Kettler Capitals Iceplex

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Structural Option

The Pennsylvania State University
Architectural Engineering
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Faculty Consultant: Dr. Linda Hanagan

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To all my friends and family, for all their support. I couldn’t have made it this far without you!

Washington Capitals Iceplex

Arlington, Virginia

Team Roster

- Owners: Lincoln Holdings, Arlington County
- Architect: Architecture, Inc.
- Structural Engineer: Rathgeber/Goss Associates
- MEP Engineer: KTA Group
- Civil Engineer: VIKA
- Construction Manager: Sigal Construction Corporation

The Stats

- Project Cost: $42.7M
- Size: 137,000 SF
- Levels: 9
- Construction Dates: January 2005 - present
- Delivery Method: Design-Bid-Build

Architectural Line

- Highest ice rink above street level in the United States
- Built on top of existing seven story parking garage
- Designed to be LEED Certified
- Two NHL regulation size rinks with training facility and corporate offices

Structural Line

- Parking Structure
  - -Five levels cast-in-place concrete
  - -One level post-tentioned concrete
  - -One level composite steel
- Ice Rink/Training Facility
  - - Composite Steel
  - - Deep long-span roof joists providing clear space for rinks

Mechanical Line

- Desiccant based dehumidification system
- Refrigerant based cooling system
- Air space that contacts ice must remain 10-20 degrees above ice surface temperature
- CO_2 sensors regulate required 8000 cfm of outside air to ice rinks
- Network of ammonia refrigerant piping runs under rink to create and maintain ice

Electrical/Lighting Line

- 4000-ampere, 277/480 volt 3-phase, 4 wire switchboard located on garage level one
- Service extended to 8th and 9th level Iceplex to distribution switch board via bus duct
- 120/208 volt step-down transformer located in mechanical rooms if needed
- Lighting served at 277 volts
- Emergency power to support life safety systems and fire pump

Fire Protection Line

- Standard wet system complying with NFPA 13, 14, 24
- Meets requirements for high-rise building structures
- Fire pump maintains required 100 psi at top of standpipes
- Fully alarmed heat trace system

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EXECUTIVE SUMMARY

The Kettler Capitals Iceplex is the practice facility for the NHL franchise, Washington Capitals. It is located in Arlington, Virginia just outside Washington D.C. The Iceplex was constructed on top of the existing parking structure for the Ballston Mall in Arlington. The original parking structure consists of concrete two-way slabs and post-tensioned concrete. The Iceplex was constructed using a composite steel system.

When the Iceplex was constructed on top of the existing parking structure, the gravity system, the lateral system, and the foundation system all needed to be reinforced. This was proven to be the most complicated part of the design.

A solution to this problem would have been to tear down the parking structure and construct the new building from scratch. This thesis examines this possibility in order to determine if this is indeed a feasible solution. The Iceplex and parking structure will be completely redesigned. The two ice rinks will be moved to the first level on a slab-on-grade, which will help limit deflections of the ice surface. The parking structure will then be designed as a separate structure constructed of precast concrete and will span over the ice rinks. This will create the need for a large transfer system.

In addition to the complete structural redesign of the Iceplex and parking garage, three additional design changes are discussed. First, a civil/site design examines the most efficient way of laying out the building and includes any changes in the locations of entrances and exits. Second, an architectural redesign accounts for any changes in the architectural layout of the spaces. Finally, a construction management assessment compares the cost and schedule of the proposed design to the actual design.

Based on the structural redesign and the three breadth topics, it was found that the proposed design is *not* a feasible solution. Although it was possible, the design of the transfer system proved to be very complicated. Extremely large steel member sizes were required to take the large loads from the parking garage above. Also, the estimated cost of the proposed design was 24% more than the actual cost of the original design. Since cost is a very important factor to building owners, it should be considered in the final decision. Finally, the estimated project schedule was about twice as long as the original project.

INTRODUCTION

The Kettler Capitals Iceplex is the practice facility for the National Hockey League team, Washington Capitals. It is located at the Ballston Common Mall in Arlington, Virginia at the intersection of Glebe Road and Randolph Street. This 137,000 square foot facility was built on top an existing parking structure and houses two regulation sized ice rinks, corporate offices, a training facility, and a pro shop. At 60ft. above street level, the Kettler Capitals Iceplex is the home of the highest ice rink in the United States.

Design for the Iceplex began in 2000; however, this was the third time the Ballston parking garage had been expanded. The original facility, which dates back to the 1950s, was a five story cast-in-place concrete structure reinforced with mild steel. Then in the 1980s, the parking garage was expanded two more times. In 1981, a five story L-shaped addition was constructed of cast-in-place post-tensioned concrete. Then in 1986, the existing five level structure was topped with two more levels, one post-tensioned concrete and the other composite steel. See Figure 1 for a schematic phasing diagram of these additions.

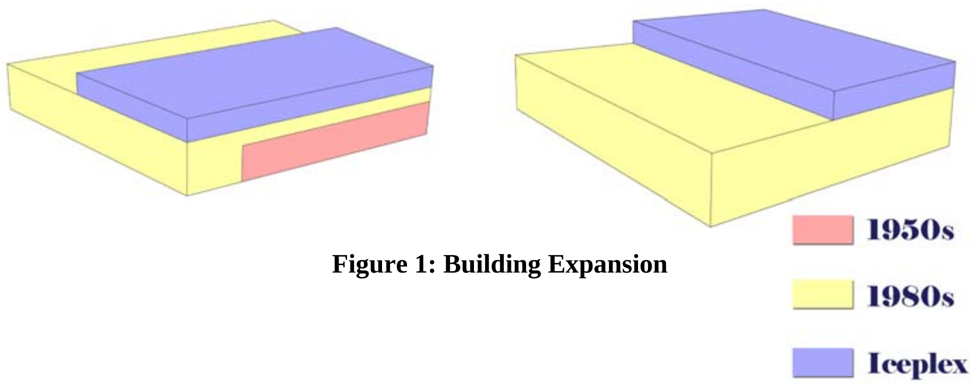


Figure 1: Building Expansion

There were several challenges when designing the Iceplex. The initial challenge was figuring out how to safely build an ice rink and roof weighing a total of 235 psf dead load plus 130 psf live load over an existing structure that was designed for a total expansion of 60 psf dead load and 50 psf live load. Another challenge was controlling deflection over the long 200ft. span of each ice rink. A consultant recommended that the deflection be as close to zero as possible in order to prevent the ice from cracking. Also, the need for large column-free spaces limited the locations where lateral members could be placed.

BUILDING BACKGROUND

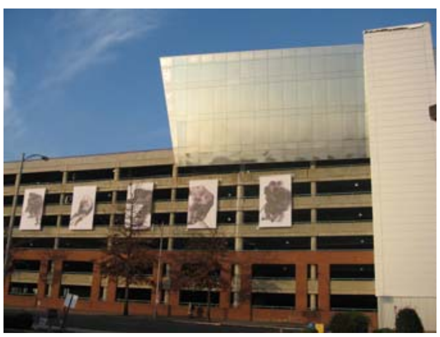
Architectural Design

The Kettler Capitals Iceplex is located on top of a parking structure at the Ballston Common Mall at the intersection of Glebe Road and Randolph Street in Arlington, Virginia. The Iceplex is connected to the mall through the parking structure and is directly linked to the Metro Orange Line.

The new rink building was constructed over the original 1951 parking garage. This garage was designed as five levels of cast-in-place concrete reinforced with mild steel with the additional capacity for two levels of vertical expansion. In the 1980s, two levels of additional parking were added. Finally in 2000, design for the eighth floor ice rinks and ninth floor corporate/training facility began. At 60ft. above grade, the Kettler Iceplex is proud to be the home of the highest rink above street level in the US.

The complex was designed to a LEED Certified level but never registered with the US Green Building Council (USGBC) in order to pursue the rating. Using a variety of recycled materials and having natural light penetrate into 90% of interior spaces are two features that make Kettler a green building.

Building Envelope



The façade of the parking structure consists of reinforced concrete and brick. Like typical parking garages, many openings in the façade are needed to ventilate the area from car exhaust. This means that the building envelope of this part of the building does not give any protection from the elements.

The building envelope of the rink and office space is made of metal paneling and glass curtain walls. The curtain wall is supported using metal studs and kickers. The wall uses 1” insulated glazing in order to obtain a sufficient thermal barrier.

The roof membrane consists of a concrete slab on metal deck using a fully adhered EPDM roof membrane. Long-span trusses were required to support the roof above the ice rinks.

Structural System

Since the Kettler Capitals Iceplex was constructed on top of an existing parking garage, there were many design challenges during the addition.

The actual load of the new Iceplex was about three and a half times that of the allowable expansion load. Consequently, reinforcing the existing structure was required. After testing the soils, it was determined that two footings needed to be expanded 3 ft. in one direction. The existing columns were analyzed for the additional load of the new structure. A total of 11 columns were wrapped in carbon fiber reinforcing. Also, all existing steel columns on levels 5 and 6 of the parking garage were encased in concrete.

Two expansion joints were used in the construction of the new Iceplex. One running in the north-south direction separates the 8th level of parking from the 8th level of the Iceplex which is where the ice rinks are located. The other expansion joint, running east-west, separates the ice rinks from the Capitals corporate offices and training facility.

It was important to limit deflection of the concrete slab supporting the rinks in order to prevent the ice from cracking. The structural engineer and the ice rink consultant compromised to limit the deflection to L/480. This slab was constructed from 3½" lightweight concrete over 3" 18 gage galvanized composite deck (total thickness = 6½") reinforced with #4 at 16"oc each way 2" below the slab. Supporting the slab are mostly composite W18x40s at 9'-0"oc spanning 30'-0". These W18s frame into larger steel composite beams which range from W21x50s to W36x150s. All shear studs are ¾" diameter by 4" long. Steel columns supporting the rinks range from W12x58s to W14x257s.

The need for long-span, column free spaces was critical in the design of the roof over the two ice rinks. The roof joists above the rinks are open web steel joists, 68DLH16. These joists have a depth of 68" and have the capacity to support large loads with extremely long spans. The span of these roof joists are 120ft. and are spaced at 5'-6"oc.

There is a mix of lateral resisting systems throughout the structure. The original parking structure that was built in the 1950s was constructed using a two-way slab system. During the 1980s expansions, moment frames were designed to resist lateral loads. Finally, during the construction of the new Iceplex, a mix of braced frames and moment connections were used.

Mechanical System

The mechanical system serving the main ice rinks consists of a desiccant based dehumidification system combined with a refrigerant based cooling system all packaged

in three separate roof mounted units. The purpose of these systems is to keep the air space that contacts the ice between 10 to 20 °F above the ice surface temperature. Since the ice surface does evaporate and contribute water vapor to the airspace, these units are also responsible for removing this moisture from the air.

A network of refrigerant piping running under the entire rink surface creates and maintains the temperature of the ice. The refrigerant used for this system is ammonia, which is cooled by industrial type chillers. The chiller room is located next to the ice melt pit in between the two ice rinks.

The locker rooms, bathrooms, and many of the utility rooms use various exhaust systems. The party rooms, faculty offices, and other remaining spaces are ventilated and air conditioned by four refrigerant based packaged constant volume rooftop units with extensive ductwork and air distribution systems.

The team offices, team locker rooms, coaches' rooms, and team training facilities are all ventilated and air conditioned by four refrigerant based packaged variable-air-volume (VAV) rooftop units. These units work in conjunction with VAV zone terminal units (VAV boxes) that vary the amount of cooling or heating into the zone they serve by varying the amount of air provided to these spaces. These VAV boxes act as a damper/valve and also include electric reheat. The VAV boxes operate based on a zone thermostat and the VAV rooftop unit is responsible for maintaining a constant supply air temperature and adequate airflow to the VAV boxes based on the demand.

Lighting/Electrical System

Electrical service consists of a 4000-ampere, 277/480 volt 3-phase, 4-wire, switchboard in the garage at level one. Service is extended to the Kettler Capitals Iceplex, at the new eighth level, through a distribution switchboard by bus ducts. Distribution throughout the facility is at 277/480 volts to several electric rooms located throughout the facility. If transformation is required to 120/208 volts, step-down transformers are provided at the electric rooms. The vast majority of lighting equipment is served at 277 volts. The majority of mechanical equipment is served at 277/480 volts. All other equipment that is not served at 277/480 volts is served at 120/208 volts. All branch circuit wiring is installed in conduit and Type AC cable was permitted when concealed. The facility has emergency power to support life safety systems and a fire pump. The majority of the lighting was required to be the same color temperature, 3500K. Although several manufacturers were used, most lighting fixtures were manufactured by Lithonia. Mounting types varied, but recessed and surface mounting were most widely used.

Construction Management

Sigal Construction Corporation was the project manager for the Iceplex. They acted as under a design-bid-build delivery method. The staging area for the new 8th and 9th levels was located on the 7th level of the existing parking garage.

Since the existing structure needed to be reinforced before constructing the new Iceplex, the architect prescribed a 6-phase plan starting in February 2005 and ending in June 2006.

- Phase 1: Expand footings
- Phase 2: Column Bolstering
- Phase 3: Underground utility work
- Phase 4: Crane installed on level 7 to erect steel for ice rinks
- Phase 5: Complete structural work for new stairs/elevators
- Phase 6: Complete construction within levels 8 and 9

Fire Protection System

The fire protection system is a standard wet system complying with NFPA sections 13, 14 and 24 and meets the requirements for high rise building structures. Based on the high rise requirements, and those of Arlington County, Virginia, 100psi must be maintained at the top of the standpipes. Thus a fire pump was required and is located at the ground level of the structure. Water was distributed to the standpipes using a fully charged wet system in unconditioned space and is protected from freezing by means of heat trace and insulation. The fire pump and heat trace are fed from an emergency generator. The heat trace system is fully alarmed to notify building engineers of failure in any portion of the system.

Project Team

Owner/Developer: Lincoln Holdings, Arlington County / Jones Lang Lasalle

Architect: Architecture, Inc.

Civil Engineer: VIKA

Structural Engineer: Rathgeber Goss Assoc.

MEP Engineer: KTA Group

Construction Manager: Sigal Construction

THE EXISTING STRUCTURAL DESIGN

Reinforcing the Existing Parking Structure

As previously mentioned, the actual load of the new Iceplex was about three and a half times that of the allowable expansion load of the existing parking structure. Inevitably, the existing parking structure needed to be reinforced before constructing the new addition.

Foundation

The structural engineer of record, Rathgeber/Goss Associates of Rockville, MD, recommended testing the soil as a first step in the reinforcing process. Engineering Consulting Services, Ltd. was hired to complete the testing. Test results showed that the allowable bearing pressure of the soil was 10,000 psf which was significantly higher than the 6,000 psf used in the original construction. Based on this information and the column loads from the new construction, it was concluded that only two footings needed to be expanded. These footings, along column line 9 (see Figure 2), were expanded 3’-0” in one direction. No increase in footing depth was necessary.

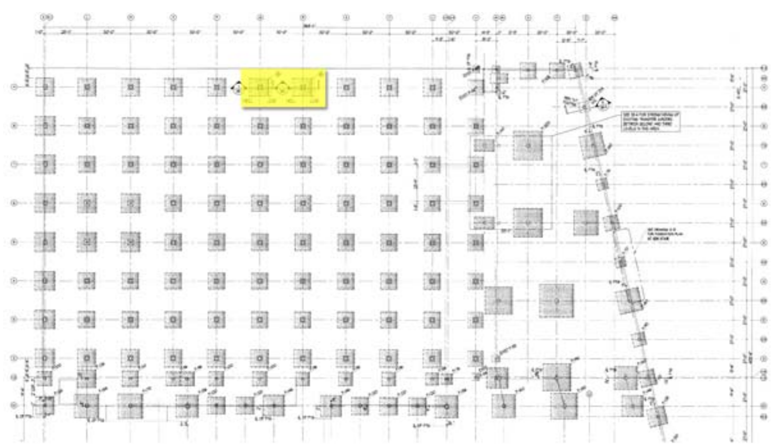


Figure 2: Footing Expansion Locations

Columns

It was also recommended by Rathgeber/Goss that the existing concrete columns be core tested in order to analyze their compressive strength. Engineering Consulting Services, Ltd. was hired to perform these tests as well. However, due to the high density of reinforcing steel in the columns, testable cores were unobtainable. Therefore, a series of Windsor Probe tests were performed throughout the structure in lieu of the originally proposed concrete coring.

A total of nine Windsor Probe tests were performed throughout the existing parking structure. Five tests were located on the first floor, four on the fourth floor, and two on the sixth floor. ECS attempted to concentrate these tests primarily in locations where column loads would increase the greatest with the vertical expansion. After completing the tests, it was recommended that a compressive strength of 5,000 psi be assumed for the existing concrete columns. Since the original concrete strength was assumed to be 3,000 psi, this showed that the concrete had gained significant strength over time.

Based on these results, the columns needing additional reinforcement were determined. A total of 11 columns on levels 3, 4, 5, and 6 were wrapped in carbon fiber reinforcing. These columns are shown in red in Figure 3. Gardner James Engineering, Inc. was commissioned to design this additional reinforcement. GJ chose a product called Aquawrap from Structural Composites, Inc. for the carbon fiber reinforcing. This allowed the ultimate axial load in the columns to be greater than the nominal capacity by a factor of 1.2.

In addition to the carbon fiber reinforcement, all existing steel columns in the parking structure (levels 5 and 6) were encased in concrete in order to provide the additional required capacity. All columns shaded in blue in Figure 3 were reinforced. See Figure 4 for a bolstering detail.

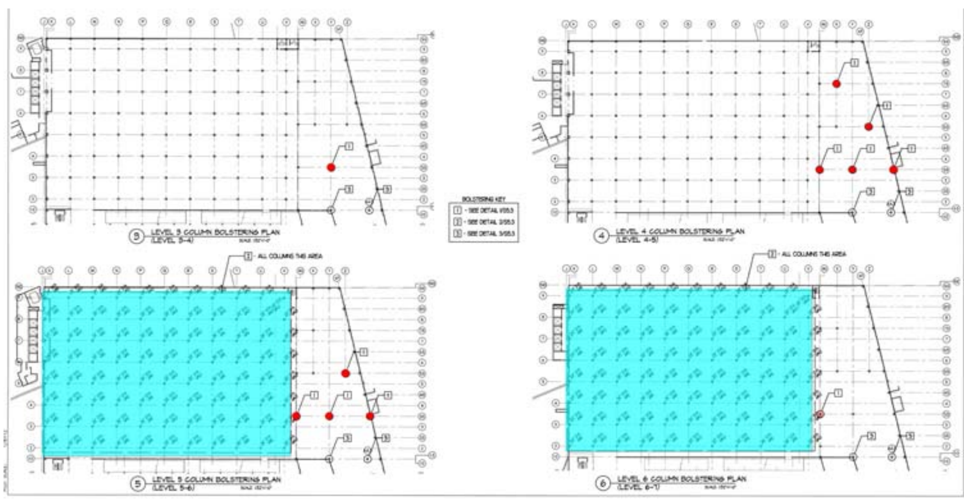


Figure 3: Column Reinforcing Locations

Figure 4: Column Bolstering Detail

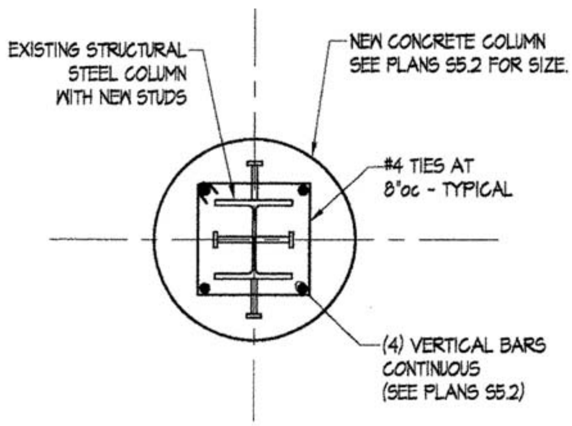


Figure 5: Column Bolstering. ***Courtesy: RGA***

The Gravity Framing System

There were two expansion joints used in the construction of the new Iceplex, one running in the north-south direction and the other in the east-west direction. See Figure 6 for the locations of these joints. Expansion joint A, running north-south, separates the 8th floor parking structure from the 8th floor of the Iceplex. Expansion Joint B, running east-west, separates the ice rinks from team facility including the team offices and locker rooms. Both these joints span vertically the entire height of the building.

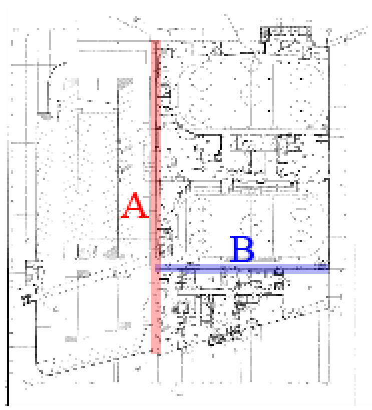
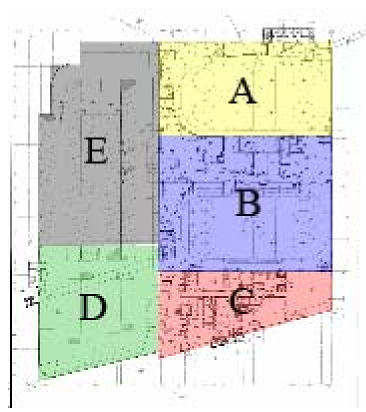


Figure 6: Location of Expansion Joints



Keyplan

The first five levels of Areas A and B are constructed of mildly reinforced cast-in-place concrete consisting of 26”-28” diameter columns. The two-way slab is 10½” thick with 5¼ ” drop panels and column capitals. Levels six and seven are constructed of 27’-0” x 30’-0” composite steel bays with W16x26s spanning the 27’ direction and W24x55s spanning the 30’ direction. Levels eight and nine of the Iceplex also consist of composite steel framing with the same 27'-0" x 30'-0" bay. Figure 7 shows a typical bay framing of level eight supporting the ice rinks.

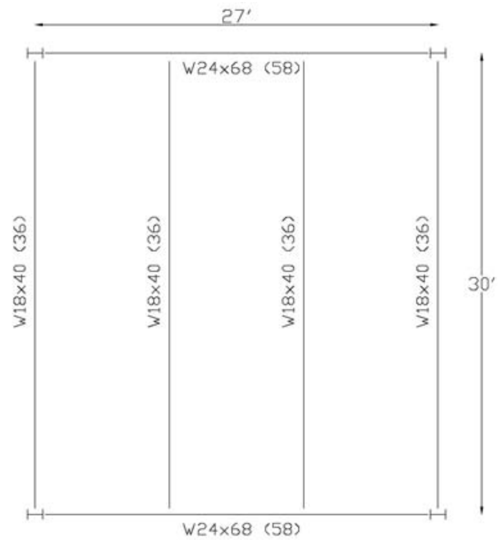


Figure 7: Enlarged Typical Framing Plan

The Lateral Framing System

The lateral system of Areas A and B is somewhat complicated due to the several expansions the structure has encountered over the years and the various materials that were used.

The first five levels of concrete were cast monolithically creating continuous concrete moment frames in each direction throughout the building footprint. In general, this lateral system has proven very stiff and efficient for resisting lateral loads but creates potential problems in seismic regions because of its heavy weight.

When the structure was expanded both horizontally and vertically in the 1980s, reinforcement of the lateral system was needed. The original lateral system is shown in yellow in Figure 8. Areas A and B on levels 7 and 8 were framed using composite steel with moment connections. There are ten moment frames spanning the east-west direction along the exterior of the building. Two frames spanning the north-south direction run the entire width of the building at both sides of the structure.

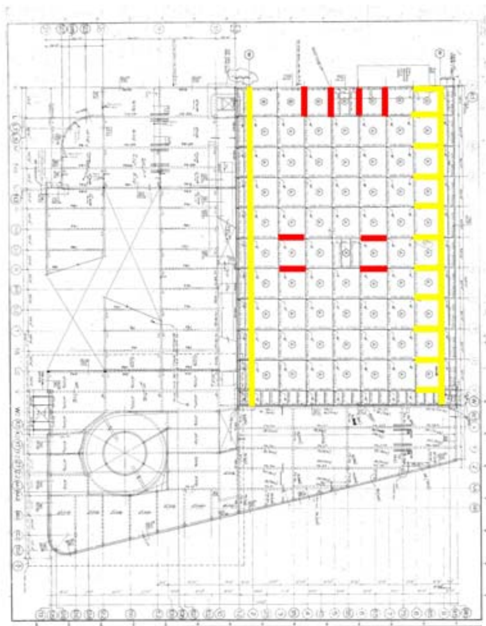


Figure 8: Level 7 Lateral System

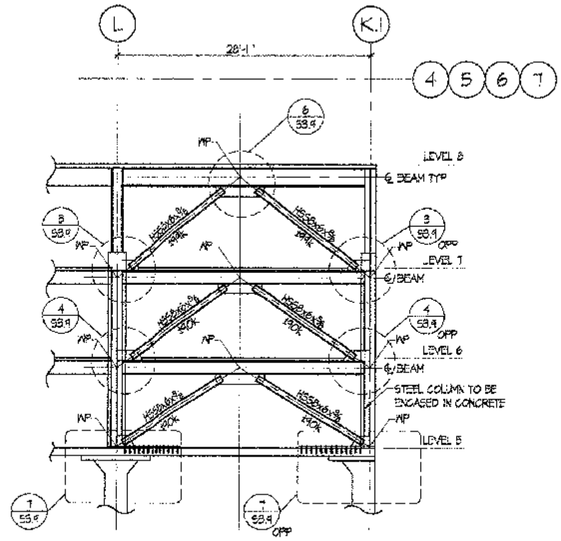


Figure 9: Braced Frame Detail

Finally, when the Iceplex was added onto the parking structure, a mix of braced frames and moment connections were used. Eight braced frames were constructed on the 7th level reinforcing the existing structure for additional lateral forces. HSS8x6x3/8 tubes were used for all cross bracing. These frames are shown in red in Figure 8 and a detail of these braced frames is shown in Figure 9. On the 8th level, there are a total of eight braced frames, four in each direction. These frames use the same tube sections and are shown in blue in Figure 10. Eight moment frames were constructed and were spaced evenly throughout with the exception of the voided areas from the ice rinks. These are shown in green in Figure 10.

All lateral resisting members on the 9th level in this area are located in Area 9B. Seven moment frames span the north-south direction and four span the east-west direction. W24s and W33s are typical of the moment frames on the 9th level. Figure 11 shows the location of all lateral resisting frames in Area 9B.

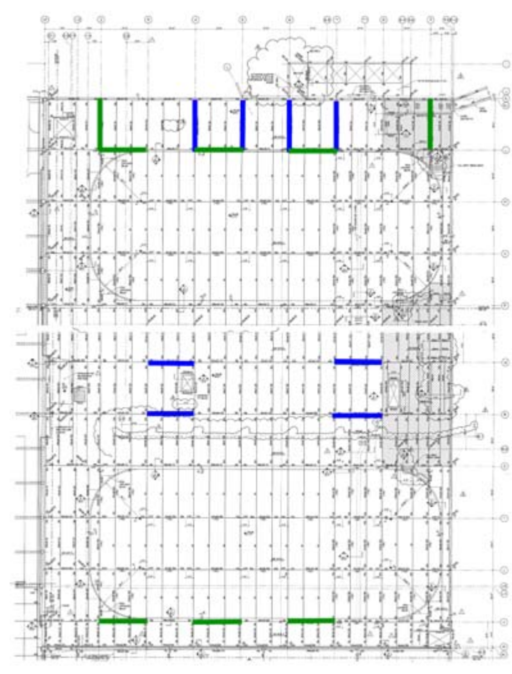


Figure 10: Level 8 Lateral System

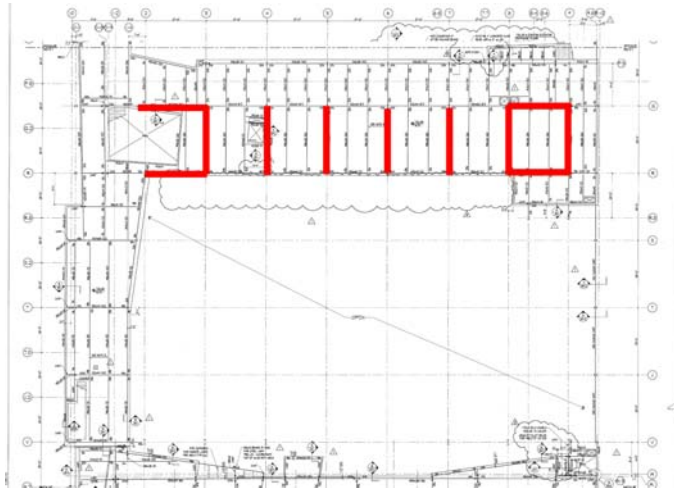


Figure 11: Level 9 Lateral System

The lateral resisting system of Areas A and B may be difficult to understand in 2-dimensions. Figure 12 shows the entire lateral system in 3D which may help to explain how the various systems work together to resist wind and seismic loads.

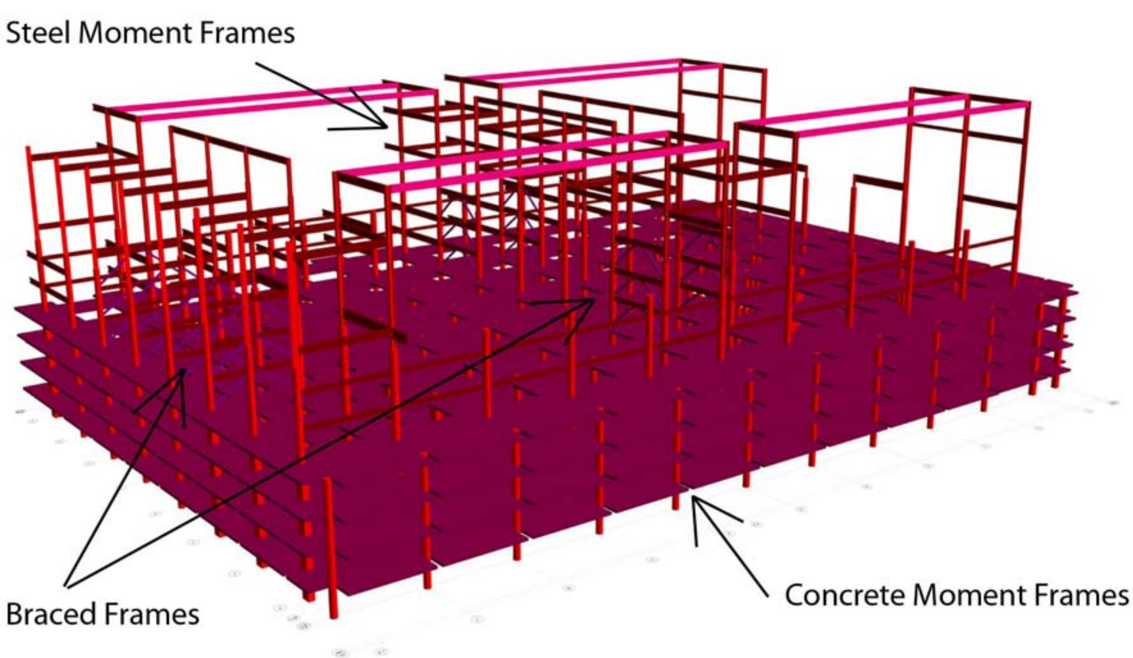


Figure 12: 3D Lateral Resisting System

PROBLEM STATEMENT AND PROPOSED SOLUTION

As previously stated, when the Iceplex was added onto the existing Ballston Mall parking garage, reinforcing the structure was required. Two footings were expanded, most columns were strengthened, and the lateral system needed to be reinforced in order to resist increased lateral loads. This proved to be the most complex part of the design. Also, minimizing deflection was crucial for the ice rinks which are located over 60ft. above grade.

There is a possibility that reinforcing the existing structure was not the most efficient and economical solution to the expansion. Instead, demolishing the existing parking garage and constructing the Iceplex from scratch may have simplified the project. This would eliminate the need for reinforcement and would simplify the lateral framing system.

Redesigning the Iceplex and parking structure would allow the two ice rinks to be relocated to the first floor. The rinks could then be supported using a slab-on-grade, therefore minimizing deflection issues. The parking garage will be built as a separate structure above the Iceplex creating the need for a large transfer system. These trusses will be used to support the parking structure and span above the rinks. The garage will be framed using a precast concrete system. The lateral system will mainly consist of shear walls.

Since the building will be built from scratch, a civil/site study will be completed. This will analyze the site and nearby vehicular traffic in order to chose the most efficient locations for garage entrances and exits. The architectural layout will also be examined and any needed changes will be designed in order to accommodate any adjustments made to the site layout. An in-depth project cost will be calculated for this proposed design and a corresponding schedule will be completed. Taking all this into consideration, a final recommendation can be made about whether or not the proposed design is in fact a feasible and economic solution.

CIVIL/SITE DESIGN

Since the entire building is to be redesigned from the ground up, the locations of existing garage entrances and exits should be evaluated for their efficiency. Currently, the parking structure, servicing both the Iceplex and the Ballston Common Mall, has two entrances and two exits. The main access point, from N. Glebe Rd., has two entrance lanes and three exit lanes. A secondary access point, from N. Randolph St., has two entrance lanes and two exit lanes.

According to *The Dimensions of Parking*, a publication from the Urban Land Institute and the National Parking Association, entrances should be located on high-volume streets in order to provide easy site access from nearby interstates and other major vehicular flow patterns. Exists should be located on low-volume streets to minimize street traffic conflicts with vehicles existing the parking garage. In order to effectively analyze the proper locations of garage entrances and exits, traffic count data was obtained from the Virginia Department of Transportation website. North Glebe Rd. has an AADT (Annual Average Daily Traffic) of 33,000 with a directional factor of 57%. This means that every day approximately 33,000 vehicles travel this stretch of road, 57% of which occur during rush hours. North Randolph St. has an AADT of 11,000 and a directional factor of 64%. Based on this information, N. Glebe Rd. has a much higher vehicular flow volume than that of N. Randolph St. From this data, it was concluded that vehicles should enter the garage from N. Glebe Rd. and exit from N. Randolph St.

It can be seen from the aerial map shown below that there is a major intersection at the existing main entrance to the garage. *The Dimensions of Parking* states that all entrances should be located at least 75 to 100ft. from any corner intersection; therefore the entrance must remain directly across from N. Carlin Springs Rd. This will allow the existing traffic signal light to continue to control vehicles entering the garage.

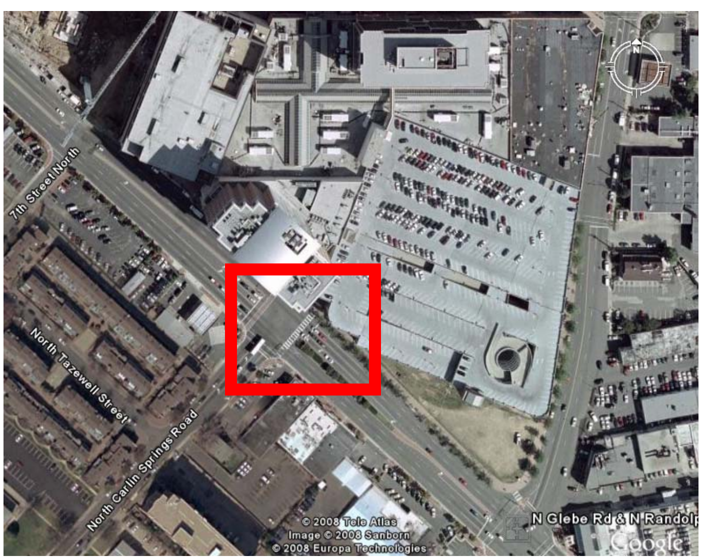


Figure 13: Aerial View Showing Existing Entrance Location

Based on this research combined with recommendations from professional engineers at The Pennsylvania State University, it was concluded that the existing entrances and exits were properly located during the original design. The garage entrance should be located at the intersection of N. Glebe Rd. and N. Carlin Springs Rd. and the garage exit should remain at the existing location on N. Randolph St.

Although *The Dimensions of Parking* recommends having more than one entrance for garages holding more than 500 vehicles, Ryan Seacrist, civil engineer at Penn State, advised that one entrance location will be sufficient as long as an adequate number of entry lanes and gates are provided. The redesign calls for three entry lanes and three gates which should be adequate.

Vehicle stacking must also be considered when designing a large parking structure. It is recommended that entry gates be located at least two car lengths, or 20ft. from the roadway. This will prevent roadway traffic from backing up while vehicles entering the garage receive their parking ticket at the gate. The proposed design has a minimum stacking distance of 101ft. and utilizes a deceleration lane for traffic entering the garage from northbound Glebe Rd. This will considerably reduce the possibility of traffic backing up onto the roadway, thus, decreasing the risks of traffic hazards. To avoid traffic congestion while exiting the parking garage, the cashier's booths are located 97ft. from the nearest parking aisle and 145ft. from Randolph St.

Finalized parking garage entrance and exit designs are shown in the enlarged plans shown below.

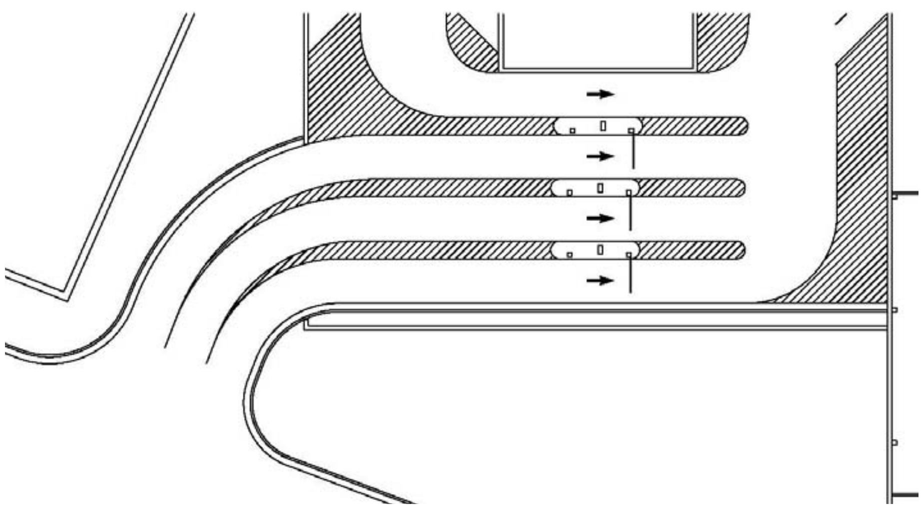


Figure 14: Garage Entrance Detail

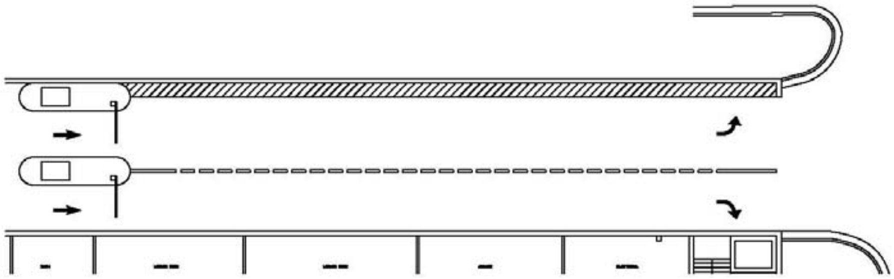


Figure 15: Garage Exit Detail

Another consideration from a civil site perspective is the building's main entrance and logistically accessing this location. An entry location for occupants not using the parking garage was determined to be both necessary and convenient. Based on the traffic count data, it was concluded that this entrance should be located off Randolph St. This will allow a drop off and pick up loop to be located off a low-volume roadway. This entrance will prove convenient for parents dropping off their children for events such as hockey practice and birthday parties. This roadway can also serve as an emergency fire lane if needed. This loop can be seen in architectural site plan shown below.

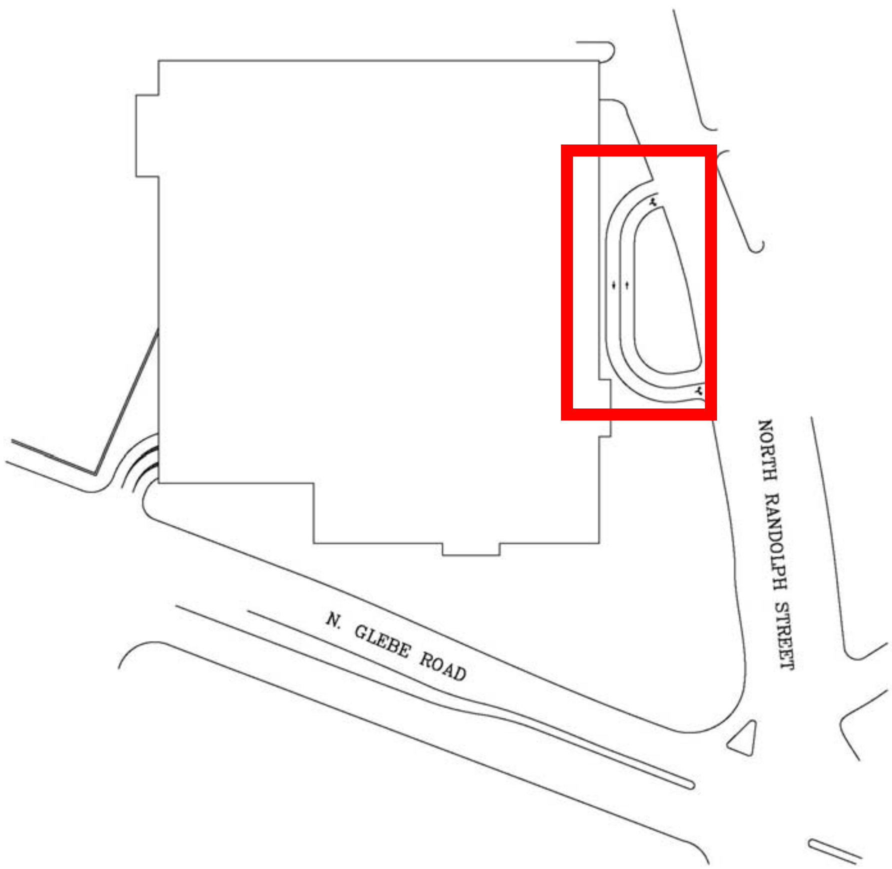


Figure 16: Architectural Site Plan

Green design is becoming more and more prominent in today's society. The amount of impervious area on a site should be minimized in order to have as little environmental impact as possible. Also, better storm water management is created on green sites. With the proposed design, the building footprint was reduced by 14.6% from the original design. This can be seen in Figure 17. This undeveloped land can serve as a large staging area for the contractor during construction. Then once construction is complete, it can be transformed into an attractively landscaped area. This corner has potential to become a prominent landmark in Arlington County. One design possibility could be to install a fountain surrounded by a park-like seating area. A large showpiece of the Capitals logo could be placed in the center of the fountain to symbolize the hockey team's unity with the county.

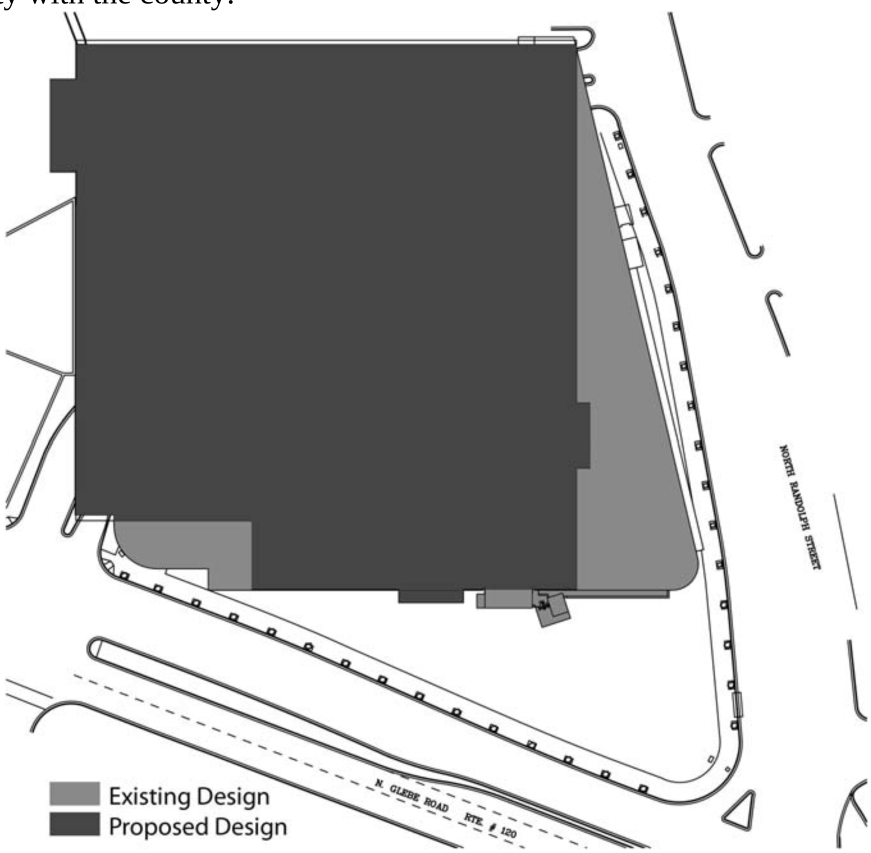


Figure 17: Impervious Area

ARCHITECTURAL DESIGN

The major architectural design change made to the Capitals' practice facility was to move the Iceplex from where it exists today on the 8th level down to ground level. Instead of the Iceplex being located on top of the parking garage, it will be relocated beneath the parking structure creating easy access from the street.

The Iceplex

The first step in the architectural redesign was to create a square footage program of the existing spaces. This will ensure that all rooms and facilities remain close to the same size as required by the owner in the original design. The Iceplex was divided into four distinct areas based on who will occupy the space: general admission/community areas, Capitals team access, visiting team access, and Capitals private offices. These areas are color coded in Figure 18.

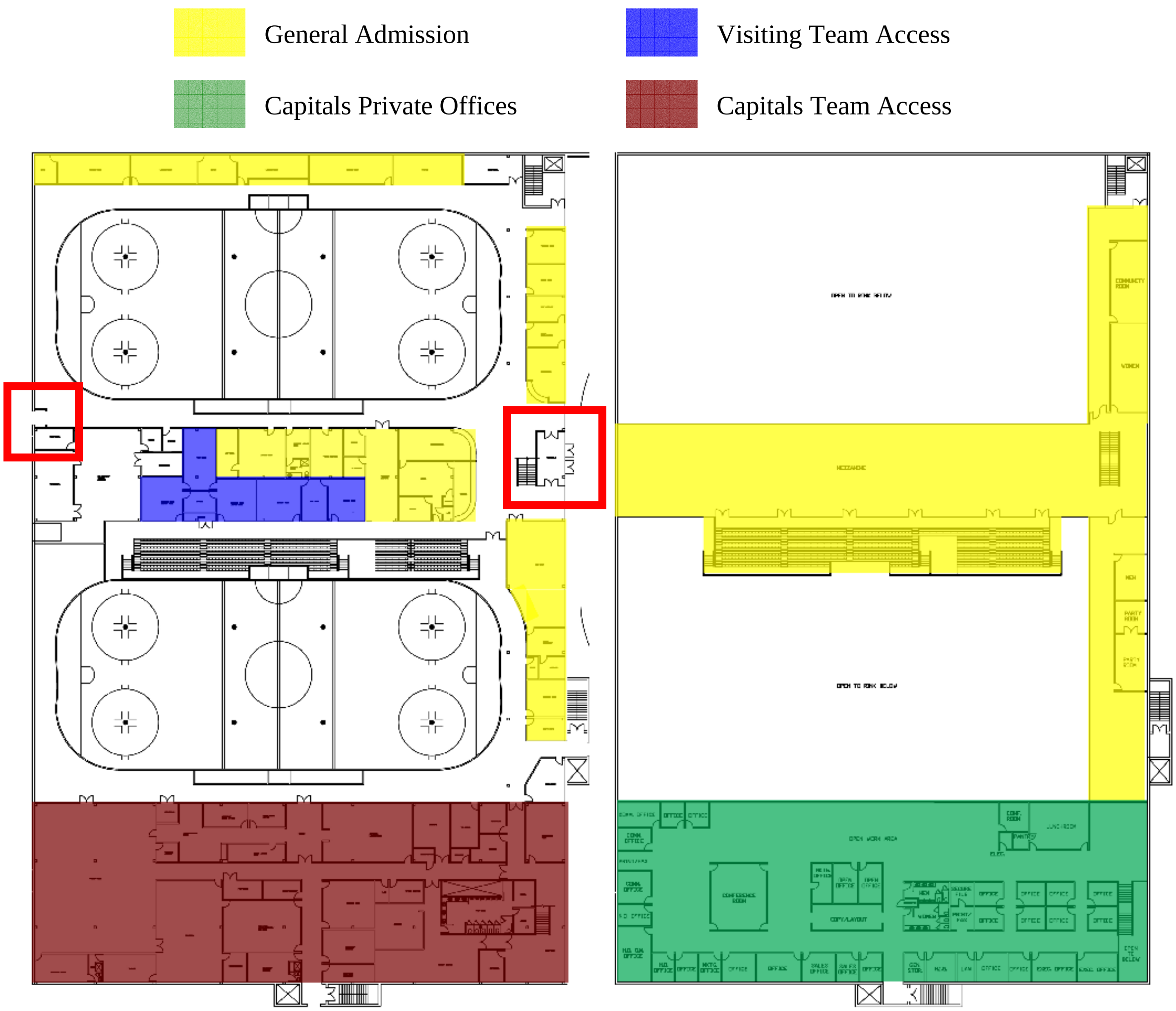


Figure 18: Iceplex Areas

Based on the garage access locations discussed previously in the civil/site section, the best orientation of the Iceplex was determined for the new design. Since the parking entrance and exit were already established, the southwest and northeast corners of the building must be reserved for the parking garage.

The main building entrance was located along the east side of the Iceplex creating easy access from N. Randolph St. This entrance will integrate with the drop off loop discussed previously. Based on these predetermined designs, the ice rinks were ideally located as shown in Figure 18. This figure also shows the proposed layout of the four main occupancy uses of the Iceplex.

A secondary entrance was located to access the parking garage. Since this is not a main entrance, a large greeting area was not provided. It was noted that this is not the most convenient situation for Capitals fans coming to watch pre-season practice and entering through the parking garage. However, for the majority of the year the main entrance will be more utilized with little league hockey practice and birthday parties. The two building entrances are outlined in red in Figure 18.

This conceptual layout and existing square footage program were then used to layout the individual rooms. As mentioned earlier, all spaces should be approximately the same size as in the original design. The spreadsheets below show the square footage areas of all spaces for both the existing and proposed designs. The difference in area size was calculated for comparison. As anticipated, room sizes were not exact, however most spaces are within a reasonable margin. The only major differences were in the visiting team locker rooms. With more time, these areas could be reanalyzed to account for the additional required space.

Table 1: Capitals Team Square Footage Program

Capitals Team Square Footage Program			
Room	Original Area	Proposed Area	Difference
Weight Room	3402	3678	276
Strength Coach's Office	180	297	117
Trainer's Office	224	294	70
Trainer's Bath	108	78	30
Training	1400	1350	50
Exam/Message	150	137	13
Equipment Manager	250	260	10
EM's Bath	104	44	60
Hydro Therapy	875	814	61
Stick Workshop	160	145	15
Pump Room	140	148	8
Training Storage	80	85	5

Weight Storage	110	82	28
Video Room	392	379	13
Asst. Coaches	216	215	1
Head Coach	180	175	5
Coach's Locker Room	300	212	88
Sticks	350	424	74
Capital's Locker Room	1204	1060	144
Lecture Room	486	495	9
Equipment Storage	288	345	57
Equipment Workroom	180	184	4
Player Lounge	1025	837	188
Changing	416	430	14
Office Storage	160	150	10
Player Locker Room	625	752	127
Steam Room	96	109	13
Sauna	130	117	13
Laundry	340	372	32
Housekeeping	54	57	3
Electric Room	60	51	9
Copy/Mail	189	160	29
Conference Room	484	464	20
Reception	300	574	274
Video Editing	156	162	6
Closing	156	174	18
TV Studio	210	250	40

Table 2: General Admission Square Footage Program

General Admission Square Footage Program			
Room	**Original Area**	**Proposed Area**	**Difference**
Office	130	119	11
Electric	90	93	3
Vending/Lockers	540	591	51
Skate Rental	665	450	215
Administration	345	380	35
Office	168	156	12
Tickets	280	232	48
Storage	25	18	7
Main Lobby	1500	1475	25
Snack Bar	440	400	40
Dry Storage	210	196	14
Food Preparation	200	187	13

Mech/Zamboni/ Ice Machine/ Ice Melt Pit/Elec	2304	2102	202
Mechanical	209	192	17
Office	90	90	0
Upper Concourse	10000	9919	81
Storage	90	92	2
Locker Room	418	456	38
Wet Room	198	211	13
Referee Locker Room	140	139	1
Referee Bath	80	89	9
Figure Skating	308	323	15
Ice Machine/Rink Storage	510	0	510
Lobby/Elevator Lobby	1000	0	1000
Bathroom	198	130	68
Locker Room	486	423	63
Electric	297	351	54
Arcade	342	419	77
Woman Bath	270	255	15
Men Bath	225	230	5
Woman Bath	680	660	20
Mezzanine	1100	1228	128
Community Room	629	602	27
Media Room	300	308	8
Men Bath	176	170	6
Woman Bath	400	281	119
Music	50	50	0
Storage	130	115	15
Skate Workshop	220	214	6
Pro-Shop	1320	1145	175
Mezzanine	1512	2358	846
Party Room	390	370	20
Party Room	225	210	15
Men Bath	300	297	3
Bleachers	4867	3993	874

Table 3: Visiting Team Square Footage Program

Visiting Team Square Footage Program			
Room	**Original Area**	**Proposed Area**	**Difference**
Locker Room	608	360	248
Office	165	134	31
Locker Room	510	366	144
Storage	75	51	24
Locker Room	540	366	174
Wet Room	360	216	144
Locker Room	400	314	86

Table 4: Capitals Private Offices Square Footage Program

Capitals Private Offices Square Footage Program			
Room	**Original Area**	**Proposed Area**	**Difference**
Balcony	144	0	144
Overlooking Area	2808	3503	695
General Offices	3207	3540	333
Print/Fax	110	136	26
NO Office	150	152	2
GM Office	400	400	0
HO Office	130	130	0
Marketing	130	130	0
Sales Office	195	195	0
G. Sales Office	117	130	13
MIG	169	176	7
LAN	130	104	26
Executive Office	260	260	0
Executive Office	260	234	26
Lunch Room	567	557	10
Waiting Room	260	930	670
Conference Room	169	169	0
Storage	32	52	20
Electric	45	41	4
Secure File	143	110	33
Print/Fax	110	110	0
Men Bath	156	167	11

Women Bath	234	227	7
Housekeeping	24	21	3
Copy/Layout	390	351	39
Marketing Office	162	162	0
Conference Room	740	750	10

In any sporting venue, floor to ceiling height is very important in the design. The distance from the ice rink surface to the bottom of the overhead truss supporting the parking structure is 26ft. This should allow for enough room to hang the “jumbo-tron” from the above structure and to avoid a claustrophobic feeling for skaters on the ice. This will give a floor to floor height of 15ft. for level one and 11ft. for level two of the two-story section of the Iceplex.

A unique feature of the new floorplan will now be pointed out. The zamboni storage area, which services both ice rinks, has direct access to the parking garage through a large sectional overhead door. This will allow easy transportation of the zamboni if it ever needs to leave the Iceplex facility for any maintenance issues. This feature was not available in the existing design and is shown in detail in Figure 19.

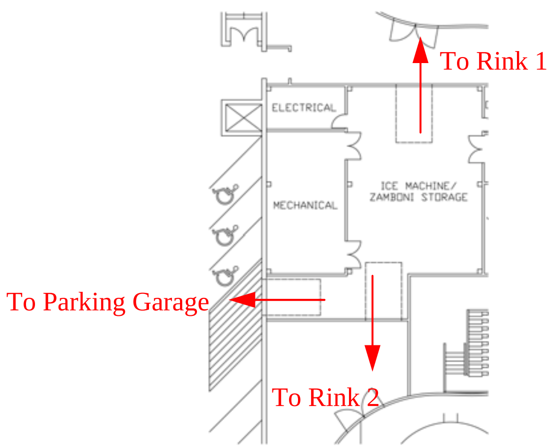


Figure 19: Zamboni Storage and Access

The Parking Garage

The International Building Code 2006 states that the minimum clear distance in a parking structure is 7’-0”. However, *The Dimensions of Parking* recommends that a clear distance of 7’-6” provides a more spacious feeling. The proposed design calls for a floor to floor height of 11ft. Assuming a 2’-6” depth for structure, this leaves a clear distance of 8’-6” which is more than the recommended distance.

Deciding on an appropriate vertical transportation route proved to be the most complicated part of the parking garage architectural design. There are many options for laying out ramps and flat parking aisles. After careful consideration, a three-bay double helix ramp with one-way traffic was chosen. A diagram showing this vertical route is shown below in Figure 20. The only logical location for these ramps was at the western side of the building. This will allow for flat parking levels above the Iceplex. Per IBC 2006, vehicle ramps that are used for vertical circulation *and* parking shall not exceed a slope of 1:15 (6.67%). The ramps used in this design run a vertical distance of 15ft. on the first tier and 11ft. on the every other tier. They all run a horizontal distance of 250ft. creating a 6% or 4.4% slope. The proposed design of the parking garage calls for a total of nine levels of parking including the three partial levels on the ramps located adjacent to the Iceplex.

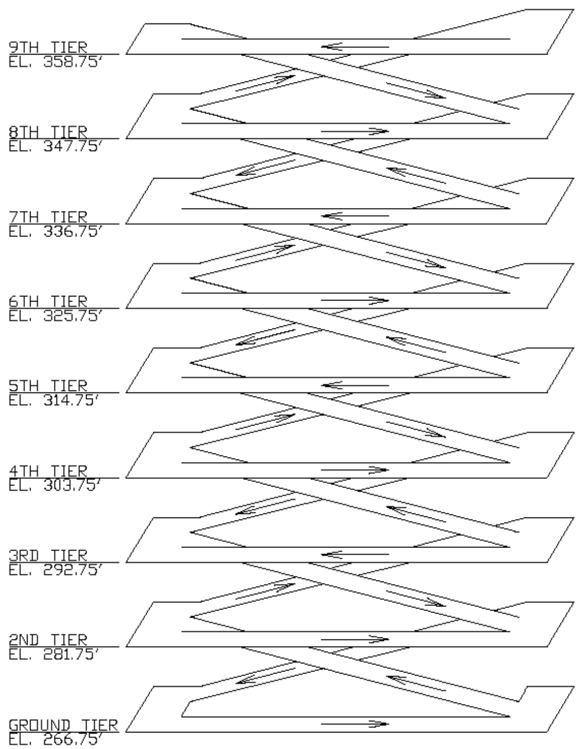


Figure 20: Vertical Transportation Isometric

One possible concern with this design is the recognizable availability of spaces. A car coming up the ramp may not know if there are any available parking spots located in the far rows of the garage. To eliminate this problem, a Smart Parking Garage System could be used. This system uses occupancy sensors to know where available parking spaces are located. The system can then inform drivers of the number of available stalls and direct them to the appropriate area. This kind of system should avoid driver's frustration in trying endlessly to find a parking space in a crowded parking garage. The two images shown below illustrate this Smart System. Figure 21 shows the proposed locations of the directional signage for the Smart System.

Beaver Ave. Parking Garage. State College, PA

BWI Airport. ***Courtesy IEEE.org***

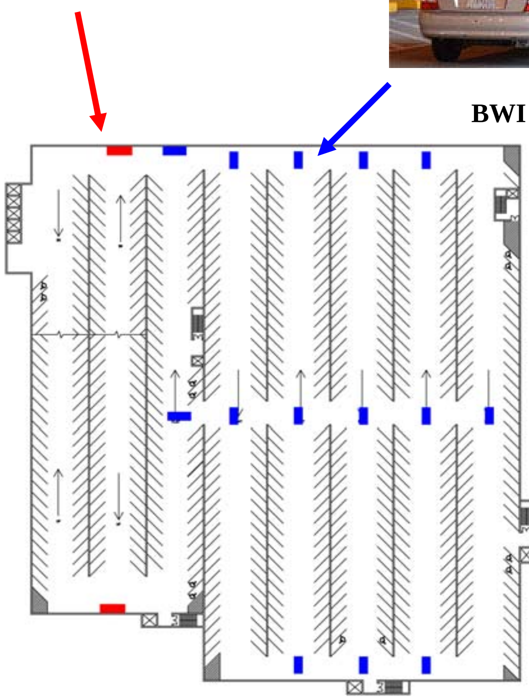


Figure 21: Directional Signage Locations

One necessary thing taken into consideration when designing the new parking garage was the number of parking stalls. Since this parking structure also services a large mall, it was important not to decrease the capacity of the garage. The structure currently carries approximately 2800 vehicles. With the new design, the parking garage has over 3800 stalls, which is a 37% increase over the existing design. Depending on mall management, these additional stalls could either be utilized, or one level of the garage could be removed creating about 3300 stalls.

The number of accessible parking spaces is regulated by the building code. According to IBC 2006 Section 1106, there must be 20 ADA spaces plus one for each 100 over 1000 total spaces. This means that for the parking garage holding 3800 vehicles, there must be 20 + (3800-1000)/100 = 48 ADA spaces. These spaces must be located along the shortest accessible route of travel from adjacent parking to an accessible building entrance or exit. Therefore, the ADA spaces in the proposed design will be located near all elevators and the Iceplex entrance.

Means of Egress

When designing a building with a large footprint, it is important to consider means of egress for building occupants. According to IBC 2006 Section 1019.1, for a building with an occupancy load of more than 1000, at least four exits per story must be provided. It also states that parking structures must have a minimum of two exits per parking tier excluding the vehicle ramps. It can be seen from Figure 22 that each level has at least six exits, including the elevator lobby connecting to the mall.

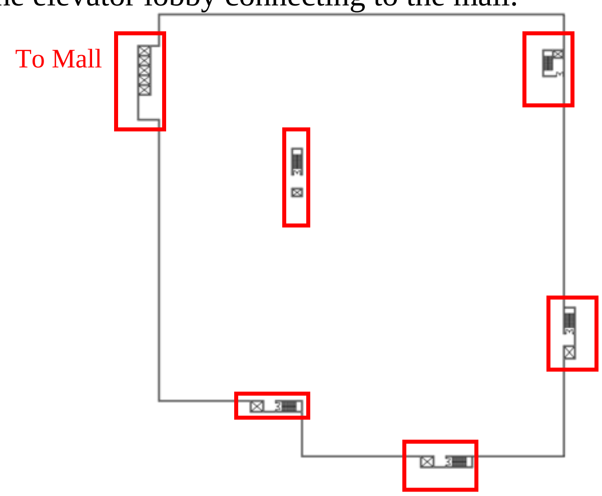


Figure 22: Exit Locations

IBC 2006 also sets limitations on exit access travel distance. According to Table 1016.1, the maximum length of exit access travel is as follows: 250ft. for ice rinks and bleachers (Occupancy Group A); 300ft. for offices (Occupancy Group B); and 300ft. for the parking garage (Occupancy Group S-2). These distances assume that the interior spaces of the Iceplex are sprinklered and the parking garage is not. Figure 23 shows the maximum travel distance for the most remote locations of the building.

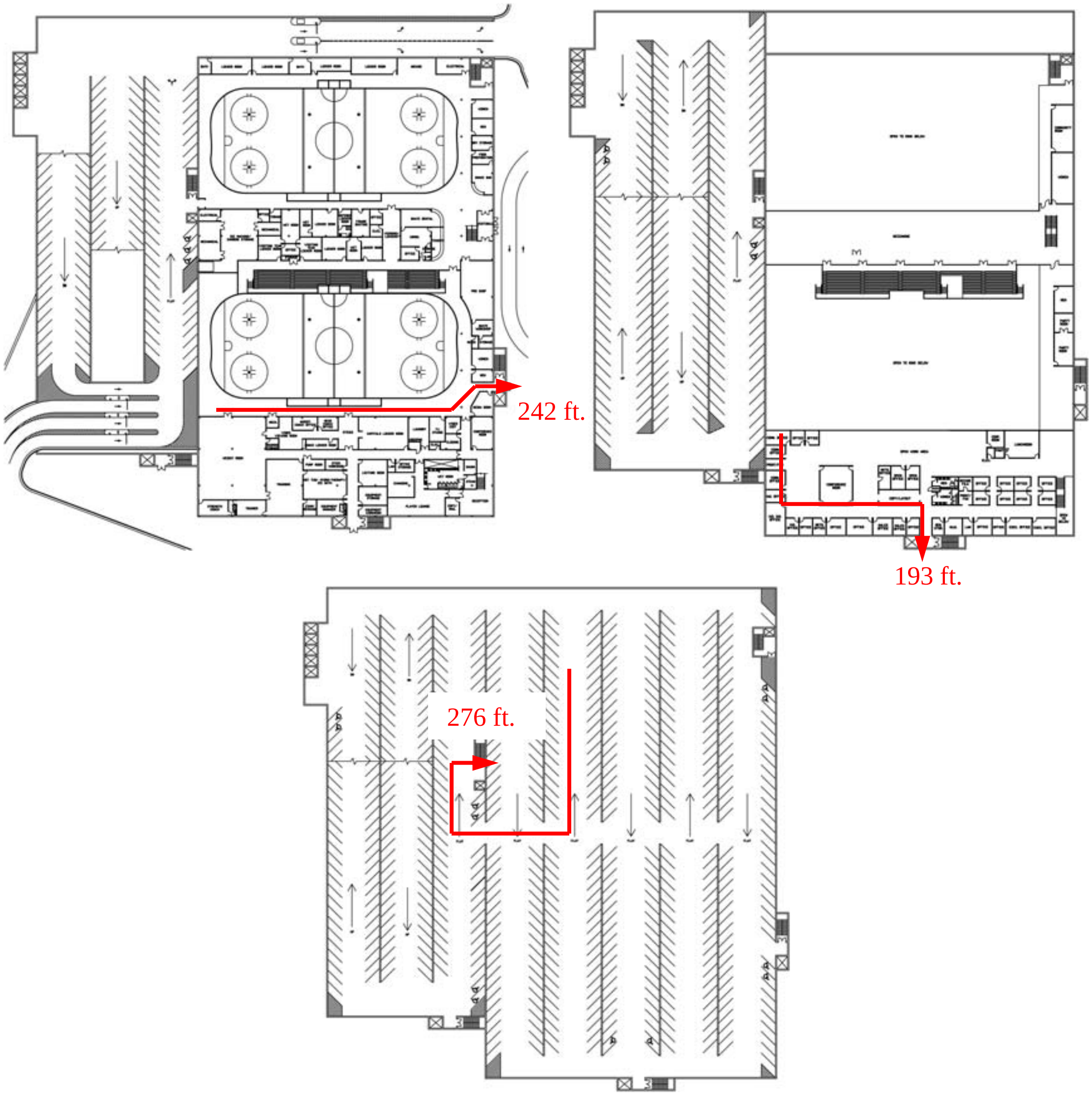


Figure 23: Maximum Egress Distances

Final Design

The finalized floorplans for the first two levels of the Iceplex and the parking garage layout are shown below. The maximum building dimensions are 372ft. x 408ft.

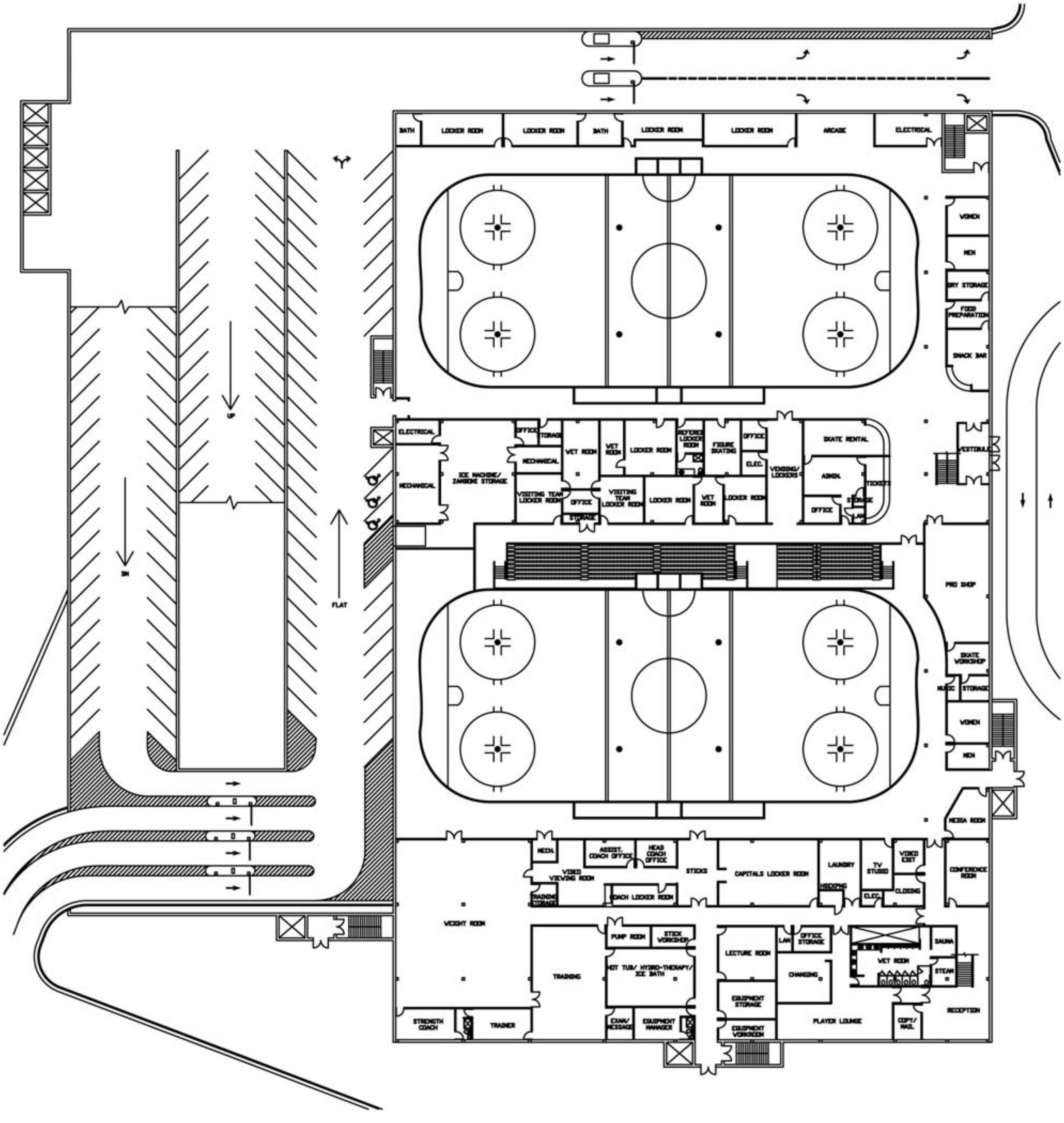


Figure 24: First Floorplan

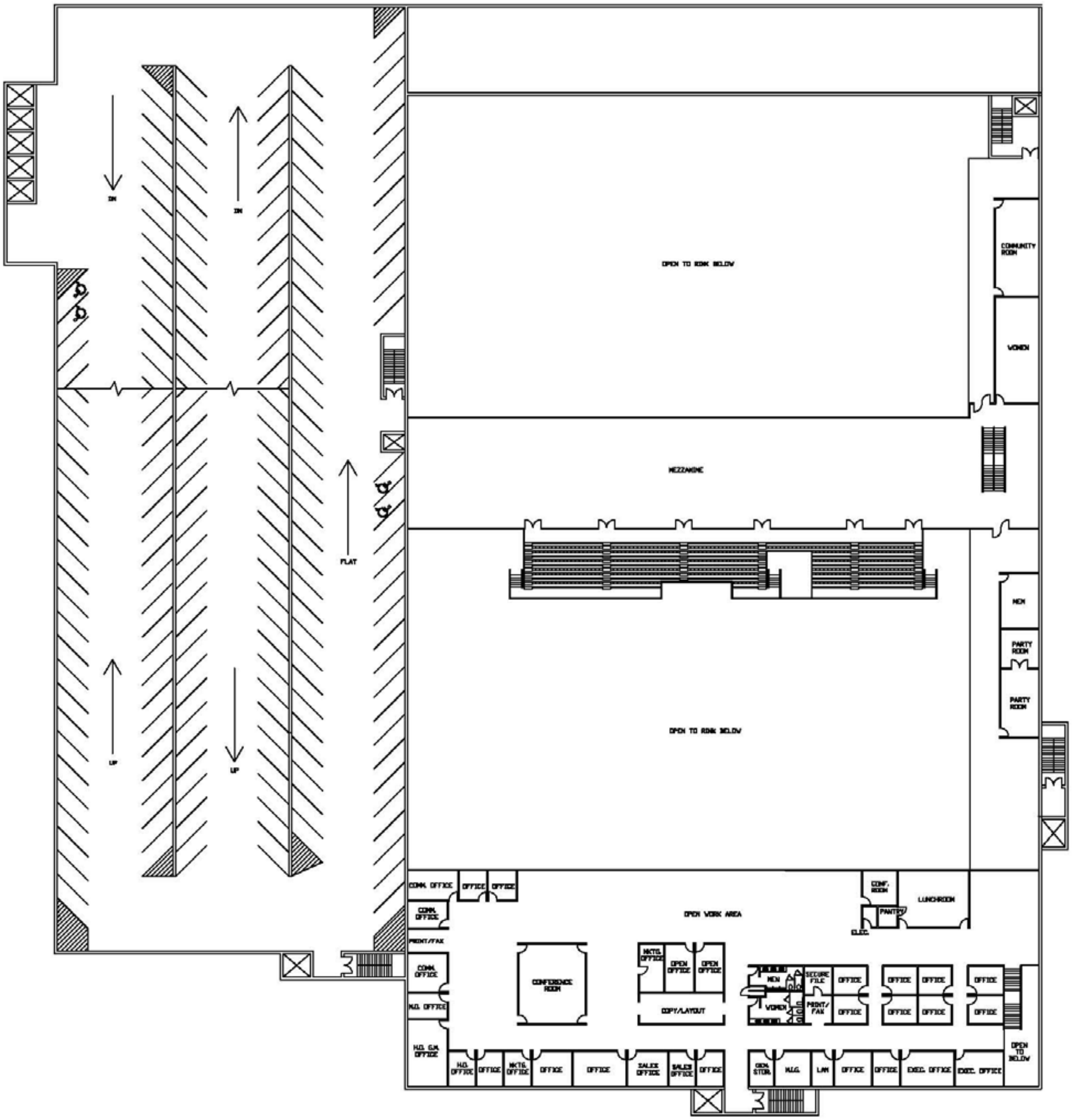


Figure 25: Second Floorplan

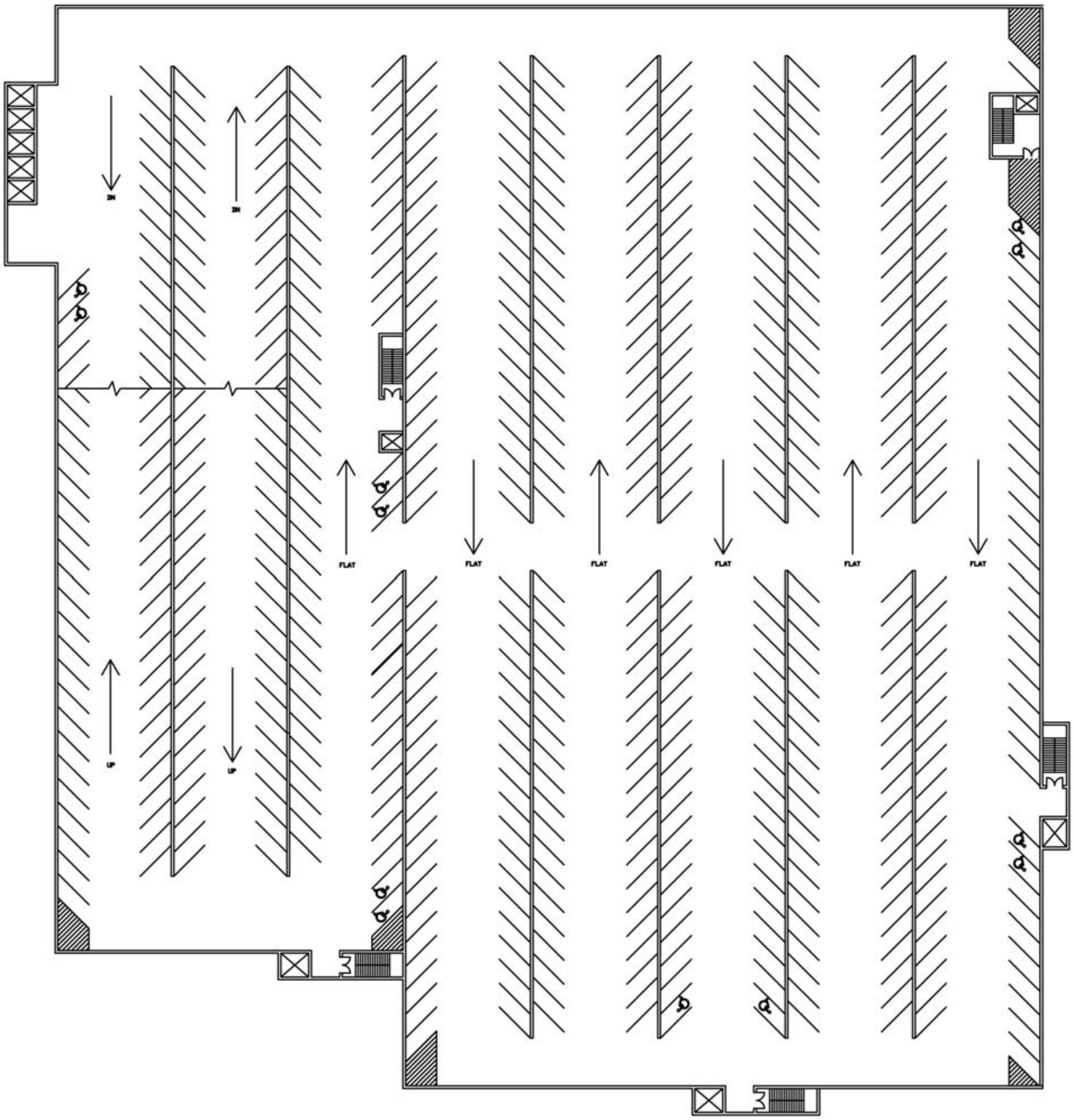


Figure 26: Typical Parking Floorplan

Below are sketches of building sections which explain how the parking garage will span over the Iceplex. It can be seen that an 11ft. clearance was allowed for any structure needed to support this section of the parking garage. A building rendering is provided to illustrate the architectural impact on the exterior. It can be seen that the main building materials include precast concrete, brick, metal paneling, and glass. It is very obvious which part of the building is the Iceplex and which part is the parking garage by the use of the various materials.

Since the Iceplex is now located on the ground and second floors, privacy must be considered. It is important to note that no private areas, such as bathrooms, are enclosed by glass. However, for other areas such as the training facility, that need to remain private only when in use, window shades are suggested. Another consideration for the final design was the openness of the parking garage. According to IBC 406.3.3.1, the openings of exterior walls in any parking garage must be at least 20% of the total perimeter wall area on any given tier. The proposed design calls for approximately 27% openness, therefore meeting this criterion.

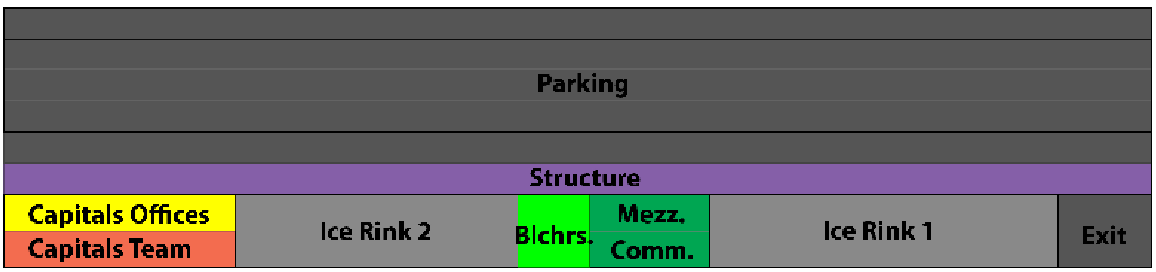


Figure 27: N-S Building Section

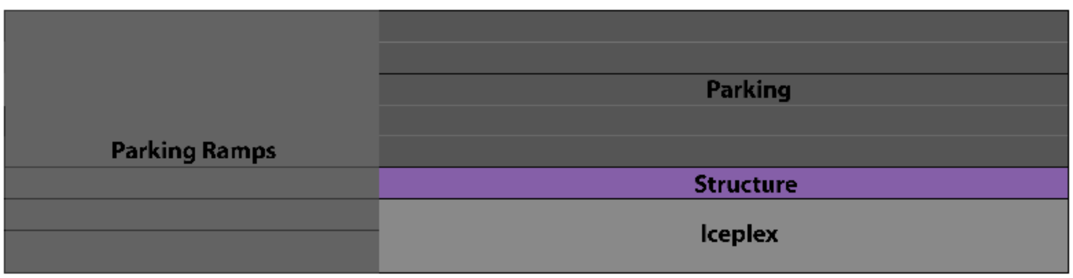


Figure 28: E-W Building Section

Figure 29: Rendering

STRUCTURAL DESIGN

The Parking Garage Structural System

Since the Ballston Common Mall will be without any parking for the duration of this proposed project, it was very important to frame the structural system with materials that can be erected quickly and efficiently. There were several options considered for the structural framing of the parking garage: cast-in-place concrete, steel, and precast concrete. Cast-in-place concrete takes time to cure, which will slow down erection time. Using CIP concrete will also push back the occupancy date, which needs to be avoided with this specific project. Steel framing has somewhat of a quick erection time, but is not typical of a large parking structure. Precast concrete was chosen to be the best building material to structure the parking garage because of its fast erection. Precast parking structures are erected one bay at a time, different from that of typical construction which erects one level at a time.

Another reason precast concrete is the ideal building material for this parking garage is its maintenance schedule. Precast parking structures that receive periodic maintenance and care can be used for decades with only moderate cost. If an owner follows a very simple schedule of maintenance, he will be protecting his investment avoiding increased repair costs down the road. There are three categories for this maintenance: housekeeping, preventative maintenance, and repairs. Housekeeping items include sweeping, trash pickup, and parking space restriping. Preventative maintenance items include floor wash down and repairing joint sealant. If an owner keeps up with these housekeeping and preventative maintenance tasks, major issues like corrosion that need serious repair may be eliminated.

Several options of precast concrete systems were researched. It was concluded that the MEGA-SPAN system from High Concrete was the best choice. This system introduces the 15ft. "MEGA-TEE" which is 5ft. wider than the typical double T. This allows for wider bays and longer spans. This system's primary benefits include faster construction time and reduced costs. In fact, this system can reduce construction time by 20-30% which will decrease the amount of time the Ballston Common Mall goes without parking. The MEGA-SPAN system is becoming more and more popular with large parking structures. Parking decks, such as East Parking Deck and Eisenhower Parking Deck on Penn State University's main campus, use this framing system. A copy of the MEGA-SPAN Precast Building System Design Guide has been provided in the appendix.

According to PCI's *Parking Structures: Recommended Practice for Design and Construction*, expansion joints are rarely used in precast parking structures unless the structure is more than 300ft. in length. The proposed design has maximum dimensions of 372'x408', which means that there should be at least on expansion joint in each direction. However, upon close examination it was very difficult to locate these needed joints because of the architectural layout of the building and the structural requirements of the ice rinks. Jim Pudleiner of Walker Parking Consultants in Wayne, PA stated that this large structure is pushing the limits, but it is possible to go without expansion joints. He stated that this may cause some issues with diaphragm cracking.

The Iceplex Structural System

The first level of the Iceplex, including both ice rinks, locker rooms, and the pro-shop, are located on ground level framed using a slab-on-grade. Although this slab was not designed in detail in this report, it is believed that locating the ice rinks on SOG will drastically improve the deflection problems that were experienced in the original design.

The second level of the Iceplex, including the Capitals corporate offices and mezzanine overlooking the rinks, was framed out of composite steel. The typical bay measures approximately 30'x30' varying slightly depending on architectural layout. The exact member sizes were not designed in detail in this report.

Gravity Loads

Gravity loads were taken from ASCE7-05 and are listed in Table 5. Even though the framing systems for the offices, mezzanine, and bleachers were not designed in detail in this report, their live loads are listed to show what would be used during this design.

Table 5: Gravity Loads

Area	LL (psf)	SDL (psf)
Parking Garage	40	3(mech) + 8 (Double T stem)
Offices	50	15 (corridors)
Mezzanine	60	
Bleachers	60	

The superimposed dead loads for the parking garage include 3psf for mechanical equipment and 8psf which accounts for the double T stems. This is in addition to a 5" slab thickness used during analysis.

Column takedowns were performed in order to determine column loads for the parking structure. Live load reduction was taken into consideration with these takedowns. The column takedown spreadsheets are provided in Appendix C. Figure 30 shows the proposed locations of gravity precast columns.

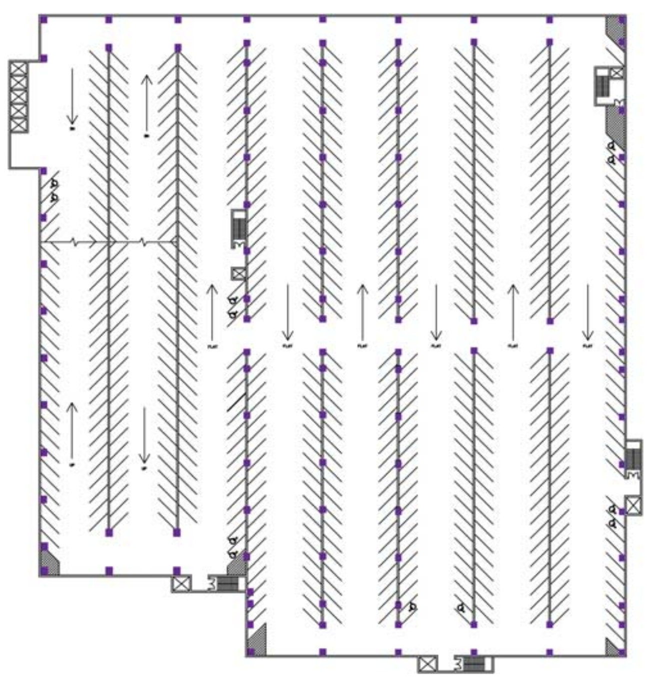


Figure 30: Column Locations

The Lateral Framing System

The main lateral force resisting system for the proposed design consists of precast concrete shear walls and lite walls. Lite walls are typically used in concrete parking structures. They are like shear walls but with openings in order to allow air flow and ventilation through the parking garage. These walls resist lateral forces as well as act as girders picking up the gravity loads from the double Ts. A photo of a lite wall used in Penn State's East Parking Deck in shown in Figure 31.

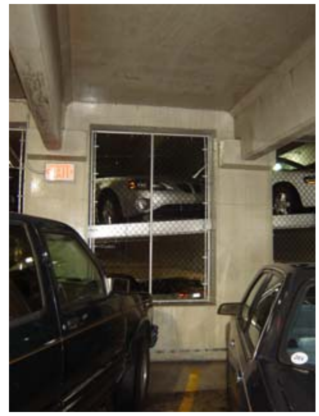
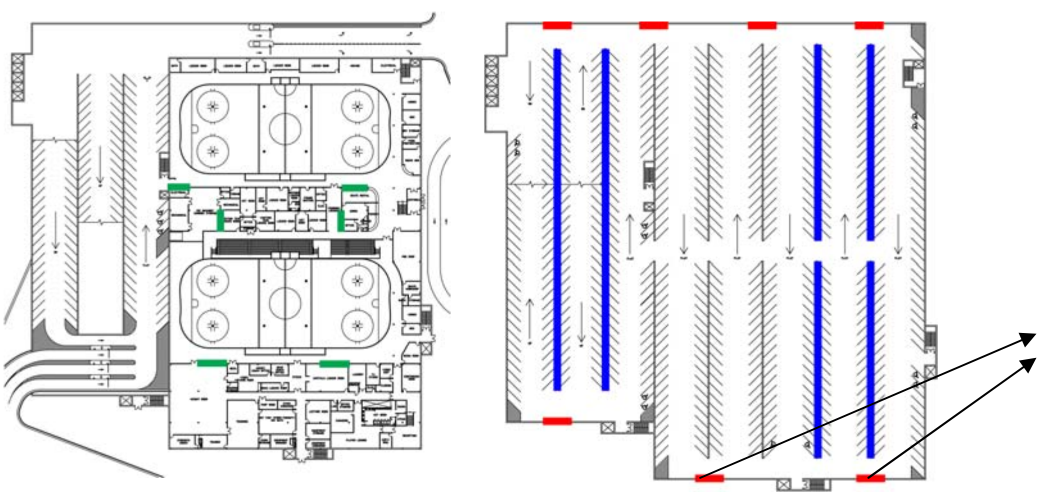


Figure 31: PSU East Parking Deck Lite Wall

One design consideration that took a great deal of thinking was the load path for lateral loads. Since the ice rinks require large open spaces, shear walls not located along the building perimeter cannot run the entire height of the building. There were also issues with the shear walls located along the exterior of the Capitals office spaces and locker rooms. The exterior walls of these spaces needed to remain glass and metal paneling for architectural reasons; therefore these walls were cut back at the first two levels of the Iceplex. After a great deal of thinking and comparing options, a final lateral resisting system was determined and is show in Figure 32. Lite walls are shown in blue, shear walls in red, and braced frames in green. Figure 33 shows how two of the shear walls will be cut back at Iceplex levels.

Figure 32: LFRS Location



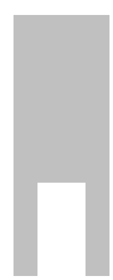


Figure 33: Shear Wall Cutback

Although this LFRS was determined to be the best solution, the location of these members may raise some other design issues. It can be seen that walls in both directions are located a reasonable distance from the center of rigidity. It is anticipated that this may cause some problems. When the building contracts, due to volume changes such as thermal expansion, the shear walls along the perimeter will want to resist this contraction resulting in serious cracking in the diaphragm. In order to avoid this problem, lateral resisting members should be relocated near the center of rigidity. Consequently, this will cause some design issues with the large open spaces needed for the ice rinks. With further consideration, there may be a solution to this problem but one was not determined in this report.

Determining Wind Loads

Wind loads were generated using ASCE7-05 Chapter 6. Much of the structure exterior on the north and east sides is blocked by adjacent buildings, therefore no windward or leeward pressure would result. This was taken into account when generating wind loads. The spreadsheets used to generate wind pressures can be found in Appendix C. Here is a list of input parameters used when calculating wind pressures:

- Basic Wind Speed, V — 90mph
- Wind Directionality Factor, K_d — 0.85
- Importance Factor, I — 1.15
- Exposure Category — B
- Internal Pressure Coefficient, C_{pi} — 0.18
- Topographic Factor, K_{zt} — 1.0
- External Pressure Coefficient, $C_{p,w}$ — 0.8
- External Pressure Coefficient, $C_{p,l}$ — -0.5
- External Pressure Coefficient, $C_{p,s}$ — -0.7

The tables below show how the wind forces will be distributed for the four different wind directions. The grayed out areas represent where there will be no windward/leeward pressures. It can be seen that wind traveling from east to west and wind traveling from south to north will control.

Table 6: North-South Wind Distribution

N-S Wind Distribution							
Level	Leeward Pressure (psf)	Windward Pressure (psf)	Wall Area-Leeward (SF)	Wall Area-Windward (SF)	Total Leeward Load (kips)	Total Windward Load (kips)	Total Load to be Applied (kips)
Mezz.	3.0		4836		14.51		14.51
Truss	3.0		4092		12.28		12.28
P1	3.0		4092		12.28		12.28
P2	3.0		4092		12.28		12.28
P3	3.0		4092		12.28		12.28
P4	3.0		4092		12.28		12.28
P5	3.0	19.70	4092	2880	12.28	56.74	69.01
Roof	3.0	20.10	2046	3720	6.14	74.77	80.91
						Base Shear =	225.81

Table 7: South-North Wind Distribution

S-N Wind Distribution							
Level	Leeward Pressure (psf)	Windward Pressure (psf)	Wall Area-Leeward (SF)	Wall Area-Windward (SF)	Total Leeward Load (kips)	Total Windward Load (kips)	Total Load to be Applied (kips)
Mezz.		15.1		4836		73.0	73.0
Truss		16.4		4092		67.1	67.1
P1		17.3		4092		70.8	70.8
P2		18.0		4092		73.7	73.7
P3		18.6		4092		76.1	76.1
P4		19.2		4092		78.6	78.6
P5	3.0	19.7	2880	4092	8.6	80.6	89.3
Roof	3.0	20.1	3720	2046	11.2	41.1	52.3
						Base Shear =	580.8

Table 8: East-West Wind Distibution

E-W Wind Distribution							
Level	**Leeward Pressure (psf)**	**Windward Pressure (psf)**	**Wall Area-Leeward (SF)**	**Wall Area-Windward (SF)**	**Total Leeward Load (kips)**	**Total Windward Load (kips)**	**Total Load to be Applied (kips)**
Mezz.		15.00		5304		79.56	**79.56**
Truss		16.30		4488		73.15	**73.15**
P1		17.20		4488		77.19	**77.19**
P2		17.90		4488		80.34	**80.34**
P3		18.50		4488		83.03	**83.03**
P4		19.10		4488		85.72	**85.72**
P5		19.60		4488		87.96	**87.96**
Roof		20.00		2244		44.88	**44.88**
					Base Shear =		**611.84**

Table 9: West-East Wind Distribution

W-E Wind Distribution							
Level	**Leeward Pressure (psf)**	**Windward Pressure (psf)**	**Wall Area-Leeward (SF)**	**Wall Area-Windward (SF)**	**Total Leeward Load (kips)**	**Total Windward Load (kips)**	**Total Load to be Applied (kips)**
Mezz.	3.1		5304		16.44		**16.44**
Truss	3.1		4448		13.79		**13.79**
P1	3.1		4448		13.79		**13.79**
P2	3.1		4448		13.79		**13.79**
P3	3.1		4448		13.79		**13.79**
P4	3.1		4448		13.79		**13.79**
P5	3.1		4448		13.79		**13.79**
Roof	3.1		2244		6.96		**6.96**
					Base Shear =		**106.13**

Determining Seismic Loads

Seismic story forces were calculated using ASCE7-05. First, the weight of each story was calculated. To view these calculations and the material loads that went into them, see Appendix C. The table below summarizes the weights of each level.

Table 10: Story Weights

Level	Weight (kip)
Base	1,976
Mezz.	6,731
Truss*	6,152
P1	16,573
P2	16,573
P3	16,573
P4	16,573
P5	16,573
Roof	15,653
Total	**113,377**

* Assuming steel trusses = 3psf

Once the weights of every level were calculated, they were used to determine to seismic story forces. The table on the following page summarizes these loads. See Appendix C for the calculations. Here is a list of input parameters used during seismic analysis:

- S_s — 0.154
- S_1 — 0.051
- Site Class — Unknown, assume D
- Occupancy Category — III
- F_a — 1.6
- F_v — 2.4
- Importance Factor, I — 1.25
- Response Modification Coefficient, R — 4 (precast shear walls)
- Approximate Period, T_a — 0.59

Table 11: Seismic Story Forces and Shears

Level (x)	Fx (kips)	Vx (kips)	Mx (ft-kips)
Roof	156.5		
P5	165.7	165.7	14401
P4	165.7	331.5	27825
P3	165.7	497.2	39426
P2	165.7	662.9	49204
P1	165.7	828.7	57159
Truss	61.5	890.2	63291
Mezz.	67.3	957.5	64891
Base	19.8	**977.2**	**65900**

Analyzing Lateral Force Resisting System

Comparing the story forces and base shears for wind and seismic loading, it can be seen that seismic controls over wind. Now that the controlling lateral force was known, a computer model was ready to be built. Through previous experience with several modeling softwares, ETABS was chosen to be the best program to use to analyze the LFRS of the proposed design. Figure 34 shows the finalized ETABS model. It can be seen how the three types of lateral members are distributed throughout the building. It should be noted that parking ramps were ignored in the model in order to avoid meshing problems.

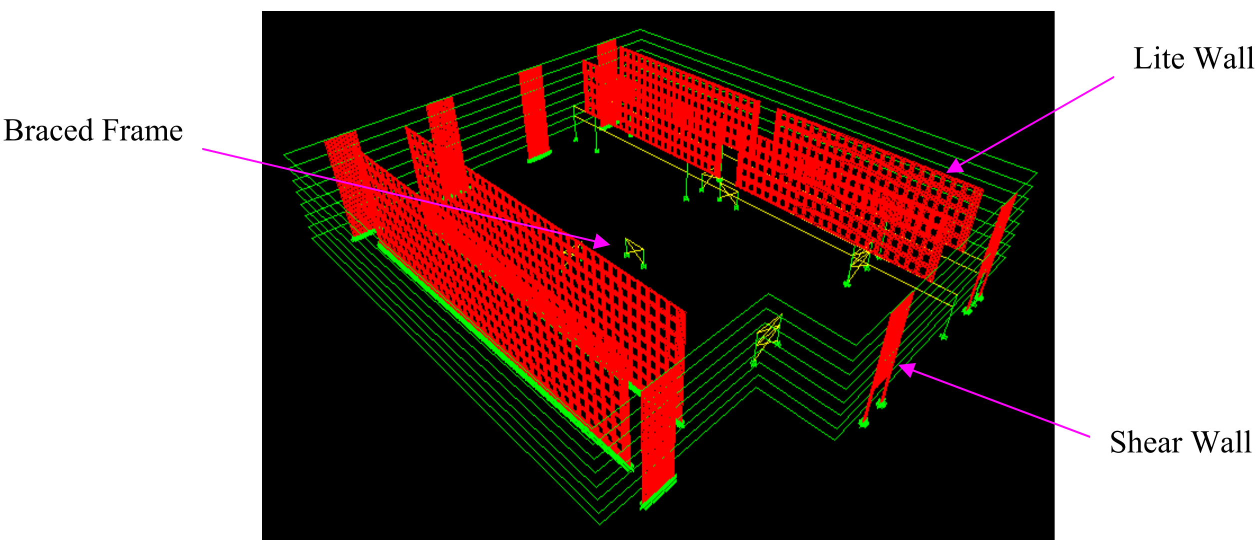


Figure 34: 3D LFRS ETABS Model

Here is a list of input parameters that were used when building this computer model:

- Manual meshing of all walls 1'x1'
- Each level assigned to its own diaphragm
- Seismic forces applied at center of rigidity with +5% eccentricity
- Diaphragm mass = DL (slab + double T stem + mech.)ksf / 32.2 / 12^3
- All members fixed at base
- P-Δ effects accounted for

Once the computer model was analyzed, the loads taken by each member were tabulated. Designing the lateral members is beyond the scope of this report, however the loads at the base of the four lite walls will be needed in order to design the transfer trusses. That is discussed later in this report. See the tables below for these forces.

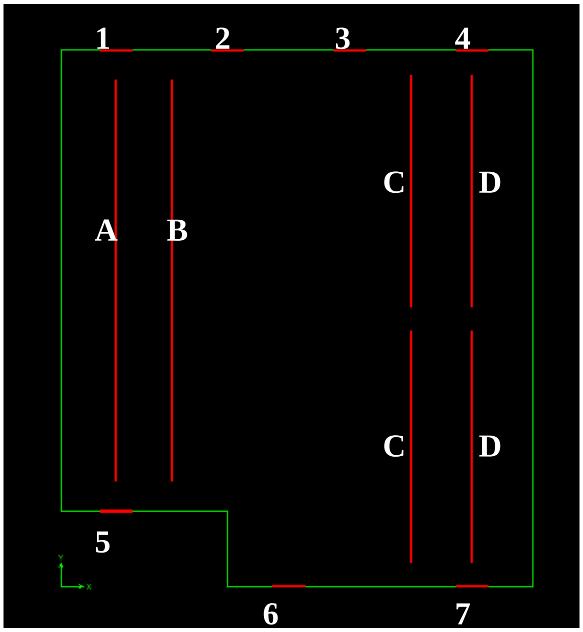


Figure 35: LFRS Labels

Table 12: Shear Wall Forces

Member	Floor (x)	Vx (kip)	Mx (kip-in)	Member	Floor (x)	Vx (kip)	Mx (kip-in)
Wall 1	Roof	9.5	71	Wall 5	Roof	3.9	17
	P5	22.9	1,238		P5	17.7	1,230
	P4	48	6,742		P4	40	4,057
	P3	71	12,705		P3	62	11,075
	P2	97.3	26,976		P2	80.2	18,804
	P1	126	46,486		P1	94.2	32,402
	Truss	137	65,475		Truss	93.1	51,373
	Mezz.	131	82,459		Mezz.	324	89,863
	Base	129	106,644		Base	360	156,868

	Roof	5.3	13		Roof	3.8	14
	P5	23.7	1,546		P5	18.3	1,244
	P4	46.4	5,595		P4	39.3	3,737
	P3	72.6	15,972		P3	62.5	9,232
Wall 2	P2	97.3	25,796	Wall 6	P2	88.2	22,534
	P1	126	47,412		P1	111	40,069
	Truss	137	65,138		Truss	169	60,094
	Mezz.	131	83,103		Mezz.	13.3	66,893
	Base	129	106,485		Base	9.4	68,665
	Roof	12.6	109		Roof	4.8	29
	P5	23.7	1,727		P5	16.7	946
	P4	46.4	5,185		P4	40.4	4,473
	P3	65.1	13,749		P3	63.4	10,242
Wall 3	P2	98.3	31,718	Wall 7	P2	87.9	23,400
	P1	126	47,412		P1	103	42,487
	Truss	137	65,307		Truss	170	63,081
	Mezz.	131	83,103		Mezz.	13.8	66,967
	Base	130	106,803		Base	10.3	68,896
	Roof	6.6	41				
	P5	23.6	1,612				
	P4	45.6	5,318				
	P3	71.5	13,896				
Wall 4	P2	97.9	31,748				
	P1	109	48,356				
	Truss	139	66,140				
	Mezz.	132	82,929				
	Base	132	107,316				

Table 13: Lite Wall Forces

Member	Floor (x)	Vx (kip)	Mx (kip-in)		Member	Floor (x)	Vx (kip)	Mx (kip-in)
Lite Wall A	Roof	11.2	55		Lite Wall C	Roof	92	385
	P5	22.8	2,830			P5	93.7	12,065
	P4	28.7	4,689			P4	134	27,640
	P3	35.3	7,208			P3	157	46,416
	P2	57.4	9,421			P2	203	68,970
	P1	49.6	16,141			P1	211	63,224
	Truss	416	66,810			Truss		
	Mezz.	417	12,216			Mezz.		
	Base	415	192,912			Base		
Lite Wall B	Roof	31.6	93		Lite Wall D	Roof	67	61
	P5	49.8	6,327			P5	127	18,436
	P4	83.3	16,997			P4	156	34,104
	P3	114	31,661			P3	183	59,122
	P2	142	49,719			P2	210	82,562
	P1	137	67,572			P1	232	93,361
	Truss	629	147,556			Truss		
	Mezz.	609	228,750			Mezz.		
	Base	609	321,953			Base		

Designing Transfer Trusses

Since the proposed design calls for six levels of parking to be constructed over top the two ice rinks, a large transfer system is required to transfer the large gravity and lateral loads from above. Based on these large loads, especially dead loads, it was determined that a truss system was the best system to use. Vertical web members were located at points of column loads and were spaced evenly under conditions of uniformly distributed loads. This should allow for the most effective transfer of axial forces throughout the truss.

Design Overview

The truss and column locations are shown in the figure below in red and blue, respectively. Trusses A, B, and E take only gravity load, whereas trusses C and D take both gravity and lateral load. The applied forces on trusses A, B, and E are point loads from the precast columns supporting the parking garage. The column takedowns were used in order determine each of these loads. The gravity loads on trusses C and D are uniformly distributed loads transferred from the lite walls above. These trusses were modeled individually using the structural design software, SAP2000.

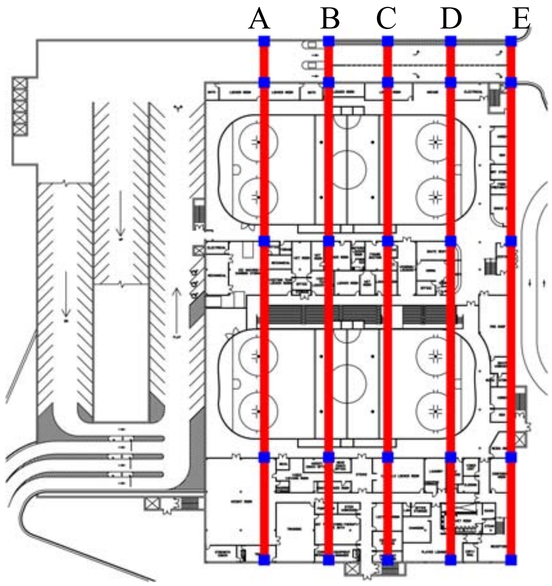


Figure 36: Design 1 Truss and Column Locations

Design 1

The first design attempt used a planar truss with a depth of 11ft. An elevation of this truss is shown in Figure 37. To test this configuration, a truss taking only gravity loads was examined.

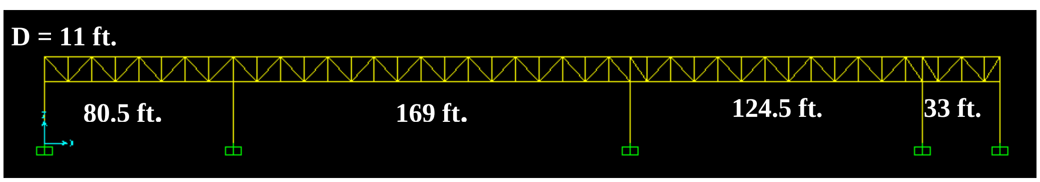


Figure 37: Planar Truss Model (Design 1)

The figure below shows the axial compressive and tensile forces in the truss members. Red indicates compression whereas yellow indicates tension. It can be seen that there are extremely high forces in several members, especially the top chords. A combined loading analysis was performed using the AISC Steel Construction Manual Chapter 6. Interaction equations H1-1a and H1-1b were used to determine what member size, if any, was adequate to carry these loads. Since the top chord spanning 169ft. carries the largest axial load, it was checked first. Using the axial and flexural loads provided by SAP, a stress level of 2.283>>1.0 was calculated using even the largest wide flange shape, the W36x800. This concludes that this truss configuration will fail. The calculations for this member can be found in Appendix C.

Figure 38: Planar Truss Axial DL

Design 2

Since the first design attempt failed by a large margin, several changes were made during the second attempt. First, the depth of the truss was increased from 11ft. to 22ft. The configuration of this truss was made into a space truss instead of a planar truss. This will provide two bottom chords with additional web members to better spread out the load from the top chord. An additional column was added in order to decrease the spans as much as the architectural layout would allow. The new longest span is still in the second bay but is now 126ft. instead of the original 169ft. This can be seen in Figure 39 with the additional columns shown in yellow. Figure 40 shows a view of this new truss configuration.

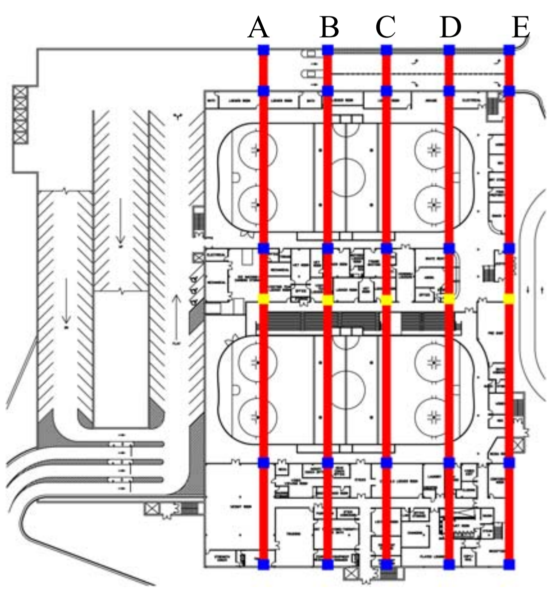


Figure 39: Design 2 Truss and Column Locations

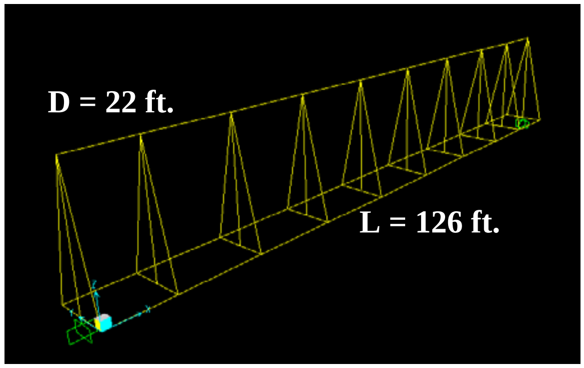


Figure 40: Space Truss Model (Design 2)

Once again, the axial and flexural loads were taken from SAP and used to check the member stresses. With this new design, the top chord still fails with a stress level of 1.18. This calculation can also be found in Appendix C.

Design 3

One last truss configuration was analyzed. With this third attempt, more diagonal members were added to even better distribute the load from the top chord. The truss depth and span remained 22ft. and 126ft. respectively. This can be seen in Figure 41 below. AISC equations H1-1a and H1-1b were again analyzed. This time, the stress levels in all members was found to be acceptable. The controlling load combination was 1.2D + 1.6L. The top chord final design for trusses A and B (taking only gravity load) uses a W40x503 (93% stressed). The bottom chords for this design are both W36x150 (also 93% stressed). The calculations for these members can be found in Appendix C. Web members range from HSS3x3x3/8 to HSS12x12x5/8. Two W14x193 members were used as vertical web members on either side of the truss. A spreadsheet showing the design and forces in each web member from the controlling bay is provided in Appendix C.

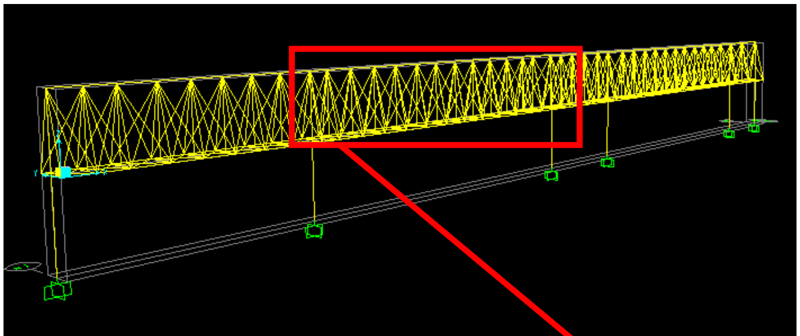
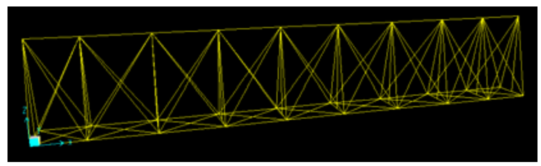


Figure 41: Space Truss Model (Design 3)



Design 3 (continued)

Now that a possible truss design configuration was determined, trusses C and D transferring both gravity and lateral load were examined. Uniformly distributed gravity loads were applied to the top chords of the truss. These loads were calculated the same way as the column takedowns and can be found in Appendix C. The moments determined from the ETABS lateral model were taken and applied to the top chords as well. These moments were applied using the truss bay's relative spans. For instance, the total moment at the base of a 176ft. lite wall is 5269 kip-ft. The truss bay spanning 126ft. will take 126' x 5269 / 176' = 3772 kip-ft. The corresponding moments were calculated this way for all truss spans and were applied at the center of the top chords. The shear in each lite wall was also applied to the edge of the truss. The model was finally analyzed in order to obtain the forces in each member. The load combination 1.2D + 1.0E + L controlled in this case.

The combined loading equations were once again used to design the top and bottom chords of the lateral trusses. The top chord of the second bay was designed as a W36x441 (82% stressed) and the bottom chords a W36x135 (76% stressed). Web members slightly increased in size and ranged from HSS3x3x3/8 to HSS14x14x5/8.

Since these trusses take lateral loads, it was assumed that the columns supporting these trusses will control over those of the gravity only trusses. These columns were then designed using the combined loading equations. It was important to use columns with as small of depth and width possible because they will be located in corridors as well as office spaces. The smallest column, Column 1, was designed as a W30x261 (96% stressed). The largest column, Columns 2 and 5, were designed as W36x441. Figure 42 shows the column designs.

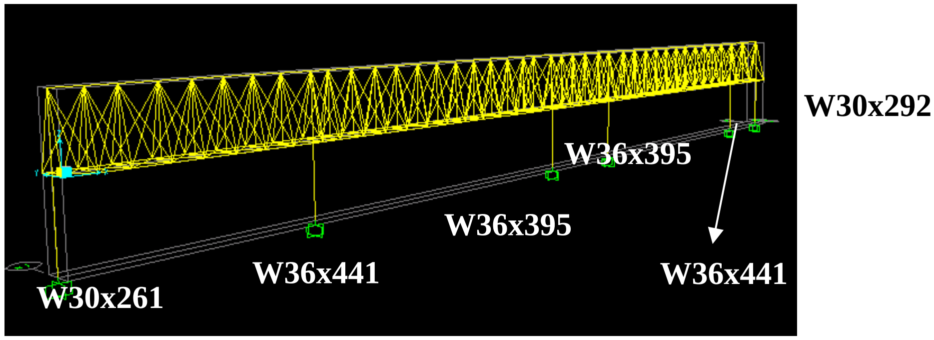


Figure 42: Column Designs

Design 4

Since the column sizes required by Design 3 were extremely large in dimension, a fourth design was analyzed. This design calls for two columns at each supporting location instead of the original one. This can be seen in Figure 43. It was thought that by adding additional columns, the dimensions would decrease to sizes that can be used without interfering with the architectural layout. After this SAP model was analyzed, the columns with the largest load, columns 3 and 5, were once again designed. Although they were able to decrease to a W30x261 and a W30x292 respectively, their 30" depth is still not ideal to use in architectural spaces. Since the depth only decrease by 3", it was assumed that this minimal change does not make up for the additional steel members. Therefore, one column at each support will be used as the final design. All column calculations can be found in Appendix C.

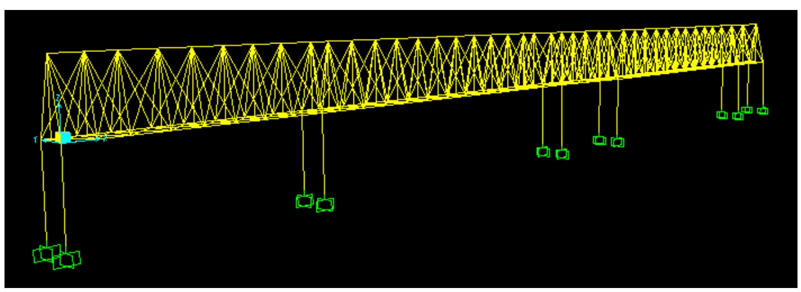


Figure 43: Space Truss with 2-Column Option

Final Design

The final design of the lateral trusses is shown below. Now that this has been determined, deflections can now be analyzed. Deflections in the top chords of these transfer trusses needed to be as small as possible because too much deflection can lead to sagging of the parking garage above. This can cause many problems from disturbing the drainage system of the garage to major issues while erecting the precast structure.

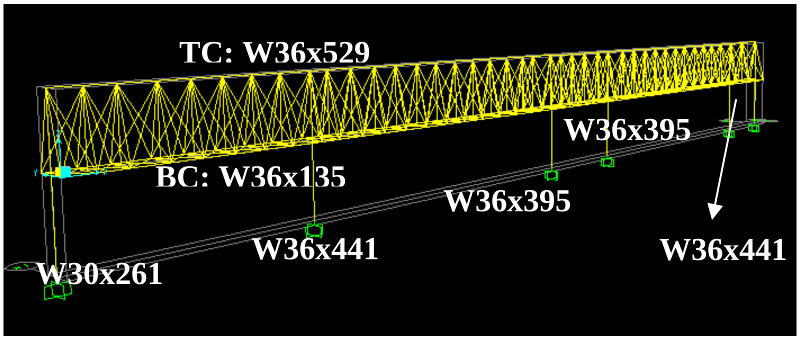


Figure 44: Final Truss Design

In order to determine how deflections will affect the structure, SAP was used to determine these deflections. Once all members were assigned there appropriate size, the model was analyzed one last time. The deflections of the top chords were as follows:

- Δ_{DL} = 2.33" in Bay 4 = L/641
- Δ_{LL} = 0.82" in Bay 4 = L/1822
- $\Delta_{Total,\ Factored}$ = 3.61" in Bay 4 = L/414

It should be noted that the live load deflection above is for the total code load of 40psf for parking garages. Whereas in reality the actual live load for a garage with cars bumper to bumper is more in the magnitude of 20psf. Therefore, it can be concluded that live load deflections should not be an issue with this truss design.

The biggest concern with these trusses is the dead load deflection. Since the deflection is relatively small, the top chords can be cambered in order to account for this deflection. It was recommended by a practicing engineer to camber about 80% of the total dead load. That would mean using a camber of about 1.9".

One major concern that must be addressed with the construction manager and precast engineer is how to properly erect the parking structure on top of these trusses. As the parking structure is erected, the trusses will deflect more and more as each piece is laid into place. One must ask the question, will the connections of the precast structure allow for such movement? Due to time constraints, this issue is not addressed thoroughly in this report, however it should be noted that it was indeed considered.

Truss Design Affects on Architecture

There were several things that changed during the many design attempts of the transfer trusses. First, the truss depth was increased from 11ft. to 22ft. This will affect both the parking layout and the architectural façade of the building. Since one level of parking must be removed to account for this larger truss, 341 parking stalls were lost. However, the new parking capacity is still greater than that of the existing parking structure. Also, the fourth level on the building exterior must now become another 11ft. of metal paneling. This is shown in Figure 45 outlined in red. Recall that the columns supporting

Figure 45: Facade Changes

these trusses are located in corridors and in office spaces. Their required 36" depth will have to either be blocked out using gypsum board or given an architectural layer of paint. Either way, these columns are taking up usable working space and will be inconvenient for building occupants.

CONSTRUCTION MANAGEMENT

Project Cost

Project cost is extremely important in the construction industry. Owners always want to get the best end result at the lowest possible price. Estimating the cost of the proposed design and comparing it to the actual design is essential in determining whether or not this is an economical solution.

The actual cost of the original design was $42.7M, but this number cannot be legally confirmed. Sigal Construction, the construction manager, is currently in litigation with the owner arguing about price. This means that the exact numbers could not legally be made public. However, a Certification for Payment document was obtained from Micheal Shevitz, project manager of the Iceplex project. It dates back to March 2005 and only addresses early estimated prices. According to this document, the total project cost is $30.4M. This document can be reviewed in Appendix D. For the sake of this report, the unconfirmed, yet more accurate, cost of $42.7M will be used.

Three RS Means 2007 publications were used to estimate the cost of the proposed design: Assemblies, Heavy Construction, and Site Work & Landscape. Unit prices of various activities were obtained from these books and applied to estimated quantities to arrive at cost data. It should be noted that all numbers include installation, overhead, and profit. Several soft costs were also estimated including contingencies and an estimated loss in mall revenue for the roughly 300 days the mall will be without the parking garage. Some numbers were taken from the cost data provided by Sigal Construction, adjusted for inflation. These numbers are noted with an * in the cost spreadsheet. The final project estimate is shown in the spreadsheet on the following pages.

Table 14: Cost Estimate

Activity		Unit	Cost/Unit	No. Units	Cost
Existing Building Demolition					
Demolition-Building		CF	$0.40	12,124,000	$4,849,600
Demolition-Foundation		CF	$5.00	102,900	$514,500
	Reinforcement: +10%				$51,450
Disposal (Assume no hazardous material)		CY	$12.00	157,500	$1,890,000
					$7,305,550
Foundation / Slab-on-Grade					
Strip Footings (Litewalls)		LF	$450.00	618	$278,100
	Estimate for 60klf, soil capacity 3ksf				
Spread Footings (Truss Columns)		EA	$61,500.00	36	$2,214,000
	Estimate for 3000k, soil capacity 3ksf				
Spread Footings		EA	$437.00	83	$36,271
	75k, soil capacity 3ksf				
Slab-on-Grade		SF	$5.45	145,000	$790,250
	6" thick, reinforced, non-industrial				
					$3,318,621
Precast Parking Garage					
Double Ts		SF	$15.00	632,000	$9,480,000
Columns		LF	$225.00	5,600	$1,260,000
Girders		LF	$200.00	9,700	$1,940,000
Shearwalls		SF	$40.00	25,000	$1,000,000
Litewalls		SF	$30.00	95,000	$2,850,000
Exterior Spandrels with Brick		SF	$50.00	48,500	$2,425,000
Exterior Spandrels		SF	$36.00	68,800	$2,476,800
Ramps		SF	$30.00	37,400	$1,122,000
Stairwells		EA	$4,500.00	40	$180,000
					$22,733,800
Iceplex Structure					
Steel Beams and Girders (Mezz./Offices)		SF	$14.40	34,500	$496,800
	25x30 Bay				
	125 psf Total Load				
Metal Deck / Concrete Fill (Mezz./Offices)		SF	$5.47	34,500	$188,715
	10' span				
	4" thickness				
Steel Columns (Mezz./Offices)		LF	$50.00	1,245	$62,250
Precast Wall Separating Garage and Iceplex		SF	$28.80	8,424	$242,611
	8" thick with 2" insulation				
	20x10 units				
					$990,376
Transfer Trusses					
Wide Flange		TON	$880.00	1,195	$1,051,600
HSS		TON	$980.00	685	$671,300
Connections (33% of bare steel)					$568,557
Erection (Assume +30%)					$516,870
					$2,808,327

Activity			Unit	Cost/Unit	No. Units	Cost
Finishes						
Curtain Wall Panels without Structure			SF	$20.50	6,500	$133,250
	Glazing Panel 5/8" thick					
Glazed Doors			EA	$3,800.00	5	$19,000
	Aluminum/Glass w/o transom					
	6'x7'					
Drywall Partitions on Metal Stud Framing			SF	$5.41	36,000	$194,760
	5/8" FR Drywall Both Faces					
	Framing 16" oc					
Steel Doors (Exits)			EA	$3,985.00	42	$167,370
	18 gage steel, 2-door with frame; 6'x7'					
	B label = A label + $600 (approx.)					
Interior Glazed Opening (Capitals Offices/Mezz)			EA	$4,500.00	45	$202,500
	Concealed Frame, 1/2" float, 16'x5'					$0
Interior Doors			EA	$394.00	125	$49,250
	Wood Door/Frame, Hollow Core					
	Luann, 2'-8"x6'-8", 3-5/8" Deep					
Wall Finishes			SF	$0.73	36,000	$26,280
	Painting Primer + 1 Coat					
Floor Finishes			SF	$7.00	81,000	$567,000
	Avg. Carpet and Tile					
Drywall Ceilings						
	5/8" FR Drywall on Metal Studs		SF	$3.32	57,300	$190,236
Ice Rink Dasher Boards			EA	$157,500.00	2	$315,000
Bleachers			EA	$138.00	675	$93,150
Miscellaneous (scoreboard, fire extinguishers, projection screens, sauna, etc.)*						$635,000
						$2,592,796
MEP						
Community Centers (Comm./Party Rooms)						
	Total Mechanical/Electrical		SF	$34.50	28,000	$966,000
Offices						
	Plumbing		SF	$4.59	19,300	$88,587
	HVAC		SF	$9.15	19,300	$176,595
	Electrical		SF	$9.65	19,300	$186,245
Parking Garage						
	Plumbing		SF	$1.36	632,000	$859,520
	Electrical		SF	$2.09	632,000	$1,320,880
Ice Skating Rinks						
	Total Mechanical/Electrical		SF	$14.75	32,700	$482,325
Clubs: YMCA (Locker Rooms/Training/etc.)						
	Total Mechanical/Electrical		SF	$33.00	19,300	$636,900
Elevators: Hydrolic			EA	$110,000.00	5	$550,000
Ice Rink Maintanence (Refrig., Plumb., Cooling)						
	55°, 5 mos., 100 ton		EA	$545,500.00	2	$1,091,000
Fire Protection*						$486,000
Smart Parking Garage Counting System						$75,000
						$6,919,052
Landscaping and Site Work						
Site Utilities*						$150,000
Chain Link Fences*						$6,850
Concrete Sidewalks			LF	$24.70	1,000	$24,700
Bituminous Roadway			LF	$91.00	232	$21,112
Lawn			1000SF	$1,105.00	67	$74,035
						$119,847
						$46,788,369
Soft Costs						
Design Fees			8.5%			$3,977,011
Permitting						$150,000
Contingency*		gen + adjustment				$1,720,000
Mobilization/Demobilization*						$172,000
Bond*						$217,500
Estimated Loss in Mall Revenue			DAY	$72,000.00	300	$21,600,000
						$74,624,881

*Numbers taken from original price estimate, adjusted for inflation

The estimated mall loss of revenue was obtained by using the 2002 revenue for total retail trade stores in Arlington, VA from the Census Bureau. The annual revenue for all retail stores in the county totaled $2.1B in 2002. It was assumed that 5% of this revenue comes from the Ballston Common Mall and that the mall would lose 25% of its customers without a parking garage. Based on these numbers, it was estimated that the mall would lose approximately $72,000 each day without the parking garage. It should be noted that this number is based on rough assumptions and cannot be labeled accurate without more information. However, when asked about its daily revenue, mall management stated that they could not release that information, therefore these assumptions must be made.

For this estimate, the foundation system was assumed to be spread footings. It was recommended that a deep foundation system, such as piles, could be used in Arlington in order to decrease any structure settlement. However, this system only works efficiently with certain soil types. According to an interactive map from the U.S. Department of Agriculture NRCS, the corner of N. Glebe Rd. and N. Randolph St. is composed mainly of construction fill material. A deep foundation system would not work properly under these soil conditions.

The estimate for the precast parking garage was taken from unit price numbers given by a salesman from High Concrete. This salesman had access to dimensions, plans, and elevations for the proposed design. The email with these numbers can be found in Appendix F.

The steel cost numbers were calculated based on a recommendation by Charlie Carter of AISC. He stated that wide flange shapes cost approximately $0.44 per pound and HSS tubes cost about $0.49 per pound. The email with these numbers can be found in Appendix F.

Without taking into consideration a loss in mall revenue, the total construction cost of the proposed design is $53M, about $10M more than the original design, a 24% increase.

Project Schedule

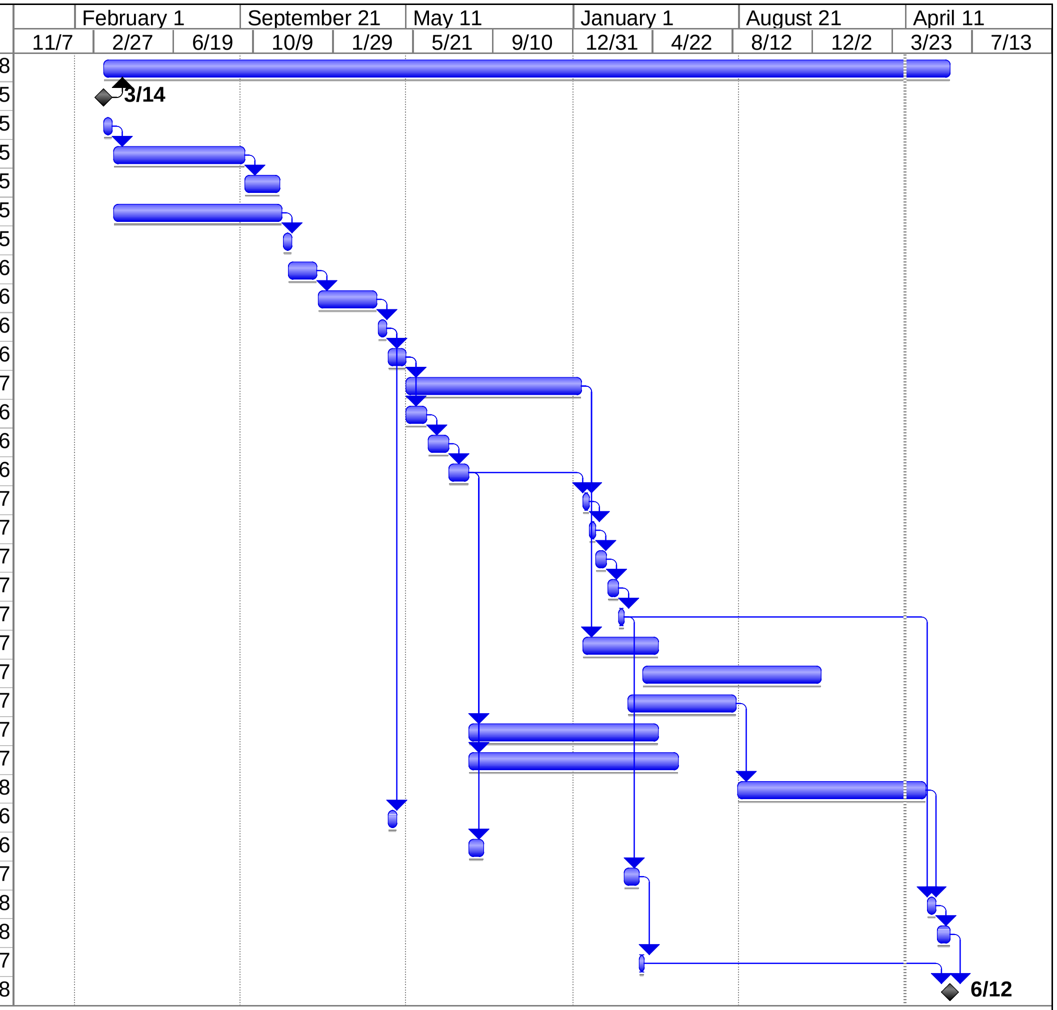
Another determining factor in the feasibility of the proposed design is the project schedule, how long construction will take. The original design called for a total duration of 360 working days, which came out to be approximately 495 days total. A detailed Gantt Chart for the original project is provided in Appendix D.

The schedule for the proposed design was created based on RS Means daily output numbers, data from the original schedule, and input from professional engineers in the industry. For instance, Ken Bauer, Director of Research and Development & Technical Sales Support at High Concrete, provided information on how their MEGA-SPAN system is erected and how long it takes to construct. This information can be found in an email in Appendix F.

The schedule for the proposed design calls for a duration of 848 working days, or a total of 1170 days. Based on engineering judgment and input from professional engineers in the industry, this does not seem accurate. One engineer stated that this project seems like a 30 month project, not a 39. Most likely there is a lot of activity overlap that was missed in this schedule.

The approximate 30 month project schedule is about twice as lengthy as the original schedule. The most likely cause of this big difference is the demolition time, which runs approximately one year in length. Also, erecting the large steel trusses makes up a large portion of the schedule. The precast parking garage, however, is erected extremely fast compared to its large size. One crew can erect an average of 22 pieces per day. The proposed schedule can be found on the following page.

ID	Task Name	Duration	Start	Finish
1	TOTAL PROJECT DURATION	848 days	Mon 3/14/05	Thu 6/12/08
2	Notice to Proceed	0 days	Mon 3/14/05	Mon 3/14/05
3	Mobilization	10 days	Mon 3/14/05	Fri 3/25/05
4	Building Demolition	132 days	Mon 3/28/05	Tue 9/27/05
5	Foundation Demolition	35 days	Wed 9/28/05	Tue 11/15/05
6	Disposal	170 days	Mon 3/28/05	Fri 11/18/05
7	Excavation	10 days	Mon 11/21/05	Fri 12/2/05
8	Site Utilities	30 days	Mon 11/28/05	Fri 1/6/06
9	Form and Pour Foundation	60 days	Mon 1/9/06	Fri 3/31/06
10	Backfill	10 days	Mon 4/3/06	Fri 4/14/06
11	Pour SOG	19 days	Mon 4/17/06	Thu 5/11/06
12	Steel Erection	175 days	Fri 5/12/06	Fri 1/12/07
13	Precast Parking Bay 1	21 days	Fri 5/12/06	Fri 6/9/06
14	Precast Parking Bay 2	20 days	Mon 6/12/06	Mon 7/10/06
15	Precast Parking Bay 3	20 days	Tue 7/11/06	Mon 8/7/06
16	Precast Parking Bay 4	7 days	Mon 1/15/07	Tue 1/23/07
17	Precast Parking Bay 5	7 days	Wed 1/24/07	Thu 2/1/07
18	Precast Parking Bay 6	11 days	Fri 2/2/07	Fri 2/16/07
19	Precast Parking Bay 7	11 days	Mon 2/19/07	Mon 3/5/07
20	Precast Parking Bay 8	6 days	Tue 3/6/07	Tue 3/13/07
21	Perimeter Framing	76 days	Mon 1/15/07	Mon 4/30/07
22	Exterior Metal Panels	180 days	Mon 4/9/07	Fri 12/14/07
23	Exterior Glass Systems	110 days	Mon 3/19/07	Fri 8/17/07
24	Stairs	190 days	Tue 8/8/06	Mon 4/30/07
25	Elevators	210 days	Tue 8/8/06	Mon 5/28/07
26	Interiors	190 days	Mon 8/20/07	Fri 5/9/08
27	Roadway	10 days	Mon 4/17/06	Fri 4/28/06
28	Sidewalks	15 days	Tue 8/8/06	Mon 8/28/06
29	Landscaping	15 days	Wed 3/14/07	Tue 4/3/07
30	Final Inspections	10 days	Mon 5/12/08	Fri 5/23/08
31	Punchlist	14 days	Mon 5/26/08	Thu 6/12/08
32	Site Punchlist	5 days	Wed 4/4/07	Tue 4/10/07
33	Substantial Completion	0 days	Thu 6/12/08	Thu 6/12/08



Project: Project1
Date: Wed 4/9/08

Task		Milestone		External Tasks	
Split		Summary		External Milestone	
Progress		Project Summary		Deadline	

CONCLUSION

When the Iceplex was built on top of an existing parking garage, much of the existing structure had to be reinforced for the additional gravity and lateral loads. This proved to be the most complicated part of the project. A proposed solution, to demolish the parking garage and build from the ground up, was analyzed for its feasibility. First a civil/site analysis was completed to determine the best site access points. Then, the architectural layout and façade of the building were redesigned. Next, large transfer trusses were designed in order to take the loads from the parking garage above when the Iceplex was relocated to ground level. Finally, a construction management analysis was completed comparing the cost and schedule of the proposed design to the original.

Based on the civil/site analysis, the proposed solution seems to be very feasible. In fact, the building footprint can be reduced by almost 15%. The traffic analysis proved that the existing garage entrance and exit locations could remain. This would eliminate any additional traffic signal issues.

The architectural redesign of the Iceplex also concluded that the proposed design was a definite possibility to the problem. When the ice rinks were relocated to ground level, entering and exiting the Iceplex became very convenient. A drop-off loop was designed to allow occupants to be dropped off without parking their vehicle or without going to the 8^{th} level of the parking garage. The layout of the spaces remained somewhat consistent with the original design. This should assist in obtaining the owner’s approval of the proposed project. Also, the square footages of most spaces were within reasonable margins of the original design.

Based on the structural design of the transfer system, the proposed design cannot be recommended. Designing the trusses was extremely difficult and took three attempts before working. If this system were to actually be used, a large design fee would be charged by the structural engineer to effectively design the members and the complicated connections. Also, the columns needed to support these trusses were extremely large. Since these columns are located in common corridors as well as office spaces, their size will be extremely inconvenient to both building occupants and the architectural layout.

The construction management analysis also proved the proposed design not feasible. The project cost was increased by $10M and the schedule was twice as long as the original design. Both play an important role to the building owner.

In conclusion, the proposed design cannot be recommended as a possible solution. The disadvantages of a complicated design and an increased cost and construction duration outweigh the advantages. However, if the owner did indeed want to relocate the Iceplex to ground level, other structural and architectural possibilities could be examined. For instance, the Iceplex could be constructed independently of the parking garage. The garage could then become an adjacent tower to the Iceplex. Of course, this all depends on whether or not the site size allows for such architecture. This would then eliminate the need to transfer out extremely high gravity and lateral loads.