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Appendix A: Lessons Learned

Many things were learned by the structural team at Synthesis, during the design of Growing Power's Vertical Farm. Throughout the whole design process, these lessons have aided in the design of an efficient and economical building as a whole. The lessons learned while working with the team at Synthesis, will continue to provide benefit to the structural team as they begin their professional careers. Following are a few of the major lessons learned during the design process.

1. Grid Layout:
   Early in design, it was discovered that by developing a rational grid system was going to be extremely beneficial. Modeling, analysis, and drafting all could be made easier with by producing an early logical structural grid. By sticking to the conventional grid for all programs, the structure could easily be organized to eliminate all differences between these platforms. Development of a grid early that stayed constant also helped diminish confusion by team members by keeping these references similar.
2. BIM Software
   BIM technology is extremely helpful in collaborative work to create graphical representations and exchange information between disciplines. Detailed graphics that can be created through BIM technologies can help develop an idea of concept by seeing how it interacts with all other systems in the building. Information exchanges that happen through BIM technologies, allows all disciplines to see the other systems that are being implemented in the building and there location. These exchanges eliminate clashes between all disciplines earlier in the design process.
3. Industry Professionals:
   It was found throughout the design process that industry professionals are willing to extend their extended amount of knowledge and information with young engineers. These conversations helped expand the structural teams' knowledge and develop reasonable assumptions when needed. These industry professional can also make great business connections as all members at Synthesis begin their young careers in all of the disciplines involved in Architectural Engineering.
4. Organization:
   With design being a continual process, it is important to be able to quickly find and access previous calculations and information. By using organizational strategies with models, documents, spreadsheets, etc, an efficient and effective file structure can save time and confusion during the iterative design process. Also, organized files made it easier for other disciplines to access any files or information used by all other disciplines especially in integrated design.
5. Communication:
   Communication between all members working on an interdisciplinary project is a necessity to ensure that the project runs as smooth as possible. It is important to keep all members updated as to what is happening with your specific system so that no surprises come up during the design process. It is also important to note that all members within one team do not have expertise in the same field's, therefore, it is important to explain information in terms that everybody can understand and interpret what is going on. Without regularly scheduled meetings, information can be overlooked between team members and changes in design may need to occur later during the design schedule.
6. Computer Modeling Software
   While design can be made easier by employing structural analysis and design software, it is also extremely important that these programs should be used with discretion. All computer output should be interpreted and verified with manual spot checks to develop areas where error occurred. Even when some models are done incorrectly, they will still produce results that if blindly followed, could be catastrophic. Therefore, all members involved in computer modeling should be knowledgeable and should communicate with other modelers on a regular basis.
7. Open to New Designs
   It was found that being open to innovation and changes in design is extremely important for a team to function properly. When working on an integrated project, all members need to keep in mind that what is best for them may greatly hurt the other design teams involved. Therefore, whenever a new system is brought up, all members need to be objective and look at the building as a whole. A specific system may be hurt by the implementation of a design, but if it makes the building function better as a whole then the team that is hurt needs to be willing to take the hit and implement the system to create the best building possible.
8. Small Changes Effect Everybody
   Throughout the entire design process, it was found that no matter how small a change may seem to an individual option, it may have a huge effect on other options. Therefore, all members should be informed about any changes in a system no matter how small they are. By telling all involved design teams the changes that occur in a specific design, unforeseen clashes and problems can be kept to a minimum. Also, it is of utmost importance to put all design changes into Revit as soon as possible, so that everyone can directly see how the new systems will coordinate with one another.

Appendix B: Applicable Codes, Standards & Software

Codes and Standards:

- International Code Council (ICC). *International Building Code.* International Code Council, Falls Church, Va. (2009).
- Milwaukee Code
- Miami Code
- American Institute of Steel Construction (AISC). *Steel Construction Manual.* 14th Edition. (2011).
- American Concrete Institute (ACI). "Building Code Requirements for Structural Concrete and Commentary." *ACI Standard 318-11.* (2011).
- American Society of Civil Engineers (ASCE). "Minimum Design Loads for Buildings and Other Structures." *ASCE/SEI Standard 7-05.* (2005).
- Masonry Standards Joint Committee (MSJC). "Building Code Requirements and Specifications for Masonry Structures." *The Masonry Society (TMS) Standard 402-08.* (2008).
- Concrete Reinforcing Steel Institute (CRSI). "CRSI Design Handbook, 2008. *Concrete Reinforcing Steel Institute.* (2008).
- Design Guide 11. "Floor Vibrations Due to Human Activity." *American Institute of Steel Construction.* (1997).

BIM and Structural Analysis Software:

- "Autodesk Revit 2015." Autodesk. (2015).
- "ETABS 2013 Ultimate." Computers and Structures, Inc. (2013)
- "RAM Structural System." Bentley Engineering. (2012).
- "RISA-2D Educational." RISA Technologies. (2002).
- "STAAD.Pro." Bentley Engineering. (2012).

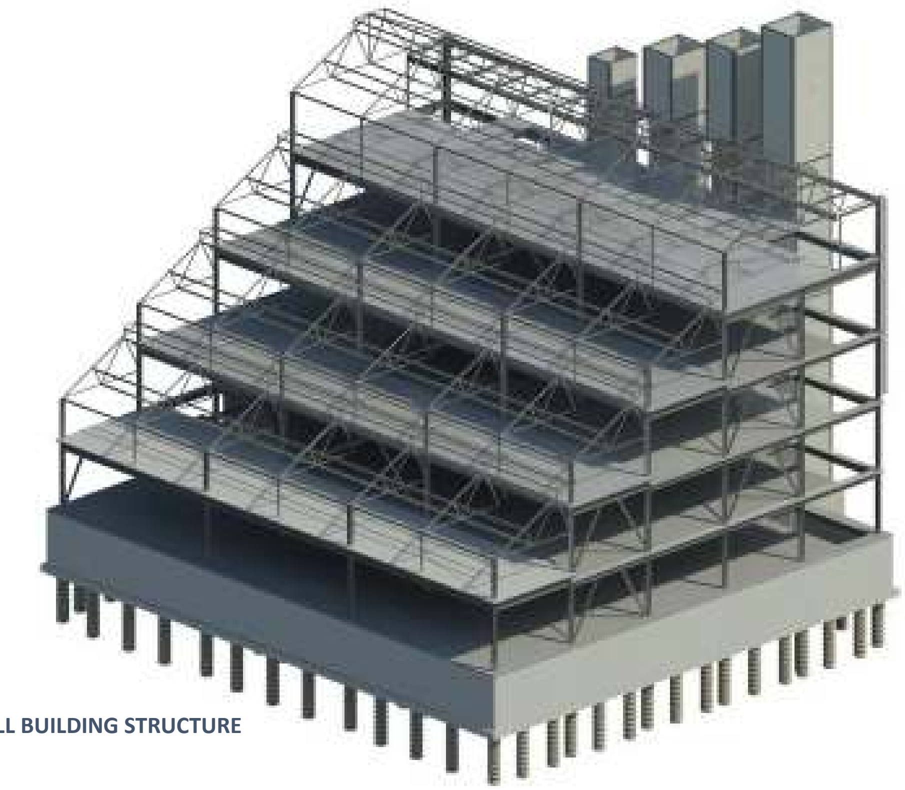

FIGURE S. 1: OVERALL BUILDING STRUCTURE

Appendix C: Design Loads, Parameters, & Analysis

TABLE S.1: FLOOR DEAD LOADS

Typical Floor Dead Loads		
Type:	**Notes:**	**Value:**
Decking	3.5" of Lightweight Concrete on Composite Metal Deck: Vulcraft 2VLI18	42 lb/ft^2
Self-Weight	Beam and Girder Allowance	7 lb/ft^2
Mechanical	Superimposed	5 lb/ft^2
Lighting/Electrical	Superimposed	5 lb/ft^2
Floor Allowance/Raised Floor System	Superimposed	8 lb/ft^2
	Total	67 lb/ft^2

TABLE S.2: FLOOR LIVE LOADS

Typical Floor Live Loads		
Type:	**Notes:**	**Value:**
Office + Partitions	Used in all areas for adaptability. Corridor above first floor is 80 lb/ft^2, the same as an office with movable partitions	80 lb/ft^2
Assembly	Used an areas with movable seating, such as the gathering and break out space	100 lb/ft^2
Stairways		100 lb/ft^2
Greenhouse	To allow for the addition of and movement of aquaponic tanks	150 lb/ft^2

TABLE S.3: FAÇADE DEAD LOADS

Façade Dead Loads		
Type:	**Notes:**	**Value:**
Architectural Precast Panel	Line load on exterior beams where precast panels are present, using 6" of concrete and 3" of insulation	1,000 lb/ft

TABLE S.4: DEAD LOAD IN THE GREENHOUSES

Dead Loads in Greenhouses		
Type:	**Notes:**	**Value:**
Greenhouse Equipment	Due to the unknown nature of what equipment is in greenhouse this load was taken from a precedent	150 lb/ft^2

Lateral Loading:

In the beginning of design, it was important for the structural team to develop preliminary lateral loads that would be experienced by the building. Preliminary wind loads were developed using Main Wind Force Resisting System (MWFRS). All parameters used during design for the Milwaukee and Miami locations can be seen below in Table S.5 and S.6.

TABLE S.5: MILWAUKEE WIND PARAMETERS

Milwaukee Wind Parameters	Value
Risk Category	III
Importance Factor	1.15
Basic Wind Speed, V	90 mph
Wind Directionality Factor, K_d	0.85
Exposure Category	C
Velocity Pressure Coefficient, K_z (At top roof level)	1.20
Topographic Factor, K_{za}	1.0
Gust Effect Factor	0.85
Enclosure Classification	Enclosed
Internal Pressure Coefficient, GCpi	± 0.18

TABLE S.6: MIAMI WIND PARAMETERS

Miami Wind Parameters	Value
Risk Category	III
Importance Factor	1.15
Basic Wind Speed, V	115 mph
Wind Directionality Factor, K_d	.85
Exposure Category	C
Velocity Pressure Coefficient, K_z (At top roof level)	1.20
Topographic Factor, K_{za}	1.0
Gust Effect Factor	.85
Enclosure Classification	Enclosed
Internal Pressure Coefficient, GCpi	±0.18

FIGURE S.2: WIND LOADING PRESSURES FOR MILWAUKEE

Seismic loads were also developed for the Vertical Farm using the Equivalent Lateral Force (ELF) Procedure. Parameters used to determine seismic loads on the building in Milwaukee and Miami are presented below in Table S.7 and S.8.

TABLE S.7: MILWAUKEE SEISMIC PARAMETERS

Milwaukee Seismic Parameters	Value
R	7
S_s	0.107
S_1	0.045
S_{ms}	0.168
S_{m1}	0.105
S_{ds}	0.179
S_{d1}	0.104
T_L	12
T_s	0.581
I_e	1.25
Seismic Design Category	B

TABLE S.8: MIAMI SEISMIC PARAMETERS

Miami Seismic Parameters	Value
R	3.5
S_s	.051 g
S_1	.019 g
S_{ms}	.082 g
S_{m1}	.046 g
S_{ds}	.055 g
S_{d1}	.072 g
T_L	8 s
T_s	.433
I_e	1.25
Seismic Design Category	B

FIGURE S.3: SEISMIC FORCES FOR MILWAUKEE

After developing the loads that the building would experience with wind and seismic loading, it was found that wind loads controlled for both the Milwaukee and Miami sites. The comparison of lateral loads can be seen in Table S.9 and S.10.

TABLE S.9: COMPARISON OF LATERL LOADS IN MILWAUKEE

Comparison of Lateral Loads (Milwaukee)			
Level	Wind(N-S)	Wind(E-W)	Seismic
1	26.86 kips	22.70 kips	0 kips
2	49.26 kips	24.88 kips	19.61 kips
3	51.77 kips	21.73 kips	26.09 kips
4	54.26 kips	18.30 kips	28.43 kips
5	33.19 kips	11.26 kips	21.75 kips
Top Greenhouse	28.81 kips	9.78 kips	0 kips
Overturn Moment	14,038 k-ft	6,254 k-ft	6,596 k-ft
Base Shear	244 kips	109 kips	95.9 kips

TABLE S.10: COMPARISON OF LATERAL LOADS IN MIAMI

Comparison of Loads (Miami)			
Level	Wind(N-S)	Wind(E-W)	Seismic
1	32.34 kips	27.43 kips	0 kips
2	99.16 kips	50.48 kips	12.27 kips
3	103.8 kips	43.67 kips	20.7 kips
4	108.9 kips	36.79 kips	24.86 kips
5	66.50 kips	22.59 kips	20.10 kips
Top Greenhouse	31.68 kips	10.78 kips	0 kips
Overturn Moment	25,440 k-ft	10,967 k-ft	5,200 k-ft
Base Shear	442 kips	192 kips	82.4 kips

Appendix D: Greenhouse Methodology, Calculations, and Description

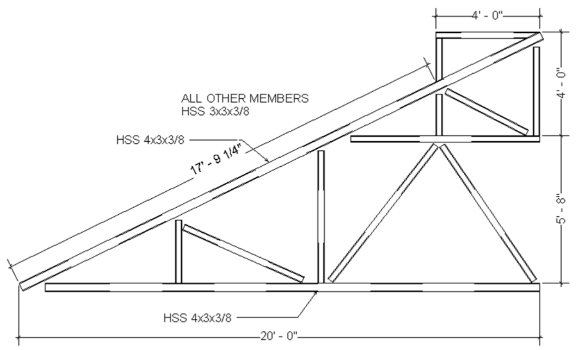

FIGURE S.4: Truss 1

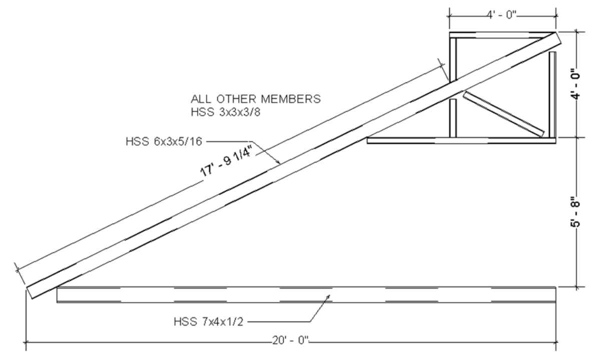

FIGURE S.5: Truss 2

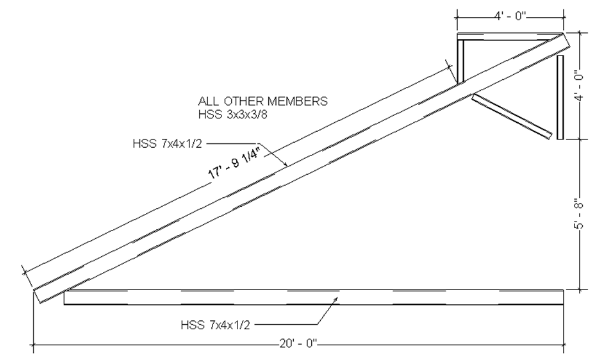

FIGURE S.6: Truss 3

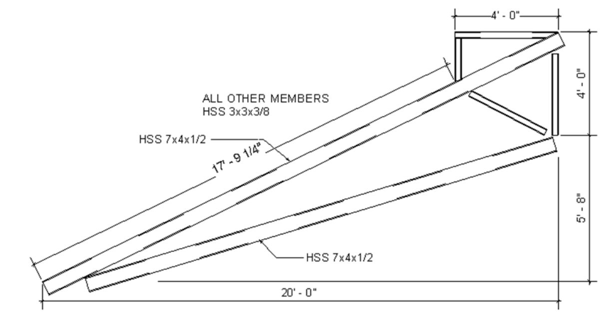

FIGURE S.7: Truss 4

Truss Design Process

The Greenhouse modules were an interdisciplinary collaboration between all members of the Synthesis team. The focus of the design was to create an effective greenhouse that optimized plant growth and represented an efficient and effective design in every aspect. For more on the collaboration involved in designing the modules, see section 2.0 in the **[Integration Report].**

The original greenhouses utilized a precast frame system, which produced large beam and column sections that imposed large shadows in the greenhouses, decreasing the effectiveness of the module. When the building super structure was reconfigured to a composite steel system, this naturally carried over to the greenhouses. The steel allowed for smaller sections and larger spans, decreasing the amount of shadows cast on the growing space. With the formulation of a mono-sloped greenhouse, a truss/ rafter system was a natural fit for the roof system. A spacing of 19'-2" was set for the modules to correspond with the structural bays of the building. This allows for flexibility in the prototype aspect of the building. If the prototype design were to be used on a more constrained site, a structural bay could be added/removed, and the corresponding greenhouse modules with it, eliminating the need to redesign the structural system for the greenhouse modules.

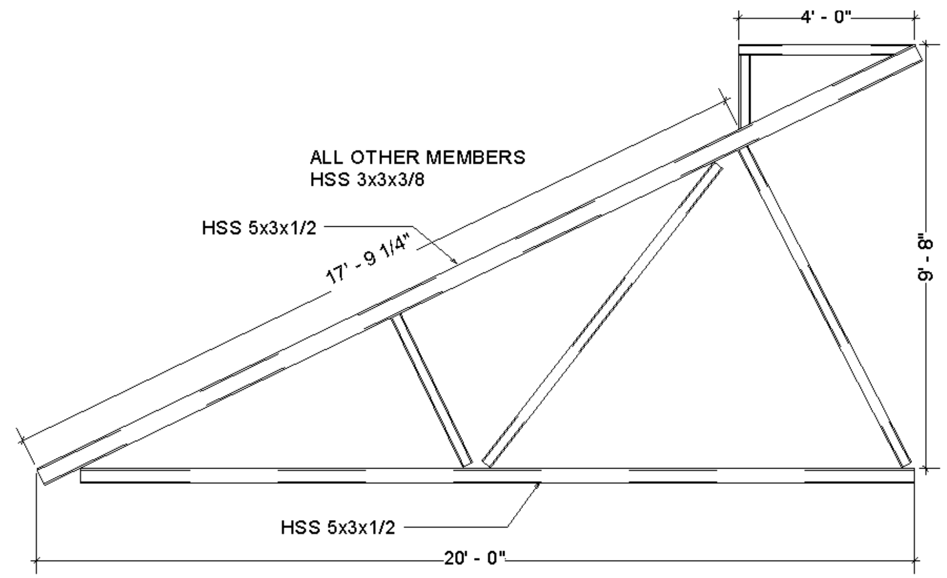

FIGURE S.8: Truss 5, Selected Truss

The structural team and the lighting team worked together to come up with an effective truss configuration that succeeded at supporting the roof glazing system, while keeping structural shadows to a minimum. With the adoption of a closed loop mechanical system for the modules, a horizontal divider was required at 8'-0" to allow the system to function properly. For more information on the closed loop system see section 8.0 in the **[Mechanical Report]**. This required a bottom chord to span 20'-0" across the greenhouses.

With the addition of the bottom chord, a truss design became the clear choice because it allowed for a larger span with smaller members that a rafter system. The final truss configuration see Figure S.8 was decided upon after a cost analysis of all possible configuration was completed, producing a design that satisfied lighting, mechanical, and structural requirements set forth, as well as being a cost effective solution.

TABLE S.11: COST COMPARISON OF THE TRUSS DESIGNS

Cost Comparison of the Truss Designs		
Truss Number	Max Member Size	Total Cost
1	4x3x3/8"	$3,766
2	7x4x1/2"	$2,214
3	7x4x1/2"	$1,164
4	7x4x1/2"	$1,198
5	5x3x1/2"	$2,452

*Truss design number 5 was selected even though it costs more because it blocked significantly less light than truss designs 2, 3, and 4

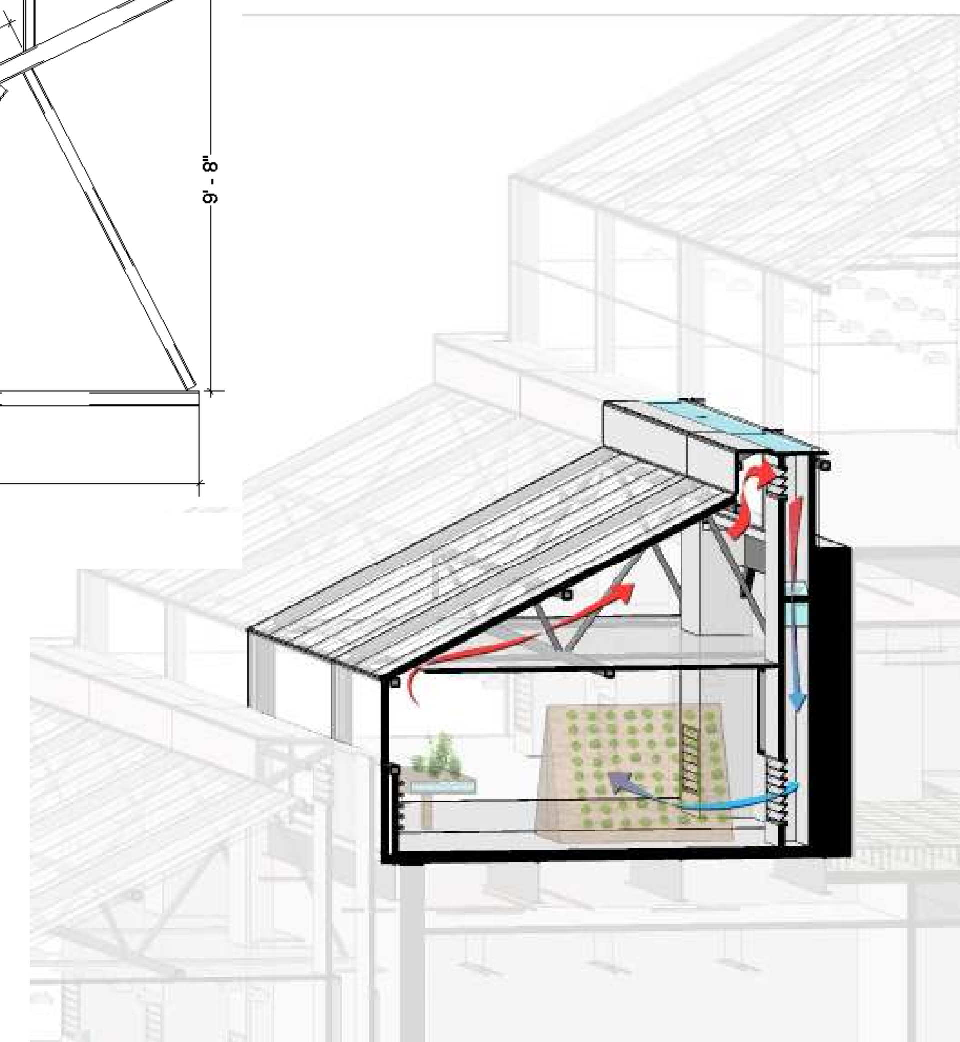

FIGURE S.9: Greenhouse Module

Appendix E: Building Enclosure Methodology, Calculations, and Description

When forming the façade for this Vertical Farm, one of the main concerns of the structural team was with moving the building to hurricane prone regions such as the proposed building site of Miami. In many of these regions, local codes state that buildings must be designed for the case that an object is picked up by the wind and turned into a missile. Therefore, developing a façade system capable of withstanding these impact forces was a necessity when designing the façade system for Growing Power. It was also important for the building to use a façade system that is easily removed and replaced in the event that it is damaged. Since the building is separated into the non-greenhouse regions and the greenhouse regions, two different façade systems were deployed on the building. The main building's façade system consisted of an architectural precast panel system, while the greenhouses consisted of a polycarbonate glazing system.

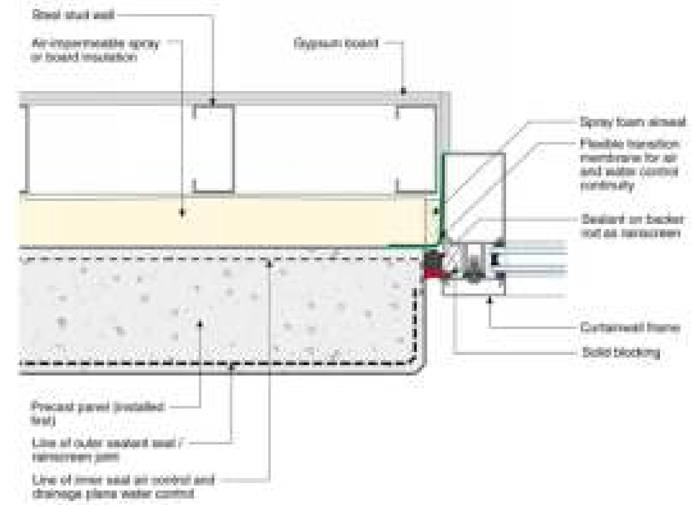

FIGURE S.10: PRECAST PANEL TO POLYCARBONATE CONNECTION (COURTESY OF JOHN STRAUBE OF BSC)

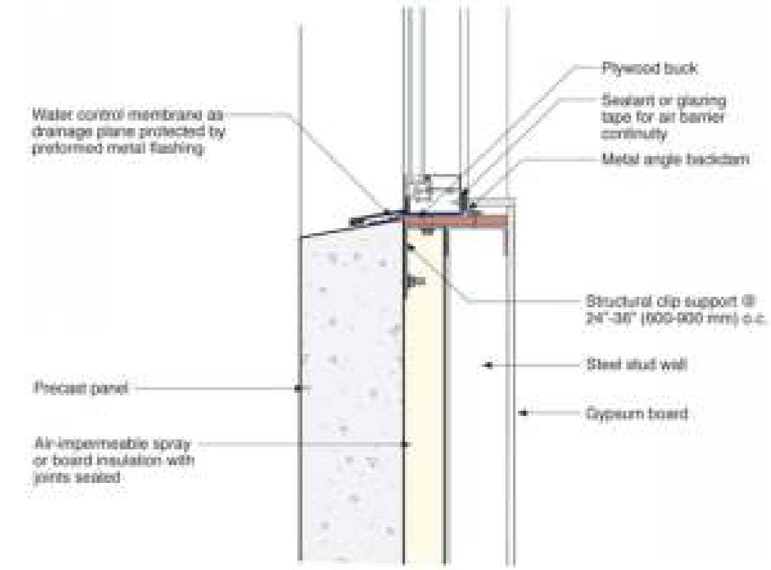

FIGURE S.11: WATER CONTROL AT WINDOW OPENINGS (COURTESY OF JOHN STRAUBE OF BSC)

Precast Panel System

The architectural precast paneling system for the main portion of the building consists of two three inch layers of concrete with a three inch layer of insulation between the concrete. By using this system, a high insulation value was able to be created for the buildings enclosure. These precast panels have also been proven to be extremely durable during impact loading. Each precast panel will span two floors and tie into the beams and columns of the structure. Since the precast panels will be connected to the perimeter beams of the building prior to the concrete floor being cured, all perimeter beams were designed as non-composite.

With the majority of building failures characterized by water penetration into the building, the structural team wanted to develop a façade system that efficiently stopped moisture penetration. Therefore, a two stage sealant system would be employed between the joints separating precast panels, see Figure S.12, Figure S.13, and Figure S.14. This system uses an exterior sealant as well as an interior sealant since it is common for the exterior sealant to break down from ultraviolent sun rays. Therefore, the joints will have additional moisture protection until the sealants get replaced. Also, with a two stage sealant, the precast panels would not need to be coated for additional resistance against moisture penetration.

To eliminate the thermal break, and moisture penetration at the windows, the window would be placed over the insulation within the wall with flashing below it. This can be seen in Figure S.11.

While the precast panels were not the most ideal for the structural system because of the weight, the benefits for all of the other disciplines at Synthesis were substantial. The mechanical team got the high R-value that they desired, the speed of construction could increase, and windows could easily be placed anywhere within the façade system.

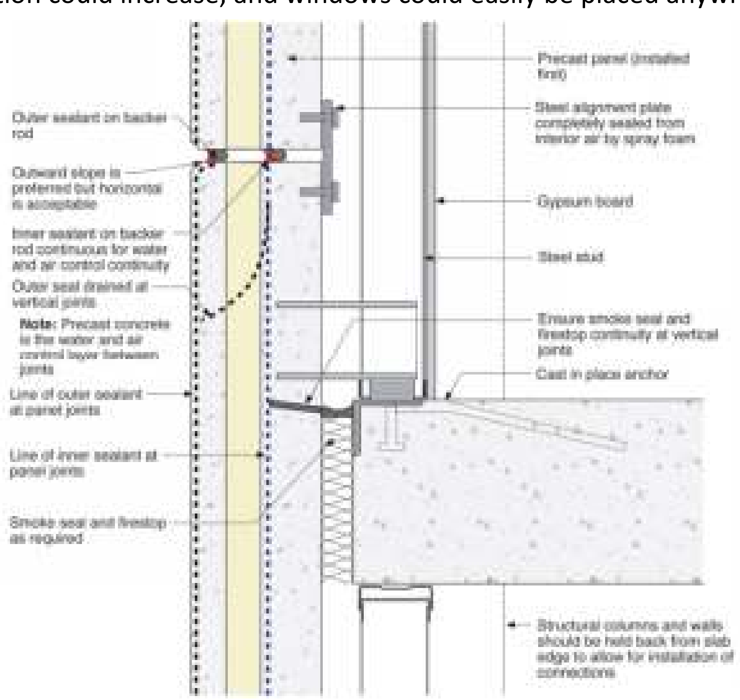

FIGURE S.12: TWO STAGE SEALANT SYSTEM (COURTESY OF JOHN STRAUBE OF BSC)

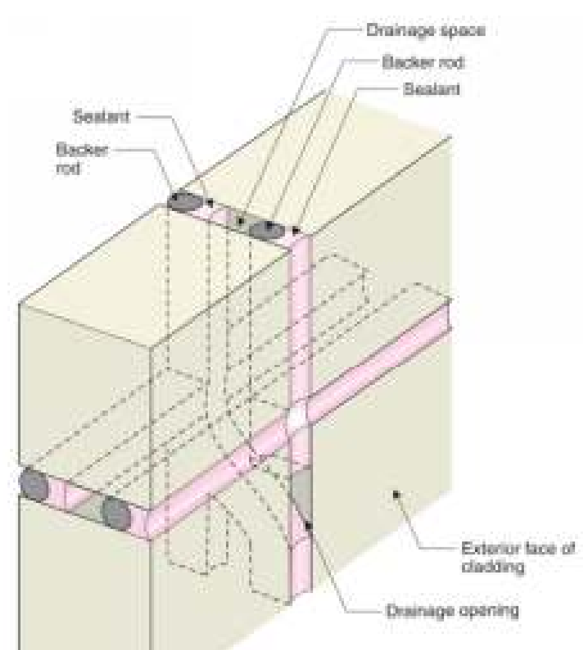

FIGURE S.13: TWO STAGE SEALANT SYSTEM (COURTESY OF JOHN STRAUBE OF BSC)

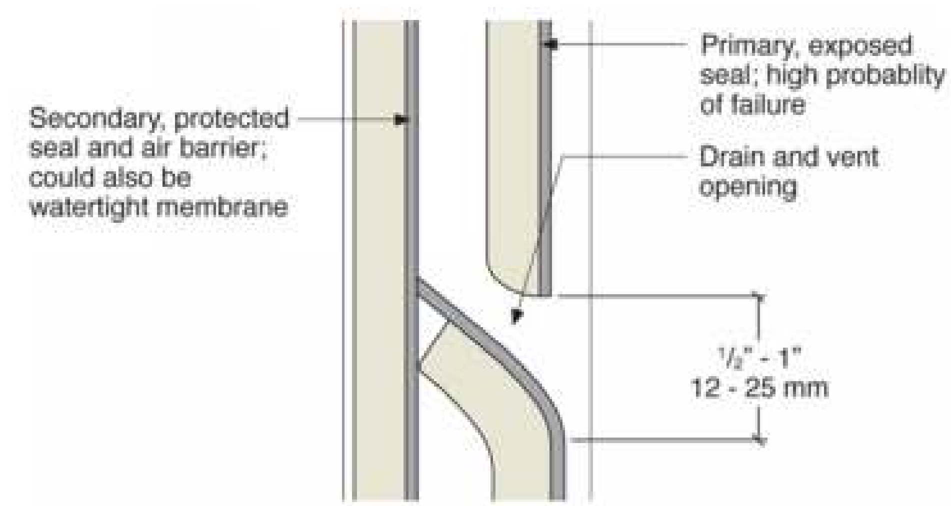

FIGURE S.14: SECTION OF TWO STAGE SEALANT SYSTEM (COURTESY OF JOHN STRAUBE OF BSC)

Polycarbonate Glazing System

The polycarbonate glazing system within the greenhouses was chosen for multiple reasons. These reason consist of their transmittance properties, durability, light-weight, and strength compared to other glazing systems. With the panels spanning 17'-9 ¼", parallel to the trusses within the greenhouses, a cross member at mid-span to support polycarbonate. By supporting the panels at the midpoint, the required loads were capable of being supported while meeting the deflection criteria for these panels of l/240. With the long spans of the polycarbonate glazing, with a support only at the mid-span, the amount of light interference could be limited. Thus allowing for maximum sun exposure for all of the plants, creating a more efficient ecosystem. The strength characteristics of the polycarbonate glazing can be seen in Figure S.15.

To create an efficient curtain wall system within the greenhouses, the structural team decided to employ a flush glazing system. With this system, all of the polycarbonate paneling will be on the outside of the steel mullions and structural members. Therefore, a flush finish on the exterior of the building could be created limiting the chance of snow build up at these points, and eliminating any thermal breaks between the interior and exterior of the greenhouses. The panels are connected to the mullions through structural sealant, as seen in Figure S.16. This design allow for easy maintenance and replacement in the event that any panels are broken. In each greenhouse module, there will be two mullions that create a thermal bridge between the interior and exterior of the greenhouse, to create an area for condensation to collect and be used within the grey water system.

A steel mullion system will be implemented within the greenhouses to eliminate the chances of corrosion from two different metals contacting each other. With a steel superstructure and steel mullions, less detailing would be needed when the two are in contact with each other. Steel mullions also resulted in smaller mullion sizes allowing for additional light to enter the greenhouses.

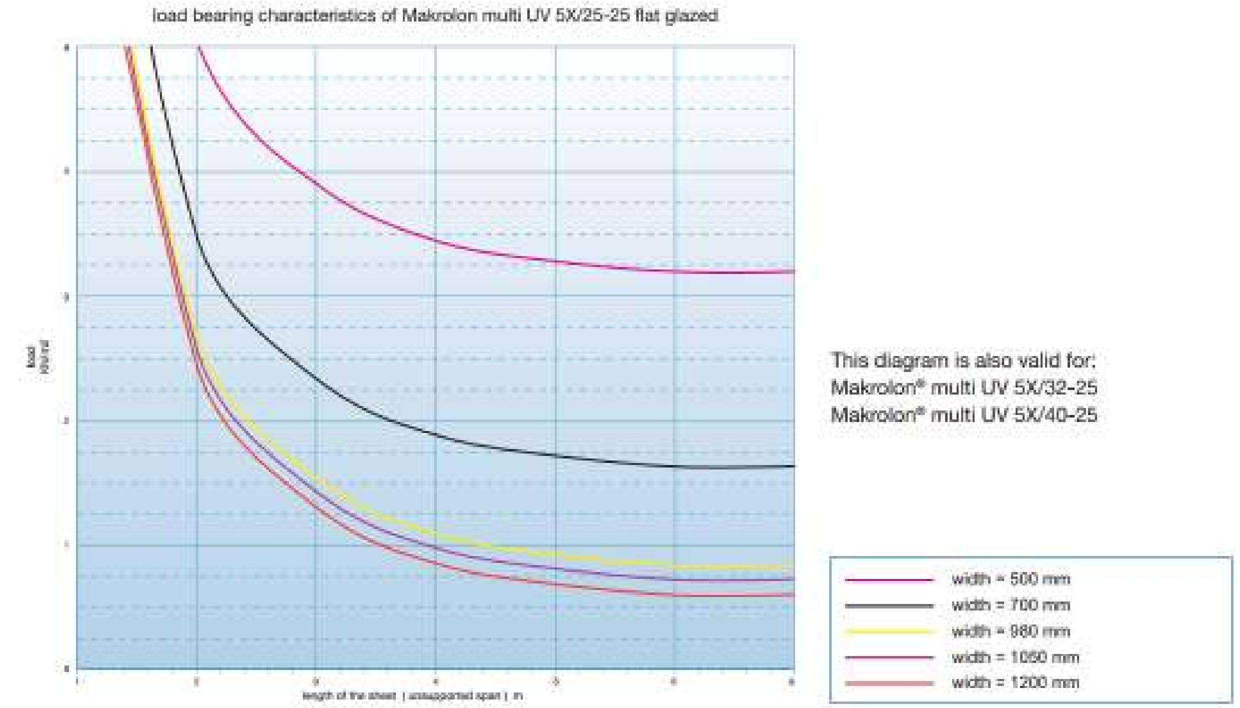

FIGURE S.15: LOAD BEARING CHARACTERISTIC OF POLYCARBONATE GLAZING (COURTESY OF MAKROLON TECH MANUAL)

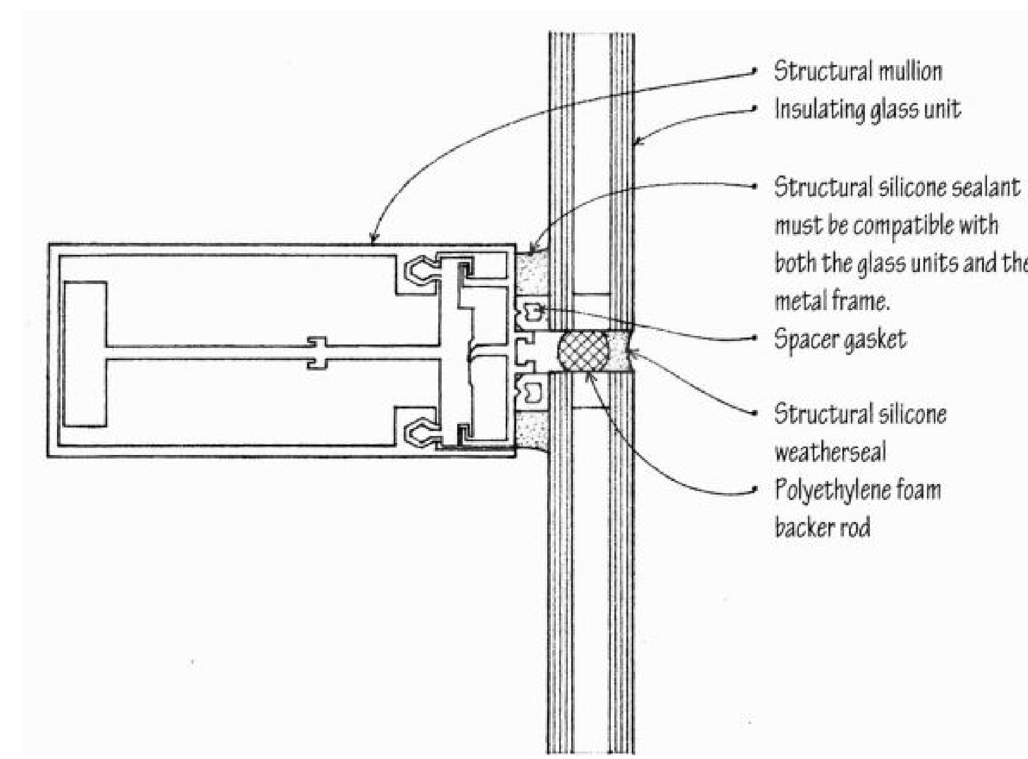

FIGURE S.16: FLUSH GLAZING SYSTEM (COURTESY OF BUILDING CONSTRUCTION ILLUSTRATED)

Appendix F: Gravity System Methodology, Calculations, and Description

After buildings loads were calculated for the building and precedent research was completed, the structural team at Synthesis started began with the design of the gravity system. A steel superstructure was used for design to reduce the seismic weight of building, reduce the amount of columns interfering with architecture, increase life-cycle efficiency, and decrease the length of the schedule. Once 2VLI18 decking with 3-¼" light-weight topping was selected, two separate structural layouts were developed and analyzed through a list of positives and negatives. The best layout was the chosen based off of cost and ability to mesh with the other structural systems. With both designs, a composite design was used to help reduce the building weight.

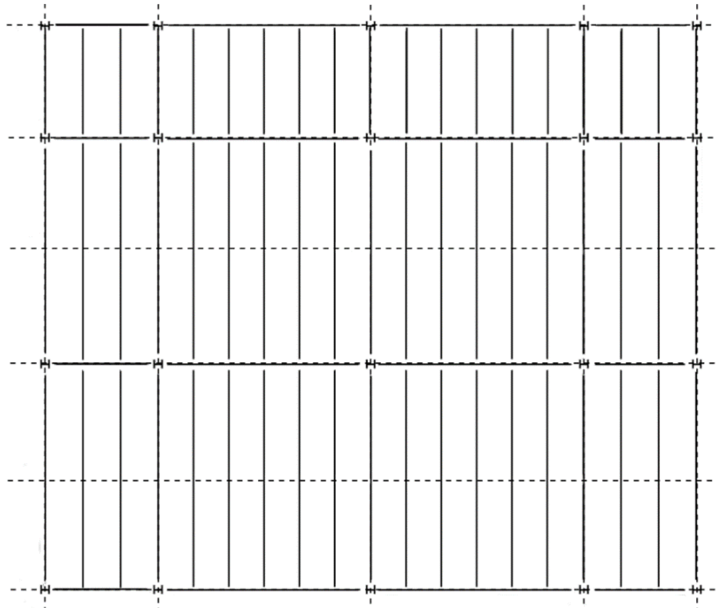

FIGURE S.17: PRELIMINARY GRAVITY DESIGN #1

Pros:

- Open floor plan
- Column layout worked with original redesign
- Less columns
- Architectural flexibility

Cons:

- Transfer girder needed for greenhouse columns
- Large members
- Deck spanning different directions
- No column line between differential floor elevations

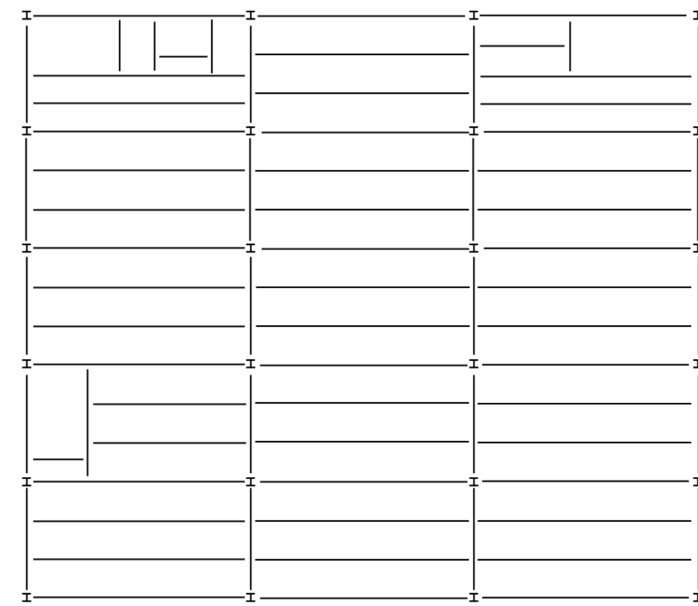

FIGURE S.18: PRELIMINARY GRAVITY DESIGN #2

Pros:

- Smaller member sizes
- Uniform bay sizes
- Column line along differential elevations
- Same deck direction for the floor
- No transfer girders needed

Cons:

- More columns
- Original floor plan redesign needed altered

*This gravity system was selected based off of the uniform layout and lower cost and developed further into the current system being used in the building.

Deck Design

- Composite Deck with Light-weight Concrete Topping
- 2" Deck with 3-¼" Topping (Vulcraft 2VLI18)
- 18 Gauge Decking
- 2-hr Fire Rating Between Floors
- 10'-6" Unshored Clear Span
- Max Beam Spacing, 10'-0" – Live Load Capacity = 205 lb/ft^2

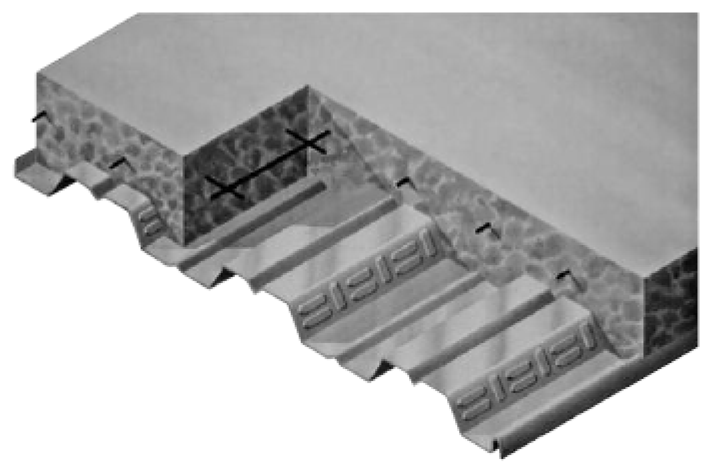

FIGURE S.19: 2VLI18 DECKING (COURTEST OF VULCRAFT)

(N=14.15) LIGHTWEIGHT CONCRETE (110 PCF)

TOTAL SLAB DEPTH	DECK TYPE	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)														
		1 SPAN	2 SPAN	3 SPAN	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0
5.25	2VLI22	7'-2	9'-3	9'-7	334	294	262	209	187	168	152	138	126	116	108	98	90	84	78
	2VLI20	8'-5	10'-7	10'-11	377	331	293	263	237	190	171	155	142	130	119	110	101	94	87
(t=3.25)	2VLI19	9'-6	11'-8	12'-1	400	368	324	289	260	236	216	198	156	143	131	121	111	103	95
42 PSF	2VLI18	10'-6	12'-7	12'-7	400	400	355	319	288	263	241	222	205	191	151	140	130	121	113
	2VLI16	10'-9	12'-10	13'-3	400	400	400	367	330	300	274	252	232	215	173	160	148	138	128

FIGURE S.20: 2VLI DECK PROPERTIES (COURTESY OF VULCRAFT)

Raised Floor System

A raised floor was developed in portions of the second, third, and fourth floors to allow for under-floor air distribution (UFAD). Due to the raised floor only a portion of the building, two differential slab elevations offset by 16 inches were created. These differing height occurred along column line B and along the column lines that separate the building from the greenhouse on each level. The areas of differing floor elevations on each level are shown in the figure to the right.

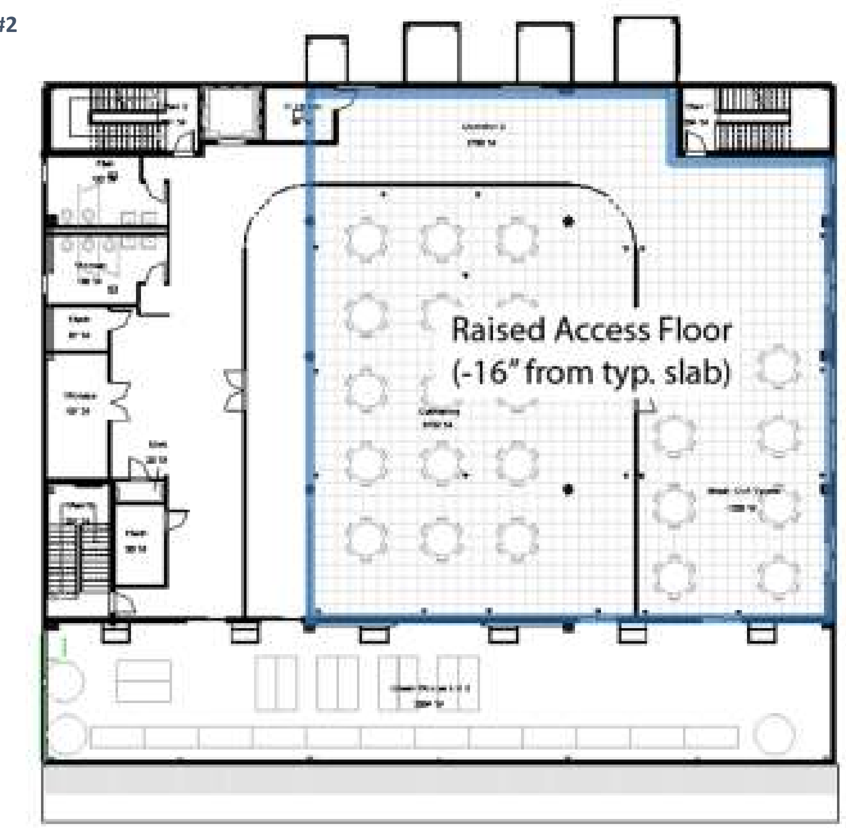

FIGURE S.21: RAISED ACCES FLOOR SEPERATION ON LEVEL 2

Composite Beam

A major consideration when designing the gravity system was the maximum allowable depth of the members. Since air would be returned in the plenum beneath the members, a uniform depth needed to be kept to allow for efficient air travel. Therefore, in the area of the building with a raised floor, the maximum member depth allowed was 16 inches. With the composite action, the structural team was able to design for smaller beam depths, without compromising on strength.

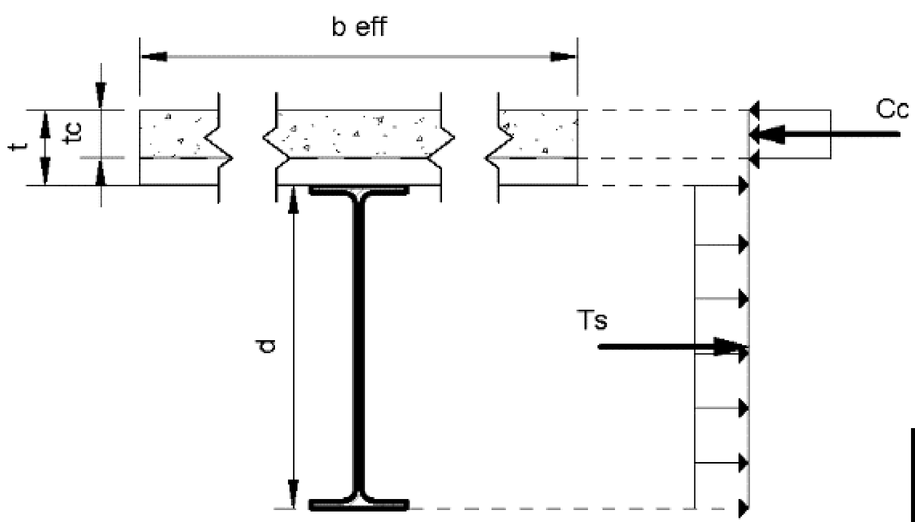

FIGURE S.22: COMPOSITE BEAM FORCES

Column Design

Column Design Considerations

- Columns are spliced every two stories for constructability purposes
- Column splices are done 4'-0" above top of slab for constructability purposes
- Columns are designed for an interaction ratio between .65-.95 for efficiency
- Columns were designed in RAM and verified through self-generated EXCEL Spreadsheets

Composite Beam Design Verification

Ram Gravity Beam Design -Size- W24x55, b_{eff} = 80", M_N=1067 k-ft

$$b_{eff} = min\begin{cases} {span}/{8} = \dfrac{38.33\ ft}{8} = 57.5\ in \\ \dfrac{spacing}{2} = \dfrac{80\ in}{2} = \mathbf{40\ in} \end{cases}$$

$$b_{eff} = 2 \times 40\ in = 80\ in$$

$$V_{c_{max}} = 0.85f'_c b_{eff} t = 0.85 \times 4\ ksi \times 80\ in \times 3.5\ in = 952\ kips$$

$$V_{s_{max}} = A_s F_y = 16.2\ in^2 \times 50\ ksi = 810\ kips \gg Case\ 1$$

$$a = \frac{A_s F_y}{0.85 f'_c b_{eff}} = \frac{16.2\ in^2 \times 50\ ksi}{0.85 \times 4\ ksi \times 80\ in} = 2.98\ in < t_c = 3.5\ in \gg Case\ 1$$

$$Mn = A_s F_y \left(\frac{d}{2} + t - \frac{a}{2}\right)$$

$$= 16.2\ in^2 \times 50\ ksi \times \left(\frac{23.7\ in}{2} + 5.5\ in - \frac{2.98\ in}{2}\right) = \mathbf{1070k - ft}$$

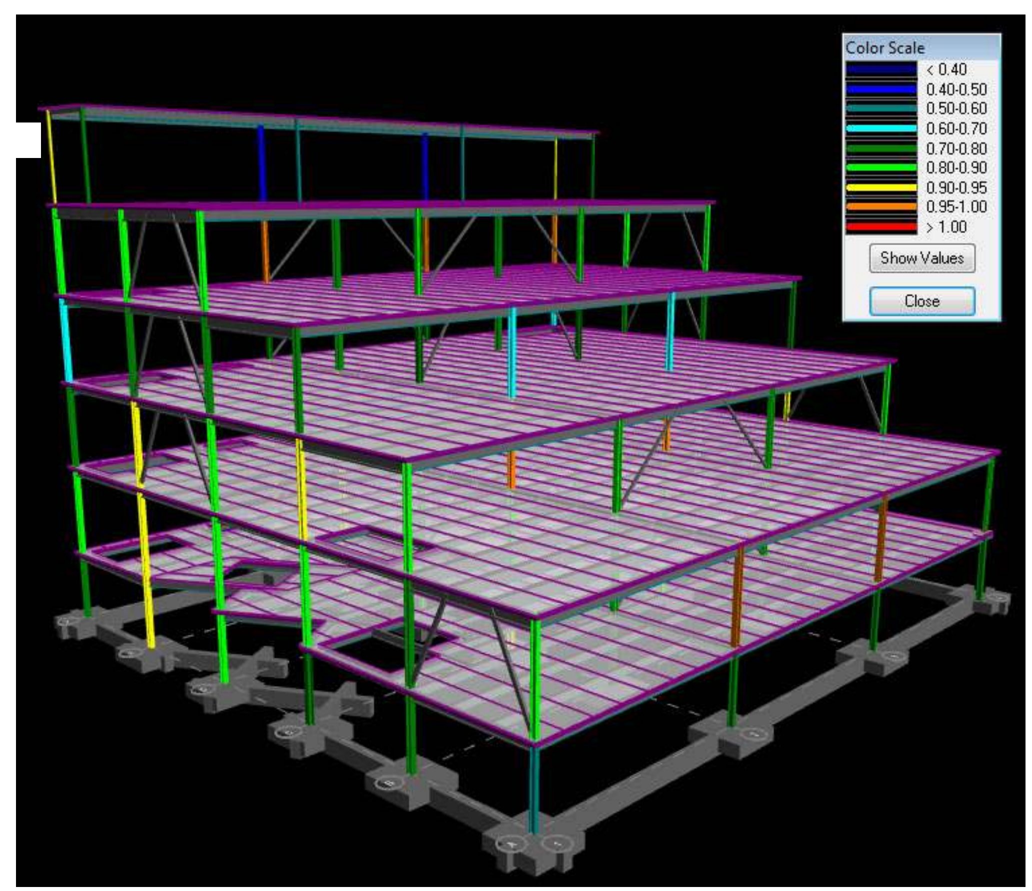

FIGURE S.23: RAM SS COLUMN DESIGN AND VERIFICATION OUTPUT

Camber

With the composite beam system being employed in addition to a beam depth maximum of 16", many of the beams required cambering to meet deflection requirements during construction. Without the use of cambering, many of the beams would need to be shored during construction, until the concrete reached full strength. The cambers were calculated to account for the self-weight of the structure and a construction live load of 20psf. Cambers are set to a ¾" minimum and specified in ¼" increments.

Equipment Access Panel

To ensure a sustainable building a needed to be design so the mechanical equipment in the basement could be accessed. To do this an opening was framed in the first floor that would then be covered with (3)4'x8" pre-stressed panels. To ensure that the precast was on the same level as the normal floor, a built up section was used to support the deck. Then a 2-¼" topping was poured on the precast panels to give it a finished look. A detail showing the framing and the built up section can be seen on the right.

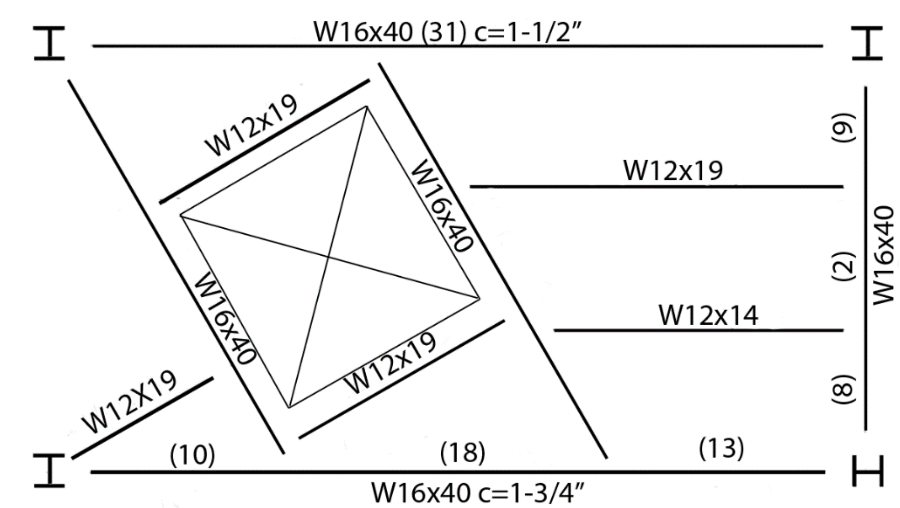

FIGURE S.24: EQUIPMENT ACCESS PANEL FRAMING

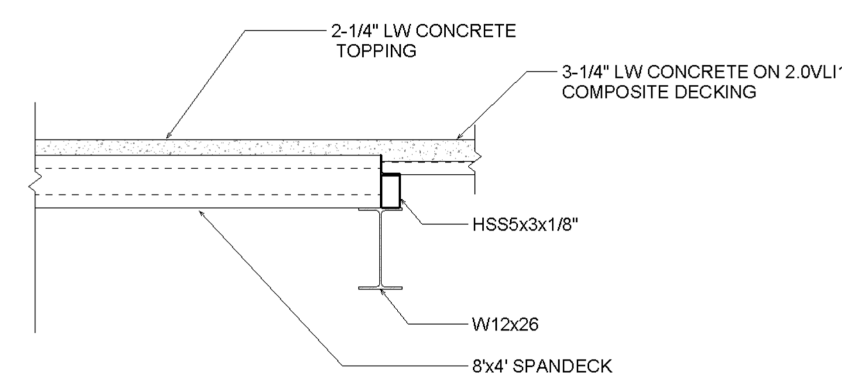

FIGURE S.25: EQUIPMENT ACCESS PANEL FRAMING BUILT UP SECTION

Vibrations Analysis

With reduced member sizes due to the composite design, the building had an increased chance of having vibration problems. Therefore, a vibrations analysis, using Design Guide 11, was done for classrooms and offices where people could be distracted by the floors shaking. The allowable % of gravity (g) allowable for a classroom or office is 0.5%. Once analysis was done, it was found that there was only 0.187% of g experience by the floors under a typical load. Therefore, the most efficient structural design was satisfactory, and well below the allowable limit for floor vibrations.

TABLE S.12: VIBRATIONS ANALYSIS CALCULATIONS

Beam Properties	
W16x26	
Area	7.68 in^2
I(x)	301 in^4
Depth	16.0 in

Girder Properties	
W16x40	
Area	11.8 in^2
I(x)	518 in^4
Depth	16.0 in

Deck Properties	
Conc. Weight	110 pcf
f'c	4000 psi
Topping	3.25 in
Rib Depth	2 in
Weight	42 psf

Building Loads	
Live	80 psf
Mech./Ceiling	15 psf

Beam Mode Properties	
Trib. Width	6'-8"
L_j	38'-4"
0.4*L_j	15'-4"
Eff. Slab Width	6'-8"
E_c	2307 ksi
Dynamic E_c	3115 ksi
E_s	29000 ksi
n	9.31
y(bar)	0.849 in Below top of form deck
I_j	923.2 in^4
w_j	939.8 plf
Δ_j	1.7 in
f_j	2.71 Hz

Beam Mode Properties (cont.)		
D_s	8.2 in^4/ft	
D_j	45.1 in^4/ft	
C_j	2.0	Eqn. 4.3a
B_j	50.1 feet	
W_j	406.0 kips	

Girder Mode Properties		
L_g	20 ft	
L_j	38'-4"	
0.4* L_g	8 ft	
Eff Slab Width	8 ft	
y(bar)	0.235 in Below effective slab	
I_g	2372.0 in^4	
w_g	5440 plf	
Δ_g	0.285 in	
f_g	6.63 Hz	
C_g	1.8	Eqn. 4.3b

Girder Mode Properties (Cont.)	
D_g	61.9 in^4/ft
B_g	33.3 ft
W_g	94.4 kips

Combined Mode Properties		
Δ_g'	.114 in	
f_n	2.62 Hz	
W	386.55 kips	
β	0.03	Table 4.1 (Office)
β W	11.6 kips	
P_o	65 lbs	Table 4.1 (Office)
A_p/g	0.00187 = 0.187% of g	

Allowable % of g for office/class	0.5 %

Therefore, the floor is considered satisfactory

Connections

The structural team worked closely with the construction team to design repeatable, constructible gravity connections for the steel framing. After analyzing the required connections for the building, a shear tab connection was selected because:

- Cost
- Ease of construction
- Less pieces involved

With the shear tab connections, half of the connection is constructed off site, saving time during erection.

Connection Design Calculations

All tables referred to in calculations are from AISC Steel Manual 14th Edition

Coped Beam

Double Cope:

$$S_{net} = \frac{t_w {h_o}^2}{6} = \frac{.395 \times (23.6-4)^2}{6} = 25.3\ in^3$$

Flexural Yielding:

$$\emptyset M_n = 0.9 F_y S_{net} \geq \emptyset V_n \times e = \frac{0.9(50)(25.3)}{1.5}$$

$$\emptyset V_n = 759\ kips$$

Local Web Buckling

$$\emptyset M_n = \emptyset F_{bc} S_{net}$$

$$F_{bc} = 56940 \left[{t_w}^2 / c h_o \right] f_d \leq F_y = 50\ ksi$$

$$f_d = 3.5 - 7.5\, {}^{d_c}/_{d} = 2.86 \gg F_{bc} = 321 \gg F_{bc} = F_y$$

≫No local web buckling

Block Shear

$$\emptyset R_n = \emptyset 0.6 F_u A_{nv} + \emptyset U_{bs} F_u A_{nt} \leq \emptyset 0.6 F_y A_{gv} + \emptyset U_{bs} F_u A_{nt}$$

Table 9.3a – 39.6 k/in Table 9.3b – 163 k/in Table 9.3c – 148 k/in

$$\emptyset R_n = (39.6 + 148) \times t_w = 74.102\ kips > 45\ kips$$

Bearing/Tear-out

Table 7-4 – (87.8 k/in)(.395)=34.7 k Table 7-5 – (49.4 k/in)(.395)=19.5 k

$$\emptyset R_n = \emptyset r_n t_w = 34.7\ k + 2(19.5\ k) = 73.7\ kips > 45\ kips$$

Bolts

Shear

$$\emptyset r_n = 17.9\ kips \gg 3\ bolts, \emptyset R_n = 53.7\ kips$$

Weld

Table 10-10a – ¼" fillet weld

Plate

Shear Yield

$$\emptyset R_n = 1.0(0.6) F_y A_g = .6 \times 36\ ksi \times 9\ in \times {}^5/_{16}\ in = 60.75\ kips > 45\ kips$$

Shear Rupture

$$\emptyset R_n = 0.75(0.6) F_u A_n = .75 \times .6 \times 58\ ksi \times 6.375\ in \times {}^5/_{16}\ in = 52\ k > 45\ kips$$

Block Shear

Table 9.3a - 46.2 k/in Table 9.3b - 117 k/in Table 9.3c - 132 k/in

$$\emptyset R_n = (42.6 + 117) \times t_w = 79\ kips > 45\ kips$$

Bearing/Tear-out

Table 7-4 – (78.3 k/in)(5/16)=24.5 k

Table 7-5 – (44.0 k/in)(5/16)=13.8 k

$$\emptyset R_n = \emptyset r_n t_w = 24.5\ k + 2(13.8\ k) = 52.1\ kips > 45\ kips$$

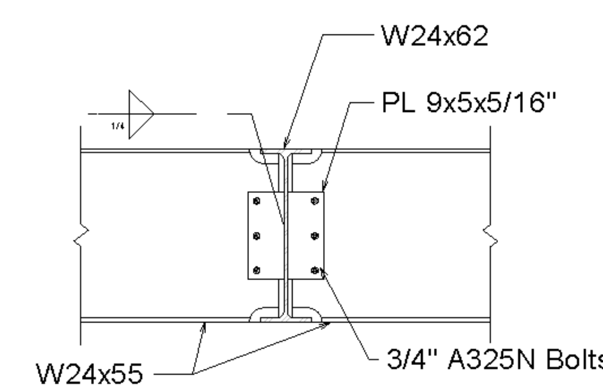

FIGURE S.26: EQUIPMENT ACCESS PANEL FRAMING BUILT UP SECTION

Appendix G: Lateral System Methodology, Calculations, and Description

After a considerable amount of research was done on the building, it was found that the lateral force resisting system for this Vertical Farm was going to be one of the major considerations for the building to be adaptable. It was essential to create a lateral system that was capable of moving to areas with high seismic or wind loads with minimal design changes being needed. With the architectural redesign of the building, it made it possible for the structural team at Synthesis to incorporate a system of eccentrically braced frames hidden within interior partitions of the wall. In areas were braced frames could not be used, a SidePlate moment connection was use to transfer the lateral loads to the columns.

Center of Rigidity

By developing a semi-symmetrical layout for the lateral force resisting elements, the center of rigidity and center of mass of the building could be kept close together. This in turn lowered the eccentricity in the building and eliminated extensive additional lateral forces from torsional effects. The COM compared to the COR of a typical floor can be seen on the left.

TABLE S.13: CENTER OF RIGIDITY CALCULATION WITH LATERAL FORCES BEING EXPERIENCED

Level 3		
Story Force	N-S	E-W
	51.77	21.73

Element	Element Direction	Distance from Ref. Datum		Rx	Ry	Rx*y	Ry*x	dx	dy	Rxdy	Ry*dx	Rx*dy^2	Ry*dx^2	Direct Shear [kips]		Torsional Shear [kips]		Total Shear [kips]	
		x	y											N-S	E-W	N-S	E-W	N-S	E-W
1	y	0.00	-	-	143.00	-	0.00	64.56	-	-	9231.40	-	595934.73	22.71	-	2.32	0.00	25.03	-
	y	115.00	-	-	143.00	-	16445.00	50.44	-	-	7213.60	-	363888.72	22.71	-	1.81	0.00	24.52	-
2	y	115.00	-	-	40.00	-	4600.00	50.44	-	-	2017.79	-	101787.05	6.35	-	0.51	0.00	6.86	-
3	x	-	40.00	5.80	-	232.00	-	-	0.96	5.57	-	5.35	-	-	0.50	0.00	0.00	-	0.50
4	x	-	80.00	125.00	-	10000.00	-	-	39.04	4879.90	-	190507.70	-	-	10.87	0.00	0.07	-	10.94
5	x	-	0.00	119.00	-	0.00	-	-	40.96	4874.33	-	199656.36	-	-	10.35	0.00	0.07	-	10.42
	Total			**249.80**	**326.00**	**10232.00**	**21045.00**			**9759.81**	**18462.79**	**390169.42**	**1061610.51**	**51.77**	**21.73**	**4.64**	**0.14**	**56.41**	**21.87**

Architectural Coordination

One of the many benefits of using eccentrically braced frames is the freedom they allow architecturally over other braced frames, while still performing well structurally. The locations of the braced frames was coordinated closely with the architecture within the building. The images below illustrates how the braced frames were embraced in certain areas by the architecture.

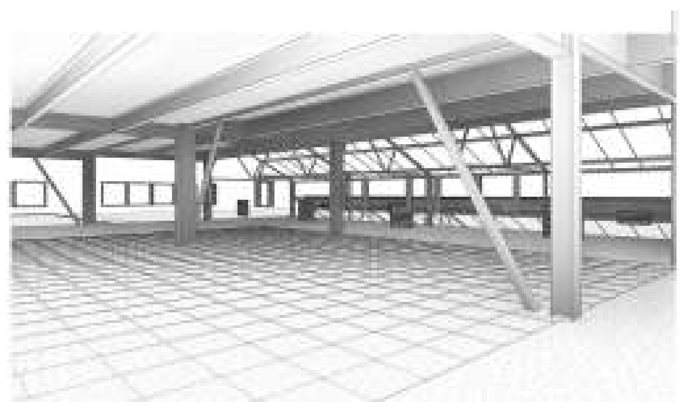

FIGURE S.27: EBF WITHOUT INTERIOR PARTITION

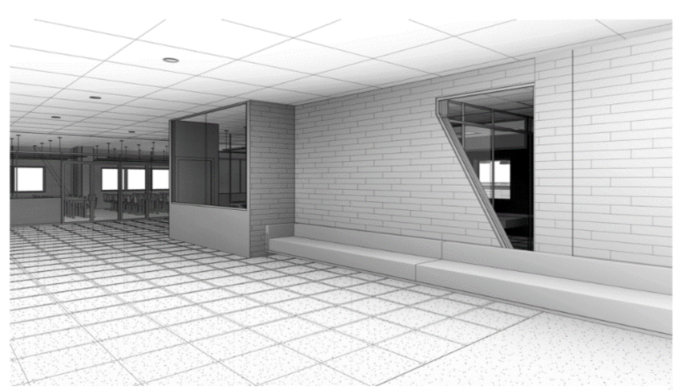

FIGURE S.28: EBF WITH ARCHITECTURAL PARTITION

Deep Beam

To transfer the lateral loads between the two differential slab elevations, a deep girder was needed to pick up both beams. The lateral loads then could be transferred through this girder into the other diaphragm. With this proper diaphragm action, the lateral loads were capable of getting to the lateral force resisting elements even though they were on different elevations.

In buildings with low lateral loads, the deep girder by itself would be capable to resist the torsional forces from the slabs. When moved to a high seismic area, the beams would need additional bracing to resist the beam from rolling from torsional effects. Therefore, a kicker could be added to the deep girders to efficiently transfer the lateral loads.

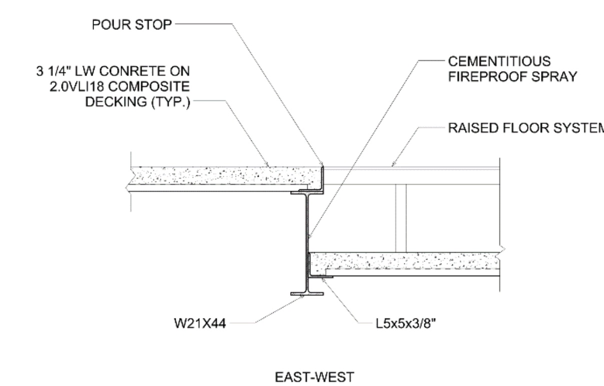

FIGURE S.29 GIRDER RUNNING EAST-WEST DIFFERENTIAL SLAB CONNECTION

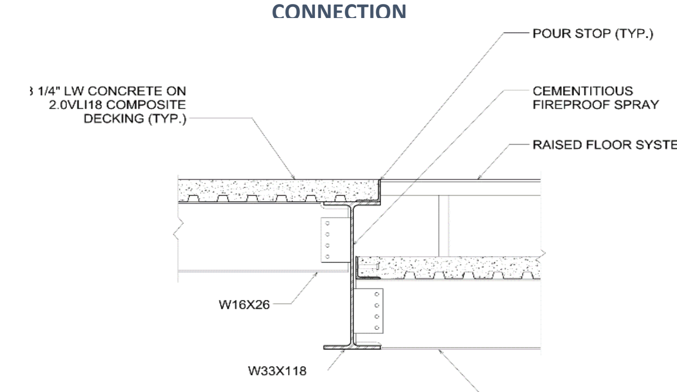

FIGURE S.30: GIRDER RUNNING NORTH-SOUTH DIFFERENTIAL SLAB CONNECTION

EBF Analysis and Sizing

To size the eccentrically based frames, the structural team analyzed a full height frame in ETABS to determine the axial forces that each brace would take. The structural team then took one of the frames and calculated the forces that each story would experience. From here, the forces for each brace were calculated to hand check the compressive forces that came from ETABS. Once these forces were confirmed, the braces were sized using the unbraced length of the members along with the axial load in the braces. A table in the steel manual was used to find the unbraced capacities for each member. After sizing the members for axial loads, it was found that the building did not meet the wind drift requirements set forth by Synthesis. Therefore, member sizes were increased until the drift requirements were met for the building.

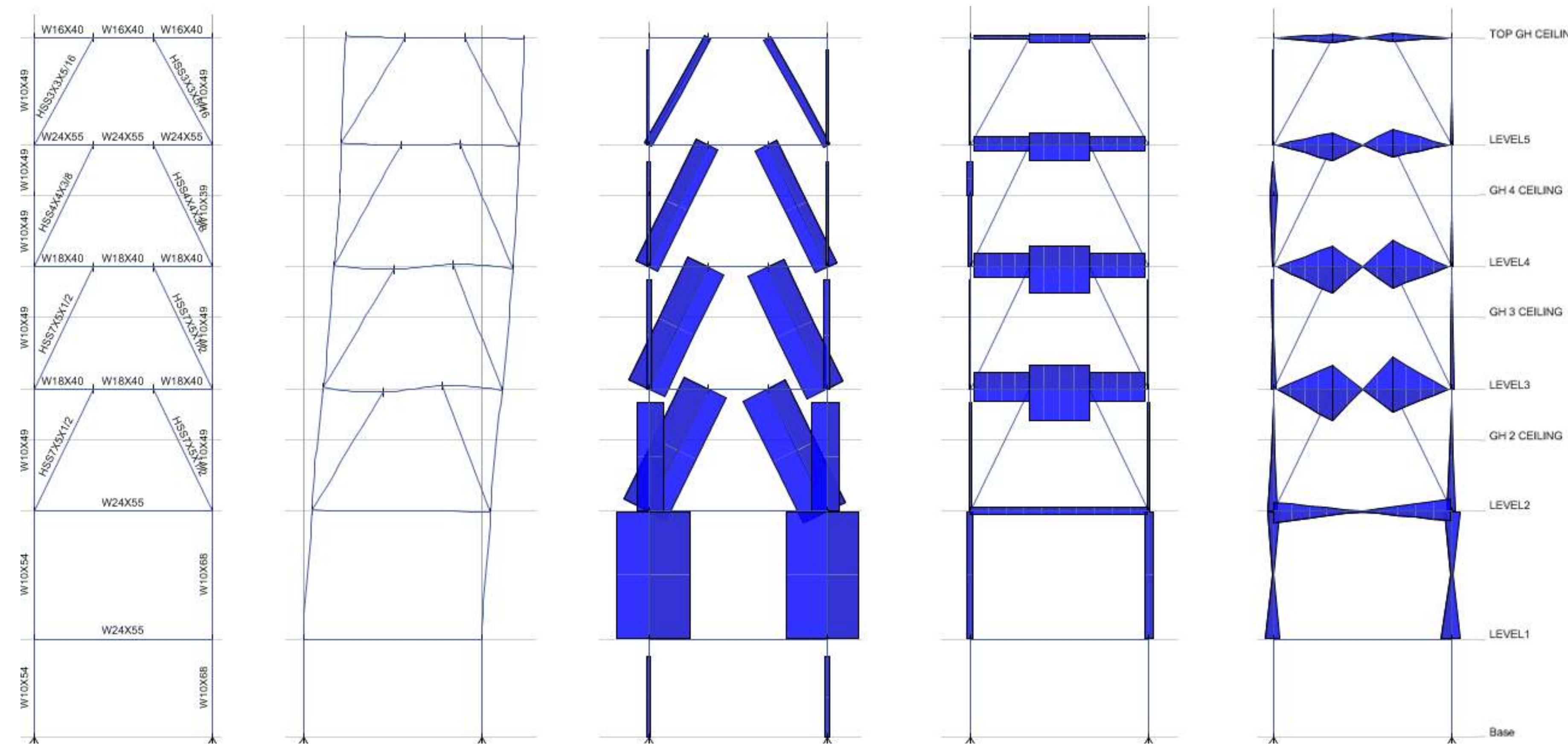

FIGURE S.31: FROM LEFT TO RIGHT, (1) LATERAL FRAME, (2) DEFLECTED SHAPE, (3) AXIAL FORCES, (4) MOMENTS, (5) SHEAR FORCES

Response Modification Factor

A major consideration when designing the lateral system was the response modification factor of all elements when moving the building to areas with high seismic loads. By developing a system that utilizes a high response modification factor, the lateral loads that are experienced by the building during an earthquake could be reduced considerably. Considering that most buildings under 10 stories are typically controlled by seismic loading, especially in buildings with high dead loads, such as this vertical farm, the structural team at Synthesis felt that it was a necessity to develop the lateral elements to have a high response modification factor. Thus allowing Growing Power to reproduce the building is high seismic regions with less changes needed for the lateral system.

With the main lateral system consisting of eccentrically braced frames, a response modification factor of 7 could be used for a majority of the floors within the building. The only problem was that in areas, such as the gathering space on the second floor, and the loading docks on the first floor, braced frames could not be used because the building would not be functional. Therefore, the lateral design was done with these frames as moment frames. The led to an issue due to the fact that a typical moment frame only has a response modification factor of 3.5. Therefore, a value of 3.5 needed to be used at the location of highest moment frame and any level below that. By having to use this response modification factor, it eliminated one of the main benefits of eccentrically braced frames, the high R-factor. Therefore, to ensure the building was fit to move to regions where the seismic load controlled, a solution needed to be developed to make the response modification factor of these moment frames be at least the same as an eccentrically braced frame. It was found through ASCE 7-05 that a moment connection deemed as a special moment frame could reach an R-factor of 8 but it came with a lot of cost and detailing to achieve that value. Thus increasing the overall budget and duration of the schedule. More research was done to develop a way of achieving the favored R-factor.

SidePlate

While doing research on special moment frames, the structural team at Synthesis came across a connection called SidePlates. SidePlates are a type of special moment connection that features a separation between the face of the column and the beam that it is supporting. The beam is then connected to the column by a system of plates allowing movement within the connection. By allowing this movement at the joints some of the seismic energy entering the lateral frame is dissipated, lowering the forces that are experienced. By using SidePlates, it was found that the connection could act similar to intermediate moment frames as well as special moment frames. Therefore, a response modification factor of 8 could be developed in regions that are seismically controlled to reduce the forces experienced by the building. When seismic loads do not control, the SidePlate connection that acts as an intermediate moment connection can be used to reduce the cost of the connection. Research showed that a SidePlate connection is favorable over special moment connections for a multitude of reasons. The biggest reasons why SidePlates are preferred is for construction purposes. By using SidePlates, money could be saved by not having to do the complicated connections that come with special moment frames. It was also found that without the complicated moment connections, SidePlates are also considerably less expensive to construct.

FIGURE S.32: SIDEPLATE CONNECTION (Courtesy of SidePlate)

Drift Analysis

Through discussions with design professionals, it was found that there is not a specific drift requirement for buildings when experiencing wind loading. These design professionals informed us that it is typical practice to limit the drift within the building to h/400 for each level and for the entire building height. Since the structural team wanted to meet this standard practice, the brace sizes were controlled by drift requirements.

For seismic loads, using table 1617.3.1, Allowable Story Drift, the building fell under the building category of all other buildings and seismic use group II. By using these parameters, the allowable drift under the seismic loading for the entire building was 12.12". The lateral design used to limit the drift under wind loading also kept the seismic drift well under then allowable drift. A breakdown of the allowable drift for seismic and wind loads can be seen in Table S.14.

TABLE S.14: ALLOWABLE VS. ACTUAL DRIFT

Drift Control			
Story	Allowable Wind Drift	Allowable Seismic Drift	Worst Case Drift
2	0.43"	2.58"	0.39"
3	0.41"	2.46"	0.32"
4	0.41"	2.46"	0.34"
5	0.41"	2.46"	0.28"
Top Greenhouse	0.36"	2.16"	0.19"
Total Structure	2.02"	12.12"	1.53"

ETABS Modeling

While original sizing of lateral members was done using RAM Structural System, the structural team at Synthesis felt that the loads that were being generated were not accurate because they did not include the greenhouses and did not align with the loads calculated by hand. Therefore, further analysis was done using ETABS by modeling the main diaphragms and lateral elements along with a semi-rigid-diaphragm at the roof of the greenhouses. Thus creating more accurate lateral loads to size the braced frames. An ETABS model with the diaphragms hidden can be seen in Figure S.33.

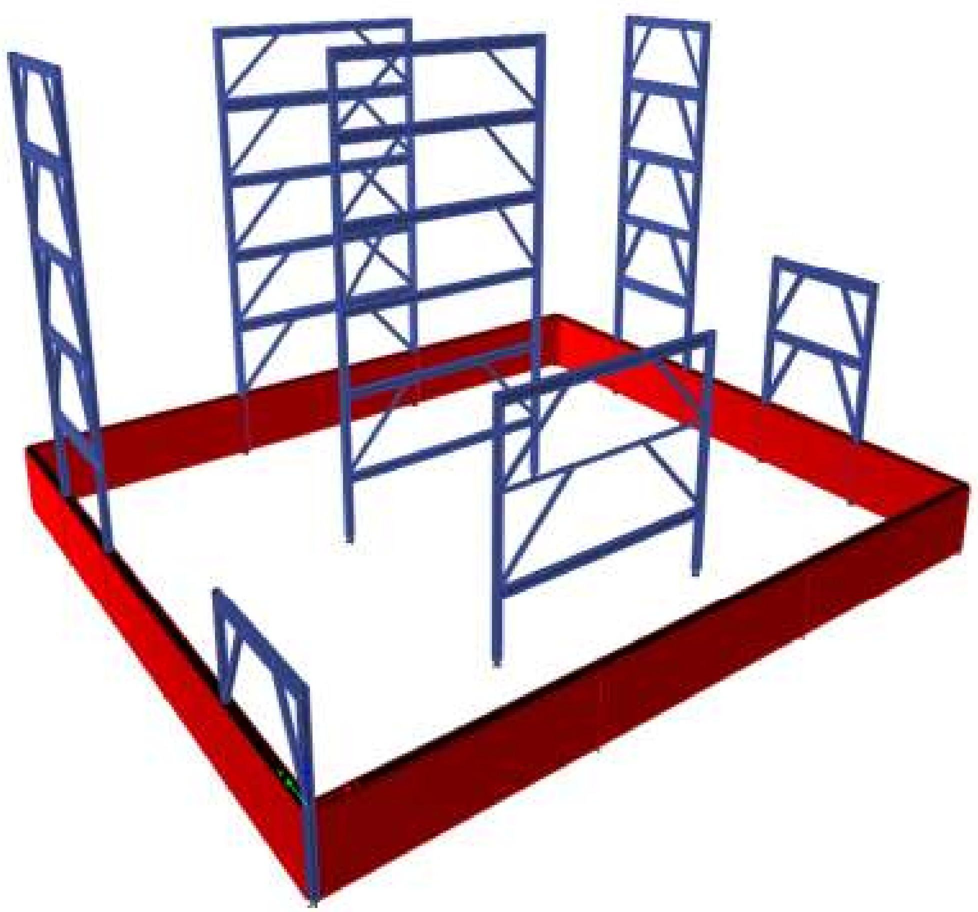

FIGURE S.33: ETABS LATERAL SYSTEM MODEL WITHOUT DIAPHRAGMS

Wind Tunnel Analysis

Due to the complicated nature of wind analysis, it may be beneficial to determine the wind pressures being experienced by the building through wind tunnel testing. Developing the wind pressure for buildings through codes and standards, such as ASCE 7-05, can often be complicated and are considered an educated guess at the wind pressures that will be experienced. Also, these standards are for developing wind pressures on "regular-shaped" buildings, which is loosely defined as a building having no unusual geometry irregularities. Due to the vagueness of the code, and the stepping back of all of the greenhouses this building could certainly be determined to be an irregular shape. Therefore, the structural engineers at Synthesis conservatively assign some of the wind loads for the building because of the unpredictability of wind. Therefore, with this building being irregularly shaped, it may be beneficial to perform Computational Fluid Dynamics (CFD) analysis to determine if the building would be ideal for wind tunnel testing. With the building being a prototype for many buildings to come, the model for the building could be used for the analysis of the wind pressures on any of the proposed building sites.

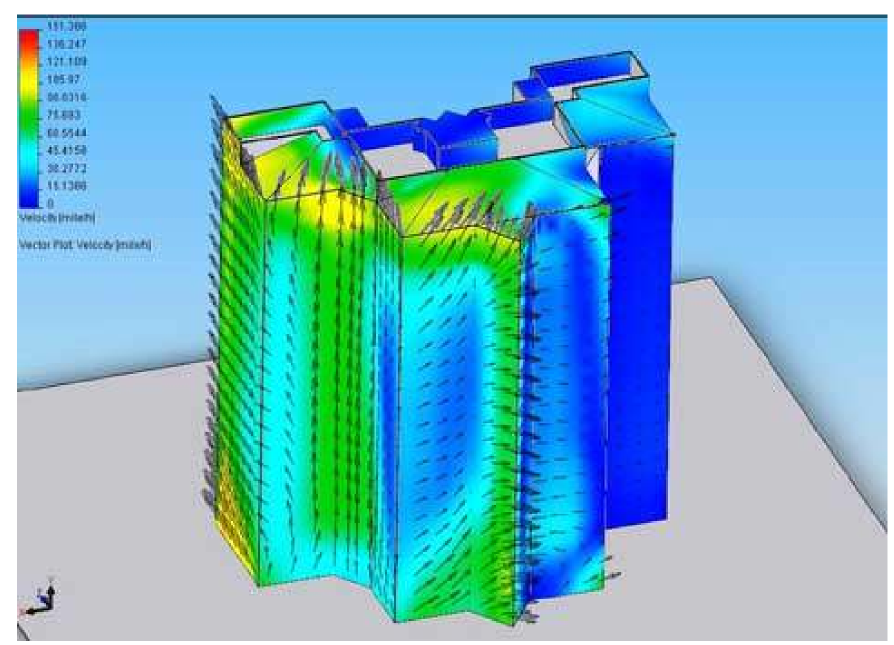

FIGURE S.34: GENERIC COMPUTATIONAL FLUID DYNAMICS ANALYSIS (COURTESY OF BUILDING ENCLOSURE CONSULTING)

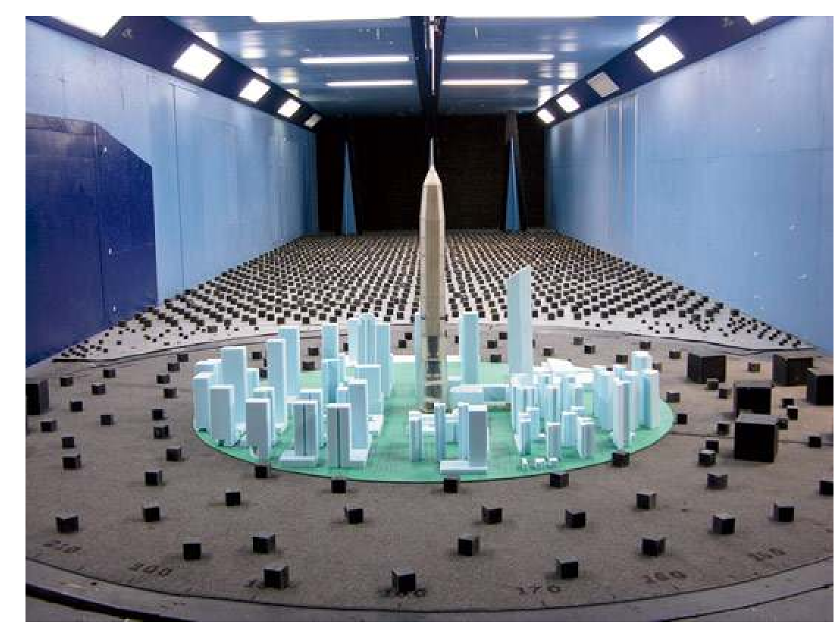

FIGURE S.35: ETABS LATERAL SYSTEM MODEL WITHOUT DIAPHRAGMS (COURTESY OF MCGRAW-HILL CONSTRUCTION)

Code Considerations

In areas where the towers did not extend above the top greenhouse, the wind pressure that was applied to the faces of the chimney like structure was conservatively assumed to be the same as the wind pressures experienced by the main building. For the portion of the structure that extended above the building, Components and Cladding in ASCE 7-05 was used to develop the wind pressure experienced by tower. By using Figure 6-21 within ASCE 7-05 it was determined that ideally the towers were round because the Force Coefficient (C_f) would be less than 1.0, resulting in a reduction in the forces that the structure would see. However, due to the mechanical team needed square faces perpendicular to the wind, a square chimney was needed which resulted in a Force Coefficient greater than 1.0. As mentioned in the lateral system section of this appendix, the wind loads experienced by the towers could be reduced through a CFD analysis and wind tunnel testing. A breakdown of the wind pressures experienced by the towers can be seen in the table to the left. It can be seen why round chimneys were desired because they could be designed for the minimum building pressure of 16 psf.

Gravity Design

To provide the needed insulation values for the mechanical system, it was preferred that the towers on the north side of the building were made out of CMU's to achieve a high R-value. Also, with a CMU façade, an exterior insulation façade system could be used to help the mechanical team achieve the needed insulation values. Due to the height of the towers, the structural team wanted to develop a way to alleviate the masonry at each level so that the walls would not have to be extremely thick. Therefore, a steel superstructure was developed to support the CMU at each level.

By breaking the CMU at each level, the thickness of the wall was greatly decreased because a lower moment and axial force were developed by all of the masonry units. Therefore, for every tower, two additional columns (W8x31) were dropped outside the building for the beams to frame into. This was preferred to cantilevering steel beams off of the existing structure to eliminate the development of torsion in the building.

When designing the masonry infill walls, the construction team did not want to have to use reinforcing so that the steel could be erected prior to the masonry walls. Knowing this, the structural team took a one foot section of the wall and assumed it would be pinned at the top and bottom, and the applicable wind pressures. Multiple design iterations were then done to find the most efficient fully grouted CMU that could be used. An example of this process is shown in the tables on the right. Through this analysis, 8" CMU's were used to infill all layers lower that did not extend above the building, and 12" CMU was used in the portion of the tower that extended above the building.

TABLE S.17: MASONRY DESIGN CALCULATIONS FOR TOWERS

Masonry Wall Design (1st Iteration)	
End Conditions	Pinned-Pinned
Height	22'-6"
Wind Pressure	33 psf
Max Moment Location	11'-3"
Self-Weight (10" Fully Grouted)	104 psf
P @ 11'-3"	1170 plf
A_N of CMU	115.5 in²/ft
S_N	185.3 in³/ft
f(b)	125.2 psi
F(b) from 2011 Code	86 psi
	125.2>86
Therefore, the wall **does not meet code**	

Masonry Wall Design (2nd Iteration)	
End Conditions	Pinned-Pinned
Height	22'-6"
Wind Pressure	33 psf
Max Moment Location	11'-3"
Self-Weight (12" Fully Grouted)	133 psf
P @ 11'-3"	1496 plf
A_N of CMU	139.5 in²/ft
S_N	270.3 in³/ft
f(b)	82.0 psi
F(b) from 2011 Code	86 psi
	82<86
Therefore, use a 12" fully grouted CMU in the exhaust tower above the building.	

TABLEE S.15: TOWER DIMENSIONS

Tower Dimensions		
Tower	Plan Dimension	Height above Top Greenhouse
2nd Story Supply	6'-5" x 6'-9"	9'-4"
3rd Story Supply	8'-5" x 8'-9"	15'-4"
4th Story Supply	8'-5" x 8'-9"	21'-4"
Exhaust	9'-5" x 9'-9"	28'-4"

TABLE S.16: WIND FORCES FOR SQUARE VS ROUND TOWERS

Tower – Wind Parameters			
Tower	Force Coeff. (C_f)		q_z
	Square	Round	
2nd Story Supply	1.64	0.64	23.9psf
3rd Story Supply	1.54	0.62	24.5 psf
4th Story Supply	1.56	0.63	23.9 psf
Exhaust	1.54	0.62	25.3 psf

*Gust Factor for all = 0.85

Tower – Wind Forces				
Tower	Pressure		Force (F)	
	Square	Round	Square	Round
2nd Story Supply	33.2 psf	13.0 psf	0.72 k	0.28 k
3rd Story Supply	31.9 psf	12.9 psf	2.31 k	0.94 k
4th Story Supply	33.0 psf	13.3 psf	3.78 k	1.52 k
Exhaust	33.1 psf	13.4 psf	5.91 k	2.39 k

*If round towers were designed for, a code required minimum of 16 psf would be used for the towers.

FIGURE S.36: TOWERS ON THE NORTH SIDE OF THE BUILDING

Once the weight of the infill walls was developed, the beams supporting them were then designed using the load of the masonry. With such little load on these members, it was found that developing the moment capacity was not going to be a problem, but the beams would be controlled by deflection since they could not deflect more than l/600. Through the design process shown in the tables on the right, it was found that a W12x19 was going to be needed to support the load and limit the deflection within acceptable limits. The only problem is that a W12x19 only has a flange width of four inches. Thus meaning the 8" and 12" masonry blocks would not be fully supported by the beam. Therefore, as opposed to overdesigning the beam so that the CMU could rest on top, the beams will be prefabricated with a 12" plate welded to the top to support the masonry. This was an ideal solution as opposed to overdesigning the beam because the beams already needed to be prefabricated to attach shear studs that engage the masonry that would be sitting on them. A detail of the connection within the chimney can be seen in Figure S.37.

TABLE S.18: BEAM DESIGN FOR STEEL IN TOWERS

Beam Properties	
Heaviest CMU Load	133psf
Height of Wall	22'-6"
Linear Load	2.99 klf
Dead Load Factor	1.2
Beam Length	9'-9"
Moment	42.6 ft-k
L/600	0.195"

Beam Design			
W12x14		W12x19	
I	88.6 in^4	I	130 in^4
$\emptyset M_n$	65.3 ft-k	$\emptyset M_n$	92.6 ft-k
deflection	0.237in	deflectio n	0.161 in
65.3ft-k > 42.6ftk-k		92.6 ft-k > 42.6 ft-k	
0.237" > 0.195"		0.161" < 0.195"	
FAILS DEFLECTION		**GOOD**	

*Therefore, W12x19 beams were used within the towers for all beams to keep uniformity.

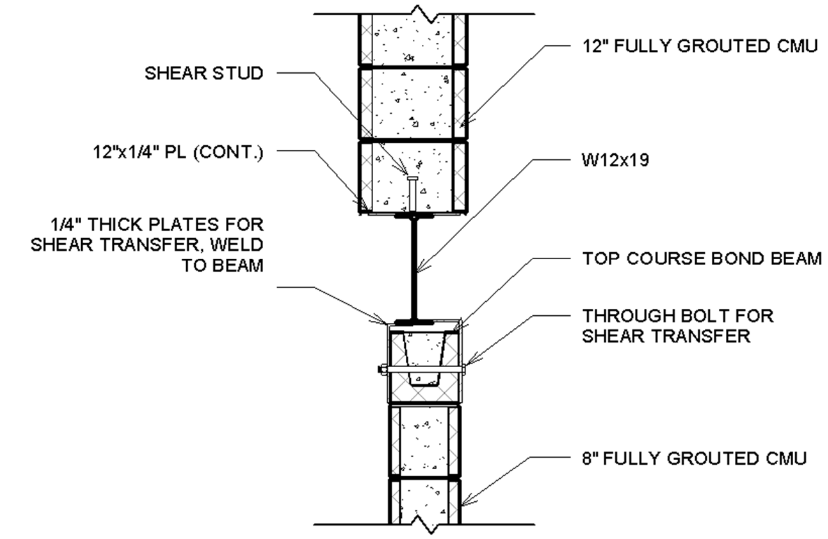

FIGURE S.37: TYPICAL MASONRY SHEAR WALL CONNECTION

Hybrid Shear Walls

Since the towers on the north side of the building already needed to be infilled with masonry for the thermal properties, the structural team wanted to avoid using braces because it would make infill with CMU's a construction nightmare. Therefore, the structural team looked into ways to use these masonry infill walls as a way to resist that lateral loads in the towers. Through research, it was found that hybrid masonry walls could be used to resist the lateral loads by using the steel frame tied into the masonry. Synthesis decided to implement Type I hybrid walls, which include a gap between the top of the wall and bottom of the steel. By using Type I walls, the construction team could build the steel superstructure and come back and infill the masonry afterwards. The top unit in the masonry shear walls will be a bond beam so that the construction team can place the unit and then fill the member with grout by hand. A detailed connection of a Type 1 hybrid shear wall connection can be seen in Figure ###. To analyze the drift that would be experienced in the towers, an equivalent brace size was calculated to use when calculating the drift that would be experienced by the towers. By calculating the equivalent area of steel for the hybrid shear walls, the analysis on drift could then be checked using ETABS by inputting the steel as cross bracing within the towers. Calculations of the equivalent steel member sizes can be seen in Table S.19 below.

TABLE S.19: EQUIVALENT STEEL BRACES OF HYBRID SHEAR WALLS

	2nd Story Supply		3rd Story Supply		4th Story Supply		Exhaust	
	Below Top Greenhouse	**Above Top Greenhouse**	**Below Top Greenhouse**	**Above Top Greenhouse**	**Below Top Greenhouse**	**Above Top Greenhouse**	**Below Top Greenhouse**	**Above Top Greenhouse**
	$t = 7.625"$	$t = 11.625"$	$t = 7.625"$	$t = 11.625"$	$t = 7.625"$	$t = 11.625"$	$t = 7.625"$	$t = 11.625"$
	$h = 164"$	$h = 82"$	$h = 164"$	$h = 154"$	$h = 164"$	$h = 226"$	$h = 164"$	$h = 310"$
	$l = 81"$	$l = 81"$	$l = 104"$	$l = 104"$	$l = 104"$	$l = 104"$	$l = 116"$	$l = 116"$
	$L = 183"$	$L = 115.3"$	$L = 194.2"$	$L = 185.8"$	$L = 194.2"$	$L = 248.8"$	$L = 200.9"$	$L = 331.0"$
	$\Delta = 3.82E-3"$	$\Delta = 4.58E-4"$	$\Delta = 1.98E-3"$	$\Delta = 1.34E-3"$	$\Delta = 1.98E-3"$	$\Delta = 3.03E-3"$	$\Delta = 1.51E-3"$	$\Delta = 5.38E-3"$
	$k = 262\ k/in$	$k = 2184\ k/in$	$k = 504\ k/in$	$k = 745\ k/in$	$k = 504\ k/in$	$k = 330\ k/in$	$k = 662\ k/in$	$k = 186\ k/in$
	$\theta = 1.11$	$\theta = 0.79$	$\theta = 1.01$	$\theta = 0.98$	$\theta = 1.01$	$\theta = 1.14$	$\theta = 0.96$	$\theta = 1.21$
	$A = 4.22in^2$	$A = 8.79in^2$	$A = 5.89in^2$	$A = 7.63in^2$	$A = 5.89in^2$	$A = 8.10in^2$	$A = 6.88in^2$	$A = 8.64in^2$
Eq. Member	**W12x14**	**W12x30**	**W10x19**	**W10x26**	**W10x19**	**W12x26**	**W10x22**	**W8x28**

$$Stiffness \rightarrow K = \frac{1}{\Delta}$$

$$Masonry\ Shear\ Wall\ Deflection \rightarrow \frac{V * h^3}{3 * E_m * I} + \frac{6 * V * h}{5 * G_m * A} = \frac{V}{E_m * t}\left[4\left(\frac{h}{l}\right)^3 + 3\left(\frac{h}{l}\right)\right]$$

$L = Diaganol\ Length\ of\ the\ Wall$

$V = Arbitrary\ Load\ Applied = 1\ kip$

$h = Height\ of\ the\ wall$

$l = length\ of\ wall$

$E_m = Modulus\ of\ Elasticity\ for\ Concrete = 900 * 1{,}500psi = 1{,}350{,}000\ psi$

$t = Actual\ thickness\ of\ the\ wall$

$\theta = Angle\ of\ Diaganol\ Length\ in\ Radians$

$$k = \frac{2AE}{L}\cos^2(\theta)$$

$A = Equivalent\ area\ of\ steel$

Connection to Main Building

Since the masonry infill walls were going to be used to resist the lateral loads within the greenhouses, there was a major concern with the different drift requirements for the main superstructure and the superstructure of the air towers. The main building was design to meet an industry standard drift of h/400 under wind loads. However, masonry elements have a lower tolerance of h/600 when it comes to deflections. The structural team felt that by designing the whole building to the drift requirements for the masonry in the towers would result in an overdesign for the lateral force resisting system. Therefore, in regions where the steel superstructure in the towers connected to the steel in the main superstructure, a slotted connection was used. With the use of slotted connections, the towers were essentially isolated from the main superstructure. Thus allowing the towers to be design for h/600 and the main superstructure to be designed for L/400. See Figure S.38 for a detail of the slotted connection.

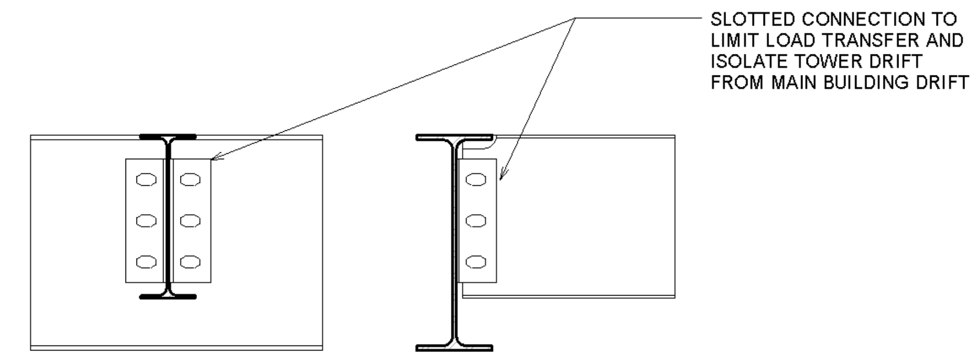

FIGURE S.38: TOWER CONNECTION TO THE MAIN SUPERSTRUCTURE

Appendix I: Foundation System Methodology, Calculations, and Description

Geotechnical Report

From the geotechnical report that was provided for the building site, it was found that the soil that was present was very poor. Multiple borings were drilled throughout the site to a depth of 15 feet. Within these borings, water was found in many of the holes at depths of five to twelve feet. Therefore, a high water table was present and was possibly above the depth of the basement. It was also found that the bearing capacity was very low at 1500 psf. On top of the high water table and poor bearing capacity it was found in some up to 80% organic material was present in some areas which could possibly lead to settlement issues within the building. The geotechnical report recommended that all the fill and organic material and fill it back in with suitable soils. It also recommended that conventional spread and strip footings with a minimum dimension of 24” be used for the foundation.

Spread Footing

By following the recommendations of the geotechnical report, the structural team at Synthesis believed that the implementation of spread and strip footings was going to be an efficient foundation for the Vertical Farm at 5500 W. Silver Spring Drive. Through preliminary analysis of the gravity load that was going to be experienced by some of the foundations, it was found that many of the foundations were going to overlap one another and combined footings were going to be needed to use spread footings. Therefore, the structural and construction teams started developing new ideas to try to create an efficient foundation system.

Mat Slab

With spread footings overlapping one another, the structural team felt that a mat slab foundation could be used to support the building as well as deal with the hydrostatic pressure that would be experience by the high water table. Through conversation with design professionals and the construction team at Synthesis, it was found that implementing a mat slab was going to be extremely expensive compared to other foundation system because of the amount of concrete that was going to be needed. Therefore, more research was done to develop the most efficient foundation system.

FIGURE S.39: TYPICAL GEOPIER WITH A SPREAD FOOTING

FIGURE S.20: FOOTING SIZES WITH 1500 PSF BEARING CAPACITY

Preliminary Footing Sizes
Column 2-c
$DL = 1.2[67psf * (38' - 4*20')*3+67psf*(38'\text{-}4X10') = 216\ kips$
$LL = 1.6[100psf * (38' - 4*20')+80psf*(38'\text{-}4 * 20') *2+80psf(38'\text{-}4''*10'0)]=368k$
$Area = \frac{(216k + 368k) * 1000\frac{lb}{k}}{1500\ psf} = 389ft^2 = 20'x20'\ footing$
Column 2-D
$DL = 1.2[67psf * (38' - 4*20')*4+67psf*(38'\text{-}4X10') = 277\ kips$
$LL = 1.6[100psf * (38' - 4*20')+80psf*(38'\text{-}4 * 20') *3+150psf(38'\text{-}4''*10'0)]=509k$
$Area = \frac{(277k + 509k) * 1000\frac{lb}{k}}{1500\ psf} = 524ft^2 = 23'x23'\ footing$

*Therefore, assuming each column is center on the footing, they will overlap in the 20'direction

FIGURE S.21: FOOTING SIZES WITH 6000 PSF BEARING CAPACITY

Preliminary Footing Sizes with Geopiers
Column 2-C
$DL = 1.2[67psf * (38' - 4*20')*3+67psf*(38'\text{-}4X10') = 216\ kips$
$LL = 1.6[100psf * (38' - 4*20')+80psf*(38'\text{-}4 * 20') *2+80psf(38'\text{-}4''*10'0)]=368k$
$Area = \frac{(216k + 368k) * 1000\frac{lb}{k}}{6000\ psf} = 97ft^2 = 10'x10'\ footing$
Column 2-D
$DL = 1.2[67psf * (38' - 4*20')*4+67psf*(38'\text{-}4X10') = 277\ kips$
$LL = 1.6[100psf * (38' - 4*20')+80psf*(38'\text{-}4 * 20') *3+150psf(38'\text{-}4''*10'0)]=509k$
$Area = \frac{(277k + 509k) * 1000\frac{lb}{k}}{1500\ psf} = 131ft^2 = 12'x12'\ footing$

Geopier

Through research, the structural and construction team at Synthesis decided that the most efficient design for the foundation was using Rammed Aggregate Geopiers. By contacting a design professional at Ground Improvement Engineering, it was deemed that out site was a perfect place to implement Geopiers. It was also found that a typical 30" Geopier could support a 100 kip load, and the center to center spacing for strip footing must not exceed 12' o/c. Also with the borings only going down 15' and the basement and footings going down a similar depth, it was unknown if more organic material was lower in the soil. Geopiers have been proven to limit total and differential settlement within the building. From Geopiers design manual, certain design parameters were found, these parameters are shown in Table ###.

FIGURE S.22: FINAL FOOTING SIZES, REINFORCING AND NUMBER OF GEOPIER

Footing Sizes and Geopier Elements				
Footing Number	Size	Reinforcing	Column Load	Geopier Elements
1-A	4'x4'x1.5'	(5) #4 Each Way	45 k	1
1-B	4'x4'x1.5'	(5) #4 Each Way	80 k	1
1-C	5'x5'x1.5'	(5) #5 Each Way	152 k	2
1-D	10'x10'x2.5'	(11) #7 Each Way	584 k	6
1-E	6'x6'x1.5'	(8) #5 Each Way	182 k	2
1-F	4'x4'x1.5'	(5) #4 Each Way	94 k	1
2-A	4'x4'x1.5'	(5) #4 Each Way	64 k	1
2-B	9'x9'x2.5'	(9) #7 Each Way	536 k	6
2-C	10'x10'x2.5'	(11) #7 Each Way	635 k	7
2-D	11'x11'x3'	(10) #8 Each Way	825 k	9
2-E	11'x11'x3'	(10) #8 Each Way	797 k	8
2-F	6'x6'x1.5'	(8) #5 Each Way	207 k	3

FIGURE S.23: FINAL FOOTING SIZES, REINFORCING AND NUMBER OF GEOPIER

Footing Sizes and Geopier Elements				
Footing Number	Size	Reinforcing	Column Load	Geopier Elements
3-A	4'x4'x1.5'	(5) #4 Each Way	64 k	1
3-B	9'x9'x2.5'	(9) #7 Each Way	539 k	6
3-C	10'x10'x2.5'	(11) #7 Each Way	637 k	7
3-D	11'x11'x3'	11) #7 Each Way	827 k	9
3-E	11'x11'x3'	11) #7 Each Way	792 k	8
3-F	6'x6'x1.5'	(8) #5 Each Way	206 k	3
4-A	4'x4'x1.5'	(5) #4 Each Way	38 k	1
4-B	4'x4'x1.5'	(5) #4 Each Way	83 k	1
4-C	4'x4'x1.5'	(5) #4 Each Way	99 k	1
4-D	5'x5'x1.5'	(5) #5 Each Way	145 k	2
4-E	4'x4'x1.5'	(5) #4 Each Way	111 k	2
4-F	4'x4'x1.5'	(5) #4 Each Way	92 k	1

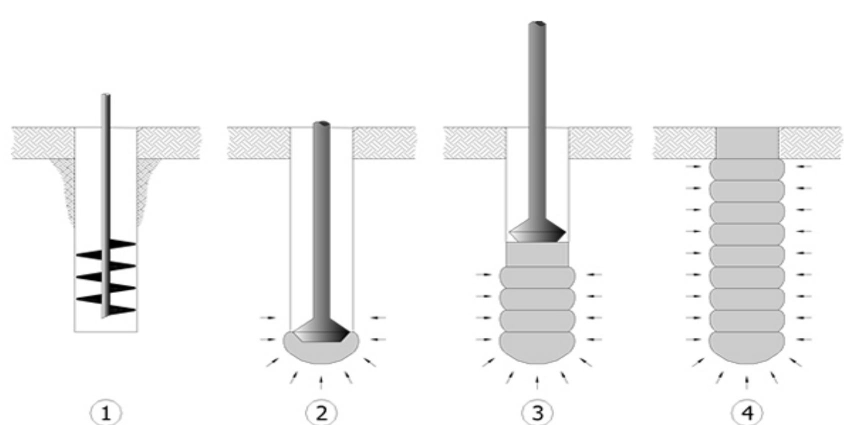

FIGURE S.40: GEOPIER INSTALLATION PROCESS (COURTESY OF GEOPIER)

Footings with Geopiers

With the use of Geopiers, the soil capacity could be quadrupled. With a quadrupled bearing capacity, the size of the spread and strip footing was greatly decreased to the point that they would not overlap with one another. Thus making Geopiers with spread and strip footings the most efficient foundation system. Prior to trying to design with Geopiers, a rough preliminary calculation as done with the same assumed column load as previously to see how much the footing size could be expected to decrease. These calculations can be seen in Table S.21. After the gravity system was designed, final footing sizes, the amount of reinforcing and number of Geopier elements was calculated. These Values can be seen in Table S.22 and Table S.23.

Retaining Walls

With the high water pressure, all of the foundation walls had the high possibility of seeing high saturated soil loads. To ensure that this high load was not present at all times though, a drainage system was developed to remove the hydrostatic pressures. A corrugated pipe will installed around the whole perimeter of the building. These corrugated pipes will then connect to a duplex sump pump that is could be serviced from the basement. This water will then be used in the buildings grey water system. In the event that these sump pumps failed, the foundation walls were designed to be able to handle completely saturated soils.

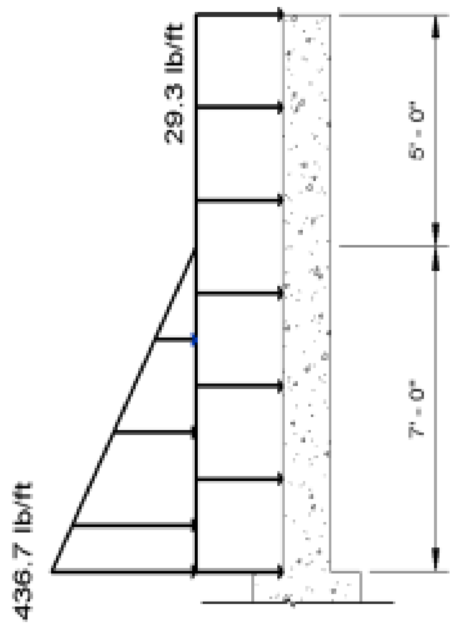

FIGURE S.41: FOUNDATION LOADING

TABLEE S.24: FOUNDATION RETAINING WALL DESIGN

Foundation Wall Design (North, South, East)					
Parameters	Values	Calculations	Values	Calculations	Values
γ(w)	62.4 pcf	$Load\ 1 = (5')^2 * 100psf = 147\ lbs$	@ 9.5'	h (thickness)	16"
Int. Friction Angle (∅)	0.79 rad	$Load\ 2 = 0.29 * 100psf * 7' = 205\ lbs$	@ 3.5'	$d(actual) = 16 - 1.5 - \frac{1"}{2}$	14"
Surcharge (q)	100 psf	$Load\ 3 = 1/2 * (62.4pcf * (7')^2) = 1529\ lbs$	@ 2.33'	$a = \frac{A_s * f_y}{0.85 * f'c * b}$	$1.96A_s$
Height (h)	12'	BASE SHEAR	1432 lbs	$M_u = 0.9 * 60 * A_s(d - \frac{a}{2})$	
Depth to Water	5'	OVER TURN MOMENT	4630 ft-lbs	A_s(required)	0.867 in^2/ft
Depth Below Water	7'	M(max)	384.9 in-k		
$k_o = 1 - \sin(0.79)$	0.29	V(max)	8481 lbs	Try #8 @ 10"o/c	0.948 in^2/ft
f'c	3000psi	M(u)=M(max)*1.6	615.82 in-k	A_s for ρ(vert. min)	0.230 in^2/ft
Clear Cover	1.5"	V(u)=V(max)*1.6	13570 k	A_s for ρ(hor. min)	0.384 in^2/ft
Assumed Rebar Dia. (#8)	1"	$d(required) = \frac{13570}{(0.75 * 2 * sqrt(3000) * 12)}$	13.76"	Use a 16" Wall w/ #8 @ 10" Vert. on Interior and #4 @ 12" Vert. on Exterior, and #4 @ 12" Hor. on Each Side	

Appendix J: Sustainability and LEED

Sustainability and LEED

One of the main goals of the entire team at Synthesis was to create a building that was sustainable and was environmentally friendly during the lifecycle of the building. Early in the design process, the structural team decided on a steel superstructure due to the chance of creating a lightweight superstructure. From here, research was done to determine ways to reduce the impact the buildings structure would have on the environment. It was found that structural steel is often referred to as the premier green construction material due to the reduction in greenhouse gas emissions by nearly 50% between 1990 and 2005. Steel is also a world leader in the use of recycled material with recycled content in structural beams and columns of up to 88%. The utilization of Geopiers in the foundation, also created a green substructure. Geopier is an advocate of using local reclaimed aggregate such as concrete and glass. The implementation of Geopiers also reduces the carbon footprint of the building as opposed to more traditional methods. While many LEED points due not come from the structural system, the structural team managed to contribute to a green building by using building materials located within 500 miles of the site (2 LEED points). For the full LEED analysis and breakdown of all the points that will be achieved during construction, see the [Construction Report].

Appendix K: Considerations to Move

With Growing Power wanting this building to be a prototype to be built throughout the United States, the structural team developed regions of where the building will be controlled seismically and where the building will be control by high winds. Areas that will need to use higher R-values for the SidePlates are shown in Figure S.43. Areas with high winds that will most likely be able to use SidePlates with a lower response modification factor are shown in Figure S.42. While many sites fall under moderate to low wind and seismic loads, these maps helped to see regions of the United States where additional lateral considerations may need to be taken into account.

The structural team developed two main categories for adaptability with the structural system. The first was consideration that may need to be changed when moving the building from one site to the next. These considerations include the truss member sizes and polycarbonate thickness within the greenhouses due to the wind pressures and snow loading being different depending on the region. When moved to a seismic regions, knee braces may be need to be added to the deep beam to ensure that it does not roll, and reinforcing may need to be added in some areas of the slab to ensure that excessive cracks do not occur in areas of high stresses. Another major consideration that would need to be changed as the building moves is the foundation sizes, types, and the implementation of Geopiers since soil properties vary drastically throughout the United States. These considerations can be seen in Table S.25.

The other category that the structural team developed is designs that will stay constant when moving the prototype from site to site. These include the uniform bay sizes and greenhouse modules so that bays can be added or subtracted easily without having to redesign the how structural system. Also, the gravity designs for the greenhouses and main building will remain the same because the assumptions made with the live loads for the building. Another major area that would stay the same is the lateral system, eccentrically braced frames with SidePlates will be used in all design throughout the United States to keep the construction of the lateral system the same. The design constants can be seen in Table S.26.

TABLEE S.25: CONSIDERATIONS FOR CHANGE

Considerations for Change	
Topic	Structural Change
Greenhouse	Truss Member Sizes
	Polycarbonate Thickness and Span
Enclosure/Foundation	Retaining Wall Sizes and Reinforcing
	Keeping/Eliminating Geopiers
	Foundation Types
Gravity/Lateral System	Knee-Brace in Seismic Regions
	Reinforcing in Areas of High Stress in Seismic Regions
	Eccentrically Braced Frames and SidePlate Sizes
	Infill Wall and Sizes in Towers

TABLEE S.26: DESIGN CONSTANTS

Design Constants	
Topic	Same Design
Greenhouse	Repeatable 19’-2” Spacing
	Gravity Design
	General Truss Layout
Enclosure/Foundation	Façade System
Gravity/Lateral System	Gravity Design
	Steel Superstructure
	Uniform Bays
	Deep Girder to Pick up Differing Elevations
	Implementation of EBF’s and SidePlates
	Layout of Lateral Elements

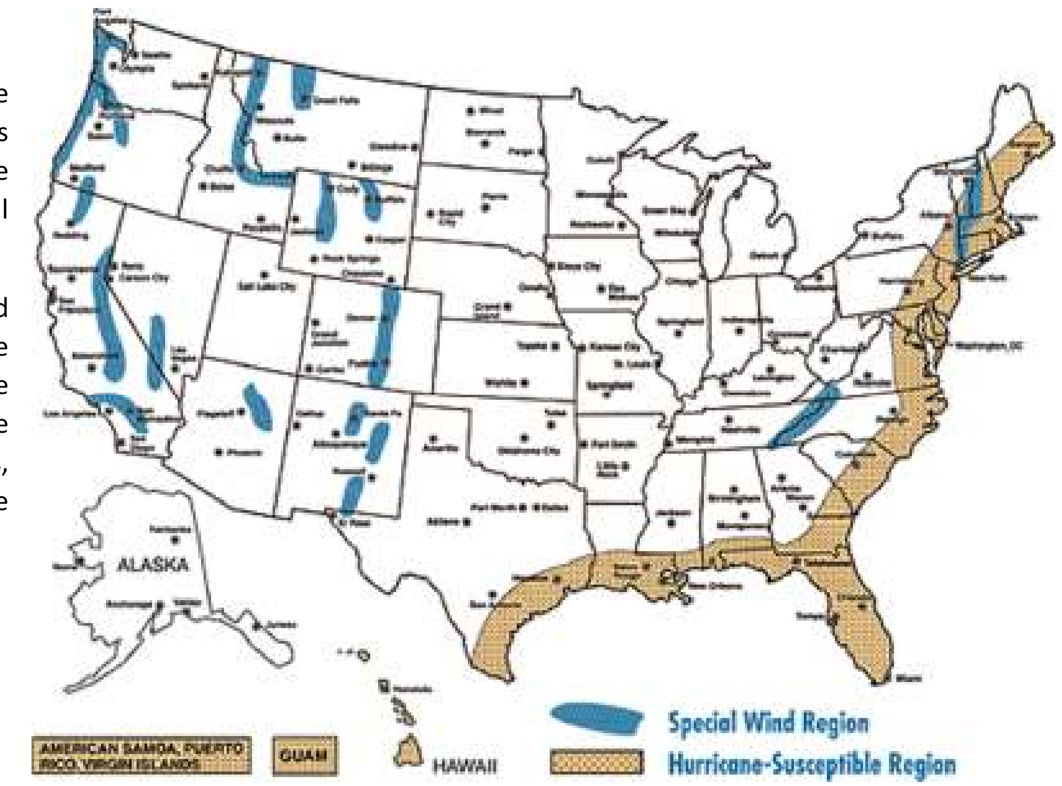

FIGURE S.42: HIGH WIND REGIONS IN THE UNITED STATES (COURTESY OF WBDG)

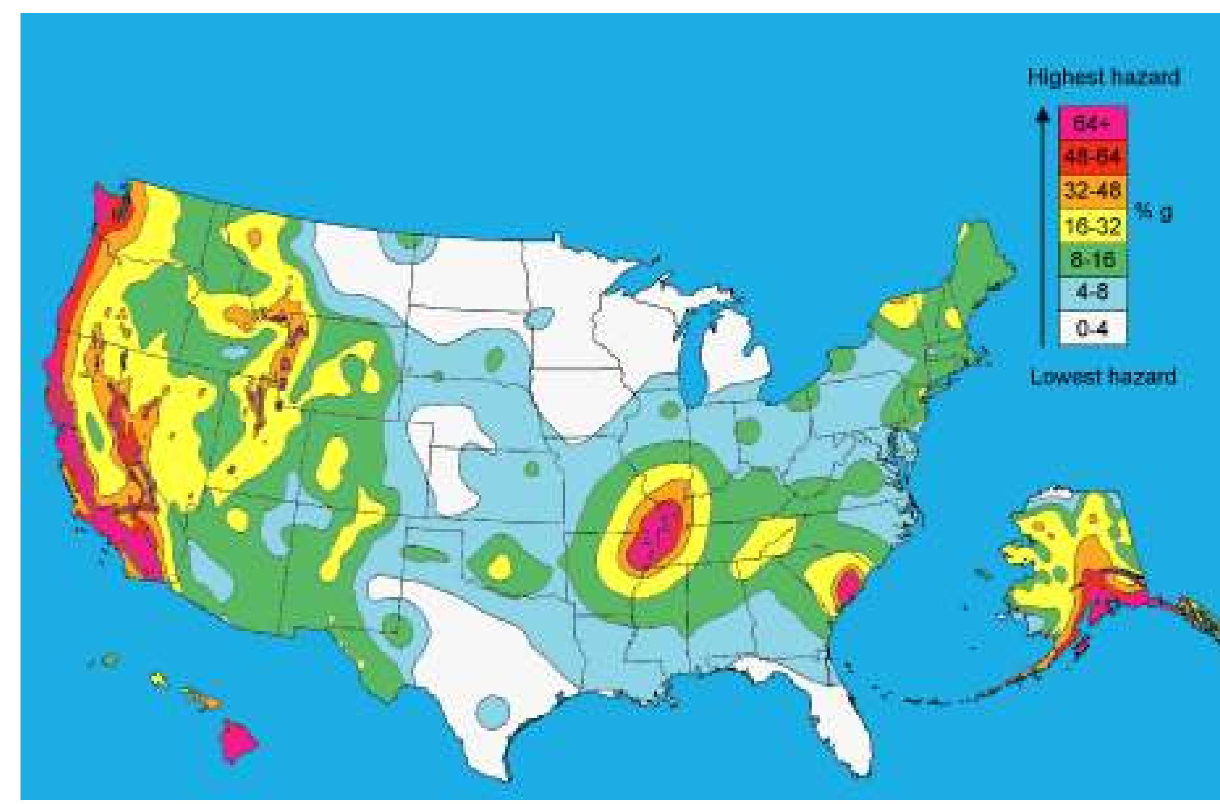

FIGURE S.43: SEISMIC MAP FOR THE UNITED STATES (COURTESY OF USGS)

Appendix L: References

Below is a list of all of the references used in this report.

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