Technical Report #3

Virginia Advanced Shipbuilding & Carrier Integration Center
Newport News, VA

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Structural Option

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Executive Summary

Technical Report III is an analysis of the existing lateral resistance within the building. The Virginia Advanced Shipbuilding & Carrier Integration Center makes use of two tall, "K" braced frames in order to resist wind and seismic loads. The frames are located along column lines 3 and 9 and consist of 3 bays. The columns and beams are designed as wide flange steel members. The cross bracing makes use of square HSS members.

In order to analyze the building's lateral resisting elements, the lateral forces were first determined. Wind pressures and forces were determined. Next, seismic loads were calculated. Analysis of load combinations found that the wind force controlled as no gravity loads were taken into account for this report. The load combination 1.6W was used.

Many things were taken into account in terms of the design of the "K" braced frame. First, due to the shape of the building, the frame acted in 3-dimensions as opposed to 2-dimensions. The outer bays of the frame are perpendicular to the façade of the building and therefore perpendicular to the wind load. The middle bay, however, is not perpendicular. Three bays of the frame make use of "X" bracing as well for added reinforcement.

Two methods of analysis were used to determine the effectiveness of the frame. First, the frames were modeled in ETABS. Member properties and loads using the 1.6W load case were input into the program to determine the deflection. Story-by-story deflection outputs were then compared to the standard h/400 requirement for building deflection. After comparison, the frames were adequate to handle the wind force applied to the building.

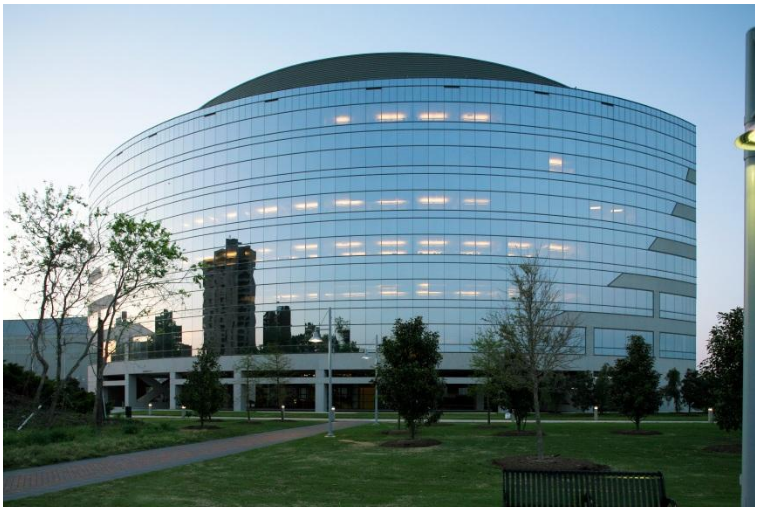
The second method of analysis made use of the cantilever method to compare moments amongst the members in the Penthouse level. The cantilever method was used to determine the forces and moments in these members, which were then compared to AISC values using LRFD criteria. These members were found to be adequate in resisting the lateral wind force.

After two methods of analysis, one using computer technology, one using hand calculations, it was determined that the "K" braced frame designed for the building is adequate for handling the lateral loads that it is designed to resist. The "K" braced frame was most likely used due to its simplicity as well as its aesthetic appeal. The frame fits in seamlessly with the rest of the VASCIC which consists of large open areas as well as steel beams and columns.

Introduction

The Virginia Advanced Shipbuilding and Carrier Integration Center was designed by Clark Nexsen. The project consists of two main buildings: the office building and the lab wing complete with lab parking and a parking deck. The office building is a typical composite steel frame design. The steel frame grid consists of wide flange beams and columns that range from W12x14 to W18x40. The Lab wing consists of concrete slab with concrete columns and precast concrete walls.

The office building is elevated on "stilts" of concrete made of concrete piles surrounding wide flange steel columns.



Source: Clark Nexsen

The first floor consists of a 5" reinforced concrete slab in the main office area, an 8" reinforced concrete slab at the front of the building and a 6" reinforced concrete slab in the stairwells. The rest of the floors consist of a grid of wide flange steel columns and beams that is shaped into the unique curved design of the Virginia Advanced Shipbuilding and Carrier Integration Center. The composite steel deck and slab is 4.5" in total thickness and consists of lightweight concrete placed on a 2" deep, .038" thick galvanized steel deck.

The lab wing consists of 24"x24" precast concrete columns, 8" precast lightweight concrete walls, and 4" reinforced concrete slabs. The roof of the lab wing consists of gable trusses with steel deck.

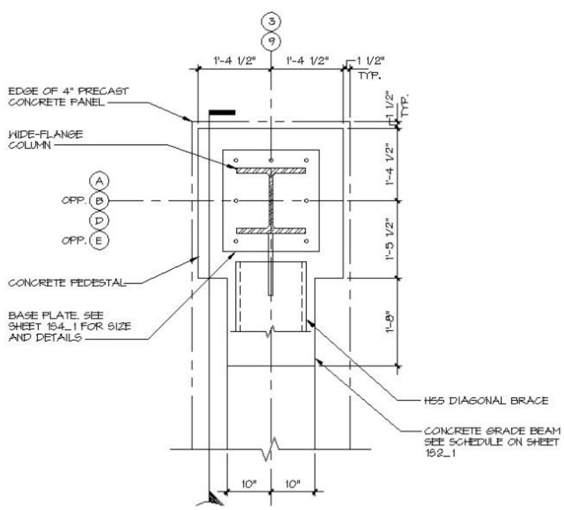
Structural Systems Overview

1. Foundation

A. Office Building

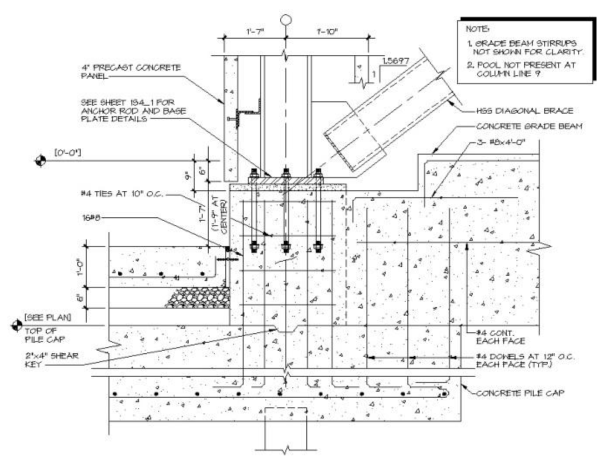
The foundation of the office building consists of a wide-flange steel column on a concrete pedestal. These concrete pedestal/steel column arrangements are placed around the perimeter of the office building in a shape that resembles a football. The soil condition on the site consists of unstable soil do to the waterfront location of the building. This shape is repeated for interior columns as well. *Figure 1* shows the plan view of the concrete pedestal/steel column arrangement and *Figure 2* shows the section view.

FIGURE 1 –CONC. PEDESTAL PLAN



Source: Clark-Nexsen

FIGURE 2 – CONC.PEDESTAL SECTION



Source: Clark-Nexsen

The concrete used in these arrangements is 3000 psi concrete. It is reinforced by #4 ties at 10"O.C, a 2"x4" shear key, and 16 #8 steel rebar. These concrete piles support the wide flange columns that are placed on them and connected with steel plates and anchor rods.

Two grade beams are used in the foundation of the office building. These grade beams are used to resist lateral column base movement as well as distribute the weight of the building over the soil. These grade beams are important due to the unstable soil condition on the site. Lateral column base movement is important in this project as it is on the coast of the James River. A bulkhead of steel sheet pile had to be constructed to resist the water pressure of the river as well as to provide slope stability and increase bearing capacity for the building foundation. They also serve to increase the bearing capacity for the building foundation. The grade beams are used to further this insurance that the building will not be affected by the river. *Table 1* shows the width, depth and reinforcing of these grade bars.

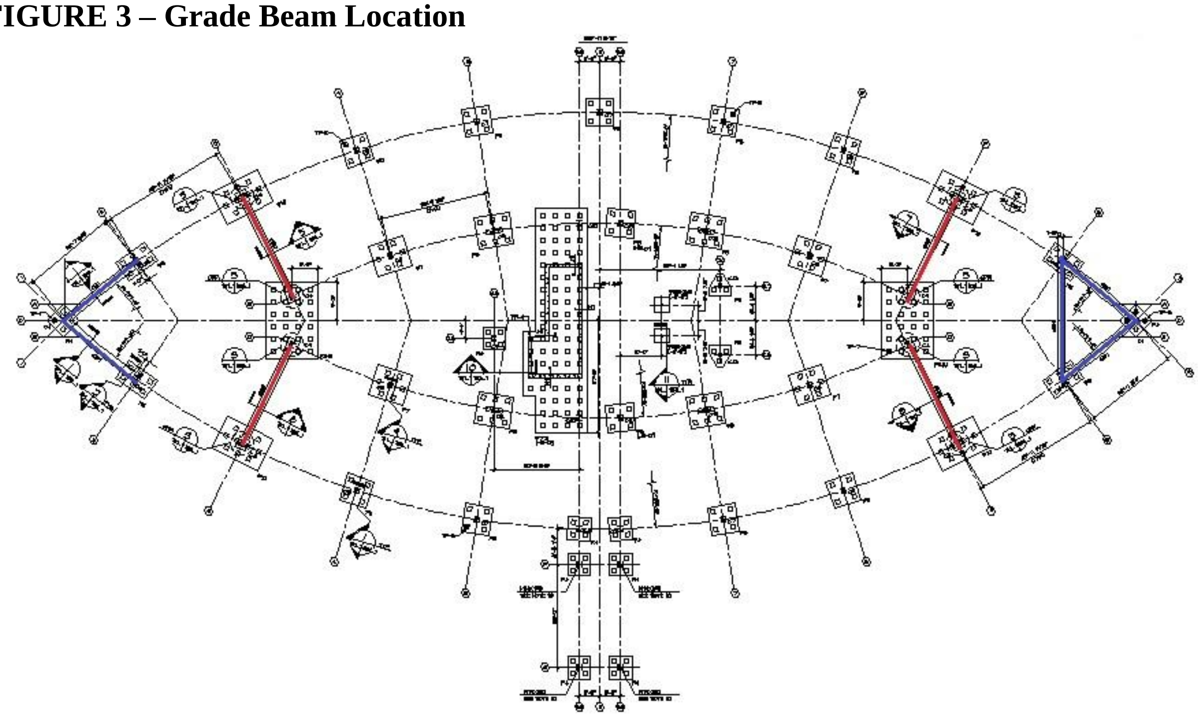
TABLE 1 – Grade Beam Schedule

GRADE BEAM SCHEDULE						
MARK	WIDTH	DEPTH	TOP BARS	BOTTOM BARS	STIRRUPS	
					SIZE	SPACING
GB1	22"	46"	4 - #8	4 - #8	#4	12" O.C.
GB2	20"	50"	4 - #7	4 - #7	#4	12" O.C.

Source: Clark-Nexsen

Figure 3 shows the locations of the grade beams. GB1 is indicated in blue and GB 3 is indicated in red.

FIGURE 3 – Grade Beam Location

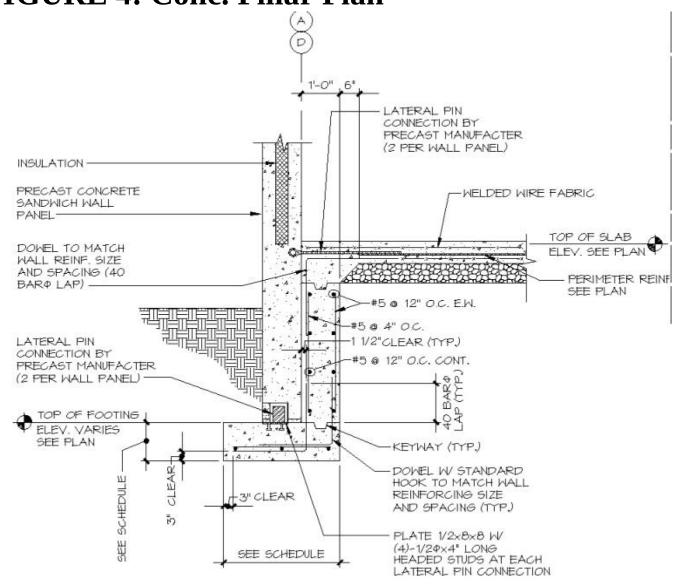


Source: Clark-Nexsen

B. Lab Wing

The lab wing foundation consists of concrete pillars attached to concrete footing. The pillars, which are continuous in length, contain #5 rebar at 12" O.C. and are attached to the footing by a lateral pin. *Figure 4* shows the plan view of the concrete pillars.

FIGURE 4: Conc. Pillar Plan



Source: Clark-Nexsen

The concrete used in the pillars for the lab wing are 3000psi concrete. They support precast concrete walls. The footings that support these walls are continuous in length. They range from 2'-0" wide by 1'-0" thick to 7'-0" by 1'-0". *Table 2* shows the footing schedule. The "A" bars indicate reinforcing in concrete deposited against the ground. The "B" bars indicate reinforcing in the concrete exposed to earth or weather.

TABLE 2 – Footing Schedule

FOOTING SCHEDULE						
MARK	DIMENSIONS			REINFORCEMENT		NOTES
	W	L	T	'A' BARS	'B' BARS	
CF2.0	2'-0"	CONT.	1'-0"	(2) #5's CONT.	#5's @ 4'-0" O/C	1
CF3.0	3'-0"	CONT.	1'-0"	(3) #5's CONT.	#5's @ 4'-0" O/C	1
CF4.0	4'-0"	CONT.	1'-0"	(4) #5's CONT.	#5's @ 6' O/C	1, 2
CF7.0	7'-0"	CONT.	1'-0"	(6) #5's CONT.	#5's @ 6' O/C	1, 2
F4.0x4.0	4'-0"	4'-0"	1'-0"	(6) #4's	(6) #4's	1
F8.5x8.5	8'-6"	8'-6"	1'-8"	(7) #7's	(7) #7's	1

Source: Clark Nexsen

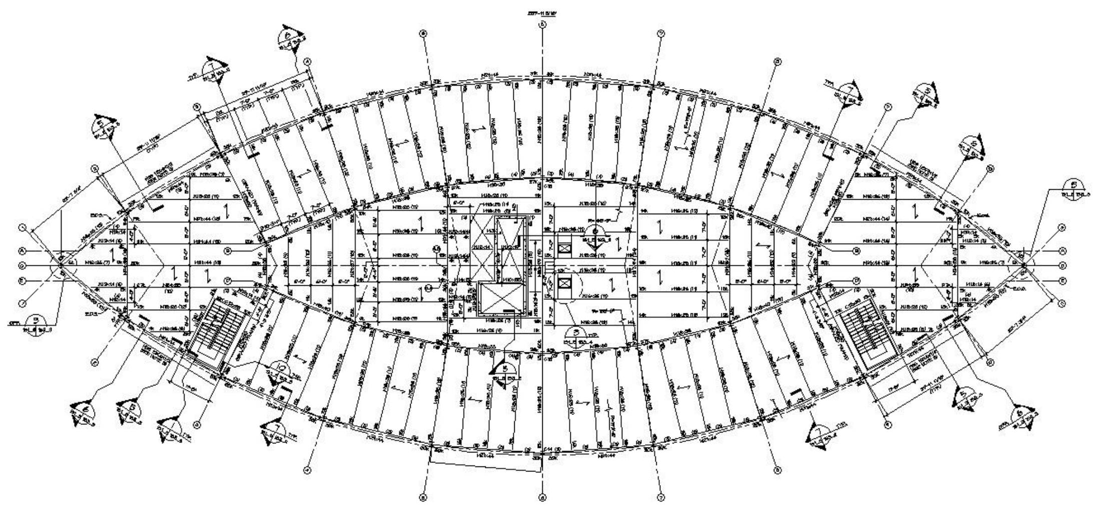
The lab wing also contains a 23"wide by 30" deep grade beam, GB1 along vertical grid line 1.5.

2. Floor System

A. Office Building

The floor system of the office building is consistent from the second floor to the seventh floor. These floors contain 4.5" total thickness composite steel deck and slab. This slab consists of lightweight concrete placed on a 2" deep, .038" thick galvanized steel deck. The steel deck conforms to ASTM A653-94 specifications and has a minimum yield strength of 33ksi. The beams are wide flange steel beams arranged in various grids that form together to fit the curved shape of the building. *Figure 5* shows the floor plan from floor 2 to floor 7.

FIGURE 5 – Floor Plan Floor 2-7

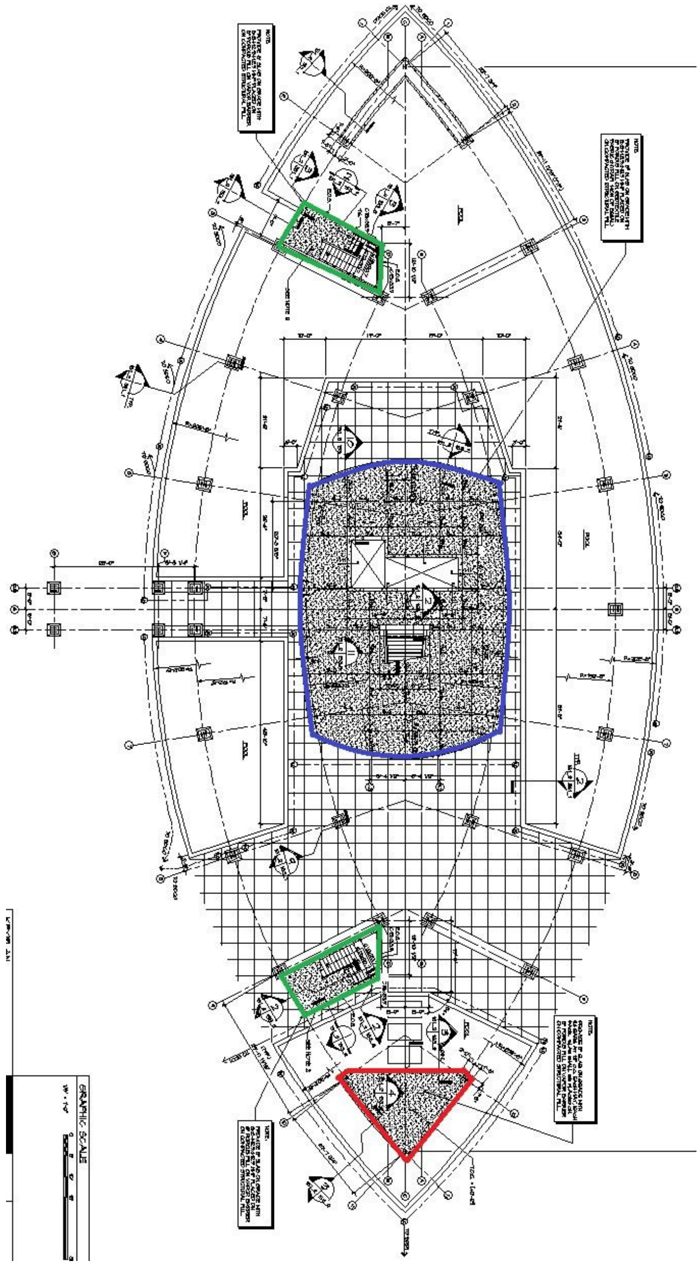


Source: Clark-Nexsen

The first floor of the office building contains three separate load-bearing reinforced concrete slabs. The first slab is at the center of the building. It consists of a 5" slab on grade with 6x6-W2.9xW2.9 WWF placed on 6" porous fill.

There is also a triangular slab in the back of the building. This slab is 8" slab on grade with #4 bars at 12" O.C. Finally, there is a slab on the floor of the stairwells. These slabs are a 6" slab on grade with 6x6-W2.9xW2.9 WWF. *Figure 6* shows the first floor plan. The 5" slab is outlined in blue, the 8" inch slab is outlined in red, and 6" slab is outlined in green.

FIGURE 6 – First Floor Plan



Source: Clark-Nexsen

B. Lab Wing

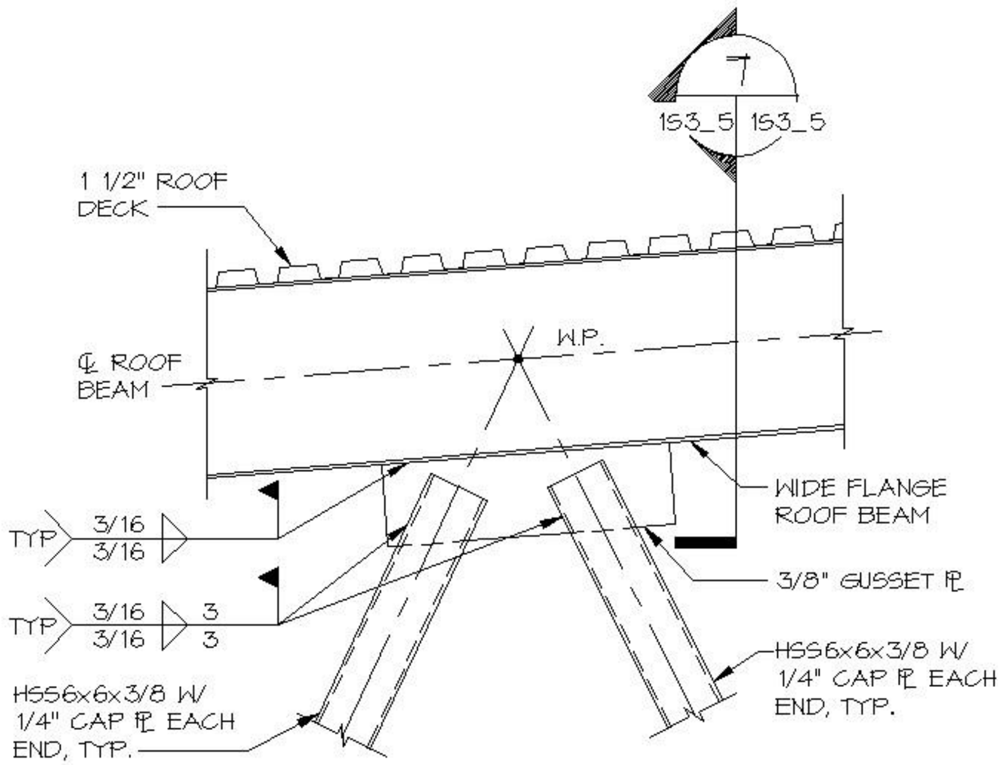
The lab wing consists of a 4" reinforced concrete slab. The slab is reinforced with 6x6 W2.0xW2.0 WWF. This concrete used in the slab is 4000psi.

3. Roof System

A. Office Building

The roof structure of the office building is 1 ½" corrugated composite steel deck. The deck sits on wide flange steel roof beams. *Figure 7* shows the section view of the roof.

FIGURE 7 – Roof Section

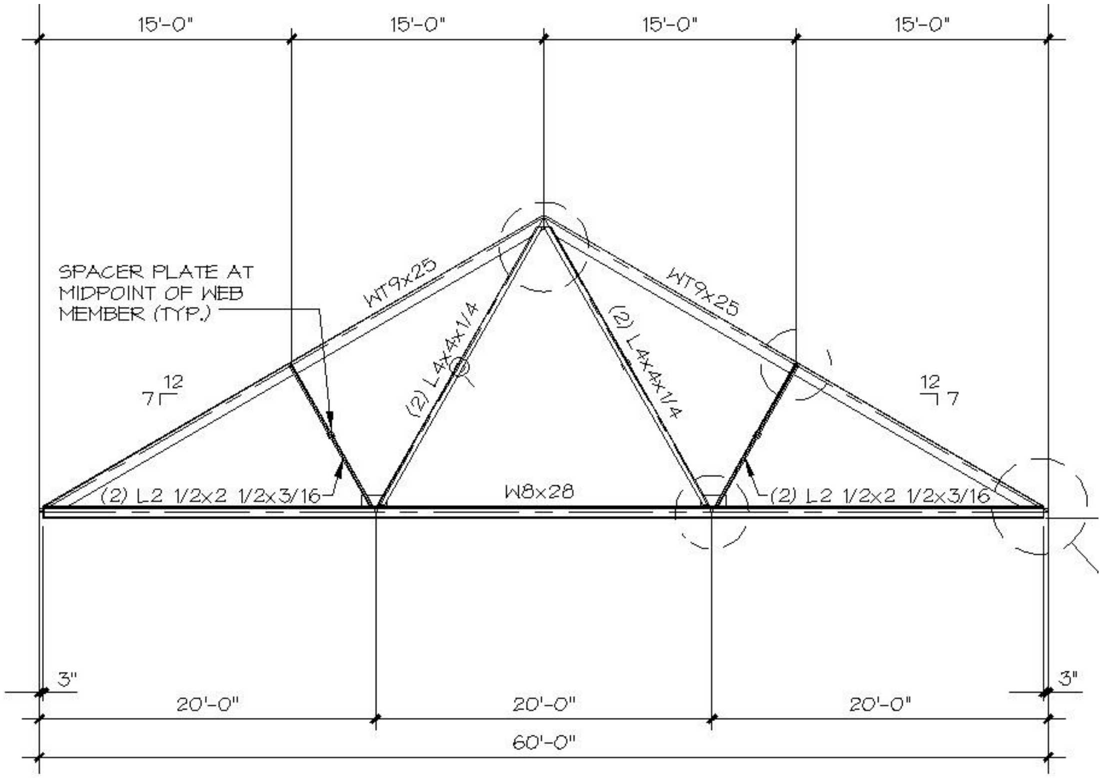


Source: Clark-Nexsen

B. Lab Wing

The roof of the lab wing involves gable trusses, spanning between concrete columns. The gable trusses are constructed using WT9x25, L2 1/2x2 1/2x3/16, and W8x28 steel members. On the gable trusses is a 20GA 1 ½" deep wide rib roof deck. *Figure 8* shows a section view of the gable trusses.

FIGURE 8 – Gable Trusses



Source: Clark-Nexsen

There is also a special truss located along column line 2.5. For these trusses, bottom chord members are W8x31 and the top chord members are WT9x27.5.

4. Columns

A. Office Building

The office building contains steel wide flange columns. 42 columns are arranged to fit the curved shape of the building. The columns used are W8, W10, W12, and W14 steel members. These wide flange columns are encased by concrete piles on the foundation to provide extra structural stability. This is important on the foundation because, as previously stated, the building is raised off the ground to provide protection against flooding. The number of piles used for each column varies from 2 to 9.

These columns on floors 1 to 7 direct gravity loads to the foundation where the columns and concrete piles direct the loads to the earth's foundation.

B. Lab Wing

The lab wing uses concrete columns. These columns vary in size, with the most common size being 24"x24" precast concrete. The columns are accompanied by concrete piles at the foundation in order to provide extra strength at the foundation of the building.

5. Lateral System

A. Office Building

The lateral system of the office building consists of a "K" braced frame. This braced frame occurs at column lines 3 and 9. The frame consists of wide-flange steel members as well as HSS steel members. The wide-flange members are used as columns. The HSS members are used as diagonal bracing. The wide-flange members are W14 and range from W14x82 at the top, W14x90 in the middle, and W14x159 at the bottom. The HSS members range from HSS 8x8 at the top to HSS 10x10 in the middle, and finally HSS 12x12 at the bottom.

"X" bracing is used in three bays of this structure: the outer bays on the bottom level as well as the middle bay in the penthouse level. "X" bracing is used on these floor as added bracing because of the loads on the floors. As discussed later in the "Wind Load" section, the penthouse sees the highest load in psf from wind. The penthouse also lacks the outer bays to help deflect the load like the floors below it have. The bays on the bottom level have the added weight of the floors above to take into consideration. The "X" bracing allows one diagonal brace to be in tension and one to be in compression. *Figure 9* shows the location of the "K" braced frame and *Figure 10* shows the "K" braced frame in section.

FIGURE 9 – Source: Clark Nexsen

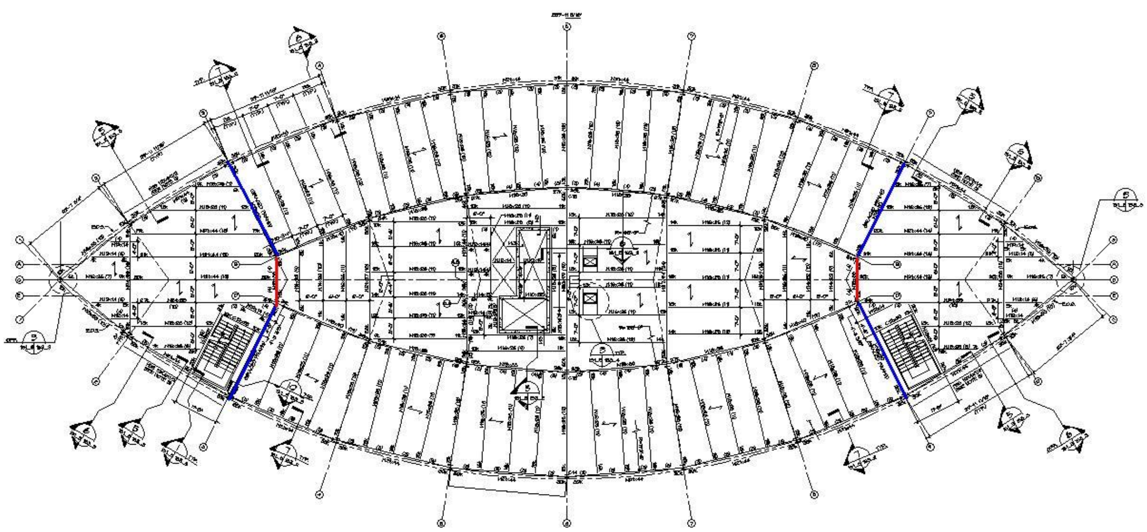
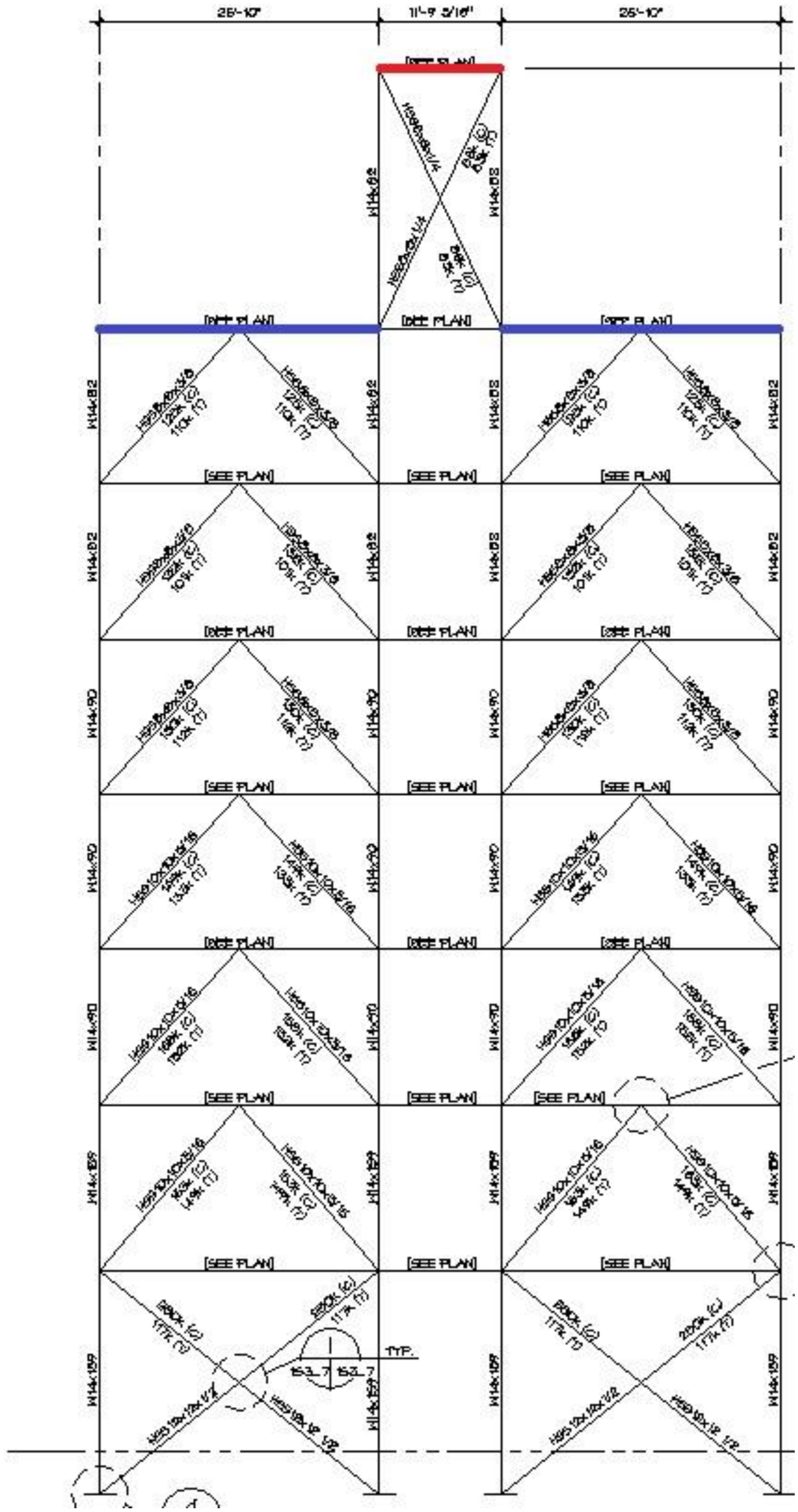
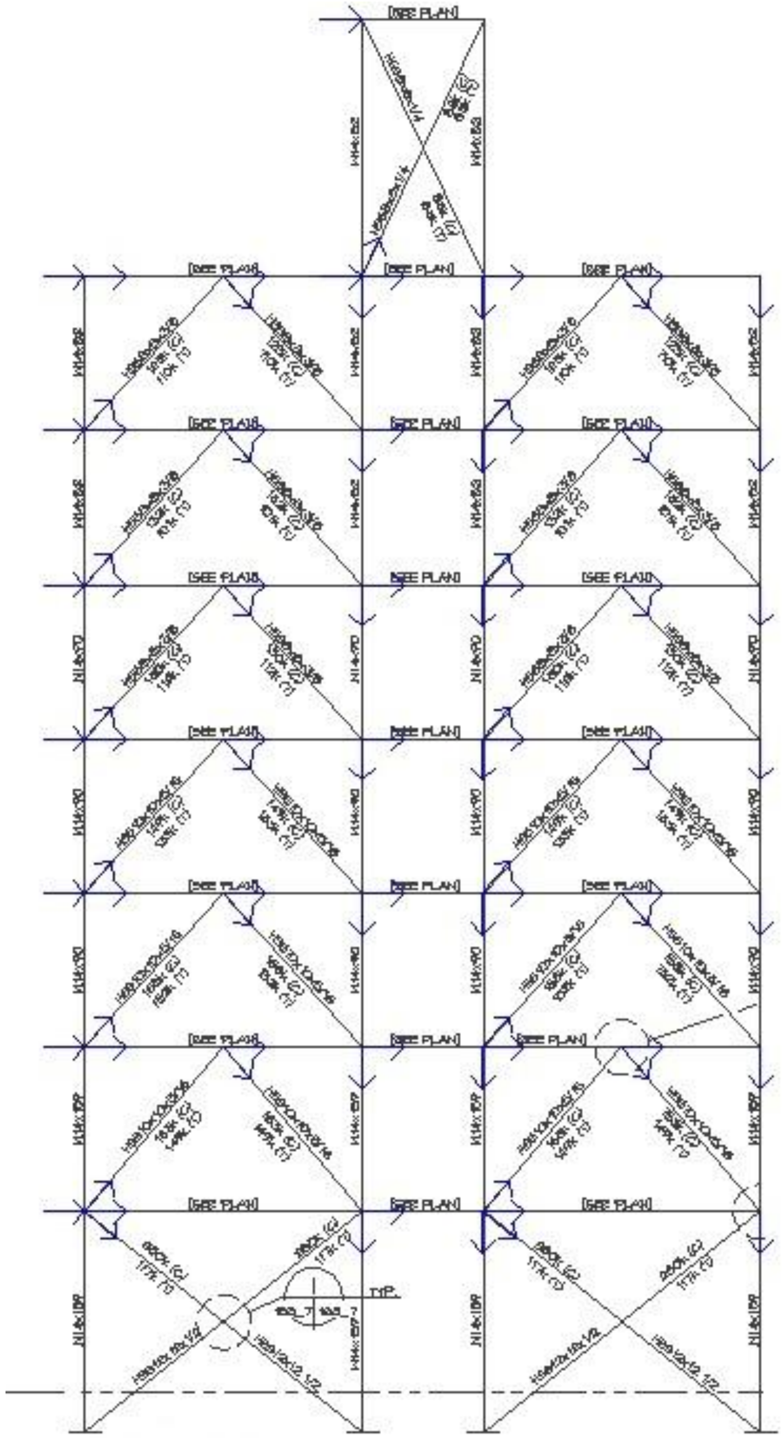


FIGURE 10 – Source: Clark Nexsen



The unique design of the building caters to the shape of the frame. The outer bays are perpendicular to the load and transfer the load to the middle bays as well as down through the cross bracing. *Figure 11* shows the load path of the frame.

FIGURE 11 – Source: Clark Nexsen



B. Lab Wing

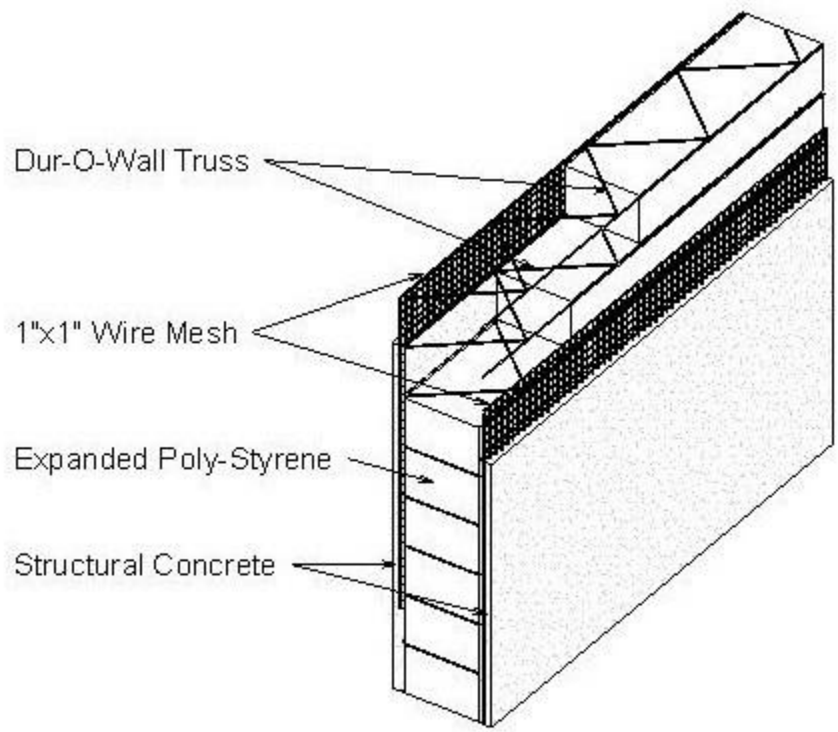
The lateral support for the lab wing is provided by shear walls. 8" precast lightweight concrete walls are used as sheer walls throughout the lab wing of the building. These walls combine with the concrete slabs to provide lateral support for the building.

6. Structural Details

A. Sandwich Wall

The lab wing makes use of concrete sandwich walls. Sandwich walls are resistant to many important forces of nature including, earthquakes, hurricanes, heat, cold, and flooding. Flooding is the most important natural force in the situation of the Virginia Advanced Shipbuilding & Carrier Integration Center. As stated earlier, the office building uses stilts with thick concrete piles to avoid problems caused by the flooding of the James River. The lab wall instead makes use of the sandwich wall in order to defend against flooding. *Figure 10* shows the sandwich wall in section.

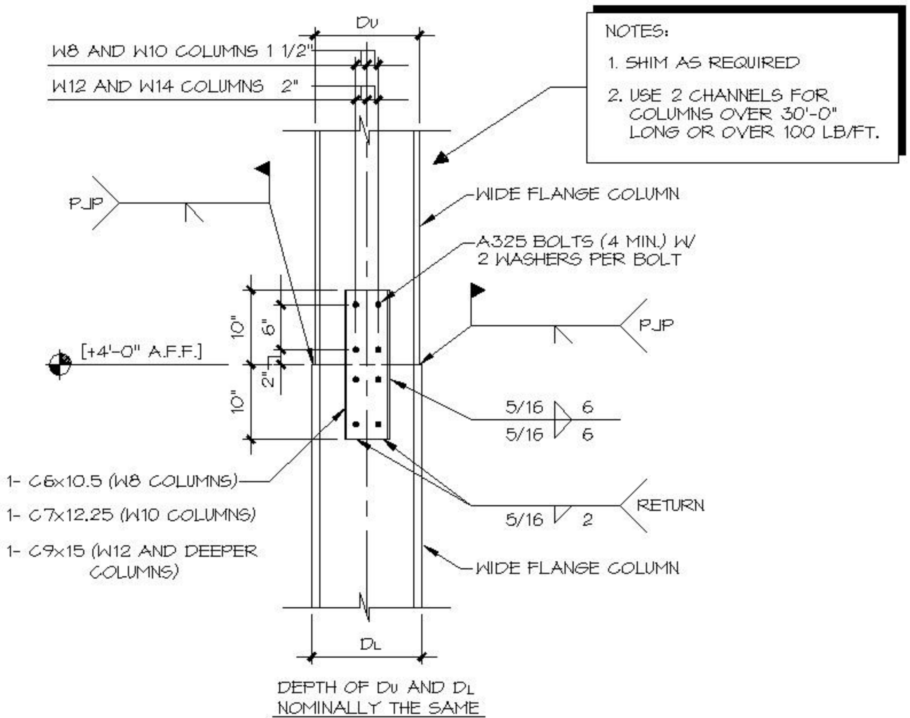
FIGURE 12 – Source: http://www.cswall.com/CSW/Walls/index.htm



B. Column Splice Connections

The height of the office building makes it necessary for column splice connections to be used. *Figure 11* shows the typical column splice details.

FIGURE 13 – Source: Clark Nexsen



It is important to note the variance of the connections from the W8 to the W14 columns. A325 bolts are used. Also, 2 channels are used for columns over 30’-0” long or over 100lb/ft.

7. Conclusions on Structural System

The first thing that was noticed when looking at the structural drawings is the vast difference between the office building and the lab wing. The office building makes use of steel columns and beams as well as diagonal steel bracing. The lab wing, however, makes use of concrete slabs and concrete columns as well as shear walls and sandwich walls.

Flooding is an important natural force that had to be accounted for in the structural design of the building. The building had to be designed to withstand flood loads. The use of large concrete areas on the ground floor are designed to resist these loads. The ground floor does not contain offices or any rooms. Instead, the offices are located above flood levels in the floors above the ground floor. This allowed the ground floor to keep an open feel to it even with the large areas of concrete. The office building makes use of stilts and thick concrete piles to remain above flood level. The Lab wing, however, makes use of sandwich walls.

The use of steel in the office building is most likely due to the architect wanting to keep the office building more open and spacious and not have to worry about large, cramping concrete columns. The steel columns and beams are complimented by the curtain wall that engulfs the building. This provides a light, spacious, and well-lit office building.

The lab wing, on the other hand, is designed as a seemingly heavier, less spacious building. Most business will be taking place in the office building and it is clear that the designer wanted the office building to feel more welcoming. The parking deck makes use of concrete because it is most likely cheaper to design a parking deck out of concrete. Also, while the laboratories will be operated during the day, they make more use of artificial lighting and rely less on natural light.

Design Codes and Standards

The design of the Virginia Advanced Shipbuilding & Carrier Integration Center followed the following codes:

The BOCA National Building Code – 1996
AISC Manual of Steel Construction, Load and Resistance Factor Design, Second Edition
ACI 318-95 Building Code Requirements for Structural Concrete

This report will make use of the following codes and standards

ASCE/SEI 7-05 – Minimum Design Loads for Buildings and Other Structures
This text will be referred to as *ASCE 7-05* from now within the report. ASCE 7-05 was used to determine appropriate Live Loads, Wind Loads, Snow Loads, Seismic Loads, as well as Load Factoring and Live Load Reduction.

AISC Steel Construction Manual Thirteenth Edition
This text well be referred to as AISC from now on within the report. AISC was used to determine loads as well as sizes of steel beams and columns. LRFD was used in the calculation and determination of these loads and steel member sizes.

ACI 318-08 Building Code Requirements for Structural Concrete
This text will be referred to as ACI 318-08 from now within the report. ACE 318-08 was used to determine loads as well as sizes of concrete structural aspects including slabs and load bearing precast concrete walls as well as concrete columns.

Material Properties

Reinforced Concrete

TYPE	F'c	Aggregate
Slab on Grade	4000psi	Normal Weight
Walls	4000psi	Light Weight
Grade Beams	3000psi	Normal Weight
Pile Caps	3000psi	Normal Weight
Composite Deck Fill	3000psi	Lightweight
All Other Concrete	3000psi	Normal Weight

Structural Steel

Shape	Fy (KSI)
Wide Flanges	50
Rectangular HSS members	46
WT members	50
Channels	50
Connectors – Angles	36
Connectors – Angles	36

Lateral Loads

1. Wind Loads

BOCA 1996 was used as the resource for wind calculations for the existing design. My analysis, however, will make use of ASCE 7-05 chapter 6. Section 6.5 (Method 2 – Analytical Procedure), specifically section 6.5.3 (Design procedure), was used as a guide for the calculation of wind load.

Basic Wind Data

- Location: Newport News, VA
- Exposure: D (Building at Shoreline)
- Occupancy: III

Design Procedure

- Basic Wind Speed (V) = 90 mph from Fig. 6-1
- Importance factor (I) = 1.0 from fig 6-1
- Exposure Category = D from Section 6.5.6.3
- Directionality Factor (K_d) = .85 from table 6-4
- Topographic Factor (K_{zt}) = 1.0 from section 6.5.7
- Gust Effect Factor (G) E/W = 1.003 from section 6.5.8 (see appendices for calculation)
- Internal Pressure Coefficient (GC_{pi}) = $\pm$ .18 from figure 6-5
- Velocity Pressure (q_z) = 25.204 @ 6th floor from section 6.5.10 (see appendices for calculation)
- Velocity Pressure (q_z) = 26.421 @ mean roof height from section 6.5.10 (see appendices for calculation)

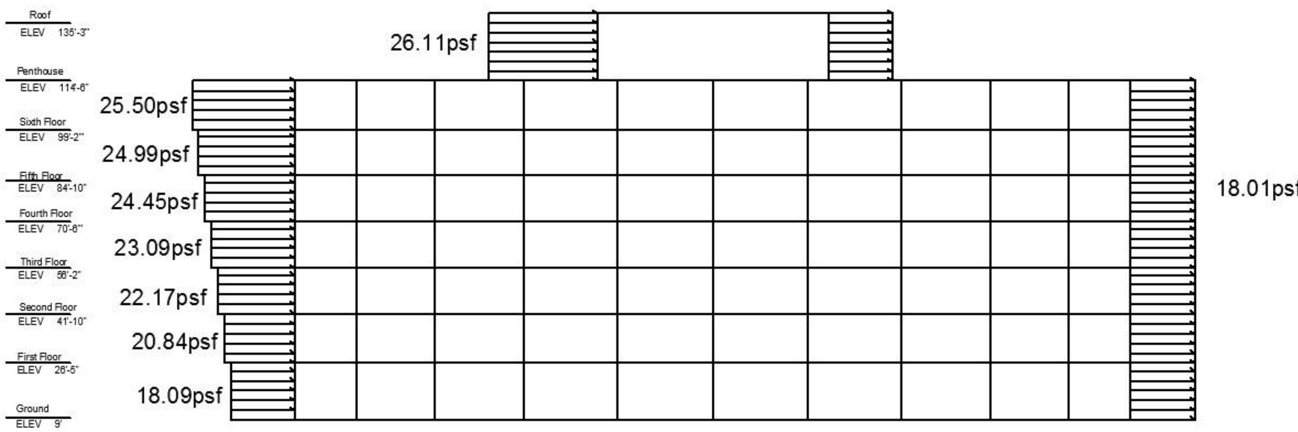
TABLE 3: Wind Loads

	Height	Kz	qz	P	Height Difference	F
Ground	9	0.943	16.62	18.09	0	0.00
First	26.5	1.137	20.05	20.84	17.5	91.60
Second	41.83	1.231	21.70	22.17	15.33	87.19
Third	56.16	1.296	22.85	23.09	14.33	87.43
Fourth	70.5	1.348	23.77	23.83	14.34	89.97
Fifth	84.83	1.393	24.54	24.45	14.33	92.11
Sixth	99.16	1.431	25.22	24.99	14.33	97.42
Penthouse	114.5	1.467	25.86	25.50	15.34	121.17
Roof	135.21	1.510	26.62	26.11	20.71	70.30

TABLE 4: Wind Forces

	Force	Shear	Moment
Ground	0	179	0
First	92	175	2427
Second	87	177	3647
Third	87	182	4910
Fourth	90	190	6343
Fifth	92	219	7813
Sixth	97	191	9660
Penthouse	121	70	13874
Roof	70	0	9506

FIGURE 12: E/W Wind Load Diagram



2. Seismic Load

My seismic analysis for the Virginia Advanced Shipbuilding & Carrier Integration Center was done using ASCE 7-05. Newport News, Virginia is not a seismic zone, however it is important to analyze the seismic loads to determine their impact on the structure of the building. The building cost enough and is important enough to the community that, if a freak earthquake were to occur, it is necessary to make sure the building would remain intact.

ASCE 7-05, sections 11, 12, and 22 were of use during the seismic analysis. Calculations for the following values can be found in the appendices. *Table 5* shows the seismic forces for each floor.

Basic Seismic Information

- Location: Newport News, VA
- Site Class: D
- Importance Factor: 1

Design Procedure

- S_s = .123 from USGS website
- S_1 = .049 from USGS website
- F_a = 1.6 from table 11.4-4
- F_v = 2.4 from table 11.4-2
- S_{MS} = .1968
- S_{M1} = .1176
- S_{DS} = .1312
- S_{D1} = .0784
- C_t = .028 from table 12.8-2
- x = .8 from table 12.8-2
- T_a = 1.419
- T_s = .598
- R = 8 from table 12.2-1
- C_u = 1.7 from table 12.8-1
- Cs = .0069
- V = 90.37
- K = 1.46 from section 12.8.3

TABLE 5: SEISMIC FORCES

Floor	Wx (k)	Hx (ft)	Hx^k	WxHx^k	ΣWxHx^k	Cvx	Fx
Ground	1019	9	25	25186	25186	0.005	0.4
First	606	26.5	120	72486	97672	0.742	67.1
Second	1809	41.83	233	421601	519273	0.812	73.4
Third	1809	56.16	358	648176	1167449	0.555	50.2
Fourth	1809	70.5	499	903412	2070861	0.436	39.4
Fifth	1809	84.83	654	1183619	3254479	0.364	32.9
Sixth	1809	99.16	822	1486555	4741034	0.314	28.3
Penthouse	598	113.5	1001	597939	5338973	0.112	10.1
			Total	5338973			

LOAD CASES

BOCA 1996 was used to design the Virginia Advanced Shipbuilding & Carrier Integration center. My analysis of the building will make use of ASCE 7-05. Section 2.3.2 has seven load cases that were considered for the analysis.

1. 1.4 (D +F)
2. 1.2(D + F + T) + 1.6(L + H) + .5(L_r or S or R)
3. 1.2D + 1.6(L_r or S or R) + (L or .8W)
4. 1.2D + 1.6W + L + .5(L_r or S or R)
5. 1.2D + 1.0E + L + .2S
6. .9D + 1.6W + 1.6H
7. .9D + 1.0E + 1.6H

Where: D = Dead Load
F = Load due to fluids with well-defined pressures and maximum lengths
T = Self-straining force
L = Live Load
H = Load due to lateral earth pressure, ground water pressure, or pressure of bulk materials
L_r = roof live Load
S = Snow Load
R = Rain Load
W = Wind Load
E = Earthquake Load

For this report only lateral wind and seismic loads were considered. Therefore only the load cases containing W and E were compared. After comparison 1.6 was found to be greater than 1.0E and was therefore used in the following analysis. *Table 6* shows the breakdown of the wind force for the model. The load was divided amongst the two braced frames. Because of the shape of the building the load had to be broken down into components of x and y before put into an analysis program.

TABLE 6: Wind Force Breakdown

	Force(k)	Force/Frame	1.6F/F	Force(Y)	Force(X)	Leeward(k)	Leeward(Y)	Leeward(X)
Ground	0	0	0	0	0	0	0	0
First	92	46	73	66	32	77	69	33
Second	87	44	70	63	31	69	62	30
Third	87	44	70	63	31	67	60	29
Fourth	90	45	72	65	30	67	60	29
Fifth	92	46	74	67	32	67	60	29
Sixth	97	49	78	70	34	67	60	29
Penthouse	121	61	97	87		84	75	37
Roof	70	35	56			51		

ETABS MODEL

In order to analyze the frames according to the loads calculated in previous sections, the frames were modeled in 3D in ETABS. The correct columns, beams, and cross bracing were modeled and the loads were applied to each floor. The results of the analysis were then compared to hand calculations preformed on the frame. *Figure 15*, *Figure 16*, and *Figure 17* shows the frame as modeled in ETABS.

FIGURE 15: 3D with grid

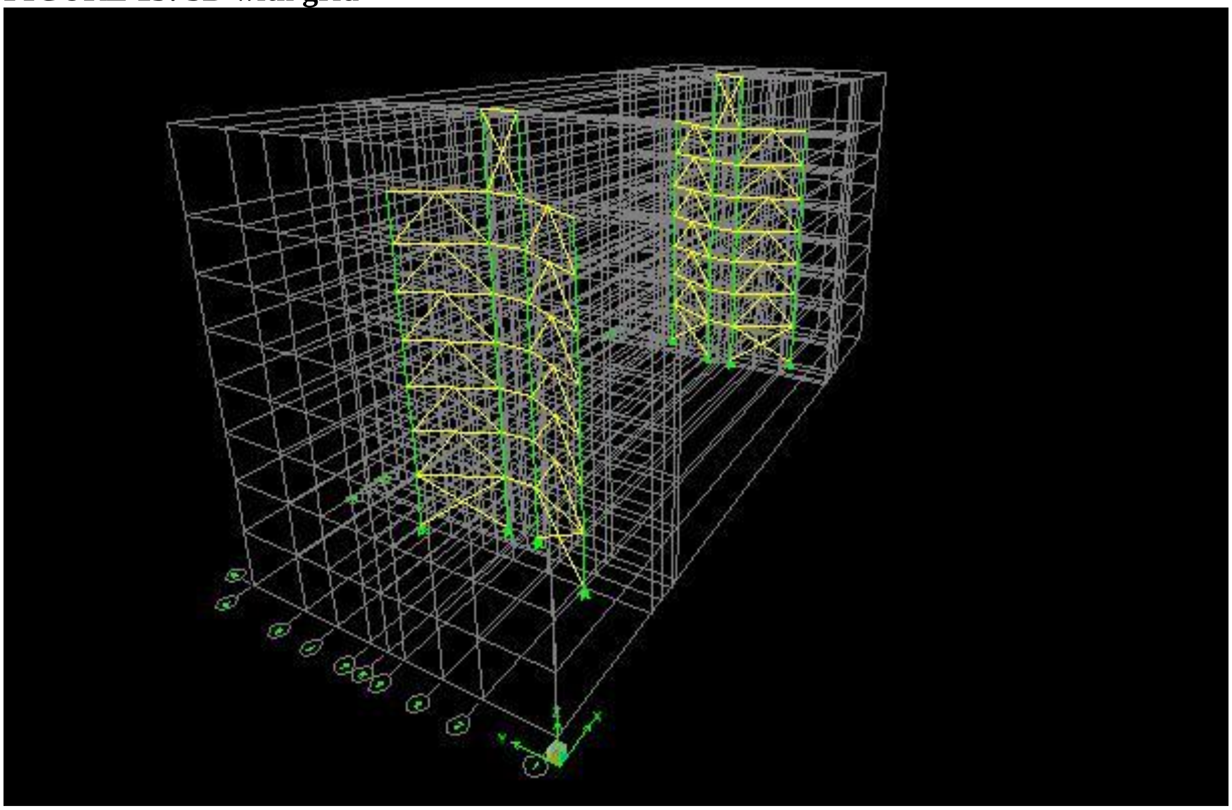


FIGURE 16: 3D without grid

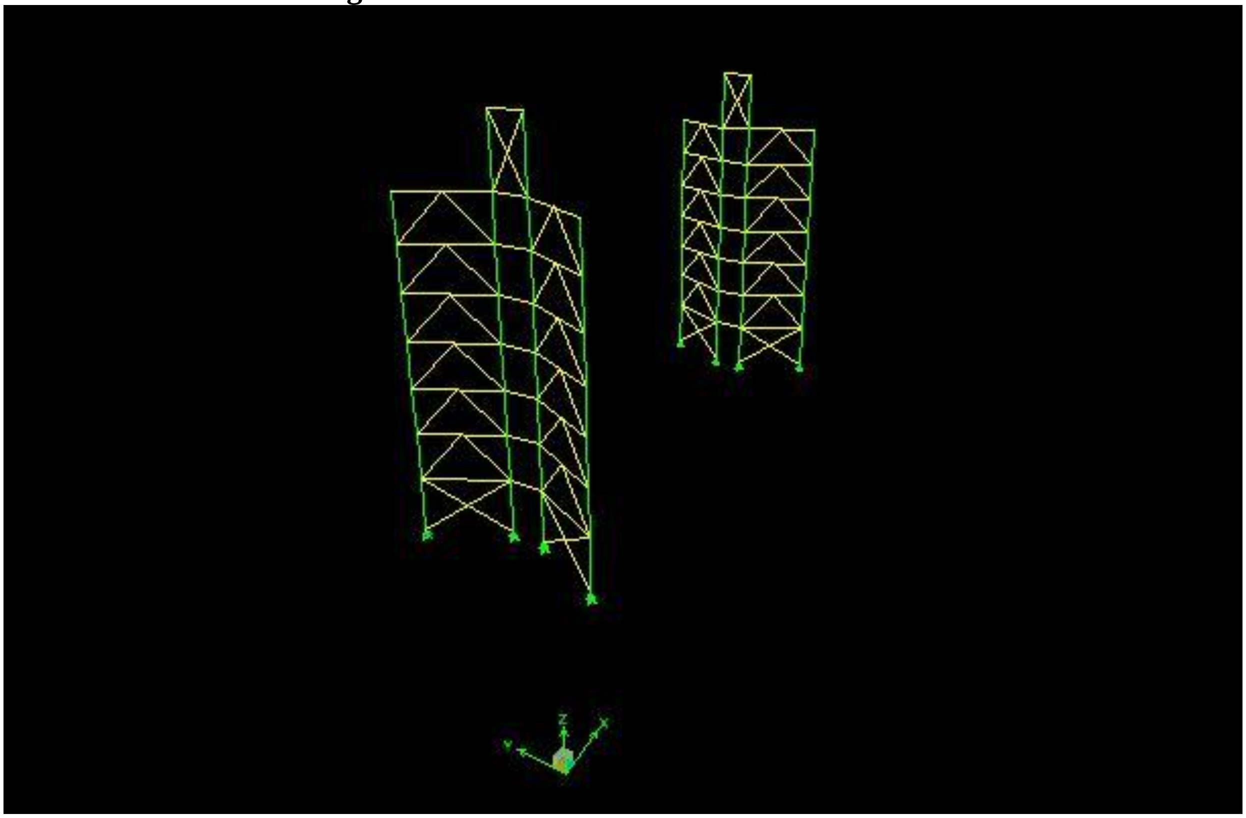
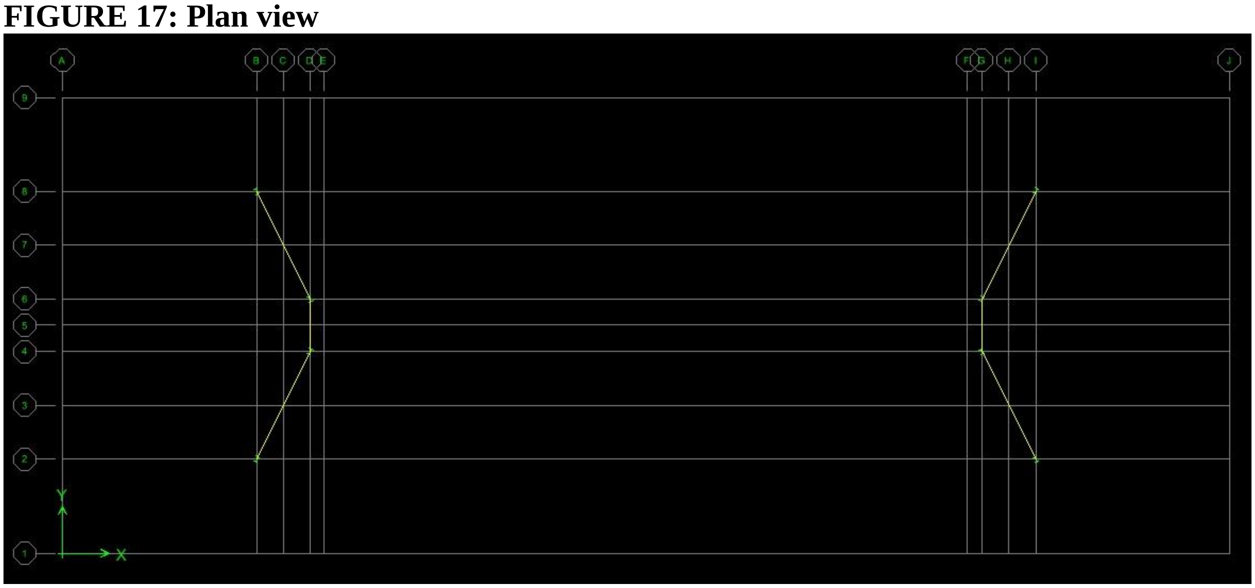


FIGURE 17: Plan view



Results

To find the stiffness of the frame, a 1k load was applied to the penthouse roof. The deflection was found to be .107 inches. Using the equation K = 1/Δ, the stiffness was found to be 9.35k/in.

Since the frames are symmetrically placed on the building the center of mass equals the center of rigidity. There is also no eccentricity. Eccentricity is based on forces acting off-center-of-mass in order to create an unbalanced moment. Since the frames are symmetrical about the center of mass, there is complete balance to the frames.

COM = COR = (Xft, Yft) = (130, 50.5)
Eccentricity = 0

Deflection results were compared to the standard H/400. *Table 7* shows the results of this comparison.

TABLE 7: Deflection Comparison

FLOOR	FLOOR HEIGHT (ft)	H/400 (in)	ETABS DEFLECTION
Ground	9.0	.27	
First	26.5	.795	.49
Second	41.8	1.25	.68
Third	56.2	1.69	.74
Fourth	70.5	2.12	.92
Fifth	84.8	2.54	1.55
Sixth	99.2	2.98	2.407
Penthouse	113.5	3.41	3.20
Roof	127.75	3.83	3.92

ETABS calculated the deflection to be up to standards for every floor with the exception of the roof. The error in the roof is .09 inches. This difference is most likely made up by the slight effect the other various columns and beams throughout the building. The frame shows that it is more than capable of carrying the load for most of the floors. The wind force on the roof was also overcalculated. The analysis of the frame is a slight overdesign as the force was calculated such that the penthouse spans the entirety of the length of the building (260 feet). This is not the case in the actual building, therefore, the wind force would actually be smaller than the force put into ETABS. This would make up for the .09 inch difference and may even lower the deflection of the floors below it as well. Therefore, the conclusion can be made that the "K" braced frame is adequately designed to handle the wind force.

Hand Checks

Hand calculations were performed as a check to the ETABS model. Hand calculations were performed by viewing the frame in 2D as pictured in *Figure 10*. The cantilever method was used to analyze the penthouse story of the building. Moments were checked on the wide-flange beams and columns as well as the square HSS members used for cross-bracing. These values were then compared to the values in AISC. The results are tabulated in *Table 8*. Cantilever method calculations can be found in the Appendices.

Member	Determined Load (ft-k)	AISC Load (LRFD)
W14x43	63.6	69.4
W14x82	9.21	168
HSS8x8x1/4	54.4	129

After using the cantilever method to analyze the penthouse story of the Virginia Advanced Shipbuilding & Carrier Integration Center, it was determined that the frame members that were checked were adequately designed. The W14x82 columns seem to be a drastic overdesign, however, the frame is not only used for lateral resistance, but is also used for gravity load resistance as well. The gravity loads most likely controlled the design of the columns.

APPENDIX:

WIND CALCS

Gust Factor Calculation (ACI 6.5.8)

ACE 6.5.8.1
z = .6(135.2) = 81.2

c = .15 (Table 6-2)

$I_z = .15\left(\frac{33}{81.12}\right)^{1/6} = .129$

$Lz = 650\left(\frac{81.12}{33}\right)^{1/8} = 727.3$

H = 129.5ft
B = 260ft

$$Q = \sqrt{\frac{1}{1+\left(\frac{260+129.5}{727.3}\right)^{.63}}} = .838$$

ACI 6.5.8.2
$g_Q = g_r = 3.4$

$n_1 = \frac{100}{135.2} = .74$

$$g_R = \sqrt{2\ln(3600(.74))} + \frac{.577}{\sqrt{2\ln(3600(.74))}} = 4.18$$

$V_z = .8\left(\frac{81.12}{33}\right)^{1/9} 90\left(\frac{88}{60}\right) = 116.7$

$N_1 = \frac{.74(727.3)}{116.7} = 4.61$

$R_n = \frac{7.47(4.61)}{\left(1+10.3(4.61)\right)^{5/3}} = .053$

$4.6n_1EB/V_z = 7.58$

$RB = \frac{1}{7.58} - \frac{1}{2(7.58^2)}\left(1 - e^{-2(7.58)}\right) = .123$

$\frac{15.4n1L}{Vz}$ = 25.39

$R_L = \frac{1}{25.39} - \frac{1}{2(25.39^2)}(1 - e^{-2(25.39)}) = .0386$

β = .004 for steel

$R = \sqrt{\frac{1}{.004}(.053)(.230)(.123)(.53 + .47(.0386))} = .453$

$G = .925\frac{1+1.7(.129)\sqrt{(3.4^2)(.838^2)+(4.18^2)(.453^2)}}{1+1.7(3.4).129} = 1.003$

Velocity Pressure ACI 6.5.10

$q_z = .00256K_zK_{zt}K_dV^2I$

$K_z = 2.01(\frac{99.161}{700})^{2/11.5} = 1.43$ @ 6th floor

$q_z = .00256(1.43)(1)(.85)(90^2)(1) = 26.421$

Pressure ACI 6.5.12

$p = qG_fC_p - q_i(GC_{pi})$

C_p = .8 winward
.5 leeward

Winward: $p = q_z(1.003)(.8) - 26.421(-.18) = .0824q_z + 4.7558$
Leeward: $p = 26.421(1.003)(-.5) - 26.421(18) = -18.06$

Force of Winward Pressure

@ 5th floor

$K_z = 2.01(\frac{84.83}{700})^{2/11.5} = 1.39$

$q_z = .00256(1.39)(1)(.85)(90^2)(1) = 24.50$

$P_{5th\ floor} = .8024(24.5) + 4.7558 = 24.415$

$P_{6th\ floor} = .8024(25.204) + 4.7558 = 24.979$

$F = \frac{260}{1000}(24.415(\frac{14.33}{2}) + 24.979(\frac{14.33}{2})) = 92k$

SEISMIC LOADS

S_s = .123
S_1 = .049

Site Class: D
Importance Factor: 1

F_a = 1.6
F_v = 2.4

S_{MS} = 1.6(.123) = .1968
S_{M1} = 2.4(.049) = .1176

$S_{DS} = \frac{2}{3}(.1968) = .1312$
$D_{D1} = \frac{2}{3}(.1176) = .0784$

C_T = .028
x = .8

$T_a = .028(135.21)^{.8} = 1.419$
$T_s = \frac{.0784}{.1312} = .598$

R = 8

C_v = 1.7

$C_s = \frac{.1312}{8} = .0164$
$C_s = \frac{.0784}{1.419(8)} = .00691$

Use Min => C_s = .0069

V = .00691(13078.09) = 90.37

K =

$Fx = \frac{WxHx^{1.46}}{\sum Wihi^{1.46}}$