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Kettler Capitals Iceplex
Arlington, Virginia

Structural Option
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TECHNICAL REPORT I
STRUCTURAL CONCEPTS & EXISTING CONDITIONS

EXECUTIVE SUMMARY

This is an existing conditions report for the Kettler Capitals Iceplex, the practice facility for the Washington Capitals in Arlington, Virginia. The report starts with an in depth description of the existing structural systems. This includes typical framing plans for the reinforcing of the pre-existing parking structure, the floor system, roof system, and lateral resisting system. Then, an analysis of the building loading is provided. These loads include live loads, dead loads, snow loads, wind loads, and seismic loads. Next, using these loads, a spot-check of four framing elements was completed including a girder, a composite deck, a roof joist, and a lateral braced frame. Finally, it can be concluded that there are a few very minor differences between the original design and the analysis provided in this report. These differences could be because two different codes were used. Also, the design of some elements was made using manufacturer catalogs that are specific to certain material properties. If a used material is not exactly the material tabulated, varying designs will result.

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INTRODUCTION

The Kettler Capitals Iceplex, which is the practice facility for the National Hockey League team, the 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 SF 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 60 ft. 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 has 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 posttentioned concrete. Then in 1986, the existing five level structure was topped with two more levels, one posttentioned concrete and the other composite steel. See Figure 1 for a schematic phasing diagram of these additions.

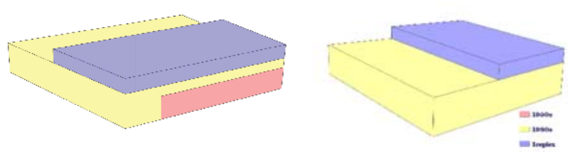


Figure 1

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 200 ft. 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. The need for large column-fee spaces limited the locations where lateral members could be placed.

This report begins to discuss these structural issues and uses various analyses to explain the existing structural system of the Kettler Capitals Iceplex.

CODES AND MATERIAL PROPERTIES

Codes and Standards

The Kettler Capitals Iceplex was designed using Building Officials and Code Administrators, Inc (BOCA), 1996 and ASCE 7-95 for building loads and structure analysis. Concrete design used American Concrete Institute, ACI 318-95 and the Manual of Steel Construction –Allowable Stress Design, 9th Edition, 1989 was used for the steel design.

This report will use a newer version of code to analyze the existing structure. The International Building Code (IBC 2006) and ASCE7-05 was used to determine loads and analysis procedures. The concrete and steel codes used will be ACI 318-05 and AISC Steel Construction Manual –Load and Resistance Factor Design, 13th Edition 2005 respectively.

Material Properties

Concrete

Slab-on-grade	3500 psi	145 pcf
Reinforced Slabs	5000 psi	145 pcf
Reinforced Beams	5000 psi	145 pcf
Fill on Metal Deck	3500 psi	115 pcf
Columns	5000 psi	145 pcf
Walls	4000 psi	145 pcf
Grade Beams	3000 psi	145 pcf
Footings	5000 psi	145 pcf
Parking Level Concrete Topping	5000 psi	145 pcf
Rink Slab	5000 psi	115 pcf

Structural Steel

Rolled Shapes	ASTM A992, Grade 50
Channels, Angles, and Plates	ATMS A36
Structural Pipe	ASTM A53, Grade B, F_y = 35 ksi
Round HSS Shapes	ASTM A500, Grade B, F_y = 42 ksi
Structural Tubing	ASTM A500, Grade B, F_y = 46 ksi
Steel Joists	per Steel Joist Institute

STRUCTURAL SYSTEMS

Reinforcing 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

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. Please see the appendix for the tabulated results.

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. See the columns shaded red in Figure 3 for the location of these columns. 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. See the columns shaded blue in Figure 3 for the locations of these columns and Figure 4 for a detail.

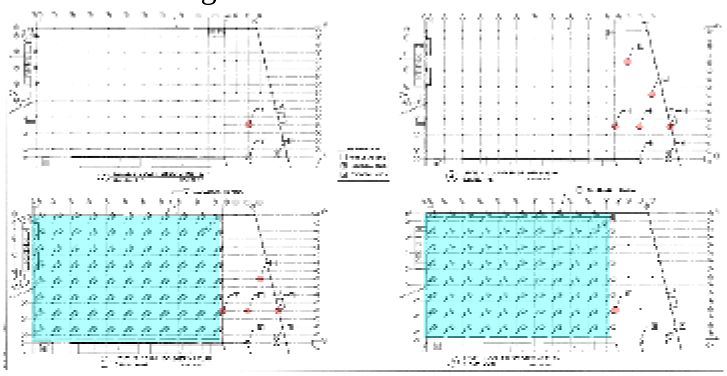


Figure 3

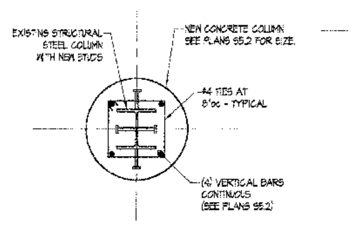
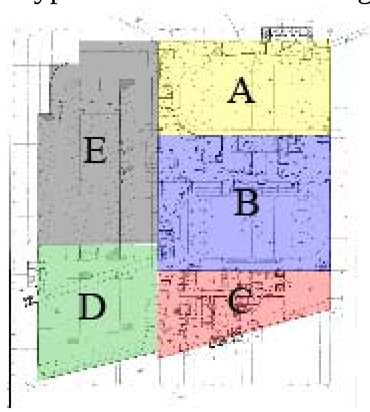


Figure 4

Floor Framing

This report will now concentrate on the framing plans for the new structure located on levels 8 and 9. Please see the keyplan below for area designations that will be used.



Keyplan

Expansion Joints

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. Please see Figure 5 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.

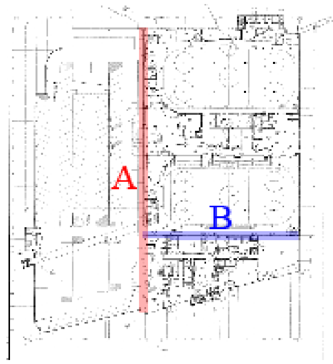


Figure 5

Areas 8A & 8B

Areas 8A and 8B are located on the 8th floor of the new Iceplex facility and are the location of both regulation-sized ice rinks. 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" dia. x 4" long. It was noted that rebar is not traditionally used in a composite slab system; however, it was necessary to properly support the ice. Steel columns supporting the rinks range from W12x58s to W14x257s. Please see Figure 6 for a typical framing plan.

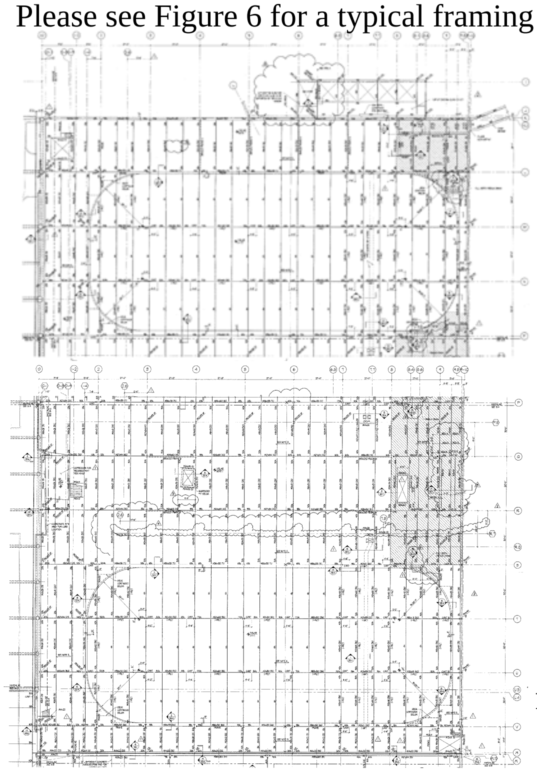


Figure 6

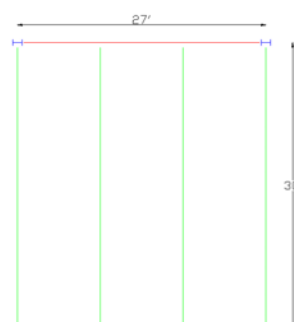
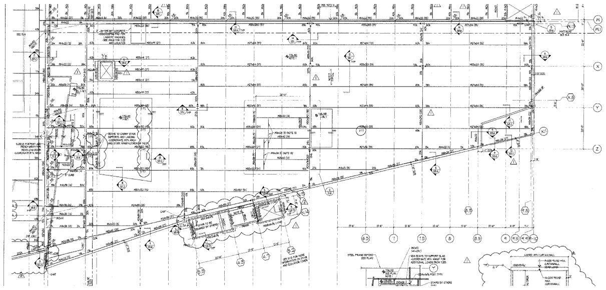


Figure 6A

Area 8C

Area 8C is located adjacent to Area 8B and ice rink no. 2. The Washington Capitals team offices, locker rooms, and weight room are located in this area. The slab in Area 8C consists of 3¼" lightweight concrete over 2" deep 18 gage galvanized metal deck (total thickness = 5¼") reinforced with 6x6 W1.4x1.4 WWF. The shear studs in this location were also ¾" x 4" long. Girders supporting the slab consist of mostly W27s and W33s. Heavier W33x201 are used to support a hot tub (see Figure 7A). These girders span 54'-0" and are spaced at 10'-0"oc. Composite beams range from a W21x44 spanning 22'-6" to a W36x439 spanning 50'-0". Steel columns supporting Area 8C are W14s weighing from 53 to 398 lb/ft and run all the way to the roof in most cases. See Figure 7 for a typical framing plan.



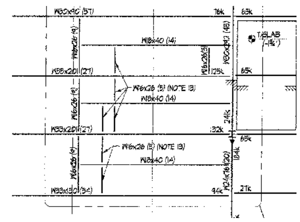


Figure 7A

Figure 7

Area 8D

Area 8D is located just to the west of area 8C and is an expansion of the existing parking garage. The addition will add approximately 60 more parking spaces. This area is constructed of a solid 5" thick normal weight concrete slab. Reinforcing consists of continuous rebar mats of #6 at 12"oc top and #4 at 12"oc bottom running in the north-south direction and #4 at 12"oc top and bottom running in the east-west direction. Composite W18s and W21s support the west side of the slab. These span 38-0" and are spaced at 11'-0"oc. Composite W21s and W30s support the east side of the slab. These span as long as 69 ft. and are also spaced at 11'-0"oc. These girders span into composite beams which range from W21x44 to W30x173 and span from 20-42 ft. The steel columns below consist of W10s, W12, and W14s. See Figure 8 for a typical framing plan of area 8D.

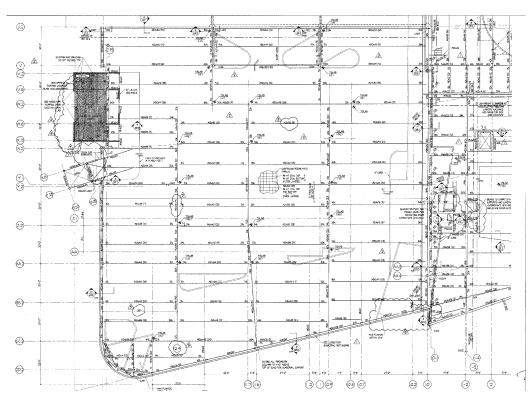


Figure 8

Areas 9A and 9B

Areas 9a and 9B support the mezzanine and bleachers that overlook the ice rinks. Both areas consist of a 3¼" lightweight concrete structural slab over a 2" deep 18 gage galvanized composite metal deck (total thickness = 5¼") reinforced with 6x6 W1.4x1.4 WWF. Composite girders are typically W16x26s and W12x14s spaced at 9'-0"oc and span 30'-'0". Composite beams range from W16s to W24s. They span 28'-0" including a 4 ft. cantilever on the west side of Area 9A. Beams in Area 9B span 27'-0". Most of the columns supporting level 9 are a continuation of the columns supporting level 8.

Area 9C

The Washington Capitals' corporate offices are located on the 9th floor of the Iceplex in Area 9C. The floor slab in this area is a 3¼" lightweight concrete structural slab over a 2" deep 18 gage galvanized composite metal deck (total thickness = 5¼") reinforced with 6x6 W1.4x1.4 WWF. Composite girders vary in size from light W24s to heavy W33s. The maximum span of these girders is 54'-0" and they are spaced at 10'-0"oc. The girders frame into very large composite beams which can be as large as a W33x291. The columns supporting Area 9C are continuous from the 7th floor.

Roof Framing

The need for long-span, column free spaces was critical in the design of the roof over the two ice rinks. Please see Figure 9 for a roof plan. 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 120 ft. and are spaced at 5'-6"oc. Three custom trusses were also designed to support the roof over Areas A and B and are shown in green. Two of these trusses are located along grid 2 and have a small slope to them. They span 120 ft. and are designed with WTs as top and bottom chords and double angles and single angle diagonals. A shorter custom truss, along grid V spans 81 ft. with no slope. This truss consists of wide flange top and bottom chords with double angle diagonals. Additional K-Series open web steel joist and wide flange shapes support the remaining part of the roof in Area A and B. The roof deck in this area is 1½" deep, wide rib 18 gage galvanized metal deck.

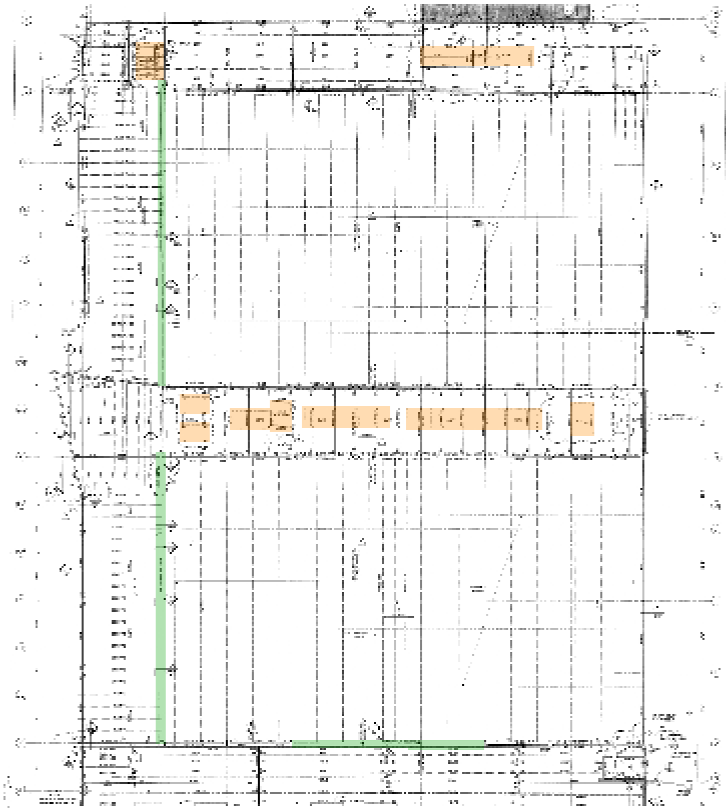


Figure 9

The roof framing of Area C consists of a mix between open web steel joists and wide flange beams. See Figure 10 for a plan. The joists are LH and K-Series joists spaced at 5'-0"oc and span a maximum of 54 ft. The wide flange section of roof consists of mostly W24s with a few W18s. These are also spaced at 5'-0"oc and span 54 ft. The roof deck in Area 8C is also 1½" deep, wide rib 18 gage galvanized metal deck.

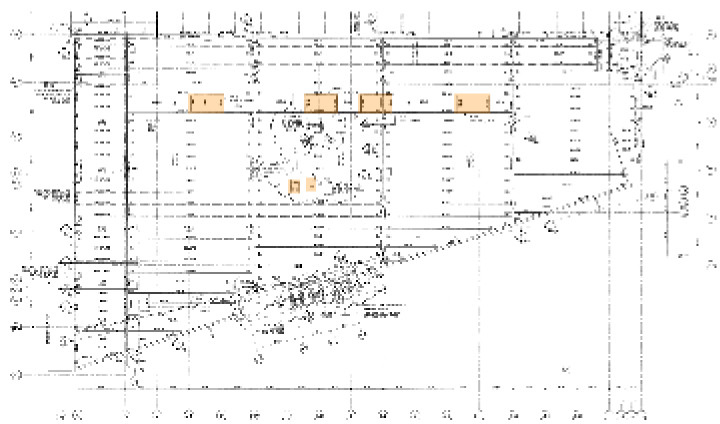


Figure 10

There are several roof top mechanical units that were taken into consideration during the design of the roof system. They are shown in orange. Here, increase steel was used to account for the additional load. These areas can be seen shaded in the above plans.

Lateral System Framing

Since the Iceplex was built on top of an existing parking structure that was also expanded, there are several different types of lateral resisting frames throughout the building. This report will now concentrate on the lateral system of all nine levels of Areas A and B of the structure.

The system of the 1950s parking garage consists of a two-way slab system. This system can be found on the entire footprint of the building and on all five levels as you can see in Figure 11. This slab has a total thickness of 10½".

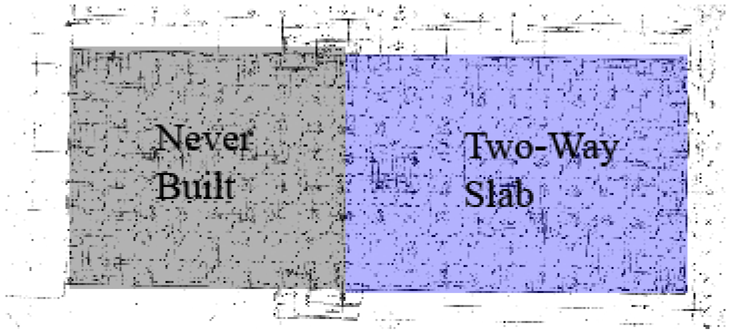


Figure 11

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 12. 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.

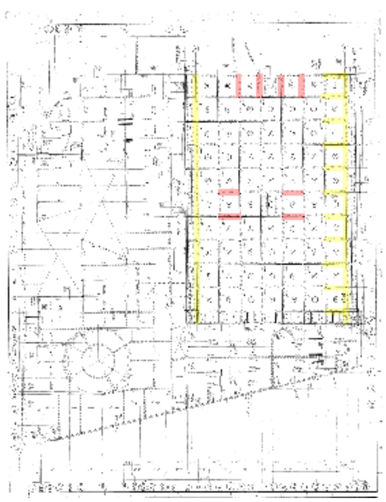


Figure 12

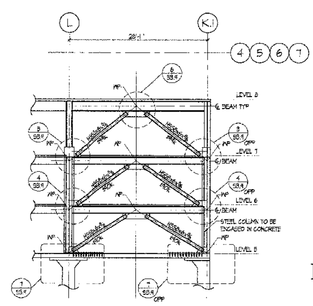


Figure 12A

Finally, when the Iceplex was added onto the parking structure, a mix of braced frames and moment connections was used. Eight braced frames were constructed on the 7th level reinforcing the existing structure for additional lateral forces. These frames are shown in red in Figure 12 and a detail of these braced frames is shown in Figure 12A. On the 8th level, there are a total of eight braced frames, four in each direction. These are shown in blue in Figure 13. 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 13.

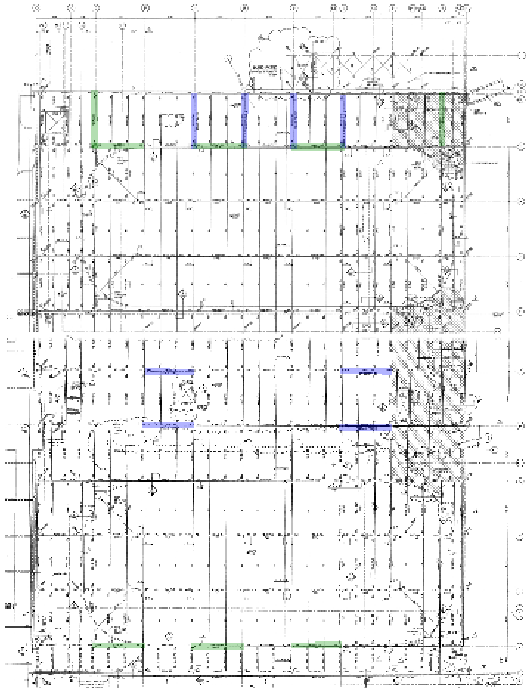


Figure 13

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. Figure 14 shows the location of all lateral resisting frames in Area 9B.

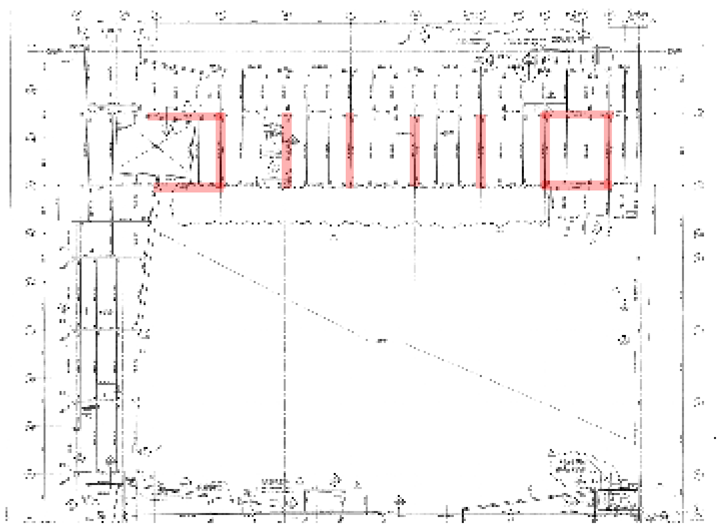


Figure 14

BUILDING LOADS

Live Loads

AREA	PSF USED BY ENGINEER OF RECORD	PSF USED IN ANALYSIS
Framed Floor Areas	100	100
Lobbies, Stairs, Exits	100	100
Mechanical	As noted on plans	As noted on plans
Ice Rink	100	100
Parking Decks	50	40
Parking Decks (Top Level)	80 (50LL + 30 snow)	70 (40LL + snow)
Roof LL	30 or snow load (whichever is greater)	25 or snow load (whichever is greater)

Dead Loads

AREA	PSF USED BY ENGINEER OF RECORD	PSF USED IN ANALYSIS
Rink	1.5" ice = 7.8	132
	5" NW Concrete = 63	
	4" Insulation = 6	
	4" Sand = 40	
	Misc = 15	
	132	
Wet Areas	30	30
Parking	3	3
Planter	440	440
Other	15	15

Snow Loads

Since the Iceplex has various roof heights, snow drift should be examined in order prevent overloading the structure. Snow drift was analyzed using ASCE7-2005. Three snow drift conditions were analyzed-first, the area where the Iceplex meets the 8th floor of the parking structure; second, along the garage parapet on the roof; and finally, the area where area B and C meet at varying roof heights. See Figure 15 for a diagram of these locations. The worst of these possible conditions was determined to be on the 8th floor parking structure near the Iceplex vertical expansion, shown in green. See the loading diagram shown below. Below is a list of the input parameters used during analysis. Please see the appendix for the calculations and loading diagrams.

- Ground Snow Load (P_g) 25 psf
- Snow Exposure Factor (C_e) 0.9
- Thermal Factor (C_t) 1.2
- Importance Factor (I) 1.1

Figure 15

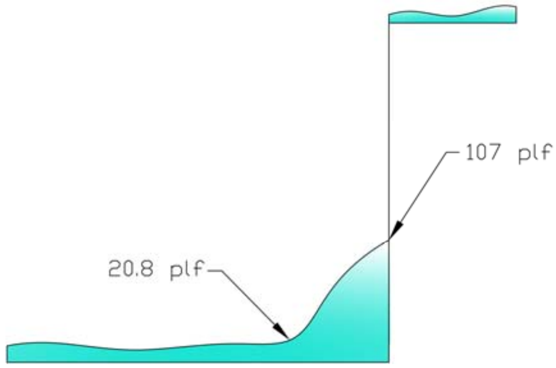
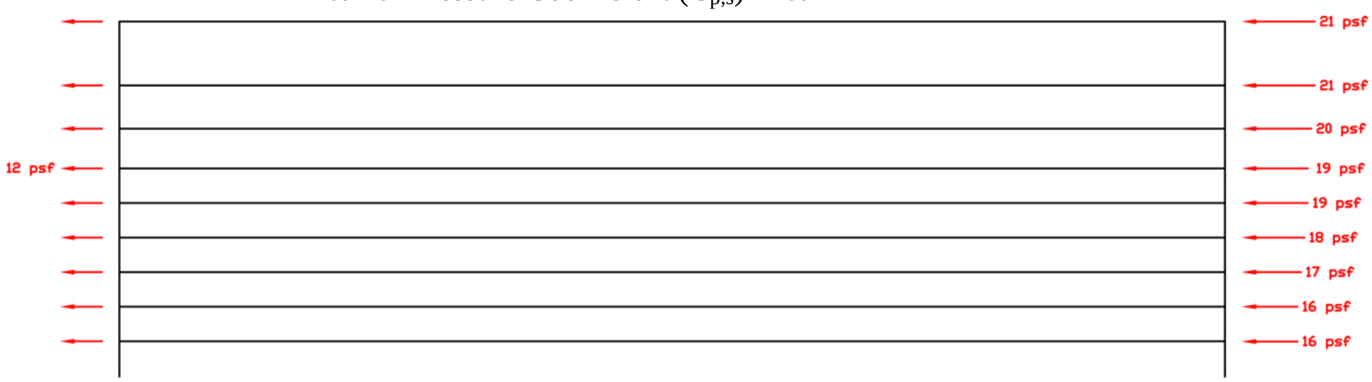


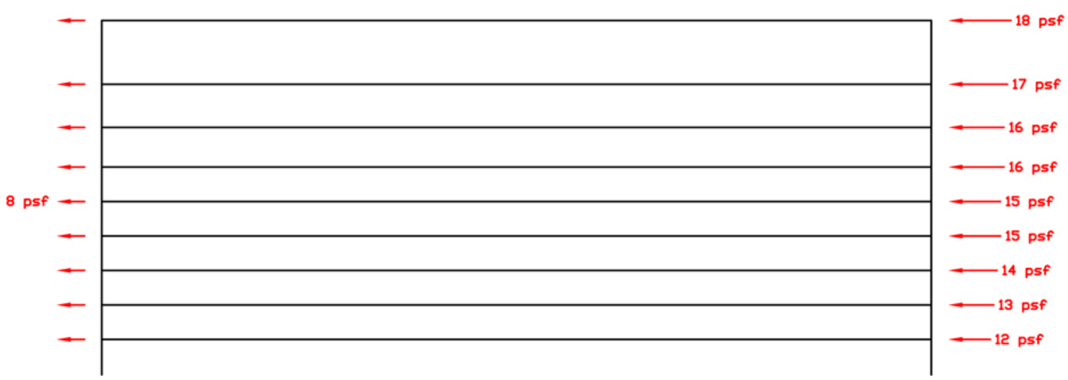
Figure 16

Wind Loads

Wind loads were calculated in accordance with ASCE7-05. A wind analysis of Areas 8A and 8B was performed. Below is a list of assumptions made during analysis. It should be noted that a partially enclosed building was assumed for the entire structure since the majority of the structure is a parking garage with large openings on the façade. It was also assumed that the entire footprint of the building extends the full nine floors. This will create a larger wind surface than in realty; therefore, giving conservative wind loads. The loading diagram is also shown below. It was assumed during analysis that wind forces will hit all four sides of Areas A and B without the interference of adjacent structures. In reality, wind forces will be blocked on the north from the Ballston Mall, on the south on all 9 levels from Area C, and on the west on levels 1-7 from the parking structure. Please see the appendix for the calculation spreadsheets.

- Basic Wind Speed (V) 90 mph
- 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





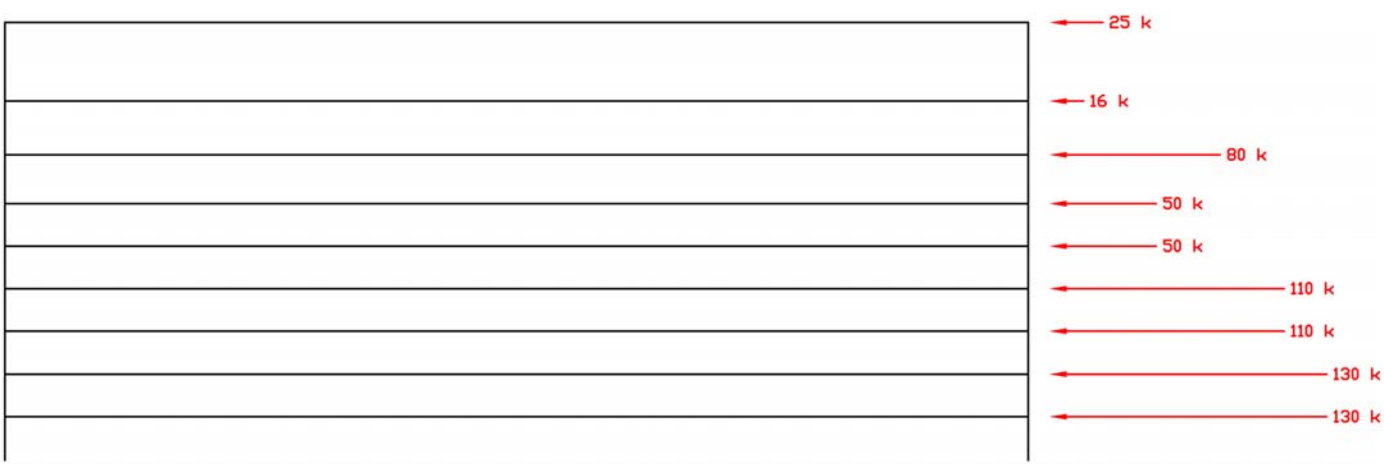
Seismic Loads

Seismic loads were calculated using ASCE7-05. A seismic analysis was completed on Areas A and B. Below is a list of input parameters used during analysis and the loading diagram. Please see the appendix for the calculation spreadsheet.

• Ss	0.154
• S1	0.0051
• Site Class	D
• Occupancy Category	III
• F_a	1.6
• F_v	2.4
• Importance Factor (I)	1.25
• Response Modification Coefficient	3 (most conservative)
• Approximate Period (T_a)	0.65

Final Results

Base Shear:	**400 kip**
Overturning Moment:	**27,000 ft k**



SPOT CHECK OF FRAMING DESIGN

Area 8B Girder

The first structural element checked was a girder supporting the ice rink in Area 8B. There were three different loading cases for this girder. First, a uniformly distributed dead load (including superimposed dead load from the excess weight of the rink slab) and a uniformly distributed live load; second, the same uniformly distributed load as in the first case, plus a live load from the zamboni repairing the ice; and finally, a load case that takes into consideration only the wet weight of the concrete. It was found that the first case will control in flexure and in the required I_{LB} needed for the composite system, and the last case will control over the required moment of inertia for deflection. Based on the analysis, using a W18x40 girder with 13 shear studs was the most economical choice. A original design required a W18x40 with 36 shear studs. The original design differed only in the number of shear studs. The probable reason for this difference is that more composite action was desired by the structural engineer of record; therefore, more shear studs were used. Please see the appendix for this design calculation.

Composite Deck

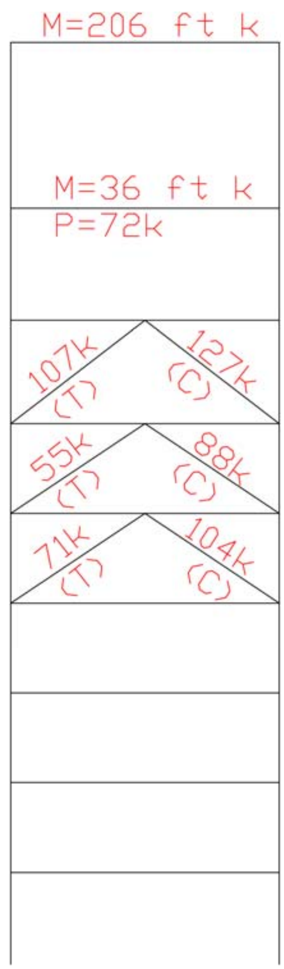
The United Steel Deck, Inc. design tables found online were used in the design of the floor deck. The same M_u from the girder design was used from the deck tables. Using this required moment capacity, an adequate deck was not found. This is because these tables use an f'_c of 3 ksi where the strength of the concrete used was 5 ksi. It is believed that this will significantly increase the moment capacity of the composite deck. A designer might need to contact USD to find design moments for concrete of this strength. Please see the appendix for the design table that was used in this analysis.

Roof Joist

Due to the required long span over the two ice rinks, there was only one practical design for the roof framing, bar joists. Since the joists span a long 120 ft., there was only one series of bar joist that would work and that was the DLH Series. Based on a roof live load of 25psf and an approximate roof dead load of 30psf, a 64DLH15 would be adequate. *(25+30)*5=275plf+43plf (weight of joist)=318plf<375plf.* The original design calls for a 68DLH16 which has load capacity of 441plf. This difference could be due to an under-estimation of the roof dead load.

Lateral Braced Frame

A RAM model was used to analyze the lateral system of the building. Using the parameters listed in the Wind Load section, member forces for the braced frame shown below were found. The original design calls for HSS8x6x3/8. As shown in the diagram the maximum tensile force these braces will see is 107 kips and the maximum compressive force is 127 kips (unfactored). According to Table 5-4 in the AISC Steel Manual, the ASD tension strength of an HSS8x6x3/8 is 195 kips. There is a slight difference between the original design but not much. One possibility for this difference could be that lower wind forces may have been calculated using BOCA rather that IBC. Another possible reason for the difference could be that drift controlled instead of axial strength. According to Table 4-3 in the AISC Steel Manual, the compressive strength of an HSS8x6x3/8 is 142 kips with an effective length of 18 ft. Once again there is a slight difference in the design which could be for the same reasons. Overall, it can be concluded that the lateral braces in this frame were designed effectively and efficiently.



CONCLUSION

After analyzing the structural systems of the Kettler Capitals Iceplex, I have made a few conclusions. First, some loading cases are different. This is because a different code was used for the design than from my analysis. BOCA 1996 was used for design and IBC 2006/ASCE 7-05 was used during the analysis for this report. This created different live, wind, and seismic loads on the structure. Second, I have concluded that, through a spot check of the design, the structure was designed accurately. There were a few small differences from the original design to my design. This was most likely due to varying criteria, such as drift, and design tables that were meant for different material properties, such as the composite deck system.

APPENDIX

Beam Strengths from Geotechnical Report

Floor	Column Location	Windsor Probe Results (psi)	Compressive Strength (psi)[1]
1	T-6	6,747	5229
1	L-8	6,526	5058
1	M-3	6,747	5229
1	N-11	4,907	3803
1	W.2-10.8	6,305	4887
	Average :	6,246	4,841
4	Beam at Y-3.5[3]	6,379	4944
4	Y-3.5	6,452	5001
4	T-9	6,158	4773
4	L-2	6,600	5115
	Average :	6,397	4,958
6	M-11	6,526	5058
6	S-8[4]	5,275	4088

Snow

DRIFTING SNOW CALCULATIONS

CODE: ASCE-7 2005 (IBC)

LOCATION: Iceplex onto Garage

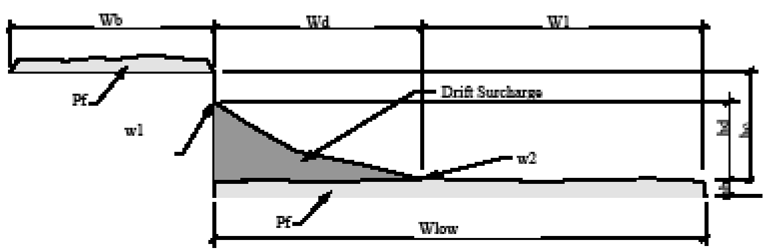
DRIFTING:

P_g (psf) = 25
C_e = 0.9
I = 1.1
Ct= 1.2

P_f (psf) = 0.7*P_g*C_e*Ct*I = 20.79
D (lb/ft^3) = 17.25
W_{low} (ft) = 170.0
W_b (ft) = 240.0
step height= h_r (ft) = 13.3
h_d (ft) = 5.00

h_b (ft) =	1.21	$(h_d)_{ww}$=	3.22 ft
W_d (ft) =	20.00	$(h_d)_{lw}$=	5.00 ft
W_1 (ft) =	150.00	$(h_d)_{max}$= h_c= h_r-h_b=	12.09 ft

w1 (plf) = **107.0**
w2 (plf) = **20.8**



DRIFTING SNOW CALCULATIONS
CODE: ASCE-7 2005 (IBC)

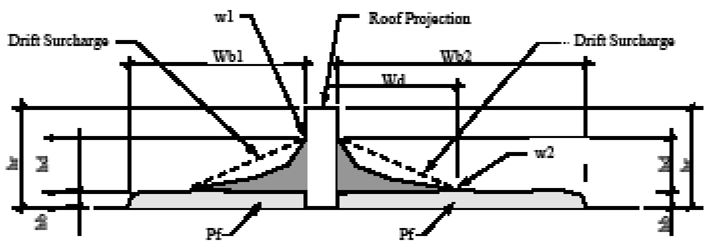
LOCATION: 4' parapet along grid 13

DRIFTING:

Pg (psf) = 30
Ce = 0.7
I = 1.1

Pf (psf) = Pg*Ce*I = 23.1
D (lb/ft^3) = 17.9
Wb (ft) = 170.0
hr (ft) = 4.0
hd (ft) = 4.49
hb (ft) = 1.29
Wd (ft) = 10.84

w1 (plf) = 103.5
w2 (plf) = 23.1



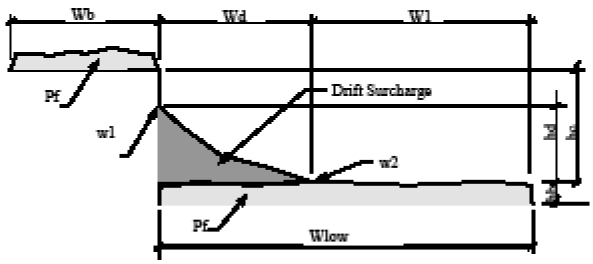
DRIFTING SNOW CALCULATIONS CODE: ASCE-7 2005 (IBC)

LOCATION: B onto C

DRIFTING:

P_g (psf) = 25
C_e = 0.9
I = 1.1
Ct= 1.2

P_f (psf) = $0.7^*P_g^*C_e^*Ct^*I$ = 20.79
D (lb/ft^3) = 17.25
W_{low} (ft) = 70.0
W_b (ft) = 120.0
step height= h_r (ft) = 14.0
h_d (ft) = 3.66
h_b (ft) = 1.21
W_d (ft) = 14.64
W_1 (ft) = 55.36



w1 (plf) = **83.9**
w2 (plf) = **20.8**

$(h_d)_{ww}$= 2.11 ft
$(h_d)_{lw}$= 3.66 ft
$(h_d)_{max}$= h_c= h_r-h_b= 12.79 ft

Wind

Main Windforce Resisting System

CODE:

International Building Code 2000 / ASCE 7-98

INPUT:

Building Height (H):	103 ft		
Building Depth (L):	240 ft		
Building Width (B):	320 ft	Wind on Broad Face of Building	
Number of Stories (N):	9		
Basic Wind Speed (V):	90 MPH	Figure 6-1 (pg 34)	
Wind Directionality Factor (K_d):	0.85	Table 6-6 (pg 61)	
Building Category:	III	Table 1-1 (pg 4)	
Importance Factor (I):	1.15	Table 6-1 (pg 55)	
Exposure Category:	B	6.5.6.1 (pg 28)	
Topographic Factor (K_{zt}):	1.00	6.5.7 (pg 29)	
Gust Effect Factor (G or Gf): Use Calculated?	Yes	6.5.8 (pg 29)	Frequency (Hz) = 1.11
Internal Pressure Coefficients (+/-GC_{pi}):	0.18	Table 6-7 (pg 62)	
External Pressure Coefficient (C_p windward):	0.8	Figure 6-3 (pg 42)	
External Pressure Coefficient (C_p leeward):	-0.3	Figure 6-3 (pg 42)	L/B = 0.75
External Pressure Coefficient (C_p sidewall):	-0.7	Figure 6-3 (pg 42)	

FORMULAS:

$p = qGC_p - q_i(GC_{pi})$ Equation 6-15 (6.5.12.2 pg 31)

$q_z = 0.00256(K_z)(K_{zt})(K_d)(V^{\wedge}2)(I)$ Equation 6-13 (6.5.10 pg 30)

CALCULATIONS:

Gust Effect Factor:

z = 61.8 ft	frequency (n_1) = 1.11 Hz	*Rigid*	$g_Q = g_v = 3.4$
	I_z = 0.270	$L_z = 394.4$	Q = 0.777

Rigid Structures: $G = 0.925[(1+1.7g_Q I_z Q)/(1+1.7g_v I_z)]$ Equation 6-2 (6.5.8.1 pg 29)

Flexible Structures: $G_f = 0.925[(1+1.7I_z{*}sqrt(g_Q{}^{\wedge}2Q^{\wedge}2+g_R{}^{\wedge}2(R^{\wedge}2)))/(1+1.7g_v I_z)]$ Equation 6-6 (6.5.8.2 pg 29)

$g_R = 4.215$	$b = 0.45$	$\alpha = 0.25$	$V_z = 69.49$
$N_1 = 6.307$	$R_n = 0.044$	$\eta_h = 7.58$	$R_h = 0.123$
$\eta_B = 23.54$	$R_B = 0.042$	$\eta_L = 59.10$	$R_L = 0.017$
$R = 0.110$			

Velocity Pressure and Wind Force Summary

Location	Height (ft)	K_z	q_z	G	Ext. Pres. qGC_p	Internal Pressure q_i	Internal Pressure $q_i(GC_{pi})$	Combined Pressure (+GC_{pi})	Combined Pressure (-GC_{pi})	Design Load Ww + Lw Height (ft)	Design Load Ww + Lw Load (psf)
Windward	10.25	0.575	11.65	0.799	7.45	20.19	3.63	3.81	11.08	10.25	12.28
	20.25	0.626	12.69	0.799	8.11	20.19	3.63	4.48	11.75	20.25	12.95
	30.25	0.702	14.23	0.799	9.10	20.19	3.63	5.46	12.73	30.25	13.94
	40.25	0.762	15.44	0.799	9.87	20.19	3.63	6.24	13.51	40.25	14.71
	50.25	0.812	16.46	0.799	10.52	20.19	3.63	6.88	14.15	50.25	15.36
	60.25	0.855	17.33	0.799	11.08	20.19	3.63	7.44	14.71	60.25	15.92
	71.75	0.899	18.22	0.799	11.64	20.19	3.63	8.01	15.28	71.75	16.48
	84.25	0.941	19.07	0.799	12.19	20.19	3.63	8.56	15.83	84.25	17.03
	102.75	0.996	20.19	0.799	12.90	20.19	3.63	9.27	16.54	102.75	17.74
Leeward	ALL	0.996	20.19	0.799	-4.84	20.19	3.63	-8.47	-1.21		
Side Walls	ALL	0.996	20.19	0.799	-11.29	20.19	3.63	-14.92	-7.66		

Main Windforce Resisting System

CODE:

International Building Code 2000 / ASCE 7-98

INPUT:

Building Height (H):	103 ft		
Building Depth (L):	320 ft		
Building Width (B):	240 ft	Wind on Narrow Face of Building	
Number of Stories (N):	9		
Basic Wind Speed (V):	90 MPH	Figure 6-1 (pg 33)	
Wind Directionality Factor (K_d):	0.85	Table 6-6 (pg 61)	
Building Category:	III	Table 1-1 (pg 4)	
Importance Factor (I):	1.15	Table 6-1 (pg 55)	
Exposure Category:	B	6.5.6.1 (pg 28)	
Topographic Factor (K_{zt}):	1.00	6.5.7 (pg 29)	
Gust Effect Factor (G or Gf): Use Calculated?	Yes	6.5.8 (pg 29)	Frequency (Hz) = 1.11
Internal Pressure Coefficients (+/-GC_{pi}):	0.18	Table 6-7 (pg 62)	
External Pressure Coefficient (C_p windward):	0.8	Figure 6-3 (pg 42)	
External Pressure Coefficient (C_p leeward):	-0.5	Figure 6-3 (pg 42)	L/B = 1.33
External Pressure Coefficient (C_p sidewall):	-0.7	Figure 6-3 (pg 42)	

FORMULAS:

$p = qGC_p - q_i(GC_{pi})$ Equation 6-15 (6.5.12.2 pg 31)

$q_z = 0.00256(K_z)(K_{zt})(K_d)(V^2)(I)$ Equation 6-13 (6.5.10 pg 30)

CALCULATIONS:

Gust Effect Factor: frequency (n_1) = 1.11 Hz *Rigid* $g_Q = g_V = 3.4$

z = 61.8 ft I_z = 0.270 L_z = 394.4 Q = 0.796

Rigid Structures: $G = 0.925[(1+1.7g_Q I_z Q)/(1+1.7g_V I_z)]$ Equation 6-2 (6.5.8.1 pg 29)

Flexible Structures: $G_f = 0.925[(1+1.7I_z*\text{sqrt}(g_Q{}^\wedge 2Q{}^\wedge 2+g_R{}^\wedge 2(R{}^\wedge 2)))/(1+1.7g_V I_z)]$ Equation 6-6 (6.5.8.2 pg 29)

g_R = 4.215	b = 0.45	α = 0.25	V_z = 69.49
N_1 = 6.307	R_n = 0.044	η_h = 7.58	R_h = 0.123
η_B = 17.65	R_B = 0.055	η_L = 78.80	R_L = 0.013
R = 0.126			

Velocity Pressure and Wind Force Summary

Location	Height (ft)	K_z	q_z	G	Ext. Pres. qGC_p	Internal Pressure q_i	Internal Pressure $q_i(GC_{pi})$	Combined Pressure (+GC_{pi})	Combined Pressure (-GC_{pi})	Design Load Ww + Lw Height (ft)	Design Load Ww + Lw Load (psf)
Windward	10.25	0.575	11.65	0.810	7.55	20.19	3.63	3.92	11.18	10.25	15.73
	20.25	0.626	12.69	0.810	8.23	20.19	3.63	4.59	11.86	20.25	16.40
	30.25	0.702	14.23	0.810	9.23	20.19	3.63	5.59	12.86	30.25	17.40
	40.25	0.762	15.44	0.810	10.01	20.19	3.63	6.38	13.64	40.25	18.19
	50.25	0.812	16.46	0.810	10.67	20.19	3.63	7.03	14.30	50.25	18.84
	60.25	0.855	17.33	0.810	11.23	20.19	3.63	7.60	14.87	60.25	19.41
	71.75	0.899	18.22	0.810	11.81	20.19	3.63	8.17	15.44	71.75	19.98
	84.25	0.941	19.07	0.810	12.36	20.19	3.63	8.73	16.00	84.25	20.54
	102.75	0.996	20.19	0.810	13.08	20.19	3.63	9.45	16.72	102.75	21.26
Leeward	ALL	0.996	20.19	0.810	-8.18	20.19	3.63	-11.81	-4.54		
Side Walls	ALL	0.996	20.19	0.810	-11.45	20.19	3.63	-15.08	-7.81		

Seismic

SEISMIC LOADING CALCULATIONS

REF: ASCE7-05

General Information

Ss=	0.154	Fa=	1.6	SDS=	0.164
S1=	0.0051	Fv=	2.4	SD1=	0.008
Site Class:	D				
Occ. Cat.	III			Seismic Design Category:	A

Seismic Response Coefficient

I=	1.25	Csa=	0.068	CS=	0.005
R=	3.00	Csmax=	0.005		
Ta=	0.65	Csmin=	0.010		

Equivalent Lateral Force Calculation

Fx = 0.01Wx
Fx = story forces

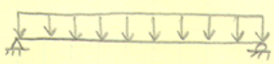
Vx = story shear
W = 76190.0
V=Cs*W 400.4

Floor(x)	Height(ft.)	Wx(kips)	Fx(kips)	Vx(kips)	Mx (ft.-kips)
Roof	102.75	2500.0	25.0	25.0	0.0
9	84.25	1600.0	16.0	41.0	2568.8
8	71.75	8000.0	80.0	121.0	3916.8
7	60.25	5000.0	50.0	171.0	9656.8
6	50.25	5000.0	50.0	221.0	12669.3
5	40.25	11000.0	110.0	331.0	15181.8
4	30.25	11000.0	110.0	441.0	19609.3
3	20.25	13000.0	130.0	571.0	22936.8
2	10.25	13000.0	130.0	701.0	25569.3
1	0.00	6090.0	60.9	761.9	26901.8
total		76190.0			

Girder Check

Design of Area 8B Girder — 1

Composite Steel
$f'_c = 5000$ psi
spacing = 9'-0"
Span = 30'-0"
$t_{slab} = 6.5''$

A)  $w_D + w_L$

$w_D = 56\,psf + 132\,psf$ S.DL $= 188\,psf$
$w_L = 100\,psf$

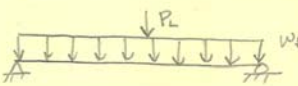
$w_u = 1.2D + 1.6L = 1.2(188) + 1.6(100) = 386\,psf \times 9' = 3.5\,k/ft$

$M_u = \frac{w_u L^2}{8} = \frac{3.5(30)^2}{8} = 394\ ft\cdot k$ * CONTROLS

$\Delta_L = L/480$ to prevent ice from cracking

$\frac{L}{480} = \frac{5 w_L L^4}{384 E I_{LB}} \Rightarrow \frac{30(12)}{480} = \frac{5(0.9)(30)^4(1728)}{384(29000) I_{LB}}$

$I_{LB} \geq 755\ in^4$ * CONTROLS

B)  P_L, w_D

$w_D = 188\,psf$
$P_L = 8000\,lb$ from zamboni
(worst case as pt. load @ midspan)

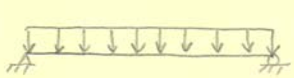
$w_u = 1.4D = 1.4(188) = 263\,psf \times 9' = 2.4\,k/ft$

$M_u = \frac{w_u L^2}{8} + \frac{PL}{4} = \frac{2.4(30)^2}{8} + \frac{8(30)}{4} = 330\ ft\cdot k$

$\Delta_L = L/480$

$\frac{L}{480} = \frac{PL^3}{48 E I_{LB}} \Rightarrow \frac{30(12)}{480} = \frac{8(30)^3(1728)}{48(29000) I_{LB}}$

$I_{LB} \geq 358\ in^4$

C)  w_D (Wet wt. of concrete) $= 56\,psf$

$w_u = 1.4D = 1.4(56) = 78.4\,psf \times 9' = .706\ k/ft$

$M_u = \frac{w_u L^2}{8} = \frac{0.706(30)^2}{8} = 80\ ft\cdot k$

$\Delta_T = L/480$

$\frac{L}{480} = \frac{5 w L^4}{384 E I} \Rightarrow \frac{30(12)}{480} = \frac{5(0.706)(30)^4(1728)}{384(29000) I}$

$I \geq 592\ in^4$ * CONTROLS

Design of Area 8B
Girder 2

Requrements: $I \geq 592 \text{ in}^4$ (W18×40 min.)
$I_{LB} \geq 755 \text{ in}^4$
ϕM_n (composite) ≥ 394 ft·k

Assume $a=1$: $Y2 = t_{slab} - a/2 = 6.5 - 1/2 = 6''$

	PNA	ΣQ_n	ϕM_n	# studs	Bm. wt.	Stud wt.	Total wt.
$\boxed{W18\times40}$	7	147	433	13	1200	130	$\boxed{1330}$
W18×50	7	183	549	16	1500	160	1660
W18×55	7	202	608	18	1650	180	1830
W21×44	7	162	528	15	1320	150	1470
W21×50	7	184	606	16	1500	160	1660
W21×55	7	203	680	18	1650	180	1830

3/4" shear stud w/ deck
$\frac{w_r}{h_r} = \frac{6}{3} = 2 > 1.5$ $Q_n = 23^k$ (Estimate - 5 ksi concrete NOT tabulated)

$\# \text{ studs} = \left(\frac{\Sigma Q_n}{Q_n}\right) 2$

$b_{eff} =$ spacing $= 9'$
1/4 span $= 7.5'$* min

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{147}{0.85(5)(7.5\times12)} = 0.38 < 1$ ∴ assumption was conservative

$Y2 = t_{slab} - a/2 = 6.5 - 0.38/2 = 6.3$

$Y2 = 6$, $\phi M_n = 433$ | $Y2 = 6.3$, $\boxed{\phi M_n}$ | $Y2 = 6.5$, $\phi M_n = 439$

by inspection $\phi M_n > 394$ ft·k ✓

$I_{LB} = 1140$ @ $Y2 = 6$ $\gg 755$ ✓

$\boxed{\text{use W18×40 w/ (13) 3/4" shear studs min.}}$

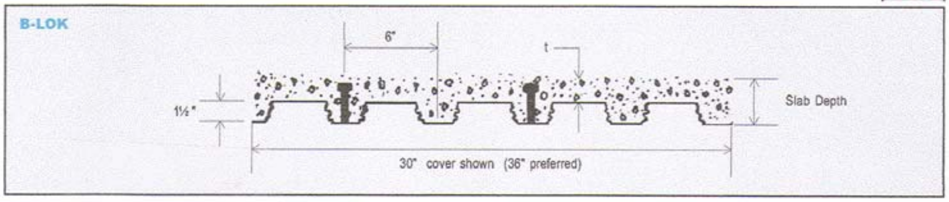
Composite Deck Check

1.5 x 6" DECK F_y = 33ksi f'_c = 3 ksi 115 pcf concrete

USD United Steel Deck, Inc.

LRFD

B-LOK



LW conc.

$M_u = 394\ \text{ft}\cdot\text{K}$

6.5" slab

19 gage →

$\phi M_n = \frac{99.39(30')}{12} = 248\ \text{ft}\cdot\text{K}$

NG!

18 gage →

$\phi M_n = \frac{111.71(30')}{12} = 279\ \text{ft}\cdot\text{K}$

NG!

16 gage →

$\phi M_n = 279\ \text{ft}\cdot\text{K}$

NG!

DECK PROPERTIES

Gage	t	w	As	I	S_p	S_n	R_b	ϕV_n	studs
22	0.0295	1.6	0.470	0.165	0.195	0.206	1320	2620	0.52
20	0.0358	1.9	0.570	0.212	0.247	0.260	1880	3170	0.63
19	0.0418	2.3	0.670	0.260	0.292	0.304	2500	3680	0.73
18	0.0474	2.6	0.760	0.308	0.337	0.349	3200	4160	0.83
16	0.0598	3.3	0.960	0.400	0.434	0.439	4750	5210	1.05

COMPOSITE PROPERTIES

	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in³	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{smf}
22 gage	4.00	38.19	21.3	0.255	29	0.91	3.1	25.66	2980	5.14	6.82	6.91	0.023
	4.50	44.78	24.8	0.297	34	1.11	4.3	31.13	3460	4.90	6.53	6.61	0.027
	4.75	48.07	26.5	0.318	37	1.21	5.1	33.93	3700	4.80	6.39	6.47	0.029
	5.00	51.37	28.3	0.339	39	1.31	5.9	36.77	3960	4.70	6.27	6.34	0.032
	5.50	57.96	32.1	0.380	44	1.52	7.9	42.53	4480	4.52	6.04	6.11	0.036
	5.75	61.26	34.0	0.401	46	1.62	8.9	45.45	4750	4.43	5.94	6.01	0.038
	6.00	64.55	36.0	0.422	49	1.73	10.1	48.39	5030	4.36	5.84	5.91	0.041
	6.50	71.15	40.1	0.464	53	1.94	12.8	54.31	5420	4.21	5.66	5.73	0.045
	6.75	74.44	42.2	0.484	56	2.04	14.3	57.30	5570	4.15	5.58	5.64	0.047
	7.00	77.74	44.3	0.505	58	2.15	15.9	60.30	5720	4.09	5.50	5.56	0.050
20 gage	4.00	45.45	21.3	0.255	29	1.09	3.3	30.45	2980	6.02	8.01	8.11	0.023
	4.50	53.44	24.8	0.297	34	1.32	4.7	36.98	3460	5.72	7.64	7.74	0.027
	4.75	57.44	26.5	0.318	37	1.44	5.5	40.33	3700	5.59	7.48	7.57	0.029
	5.00	61.44	28.3	0.339	39	1.56	6.4	43.73	3960	5.47	7.33	7.41	0.032
	5.50	69.43	32.1	0.380	44	1.81	8.5	50.64	4480	5.25	7.05	7.13	0.036
	5.75	73.43	34.0	0.401	46	1.93	9.7	54.14	4750	5.15	6.92	7.00	0.038
	6.00	77.43	36.0	0.422	49	2.06	11.0	57.67	5030	5.06	6.80	6.88	0.041
	6.50	85.42	40.1	0.464	53	2.31	13.8	64.79	5600	4.89	6.58	6.66	0.045
	6.75	89.42	42.2	0.484	56	2.44	15.4	68.38	5890	4.81	6.48	6.56	0.047
	7.00	93.42	44.3	0.505	58	2.57	17.1	71.98	6190	4.73	6.39	6.46	0.050
19gage	4.00	52.41	21.3	0.255	29	1.25	3.6	35.09	2980	6.70	8.94	9.05	0.023
	4.50	61.81	24.8	0.297	34	1.52	5.1	42.65	3460	6.36	8.52	8.62	0.027
	4.75	66.51	26.5	0.318	37	1.66	6.0	46.54	3700	6.21	8.33	8.43	0.029
	5.00	71.20	28.3	0.339	39	1.80	6.9	50.49	3960	6.08	8.16	8.25	0.032
	5.50	80.60	32.1	0.380	44	2.09	9.1	58.52	4480	5.83	7.84	7.93	0.036
	5.75	85.30	34.0	0.401	46	2.23	10.4	62.60	4750	5.71	7.69	7.78	0.038
	6.00	90.00	36.0	0.422	49	2.38	11.7	66.71	5030	5.61	7.56	7.65	0.041
	6.50	99.39	40.1	0.464	53	2.67	14.8	75.02	5600	5.41	7.31	7.39	0.045
	6.75	104.09	42.2	0.484	56	2.82	16.5	79.20	5890	5.32	7.19	7.28	0.047
	7.00	108.79	44.3	0.505	58	2.97	18.3	83.41	6190	5.24	7.08	7.17	0.050
18 gage	4.00	58.42	21.3	0.255	29	1.40	3.8	39.20	2980	7.34	9.64	9.93	0.023
	4.50	69.07	24.8	0.297	34	1.70	5.4	47.67	3460	6.96	9.19	9.44	0.027
	4.75	74.40	26.5	0.318	37	1.85	6.3	52.03	3700	6.79	8.99	9.23	0.029
	5.00	79.73	28.3	0.339	39	2.01	7.3	56.46	3960	6.64	8.80	9.03	0.032
	5.50	90.39	32.1	0.380	44	2.33	9.6	65.49	4480	6.36	8.46	8.67	0.036
	5.75	95.72	34.0	0.401	46	2.50	11.0	70.08	4750	6.24	8.30	8.51	0.038
	6.00	101.05	36.0	0.422	49	2.66	12.4	74.71	5030	6.12	8.15	8.35	0.041
	6.50	111.71	40.1	0.464	53	3.00	15.6	84.06	5600	5.90	7.88	8.07	0.045
	6.75	117.04	42.2	0.484	56	3.16	17.4	88.78	5890	5.80	7.75	7.94	0.047
	7.00	122.37	44.3	0.505	58	3.33	19.3	93.52	6190	5.71	7.63	7.82	0.050
16 gage	4.00	58.42	21.3	0.255	29	1.70	4.3	39.20	2980	8.58	10.76	11.12	0.023
	4.50	69.07	24.8	0.297	34	2.08	6.0	47.67	3460	8.12	10.27	10.61	0.027
	4.75	74.40	26.5	0.318	37	2.27	7.0	52.03	3700	7.92	10.05	10.38	0.029
	5.00	79.73	28.3	0.339	39	2.46	8.1	56.46	3960	7.73	9.84	10.17	0.032
	5.50	90.39	32.1	0.380	44	2.86	10.7	65.49	4480	7.40	9.46	9.78	0.036
	5.75	95.72	34.0	0.401	46	3.07	12.1	70.08	4750	7.25	9.28	9.60	0.038
	6.00	101.05	36.0	0.422	49	3.27	13.7	74.71	5030	7.11	9.12	9.43	0.041
	6.50	111.71	40.1	0.464	53	3.69	17.2	84.06	5600	6.85	8.81	9.11	0.045
	6.75	117.04	42.2	0.484	56	3.90	19.2	88.78	5890	6.74	8.67	8.96	0.047
	7.00	122.37	44.3	0.505	58	4.11	21.3	93.52	6190	6.62	8.54	8.82	0.050

B - LOK

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Roof Bar Joist Check

STANDARD LOAD TABLE/DEEP LONGSPAN STEEL JOISTS, DLH SERIES
Based on a Maximum Allowable Tensile Stress of 30 ksi

Joist Designation	Approx. Wt. in Lbs. per Linear Ft. (Joists Only)	Depth in Inches	SAFE LOAD* in Lbs. Between		CLEAR SPAN IN FEET															
			70-99	100-104	105	106	107	108	109	110	111	112	113	114	115	116	117	118	119	120
60DLH12	29	60	31100	31100	295 168	289 163	284 158	279 154	274 150	270 146	265 142	261 138	256 134	252 131	248 128	244 124	240 121	236 118	232 115	228 113
60DLH13	35	60	37800	37800	358 203	351 197	345 191	339 187	333 181	327 176	322 171	316 167	311 163	306 158	301 154	296 151	291 147	286 143	282 139	277 135
60DLH14	40	60	42000	42000	398 216	391 210	383 205	376 199	370 193	363 189	356 183	350 178	344 173	338 170	332 165	327 161	321 156	316 152	310 149	305 145
60DLH15	43	60	49300	49300	467 255	458 248	450 242	442 235	434 228	427 223	419 216	412 210	405 205	398 200	392 194	385 190	379 185	373 180	367 175	361 171
60DLH16	46	60	54200	54200	513 285	504 277	494 269	485 262	476 255	468 247	460 241	451 235	444 228	436 223	428 217	421 211	414 206	407 201	400 196	393 190
60DLH17	52	60	62300	62300	590 324	579 315	569 306	558 298	548 290	538 283	529 275	519 267	510 261	501 254	493 247	484 241	476 235	468 228	460 223	453 217
60DLH18	59	60	71900	71900	681 366	668 357	656 346	644 337	632 327	621 319	610 310	599 303	589 294	578 286	568 279	559 272	549 266	540 259	531 252	522 246
			75-99	100-112	113	114	115	116	117	118	119	120	121	122	123	124	125	126	127	128
64DLH12	31	64	30000	30000	264 153	259 150	255 146	251 142	247 138	243 135	239 132	235 129	231 125	228 122	224 119	221 116	218 114	214 111	211 109	208 106
64DLH13	34	64	36400	36400	321 186	315 181	310 176	305 171	300 168	295 163	291 159	286 155	281 152	277 148	273 144	269 141	264 137	260 134	257 131	253 128
64DLH14	40	64	41700	41700	367 199	360 193	354 189	349 184	343 179	337 174	332 171	326 166	321 162	316 158	311 154	306 151	301 147	296 143	292 140	287 136
64DLH15	43	64	47800	47800	421 234	414 228	407 223	400 217	394 211	387 206	381 201	375 196	369 191	363 187	358 182	352 177	347 173	341 170	336 165	331 161
64DLH16	46	64	53800	53800	474 262	466 254	458 248	450 242	443 235	435 229	428 224	421 218	414 213	407 208	401 203	394 198	388 193	382 189	376 184	370 180
64DLH17	52	64	62000	62000	546 298	536 290	527 283	518 275	509 268	501 262	492 255	484 248	476 243	468 237	461 231	454 226	446 220	439 215	432 210	426 205
64DLH18	59	64	71600	71600	630 337	619 328	608 320	598 311	587 304	578 296	568 288	559 282	549 274	540 267	532 261	523 255	515 249	507 243	499 237	491 232
			80-99	100-120	121	122	123	124	125	126	127	128	129	130	131	132	133	134	135	136
68DLH13	37	68	35000	35000	288 171	284 168	279 164	275 159	271 155	267 152	263 149	259 145	255 142	252 138	248 135	244 133	241 130	237 127	234 124	231 121
68DLH14	40	68	40300	40300	332 184	327 179	322 175	317 171	312 167	308 163	303 159	299 155	294 152	290 148	286 145	281 141	277 138	273 135	269 133	266 130
68DLH15	40	68	45200	45200	372 206	365 201	360 196	354 191	348 187	343 182	337 178	332 174	327 170	322 166	317 162	312 158	308 155	303 152	299 148	294 145
68DLH16	49	68	53600	53600	441 242	433 236	427 230	420 225	413 219	407 214	400 209	394 204	388 199	382 195	376 190	371 186	365 182	360 178	354 174	349 171
68DLH17	55	68	60400	60400	497 275	489 268	481 262	474 256	467 249	460 244	453 238	446 232	439 228	433 222	427 217	420 212	414 208	408 203	403 198	397 194
68DLH18	61	68	69900	69900	575 311	566 304	557 297	549 289	540 283	532 276	524 269	516 263	508 257	501 251	493 246	486 240	479 234	472 230	465 225	459 219
68DLH19	67	68	80500	80500	662 353	651 344	641 336	631 328	621 320	611 313	601 305	592 298	583 291	574 285	565 278	557 272	548 266	540 260	532 254	525 248
			84-99	100-128	129	130	131	132	133	134	135	136	137	138	139	140	141	142	143	144
72DLH14	41	72	39200	39200	303 171	298 167	294 163	290 159	285 155	281 152	277 149	274 146	270 143	266 139	262 136	259 133	255 131	252 128	248 128	245 129
72DLH15	44	72	44900	44900	347 191	342 187	336 183	331 178	326 174	322 171	317 167	312 163	308 160	303 156	299 152	295 150	291 147	286 143	282 140	279 137
72DLH16	50	72	51900	51900	401 225	395 219	390 214	384 209	378 205	373 200	368 196	363 191	358 188	353 183	348 179	343 175	338 171	334 169	329 165	325 161
72DLH17	56	72	58400	58400	451 256	445 250	438 245	432 239	426 233	420 228	414 224	408 218	402 213	397 209	391 205	386 200	381 196	376 191	371 188	366 184
72DLH18	59	72	68400	68400	528 289	520 283	512 276	505 270	497 265	490 258	483 252	479 247	470 242	463 236	457 231	450 227	444 222	438 217	432 212	426 209
72DLH19	70	72	80200	80200	619 328	609 321	600 313	591 306	582 300	573 293	565 286	557 280	549 274	541 268	533 263	526 257	518 251	511 247	504 241	497 236

*The safe uniform load for the clear spans shown in the Safe Load Column is equal to (Safe Load)/(Clear span + 0.67). (The added 0.67 feet (8 inches) is required to obtain the proper length on which the Load Tables were developed).

In no case shall the safe uniform load, for clear spans less than the minimum clear span shown in the Safe Load Column, exceed the uniform load calculated for the minimum clear span listed in the Safe Load Column.

To solve for *live* loads for clear spans shown in the Safe Load Column (or lesser clear spans), multiply the live load of the shortest clear span shown in the Load Table by (the shortest clear span shown in the Load Table + 0.67 feet)^2 and divide by (the actual clear span + 0.67 feet)^2. The live load shall *not* exceed the safe uniform load.

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