0.00	0.00	674.14	0.00
Floor 2	100	41055	26	5027.8	130722.8	0.16	107.49	674.14	17527.63
Floor 3	100	41055	51	4727.7	241112.7	0.29	198.26	566.65	28899.18
Floor 3 (Mezz)	100	13294	65	1359.4	88361	0.11	72.66	368.39	23945.47
Roof	100	27761	79	2966	234314	0.29	192.67	295.74	23363.12
High Roof	100	13294	93	1347.8	125345.4	0.15	103.07	103.07	9585.25
								Total Overturning Moment:	103320.65

E-W Seismic Analysis									
Floor	Distributed Floor Dead Load (psf)	Area (ft^2)	Elevation (ft):	Weight (k):	$w_x h_x^k$	C_{vx}	EW Story Force	EW Story Shear (k)	EW Overturning
Floor 1	200	41055	0	8713.5	0.00	0.00	0.00	674.14	0.00
Floor 2	100	41055	26	5027.8	233462.21	0.14	92.82	674.14	2413.37
Floor 3	100	41055	51	4727.7	485475.79	0.29	193.02	581.32	9843.99
Floor 3 (Mezz)	100	13294	65	1359.4	185762.99	0.11	73.86	388.30	4800.71
Roof	100	27761	79	2966	510006.68	0.30	202.77	314.44	16019.03
High Roof	100	13294	93	1347.8	280865.48	0.17	111.67	111.67	10385.19
								Total Overturning Moment:	43462.29

Figure 20: Building C seismic analysis results for story forces, story shears, and overturning moments.

Story forces are than taken and divided to each frame according to relative stiffness. This calculation can be seen in Appendix D.

Torsion Check

With new member sizes determined, the SAP models are updated to again confirm stiffness. This is necessary because, though relative stiffness remains similar, the actual stiffness values, which are necessary to determine the rigidities for torsion calculations, change. The values are fairly significant due to the asymmetrical nature of the bracing layout. Appendix E shows the locations of the centers of rigidity in relation to the centers of mass for each floor in each building. The calculation for torsional shear is performed using the equation below:

$$T = \frac{V_{tot} \cdot e_x \cdot d_i \cdot R_i}{\Sigma(R_i d_i^2)}$$

V_{tot} = Story Force
e_x = distance from center of mass to center of rigidity
d_i = distance from frame to center of rigidity
R_i = relative stiffness of frame

Below, in Table 9, a sample calculation of shear caused by torsion is shown for Building A. The other values are shown in Appendix E.

Table 9: Sample calculation of torsional shear values.

Building A: Frame Torsional Shear							
Level	Frame	V_{tot} (k)	R_i	e_x (ft)	d_i (ft)	$R_i d_i^2$	Torsional Shear (k)
Roof	B	200.02	0.75	11.18	45.83	1575.29	20.28
	F	200.02	0.25	11.18	94.12	2214.64	13.88
	2	215.22	0.61	21.45	41.00	1030.45	42.05
	4a	215.22	0.39	21.45	66.83	1728.44	43.27
Third	B	421.58	0.75	16.42	40.83	1250.32	57.19
	F	421.58	0.25	16.42	99.12	2456.19	46.28
	2	435.75	0.61	20.78	42.00	1081.33	84.49
	4a	435.75	0.39	20.78	65.83	1677.10	83.61
Second	B	542.56	0.75	20.67	36.83	1017.34	84.28
	F	542.56	0.25	20.67	103.12	2658.43	78.65
	2	542.56	0.61	20.48	42.00	1081.33	103.69
	4a	542.56	0.39	20.48	65.83	1677.10	102.60

ANALYSIS OF WORST-CASE FRAMES (BRACE DESIGN)

With the distribution of lateral forces and torsional forces accounted for, the worst-case lateral force resisting frame is assessed and used for calculation of a typical brace size. This is taken to be the frame with the highest seismic shear value.

Frame L, level one, has the worst-case bracing scenario, as it had the highest lateral shear force to resist. A buckling restrained brace is designed using the following formula (courtesy of Structural Engineering Associates of California):

$$A_{brace} = \frac{\mathrm{P_{brace}}}{\varphi \mathrm{F_y}}$$

From this, it is determined that a buckling restrained brace with a steel core area of 10in^2 is needed to satisfy the lateral requirements of Frame L, level one, as seen in the figure 21 below. This size is applied to all bays and frames, as there was not time for economization for all building frames in this report. For a realistic sample of a finalized design coordinated with a buckling restrained brace manufacturer, please see the “Building A Economization” section of this report.

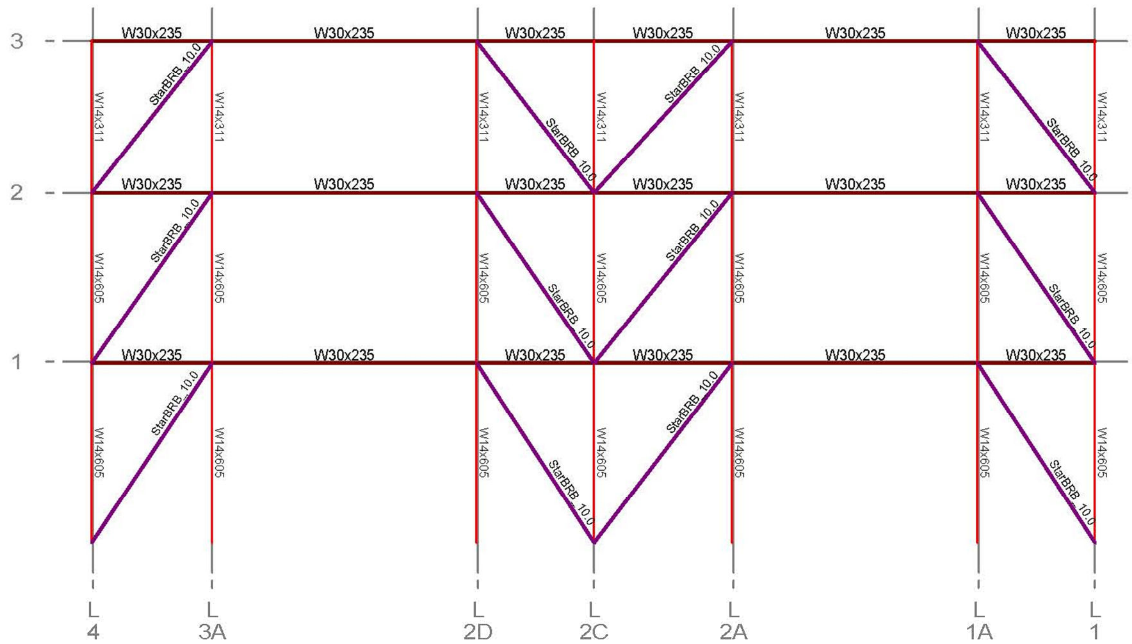


Figure 21: Frame L in Building B with member designations, including the assigned buckling restrained braces.

GRAVITY CHECK

Due to a shift in bay size, a gravity check is necessary to resize the new members extending between column lines S.1 and U. The original beams span 35'-0" between column lines U and T; the new members are resized to span 46'-6", causing an increase in the beam capacities. In terms of efficiency, many of the beams increased in linear weight, but decreased in depth. Column size, on the other hand, decreased. Figure 22 below is a comparison between the new and old frame designs and locations.

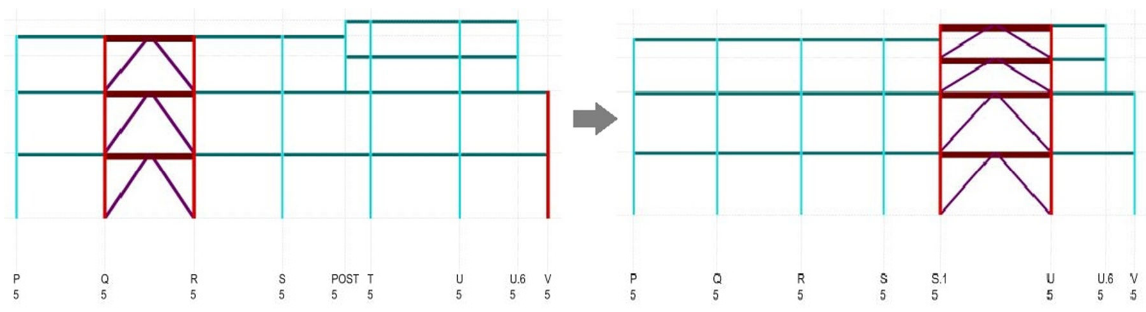


Figure 22: Old and new brace locations along southern façade.

The slab design from the original system is still deemed to be efficient. The slab is controlled by punching shear due to the loads from the rear axle of a tow truck, while lifting a double-decker bus. This imposes a load of 15.45 kips spread over a 4.5 inch diameter circle. This calculation can be seen in Appendix G and further discussed in Technical Report 1.

IMPLEMENTATION INTO RAM

To complete the analysis of the New York City Bus Depot, the lateral system analysis must be finalized. The design changes made above need to be implemented in to Bentley's RAM Structural System model originally created in Technical Report 3. The three structures of the New York City Bus Depot are all modeled in separate files to ensure that the properties and loads of the separate buildings may be applied properly. In Figures 23, 24, and 25, the gravity system can be seen modeled in blue, the lateral system in red, buckling restrained braces in purple, and the rigid slabs in a light gray.

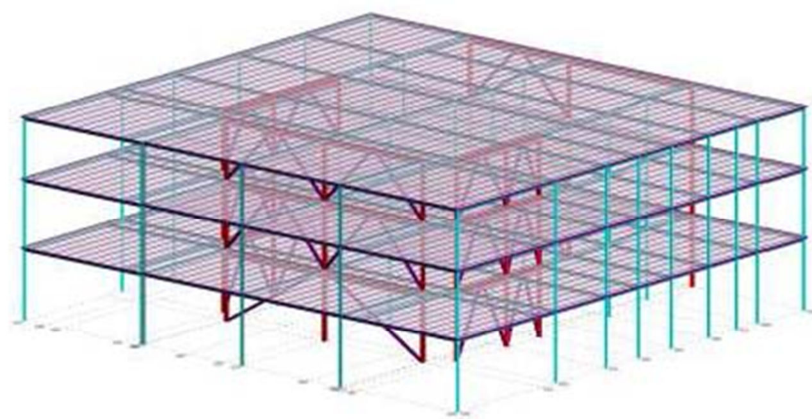


Figure 23: Building A RAM Structural System Model

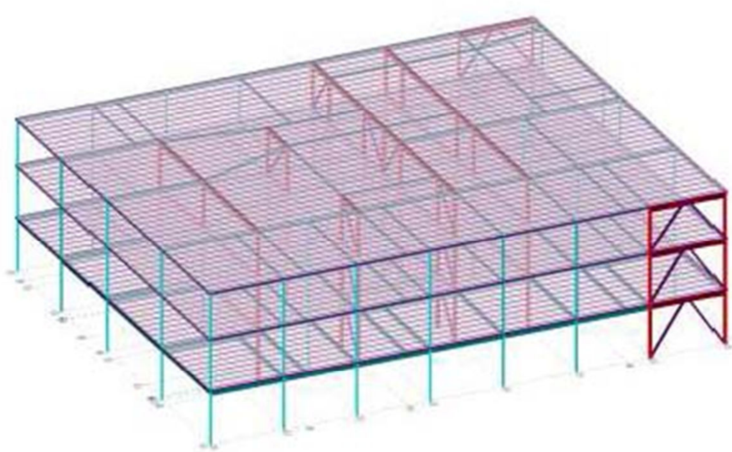


Figure 24: Building B RAM Structural System Model

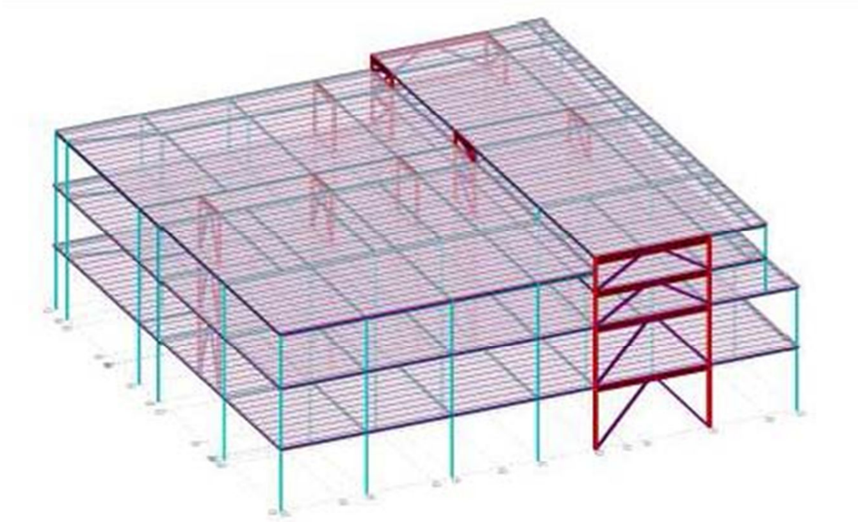


Figure 25: Building C RAM Structural System Model

Loads are applied to the structure from ASCE7-05 and IBC2006. The major change from the original model includes an alteration in the redundancy factor to be seven in both directions (north-south and east-west). Changes in brace designations from lateral members to gravity members were also made where necessary. Rather than being remodeled, some newly designated gravity members connected to remaining lateral frames are changed from having a fixed moment connection to a pinned connection typical of a gravity member.

Other assumptions in this model include the following, but are not limited to:

- Inclusion of PΔ effects
- Inclusion of Rigid-End Offset effects
- Diaphragms modeled rigid
- Columns fixed to ground

Building C contains the most changes, as its deflections in the original model are above the allowable according to analysis in technical report three. For this, the posts originally providing lateral support to the third floor mezzanine and the high roof are removed. Figure 22 shows this change earlier in the report. The east-west oriented braces along column lines 1 and 5 are shifted down to the mezzanine and stretched to span from column lines S.1 to U, as described in the gravity system analysis.

This change helps to stabilize the deflections of the third floor mezzanine and the high roof by tying these levels to the roof and other floors below. The new story drift and displacement values can be reviewed in Appendix H. Maximum and permitted drift and displacement values are discussed later in the outcome section of the structural depth.

BUCKLING RESTRAINED BRACE

A major advantage of Bentley's RAM Structural System Program is that it contains the section information of the Star Seismic™ buckling restrained braces within the program. The POWERCAT™ buckling restrained braces are analyzed in this project due to their simple pinned connections that allow for quick frame assembly and a longer yielding core in comparison to traditional bolted buckling restrained frames.

Once the size of the buckling restrained brace is determined, as discussed above in the lateral brace design, then the braces that are currently modeled as conventional steel members need to be altered to represent buckling restrained members. This is completed following *The RAM Structural System V8i SELECTseries 3™*user manual for RAM Frame.

When generic braces are modeled in their proper locations, buckling restrained braces can be assigned. This is done by following the commands **Assign-Braces-Buckling Restrained-Generic**. This assigns an axial stiffness modifier to the braces. Generally, an initial multiplier of 1.5 is used for preliminary design; however, coordination with the buckling restrained brace manufacturer is required for a more accurate modifier. The modifier accounts for the addition of stiffness from other material surrounding the steel core and the fact that the yielding steel core does not extend the entire length of the brace. The technical calculation for the axial stiffness modifier is as follows:

$$Modifier = \frac{E_{brace}\,A_{yield\ core}/L_{yield\ core}}{E_{total\ brace}\,A_{yield\ core}/L_{total\ brace}}$$

From here, the size of the buckling restrained brace needs to be assigned. This was earlier determined to be a brace with a 10in^2 steel core. To attain this section, the master steel table must be changed from the RAMAISC.TAB master steel table to the BRB_STARSEISMIC.TAB in the **Criteria-Master Steel Table** in the RAM Manager window. This brings up the same AISC members that were in the previous table, but add additional Star Seismic™ buckling restrained braces to the table. These sections are labeled by steel core area. Once this is applied, the brace sections can be assigned in the elevation views.

With the braces and all other structural members assigned, the structure can be analyzed. This is done by using the **Process-Analyze** command followed by **the Process-Member Code** check as is done with conventional structures. Keep in mind that the unique aspect of buckling restrained braces is that the member capacity is always calculated as though the forces are acting in tension. This is because the compression and tension capacities are similar, but the member cannot buckle. Because of this, the capacity is always plastic, just like the capacity that is calculated for a member in tension.

Buckling Restrained Braced Frame Modeling Results

Load Path Analysis

Structures follow a load path that takes both lateral and vertical loads into the ground. In the New York City Bus Depot, lateral loads are absorbed by the building's braced frames according to their stiffness values relative to one another. These stiffness values are calculated by applying a unit of a force to the frame at the level of analysis and recording the deflections at that respective level. Each frame stiffness value is set to a ratio of the sum of the stiffness values on that level acting in that direction to obtain the relative stiffness. The relative stiffness values for this building can be found in Appendix D. This is accomplished in the SAP2000 analysis of the structure and applied when recalculating the later force distributions which are again found in Appendix D. These lateral forces are distributed to the braces implemented in the structure, which then take these loads down into the columns and into the ground.

Gravity loads are distributed across the slab, which transfer loads to the beams, and then from the beams into the columns and into the ground.

Drift and Displacements

Upon analysis of the building lateral systems, drift values are obtained for each load case. The drift values determine the controlling load case. Each deflection is set in a ratio to the maximum allowable deflection for the type of load. The maximum allowable load cases are as follows:

$$\frac{H}{400}\ (Wind\ per\ Code)$$

$$0.020h_{sx}\ (Seismic\ per\ Code)$$

$$\frac{H}{240}\ (Seismic\ for\ Nonstructural)$$

For the New York City Bus Depot, H/240 is used to control the seismic design because it is a good limit to prevent damage to other non structural elements of the building. This standard is frequently applied in the professional world according to Chris Cerino, a structural engineer at STV Incorporated. Below is a chart of the maximum wind and seismic deflections for each of the three buildings:

Table 10: Drift Ratios

Structure	Load	Max (in)	Permitted (in)	Ratio
A	EQ	1.28	3.634	0.35
	W	0.312	2.180	0.14
B	EQ	0.520	3.634	0.14
	W	0.315	2.180	0.14
C	EQ	0.724	3.934	0.18
	W	0.386	2.557	0.15

The ratios for the earthquake deflections are higher than those for the wind deflections, but the low ratios of deflection indicate that strength is really in control of the design, as opposed to deflection. This is also seen by the seismic forces that are much higher than the wind shears when distributed.

PERIOD

The change in period of the building from the original design with the moment frames to the current design with buckling restrained braced frames can be seen below in table 11. Notice that the period in the East-West (Tx) direction decreases, but the periods for the North-South direction and rotation increase. This is potentially due to change in quantity of frames and the change in fixity of the connections. All of the periods are still acceptable.

Table 11: Period Calculations

	Period Calculations					
	Building A		Building B		Building C	
	Original	New	Original	New	Original	New
Tx	1.57	1.31	1.30	1.20	2.20	1.49
Ty	0.63	1.23	0.76	1.08	1.26	1.47
Tz	0.63	1.08	0.68	0.93	0.90	1.10

CONTROLLING LOAD CASE

This analysis utilizes load cases and combinations from ASCE 7-05 and IBC 2006. The controlling load case is determined by the largest ratio of actual to allowable deflection and the greatest lateral forces acting on the structures. The deflections from the previous section in this report are from the cases highlighted in orange on tables 12 and 13: the controlling cases and combinations. The variables indicate how these loads are represented in RAM Structural System.

Table 12: Load Cases with the controlling highlighted in orange

Load Case Definitions: IBC 2006		
Variable	**Type**	**Definition**
D	Dead Load	User
Lp	Live Load	User
W1	Wind	X
W2	Wind	Y
W3	Wind	X + e
W4	Wind	X - e
W5	Wind	Y + e
W6	Wind	Y - e
W7	Wind	X + Y
W8	Wind	X - Y
W9	Wind	X + Y CW
W10	Wind	X+ Y CCW
W11	Wind	X - Y CW
W12	Wind	X - Y CCW
E1	Seismic	X + e
E2	Seismic	X - e
E3	Seismic	X + e
E4	Seismic	X - e

Table 13: Load Combinations with the controlling highlighted in orange.

Allowable Stress Design Load Combinations: ASCE 7-05		
Code Defined Loads		**Lateral Loads**
1	**D** + **F**	--
2	**D** + **H** + **F** + **L** + **T**	--
3	**D** + **H** + **F** + ($\mathbf{L_r}$ or **S** or **R**)	--
4	**D** + **H** + **F** + 0.75(**L** + **T**) + 0.75($\mathbf{L_r}$ or **S** or **R**)	--
5	**D** + **H** + **F** + (**W** or 0.7**E**)	(**W** or 0.7**E**)
6	**D** + **H** + **F** + 0.75(**W** or 0.7**E**) + 0.75**L** + 0.75($\mathbf{L_r}$ or **S** or **R**)	(**W** or 0.7**E**)
7	0.6**D** + **W** + **H**	**W**
8	0.6**D** + 0.7**E** + **H**	0.7**E**

D = Dead Load; **E** = Earthquake; **F** = Well-defined Fluids; **H** = Lateral Earth Pressure; **L** = Live Load; $\mathbf{L_r}$ = Roof Live Load; **R** = Rain Load; **S** = Snow Load; **T** = Self-straining Force; **W** = Wind Load

DIRECT SHEAR VALUES

Direct shear values are discussed earlier in this report. They are a result of the controlling seismic loads' maximum base shear and the torsional shear on the building both distributed according to brace relative stiffness. The values for the shear forces on individual frames and stories on the building can be seen in Appendices D and E.

OVERTURNING MOMENTS VS. RESISTING MOMENTS

Overturning moments can be seen in Structural Depth-Process-Later Load Calculation section of this report. These values are far less than the resisting moments presented in table 14 below:

Table 14: Resisting Moments

Resisting Moments			
Structure	**Property**	**N/S**	**E/W**
A	Weight	12,272	12,272
	Width	196	184
	M_{Resist}	**1,199,588**	**1,130,067**
B	Weight	12,272	12,272
	Width	196	245
	M_{Resist}	**1,199,588**	**1,503,320**
C	Weight	12,272	12,272
	Width	196	210
	M_{Resist}	**1,199,588**	**1,288,560**

It is important to note that the values for the resisting moments are obtained from unfactored loads, where as the overturning moments calculated for the seismic loads above are obtained from factored loads. This only further proves the point that the building has the capacity to resist the overturning moments imposed on it from the controlling seismic loads.

CONNECTION DESIGN

The design of a connection is beyond the scope of this thesis, but the strength required by a connection can be easily found according to the following equation:

$$Connection\ Strength = \beta\omega P_{yielding\ core}$$

β = Compression Max : Tension Max

ω = Tension Max : Yield Strength*Steel Area

Using beta and omega values from the economized value from Building A, a sample calculation was done for ten square inch steel yielding core brace present in that design. With a beta value of 1.09, an omega value of 1.29, and an axial force of 460 kips, the connection required a strength of 647 kips. The connection strength is greater than the maximum strength of the brace which is important so that the connection does not fail, but instead, the brace acts as a fuse. The values used for this calculation can be found in Appendix I. A sample connection is shown below in figure 26.

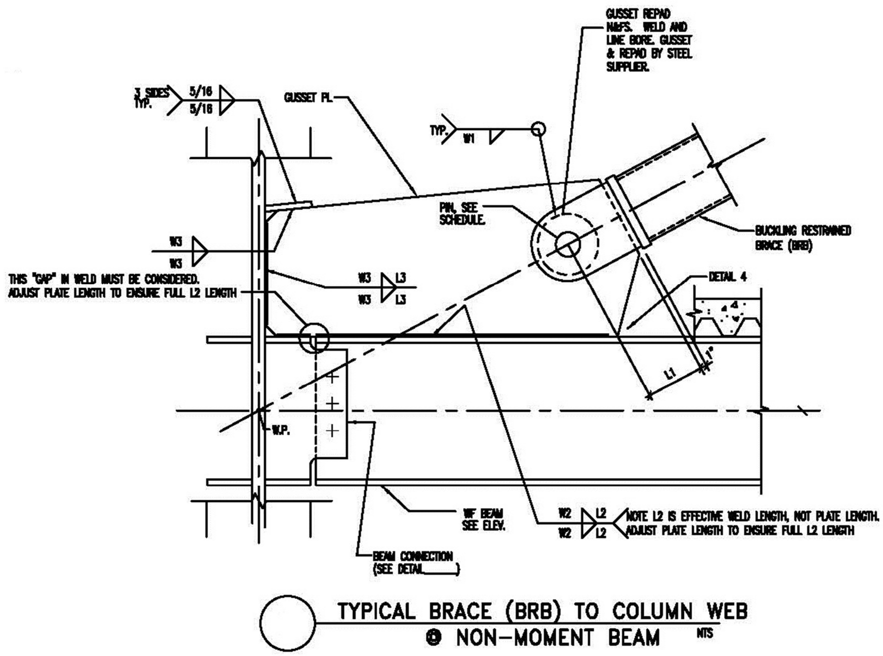


Figure 26: Typical buckling restrained braced connection
(Image courtesy of Star Seismic™

Economized Model

Through coordination with Kim Robinson of Star Seismic™ an economized buckle restrained brace frame lateral design was achieved for Building A. The results of this analysis can be seen in Appendix I. Below is figure 27 summarizing the results of the coordinated analysis.

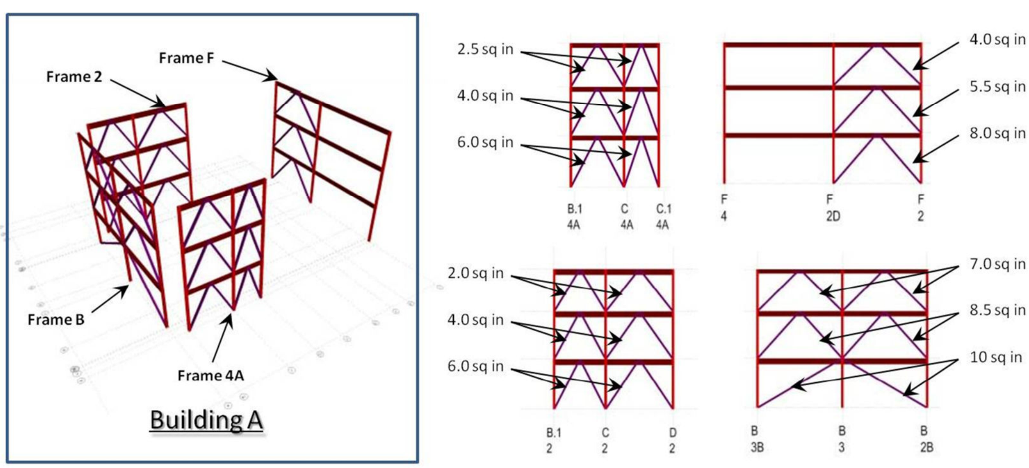


Figure 27: Building A and its frames labeled by yielding steel core size.

OUTCOME

The goals of the structural analysis are as follows:

- ◊ Create a bracing scheme to replace the moment frame scheme
- ◊ Do not interfere with the bus flow within the building
- ◊ Utilize buckling restrained braced frames to lower seismic forces on the structures
- ◊ Control drift of the 3rd Floor Mezzanine and High Roof
- ◊ Decrease lateral loads on the building to help with design on poor soils

These goals are all met in the structural depth of this thesis. The buckling restrained braced frames prove to be an effective solution in creating a bracing scheme that does not interfere with the bus flow of the building and effectively distributes decreased lateral loads on the buildings to the foundations. The new scheme is also able to control drifts throughout the building, including those originally resisted by posts on the third floor mezzanine and high roof. To further prove the effectiveness of the newly designed system, a construction management breadth must be studied to deem the effectiveness of the effects on cost, schedule, and construction sequence. The daylighting breadth also deems whether or not the new scheme aids in bringing higher light levels into the building. These studies can be seen in the following pages of the report.

CONSTRUCTION MANAGEMENT BREADTH:

PROBLEM STATEMENT:

The New York City Bus Depot is an important transportation hub. Its temporary decommissioning displaces 150 buses to various other depots in the city, causing overcrowding and worker displacement. Because of this, the New York City Bus Depot requires efficient and timely construction methods. More efficient and cost effective methods will allow the New York City Mass Transit Authority to return its focus to providing the residents in the neighborhood of the depot improved, convenient public transportation throughout the city.

PROPOSED SOLUTION:

Given that steel erection and connection times often form a major portion of a project schedule's critical path, the lateral force resisting system is selected for study in order to decrease the cost and construction time of the New York City Bus Depot. The current design for the bus depot utilizes labor intensive and expensive moment connections for a majority of its lateral system. Utilizing a braced lateral system is proposed in order to decrease the cost of the project. Research from Dasse Design, Incorporated indicates this will be the outcome for the New York City Bus Depot as seen in Figure 28 below, highlighting the cost differences between special concentrically braced frame structures to buckling restrained brace frame structures. Cost savings from a moment frame system are expected to be higher than the braced frame in the comparison.

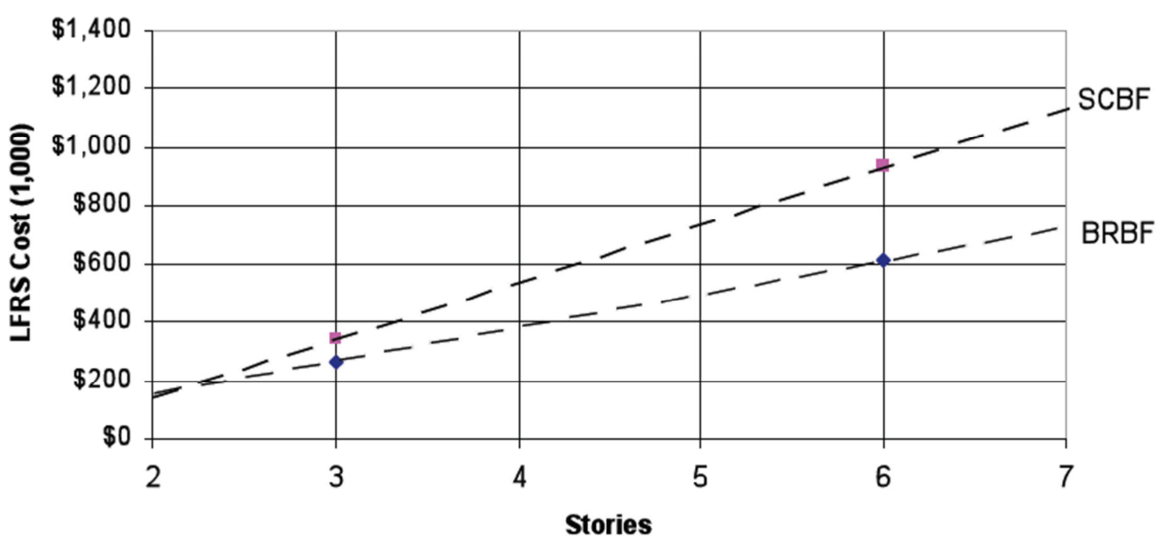


Figure 28: LRFS Cost relative to building Height (Image courtesy of Dasse Design Incorporated).

Not only does this change in framing scheme decrease the cost per unit of stiffness, but the number of lateral force resisting frames decreases as well. This results in a quicker erection time for the lateral force resisting system. Because no project schedule is available for the study of the New York City Bus Depot, connection times for individual frames are compared to each other for analysis. Connection cost is then further decreased by the POWERCAT™ buckling restrained braces that utilize a true pinned connection in place of high strength bolts.

Buckling restrained brace frames are gaining more recognition for their construction ease and streamlined design process. According to a study by Dasse Design Inc., buckling restrained brace frame systems can save the average three-story building utilizing a special concentrically braced frame system approximately $1.10/ft^2. Most of this savings occurred in reference to construction time and material for the connections. This savings would only increase in comparison to the welded moment frames.

PROCESS:

Changes in cost and schedule are analyzed by utilizing RS Means data. First, analyses are performed on the original lateral force resisting system with moment frames and traditional, ordinary chevron-braced frames to form base estimates. Once the new structural system is laid out, a new set of analyses are performed on the new system. This new takeoff includes members that are part of the original lateral force resisting system, such as the original beams and columns, in addition to the new braced members.

COST ANALYSIS

Cost analysis is performed through a tonnage take-off for steel, followed by an analysis of each type of typical connection. Typical weld lengths, bolt amounts, plate and angle quantities et al, are taken from the detail pages of the structural drawings as shown in figures 29-31 below:

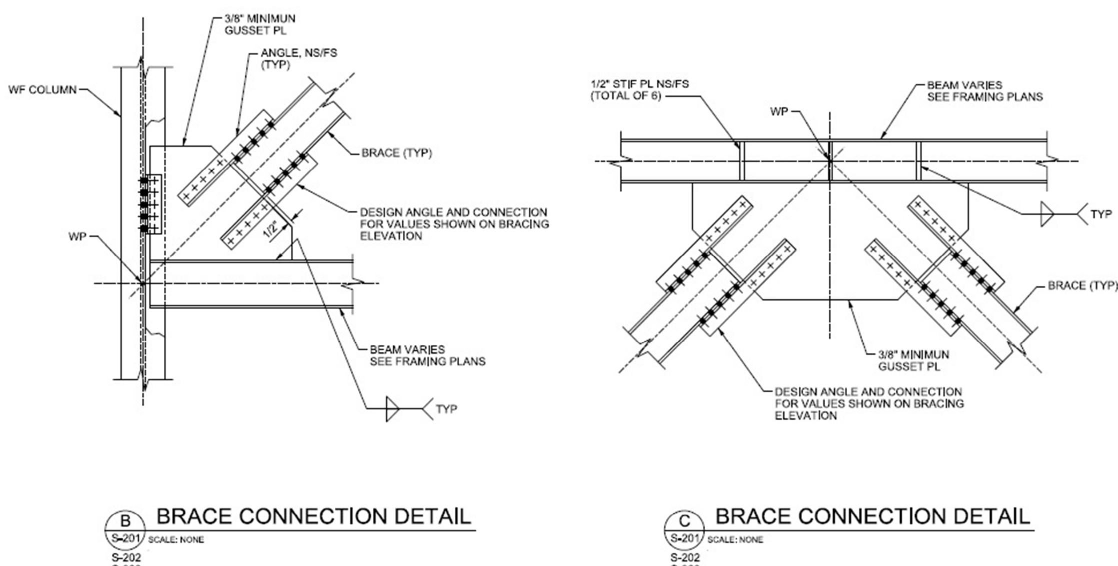


Figure 29: Typical connection details for conventional steel braced frames.

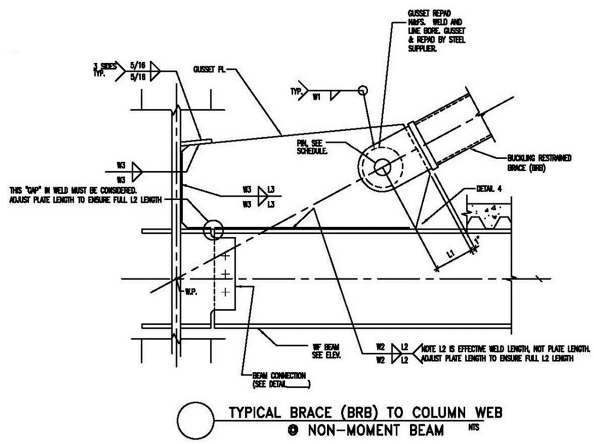


Figure 30: Typical buckling restrained brace connection detail

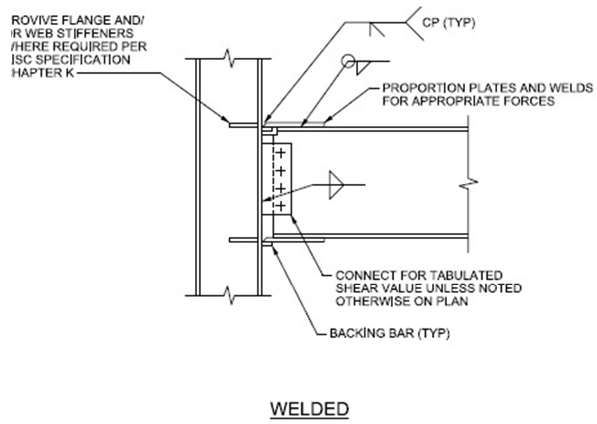


Figure 31: Typical moment connection detail

Pricing values are taken from the bare materials, bare labor, and bare equipment costs from RS Means with the exception of the pins and braces for the buckling restrained frames. For the cost of the buckling restrained braces, the Dasse Design Inc. Report is used for information. The value of a normal steel brace is taken and multiplied by the ratio 13/9 according to the cost ratios shown in the pie charts in figure 32 comparing buckling restrained braces to special concentrically braced frames.

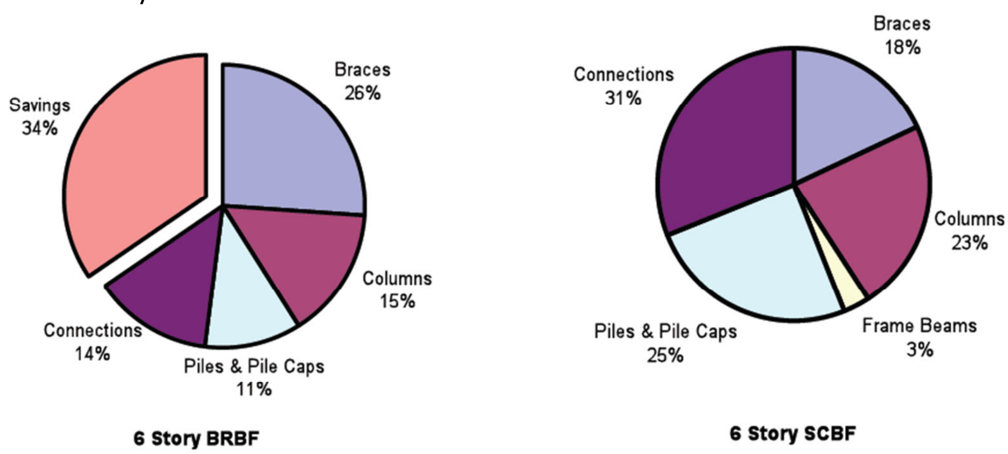


Figure 32: Relative Cost of LRFS Elements
6 Story Building with a Pile Foundation
(Image courtesy of Dasse Design, Incorporated).

Assumed pin sizes are taken from a typical detail sheet provided by the manufacturer, StarSeismic™; pins were assumed to be 4” in diameter and 9” in length. Material estimates for the pins are taken from structural steel tonnage estimates to achieve a value of $13.50 per pin. The results of these cost analyses are seen below in the outcome section of the report.

SCHEDULE ANALYSIS

The schedule is also analyzed by using RS Means data. Daily output and labor hours are assessed from RS Means and applied to each portion of the connection assembly, as seen in appendices L, M, and N. Microsoft Excel is utilized to compare the time differences of each style of bay construction, from column erection to connection completion. In Figures 33 through 37 below are schedule analyses of individual frame types. Again, individual frame erection times are shown to compare for steel erection due to a lack of project schedule.

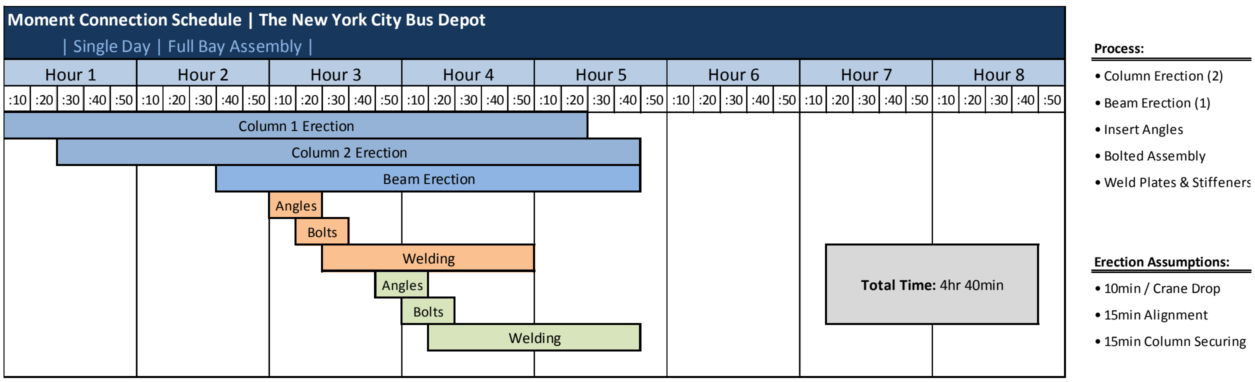


Figure 33: Full Bay Connection Schedule for an Ordinary Moment Frame

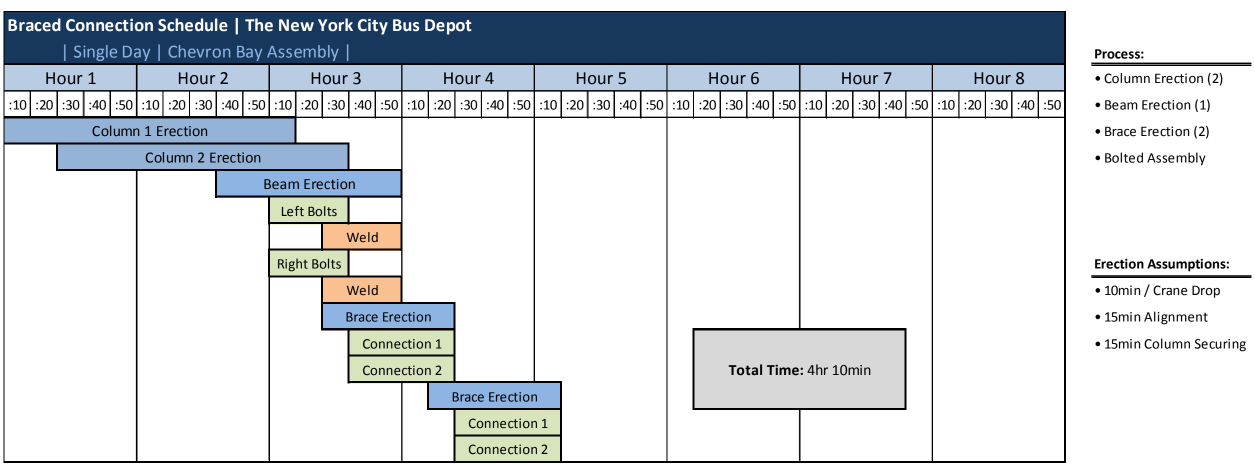


Figure 34: Full Bay Connection Schedule for Ordinary Steel Chevron Braced Frames.

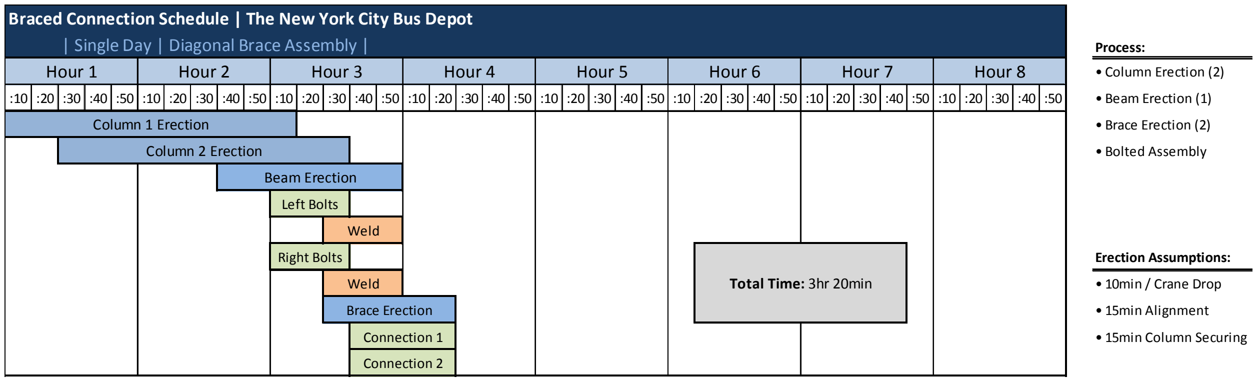


Figure 35: Full Bay Connection Schedule for Ordinary Steel Diagonally Braced Frame

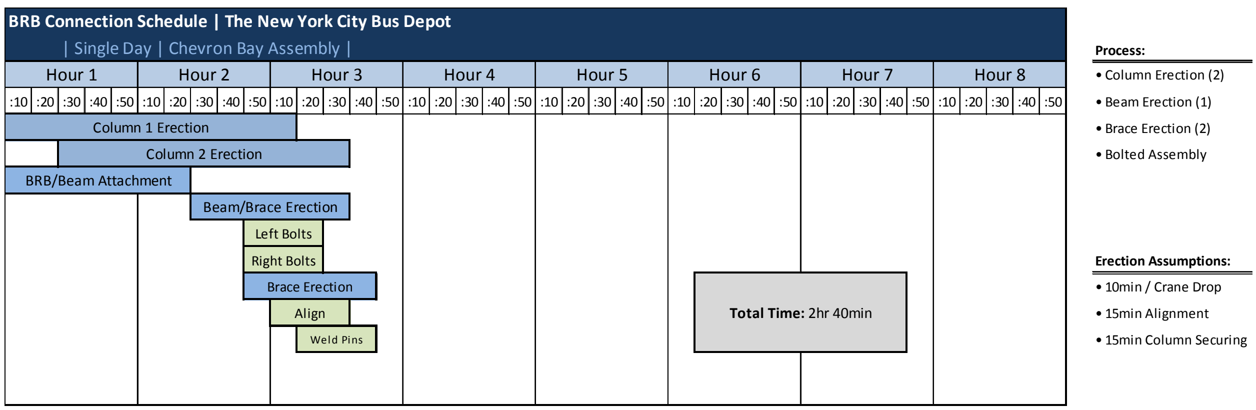


Figure 36: Full Bay Connection Schedule for Buckling Restrained Brace Chevron Assembly

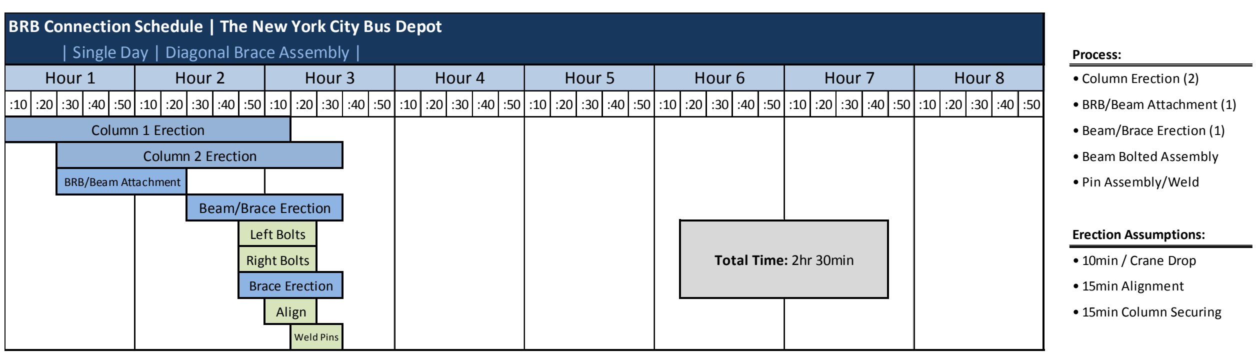


Figure 37: Full Bay Connection Schedule for Buckling Restrained Brace Diagonal Brace Assembly

From the charts above, it can be seen that the erection time for a buckling restrained brace member is significantly shorter than for a conventional braced frame or a moment connection. The time difference from the welded frame to the bolted frame is a decrease of 57%. It should also be noted that, in addition to this decrease in assembly time, the buckling restrained brace requires fewer laborers for the erection than a welded moment connection. Due to lack of availability of the project schedule, times are compared relatively to each frame style, looking only at the ability to shorten steel lateral system frame erection time, not the effect on the critical path of the schedule.

OUTCOME:

The following list outlines the goals set forth and achieved for the construction management study of the New York City Bus Depot:

◊ Decrease the construction time required for making connections and assembling frames
◊ Decrease the cost of lateral system erection
◊ Decrease the skilled laborers necessary on site

The use of buckling restrained braced frames is beneficial for the New York City Bus Depot. Cost analysis shows that the connection costs for buckling restrained braces are much cheaper than those for moment connections. In fact, the use of buckling restrained braced frames in place of moment frames decreases the lateral resisting system's construction costs by 8%. In this case, that is over a $250,000 reduction in lateral system costs. The comparison is seen in table 15 below:

Table 15: Prices per connection and per project

Pricing Connections			
Unit	**BRB**	**Weld**	**Bolt**
per connection	$200.50	$672.76	$1,164.08
per project	$2,995,548.31	$3,346,611.66	

* Note : per project costs include braces, connections, columns, and beams

As for the schedule, the connection cost time is greatly decreased. According to the time assessed per connection, the erection time for individual frames decreases by over 50%. Much of this is due to the greatly decreased amount of welding and simplification of the connections. This lack of welding creates a much simpler construction sequence, though it requires much stricter construction tolerances. Below, in figure 38, is a series of images depicting the construction sequence of the buckling restrained braces:

Figure 39: Construction sequence of installing a buckling restrained brace. This shows the Wildcat brace system, but the processes between the Powercat and Wildcat braces are very similar.

Daylighting Breadth:

Problem Statement:

The New York City Bus Depot takes pride in its design to be one of the greenest bus depots ever built, and in order to be sustainable, sufficient amounts of daylight must enter the building when it is available. With such large, expansive spaces with minimally reflective surfaces, vast expanses of windows are necessary to light the parking spaces adequately during the solar window. Seeing as adequately lit spaces are necessary for operation, utilizing sunlight when possible is particularly important because the depot is a 24 hour facility that will be required to use electric light at certain points of the day.

Proposed Solution:

The set of two east-west oriented braced frames in building C are relocated to the stairwell's exterior facade. This allows room for an additional set of windows to be added in the bay between column lines Q and R.

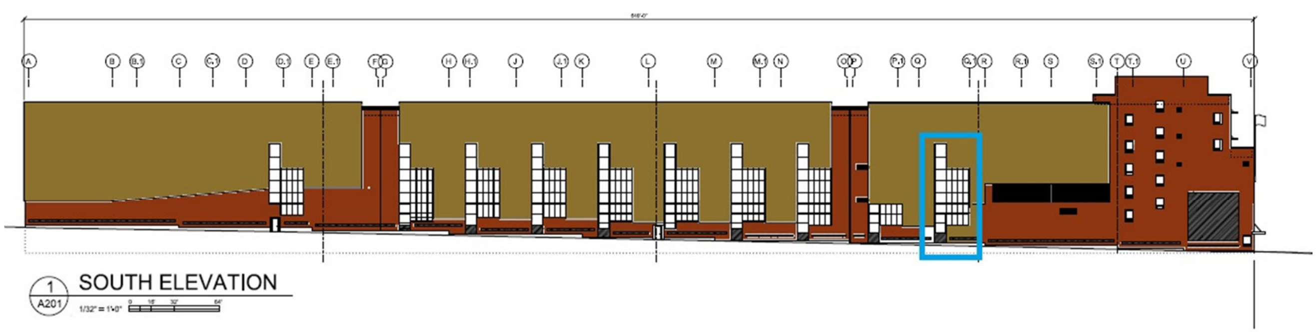


Figure 40: South elevation of bus depot with new window highlighted in blue

With this additional set of windows, as seen in Figure 40, the bus depot's third floor parking area can be analyzed for its foot-candle levels throughout a day. The maximum value is taken on the twenty-first of June, as it is the summer solstice where the sun will be brightest. Since the facility is run on a 24-hour operation, the minimum value considered is zero footcandles, rather than a low light value in the winter solstice. The study of the parking area shows whether or not an additional bay of windows is worth the cost and effort for the extra light.

PROCESS:

To determine the proper light levels in the New York City Bus Depot, both interior and exterior daylight levels are calculated. This is done mainly using Microsoft Excel. Since the depot operates around the clock, the maximum lighting availability is the only value calculated. The minimum value for available lighting during operating hours is zero footcandles.

EXTERIOR DAYLIGHT VALUE:

The exterior daylighting quantities are calculated using a spread sheet developed in Microsoft Excel in ARCH497E: Daylighting Analysis of Ancient Roman Architecture. This template contains cells filled with information pertaining to the time, site location, and building orientation. Appendix O shows a sample calculation for the lighting values obtained for noon on June 21st along with the accompanying calculations and input values.

In the spread sheet, computed values are highlighted in orange and user input values are highlighted in yellow. For the proper outputs, the user needs to be aware of the date and time desired, the time zone of the site, whether or not daylight savings time is in effect, the latitude and longitude of the site, the building elevation azimuth, and the sky condition. From here, solar time, sunrise, sunset, and all solar angles are computed. Also, important for evaluation of the interior lighting, horizontal and vertical illuminance values are calculated. A table of these values can be seen in the outcome section of the daylighting analysis section of this report.

INTERIOR DAYLIGHT VALUES:

From the exterior daylighting quantities, the vertical and horizontal illuminance values are obtained for calculating the light values at varying depths of the room. The parking area is selected for study in this case. Of the three methods available for interior light level calculations, the IESNA method is used for this calculation because it is the most accurate and flexible. This method takes into consideration reflected light from surfaces and structures, accounts for shading, glazing types, window controls , zonal cavities due to window height, and varying distances in the room. A limitation of this method is that the calculation cannot be done for a room with undetermined dimensions. All dimensions and building materials must be known for a proper calculation. Variables used in the calculation can be found in *Mechanical and Electrical Equipment for Buildings, 10th Edition* on pages 606-609. Below, Table 16 contains a chart of constants utilized in the computation:

Table 16: Constraints utilized in IESNA indoor Daylight Calculation Method

CONSTANTS		
Window Transmittance	0.85	(Tempered Insulated Glass)
Light Loss Factor	0.8	(Clean Industrial Zone)
Net Glazing Area	0.87	(Assuming 3" Mullions)
Net Transmittance (τ)	0.5916	
Ground Reflection (RF_g)	0.2	
Room Depth	196	
Window Height	9	
Window Length	54	9 windows

The obtained daylighting values can be seen in the outcome section of the Daylighting breadth of this report.

OUTCOME:

The following outlines the goals set forth and achieved for the daylighting analysis of the New York City Bus Depot:

- ◊ Examine the effects of an additional window on the southern building façade
- ◊ Obtain maximum available daylighting values outdoors
- ◊ Obtain light levels indoors
- ◊ Determine adequacy of interior lighting in parking area

After analysis of the interior daylight levels within the building, it is deemed that the addition of another window makes little to no difference in the light levels present in the building. The slight increase in window area compared to the full area of the wall does not increase the light values or change any of the coefficients to alter the daylight levels in the parking area of the building. The coefficients of utilization can be seen in Tables 17 and 18 below for the varying depths of the parking area.

Table 17: Values used to enter the chart to determine the Coefficients of Utilization

Coefficient of Utilization Chart Values	
Window Length/ Window Height	6
Rounded Value	6
50% Room Depth/ Window Height	10.888889
Rounded Value	10
30% Room Depth/ Window Height	6.5333333
Rounded Value	7
10% Room Depth/ Window Height	2.1777778
Rounded Value	3

Table 18: Coefficients of Utilization

Coefficients of Utilization according to Sky Condition (SC)					
0.75		1.00		1.25	
CU_k	CU_g	CU_k	CU_g	CU_k	CU_g
0.022	0.023	0.03	0.023	0.035	0.023
0.093	0.098	0.118	0.098	0.133	0.098
0.749	0.185	0.616	0.185	0.536	0.185

Even without the additional window, the light levels are sufficient according to the IES Footcandle Recommendations table seen in Appendix Q. These state that a footcandle level of 20 is sufficient for a service garage. Figure 41 below shows the varying light levels at different depths in the room through color value (white being the highest light level and blue being the lowest). The black line represents the 20 footcandle minimum acceptable light value. All values beyond this line are not deemed acceptable and will need to be supplemented with electronic lights. Table 19 quantifies these values later in the report. The illuminance values are satisfactory for the 20 footcandle requirement except for in the morning hours and the late evening hours just before sunset.

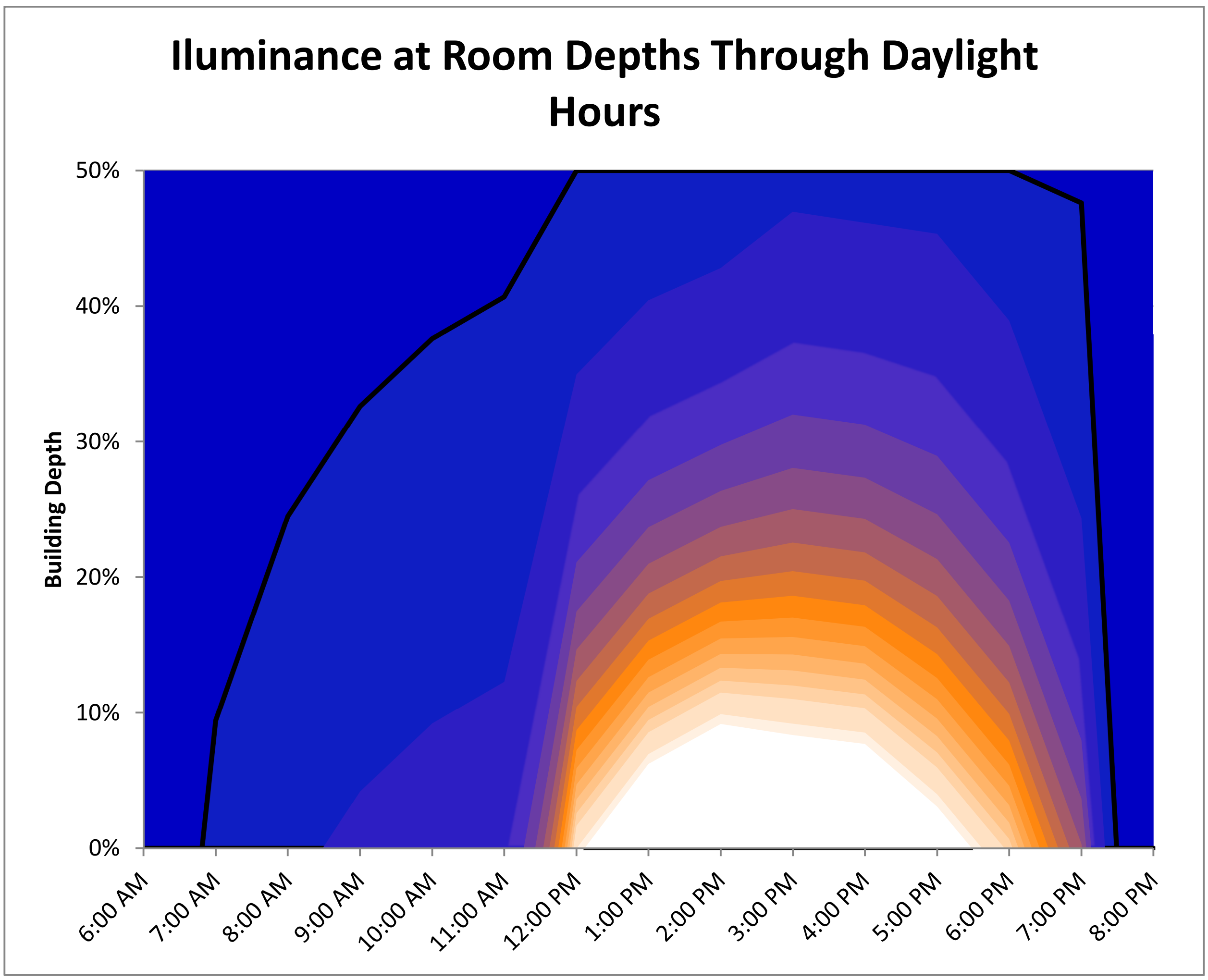


Figure 41: This chart gives color value to the light values in the depot. The black lines shows the 20footcandle cut off.

Daylighting Values			
Time	**6:00 AM**	**7:00 AM**	**8:00 AM**
Solar Altitude (degrees)	5.1	15.6	26.6
Vertical Window Illuminance (fc)	0	0	0
Horizantal Illuminance: Full Sky (fc)	97	1465	3349
Horizantal Illuminance: Half Sky (fc)	48.5	732.5	1674.5
Vertical Illuminance from Ground (fc)	0.0	0.0	0.0
SC	0.00	0.00	0.00
Closest Chart Value	0.75	0.75	0.75
Illuminace at 50% depth (fc)	0.13	1.99	4.56
Illuminace 30% depth (fc)	0.56	8.49	19.42
Illuminace 10% depth (fc)	1.06	16.03	36.65

Daylighting Values					
9:00 AM	**10:00 AM**	**11:00 AM**	**12:00 PM**	**1:00 PM**	**2:00 PM**
37.9	49.2	60.0	68.9	72.7	68.2
0	0	0	1393	2595	3416
5220	6859	8122	8909	9161	8859
2610.0	3429.5	4061.0	4454.5	4580.5	4429.5
0.0	0.0	0.0	0.0	0.0	0.0
0.00	0.00	0.00	0.31	0.57	0.77
0.75	0.75	0.75	0.75	0.75	0.75
7.10	9.33	11.05	30.25	46.24	56.51
30.26	39.77	47.09	128.29	195.89	239.31
57.13	75.07	88.89	714.76	1250.13	1610.62

Daylighting Values					
3:00 PM	**4:00 PM**	**5:00 PM**	**6:00 PM**	**7:00 PM**	**8:00 PM**
59.0	48.2	36.9	25.6	14.6	4.1
3775	3632	3001	1964	735	0
8026	6724	5057	3174	1306	47
4013.0	3362.0	2528.5	1587.0	653.0	23.5
0.0	0.0	0.0	0.0	0.0	0.0
0.94	1.08	1.19	1.24	1.13	0.00
1.00	1.00	1.25	1.25	1.25	0.75
77.92	73.61	69.02	44.99	17.00	0.06
310.06	292.53	265.45	172.93	65.40	0.27
1463.55	1397.19	1006.96	657.52	247.36	0.51

Table 19: Daylighting levels by time of day and location.

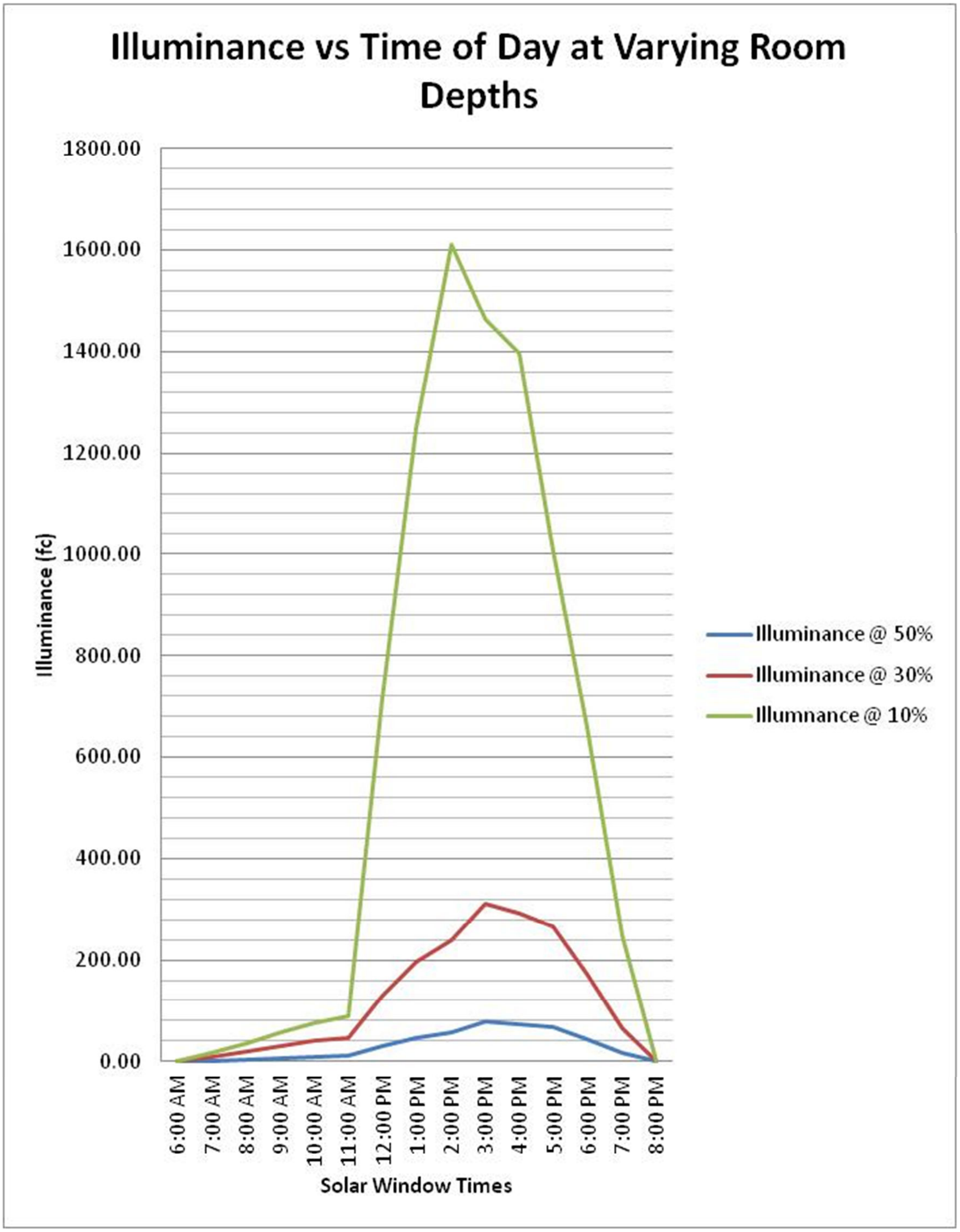


Figure 42: Illuminance vs. Time of Day at varying Room Depths. Note that 20 footcandles is the minimum acceptable light level.

CONCLUSION AND SUMMARY:

The buckling restrained braced frame redesign of the lateral system of the New York City Bus Depot creates a viable solution that lowers construction costs, stabilizes the third floor mezzanine, and does not interfere with the architectural flow of the depot.

The goals stated and achieved in the thesis study are as follows:

- ◊ Create a bracing scheme to replace the moment frame scheme
- ◊ Do not interfere with the bus flow within the building
- ◊ Utilize buckling restrained braced frames to lower seismic forces on the structures
- ◊ Control drift of the 3rd Floor Mezzanine and High Roof
- ◊ Decrease lateral loads on the building to help with design on poor soils
- ◊ Decrease the construction time required for making connections and assembling frames
- ◊ Decrease the cost of lateral system erection
- ◊ Decrease the skilled laborers necessary on site
- ◊ Examine the effects of an additional window on the southern building façade
- ◊ Obtain maximum available daylighting values outdoors
- ◊ Obtain light levels indoors
- ◊ Determine adequacy of interior lighting in parking area

In order to convert the lateral force resisting system from one primarily of moment frames to one of buckling restrained brace frames, several steps are followed. The bays that could not maintain braced frames are assessed by critiquing the first floor plan of the bus depot. Frames are then modeled in SAP2000 to determine which ones are unnecessary. Once the frames to be utilized are determined, the lateral and torsional forces are recalculated. Member recalculations are then done to determine brace sizing, column sizing, and beam sizing. Once implemented into RAM, the success of the design is thoroughly evaluated. The design is acceptable, seeing as the drifts on the upper levels are under control and the bus flow is not interrupted.

From here, the construction management breadth displays the lower costs and erection times of the elements of the bus depot. RS means data is used with a detailed material take off to determine construction times, labor costs, material costs, and equipment costs. The evaluation of individual frames of the bus depot show that erection time for an individual frame can be decreased by over 50%, and the cost for erecting the entire lateral system can be decreased by over $250,000: an 8% savings.

The lighting study shows that, though there is room for an additional set of windows, the windows will not make an impact on the lighting levels of the bus depot. The study, done by applying IESNA equations into excel spread sheets, shows that the lighting levels are plenty acceptable for the parking garage, further proving the statement that the New York City Bus Depot is a green building.

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