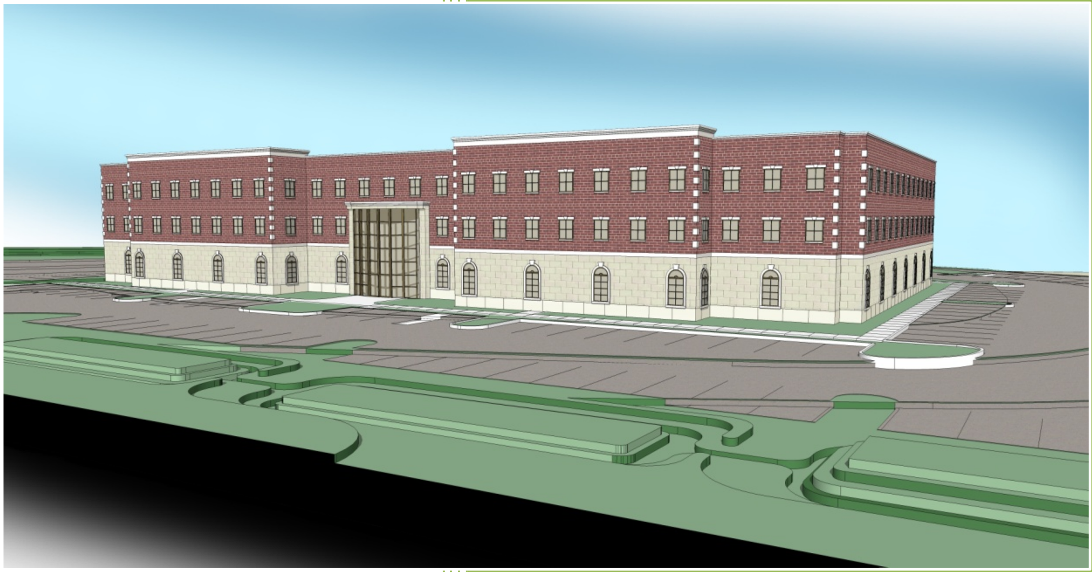
ae Senior Thesis

Crocker West Building



"A Steel Experience"

Eric M. Foster, Structural Option

Prepared for: Dr. Linda M. Hanagan

4/13/2009

(This Page Reserved for *Abstract*)

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- Land/Building Owner
- Structural Engineer
- Employer

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- Structural Engineer
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- Land/Building Owner

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Executive Summary:

This report is slated to assess the current structural system of the Crocker West Building and attempt to document any and all necessary information relating to this thesis study of an alternative lateral framing system and its implications on other systems and overall program layout. The goal of this report is to conduct an in depth study and redesign of the main force resisting system in the building with the addition of 3-stories while trying to avoid severe ramifications to the existing structural integrity or program layout, yet introducing a new style of architecture to the surrounding area.

The Crocker West Building will be used as a highly classified research facility, specializing in the development and testing of underwater weapons for the U.S. Department of Defense. Located in State College, Pa, the structure will be a 3-story low-rise building with areas classified as office, light industrial, and warehouse totaling nearly 120,000 square feet. The first floor of the CWB will consist mainly of 'closed' lab area, along with technician offices, locker rooms and special test areas. The second floor will include office space, another lab area, computer lab, student room and a room designated to SCIF (Sensitive Compartmented Information Facility), while the third floor will be devoted mostly to office space. The existing building will be constructed of Architectural Precast Concrete (APC) systems, including: columns, beams, walls, and floor & roof diaphragms. Lateral loads applied to the structure will be collectively distributed throughout the building to specially designed APC shear walls.

The depth study of this report focuses on the proposed alternate framing system, evaluated and designed using conventional composite steel framing methods while utilizing concentrically braced frames (CBF) for the main lateral resisting system. These designs shall adhere to the provisions of ASCE 7-05, the 13th edition AISC Steel Construction Manual, and any other applicable design codes and standards. Two breadth topics are also examined with respect to a new lateral system. An architectural breadth that addresses conflict between CBF locations and programmatic space was conducted with an additional 3-D BIM that was created to visually display the additional three stories. Relative cost and schedule effects due to the additional levels and construction time of the proposed building were also observed. Culmination of this report shows that the proposed system can be considered a feasible alternative for the Crocker West Building.

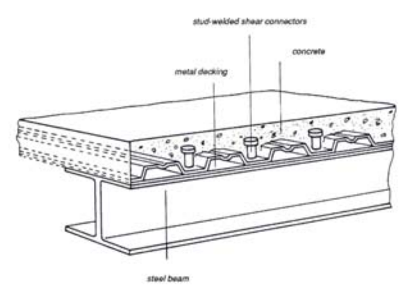

Figure 1: Composite Steel Beam Perspective

Project Background:

The Crocker West Building (CWB) Project is located in Ferguson Township just off SR 26, west of State College, Pa on Science Park Road. The site falls in an area zoned as Light Industry, R & D by the township. Construction for Crocker West was initiated in late February 2009 and will conclude in December 2009. Local State College businessmen Scott A. Smith, P.E. of Civilsmith Engineering, Inc. and Mike D. Coyle of Northwestern Mutual Financial purchased the plot of land and will finance construction costs for the 3-story (45-ft), 121,000 SF building. The structure is being designed by Civilsmith Engineering, Inc. in unison with The Pennsylvania State University for one of its divisions of special research; as Penn State plans to lease-to-own the building.

Crocker West will be used as a highly classified research facility, specializing in the development and testing of underwater weapons for the U.S. Department of Defense. The 3-story low-rise building will house office space, warehouse storage, 'closed' lab areas, and light industrial workspace. The first floor of the CWB is comprised mostly of 'closed' lab area, along with technician offices, locker/restrooms, and specially required test areas. The second floor will include office space, lab area, a computer lab and student rooms, as well as a security sensitive room (designated S.C.I.F.), while the third floor plans to be devoted to mostly office space. Furthermore, the first floor has a 16-ft floor-to-floor height while the second and third floors typical floor-to-floor height is 12-ft.

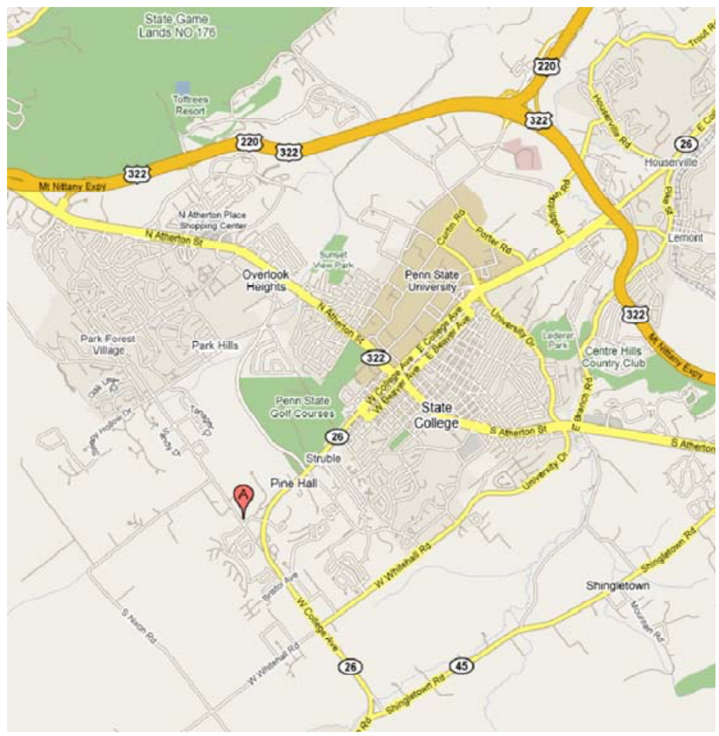

Figure 2: Location Map of Crocker West Building site location, 224 Science Park Road (Designated by bubble A)

Architectural Precast Concrete (APC) has been selected as the primary building material for the Crocker West Building based on its design flexibility, aesthetics and relative speed of construction. APC will be used as a variety of structural systems including columns, beams, wall panels, floor & roof diaphragms, and shear walls. In fact, cast-in-place concrete spread footings will be the only non-precast structural elements on the project.

Existing Structural System -

As stated above, the superstructure of CWB is almost entirely a precast. The following are detailed explanations of the individual precast members and systems, along with a description of the cast-in-place foundations.

FOUNDATION: The foundation system(s) being implemented consist of 3000 psi and 4000 psi air-entrained, cast-in-place (CIP) strip and spread footings, as well as a CIP slab-on-grade (4000 psi). Fifteen inch deep strip footings ranging from 3'-3" to 6'-6" wide are used along the perimeter of the structure. These footings help distribute wall panel loads to the ground. Additionally, the East walls strip footing of the structure will also be used as a part of the underground water cistern that will be used to collect treatable storm water runoff for reuse. Spread footings will be used throughout the interior portion of the building and will be used to pick up loads from columns and stair-towers. Spread footing located under columns vary in size from 12' square to 14'-5 square, while the four typical stair-tower footings are 12'-0 x 25'-6. All spread footings are 2 foot thick unless noted otherwise. A six-inch thick slab-on-grade reinforced with W4.0 x W4.0 welded-wire fabric (W.W.F.) will complete the foundation system(s) and will be used as the ground floor level of the building. The 12' deep by 14' wide cistern used to house the storm water will be constructed of precast and will span the length of the East wall.

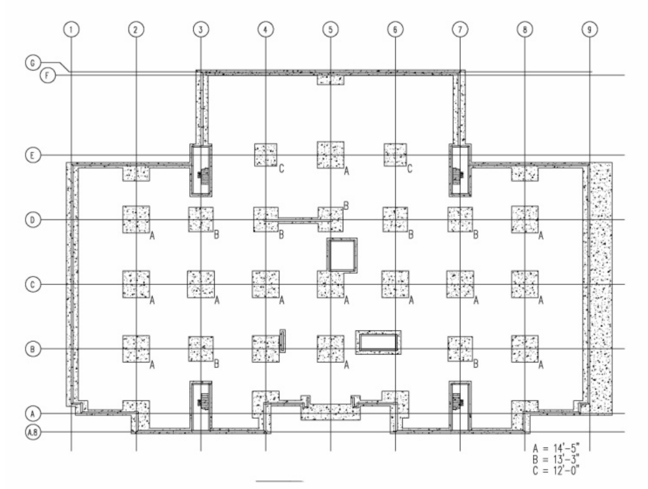

Figure 3: Existing Foundation Plan

COLUMNS: The vertical supporting members for the entire structure are reinforced, precast concrete columns. All columns are 24" x 24" square columns with four (4) #11 longitudinal reinforcing bars and #4 stirrups spaced accordingly, both Grade 60 steel. Columns will be cast for lengths up to 42 feet. Each column will contain haunches and haunch reinforcing cast monolithically at each floor level and in the required position for beam bearing and load transfer. The columns are spaced on a 35'-0 x 35'-0 typical bay grid and are connected to the spread footings with four (4) 1 ¼" dia. ASTM A193 threaded rods.

FLOOR SYSTEM: The 1st Floor (or Ground Level) floor system is a 6" thick slab-on-grade with W4.0 x W4.0 WWF reinforcing. The framed floor levels are constructed of precast, prestressed hollow-core planks with similar sections depicted in Figure 4 below. The 2nd Floor Level will consist of 12-in. thick by 4-ft. wide plank and the 3rd Floor Level will be comprised of 10-in. thick by 4-ft. wide plank, each with seven (7) 7-wire, ½" dia. 270 ksi low-relaxation prestressing strands and a 2" topping. Some of the hollow-core floor system clear spans are nearly 33'-0, with individual panels running transverse to Project North.

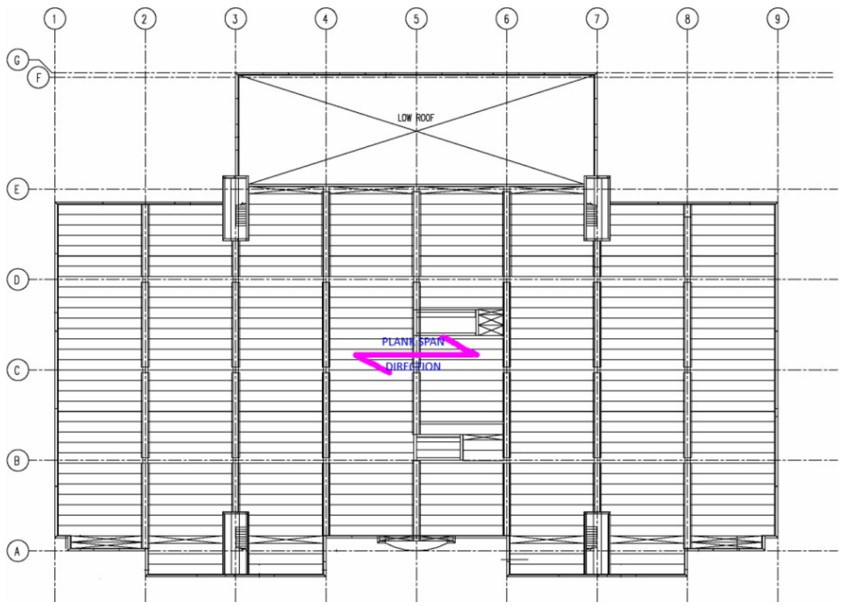

Figure 4: Precast Hollow-Core Plank Layout

Furthermore, these hollow-core slabs are supported by one of two methods. If the floor slab is to bear at an exterior wall panel location, a specially designed bearing ledge will be cast into the precast wall panel with proper reinforcing. For interior bay supports, the hollow-core slabs will be supported by precast, prestressed concrete inverted-tee (IT) beams bearing on column haunches. IT beams for the 2nd Floor were designed to be 28" deep, while 3rd Floor beams are 20" deep due to dissimilar live loads.

ROOF SYSTEM: The roofing system for the Crocker West Building main roof will be constructed by means of similar materials used in erecting floors two and three. The main roof will consist of 8-in. thick by 4-ft. wide hollow-core plank with (7) 7-wire, ½" dia. 270 ksi low-relaxation strands supported by 18" deep inverted-tee beams. The low roof, located in the rear storage area of the building, will also be constructed of 4-ft. wide hollow-core planks. A portion of the low roof is being designed as a roof garden and will consist of 12-in. thick plank; while the remaining half is comprised of 8-in. thick plank. In addition, each roof will receive a layer of 4" tapered rigid insulation and a 60 mil EPDM roofing membrane rather than a 2" topping which is not needed on the roof.

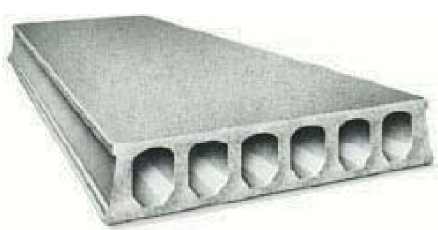
Figure 5: Precast Concrete Hollow-Core Plank

LATERAL SYSTEM: One of the key design issues of a total precast structure is the makeup of the main lateral force resistance system. Crocker West was no different; its lateral system was designed as displayed in Figure 6 using a compilation of precast shear walls positioned around the perimeter and throughout the building. These precast shear walls are constructed with several different thicknesses of insulated and non-insulated precast panels. Exterior wall panels (all insulated) acting as shear walls in the N-S direction are 12 ½" thick, while E-W direction walls are 9 ½" thick. Shear walls located on the interior of the structure and around stair-towers are 9" thick and non-insulated. Due to the fact that every panel is individually erected, specially designed connections are required for each piece. These connections, not specified in this report, are designed to ensure the applied load is safely distributed to the lateral system.

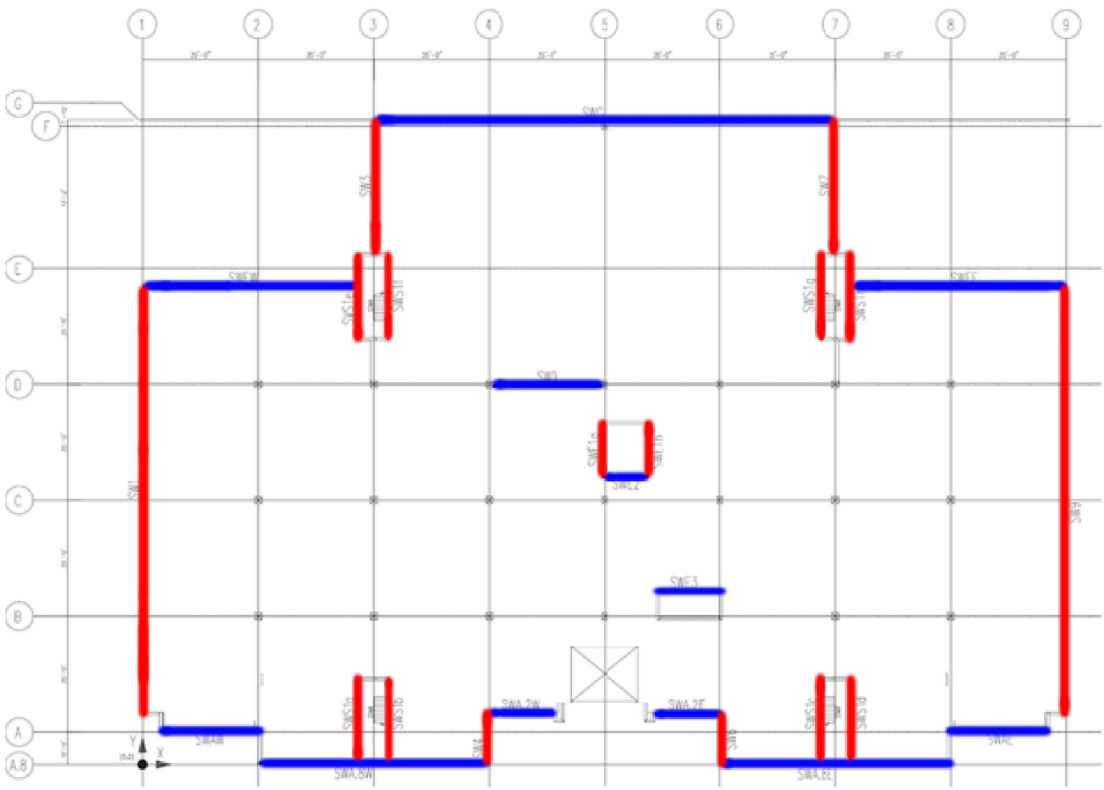

Figure 6: APC Panels used as Shear Walls

Architecture & Site -

ARCHITECTURE: The Crocker West Building will be used as a highly classified research facility, specializing in the development and testing of underwater weapons for the U.S. Department of Defense. For this reason, the façade of the structure only contains a minimal amount of repetitious Pella windows. The planned tenants will tint all windows as they see fit to satisfy their security requirements. The APC panels may be designed in a variety of ways to create the authentic look the owner desires. Although the veneer will look like real brick and stone, individual panels will be designed with either a brick or stone veneer in-lay; where a mold resembling the face of a brick or stone are laid in place prior to casting the panel. This allows for a controlled veneer and joint pattern. Also, the panels will be cast using specific concrete colors and finished using an assortment of sand-blasts to provide a clean, natural masonry-looking façade.


Figure 7: Example of Brick In-lay APC Panel System

Figure 8 & Figure 9 below shows the breakdown of the program requirements. The 121,000 s.f. facility will roughly consist of 70,000 s.f. of office space (Red), 40,000 s.f. of light industrial space (Blue), and 10,000 s.f. of warehouse storage (Yellow). Note, the (Green) designates public space and the third floor was omitted having been designated solely to office space.


Figure 8: Ground Floor Program Requirements


Figure 9: 2nd Floor Program Requirements

SITE & ZONING: The parking lot will consist of roughly 300 spaces, 8 of which are designated for handicapped parking. Approximately 100 of these spaces utilize a pervious parking section with a geotextile reinforcing; while the remaining parking lot will be developed using a 12-in. paving section consisting of 6" PADOT 2B compacted stone, 4 ½" BCBC, and 1 ½" ID-2. Geotextiles are manufactured, woven & nonwoven, fiber materials made into a variety of fabric constructions. Due to variable soil conditions concerning rock and sink-holes, the net allowable bearing capacity of the soil ranges from 1,500 psf to 10,000 psf. Figure 11 displays the problem areas as well as defines the allowable bearing capacity in these locations.

Figure 10: Example of Geotextile Reinforcing

As established, Ferguson Township has zoned the project location as Light Industry, R & D. As such, the site landscaping will comply with Ferguson Townships requirements for the "Corridor Overlay District", and the "IRD Zoning Ordinance for buffer yards".

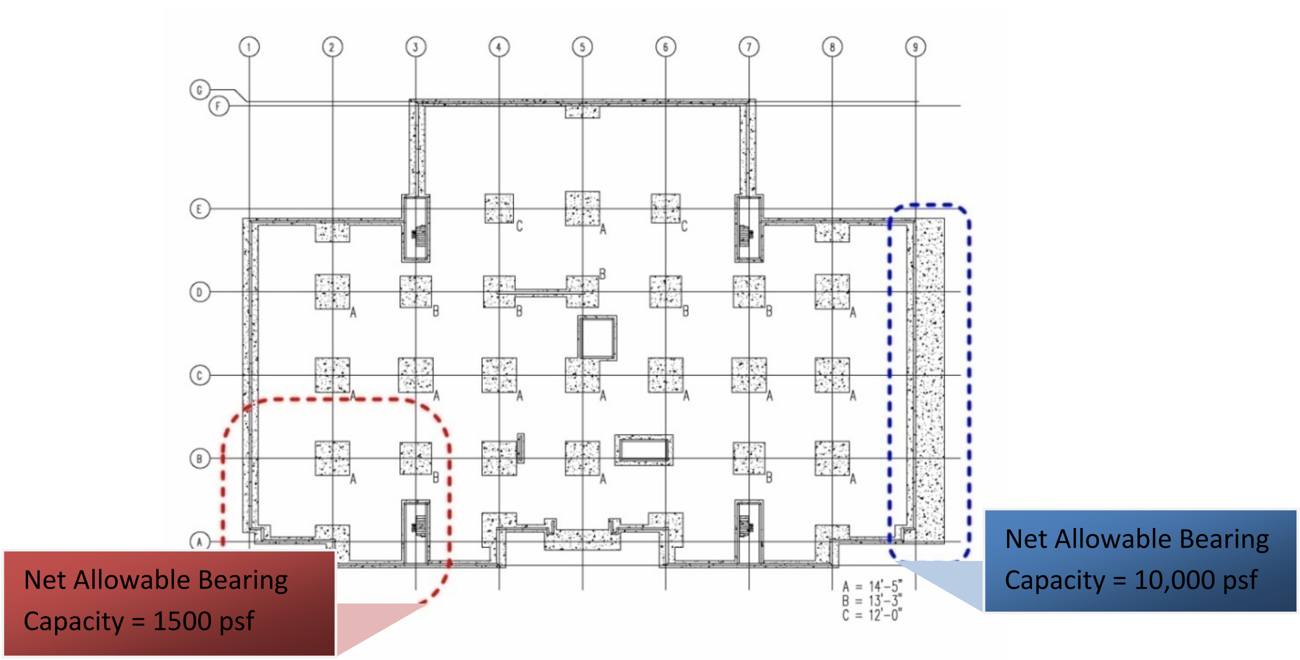

Figure 11: Soil Bearing Capacities

Net Allowable Bearing Capacity = 3500 psf
Where Not Noted

Design Criteria -

The following is a list of material properties used for the design of the individual structural elements for new & existing structure.

STRENGTH of MATERIALS:

CONCRETE -	
Foundation Walls	f'_c = 4,000 psi (Air Entrained)
Footings, Exterior & Below Grade	f'_c = 3,000 psi (Air Entrained)
Slab on Grade & Locations Not Specified	f'_c = 4,000 psi
Mud Slabs and Over-Excavated Areas	f'_c = 2,000 psi
REINFORCING STEEL -	
Deformed Individual Bars	ASTM A615, Grade 60
Welded Wire Fabric (W.W.F.)	ASTM A185, Fy = 65 ksi
STRUCTURAL STEEL -	
Wide Flange Shapes (W-Shapes)	ASTM A992, Fy = 50 ksi
Channels, Angles, Plates	ASTM A36
HSS Structural Tubes	ASTM A500, Fy = 46 ksi
Anchor Bolts	ASTM F1554, Fy = 36 ksi
ARCHITECTURAL PRECAST CONCRETE -	f'_c = 4,000-6,000 psi
PRESTRESSING STRANDS -	
½" Special (7-Wire), Low-Relaxation Strands	f_{ps} = 270 ksi

Design Codes and Standards -

The subsequent codes listed were used for both, the original design and thesis redesign of the structure due to local code regulation and the building being of 'new construction'.

Centre Region Building Code.

<www.centreregioncode.org>

International Building Code (IBC), 2006.

American Society of Civil Engineers (ASCE) 7-05

Design Loads for Building and Other Structures.

Aisc.ico AISC Manual of Steel Construction – 13th Edition.

ACI Building Code Requirements for Reinforced Concrete, ACI 318-05.

Load Combinations -

Design load combinations are in accordance with the 2006 International Building Code, Section 1605.2.1 and ASCE 7-05, Ch. 2.

Basic Combinations	
1.	1.4(D +F)
2.	1.2(D + F + T) + 1.6(L + H) + 0.5(L_r or S or R)
3.	1.2D + 1.6(L_r or S or R) + (L or 0.8W)
4.	1.2D + 1.6W + L +0.5(L_r or S or R)
5.	1.2D + 1.0E + L +0.2S
6.	0.9D + 1.6W + 1.6H
7.	0.9D + 1.0E + 1.6H

Design Loads -

The following loads listed were analyzed and used for the original design of the structure.

LIVE LOADS			(GRAVITY)
1. Roof - Minimum		=	20 psf
2. Roof - Snow:			
	Ground Snow Load, P_g	=	40 psf
	Importance Factor, I	=	1.0
	Snow Exposure Factor, C_e	=	1.0
	Roof Thermal Factor, C_t	=	1.0
Flat Roof Snow Load, P_f		=	30 psf
	(Drift Loads, where required per ASCE 7-05)		
3. Stairs		=	100 psf
4. 1^{st} Floor (Slab-on-Grade)		=	250 psf
5. 2^{nd} Floor		=	125 psf
6. 3^{rd} Floor		=	80 psf
7. Live Load Reductions per IBC Section 1607.9			
	No Live Load Reduction for LL > 100 psf.		
	No Live Load Reduction at Roof.		

DEAD LOADS			(GRAVITY)
1. Roof -			
	Roofing & Insulation	=	5 psf
	8" Hollow-Core Plank	=	62 psf
	MEP	=	5 psf
	Ceiling	=	2 psf
	Total	=	74 psf
2. 3^{rd} Floor -			
	10" Hollow-Core Plank	=	67 psf
	Concrete Topping	=	22 psf
	MEP	=	5 psf
	Ceiling	=	2 psf
	Flooring	=	5 psf
	Total	=	101 psf
3. 2^{ND} Floor -			
	12" Hollow-Core Plank	=	72 psf
	Concrete Topping	=	22 psf
	MEP	=	5 psf
	Ceiling	=	2 psf
	Flooring	=	5 psf
	Total	=	106 psf

WIND LOADS			(LATERAL)
1. Wind Load (IBC 2000, Section 1609)			
	Basic Wind Speed, V	=	90 mph
	Exposure Category	=	C
	Importance Factor, I	=	1.0
Wind Design Method, Simplified Wind Procedure		=	20 psf

SEISMIC LOADS			(LATERAL)
1. Seismic Load (IBC 2006, Sect. 1613 & ASCE 7-05, Ch. 11 & 12)			
	Occupancy Category/Seismic Use Group	=	II
	S_S	=	0.17
	S_1	=	0.06
	S_{DS}	=	0.182
	S_{D1}	=	0.096
	Site Class	=	D
	Importance Factor	=	1.0
	Seismic Design Category	=	B
Analysis Procedure, Equivalent Lateral Force			
Basic Seismic Force Resisting System			
	"Ordinary Precast Shear Walls in Bearing Wall", R	=	3
SEISMIC DESIGN BASE SHEAR, $V = C_S W$		=	883^k

As proven from tables above, the controlling design loads for the Crocker West Building are based on seismic criteria. Good engineering judgment allows this to be predicted based on the overall height of the structure – to – seismic weight ratio. The magnitude of the concrete weight is far more sufficient against wind forces than earthquake forces.

Depth Study: Alternate Lateral System

This portion of the report examines a complete modification of the existing structure from precast concrete to composite steel framing, utilizing concentrically braced frames in both orthogonal directions.

Problem Statement –

Figure 12: Concentrically Braced Steel Framing (CBF)

PROPOSAL: Having performed several of the actual designs used for the Crocker West building and the required technical reports for this senior thesis project proved that the structural system utilized in the existing building is adequate of resisting the calculated gravity, wind, and seismic loads. For the purpose of fulfilling senior thesis project requirements, a petition will assumed to be deemed granted for the addition of 3-stories needed for future office space. The acquired permission was issued with the understanding and intent that the redesign takes on a more architectural appeal than the existing precast envelope.

SOLUTION: Due to the additional levels proposed for the structure, a redesign of the lateral system using CBF's, concentrically braced steel frames as the main lateral force resisting system shall be examined. The lateral system will be modeled in RAM to properly apply and analyze the aforementioned load combinations for criteria such as drift, torsion, and vibration. RAM is a computer modeling program designed to model the lateral load affects in order to determine the controlling forces needed for design. These situations will be carefully modeled to ensure proper design of CWB's new structural system, with the least amount of impact as possible to the overall building layout.

RAMIFICATIONS: The proposed expansion will no doubt provoke new challenges to the project. The addition of 3-stories could possibly add to the seismic weight of the building; thus inducing higher seismic loads applied at each story and a redesign of the existing foundation system for the extra axial loads. Further site investigation would likely be required for required landscape and parking criteria due to the increase in building area.

Design Criteria -

INTRODUCTION: Concrete and steel are predominantly two of the most widely used construction materials in the State College region due to adverse weather effects on materials. Upon initiation of this project, it was decided that the overall architectural layout be as undisturbed as possible due to special programming requirements by the leasing tenant while maintaining structural integrity. Typical bay dimensions for Crocker West are thirty-five foot (35') x thirty-five foot (35') to column center-lines in both orthogonal directions. Preserving the open floor layout with another concrete floor system would have been possible; however the span-to-depth ratio for the alternative system ultimately would have thickened the slab, increasing the weight of the superstructure and raising potential for punching shear failure around columns. This increase in weight due to the additional levels will undoubtedly have a substantial effect on the cost of the new structure. Design issues such as these were analyzed and researched in previous Tech II Report and consequently a substitute concrete system was not pursued.

Steel was chosen as the alternative framing material for redesign of the lateral force resisting system based on several factors. Steel offers a great deal of resistance over long spans due to its high strength-to-weight ratio, allowing for minimum floor plenum depths. One element of the lateral system redesign was to optimize the amount of braced frames used throughout the structure. The existing APC superstructure consisted of nearly 30 specially designed shear walls scattered in plan and along the perimeter. Using concentrically braced steel frames as the main lateral system may slightly hinder the existing architectural layout based on the positioning of the lateral braces, however these CBF's will try to be incorporated into the architectural aspect of the design by leaving them exposed in selected glazed areas. The Crocker West Building was also reconsidered using steel based on similar speed of erection time to precast concrete. Maintaining the implemented 13 month schedule would have been difficult with an alternative cast-in-place system due to concrete curing time required. Lastly, steel was chosen as the basis of redesign for the purpose of ultimately decreasing the weight of the structure and reducing applied building forces.

The goal of the structural redesign is to replace the current architectural precast concrete shear walls with precisely placed concentrically braced frames. This includes a redesign of the current hollow-core plank floor system to a composite system utilizing 3" metal deck with 3" light-weight concrete (LWC) cover for a total slab depth of 6 inches. The overall design of the steel system will be ultimately compared to the current concrete system. Conclusions will be based upon construction impacts, performance, architectural impacts, cost & scheduling. The structural redesigns presented herein have been designed with various assumptions to accelerate the analysis and design process.

Design Codes and Standards -

The subsequent codes listed were used for both, the original design and thesis redesign of the structure due to local code regulation and the building being of 'new construction'.

Centre Region Building Code.

<www.centreregioncode.org>

International Building Code (IBC), 2006.

American Society of Civil Engineers (ASCE) 7-05

Design Loads for Building and Other Structures.

Aisc.ico AISC Manual of Steel Construction – 13th Edition.

ACI Building Code Requirements for Reinforced Concrete, ACI 318-05.

Load Combinations -

Design load combinations are in accordance with the 2006 International Building Code, Section 1605.2.1 and ASCE 7-05, Ch. 2.

Basic Combinations	
1.	1.4(D +F)
2.	1.2(D + F + T) + 1.6(L + H) + 0.5(L_r or S or R)
3.	1.2D + 1.6(L_r or S or R) + (L or 0.8W)
4.	1.2D + 1.6W + L +0.5(L_r or S or R)
5.	1.2D + 1.0E + L +0.2S
6.	0.9D + 1.6W + 1.6H
7.	0.9D + 1.0E + 1.6H

Design Loads -

The following loads listed were analyzed and used for the redesign of the structure.

GRAVITY: Gravity loads for the newly proposed steel framed structure differ from that of the existing design. While the live loads remain nearly identical, for the exception of the additional green roof loading, the calculated dead loads for the redesign are substantially less than the original. Snow loads were calculated by means of ASCE 7-05, Ch. 7.

LIVE LOADS		(GRAVITY)
1. Roof - Minimum	=	20 psf
2. Roof - Snow		
Ground Snow Load, P_g	=	40 psf
Importance Factor, I	=	1.0
Snow Exposure Factor, C_e	=	0.9
Roof Thermal Factor, C_t	=	1.0
Flat Roof Snow Load, P_f	=	25.2 psf
(Drift Loads, where required per ASCE 7-05)		
3. Green Roof	=	100 psf
4. Stairs	=	100 psf
5. 1st Floor (Slab-on-Grade)	=	250 psf
6. 2nd Floor	=	125 psf
7. 3rd - 6th Floor	=	80 psf
8. Live Load Reductions per IBC Section 1607.9		
No Live Load Reduction for LL > 100 psf.		
No Live Load Reduction at Roof.		

DEAD LOADS		(GRAVITY)
1. Green Roof -		
Roofing & Insulation	=	5 psf
Green Roof (Assumed Wt. of Sat. Soil)	=	30 psf
MEP	=	5 psf
Ceiling	=	2 psf
Total	=	42 psf
2. 2nd thru 6th Floor -		
3" USD Metal Deck w/ 3" LWC Concrete	=	43 psf
Partitions	=	20 psf
MEP	=	10 psf
Ceiling	=	2 psf
Flooring	=	5 psf
Total	=	80 psf
3. Exterior Cladding -		
Glass Curtain Wall/Metal Panel (Assumed)	=	15 psf

WIND: A wind analysis was prepared using the ASCE 7-05 Analytical Procedure, as depicted in Section 6.5. Wind pressures were assumed to be uniformly (horizontally) & exponentially (vertically) distributed across each of the six stories to calculate the design wind forces. Conservative standards for K_Z were taken when floor heights fell between differing wind pressures. The wind pressures were multiplied by the corresponding façade areas to then find the appropriate story shears. These individual story shears are then summed up to generate the total base shear per respective elevation of the building due to wind. Below are the results for the design wind pressures, design base shears and over-turning moments for wind loading generated in the E-W & N-S directions, respectively. Wind forces do not govern in any of the categories or directions considered as expected due to the ratio of height added to total square footage of the building added.

WIND LOADS			(LATERAL)
1. Wind Load (ASCE 7-05, Ch. 6)			
	Basic Wind Speed, V	=	90 mph
	Importance Factor, I	=	1.0
	Topographic Factor, K_{zt}	=	1.0
	Directionality Factor, K_d	=	0.85
	Exposure Category	=	C
	Building Classification	=	Category II
	Gust Effect Factor, G	=	0.86
Analysis Method: Analytical Procedure			

E-W Design Wind Pressures (Normal to 150-ft. Face)						
Story	Height Above Grade	Floor Tributary Height	Wind Pressure	Building Width 'X' Wind Dir.	Windward 'X' Story Force	Overturning Moment 'X'
	(ft.)	(ft.)	(psf)	(ft.)	(kips)	(ft.-kips)
Roof	79	6	14.7	150	13.2	1045.2
6th	67	12	14.2	150	25.6	1712.5
5th	55	12	13.7	150	24.7	1356.3
4th	43	12	13.2	150	23.8	1021.7
3rd	31	12	12.7	150	22.9	708.7
2nd	19	15.5	10.9	150	25.3	481.5
				Wind Base Shear 'X': ΣV =	135.4	
					OT Moment 'X': ΣOTM =	6325.8
				Factored Base Shear 'X': ΣV = 1.6 x V =	216.7	kips

N-S Design Wind Pressures (Normal to 280-ft. Face)						
Story	Height Above Grade	Floor Tributary Height	Wind Pressure	Building Width 'Y' Wind Dir.	Windward 'Y' Story Force	Overturning Moment 'Y'
	(ft.)	(ft.)	(psf)	(ft.)	(kips)	(ft.-kips)
Roof	79	6	14.7	280	24.7	1951.0
6th	67	12	14.2	280	47.7	3196.7
5th	55	12	13.7	280	46.0	2531.8
4th	43	12	13.2	280	44.4	1907.1
3rd	31	12	12.7	280	42.7	1322.8
2nd	19	15.5	10.9	280	47.3	898.8
				Wind Base Shear 'Y': ΣV =	252.8	
					OT Moment 'Y': ΣOTM =	11808.2
			Factored Base Shear 'Y': ΣV = 1.6 x V =		404.4	kips

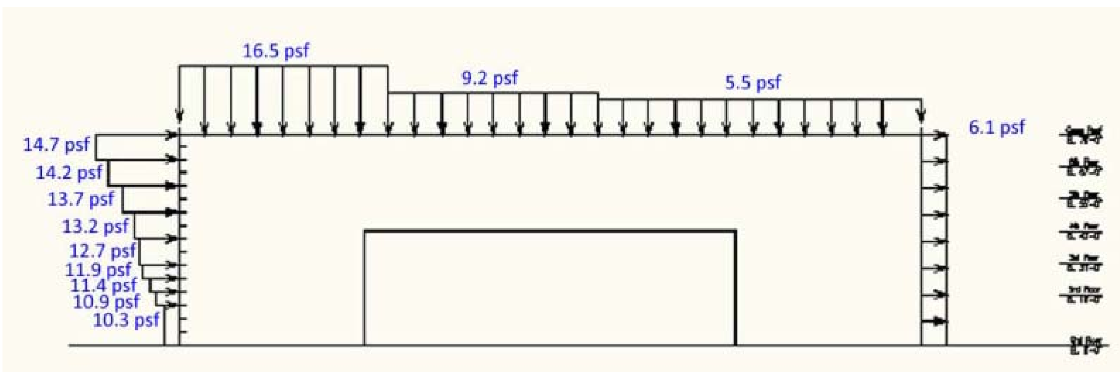

Figure 13: E-W Design Wind Pressures

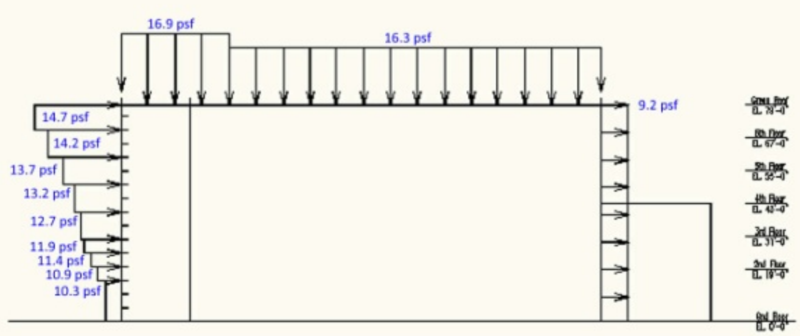

Figure 14: N-S Design Wind Pressures

SEISMIC: Similar to the existing design, seismic loads were calculated in accordance with ASCE 7-05, Section 12.8 – Equivalent Lateral Force Procedure (ELF). The ELF procedure was used to analyze and determine the total seismic base shear (V), as well as the applied seismic shear forces at each floor. Centre Region Code's website (www.centreregioncode.org) was used to specify spectral response acceleration parameters, S_S and S_1. The new design features 'Ordinary Steel Concentrically Braced Frames' which yields a value of 3 ¼ for both, the response modification coefficient (R=3.25) & deflection amplification factor (C_d = 3.25) from ASCE, Table 12.2-1. These, and the rest of the values listed in the following seismic loads table are then used in conjunction with the effective seismic weight to determine the unfactored seismic base shear.

SEISMIC LOADS		(LATERAL)
1. Seismic Load (ASCE 7-05, Ch. 11 & 12)		
Occupancy Category/Seismic Use Group	=	II
S_S	=	0.17
S_1	=	0.06
S_{DS}	=	0.136
S_{D1}	=	0.068
Site Class	=	C
Importance Factor	=	1.0
Seismic Design Category	=	B
Effective Seismic Weight	=	$24{,}770^k$
Analysis Procedure, Equivalent Lateral Force		
Basic Seismic Force Resisting System		
"Ordinary Steel Concentrically Braced Frames", R	=	3.25
SEISMIC DESIGN BASE SHEAR, $V = C_S W$	=	991^k

The above base shear is then collectively distributed vertically over the entire building height during any seismic activity (i.e.- during an earthquake). The vertical distribution of seismic forces (F_X), seen in the table below, is based on a ratio of each individual story height and weight divided by the summation of every levels heights and weights. These seismic forces are assumed to be laterally applied at the respective floor levels because the structures diaphragm acts as a load path for these earthquake forces to travel to the required vertical resisting elements, where the loads will ultimately be transferred back into the ground. It is important to note that the redesign of this structure was assumed being classified as 'rigid', thus having a 'rigid' diaphragm to properly transfer the seismic forces.

After the seismic shear force contribution to each floor level has been determined using ASCE Eq. 12.8-11, these story shears can then be broken-down further and effectively distributed throughout designed framing members like CBF's, girts/beams, and columns. RAM Structural System was used to aid in the modeling and analysis of these individual framing components. Comparing values of the two lateral system analyses performed, wind & seismic, it is apparent the seismic loading conditions control any and all lateral system designs in both orthogonal directions.

Vertical Distribution of Seismic Forces						
	Height from Base to Level x, h_x	Portion of Total Effective Seismic Wt. at Level x, w_x	$w_x h_x^k$	$C_{vx} = w_x h_x^k / \Sigma w_i h_i^k$	$F_x = C_{vx} V$	Overturning Moment
	(ft.)	(kips)	(ft.-k)		(kips)	(ft.-k)
Green Roof	79	3111	32908	0.22	222.5	17578.9
6th Floor	67	4054	34040	0.23	230.2	15421.5
5th Floor	55	4060	27974	0.19	189.2	10403.6
4th Floor	43	4600	24018	0.16	162.4	6983.4
3rd Floor	31	4728	17675	0.12	119.5	3705.1
2nd Floor	19	4217	9943	0.07	67.2	1277.4
Gnd. Floor	0	N/A	N/A	N/A	N/A	0
				Seismic Base Shear, V =	991	
`				Overturning Moment at Base =		55369.88

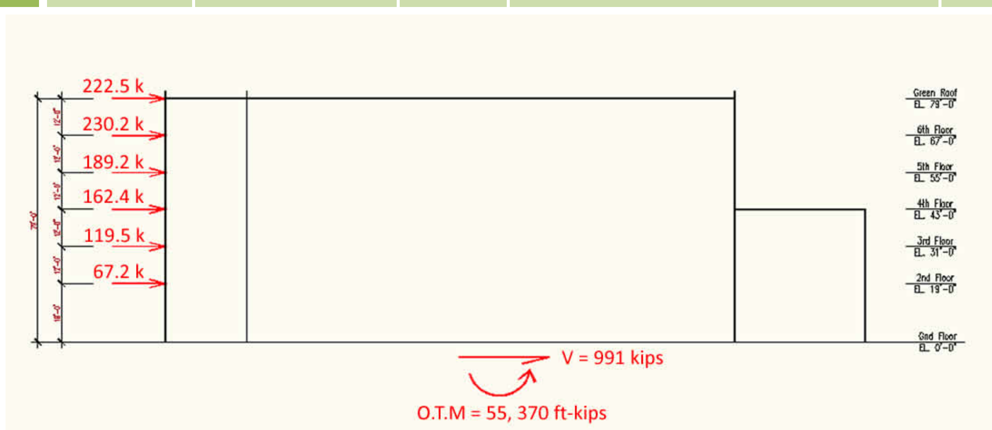

Figure 15: Applied Seismic Story Forces, Base Shear & Overturning Moment

BIM Modeling Procedure -

The following information relates to the design process of the redesigned Crocker West Building including the use of BIM software and thought process of lateral system redesign.

RAM OVERVIEW: The RAM Structural System building information modeling (BIM) software developed by Bentley Systems, Inc. is a fully integrated 3-D modeling, analysis, design, and drafting program for both steel and concrete framed structures. The RAM Manager is divided into separate modules for foundation design, beam design, column design, and frame design; this program aids engineers in the creation of a faster, more productive design.

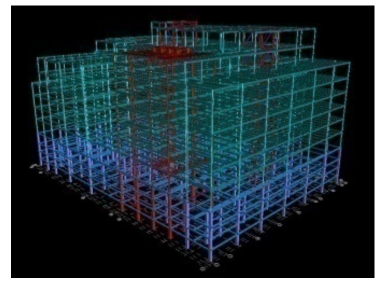

Figure 16: Photo Courtesy of RAM Bentley

RAM MODELING: Making use of available tutorials and understanding the fundamentals of RAM will be of great benefit to any user at the start of a BIM based building design. The first step in a RAM based design involves using the RAM Modeler to construct the building in question. Here grid lines are laid out, all gravity columns and beams are assigned framing locations, loads and floor systems are designated & assigned to particular areas, and proposed lateral framing members are allocated. Figure 16 (above right) shows a basic example of a constructed RAM model and how each member is designated by color schemes. The purple/blue colors represent concrete framing, while the red designates lateral steel framing and the teal gravity framing. As shown in Figure 17, The RAM Modeler was basically designed for the 3-D construction of the working project prior to analysis and design.

Upon successful completion of model status without discrepant errors or warnings in RAM Modeler, the RAM Steel Beam or RAM Steel Column module may then be selected for the progressive analysis. Both of these software design elements conduct full-scale analysis and design of the '<u>gravity only</u>' framing members based on user or default defined criteria and parameters. As part of good engineering practice, the steel beam module should be executed prior to selecting the steel column module; as this method will be sure to account for the proper beam designs and loads assigned in RAM Beam.

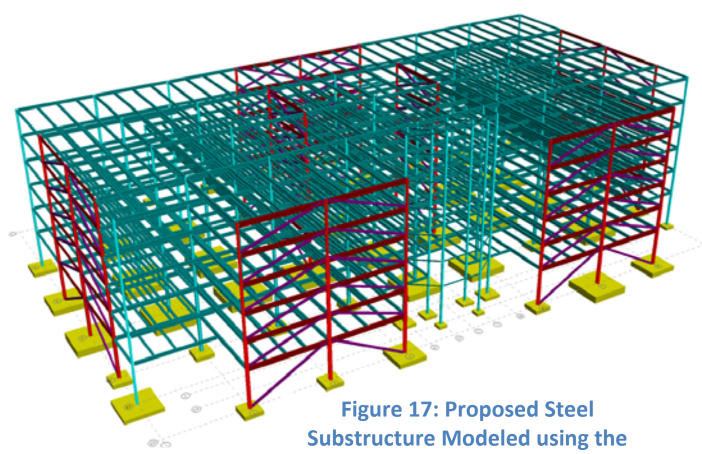

Figure 17: Proposed Steel Substructure Modeled using the RAM SS Modules

RAM Frame will be the next module utilized by the integrated RAM Structural System. The RAM Frame component of the program is somewhat of an extension of the beam and column modules. Simply put, RAM Frame will design members designated as 'lateral' in the Modeler that were not designed by the Beam or Column modules because they were part of a lateral force resisting system. Standard & seismic steel provisions, and drift control are all criterion used to properly account for the various load cases and combinations applied to the structure.

Assuming all the designs were analyzed, and effectively designed by RAM Frame, the RAM Manager should indicate green lights next to completed modules. RAM Foundation will conclude these modules with a design of the projects substructure. This particular module allows for site specific soil bearing capacities, optimization of column base plates and spread footings using an array of design parameters such as: including moments due to shear in column footings, and designing footings based on applied forces or soil capacity. The module is equipped to model spread footings & continuous (strip) footings for shallower substructures, while piles and pile caps are model ready for poor site conditions with little or no bearing capacity. Piles and pile caps are not typically used in the State College region due to the amount of rock beneath the soil and the allowable bearing capacities that accompany it.

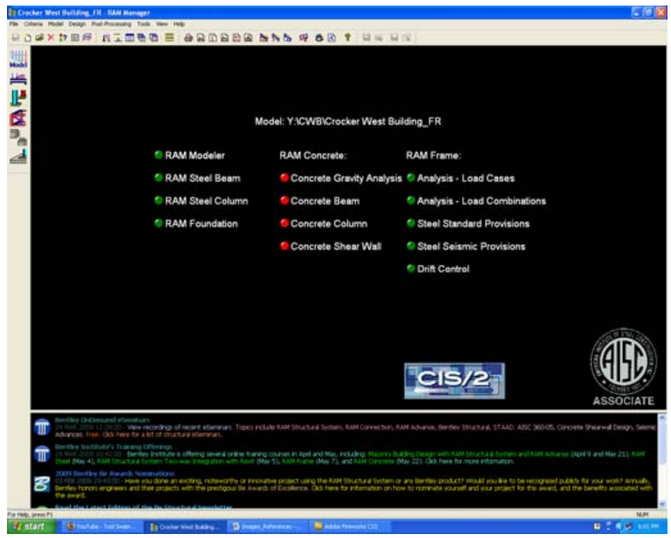

Figure 18: RAM Manager Design Confirmation

RAM CONCLUSION: Using RAM Structural to model, analyze, and then design the proposed composite steel superstructure with concentrically braced frames proved to be profitable for several reasons. The experience gained using a BIM program seems to be more and more viable as practicing engineers and their firms are slowly shifting toward this type of modeling method. Also, the use of RAM Structural System as a design 'aid' lends itself to the possibilities of quickly checking alternative construction systems without the hassle of re-entering all the necessary loads repeatedly. The program allows existing element properties and materials to be changed within minutes and few specified parameters. Overall, the RAM Structural Systems software offers the design professional a powerful, integrated building information modeling program that has the capability of complete building design for limitless possibilities. However, as with any computer based design program, the user of this program should have a fundamental understanding of the specified design input & output parameters assumed and calculated by the 'black box' program methodology.

Design Procedure -

The goal of the lateral system design is to replace the existing APC shear wall lateral system with an efficient design of concentrically braced frames.

GRAVITY: Using the RAM Structural Systems described above, the existing architectural floor plan layout was converted and modeled as a composite steel gravity system. Varying trials of typical bay in-fill beams were analyzed in order to keep the overall floor plenum to a minimum. However, other issues arose with these preliminary design considerations.

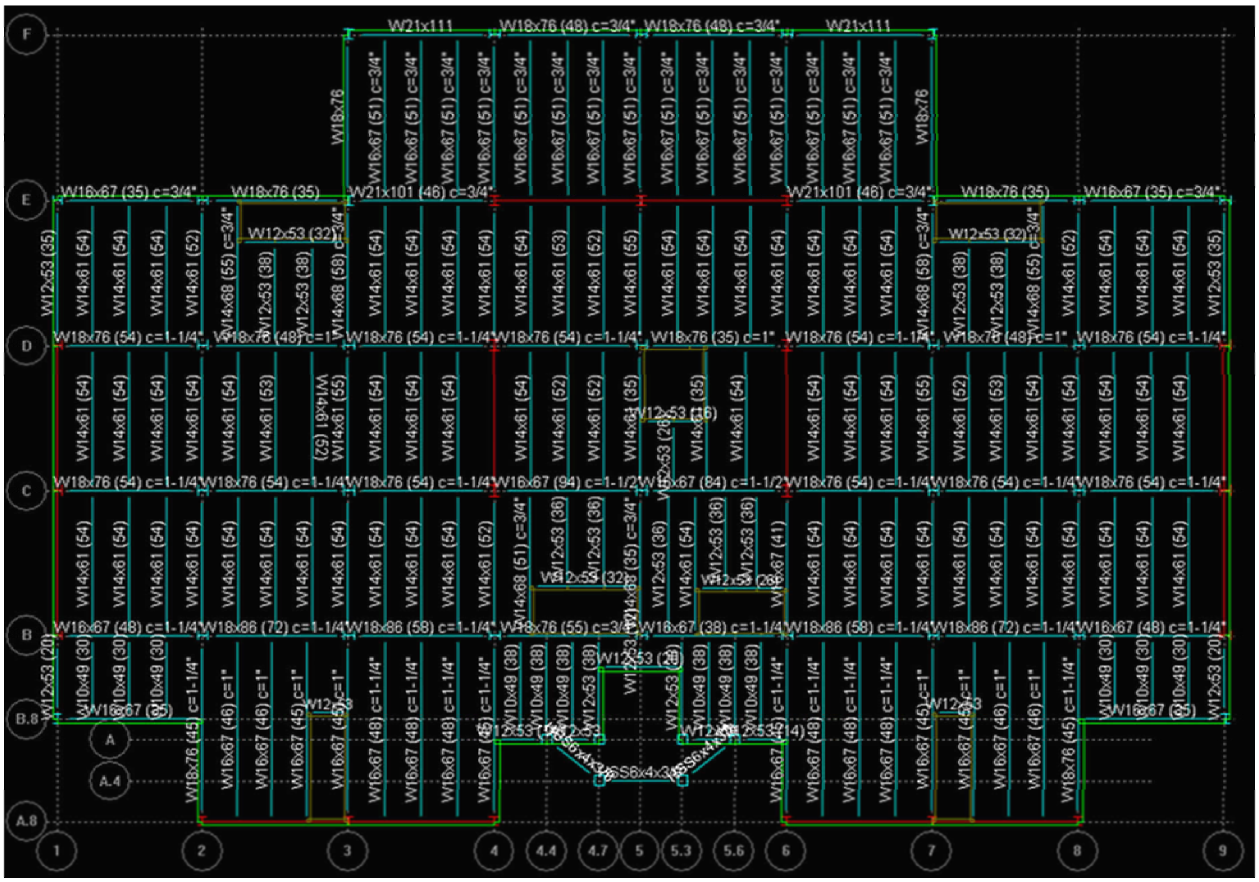

Figure 19: 2nd Floor Plan of Proposed Structure with 4-span In-fill Beam Layout

For instance, based on the same existing architectural floor layout with 3" LWC concrete over 3" USD metal deck, one gravity floor system was modeled with the typical 35' x 35' bay consisting of two (2) in-fill beams (3-span) and another implementing the same size bay with a three (3) in-fill beam (4-span) system. Results from this analysis yielded 124 <u>more</u> steel beams being used in the 4-span system; however the number of shear studs required for composite action decreased by more than 2400 studs. Although 2400 less shear studs would amount to a slight reduction in seismic weight, the major benefits of switching to the 4-span system is a combination of the seismic weight and floor depth of the system. In reference to seismic weight, the 4-span system alleviates the effective seismic weight by over 176,000 lbs. less than the 3-span system. Assuming 10 lbs./stud and combine it with the 176,000 lbs. of steel saved, this particular 4-span system will save the effective seismic weight nearly 20 kips. Considering the seismic weight of the existing APC structure, the proposed steel building was based on 3 additional floor levels and was slightly less than double the original weight. Figure 20 display some of the typical sizes of the redesigned gravity systems. The largest W-shape columns used for gravity loads were a W14 x 176.

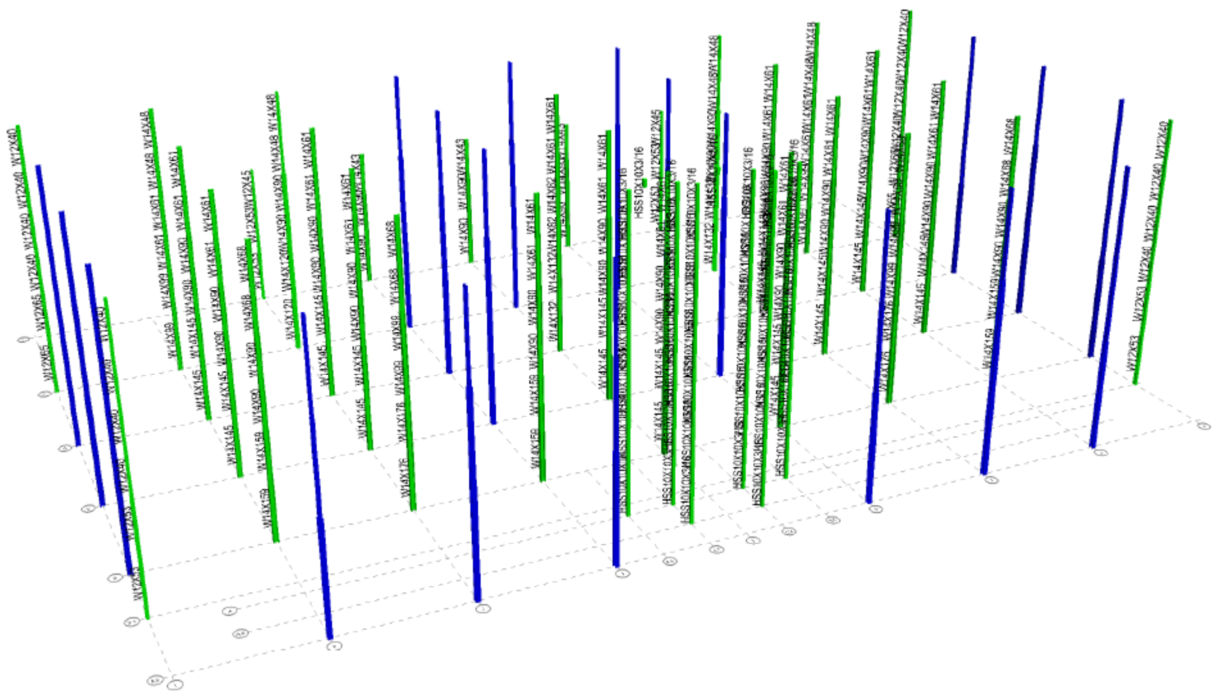

Figure 20: Confirmed Gravity Designed Columns with Sizes

LATERAL: Determining where to place the proposed concentrically braced frame locations in the redesigned structure was also of high priority in the design process. The increased building height will require a close look at the building's lateral system in both directions. Also noted, the architectural layout is to be as undisturbed as possible. Similarly to the APC shear walls used in the existing design, the CBF's were preliminarily placed using RAM Modeler in identical positions to that of the precast shear wall and an analysis was executed. Following the analysis, it was concluded that there was an overabundance of braced frames throughout the structure. This was determined by sifting through the designed member sizes of the analysis and noticing that some of the CBF's were not needed. Several trial and error analyses were then run with several design options, such as Tension-only braced members and varying frame types (i.e. chevron, 'X' brace..) changed and altered. Figures #22 below shows the final CBF number and layout of the proposed 6-story steel building.

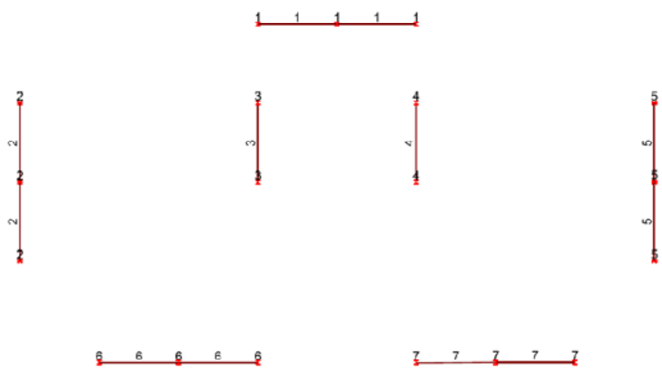

Figure 21: CBF Frame Numbers & Location

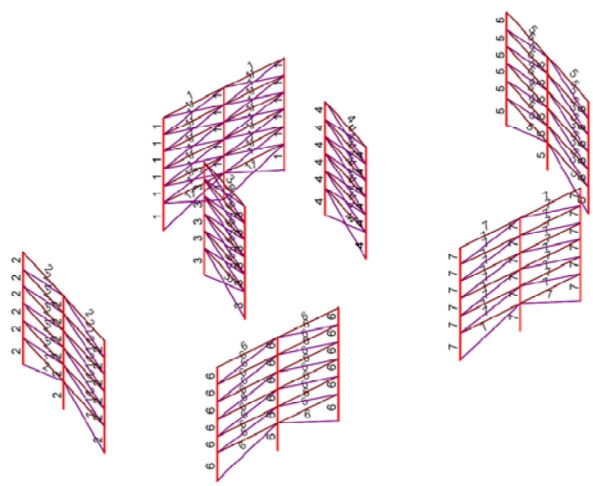

Figure 22: CBF Elevations with Frame Numbers

The design of the system was determined by inputting the design wind and seismic loads calculated via ASCE 7-05 design code into the RAM Frame module. Once this information is entered into the program, RAM analyzed the desired braced frames and their locations via the established design loads and criteria set-forth in the prior modules. RAM first analyzed the structure based on required strength, and then performs several checks based on serviceability before a final strength. The determined locations of the concentrically braced frames in plan seems viable, as the orientation due to symmetry and location will ensure that torsional effects on the building will be kept to a minimum by minimizing the eccentricity induced to the system from applied loading, whether from wind or seismic. Torsion is the result of loading applied to an eccentricity between the centers of mass and rigidity that can result in additional loading on the frame structure. By placing the braced frames symmetrically about the central core of the plan layout, torsion and other serviceability standards such as drift can be limited, reducing overall frame loading and increasing the efficiency of the structure.

STRENGTH & SERVICEABILITY: The first step in checking strength and serviceability was to assign member sizes to the designated frames. RAM requires these members be initially assumed during the RAM Modeler module, as they are not designed during gravity framing analysis. Upon completion of the RAM Beam and Column gravity designs, RAM Frame was then used to analyze the CBF's for strength. Rectangular HSS Structural Tube steel was used as braces. HSS brace strength was based on axial forces only as they were designed as tension-only members, while W-shape beam and columns were analyzed according to the interaction equations for combined axial and flexural loading. These equations, seen below, can be found in Chapter H of the Steel Construction Manual (13th ed.) as they address members subject to axial force and flexure about one or both axes, with or without torsion, and to members subject to torsion only.

Eq. H1-1a: For $\frac{Pr}{Pc} \geq 0.2$ → $$\frac{Pr}{Pc} + \frac{8}{9}\left(\frac{Mrx}{Mcx} + \frac{Mry}{Mcy}\right) \leq 1.0$$

Eq. H1-1b: For $\frac{Pr}{Pc} < 0.2$ → $$\frac{Pr}{2Pc} + \left(\frac{Mrx}{Mcx} + \frac{Mry}{Mcy}\right) \leq 1.0$$

Note: Equation H1-1a controlled most column designs, while equation H1-1b controlled beam design.

After analysis, RAM Frame displays these interaction values for the various members of each frame. These values can be used to determine the adequacy of design. A target interaction value of less than 1.0 for all members is required to provide sufficient strength to the system. Values greater than 1.0 are graphically displayed red in RAM Frame module to alert the user of any deficient designs, while under-stressed member's colors vary from dark blue to orange as the interaction value increases. Over-stressed elements are then properly resized using RAM Frame and the analysis is repeated as needed until all design criteria has been satisfied. As you can see below, the new design of Crocker West satisfies these requirements.

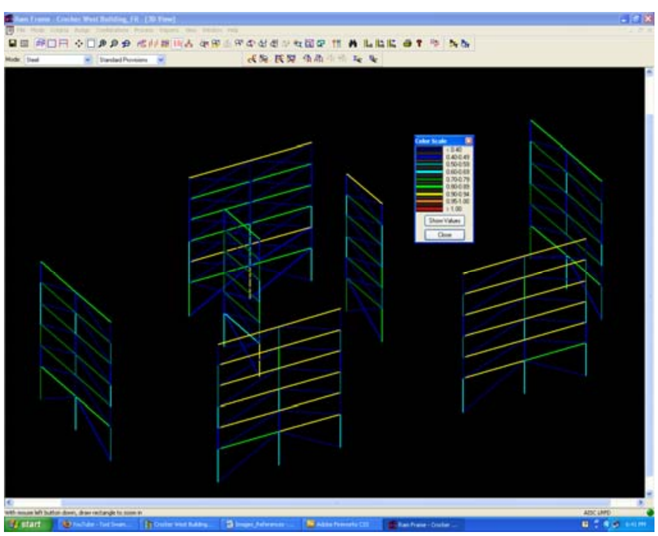

Figure 23: Frame Interaction Values

Serviceability considerations for the new redesigned Crocker West building included torsion, drift and vibration. However, due to the location, use and height of the building and the overall schematic of the symmetrical footprint these issues raised very little concern to the structural integrity of the building. Based on the RAM modeling programs values for center of mass (CoM) and center of rigidity (CoR), a 6% eccentricity was the largest eccentricity posed to the structure. Considering design criteria for eccentricity run within the module was set to 15% for accidental torsion as a conservative assumption, this resultant load due to the 6% eccentricity can be considered insignificant. Although drift was considered in the analysis of the new structure, it proved to not be of major significance to the steel superstructure. Story displacements from the analysis of each floor were checked against the acceptable limit of H/400. The calculated allowable building drift for the proposed 6-story, 79' mean roof height Crocker West Building was 2.37". Displacement reports from the RAM model displayed maximum displacements of close to 1.25" which is considerably less than the allowable. Allowable story drift limits are were computed based on ASCE 7-05, Table 12.12-1 using Occupancy Category II for 'All other structures'.

$$\Delta_{max} = 0.020 \text{ x } h_{sx} \qquad \text{where: } h_{SX} \text{ is the story height below Level x.}$$

Allowable story drift limits for a typical 12' floor-to-floor height for floors 3 thru 6 were calculated to be Δ = 2.88", while Δ = 4.56" for the 19' floor-to-floor height at ground level. Analyzed story drift values computed for the entire building were all found to be less that 1", thus proving drift is not a major issue in the redesign process.

SUMMARY: The overall redesign of the lateral force resisting system proved to be efficient based on the predetermined criteria and program requirements. Orientation of the introduced braced frames did not pose any threat in forcing a new architectural layout. The existing plan was maintained by placing 5 of the 7 CBF's around the perimeter of the building and implementing the other two frames to form a lateral core of supports. As discussed, reducing the total seismic weight, while reducing floor plenum depth, was a main goal of the study. Typical bay member sizes for gravity loads are shown in Figure 24, while Figures 25-28 depict frame layouts and sizes. Although vibration was not checked against actual values, it was assumed to be satisfactory due to the addition of the 4th in-fill gravity beams.

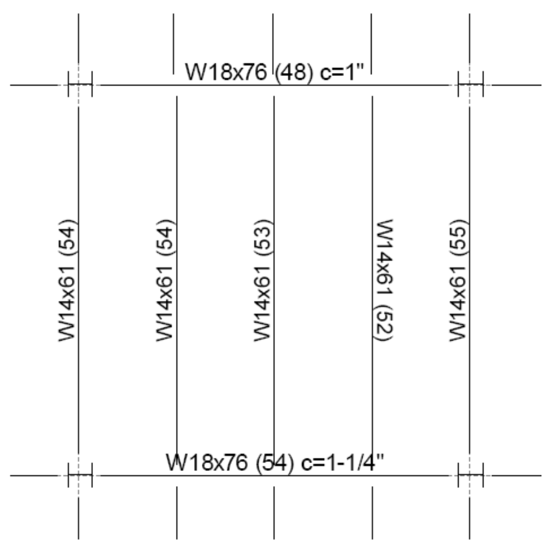

Figure 24: Typical Bay Gravity Members

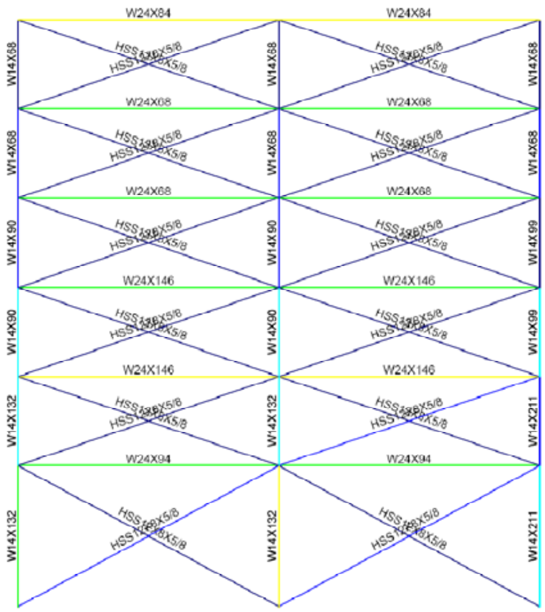

Figure 25: CBF Frame #1 ('X' Brace)

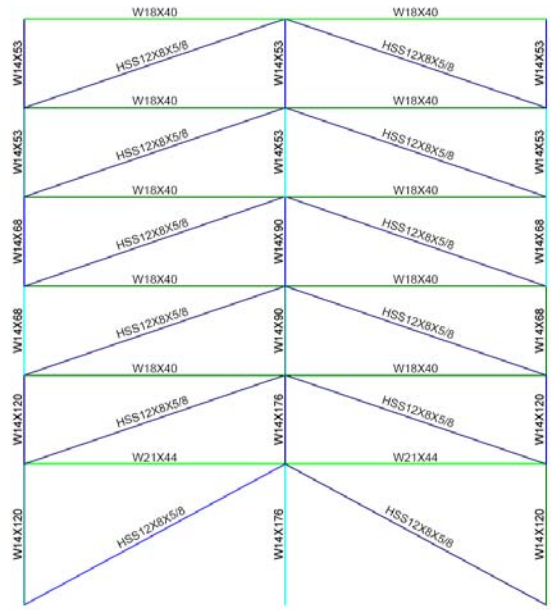

Figure 26: CBF Frame #2 & #5

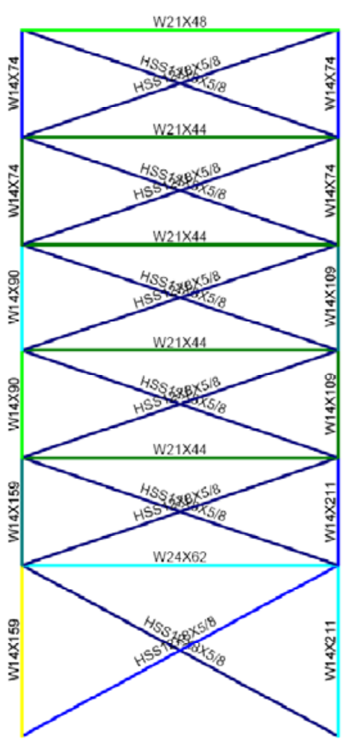

Figure 27: CBF Frame #3 & #4 ('X' Brace)

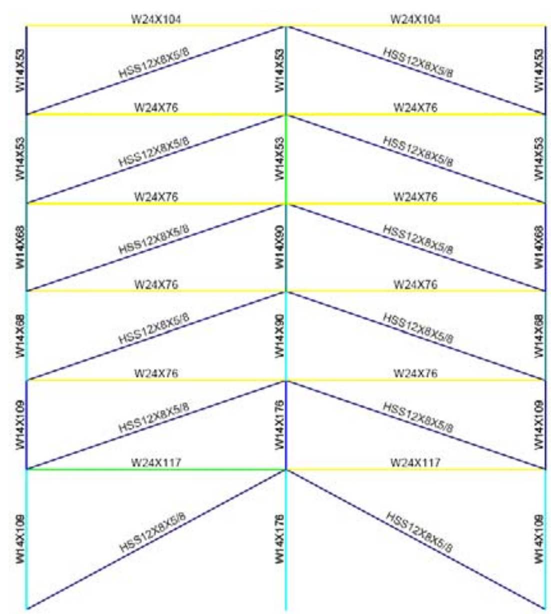

Figure 28: CBF Frame #6 & #7

Architectural Breadth Study:

The goal of this architectural study is to avoid a major overhaul to the existing program requirements while introducing a dissimilar style of architecture to the surrounding area.

INTRODUCTION: Negotiating height for aesthetics creates several areas of concern. Based on an analysis of the tenant's future needs, they have requested the addition of several levels of office space. The use of precast for the entire building envelope could prove to be insufficient and uneconomical. Also, a conversion of the existing precast concrete system to a concentrically braced steel framing system (CBF) could also introduce architectural issues such as grid layout, core location, and façade aesthetics.

Figure 29: Rendering of Exposed CBFrames on West Wall Elevation

Based on this hypothetical analysis of the tenants needs, a petition will assumed to be deemed granted for the addition of several stories needed for office space. The acquired permission was issued with the understanding and intent that the redesign takes on a more modern, architectural appeal than the existing precast envelope. Also, the traditional form of brick veneer construction used near the project site and surrounding area shall be avoided.

The change induced on the exterior of the edifice due to the redesigned steel framing and glass curtain façade will be analyzed and incorporated in the appearance overhaul. For this reason, an architectural study was conducted in order to allocate the proposed redesign. This study included an investigation of the architecture in the surrounding area and ramifications the vertical expansion poses to the structure and program layout.

SCHEMATIC: Research of the surrounding area provided insightful detail on the type of construction material used in the region around the project site. It was found that brick veneer was the most typical style of architectural façade used, with the exception of a few buildings. Using steel braced frames as the main lateral system did not completely alter the existing floor plan, thus maintaining the overall architectural layout and vision of the structure. The majority of the CBF's were placed around the perimeter of the redesigned Crocker West Building optimizing the structural integrity of the building without hindering program layout. The braced frames were also worked into the façade aesthetics to render the proposed building in a more modern style of architecture. Switching from the existing APC framing to steel framing will also benefit the architectural aspect of CWB. The new design will employ an aluminum metal rain-screen panel system with dark-blue reflective glazing panels and glass curtain walls. The glass was chosen based on the level of security the tenant requires. In any case, window treatments will be conducted by the tenant as they see fit.

MODELING: As mentioned, the redesign of the Crocker West Building will include the addition of several levels of office space for a total of 6-stories. The vertical expansion of the 3-stories will be modeled using another type of building information modeling (BIM) program. The Autodesk Revit Architecture (ARA) developed software is an integrated BIM program that provides to support sustainable design, construction planning and fabrication. Revit allows architects and design engineers to remain coordinated thru all phases of design due to their parametric change technology, any change made is automatically updated throughout the project.

ARA was used in this report to construct a visual representation of the architectural, vertical expansion of Crocker West. The existing grid lines used on the project, along with any newly established grids from RAM were laid out and structural columns and beams were then placed accordingly in conjunction with the new floor plan designed in RAM. These members were modeled using the exact section properties as the designed RAM members to provide the closest visual scale of the model. Upon complete modeling of the substructure, the exterior façade of the new building was then created using an array of different wall components. Figure 29 is a rendering created using ARA to demonstrate the types and locations of the new construction materials.

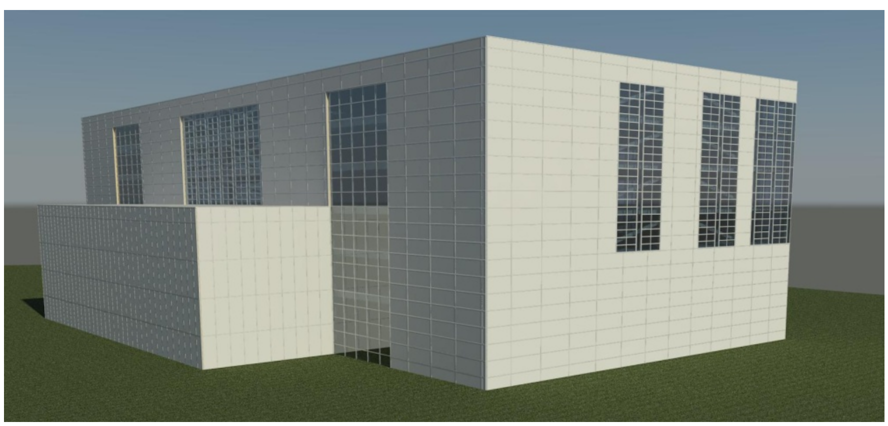

Figure 30: 3-D Rendering of Proposed Revit Structure (Rear)

IMPLICATIONS: The new façade design of the Crocker West Building includes replacing the existing Architectural Precast Concrete wall panels that contain a traditional brick in-lay pattern representational of brick veneer. These wall panels will be replaced with modern steel framed construction, including concentrically braced frames and curtain wall systems scattered amongst the floor plan. The introduced metal panel rain-screen system will be secured to a proper metal stud wall construction assembly similar to Figure 30 below. Citadel Architectural Products was used as a basis for design guidelines and requirements. Panels were modeled in Revit using 4' x 10' metal panels, however the dimensions of the glass were merely roughed in until the author was pleased with the design.

As can be seen in the Depth Study portion of this report, 5 of the 7 concentrically braced frames were positioned around the exterior of the building to provide adequate structural resistance to the applied loads. In addition to resisting the applied lateral loads exerted on the building, these perimeter frames will also be required to carry the 'hanging' curtain wall gravity loads resulting from the new exterior façade. As previously established this load was assumed, however applied during redesign of the new structure in RAM SS. The remaining two concentrically braced frames were located in the interior of the building near the symmetrical center along grid lines 4 & 6, between grids C and D. These are the only two braced frames that could possibly interrupt the architectural layout based on their conventional 'X'-bracing. A possible solution to this problem would be to utilize a special braced frame in these locations to maximize the open area under the intersection of the links.

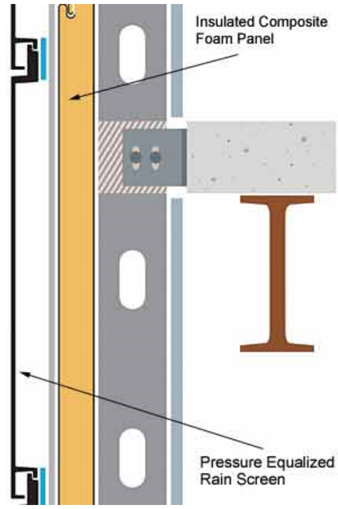

Figure 31: Superior Wall System Section

Along with the vertical growth of the building from this architectural change comes increased design pressures and applied loads. The increase in height resulted in higher wind design pressures than the existing analysis, thus leading to the applied wind forces increasing. This increases the amount of strength the frames and gravity members are required to resist, resulting in larger size members.

SUMMARY: The purpose of the architectural breadth was to create a design that met the program requirement while dealing with several other design issues. The schematic design successfully provided the programmatic requirements of increased square footage and introduced a fairly modern architectural style into the area with an unconventional façade material. The overall existing floor plan may have slight altercations near Frames 3 & 4; however, due to security issues these areas are not specifically detailed and were assumed to be sufficient for the purpose of this design.


Figure 32: Rendering of Proposed CWB Front Elevation

Construction Management Breadth Study:

The goal of this CM study is to compare the relative cost and schedule of the existing project to that of the proposed structure.

INTRODUCTION: Transforming the existing structural system from architectural precast concrete to a modern steel framed design, plus the accumulation of construction materials for the additional levels will lend itself to an assortment of construction management issues. A project schedule shall be assessed based on the new steel system and related materials; also, a relative cost shall be determined from the new specified material list. Information gathered regarding each systems schedule & cost impact shall be compared and used to recommend one system over the other.

Construction management is becoming more and more popular these days due to the instability of the economy. The design of a lateral system can prove to be very bi-costly by means of construction type, material, and speed of erection. A structural superstructure that can be assembled quickly may not be feasible for design based on loading conditions. Over-designing the system for forces it will never experience is also unconventional. As design engineer, one should always be as resourceful and feasible as possible, making the proper assumptions in the design process by utilizing the strength of materials used.

The goal of this study is to compare the costs and construction times of the original superstructure and the proposed changes to the structure. An attempt will be made to evaluate the systems in the most similar fashion as possible.

EXISTING SYSTEM: Architectural Precast Concrete (APC) was selected as the primary building material for the Crocker West Building for many practical reasons. It provides a myriad of life-cycle and ancillary benefits that are difficult to match with other materials. Speed of erection was one of these factors as APC's speed of assembly and its ability to be cast and erected in all kinds of weather aid the entire construction team. Since the casting process does not rely on other critical path activities to begin, units can be produced as soon as construction documents are approved. As a single unit, precast panels provide one source for supplying the entire exterior wall system. When load-bearing precast structural floors along with panels are specified, it concentrates the complete shell of the building with one certified precaster responsible for the manufacturing and constructability issues. This reduces the number of subcontractors and minimizes trade coordination for the project.

The Crocker West project consisted of nearly 1,200 pieces of precast, with close to 800 of those being the hollow-core plank. Discussing the project with the owner, it was observed that the total price for the precast only is estimated to be around $3.95 million. This price is based on the precast only and does not include cost and time of erection.

PROPOSED SYSTEM: The newly designed steel system is also considerably fast and easy in terms of erection and is probably the most comparable to precast. Similar to APC, steel requires a significant lead time for shop drawings and fabrication of the members. The composite steel framing layout designed in RAM SS required an estimated 4,800 steel components and nearly 82,000 s.f. of exterior façade. Lead time can become more arduous if any of the steel elements have to 'built-up' or manufactured in special plants located outside the U.S. Appreciatively, none of the members designed for this thesis report were required to sustain that amount of loading.

Braced frames also contribute to the ease and cost of steel framing erection. Compared to moment frame connections which are extremely costly and labor intensive, braced frame connections are noticeably simpler to install and less expensive due to the lack of welding labor required. The CBF connections may require the use of gusset plates that are usually attached in the shop. The diagonal braced member is then framed into and attached to the gusset with a series of bolts. Braced frame connections should be designed in accordance with Part 13 of AISC's Steel Construction Manual.

Cost comparison between the existing structure and the one proposed in this report proved to be difficult to contrast due to the additional 3-stories of office space. Using RSMeans Assemblies Cost Data, an estimated cost consisting only of the steel superstructure was assembled based on square footages in order to try and assess the two systems. Using RSMeans>Superstructures>A3.5-530>W Shape, Composite Deck, & Slab for the cost analysis, a 35' x 35' bay size of beams and girders resisting a 200 psf superimposed load was assumed and priced as follows:

COST PER S.F.		
MATERIAL	INSTALLATION	TOTAL
$16.30	$9.35	$25.65

For comparison purposes, assume the square footage of the building was exactly doubled with the vertical expansion, thus equaling 242,000 s.f. of rentable space.

242,000 sf x $25.65 = $6,207,300

$$\frac{\$6,207,300}{2} = \$3,103,650$$

In order to try and compare the new superstructure to that of the precast superstructure, the estimated price was divided in half to assume a price for a 3-story steel framed building.

SUMMARY: The economy of a steel frame depends largely upon the difficulty of the fabrication and erection. Connections for steel framing seem to be one of the largest contributors when considering the type of lateral framing system to use. As stated, braced frame connections are considerably less expensive and much easier to connect due to the use of high strength bolts and gusset plates. Eliminating the welding labor involved with moment connections will not only decrease cost, but will also prove beneficial to erection time. Assuming the preceding cost analysis was assessed properly, the prices calculated for the existing system and proposed systems are relatively close. However, without conducting a full-scale cost and schedule analysis including ALL the relative building materials needed for the project an exact comparison is inaccessible. Ultimately, the cost of the steel structure would more than like end up being more expensive due to the required number of steel members and façade area omitted from this analysis, and the construction schedule would be very stringent in order to complete this project in the 13 month time frame.

Conclusion:

The main goal of this senior thesis report was to efficiently design an alternate framing system capable of withstanding the increased applied load caused by the addition of 3-stories of office space requested. After careful deliberation, the proposed alternate composite steel system for the Crocker West Building proved to be a viable solution to the existing APC superstructure. The new system was modeled, analyzed, and designed with adequate strength capacity to resist the prescribed gravity and lateral loads determined in this report. This model was edited for a number of strength and serviceability requirements and was found to be in compliance with all major codes and strength checks.

Had the existing 3-story structure actually been constructed using composite steel framing with CBF's, the effective seismic weight of the structure would have been greatly reduced. Lowering the seismic weight can be very cost effective on several accounts. A lower effective seismic weight results in a lower base shear, effectively decreasing the amount story forces applied to the structure. The reduction in weight would also facilitate changes to the foundation system; less weight means less gravity loads required to be resisted, resulting in smaller spread footings beneath the columns. Depending on the manner the two systems are compared, the foundation system would need closer evaluation of the 6-story building in order to determine the ramification it would render to the existing system. The 6-story building designed was determined to be nearly 10,000 kips heavier than the existing 3-story structure. Conservatively dividing and distributing this additional weight amongst the 44 columns included in the main body of the structure results in an estimated supplementary axial load of 230 kips per column. This can be assumed to induce a substantial impact upon the existing foundations; however, a detailed analysis based on soil bearing capacity and other site specific information should be conducted to make an educational decision. Now if the comparison of the two framing systems were to be based on the 3-story structure, the existing foundation system would prove sufficient and possibly reducible based on the assumption the seismic weight was lowered.

Architectural issues related with the redesign of the structural system were kept to a minimum by replicating the existing shear wall locations and removing frames as they were deemed unnecessary from analysis. The minor discrepancy encountered with the two interior braced frames is not enough to discard this alternative lateral system. Further research and investigation of design criteria is required for impacts the proposed modified framing system would pose on other trade systems (mechanical, electrical …etc.). Although unmentioned in this report, these systems must be taken into consideration when dealing with the optimization of a new facility.

In the end, both of the systems discussed throughout this report have proven to be reliably sustainable for the examined designs and redesigns of the Crocker West Building. Recommending one system as opposed to the other seems to be justified better by associating what type of taste and style the owner has and his functional vision of the design. For example, in the case of the Crocker West

Building, the owner is also the structural engineer of the project who just so happens to specialize in the design of Architectural Precast Concrete!

Figure 33: Final Perspective Rendering of CWB with Proposed Composite Steel Framing and Concentrically Braced Frames

**On a side note, the redesign of the lateral system to steel was chosen by the author of this report to gain a better understanding and personal practice experience in the design of a main force resisting system composed of steel. Many design assumptions were both corrected and confirmed during this design process. In addition, the goal of learning an abundance of awareness proved to be successful, making this a positive experience.