2.0 STRUCTURE

2.1 INTRODUCTION

This section presents the structural basis of design. It includes design data, material specifications, loadings and other parameters, and a discussion of the proposed structural systems.

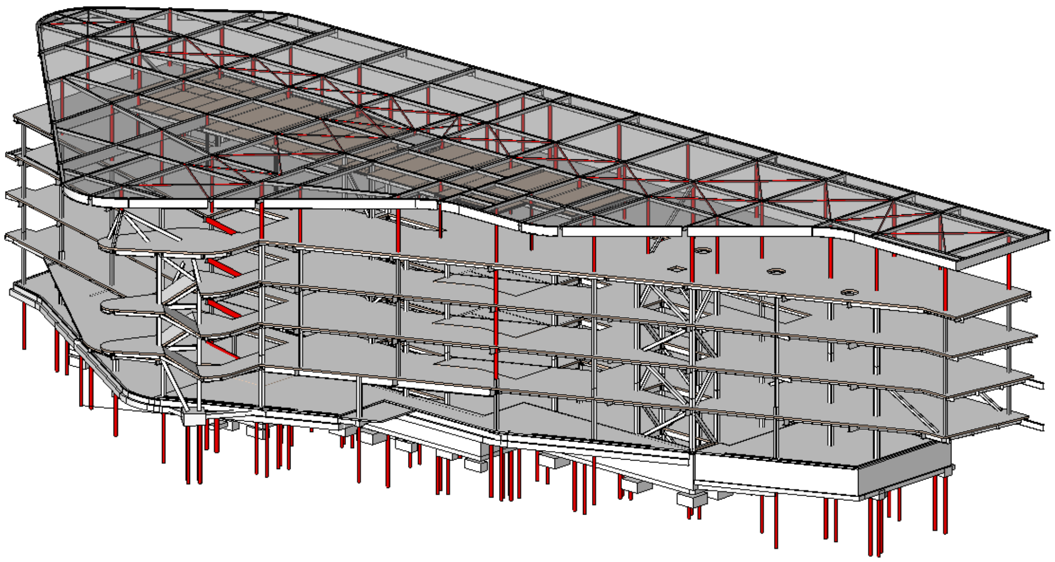


Figure 1 - 3D View at DD

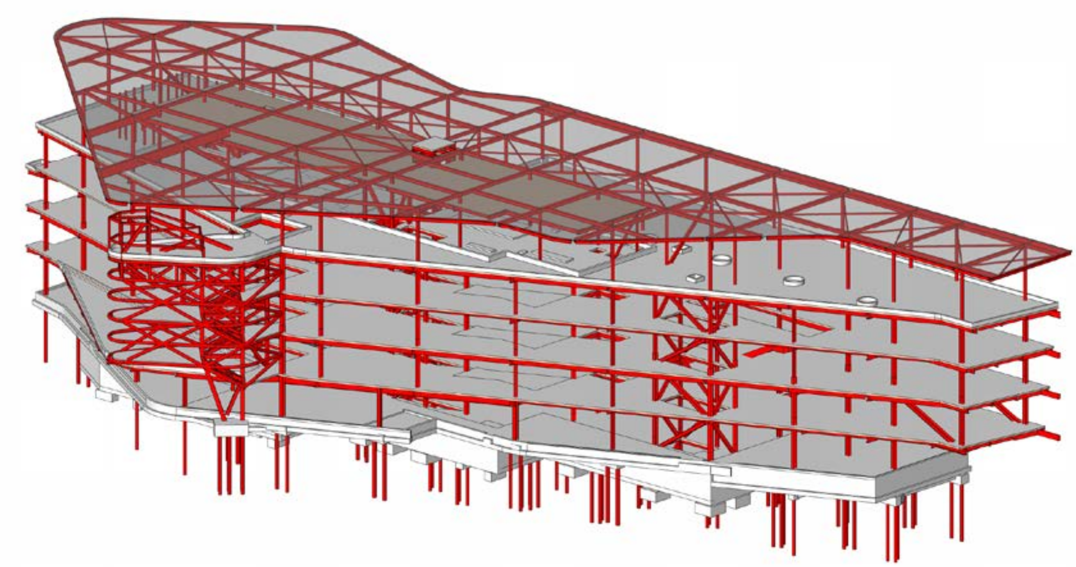


Figure 2 – 3D view at CD

This section builds on the structural reports issued at Schematic Design and Design Development stages. In this section, SD denotes Schematic Design, DD denotes Design Development, and CD denotes Construction Documentation.

The report should be read together with the structural CD drawings.

The primary aims of the structural engineering design are to:

- support the design loads specified whilst meeting the serviceability criteria
- be flexible and adaptable to meet future needs
- allow cost effective and economic construction through structural optimization
- implement the sustainability goals of the project as much as possible.

2.2 DESIGN CRITERIA

2.2.1 GENERAL

All structural elements conform to the 2008 Building Code of the City of New York (BCCNY) and all applicable Federal, State and Local laws, standards and regulations. In addition, the following codes, standards and specifications apply where more stringent and as modified by the Building Code.

- AISC 360, “Specification for Structural Steel Buildings”
- ACI 318, “Building Code Requirements for Structural Concrete” and Commentary
- AISC, “Code of Standard Practice for Steel Buildings and Bridges”
- AISC, “Specification for Structural Joints Using ASTM A325 or A490 Bolts”
- AWS D1.1, “Structural Welding Code”
- ACI 530, “Building Code Requirements for Masonry Structures”
- AISC Design Guide 11 “Floor Vibrations Due to Human Activity”

2.2.2 DESIGN LOADS

The following basic load allowances are proposed for the design of this project. Suitable load factors, load reductions and load combinations are to be applied as permitted or required by code.

Dead Loads
All dead loads are calculated as the self-weight of the structure using the following unit weights:

Normal Weight concrete	150 pcf
Structural steelwork	490 pcf.

Superimposed Dead Loads
Superimposed dead loads are the weight of other permanent building construction.

Finishes and superimposed dead load assumptions have been revised during CD stage design to reflect architectural finishes and requirements for mass in floors for acoustic requirements. All non-structural topping slab and housekeeping or equipment pads are expected to be normal weight concrete with a unit weight of 150 pcf.

As of SD stage, the assumed superimposed dead load allowance for partitions was 20 psf. As of DD and CD, however, the partition allowance is 12 psf, based on the actual partition weights estimated from the architectural drawings and to maintain consistency with the load used on the adjacent colocation building. As recommended in ASCE 7, this partition load allowance is not applied where live loads exceed 80 psf.

Refer to the structural loading plans for assumptions on superimposed dead loads.

Live Loads
Indicative live loads are shown in the table below. Refer to the structural loading plans for full details of design live loads and the areas in which they are applied.

Table 1 - Live Load

Occupancy		Uniform Live Loads	Concentrated
	Reducible	Not Reducible	
Classrooms and Auditoria (mobile seating)	100 psf	-	1000 lbs
Classrooms and Auditoria (seating fixed to the floor)	60 psf	-	1000 lbs
Corridors, Stairs and Exits	-	100 psf	1000 lbs
General Assembly Areas (cafés, lobbies, etc)	-	100 psf	2000 lbs
Min Roof Load (L_r)	-	20 psf	-
Mechanical / File Rooms	-	based on actual weight, but not less than 150 psf	based on actual weight, but not less than 2000 lbs
Offices	100 psf (see below)		2000 lbs
Circulation areas (bridges, walkways etc.)	60 psf	-	-
Roof areas with mechanical equipment	-	based on actual weight, but not less than 150 psf	based on actual weight, but not less than 2000 lbs
Cellar areas supporting MEP equipment	-	based on actual weight, but not less than 150 psf	based on actual weight, but not less than 2000 lbs

The minimum live loading required by BCCNY for office occupancy is 50 psf. The code requires that assembly spaces, such as function rooms, or classrooms without fixed seating be designed for a live load of 100psf.

One of the design aims is to create a flexible space. In support of this aim, it is proposed to adopt a minimum live load of 100 psf across the majority of the floors area as indicated on structural loading plans so that, for example, office spaces could be converted to classrooms without fixed seating or to assembly spaces in the future without the need for re-assessment of the structure or structural intervention.

This design for flexibility results in an increase in factored area loads used for beam and slab design of circa 20 to 25% and a similar percentage increase in weight (and therefore construction cost) for the steel structure supporting the office areas (i.e. areas which could, in compliance with the BCCNY, be designed for the lighter loading). Note, however, that the relationship between loading and steel tonnage is not linear and the percentage increase will generally be less due to other design criteria such as footfall vibration.

Live load reduction is applied as permitted by the BCCNY.

Wind Loads

Wind load effects on the structure as a whole and on individual elements are considered with recognition of the variation of wind pressures over the height of the building and orientation to the wind.

Design wind pressures are calculated in accordance with the BCCNY based on the following parameters:

Building - Enclosed
Exposure Category = C
Wind Importance Factor = 1.15
Base Wind Speed = 98 mph

Snow Loads

Design snow loads are calculated in accordance with the BCCNY based on the following parameters.

Ground Snow Load = 25 psf
Exposure Category = C
Snow Importance Factor = 1.1
Drifting per code recommendations

Seismic Loads

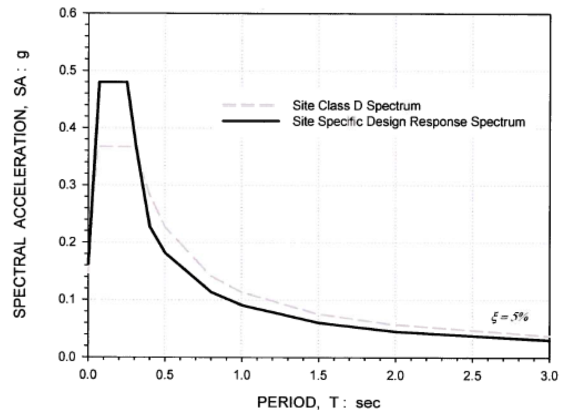
Design seismic loads were calculated during SD and DD phases in accordance with the BCCNY based on appropriate code base parameters. At these earlier stages, S_{DS} = 0.37g and S_{D1} = 0.11g.

At CD stage, a site specific response spectrum was developed by geotechnical consultant Mueser Rutledge and is shown on figure below (Refer to Mueser Rutledge – Site Specific Spectra Analysis Letter dated July 29, 2013).
The design spectrum considered supersedes any generic site classification used during previous stages of design. As specified by geotechnical consultant, the values of Spectral Acceleration (SA) at 0.2sec, S_{DS} (0.48 g) and SA at 1sec, S_{D1} (0.09g), cannot be used to generate an alternate response spectrum for site.
Seismic Importance Factor = 1.25
Design Spectral Accelerations: as per site specific study *
Site Class = as per site specific study*

Seismic Design Category = C
Response Factor = 3 (no specific seismic detailing)

*Determined by the project geotechnical consultant Mueser Rutledge.

Because the building has a fundamental natural frequency hovering around 1 sec, the effect of Mueser Rutledge's site-specific spectra analysis has been to decrease the seismic loads on the structure.



Period T(s)	SA (g) ξ = 5%
0.00	0.16
0.07	0.48
0.25	0.48
0.31	0.37
0.40	0.23
0.50	0.18
0.80	0.11
1.00	0.09
1.50	0.06
2.00	0.05
3.00	0.03

NOTES:

1. Recommended design spectrum developed for the foundation level based on a site-specific study performed by MRCE for 2/3 of the MCE within the guidelines of the 2008 NYC Building Code (NYCBC).
2. The design spectrum should be applied at the foundations, at approximately Elev. +5. Should the foundation level elevation change, MRCE shall review the changes and revise the design spectrum accordingly.
3. The design spectrum shall be considered in its entire period range and supersede any generic code site classification. The values of Spectral Acceleration (SA) at 0.2 sec, S_{DS} (0.48 g), and SA at 1 sec, S_{D1} (0.09 g) cannot be used to generate a response spectrum for the site.
4. No site class should be provided on the design drawings. Site class should indicate "As per a site-specific study."
5. Structural damping ξ of 5%.

CORNELL NYC TECH NEW YORK NEW YORK		
MUESER RUTLEDGE CONSULTING ENGINEERS 14 PENN PLAZA - 225 W. 34TH STREET, NY, NY 10012		
MADE BY: JR CH'KD BY: SN	DATE: 7/29/13 DATE: 7/29/13	FILE NO. 11856
DESIGN RESPONSE SPECTRUM		FIGURE NO. S-1

Groundwater and Flood Loading
Design groundwater level in normal service conditions is +6.63ft NAVD88 per the geotechnical consultant's recommendation. Where the effect of groundwater loading would be beneficial (e.g. where water uplift pressure would reduce foundation loads) it is ignored.

During a flood condition there is potential for the ground around the building to become saturated to the prevailing flood level. This is the design flood groundwater level. The recommended minimum building first floor elevation for the campus taking account of revised flood level predictions after Hurricane Sandy is +15.63ft NAVD88. The +15.63ft elevation stems from FEMA's "advisory base flood elevations" (ABFE) issued in February 2013, which gives +13' NAVD88 as the 100-yr ABFE (ref: Phase 1 Site Development 50% Design Development Report March 2013, Appendix H). As of SD and DD stages, a +15.63' elevation was also taken as the design flood groundwater level until a 500-yr flood elevation could be confirmed. This is an extreme-case design condition for which reduced safety factors are reasonable.

In CD stage, a revised design flood groundwater elevation of +14' NAVD88 is assumed. This elevation is the revised 500-yr flood from FEMA, dated December 2013. Using the same philosophy as in SD and DD stage, reduced safety factors are applied to the hydrostatic loading resulting from the +14’ flood, as this represents an extreme event.

Design Load Combinations
Load combinations for elements designed according to strength design or allowable stress design are in accordance with the BCCNY.

2.3 PERFORMANCE CRITERIA

2.3.1 SETTLEMENT

Building settlement is influenced by the type of site soil, foundation construction and building loads. The foundation design adopts footings bearing on rock and mini-piles to rock and settlements are expected to be minimal and not structurally significant.

2.3.2 DEFLECTIONS

The structural elements are sized to limit deflections to the values specified in the BCCNY.

Anticipated deflections of floors are limited to the following maximum values unless more stringent requirements are noted on structural drawings.

Table 2- Structural Deflection Limits

Structural Deflection Limits		
Floor Beams	Live Load Live Load + SDL	Span/360 Span/240 or 1.5”
Spandrel Beams	Live Load + SDL	Span/480 or 0.75”
Roof Members *In addition to limits for Floor Beams	Total Uplift Load	Span/240 or 1.5”
Floor Supporting Masonry Walls	Live Load + SDL Total Load	Span/600 Span/360
Structure Supporting PV Panels	Live Load, Snow or Wind Live Load + SDL	Span/180 Span/120
Transfer Trusses and girders	Live Load + SDL Total Load	Span/600 or 1.0” Span/480
Atrium Glass wall Horizontal member	Wind (Horizontal Defl) Live or Snow Load+Total Dead oad	Span/180 Span/480

Note: SDL = Superimposed Dead Load

2.3.3 SIDESWAY

The lateral stability elements are sized to limit horizontal movement due to seismic loads to the values specified in the BCCNY.

For the base building, the wind interstory drift is limited to h/425, where "h" is story height, under 10-yr wind. Overall wind building drift is limited to H/500, where "H" is total building height, under 10-yr wind.

For the PV Canopy, where the façade is composed of metal panels able to withstand more distortion, the wind interstory drift is limited to h/180, where "h" is story height, under 10-yr wind.

2.3.4 BUILDING DESIGN LIFE

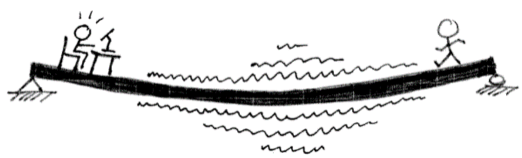
It is assumed that the building design life is 50 years.

2.3.5 FIRE RESISTANCE PERIODS

Fire resistance periods for building structural elements are as determined by the architect or code consultant.
Arup has been advised that structural fire rating required for slab design is 1 hour.

2.3.6 VIBRATION CRITERIA (HUMAN COMFORT)

The perception of vibration is both subjective and variable between individuals. Acceptability criteria for various occupancies have been developed using acceleration limits by both the AISC and the International Organization for Standardization (ISO).



The performance of steel-framed floors deemed susceptible to excessive vibrations due to walking excitation are assessed with reference to the provisions and criteria published by the American Institute of Steel Construction and accepted international design practice.

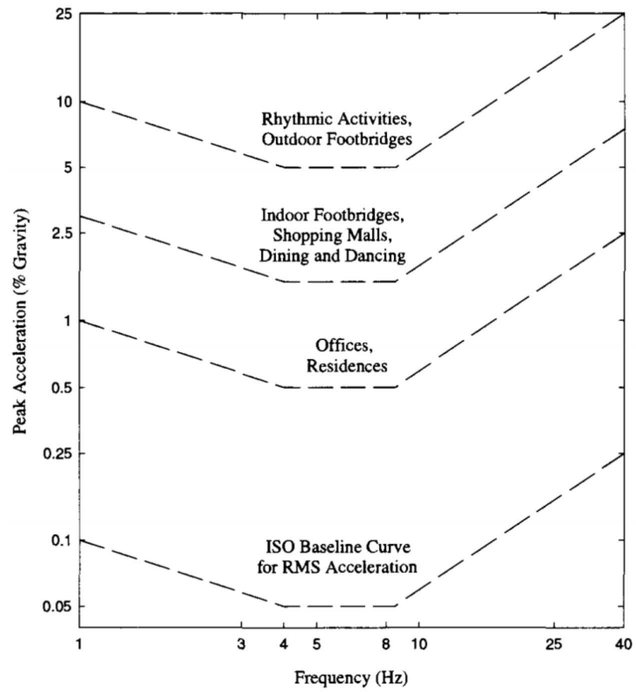


Figure 3 - Recommended Peak Acceleration for Human Comfort for Vibrations due to Human Activities (Allen and Murray, 1993; ISO 2631-2)

The design team has been advised by Cornell that the building will not house any equipment or processes which require special vibration control. Up to DD stage, the building had a multi-purpose space at second floor level, where anticipated uses included physical exercise and games. Restrictions on the use of that space and adjacent spaces were to be determined based on the available vibration performance of structure designed without special vibration control, and it was acknowledged this could limit the available use of the space. This area was cantilevered from the south end of the building at schematic design stage, creating significant issues for vibration control. The cantilever was reduced during design development stage and the vibration control issues became less significant. At CD stage, the space is a regular assembly space with no special vibration performance requirements.

Optimization of vibration performance

It should be noted that the published criteria represent the intended minimum performance level. While the analytical methods used to compute the performance are accepted within the industry, there are intangible effects, such as the random nature of normal concrete cracking, and the position of partitions and furniture that will have an effect upon the actual dynamic response of the structure.

The dynamic response of a continuous suspended floor structure varies greatly over the area of the floor plate. Vibrations will generally be lowest at columns, larger at mid spans and greatest at free edges of the structure near the last column bay or near atrium large floor openings. There are normally marked differences in the magnitude of vibrations at mid-span of an internal bay compared to mid-span at an end or corner bay. For certain structures the difference could be in the order of 2-4 times. In cases, it may be prudent to relax the desired vibration performance criteria in certain positions in order to maintain cost efficiency of the structure while achieving the desired performance over the greater proportion of the structure.

The location of walking routes across the floors and the walking space at which people are expected to walk will also affect the induced vibrations.

Vibration performance targets for the floor systems in the main program area in the building are shown in the following table. These criteria are expressed in terms of response factor R and, for comparison, rms velocity. The response factor R is the multiplier on the ISO baseline curve for rms acceleration and therefore reflects the variation in acceptable acceleration occurring at varying frequencies. R is used as the basis of design for human perception of comfort.

Criterion Curve	**V_{rms} Velocity Level**		**Response factor (3)**	**Description of User**
	(μin/s)	**(dB) Ref: μin/s**		
Workshop (ISO)	32,000 (1)	90 (1)	R = 8	Distinctly felt vibration. Appropriate to workshops and non-sensitive areas.
Office (ISO)	16,000 (1)	84 (1)	R = 4	Felt vibration. Appropriate to offices and non-sensitive areas.
Atrium balcony	(2)	(2)	R=16 -20	Adequate for light usage/light structure public spaces
Stair	(2)	(2)	R = 24 - 32	Adequate for heavy (public spaces) to light (office) stair usage

Note: (1) Reference codes ISO2631, BS6472, ASHRAE
(2) Codes in reference (1) do not specify value,
(3) Arup considers R factor the most appropriate metric for human perception of comfort.

2.3.7 FLOTATION STABILITY

The BCCNY does not provide specific guidance on the overall stability factor of safety against flotation. By reference to ASCE7 2.4.1 Case 7, the BCCNY requires the following strength check:
Design satisfies: $0.6D \geq H$

This is too onerous for a stability check against flotation in an extreme design event such as the design floodwater ground level (and the code does not intend it for such a check). Arup proposes to check the building for flotation stability on the following basis:

Design satisfies:
1.0D + 0.8SF ≥ 1.1 H

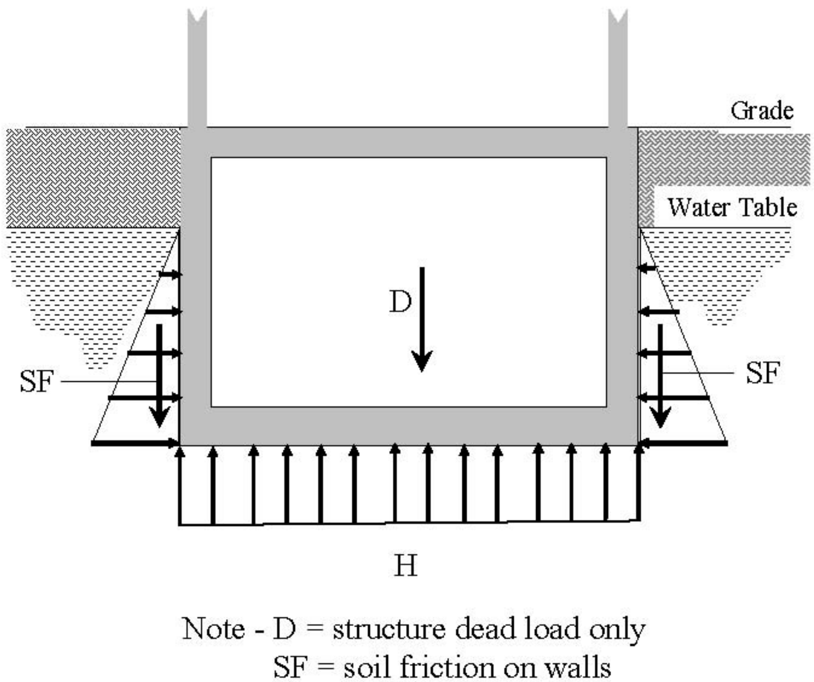


Figure 4 - Flotation Stability

This approach is consistent with common practice in design of underground structures.

In this relationship:
D = dead load of structure plus reasonable allowance for superimposed dead load that is likely to be permanent

H = water uplift force based on the design extreme flood elevation (namely, +15.63’ NAVD88 as of SD and DD stages, reduced to +14’ as of CD stage).

SF – the design does not rely on SF for this building as there is limited friction between soil and wall waterproofing membrane, hence SF = 0

If additional resistance to flotation is needed, options include:
- provision of additional dead weight (for example, by thickening slabs)
- permanent tie-down solutions (for example, permanent rock anchors/ tension piles)

During the design development stage it was determined that the foundation system included mini piles socketed into rock. These piles supplied the required uplift capacity. During CD stage of design foundation system includes footings with rock tie anchors and mini piles socketed into rock. Combination of tie anchors and mini pile can supply required uplift capacity.

2.4 STRUCTURAL MATERIALS

The selection of materials responds to the structural performance objectives (strength, stability, stiffness, architectural intent and the sustainability considerations of the project, etc). In addition, the future maintenance of the structure is considered and materials selected to reduce on-going maintenance to a practical minimum.

2.4.1 REINFORCED CONCRETE

Normal weight concrete is used for all structural elements. The minimum compressive strength f'c at 28 days is:

- f'c = 5000 psi for cellar walls, cellar base slab, columns and foundations
- f'c = 4000 psi for all other concrete (including slabs on metal deck)

The use of supplementary cementing products such as high volume fly ash and blast furnace slag is included in the construction specifications. Fly ash is available locally.

Some points to note regarding the use of high volume fly ash concrete (HVFAC) include:

- Slower rate of strength gain – note effect on strike times especially for flat works, walls, columns, etc.
- Proper curing becomes more important.
- HVFA concrete sets quite differently to 'normal' concrete. This needs to be understood by the concrete finisher before the process starts.

2.4.2 CONCRETE REINFORCING

ASTM A615, Grade 60 deformed bar is the typical rebar used. ASTM A706 is used for rebar requiring welding.
ASTM A185 is used for welded wire fabric.
A high volume of recycled steel is specified.

2.4.3 STRUCTURAL STEEL

Plates, Channels and Bars: ASTM A572, Grade 50, typical.
Wide flange shapes: ASTM 992.
Structural tubing: ASTM A500 Grade B.
Pipes: ASTM A53, Type E.
Miscellaneous shapes and angles: ASTM A36.
A high volume of recycled steel is specified.

High Strength Bolts
ASTM A325 Type N connection typical.
ASTM A325 Type SC connection at moment connections.
ASTM A490 Type SC connection at brace end connections where appropriate.
Machine bolts: ASTM A307 typical.
Anchor bolts: ASTM F1544 Grade 35

Welding electrodes: E70xx for structure steel.
Metal decking and accessories: ASTM A611, Grade C and D or ASTM A653-94.
Welding electrodes: E70xx low hydrogen for metal deck.

2.5 FOUNDATION MATERIALS

The building has a single level cellar extending under part of its footprint.

At 100%SD stage, geotechnical investigation of the site was incomplete and at a preliminary stage. In particular, there was significant uncertainty over the elevation of rock layers and competent bearing strata. The project geotechnical engineer – Mueser Rutledge – provided initial recommendations for foundation design including the use of spread footings. Following SD stage the Geotechnical Summary Report was issued on February 15, 2013 and the design of the cellar and foundations was changed to include support on mini-piles.

Data on groundwater elevations is provided in the Geotechnical Report. In addition to the general design groundwater level, the design considers a higher (extreme-case) design flood groundwater level. This is discussed in the Flood Loading section of this report. The design flood groundwater level is used to determine lateral water pressures on cellar walls, uplift pressures on the cellar base slab and any measures needed to counteract overall flotation (such as additional dead weight or pile uplift capacity).

The previous design assumptions adopted for SD stage are summarized below. Some of these assumptions are now superseded.

- Foundations at cellar level are spread footings on rock (or on mass concrete fill to rock if competent rock is at shallow depth below the footings). Foundations of building columns that terminate at Level 01 are extended to rock so that the entire building is supported on the same stratum (bedrock), to minimize differential settlements.

- The seismic site class is D.

- Top of competent rock is around -0.4ft NAVD88 elevation. In practice, there may be considerable variability in the top of rock elevation. The allowable bearing pressure on rock will be 20tsf, which is intermediate between the range of values indicated in the geotechnical section of the schematic design report.

- The extent to which groundwater levels around and below the cellar will respond to floodwater elevation is not known and this information is required to finalize the cellar and foundation design. In the meantime, Arup has assumed - perhaps conservatively - that the ground around the cellar will become saturated to the extreme flood event elevation which is taken as +15.63 ft NAVD88. This is therefore the water head that has been used to calculate hydrostatic pressures on cellar walls and on the base slab.

The design assumptions adopted for Design Development, and based on the detailed geotechnical investigation and report, were different in some respects from those considered at schematic design as outlined above. The design assumptions adopted for

DD are summarized below. Some of these assumptions are now superseded in CD stage:

- The building will be supported on mini-pile foundations. Per the Geotechnical Summary Report dated February 15, 2013, the borings indicated that the depth to bedrock was greater and more variable than assumed for the initial foundation concept (at SD). Because of the need for deep foundations over at least a portion of the building footprint and significant uplift forces - and taking account of constructability comments received from the construction manager which indicated that a piled solution to the entire building would be cost effective - - the foundation concept has been amended to support the entire building on drilled mini-piles socketed into bedrock. The piles provide compressive, tensile (uplift) and lateral capacity.

- The seismic site class is assumed as D. The geotechnical engineer is studying the potential to reduce the response spectrum accelerations from the code–specified values via a site specific analysis. The results of this study have not been received in time for incorporation into the structural design. A reduction in spectral accelerations will likely reduce the weight / cost of some of the structural steel components in the building.

- Top of competent rock is assumed as per Geotechnical Summary Report.

- Design groundwater level is at El. +6.63ft NAVD88. As discussed in the Flood Loading section of this report, the design assumes that the ground around the cellar will become saturated in an extreme flood event and the design flood groundwater elevation is taken as +15.63ft NAVD88. This is therefore the water head that has been used to calculate hydrostatic pressures on cellar walls and on the base slab.

The design assumptions adopted for Construction Documentation, and based on the detailed geotechnical investigation and report, are different in some respects from those considered at schematic design and design development as outlined above. The design assumptions adopted for CD are:

- The building is supported on footings bearing on rock and drilled mini-pile foundations. Per the Geotechnical Summary Report dated February 15, 2013, the borings indicated that the depth to bedrock is expected to be closer to cellar level foundation on the east side of footprint and consistently deeper on west side of cellar footprint. Because of the expected high variation of top of bedrock across the site, the close proximity of the top of rock elevation to the foundation level on the east side of the cellar level footprint, and significant uplift forces - and taking account of constructability and technical feasibility comments received from the construction manager and geotechnical consultant - the foundation concept has been revised to support the entire building on a combined system including footings bearing on rock (with rock anchors) and pilecaps with drilled mini-piles socketed into bedrock. The combination of footings with tie-down anchors and piles provides all required compressive and tensile (uplift) capacity. Due to larger footing contact area with rock and the pre-stress provided by anchors, friction under the footings now provides the entire lateral capacity needed for the base building. Except for Stair #5, the piles are no longer designed to provide lateral resistance. Because the whole building bears on rock, differential settlements are minimized.

- The response spectrum accelerations determined via a site specific analysis by geotechnical engineer are implemented in the CD stage of design. A reduction in spectral accelerations slightly reduced the cost of some of the structural steel components in the building.

- Top of competent rock is assumed as per Geotechnical Summary Report.

- Design groundwater level is at El. +6.63ft NAVD88. As discussed in the Flood Loading section of this report, the design assumes that the ground around the cellar will become saturated in an extreme flood event and the design extreme flood groundwater elevation is taken as +14ft NAVD88. This is therefore the water head that has been used to calculate hydrostatic pressures on cellar walls and on the base slab.

2.6 GRAVITY STRUCTURAL SYSTEM

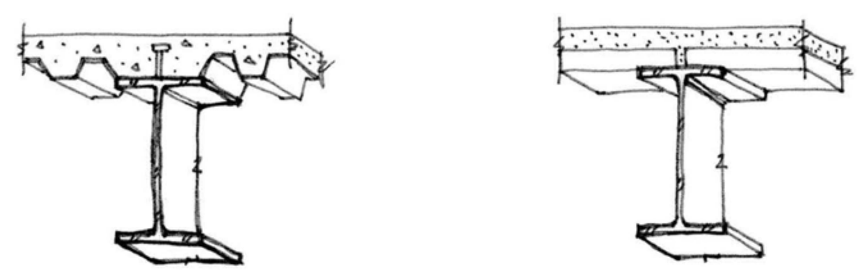
2.6.1 STEEL FRAME WITH COMPOSITE CONCRETE SLAB ON METAL DECK

The selection of the floor system warrants careful consideration as it accounts for a large component of structural cost. In addition, it has a direct impact on construction time as well as on the future flexibility of the space. The options for supporting the gravity loads of the building are numerous. Several common structural systems were discussed in the pre-Schematic phase report and pertinent aspects of that discussion are repeated here.

The general structural grid was agreed to be typically 30ft x 30ft, however the form of the building dictates that there is considerable variability in actual grids. The base 30ft x 30ft grid is an efficient size for a variety of common structural steel and reinforced concrete floor systems and integrates effectively with the architectural design module.

Floor System Types
Several common structural systems evaluated at SD stage are illustrated below.

Common Steel Systems
Steel beams with composite concrete slab on metal deck
Steel beams with precast planks and topping

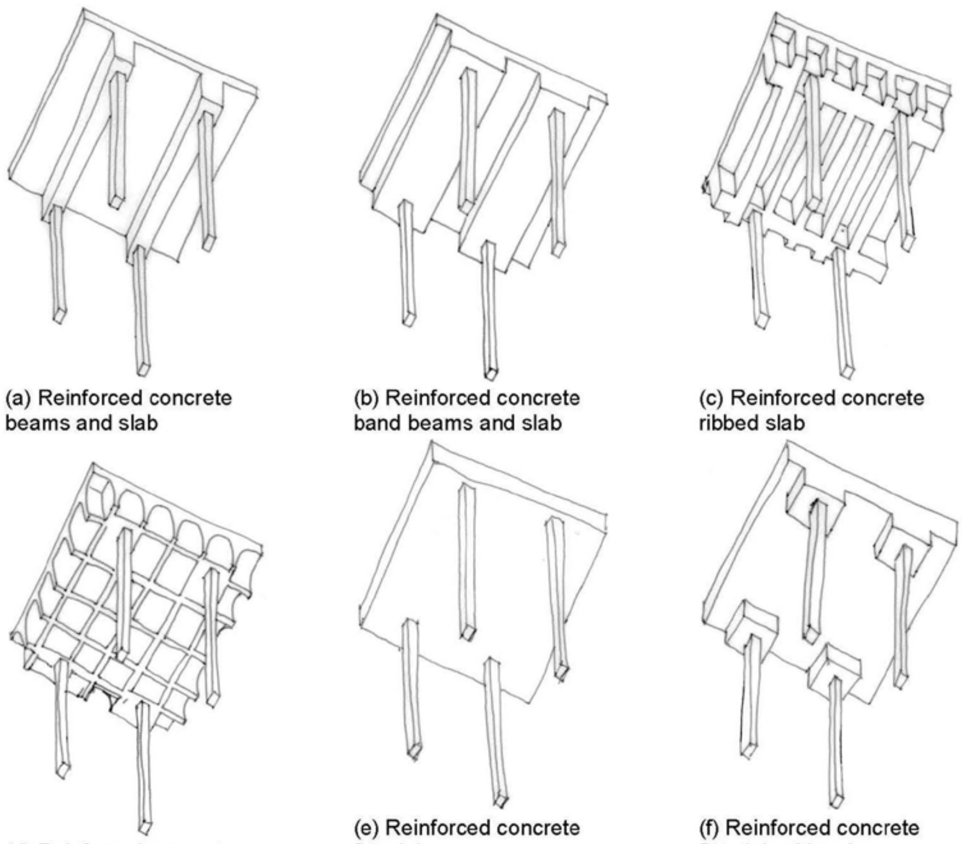


(a) Steel beams with slab on metal deck (b) Steel beams with precast plank flooring

Figure 5 - Common Steel Floor Systems

Common Concrete Systems

- Reinforced concrete beams with reinforced concrete slab
- Reinforced concrete band beams with reinforced concrete slab
- Reinforced concrete ribbed slab
- Reinforced concrete waffle slab
- Reinforced concrete flat slab
- Reinforced concrete flat slab with capitals

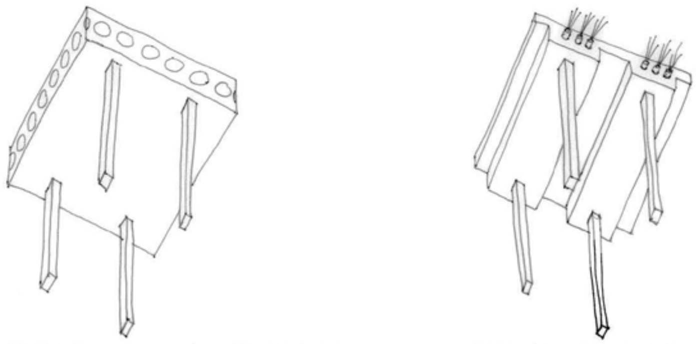


(a) Reinforced concrete beams and slab
(b) Reinforced concrete band beams and slab
(c) Reinforced concrete ribbed slab
(d) Reinforced concrete waffle slab
(e) Reinforced concrete flat slab
(f) Reinforced concrete flat slab with column capitals

Figure 6 - Common Concrete Floor Systems

Special Concrete Systems (may require specialty contractors):

- Reinforced concrete voided flat slab
- Post-tensioned reinforced concrete slabs with or without beams



(a) Reinforced concrete voided flat slab

(b) Post tensioned reinforced concrete slab with band beams

Figure 7 - Special Concrete Floor Systems

Floor System Considerations

Selecting the structural floor system involves a number of considerations which are outlined below.

<u>Design Considerations</u>

- Ability to accommodate long spans
- Ability to accommodate large cantilevers
- Flexibility to accommodate future changes in program
- Ability to accommodate openings near columns
- Ability to accommodate irregular geometry
- Structural depth (affects floor heights and MEP coordination)
- Thermal mass
- Fire resistance
- Soffit appearance
- Response to floor vibrations

<u>Material Considerations</u>

- Sustainability (embodied energy, recycled content)
- Material cost
- Weight (and effect on foundations)

<u>Constructability Considerations</u>

- Local Skills (contractor familiarity)
- Ease/speed of construction
- Ease of transport to island site (road vs. barge delivery)
- Potential for prefabrication
- Weather dependency of erection

Structural Floor System	Design								Material		Constructability				
	Long Spans	Long Cantilevers	Openings Near Columns	Irregular Geometry / Irregular Grids	Structural Depth	Response to Floor Vibrations	Supports Non-conventional Structural Action	Flexibility - Post-Construction Openings and Strengthening	Embodied Energy / Recycled Content	Material Cost	Weight	Ease/Speed of Erection	Potential for Prefabrication	Erection Independent of Weather Conditions	System Cost
Steel beams with slab on metal deck	■	■	■	■	□	▣	■	■	Depends on local market	Depends on local market	■	■	■	▣	■
Steel beams with precast plank flooring	■	■	▣	□	□	▣	■	▣			■	■	■	▣	▣
Reinforced concrete beams and slab	□	□	▣	■	□	■	▣	□			□	▣	□	□	□
Reinforced concrete band beams and slab	□	□	▣	▣	▣	■	▣	□			□	▣	□	□	□
Reinforced concrete ribbed slab	▣	▣	▣	□	▣	■	□	□			▣	□	□	□	□
Reinforced concrete waffle slab	▣	▣	▣	□	▣	■	□	□			▣	□	□	□	▣
Reinforced concrete flat slab	□	□	□	■	■	■	□	□			□	▣	□	□	■
Reinforced concrete flat slab with column capitals	□	□	□	■	■	■	□	□			□	▣	□	□	▣
Reinforced concrete voided flat slab	■	▣	□	□	□	■	□	□			▣	□	□	□	□
Post tensioned reinforced concrete slab	▣	▣	□	▣	■	■	□	□			▣	□	□	□	□

Better	⟷	Worse
■	▣	□

Figure 8 - Relative Comparison of Floor System Attributes

Selected Floor System

During SD stage and subsequent value engineering, the structural floor system was selected in consultation with the owner, the architect and the cost consultants / CM. The selected system is steel framing with composite concrete slab on metal deck. This system is favorable in the key categories indicated above and is the basis of current structural system cost allowances.

The thickness of floor slabs varies to suit loading requirements and the mass required for acoustic and vibration control.

The floor slab is designed for increased loading in areas designated as routes for installation and removal of equipment.

Alternate Floor System for Cost Comparison

During schematic design, it was agreed within the design team and in discussion with the cost consultants that a reinforced concrete flat slab floor scheme had potential to be a favorable alternate scheme. In response, Arup supplied an alternate floor scheme for one typical floor framed as a concrete flat slab with concrete columns to enable a cost comparison between this flat slab and the steel framed system. This scheme was supplied as a separate document to the cost consultants / CM in January 2013, and its cost was estimated in parallel with the steel frame scheme shown on the SD drawing set. During the ensuing VE discussions, it was agreed that the concrete alternate scheme did not offer significant overall advantages and it was agreed to retain the steel framed scheme as the basis of design.

2.6.2 BUILDING CANTILEVERS

The building has various cantilever conditions:

Floor levels L02 - mR at the north end of the building

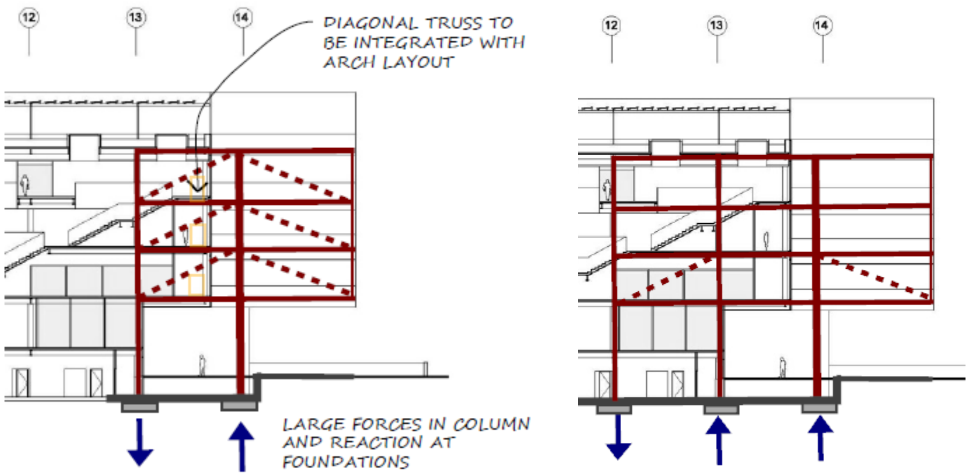


Figure 9 - North Cantilever Overhang at SD (on left) and at DD (on right)

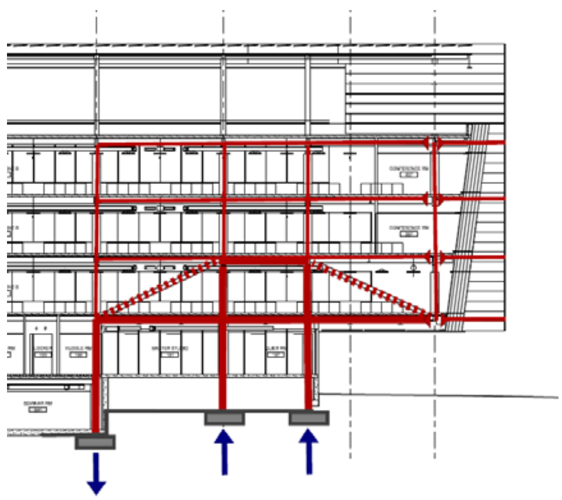


Figure 10 - North Cantilever Overhang at CD stage

At SD and DD stage, the cantilever was uniform at all the floors and approximately 30 feet. At SD it was structured with full story height trusses. In DD, following further coordination and studies, the cantilever structure was changed to a single story-height truss between level L02 and L03, to minimize the architectural impact of diagonal members and improves constructability.

In CD, the cantilever structure scheme remains essentially a single story-height truss between L02 and L03. The overall building overhang now varies from approximately 30 feet at L02 to and 45 feet at mR level. This variations is structured by varying the cantilever of floor beams at each floor level.

Floor levels L02 - mR at the south end of the building

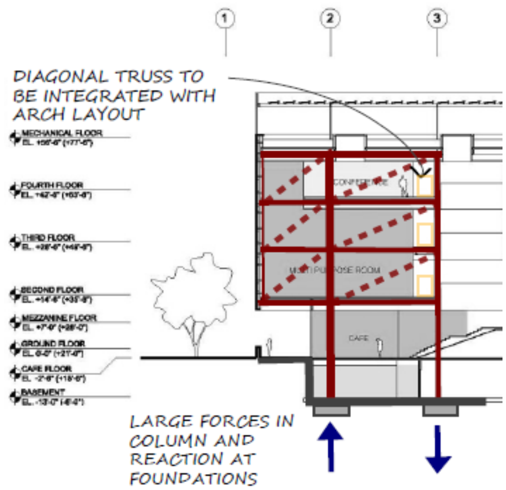


Figure 11 - South Cantilever Overhang (at SD stage)

At SD stage, this cantilever was approximately 20 feet and structured with full story height trusses. The cantilevered space included a multi-purpose room at second floor and conference / meeting rooms. One of the potential uses of the multi-purpose room was basketball games. Control of dynamic effects in – and transmission through – the structure due to activities such as basketball would be challenging and require significant stiffening of the structure and coordination with the acoustic consultant. Appropriate design strategies for this area were considered to include:

- stiffening the structure
- isolating the structure

At DD stage, the south cantilever was reduced significantly and structured with cantilevering beams at each floor level (full story height trusses are no longer required). The room designation at Second Floor was changed to multipurpose room, where anticipated uses included physical exercise / games. To avoid incurring a cost premium on the structure in this area, it was agreed that the program for and restrictions on use of that space and adjacent spaces would be determined based on the available vibration performance of structure which is designed for regular strength and serviceability criteria without special vibration control. This would potentially limit the available use of the space, particularly for physical and high impact or vibration-inducing activities. The design strategy also assumed an isolating floor system above the structural slab.

At CD stage, the south cantilever geometry and design assumptions remained the same as DD stage. The isolating floor system above the structural slab was removed.

Planters at Façade (west side) and Green Roof at Second Floor (east side)

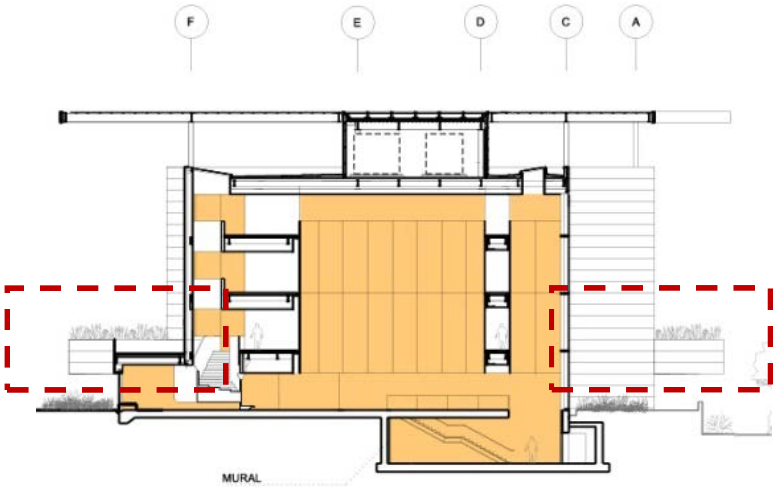


Figure 12 - Planter Cantilevers at Façade

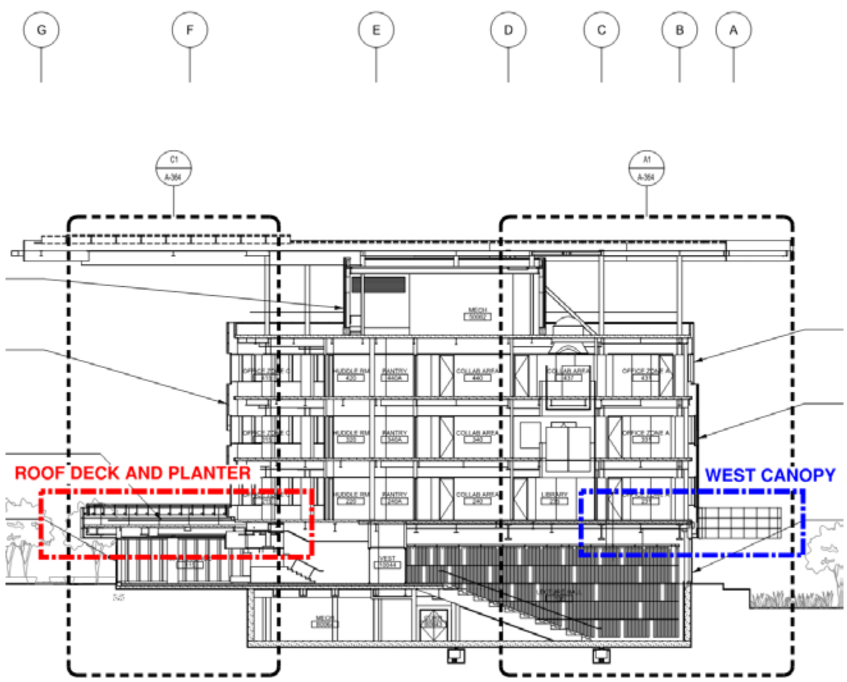


Figure 13 - Planter cantilever at east façade and West Canopy

At SD stage, these cantilevers were variable length up to a maximum of ~21 ft. They were supported by the steel floor beams extending through the cantilever at each location. The cantilevers were heavily loaded due to the combination of soil weight and heavy precast concrete cladding panels supported at the tip of the cantilever.

During DD stage, the west façade cantilevers were significantly reduced in scope and length, straightened in plan and the precast concrete façade cladding was replaced by a lighter metal façade system. Planter cantilevers remained on Level 02 only.

In CD stage, the planters on the west cantilever were removed and it was changed to a west canopy structure cantilevering off from the building. Specific details have been developed to accommodate the thermal break between building interior and exterior. Planter cantilevers remain on Level 02 east side only.

2.6.3 SOLAR PANEL CANOPY STRUCTURE

The building columns are extended through the building roof to support the solar panel canopy structure. The canopy structure is a grillage of steel beams in one horizontal plane supported on steel columns.

The configuration of the canopy grillage has been developed to achieve an optimal combination of:

- Form and aesthetics
- Strength and stiffness with economy
- Durability
- Integration with support requirements of the PV panel system selected
- Daylight penetration where applicable

<u>Schematic Design Stage</u>
Alternative framing / grillage systems were studied in schematic design (SD). A number of the principal systems studied were discussed in the SD report and are reproduced below for completeness. Some of these systems were developed when the orientation of the solar panels was skewed to the structural grid. Although that is no longer the case (panels are now aligned to grid) the structural principles indicated in these diagrams remain applicable.

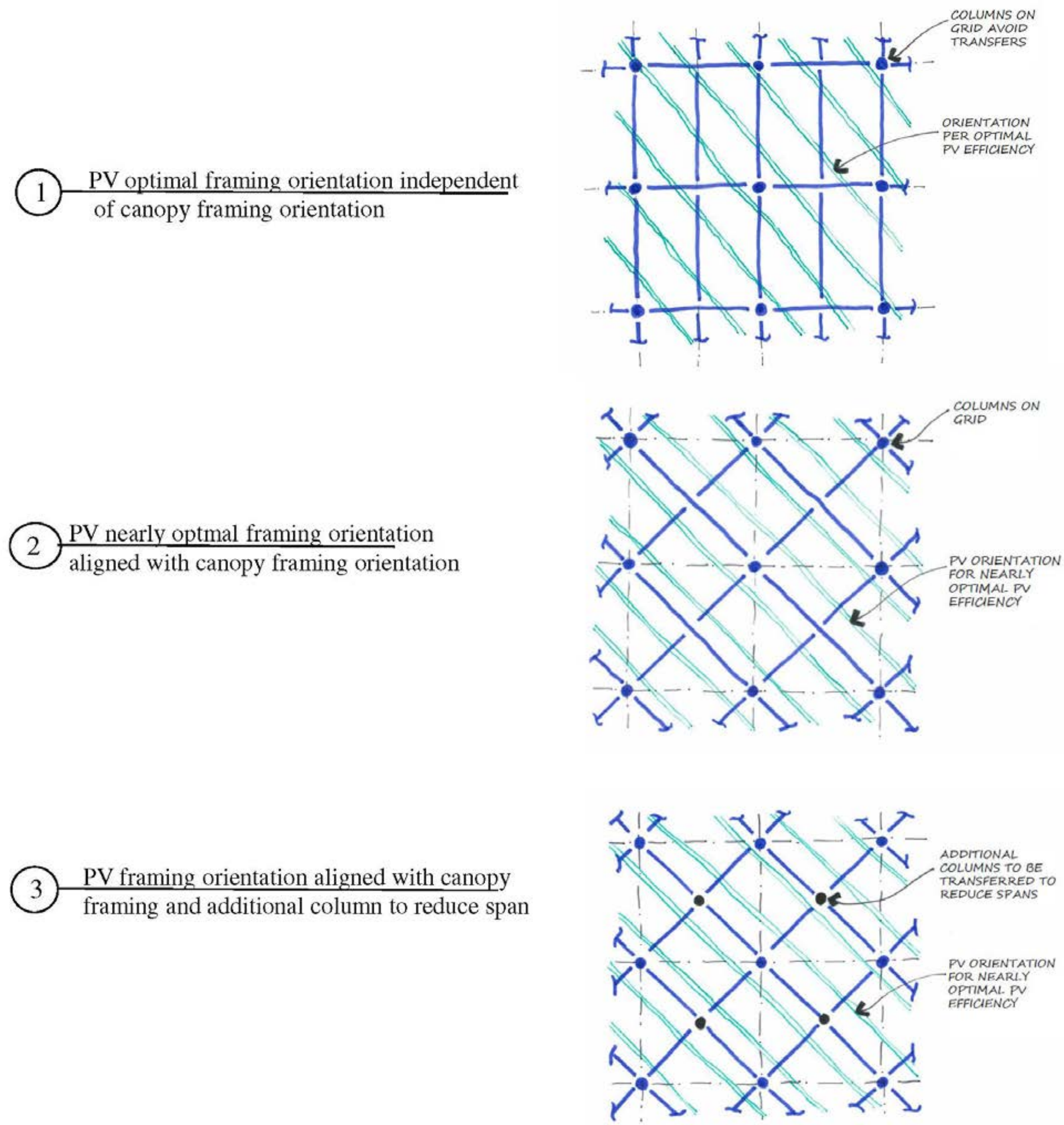


Figure 14 - PV canopy structure framing options (from SD Report)

One of the additional grillage options that was studied on a preliminary basis is a “basket- weave” system (as sketched below). This option has potential for some efficiency in terms of tonnage and consistency in the overall structure depth profile, however it is potentially more complex in terms of constructability and the number of connections / pieces, and additionally it did not at DD - and currently does not at CD - align with the expected scope split between primary / secondary steel provided in the main building contract and tertiary steel in PV package, hence this alternate has not been pursued.

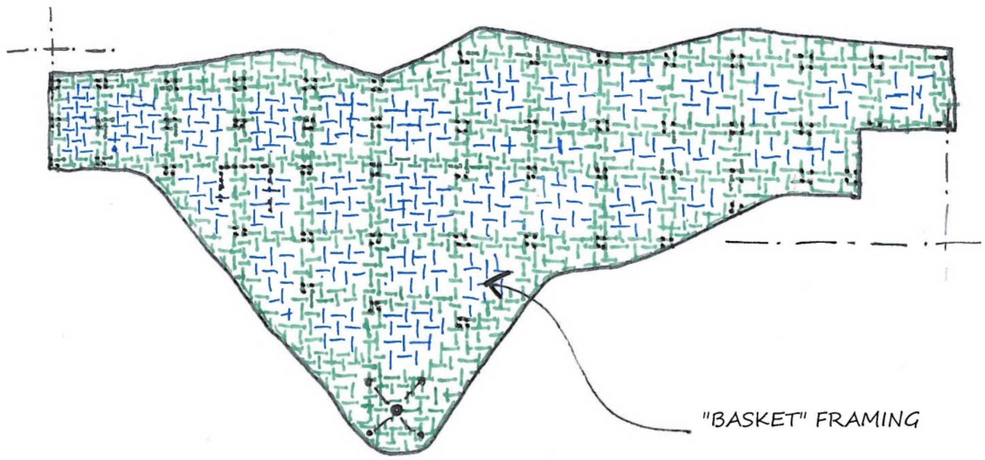


Figure 15 - PV Canopy "Basket-Weave" Structure – Preliminary Scheme Option

<u>Design Development</u>
The scope split between steel which is provided in the building construction package and steel which is provided as part of the PV installation contract is not yet known by the design team. The design team has discussed various alternate framing systems – including arrangements and layers of primary / secondary / tertiary steel - with the project PV consultant Distributed Sun. The actual scope spilt needs to be agreed and confirmed to the design team.

The design team designed the canopy steel to cover two possible schemes / scope scenarios:

1. Primary Only Option
   Building steel provides canopy columns plus primary beams between columns and in-plane stability bracing. All other PV support steelwork is provided by PV contractor.

2. Primary and Secondary Option
   Building steel provides canopy columns, primary beams between columns, secondary beams at 10 ft spacing and in-plane stability bracing. All other PV support steelwork is provided by PV contractor.

These options are illustrated below:

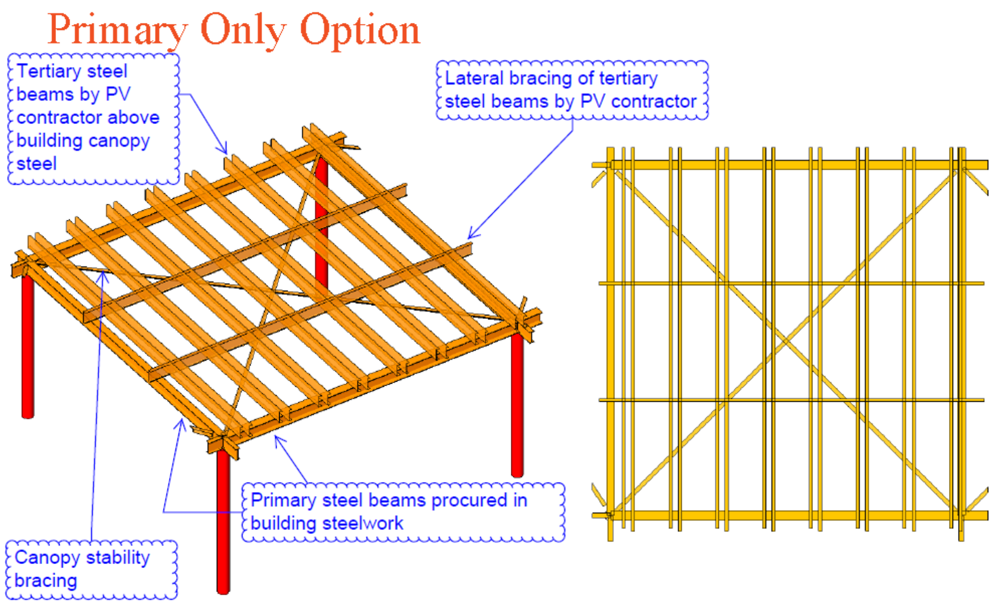


Figure 16 - PV canopy structure – Primary Only option

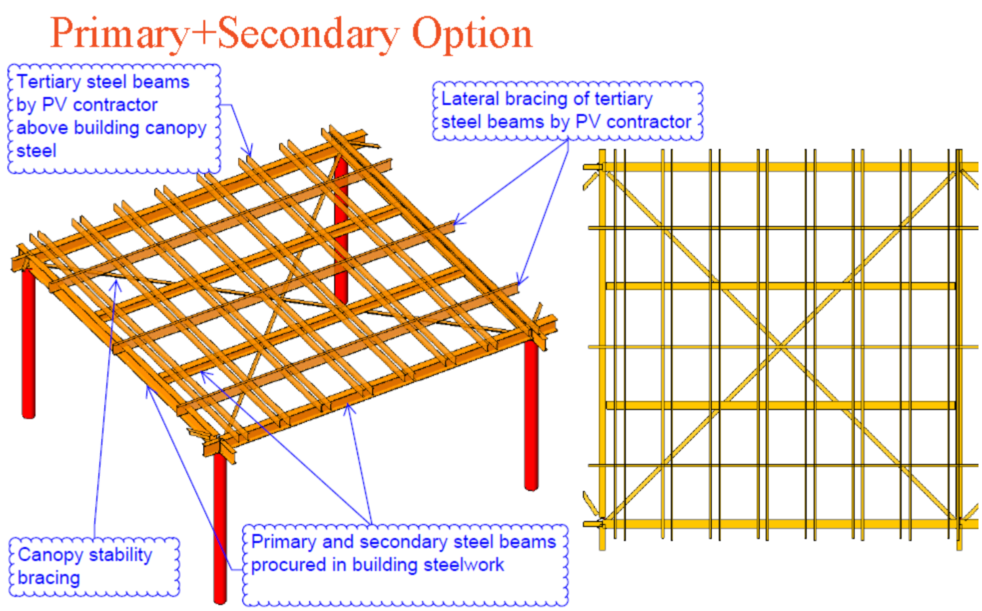


Figure 17 - PV canopy structure – Primary & Secondary option

The architect has instructed that the canopy design should be based on the Primary Only option, but that it should be designed such that it could also accommodate the Primary and Secondary Option to provide additional options for the future PV contractor structure.

Protective coatings will need to be applied to extend the life to first maintenance. Environmental protection for the bare steel framing is assumed to be provided by galvanization or other exterior-grade coating, and is specified by the architect.

The in-plane bracing is connected to the primary building stability system. An alternate system for the canopy utilizing moment frames for lateral stability was also considered, but the braced scheme is more efficient and is currently preferred. The PV canopy structure is a single structure framed without movement joints.

The canopy is designed to cantilever beyond the perimeter columns as indicated on structural drawings. The eastern tip of the canopy above the Tech Plaza is supported on a trussed column that is part of Stair #5 structure.

Other issues including integration at mechanical penthouse roof and thermal bridge detailing are being developed with the architect. The mechanical penthouse roof will have lower steel framing supporting a metal deck. Thermal bridge detail is developed for columns that exist within mechanical rooms and extend to support higher canopy steelwork

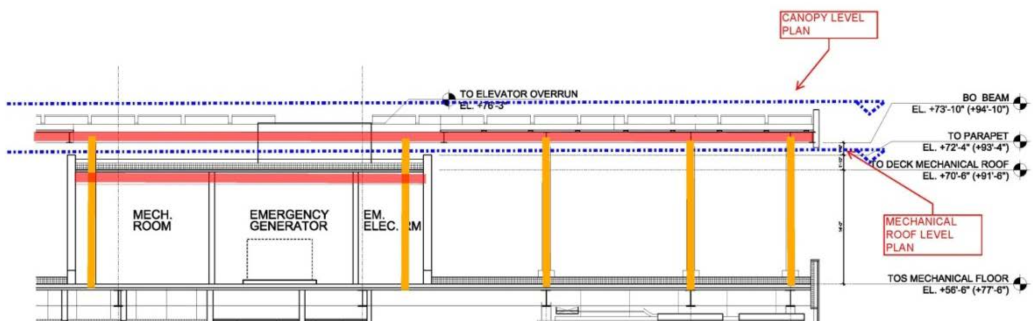


Figure 18 – Schematic of PV canopy structure and MER room roof

<u>Construction Documentation</u>
The canopy structural system has been further developed and coordinated in line with the design and scope assumptions developed in the previous DD stage.

The edge profile on plan of the canopy has been revised to achieve a more rational coordination between architectural form and structural efficiency. Maintenance strategy has been discussed with the project PV consultant. Structural scope distinction versus PV panels and PV supports scope is clearly shown on structural drawings and details. We understand the maintenance strategy will rely on access to PV panels:

- from the mR floor below for areas of the canopy above main building roof,

- by catwalks integrated with tilted panels on the canopy portion over the plaza,
- by crane access from street level for north, west and south canopy cantilevers not accessible from main building mR floor

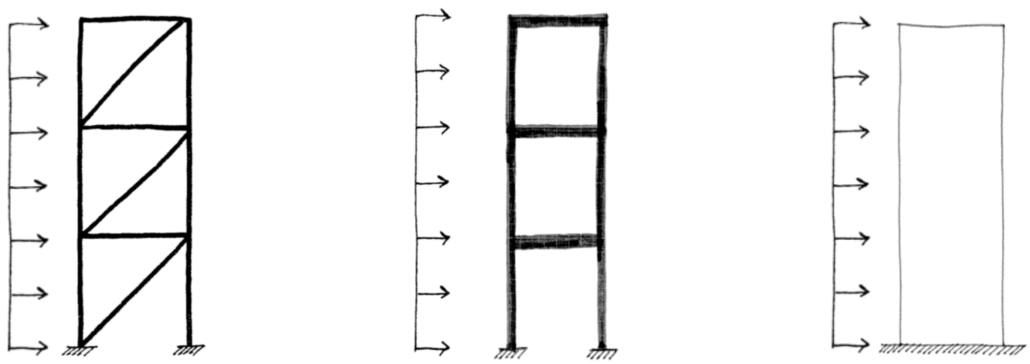
2.7 LATERAL STRUCTURAL SYSTEM

There are various options for resisting the lateral loads acting on the building. The common structural systems were discussed in the pre-Schematic and SD phase reports and pertinent aspects of those discussions are repeated here.

The basic descriptions presented below are for lateral systems which are appropriate for relatively low-rise buildings.

Lateral System Types

- Steel braced frame
- Steel moment frame
- Reinforced concrete shear wall



(a) Steel braced frame (b) Steel moment frame (c) Reinforced concrete shear wall

Figure 19 - Common lateral system types for relatively low-rise buildings.

Lateral System Considerations

The basic considerations for lateral system selection are presented below.

<u>Design Considerations</u>

- Ability to accommodate openings in walls (for doors, MEP, etc)
- Thickness of structural profile (width of structural zone required)
- Ability to receive concentrated forces (such as the tension or compression generated by a truss chord)
- Impact on program layout flexibility (space requirement in the vertical plane)

<u>Material Considerations</u>

- Sustainability (embodied energy, recycled content)
- Material cost

<u>Constructability Considerations</u>

- Phasing of trades (if one trade must proceed in advance of another)
- Local skills (contractor familiarity)
- Ease/speed of construction
- Ease of transport to island site (road vs. barge delivery)
- Potential for prefabrication
- Weather dependency of erection

Structural Lateral System	Design				Material			Constructability		
	Openings in Walls	Thickness	Impact on Program Layout Flexibility	Support of Concentrated Forces	Embodied Energy / Recycled Content	Material Cost	Weight	Ease/Speed of Erection	Potential for Prefabrication	Erection Independent of Weather Conditions
Steel braced frame	■	□	▣	■	Depends on local market	Depends on local market	■	■	■	▣
Steel moment frame	■	□	■	■			▣	▣	■	▣
Reinforced concrete shear wall	▣	■	□	□			□	▣	□	□

Better	⟷	Worse
■	▣	□

Figure 20 - Relative comparison of lateral system attributes.

2.7.1 LATERAL SYSTEM SELECTION AND DESIGN

<u>Schematic Design and Design Development</u>
At SD stage, the structural lateral system was selected in consultation with the architect. The selected system was concrete shear walls. This system is favorable in terms of stiffness, strength and thickness.

Steel braced frames were studied as alternative to concrete shear walls, both during and after SD stage. Either system can achieve the required structural performance in this building. During the value engineering exercise which followed the SD submission, constructability and cost factors were further reviewed with the architect, cost consultant and construction manager and steel braced frames were selected over concrete shear walls for constructability and cost.

Steel moment frames were also considered in SD stage but not pursued, since the alternate of a braced system (either shear walls or bracing) is more efficient and can be readily incorporated in the building.

Wind loads applied to the façade are transferred through the floor diaphragms to the braced frames which in turn transfer the loads to the foundations. Seismic loads caused by the restraint of mass within the buildings are resisted in the same manner. As of DD, foundations resist lateral loads through lateral pile capacity and passive resistance of the earth on the foundations and cellar walls.

The building acts as one continuous structure i.e. there are no movement joints.

Details for transfer of lateral forces in slabs around openings in the diaphragm and where the diaphragm reduces in plan area (such as along the central galleria and above the main lobby / entrance area) have been developed.

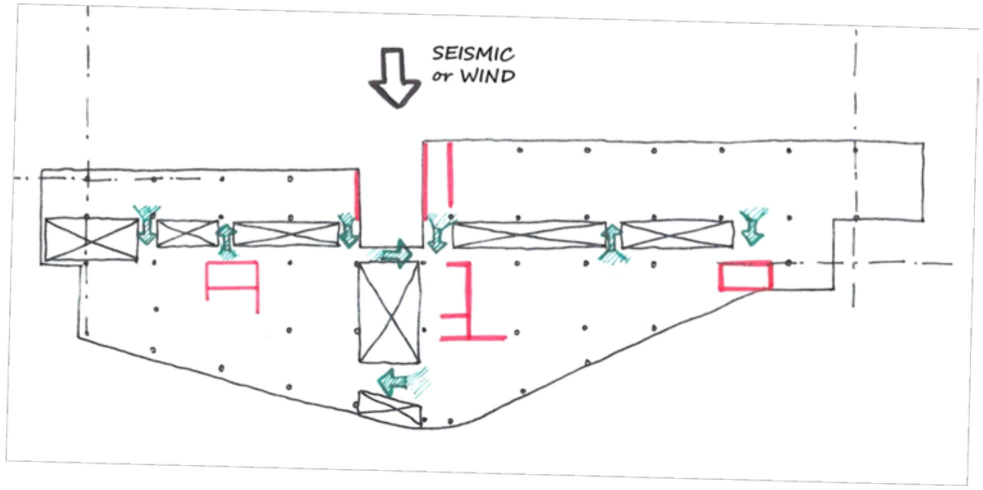


Figure 21 – Lateral structural system frames (indicative) and force transfer around openings in slab diaphragm requiring defined load path.

<u>Construction Documentation</u>
In CD stage, a braced frame system is still utilized. A few braced frame bays, such as at Grid 7, have been eliminated as part of normal design progression and optimization. The natural periods of the buildings are:

- 1st Mode: 1.5s (torsional)
- 2nd Mode: 1.1s (N-S translation)
- 3rd Mode: 1.0s (E-W translation)

The unfactored base shears are:

- Seismic: 860k (each direction)
- Wind:
    - N-S: 550k
    - E-W: 880k

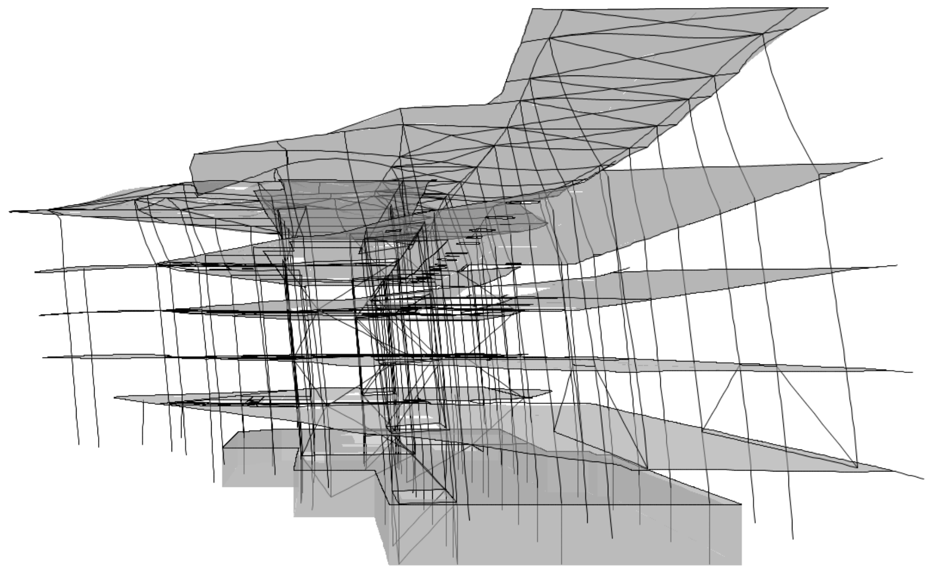


Figure 22 – Lateral system model showing amplified E-W wind deflection

As noted in the SD/DD section of this report, there are large openings in the slab diaphragm which merit special consideration. At the galleria and atrium openings, steel trusses are provided across the effective “bridges”. See figure below, which shows a bridge with greatly amplified displacement. Diaphragm load arising from wind or seismic actions are transferred from the slab to steel drag beams via shear studs welded to the top flange. The drag beams then feed the load to steel trusses placed across the bridge. The truss bridge, in effect, acts as a link beam or coupling beam between portions of the building. On the other side, of the bridge the process is reversed – load is transferred to the drag beam and back into the slab. From there, the load is carried by the slab to the vertical braced frame system (not shown in figure).

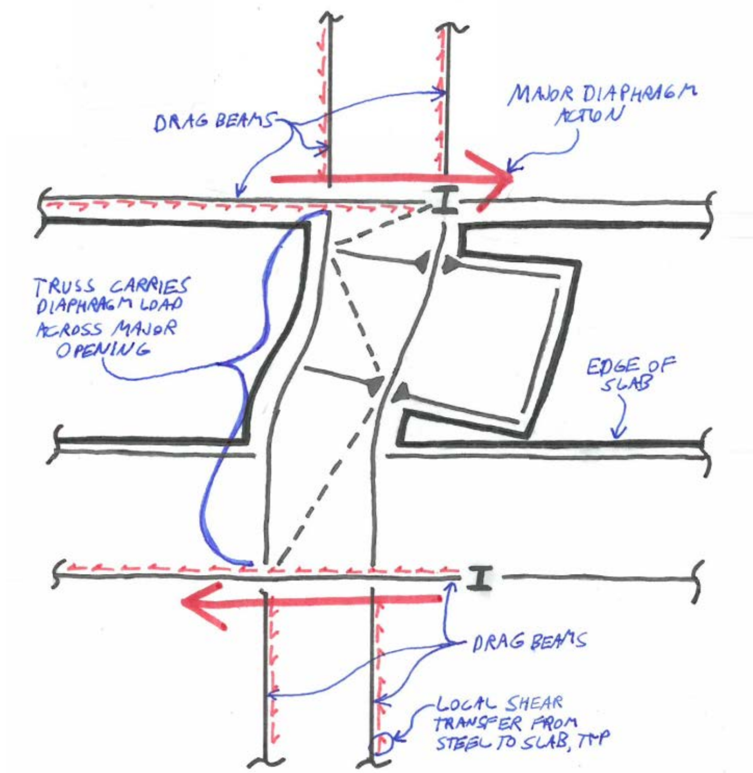


Figure 23 – Diagram showing how diaphragm loads are carried across major openings.

At the foundation level, lateral loads are no longer taken by the piles. Instead, they are taken by prestressed anchored footings, which have much higher lateral stiffness. Due to the addition of these footings, the lateral deflection at ground level is greatly reduced, thereby diminishing the passive resistance of the earth on the cellar walls. Therefore, footings have been designed to take the entire lateral load into bedrock (using friction on the footing / concrete fill / rock interfaces), without relying on the cellar walls or piles to share this load.

2.7.2 SHRINKAGE AND THERMAL EFFECTS

The length of the building is such that movements due to shrinkage and thermal effects must be taken into consideration for the design of the structure. The stability system is arranged in a manner which will reduce the effects of these shrinkage and thermal issues.

Although the building is relatively long, the points of lateral restraint (the braced frames) are kept relatively close (or flexible) enough so not to provide undue restraint against shrinkage and thermal movement. Special consideration is given to areas where restraint is increased between cores.

Consideration has been given to late-pour (leave out) strips of slab in the central area of the building around the atrium. These sections of slab would be cast later to allow some relief of shrinkage effects on either side, before the two sides are joined. Refer to CD drawings for details.

Any resulting movements due to shrinkage and thermal affects are expected to be within typical movement tolerances for cladding systems and finishes, and have been reviewed.

2.8 CELLAR LEVEL AND FOUNDATIONS

The building has a single level cellar extending under part of its footprint.

The cellar is a monolithic reinforced concrete box. Footings and pile caps are integrated into the base slab and the base and wall slabs are cast integrally. As of SD and DD, the concrete retaining walls around the perimeter were laterally braced at the top and bottom by the cellar and ground level slabs respectively. Where lack of continuity of ground level slab occurred due to major openings in cellar wall or slab, the retaining wall was designed to cantilever from the foundation or span horizontally to piers on the inside face of the cellar walls, or a combination of these two. As of CD, all foundations were designed as cantilever walls, given the high degree of perforation of the ground floor elevated slab and agreed with CM based on his preferences in terms of construction sequence .
Monolithic construction provides enhanced protection against ingress of groundwater, in addition to the protection provided by the primary membrane waterproofing system (not included in structural design).

The following sketch shows the primary structural systems and design assumptions for cellar and foundations.

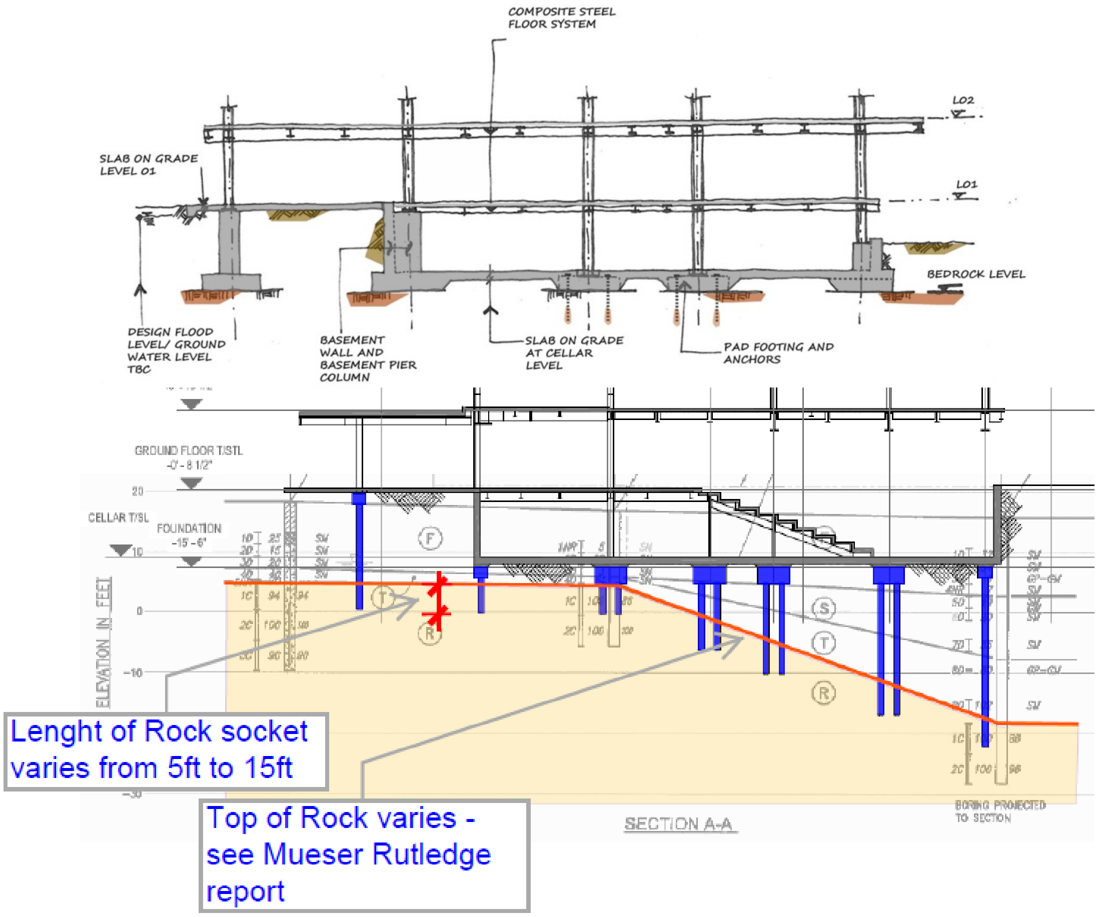


Figure 24 – Indicative substructures at SD stage (above) and at DD stage (below)

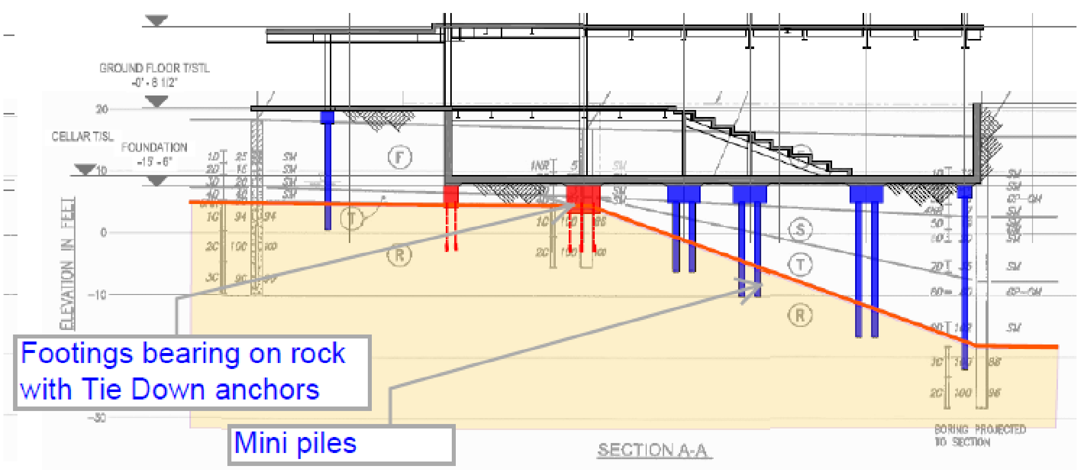


Figure 25 - Indicative substructures at CD stage

If the groundwater level around the cellar rises to reflect the extreme case floodwater elevation (noted as potentially up to +13ft NAVD88 as of DD; +14ft as of CD in Section 2.5 of report), and if the resulting hydrostatic uplift pressure is exerted on the base slab of the cellar level, the permanent weight of the building will not be sufficient to provide the factor of safety against flotation (buoyant forces) noted in Section 2.5. Additional resistance to buoyant forces is required in the form of either:

- additional structural self-weight (for example, thicker floor slabs), or
- physical tie-down (e.g. rock anchors or tension piles)

The SD structural drawings indicated tie-down anchors at each cellar column foundation (in combination with spread footings). The DD foundation design used mini-piles which can also act against uplift, eliminating the need for additional tie-down anchors. The number and layout of piles was designed in order to resist compression (downward) loads, tension (uplift) force and building lateral forces. The CD foundation solution integrates footings bearing on rock (with rock anchors) and mini piles socketed into rock that can also act against uplift. Where top of rock is expected to be less than roughly 6ft below bottom of pile cap, footings bearing on rock are the preferred solution from a technical and constructability prospective (this applies over about 1/3 of cellar level footprint). The combined foundation system of footing bearing on rock and mini piles has been designed to resist compression (downward) loads and tension (uplift). The footings bearing on rock resist the building lateral forces by way of prestressing the rock anchors. As shown in the figure below, the anchor prestressing (P_i) serves to counterbalance the uplift force (F) generated from hydrostatic uplift on the cellar slab and wind/seismic uplift from the supported column. P_i was initially calibrated by MRCE so that $P_i = F+V\mu$, where V is the shear demand on the particular footing, and μ is the friction coefficient between rock and concrete.

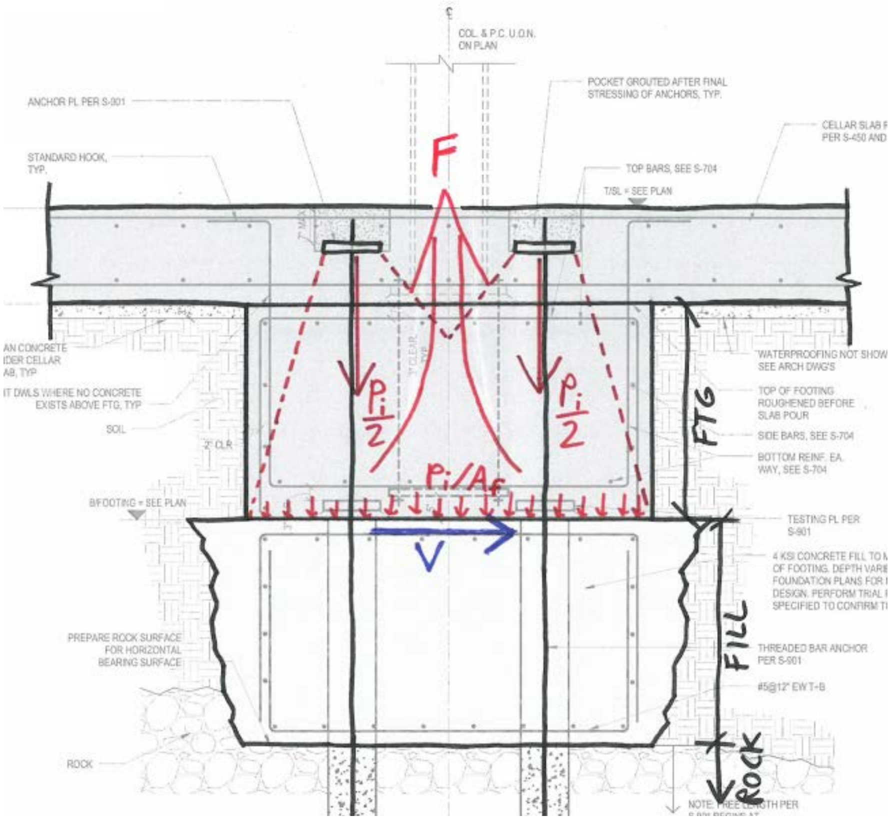


Figure 26 – Force Diagram for Anchored Footings

Recognizing that some footings, where rock is at an “intermediate” depth, require deeper excavation, a separate “concrete fill” pour was added to make up the difference between the bottom of the structural footing and the top of bedrock. This also minimizes the amount of work required at the deeper top of rock elevation by decreasing dewatering effort and elevating the anchor testing operation to the top of fill. As a result, the prestressing testing is accomplished by drilling directly through the fill into the rock below. The structural design of the footing is predicated on final prestressing done above the ftg as indicated in the figure above, in order to fully pre-load the interface between footing and fill, ensuring shear transfer between the two.

For braced frames landing on pile caps, the cellar slab is designed to carry the horizontal load from the braced frames to the anchored footings. At the grid 6 braced frame, for example, a steel drag beam is embedded within the cellar slab to distribute the load into the slab. At the grid 8 BF, however, beams embedded within the cellar slab carry the loads directly into adjacent footings, bypassing the need for the cellar slab to transfer the load. Beams embedded within the cellar slab have been sized and positioned to avoid interference with cellar slab rebar.

Grade beams are provided at the ground floor level as required by the BCCNY for Seismic Design Category C. Beyond their prescriptive requirement, the grade beams also serve to absorb moment generated by a maximum 3” pile tolerance – that is, a maximum of 3” between design center of pile and constructed center of pile, as stated in the geotechnical engineer’s specifications.

For comparison purposes, if actual maximum groundwater level does not exceed an elevation of approximately +9.6ft NAVD88, the building will have sufficient weight as designed to provide the required factor of safety against flotation without supplementary self- weight or tie-down. This analysis assumes top of cellar slab is at elevation circa +4.6ft NAVD88.

2.8.1 SLAB ON GRADE AT FIRST FLOOR LEVEL

The slab on grade at first floor (where there is no cellar below) will typically be 6" thick. The slab is thickened locally where required by special loadings, such as under heavy walls or within routes identified for installation of replacement of heavy equipment. The slab will also be thickened 4 ft at exterior edges to provide edge frost protection. At the north end of the building, the exterior grade elevation slopes down below the slab level and a short retaining wall is required around the edge.

2.9 MISCELLANEOUS CONSIDERATIONS

2.9.1 EXPANSION JOINTS

In structures with considerable length (~200 ft +) in one or more dimensions (such as this building in the north- south direction), there are advantages and disadvantages to subdividing the structure with expansion joints. Various alternatives were reviewed and coordinated with the architect. The options are indicated below:

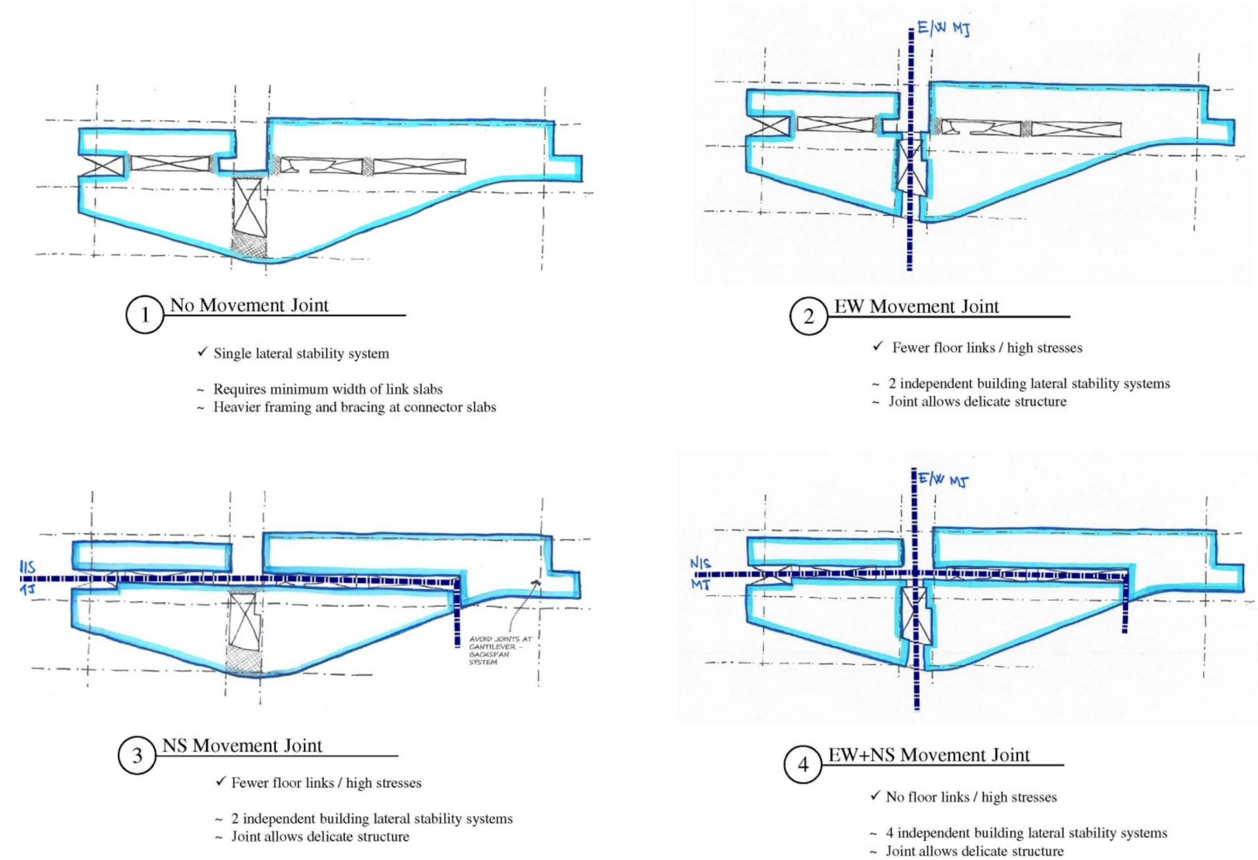


Figure 27 - Expansion Joint Options

Following design team coordination, the structure will built continuous without expansion joints. (Refer also to the discussion on shrinkage and thermal effects in Section 2.7.2)

2.9.2 STAIR #5

During DD stage a new main stair was added (Stair #5) which is a key component of the form of the building on its eastern elevation (campus plaza side).

At DD stage the structural system of Stair #5 was articulated in a single trussed spine column (oriented approximately east – west) from which stair ramps and landings were supported on cantilevers and connected to the main building floor system at each level. The roof was a horizontal slab extending from the main building mR level. The single trussed column was extended to support the east tip of the PV canopy. The stair system was laterally restrained by the floor landings that connect the stair to the main building. The architectural program for the stair indicated meeting places on intermediate landings of the stair.

Structural analysis and design was developed for gravity and lateral load effects lateral and design for human comfort under footfall induced vibration.

Deflection and vibration performance of the cantilevering landings relied on a hanger system that connected landings to the roof (at the further end of the cantilever) and connected the roof and landings to the foundation (at the two sides of the intermediate landings).

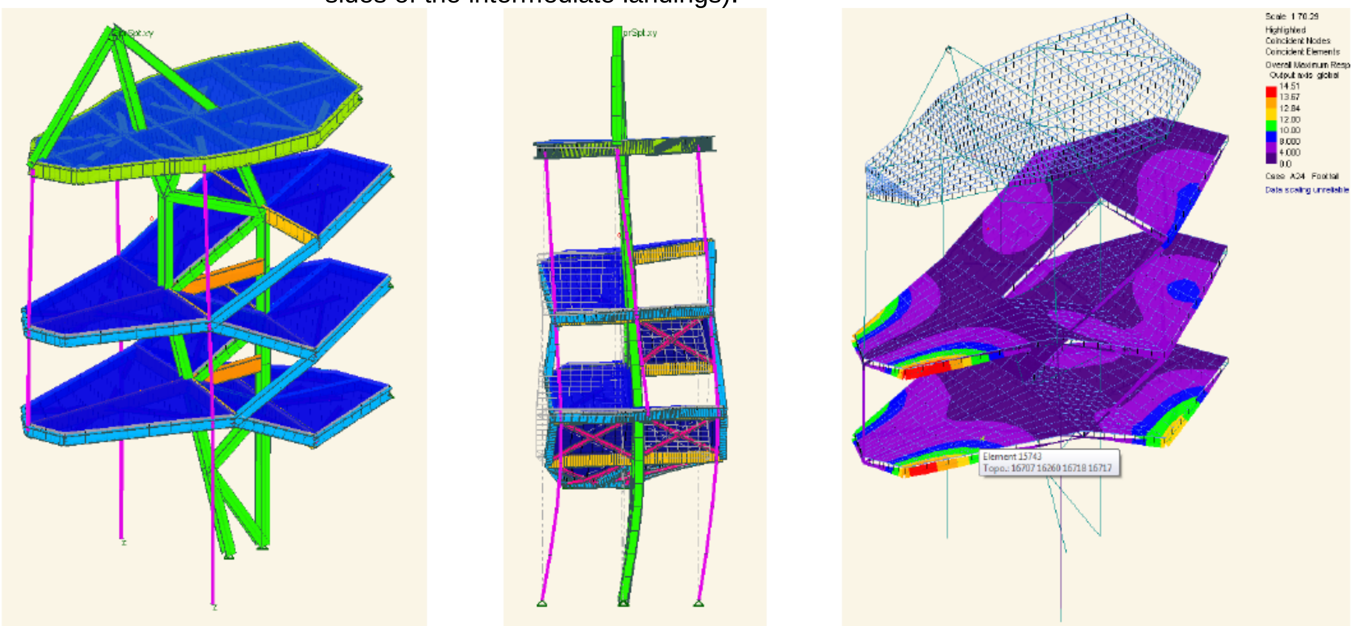


Figure 28 - Geometry, 1st Modal shape, Footfall results analysis of Stair #5 at DD stage.

The architect has confirmed that it is acceptable and in fact desirable for the stair including the meeting places (on the east end of the stair landings) to feel lively and active in terms of its footfall-induced vibration performance. The meeting places on landings are designed for a vibration criterion in the range of R=8 to 16 which is the range in which vibration is expected to be perceptible to clearly perceptible. It is emphasized that these areas are not designed to satisfy the more restrictive vibration performance typically adopted for office / work spaces (R=4 to 8, just perceptible to perceptible); this could only be achieved with additional structure, which would add cost and affect the form of stair. Stairs and landings (except in areas shown as meeting places) are designed for R = 24 which is typically considered to provide an acceptable level of performance.

During CD phase, the design of Stair #5 was further refined to optimize the form and structural system. The structural system includes:

- A central spine truss oriented longitudinally along the stair axis(extending to the canopy roof);
- Truss systems which include transverse trusses perpendicular to central spine truss, an edge truss which vertically supports the perimeter diagrid system described below and is connected to the transverse and central spine truss, and vertical truss on the south end which connects laterally the diagrid system and central column to the main building;

- An outer, partially curved diagrid truss system that supports and provides additional stiffness to the façade and the structure. This outer diagrid system is now part of the primary structural system. It provides the gravity support of stair edges and facade. Similarly to an outrigger system, the exterior diagrid system also helps to brace the overall stair structure laterally and transfer the lateral forces to the main building structure. The introduction of the diagrid, as an exterior triangulated structure connected to stair landings, improves vibration performance. This improvement is somewhat offset by lengthening of the stair structure which has also occured, however in overall terms the structure is now better integrated with the form and required function.

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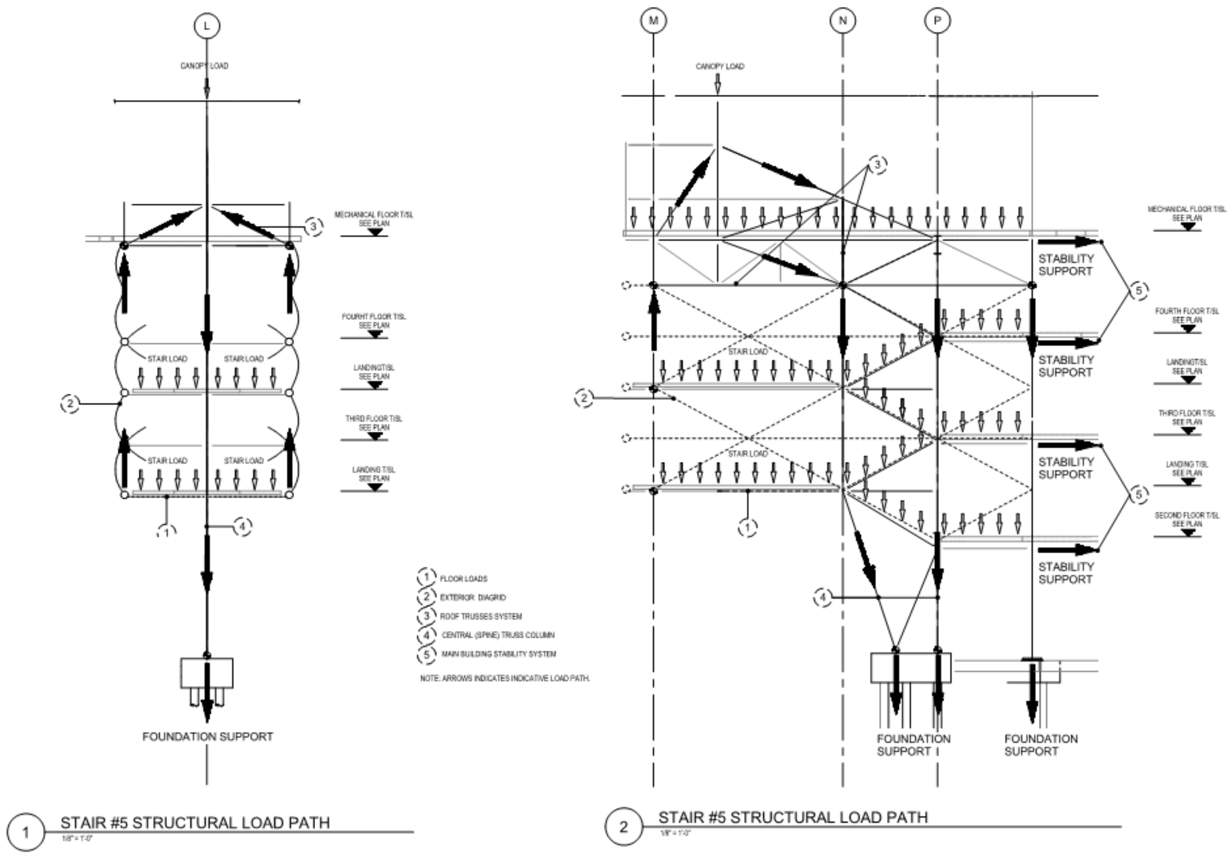

Figure 29 – Stair #5 -Structural indicative load path.

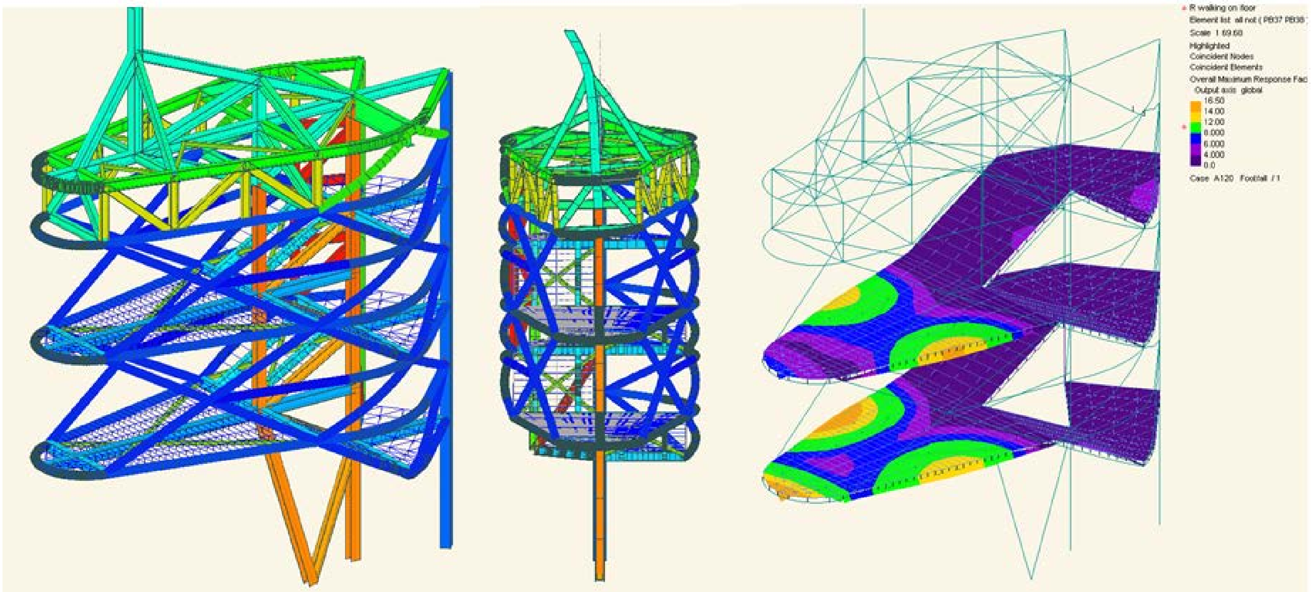

Figure 30 – Geometry, 1st Modal shape, Footfall results analysis of Stair #5 at CD stage